Combined Gravity and Lateral Loading of Light Gauge Steel Frame / Wood Panel Shear Walls

by

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ABSTRACT

Methods for the design of steel frame / wood panel shear walls used as a seismic force resisting system have been developed. These methods, which can be used in conjunction with the 2005 NBCC, were based on the results of shear wall tests carried out using lateral loads alone. The research program was extended to determine the influence of gravity loads on the lateral performance of the shear wall. An initial series of stud column tests was completed to determine an appropriate predication method for the axial capacity of the principal vertical load carrying members. Recommendations for appropriate effective length factors and buckling lengths were derived from the results of 40 tests. A subsequent series of five single-storey shear wall configurations were designed using capacity based methods. These shear walls were tested under monotonic and cyclic lateral loading, where two of three shear walls were also subjected to a constant gravity load. In total, 32 steel frame / wood panel shear walls composed of 1.09 – 1.37 mm thick steel studs sheathed with DFP, CSP or OSB panels were tested and analyzed. The equivalent energy elastic-plastic analysis approach was used to determine design values for stiffness, strength, ductility and overstrength. The data from this most recent series of tests indicates that the additional gravity loads do not have a detrimental influence on the lateral behaviour of a steel frame / wood panel shear wall if the chord studs are designed to carry the combined lateral and gravity forces following a capacity based approach. A resistance factor of 0.7 was found to be in agreement with previous tests that did not include gravity loads. The calculated seismic force modification factors also agreed with the previous test results, which suggest that $R_d = 2.5$ and $R_o = 1.7$.

RÉSUMÉ

Cette étude présente le développement de méthodes appliquées au design de murs de refend (panneau de bois combiné à un cadre d'acier) conçus dans l'objectif de créer un système à caractère antisismique. Ces méthodes, pouvant être utilisées en conjonction avec le CNBC 2005, ont été élaborées à partir des résultats obtenus lors de tests pratiqués sur des murs de refend par application de forces latérales. Le programme de recherche a été étendu à la détermination de l'influence de forces de gravité sur les performances latérales des murs de refend. Une première série de tests a été effectuée sur des montants afin de mettre en place une méthode permettant de prévoir la capacité de résistance axiale des principaux éléments verticaux porteurs de charge. L'optimisation des facteurs de longueur effective a été déduite des résultats de 40 tests. Une série ultérieure de cinq configurations de murs de refend uni-étage a été conçue en utilisant des méthodes basées sur les capacités de résistance du système. Ces murs de refend ont été testés par application de forces latérales monotoniques et cycliques alors que deux des trois murs étaient simultanément soumis à une force de gravité constante. Au total, 32 murs faits de montants d'acier d'épaisseur 1.09 - 1.37mm combinées à des panneaux DFP, CSP ou OSB ont été testés et analysés. L'analyse énergie équivalent élastique-plastique (EEEP) a été utilisée afin de déterminer les valeurs de rigidité, capacité, ductilité et la sur-réssistance appropriées au design. Les données provenant des plus récentes séries de tests indiquent que les forces de gravité supplémentaires n'ont pas d'influence négative sur les propriétés latérales des murs de refend si les montants ont été conçues pour supporter la combinaison de forces latérales et de gravités en suivant l'approche basée sur les capacité de résistance du système. Un facteur de résistance de 0.7 a été calculé, résultat en accord avec de précédents tests dans lesquels les forces de gravités n'étaient pas appliquées. Les facteurs de modifications dus aux forces sismiques qui ont été calculés sont également en accord avec de précédents tests, ce qui amène la suggestion suivante : $R_d = 2.5$ et $R_o = 1.7$.

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CHAPTER 1 INTRODUCTION

1.1 General Overview

The expanding capability of engineers to design light gauge steel structures has created a growing opportunity for construction in North America (Yu, 2000). Light gauge steel sections can be used for a number of purposes, such as floor decks, framing members, cladding and concrete forms. Figure 1.1 depicts a typical light gauge steel frame house with plywood sheathing.

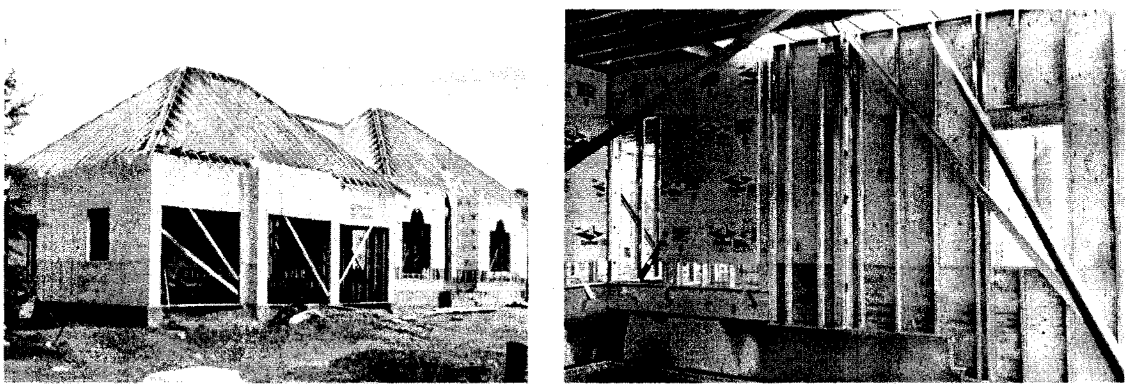


Figure 1-1 : Light gauge steel stud wall using platform framing technique (left: exterior view; right: interior view of side wall) ***(Branston, 2004)***

Walls framed with light gauge steel members can be designed as load bearing systems and as shear walls. Shear walls are designed to transmit in-plane lateral forces due to wind and earthquakes through the structure to the foundation, and to provide overall stability to the gravity load carrying system. To develop lateral resistance light gauge steel frames are sheathed with a structural grade of plywood or oriented strand board (OSB) panels. The wood paneling is affixed to the steel frame by means of screws, the size and number of which will determine the stiffness and shear resistance of the shear wall. The minimum size of the screw is generally dictated by the base metal thickness of the framing and can be increased depending on predicted loads. In order to transfer the lateral forces through the structure it is necessary to ensure that the wall is sufficiently attached to adjoining wall segments and/or the supporting foundation by means of shear anchors and hold downs. The anchorage of the walls in this fashion creates a structural element that behaves in essence as a vertical cantilever beam.

A progressive increase in the use of light gauge steel as a building product has been driven by a number of factors including market price, consistent quality, performance and knowledge. In 1993 a spike in North American lumber prices made light gauge steel a cost effective alternative (Gorte, 1994). The consistent quality and uniformity that can be expected from steel frame construction is beneficial for both builders and owners due to the ease of construction. As the popularity of light gauge steel construction increases the number of designers and builders familiar with this type of structure grows. Consequently, there is pressure on industry and researchers to develop standards and codes for the design and construction of these structures. More specifically, there is currently a demand for the development of design guidelines for shear walls that are expected to undergo seismic loading. At present, in Canada there exists no standards or codes with which engineers can design light gauge steel frame / wood panel shear walls. In order to develop a Canadian design standard a study of light gauge steel frame / wood panel shear walls was undertaken at McGill University in 2000. The overall aim of the research project is to develop a design method for light gauge steel shear walls to be used in conjunction with the National Building Code of Canada (NBCC) (NRCC, 2005). Completed research at McGill University includes the physical testing of single-storey shear walls under monotonic and reversed cyclic lateral loading (Boudreault et al. 2006; Branston *et al.*, 2006a; Branston *et al.*, 2006b; Chen *et al.*, 2006a; Blais, 2006; Boudreault, 2005; Branston, 2004; Chen, 2004; Rokas, 2006; Zhao, 2002). These past test programs reviewed the effects of: lateral loading protocol, fastener schedule, aspect ratio as well as, sheathing type and thickness. Methods have been developed for the interpretation of test results such that design parameters for wind and earthquake loads could be recommended. This includes shear strength and stiffness for a variety of wall configurations, as well as an appropriate resistance factor, seismic force modification factors and an overstrength factor. As well, hysteretic element models of these shear walls have been calibrated using the test data. These elements have been incorporated into the design of two typical buildings and analyzed under a limited number of dynamic loading conditions. There has been a substantial amount of research accomplished since 2000 regarding the design and performance of steel frame / wood panel shear walls. However, there is still further physical testing and analysis required to confirm and extend the applicability of the design methods developed through this research program.

1.2 Statement of the Problem

The 2005 National Building Code of Canada (NRCC, 2005) contains a procedure for the calculation of equivalent static seismic design loads, but does not list appropriate force modification factors (R_d and R_o) for light gauge steel frame / wood panel shear walls. As well, the North American Specification for the Design of Cold-Formed Steel Structural Members (NASPEC) (CSA S136, 2001) does not contain information concerning the design of shear walls. In addition, the current literature does not provide sufficient guidance, with respect to Canadian seismic design requirements, regarding the design and performance of light gauge steel frame / wood panel shear walls in structures.

In contrast, for engineers in the United States ASCE/SEI 7 (2005) lists response modification factors (R values) for light framed seismic force resisting systems. A shear wall design guide has also been developed by the American Iron and Steel Institute (AISI, 1998). As well, the AISI Committee on Framing Standards has introduced a standard for the lateral design of cold formed steel framing (AISI, 2004) and included shear wall design information in the IBC (ICC, 2006) and UBC (ICBO, 1997) model building codes. However, because this ASCE/SEI and AISI design information is based on the test results of American building products it is of great importance that studies be carried out with Canadian materials to develop an appropriate design method for light gauge steel frame / wood panel shear walls. The design method should reflect the limit states design philosophy as set forth in the 2005 National Building Code of Canada. Investigations have focused on the in-plane lateral loading of shear walls constructed with 1.09 mm thick steel framing and various types of wood sheathing.

In a structure the steel frame / wood panel shear wall can be used as the principal lateral load resisting system (Seismic Force Resisting System (SFRS)). Boudreault et al. (2006) recommended that steel frame / wood panel shear walls be considered as a moderately ductile system for which $R_d = 2.5$ and $R_o = 1.7$. Under this classification the system must provide a ductile response during repeated inelastic cycles, and a capacity based approach must be implemented in design. Capacity based design aims to have the most ductile response in a sacrificial element while limiting the inelastic displacement and force demand in other SFRS elements. The resulting structural system has a controlled yielding pattern to allow for maximum energy dissipation (Mitchell et al., 2003). In the design of the steel frame wood / panel shear walls the desired energy dissipation mechanism is the failure of the sheathing screw connections around the perimeter of the wall, either by bearing or plug shear failure of the plywood or OSB. During previous physical testing of these types of shear wall systems it was revealed that in certain configurations where sheathing screws were placed at a 75 mm spacing the back-to-back double chord studs could fail in compression, which is highly undesirable (Branston *et al.*, 2006b). The overall shear strength and ductility of the system could be reduced because of this failure mode; as well, the structure may not have the capacity to carry post earthquake gravity loads. Furthermore, these chord stud failures were observed for test specimens in which only lateral loads were applied. This recorded behaviour indicates that the inclusion of gravity loading could prove to have a critical influence on the performance of the SFRS in cases where the chord studs are not adequately sized.

Research into the effect of combined gravity and lateral loading on light gauge steel frame / wood panel shear walls has not been pursued before in North America. As mentioned earlier, the demand for research into this building system is fairly recent with respect to similar systems such as wood framed / wood panel shear walls. Also, previous research of the effect of combined loading on similar forms of construction, namely wood framing, concluded that gravity loading was beneficial to the system by increasing its capacity and ductility (Durham et al., 2001). Thus, there was no pre-existing basis for concern about the possible effects of combined loading on light gauge steel frame / wood panel shear walls. In contrast, for steel frame walls the presence of gravity loads has not proven to be beneficial given the occurrence of chord stud failure as reported by Branston (2004), and hence there is a definite need for the evaluation of shear walls subjected to combined gravity and lateral loading.

Critical to the performance of such shear walls is the ability of the chord studs to carry the compression forces imposed by the combined lateral and gravity loads. An accurate prediction of the sheathed double chord stud's axial compression capacity is difficult to achieve. The North American Specification for the Design of Cold-Formed Steel Structural Members (CSA S136, 2001) includes Section C4.6, which describes a procedure to calculate the capacity of a built-up member sheathed on one side. The 2004 Supplement to the Specification (AISI, 2005), however, no longer contains this procedure due to evidence that this method was not reliable nor typically used by designers. Thus, at present a bare steel design method is implemented, in which the predicted axial capacity of a double chord stud does not account for any increase in strength due to the contribution of the sheathing. The Direct Strength Method (Schafer, 2004), which was also introduced in the 2004 NASPEC supplement, is not yet advanced enough to provide designers with a readily accessible solution. Hence, a lack of information on the accuracy of the existing design methods for the sheathed chord studs required that testing be carried out on a range of representative specimens.

1.3 Objectives

The objectives of the research documented in this thesis include: 1) To verify the accuracy of calculating the axial compression capacity of back-to-back double chord studs by use of the effective width method as prescribed in the North American Specification for the Design of Cold-Formed Steel Structural Members (CSA S136, 2001, 2005), with physical testing; 2) To design and test, under combined lateral and gravity loads, a series of shear walls using the shear strength recommendations from Branston (2004) and the chord stud test results; 3) To determine design values for the tested walls using the equivalent energy elastic-plastic (EEEP) analysis approach; 4) To evaluate the results of the shear wall tests in order to identify the impact of gravity loads on lateral load carrying performance; 5) To calibrate hysteretic models for each of the wall configurations used in testing; And finally 6) to provide recommendations for future studies for the testing and analysis of shear walls to further expand the database of knowledge regarding the behaviour of light gauge steel frame / wood panel shear walls.

1.4 Scope of Study

A total of 40 tests were completed to determine the axial load carrying capacity of back-to-back double chord studs made of light gauge steel C-sections. The studs were constructed with Canadian cold-formed steel in four different thicknesses 0.84 mm, 1.09 mm, 1.37 mm and 1.72 mm. To address the fact that the studs are typically incorporated into a system they were built as part of a 610 x 1220 mm frame and sheathed with either 9.5 mm or 12.5 mm thick OSB or CSP panels on one side. As a control, one unsheathed chord stud of each thickness was tested to identify the effects of the sheathing and framing on the load carrying capacity. The results were compared with predicted design strengths and then used in the design of the light gauge steel frame / wood panel shear walls that would undergo combined lateral and gravity loading.

A series of 32 1220 x 2440 mm shear walls were constructed and then tested under combined lateral and gravity loading. The series was composed of 17 monotonic tests, the results of which were used to calibrate the Consortium of Universities for Research in Earthquake Engineering (CUREE) reversed cyclic loading protocol (Krawinkler et al, 2000; ASTM E2126, 2005) for the subsequent 15 shear walls. Each wall consisted of a light gauge steel frame and a sheathing panel of 12.5 mm plywood or 11 mm oriented strand board (OSB). The wall configurations that were tested matched those included in the research program carried out by Branston (2004), except that the thickness of the chord studs was selected to accommodate for the anticipated compression forces due to gravity and lateral loads. Design strength and stiffness values were derived from the test results, and then used in a comparison to evaluate the impact of gravity loads on lateral load carrying performance. Calibration of hysteretic models, for use in the subsequent dynamic modeling of representative light framed structures, was also included.

1.5 Thesis Outline

This thesis describes a study in which the effect of combined gravity and lateral loading on the behaviour of light gauge steel frame / wood panel shear walls was investigated. The study consists of four main parts; Chapter 2 includes a literature review of prior studies on the axial compression capacity of sheathed C-channels, an examination of previous shear wall testing with combined gravity and lateral loading and a brief description of previous tests and modeling of steel frame / wood panel shear walls at McGill University. Chapter 3 follows with a description of the chord stud experimental program. A summary of the results which provides a basis for the design of the combined loading wall test specimens is included. In Chapter 4 the combined loading shear wall experimental program is summarized. Chapter 5 consists of a review of the resulting shear wall test data in terms of the recommended design values. Chapter 6 deals with the calibration of hysteretic shear wall models using the software program HYSTERES (Carr, 2000). Chapter 7 provides conclusions and recommendations for future studies of the testing and modeling of light gauge steel frame / wood panel shear walls.

CHAPTER 2 LITERATURE REVIEW

2.1 SUMMARY OF SHEAR WALL TESTING

In 2000 a research program involving the performance and design of light gauge steel frame / wood panel shear walls began at McGill University. Since then, Zhao (2002), Branston (2004), Chen (2004), Boudreault (2005), Blais (2006) and Rokas (2006) have each written a thesis relating directly to the research program and provided a thorough literature review on topics related to shear walls. Hence, only a brief summary of their reviews and work will be presented herein. A more detailed review of the literature relating to the axial capacity of light gauge steel studs and the combined loading of shear walls is provided.

The testing of light gauge steel frame / wood panel shear walls began in the 1970s with Tarpy at Vanderbilt University (McCreless & Tarpy, 1978; Tarpy & Hauenstein, 1978). Subsequent studies were pursued by Tissel (1993). Largely as a result of the performance of light framed residential buildings in the 1994 Northridge CA earthquake a number of studies ensued; Serrette et al. (1996a, 1996b, 1997a, 1997b), Serrette and Ogunfunmi (1996), Serrette (1997), National Association of Home Builders (NAHB) (1997), Salenikovich and Dolan (1999), Salenikovich et al. (2000) and the City of Los Angeles (CoLA) – University of California at Irvine (UCI) (2001) all carried out physical testing of shear walls. From these studies, design guides and standards were developed for use by structural engineers in the United States. Design information is available in the 1997 UBC (ICBO, 1997), the 1998 Shear Wall Design Guide (AISI, 1998) the 2006 International Building Code (ICC, 2006) and the Standard for Cold-Formed Steel Framing – Lateral Design (AISI, 2004). A Canadian design standard for steel frame / wood panel shear walls has yet to be written.

Zhao (2002) completed a literature review of the existing shear wall test programs in North America and Australia. The researchers included were: McCreless and Tarpy (1978), Tarpy and Hauenstein (1978), Tarpy (1980), Tarpy & Girard (1982), Tissell

(1993), Serrette et al. (1996a, 1996b) and Serrette & Ogunfunmi (1996), Serrette (1997), NAHB (1997), Serrette et al. (1997a, 1997b), Gad et al. (1997, 1998, 1999a, 1999b, 1999c, 2000), Salenikovich and Dolan (1999), Salenikovich et al. (2000) and CoLA – UCI (2001). Zhao derived a ductility related R value for seismic design according to the 1995 NBCC (NRCC, 1995) from these past shear wall tests. The design of a shear wall testing frame, which was used for the test program described in Chapter 4 of this thesis, was also completed by Zhao.

In 2004 Branston described the test programs of Morgan et al. (2002) and Fulöp and Dubina (2002, 2003). This literature review also included a survey of the existing shear wall test programs in North America, Australia and Europe, plus a comparison of the standards for structural wood panels in Canada and in the United States. A series of 109 light gauge steel frame / wood panel shear wall specimens were tested under lateral loading during the summer of 2003 in the Jamieson Structures Laboratory at McGill University by Boudreault (2005), Branston (2004) and Chen (2004). Branston, who was responsible for 43 of the tests, used the complete data set to propose design parameters for in-plane strength and stiffness using the equivalent energy elastic-plastic (EEEP) method. The EEEP method was originally developed by Park (1989) and then modified by Foliente (1996) to identify a yield point. Based on the data for specimens sheathed with 12.5 mm CSP and DFP, as well as 11 mm OSB panels, Branston determined the walls exhibited an approximate overstrength of 1.2 and recommended a resistance factor of 0.7 for shear walls with an aspect ratio of 2:1 or less (Branston et al., 2006a,b).

Within the same testing series Chen (2004), who was responsible for 46 tests, examined the impact of varying the aspect ratio, sheathing fastener pattern and the sheathing material (12.5 mm CSP & DFP and 11 mm OSB) on the shear wall ultimate shear resistance, yield shear resistance, stiffness, ductility and energy dissipation (Chen et al., 2006a). Chen also recommended an analytical model to calculate the resistance and lateral deflection of various configurations of light gauge steel frame / wood panel shear walls based on the strength and stiffness characteristics of the sheathing connections (Chen et al. 2006b).

Using the complete data set from the 109 specimens as presented in Branston et al. (2004), Boudreault (2005) determined appropriate test-based seismic force modification factors (R_d & R_o) for use with the 2005 NBCC. Boudreault also reviewed existing reversed cyclic loading protocols for shear walls including the sequential phased displacement (SPD) (Porter, 1987), Applied Technology Council ATC-24 (1992), International Organization for Standardization ISO 16670 (2002) and the CUREE ordinary ground motions (Krawinkler et al., 2000; ASTM E2126, 2005). As well, Boudreault summarized the reversed cyclic protocols used by the following researchers: Karacabeyli & Ceccotti (1998), Dinehart & Shenton III (1998), Karacabeyli et al. (1999), Heine (2001), Gatto & Uang (2002) and Landolfo et al. (2004). A complement of 20 steel frame / wood panel shear wall specimens constructed with 12.5 mm CSP and DFP panels were tested by Boudreault. Using the resulting data as a benchmark, a review of existing hysteresis models for dynamic analyses and comments on their applicability to light gauge steel frame / wood panel shear walls were made. Boudreault concluded that the Stewart hysteretic element (1987) should be used to model the shear wall experimental data and provided appropriate calibrations for the 109 shear wall test specimens.

Blais (2006) augmented the light gauge steel frame / wood panel shear wall data set with a suite of 18 tests consisting of walls constructed with 9 mm OSB sheathing. These shear walls were tested in the same fashion as those by Branston, Boudreault and Chen; that is subjected to lateral in-plane loads alone. She calculated the relevant design parameters: strength, stiffness, resistance factor, overstrength, factor of safety and ductility, as well as the ductility and overstrength-related force modification factors according to the approaches recommended by Branston and Boudreault. Blais then compared the Stewart hysteretic element (1987) calibrated by Boudreault with the additional 18 OSB shear wall test results. Using Ruaumoko (Carr, 2000), non-linear time history dynamic analyses were carried out for two representative buildings modeled with Stewart hysteretic elements. Ten earthquake ground motion records from the west coast of North America were scaled to the uniform hazard spectrum for Vancouver BC from the 2005 NBCC.

The results of the dynamic analyses showed that the wall-segment rotations remained within the limits suggested by full-scale testing, and hence verified the test-based R_d = 2.5 and R_o = 1.7 force modification factors (Boudreault et al., 2006, 2007).

Rokas (2006) expanded the existing data set by testing 25 light gauge steel frame shear walls sheathed with 9.5 mm CSP. His walls were also tested with lateral in-plane loads following the procedures implemented by Branston, Boudreault, Chen and Blais. The relevant design parameters: strength, stiffness, resistance factor, overstrength, factor of safety and ductility, as well as the ductility and overstrength-related force modification factors were calculated according to the approaches recommended by Branston and Boudreault. Rokas' findings agreed with Blais' (2006) recommended force modification factors of R_d = 2.5 and R_o = 1.7.

In reviewing the past research a fundamental shortcoming with the approach used for testing steel frame / wood panel shear walls became apparent. Physical testing by Serrette et al. (1996b), Morgan et al. (2002) and Branston (2004) of this type of shear wall under lateral in-plane loading with fastener schedules of 50 mm (2") or 75 mm (3") along the panel perimeter exhibited local buckling compression failures in the end studs. This type of failure is not consistent with the intended capacity based design philosophy, which denotes the frame to sheathing fastener as the sacrificial element in the seismic force resisting system (SFRS) (Branston et al., 2006a). Failure of the chord studs does not provide the same degree of ductility, considering the force vs. deformation hysteretic behaviour of a shear wall, compared with the sheathing fastener failure mode. In addition, these vertical chord stud members are relied upon to maintain vertical load resistance once ground motion shaking has stopped. Furthermore, Branston et al. (2006a) stated that "The designer must be aware that compression failure of the chord studs may occur if gravity loads are present during wind or seismic events. These studs must be designed to resist the total expected compression force from gravity and lateral loads in order to preserve the overall structural integrity of the building". Thus far, no research in North America has addressed the effects of gravity forces during the lateral loading of steel frame shear walls. All of the testing that has been carried out has comprised walls subjected to lateral in-plane loads alone.

2.2 Axial Compression Capacity of Sheathed Light Gauge Steel Studs

2.2.1 Tian, Wang, Lu & Barlow (2004)

This experimental research program was aimed at determining the effects of one and two side sheathing on the vertical load carrying capacity of light gauge steel frames. The focus of the study was on walls sheathed on one side, since previous testing had considered two-sided walls, although unsheathed and fully sheathed walls were also included in the test program. The influence of screw spacing, stud dimensions and loading types were investigated. Tian et al. tested a group of 30 frames, 1250 mm x 2450 mm in dimension, for which the sheathing screw spacing of the interior studs was varied. During testing the loading pattern was either distributed as three simply supported point loads or as a single point load in the centre of the wall.

The studs were loaded individually under simply supported conditions for ten of the walls, and then compared with the predicted compression resistance obtained from the British Standard BS5950-5 (1998) entitled: "Structural use of steelwork in building. Code of practice for design of cold formed thin gauge sections". Using an effective length factor of K=1, the overall average failure load was less than that predicted by BS5950-5, however, in almost all cases the difference between the experimental and predicted loads was within 6%. The failure modes included overall flexural buckling in the unsheathed frames, flexural buckling and torsional-flexural buckling in the one side sheathed frames and crushing in the two side sheathed frames. An increase in the density of the screw spacing was found to increase the compression capacity of the interior studs. Under three point loading the panels provided more overall resistance because the distribution of the load was more efficient. On average, the panels with only a single distributed load carried 70% of the load of a wall loaded with three point loads.

2.2.2 Miller and Pĕkoz (1993)

The focus of this study was the overall behaviour of cold formed steel wall stud assemblies under axial loading. A series of unsheathed wall stud assemblies with mid-height channel bridging, mid-height strap bracing or no bracing were loaded in compression at their geometric centre to provide realistic conditions. Metal shims were placed between the tracks and the studs at the top and bottom of the walls to create more uniform bearing conditions. The studs used in the test program were both lipped channels either 92 mm (3 5/8") deep and 1.90 mm (0.075") thick or 152 mm (6") deep and 0.91 mm (0.036") thick. All of the observed failure modes agreed with the predicted flexural-torsional mode. Experimental results were compared with predicted values obtained using the AISI Specification (1986). The authors used effective length factors of $K_x = 0.7$ for all tests, $K_y = 0.5$ and $K_t = 0.7$ when unbraced, and $K_y = 0.35$ and $K_t = 0.44$ when braced to calculate the predicted loads. Normalized test-to-predicted ratios for the 92 mm channels were closer to unity than those for the 152 mm channels. Additional individual long column tests were performed to examine the sensitivity of strength to loading eccentricities. It was found that eccentricities as small as 2.5 mm could reduce AISI (1986) predicted failure loads by as much 40% below the concentric failure load. As well, a pilot series of stud tests with flat ends were tested to simulate conditions in construction where top and bottom tracks bear directly on concrete floors. Test results indicated that the assumption of full fixity was unconservative and that pinned-ended conditions with an assumed eccentric load were more conservative. The general conclusions drawn from this study were recommended effective length factors of $K_x = K_y = K_t = 0.65$ for unbraced studs and $K_x = 0.65$, $K_y = K_t = 0.4$ applied to the overall height of the wall.

2.2.3 Telue and Mahendran (2004)

This study included the development of a finite element model (FEM) of cold-formed C-channels lined on both sides with plasterboard, and physical testing to validate the model. The research was aimed at understanding the structural behaviour of steel wall frame systems with gypsum plasterboard and how axial strength is increased with a non-structural lining.

A finite element model of the steel frame / plasterboard walls using MSC/PATRAN and ABAQUS (Hibbitt, Karlsson and Sorensen Inc., 1996) was developed. Two different sizes and grades of channel were used in the comparative physical experiments. The C-sections had nominal dimensions of 75 x 30 x 1.2 mm and 200 x 35 x 1.2 mm (web x flange x thickness) in both G2 (minimum yield stress 175 MPa) and G500 (minimum yield stress 500 MPa) steel grades. The experiments were in good agreement with the Finite Element Analysis (FEA) in terms of predicted ultimate strength and failure mode. The study confirmed that stud spacing does not affect the ultimate load of studs in compression unlike the shear diaphragm model assumed by the 1986 AISI Specification (AISI, 1986) (Telue and Mahendran, 2001). The effective length factors used in their calculations came from design charts and equations, more specifically K_x (see Figure 2-1) from a design chart previously established by Telue and Mahendran (2002) and $K_y = K_t = nS_f/L$, where n represents the spacing factor. The mean ratio between the FEA and the experimental results was 0.99 with a COV of 0.11. However, Telue and Mahendran recommended an n = 2.0 to agree with the AISI design rules to allow for a defective adjacent screw fastener. The results were found valid for fastener spacings between 140 and 300 mm. The mean test-to-predicted ratio of results with the more conservative spacing factor was 0.94. The predicted ultimate loads were slightly underestimated, but more consistent with a COV of 0.07.

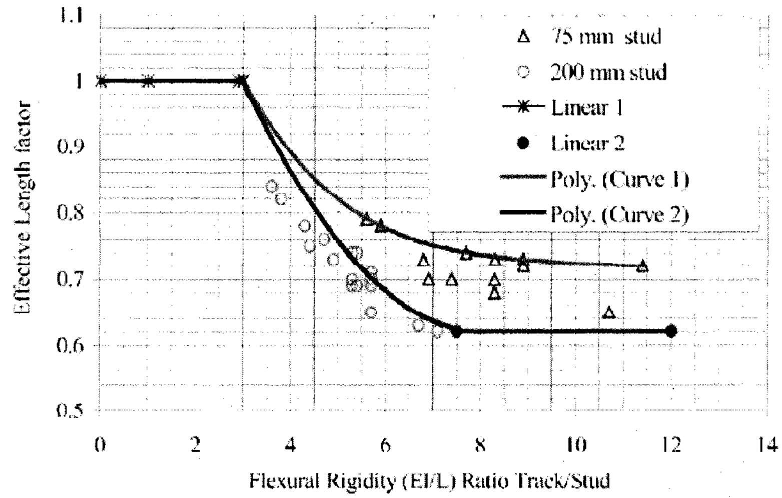


Figure 2-1 : Effective length factor for out-of-plane major axis flexural buckling versus flexural rigidity ratio (*Telue and Mahendran, 2002*)

2.2.4 Lee & Miller (2001)

Lee and Miller reviewed the theory behind the torsional and flexural stability of a centrally compressed bar as it relates to C-section stud walls sheathed with gypsum wallboard on both sides. The authors used the differential equation of equilibrium to determine the flexural and flexural-torsional buckling loads of C-sections sheathed with gypsum wallboard on both sides. This method was compared with the prediction methods for compression capacity contained in the 1986 and 1996 AISI Specifications. Both Specifications include Section D4 on Wall Studs and Wall Stud Assemblies, which has since been removed from the most current AISI Specification (2004) and S136 Standard (CSA S136, 2004). Section D4 allows the designer to account for the "bracing action due to both the rigidity and the rotational restraint supplied by the sheathing material", (CSA S136, 2004) by using K=1.0 and a length equal to twice the fastener spacing (2s) in the evaluation of the buckling load. These predicted capacities were then compared with the existing database of buckling loads for steel frame wall system assemblies sheathed with gypsum wallboard. The results confirmed the trend observed during experimental studies that stud spacing does not have an influence on axial capacity. Furthermore, the test assemblies reflected at least a 70% increase in strength when sheathed with gypsum wallboard in comparison to non-braced studs. The method of evaluation proposed by Lee and Miller results in compression capacities that are much closer to those found in experimental testing than the 1986 AISI Specification. However, the need for a more simplified approach is recognized, as well as a larger experimental database.

2.2.5 Stone & LaBoube (2005)

An evaluation of the current design provisions for built-up members in the North American Specification for the Design of Cold-Formed Steel Structural Members (AISI, 2004) was completed through experimental investigation. A series of 32 built-up I sections consisting of two C-sections attached back-to-back were tested using the Structural Stability Research Council Technical Memorandum No. 4: Procedure (Galambos, 1998). The specimens all measured 2.1 m in length, however the base metal thickness, web depth and screw spacing parameters were varied. A comparison of normalized test results (P_{test}/P_n) concluded that the design equations using the unmodified slenderness ratio, (KL/r), gave results that were conservative (by 43%) for sections with a base metal thickness greater than 0.89 mm. For sections with a base metal thickness less than 0.89 mm the capacity was only underestimated by 1% with the unmodified slenderness ratio. In the case where the modified slenderness ratio, required when the buckling mode produces shear forces in the connectors (Eq. 2-6), was used the range of the results was wider. In sections thicker than 0.89 mm the predictions were 65% conservative. The specimens consisting of sections thinner than 0.89 mm the capacities were overestimated by 16%.

2.2.6 Wang, Tian & Lu (2005)

This experimental study was conducted to determine the stress/strain distributions for the elements of a steel frame wall panel sheathed on one side. The Australian Standard for Cold-Formed Steel Structures AS4600 (Standards Australia, 1996) considers the lining materials of a wall sheathed on both sides to provide lateral and rotational support. The British Standard BS5950-5 (1998) does not consider any contribution of the panel to the load bearing capacity of the wall. Wang et al. studied the function of each structural member to give insight into the behaviour of the system and to develop a method for evaluating its capacity.

A total of 16 tests were performed on 2450 x 1250 mm steel frames, which were composed of four 93 x 67 x 1.2 mm perimeter tracks riveted together at each contact point. A central stud of 90 x 39/42 x 7.8 x 1.5 mm was connected to the tracks by 3 rivets at each point of contact. The frames were sheathed with Calcium Silicate Board (CSB), Cement Particle Board (CPB) and Oriented Strand Board (OSB). Only the screw spacing along the central stud was varied. (300, 400 or 600 mm) The tests were loaded in two different patterns. The first was the direct loading of the middle stud through a single point. The second pattern was through three point loads, each centered vertically over the stud or track.

Results showed that walls loaded through a single point redistributed about 20% of the load through the panel. Test walls loaded with three point loads only redistributed 5-10% of the load through the panel. Strain gauge measurements revealed that there is a very complex stress/ strain state in the top track. Thus, the design and quality of construction of the top and bottom track and connections is influential on the efficiency of the load redistribution. The measured strains of the middle studs were much higher along the flange that was attached to the sheathing. This uneven distribution of strain would need to be accounted for any design method developed. It was also found that as the screw spacing decreased the capacity of the panel increased. However, the authors commented that the gain in strength is marginal relative to the increase in cost. The main mechanism of failure for the system was the overall buckling of the stud accompanied by the pull-through of fasteners. The authors noted that the pull-out capacity of the fasteners relative to the thickness of sheathing and steel needs to be addressed in a design method for this style of wall.

2.3 Methods of Evaluating Axial Capacity of Built-Up Chord Studs

2.3.1 CSA-S136 (2004) Design Standard & Commentary

The provisions used to calculate the axial compression capacity of a cold-formed steel stud member are based on the effective width concept, which was initially developed by Von Karman et al. (1932). The approach used to determine the effective width of an element accounts for the non-uniform stress distribution across a compressed plate once the elastic local buckling stress level is reached. The Standard has evolved over the years to include design methods for many different sections and systems. Among those is a method to calculate the compression resistance of built-up members, provided in Chapter 4.5 of the Standard. Chapter 4.5 allows for an increase in strength of members connected by an adequate number of fasteners. Similar provisions to those described in Section 2.3.1 and 2.3.2 are included in the AISI Specification for the Design of Cold-Formed Steel Structural Members (AISI, 2004).

The S136 Standard (CSA, 2004) states that for compression members built of two sections in contact the nominal axial strength, P_n, shall be calculated as follows:

$$P_n = A_e F_n \quad (2\text{-}1)$$

where F_n:

For inelastic buckling, $\lambda_c \leq 1.5$

$$F_n = (0.658^{\lambda_c^2})F_y \quad (2\text{-}2)$$

For elastic buckling, $\lambda_c > 1.5$

$$F_n = \left[\frac{0.877}{\lambda_c^2}\right] \quad (2\text{-}3)$$

The lower of:

$$F_{e(flexural)} = \frac{\pi^2 E}{(KL/r)^2} \quad (2\text{-}4)$$

$$F_{e(torsional)} = \frac{1}{Ar_o}\left[GJ + \frac{\pi^2 EC_w}{(K_t L_t)^2}\right] \quad (2\text{-}5)$$

where $\lambda_c = (F_y / F_e)^{0.5}$

If the buckling mode produces shear forces in the connectors between members then, KL/r should be replaced with the modified slenderness $(KL/r)_m$:

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\left(\frac{KL}{r}\right)^2_o + \left(\frac{a}{r_i}\right)^2\right)} \quad (2\text{-}6)$$

where:

A_e	Effective area at stress F_n, with web perforation consideration
A	Full unreduced cross section
F_e	The least of the elastic flexural and torsional buckling stress
E	Modulus of Elasticity
F_y	Yield Strength
K	Effective length factor
L	Unbraced length of the member
$\left(\frac{KL}{r}\right)_o$	Overall slenderness ratio of entire section about built-up member axis
a	Intermediate fastener spacing
r_i	Minimum radius of gyration of full unreduced cross-sectional area of an individual shape (component of built-up member)
r_o	polar radius of gyration of cross section about shear centre
J	Saint-Venant torsion constant of cross section
C_w	Torsional warping constant of cross section
G	Shear modulus

Equations 2-1 through 2-6 were adopted from the CSA S136 (2004) Chapter C.4.

2.4 COMBINED GRAVITY AND REVERSED CYCLIC LOADING OF SHEAR WALLS

2.4.1 Ni & Karacabeyli (2000)

Ni and Karacabeyli used the results of full-scale shear wall test specimens, constructed with Spruce-Pine-Fir 38 mm x 89 mm lumber framing and 9.5 mm Canadian Softwood Plywood (CSP) sheathing, to compare with two methods that account for the effect of vertical and perpendicular loads on the performance of shear walls. The wall specimens were tested under lateral loads with and without hold -down connections, and with and without dead loads. Initially, a mechanics based method of evaluation that is a function of the end stud uplift restraint and the percent of nails resisting lateral loads along the bottom plate was evaluated. Force values were developed based on lateral load capacity per nail and the required uplift restraint at the ends of a wall. The method is dependent on the wall length, and hence indicates that the shorter the wall, the greater the influence from uplift restraint. This method of evaluation was found to be more conservative than an empirical method, which fit an equation to test data.

The shear wall testing was divided into two sections. The first series consisted of walls tested under monotonic and cyclic lateral loading with varied aspect ratios, panel orientation and fasteners. The second series of walls were tested under combined lateral and vertical (gravity) monotonic loading. The configuration for this series was not varied, with the exception of the inclusion of hold downs in two cases. The intensity of the vertical load ranged between 0 kN/m and 18.2 kN/m. It was found that a vertical load of 18.2 kN/m in a wall without hold downs was sufficient to resist the overturning moment and develop the full lateral load capacity.

2.4.2 Landolfo, Fiorino & Della Corte (2006)

This paper describes part of a research project entitled "A Theoretical and Experimental Study on the Feasibility of Using Cold-Formed Steel Members in Seismic Zones", sponsored by the Italian Ministry of Universities and Research. Two nominally identical shear wall prototypes were built using stick construction. The prototype consisted of two parallel shear wall diaphragms spaced 2 m apart and connected with a flat sheathed roof. Each wall segment measured 2700 x 2505 mm with lipped C-sections (100 x 50 x 10 x 1 mm) (web x flange x lip x thickness) as vertical members spaced at 600 mm c/c in the interior of the wall and doubled back-to-back chord studs at the ends. The frames were connected with screws and sheathed on one side with vertically oriented 12.5 mm Gypsum Wall Board (GWB) and 9 mm Oriented Strand Board (OSB) on the opposing side. The prototype was designed as part of a typical single family dwelling in a medium seismic zone in central Italy. A lateral seismic force of v_s = 11.0 kN/m was calculated in order to design the system according to capacity based methods, using the sheathing to frame connections as a fuse. To account for the loading of a typical home a gravity load was added to the top of the prototype that was equivalent to 8.33 kN/m along the wall length. Two synchronized lateral loads were applied in parallel to the top corners of each wall for both the monotonic and reversed cyclic loading. The monotonic test was composed of two phases. The first phase looked at the permanent offset of the system after small displacements (2, 4, 6, 10 mm). The second phase consisted of the testing of the prototype to a displacement of 150 mm. Both walls behaved similarly up until the maximum shear resistance of 18.5 kN/m was reached at about a 30 mm displacement. The sheathing to panel connections failed as the deformations progressed past this point. There was no damage to the chord studs whatsoever and only the tracks connecting the flooring buckled locally after the maximum shear resistance was obtained.

The cyclic loading protocol was based on a numerical study by Della Corte et al. (2006) in accordance with the Applied Technology Council (ATC 24, 1992) for a multi-step test. The protocol cycled three times at increasing displacement steps with the final step being 78.0 mm at a rate of 2 mm/s. A maximum load of $v_{r+1,\,Test}$ = 16.4 kN/m was obtained at a displacement of 36 mm in the direction of initial loading. The minimum load, in the opposite direction, was $v_{r-1,\,Test}$ = -14.8 kN/m at a displacement of -24 mm. The first cycle of each step always achieved the highest resistance. On average there was a 38% difference in shear strength between the first and third cycle of any step in the positive direction. In the negative direction this behaviour was less pronounced with an average difference of about 16%. Damage to the walls focused on the sheathing to panel connections. Tilting of the screws and the pull through failure of the screw heads were characteristic of the OSB panels. Screw bearing and pull through failure of the screw heads were typical for the GWB panels. At large displacements both panels came unzipped along the panel edges allowing the frame to rotate into a parallelogram shape and causing distortional buckling at the ends of the double chord studs. An energy analysis revealed that the walls dissipated approximately 50% of their energy before the maximum resistance was obtained. This experimental program proved that this structural system could be efficiently designed according to capacity based criteria imposing the failure of the sheathing to panel connectors.

2.4.3 Durham, Lam & Prion (2001)

A study of the seismic resistance of wood shear walls sheathed with large 2400 x 2400 mm and standard 1200 x 2400 mm size oriented strand board (OSB) panels was presented by Durham et al. (2001). The walls were tested under quasi-static monotonic and cyclic conditions as well as under dynamic loads. For the construction of the walls No.2 Grade Spruce-Pine-Fir 38 x 89 mm dimensional lumber was used for all framing members. Pneumatically driven 76 mm common nails were used to assemble the frames. The edge studs and top plate members were doubled and the frame was sheathed with 9.5 mm thick performance rated W24 OSB. A gravity load of approximately 9.1 kN/m was used, representative of a 5 kPa uniform load on a 7.3 x 7.3 m room. The load was applied across the top of the wall using a stiff beam that had two point loads applied to it through a tensioned cable connected to a pulley system.

The dominant failure mode of the walls for all loading types involved the nails pulling out of the frame or the nails pulling through the sheathing. This indicated that the overall behaviour of the shear wall is dictated by the fastener configuration. The gravity loads played an increasingly larger role in the behaviour of the wall as the aspect ratio (height to width) increased. The vertical dead load restrained the wall corners, thus resisting the over turning moment. However, the importance of installing hold down brackets to increase tensile capacity and to achieve the full racking resistance of the wall was also confirmed.

2.5 SUMMARY

Building on the previous research from this program, looking specifically at the potential performance of steel / frame shear walls under combined gravity and lateral loading, the capacity based design approach was reviewed because of concerns about the chord stud capacity raised by Branston, 2004 and Branston et al., 2006a. The preliminary test program on the axial capacity of light gauge steel studs (Chapter 3) was shaped by the previous research conducted by Telue and Mahendran (2001, 2004) and Lee and Miller (2001). Aspects of the test set-up used by these researchers were used to develop an applicable approach to the evaluation of the double chord stud (DCS) sheathed on one side. It should be noted, that to date no standard or codified method exists for evaluating the axial capacity of steel frame panels sheathed on one side. Suggestions for axial capacity predictions made by the previously mentioned authors as well as Miller and Pěkoz (1993) and Stone and LaBoube (2005) were applied to determine the appropriate fixity and slenderness ratio. Studies on the behaviour of sheathed steel frame panels by Tian *et al.* (2004) and Wang *et al.* (2005) were used to help explain behaviour and failure modes that occurred in the preliminary test series (Chapter 3). Applying the conclusion of the preliminary test series and the analytical work by Chen (2004) the main test series of steel frame / wood panel shear walls for combined gravity and lateral loading was designed (Chapter 4). Investigations into the combined gravity and lateral loading of similar shear wall systems, i.e. Durham *et al.* (2001) and Ni and Karacabeyli (2000) were used to develop an idea of appropriate gravity load levels and means of applying the gravity load. The methods for data analysis proposed by Branston (2004) and Boudreault (2005) were applied to these test results, so the influence of combined gravity loads could be contrasted with previous laterally loaded shear wall results (Chapters 5 & 6).

CHAPTER 3 CHORD STUD COMPRESSION TESTING

3.1 INTRODUCTION

Previous testing of steel frame / wood panel shear walls under in-plane lateral loading has revealed that buckling of the chord studs is of concern for certain configurations. In studies performed by Serrette et al. (1996b), Morgan et al. (2002) and Branston (2004) some of the walls with framing to sheathing fastener schedules of 50 mm (2") or 75 mm (3") along the perimeter exhibited local buckling failure of the double chord studs (DCS) (Figure 3-1), which compromised the vertical load carrying capacity of the system. Furthermore, the shear capacity and ductility of the shear walls that failed in this mode were reduced to some extent compared with similar walls in which the sheathing-to-framing connections failed instead. Branston et al. (2006a) stated that "chord stud compression buckling is an unfavourable governing failure mode for a lateral force resisting shear wall because, in almost all situations, 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 carry the gravity loads, which may lead to a possible collapse in part of the structure." Hence, the designer must select the chord studs to resist the total expected compression force obtained from the combination of lateral and gravity loads placed on the shear wall. In terms of seismic lateral loads, with capacity based design concepts in mind, the relevant forces imposed on the chord studs can be obtained from the expected ultimate capacity of the shear wall when the sheathing-to-framing connections fail and the companion gravity load.

In the selection of the chord studs it is necessary to accurately estimate their compression load carrying capacity, accounting for the possible bracing effect of the attached sheathing. Studies by Tian et al. (2004), Lee & Miller (2001) and Wang et al. (2005) have indicated that sheathed chord studs routinely exhibit higher axial load capacities relative to unsheathed members. However, the most recent version of the CSA S136 Design Standard does not contain a method to account for the contribution of strength from sheathing on one side of a panel. For this reason a series of tests was carried out to investigate the behaviour and capacity of sheathed built-up chord stud members. The ultimate loads measured by means of physical testing were then compared to capacities calculated using the S136 Standard and its most recent supplement (CSA, 2001, 2004). These calculated values were later applied in the design of the steel frame / wood panel shear wall test specimens that were subjected to combined gravity and lateral loading (Chapter 4).

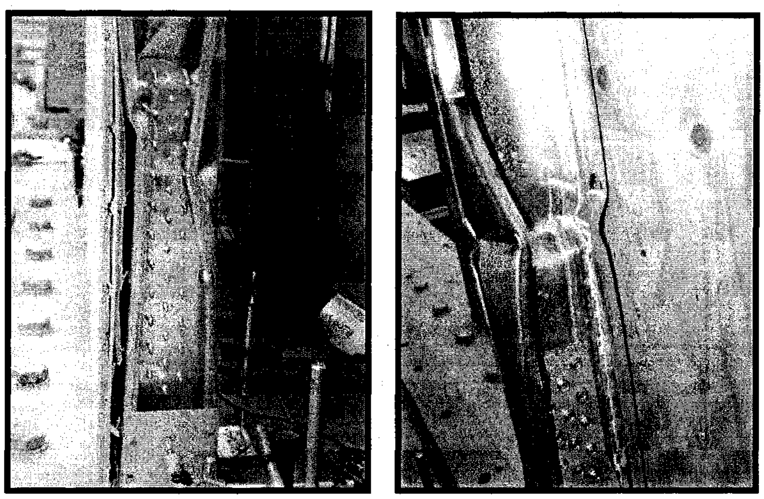


Figure 3-1 : Compression chord local buckling in Test 13B ***(Branston, 2004)***

3.2 CHORD STUD TEST PROGRAM

During the winter of 2005 a total of 40 full height double chord studs (DCS) were tested to determine their axial compression capacity. Each double chord stud specimen consisted of two light gauge steel 92.1 mm deep C-sections connected back-to-back with two No. 10 long Hex head self-drilling screws at various intervals. Simpson Strong-Tie S/HD10 (Simpson, 2001) hold down connectors were attached at the base of the DCS with No. 10 long Hex head self-drilling screws.

The tests were separated into two groups; sheathed and bare steel chord studs. The first group of 36 tests (18 configurations x 2 specimens) was composed of 610 x 2440 mm sheathed wall specimens (representing the maximum allowable height to width aspect ratio 4:1 (AISI, 2004)) with one DCS at the loaded panel edge and a single 1.09 mm

(0.043") thick C-section stud at the opposing panel edge. The studs were connected at top and bottom with 1.09 mm (0.043") thick light gauge steel tracks. The sheathing to frame fasteners were installed with a denser spacing along the panel edge aligning with the DCS and the tracks (3 edges) compared with the single stud. Each specimen was designed as one half of a 1220 x 2440 mm wall, thus the single stud represented the interior stud and therefore had a lighter screw pattern. The second group comprised four test specimens in which the DCS was constructed without any sheathing or attached steel framing. The purpose of these four tests was to provide information such that a comparison of the sheathed and unsheathed stud capacity could be made. Note, that for the sheathed chord stud specimens a wood panel was installed on only one side of the wall and assumed to provide sufficient in-plane stiffness. No attempt was made to simulate the gypsum panels that would typically be installed on the interior side of a wall. It is possible that the gypsum would have allowed for additional lateral support, however the intent was to provide information for engineers who take a conservative approach in the design of these shear walls, that is assume the gypsum does not act as a structural or bracing member. Furthermore, all of the shear wall tests (Chapter 4) were constructed with wood structural panels on one side only, thus the chord stud specimens were similar in configuration.

A number of parameters were varied from specimen to specimen including: thickness of the DCS (0.84 mm (0.033"), 1.09 mm (0.043"), 1.37 mm (0.054") and 1.72 mm (0.068")), panel material (9.5 or 12.5 mm thick Oriented Strand Board (OSB) or Canadian Softwood Plywood (CSP)), fastener schedule (either 75 mm or 152 mm along the exterior edges and 305 mm or 610 mm on the "interior" stud). A summary of the test configurations is listed in Table 3-1.

Table 3-1 : Summary of chord stud test configurations

Test	Stud Thickness	Sheathing Type	Sheathing Thickness (mm)	Interior Stud Screw Spacing	Screw Spacing in DCS[1]	Screw Spacing in Sheathing
1. 043OSB1-12-3A	1.09 mm	OSB	12.5 mm	305 mm	305 mm	75 mm
2. 043OSB1-12-3B	1.09 mm	OSB	12.5 mm	305 mm	305 mm	75 mm
3. 043OSB2-12-3A	1.09 mm	OSB	9.5 mm	305 mm	305 mm	75 mm
4. 043OSB2-12-3B	1.09 mm	OSB	9.5 mm	305 mm	305 mm	75 mm
5. 043OSB1-24-3A	1.09 mm	OSB	12.5 mm	305 mm	610 mm	75 mm
6. 043OSB1-24-3B	1.09 mm	OSB	12.5 mm	305 mm	610 mm	75 mm
7. 043OSB2-24-3A	1.09 mm	OSB	9.5 mm	305 mm	610 mm	75 mm
8. 043OSB2-24-3B	1.09 mm	OSB	9.5 mm	305 mm	610 mm	75 mm
9. 043OSB1-12-6A[2]	1.09 mm	OSB	12.5 mm	305 mm	305 mm	152 mm
10. 043OSB1-12-6B	1.09 mm	OSB	12.5 mm	305 mm	305 mm	152 mm
11. 043OSB2-12-6A	1.09 mm	OSB	9.5 mm	305 mm	305 mm	152 mm
12. 043OSB2-12-6B	1.09 mm	OSB	9.5 mm	305 mm	305 mm	152 mm
13. 043OSB1-24-6A	1.09 mm	OSB	12.5 mm	305 mm	305 mm	152 mm
14. 043OSB1-24-6B	1.09 mm	OSB	12.5 mm	305 mm	305 mm	152 mm
15. 043OSB2-24-6A	1.09 mm	OSB	9.5 mm	305 mm	305 mm	152 mm
16. 043OSB2-24-6B	1.09 mm	OSB	9.5 mm	305 mm	305 mm	152 mm
17. 033OSB1-12-3A	0.84 mm	OSB	12.5 mm	305 mm	305 mm	75 mm
18. 033OSB1-12-3B	0.84 mm	OSB	12.5 mm	305 mm	305 mm	75 mm
19. 033OSB1-12-6A	0.84 mm	OSB	12.5 mm	305 mm	305 mm	152 mm
20. 033OSB1-12-6B	0.84 mm	OSB	12.5 mm	305 mm	305 mm	152 mm
21. 043CSP1-12-3A	1.09 mm	CSP	12.5 mm	305 mm	305 mm	75 mm
22. 043CSP1-12-3B	1.09 mm	CSP	12.5 mm	305 mm	305 mm	75 mm
23. 043CSP2-12-3A	1.09 mm	CSP	9.5 mm	305 mm	305 mm	75 mm
24. 043CSP2-12-3B	1.09 mm	CSP	9.5 mm	305 mm	305 mm	75 mm
25. 043CSP1-12-6A	1.09 mm	CSP	12.5 mm	305 mm	305 mm	152 mm
26. 043CSP1-12-6B	1.09 mm	CSP	12.5 mm	305 mm	305 mm	152 mm
27. 043CSP2-12-6A	1.09 mm	CSP	9.5 mm	305 mm	305 mm	152 mm
28. 043CSP2-12-6B	1.09 mm	CSP	9.5 mm	305 mm	305 mm	152 mm
29. 054OSB1-12-3A	1.37 mm	OSB	12.5 mm	305 mm	305 mm	75 mm
30. 054OSB1-12-3B	1.37 mm	OSB	12.5 mm	305 mm	305 mm	75 mm
31. 054CSP1-12-3A	1.37 mm	CSP	12.5 mm	305 mm	305 mm	75 mm
32. 054CSP1-12-3B	1.37 mm	CSP	12.5 mm	305 mm	305 mm	75 mm
33. 068OSB1-12-3A	1.72 mm	OSB	12.5 mm	305 mm	305 mm	75 mm
34. 068OSB1-12-3B	1.72 mm	OSB	12.5 mm	305 mm	305 mm	75 mm
35. 068CSP1-12-3A	1.72 mm	CSP	12.5 mm	305 mm	305 mm	75 mm
36. 068CSP1-12-3B	1.72 mm	CSP	12.5 mm	305 mm	305 mm	75 mm
37. 033DoubleChordStud	0.84 mm	N/A	N/A	305 mm	305 mm	N/A
38. 043DoubleChordStud	1.09 mm	N/A	N/A	305 mm	305 mm	N/A
39. 054DoubleChordStud	1.37 mm	N/A	N/A	305 mm	305 mm	N/A
40. 068DoubleChordStud	1.72 mm	N/A	N/A	305 mm	305 mm	N/A

[1] Double Chord Stud (DCS)
[2] Two extra fastners in web-to-web connection at 19 mm (0.75") from the top

3.3 Wall Fabrication, Materials and Components

The materials used to construct the 40 specimens listed in Table 3-1 were:

- 9.5 mm and 12.5 mm CSA O151 Exterior Canadian Softwood Plywood (CSP) *(CSA O151, 1978)*, 9.5 mm and 12.5 mm CSA O325 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). See Figure 3-2 for mill and grade stamps.

- Light gauge steel studs manufactured in Canada to ASTM A653 (2002) with the following four nominal grades and thicknesses: 1. 230 MPa (33 ksi) and 0.84 mm (0.033"), 2. 230 MPa (33 ksi) and 1.09 mm (0.043"), 3. 340 MPa (50 ksi) and 1.37 mm (0.054") and 4. 340 MPa (50 ksi) and 1.72 mm. All studs had nominal dimensions of: 92.1 mm (3-5/8") web, 41.3 mm (1-5/8") flanges and 12.7 mm (1/2") lips.
- Light gauge steel top and bottom tracks manufactured in Canada to ASTM A653 (2002) with nominal grade of 230 MPa (33 ksi) and a thickness of 1.09 mm (0.043"). All tracks had nominal dimensions of 92.1 mm (3-5/8") web and 31.8 mm (1-1/4") flange.
- The double chord stud (DCS) consisted of two studs connected back-to-back and connected by two No. 10—16 x 19.1 mm (3/4") long Hex head self-drilling screws at 305 mm (12") on centre. The interior stud was spaced at 610 mm (24") on centre.
- Industry standard Simpson Strong-Tie S/HD10 (Simpson, 2001) hold down connectors were attached to the DCS with 33 No. 10-16 x 19.1 mm (3/4") long Hex head self-drilling screws.
- No. 8 x 12.7 mm (1/2") long wafer head self-drilling framing screws were used to connect the track and studs.
- No. 8 x 38.1 mm (1-1/2") Grabber SuperDrive (SuperDrive, 2003) Bugle head self-piercing sheathing screws were used to affix the sheathing to the light gauge steel frames. The sheathing-to-framing screws were installed 12.7 mm (1/2") away from the edge of each sheathing panel. The screw spacing / fastener schedule was 75 mm (3") or 152 mm (6") along the panel edges and 305 mm (12") in the interior.

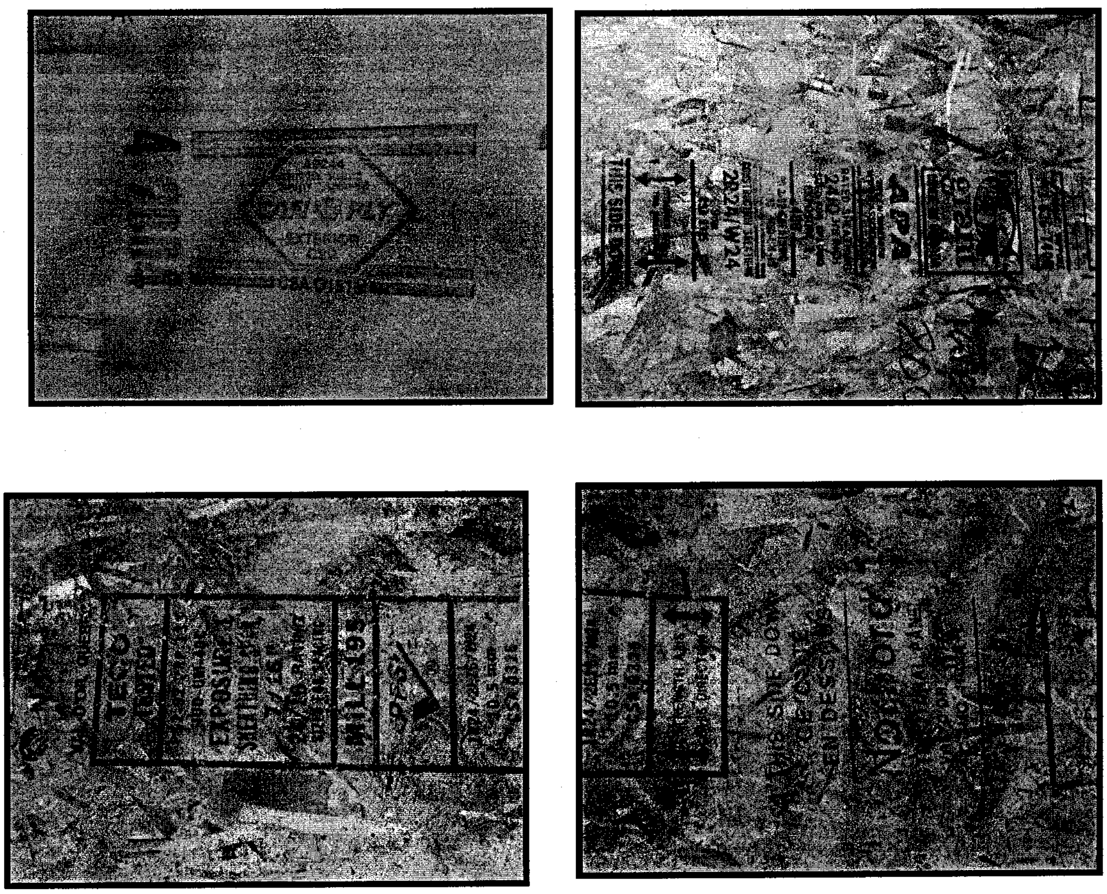


Figure 3-2 : Mill and grade stamps from sheathing

This study utilized as many Canadian products as possible in order to best reflect the quality and material property standards available in the country. The specimens for the axial chord stud tests were fabricated in the same manner as described in Branston (2004) with some small modifications. Due to the nature of the loading of the DCS it was important to match the tops of the chord studs before they were fastened together. Otherwise, the load would not be evenly distributed through both members. The tracks were fitted as tightly as possible to the DCS to avoid any gap between the studs and the web of the track. Prior to testing any remaining gaps along the load path were shimmed to ensure an even transfer of force. To prevent the wood panels from carrying compression load through contact with the loading and reaction plates of the test frame, a section approximately 3 mm (1/8") in width was shaved off the top and bottom panel edges as shown in Figure 3-3.

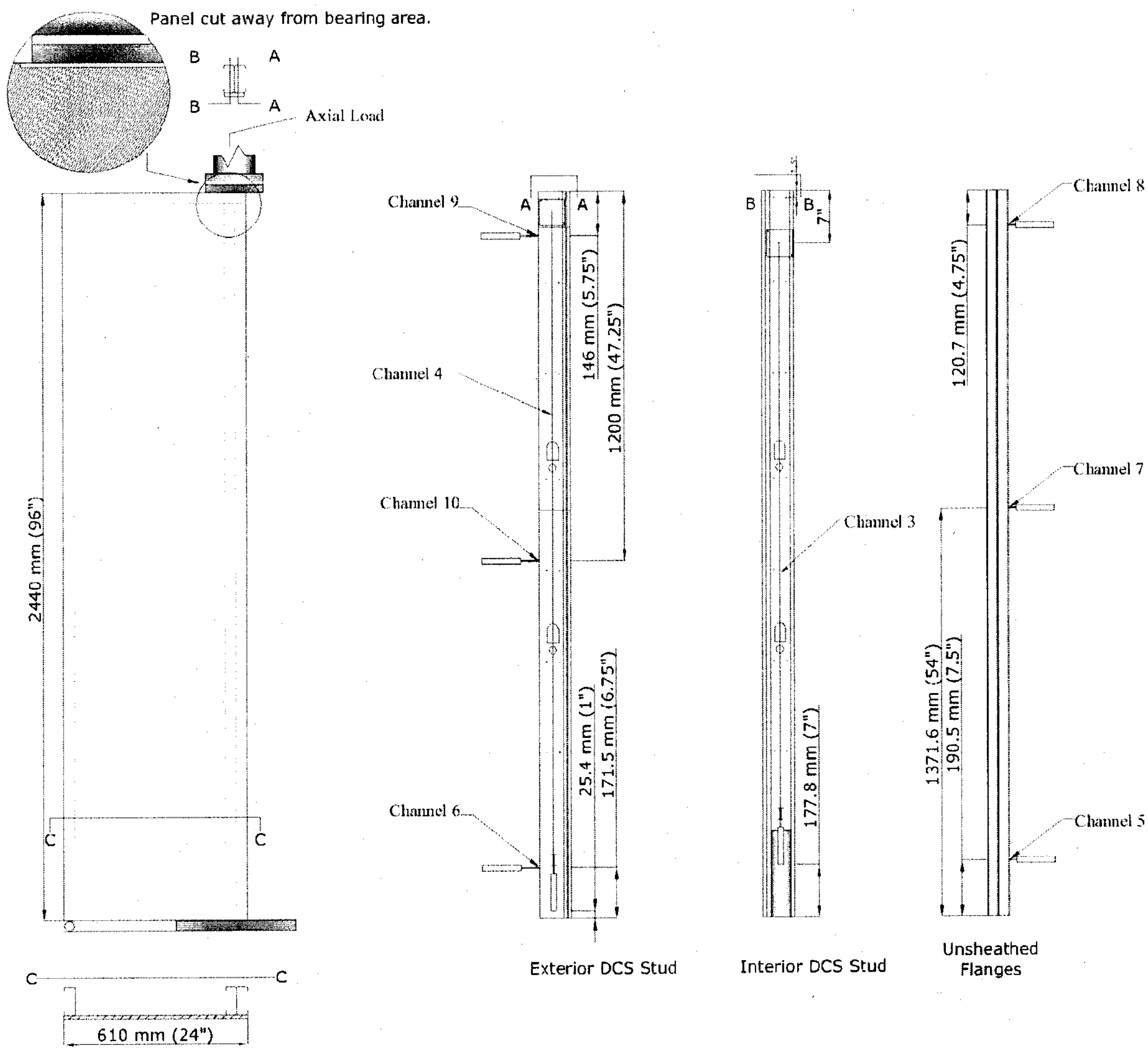


Figure 3-3 : Diagram of wall panel edges at bearing surfaces shaved off and locations of LVDTs on the DCS

3.4 TEST SET-UP AND LOADING PROTOCOL

The test set-up was located in the Jamieson Structures Laboratory at McGill University. A Baldwin universal testing machine fitted with a 600 kN actuator was used to perform this series of testing. The actuator was positioned to exert a concentric downward displacement on the top of the double chord stud. The interior stud and wood sheathing were not connected to the actuator, hence were not directly loaded by movement of the actuator piston. Attached to the actuator was a set of two steel plates that were separated by two layers of a 12.5 mm thick dense rubber. The rubber was used to limit the lateral force due to bending of the test specimens from being transferred to the actuator. The face of the plate in contact with the test specimen was milled, hardened and then milled flat for optimal load transfer. The base of the wall was placed on a hardened reaction plate, whose top surface was also milled flat. The wall specimen was installed in the vertical position and geometrically centered between the loading plate and the reaction plate. The end condition at the bottom restricted any rotations beyond what the track to DCS connection would allow for. At the top of the test specimen the rotations were similarly restrained except for the minute rotations allowed by deformation of the rubber. These end conditions were chosen over pin ended assemblies because they more realistically reflected the end conditions in an actual structure. A simple support of a half round was placed under the single chord stud at the base such that the wall would not rack in-plane while the chord stud was loaded. No support was provided to the top of the single chord stud. In some cases it was necessary to place shims under the base of the track to ensure the wall stood perfectly vertical. The test set-up for specimens 37 through 40 was slightly different from what was previously described as they were unsheathed DCS without framing. These tests were used to compare the difference in load carrying capacity between the sheathed and unsheathed member. For these particular tests the DCS was centered geometrically between the milled flat steel surfaces of the loading and reaction plates. Similarly to the tests previously mentioned the contact surfaces between the stud and plates were shimmed to ensure the member stood vertically and attained even load transfer. A small load was placed on the wall/DCS, less than 2 kN, to secure it in place while transducers (LVDTs) were positioned. Since there was no track connected to the top or bottom of the DCS in tests 37 through 40 the fixity in the end conditions was greater compared to the rest of the tests in this series. Figure 3-4 depicts the typical test set-up for tests 1 through 36. Figure 3-5 shows the typical test set-up for test 37 through 40.

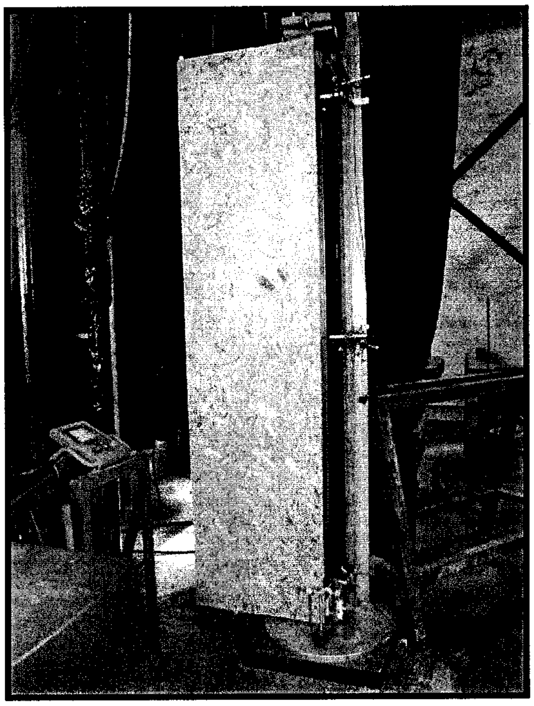


Figure 3-4: Typical set-up for tests 1-36

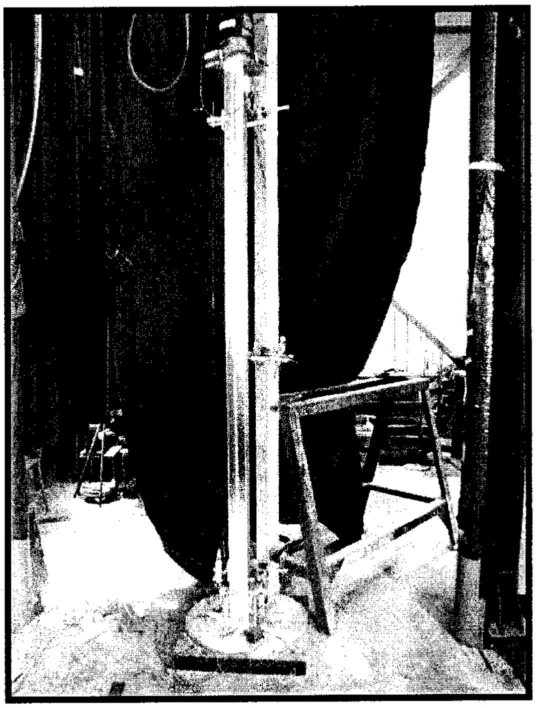


Figure 3-5: Typical set-up for tests 37-40

The recommended loading rate of 0.013 mm/sec from ASTM E72 (1998) for compression loads was used for specimens 1 to 36. The remaining unsheathed specimens were tested at a slower rate of 0.0066 mm/sec for safety reasons.

3.5 Instrumentation and Data Acquisition

To record the behaviour of the DCS during loading a total of 10 channels were used for data acquisition. Two channels were connected to the actuator recording the force and displacement. Eight LVDTs were placed in contact with the DCS. One LVDT was positioned vertically (Z-axis) on either side of the weak axis to record any changes in length or bending and the remaining six LVDTs were arranged to record the displacements in the X and Y-axes at the top, middle and bottom of the DCS. The LVDT at the bottom of the wall specimen responsible for recording changes in height on the exterior weak z-axis can be seen in Figure 3-6. Figure 3-7 shows the attachment for the pre-tensioned wire at top of the wall along the same axis. The placement of the LVDTs is depicted in Figure 3-3.

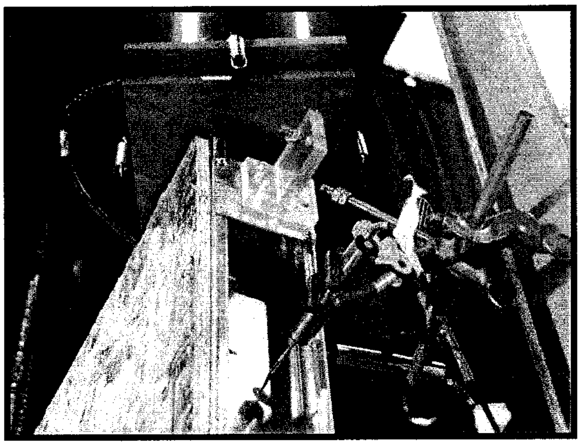


Figure 3-6 : Top of exterior z-axis of specimen with loading plate and LVDT

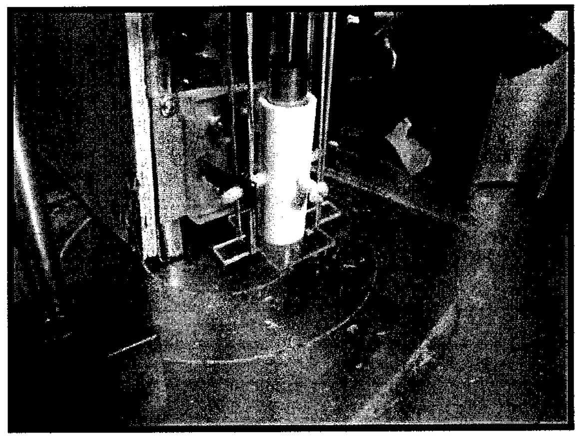


Figure 3-7 : Bottom of exterior z-axis of specimen with LVDT measuring axial displacement

All measuring devices were connected to Vishay Model 5100B scanners to record the data. The data acquisition system was operated using the Vishay System 5000 StrainSmart software.

After each DCS test was completed the sheathing was tested in accordance with APA Test Method P-6 (APA PRP-108, 2001) to determine its moisture content. Two specimens were cut using a 75 mm (3") diameter hole saw. The samples were weighed wet and then placed in drying oven at approximately 93.3°C (200°F) for 24 hours. Once oven-dry the specimens were weighed again and then the moisture content was determined.

3.6 Observed Failure Modes

A number of failure modes were observed during testing. The LVDTs in contact with the DCS helped predict and identify the failure modes, which can be classified into six major groups as follows:

1. Local buckling / crushing of the DCS at the top of the wall

This failure mode generally began with the local buckling of the chord stud lips in contact with the upper track (Figure 3-8). Local buckling of the flange and web followed within a 200 mm (8") region at the top of the wall, which led to a dramatic loss in capacity.

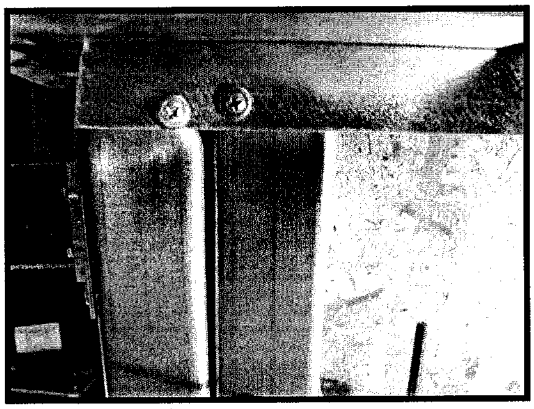


Figure 3-8 : Local buckling of DCS at top

2. Local buckling of diverging chord studs

The divergence of the chord studs in the DCS (Figure 3-9), between the fastener connections, led to local and distortional buckling. In this failure mode the individual chord studs buckled in a flexural / torsional mode between the web screw fastener connections. Local buckling was also observed at these locations. This failure mode was observed for test members with a web fastener spacing of both 305 mm and 610 mm.

3. Flexural-torsional failure at the perforations or failure due to small moment from an eccentric load

It is difficult to differentiate between these two possible failure modes due to their similarity in distorted shape and location (adjacent to the web cut-outs). An indicator of this failure mechanism was sometimes the local buckling of the web around the cut-outs that led to a visible gap between the webs of the two members. The local buckling of the stud flanges not connected to the wood sheathing was an indicator that this failure mode could also be attributed to the bending effect of an eccentric

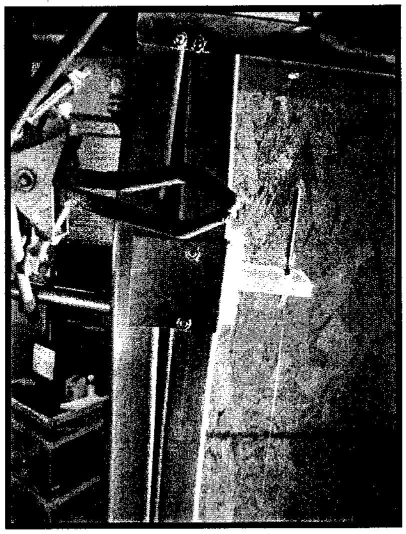


Figure 3-9 : Buckling of diverging chord studs

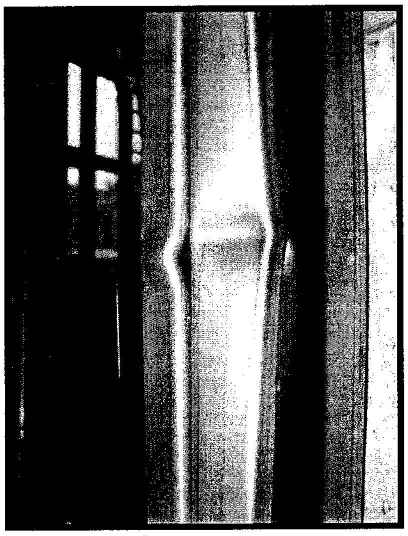


Figure 3-10: Flexural-torsional failure

load. The failure was most pronounced in the unrestrained flanges as shown in Figure 3-10. Distortional buckling along the web-flange axis and the flange-lip axis occurred as the failure progressed. In some cases a warping of the entire wall was noticeable following the twisting of the chord studs.

4. Initial buckling of a single chord stud at the top of the wall

Small variations in the lengths of the individual chord studs were common, which in some cases made it difficult to obtain even bearing on the built-up section. If the axial load was not able to distribute itself evenly over the area of the DCS, then it would concentrate in the higher of the two studs. In this case the single stud would compress until even with the second shorter stud, which would then begin to carry load as shown in Figure 3-11.

5. Moment induced failure

In some instances the DCS slipped a small amount in the initial stages of testing, i.e. the test specimen moved horizontally at its ends so that it was no longer being loaded at the geometric centroid. This movement induced a moment type failure that was

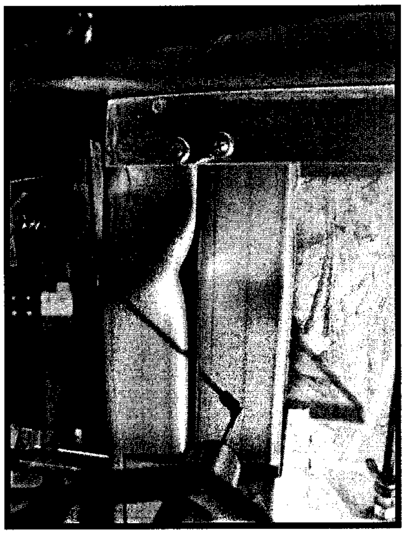


Figure 3-11 : Initial buckling of single chord stud

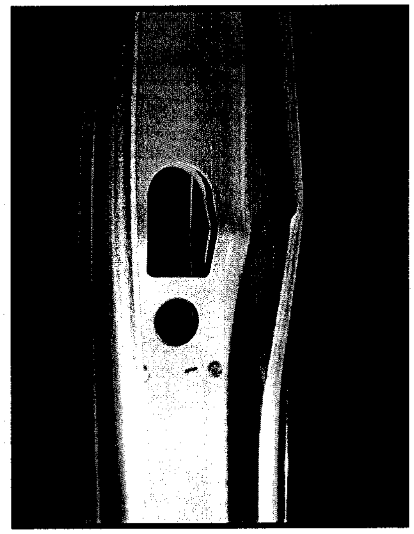


Figure 3-12 : Moment induced failure

most noticeable in the unsheathed double chord studs. Failure generally began with the local buckling of the lip elements on one side of the test member, which then progressed to distortional buckling (Figure 3-12). The drop in capacity was quite sudden when this failure mode was observed.

6. Failure of unsheathed studs

It was possible for the unsheathed stud to fail in a manner similar to that described in mode 5 without the presence of an eccentric load. Flexural buckling of the DCS took place in one direction (weak axis), which caused the compression stresses on one side of the member to be slightly higher than the other. The elements of the DCS that carried the higher stresses would fail first. Progressive damage from additional deformations applied by the actuator would then cause the member to bend. The final failure pattern was as shown in Figure 3-12.

3.7 General Test Results

The distribution of failure modes throughout the test series indicated that there was no distinct or systematic weakness of the DCS in the steel frame / wood panel shear wall system under gravity loads. The peak axial loads obtained from the 40 test specimens are presented in Table 3-2. Test damage sheets that contain a description of the failure modes for each test specimen are available for review in Appendix A. To illustrate the difference in behaviour of the sheathed and the bare DCS tests, Figures 3-13, 3-14, 3-15 and 3-16 were prepared for studs 0.84, 1.09, 1.37 and 1.72 mm in thickness, respectively. The load carrying capacity of the sheathed studs was always higher than that of the bare studs with the exception of the 1.09 mm thick test specimens. One reason the bare 1.09 mm thick test specimen was able to surpass the capacity of a sheathed member was the difference in the end conditions between the two test set-ups. The increased fixity of the bare DCS may have lowered the effective length factor, increasing the overall capacity. As well, this particular stud thickness composed 60% of the test program and were the first to be completed leading to an increased distribution.

In Figures 3-13 through 3-16, a graph of load vs. displacement for a sheathed and unsheathed DCS for each stud thickness, shows the bare DCS reaches its ultimate load at smaller deflections than the sheathed DCS. The extra deflection in the sheathed DCS was attributed to the flattening of the tracks and the overall compression of the connection between the track and stud. From visual inspection, once the loading has reached a stabilized response the stiffness of each system are almost parallel. In all cases shown the capacity of the sheathed DCS exceeds the unsheathed DCS while maintaining a linear stiffness.

Table 3-2 : Test results for chord stud compression tests

Test	Maximum Load (kN)	Test	Maximum Load (kN)
1. 043OSB1-12-3A	94.3	21. 043CSP1-12-3A	82.6
2. 043OSB1-12-3B	83.0	22. 043CSP1-12-3B	89.8
3. 043OSB2-12-3A	91.5	23. 043CSP2-12-3A	84.9
4. 043OSB2-12-3B	83.3	24. 043CSP2-12-3B	80.3
5. 043OSB1-24-3A	82.0	25. 043CSP1-12-6A	83.9
6. 043OSB1-24-3B	84.2	26. 043CSP1-12-6B	80.4
7. 043OSB2-24-3A	78.4	27. 043CSP2-12-6A	78.7
8. 043OSB2-24-3B	84.0	28. 043CSP2-12-6B	91.0
9. 043OSB1-12-6A[1]	80.8	29. 054OSB1-12-3A	125.0
10. 043OSB1-12-6B	74.0	30. 054OSB1-12-3B	125.0
11. 043OSB2-12-6A	78.1	31. 054CSP1-12-3A	119.0
12. 043OSB2-12-6B	77.2	32. 054CSP1-12-3B	114.3
13. 043OSB1-24-6A	85.8	33. 068OSB1-12-3A	173.0
14. 043OSB1-24-6B	70.4	34. 068OSB1-12-3B	179.2
15. 043OSB2-24-6A	85.6	35. 068CSP1-12-3A	172.3
16. 043OSB2-24-6B	85.7	36. 068CSP1-12-3B	183.0
17. 033OSB1-12-3A	60.3	37. 033DoubleChordStud	56.2
18. 033OSB1-12-3B	62.4	38. 043DoubleChordStud	78.8
19. 033OSB1-12-6A	62.6	39. 054DoubleChordStud	109.7
20. 033OSB1-12-6B	62.3	40. 068DoubleChordStud	146.0

[1] Two extra fastners in web-to-web connection at 19 mm (0.75") from the top

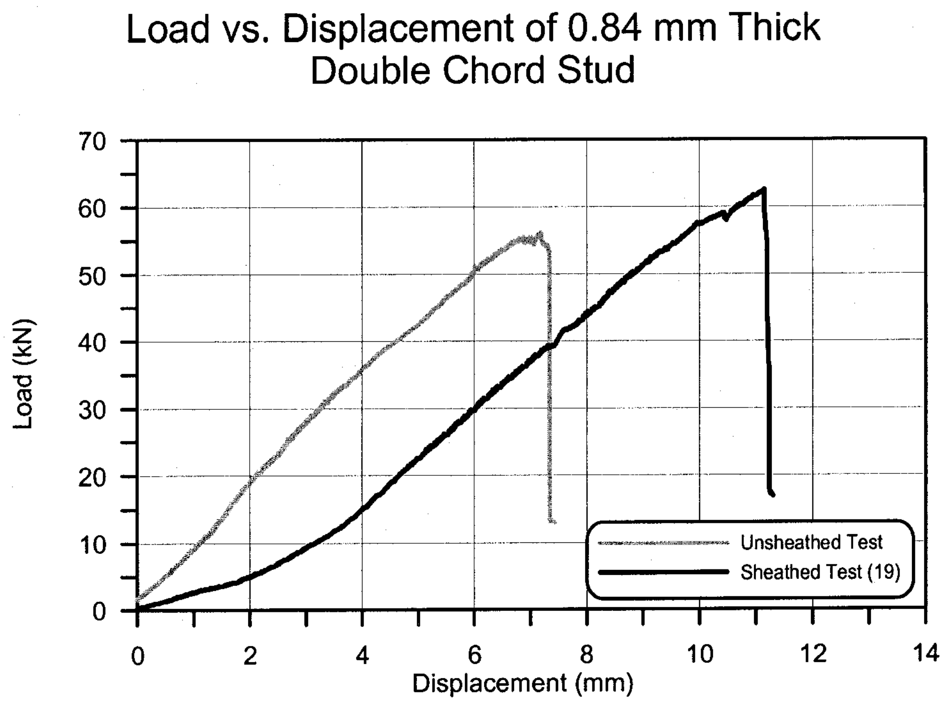


Figure 3-13 : Load vs. displacement of 0.84 mm thick double chord stud

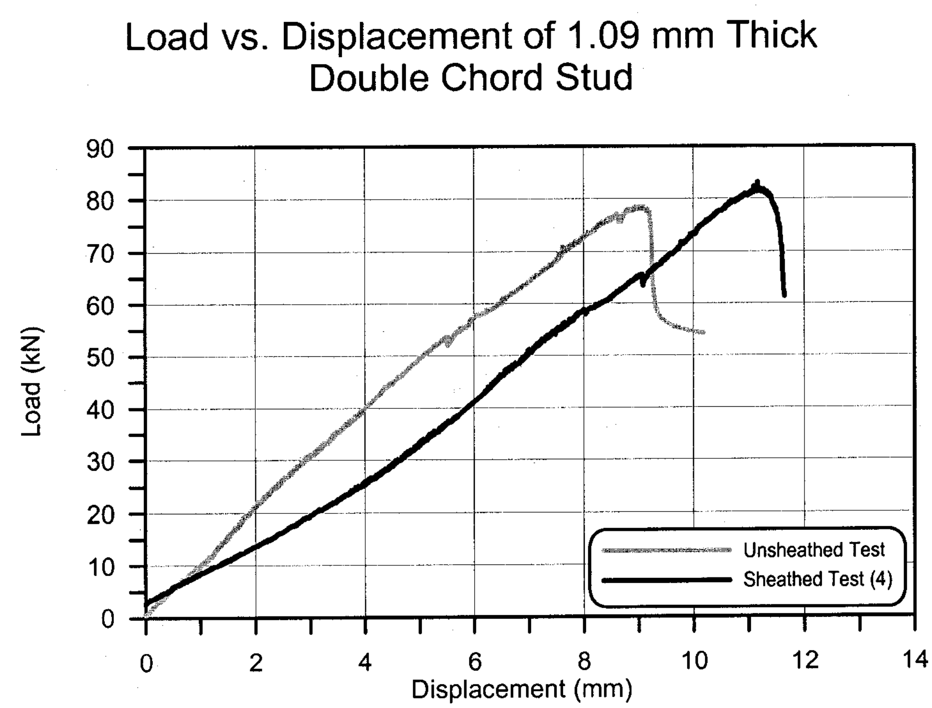


Figure 3-14 : Load vs. displacement of 1.09 mm thick double chord stud

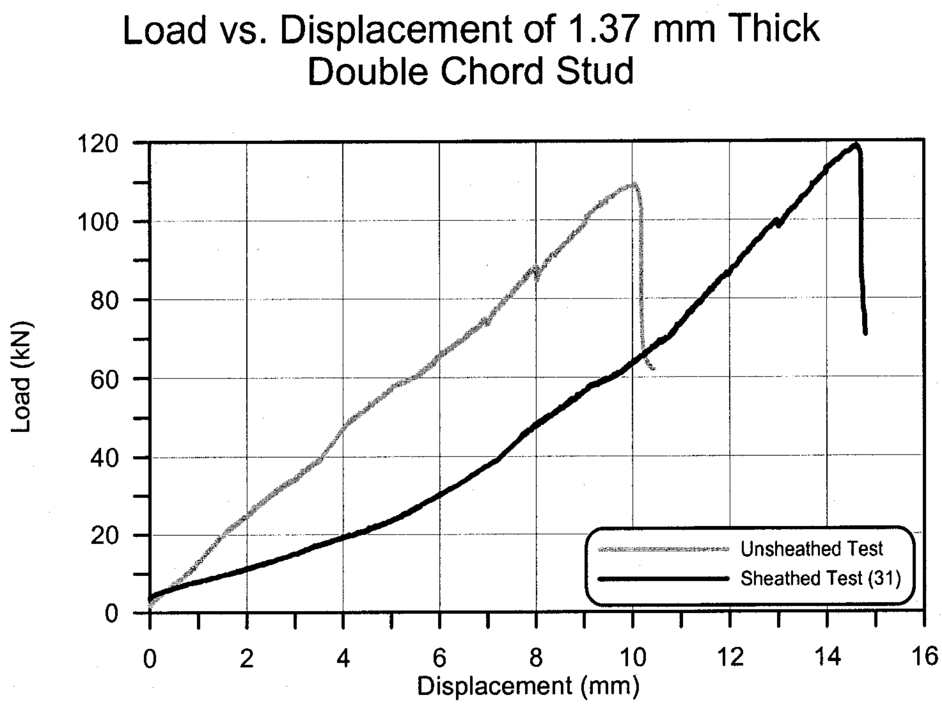


Figure 3-15 : Load vs. displacement of 1.37 mm thick double chord stud

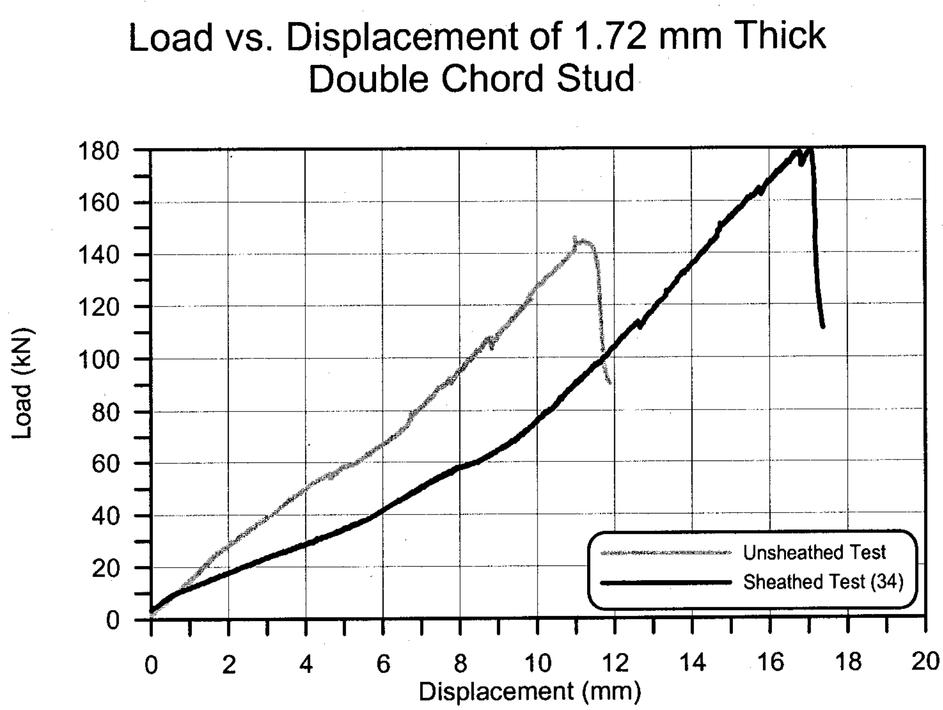


Figure 3-16 : Load vs. displacement of 1.72 mm thick double chord stud

It was suspected that the density of web screw fasteners in the DCS may influence the local buckling / crushing of the DCS at the top of the wall; for this reason two extra fasteners were added to Test 9 at 19 mm (0.75") from the top of the chord stud. The failure mode was similar to previous tests with no increase in capacity (Figure 3-17) indicating that an increase in fastener density at the end of the member does not improve load carrying abilities.

Figure 3-17 : Comparison of DCS local buckling with decreased web fastener spacing

Furthermore results for peak axial load showed that a decrease in the screw spacing in the web of the double chord stud from 610 to 305 mm did not consistently increase the axial capacity. Table 3-3 lists the maximum axial capacities obtained for two pairs of configurations where the screw spacing in the web was varied. The values for average capacity are somewhat misleading because they do not reflect the range and consistency of the results. For example Test 4's maximum axial capacity was below that of Test's 6 and 8 despite having twice the number of screws in the web.

At the beginning of the construction process it was thought that ideally the base of the chord studs should be aligned because when the combined gravity and lateral loads would be applied the highest compression forces would exist at the base of the wall. Initial chord stud tests showed that this thinking would not provide the desired response, where the two studs worked together as a built-up member. As previously mentioned, Figure 3-11 depicts how the taller stud failed before the built-up chord member had the opportunity to share the load. Test 2 listed in Table 3-3 was one of the tests to fail in this manner; its capacity was 11.3 kN less than Test 1 of the same configuration. The only

Table 3-3 : Comparison of capacities with change in web screw spacing in DCS

Test	Sheathing Thickness (mm)	Screw Spacing in DCS[1]	Maximum Load (kN)	Average Value
1. 043OSB1-12-3A	12.5 mm	305 mm	94.3	88.6
2. 043OSB1-12-3B	12.5 mm	305 mm	83.0	
3. 043OSB2-12-3A	9.5 mm	305 mm	91.5	87.4
4. 043OSB2-12-3B	9.5 mm	305 mm	83.3	
5. 043OSB1-24-3A	12.5 mm	610 mm	82.0	83.1
6. 043OSB1-24-3B	12.5 mm	610 mm	84.2	
7. 043OSB2-24-3A	9.5 mm	610 mm	78.4	81.2
8. 043OSB2-24-3B	9.5 mm	610 mm	84.0	

[1] Double Chord Stud (DCS)

other test that failed in this manner was Test 10 at a load of 74.0 kN. Test 9 of similar configuration had shims inserted between the top of chord studs and the top track to ensure even bearing during loading. As well, Test 9 had 2 extra screws inserted to the webs of the DCS about 19 mm (0.75") below the top to determine if an increased fastener density could prevent the local buckling/crushing of the DCS at the top of the wall. Test 9 failed at a load of 80.8 kN due to the local buckling/crushing of the DCS at the top of the wall indicating that an increase in fastener density does not necessarily deter failure in the region. The variation in results of these two pairs of tests indicates that the reduction of axial carrying capacity due to the varying heights of the chord studs in the DCS is highly variable, between 6.3 and 11.3 kN for this test program. Immediately after the shortcoming in performance of uneven chord studs at the top of the wall was identified all DCS were either shimmed or constructed with the top of the chord studs at matching heights. This change in construction allowed for the two stud members to more evenly share the applied compression loads, and hence failures of single studs were no longer observed. Table 3-4 lists the failure modes of Tests 1 through 8 to show the variety of modes and how no two tests of the same configuration failed in the same manner. The prevention of the initial buckling of a single chord stud at the top of the wall helped isolate failure modes to those concerning the DCS system, but did not necessarily improve load carrying capacities. Comparing the values from Table 3-3 with the modes of Table 3-4 for Test 4 and Test 5 shows that the shimming of the gap between the top of the DCS and track, nor changing the construction method to align the top of the studs provided a distinct improvement in load carrying capacity.

Table 3-4 : Summary of failure modes for Tests 1-8

Test	Failure Mode
1. 043OSB1-12-3A	• Flexural-torsional failure at the perforations (Mode 3)
2. 043OSB1-12-3B	• Initial buckling of a single chord stud at the top of the wall (Mode 4)
3. 043OSB2-12-3A	• Flexural-torsional failure at the perforations (Mode 3)
4. 043OSB2-12-3B	• Local buckling/crushing of the DCS at the top of the wall (Mode 1) • Top of chord studs height matched during construction
5. 043OSB1-24-3A	• Local buckling/crushing of the DCS at the top of the wall (Mode 1) • Top of the chord studs shimmed for even bearing during loading
6. 043OSB1-24-3B	• Flexural-torsional failure at the perforation (Mode 3)
7. 043OSB2-24-3A	• Flexural-torsional failure at the perforation (Mode 3)
8. 043OSB2-24-3B	• Local buckling of diverging chord studs (Mode 2)

There were two instances where secondary sources of damage, which were not typical among the test group, were observed. In Test 35 it was found that the shank of a No. 8 x 12.7 mm (1/2") long wafer head self-drilling framing screw connecting the 1.72 mm (0.068") double chord stud to the 1.09 mm (0.043") upper track had failed in shear and / or tension during testing (Figure 3-18). Wang et al. (2005) found that the design of the top track is critical in the redistribution of loads through the wall system. This failure indicates that for thicker chord stud sections it may be necessary to change the design of the track and connection to accommodate higher loads, for example a larger diameter framing screw may be required.

In Test 36, also constructed with 1.72 mm chord studs, there was some bearing and pull through damage from the sheathing to framing fastener at the base of the wall near the double chord stud (Figure 3-19). Through physical testing Wang et al. (2005) found that on similar walls 5-20% of the axial load would be redistributed through the panel depending on the loading pattern and fastener schedule. However, this load carrying ability of the sheathing cannot be easily accounted for in any design method. It should be noted that the ultimate failure modes for both Tests 35 and 36 was flexural-torsional failure about the web cut-out (Mode 3), similar to that shown in Figure 3-20.

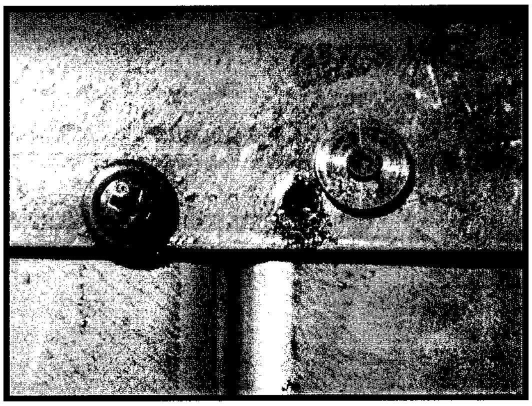


Figure 3-18 : Framing screw failed in shear

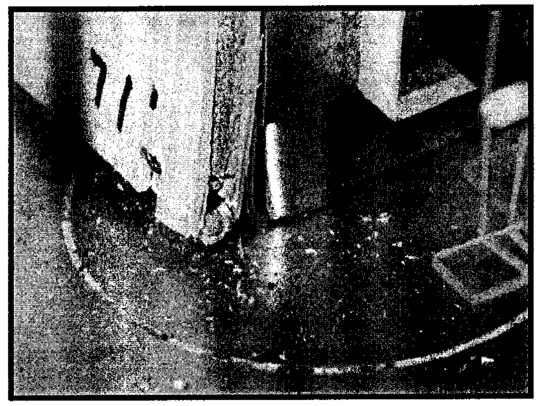


Figure 3-19 : Bearing / pull through damage of sheathing to frame connector

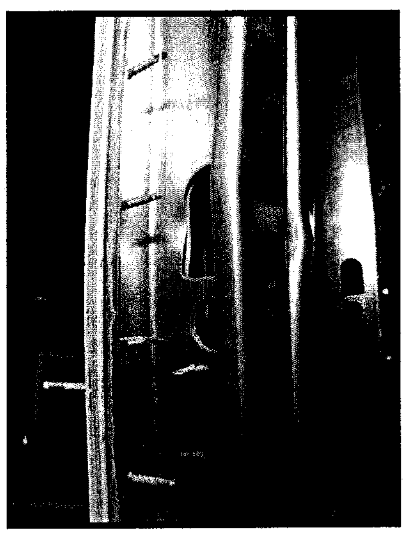


Figure 3-20 : Typical torsional/flexural failure mode of tests 35 and 36

3.8 Ancillary Material Testing

The material properties of the light gauge steel studs and tracks used in this test series were measured according to ASTM A370 (2002). Upon receipt of the materials they were immediately verified for the appropriate base metal thickness to ensure they were close to the nominal thickness. This was the only material evaluation completed before the chord stud specimens were tested. Coupon tests of each of the five types of steel framing were carried out after the chord stud tests had been completed.

Three replicate coupons for each of the five steel types were tested at a cross-head rate of 0.5 mm per minute in the elastic range, which was increased to a cross-head rate of 4 mm per minute in the plastic range. The cross-head movement was paused for 60 second intervals in the yield plateau to measure the static yield and ultimate stress of each steel coupon. After testing, the coupons were soaked in a 25% Hydrochloric Acid (HCl) solution to remove the zinc coating, which facilitated measurement of the base metal thickness. The yield stress (F_y), ultimate stress (F_u) and Young's modulus (E) were all determined using the base metal thickness (Table 3-5). The ratio of F_u to F_y and the percent elongation over a 50 mm gauge length are also provided.

Table 3-5 : Measured material properties of steel framing

Specimen	Member	Base Metal Thickness (mm)	Fy (MPa)	Fu (MPa)	Fu / Fy	E (GPa)	% Elong. 50 mm Gauge
0.84 mm	Stud	0.89	293	338	1.15	196	35.7
1.09 mm	Stud	1.12	246	321	1.30	220	32.4
1.37 mm	Stud	1.43	286	395	1.38	205	33
1.72 mm	Stud	1.81	299	395	1.32	215	35.5
1.09 mm	Track	1.12	254	321	1.26	196	30.7

The North American Specification for Cold-Formed Steel Members (CSA, 2004; AISI, 2001) requires that the ratio $F_u/F_y \geq 1.08$ and the elongation of a 50 mm gauge length be at least 10% for all members. All steel coupons exceeded these minimum requirements. However, the 1.37 mm (0.054") and 1.72 mm (0.068") steels were measured to have lower yield strengths than what would typically be expected, i.e. the minimum F_y should be 340 MPa (50 ksi), whereas the test results for the 1.37 mm and 1.72 mm studs were 286 MPa and 299 MPa, respectively. Therefore these studs were understrength in terms of expected material properties and could possibly carry higher loads if manufactured to the typical minimum specified F_y. As well, the modulus of elasticity for the stud section with 0.89 mm base metal thickness and the track section with a 1.12 mm base metal thickness were found to be slightly lower than the expected nominal value of 203 GPa.

3.9 Existing Method for Determining Axial Capacity for Design

To evaluate the axial carrying capacity of the DCS the current version of the S136 Standard was applied (CSA, 2001, 2004). The DCS, two chord studs connected back-to-back by their webs with screws, was treated as a built-up section. The capacities of the full section and the perforated section were both calculated. A number of different approaches were used for the effective length factors for the x, y and torsional axes as well as the buckling lengths for the y and torsional axes.

Chapter C Section 4.5 of the S136 Standard (CSA, 2004) allows for the increased capacity of a built-up section. To qualify as a built-up member the intermediate fastener spacing, a, shall be limited such that the ratio of a/r_i, where r_i is the radius of gyration of a single member, does not exceed half the governing slenderness ratio of the built-up member, the ends of the built-up member shall be connected by fasteners, such as screws, spaced longitudinally no more than 4 diameters apart for a distance equal to 1.5 times the maximum width of the member and each discrete connector shall be capable of transmitting a longitudinal shear force of 2.5% of the total force in the built-up member. Though the DCS specimens used in this study did not technically fall within this qualification, it was nonetheless applied. The design of the built-up section was not altered in the interest of maintaining the direct link between the existing database of information for the style of wall and this current body of research. The nominal capacities of the No. 10 screws in shear are 3.69 kN (for 0.84 mm sections), 5.36 kN (for 1.09 mm sections), 5.64 kN (for 1.37 mm sections) and 6.90 kN (for 1.72 mm sections) according to the manufacturer and should account for screw shear, bearing of the sheet steel and tilting of the fastener. The S136 (CSA, 2001) applies a resistance factor of $\phi =$ 0.4 to these values. With this consideration, if these walls were to be redesigned the fasteners should be enlarged and their pattern made denser to meet the criteria for built-up members. Section C4.5 of the CSA S136 (2004) also includes an approach for determining the axial compression capacity of a perforated member. The approach consists of treating the steel bordering the free edge of the web as an unsupported flange.

This is a conservative method because it does not account for the fixity provided by the web surrounding the perforation.

The use of appropriate effective length factors, representing the end conditions of the DCS, was integral in determining an accurate prediction method for the axial capacity. Previous research on the capacity of the light gauge steel studs and DCSs has indicated that the assumption of pinned end conditions for the effective length factor maybe incorrect. Table 3-6 summarizes conclusions drawn from past test programs in regards to effective length factors.

The S136 Standard (CSA, 2004) method for determining axial capacity of a built-up member was calculated for each configuration using an effective length factor of one, as well as, all the suggestions made by previous researchers (Table 3-7). Table 3-6 lists the effective length factors used in calculations with the modified slenderness ratio with the exception of Stone and LaBoube (2005) as noted. All calculations were carried out according to the S136 Standard (CSA, 2001, 2004) and verified using CFS 5.0 with the 2004 S136 (CFS, 2005).

To evaluate each of the prediction methods listed in Table: 3–7 for the axial compression capacity of the DCS within the steel frame / wood panel shear wall system two selection criteria were applied. The first criterion requires the predictions to have a reliability/safety factor, β_o, of at least 2.5 as recommended in the Commentary to the 2001 North American Cold-Formed Steel Specification (AISI, 2001). The second selection criteria evaluated the accuracy of the nominal axial capacity prediction with the experimental values.

Table 3-6 : Summary of effective length factors from previous test programs

Authors	Summary	Conclusions
Miller and Pěkoz (1993)	• Investigation of AISI Cold-Formed Specification methods 1986 applied to the design of wall studs through comparing observed behaviour • Stud dimensions (92 x 41 x 13 x 1.811 mm) (web x flange x lip x thickness) • Frame connected by self-drilling screws • Gypsum wallboard sheathed on both sides • Applied load at geometric centroid (gross section)	• $K_x = 0.65$, $K_y = 0.8$, $K_z = 0.8$ (wall stud assemblies) • $K_x = K_y = K_t = 0.65$ (unbraced wall stud assemblies) • Sensitive to loading eccentricities as small as 2.5 mm
Lee and Miller (2001)	• Stud dimensions (92 x 35 x 3.8 x 1.91 mm) (web x flange x lip x thickness) • No tracks used • Gypsum wallboard on both sides • Axial load applied to gross cross section of the stud • Tests showed that the sheathing increased stud strength by 1.7 times, but using the theoretical model developed it increased 1.9 times	• $K_x = 0.7$, $K_y = 0.5$, $K_t = 0.7$ • Ky = 1.0 over a length of 2s (s = fastener spacing) in the prediction of weak axis flexural buckling (with sheathing in place)
Telue and Mahendran (2001)	• Stud dimensions (75 x 30 x 1.15 mm) (web x flange x thickness) • Frame connected by an 8-18 gauge 12 mm long wafer head screw at each connection • Plasterboard lining fixed in the horizontal position joined together in accordance with the Plasterboard Manufacturers Installation Manual (CSR Plasterboard, 1990) • Load applied uniformly through top track	• $K_x = K_y = K_t = 1.0$ • $K_x = K_y = K_t = 0.75$ with a 2 mm eccentricity (resulting in better predictions)
Telue and Mahendran (2004)	• Stud dimensions (75 x 30 x 1.2 mm) (web x flange x thickness) • Frame connected by an 8-18 gauge x 12 mm long wafer head screw • Plasterboard lining on both sides • Loaded at geometric centroid of stud	• See Fig. 3-13 Effective length factor for out-of-plane major axis flexural buckling versus flexural rigidity ratio (from Telue and Mahendran, 2002) Figure 2-1 • $K_y = K_t = nS_f/L$ (where n = fastener spacing factor = 1.0)
Stone and LaBoube (2005)	• Stud dimensions (92.08 x 41.28 x 9.53 x 1.155 mm) and (92.08 x 40.46 x 9.53 x 0.88 mm) (web x flange x lip x thickness) • Studs attached back-to-back with pairs of screws at intervals of 305 mm, 610 mm and 914 mm • Track connected with one screw in each flange • No sheathing • Loaded axially with pin ended connections within apparatus	• $K_x = 1.0$ • $\left(\frac{KL}{r}\right)_m = \sqrt{\left(\left(\frac{kL}{r}\right)_o^2 + \left(\frac{a}{r_i}\right)^2\right)}$ only necessary for materials with a thickness ≤0.89 mm

Table 3-7 : Effective length factors utilized in capacity calculations

	K_x	L_x (mm)	K_y	L_y (mm)	K_t	L_t (mm)	Notes
1.	1.0	2440	1.0	$2s$[1]	1.0	$2s$	• Pinned end conditions assumed • Weak and torsional axis buckling length based on twice the maximum screw spacing along the panel edge
2.	0.9	2440	0.9	s	0.65	s	• Partial fixity of strong and weak axis and fixed end conditions for torsional axes • Strong axis buckling length equal to full height of the wall • Weak and torsional buckling length equal to screw spacing in panel edge
3.	0.80	2440	0.8	s	0.65	s	• Partial fixity of strong and weak axis and fixed end conditions for torsional axes • Strong axis buckling length equal to full height of the wall • Weak and torsional buckling length equal to screw spacing in panel edge
4.	0.65	2440	0.65	2440	0.65	2440	• Effective length factors suggested by Miller and Pekoz (1993) • Buckling lengths equal to full height of members
5.	0.7	2440	0.5	2440	0.7	2440	• Effective length factors suggested by Lee and Miller (2001) • Buckling lengths equal to full height of members
6.	0.75	2440	0.75	2440	0.75	2440	• Effective length factors from Telue & Mehendran (2001) • Buckling lengths equal to full height of members
7.	1.0	2440	1.0	$2s$	1.0	$2s$	• Effective length factors from Telue & Mehendran (2004) • Strong axis buckling length equal to full height of members • Weak and torsional axis buckling lengths equal to twice the screw spacing along the panel edge
8.[2]	0.9	2440	0.9	$2s$	0.65	$2s$	• Effective length factors and buckling lengths from Method 2 applied • Stone and Laboube's (2005) recommendation for the limited application of the modified slenderness ratio to members 0.89 mm in thickness or less
9.[2]	0.8	2440	0.8	$2s$	0.65	$2s$	• Effective length factors and buckling lengths from Method 3 applied • Stone and Laboube's (2005) recommendation for the limited application of the modified slenderness ratio to members 0.89 mm in thickness or less

[1] Where s = fastener spacing at edge of the panel
[2] Modified slenderness ratio $(KL/r)_m$ only applicable to members $\leq$ 0.89 mm.

The reliability/safety factor, β_o, was calculated using Equation 3-1. The recommended value for the calibration coefficient, C_ϕ, of 1.42 was used from the Commentary to the

S136 NAS (AISI, 2001). This source was also used for the statistical data of M_m, V_M, F_m and V_F corresponding to wall studs in compression, which were 1.10, 0.10, 1.00 and 0.05, respectively. For the design of columns the resistance factor, ϕ, is 0.8 (CSA, 2004). The mean value of professional factor for tested components, P_m, and the coefficient of variation of the professional factor, V_P, were taken from the statistical analysis of the experimental values normalized with the predicted capacities. Chapter F "Tests for Special Cases" of the Commentary to the S136 NAS (AISI, 2001) recommends that the coefficient of variation of the professional factor, V_P, be at least 0.065 for regular cold-formed steel components. For the statistical analysis the normalized values were first separated by chord stud thickness to determine the reliability/safety factor and then a combined value was determined for the entire sheathed DCS data set.

The second criterion was the evaluation of the accuracy of the axial capacity prediction. The purpose of this test program was to develop a better understanding of the load carrying capacity of the DCS member to improve the efficiency of the overall design of the steel frame / wood panel wall for seismic design. For an efficient and cost-effective design it is helpful to know the capacity of elements within the system as accurately as possible, thus the prediction methods listed in Table : 3-7 based on the equations presented in Section 2.3.1 were reviewed for how closely they reflected the experimental results collected in this study.

$$\beta_o = -\left[\ln\left(\frac{\phi}{C_\phi(M_m F_m P_m)}\right) \Big/ \left(\sqrt{V_M^{\ 2} + V_F^{\ 2} + C_p V_P^{\ 2} + V_S^{\ 2}}\right)\right] \tag{3-1}$$

where,

ϕ = Resistance factor (= *0.8 for columns*)

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

β_o = Reliability/safety index

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 $n = 3$

V_S = Coefficient of variation of the load effect

m = Degrees of freedom = n – 1

n = number of tests

ln = Natural logarithm

For both of the selection criteria mentioned above the maximum measured loads of the experimental sections were compared to the predictions for a full built-up section. This approach was chosen for three reasons; first the DCS did not consistently fail at the perforated section. While the predicted capacity of the perforated section always controlled, all of the loads at failure experienced during testing tended towards the full section capacity. This trend indicates that the perforated section capacity underestimates the true capacity of the section, possibly due to the disregard of the fixity provided by the surrounding web as previously mentioned. Thirdly, the same manufacturer was used for all sections tested and therefore only one perforation pattern was evaluated. Since there is variability in the performance of the section related to the size and shape of the perforation (Pu et al., 1999) it was decided that the full section prediction would be used for calibration.

A statistical comparison between the experimental load of the sheathed DCS normalized by the predicted capacity according to Method 2, deemed most appropriate in the following discussion, for the nominal and measured sections sizes and material properties is presented in Table 3-8. The statistical comparison for the remaining methods are located in Appendix G.

Table 3-8 : Comparison of predicted ultimate load based on nominal and actual material properties using Method 2 of Table 3-7 with experimental loads

Test	Ultimate load (kN)			Expt/Pred(Nominal)	Expt/Pred(Measured)
	Predicted		Experimental		
	Nominal Properties	Measured Properties			
1. 043OSB1-12-3A	67.1	74.3	94.3	1.41	1.27
2. 043OSB1-12-3B	67.1	74.3	83.0	1.24	1.12
3. 043OSB2-12-3A	67.1	74.3	91.5	1.36	1.23
4. 043OSB2-12-3B	67.1	74.3	83.3	1.24	1.12
5. 043OSB1-24-3A	67.1	74.3	82.0	1.22	1.10
6. 043OSB1-24-3B	67.1	74.3	84.2	1.26	1.13
7. 043OSB2-24-3A	67.1	74.3	78.4	1.17	1.05
8. 043OSB2-24-3B	67.1	74.3	84.0	1.25	1.13
9. 043OSB1-12-6A[1]	67.1	74.3	80.8	1.20	1.09
10. 043OSB1-12-6B	67.1	74.3	74.0	1.10	1.00
11. 043OSB2-12-6A	67.1	74.3	78.1	1.16	1.05
12. 043OSB2-12-6B	67.1	74.3	77.2	1.15	1.04
13. 043OSB1-24-6A	67.1	74.3	85.8	1.28	1.15
14. 043OSB1-24-6B	67.1	74.3	70.4	1.05	0.95
15. 043OSB2-24-6A	67.1	74.3	85.6	1.28	1.15
16. 043OSB2-24-6B	67.1	74.3	85.7	1.28	1.15
17. 033OSB1-12-3A	47.1	57.6	60.3	1.28	1.05
18. 033OSB1-12-3B	47.1	57.6	62.4	1.33	1.08
19. 033OSB1-12-6A	47.1	57.6	62.6	1.33	1.09
20. 033OSB1-12-6B	47.1	57.6	62.3	1.32	1.08
21. 043CSP1-12-3A	67.1	74.3	82.6	1.23	1.11
22. 043CSP1-12-3B	67.1	74.3	89.8	1.34	1.21
23. 043CSP2-12-3A	67.1	74.3	84.9	1.27	1.14
24. 043CSP2-12-3B	67.1	74.3	80.3	1.20	1.08
25. 043CSP1-12-6A	67.1	74.3	83.9	1.25	1.13
26. 043CSP1-12-6B	67.1	74.3	80.4	1.20	1.08
27. 043CSP2-12-6A	67.1	74.3	78.7	1.17	1.06
28. 043CSP2-12-6B	67.1	74.3	91.0	1.36	1.22
29. 054OSB1-12-3A	115.2	109.0	125.0	1.08	1.15
30. 054OSB1-12-3B	115.2	109.0	125.0	1.08	1.15
31. 054CSP1-12-3A	115.2	109.0	119.0	1.03	1.09
32. 054CSP1-12-3B	115.2	109.0	114.3	0.99	1.05
33. 068OSB1-12-3A	152.8	152.0	173.0	1.13	1.14
34. 068OSB1-12-3B	152.8	152.0	179.2	1.17	1.18
35. 068CSP1-12-3A	152.8	152.0	172.3	1.13	1.13
36. 068CSP1-12-3B	152.8	152.0	183.0	1.20	1.20
			Average	1.22	1.12
			Standard Deviation	0.099	0.0662
			CoV	0.0811	0.0593

[1] Two extra fastners in web-to-web connection at 19 mm (0.75") from the top

The minimum coefficient of variation for the professional factor recommended by the S136 (AISI, 2001) was frequently higher than the values calculated for each normalized data set as illustrated in Table 3-8 for the Expt/Pred(Measured) column. All the calculations for the safety/reliability factor were performed using both the coefficient of variation for the professional factor directly from the data set and the recommended $V_P \geq 0.065$ when necessary.

Both the nominal properties and the measured properties from the sections tested were used to develop the recommended prediction method. From Table 3-8 the mean value of professional factor for tested components, P_M, and the coefficient of variation of the professional factor, V_P, were used to calculate the overall reliability/safety factor, β_o, as presented in Table 3-9. The reliability/safety factors for each section thickness were also calculated with $V_P \geq 0.065$ to ensure that they each, individually, met the $\beta_o \geq 2.5$ requirement. The reliability/safety factors calculated using the measured section properties for Method 2with $V_P \geq 0.065$ are all above 2.5 as well, supporting the nominal results.

Table 3-9 : Reliability/Safety Factor, β_o, for prediction Method 2 (nominal section, $V_P \geq 0.065$)

Stud Thickness	α	$\overline{Q}/Q$	C_Φ	Φ	M_m	F_m	P_m	V_m	V_f	V_s	n	C_p	V_p	β_o
0.84	1.5	0.76	1.42	0.8	1.1	1	1.31	0.1	0.1	0.21	4	3.75	0.07	3.32
1.09	1.5	0.76	1.42	0.8	1.1	1	1.24	0.1	0.1	0.21	24	1.14	0.07	3.36
1.32	1.5	0.76	1.42	0.8	1.1	1	1.05	0.1	0.1	0.21	4	3.75	0.07	2.54
1.57	1.5	0.76	1.42	0.8	1.1	1	1.16	0.1	0.1	0.21	4	3.75	0.07	2.89
AVG	1.5	0.76	1.42	0.8	1.1	1	1.22	0.1	0.1	0.21	36	1.09	0.07	3.31

Tables of the experimental values normalized by each prediction method and the subsequent table for the calculation of the reliability/safety factors for prediction methods 1 through 9 are located in Appendix G. Overall, the sections met and/or exceeded the requirements for the reliability/safety factor, β_o, corresponding to the resistance factor, $\phi = 0.8$, for columns regardless of prediction method, with the exception of three cases which are discussed later in the chapter. A summary of the reliability/safety factor using the nominal sections for each prediction method is listed in Table 3-10.

Table 3-10 : Summary of average Reliability/Safety Factors (nominal sections, $V_P \geq 0.065$)

Method	α	$\overline{Q}/Q$	C_Φ	Φ	M_m	F_m	P_m	V_m	V_f	V_s	n	C_p	V_p	β_o
1	1.5	0.76	1.42	0.8	1.1	1	1.26	0.1	0.1	0.21	36	1.09	0.08	3.38
2	1.5	0.76	1.42	0.8	1.1	1	1.22	0.1	0.1	0.21	36	1.09	0.07	3.31
3	1.5	0.76	1.42	0.8	1.1	1	1.15	0.1	0.1	0.21	36	1.09	0.09	2.98
4	1.5	0.76	1.42	0.8	1.1	1	1.35	0.1	0.1	0.21	36	1.09	0.07	3.68
5	1.5	0.76	1.42	0.8	1.1	1	1.21	0.1	0.1	0.21	36	1.09	0.08	3.22
6	1.5	0.76	1.42	0.8	1.1	1	1.47	0.1	0.1	0.21	36	1.09	0.07	4.02
7	1.5	0.76	1.42	0.8	1.1	1	1.26	0.1	0.1	0.21	36	1.09	0.08	3.39
8	1.5	0.76	1.42	0.8	1.1	1	1.20	0.1	0.1	0.21	36	1.09	0.09	3.15
9	1.5	0.76	1.42	0.8	1.1	1	1.16	0.1	0.1	0.21	36	1.09	0.09	3.03

Table 3-11 contains a comparison of the reliability/safety Factors based on nominal and actual sections. The values for β_o derived from the measured section properties were on average 0.31 less than those calculated using the nominal sections. Their mean value of professional factor for tested components, P_m, and the coefficient of variation of the professional factor, V_P, were lower than those based on nominal sections. This shows that the use of the actual cross section properties resulted in a more exact, less conservative prediction for the axial capacity. All methods of prediction, with the exception of the 1.32 mm thick section normalized using the prediction Methods 3, 8 and 9, had an overall $\beta_o \geq$ 2.5, indicating that they could all be considered acceptable methods of evaluation. Method 8 and 9 applied the recommendation that the modified slenderness ratio, $(KL/r)_m$, for built-up sections only be used for members with a section thickness of less than or equal to 0.89 mm. The modified slenderness ratio result is slightly higher than the normal slenderness ratio and in turn decreases the predicted values. The 1.32 mm thick nominal sections normalized using the prediction Methods 8 and 9 had reliability/safety factors less than 2.5. The predicted capacities not applying the modified slenderness ratio were as much as 9.3 kN higher than the experimental capacities.

Similarly, for Method 3 a partial fixity of $K_{x,y} = 0.8$ was assumed for the strong and weak axes and design level fixity, $K_t = 0.65$, in the torsional axis was used. The buckling lengths of the weak and torsional axes were reduced from the full height of the wall to reflect the sheathing screw spacing distance, while the strong axis buckling length remained equal to the height of the wall. The slenderness ratio of the strong axis controlled the prediction of the axial capacity of the member. The results using the nominal sections were higher than the experimental values for Tests 31 and 32 (section thickness 1.32 mm) (Table G3). However, unlike Methods 8 and 9, Method 3 had an acceptable reliability factor using the coefficient of variation derived from the test data, but fell short of 2.5 when the minimum $V_P = 0.065$ was imposed. It should be noted that the experimental results from this particular thickness, 1.32 mm, consistently had the lowest β_o for all prediction methods.

Table 3-11 : Comparison of Reliability/Safety Factors calculated with nominal and actual properties ($V_P \geq 0.065$)

Method	β_o (Nominal)	β_o (Measured)
1	3.38	3.12
2	3.31	2.99
4	3.68	3.34
5	3.22	2.95
6	4.02	3.69
7	3.39	3.12
8	3.15	2.84
9	3.03	2.70

The results of comparing the test-to-predicted were also used to evaluate the suitability of each prediction method. Table 3-12 lists the average test-to-predicted values for the 9 design methods (as given in Table 3-8) for the nominal and measured sections.

The predictions from Method 1 were conservative because the assumption of pinned end conditions did not reflect the supports provided during testing. Method 2 used a partial fixity of $K_{x,y} = 0.9$ for the strong and weak axes and design level fixity equal to $K_t = 0.65$ for the torsional axis. The strong axis controlled the axial capacity prediction with a buckling length equal to the full height of the wall. The predictions for this method were the most accurate without being unconservative and met the requirements for the reliability/safety factor for all sections in all situations. Method 3 applied partial fixity $K_{x,y} = 0.8$ to the strong and weak axis of the member and full fixity in the torsional axis. The buckling lengths for the weak and torsional axis were reduced to 152 mm to reflect the maximum screw spacing connecting the sheathing and stud. The predicted values were close to the experimental results, but with the requirement for the coefficient of

Table 3-12 : Comments of the results of methods of prediction

Method	Average Expt/Pred (Nominal)	Average Expt/Pred (Measured)	Comments
1.	1.26	1.16	• Predictions for nominal sections 68% to 95% of experimental loads • Predictions for actual sections 76% to 102% of experimental • Assumption of pinned end conditions conservative
2.	1.22	1.12	• Predictions for nominal sections 71% to 101% • Predictions for actual sections 79% to 106% • Only one normalized below unity at 0.99 (Test 32 with1.32 mm thick sections)
3.	1.17	1.08	• Predictions for nominal sections 73% to 106% • Predictions for actual sections 81% to 109% • Only two normalized values below unity (Tests 31 and 32) • Reliability/safety factor less than 2.5 for 1.32 mm sections when $V_P \geq 0.065$
4.	1.35	1.23	• Predictions for nominal sections 65% to 87% • Predictions for actual sections 72% to 96% • Tests exceed predicted values by up to 55%
5.	1.21	1.11	• Predictions for nominal sections 71% to 101% • Predictions for actual sections 79% to 106% • Only one normalized prediction for nominal sections is less than one at 99% (Test 32 with 1.32 mm thick sections)
6.	1.47	1.35	• Predictions for nominal sections 60% to 80% • Predictions for actual sections 66% to 89% • Experimental values at least 25% higher than predicted capacities • Predictions for 14 out of 36 test either equal to or exceeded by 50% during testing
7.	1.26	1.16	• Predictions for nominal sections 69% to 96% • Predictions for actual sections 76% to 102% • Nominal sections underestimated by at least 5% and up to 45%
8.	1.20	1.10	• Predictions for nominal sections 73% to 103% • Predictions for actual sections 80% to 108% • Normalized predictions for Test 32 with nominal 1.32 mm thick sections falls below unity to 0.97 • The Relability/Safety factor below 2.5 for the nominal 1.32 mm thick section
9.	1.16	1.06	• Predictions for nominal sections 75% to 108% • Predictions for actual sections 83% to 111% • Normalized predictions for Tests 31 and 32 fall below unity to 0.97 • The Relability/Safety factor below 2.5 for the nominal 1.32 mm thick section

variation of the professional factor $V_P \geq 0.065$ did not meet the reliability/safety factor for the 1.32 mm thick sections. Method 4 used effective length factors reflecting a design level of fixity in all axes with buckling lengths equal to the full height of member in all cases. This approach predicted that failure would occur in the weak, *y*, axis and generally underrated the capacity of the section by 15% to 55%. Method 5 employed buckling lengths equal to the full height of the member for all axes. An effective length factor reflecting partial fixity $K_x = K_t = 0.7$ was assigned to the strong and torsional axes, in the weak axis the end conditions was represented as having the theoretical level of fixity $K_y = 0.5$. The results were relatively accurate and very similar to those found for Method 3, but the predicted values were controlled by the weak axis. Method 6 was very similar to Method 4 except that the end conditions allow for more rotation with $K_x = K_y = K_t = 0.75$. This approach underestimates the axial capacity of the members even further than Method 4 with the predicted values being less than 80% of the experimental results. Method 7 again applied pinned end conditions, but reduced the buckling length of the member in the weak (*y*) and torsional axes to twice the screw spacing connecting the DCS to the sheathing. Since the slenderness ratio of the x-axis governed the axial capacity calculation the results for this method were identical to those in Method 1. This method, 7, was intended to reflect recommendations made by Telue and Mehendran (2004) that recommended a change in the effective length factor relative to the ratio of flexural rigidity between the track and stud. However the relative ratio of track to stud in the configurations tested did not warrant an improvement in fixity for the strong axis. Method 8 was a variation on Method 2 employing the same end conditions, but doubled the buckling lengths in the weak and torsional axes and overlooked the modified slenderness ratio recommended for built-up sections for those greater than or equal to 0.89 mm thick. The results for this method were accurate, but slightly unconservative for the 1.32 mm section such that it did not meet the $\beta_o \geq 2.5$ requirement. Method 9 applied the same end conditions as Method 3, but used a buckling length equal to twice the maximum fastener spacing attaching the sheathing to the DCS in case of variability during construction. Similarly to Method 8, Method 9 also used the recommendation that the modified slenderness ratio, $(KL/r)_m$, only be applied to built-up sections less than or equal to 0.89 mm thick from Stone and LaBoube (2005). This approach did not agree with the findings from this study since the capacity of the thicker, 1.32 mm thick sections had the lowest experimental capacities relative to the predicted capacities using the S136 approach (CSA, 2001, 2004).

Methods 2 and 5 for predicting the axial capacity of DCS for were both quite accurate. As discussed earlier, the failure modes of the sheathed chord studs were unpredictable and varied in location. In Method 2 the strong (x) axis controls the capacity prediction with a fixity of $K_x = 0.9$ reflecting limited rotation of the DCS. The y-axis could be considered as having the same fixity, but with the shorter buckling length due to the connectors in the sheathing acting as braces it does not control. The predictions by Method 5 were controlled by the weak (y) axis which had the full theoretical restraint of $K_y = 0.5$ and a buckling length equal to the full height of the wall. The predictions of the Methods 2 and 5 do not vary more than 0.6 kN between one another for each section as shown in Table 3-13.

Table 3-13 : Summary of test results and prediction methods 2 and 5

Section Thickness (mm)	Avg. Expt. Sheathed Capacity (kN)	Method 2 Prediction (kN)	Method 5 Prediction (kN)
0.84	61.9	47.1	47.2
1.09	82.9	67.1	67.3
1.32	120.8	115.2	115.8
1.57	176.9	152.8	153.3

Though numerically Method 5 predicts the axial capacity of the DCS well, the behaviour the calculations reflect is not feasible since from testing it was shown that the DCS was able to undergo small rotations/movements due to the gap that existed between the track and studs.

The capacities and failure modes of the unsheathed sections are summarized in Table 3-14. The test set-up for the unsheathed series allowed for less rotation at the stud ends because they were not connected to tracks; rather they bore directly on the loading plate. These specimens also did not have the benefit of the support of the sheathing along their length. Thus, they should reflect capacities of members with higher fixity in their end conditions than the sheathed members and buckling lengths equal to the full height of the wall for all axes. The failure modes were not consistent, the 0.84 mm and 1.32 mm sections the failed in weak axis buckling about the perforation, which Method 5, represents and the 1.72 mm section buckled in the strong axis with the collapse of the flanges indicating interaction with torsion forces. The inconsistency of the failure modes for the unsheathed DCS indicates that capacities controlling the various failure modes are probably very similar. For the case of the unsheathed DCS tested while bearing directly on the loading plate the prediction Method 5 could be considered appropriate since weak axis buckling was dominant over the entire height of the member. However, for the sheathed DCS Method 2, which reflects partial fixity in both the strong and weak axis as well as the shortened buckling length in the weak axis due to the support provided by the sheathing, represents the observed behaviour of these particular members more accurately and is better suited for design applications.

Table 3-14 : Summary of failure modes for unsheathed DCS Test 37- 40

Unsheathed Test	Experimental Axial Capacity (kN)	Failure Mode
Test 37 033Double Chord Stud	56.2	• Moment failure about weak axis at a perforation • Rotation about weak axis at base • Flanges unstiffened by hold down at the base buckle • Sections move together
Test 38 043Double Chord Stud	78.8	• Failure at perforation • Buckling in the strong axis on compression side • Flange deformation indicates torsion
Test 39 054 Double Chord Stud	109.7	• Moment failure about weak axis at a perforation • Sections move apart from one another at failure
Test 40 068 Double Chord Stud	146.0	• Weak axis failure at the centre of the member (not perforation)

Figure 3-21 illustrates the variation in predicted axial capacities for nominal properties, measured properties and how they compare to the average capacities of the sheathed tests and unsheathed test for each thickness. It should be noted that the recommended design Method 2 to is only intended for predicting the axial capacity of sheathed DCS.

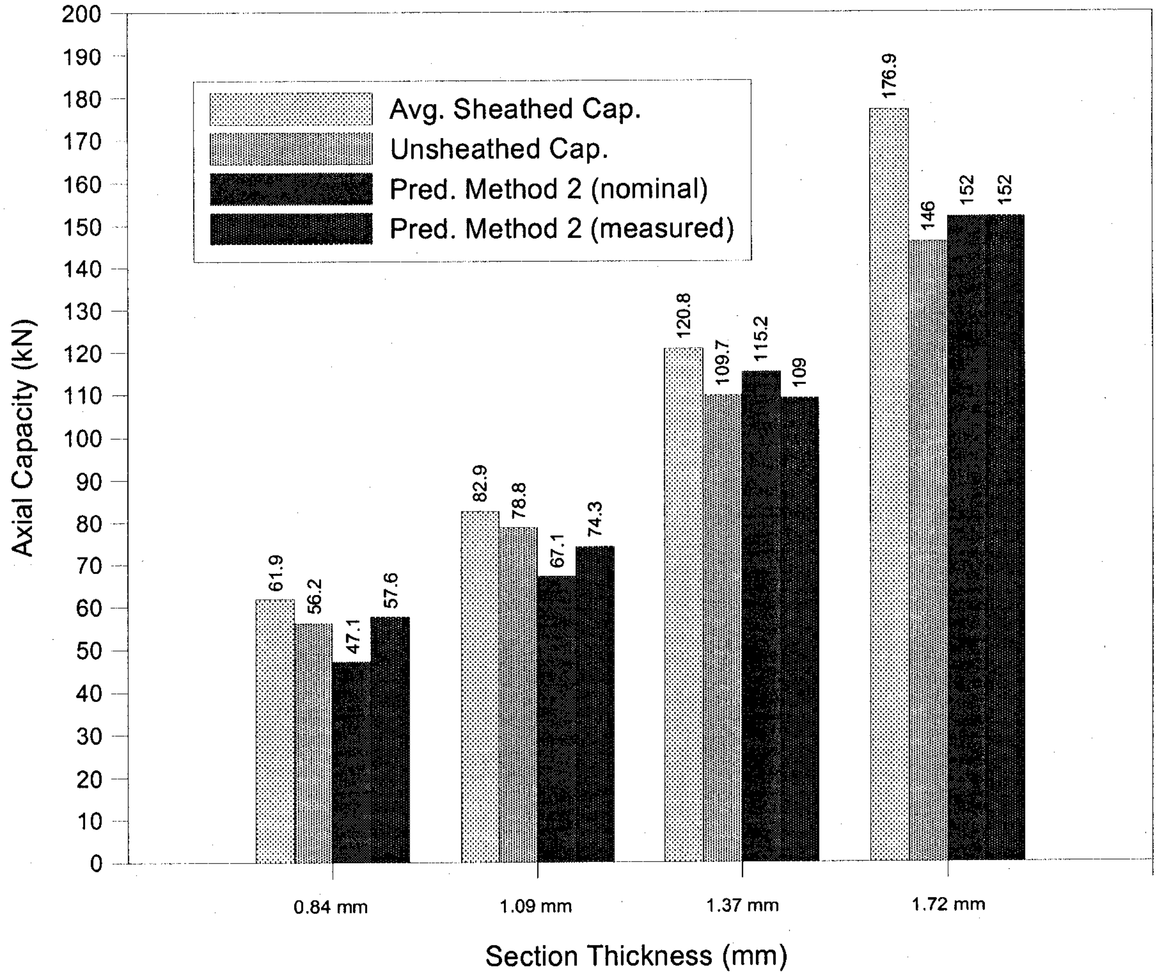


Figure 3-21 : Comparison of actual and nominal axial capacity with average test results

Another aspect of this test program was to identify the influences of various details of each wall configuration including sheathing type, sheathgin thickness, screw spacing in the field and around the edge. Table 3-15 lists the average capacities of 1.09 mm DCS according to these details. There was little variation in the results or evidence that these particular properties were responsible for any change in the trend of the axial capacity.

Table 3-15 : Average axial capacities of DCS based on configuration details for 1.09 mm sections

Variable	CSP Sheathing		OSB Sheathing		CSP / OSB Mixed Sheathing			
Average Capacity (kN)	84.0		82.4					
Variable	12.5 mm thick	9.5 mm thick	12.5 mm thick	9.5 mm thick	305 mm Screw Spacing in Field	610 mm Screw Spacing in Field	Edge Screw Spacing 75 mm	Edge Screw Spacing 150 mm
Average Capacity (kN)	84.7	83.7	81.8	83.0	83.4	82.0	84.9	81.0

This test series limited the sheathing to frame fastener schedule to a maximum edge screw spacing of 152 mm (6"). Therefore this recommended design method should not be applied for walls with larger screw spacings outside the scope of testing. For the recommend method of prediction, 2, listed in Table 3-16 the buckling length of the weak and torsional axes should be increased to twice the screw spacing to account for errors in construction. To verify the applicability of this method beyond the scope of testing presented in this report further physical testing should be conducted.

Table 3-16 : Recommended method of axial capacity prediction for sheathed DCS

	K_x	L_x (mm)	K_y	L_y (mm)	K_t	L_t (mm)	Notes
2.	0.9	2440	0.9	*2s*	0.65	*2s*	• Partial fixity of strong and weak axis and fixed end conditions for torsional axes • Strong axis buckling length equal to full height of the wall • Weak and torsional buckling length equal to screw spacing in panel edge

It should be noted that although distortional buckling was observed in some tests no design checks exists for this particular configuration in the current North American cold-formed steel specification, although this may change in future versions.

CHAPTER 4 EXPERIMENTAL PROGRAM: SHEAR WALLS UNDER COMBINED GRAVITY AND LATERAL LOADING

The Department of Civil Engineering and Applied Mechanics at McGill University has installed a frame specifically for the testing of shear walls (Figure 4-1). Originally designed by Zhao (2002) to apply in-plane lateral loads to shear walls, the frame is able to provide a rigid reaction against loads which are applied to the top of an anchored shear wall specimen. All lateral forces are transferred within the frame and only vertical forces are transferred into the floor eliminating the need for a strong floor. The test frame measures 11 m wide and 5 m high and can accommodate walls up to 3.66 m in height and 4.27 m in length. In the summer of 2005 the testing frame was modified to be able to apply combined gravity and lateral loading to the shear wall test specimens.

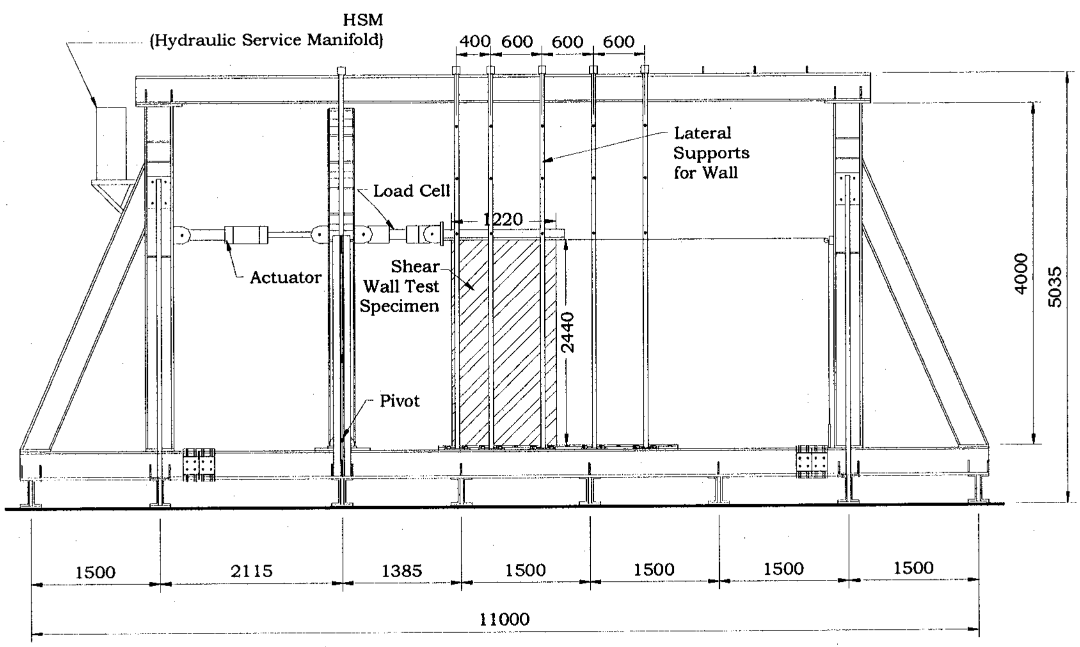


Figure 4-1 : Original test frame with 1220 x 2440 mm (4' x 8') wall specimen (*Branston, 2004*)

This chapter features the second testing phase concerned with the combined gravity and reversed cyclic loading of steel frame / wood panel shear walls. An overview of the test matrix, construction process, test set-up and results is provided.

4.1 Overview of Design Parameters for Test Matrix

The test program consisted of five different shear wall configurations. All walls maintained the same dimensions (1220 x 2440 mm), but varied in DCS thickness, sheathing type and fastener schedule. The most critical wall configuration of the test program was the 12.5 mm DFP sheathed panel with 75 mm (3") screw spacing around the perimeter. Test 13B (monotonic) by Branston (2004), a wall constructed in this configuration, failed due to the local buckling of the double chord stud when lateral loads alone were applied. Using the results from the first phase of testing described in Chapter 3 the wall was redesigned; instead of 1.09 mm (0.043") thick C-section chord studs as used by Branston, DCSs with a thickness of 1.37 mm (0.054") were specified. The increase in thickness of the DCS was also found necessary for the 12.5 mm CSP and 11 mm OSB sheathed walls with the 75 mm screw pattern. To analyze the possible influence of gravity loads on the performance of walls with 1.09 mm thick chord studs walls with 12.5 mm CSP and 11 mm OSB panels connected with screws at a 152 mm perimeter spacing were also included in the test program.

To calculate the loading of the DCS the nominal shear yield strength design values, S_y, from Branston (2004) were used in combination with the recommended overstrength value of 1.2. These shear yield strength values are associated with the sheathing connection mode of failure for all tests except 13B (Branston, 2004). The corresponding maximum axial compression force in the chord studs was then determined based on this lateral load. In addition to this, a gravity load for each wall of 18 kN or 14.8 kN/m was used based on a review of research. Ni & Karacabeyli (2000) found a load of 18.2 kN/m compensated for the omission of hold downs in wood frame / wood panel shear walls. Landolfo et al. (2006) used a load equivalent to 8.33 kN/m in their testing of steel frame / wood panel shear walls sheathed on both sides. Durham et al. (2001) applied a distributed load of 9.1 kN/m, calculated to be the equivalent of a second storey (5 kPa) room measuring 7.3 by 7.3 m. The 14.8 kN/m load applied to the steel frame / wood panel shear walls reflects bottom storey wall load of a three-storey commercial structure with the typical snow load in the Vancouver region. Although the gravity load on a wall can vary depending on occupancy, materials, span, etc., it was felt that the 14.8 kN/m represented a conservative estimate of the expected vertical force of a second and third storey, roof and snow load along a shear wall during a seismic event.

To estimate an appropriate gravity load to apply to the shear wall during testing a simple square three-storey commercial building with a flat roof (Fig. 4-2) was designed. The building is assumed to be located in Vancouver, BC, an area with potentially high seismic loads in Canada. To determine the appropriate loads the Wood Design Manual (2005) was referenced and the 2005 National Building Code of Canada was used to apply the appropriate load combination.

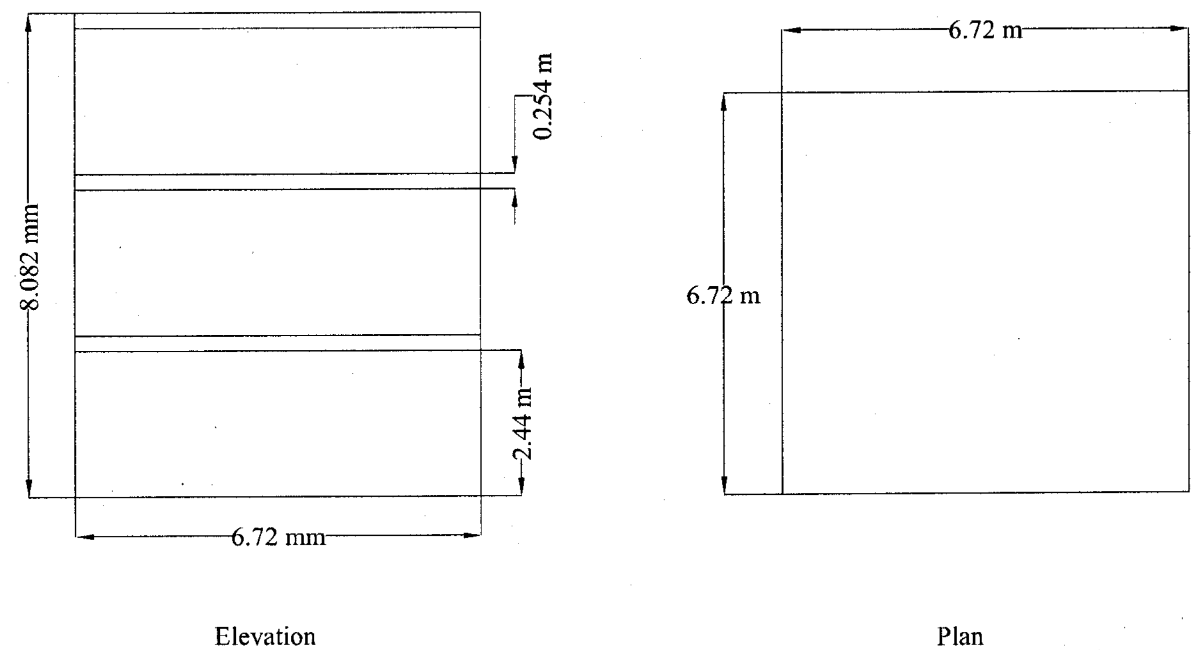


Figure 4-2 : Drawing of commercial structure used for determination of gravity loads

The snow load for this structure was computed using Equation 4-1described in the 2005 NBCC Clause 4.1.6.2 (NRCC, 2005) for Vancouver, BC.

$$S = I_S[S_S(C_b C_W C_S C_a) + S_r] \tag{4-1}$$

where,

S = design snow load [kPa]

$I_S = 1.0$, importance factor for snow loads

$S_S = 1.8$ kPa, ground snow load (1/50 year return period)

$C_b = 0.8$, basic roof snow load factor

$C_W = 1.0$, wind exposure factor

$C_S = 1.0$, roof slope shape

$C_a = 1.0$, shape factor

$S_r = 0.2$ kPa, associated rain load

(1/50 year return period and not greater than $S_s[C_b C_W C_S C_a]$)

Then the snow load for the building is equal to:

$S = 1.0[1.8 \times (0.8 \times 1.0 \times 1.0 \times 1.0) + 0.2] = 1.64$ kPa.

The live loads used for this design were taken from Table 4.1.5.3 in the 2005 NBCC.

Floors above ground (Office area) = 2.4 kPa

First storey (Retail) = 4.8 kPa

The values in Table 4-1 were obtained from Tables 11.31 and 11.32 of the Wood Design Manual (CWC, 2005) and Tables C3-1 and C3-2 from the Minimum Design Loads for Buildings and Other Structures (ASCE, 2005).

To calculate the line load along the perimeter wall of the structure in a three storey building it was assumed that in the worst case at the ground level one pair of parallel walls would intercept the loads from the snow, roof, 2 storey heights of exterior walls and one floor. It is assumed in that the joists would alternate directions between storeys and therefore the alternate pair of exterior walls would carry the remaining load from the 2 storey heights above and the remaining floor. A 25.4 cm (10") inter-storey clearance was allotted for joists and duct work (HVAC).

Table 4-1: Dead loads for the design of sample structure

	Materials Description	Distributed Load in kPa
Roof		
	Felt and Gravel (4-Ply)	0.26
	Insulation Rigid 104 mm thick	0.32
	38 mm x 0.91 mm steel deck	0.12
	Light gauge steel joists	0.24
	HVAC allowance	0.20
	Gypsum Board 13 mm	0.10
	Acoustical Fiber Board	0.05
	Total	1.29
Floor		
	Partitions	0.50
	Flooring - Linoleum 6 mm	0.05
	Sheathing - Plywood 12.5 mm	0.08
	Concrete topping - 20 mm	0.34
	Light gauge steel joists	0.28
	HVAC allowance	0.20
	Gypsum Board 13 mm	0.10
	Total	1.55
Exterior Walls		
	Interior finish (gyproc)	0.10
	Light gauge steel studs and tracks	0.13
	Insulation (fiberglass batts)	0.02
	Sheathing - Plywood 9.5 mm	0.05
	Siding	0.07
	Waterproofing Membrane	0.03
	Total	0.40

The 2005 NBCC (NRCC, 2005) load case for earthquake loading is:

Case 5: 1.0E + 1.0D + 0.5L +0.25S

Where,

E = Earthquake Load (Principal Load)

D = Dead Load (Principal Load)

L = Live Load (Companion Load)

S = Snow Load (Companion Load)

Therefore to compute the gravity load along the ground level walls most heavily loaded, the following loads were added:

- 25% of the snow load (S) equal to 1.64 kPa was multiplied by the area of the roof (7.62 m x 7.62 m) and then divided by walls design length (7.62 m + 7.62 m = 15.24 m)
- 100% of the dead load (D) at the roof multiplied by the area of the roof and then divided by the walls design length (15.24 m)
- 50% of the live load (L) for an office area multiplied by the area of one storey and then divided by the walls design length (15.24 m)
- 100% of the dead load (D) from one floor multiplied by the area of one storey and then divided by the walls design length (15.24 m)
- 100% of the dead load (D) from the exterior walls multiplied by two times the storey height (2 x 2.69 m = 5.38 m)

The total summation of these loads was equal to 14.8 kN/m. It was felt that this load represented a reasonable and realistic estimation of that which may be in place in a typical building during an earthquake. The gravity load was higher than that from the studies previously reviewed, which focused on residential structures, in order to broaden the range of application of the results.

To determine the expected compression force in the DCS the nominal shear yield strength of the selected wall was multiplied by the recommended overstrength and then multiplied by the ratio of wall height to wall length. To this value half of the gravity load (9 kN) was added. This value was then used to select the appropriate thickness of section for the double chord stud. Note, the central stud of each test wall would carry a portion of the applied gravity loads, however for the selection of chord studs in this study the contribution of the central stud was ignored. A sample calculation for the nominal axial capacity using the recommended end conditions and buckling lengths from Chapter 3 for the DCS with and without perforations is included in Appendix F.

Table 4-2 lists the shear yield values from previous testing (Branston, 2004) for each wall configuration, which were used to predict the probable loads that would be subjected to each shear wall configuration in this study. By multiplying the shear yield strength (S_y in kN / m) of the wall by the estimated overstrength of the system, 1.2 (Branston, 2004) and then by its length and the ratio of the height to length, and adding half of the gravity load it was possible to determine a reasonable estimate of the maximum compression force applied to the chord studs. Note, the gravity load would be a constant along the length of the studs; however the compression force due to the lateral load increases from zero at the wall top to a maximum at the base of the wall through the contributions from each fastener connecting the sheathing to the frame. Due to this vertically increasing load it was necessary to check whether the full section or the perforated section capacity controlled the design. The lowest perforated section in the studs used in this study occurred 837 mm above the ground. Using similar triangles the load on the stud at its base was determined as if the perforated section capacity controlled. In all cases the load at failure was governed by the full section capacity. The nominal axial compression capacity of the DCS based on the recommended effective length factors and buckling length from Chapter 3 for both the full and perforated section were compared with the estimated maximum compression load as depicted in Figure 4-3. The nominal values, where the ϕ factors are not included, in the calculation for capacity are used in capacity based design because it is expected that full lateral capacity of the shear wall would be reached during a major earthquake. This approach to design combined with the applied gravity load resulted in a required increase in the chord stud thickness to the next nominal size 1.37 mm (0.054") for all configurations with a 75 mm (3") screw spacing around the perimeter of the panel. By designing the walls in this fashion it was assumed that failure would occur at the sheathing connection locations instead of in the chord studs, even when gravity loads were to be applied. A sample calculation for the capacity based design is included in Appendix F.

Table 4-2 : Design of the double chord studs for shear wall test program

Panel Type	DFP	OSB	OSB	CSP	CSP
Panel Thickness (mm)	12.5 mm	11 mm	11 mm	12.5 mm	12.5 mm
Fastener Schedule (mm)	75/305	152/305	75/305	152/305	75/305
Design S_y[1] (kN/m)	24.5	10.6	21.6	11.0	20.6
Overstrength[1]	1.2	1.2	1.2	1.2	1.2
S_y for Capacity Based Design (kN/m)	29.4	12.7	25.9	13.2	24.7
Total Gravity Load (kN)	18	18	18	18	18
Height of Wall (m)	2.44	2.44	2.44	2.44	2.44
Width of Wall (m)	1.22	1.22	1.22	1.22	1.22
Total Potential Load[2] (kN)	80.7	40.0	72.2	41.2	69.3
Load[2] at Perforated Section (kN)	56.1	29.4	50.5	30.2	48.6
Required Stud Thickness (mm)	1.37	1.09	1.37	1.09	1.37
DCS Capacity (CSA S136)[3,4]	115.2	67.1	115.2	67.1	115.2
DCS Capacity (CSA S136)[3,5]	99.9	58.7	99.9	58.7	99.9

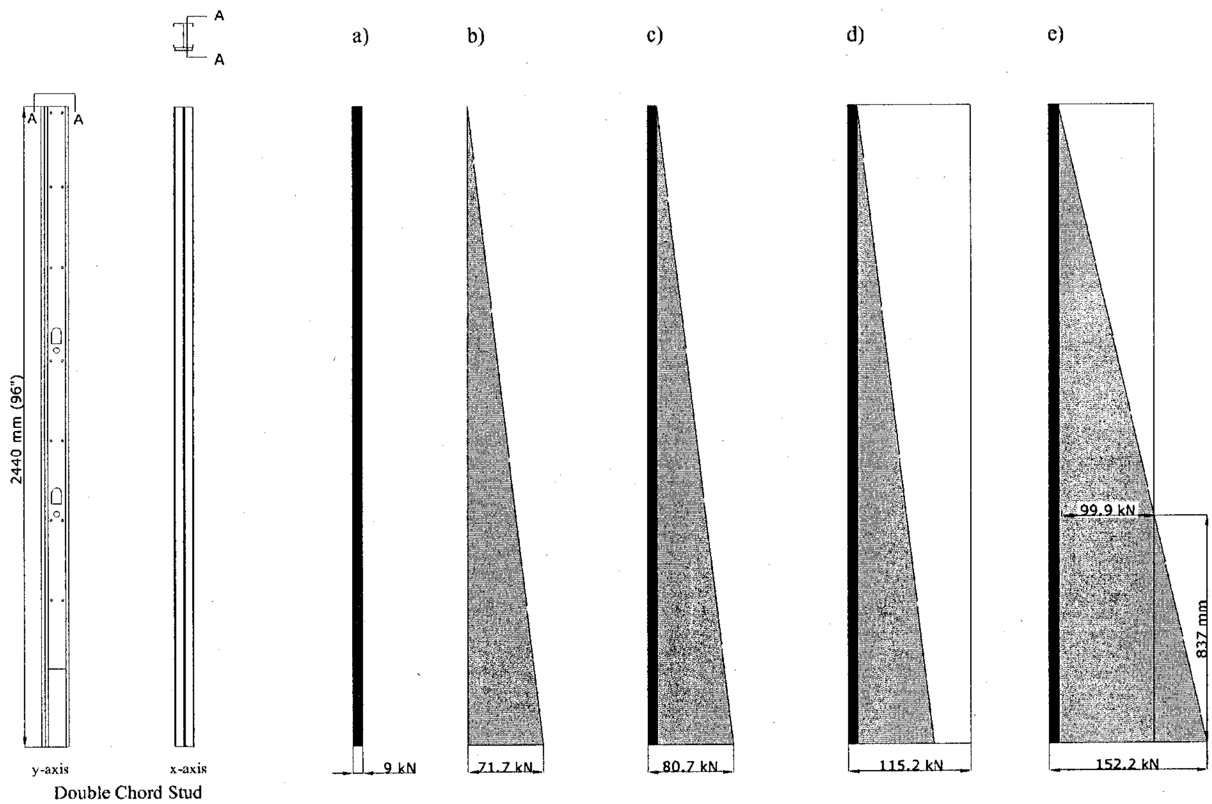
[1] From Branston (2004)

[2] For a single DCS

[3] Calculated with Φ=1.0, K_x=K_y=0.9, K_t=0.65 ended conditions and buckling lengths L_x=2.44, L_y=L_t=2**s**. Where **s** represents the spacing between the edge fasteners.

[4] Full Section Capacity

[5] Perforated Section Capacity



a) Gravity force
b) Compression force due to lateral load.
c) Combined axial force on chord stud.
d) Capacity of unperforated chord stud section with combined axial forces superimposed.
e) Capacity of perforated chord stud section with the required axial force due to lateral loads to cause failure superimposed.

Figure 4-3: Determination of the governing failure mode for the double chord stud for a DFP sheathed wall with 75/305 mm fastener schedule

4.2 TEST MATRIX

In the summer of 2005 a total of 32 light gauge steel frame / wood panel shear walls were tested. Typically, each test group consisted of six specimens (3 monotonic and 3 reversed cyclic). However, in two cases the monotonic tests exhibited variations larger than 10% and it was deemed necessary to complete an additional test. The walls were tested with a constant gravity load and either a monotonic or reversed cyclic lateral displacement. However, for each group of three monotonic or three cyclic tests at least one specimen was tested with the lateral load alone for comparison purposes.

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

- Either 12.5 mm CSA O121 Exterior Douglas Fir Plywood (DFP) *(CSA O121, 1978)*, 12.5 mm CSA O151 Exterior Canadian Softwood Plywood (CSP) *(CSA O151, 1978)* or 11 mm CSA O325 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 the following two nominal grades and thicknesses: 1. 230 MPa (33 ksi) and 1.09 mm (0.043") and 2. 340 MPa (50 ksi) and 1.37 mm (0.054'). All studs had nominal dimensions of: 92.1 mm (3-5/8") web, 41.3 mm (1-5/8") flanges and 12.7 mm (1/2") lips.
- Light gauge steel top and bottom tracks manufactured in Canada to ASTM A653 (2002) with nominal grade of 230 MPa (33 ksi) and a thickness of 1.09 mm (0.043"). The track's nominal dimensions were 92.1 mm (3-5/8") web and 31.8 mm (1-1/4") flange.
- The double chord stud (DCS) consisted of two studs connected back-to-back and connected by two No. 10—16 x 19.1 mm (3/4") long Hex head self-drilling screws at 305 mm (12") on centre. The built-up member was used to prevent the flexural and/or local buckling failure of a single chord stud alone. The remaining interior stud was spaced at 610 mm (24") on centre.
- Industry standard Simpson Strong-Tie S/HD10 (Simpson, 2001) hold down connectors were attached to the DCS with 33 No. 10-16 x 19.1 mm (3/4") long Hex head self-drilling screws. To fasten the hold down to the test frame ASTM A193 (2006) 22.2 mm (7/8") anchor rods were used.
- Bolts, 19.1 mm (3/4") diameter ASTM A325 (2002), were used as shear anchors
- No. 8 x 12.7 mm (1/2") long wafer head self-drilling framing screws were used to connect the track and studs.
- No. 8 x 38.1 mm (1-1/2") Grabber SuperDrive (SuperDriver, 2003) Bugle head self-piercing sheathing screws and No. 8 x 31.8 mm (1-1/4") Grabber SuperDrive

(2003) Bugle head self-drilling sheathing screws were used to affix the sheathing to the light gauge steel frames. The self drilling screws were used for the thicker chord studs. The sheathing-to-framing screws were installed 12.7 mm (1/2") from the edge of each sheathing panel. The screw spacing / fastener schedule was 75 mm (3") or 152 mm (6") along the panel edges and 305 mm (12") in the interior.

Table 4-3 lists the variables for the five different wall configurations: wood sheathing type, loading protocol, chord stud thickness, fastener schedule, sheathing-to-framing screws and gravity loading. Note: the field studs and track for all test specimens were rolled from 1.09 mm (0.043") thick steel. Individual test data sheets documenting the details of each wall specimen have been included in Appendix C for reference purposes.

Table 4-3 : Matrix of shear wall tests

Specimen	Protocol	Wall Length (mm)	Wall Height (mm)	Sheathing Type	Sheathing Thickness (mm)	Fastener[2] Schedule (mm)	DCS[4] Thickness (mm)
47 - A,B,C[3]	Monotonic	1220	2440	DFP	12.5	75/305	1.37
48 - A,B,C[3]	CUREE[1]	1220	2440	DFP	12.5	75/305	1.37
49 - A,B[3],C,D	Monotonic	1220	2440	OSB	11	152/305	1.09
50 - A,B,C[3]	CUREE	1220	2440	OSB	11	152/305	1.09
51 - A,B[3],C	Monotonic	1220	2440	OSB	11	75/305	1.37
52 - A,B,C[3]	CUREE	1220	2440	OSB	11	75/305	1.37
53 - A,B,C[3]	Monotonic	1220	2440	CSP	12.5	152/305	1.09
54 - A,B,C[3]	CUREE	1220	2440	CSP	12.5	152/305	1.09
55 - A,B[3],C,D	Monotonic	1220	2440	CSP	12.5	75/305	1.37
56 - A[3],B,C	CUREE	1220	2440	CSP	12.5	75/305	1.37

[1] CUREE reversed cyclic protocol for ordinary ground motions (Krawinkler et al. 2000; ASTM E2126 2005)
[2] Fastener schedule (ie.75/305) refers to the approximate spacing in millimetres between the sheathing to framing screws along the panel perimeter and the field spacing, respectively.
[3] Test did not include gravity loading.
[4] Double Chord Stud

4.3 SHEAR WALL TEST FRAME MODIFICATIONS

The test frame was ordinarily configured to perform in-plane lateral loading on shear walls (Fig. 4-1). A number of steel frame / wood panel shear walls were tested in this manner by Blais and Rokas in 2004, as well as Boudreault, Branston and Chen in 2003. The design and configuration of the original apparatus is described in more detail by Zhao (2002) and Branston (2004). In order to perform combined gravity and in-plane lateral loading on the shear walls modifications to the test frame were necessary (Figs. 4-4 & 4-5).

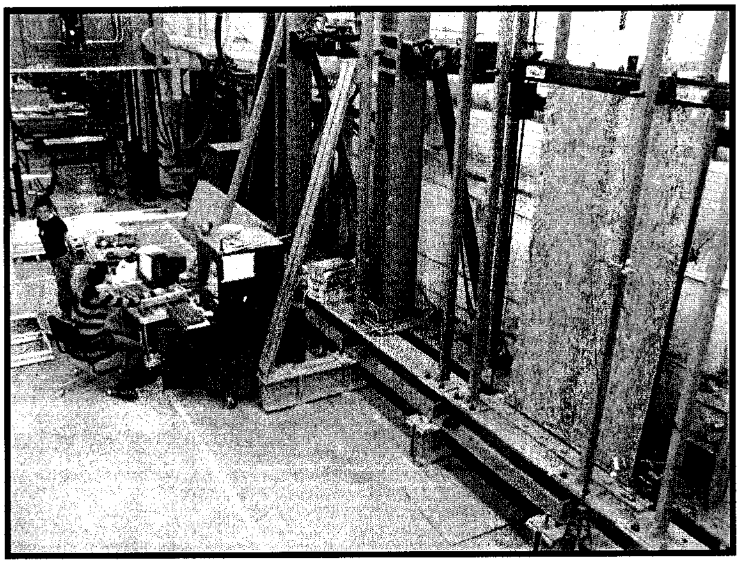


Figure 4-4 : Shear wall test specimen undergoing lateral and gravity loading.

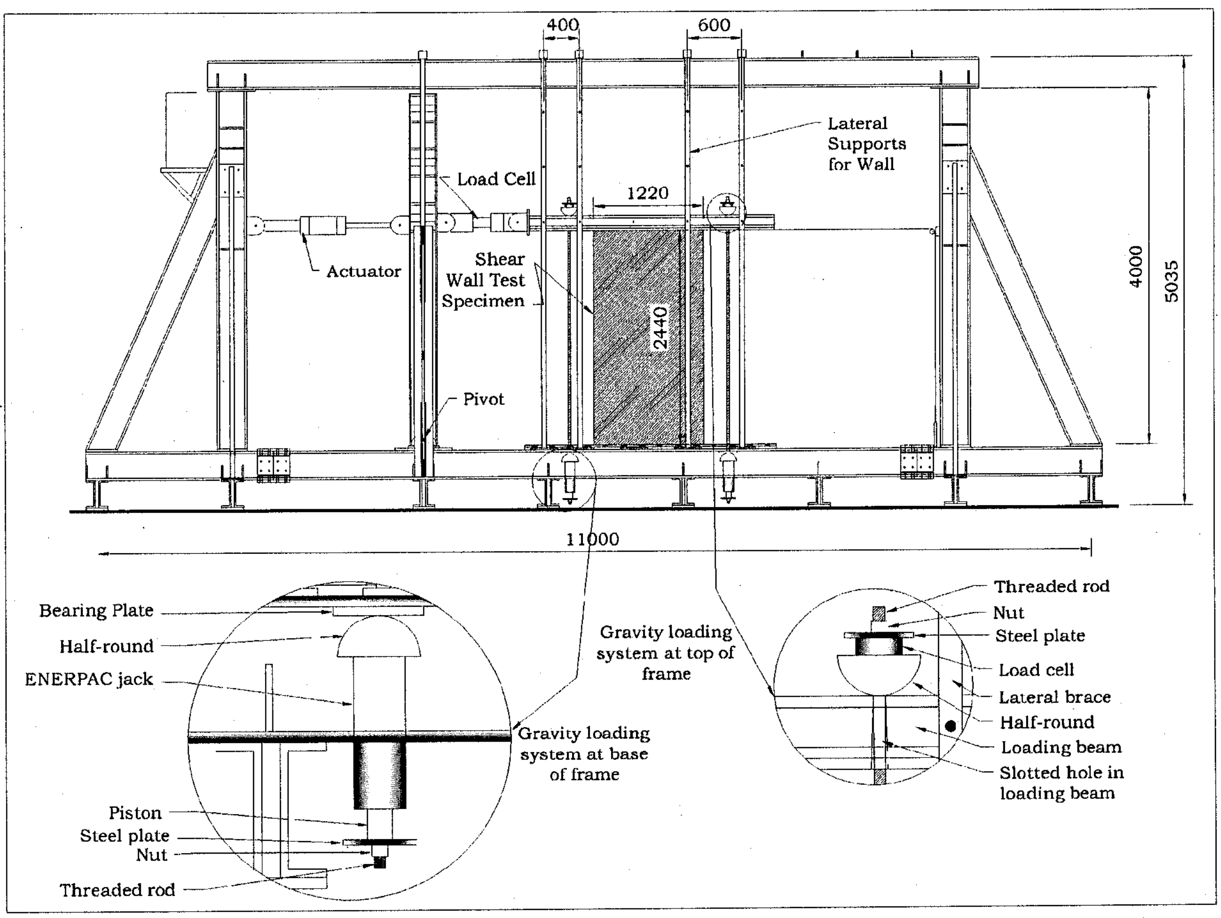


Figure 4-5 : Modified test frame with 1220 mm x 2440 mm (4' x 8') wall specimen

The frame maintained its existing 250 mm (10") stroke (± 125 mm (5")) dynamic actuator and 250 kN (55 kip) load cell for lateral loading purposes. This system continued to operate under the same principles as explained in Branston (2004). However, the loading beam transferring the lateral load to the shear wall specimen was redesigned to incorporate a gravity loading system. An Enerpac loading jack was placed at each end of the shear wall specimen below the main beams of the test frame. These jacks were connected to a servo controlled hydraulic system such that they could be controlled independently through use of the MTS IIs computer setup. Threaded rods were used to connect the hydraulic jacks to the loading beam, which was extended and stiffened to limit flexural deflections. The rods were 22.2 mm (7/8") in diameter and 3.65 m (12') in length. Half-rounds 92.1 mm (3 5/8") in diameter were placed at each reaction surface to allow the gravity loading system to pivot and follow the lateral displacement of the shear wall. Load cells (227 kN / 50 kip) were installed above the horizontal loading beam such that a constant force from the Enerpac jacks could be applied to the test wall. Figure 4-6 shows the top section of the gravity loading assembly, while Figure 4-7 shows the Enerpac jack located between the two lower members of the test frame.

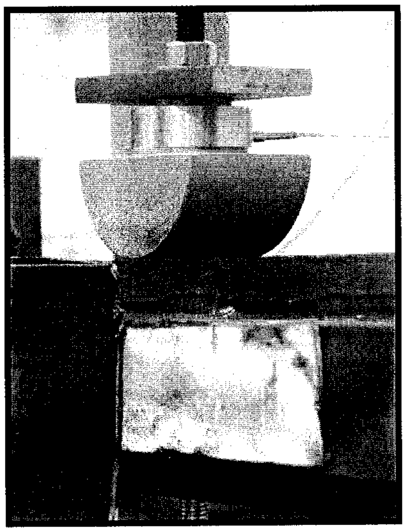


Figure 4-6 : Top section of gravity loading scheme (top to bottom): 22.2 mm ϕ rod, nut, 19 mm plate, load cell, 92.1 mm ϕ half-round and loading beam (red) braced by lateral supports sliding against greased Teflon (white).

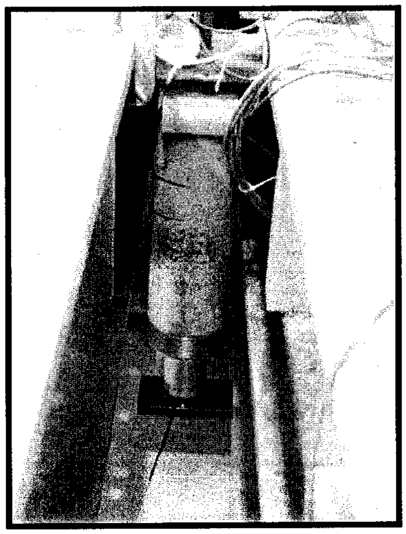


Figure 4-7 : Bottom section of gravity loading scheme (top to bottom): 22.2 mm Φ rod, 19 mm reaction plate, 92.1 mm ϕ half-round and Enerpac jack

4.4 Shear Wall Fabrication, Materials and Components

All wood panels intended for testing were stacked to allow for air circulation to achieve equilibrium moisture content (EMC) before assembling with the steel wall framing. The acclimatization was necessary to prevent cracking or splitting of the panel that may occur if attached to the frame before all dimensional changes were complete.

Prior to assembly of the frame holes were drilled into the bottom and top tracks to accommodate two shear anchors and two hold downs on the bottom and six shear anchors on the top. The shear anchors were 19.1 mm (3/4") ASTM A325 (2002) bolts 90 mm (3 ½") in length. The hold downs were attached to the test frame with 22.2 mm (7/8") threaded rods. All holes were drilled 1.6 mm (1/16") larger than necessary to facilitate installation of the shear wall specimens.

The double chord stud, the set of two chord studs connected back-to-back using No. 10-16 Hex washer head self-drilling screws at 305 mm (12") on centre, had hold downs installed at the base with 33 No. 10-16 Hex washer head screws. These Simpson Strong-Tie S/HD10 products (Simpson, 2001) were installed according to the manufacturer's literature so as to reach the allowable load of 43 kN for a hold down connected to 1.09 mm (0.043") or 1.37 mm (0.054") studs. Note, the average ultimate load for the hold downs is 129 kN.

Each steel frame was composed of a pre-drilled top and bottom track, screw connected to two DCS of the appropriate thickness and an intermediate stud. The sheathing panel was checked for moisture content prior to assembly with the frame to ensure < 15% EMC. A total of five readings were taken with an electronic moisture meter (Delmhorst Instrument Co. RDM-2 (Delmhorst, 2003)) and then averaged. The screw fasteners were then installed flush to the surface of the wood and in positions as dictated by the test matrix, which was marked on to the face of the panel at a 12.7 mm (1/2") edge distance (Figure 4-8).

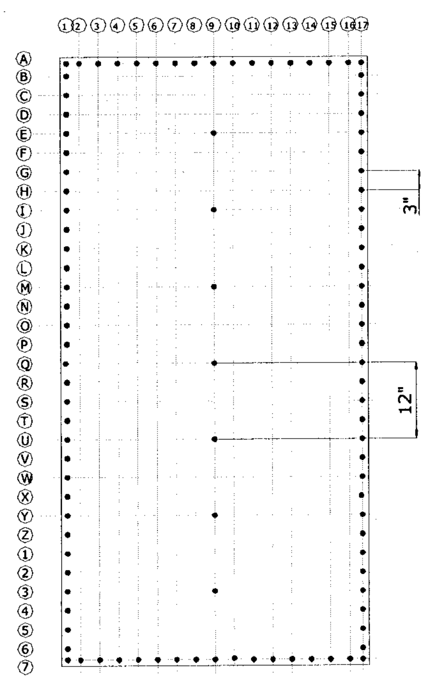


Figure 4-8 : Screw Schedule for a 75 mm/305 mm (3"/12") spacing (*Branston, 2004*)

In locations where it was necessary for fastener screws to penetrate two layers of steel holes were pre-drilled to prevent the second layer of steel (stud flange) from bending away from the track. For walls with 75 mm (3") fastener spacing, as shown in Figure 4-8, around the perimeter pilot holes at the location adjacent to the bottom corner (locations 7-2 and 7-16) were drilled and No. 9 x 1" (25.4 mm) long bugle head screws were used because of limited clearance due to the hold down. All details of the fastener schedule were recorded on Test Data Sheets located in Appendix C. The five types of fasteners used in testing are shown in Figure 4-9.

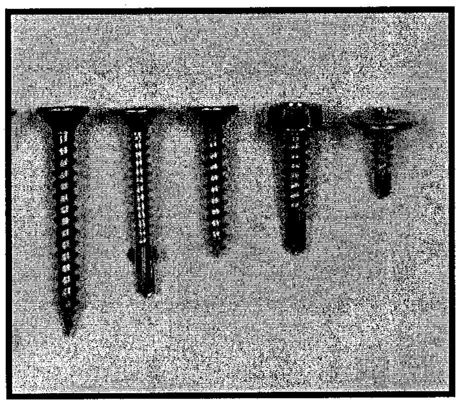
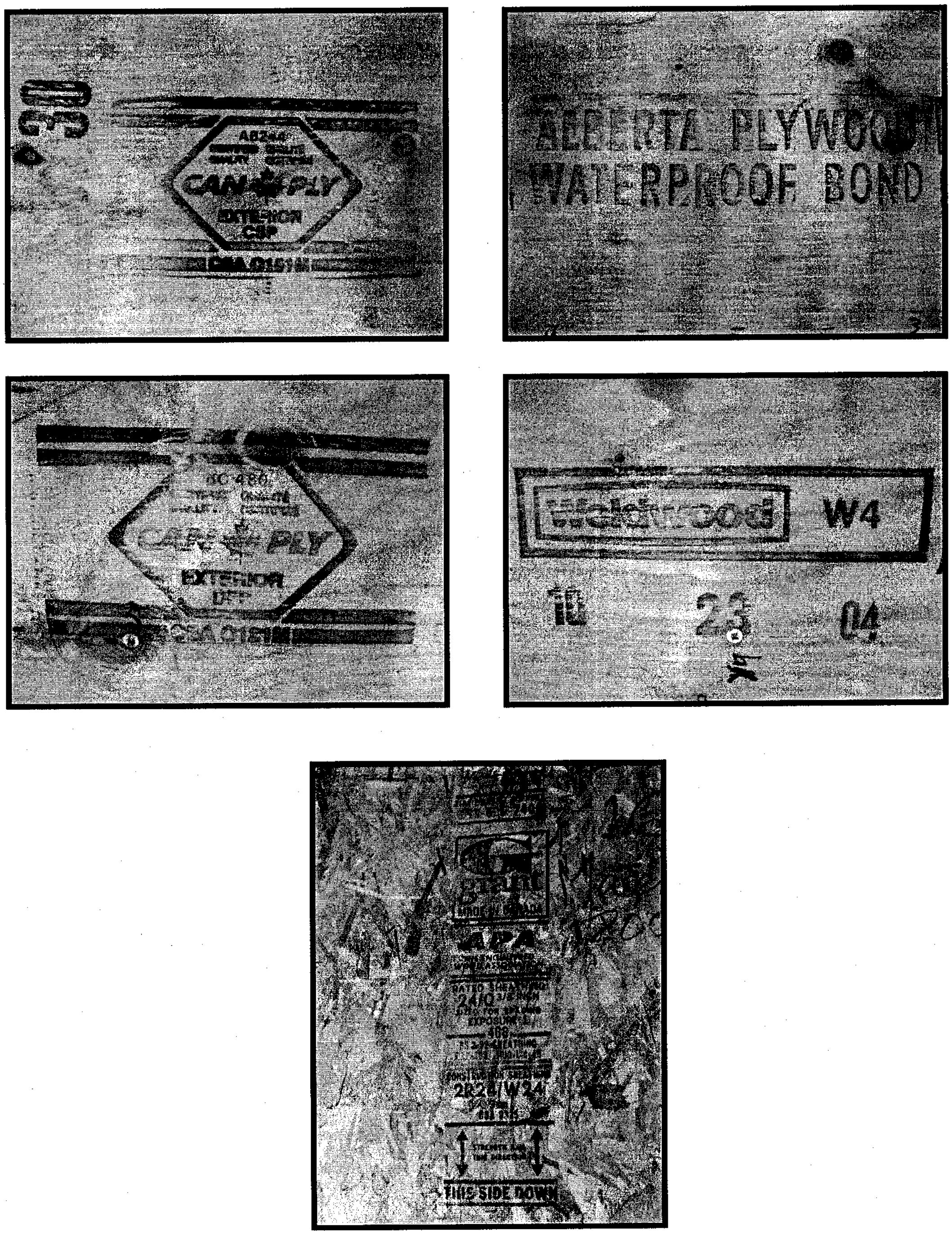


Figure 4-9 : From left to right: No. 8 x 38.1 mm (1-1/2") Grabber SuperDrive (SuperDrive, 2003) Bugle head self-piercing sheathing screw, No. 8 x 31.8 mm (1-1/4") Grabber SuperDrive (SuperDrive, 2003) Bugle head self-drilling sheathing screw,, No. 9 x 1" bugle head self-piercing sheathing screw, No. 10-16 x 3/4" Hex head self-drilling screw and No. 8 x 1/2" wafer head framing screw

Sample grade stamps for the three types of sheathing are shown in Figure 4-10. Any damage to panels prior to testing was recorded on Test Damage Sheets for each wall that can be found in Appendix C. Whenever possible, panels with prior damage to the edges or corners were not used because of the dependence of wall behaviour on the performance of the perimeter sheathing connections.

Upon completion of each test the moisture content of the sheathing was determined using APA Test Method P-6 (APA PRP-108, 2001). Two specimens, each 75 mm (3") in diameter, were taken from the centre of the panel. The weight of each specimen was



Figures 4-10 : Grade stamps of sheathing panels for CSP, DFP and OSB (from top to bottom)

recorded immediately after it was removed (W_w) from the panel. The specimens were then placed in a drying oven at approximately 93.3°C (200°F) for 24 hours. Once oven-dry each specimen was weighed again (W_d), and the moisture content (*MC*) was determined according to Equation (4-1).

$$MC = \left(\frac{W_w - W_d}{W_d} \right) x100 \qquad (4\text{-}1)$$

where,

MC = moisture content of specimen [%]

W_w = initial weight [g]

W_d = oven-dry weight [g]

4.5 Test Set-up

To install a shear wall specimen into the test apparatus it was lifted vertically on to the test frame and maneuvered between the lateral braces such that the pre-drilled holes on the bottom track were aligned with the base plate. The loading beam, initially suspended from chain blocks to allow for clearance of the wall specimen, was then lowered into position (Figure 4-11).

Figure 4-11 : Wall specimen installed in test frame

The shear wall specimen was carefully aligned vertically and in-line with the test frame ensuring that in-plane panel rotation was not restricted. Once aligned the shear anchors and hold down anchor rods with load cells were installed. The steel plate washers, previously mentioned, were used with each bolted connection between the track and the bolt head or nut (Figures 4-12 & 4-13).

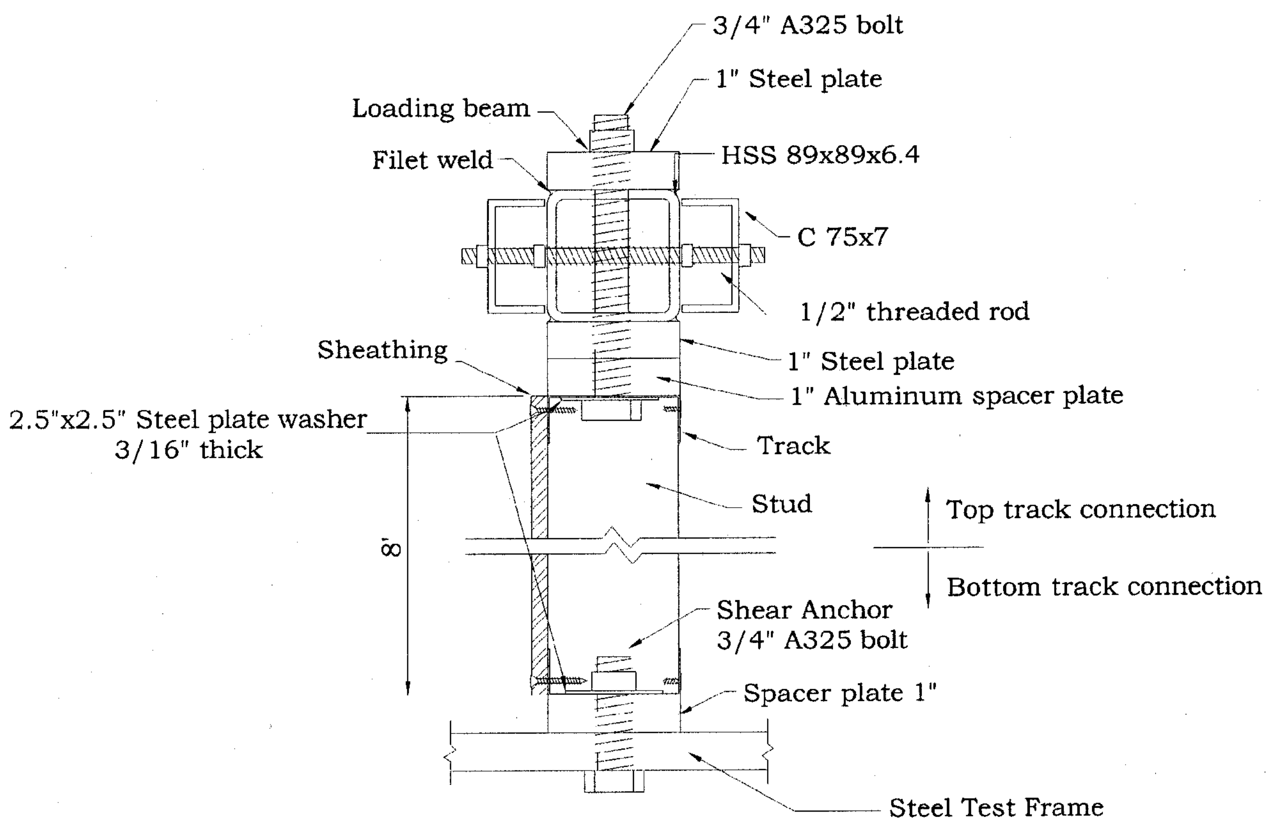


Figure 4-12: Detail of altered loading beam and its components as well as the top and bottom track connections

Once all connectors were placed and hand-tight the shear anchors were tightened using an electric impact wrench with a capacity of 0.4 kN-m (300 lb·ft). The hold downs anchor rods were then secured using the turn-of-the-nut method as specified by the manufacturer. Additionally, the force in the hold down threaded rods at each end of the wall was balanced at approximately 9 kN.

The loading beam depicted in Figure 4-10 was made of several sections: at the centre a hollow square section (HSS 89 x 89 x 6.4 mm) with 25.4 mm x 92.1 (1" x 3 5/8") steel plates centered and welded to the top and bottom of the section. This built-up section was welded to a 25.4 mm (1") thick base plate that bolted to the swivel joint connected to the load cell. Channel sections (C75 x 7) were attached to the loading beam using threaded rods. These channels were coated with Teflon at lateral brace locations such that guides projecting from the braces were able to slide with a minimal amount of friction (Figure 4-4). A 25.4 mm (1") aluminum spacer plate located between the loading beam and the wall specimen was installed to allow for free rotation of the sheathing panel during testing. A similar spacer plate was placed beneath the wall, again to allow for rotation of the sheathing.

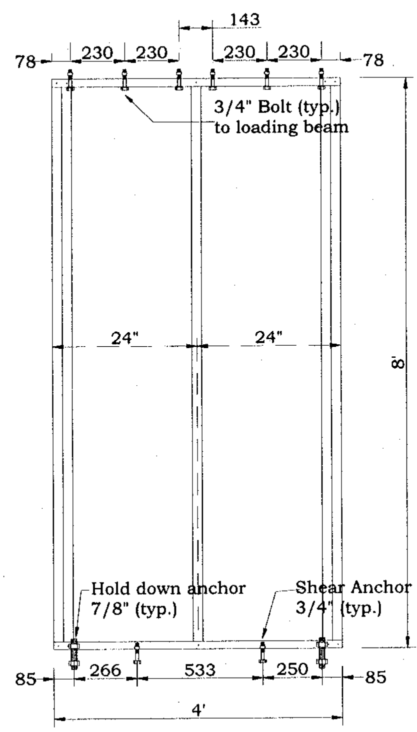


Figure 4-13 : Placement of anchorage for wall test specimen to frame (*Branston, 2004*)

4.6 Instrumentation and Data Acquisition

The forces, displacements and accelerations of each shear wall were monitored and recorded during testing. Seven transducers (LVDTs) were directly connected to the wall specimen to measure the uplift (2 LVDTs) and in-plane slip (2 LVDTs) at each bottom corner, in-plane lateral displacement at the top corner (1 LVDT) and the shear deformation of the wood sheathing (2 LVDTs) (Figure 4-14). Two additional LVDTs were placed on the lateral braces of the test frame to record any out-of-plane movement of the shear wall.

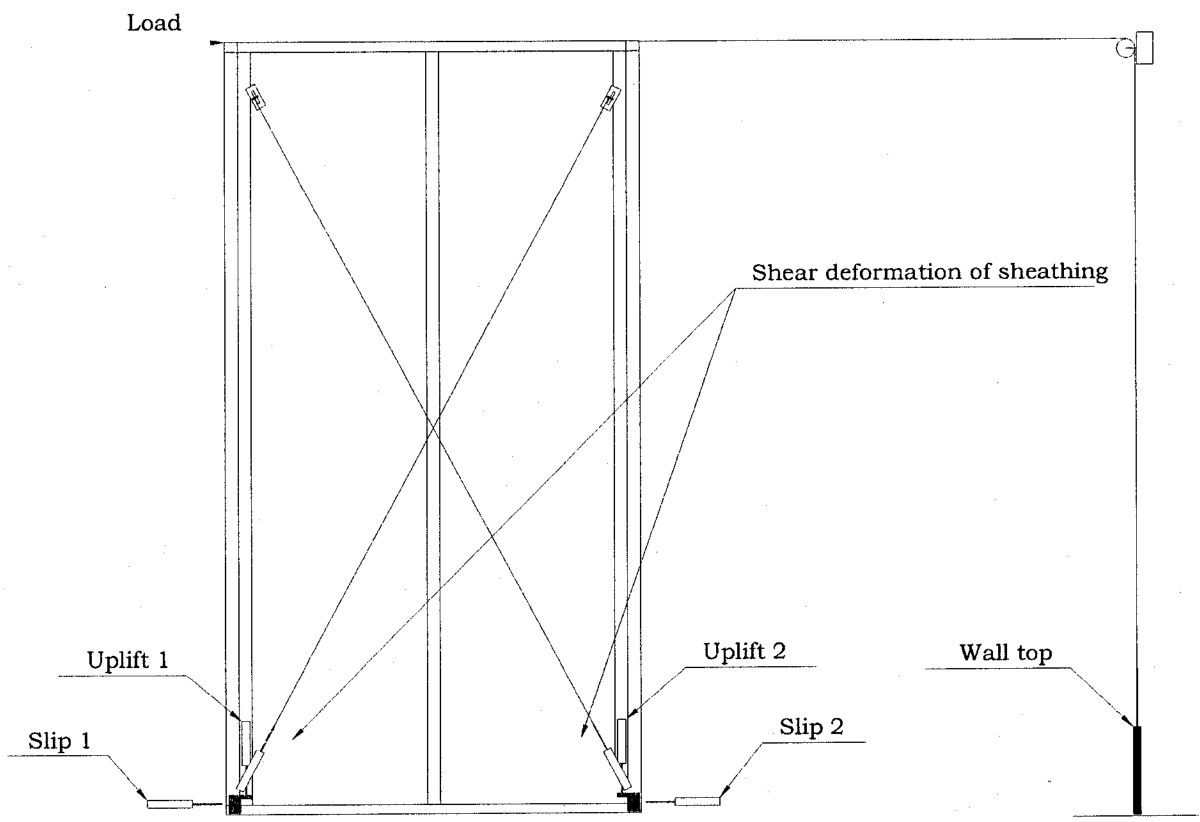


Figure 4-14: Positioning of LVDTs on wall specimen

Load cells were used in five different locations. Two load cells were connected directly to the wall through the hold down anchor rods to measure the up-lift force. At the top of the gravity load set-up on each side of the wall a load cell was also placed, as shown in Figure 4-5 and 4-6. These load cells were used to monitor and control the gravity loads that were applied to the wall. Another load cell was attached in-line with the loading beam to measure the shear force as the wall was displaced. At the same location on the loading beam an accelerometer was also attached so that the inertial effects of the test apparatus could be obtained.

The data acquisition system consisted of Vishay Model 5100B scanners to record data and Vishay System 5000 Strainsmart software to control the acquisition. Monotonic test data was recorded at a rate of 2 scans per second and reversed cyclic test data at 50 scans per second.

4.7 Loading Protocols

4.7.1 Monotonic Tests

A stroke controlled monotonic test protocol was used to displace the top of the wall at a constant rate to simulate a "static" type loading as assumed to occur in certain wind loading conditions. Once the wall had been installed in the test frame it was placed at the zero force position by making slight adjustments to the position of the actuator. Each wall was loaded at a rate of 7.5 mm (0.3") per minute until failure. This protocol was similar to the one used by Serrette et al. (1996b) and by previous researchers in this test program including Branston (2004), Chen (2004), Boudreault (2005), Blais (2006) and Rokas (2006). However, unlike the previous researchers the permanent offset of the walls was not evaluated. Figure 4-15 provides an example of a typical monotonic test force vs. deformation curve.

4.7.2 Reversed Cyclic Tests

The CUREE protocol for ordinary ground motions (Krawinkler et al., 2000; ASTM E2126, 2005) was deemed most suitable for the testing of the steel frame / wood panel shear walls (Boudreault, 2005). The protocol is based on the results of nonlinear time history dynamic analyses of structures relying on wood frame shear walls for lateral force resistance. The time history responses of the modeled buildings were converted to representative deformation controlled loading histories based on cumulative damage concepts. The protocol is representative of the expected demand to be imposed on this type of building component during an earthquake. Furthermore, it was developed to account for multiple earthquakes that might occur over the lifetime of a structure and

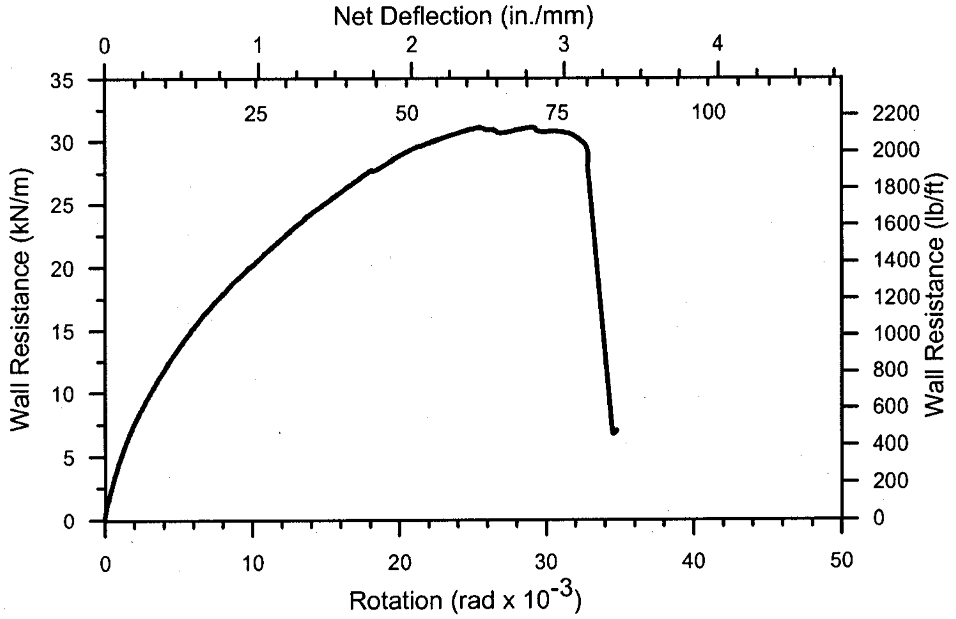


Figure 4-15 : Typical wall resistance curve for a monotonic test (Test 47A)

subjects elements to ordinary ground motions (not near fault) with the probability of exceedance of 10% in 50 years.

The cyclic test protocol was calibrated from the average ultimate deformation capacity of a monotonic test. The ultimate deformation capacity, Δ_m, was defined as the post-peak displacement corresponding to 80% of the maximum (peak) corrected resistance (Section 4.7) (Figure 4-16). For reversed cyclic testing a fraction of Δ_m, $\gamma\Delta_m = \Delta$ $(\gamma = 0.6)$, was used as a reference deformation in order to define the maximum deflection that the wall would sustain. The protocol was composed of a series of initiation, primary and trailing cycles. A cycle is defined as an excursion starting from zero to a positive displacement then reversing directions, passing through zero to a negative displacement and returning to the zero position. The displacements for each cycle, known as the loading history, were based on multiples of the reference deformation, Δ.

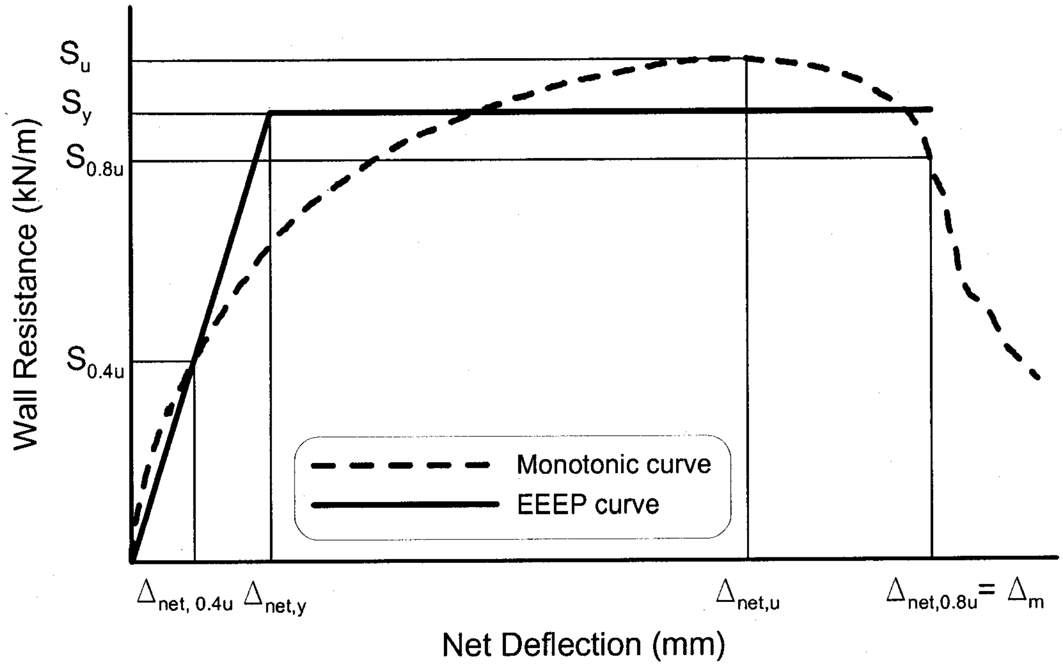


Figure 4-16 : Determination of reference deformation (Δ) from a monotonic test

The initiation cycles, the first six, were typically well within the elastic range of the shear wall specimen so that all apparatus functions and data collection processes could be verified. The primary cycles grew subsequently larger and usually led into the non-linear behaviour range of the shear wall specimen. The trailing cycles were 75% of the preceding peak primary amplitude cycle. The exact sequence of the three phases of loading and a sample protocol are presented in Table 4-4 and Figure 4-17. For the complete set of cyclic protocols used in this test series please refer to Appendix B.

In Table 4-3 there are two columns of displacements, the first named *Target (corr.)* and the second named *Actuator Input*. The first column represents the desired displacement for the shear wall specimen, but because of losses due to slip and uplift of the wall there was always a small difference between actuator input and top of wall displacement.

Table 4-4 : Displacement amplitudes following CUREE protocol for 1220 x 2440 mm (4' x 8') shear wall tests 48-A,B,C with DFP sheathing and a screw schedule of 75/305 mm (3"/12")

CUREE Quasi-Static Cyclic Testing Protocol (Deformation Controlled)

Loading History for Ordinary Ground Motions

$\Delta=0.6^*\Delta_m$	47.61	Screw Pattern: 3"/12"	
		Sheathing: DFP	
	Target (corr.)	Actuator Input	
Displ.	mm	mm	No. Of cycles
0.050 Δ	2.380	3.065	6
0.075 Δ	3.571	4.597	1
0.056 Δ	2.678	3.448	6
0.100 Δ	4.761	6.129	1
0.075 Δ	3.571	4.597	6
0.200 Δ	9.521	12.258	1
0.150 Δ	7.141	9.194	3
0.300 Δ	14.282	18.387	1
0.225 Δ	10.712	13.790	3
0.400 Δ	19.043	24.516	1
0.300 Δ	14.282	18.387	2
0.700 Δ	33.325	42.904	1
0.525 Δ	24.994	32.178	2
1.000 Δ	47.607	61.291	1
0.750 Δ	35.705	45.968	2
1.500 Δ	71.410	91.936	1
1.125 Δ	53.558	68.952	2
2.000 Δ	95.214	122.581	1
1.500 Δ	71.410	91.936	2

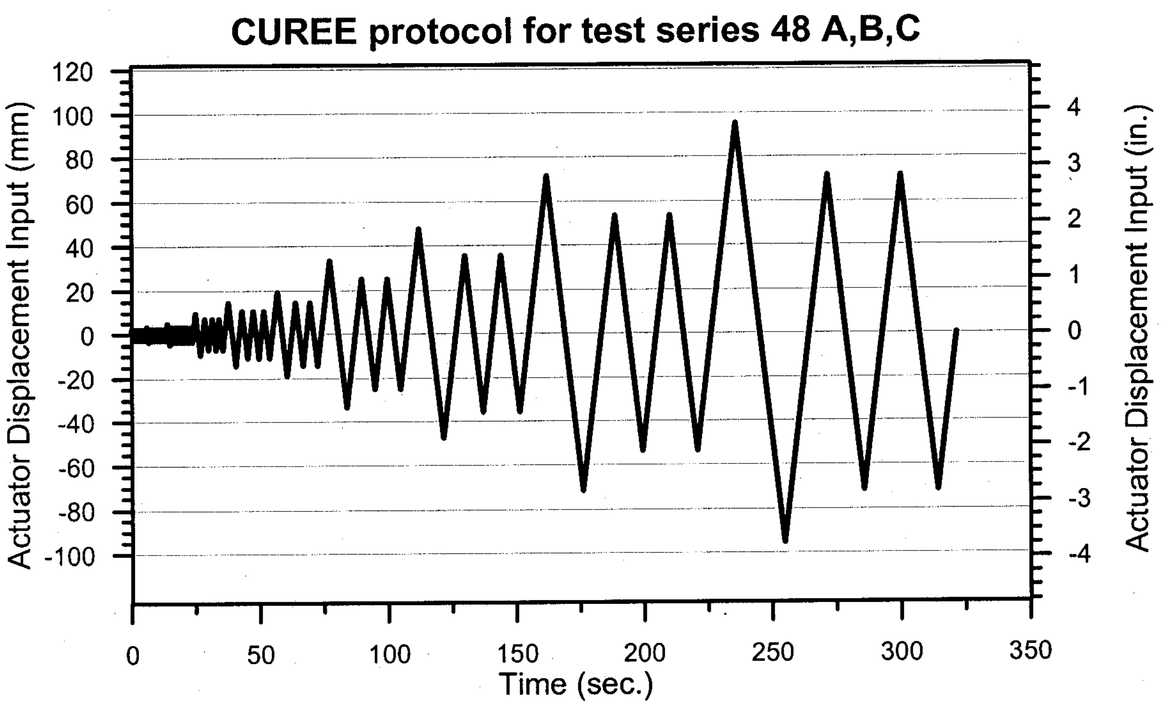


Figure 4-17 : CUREE ordinary ground motions protocol for shear wall tests 48-A,B,C 1220 x 2440 mm (4'x8') DFP 75/305 mm (3"/12")

Using the linear relationship existing between actuator input and from monotonic tests correction values were calibrated and applied to the *Target (corr.)* column resulting in *Actuator Input* values.

The testing rate was limited to a maximum of 10 mm/s due to the limitations of the hydraulic pump and oil supply at the actuator and two jacks. However, at lower displacements of the reversed cyclic protocol a frequency of 0.5 Hz was used. This frequency was chosen with the minimization of inertial effects in mind. To connect the displacement amplitudes (Table 4-4) a sine curve pattern was used, unlike the straight line ramps depicted in Figure 4-17. A sample of a reversed cyclic test response curve corrected for slip, uplift, gravity load components and inertial effects (Section 4.7) is shown in Figure 4-18.

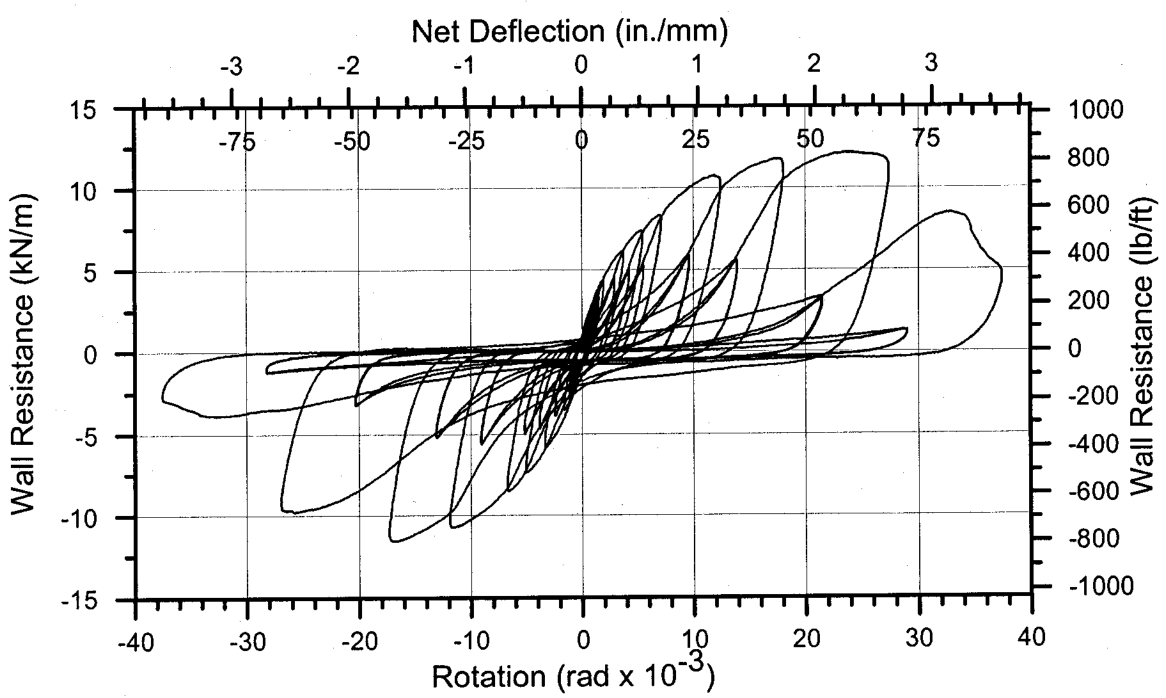


Figure 4-18: Typical wall resistance vs. deflection curve for a reversed cyclic test (Test 54B)

4.8 Data Reduction

Due to slip and rigid body rotation of the test wall the wall top displacement measured by the LVDT was not representative of the net lateral in-plane displacement. Gravity loads did help in minimizing these additional displacements, but in order to derive the mechanical properties of the wall it was necessary to correct the measured displacement. It was possible to calculate the net lateral in-plane displacement, accounting for the measured slip and uplift at the bottom corners of each specimen, as shown in Equation 4-2.

$$\Delta_{net} = \Delta_{walltop} - \left[\left(\frac{\Delta_{baseslip1} + \Delta_{baseslip2}}{2}\right)\right] - \left[\left(\Delta_{uplift1} - \Delta_{uplift2}\right) \times \frac{H}{L}\right] \tag{4-2}$$

where,

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

$\Delta_{wall\ top}$ = Total measured wall-top displacement, [mm]

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

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

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

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

The rotation of the wall in radians was defined by Equation (4-3) as:

$$\theta_{net} = \frac{\Delta_{net}}{H} \tag{4-3}$$

where,

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

Δ_{net} = as calculated using Eq. (4-1)

An illustration of what these calculations represent with reference to the wall can be seen in Figure 4-19.

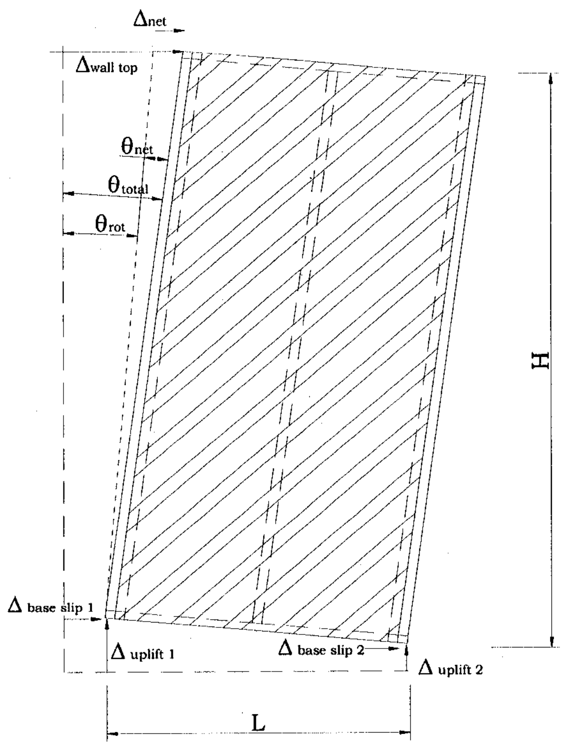


Figure 4-19 : Deformed configuration of shear wall (*Branston, 2004*)

The shear flow through the top of the wall was calculated using Equation (4-4).

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

where,

S = Wall resistance, [force per unit length, (kN/m)]

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

L = Length of the wall, [m]

For reversed cyclic loading a correction to the lateral resistance acquired from the load cell for the inertial effects was necessary. The force due to the inertial effect was calculated using a mass of 310 kg (3.04 kN) to account for the weight of the load cell, loading beam, gravity loading apparatus, nuts and bolts. Equation 4-5 was used to remove the inertia component from the wall resistance.

$$S_e' = S \pm \left(\frac{a \times g \times m}{1000 \times L} \right) \tag{4-5}$$

where,

S_e' = Wall resistance (corrected for inertia), [force per unit length, (kN/m)]

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

a = acceleration as measured by accelerometer, [g]

g = acceleration due to gravity [9.81 m/s^2]

m = mass [310 kg]

L = length of top of the wall [m]

The gravity loads also contributed to the lateral loads monitored by the load cell connected in line with the loading beam because of the inclination of the threaded rods as the wall displaced laterally. The inclined position of the shear wall in the test frame is depicted in Figure 4-20.

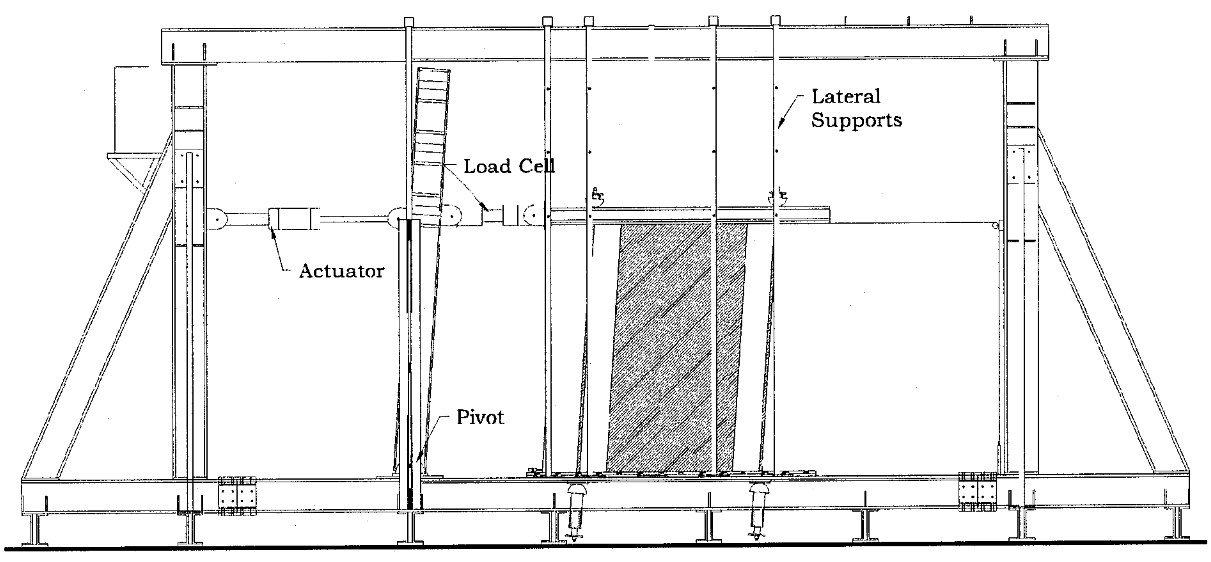


Figure 4-20: Inclined 4' x 8' (1220 x 2440 mm) shear wall in modified test frame

Equation 4-6 was utilized to remove the horizontal component of the force attributed to the gravity loading system.

$$S_g' = S - \left(\frac{v_1 \sin r_1 + v_2 \sin r_2}{L}\right) \tag{4-6}$$

where,

S_g' = Wall resistance (corrected for gravity component), [force per unit length, (kN/m)]

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

v_1, v_2 = gravity load on either side of the wall [kN]

r_1, r_2 = rotation of wall [radians]

L = length of the top of the wall [m]

During monotonic testing there were no accelerations to correct for since the test protocol ran at a constant velocity. However, for monotonic tests that included gravity loads the lateral components of these forces were corrected for using Equation (4-5). In cases where gravity loads were combined with reversed cyclic loading the lateral components produced due to rotations of the gravity loads were removed in addition to the inertial effects.

4.8.1 General Test Results

The direct results from the 32 test specimens listed in Table 4-1 are compiled in Table 4-5 (monotonic tests) and Tables 4-6 and 4-7 (cyclic tests) with corrections as described in Section 4.6 completed. Chapter 5 contains a more detailed discussion and interpretation of the test data. The test data sheets, loading protocols, table of mechanical properties and response curves can be found in Appendix C.

4.8.2 Energy Dissipation

The energy dissipation values, defined as the component of force acting through a deflection in a given direction, are listed in Table 4-5 through Table 4-7. In a plot of wall resistance (kN) versus net deflection (mm) the total dissipated energy (E, in joules) is represented by the area under the load displacement curve for a monotonic test up to $\Delta_{net0.8u}$. Likewise, for reversed cyclic tests the cumulative area enclosed by the hysteretic loops represents the cumulative energy dissipated during the test. The shear wall, in a structure designed for earthquake loading, is considered the sacrificial element (fuse) and is designed to act as the energy dissipater for the structure during earthquake loading. The structure's design should allow for a significant amount of energy to be absorbed by the fuse element such that the rest of the buildings integrity remains intact.

Recorded displacements and corrected wall resistance values were used to construct Riemann integrals (Equation 4-7) to calculate the total cumulative energy for each test (Equation 4-8).

$$\Delta E_i = \frac{F_i + F_{i-1}}{2} \times \left(\Delta_{net,i} - \Delta_{net,i-1}\right) \tag{4-7}$$

$$E = \sum \Delta E_i \tag{4-8}$$

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]

$\Delta_{net,\,i,\,i-1}$ = Net lateral displacement at data points (i) and (i-1), [mm]

E = Total cumulative energy [J]

Table 4-5 : Test results for monotonic tests

Test	Panel Type	Fastener Schedule	Maximum Wall Resistance (S_u) kN/m	Displ. @ 0.4S_u ($\Delta_{net,0.4u}$) mm	Displ. @ S_u ($\Delta_{net,u}$) mm	Displ. @ 0.8 S_u ($\Delta_{net,0.8u}$)	Rotation at S_u ($\theta_{net,u}$) rad (x 10^{-3})	Rotation at 0.8S_u ($\theta_{net,0.8u}$) rad (x 10^{-3})	Energy Dissipation (E) in Joules
47A	DFP	75/305	31.11	10.83	70.98	80.77	29.1	33.1	2400
47B	DFP	75/305	28.98	10.63	72.80	82.83	29.9	34.0	2302
47C[1]	DFP	75/305	32.40	11.25	70.12	74.43	28.8	30.5	2187
Average			30.83	10.90	71.30	79.34	29.2	32.5	2296
49A	OSB	152/305	10.92	3.31	38.41	60.12	15.8	24.7	691
49B[1]	OSB	152/305	11.75	4.13	40.61	52.67	16.7	21.6	617
49C	OSB	152/305	13.32	4.28	45.96	64.17	18.8	26.3	861
49D	OSB	152/305	12.13	3.71	40.32	52.16	16.5	21.4	625
Average			12.03	3.86	41.33	57.28	16.9	23.5	698
51A	OSB	75/305	22.17	4.88	42.01	54.47	17.2	22.3	1221
51B[1]	OSB	75/305	23.11	4.69	36.66	41.12	15.0	16.9	902
51C	OSB	75/305	22.35	4.02	36.86	48.5	15.1	19.9	1103
Average			22.54	4.53	38.51	48.03	15.8	19.7	1075
53A	CSP	152/305	13.39	6.67	57.06	77.44	23.4	31.8	1001
53B	CSP	152/305	12.41	9.25	55.71	81.58	22.8	33.5	976
53C[1]	CSP	152/305	13.15	5.71	55.84	76.12	22.9	31.2	951
Average			12.98	7.21	56.20	78.38	23.0	32.1	976
55A	CSP	152/305	25.68	11.01	70.19	88.80	28.8	36.4	2220
55B[1]	CSP	75/305	28.36	11.31	69.91	75.91	28.7	31.1	1932
55C	CSP	75/305	24.70	11.56	68.44	83.94	28.1	34.4	1982
55D	CSP	75/305	27.08	12.26	71.94	84.14	29.5	34.5	2108
Average			26.46	11.54	70.12	83.20	28.8	34.1	2061

[1] Test did not include gravity loads.

Table 4-6 : Test results for reversed cyclic tests (positive cycles)

Test	Panel Type	Fastener Schedule	Maximum Wall Resistance ($S_{u'+}$) (positive cycle) kN/m	Displacement as Su'+ ($\Delta_{net,u+}$) mm	Δnet, 0.8u+ (mm)	Rotation at S_{u+} ($\theta_{net,u+}$) rad (x 10^{-3})	Rotation at 0.8S_{u+} ($\theta_{net,0.8u+}$) rad (x 10^{-3})	Energy Dissipation, E Joules
48A	DFP	75/305	29.26	65.39	69.67	26.8	28.6	8492
48B	DFP	75/305	29.14	66.44	69.05	27.2	28.3	7777
48C[1]	DFP	75/305	28.35	50.05	74.47	20.5	30.5	7088
AVERAGE			28.92	60.63	71.06	24.8	29.1	7786
50A	OSB	152/305	10.77	33.24	51.68	13.6	21.2	3502
50B	OSB	152/305	10.49	30.84	48.83	12.6	20.0	3316
50C[1]	OSB	152/305	11.11	46.66	48.51	19.1	19.0	3294
AVERAGE			10.79	36.91	49.67	15.1	20.1	3371
52A	OSB	75/305	22.18	26.79	39.94	11.0	16.4	4617
52B	OSB	75/305	22.12	37.11	39.34	15.2	16.1	5394
52C[1]	OSB	75/305	25.64	41.25	42.21	16.9	17.3	5045
AVERAGE			23.31	35.05	40.50	14.4	16.6	5019
54A	CSP	152/305	11.95	56.06	65.12	23.0	26.7	4300
54B	CSP	152/305	12.16	58.66	66.4	24.1	27.2	4470
54C[1]	CSP	152/305	12.96	41.29	65.2	16.9	26.7	4118
AVERAGE			12.36	52.00	65.57	21.3	26.9	4296
56A[1]	CSP	75/305	26.90	57.13	58.05	23.4	23.8	7529
56B	CSP	75/305	25.56	60.29	63.78	24.7	26.2	7150
56C	CSP	75/305	25.85	57.6	59.52	23.6	24.4	6674
AVERAGE			26.10	58.34	60.45	23.9	24.8	7117

[1] Test did not include gravity loading.

Table 4-7 : Test results for reversed cyclic tests (negative cycles)

Test	Panel Type	Fastener Schedule	Maximum Wall Resistance ($S_{u'-}$) (negative cycle) kN/m	Displacement as Su'- ($\Delta_{net,u-}$) mm	Δnet, 0.8u- (mm)	Rotation at Su- (θnet,u-) rad (x 10-3)	Rotation at $0.8S_{u-}$ ($\theta_{net,0.8u-}$) rad (x 10^{-3})	Energy Dissipation, E Joules
48A	DFP	75/305	-27.99	-49.09	-47.68	-20.1	-19.6	8492
48B	DFP	75/305	-27.20	-47.25	-45.98	-19.4	-18.9	7777
48C[1]	DFP	75/305	-27.85	-41.68	-40.32	-17.1	-16.5	7088
AVERAGE			-27.68	-46.01	-44.66	-18.9	-18.3	7786
50A	OSB	152/305	-10.70	-33.01	-52.41	-13.5	-21.2	3502
50B	OSB	152/305	-10.06	-34.26	-53.3	-14.0	-21.9	3316
50C[1]	OSB	152/305	-10.79	-31.22	-49.59	-12.8	-20.3	3294
AVERAGE			-10.52	-32.83	-51.77	-13.4	-21.1	3371
52A	OSB	75/305	-20.83	-30.82	-30.7	-12.6	-12.6	4617
52B	OSB	75/305	-22.23	-27.1	-26.72	-11.1	-11.0	5394
52C[1]	OSB	75/305	-22.99	-29.58	-45.75	-12.1	-18.8	5045
AVERAGE			-22.01	-29.17	-34.39	-11.9	-14.1	5019
54A	CSP	152/305	-10.93	-43.43	-43.57	-17.8	-17.9	4300
54B	CSP	152/305	-11.56	-40.49	-65.71	-16.6	-26.9	4470
54C[1]	CSP	152/305	-11.59	-39.97	-40.45	-16.4	-16.6	4118
AVERAGE			-11.36	-41.30	-49.91	-16.9	-20.5	4296
56A[1]	CSP	75/305	-21.47	-45.98	-69.46	-18.9	-28.5	7529
56B	CSP	75/305	-22.60	-39.975	-63.82	-16.4	-26.2	7150
56C	CSP	75/305	-20.22	-39.13	-62.1	-16.0	-25.5	6674
AVERAGE			-21.43	-41.70	-65.13	-17.1	-26.7	7117

[1] Test did not include gravity loading.

4.9 Observed Failure Modes

The failure of all specimens was attributed to the deterioration in load carrying capacity of the sheathing panel to light gauge steel frame connections, as was typically observed for the tests carried out by Branston (2004), Chen (2004), Boudreault (2005), Blais (2006) and Rokas (2006). In no test wall did failure by local buckling of the chord studs take place. The failure modes of the sheathing connections can be classified into five main categories, which are described as follows:

1. Pull-through sheathing (PT)

   Rotation of the screw during testing and tension force in the screw caused the head to penetrate the sheathing and eventually pull-through (Figure 4-21). The rocking motion of the screw expanded the fastener hole allowing the head to pass through. The edge of the panel remained undamaged. Partial pull-through (PPT) of the fasteners was also observed; that is, when the screw head remained embedded within the panel thickness. (Figure 4-22)

2. Wood bearing / plug shear failure (WB)

   This type of failure was only characteristic of walls with plywood panels. One or more plies of the panel would fail with at least one remaining intact (Figure 4-23). Further loading or cycling of the wall would likely cause a complete tear-out at the edge of the panel in that region.

3. Tear-out of sheathing (TO)

   As noted above, extensive bearing or plug shear damage to an edge connection would typically result in the screw fastener tearing out of the wood panel. This was common for plywood panels, and was also observed for the OSB panels (Figure 4-24).

4. Pull-through framing

   In plywood walls with a 75/305 mm fastener pattern which used self-drilling screws (test series 47, 48, 55 and 56) a sudden pull-through of the screws through the a framing member occurred. In general, the failure occurred all along one edge of the panel after the peak resistance had been obtained by the wall. This failure mode was due to the self-drilling removing more steel surrounding the fastener in comparison to the self-piercing screws and the thread pattern was not deep enough to compensate for the loss. (These walls were configured with 1.37 mm (0.54") and No. 8 x 31.8 mm (1-1/4") Grabber SuperDrive (SuperDriver, 2003) Bugle head self-drilling sheathing screws.) (Figure 4-25)

5. Unzipping of plywood

   The unzipping of the fasteners along the edge of the panel could occur with any one or a combination of the four failure modes listed above (Figure 4-26). This event generally occured after the peak resistance of the wall had been obtained. If the panel edge does not unzip fully along one side the framing can undergo bending at the hinge created by the variation in bracing in the member as described in Branston et al. (2006b).

6. Shear of screws

   In some instances the fasteners failed in shear. This failure occurred in the areas where two layers of steel overlapped in the frame. The increased thickness in the steel restricted the screws ability to rotate and forcing it to undergo higher shear loads than experience by fasteners in other regions of the wall. Also, in shear walls configured with 1.37 mm studs and No. 8 screws the screws were observed to also fail in shear. The thicker stud in conjunction with a smaller diameter screw created higher shear stresses similarly to the previously mentioned scenario.

Figure 4-21 : Pull-though sheathing

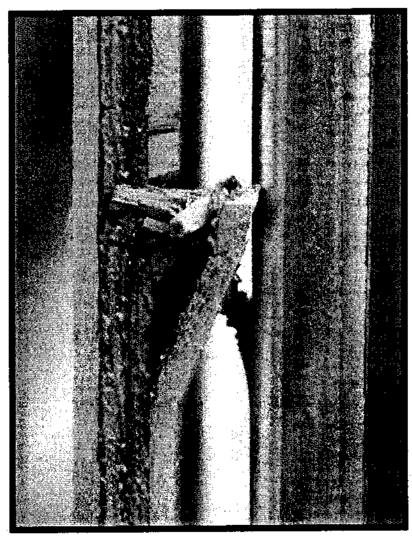


Figure 4-22 : Partial pull-through sheathing

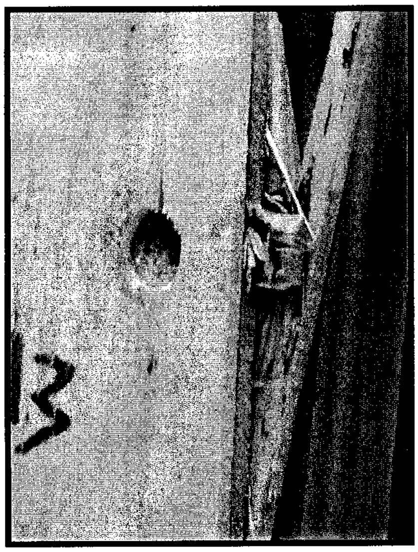


Figure 4-23 : Wood bearing / plug shear failure

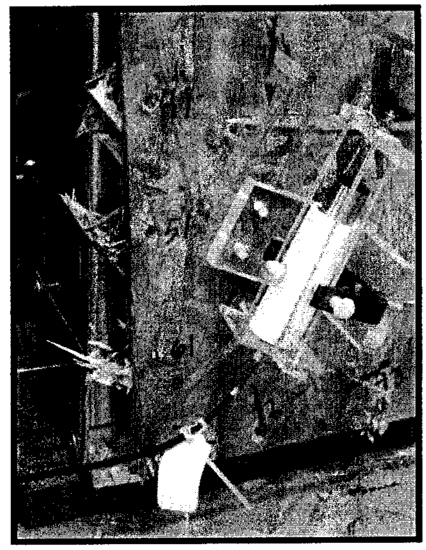


Figure 4-24 : Tear-out of sheathing

Figure 4-25 : Pull-through from framing

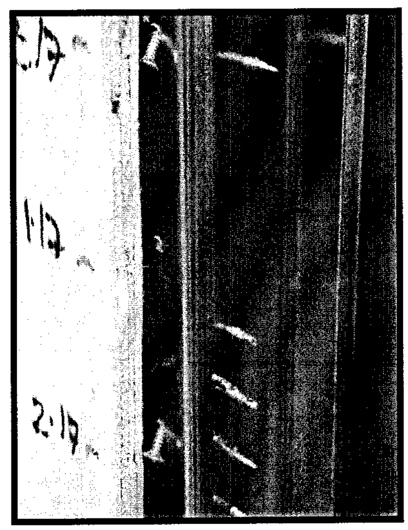


Figure 4-26 : Unzipping of sheathing connections

All tests resulted in a combination of these previously mentioned mechanisms. No damage to the hold downs, the hold down anchors or the shear anchors was observed. Any damage occurring to the tracks was minor and did not compromise the resilience of the frame. In locations where the sheathing screws penetrated two layers of steel (track to stud overlap), shear failures occurred in some instances. In these cases the shank of the screw remained embedded in the steel and the head of the screw remained in the panel. As well as the higher shear forces, the reversed cyclic loading protocol may have also caused strength degradation in these elements due to fatigue.

The performance of all the walls was governed by the sheathing to framing connections. In general, the unzipping of at least one panel edge led to a significant degradation in strength, which signaled failure in the system. Once this event occurred the wall was forced to transfer lateral loads by means of stud bending and through its framing connections to the hold downs and shear anchors joined to the loading frame. The flexural load transfer as well as the increased ability of the wall to rotate sometimes caused local buckling in the lips and flanges of the DCS, as reported by Branston (2004). Even with this extensive damage the light gauge steel frame remained able to carry the applied gravity loads without a loss in capacity.

4.10 Ancillary Material Testing

The average material properties for the studs, track and sheathing panels were measured, and are presented herein. Multiple specimens were tested for each of the three different steel products and three wood types.

4.10.1 Wood Panel Properties

The wood specimens were tested in shear following ASTM D1037 (edgewise shear) (1999). A complete description of the test procedure can be found in the work of Boudreault (2005) and Blais (2006). For each type of wood a total of six specimens were prepared; three specimens were cut parallel to the surface grain of the panel and the other three specimens perpendicular to grain. It was found that the shear properties were not directionally dependent. The resulting average values are shown in Table 4-8.

Table 4-8 : Measured material properties for wood panels

Specimen	Thickness (mm)	Ultimate Shear Strength (MPa)	Shear Modulus (MPa)	Rigidity (N/mm)
12.5 mm DFP	12.54	4.97	923	11584
12.5 mm CSP	12.41	4.24	814	10080
11 mm OSB	11.25	8.12	1402	15755

4.10.2 Light Gauge Steel Stud Properties

The steel coupons underwent tension tests in accordance with ASTM A370 (2002) requirements. A cross-head rate of 0.5 mm/min in the elastic range and 4 mm/min in the inelastic range was applied. All coupons were taken from the centre of the web of the stud and track members, and in the direction of rolling. After testing the coupons were soaked in a 25% hydrochloric acid (HCL) solution to remove the zinc coating. The base metal thickness was then measured and used for the calculation of all material properties. The North American Specification for Cold-Formed Steel Members (AISI, 2001) requires that the coupon elongation over a 50 mm gauge length must be at least 10% and that the $F_u/F_y \geq 1.08$. Table 4-9 lists the material properties of the studs and tracks used in this test series.

Note, the measured yield strength, F_y, for the 1.37 mm thick studs was below the minimum specified value of 340 MPa. However, in this case because the base metal thickness was greater than the minimum specified value (1.46 mm vs. 1.37 mm) the chord studs were able to attain an axial capacity that corresponded to the nominal level expected using calculations documented in the CSA S136 Specification (2001). As well, the test values for the modulus of elasticity for two of the three specimens (1.37 mm section and 1.09 mm track) were found to be under the nominal value of 203 GPa. The calculations were carried out to determine if this lowered value caused the axial capacity to be lower than that of a nominal section using the calculations documented in the CSA S136 Specification (2001). Both sections were found to have a higher than nominal axial capacity.

Table 4-9 : Measured material properties for steel products

Specimen	Member	Base Metal Thickness (mm)	Fy (MPa)	Fu (MPa)	Fu / Fy	E (GPa)	% Elong. 50 mm Gauge
1.09 mm	Stud	1.12	246	321	1.30	220	32.4
1.37 mm	Stud	1.46	324	422	1.30	200	31.5
1.09 mm	Track	1.12	254	321	1.26	196	30.7

4.11 Influence of Gravity Loading on Shear Wall Behaviour

The objective of carrying out the shear wall tests was to evaluate the performance of steel frame / wood panel walls that are subjected to combined lateral and gravity loading. One wall per configuration was tested with only a lateral load such that its performance could be compared with the remaining walls that carried both lateral and gravity loads. In this fashion a direct comparison of nominally identical walls could be achieved. However, previous testing has indicated some degree of variability exists in the measured performance from one test specimen to another even if they are considered to be nominally identical in terms of construction and loading. This variability may be attributed to a change in the manufacturer of a particular type of sheathing (Chen, 2004 and Rokas, 2006), the natural variation in the material properties of a particular wood species, a variation in placement (location and quality) of sheathing fasteners, pre-existing damage at sheathing connection locations, etc. This section contains a discussion of the measured shear wall parameters as listed in Tables 4-5 to 4-7. A discussion of the influence of gravity loads on design parameters is contained in Section 5.8.

Table 4-10 features the normalized displacement at $0.8S_u$, dissipated energy and the ultimate shear resistance for all tests within this study. The tabulated ratios were determined by dividing the result (displacement, energy, or resistance) for each test specimen by the result for the specimen that was subjected to lateral loading alone, within the same configuration and loading protocol (monotonic or cyclic).

Table 4-10 : Normalized properties of shear walls (combined loading/lateral loading)

Specimen	47 Series[1]	48 Series[2]	49 Series[1]	50 Series[2]	51 Series[1]	52 Series[2]	53 Series[1]	54 Series[2]	55 Series[1]	56 Series[2]
Displacement at 0.8S_u										
A	1.09	1.02	1.14	1.06	1.32	0.80	1.02	1.03	1.17	1.00*
B	1.11	1.00	1.00*	1.04	1.00*	0.75	1.07	1.25	1.00*	1.00
C	1.00*	1.00*	1.22	1.00*	1.18	1.00*	1.00*	1.00*	1.11	0.95
D	N/A	N/A	0.99	N/A	N/A	N/A	N/A	N/A	1.11	N/A
Dissipated Energy										
A	1.10	1.20	1.12	1.06	1.35	0.92	1.05	1.04	1.15	1.00*
B	1.05	1.10	1.00*	1.01	1.00*	1.07	1.03	1.09	1.00*	0.95
C	1.00*	1.00*	1.40	1.00*	1.22	1.00*	1.00*	1.00*	1.03	0.89
D	N/A	N/A	1.01	N/A	N/A	N/A	N/A	N/A	1.09	N/A
Ultimate Resistance, S_u										
A	0.96	1.02	0.93	0.98	0.96	0.88	1.02	0.93	0.91	1.00*
B	0.89	1.00	1.00*	0.94	1.00*	0.91	0.94	0.97	1.00*	1.00
C	1.00*	1.00*	1.13	1.00*	0.97	1.00*	1.00*	1.00*	0.87	0.95
D	N/A	N/A	1.03	N/A	N/A	N/A	N/A	N/A	0.95	N/A

[1] Monotonic Test Series

[2] Reversed Cyclic Test Series

"*" Shear wall specimen not subjected to gravity load. Energy, Displacement at $0.8S_u$ and Ultimate Resistance S_u of this wall used to calculate ratio for other walls within same configuration.

A visual comparison of the measured properties of the test walls was made possible through the creation of a series of bar charts comparing the displacement at $0.8S_u$, energy and ultimate shear resistance (Figs. 4-27 – 4-32). The asterisk "*" in these figures denotes that no gravity loads were applied to the wall during testing. Figure 4-27 illustrates that the walls under combined monotonic loading consistently had larger displacements at failure, defined earlier as the displacement corresponding to 80% of the ultimate load (post peak), than walls that were not subjected to a gravity load. An exception exists for Test 49D, which reached 99% of the displacement attained by 49B. The CSP and DFP sheathed shear walls were more consistent than the OSB walls in terms of displacement at $0.8S_u$. The normalized ratios had a range of 0.07 and 0.11 for test series 53 and 49 respectively, while test series 51 had a range of 0.32. Series 55 with CSP sheathing and a screw schedule of 75/305 mm reached the highest displacement of 88.80 mm under combined gravity and lateral loading.

The average value of the positive and negative displacement at $0.8S_u$ for each individual reversed cyclic test was used to obtain the values shown in Figure 4-28. Unlike the monotonic tests, the $\Delta_{net0.8u}$ values for the reversed cyclic tests did not show a consistent pattern (Figure 4-27). In some cases the tests with lateral loading alone failed at displacements beyond those recorded for tests with combined loading, eg. 52C* vs. 52A and 52B, while for other configurations the opposite was true. In the case of test series56 the failure deformation of the laterally loaded specimen fell between the two walls

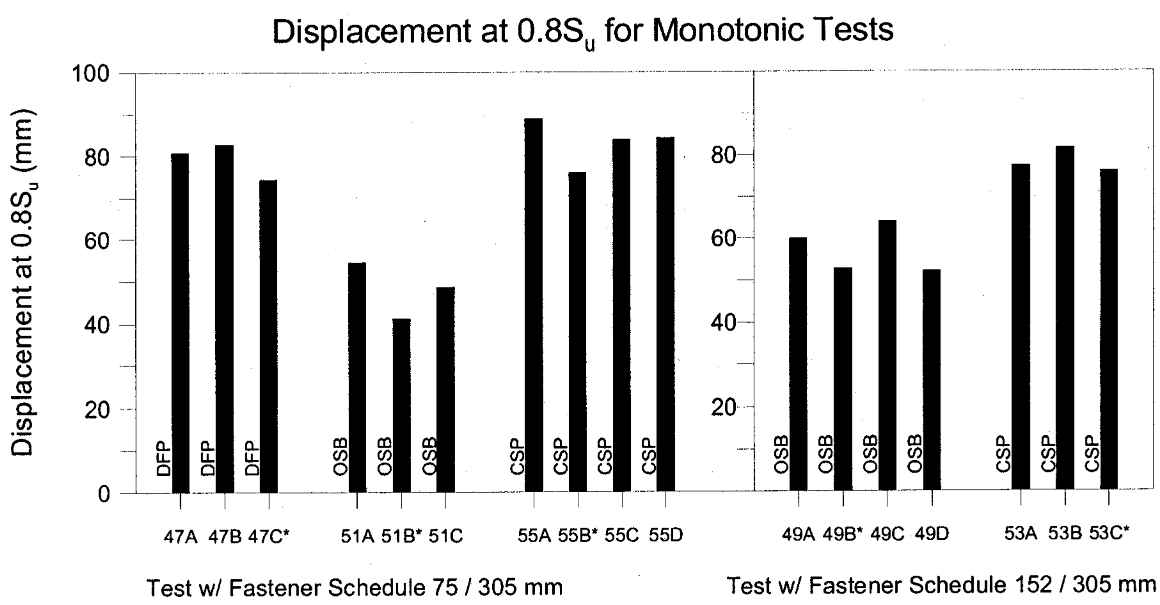


Figure 4-27 : Comparison of displacement at $0.8S_u$ for monotonic tests

with combined loading. The DFP sheathed panels, series 48, were extremely consistent regardless of the loading combination. The normalized results ranged from 1.00 to 1.02 for all three tests. Reviewing all the cyclic tests the normalized values varied 0.5 from 0.75 to 1.25.

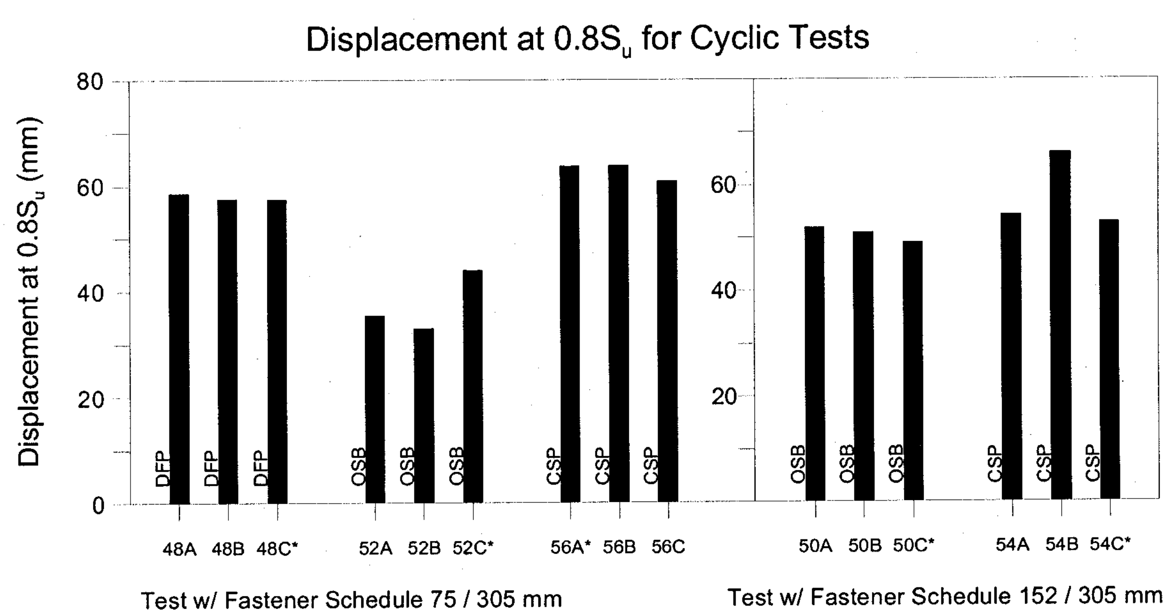


Figure 4-28 : Comparison of displacement at $0.8S_u$ for cyclic tests

Figure 4-29 illustrates the range in energy dissipation between the various monotonic tests. In general, the energy was higher for the walls with combined loading because as noted above these specimens displaced further than those that were only laterally loaded, including test 49D. The DFP sheathed series 49 dissipated the most energy overall with test 47A releasing 2399 J. The CSP sheathed walls (series 53 and 55) doubled their ability to dissipate energy when the screw schedule changed from 152/305 mm (6"/12") with 1.09 mm (0.043") thick chord studs to 75/305 (3"/12") mm with 1.37 mm (0.054") thick chord studs. The normalized results from these tests were also more consistent with a maximum range of 1.00-1.15 between results. In contrast, the shear walls sheathed with 11 mm OSB (series 49 and 52) did not experience as a dramatic improvement in performance with the increased in density of screws around the perimeter. Their range of normalized results of 1.00-1.40 also indicated that OSB sheathing is more sensitive to the loading combination than the CSP.

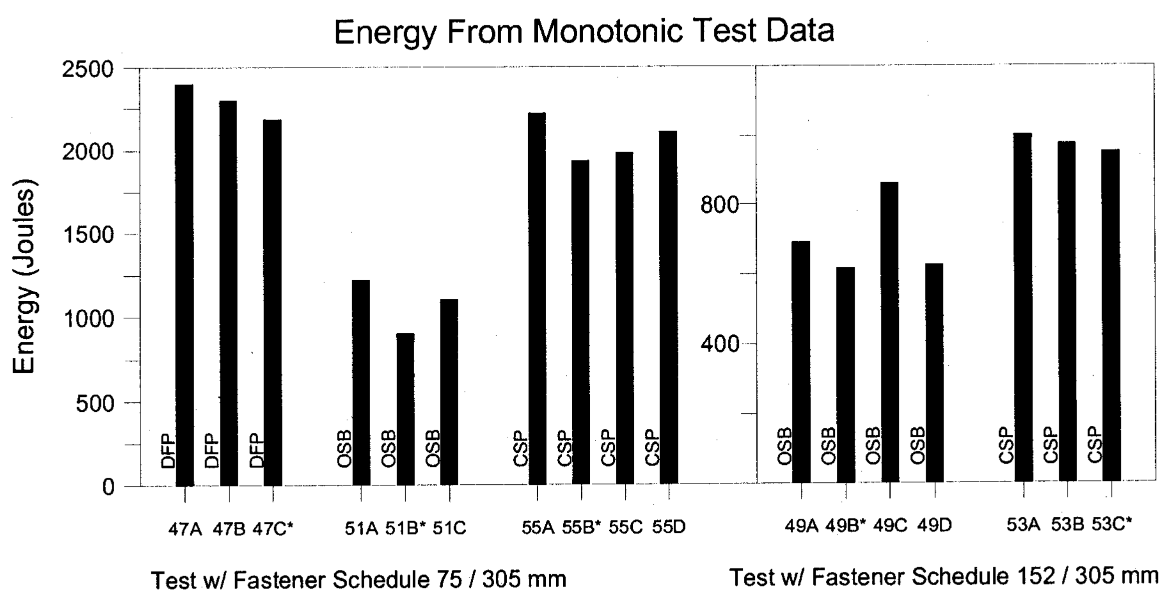


Figure 4-29 : Comparison of energy dissipated for monotonic tests

A comparison of energy dissipation results for the reversed cyclic tests is provided in Figure 4-30. In this comparison the hysteretic energy within all of the positive and negative loops of the force vs. deformation curve for a particular wall was used. Three of the five series (48, 50 & 54) showed that the walls with gravity and lateral loads were able to dissipate more energy than the walls that underwent lateral deformations alone. Similar to the monotonic results, the specimens that displaced further dissipated more energy. The energy dissipated by the specimens in series 48 under combined loading was up to 20% greater than the specimen under lateral loads only. However, the amount of dissipated energy for tests 50B and 50C were very similar even though test 50C did not carry gravity loads. The change in fastener schedule seemed to influence the energy dissipation of the CSP sheathed walls. The sparser schedule 152/305 mm specimens (series 54) saw improved energy dissipation with combined loading while the denser schedule, 75/305 mm (series 56), had reduced energy dissipation when gravity loading was included. A similar, but less distinct pattern is noted in the results of the OSB sheathed series (series 50 and 52), however the DFP sheathed configuration (series 48) does not follow this trend set by the denser screw spacing.

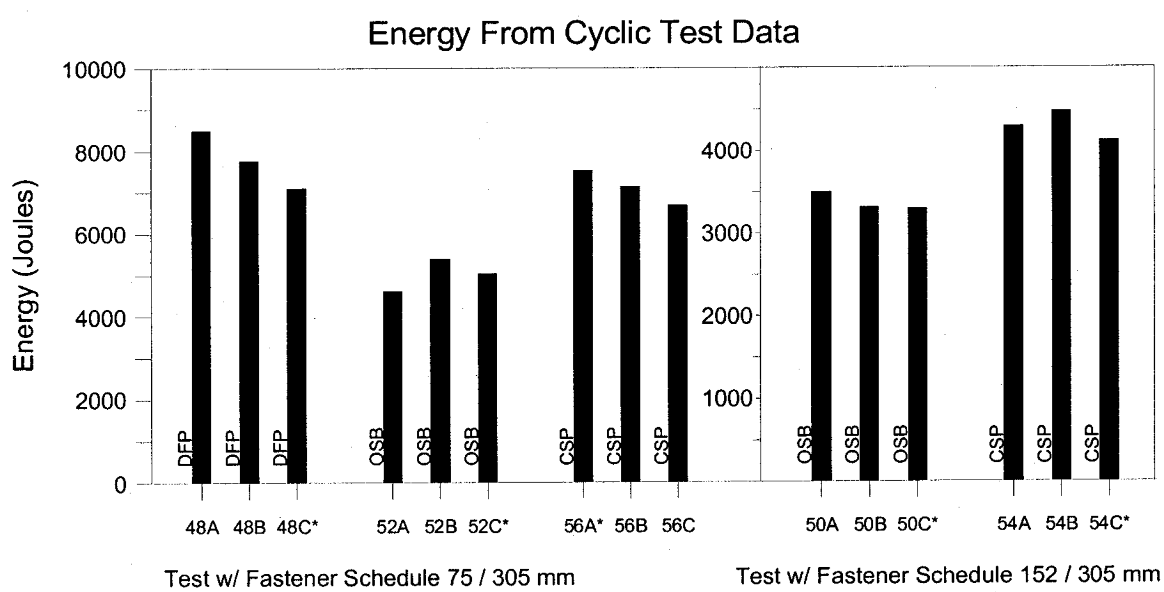


Figure 4-30 : Comparison of energy dissipated for cyclic tests

A comparison of the ultimate shear resistance reached by the monotonic tests is pictured in Figure 4-31. In general the inclusion of gravity loads did not increase the ultimate shear resistance of the shear walls. In three of the series (47, 51, 55) the test which did not include gravity loads during testing obtained the highest shear resistance. Within the remaining two series (49, 53) these tests were in the middle of the observed range. The trend in ultimate shear resistance can be linked to the density of the screw schedules the series 47, 51 and 55 all had 75 / 305 mm screw schedules and improved performance without the inclusion of gravity loads. The range of the normalized values for series 49 and 53, with a less dense fastener schedule of 150 / 305 mm, contained tests above and below unity

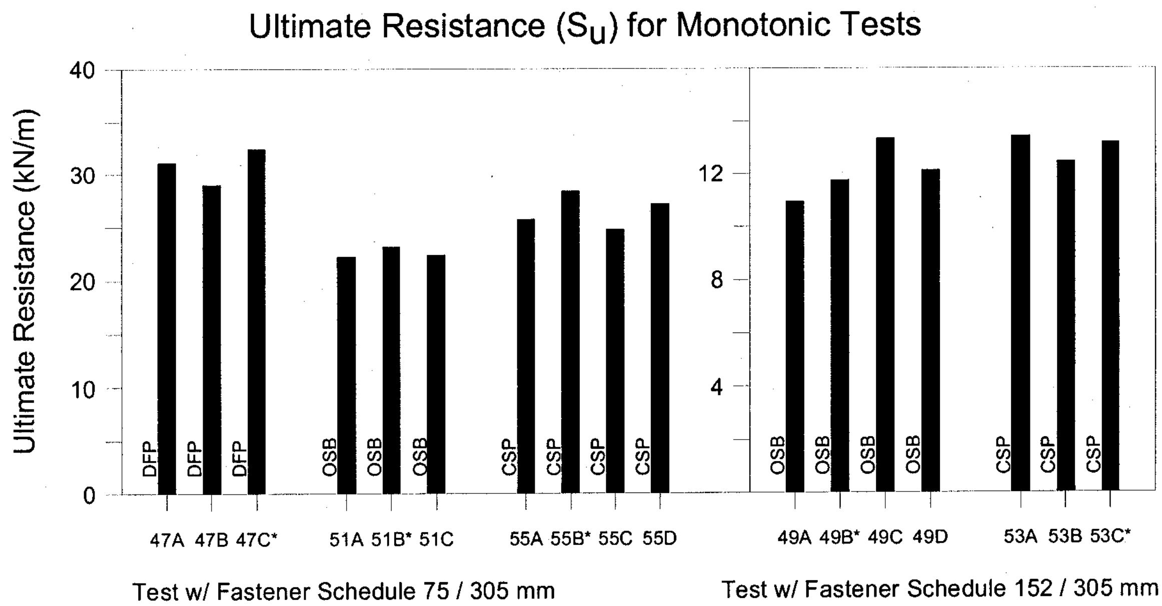


Figure 4-31 : Comparison of ultimate resistance for monotonic tests

Figure 4-32 contains the ultimate shear strength measures of the reversed cyclic shear wall tests. Note, the average of the absolute values of the ultimate force in each direction recorded during testing was used. The test walls without gravity loads reached a higher ultimate resistance than those with combined loads, except for series 48. Nonetheless, the variation in ultimate resistance between the different loading types was minor (Table 4-9), sometimes as small as 0.12 kN/m, and thus it cannot be concluded that any distinct and systematic difference existed between the combined and lateral loading wall specimens. The minimum normalized ratios for series 50, 52 and 54 were 0.94, 0.88 and 0.93 respectively. The results of series 48 were very consistent for ultimate resistance with a normalized range between 1.00 – 1.02 (28.10 – 28.62 kN/m). Overall, since the values for ultimate shear resistance are not highly deviant for monotonic and reversed-cyclic testing this variation in results can more than likely be attributed to variations in material properties and construction methods and not screw spacing.

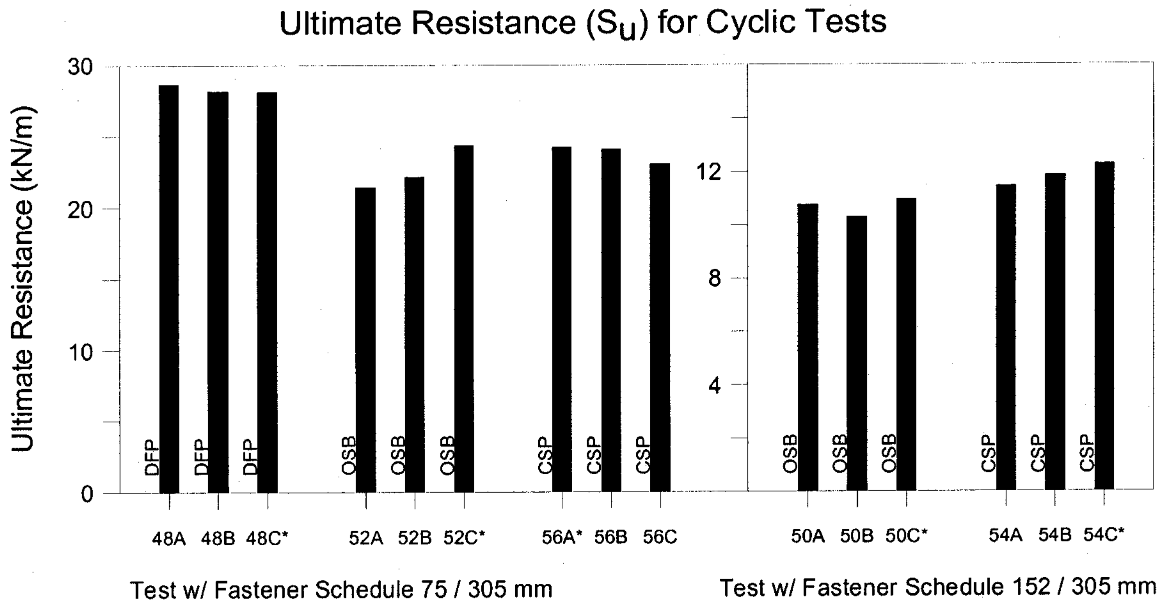


Figure 4-32 : Comparison of ultimate resistance for cyclic tests

The comparison of the measured shear wall deformation, energy and strength parameters indicates that there was no consistent or distinct influence on the lateral load carrying performance of steel frame / wood panel shear walls due to the inclusion of gravity loads. In some cases the gravity loads improved the measured parameters, whereas in others the gravity loads caused the opposite to occur; for example, the ultimate shear resistance was slightly lower for the walls with a gravity load compared with that recorded for the walls subjected to lateral loads alone. Much of the variation in the measured parameters can likely be attributed to a variation in the material properties, construction, etc, from one wall specimen to another.

The sustained lateral performance under gravity loads is dependent on the fact that the chord studs were designed following a capacity based approach (see Section 4.1), whereby failure of the sheathing connections was forced to occur. If an inappropriately sized chord stud were selected, because the ultimate shear load of the wall were incorrectly predicted or if gravity companion loads were not considered, then it would be possible for local buckling of the stud members to occur. In this instance a degradation of the strength, deformation capacity (ductility) and energy dissipation capability of the walls may be observed when gravity loads are imposed with lateral loads. Nonetheless, if steel frame /wood panel shear walls are properly designed and constructed, such that chord stud failure can be avoided, then the lateral performance of the walls can be represented by shear wall tests in which lateral loads alone are applied.

CHAPTER 5 RECOMMENDED DESIGN PARAMETERS FOR SHEAR WALLS

5.1 INTRODUCTION

The shear wall test program described in Chapter 4 is a continuation of research carried out by Blais (2006), Boudreault (2005), Branston (2004), Chen (2004) and Rokas (2006) at McGill University. To parallel the previous research the same methods of data interpretation were used to help expand the data set and draw conclusions from a larger body of work. The recommended design parameters were determined using the equivalent energy elastic-plastic (EEEP) analysis approach for the 32 shear wall specimens. Branston (2004) concluded that this approach was the best suited to the steel-frame / wood panel shear wall after reviewing multiple analysis methods. The successful use of the design parameters to determine the wall configurations for this test program, specifically the selection of the chord studs, reinforce their applicability and continued use. Figure 5-1 illustrates the non-linear behaviour of the wall in a measured resistance vs. deflection graph. The bi-linear EEEP curve, also included in Figure 5-1, is intended to represent the behaviour of the wall based on its energy dissipation capabilities. The EEEP data interpretation method provides nominal design values for strength and stiffness. It also provides a measure of the inherent ductility of the wall that can be used to define a test based seismic force modification for design. The EEEP method was chosen for this study because historically it has been used to analyze structural systems exhibiting non-linear behaviour and can be applied irrespective of the loading protocol implemented (Branston, 2004 and 2006b).

The analysis of data from the test series described in Chapter 4 has been combined and compared with previous applicable test information to provide a more comprehensive recommendation for design parameters. An in-depth summary of the EEEP analysis approach can be found in Branston (2004), therefore only a summary has been provided in this Chapter.

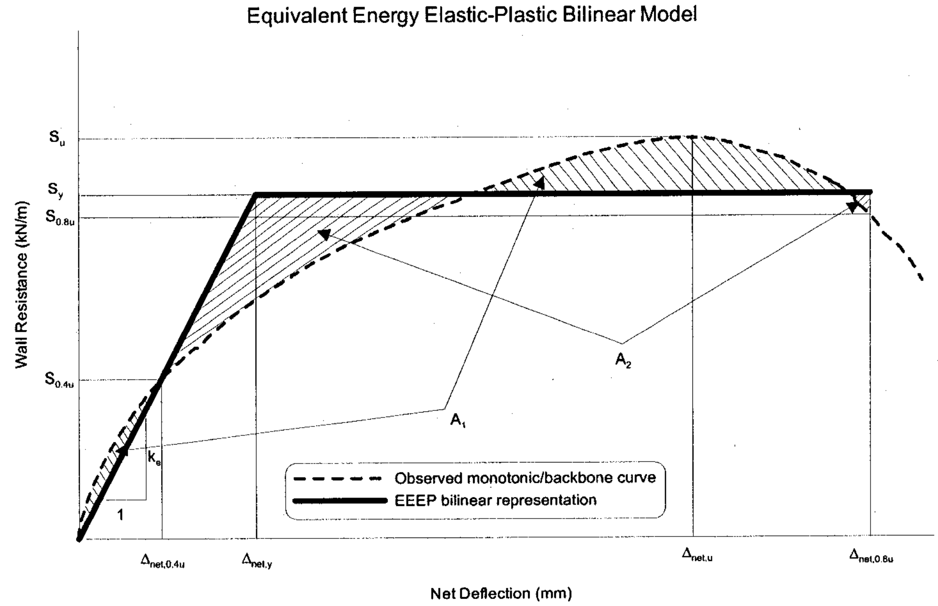


Figure 5-1 : EEEP model ***(Park, 1989; Salenikovich el at. 2000b; Branston, 2004)***

5.2 YIELD STRENGTH AND STIFFNESS

To determine the nominal yield strength, initial stiffness and ductility of each shear wall specimen a bi-linear curve was determined using the EEEP approach. The curve is based on the dissipated energy, that is, the area under a monotonic test curve or the backbone curve of a reversed cyclic test. An integration of the area under the curve is performed up to the 80% post-peak wall resistance ($0.8S_u$) level (Figure 5-1). This point denotes the limit of the useful capacity of the shear wall. To determine the stiffness of a specimen a line is constructed between the origin and the point on the test curve representing 40% of the ultimate resistance ($0.4S_u$). This load level is considered an appropriate estimate of the displacement at the maximum service load. The yield plateau (S_y) is then determined by matching the area under the test curve up to the point of failure with the area under the bi-linear curve. In a similar fashion the energy balance can also be determined by using the areas represented by A_1 and A_2 (Figure 5-1).

To efficiently calculate the value of S_y a Microsoft Excel Marco was written by Boudreault (2005) and then modified for combined loading for the purposes of this research. The result of the following mathematical derivation of the wall resistance at yield as follows was applied in the program:

The area under the EEEP curve up to $0.8S_u$,

$$A_{EEEP} = \frac{S_y \times \Delta_{net,y}}{2} + \left[S_y \times \left(\Delta_{net,0.8u} - \Delta_{net,y}\right)\right] \tag{5-1}$$

Applying the equivalent energy concept the area under the test curve, A, is equal to A_{EEEP},

$$A_{EEEP} = A = \frac{S_y \times \Delta_{net,y}}{2} + \left[S_y \times \left(\Delta_{net,0.8u} - \Delta_{net,y}\right)\right] \tag{5-2}$$

Since $\Delta_{net,y} = S_y / k_e$ using the definition of initial elastic stiffness Equation 5-2 can be reduced and the following quadratic relationship results:

$$A = -\left(S^2{}_y / 2k_e\right) + S_y \times \Delta_{net,0.8u} \tag{5-3}$$

Solving for S_y gives,

$$S_y = \frac{-\Delta_{net,0.8u} \pm \sqrt{\Delta_{net,0.8u}{}^2 - \dfrac{2A}{k_e}}}{-\dfrac{1}{k_e}} \tag{5-4}$$

where,

$$k_e = \frac{0.4 \times S_u}{\Delta_{net,0.4u}} \tag{5-5}$$

k_e = Unit elastic stiffness, [force per length per wall length (kN/m/mm)]

S_y = Wall resistance at yield, [force per unit length (kN/m)]

S_u = Ultimate wall resistance, [force per unit length (kN/m)]

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

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

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

To determine the ductility of the wall from the EEEP model curve Equations 5-6 and 5-7 were calculated using test data.

$$\Delta_{net,y} = \frac{S_y \times L}{K_e} \tag{5-6}$$

$$\mu = \frac{\Delta_{net,0.8u}}{\Delta_{net,y}} \tag{5-7}$$

where,

K_e = Elastic stiffness, [force per unit length (kN/mm)]

L = Length of the wall specimen [1.22 m (4')]

μ = Ductility

The ductility of the specimen is integral in the latter calculations of the ductility related force modification factor (Section 5.7). Examples of EEEP curve construction can be seen in Figure 5-2 for a monotonic response curve and Figure 5-3 for a reversed cyclic backbone curve.

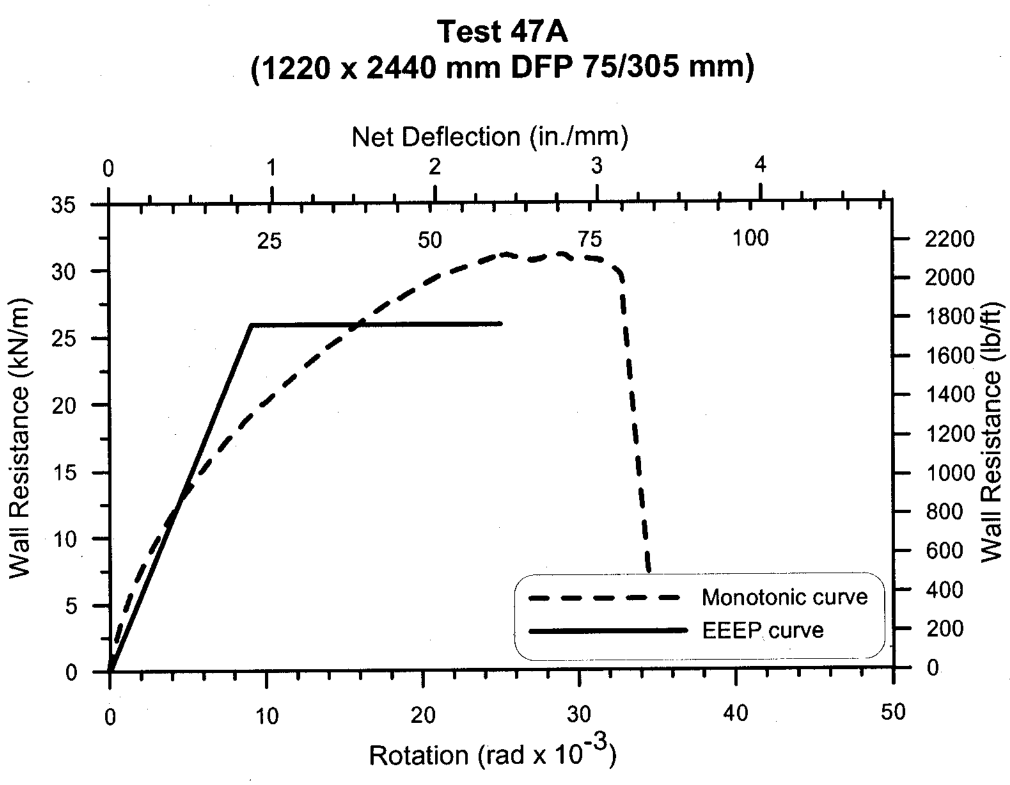


Figure 5-2 : EEEP curve for monotonic test 47A

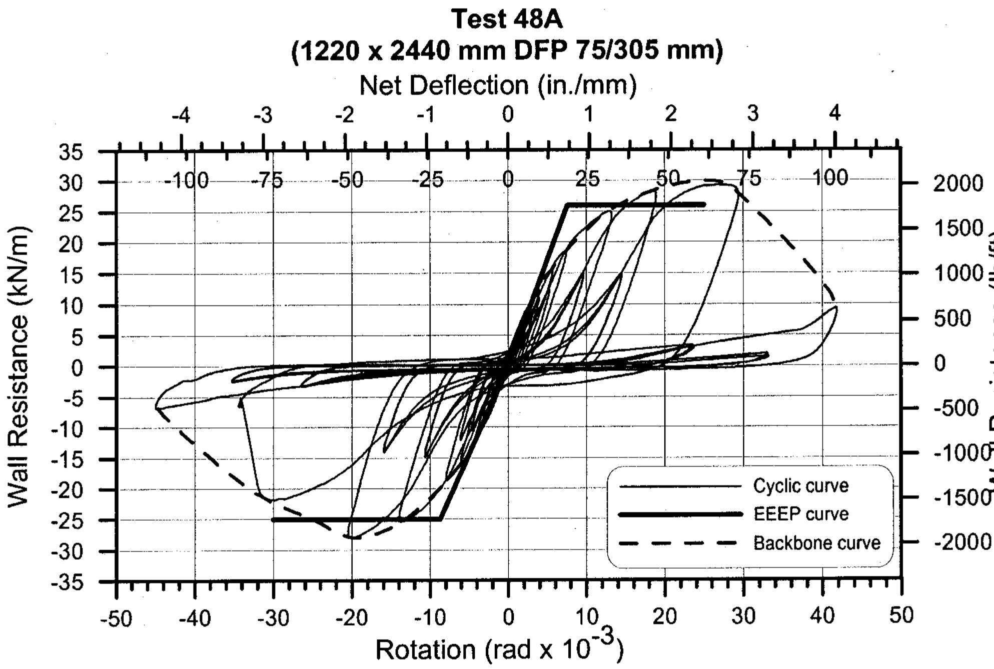


Figure 5-3 : EEEP curve for cyclic test 48A

As previously mentioned, the EEEP curve is typically limited by $\Delta_{net,0.8u}$. It is possible that the deformation at a post peak load of $0.8S_u$ exceeds that of the 2.5% inelastic drift limit prescribed by the 2005 NBCC (NRCC, 2005), in which for a wall 2440 mm (8') in height a maximum drift of 61 mm is permitted. In Figure 5-2 the drift limit was less than $\Delta_{net,0.8u}$, as well as being less than the deformation at ultimate load $\Delta_{net,u}$, hence, the EEEP curve was determined following a Case 1 approach (Figure 5-4). It is also possible for a Case 2 analysis situation to exist, where the inelastic drift limit falls between $\Delta_{net,u}$ and $\Delta_{net,0.8u}$. In this case $\Delta_{net,0.8u}$ was used in the definition of the EEEP curve (Figure 5-5), which is the same approach as used for the general case shown in Figure 5-1. A more in-depth description of these analyses cases can be found in Branston (2004). For all specimens the EEEP procedure was implemented, with the resulting parameters tabulated in Tables 5-1 through 5-3.

Case 1: 61 mm < $\Delta_{net,u}$

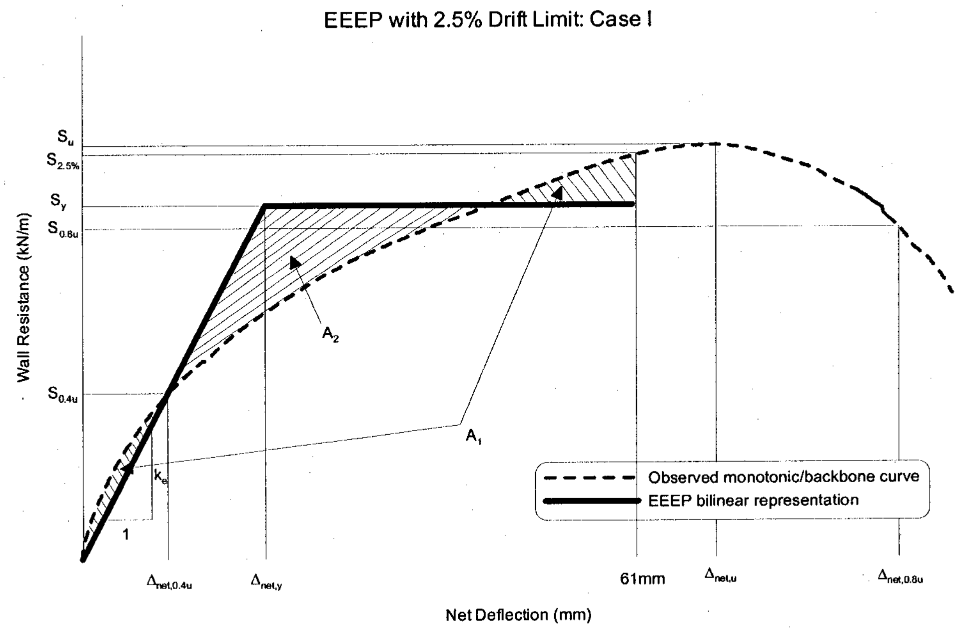


Figure 5-4 : EEEP design curve with imposed 2.5 % drift limit (Case I) (*Branston, 2004*)

Case 2: $\Delta_{net,u}$ < 61mm < $\Delta_{net,0.8u}$

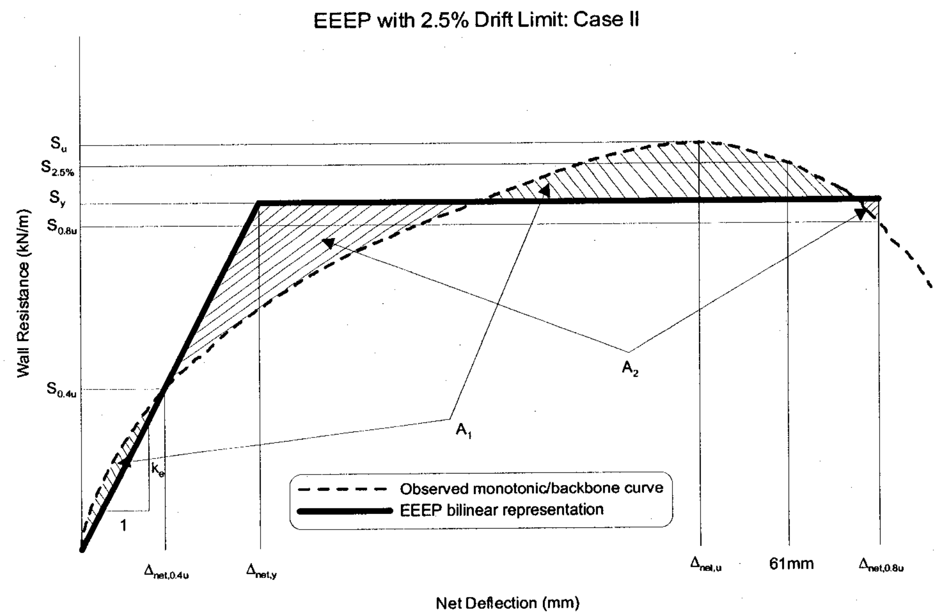


Figure 5-5 : EEEP design curve with imposed 2.5 % drift limit (Case II) (*Branston, 2004*)

Table 5-1 : Design values for monotonic tests

Test	Panel Type	Fastener Schedule	Yield Load (S_y) kN/m	Displacement at 0.4Su ($\Delta_{net, 0.4u}$) 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	Governing Case
47A	DFP	75/305	25.86	10.66	21.16	1.42	8.68	2.75	1573	1
47B	DFP	75/305	23.86	10.31	21.24	1.37	8.71	2.87	1465	1
47C[1]	DFP	75/305	26.14	11.25	22.69	1.40	9.31	2.69	1581	1
Average 47			25.29	10.74	21.70	1.40	8.90	2.77	1540	
49A	OSB	152/305	9.88	3.16	7.09	1.69	2.91	8.36	668	General
49B[1]	OSB	152/305	10.51	4.08	9.13	1.40	3.74	5.77	617	General
49C	OSB	152/305	11.63	4.12	9.00	1.58	3.69	7.07	838	General
49D	OSB	152/305	10.44	3.60	7.74	1.64	3.17	6.73	614	General
Average 49			10.62	3.74	8.24	1.58	3.38	6.98	684	
51A	OSB	75/305	20.27	4.80	10.98	2.25	4.50	4.96	1209	General
51B[1]	OSB	75/305	20.60	4.68	10.43	2.41	4.28	3.94	902	General
51C	OSB	75/305	20.37	3.92	8.93	2.78	3.66	5.43	1092	General
Average 51			20.41	4.47	10.11	2.48	4.15	4.78	1068	
53A	CSP	152/305	11.38	6.41	13.61	1.02	5.58	5.66	973	General
53B	CSP	152/305	10.85	8.85	19.35	0.68	7.94	4.19	945	General
53C[1]	CSP	152/305	11.13	5.69	12.03	1.13	4.93	6.33	951	General
Average 53			11.12	6.98	15.00	0.94	6.15	5.39	956	
55A	CSP	75/305	20.90	10.85	22.08	1.15	9.06	2.76	1272	1
55B[1]	CSP	75/305	22.81	11.29	22.69	1.23	9.31	2.69	1380	1
55C	CSP	75/305	20.74	11.38	23.88	1.06	9.79	2.55	1239	1
55D	CSP	75/305	21.33	12.06	23.70	1.10	9.72	2.57	1277	1
Average 55			21.45	11.40	23.09	1.14	9.47	2.64	1292	

[1] Gravity loads not applied to test wall

Table 5-2 : Design values for reversed cyclic tests (positive cycles)

Test	Panel Type	Fastener Schedule	Yield Load (S_{y+}) kN/m	Displacement at S_{y+} ($\Delta_{net, y+}$)	Elastic Stiffness (K_{e+}) kN/mm	Rotation at S_{y+} ($\theta_{net,+}$) rad (x 10^{-3})	Ductility μ_+	Energy Dissipation[2] E Joules +	Governing Case+
48A	DFP	75/305	25.96	18.41	1.72	7.55	3.31	1638	1
48B	DFP	75/305	25.74	18.11	1.73	7.43	3.37	1629	1
48C[1]	DFP	75/305	26.78	26.37	1.19	10.81	2.98	2056	General
AVERAGE			26.16	20.96	1.55	8.60	3.22	1774	
50A	OSB	152/305	9.96	8.55	1.42	3.51	6.68	641	General
50B	OSB	152/305	9.67	5.99	1.97	2.46	9.49	634	General
50C[1]	OSB	152/305	10.41	7.26	1.75	2.98	7.40	635	General
AVERAGE			10.01	7.27	1.71	2.98	7.86	637	
52A	OSB	75/305	20.56	15.06	1.66	6.18	2.92	914	General
52B	OSB	75/305	20.43	10.62	2.35	4.36	3.91	901	General
52C[1]	OSB	75/305	22.69	12.84	2.16	5.27	4.06	1264	General
AVERAGE			21.23	12.84	2.06	5.27	3.63	1026	
54A	CSP	152/305	10.98	14.02	0.96	5.75	5.74	983	General
54B	CSP	152/305	11.03	15.65	0.86	6.42	4.98	942	General
54C[1]	CSP	152/305	11.63	10.99	1.29	4.51	6.23	893	General
AVERAGE			11.21	13.55	1.04	5.56	5.65	939	
56A[1]	CSP	75/305	23.35	17.35	1.64	7.12	4.19	1825	General
56B	CSP	75/305	22.52	19.60	1.40	8.04	3.56	1647	General
56C	CSP	75/305	22.29	19.40	1.40	7.96	3.69	1682	General
AVERAGE			22.72	18.78	1.48	7.70	3.81	1718	

[1] Gravity loads not applied to test wall

[2] Energy calculation based on area below backbone curve

Table 5-3 : Design values for reversed cyclic tests (negative cycles)

Test	Panel Type	Fastener Schedule	Yield Load (S_{y-}) kN/m	Displacement at S_{y-} ($\Delta_{net,y-}$)	Elastic Stiffness (K_{e-}) kN/mm	Rotation at S_{y-} (θnet,-) rad	Ductility μ -	Energy Dissipation[2] E Joules -	Governing Case-
48A	DFP	75/305	-25.01	-21.23	1.44	-8.71	3.45	1908	General
48B	DFP	75/305	-24.26	-16.73	1.77	-6.86	4.22	1841	General
48C[1]	DFP	75/305	-24.47	-17.58	1.70	-7.21	3.64	1644	General
AVERAGE			-24.58	-18.51	1.64	-7.59	3.77	1798	
50A	OSB	152/305	-9.95	-7.67	1.58	-3.15	7.01	606	General
50B	OSB	152/305	-9.38	-7.92	1.44	-3.25	6.59	552	General
50C[1]	OSB	152/305	-9.86	-8.23	1.46	-3.38	7.09	651	General
AVERAGE			-9.73	-7.94	1.49	-3.26	6.90	603	
52A	OSB	75/305	-20.77	-14.96	1.69	-6.14	2.22	651	General
52B	OSB	75/305	-19.68	-8.85	2.71	-3.63	3.52	642	General
52C[1]	OSB	75/305	-21.76	-14.90	1.78	-6.11	3.33	1118	General
AVERAGE			-20.74	-12.90	2.06	-5.29	3.02	804	
54A	CSP	152/305	-9.72	-11.11	1.07	-4.56	5.60	671	General
54B	CSP	152/305	-10.51	-12.95	0.99	-5.31	5.06	756	General
54C[1]	CSP	152/305	-10.17	-12.28	1.01	-5.04	3.49	456	General
AVERAGE			-10.13	-12.11	1.02	-4.97	4.72	628	
56A[1]	CSP	75/305	-20.49	-25.52	0.98	-10.47	3.11	1662	General
56B	CSP	75/305	-20.24	-15.67	1.57	-6.43	4.38	1502	General
56C	CSP	75/305	-18.82	-16.06	1.43	-6.59	4.39	1433	General
AVERAGE			-19.85	-19.08	1.33	-7.83	3.96	1533	

[1]Gravity loads not applied to test wall

[2] Energy calculation based on area below backbone curve

5.3 Calibration of Resistance Factor

There is no approach for the design of light gauge steel frame / wood panel shear walls subjected to in-plane lateral loading in the current CSA S136 Standard (2002) for the design of cold-formed steel structures. There are also no appropriate values listed for the nominal shear capacity S_y, or a resistance factor, ϕ, calibrated in accordance with the 2005 National Building Code of Canada (NRCC, 2005). It was therefore necessary to use the design values listed in Tables 5-1 through 5-3 for S_y to calibrate a resistance factor with respect to the one in fifty years, $q_{1/50}$, NBCC factored wind load. Branston (2004) documents the derivation of the calibration procedure and the explanation for the use of particular values assigned to statistical parameters in the model (Eq. 5-8).

This section provides a summary of the approach and recommended values from the test data.

$$\phi = C_\phi (M_m F_m P_m) e^{-\beta_o \sqrt{V_M^2 + V_F^2 + C_P V_P^2 + V_S^2}} \qquad (5\text{-}8)$$

where,

ϕ = Resistance factor

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

β_o = Reliability/safety index

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 $n=3$

V_S = Coefficient of variation of the load effect

m = Degrees of freedom = n – 1

n = number of tests

e = Natural logarithmic base = 2.718...

Branston (2004) used the values for M_m, F_m, V_M and V_F specified in Table F1 of the North American Specification for the Design of Cold-Formed Steel Structural Members (AISI, 2002) in his development of the design methodology for steel frame wood panel shear walls. The mean value of 1.05 for the material factor, M_m, for the sheathing material accounts for a possible 5% overstrength in the sheathing material. A mean value of 1.00 for the fabrication factor, F_m, was used assuming that the average thickness of the sheathing material was equal to the nominal thickness. The variables V_M and V_F, representing the variation in material and fabrication, were 0.11 and 0.10 respectively, to account for a coefficient of variation of 15% found for the strength distribution of the sheathing.

The professional factor, P_m, and the coefficient of variation of the professional factor, V_P, were derived from the average wall resistance at yield $S_{y,avg}$. In this study two approaches to the calculation of $S_{y,avg}$ were used, and the resulting resistance factors, ϕ, were compared. In the first method of calculation, Eq. 5-9, the positive and negative nominal shear capacities, $S_{y+,avg}$ and $S_{y-,avg}$, from each cyclic test were averaged before being added to the nominal shear capacity of the monotonic test. The second method calculated the average nominal shear capacity with the positive nominal shear capacity, $S_{y+,avg}$, from the cyclic test and nominal shear capacity from the monotonic shear test (Eq. 5-10).

$$S_{y,avg} = \frac{S_{y,mono,avg} + \frac{S_{y+,avg} + S_{y-,avg}}{2}}{2} \tag{5-9}$$

$$S_{y,avg} = \frac{S_{y,mono,avg} + S_{y+,avg}}{2} \tag{5-10}$$

The professional factor, P_m, was calculated for all test series with the same fastener schedule using the following equation:

$$P_m = \frac{\sum_{i=1}^{n} \left({S_y}/{S_{y,avg}} \right)_i}{n} \tag{5-11}$$

To determine the coefficient of variation of the professional factor (Equation 5-13), V_P, the standard deviation of the data set was calculated with Equation 5-12 where S_y is the nominal shear value for each individual test included in the data set.

$$\sigma^2 = \frac{1}{n-1} \sum_{i=1}^{n} \left[\left({S_y}/{S_{y,avg}} \right)_i - P_m \right]^2 \tag{5-12}$$

$$V_P = \frac{\sigma}{P_m} \tag{5-13}$$

The calibration coefficient, C_ϕ, was obtained using Eq. 5-14:

$$C_\phi = \frac{\alpha}{\bar{S}/S} \tag{5-14}$$

where,

α = Load factor for wind loads and is equal to 1.4 according to the 2005 NBCC (NRCC, 2005)

$\bar{S}/S$ = Mean-to-nominal ratio of the wind load

Applying the approach described by Branston (2004), which was also used by Blais (2006) and Rokas (2006), the mean-to-nominal ratio of the wind load, $\bar{S}/S$, and its coefficient of variation, V_S, were equal to 0.76 and 0.37 respectively. In the Commentary of the 2001 North American Cold-Formed Steel Specification (AISI, 2002) the recommended range for the reliability/safety factor, β_o,, is 2.5 to 4.0 when a failure at the connection is unacceptable. A value of 2.5 for the reliability/safety factor, β_o, was proposed with the assumption that the walls have a built-in overstrength greater than 10% on average, where overstrength is the percentage exceeding unity of the ratio of ultimate shear strength over the yield shear strength determined using the previously described EEEP methods (See Section 5.6). Using this approach with the statistical values described above a resistance factor, ϕ, was calculated for the two different fastener schedules and the entire series. The resulting values of approximately 0.7 in all cases are listed in Table 5-4. These values are in agreement with the value of ϕ=0.7 recommended by Branston (2004), Blais (2006) and Rokas (2006).

Table 5-4 : Resistance factor calibration for 2005 NBCC wind loads

Mono/Cyclic +/-

Fastener Schedule (mm)	α	$\overline{S}/S$	C_Φ	M_m	F_m	P_m	β_o	V_m	V_f	V_s	n	C_p	V_p	Φ
152/305	1.4	0.76	1.842	1.05	1	1	2.5	0.11	0.1	0.37	19	1.18	0.049	0.707
75/305	1.4	0.76	1.842	1.05	1	1	2.5	0.11	0.1	0.37	13	1.29	0.034	0.711
All tests	1.4	0.76	1.842	1.05	1	1	2.5	0.11	0.1	0.37	32	1.10	0.040	0.710

Mono/Cyclic positive

152/305	1.4	0.76	1.842	1.05	1	1	2.5	0.11	0.1	0.37	19	1.18	0.047	0.708
75/305	1.4	0.76	1.842	1.05	1	1	2.5	0.11	0.1	0.37	13	1.29	0.048	0.707
All tests	1.4	0.76	1.842	1.05	1	1	2.5	0.11	0.1	0.37	32	1.10	0.047	0.707

The resistance factor, ϕ, calculated for the 2005 NBCC wind loads is recommended to also be used in seismic design. This approach is valid because ϕ is also used in the definition of R_o of the equivalent static base shear (V) (Eq. 5-15) as well as the factored wall resistance.

$$\phi S_y \geq \frac{S(T)M_v I_E W}{R_d R_o} \tag{5-15}$$

where,

$S(T)$ = Design spectral response acceleration (function of structure's period and location)

M_v = Factor for higher mode effect

I_E = Importance factor of the structure = 1.0 for normal buildings

W = Seismic weight

R_o = Overstrength-related force modification factor

R_d = Ductility-related force modification factor

ϕ = Resistance factor

Seismic resistant design is currently based on a return period of 2500 years for the design level earthquake (a probability of exceedance of 2% in 50 years) (Mitchell *et al., 2003).* For this reason the overstrength-related force modification factor, R_ϕ, is a function of the inverse of ϕ. Since earthquakes represent a rare loading event a nominal resistance is considered adequate for design rather than a factored design. Therefore a resistance factor consistent with the factor calibrated for wind loads of ϕ = 0.7 is recommended, which is consistent with the value of R_ϕ used in the calculation of R_o (see Section 5.6).

5.4 RECOMMENDED SHEAR AND STIFFNESS VALUES FOR LIGHT GAUGE STEEL FRAME / WOOD PANEL SHEAR WALLS

An average nominal shear resistance, $S_{y,avg}$, and an average unit elastic stiffness, $k_{e,avg}$, were computed for each wall configuration from the results complied in Section 5.2. For each design parameter the monotonic results were weighted at 50% and the positive and negative components of the cyclic results at 25% each as described in Eqs. 5-16 and 5-17.

$$S_{y,avg} = \frac{S_{y,mono} + (S_{y,+cyclic} + S_{y,-cyclic})/2}{2} \tag{5-16}$$

$$k_{e,avg} = \frac{k_{e,mono} + (k_{e,+cyclic} + k_{e,-cyclic})/2}{2} \tag{5-17}$$

where,

$S_{y,avg}$ = average nominal shear resistance [kN/m]

$S_{y,mono}$ = average shear resistance for monotonic tests [kN/m]

$S_{y,+cyclic}$ = average shear resistance for cyclic tests in the positive direction [kN/m]

$S_{y,-cyclic}$ = average shear resistance for cyclic tests in the negative direction [kN/m]

$k_{e,avg}$ = average unit elastic stiffness [kN/mm/m]

$k_{e,mono}$ = average unit elastic stiffness for monotonic tests [(kN/m)/mm]

$k_{e,+cyclic}$ = average unit elastic stiffness for cyclic tests in the positive direction [(kN/m)/mm]

$k_{e,-cyclic}$ = average unit elastic stiffness for cyclic test in the negative direction [(kN/m)/mm]

The results of these calculations are presented in Table 5-5 for each configuration tested. In comparison to the results determined by Branston (2004) for the same wall configurations tested under lateral loads the average yield shear resistances for the walls described in this thesis are slightly higher with the exception of the 11 mm OSB sheathed panel with 150 mm screw spacing around the edge (10.2 kN/m vs. 11.0 kN/m) and the walls with 12.5 mm CSP panels attached at a screw spacing of 75 mm (21.4 kN/m vs 21.6 kN/m). The source and condition of the sheathing panel has been found to be influential in the performance of the shear wall (Chen, 2004). Table 5-6 shows the difference in measured material properties of the sheathing between Branston (2004) and this test series. Since there is no trend linking nominal shear yield resistance, $S_{y,avg}$, of walls with the same paneling and different screw schedules relative to the results from Branston (2004) the difference in panel properties is not the only parameter responsible for the variation in response. To obtain test results that isolated the influence of gravity loads from the variability of materials and in testing a control group within the test matrix was developed, one specimen out of each test series was tested under lateral loads only. An in depth discussion of these relative results is included in Section 5.8.

The higher average recommended shear yield resistance values from this test series in comparison to Branston (2004) could also be a result of an increase in stud thickness for those walls with 75 mm screw spacing around the perimeter and the use of different screws in the walls with thicker studs and plywood paneling. However, there are no data trends of this nature to support the possibility. The higher values in shear yield resistance may also be a result of the lower stiffnesses of the walls coupled with higher energy dissipation within the design range of the wall. Table 5-7 shows a comparison of the EEEP energy dissipation and the displacement at failure, $\Delta_{net,0.8u}$, between the test results from Branston (2004) and the control group from this test series which did not include gravity loads. The energy derived from the EEEP analysis (Section 5.2) for the monotonic tests was higher, approximately 8%, in previous testing with the exception of the Series 43 (CSP 75/305 mm). Conversely, the EEEP energy calculated for the reversed-cyclic tests was 18% higher for this test series relative to the previous study with the exception Series 44 (CSP 75/305 mm). The displacement at failure for the monotonic tests did not show any consistent trends, but the values from the reversed-cyclic tests were all lower than those from previous testing. Thus, due to the way the shear yield resistance of the wall is determined through equating the energies under the curve, the trend in lower effective stiffnesses, higher energy dissipation and smaller displacements at failure for the reversed cyclic tests relative to Branston (2004) means that a larger area would have to be contained within a narrower width. Therefore the plateaus defining shear yield resistance, S_y, for these tests are comparatively higher.

In all cases the average stiffness of the walls described herein was lower than that presented by Branston (2004). The decrease in stiffness of the walls for this test series could be attributed to the variability of the sheathing. The comparison of material properties between Branston and this study indicates that the shear rigidity of the panels used in this study were consistently lower than those tested by Branston (Table 5-6). Another major factor in the stiffness of the system is the condition of the panel around the edges, especially at the corners. However, the use of damaged panels was avoided during testing. Any damage that occurred to the panel prior to testing was recorded on test damage sheets which are located in Appendix C, but these events were minimal.

The rate of loading has also been found to affect the effective stiffness of the shear wall and in general the K_e for reversed-cyclic tests are higher than monotonic (Chen ,2004). The loading of the reversed-cyclic tests in this study was slightly different than those from previous studied because it had to be limited to a rate of 10 mm/s rather than a frequency of 0.5 Hz due to constraints of the test set-up. Previous steel frame / wood panel shear wall studies reached rates of 20 mm/s on average (Chen, 2004). In order to determine the validity of this finding for these particular wall configurations Table 5-8 was prepared. The table contains the effective stiffnesses from the control group of tests from this study, which did not include gravity loads, and average effective stiffnesses for the corresponding configurations from Branston (2004). In general both studies support the finding of higher stiffnesses for the reversed-cyclic tests relative to the monotonic results. This finding also indicates the lower rate of loading was not significant. The trend can be explained through the more even distribution of forces through out the wall and strain rate effects during the reversed-cyclic protocol. Through the comparison of the effective stiffness values between Branston (2004) and the control group from this study in Table 5-8 it can be seen that the values from this study are lower with the exception of tests 50C 51B, 53C and 54C. Therefore the inclusion of gravity loads is not the sole reason for the lower stiffness of this test series it seems to moreover be the influence of the variation in materials. The higher stiffness of shear walls 53C and 54C, both of the same configuration with CSP sheathing and 152/305 mm screw schedule, relative to the results from Branston, 2004 is the only configuration on which the inclusion of gravity loads could be considered to have caused a reduction in stiffness since the overall recommended stiffness for this configuration from this study is lower than that quoted by Branston (2004). However, since this is only one configuration among five and this particular series has a rather high variability in stiffness between specimens the gravity loads should not be considered the main contributing factor.

It was suspected that the configurations with thicker chord studs would exhibit higher stiffnesses than the previous test series because from a mechanical perspective the bulkier section provides a greater restriction against rotation of the screw fastener. The design values developed from this study do not indicate this trend, but the more frequent shearing of screws during testing indicates that this condition was present.

Table 5-5 : Nominal shear strength, S_y (kN/m), and unit elastic stiffness, k_e ((kN/m)/mm), 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		150	
	S_y (kN/m)	k_e ((kN/m)/mm)	S_y (kN/m)	k_e ((kN/m)/mm)
12.5 mm Canadian Softwood Plywood (CSP) CSA O151	21.4	1.04	10.9	0.81
12.5 mm Douglas Fir Plywood (DFP) CSA O121	25.3	1.22	N/A	N/A
11.0 mm Oriented Strand Board (OSB) CSA 0325	20.7	1.86	10.2	1.30

(1) $\Phi = 0.7$ used to calculate factored resistance for design

(2) Maximum aspect ratio of 2:1 for full-height shear wall segments shall be included in resistance calculations. No increase of nominal strength permitted for sheathing installed on both sides of the wall.

(3) Tabulated values are valid for short-term load duration ($K_d = 1.0$) and dry service conditions. For shear walls under standard term loads or permanent loading tabulated values must be multiplied by 0.870 or 0.565 respectively.

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

(5) Edge fasteners shall be install at not less than 12.7 mm (1/2") along all panel edges to provide full blocking. Fasteners along intermediate supports shall be spaced at 305 mm (12") o.c. Sheathing panels must be installed vertically such that the strength axis is parallel to framing members.

(6) Minimum No.8 x 1/2" (12.7 mm) framing and No.8 x 3/2" (38.1 mm) sheathing screws shall be used

(7) ASTM A653 Grade 230 MPa (33 ksi) (min.) of minimum uncoated base metal thickness 1.09 mm (0.043") (min.) steel shall be used dependent on wall design.

(8) Chord studs for wall with 75 mm (3") panel edge screw spacing shall be ASTM A653 Grade 340 MPa (50 ksi) (min.) of uncoated base metal thickness 1.37 mm (0.054") (min.) steel.

(9) For S136 calculations the nominal value of 50 ksi is generally converted into 345 MPa, but in this case to meet ASTM standards 340 MPa should be used

(10) Studs: 92.1 mm (3-5/8") web, 41.3 mm (1-5/8") flange, 12.7 mm (1.2") return lip. Tracks: 92.1 mm (3-5/8") web, 31.8 mm (1-1/4") flange.

(11) Plywood: CSA 0151 or CSA 0121 (sheathing). OSB: CSA 0325 minimum end use 1R24/2F16/W24.

(12) The above values are for lateral loading only. The compression chords must be designed to account for loads from lateral and gravity loading combined.

Table 5-6 : Relative difference in material properties of panels

Specimen	Thickness (mm)	Ultimate Shear Strength (MPa)	Shear Modulus (MPa)	Rigidity (N/mm)
12.5 mm DFP[1]	12.54	4.97	923	11584
12.5 mm DFP[2]	12.55	5.00	825	10371
Difference[3]	**-0.01**	**0.03**	**98**	**1213**
12.5 mm CSP[1]	12.41	4.24	814	10080
12.5 mm CSP[2]	11.56	4.44	497	5738
Difference[3]	**0.85**	**-0.2**	**317**	**4342**
11 mm OSB[1]	11.25	8.12	1402	15755
11 mm OSB[2]	11.15	9.09	925	10303
Difference[3]	**0.1**	**-0.97**	**477**	**5452**

[1] Measured panel properties from Branston 2004
[2] Measured panel properties from test series
[3] Relative difference between panel properties

Table 5-7 : A comparison of the EEEP energy dissipation and the displacement at failure between control group and Branston (2004)

Monotonic Test Series	EEEP Energy (J)	$\Delta_{0.8Su}$ (mm)	Reversed-Cyclic Test Series	EEEP Energy (J)	$\Delta_{0.8Su}$ (average) (mm)
13[1]A,B,C DFP 75/305 mm	1600	62.8	14[1]A,B,C DFP 75/305 mm	3372	62.5
47[2]C DFP 75/305 mm	1581	74.3	48[2]C DFP 75/305 mm	3700	57.4
Difference[3]	**19**	**-11.5**	**Difference[3]**	**-328**	**5.1**
21[1]A,B,C OSB 152/305 mm	727	54.7	22[1]A,B,C OSB 152/305 mm	1184	54.7
49[2]B OSB 152/305 mm	617	52.7	50[2]C OSB 152/305 mm	1286	49.1
Difference[3]	**110**	**2.0**	**Difference[3]**	**-102**	**5.7**
25[1]A,B,C OSB 75/305 mm	1019	46.8	26[1]A,B,C OSB 75/305 mm	1930	46.8
51[2]B OSB 75/305 mm	902	41.1	52[2]C OSB 75/305 mm	2382	44.0
Difference[3]	**117**	**5.7**	**Difference[3]**	**-452**	**2.8**
7[1]A,B,C CSP 152/305 mm	825	67.1	8[1]A,B,C CSP 152/305 mm	1377	67.1
53[2]C CSP 152/305 mm	951	76.1	54[2]C CSP 152/305 mm	1349	52.8
Difference[3]	**-126**	**-9.0**	**Difference[3]**	**28**	**14.3**
9[1]A,B,C CSP 75/305 mm	1426	75.9	10[1]A,B,C CSP 75/305 mm	2706	69.7
55[2]B CSP 75/305 mm	1380	70.0	56[2]A CSP 75/305 mm	3487	63.8
Difference[3]	**46**	**5.9**	**Difference[3]**	**-781**	**5.9**

[1] Test results from Branston, 2004
[2] Test result that did not included gravity loads during testing
[3] Relative difference between values from each study

Table 5-8 : Comparison of stiffnesses from control group with Branston (2004)

Monotonic Test Series	Elastic Stiffness K_e kN/mm	Reversed - Cyclic Test Series	Elastic Stiffness K_e kN/mm	Percent (%) Difference ($K_{e, cyclic}$ / $K_{e, mono.}$)
13[1]A,B,C DFP 75/305 mm	1.59	14[1]A,B,C DFP 75/305 mm	1.72	8
47[2]C DFP 75/305 mm	1.40	48[2]C DFP 75/305 mm	1.45	4
Difference[3]	**0.19**	**Difference[3]**	**0.27**	
21[1]A,B,C OSB 152/305 mm	1.78	22[1]A,B,C OSB 152/305 mm	1.59	-11
49[2]B OSB 152/305 mm	1.40	50[2]C OSB 152/305 mm	1.61	15
Difference[3]	**0.38**	**Difference[3]**	**-0.02**	
25[1]A,B,C OSB 75/305 mm	1.96	26[1]A,B,C OSB 75/305 mm	2.63	34
51[2]B OSB 75/305 mm	2.41	52[2]C OSB 75/305 mm	1.97	-18
Difference[3]	**-0.45**	**Difference[3]**	**0.66**	
7[1]A,B,C CSP 152/305 mm	1.05	8[1]A,B,C CSP 152/305 mm	1.11	6
53[2]C CSP 152/305 mm	1.13	54[2]C CSP 152/305 mm	1.15	2
Difference[3]	**-0.08**	**Difference[3]**	**-0.04**	
9[1]A,B,C CSP 75/305 mm	1.33	10[1]A,B,C CSP 75/305 mm	1.5	13
55[2]B CSP 75/305 mm	1.23	56[2]A CSP 75/305 mm	1.31	7
Difference[3]	**0.10**	**Difference[3]**	**0.19**	

1 Averaged test results from Branston 2004
2 Test result did not included gravity loads during testing
3 Relative difference between stiffnesses

5.5 Factor of Safety

The factored shear resistance presented in Section 5.4 was used in the calculation of the factor of safety associated with light gauge steel frame / wood panel shear walls. The factor of safety for limit states design (LSD) and allowable stress design (ASD) were both determined. The LSD approach is a simple comparison of the measured ultimate shear resistance with the nominal shear capacity (Eq. 5-20) (Fig. 5-6). The ASD approach incorporates the wind load factor with the ratio of ultimate shear resistance to nominal capacity. A factor of 1.4 from the 2005 NBCC (NRCC, 2005) was utilized in Eq. 5-21.

$$F.S.(LSD) = \frac{S_u}{S_r} \tag{5-20}$$

$$F.S.(LSD) = 1.4\frac{S_u}{S_r} \tag{5-21}$$

where,

$F.S.$ = Factor of Safety

S_u = Ultimate wall resistance observed during test (+ direction for reversed cyclic tests)

S_r = Factored wall resistance (ϕ = 0.7)

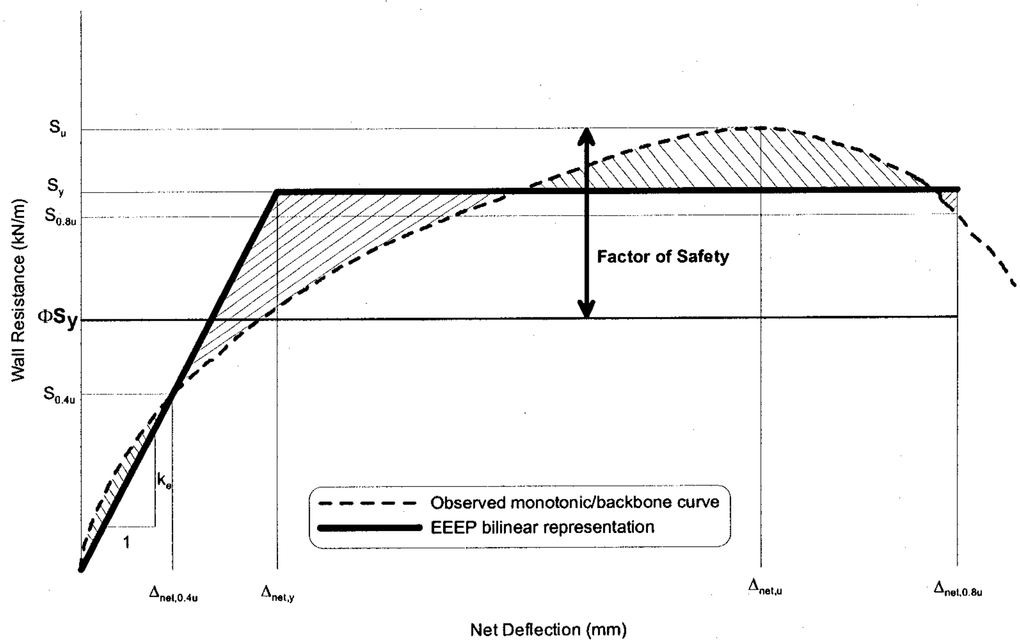


Figure 5-6 : Factor of safety inherent in limit states design (Branston, 2004)

The value of the ultimate shear resistance, S_u, was chosen from the positive direction for the reversed cyclic tests because it was felt that it best represented the value at which the wall would ultimately reach failure. The factor of safety for steel frame / wood panel shear walls for allowable stress design should be between 2.0 -2.5, according to Branston (2004). This recommendation is based on the suggestion by the 2000 IBC (ICC, 2000) of $F.S.$ = 2.0 for light gauge steel frame shear walls and by the IBC 2000 Handbook (Ghosh and Chittenden, 2001) of $F.S.$ = 2.5 for wood shear walls.

The ASD factor of safety for this test series ranged from 2.05 to 2.66 (Tables 5-9 & 5-10). For monotonic values tests the ASD factor of safety ranged between 2.13 to 2.66 with an average value of 2.37 having a standard deviation of 0.16 and a coefficient of variation of 6.8% as shown in Table 5-9. The reversed cyclic tests had similar results with a range for the factor of safety between 2.05 and 2.66, an average of 2.27, a standard deviation of 0.14 and a coefficient of variation of 6.3% as listed in Table 5-10. These values are generally well within the recommended range with no test falling below *F.S.* = 2.0. The average values fall between those calculated by Branston (2004) and Blais (2006).

Table 5-9 : 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) Su/Sr*1.4
47A	DFP	75/305	31.11	25.33	17.73	1.75	2.46
47B	DFP	75/305	28.98	25.33	17.73	1.63	2.29
47C[1]	DFP	75/305	32.40	25.33	17.73	1.83	2.56
Average 47			30.83	25.33	17.73	1.74	2.43
49A	OSB	152/305	10.92	10.24	7.17	1.52	2.13
49B[1]	OSB	152/305	11.75	10.24	7.17	1.64	2.29
49C	OSB	152/305	13.32	10.24	7.17	1.86	2.60
49D	OSB	152/305	12.13	10.24	7.17	1.69	2.37
Average 49			12.03	10.24	7.17	1.68	2.35
51A	OSB	75/305	22.17	20.70	14.49	1.53	2.14
51B[1]	OSB	75/305	23.11	20.70	14.49	1.60	2.23
51C	OSB	75/305	22.35	20.70	14.49	1.54	2.16
Average 51			22.54	20.70	14.49	1.56	2.18
53A	CSP	152/305	13.39	10.90	7.63	1.75	2.46
53B	CSP	152/305	12.41	10.90	7.63	1.63	2.28
53C[1]	CSP	152/305	13.15	10.90	7.63	1.72	2.41
Average 53			12.98	10.90	7.63	1.70	2.38
55A	CSP	75/305	25.68	21.37	14.96	1.72	2.40
55B[1]	CSP	75/305	28.36	21.37	14.96	1.90	2.66
55C	CSP	75/305	24.70	21.37	14.96	1.65	2.31
55D	CSP	75/305	27.08	21.37	14.96	1.81	2.54
Average 55			26.46	21.37	14.96	1.77	2.48
					AVERAGE	**1.69**	**2.37**
					STD DEV	**0.11**	**0.16**
					CoV	**0.068**	**0.068**

[1] Gravity loads not applied to test wall

Table 5-10 : Factor of safety inherent in design 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) Su/Sr*1.4
48A	DFP	75/305	29.26	25.33	17.73	1.65	2.31
48B	DFP	75/305	29.14	25.33	17.73	1.64	2.30
48C[1]	DFP	75/305	28.35	25.33	17.73	1.60	2.24
AVERAGE			28.92	25.33	17.73	1.63	2.28
50A	OSB	152/305	10.77	10.24	7.17	1.50	2.10
50B	OSB	152/305	10.49	10.24	7.17	1.46	2.05
50C[1]	OSB	152/305	11.11	10.24	7.17	1.55	2.17
AVERAGE			10.79	10.24	7.17	1.50	2.11
52A	OSB	75/305	22.18	20.70	14.49	1.53	2.14
52B	OSB	75/305	22.12	20.70	14.49	1.53	2.14
52C[1]	OSB	75/305	25.64	20.70	14.49	1.77	2.48
AVERAGE			23.31	20.70	14.49	1.61	2.25
54A	CSP	152/305	11.95	10.90	7.63	1.57	2.19
54B	CSP	152/305	12.16	10.90	7.63	1.59	2.23
54C[1]	CSP	152/305	12.96	10.90	7.63	1.70	2.38
AVERAGE			12.36	10.90	7.63	1.62	2.27
56A[1]	CSP	75/305	26.90	21.37	14.96	1.80	2.52
56B	CSP	75/305	25.56	21.37	14.96	1.71	2.39
56C	CSP	75/305	25.85	21.37	14.96	1.73	2.42
AVERAGE			26.10	21.37	14.96	1.75	2.44
					AVERAGE	**1.62**	**2.27**
					STD DEV	**0.10**	**0.14**
					CoV	**0.0629**	**0.0629**

[1]Gravity loads not applied to test wall

The LSD factor of safety results ranged between 1.52-1.90 with an overall average of 1.69 for monotonic tests with a standard deviation of 0.11 and a coefficient of variation of 6.8%. For the reversed cyclic tests the results ranged from 1.46 to 1.80, averaging to $F.S. = 1.62$ with a standard deviation equal to 0.10 and a coefficient of variation of 6.3%. These results are directly related to those previously mentioned for allowable stress design. They also follow the same trends as Branston (2004) and Blais (2006) and support the suggested factor of safety value.

5.6 Capacity Based Design and Overstrength

The light gauge steel frame / wood panel shear walls in this study are typically used to withstand lateral loads from wind and earthquake. When considering seismic loads on a structure a capacity based design approach should be implemented. As recommended by Branston (2004), the shear wall is expected to perform in the inelastic range of behaviour because of the ductile failure mode experienced by the wood sheathing to steel frame screw connections. The sheathing to frame connections alone are intended to fail during a design level earthquake, hence, the steel frame needs to remains intact to carry gravity loads post-disaster. In order for the frame to maintain its gravity load carrying capacity it must be designed to transfer the forces associated with failure of the sheathing connections in addition to the expected companion gravity loads. Other elements in the seismic force resisting system (SFRS), including, the tracks, framing connections, hold downs, anchor rods, shear anchors and foundation would also need to be designed for probable failure load of the sheathing connections.

In order to estimate the ultimate shear capacity of the wall based on the sheathing connection failure mode the nominal shear resistance, S_y, of the wall can be multiplied by the overstrength factor as illustrated in Figure 5-7. The S_y values used for this purpose are those listed in Table 5-5. These values can be used because they were derived from test walls for which sheathing connection failure controlled the behaviour and ultimate force level reached.

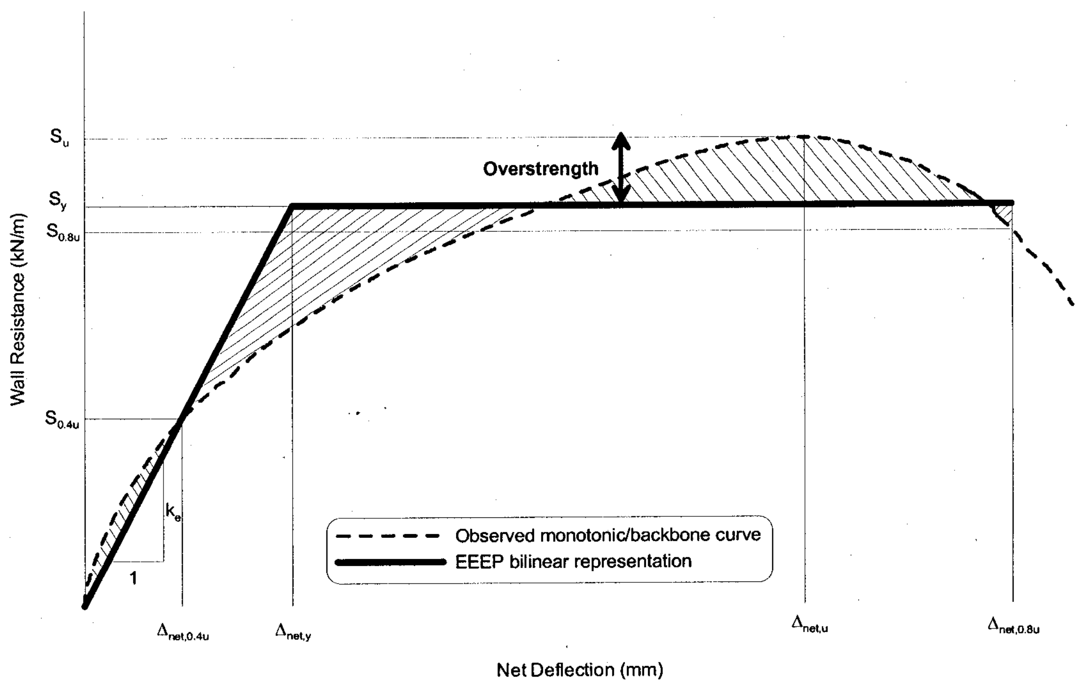


Figure 5-7 : Overstrength inherent in design (*Branston, 2004*)

The overstrength factor is calculated by dividing ultimate shear resistance, S_u, by the nominal shear resistance, S_y (Eq. 5-22).

$$overstrength = \frac{S_u}{S_y} \tag{5-22}$$

where,

S_u = Ultimate wall resistance observed during test (+ direction for reversed cyclic tests)

S_y = Nominal shear strength

Table 5-11: 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
47A	DFP	75/305	31.11	25.33	1.23
47B	DFP	75/305	28.98	25.33	1.14
47C[1]	DFP	75/305	32.40	25.33	1.28
Average 47			30.83	25.33	1.22
49A	OSB	152/305	10.92	10.24	1.07
49B[1]	OSB	152/305	11.75	10.24	1.15
49C	OSB	152/305	13.32	10.24	1.30
49D	OSB	152/305	12.13	10.24	1.18
Average 49			12.03	10.24	1.17
51A	OSB	75/305	22.17	20.70	1.07
51B[1]	OSB	75/305	23.11	20.70	1.12
51C	OSB	75/305	22.35	20.70	1.08
Average 51			22.54	20.70	1.09
53A	CSP	152/305	13.39	10.90	1.23
53B	CSP	152/305	12.41	10.90	1.14
53C[1]	CSP	152/305	13.15	10.90	1.21
Average 53			12.98	10.90	1.19
55A	CSP	75/305	25.68	21.37	1.20
55B[1]	CSP	75/305	28.36	21.37	1.33
55C	CSP	75/305	24.70	21.37	1.16
55D	CSP	75/305	27.08	21.37	1.27
Average 55			26.46	21.37	1.24
				AVERAGE	**1.18**
				STD DEV	**0.08**
				CoV	**0.068**

[1] Gravity loads not applied to test wall

The overstrength values for the monotonic tests are listed in Table 5-11. The values range between 1.07-1.33, with an average of 1.18, a standard deviation of 0.08 and a coefficient of variation of 6.8%. This range is slightly larger than that reported by Blais (2006), but significantly smaller than that found by Branston (2004). The overall average for overstrength was between the 1.15 determined by Blais (2006) and the 1.24 obtained by Branston (2004).

The results for the reversed cyclic overstrength factors followed trends similar to the monotonic results. The factors, listed in Table 5-12, for overstrength vary between 1.02-1.26, have an average of 1.14, a standard deviation of 0.07 and a coefficient of variation of 6.3%. The result from Blais' study of 9 mm OSB sheathed walls (2006) was 1.07, slightly lower value than the value obtained from the results described herein. Branston (2004) calculated an overstrength of 1.20 for the tests performed in the summer of 2003. As mentioned earlier the trends for the reversed cyclic overstrength factors with Blais (2006) and Branston (2004) were similar to those discussed for monotonic testing.

Branston (2004) calculated overstrength values for walls with very similar (thickness of the chord studs was 1.12 mm instead of 1.37 mm in three cases) or the same configurations as those included in this test program. Table 5-13 combines these like results to calculate an average overstrength of 1.19 for monotonic tests. In Table 5-14 the cyclic results are combined and average to a value of 1.14. These values have an even lower standard deviation and coefficient of variation than for the overstrength computed from this data series alone. The consistency of the overstrength values over multiple series of testing supports the recommended value.

The wood species used for the plys in CSP panels can vary, given the provisions of CSA O151 (1978). It is common to use Spruce plys, which based on the findings of this study

Table 5-12: 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
48A	DFP	75/305	29.26	25.33	1.16
48B	DFP	75/305	29.14	25.33	1.15
48C[1]	DFP	75/305	28.35	25.33	1.12
AVERAGE			28.92	25.33	1.14
50A	OSB	152/305	10.77	10.24	1.05
50B	OSB	152/305	10.49	10.24	1.02
50C[1]	OSB	152/305	11.11	10.24	1.08
AVERAGE			10.79	10.24	1.05
52A	OSB	75/305	22.18	20.70	1.07
52B	OSB	75/305	22.12	20.70	1.07
52C[1]	OSB	75/305	25.64	20.70	1.24
AVERAGE			23.31	20.70	1.13
54A	CSP	152/305	11.95	10.90	1.10
54B	CSP	152/305	12.16	10.90	1.12
54C[1]	CSP	152/305	12.96	10.90	1.19
AVERAGE			12.36	10.90	1.13
56A[1]	CSP	75/305	26.90	21.37	1.26
56B	CSP	75/305	25.56	21.37	1.20
56C	CSP	75/305	25.85	21.37	1.21
AVERAGE			26.10	21.37	1.22
				AVERAGE	**1.14**
				STD DEV	**0.07**
				CoV	**0.063**

[1] Gravity loads not applied to test wall

and previous shear wall research at McGill University provide for a lower bound shear resistance. All of the CSP design values recommended herein and by Branston (2004) were determined through the testing of walls whose sheathing was made entirely of Spruce plys. In the case where the plys of a CSP panel are made of Douglas fir the overall shear strength and stiffness of the shear wall will likely be similar to a wall sheathed with DFP panels. Since it is not possible for an engineer to specify the exact wood species for the plies of a CSP panel in design, it is necessary that the overstrength based on Douglas fir be established. To illustrate the potential increase in overstrength of a CSP sheathed wall the ultimate shear resistance of DFP sheathed test specimens were compared with the nominal shear resistance of an identical configuration except sheathed with CSP (Spruce plys). The resulting average overstrength values was 1.47 with a standard deviation of 0.13 and a coefficient of variation of 0.087 (Table 5-15).

Table 5-13 : Relative overstrength 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
47A	DFP	75/305	31.11	25.33	1.23
47B	DFP	75/305	28.98	25.33	1.14
47C[1]	DFP	75/305	32.40	25.33	1.28
13A[2]	DFP	75/305	28.00	24.50	1.14
13B[2]	DFP	75/305	30.80	24.50	1.26
13C[2]	DFP	75/305	30.40	24.50	1.24
Average			30.28	24.91	1.22
49A	OSB	152/305	10.92	10.24	1.07
49B[1]	OSB	152/305	11.75	10.24	1.15
49C	OSB	152/305	13.32	10.24	1.30
49D	OSB	152/305	12.13	10.24	1.18
21A[2]	OSB	152/305	13.40	11.00	1.22
21B[2]	OSB	152/305	13.10	11.00	1.19
21C[2]	OSB	152/305	13.30	11.00	1.21
Average			12.56	10.57	1.19
51A	OSB	75/305	22.17	20.70	1.07
51B[1]	OSB	75/305	23.11	20.70	1.12
51C	OSB	75/305	22.35	20.70	1.08
25A[2]	OSB	75/305	23.70	20.60	1.15
25B[2]	OSB	75/305	22.20	20.60	1.08
25C[2]	OSB	75/305	24.70	20.60	1.20
Average			23.04	20.65	1.12
53A	CSP	152/305	13.39	10.90	1.23
53B	CSP	152/305	12.41	10.90	1.14
53C[1]	CSP	152/305	13.15	10.90	1.21
7A[2]	CSP	152/305	12.00	10.60	1.13
7B[2]	CSP	152/305	12.60	10.60	1.19
7C[2]	CSP	152/305	13.60	10.60	1.28
Average			12.86	10.75	1.20
55A	CSP	75/305	25.68	21.37	1.20
55B[1]	CSP	75/305	28.36	21.37	1.33
55C	CSP	75/305	24.70	21.37	1.16
55D	CSP	75/305	27.08	21.37	1.27
9A[2]	CSP	75/305	27.20	21.60	1.26
9B[2]	CSP	75/305	23.50	21.60	1.09
9C[2]	CSP	75/305	24.70	21.60	1.14
Average			25.89	21.47	1.21

[1] Gravity loads not applied to test wall	**AVERAGE**	**1.19**
[2] Data from Branston (2004)	**STD DEV**	**0.07**
	CoV	**0.060**

Table 5-14 : Relative overstrength 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
48A	DFP	75/305	29.26	25.33	1.16
48B	DFP	75/305	29.14	25.33	1.15
48C[1]	DFP	75/305	28.35	25.33	1.12
14A[2]	DFP	75/305	31.00	24.50	1.27
14B[2]	DFP	75/305	29.00	24.50	1.18
14C[2]	DFP	75/305	29.50	24.50	1.20
14D[2]	DFP	75/305	29.10	24.50	1.19
AVERAGE			29.34	24.86	1.18
50A	OSB	152/305	10.77	10.24	1.05
50B	OSB	152/305	10.49	10.24	1.02
50C[1]	OSB	152/305	11.11	10.24	1.08
22A[2]	OSB	152/305	11.70	11.00	1.06
22B[2]	OSB	152/305	11.90	11.00	1.08
22C[2]	OSB	152/305	11.50	11.00	1.05
AVERAGE			11.25	10.62	1.06
52A	OSB	75/305	22.18	20.70	1.07
52B	OSB	75/305	22.12	20.70	1.07
52C[1]	OSB	75/305	25.64	20.70	1.24
26A[2]	OSB	75/305	24.00	20.60	1.17
26B[2]	OSB	75/305	22.60	20.60	1.10
26C[2]	OSB	75/305	23.90	20.60	1.16
AVERAGE			23.41	20.65	1.13
54A	CSP	152/305	11.95	10.90	1.10
54B	CSP	152/305	12.16	10.90	1.12
54C[1]	CSP	152/305	12.96	10.90	1.19
8A[2]	CSP	152/305	12.00	10.60	1.13
8B[2]	CSP	152/305	11.90	10.60	1.12
8C[2]	CSP	152/305	11.80	10.60	1.11
AVERAGE			12.13	10.75	1.13
56A[1]	CSP	75/305	26.90	21.37	1.26
56B	CSP	75/305	25.56	21.37	1.20
56C	CSP	75/305	25.85	21.37	1.21
10A[2]	CSP	75/305	26.10	21.60	1.21
10B[2]	CSP	75/305	26.90	21.60	1.25
10C[2]	CSP	75/305	25.50	21.60	1.18
AVERAGE			26.14	21.48	1.22

AVERAGE	**1.14**
STD DEV	**0.07**
CoV	**0.059**

[1] Gravity loads not applied to test wall
[2] Data from Branston (2004)

Table 5-15 : Possible Overstrength Values for CSP containing DFP

Test	Panel Type	Fastener Schedule	Ultimate Resistance (S_u) kN/m	Yield Load (S_y) kN/m [4]	Overstrength S_u/S_y
47A[1]	DFP	75/305	31.1	21.4	1.45
47B[1]	DFP	75/305	29.0	21.4	1.35
47C	DFP	75/305	32.4	21.4	1.51
13A[2]	DFP	75/305	28.0	21.4	1.31
13B[2]	DFP	75/305	30.8	21.4	1.44
13C[2]	DFP	75/305	30.4	21.4	1.42
48A[1]	DFP	75/305	29.3	21.4	1.37
48B[1]	DFP	75/305	29.1	21.4	1.36
48C	DFP	75/305	28.4	21.4	1.32
14A[2]	DFP	75/305	31.0	21.4	1.45
14B[2]	DFP	75/305	29.0	21.4	1.36
14C[2]	DFP	75/305	29.5	21.4	1.38
14D[2]	DFP	75/305	29.1	21.4	1.36
AVERAGE			29.8	21.4	1.39
5A[3]	DFP	102/305	21.1	14.4	1.46
5B[3]	DFP	102/305	25.7	14.4	1.78
5C[3]	DFP	102/305	23.9	14.4	1.66
5D[3]	DFP	102/305	24.5	14.4	1.70
6A[3]	DFP	102/305	22.6	14.4	1.57
6B[3]	DFP	102/305	22.9	14.4	1.59
6C[3]	DFP	102/305	22.3	14.4	1.55
AVERAGE			23.3	14.4	1.62
11A[2]	DFP	152/305	15.8	10.6	1.49
11B[2]	DFP	152/305	16.9	10.6	1.59
11C[2]	DFP	152/305	15.3	10.6	1.44
12A[2]	DFP	152/305	13.5	10.6	1.27
12B[2]	DFP	152/305	16.0	10.6	1.51
12C[2]	DFP	152/305	14.4	10.6	1.36
AVERAGE			15.3	10.6	1.44

Potential range of overstrength 1.27 - 1.78 **AVERAGE** **1.47**

[1] Gravity loads applied to test wall **STD DEV** **0.13**

[2] Data from Branston (2004) **CoV** **0.087**

[3] Data from Boudreault (2005)

[4] Lower bound value from Boudreault (2005), Branston (2004) or Hikita (2006)

5.7 EVALUATION OF FORCE MODIFICATION FACTORS FOR NBCC 2005

The 2005 NBCC allows for the seismic elastic base shear imposed on a building to be reduced given the proven level of ductility and overstrength exhibited by the lateral framing system. Boudreault (2005) first made recommendations for the ductility related, R_d, and overstrength related, R_o, seismic force modification factors for steel frame / wood panel shear walls based on an evaluation of shear wall test results. Subsequent nonlinear time-history dynamic analyses of two representative buildings subjected to scaled ground motions by Blais (2006) confirmed these values on a preliminary basis (Boudreault et al., 2006, 2007). The approach adopted for determining these test-based R-values is described and a comparison is made between the 32 tests included in this research and the findings of previous studies.

5.7.1 Ductility-Related Force Modification Factor, R_d

The ductility-related force modification factor was evaluated using the same approach described by Boudreault (2005). The approach is based on Equations 5-24 to 5-26, originally derived by Newmark and Hall (1982), which are each specific to a range of periods for a structure. For the light gauge steel framed residential housing with shear walls the natural period of the structure is estimated to be between 0.1 to 0.5 seconds (Table 5-16). Hence, Eq. 5-25 was utilized with the test based ductility, μ, values (Tables 5-1 to 5-3) to obtain a value for R_d.

$$R_d = \mu \qquad \text{for } T > 0.5 \text{ sec} \tag{5-24}$$

$$R_d = \sqrt{2\mu - 1} \qquad \text{for } 0.1 < T < 0.5 \text{ sec} \tag{5-25}$$

$$R_d = 1 \qquad \text{for } T < 0.03 \text{ sec} \tag{5-26}$$

where,

μ = ductility ratio (Tables 5-1, 5-2 and 5-3)

Table 5-16 : Natural period for light-framed buildings (*Blais, 2006*)

Building Type	Natural Period, T_n (sec.)	Reference
Typical 1.22m x 2.44m shear wall (single storey)	0.10	NRCC 2005
Typical 1.22m x 2.44m shear wall (two-storey)	0.17	NRCC 2005
Typical 1.22m x 2.44m shear wall (three-storey)	0.24	NRCC 2005
Residential house (Univ. Of BC code estimate)	0.18	Folz & Filiatrault *(2001)*
Residential house	0.25	Gad *et al. (1999a)*

In past studies within this research program shear walls with a displacement at failure, $\Delta 0.8S_u$, greater than the drift limit had ductility factors calculated using the drift limit as their maximum load carrying deflection. However, in review of this practice it seems unreasonable to dock the ductile performance of the walls due to the drift limit, as it does not reflect the true inelastic behaviour of the system. Tables 5-17 and 5-18 list the ductility and ductility related force modification factor for each test with and without the drift limit imposed. This change in calculation has the most dramatic affect on the monotonic data set increasing the ductility force modification factor, R_d, from 2.77 to 2.91.

These "test-based" values for R_d are higher than the value of 2.5 suggested by Boudreault (2005) which were based on 78 shear wall tests, some with configurations similar to those tested in this test series. However, these values are lower than those found by Blais (2006) who calculated R_d values greater than 3.3 for walls sheathed with 9 mm OSB panels. Since only a single value of R_d is given for steel frame / wood panel shear walls a value of R_d = 2.5 is recommended.

Table 5-17: Ductility and R_d values for monotonic tests

Test	Panel Type	Fastener Schedule	No drift limit imposed		2.5% drift limit imposed	
			Ductility μ	Rd	Ductility μ	Rd
47A	DFP	75/305	3.67	2.52	2.75	2.12
47B	DFP	75/305	3.72	2.54	2.87	2.18
47C[1]	DFP	75/305	3.41	2.41	2.69	2.09
Average 47			3.60	2.49	2.77	2.13
49A	OSB	152/305	8.36	3.96	N/A	N/A
49B[1]	OSB	152/305	5.77	3.25	N/A	N/A
49C	OSB	152/305	7.07	3.63	N/A	N/A
49D	OSB	152/305	6.73	3.53	N/A	N/A
Average 49			6.98	3.59		
51A	OSB	75/305	4.96	2.99	N/A	N/A
51B[1]	OSB	75/305	3.94	2.62	N/A	N/A
51C	OSB	75/305	5.43	3.14	N/A	N/A
Average 51			4.78	2.92		
53A	CSP	152/305	5.66	3.21	N/A	N/A
53B	CSP	152/305	4.19	2.72	N/A	N/A
53C[1]	CSP	152/305	6.33	3.41	N/A	N/A
Average 53			5.39	3.11		
55A	CSP	75/305	3.72	2.54	2.76	2.13
55B[1]	CSP	75/305	3.16	2.31	2.69	2.09
55C	CSP	75/305	3.35	2.39	2.55	2.03
55D	CSP	75/305	3.22	2.33	2.57	2.04
Average 55			3.36	2.39	2.64	2.07

[1] Gravity loads not applied to test wall

AVERAGE	**2.91**
STD DEV	**0.52**
CoV	**0.18**

Table 5-18: Ductility and R_d values for reversed cyclic test (average of both cycles)

Test	Panel Type	Fastener Schedule	No drift limit imposed		2.5% drift limit imposed	
			Ductility μ	Rd	Ductility μ	Rd
48A	DFP	75/305	4.28	2.75	3.38	2.40
48B	DFP	75/305	4.46	2.81	3.80	2.56
48C[1]	DFP	75/305	3.31	2.37	N/A	N/A
AVERAGE			4.02	2.64	3.59	2.48
50A	OSB	152/305	6.85	3.56	N/A	N/A
50B	OSB	152/305	8.04	3.87	N/A	N/A
50C[1]	OSB	152/305	7.25	3.67	N/A	N/A
AVERAGE			7.38	3.70		
52A	OSB	75/305	2.57	2.03	N/A	N/A
52B	OSB	75/305	3.72	2.54	N/A	N/A
52C[1]	OSB	75/305	3.70	2.53	N/A	N/A
AVERAGE			3.33	2.36		
54A	CSP	152/305	5.67	3.22	N/A	N/A
54B	CSP	152/305	5.02	3.01	N/A	N/A
54C[1]	CSP	152/305	4.86	2.92	N/A	N/A
AVERAGE			5.18	3.05		
56A[1]	CSP	75/305	3.65	2.50	N/A	N/A
56B	CSP	75/305	3.97	2.63	N/A	N/A
56C	CSP	75/305	4.04	2.66	N/A	N/A
AVERAGE			3.89	2.60		

[1] Gravity loads not applied to test wall

AVERAGE	**2.87**
STD DEV	**0.51**
CoV	**0.18**

5.7.2 Overstrength-Related Force Modification Factor, R_o

The overstrength-related force modification factor takes into account the various aspects that lead to a reserve of strength in a structure. The 2005 NBCC equation for the overstrength-related force modification factor is a product of modification factors for member size, resistance factor, yield strength, strain hardening and collapse mechanism (Mitchell et al., 2003). The equation for this modification factor is:

$$R_o = R_{size} \times R_{\phi} \times R_{yield} \times R_{sh} \times R_{mech} \tag{5-27}$$

where,

R_{size} = Overstrength from the restricted choices of member sizes and dimension rounding.
R_{ϕ} = Factor accounting for the difference between nominal and factored resistances
R_{yield} = Ratio of real yield strength to specified yield strength
R_{sh} = Overstrength coming from the development of strain hardening
R_{mech} = Overstrength arising from the development of a collapse mechanism

Boudreault (2005) recommended an overstrength factor related to member size, R_{size}, equal to 1.05 since the fastener spacings used in construction are often smaller than what is required by the design. The factor accounting for the difference between nominal and factored resistances, R_{ϕ}, is the inverse of the resistance factor ϕ=0.7 calculated in Section 5.3. Thus, R_{ϕ} = 1/0.7 = 1.43, should be used in Equation 5-27. The ratio of real yield strength to the specified yield strength, R_{yield}, was determined by calculating the ratio of ultimate shear resistance, S_u, to nominal shear resistance, S_y, measured during testing. The overall average of R_{yield} for the 32 tests was 1.14 (SD of 0.056 and CoV of 4.9%) as listed in Table 5-19. Note, the overstrength obtained for CSP walls of 1.4 due to the use of Douglas fir plies was not considered because it would not occur in all situations. The factor accounting for the overstrength due to strain hardening, R_{sh}, was chosen as 1.0 since no strain hardening occurs during the loading of a light gauge steel frame / wood panel shear wall. A value of 1.0 was also suggested for the overstrength developing from a collapse mechanism, R_{mech}, since no codified design procedures presently exist in Canada.

Table 5-19: Overstrength-related force modification factors

Reference	Calculation of R_o						Proposed R_o (NBCC)
	R_{size}	R_Φ	R_{yield}	R_{sh}	R_{mech}	R_o	
Boudreault (2005)	1.05	1.43	1.22	1	1	1.83	1.7
Blais (2006)	1.05	1.43	1.11	1	1	1.67	
Boudreault/Blais	1.05	1.43	1.20	1	1	1.80	
Rokas (2006)	1.05	1.43	1.14	1	1	1.72	
Hikita (2006)	1.05	1.43	1.14	1	1	1.71	

Table 5-19 includes a summary of all *R*-factors used to determine the overstrength-related force modification factor by all applicable researchers (Boudreault, 2005 and Blais, 2006). A single test-based R_o value of 1.70 is recommended for design regardless of sheathing and fastener schedule.

5.8 Influence of Gravity Loads on Lateral Performance of Shear Walls

The steel frame / wood panel shear wall is intended to resist the lateral loads from wind and earthquakes in a structural system. It also is generally expected to carry gravity loads from the storeys above, including; dead loads, occupancy live loads, snow and rain, etc. This research program, dedicated to develop and evaluate a seismic design method for steel frame / wood panel shear walls, has revealed that gravity and lateral loading should not be considered independently. Previous testing has demonstrated that the compression loads on the wall, if not designed for properly, may result in the failure of the chord studs by local buckling (Serrette et al., 1996b; Morgan et al., 2002; Branston, 2004). Hence, for the series of tests described herein the chord studs were selected based on an estimate of the compression force associated with the sheathing fastener failure mode in addition to an applied gravity load (Chapter 4). As noted previously the chord studs did not fail, rather damage to the walls was limited to the sheathing connections (Chapter 4). Nonetheless, a comparison of the test results is presented in order to identify any possible impact on lateral load carrying capacity, stiffness and behaviour due to the application of combined gravity and lateral loads, even when sheathing connection failure occurs. A single shear wall within each configuration was tested with the lateral load alone; while the other walls were subjected to combined lateral and gravity loads. Table 5-20 lists normalized values comparing the energy, stiffness, yield resistance and ductility of each test wall to that obtained for the related wall that did not carry gravity loads. The tabulated ratios were determined by dividing the result (energy, stiffness, yield resistance or ductility) for the each test specimen by the result for the specimen that was subjected to lateral loading alone, within the same configuration and loading protocol (monotonic or cyclic).

Table 5-20 : Normalized properties of shear walls (combined loading / lateral loading)

Specimen	47 Series[1]	48 Series[2]	49 Series[1]	50 Series[2]	51 Series[1]	52 Series[2]	53 Series[1]	54 Series[2]	55 Series[1]	56 Series[2]
Dissipated Energy										
A	0.99	0.96	1.08	0.97	1.34	0.66	1.02	1.23	0.92	1.00*
B	0.93	0.94	1.00*	0.92	1.00*	0.65	0.99	1.26	1.00*	0.90
C	1.00*	1.00*	1.36	1.00*	1.21	1.00*	1.00*	1.00*	0.90	0.89
D	N/A	N/A	1.00	N/A	N/A	N/A	N/A	N/A	0.93	N/A
Stiffness, K_e										
A	1.01	1.09	1.21	0.93	0.93	0.85	0.90	0.88	0.93	1.00*
B	0.98	1.21	1.00*	1.06	1.00*	1.28	0.60	0.80	1.00*	1.13
C	1.00*	1.00*	1.13	1.00*	1.15	1.00*	1.00*	1.00*	0.86	1.08
D	N/A	N/A	1.17	N/A	N/A	N/A	N/A	N/A	0.89	N/A
Yield Resistance, S_y										
A	0.99	0.99	0.94	0.98	0.98	0.93	1.02	0.95	0.92	1.00*
B	0.91	0.98	1.00*	0.94	1.00*	0.90	0.97	0.99	1.00*	0.98
C	1.00*	1.00*	1.11	1.00*	0.99	1.00*	1.00*	1.00*	0.91	0.94
D	N/A	N/A	0.99	N/A	N/A	N/A	N/A	N/A	0.94	N/A
Ductility, +										
A	1.08	1.29	1.45	0.94	1.26	0.70	0.89	1.17	1.18	1.00*
B	1.09	1.35	1.00*	1.11	1.00*	1.01	0.66	1.03	1.00*	1.09
C	1.00*	1.00*	1.23	1.00*	1.38	1.00*	1.00*	1.00*	1.06	1.11
D	N/A	N/A	1.17	N/A	N/A	N/A	N/A	N/A	1.02	N/A

[1] Monotonic Test Series

[2] Reversed Cyclic Test Series

* Shear wall specimen not subjected to gravity load. Energy, stiffness, yield resistance and ductility values of this wall used to calculate ratio for other walls within same configuration.

A series of bar charts comparing each shear wall test's mechanical properties grouped by series and by fastener schedule have also been prepared. Charts of particular interest are presented in this section, while the remaining are included in Appendix E. In the following figures the asterisk "*" denotes that no gravity loads were applied to the wall during testing.

5.8.1 Comparison of Energy Dissipation

Figure 5-8 shows the energy under the EEEP backbone curve dissipated by each of the monotonic tests (Table 5-20). These measured values representative of the design level energy of the system. They differ from those quoted in Chapter 4 because they are limited by the failure displacement of $\Delta_{0.8Su}$ or the 2.5% drift limit as shown in Figures 5-4 and 5-5.

The shear walls tested under monotonic loading do not indicate any consistent enhancement of design level energy due to the inclusion of gravity loads. The control test (which did not include gravity loads) varied between being the lowest, mid and highest energy dissipater between each series.

The energy for the reversed-cyclic tests was measured as the area under the backbone curve developed through the application of the EEEP method. The values represented in Table 5-17 are the sum of the absolute values of the energy from the positive and negative regions. Figure 5-9 shows these values in bar chart format for visual comparison. Unlike the monotonic tests, the reversed-cyclic tests showed that the inclusion of gravity loads reduced the design level energy with the exception of Series 54.

The OSB sheathed walls with 75/305 mm screw schedules under combined monotonic loading (51A & 51C) were able to dissipate more energy than the walls subjected to lateral loading alone (51B). However, the improved behaviour of the OSB sheathed walls under combined loading was not seen in the reversed cyclic tests, as shown in Figure 5-9.

The CSP sheathed walls (Series 53-56) results showed no definitive trend in energy dissipation under combined monotonic loading with energies both higher and lower than the control test. Monotonic series 53 and 55 had normalized ranges between 0.99 – 1.02 (944 – 973 J) and 0.92-1.00 (1239 – 1380 J) respectively.

The DFP paneled shear wall test results, Series 47 and 48, indicate that energy dissipation was reduced with combined gravity and lateral loading for both the monotonic and reversed-cyclic loading regimes with ratios of 0.93 – 1.00 for the monotonic tests and 0.96 – 1.00 for the cyclic tests.

A similar comparison of the dissipated energy from test data was completed in Section 4.7.3. Unlike the EEEP energies shown in Figure 5-8 the energy from the monotonic test data were consistently improved by the inclusion of gravity loads. The reversed-cyclic results were similar with the exception of test Series 52 and 56. This difference in results indicates that improvement in energy dissipation for walls under combined gravity and lateral loads is not relevant at the design level.

The monotonic test EEEP energy results showed no trend in either improved or reduced performance of shear wall under combined gravity and lateral loading. The reversed-cyclic EEEP energy indicates that the inclusion of gravity loads during testing reduces the walls ability to dissipate energy. In Chapter 4, the results of the monotonic testing pointed toward improved energy dissipation in specimens under combined gravity and lateral loading. As well, with the exceptions of test Series 51 and 55 the cyclic results reinforced the findings from the monotonic tests. This indicates that the decisive amount of energy differentiating the improved performance of the wall specimens lies within the trailing cycles and residual energy that is dissipated past the point of the failure defined by the EEEP method. It can be concluded that the combined gravity and lateral loading does improve the energy dissipation of the steel frame / wood panel shear wall, but the region where performance is improved is not useful to the designer and therefore should not be considered in design.

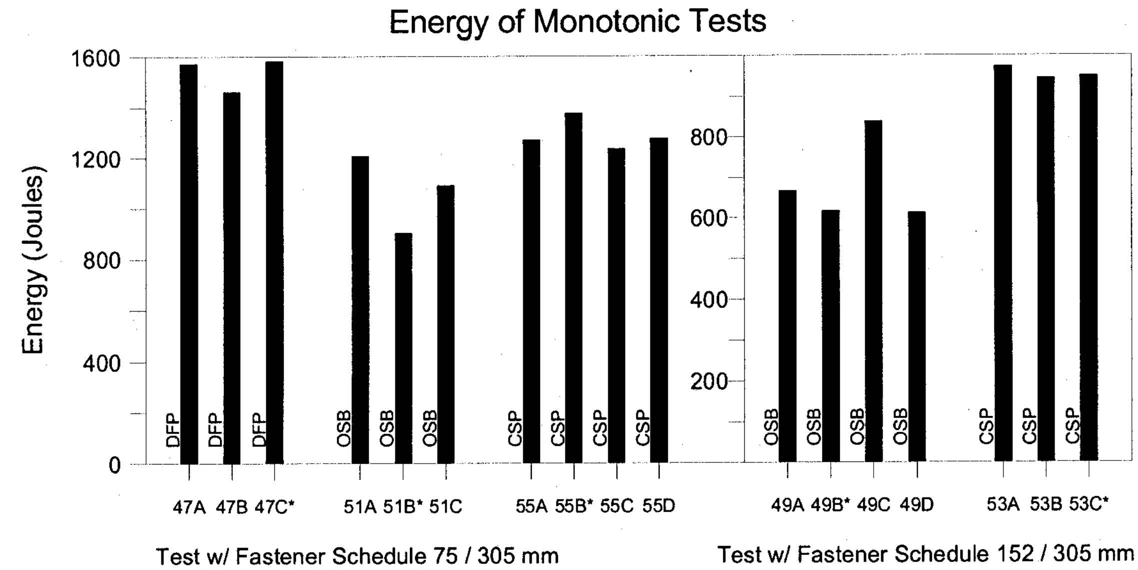


Figure 5-8 : Graph of energy for monotonic tests

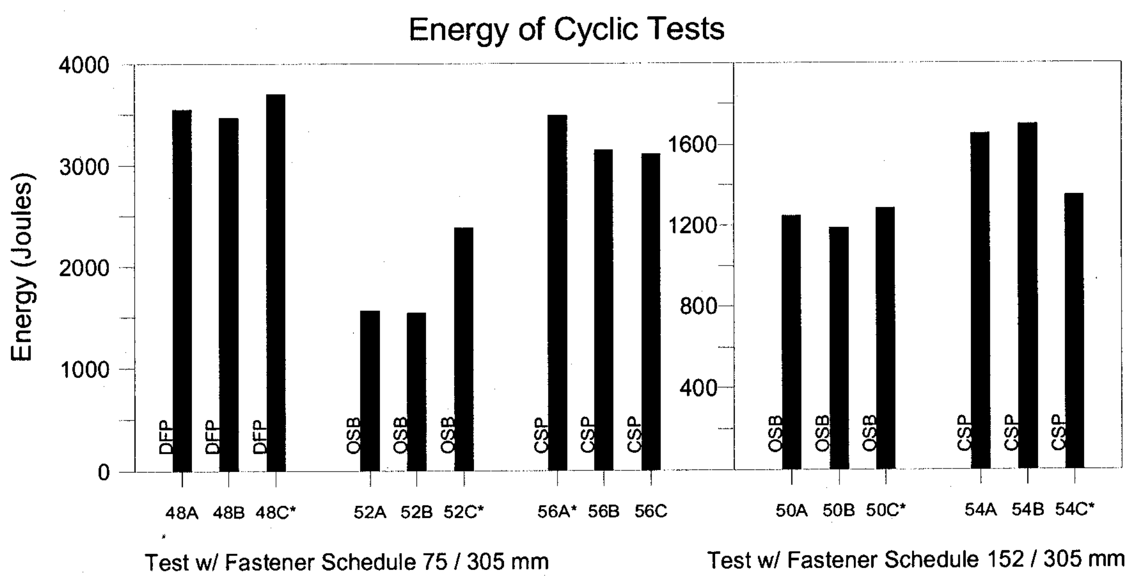


Figure 5-9 : Graph of cumulative dissipated energy from under EEEP backbone curve for reversed-cyclic tests

5.8.2 Comparison of Initial Shear Stiffness

To establish the influence of gravity loads in combination with lateral loads on the initial shear stiffness of the walls a comparison of the monotonic and reversed-cyclic results was prepared. Through a visual comparison of the monotonic shear stiffness results presented in Figure 5-10 the affect of gravity loads can be observed to be inconsistent. The test results for combined loading are both above and below the shear stiffnesses determined from the control group which did not include gravity loads. This wide range of behaviour was not expected because it was thought that the inclusion of the gravity loads would have restricted the rotation of the exterior corner connections of the frame due to the geometry of the system causing an increase in stiffness. As well, the gravity loads should have reduced tension forces in the chord stud limiting overturning and possibly lowering the stress in the connectors. This experimental data has shown that the impact of these factors are not significant.

Inspecting each configuration individually, Series 55 shows improved performance with gravity loads but Series 53 with the same fastener schedule does not, nor does Series 51 with the same sheathing type. The inability to associate trends with other related configurations suggests that change in affect of the gravity loads between configurations is an arbitrary result. The inconsistency in results is more likely to be a function of the variability in materials and construction.

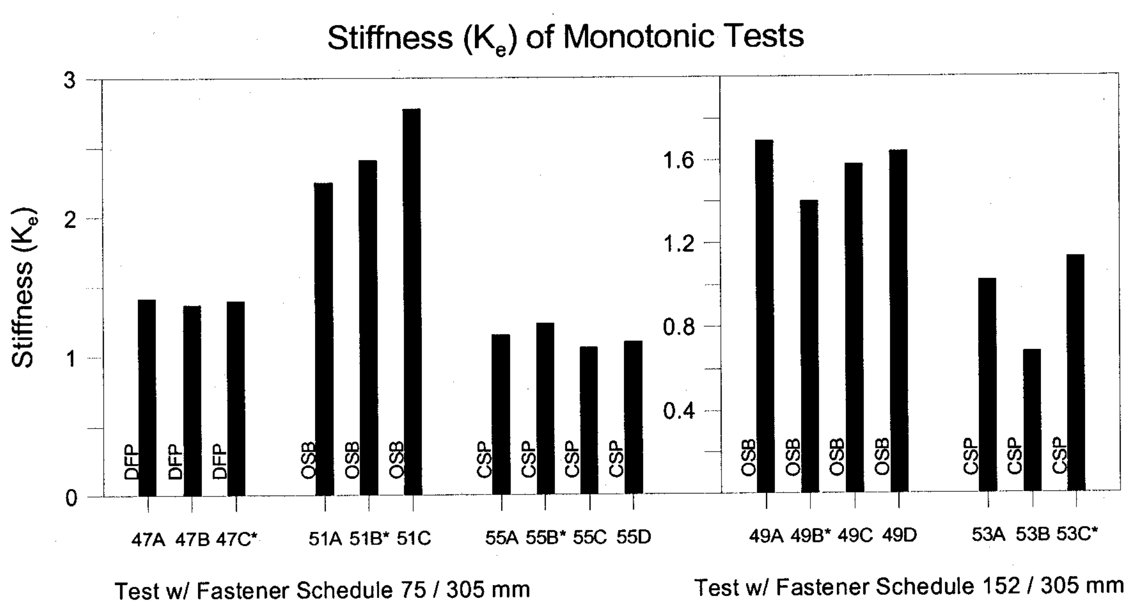


Figure 5-10 : Graph of stiffness for monotonic tests

In Figure 5-11 the effective stiffness of the shear walls tested under reversed cyclic loading are shown. The absolute stiffness for each direction, positive and negative, have been averaged. Similarly to the monotonic results the reversed-cyclic initial shear stiffness do not display results indicating a consistent influence of the gravity loads. Even the improved performance in stiffness of the individual Series 49 from monotonic testing did not carry through to the reversed-cyclic results. The corresponding Series 50 had test results with higher and lower shear stiffnesses than the control test within the group.

It is possible to link the reversed-cyclic shear stiffness results with each sheathing type. The CSP sheathed Series 54 and 56 display a lower stiffness due to the inclusion of gravity loads, while the DPF sheathed Series 48 shows the opposite. These test groups are individually so small and the variability in material is so high it would not be conservative to draw any conclusions from such a small subset of data. As well, the relative unpredictability of the affect of gravity loads displayed for other design properties coupled with the relatively small variation in test results demonstrates that they are not significant if designed for properly.

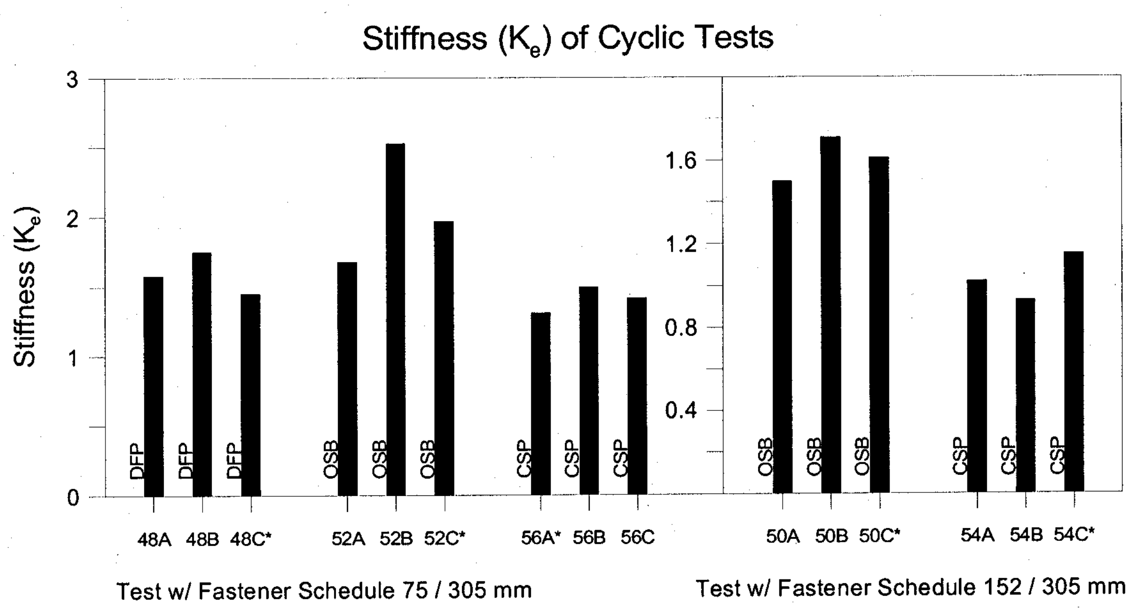


Figure 5-11 : Graph of average directional stiffness for reversed cyclic tests

5.8.3 Comparison of Yield Shear Resistance

The yield resistances calculated using the EEEP method, presented in Figure 5-12, showed an overall trend of lowered resistance in specimens under combined loading with outliers in Series 49 and 53. The normalized values indicated that the results are fairly consistent, all within normalized range of 0.90-1.11 relative to the benchmark tests. The trend in yield resistance may be related to naturally occurring imperfections in the wood or variation in loading.

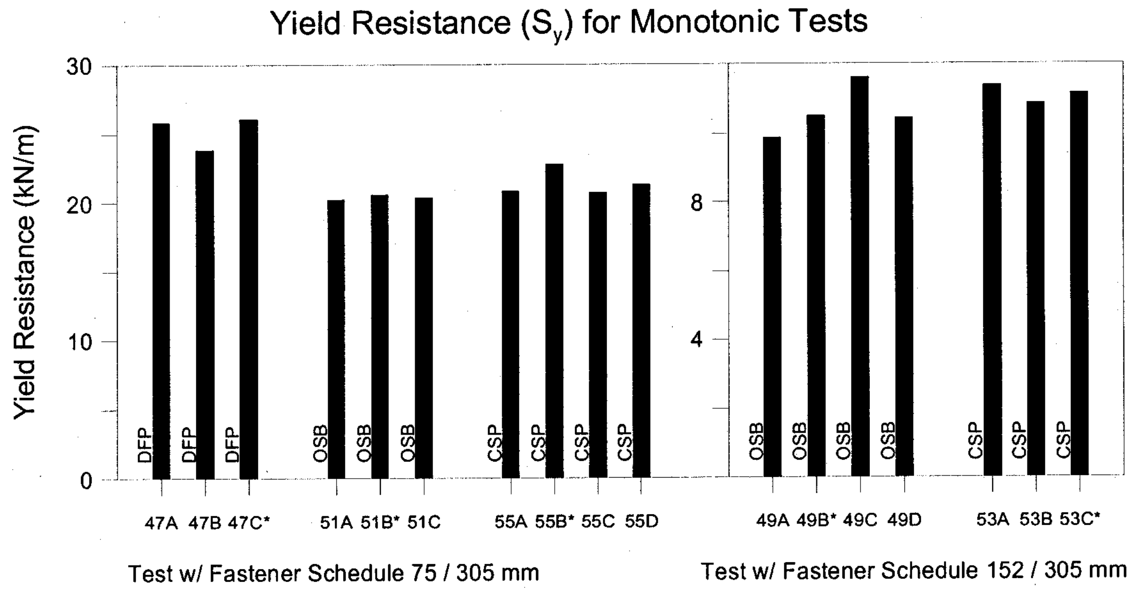


Figure 5-12 : Graph of yield resistance for monotonic tests

Figure 5-13 represents the normalized values of the yield resistance for the monotonic tests previously presented in Table 5-17. This bar chart for the monotonic tests shows the consistency of the design level values. The stability of the results between tests with and without gravity loads suggests that the affect, if any, of the variation in loading is not remarkable. The results have a similar range of variation as those found by Branston (2004) which did not include gravity loads.

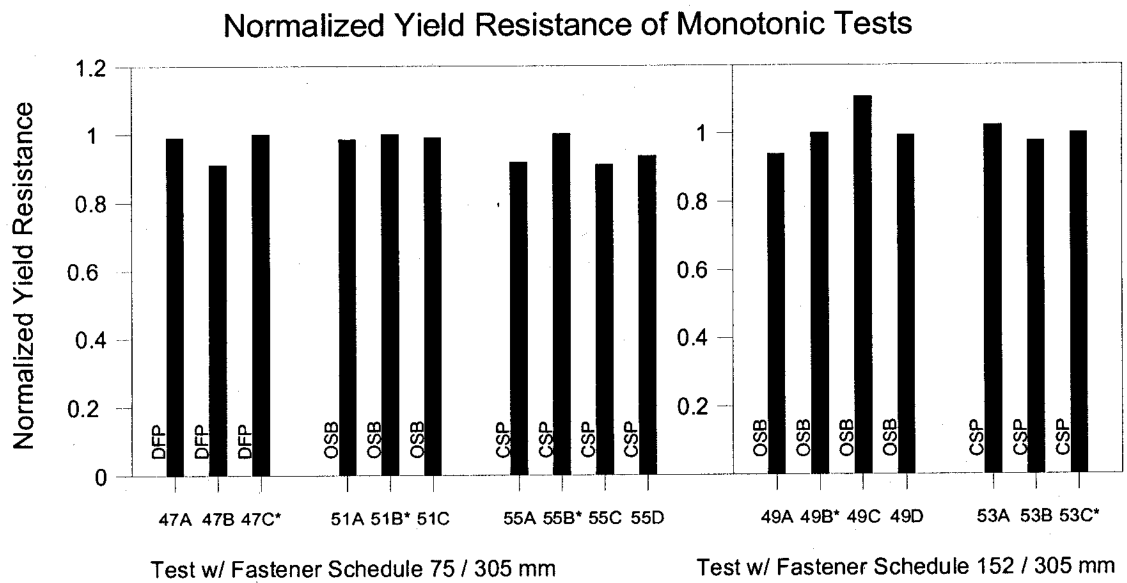


Figure 5-13 : Graph of normalized yield resistance for monotonic tests

The yield resistances shown in Figure 5-14 are the averaged absolute values from the positive and negative direction derived from the reversed-cyclic tests. The tests that did not include gravity loading consistently had higher yield resistances than those which did. However, the normalized range of values, 0.90-1.00, is almost half that of the yield resistances from the monotonic test series. The limited variation in results is within an acceptable range a test series consisting of nominally identical specimens and identical loading protocols. Thus, the trend in higher yield resistances for the walls which underwent lateral loads only could just as likely be a result in material discrepancies as the inclusion of gravity loads. In fact, for Series 48 the normalized values are 0.98, 0.99 and 1.00, with such a narrow band deviation it would be difficult to justify any distinct influence of the inclusion of gravity loads on the shear yield resistance performance of the shear walls.

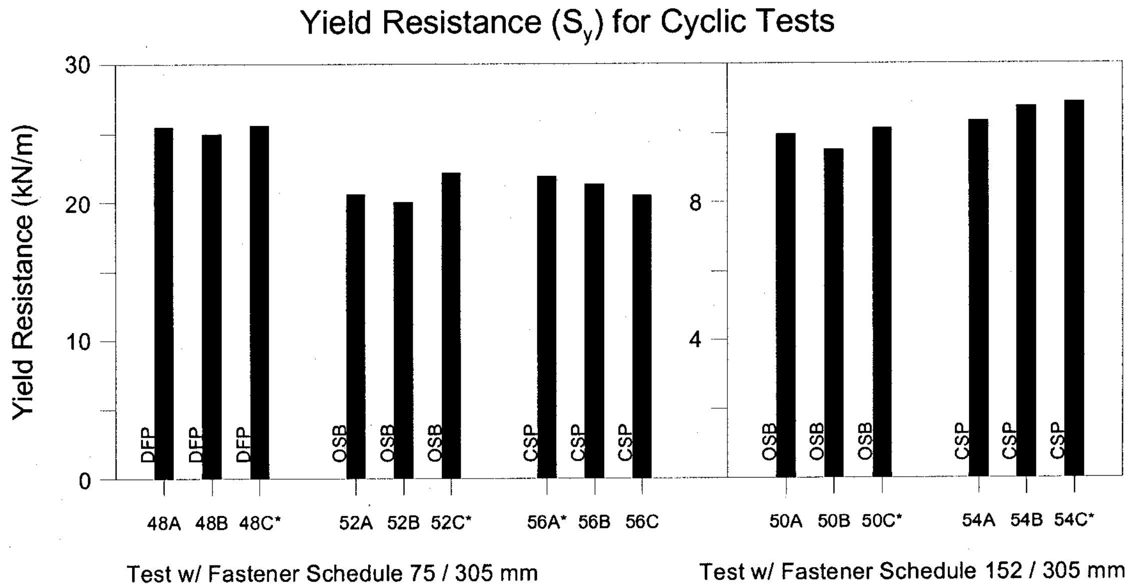


Figure 5-14 : Graph of average directional yield resistance for reversed-cyclic tests

5.8.4 Comparison of Ductility

The results of testing indicate that the ductility of each configuration was more highly influenced by the sheathing type than the inclusion of gravity loads. The ductility of the OSB sheathed walls was improved in most instances with the combined lateral and gravity in comparison to the test specimens that underwent lateral loading only as shown in Figure 5-15. The degree to which the ductility increased varied between configurations. Series 51, sheathed with OSB panels, shows the most dramatic improvement in ductility with a normalized range of 1.00 – 1.38. Series 53 is the exception to the trend with test 53C*, which did not include gravity loads, having the highest ductility. Its normalized range, 0.66 - 1.00, is the other extreme of Series 51 which indicates that either the influence of the gravity load is inconsistent or the material factors overwhelm the effect any change in loading may have.

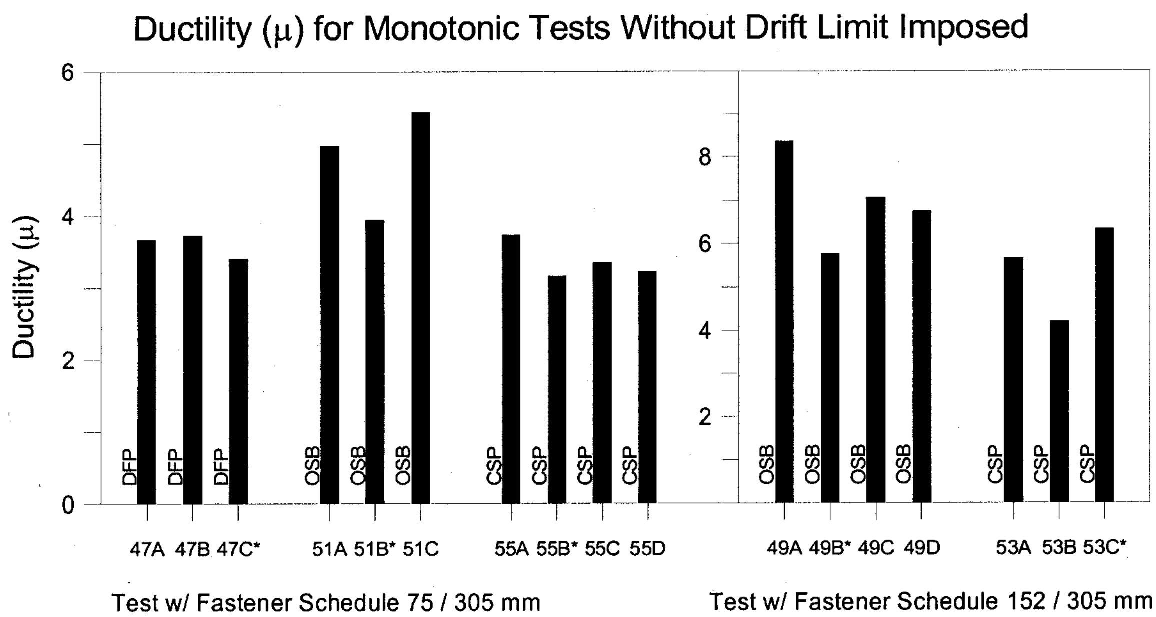


Figure 5-15 : Comparative graph of ductility for monotonic tests

The ductility derived from the reversed-cyclic testing were also generally improved by the inclusion of gravity loads as illustrated in Figure 5-16. There were two exceptions, Series 50 and 52, where the control test which did not include gravity loads, had a higher ductility than one of the two tests within the series. Series 50 had a normalized range of ductility of 0.94 – 1.11 and Series 52, a normalized range of 0.70 – 1.01. This behaviour suggests the influence of gravity loads is within the previously expected variation in results due material differences. To help support this point, Series 54, the same wall configuration as Series 53 from monotonic testing the opposite behaviour from monotonic behaviour with larger ductilities under combined loading. The remaining Series, 48 and 56, were consistent in performance between loading regimes.

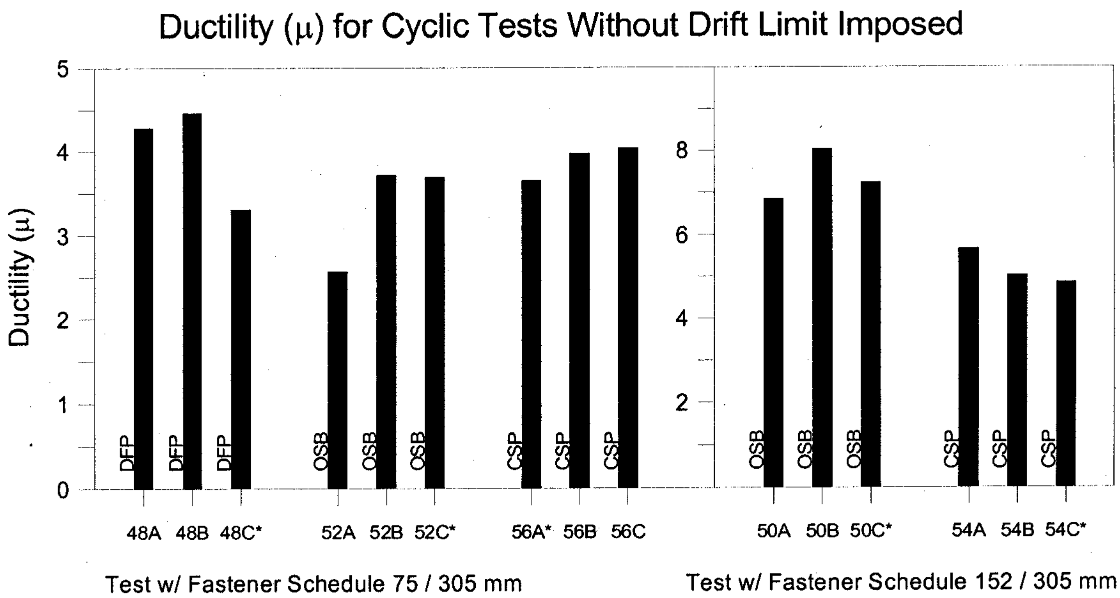


Figure 5-16 : Graph of averaged directional ductility for cyclic tests

5.8.5 Summary

The combined gravity loads and lateral loading of steel frame / wood panel shear walls did not have a significant impact on the design level energy, stiffness, yield resistance or ductility of the system. The influence of the gravity load in conjunction with lateral loads was found to increase the shear walls ability to dissipated energy past the point of failure. This energy is not included at the design level but implies an additional reserve relative to walls under lateral loading only. The results of the initial effective stiffness showed no consistent influence of the combined gravity and lateral loading. The range of results were within that which normally encompasses the variability in the construction and materials. Thus, it was concluded that the influence gravity loading was minor and inconsistent. The shear yield resistance values calculated for the test program were very consistent within each series, but displayed a minor trend towards higher yield resistance in the walls which underwent lateral loading only. The trend was so slight that it was always well within the range expected for material variation and sometimes as small as a 1% improvement. The ductility of the shear walls had far more varied results with respect to the shear yield resistance. In general the loading protocols including gravity increased the ductility. The exceptions to this trend indicate that the sheathing quality possibly plays a larger role in the ductile behaviour of the shear wall system than the inclusion of gravity loads. In summary, the inclusion of gravity loads during lateral testing should be considered as influential on the performance the steel frame / wood panel shear walls as the variation in wood properties and details of the construction if adequately designed. Overall, this test program has shown that if the steel frame / wood panel shear wall is properly designed following capacity based methods for combined vertical and lateral loads this type of shear wall maintains similar performance levels as those walls tested under lateral loads only.

CHAPTER 6 HYSTERETIC SHEAR WALL MODELS

6.1 INTRODUCTION

An important phase of the steel frame / wood panel shear wall research program at McGill University is the use of non-linear time history dynamic analyses to evaluate the performance of the tested systems and more realistic buildings under seismic ground motion records (Blais, 2006). Preliminary dynamic analyses of two representative buildings have been carried out by Blais based on hysteretic models that were calibrated using shear wall test data in which lateral loads alone were applied to the test walls. The steel frame / wood panel shear walls were calibrated using the Stewart hysteretic model (1987), chosen by Boudreault (2005), which was originally developed for the analysis of timber framed shear walls with nailed plywood sheathing. It was considered to be the most applicable of the five models considered: The Bouc-Wen-Baber-Noori (BWBN) (Baber & Noori, 1986), the Stewart (Stewart, 1987), the Florence (Ceccotti & Vignoli, 1989), the Dolan (Dolan, 1989) and the Folz & Filitrault [Cashew] (Folz & Filiatrault, 2001). It also is easily calibrated to account for the pinching and stiffness degradation observed during testing and is integrated into the dynamic analysis program, Ruaumoko (Carr, 2000), which was also selected for this research program.

Since all of the previous hysteretic models were calibrated using data from laterally loaded shear walls, the objective of this phase of the research was to determine whether the existing calibration factors recommended by Boudreault (2005) would be appropriate for shear walls for which lateral and gravity loads were applied. The resulting shear wall models could then be used in the future for additional non-linear time history dynamic analyses of cold-formed steel buildings constructed with wood sheathed shear walls.

6.2 RELEVANT SHEAR WALL CHARACTERISTICS

This section briefly describes the force vs. deformation characteristics that need to be considered for calibration of the steel frame / wood panel shear walls using the Stewart model. More in-depth descriptions of this procedure can be found in Boudreault (2005). A description of the dynamic analyses that can be carried out using the calibrated models is contained in the work of Blais (2006).

6.2.1 Stiffness Degradation

Stiffness degradation is the reduction in shear capacity of a wall over two successive cycles at the same displacement. The slope of the excursion into each region measures the stiffness of a cycle, which can be calculated by dividing the peak force for a direction by the corresponding displacement. As the loading cycle repeats itself the stiffness decreases and the area enclosed by the hysteresis loop (dissipated energy) is also reduced. The stiffness of the system decreases as the bearing distortion in the wood increases and as either bearing failure in the wood, the pull-through of the fastener or the shear failure of the fastener occurs and the system stiffness tends towards zero.

6.2.2 Pinching

Pinching is the inability of the shear wall to resist load at low displacements due to the bearing distortion at the sheathing connections caused by previous loading cycles. The permanent slotted deformations of the wood sheathing due to bearing from the screw connections can only develop shear resistance when the screw fastener comes into contact with the end of the slot. As the displacements of a test protocol increase the effects of pinching become progressively more evident. The overall effects of pinching are the reduced ability to dissipate energy and the lack of in-plane stiffness through the zero displacement region.

6.2.3 Strength Degradation

Strength degradation is similar to stiffness degradation. It can be identified when a wall is unable to maintain its peak shear resistance for consecutive cycles at the same displacement level. This effect is caused by the permanent bearing distortion of the wood surrounding the frame to panel fasteners, which prevents the wall from developing the same level of resistance during repeated cycles.

6.3. STEWART DEGRADING HYSTERESIS MODEL

The Stewart degrading hysteresis model, originally developed for wood shear walls with nailed connections (Stewart, 1987), has also been applied to reinforced concrete walls, steel shear walls (Carr, 2000) and to steel diaphragm systems (Martin, 2002). The model incorporates parameters to account for characteristics including; stiffness degradation, pinching, ultimate and yield force, slackness, softening and reloading. However, the formulation of the model used is not able to relate the strength degradation aspect of the shear wall behaviour. According to Boudreault (2005), Stewart (1987), Ceccotti & Viginoli (1989) and Dolan (1989) strength degradation should be considered less significant than stiffness degradation and pinching, and hence its exclusion should not be considered critical. This limitation does affect other aspects of shear wall performance which will be discussed in further detail in Section 6.3.2.

6.3.1 Experimental Data Matching

The Stewart model requires over 30 parameters to accurately replicate the experimental hysteresis of a shear wall. The majority of these parameters, which concern the frame type properties, were obtained following the Ruaumoko manual. Seven parameters were determined on a trial and error basis by matching the test hystereses to the model, and the ultimate force, F_u, parameter was calculated as described in Section 6.3.2. Figure 6-1 illustrates some of the model parameters on a force vs. displacement graph.

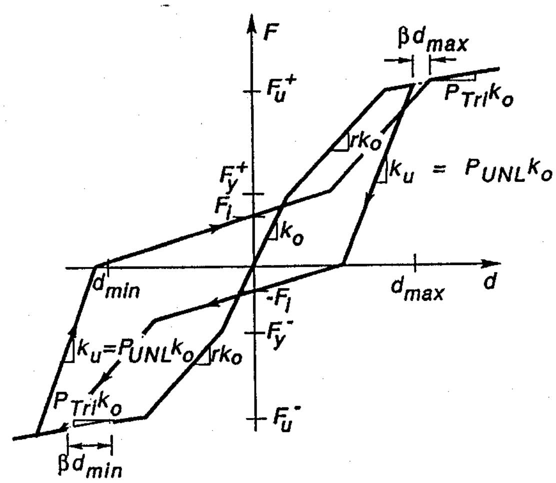


Figure 6-1 : Stewart degrading hysteresis (*Carr, 2000*)

Matching the test results with the Stewart hysteresis model was a multi-step process. The monotonic test curves for a particular wall configuration and test series were first superimposed. The program Hysteres (Carr, 2000), within the non-linear time history dynamic analysis program Ruaumoko (Carr, 2000), was then run inputting the parameters listed in Table 6-1. Initially, values for these parameters were obtained from the recommendations of Boudreault (2005), however in some cases it was necessary to modify the variables to obtain a curve of best fit (visual inspection) to the superimposed monotonic curves. Only a single set of parameters per wall configuration was permissible thus a compromise between the three test curves was made. A comparison of dissipated energy between the test and model was also carried out. Table 6-1 shows the recommended parameter values for the five shear wall configurations studied in this body of research. Figure 6-2 shows the hysteresis behaviour for a typical shear wall test with the Stewart model superimposed, and Figure 6-3 contains the corresponding cumulative dissipated energy from the test data and the Stewart model. The hysteretic comparison plots for the remaining monotonic test configurations can be found in Appendix D.

Table 6-1 : Stewart hysteresis parameters for light gauge steel frame / wood panel shear walls

Group	Wood Panel	Size	Screw Pattern	K_o (kN/mm)	r	F_y (kN)	F_u (kN/m)	F_i (kN/m)	P_{UNL}	β	α
47 & 48	DFP	1220 x 2440 mm	75/305 mm	1.85	0.26	18.00	34.20	3.00	1.55	1.09	0.23
49 & 50	OSB	1222 x 2440 mm	152/305 mm	1.60	0.20	7.70	12.70	1.00	1.75	1.10	0.45
51 & 52	OSB	1224 x 2440 mm	75/305 mm	2.60	0.28	15.00	26.70	2.40	1.25	1.10	0.45
53 & 54	CSP	1226 x 2440 mm	152/305 mm	1.08	0.21	8.20	13.83	1.40	1.65	1.10	0.41
55 & 56	CSP	1228 x 2440 mm	75/305 mm	1.50	0.33	15.00	27.00	2.45	1.65	1.09	0.23

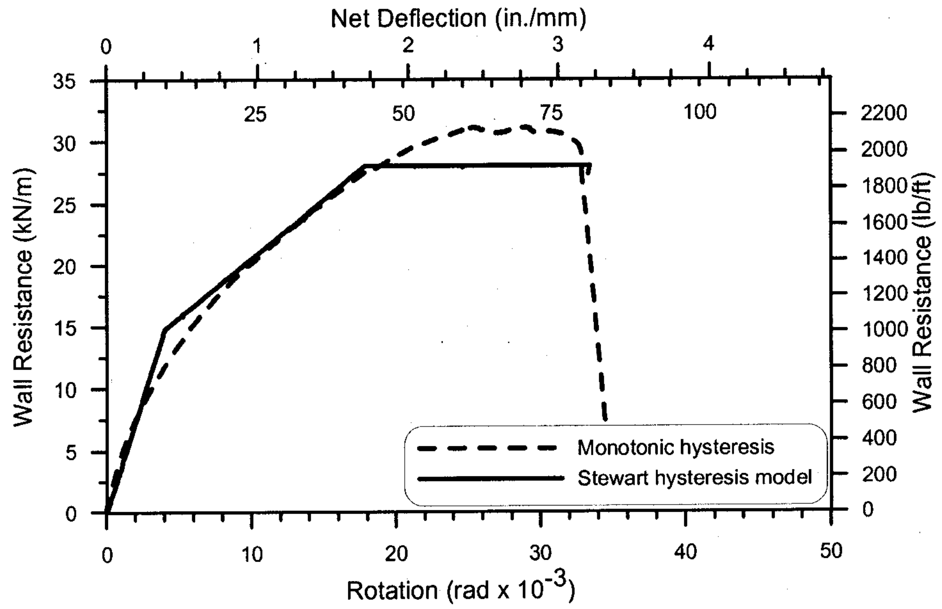


Figure 6-2 : Superposition of Stewart model and experimental monotonic curve for graph of resistance vs. displacement (Test 47A)

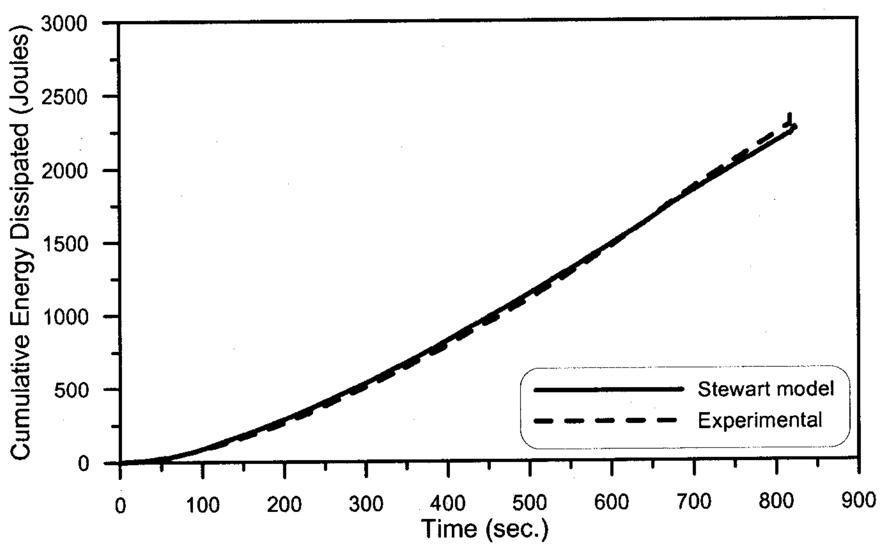


Figure 6-3 : Superposition of Stewart model and experimental monotonic curve for graph of cumulative energy dissipation (Test 47A)

Note, the initial stiffness, k_o, and the yield force, F_y, were obtained by visual inspection of the test results, and hence will be different from the values calculated with the EEEP method in Section 5.1.

Following a similar approach the data for each reversed cyclic test was graphed and superimposed for a particular configuration. Starting with the parameters derived from the monotonic test results a process of trial and error was again used to obtain a best fit resistance vs. displacement hysteresis using the Hysteres program. Since there can only be one set of parameters for each wall configuration a compromise between the set of monotonic and cyclic parameters was sometimes necessary. The overall accuracy of the cyclic performance of the wall was considered more critical and therefore favoured in the selection of final parameter variables. Table 6-2 contains the final parameters for the shear walls contained in this test series and the parameters for walls with the same configuration from Boudreault 2005. To draw a comparison between the results the most recent parameters have been divided by Boudreault's (2005) values to determine a change in terms of percentage. The average percentage for the entire series was calculated along with the standard deviation and the coefficient of variation. Between the data sets the values for P_{UNL}, β, and α remained unchanged. The values for the initial stiffness, k_o, were slightly lower than the previous calibration. The lower initial stiffness agrees with the test results from Chapter 5 which showed a tendency for this test series to have slightly lower effective stiffness than the previous study (Branston, 2004). The stress through the zero displacement axis, F_i, was also lower on average. However, no data regarding this property was formally collected from testing to provide a comparison. The ultimate stress, F_u, was similar between the two calibrations, but did not consistently reflect the differences between the two sets of test data. The yield stresses, F_y, standardized for this study were lower in comparison to the one previously calibrated by Boudreault (2005). In general, this study had higher yield resistances, S_y, than Branston (2004) which Boudreault's (2005) calibrations are based upon. This indicates a difference in either judgment or some variation in the shape of the curve in this region. The change in stiffness past the yield stiffness was represented by the parameter, *r*, in the model. This variable was the only value to increase relative to Boudreault's calibration.

The difference in estimations of this particular parameter for the OSB and CSP sheathed panels with 75 / 305 mm fastener schedules 27% and 43%, respectively. However, the average variation in parameter, *r*, was only 15.2% with standard deviation of 19.4%. In comparison, the parameter, F_y, also deviated by 15.2% in comparison to Boudreault (2005), but its standard deviation is limited to 7.4%. In contrast, the estimates for initial stiffness, k_o, on average matched very closely, 97%, with a standard deviation 7.2%. The ultimate yield strength, F_u, was previously overestimated by 6% on average with a standard deviation of only 4.3%. Taking into consideration the accuracy of the parameters F_i, P_{UNL}, α and β the overall agreement of the parameters between the two series is reasonable. It is recommended that the lower bound values for parameters be used in application and in further model development. Figure 6-4 depicts the hysteretic behaviour of a typical reversed cyclic test with the superimposed Stewart model for the particular configuration. Figure 6-5 illustrates the cumulative dissipated energy from the test data and from the model. The hysteretic comparison plots for the remaining reversed cyclic test configurations can be found in Appendix D.

Table 6-2 : Comparison of Stewart hysteresis parameters for light gauge steel frame / wood panel shear walls (Boudreault, 2005 and Hikita, 2006)

Group	Wood Panel	Size	Screw Pattern	K_o (kN/mm)	r	F_y (kN)	F_u (kN/m)	F_i (kN/m)	P_{UNL}	β	α
47 & 48[1]	DFP	1220 x 2440 mm	75/305 mm	1.85	0.26	18.00	34.20	3.00	1.55	1.09	0.23
13 & 14[2]	DFP	1220 x 2440 mm	75/305 mm	1.75	0.26	22.00	36.20	3.00	1.55	1.09	0.23
49 & 50[1]	OSB	1220 x 2440 mm	152/305 mm	1.60	0.20	7.70	12.70	1.00	1.75	1.10	0.45
21 & 22[2]	OSB	1220 x 2440 mm	152/305 mm	1.60	0.20	8.40	13.50	1.00	1.75	1.10	0.45
51 & 52[1]	OSB	1220 x 2440 mm	75/305 mm	2.60	0.28	15.00	26.70	2.40	1.25	1.10	0.45
25 & 26[2]	OSB	1220 x 2440 mm	75/305 mm	3.00	0.22	17.00	28.50	2.50	1.25	1.10	0.45
53 & 54[1]	CSP	1220 x 2440 mm	152/305 mm	1.08	0.21	8.20	13.83	1.40	1.65	1.10	0.41
7 & 8[2]	CSP	1220 x 2440 mm	152/305 mm	1.15	0.20	9.20	13.83	1.50	1.65	1.10	0.41
55 & 56[1]	CSP	1220 x 2440 mm	75/305 mm	1.50	0.33	15.00	27.00	2.45	1.65	1.09	0.23
9 & 10[2]	CSP	1220 x 2440 mm	75/305 mm	1.50	0.23	20.45	30.70	2.25	1.65	1.09	0.23
			Average% of Boudreault, 2005	97.3	115.2	84.8	94.0	99.6	100.0	100.0	100.0
			Standard Deviation	7.2	19.4	7.4	4.3	5.9	0.0	0.0	0.0
			CoV	0.1	0.2	0.1	0.0	0.1	0.0	0.0	0.0

[1] Hikita, 2006
[2] Boudreault, 2005

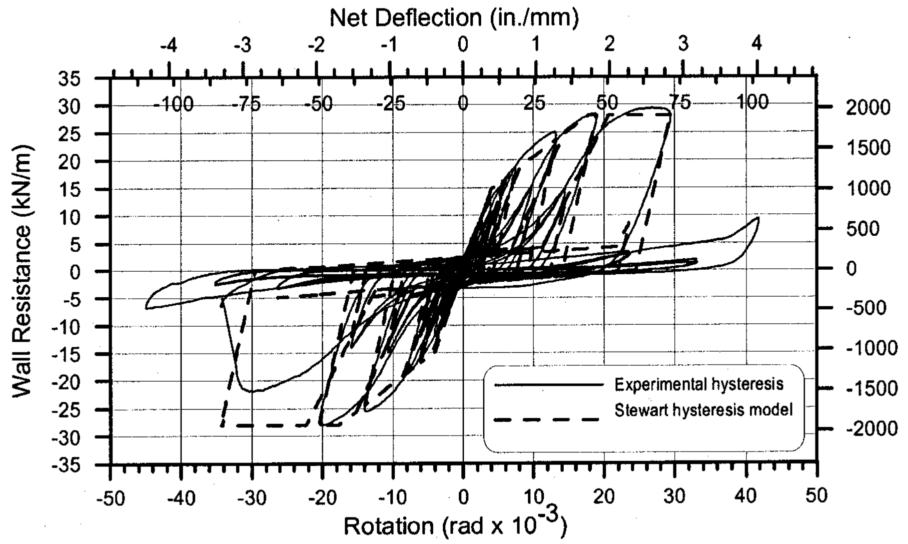


Figure 6-4 : Superposition of Stewart model and experimental reversed cyclic curve for graph of resistance vs. displacement (Test 48A)

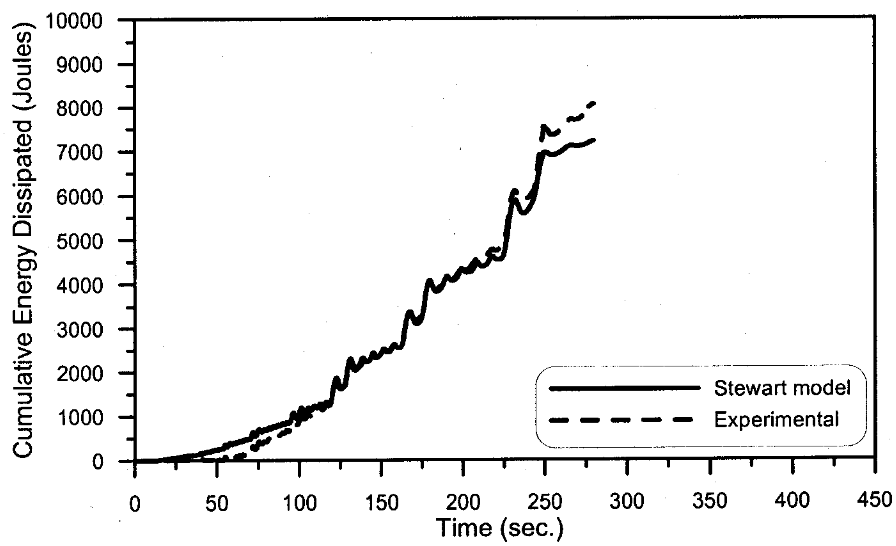


Figure 6-5 : Superposition of Stewart model and experimental monotonic curve for graph of cumulative energy dissipation (Test 48A)

The variation in model parameters recommended by Boudreault (2005) and what was obtained for the shear wall tests described in this thesis does not indicate a drastic change in the wall properties regardless in variation of sheathing, except maybe in yield and ultimate stress in the initial estimates for the parameters. Figure 6-6 shows the experimental hysteresis from test 48A, which included gravity loads during testing, with the Stewart model calibrated by Boudreault (2005) and Hikita (2006) superimposed.

There is a noticeable variation between the two models, but taking into consideration that one calibration must apply to all the test results of a configuration and the calibration is done mainly by visual inspection the differences are minor. The combined gravity and lateral loading does not seem to have had an overall influence on the outcome of the parameters. In Chapter 5 it was concluded that influence of gravity loads on the lateral performance of the steel frame / wood panel shear wall, if designed adequately, was well within the deviation expected for variation in materials. Figure 6-7 shows the experimental hysteresis from test 48C*, which did not include gravity loads during testing, with the Stewart model calibrated by Boudreault (2005) and Hikita (2006) superimposed. The calibrated Stewart models appear to be in good agreement with the test hysteresis regardless of the inclusion of gravity loads during testing. Overall, the variation between the two calibrations of parameters is to be expected since the recommended values were based on visual inspection independently carried out by two people and based on two different data sets. The variability between these data sets due to test procedure and variation in materials is discussed in Section 5.4. In the future, an estimate of the Stewart parameters by a panel of researchers may help improve estimates by giving a better indication of the variability of the parameters due to interpretation.

Visual inspection of the hysteretic model superimposed on the test data (Figure 6-4) illustrates that the calibrated Stewart model matches the behaviour of the tested shear wall until the final loop. The cumulative dissipated energy is also very similar up to approximately 7000 J.

To help illustrate the consistency of results from the Stewart model between test data which underwent lateral loading alone or in combination with gravity loading Figures 6-8 and 6-9 were prepared. Figure 6-8 shows the Stewart models for Tests 47A, 47B and 47C* (47C* did not include gravity loading) superimposed. The resulting models overlap one another and are essentially identical up to displacement of approximately 70 mm, well past the 2.5% drift limit. Similarly, Figure 6-9 represents the Stewart hysteresis models for the cyclic series 48 superimposed. The Stewart models for tests 48A and 48B are very similar in shape and placement. The Stewart model for test 48C* (48C* did not include gravity loading) is very similar in shape to tests 48A and 48B, but appears to be shifted slightly to the right relative to the two other tests. This offset is not exclusive to Test 48C* because it did not include gravity loads since this event is not constant between data sets. To help illustrate the inconsistent behaviour of the Stewart hysteresis models derived from tests which did not include gravity loading relative to those which did Figure 6-10 was prepared. It is representative of the Stewart hysteresis models for Series 52. There is no distinct difference in the shape or position of the Test 52C*, which did not include gravity loads, with regards to Tests 52A and 52B. It maintains a similar shape and its excursions reach displacements in between those obtained by Tests 52A and 52B.

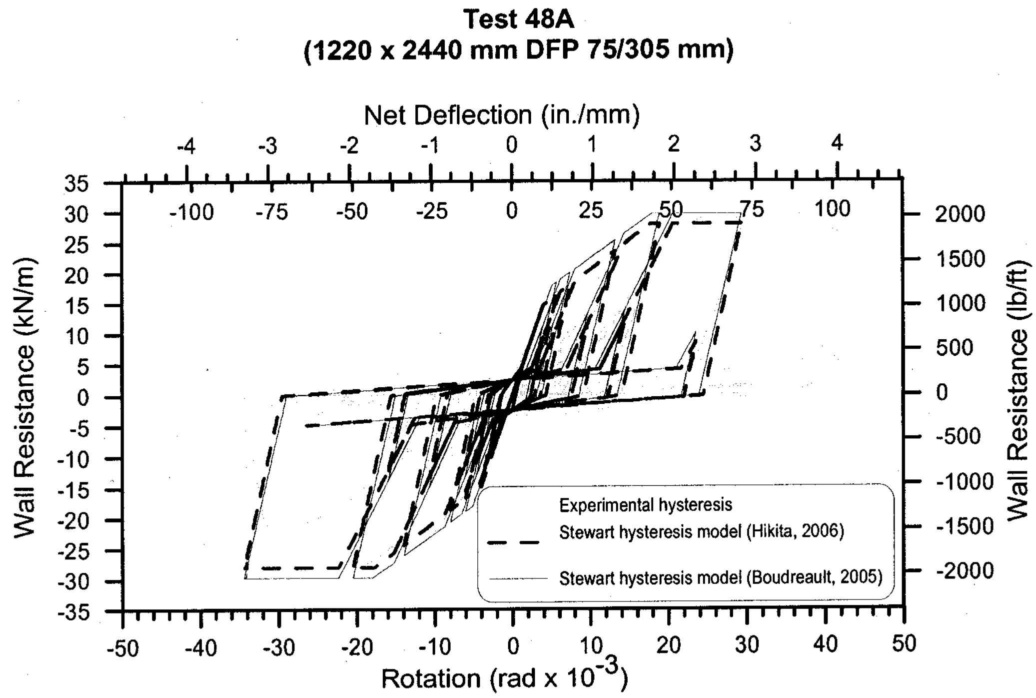


Figure 6-6 : Superposition of Stewart models by Hikita, 2006 and Boudreault, 2005 and experimental reversed cyclic curve for graph of resistance vs. displacement (Test 48A)

Test 48C
(1220 x 2440 mm DFP 75/305 mm)

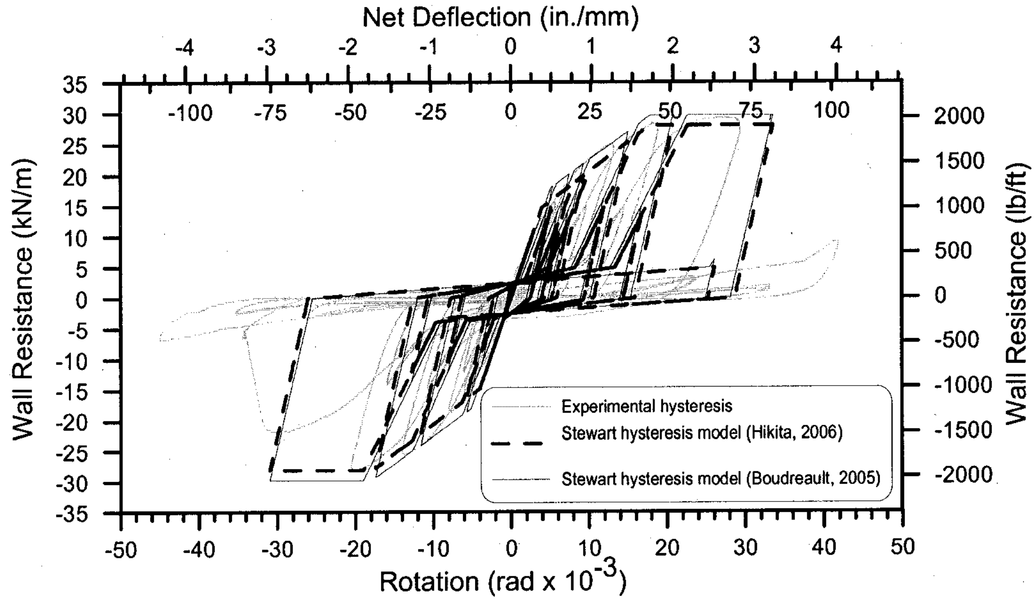


Figure 6-7 : Superposition of Stewart models by Hikita, 2006 and Boudreault, 2005 and experimental reversed cyclic curve for graph of resistance vs. displacement (Test 48C – Test did not include gravity loads)

Series 47
(1220 x 2440 mm DFP 75/305 mm

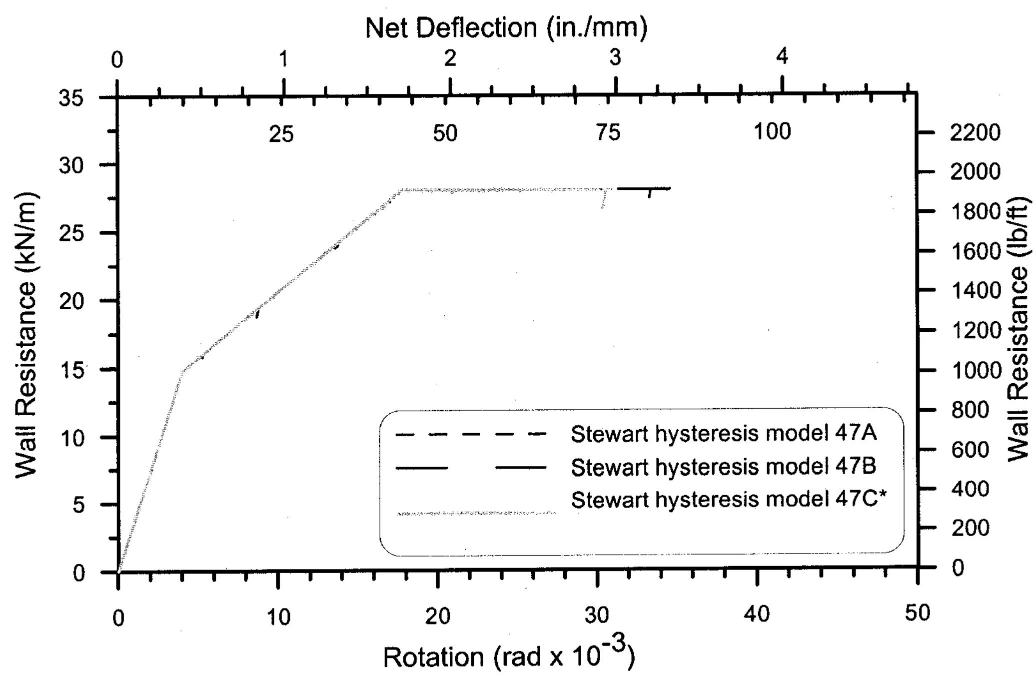


Figure 6-8 : Superposition of Stewart models for tests 47A, 47B and 47C* (Test 47C* - Test did not include gravity loads)

Series 48
(1220 x 2440 mm DFP 75/305 mm)

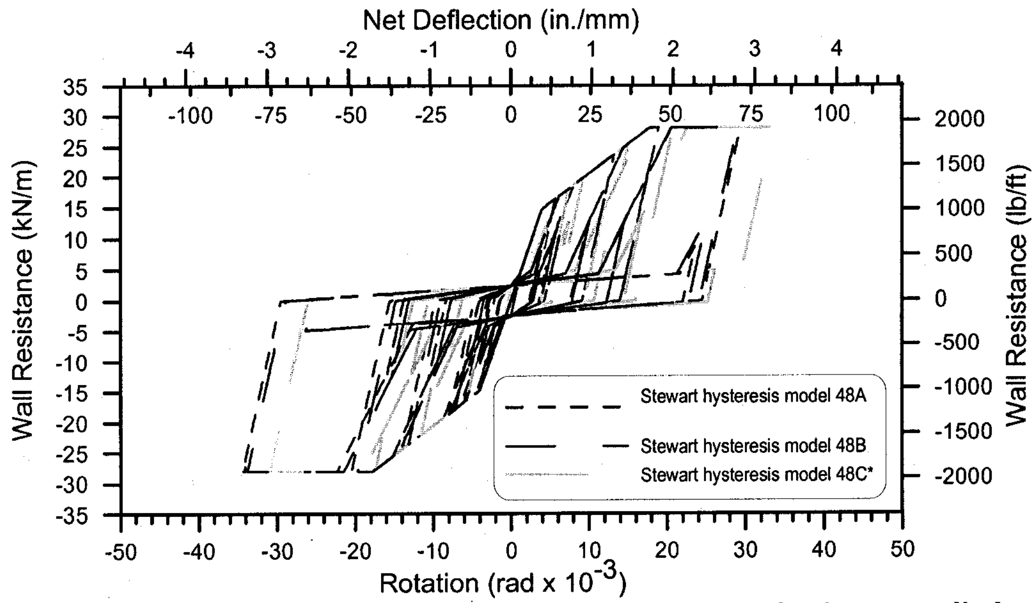


Figure 6-9 : Superposition of Stewart models 48A, 48B, 48C* for graph of resistance vs. displacement (Test 48C – Test did not include gravity loads)

Series 52
(1220 x 2440 mm OSB 75/305 mm)

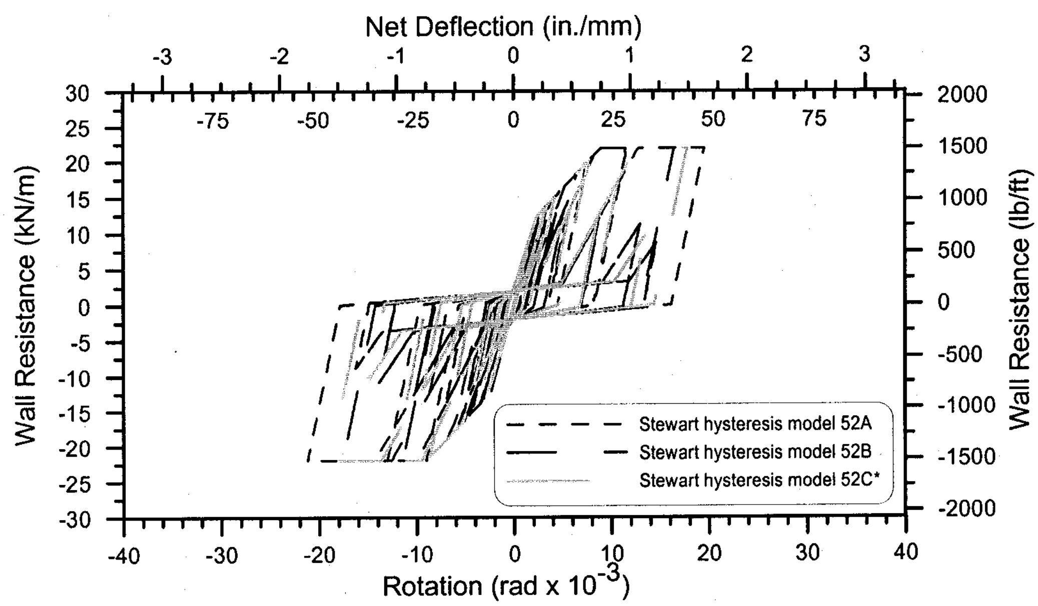


Figure 6-10 : Superposition of Stewart models 52A, 52B, 52C* for graph of resistance vs. displacement (Test 52C – Test did not include gravity loads)

6.3.2 Limitations and Conformity

Previous studies in this research program compared the cumulative energy dissipation of the test shear wall with the model to verify its accuracy. A difference of less than 10% between the cumulative energy of the wall and model up to failure indicated that the model was considered acceptable. This approach was used to balance the discrepancy in energy dissipation between the positive and negative directions because the model lacks the ability to account for strength degradation. Therefore the dissipated energy was not always the best indicator of the accuracy of the model. For a number of tests in this series the 10% limit did not make sense because it forced the model to terminate at a point where the cycles in either direction were uneven or extended the load carrying capacity beyond the test envelope. This event is illustrated in Figure 6-5 where the cumulative energy dissipation plots diverge past the 250 s point because of the disparity between the area under the test curve and the Stewart model in the last loop on the negative side (Figure 6-4). It was decided that the curve of best fit would govern the parameters with a reasonable agreement in energy based on visual inspection of the areas under the curve for monotonic tests, and within the hysteretic loops for cyclic tests.

Given that it was not possible to define the degradation in shear strength the Stewart model as used did not include a parameter to define the failure point of a shear wall. Therefore, if the recommended parameters were to be used in the definition of a hysteretic model the wall would deform indefinitely maintaining the ultimate shear load level. This behaviour is unlike that observed during testing, where a decrease in capacity takes place once the ultimate shear load is reached. Furthermore, as defined in Section 5.2, the failure point of a wall is considered to occur when the post peak deflection that corresponds with 80% of the peak shear load is measured.

To compensate for these shortcomings of the Stewart model the maximum rotation that the shear wall can undergo based on the 80% post ultimate load has been defined for each configuration (Table 6-3). This limitation is useful to determine whether building models analyzed in Ruaumoko remain within their useful performance range under dynamic loading conditions.

Table 6-3 : Maximum measured rotation of shear walls

Test Series	Maximum Rotation (10^{-3} rad)
47 & 48	19.0
49 & 50	11.5
51 & 52	10.5
53 & 54	17.7
55 & 56	16.6

It should be noted that the Stewart model does not use the recommended design values for stiffness and yield strength (Chapter 5); rather the values that most closely represent the measured test curves were relied upon to define the model parameters. Consequently, these values are not interchangeable and should only be incorporated in non-linear time history dynamic analyses. The model parameters derived for the wall configurations presented in this thesis cannot be used for other configurations. In order to analyze another wall configuration one should consult the recommendations made by Blais (2006) and Boudreault (2005). If the configuration is not included in these documents then additional testing would be required to determine appropriate hysteretic parameters.

CHAPTER 7 CONCLUSIONS AND RECOMMENDATIONS

7.1 CONCLUSIONS

During the spring of 2005 a total of 40 DCS were tested under axial loading to gain a better understanding of their behaviour and load carrying capacity as an element in a steel frame / wood panel shear wall sheathed on one side. From this study a test series of five wall configurations were developed to evaluate the combined gravity and lateral load carrying capacity of the same style shear walls. In total, 32 shear walls were tested and analyzed according to the methods proposed by Branston (2004) and Boudreault (2005). This body of work expands on the shear wall research program at McGill University and adds to the database of wall configurations begun in the summer of 2003 with Boudreault (2005), Branston (2004), Chen (2004) and augmented by Blais (2006) and Rokas (2006) the following summer.

An evaluation of the experimental load carrying capacity of the double chord stud included 18 wall configurations. The intent was to identify a means to better predict the axial compression resistance when needed for capacity based design. The results from these tests indicated that the influence of the type of structural wood sheathing and fastener patterns in the DCS and connecting the sheathing to the frame were minor. A variation in the height of the studs composing the DCS also influenced the axial compression resistance due to the load initially bearing on a single stud. The DCS of test specimens were shimmed or matched to avoid this behaviour. The base metal thickness of the chord stud was the distinct denominator between the load carrying capacity of the various configurations. The test results were used to determine appropriate effective length factors and buckling lengths that could be used to accurately reflect the capacity and behaviour of the DCS; that is $K_x = K_y = 0.9$, $K_t = 0.65$, L_x = wall height, and $L_y = L_t = 2s$. These recommendations apply to DCS with minimum 9.5 mm thick OSB, CSP or DFP structural sheathing fastened at maximum 152 mm (6") intervals.

A subsequent test series of shear walls was designed following a capacity based design approach for which the sheathing connections were selected as the fuse elements. The chord studs were chosen using the anticipated shear capacity of the wall, as defined by Branston (2006), and the recommended DCS design. The purpose of this phase of physical testing was to address concerns about combined lateral and gravity loading on the shear walls due to unfavourable behaviour of the DCS in previous lateral load testing by Serrette et al. (1996b), Morgan et al. (2002) and Branston (2004). The test results showed that the shear walls can be designed to carrying combined loading and fail in a manner in agreement with capacity based design methods.

The results from this test data were used in conjunction with the equivalent energy elastic-plastic (EEEP) analysis approach to develop design values for each wall configuration. The design values included shear stiffness, strength, ductility and a resistance factor. Force modification factors to be used in conjunction with the 2005 NBCC (NRCC, 2005) were also calculated. Recommended design values based on this study include a resistance factor of, $\phi = 0.7$ and force modification factors $R_d = 2.5$ and $R_o = 1.7$.

As illustrated in Chapter 5 the overall effects of combined gravity and lateral loading of steel frame / wood panels shear walls displayed no conclusive trend in behaviour at the design level that varied from mere lateral loading. However, it is very important that the DCS be designed appropriately to account for any anticipated gravity loads as well as the ultimate capacity of the shear wall as controlled by sheathing connection failure. Based on this finding the recommended values for lateral design determined from previous testing which did not consider gravity loads should be considered valid.

Parameters for the Stewart degrading hysteretic element (Stewart, 1987) were developed from the shear wall test data following the work of Boudreault (2005). The results add to the existing database of 22 configurations, which have been developed for the purpose of non-linear time history dynamic analysis. To date, limited studies by Blais (2006), have shown that the use of steel frame / wood panel shear walls as an SFRS is adequate using Ruaumoko (Carr, 2000).

7.2 Recommendations

In the 2004 supplement to the CSA S136 Standard the Direct Strength Method (DSM) has been included for simply supported pre-qualified members. Further development of this method to include a calibration curve and modified end conditions DCS is recommended. That would possibly allow for more accurate predictions of load carrying capacities. Ultimately, the construction of a finite element model and an analysis of the steel frame / wood panel shear wall would assist in the understanding of the system's behaviour, as the finite strip method used by the DSM limits the inclusion of the perforations and intermediate fasteners.

The cyclic loading regime used by this research program, CUREE Ordinary Ground Motion Protocol (Krawinkler et al, 2000), was developed for a design earthquake from California (US) with a 10% probability of exceedance in 50 years. However, the most recent, 2005, version of National Building Code of Canada has moved to ground motions with a 2% probability of exceedance in 50 years (Heidebrecht, 2003). A revised version of this protocol is required in order to progress this research program into agreement with the current expectations.

The incorporation of strength degradation parameters in the Stewart hysteretic model would improve the agreement with the experimental data and reduce the difficulties in matching the cumulative energy dissipation. As well, the inclusion of a parameter signifying failure by limiting maximum rotation would make interpretations of dynamic analysis results more simple and direct.

Further steps into the non-linear time history dynamic analysis of multi-storey structures using the Stewart hysteretic models developed in the study are recommended. The impact of the variations between the same configurations of wall with different parameters would assist in the understanding of the possible range in performance of steel frame / wood panel shear walls. Ideally, as mentioned by Boudreault (2005), uniformity in research and analysis of shear walls would improve the opportunity for a probabilistic approach in design.

To verify the recommended modification factors R_d and R_o the physical testing of steel frame / wood panel systems under dynamic conditions should be carried out. A study of the nature would help establish the natural period of the system and verify the accuracy of the hysteretic model developed thus far. Testing of multi-storey configurations could confirm the influences of combined gravity loading and the adequacy of the capacity based design methods currently practiced.

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APPENDIX A

SUMMARY OF CHORD COMPRESSION TESTS

Table A-1 : Summary of Chord Compression Tests

Test	Stud Thickness	Sheathing Type	Sheathing Thickness (mm)	Screw Spacing in DCS[1]	Screw Spacing in Sheathing	Maximum Load (kN)
1. 043OSB1-12-3A	1.09 mm	OSB	12.5 mm	305 mm	75 mm	94.27
2. 043OSB1-12-3B	1.09 mm	OSB	12.5 mm	305 mm	75 mm	83.00
3. 043OSB2-12-3A	1.09 mm	OSB	9.5 mm	305 mm	75 mm	91.50
4. 043OSB2-12-3B	1.09 mm	OSB	9.5 mm	305 mm	75 mm	83.34
5. 043OSB1-24-3A	1.09 mm	OSB	12.5 mm	610 mm	75 mm	81.99
6. 043OSB1-24-3B	1.09 mm	OSB	12.5 mm	610 mm	75 mm	84.23
7. 043OSB2-24-3A	1.09 mm	OSB	9.5 mm	610 mm	75 mm	78.36
8. 043OSB2-24-3B	1.09 mm	OSB	9.5 mm	610 mm	75 mm	83.96
9. 043OSB1-12-6A	1.09 mm	OSB	12.5 mm	305 mm	152 mm	80.82
10. 043OSB1-12-6B	1.09 mm	OSB	12.5 mm	305 mm	152 mm	73.99
11. 043OSB2-12-6A	1.09 mm	OSB	9.5 mm	305 mm	152 mm	78.08
12. 043OSB2-12-6B	1.09 mm	OSB	9.5 mm	305 mm	152 mm	77.22
13. 043OSB1-24-6A	1.09 mm	OSB	12.5 mm	305 mm	152 mm	85.80
14. 043OSB1-24-6B	1.09 mm	OSB	12.5 mm	305 mm	152 mm	70.42
15. 043OSB2-24-6A	1.09 mm	OSB	9.5 mm	305 mm	152 mm	85.59
16. 043OSB2-24-6B	1.09 mm	OSB	9.5 mm	305 mm	152 mm	85.70
17. 033OSB1-12-3A	0.84 mm	OSB	12.5 mm	305 mm	75 mm	60.29
18. 033OSB1-12-3B	0.84 mm	OSB	12.5 mm	305 mm	75 mm	62.40
19. 033OSB1-12-6A	0.84 mm	OSB	12.5 mm	305 mm	152 mm	62.62
20. 033OSB1-12-6B	0.84 mm	OSB	12.5 mm	305 mm	152 mm	62.33
21. 043CSP1-12-3A	1.09 mm	CSP	12.5 mm	305 mm	75 mm	82.61
22. 043CSP1-12-3B	1.09 mm	CSP	12.5 mm	305 mm	75 mm	89.79
23. 043CSP2-12-3A	1.09 mm	CSP	9.5 mm	305 mm	75 mm	84.90
24. 043CSP2-12-3B	1.09 mm	CSP	9.5 mm	305 mm	75 mm	80.31
25. 043CSP1-12-6A	1.09 mm	CSP	12.5 mm	305 mm	152 mm	83.87
26. 043CSP1-12-6B	1.09 mm	CSP	12.5 mm	305 mm	152 mm	80.41
27. 043CSP2-12-6A	1.09 mm	CSP	9.5 mm	305 mm	152 mm	78.67
28. 043CSP2-12-6B	1.09 mm	CSP	9.5 mm	305 mm	152 mm	90.96
29. 056OSB1-12-3A	1.37 mm	OSB	12.5 mm	305 mm	75 mm	124.96
30. 056OSB1-12-3B	1.37 mm	OSB	12.5 mm	305 mm	75 mm	124.95
31. 056CSP1-12-3A	1.37 mm	CSP	12.5 mm	305 mm	75 mm	119.02
32. 056CSP1-12-3B	1.37 mm	CSP	12.5 mm	305 mm	75 mm	114.34
33. 068OSB1-12-3A	1.72 mm	OSB	12.5 mm	305 mm	75 mm	172.98
34. 068OSB1-12-3B	1.72 mm	OSB	12.5 mm	305 mm	75 mm	179.20
35. 068CSP1-12-3A	1.72 mm	CSP	12.5 mm	305 mm	75 mm	172.34
36. 068CSP1-12-3B	1.72 mm	CSP	12.5 mm	305 mm	75 mm	182.97
37. 033DoubleChordStud	0.84 mm	N/A	N/A	305 mm	N/A	56.25
38. 043DoubleChordStud	1.09 mm	N/A	N/A	305 mm	N/A	78.79
39. 054DoubleChordStud	1.37 mm	N/A	N/A	305 mm	N/A	109.75
40. 068DoubleChordStud	1.72 mm	N/A	N/A	305 mm	N/A	145.95

[1] Double Chord Stud (DCS)

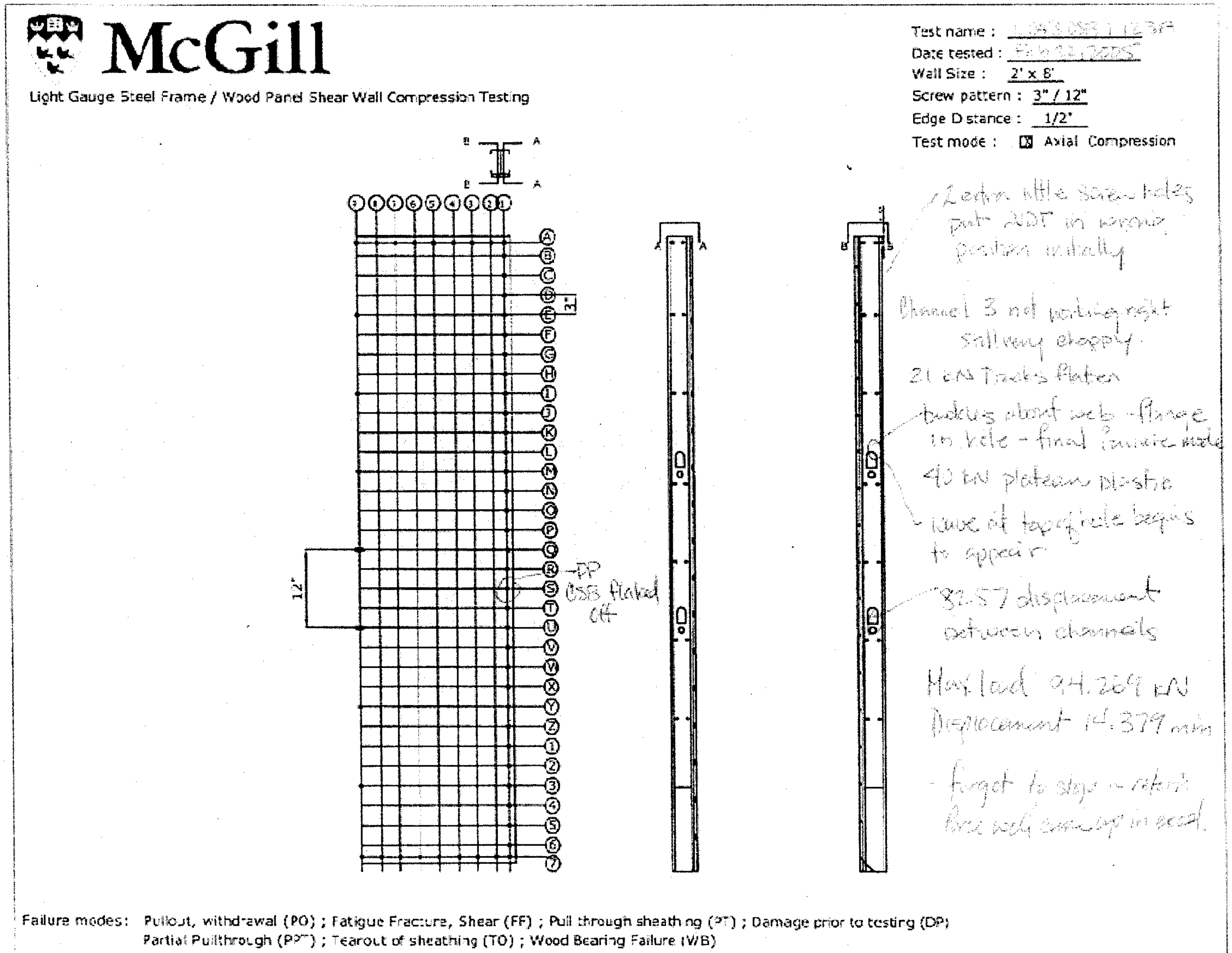


Figure A-1: Test 1 damage record

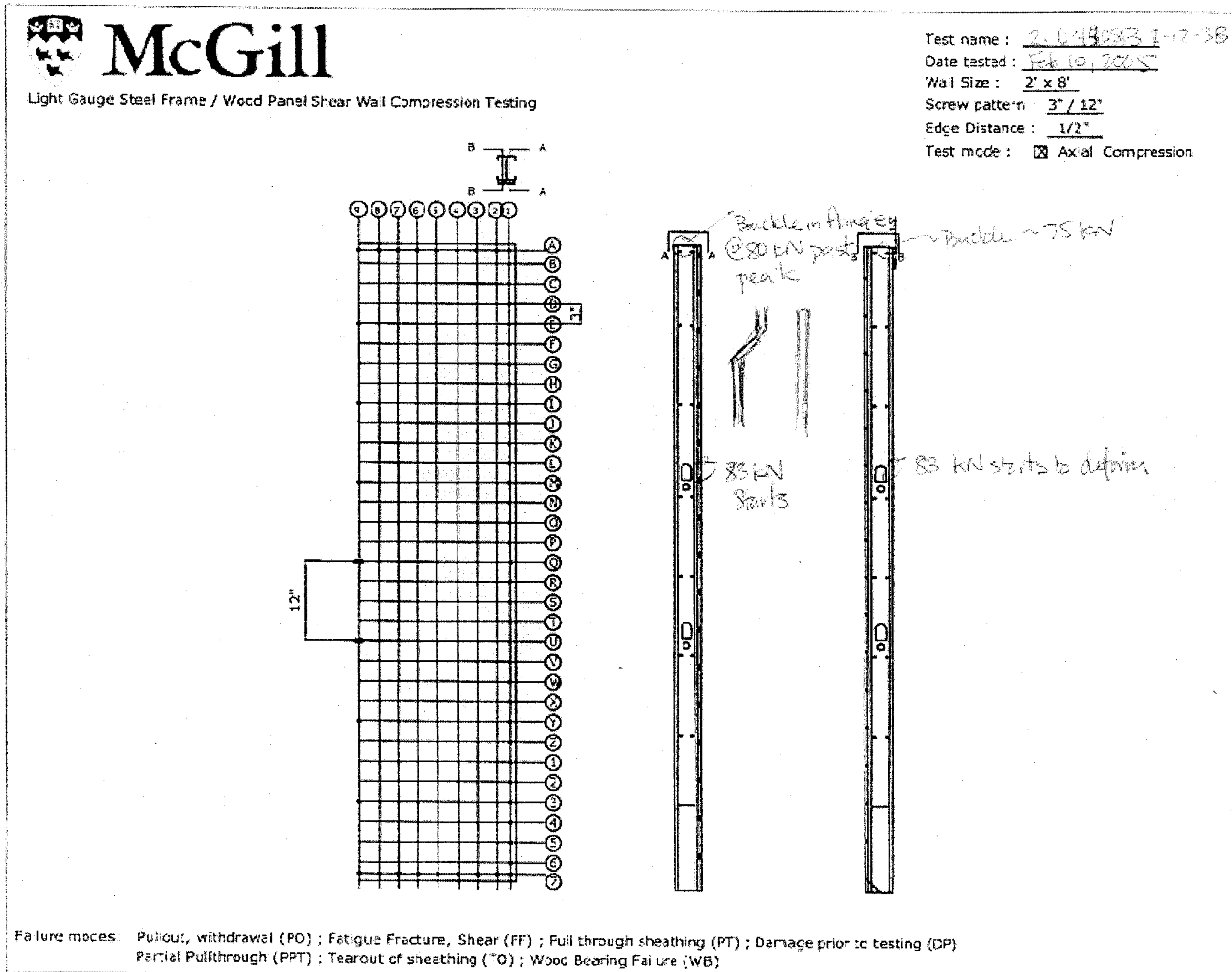


Figure A-2: Test 2 damage record

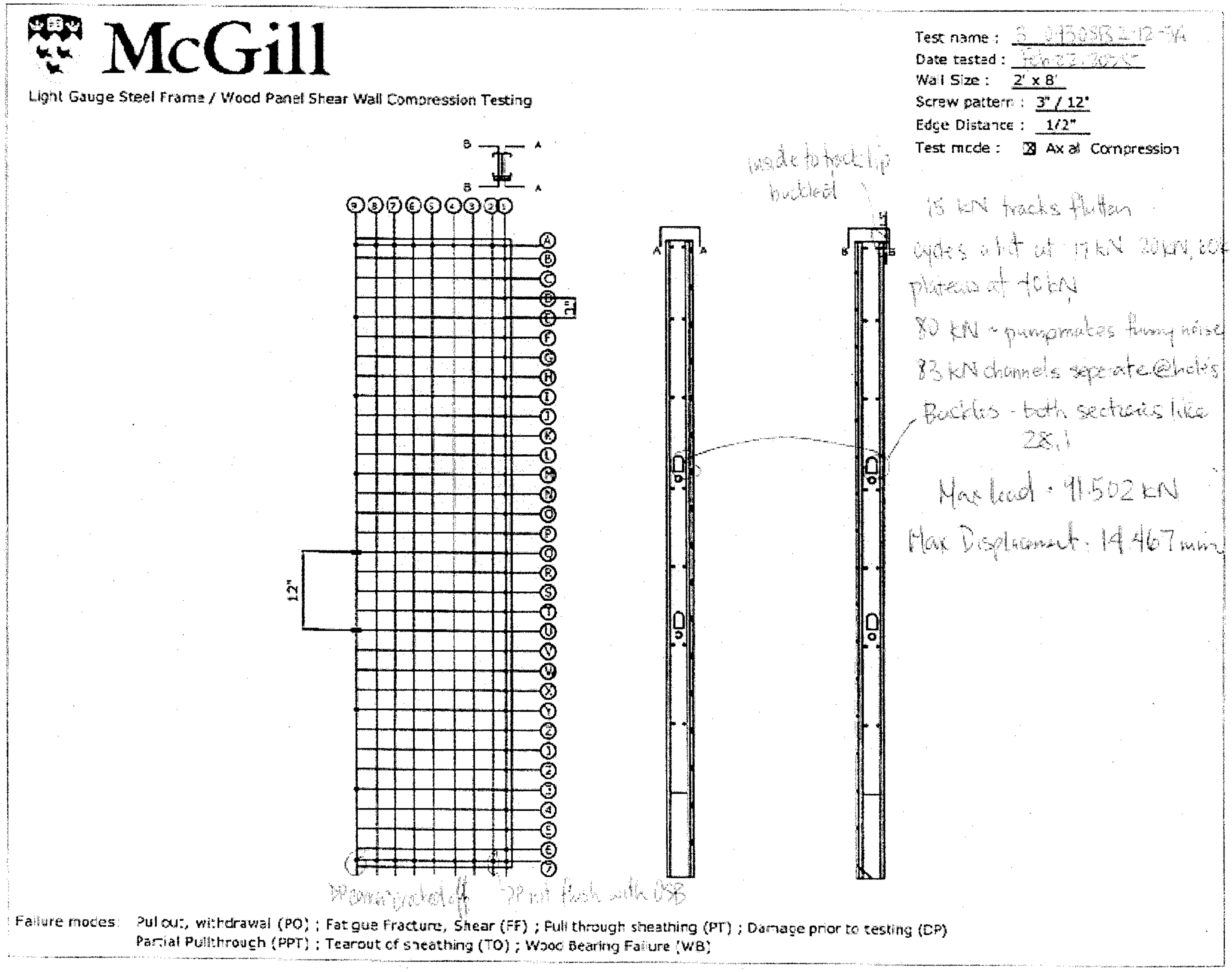


Figure A-3: Test 3 damage record

McGill

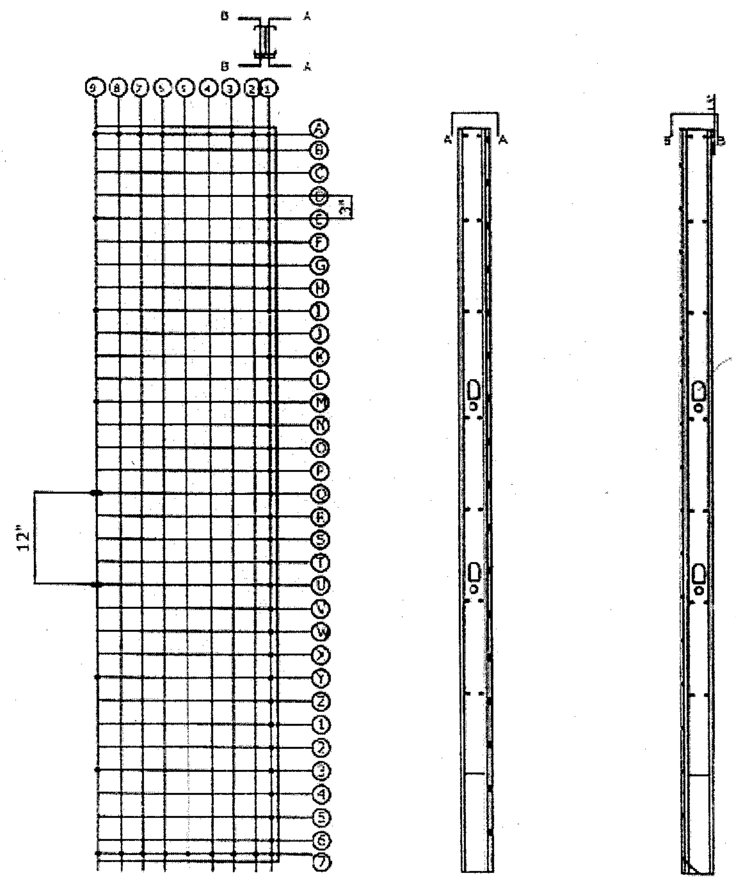
Light Gauge Steel Frame / Wood Panel Shear Wall Compression Testing

Test name : Test 4
Date tested : March 31, 2005
Wall Size : 2' x 8'
Screw pattern : 3" / 12"
Edge Distance : 1/2"
Test mode : ☒ Axial Compression

Channel 9 is choppy at beginning
→ they run ~ parallel by 40kN

Test rate = 0.3mm/min

40 kN Plateau



moves apart at these holes ~60kN

Channel 9 has moved 3mm @ 66 kN

Channel 3 and 4 are almost exactly the same @ ~ load of 67 kN

Channels 4 3
-1.26 mm -1.36 mm

Max. Load 83.342 kN
Max. Displ. 7.8265 kN

Failure modes Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Figure A-4: Test 4 damage record

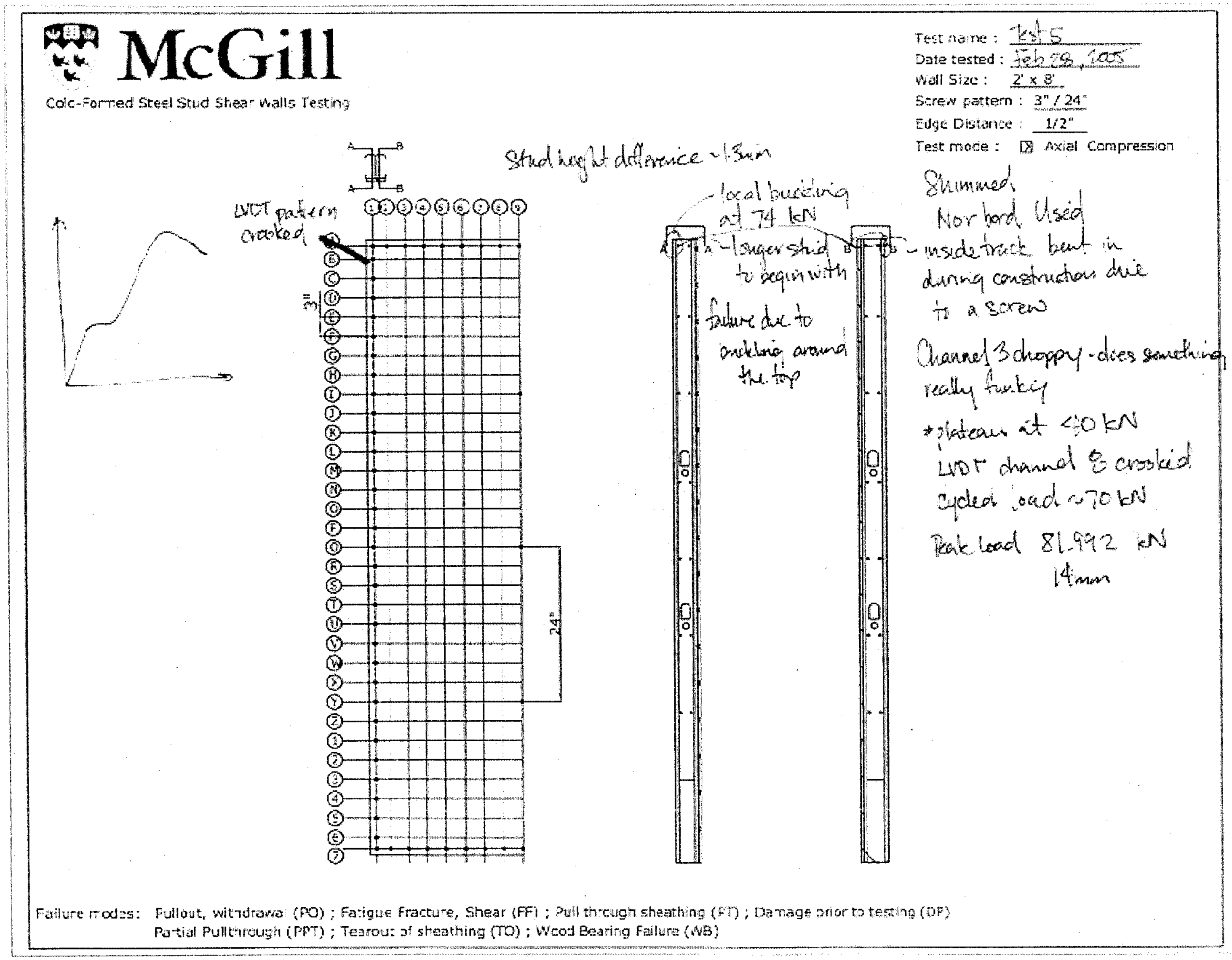


Figure A-5: Test 5 damage record

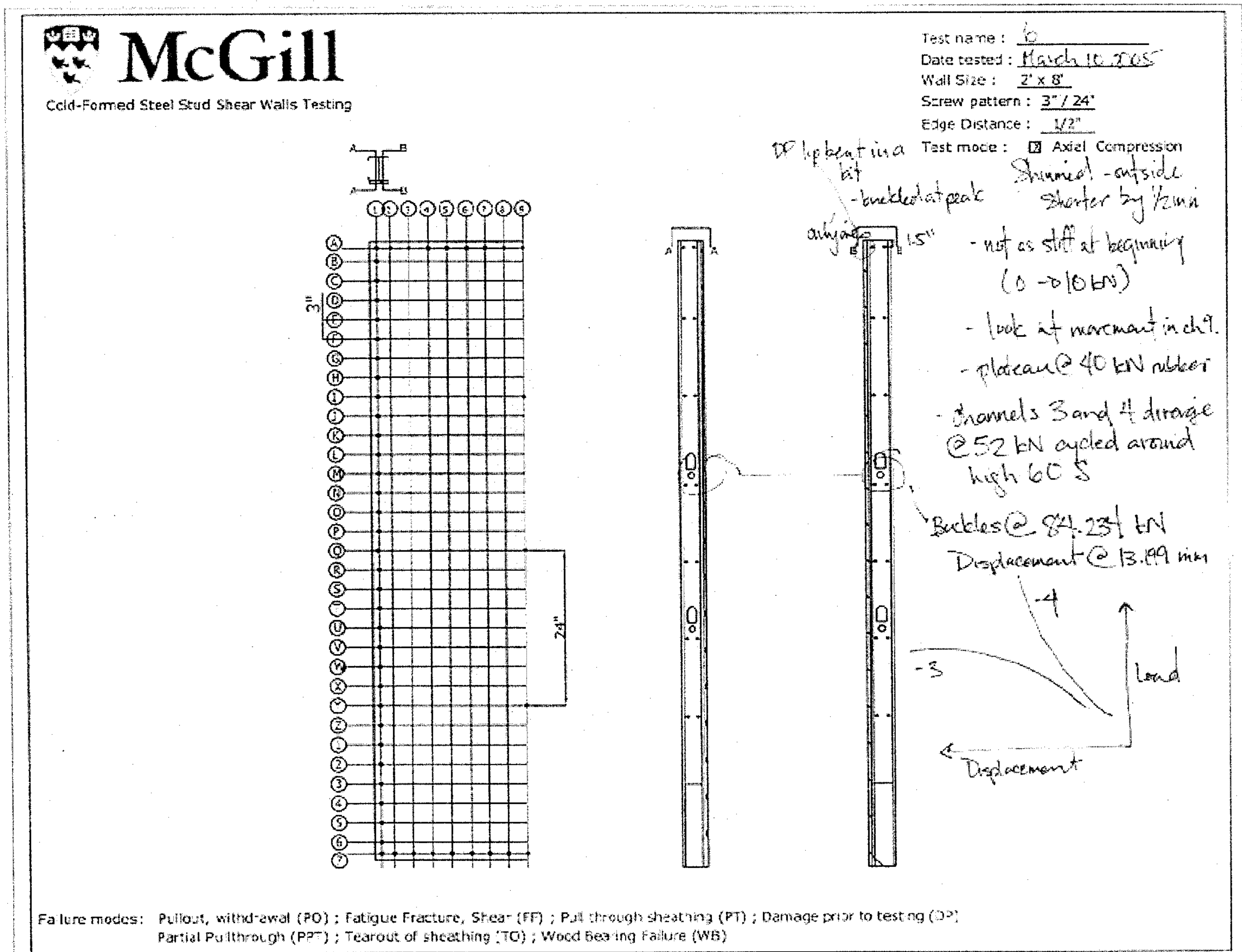


Figure A-6: Test 6 damage record

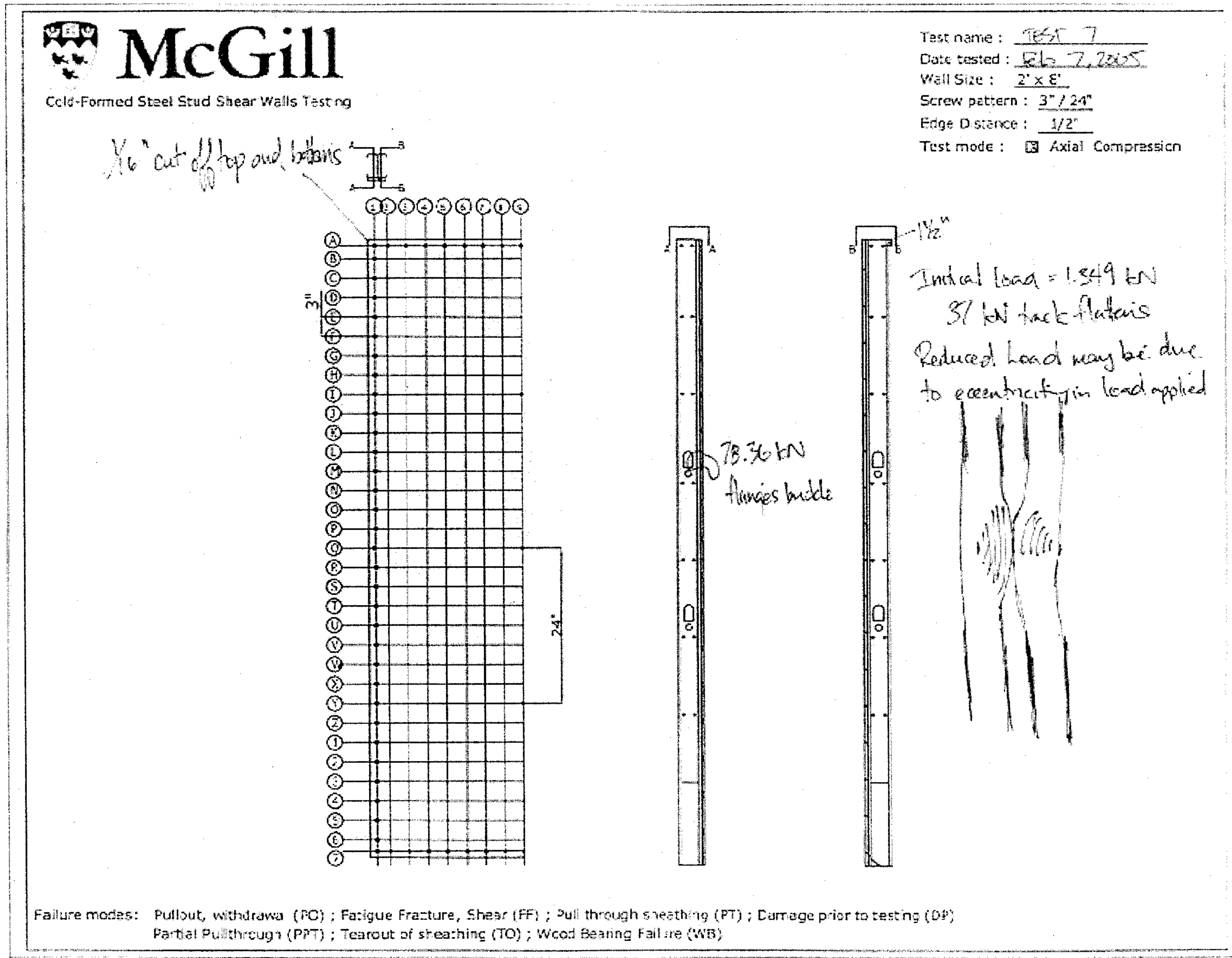
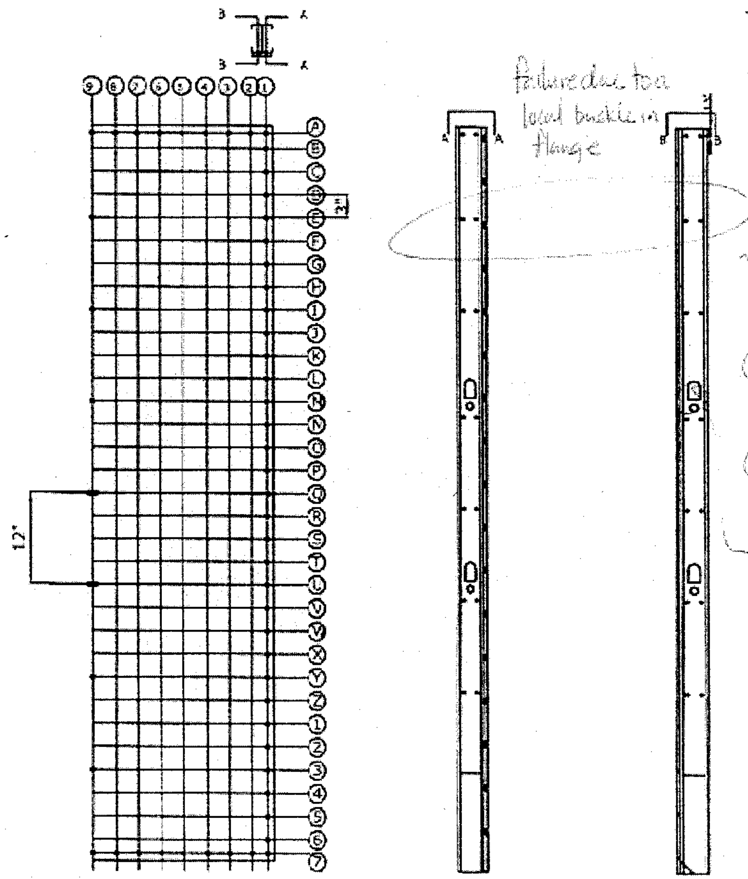


Figure A-7: Test 7 damage record

McGill

Light Gauge Steel Frame / Wood Panel Shear Wall Compression Testing

Test name : Test 8
Date tested : April 4, 2005
Wall Size : 2' x 8'
Screw pattern : 3" / 12"
Edge Distance : 1/2"
Test mode : ☒ Axial Compression



90 kN Plateau

Cycles at 50 kN

Channel 9 2.39mm in forward @ 64.4 kN

~400 lbs of garbage at beginning of test

Channel 9 3.05mm @ 74.5kN
3.83 mm @ 80.0 kN

Cycles at 78 kN

Holes displace @ 83 kN
Buckles @ 84.099 kN
11.686 mm

Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Figure A-8: Test 8 damage record

McGill

Light Gauge Steel Frame / Wood Panel Shear Wall Compression Testing

Test name : 9-C430SB1-12-6A
Date tested : March 7 2006
Wall Size : 2' x 8'
Screw pattern : 6" / 12'
Edge Distance : 1/2"
Test mode : ☒ Axial Compression

SHIMMED

* extra screws placed 1/2 way between top and initial first set ~0.25"

• system screwed up start with initial load of 3.5kN ~300 scans into recording

- plateau @ 40 kN

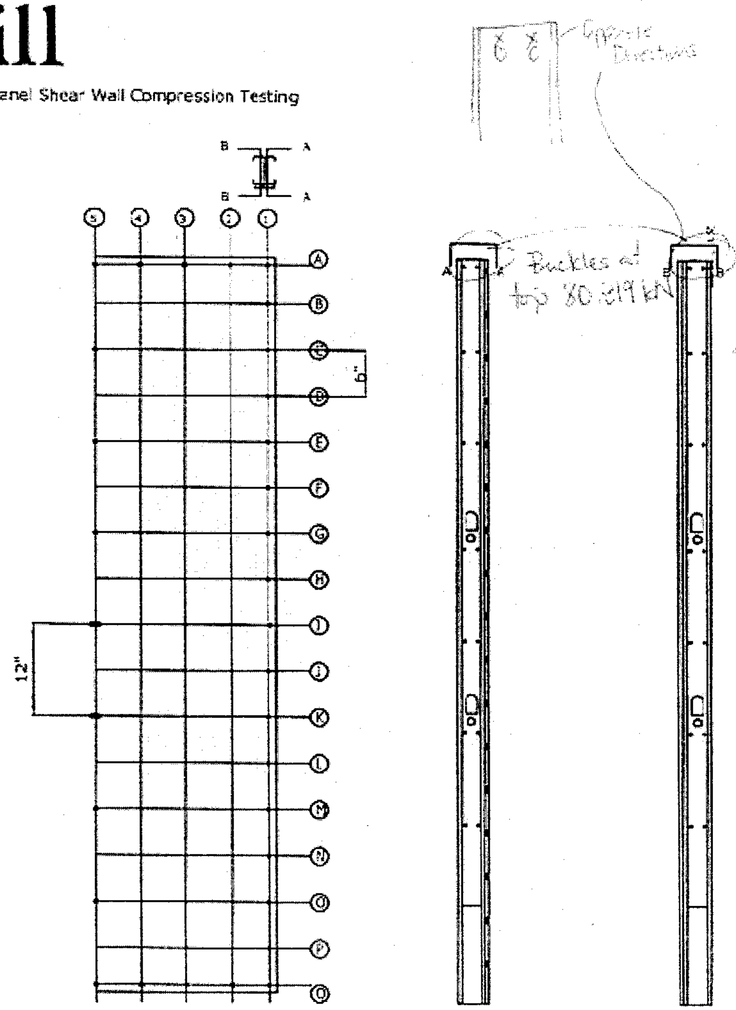
@ 40 kN the top has moved >1mm <2mm

Channels 3 and 4 diverge after about a 1mm displacement in Y!

~ doing something really funky @ 80 kN

Max load 80.819 kN

Displ. 12mm



Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Figure A-9: Test 9 damage record

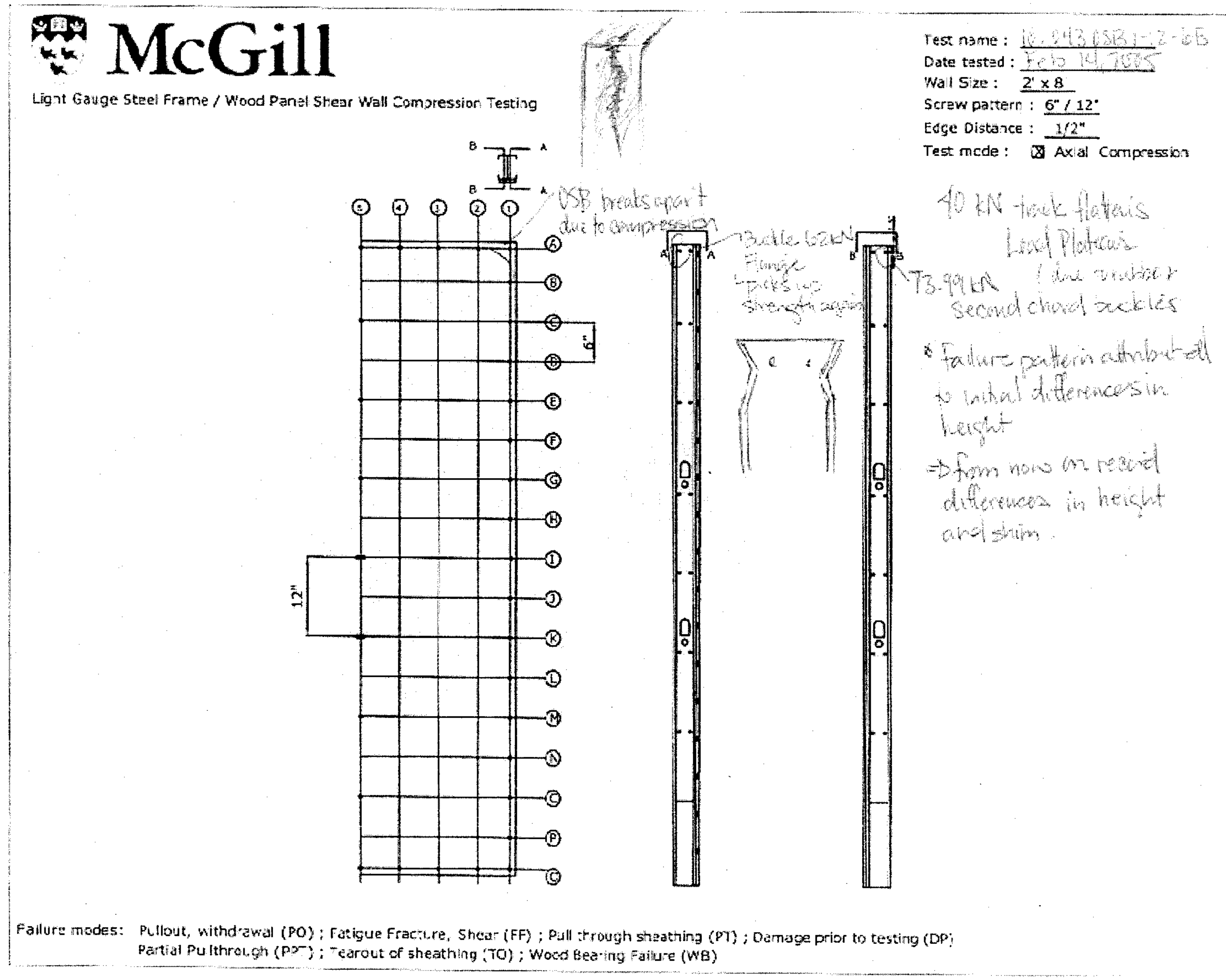


Figure A-10: Test 10 damage record

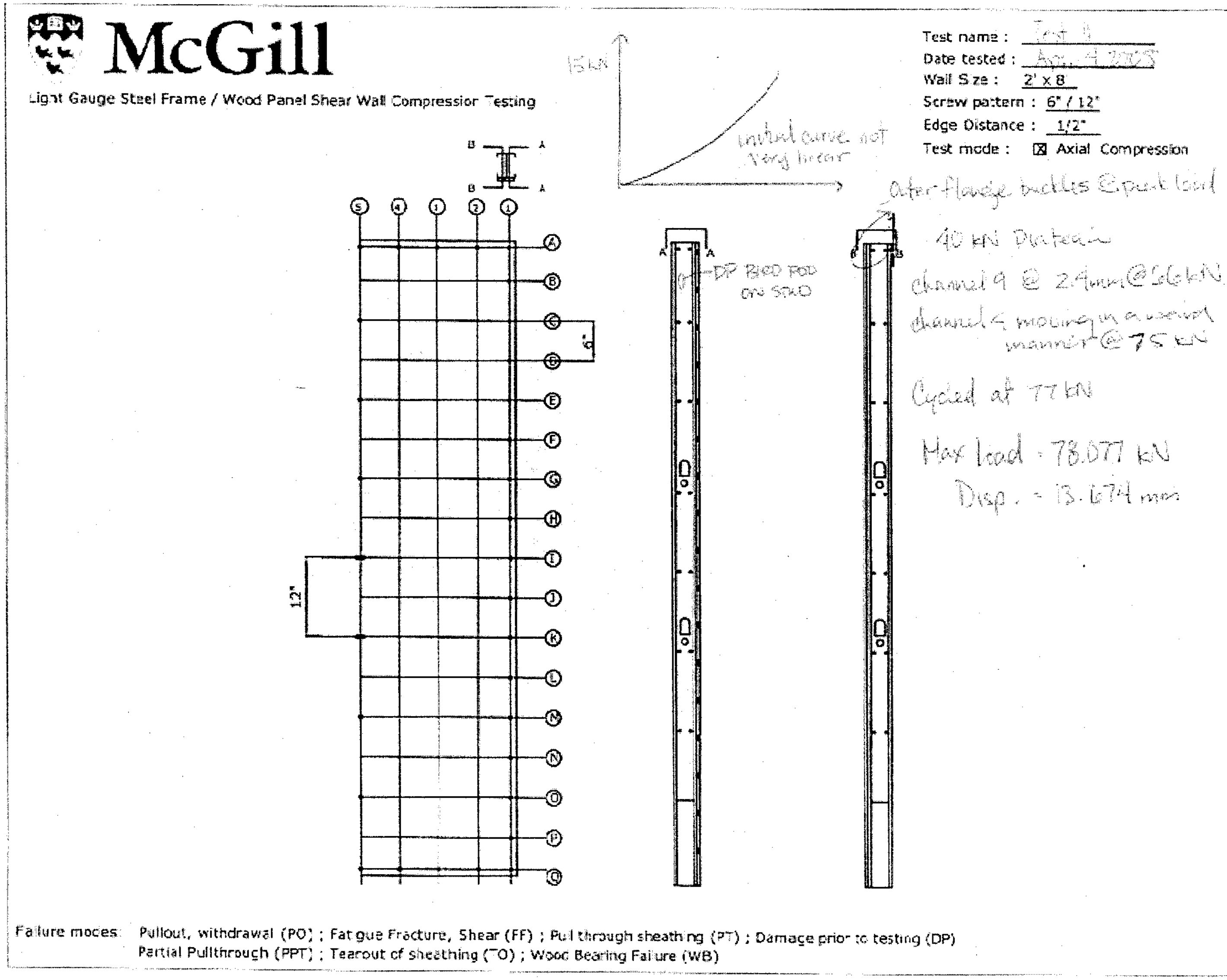
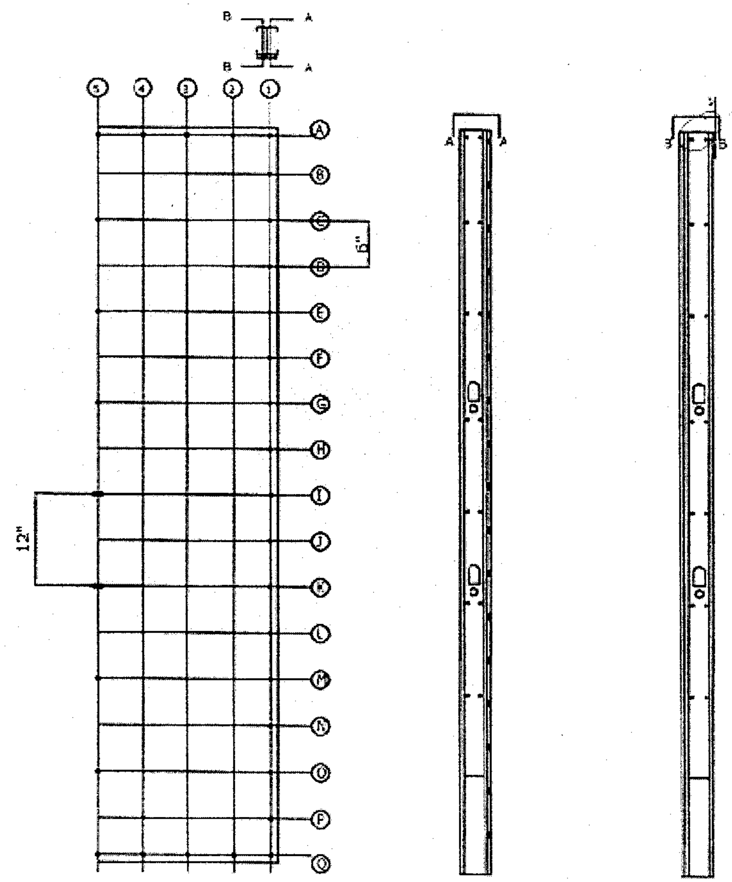


Figure A-11: Test 11 damage record

McGill

Light Gauge Steel Frame / Wood Panel Shear Wall Compression Testing

Test name : Test 12
Date tested :
Wall Size : 2' x 8'
Screw pattern : 6" / 12"
Edge Distance : 1/2"
Test mode : ☒ Axial Compression



— Buckles at 77.216 kN

Channel 9 1.33 mm @56 kN

40 kN Plateau

Cycles at 57 kN

Channel 4 - is doing something weird ~74 kN

Max load = 77.216 kN

Disp = 11.439 mm

Failure modes: Pul out, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Figure A-12: Test 12 damage record

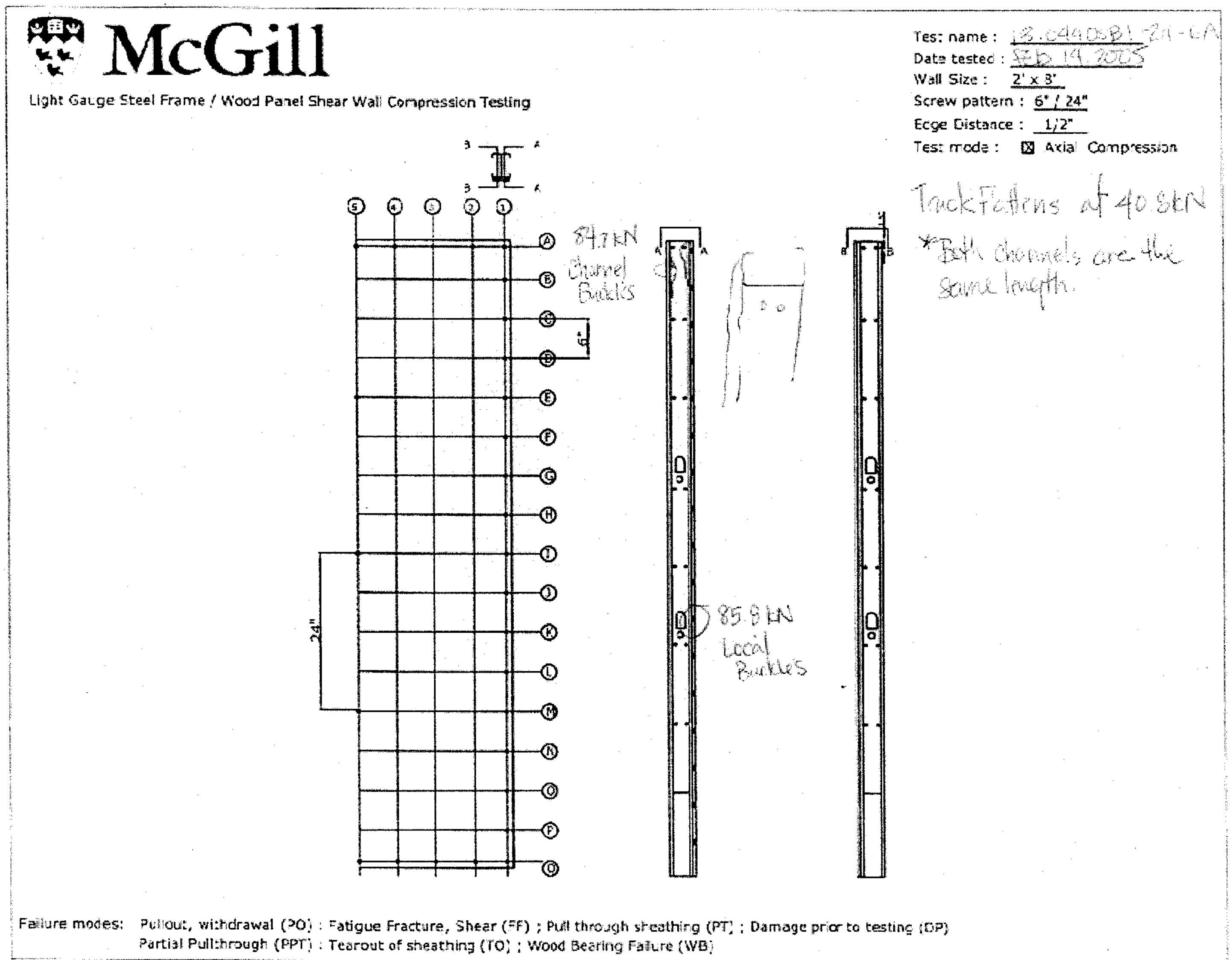


Figure A-13: Test 13 damage record

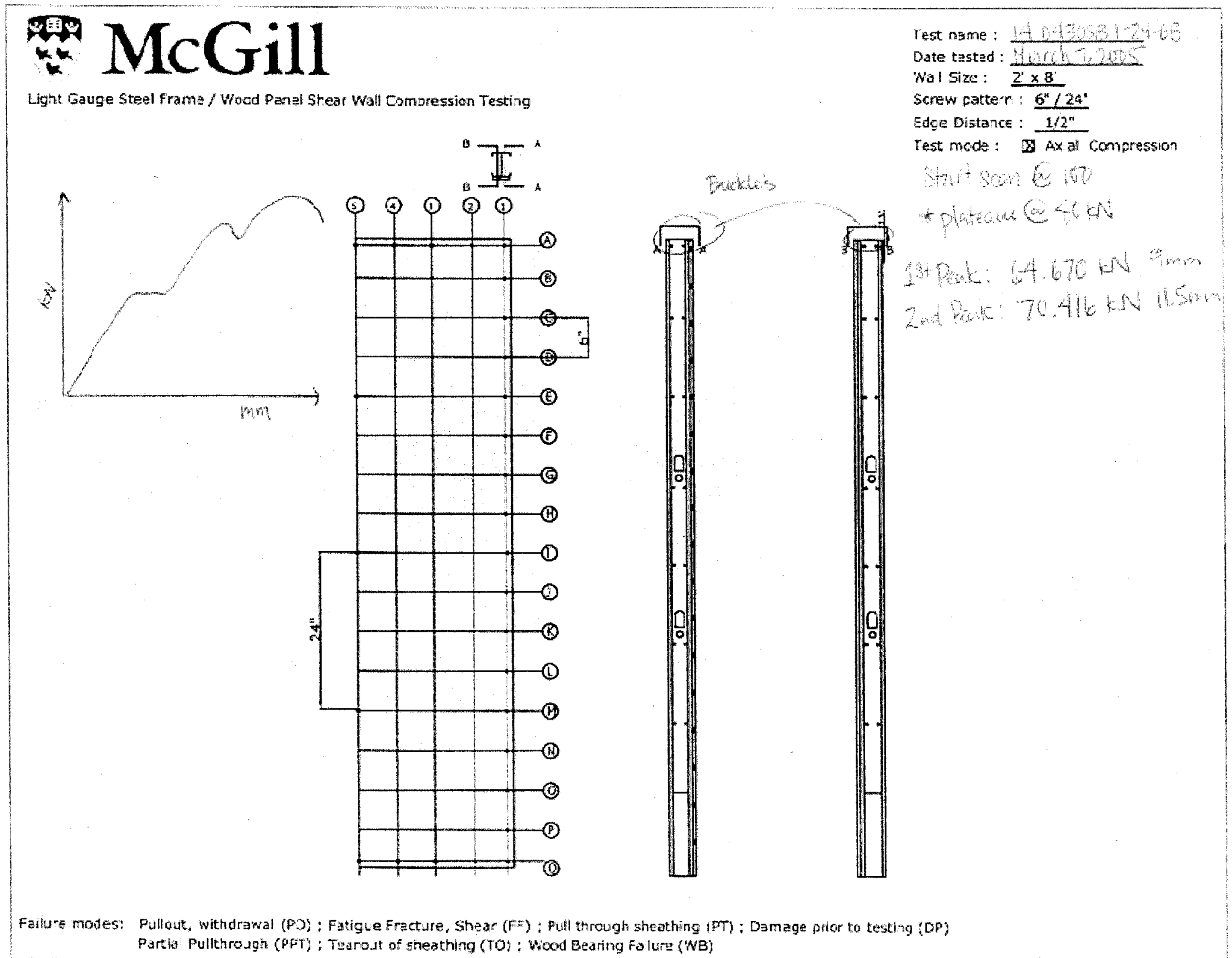


Figure A-14: Test 14 damage record

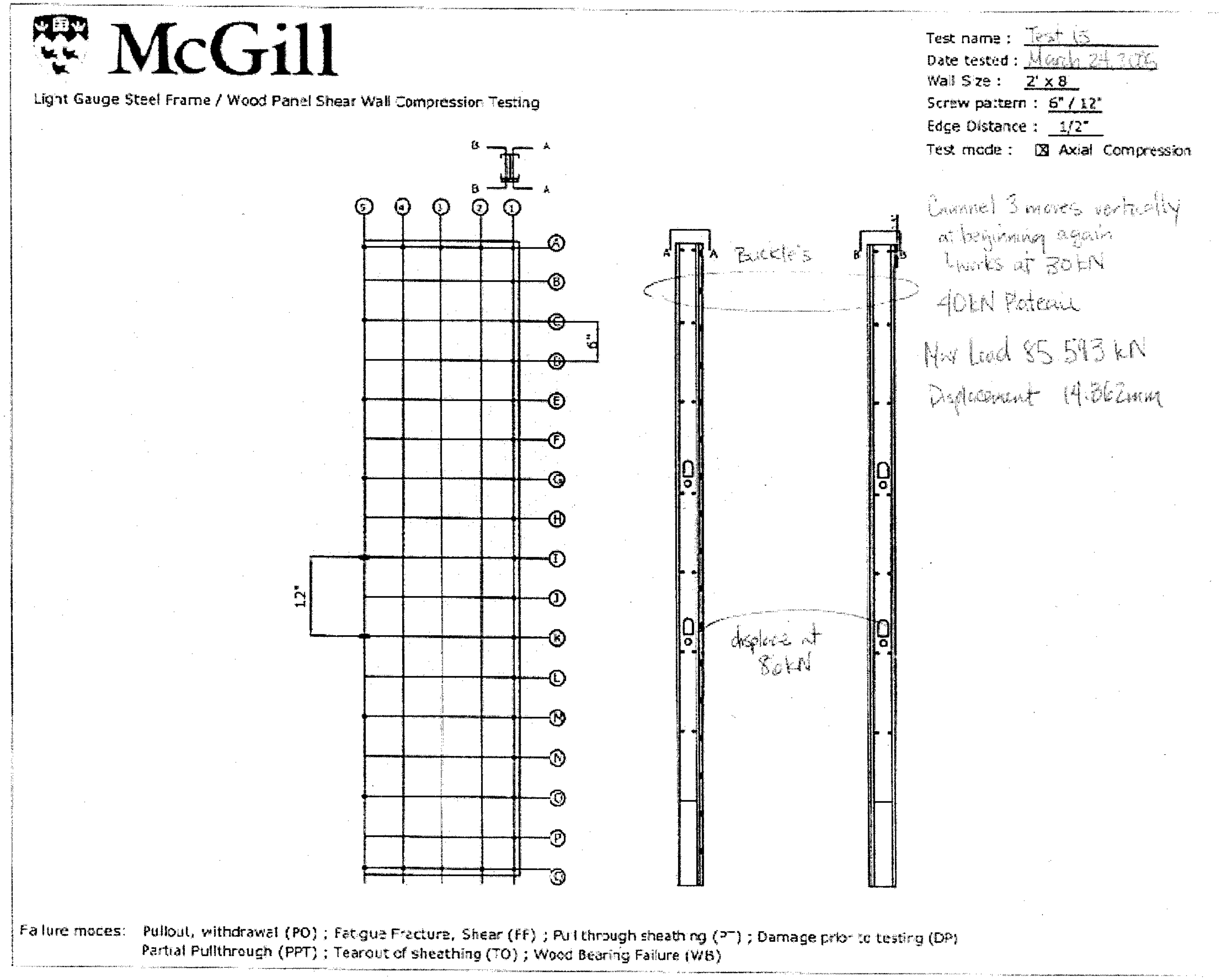
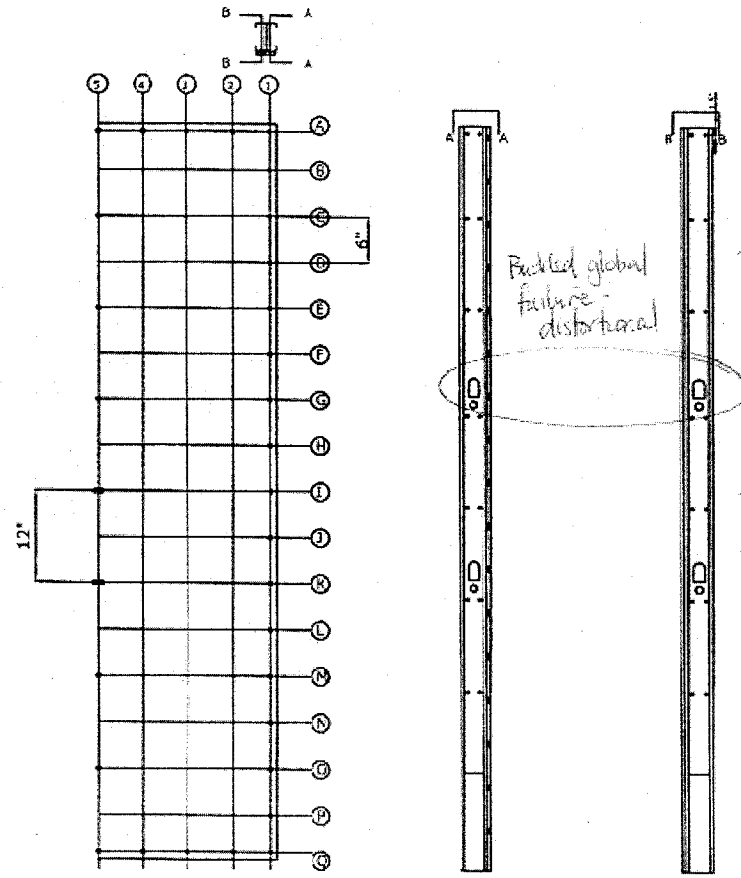


Figure A-15: Test 15 damage record

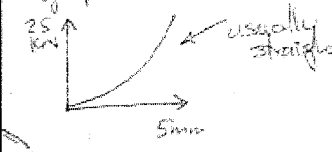
McGill

Light Gauge Steel Frame / Wood Panel Shear Wall Compression Testing

Test name : 16
Date tested : March 24, 2005
Wall Size : 2' x 8'
Screw pattern : 6" / 12"
Edge Distance : 1/2"
Test mode : ☒ Axial Compression



GRANT PLYWOOD
· makes a lot of noise at the very beginning
Channel 3 goes vertical on graph



Channel 4 wire is just touching LVDT channel 10
40 kN plateau
Channels 3 and 4 in parallel at 50 kN
Cycles ~60 kN

Max load 85.704 kN
Deflection 13.612 mm

Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Figure A-16: Test 16 damage record

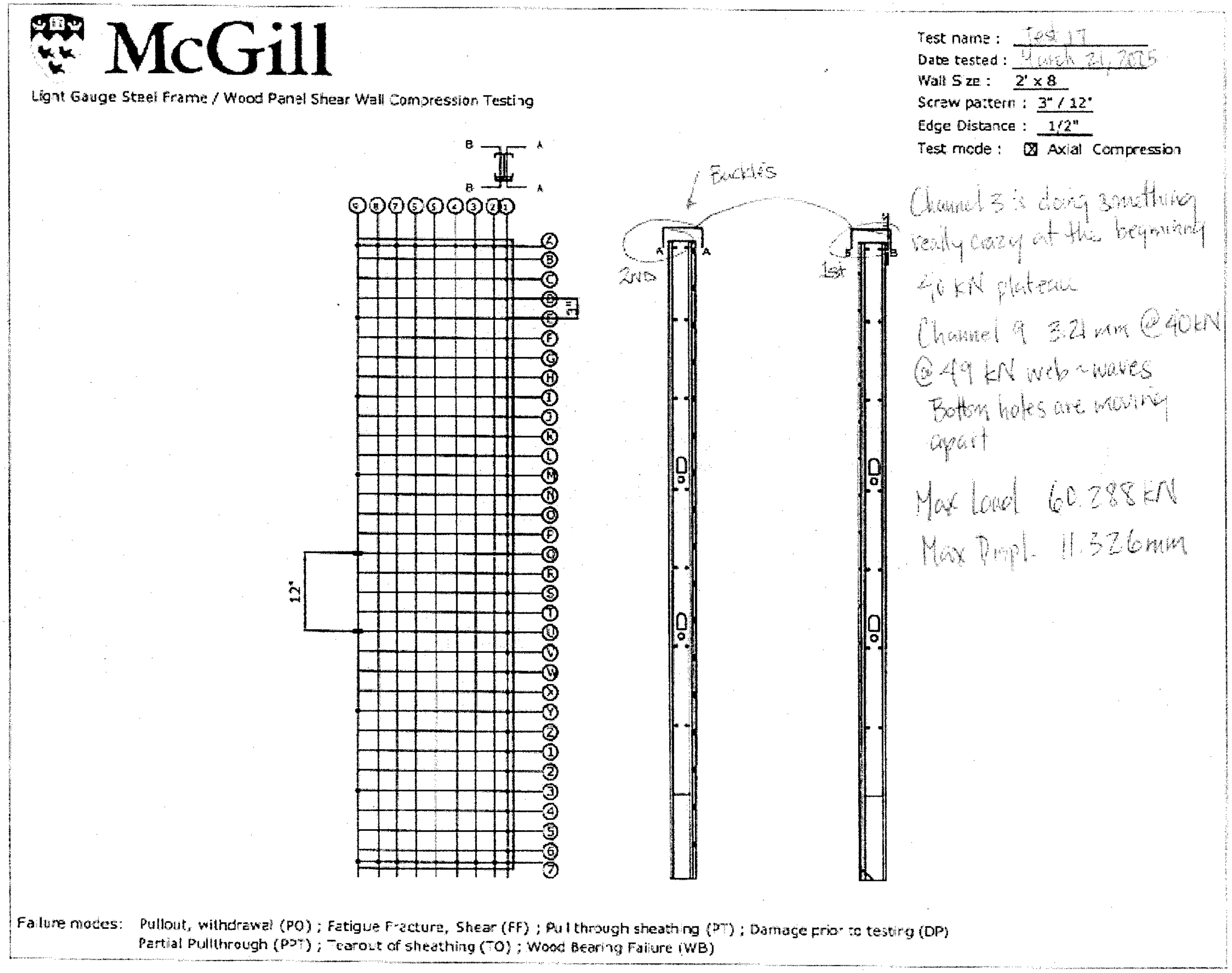


Figure A-17: Test 17 damage record

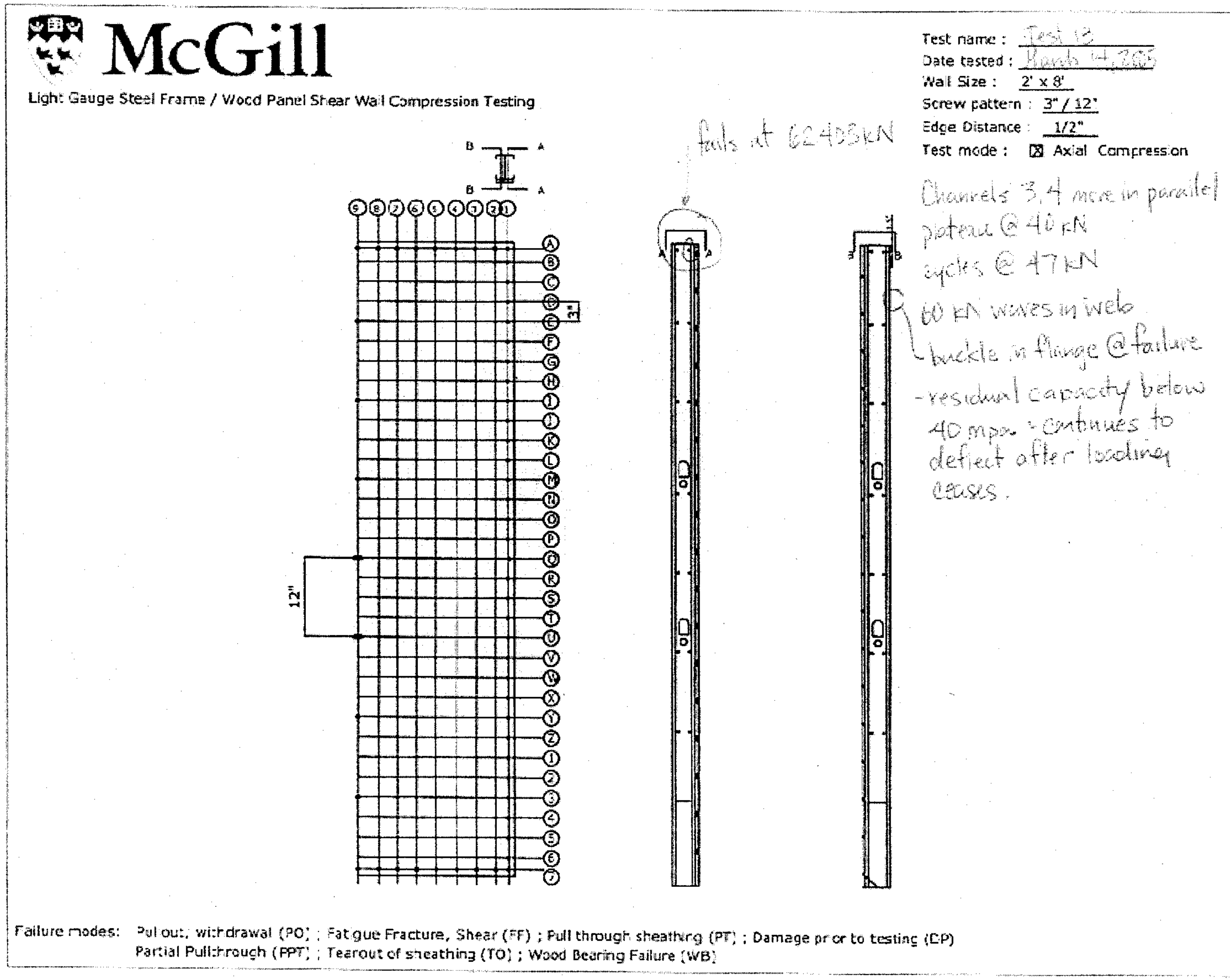


Figure A-18: Test 18 damage record

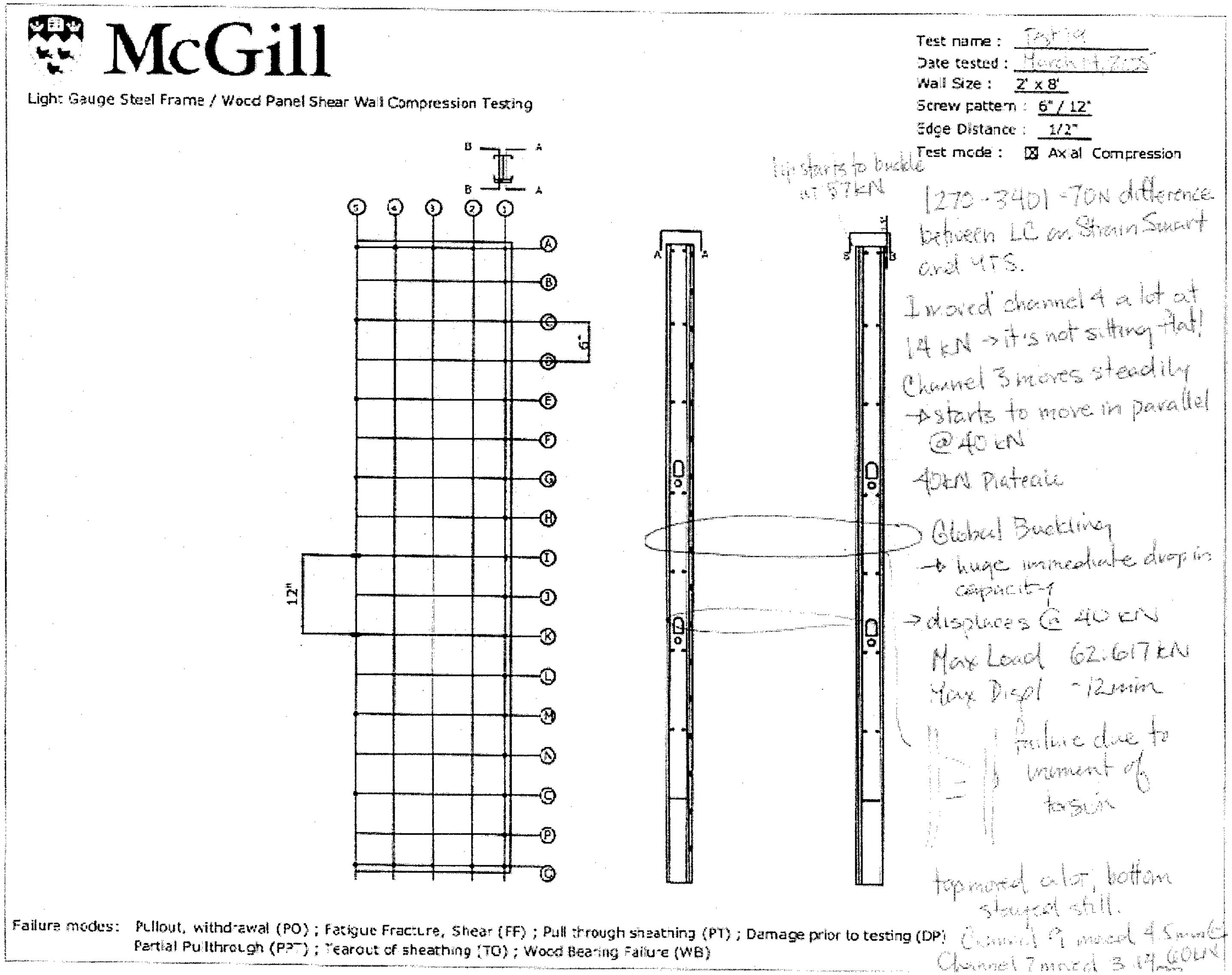
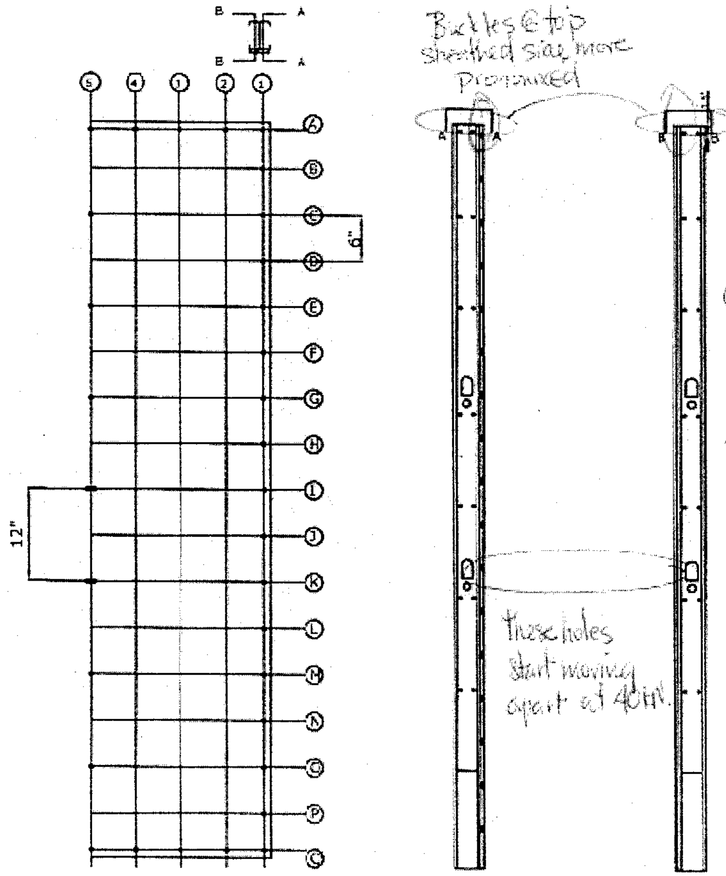


Figure A-19: Test 19 damage record

McGill

Light Gauge Steel Frame / Wood Panel Shear Wall Compression Testing

Test name : Test 20
Date tested : March 14, 2005
Wall Size : 2' x 8'
Screw pattern : 6" / 12"
Edge Distance : 1/2"
Test mode : ☒ Axial Compression



* cut off first 50 scan rec start problems - forgot to zero!
Matched Top's during construction
Larger bottom opposite hold-down
Channel 3, 4 greater Buck - almost a mm apart - running parallel at higher loads
40 kN plateau (shorter than others I think...)
12.064mm @ 43 kN (larger displacement than 0.45mm studs)
Cycles at 50 kN
Max load 62.333 kN
Displacement 15mm

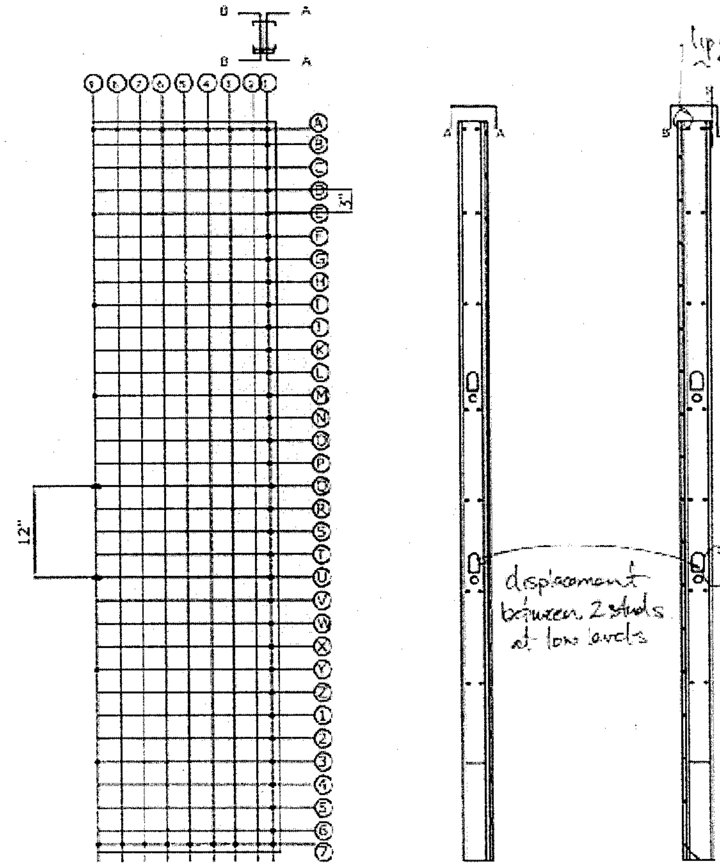
Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP) Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Figure A-20: Test 20 damage record

McGill

Light Gauge Steel Frame / Wood Panel Shear Wall Compression Testing

Test name : Test 21
Date tested : March 10, 2005
Wall Size : 2' x 8'
Screw pattern : 3" / 12"
Edge Distance : 1/2"
Test mode : ☒ Axial Compression



lip curls ~80 kN

Shimmed - very close inside 0.2 mm shorter
- plateau @ 45 kN
- cycles at 9 min
· Max load ·82.606 kN ~12.2 min

displacement between 2 studs at low levels

- buckles out @82 kN

Failure modes: Pullout, withdrawal (FC) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Figure A-21: Test 21 damage record

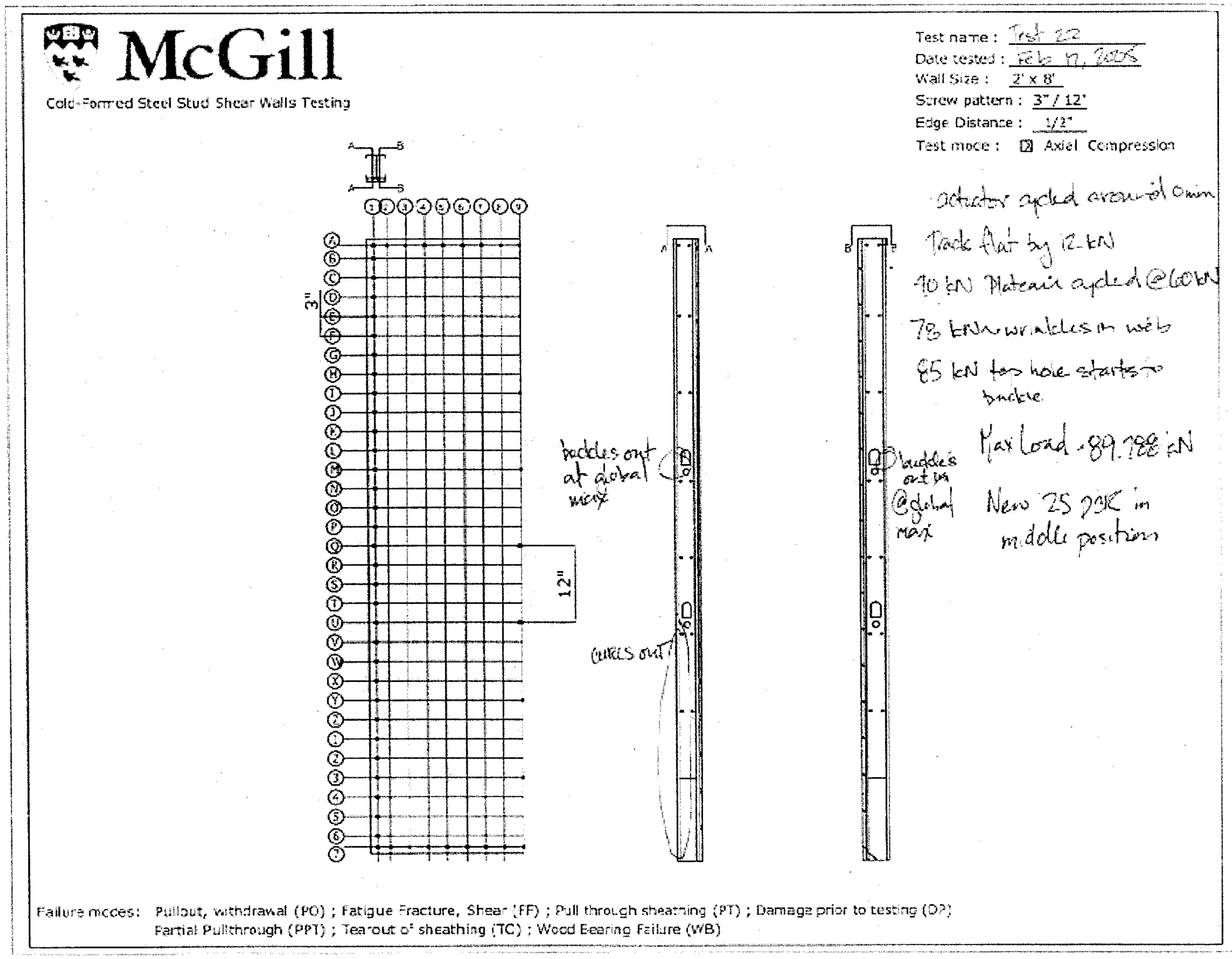


Figure A-22: Test 22 damage record

Loading Rate 0.83 mm/

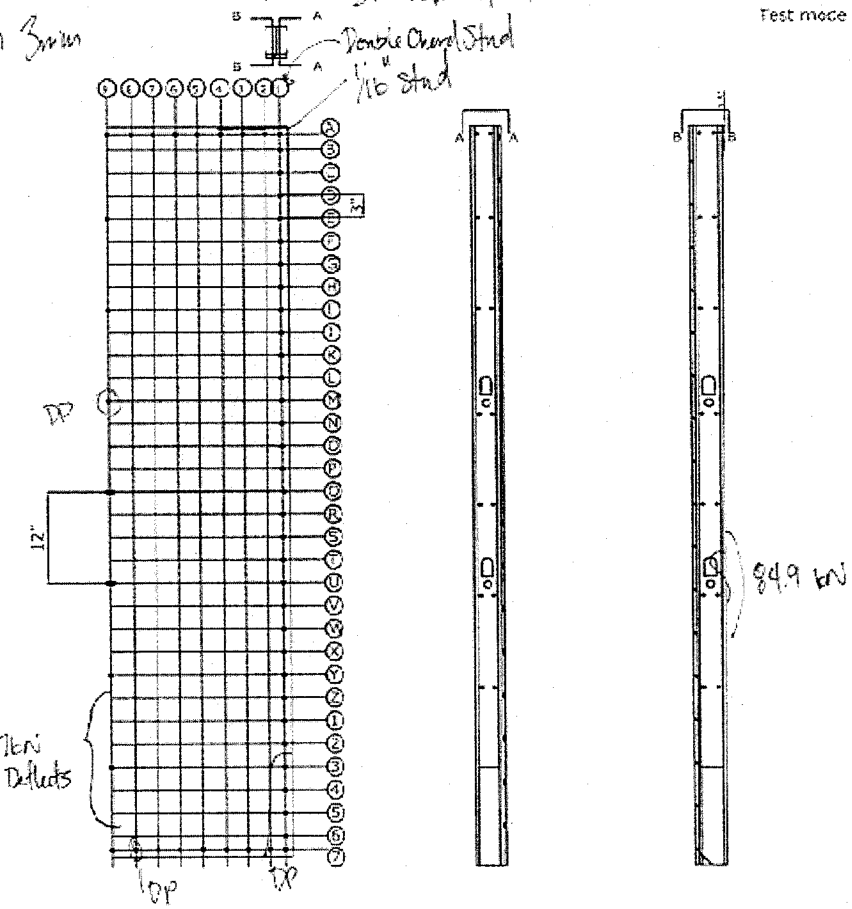
McGill

Light Gauge Steel Frame / Wood Panel Shear Wall Compression Testing

Test name : Test 23
Date tested : Feb/19
Wall Size : 2' x 8'
Screw pattern : 3" / 12"
Edge Distance : 1/2"
Test mode : ☒ Axial Compression

@ 15 kN tracks platen 3mm

Labelled Backwards
3 inch spacing all around



Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Figure A-23: Test 23 damage record

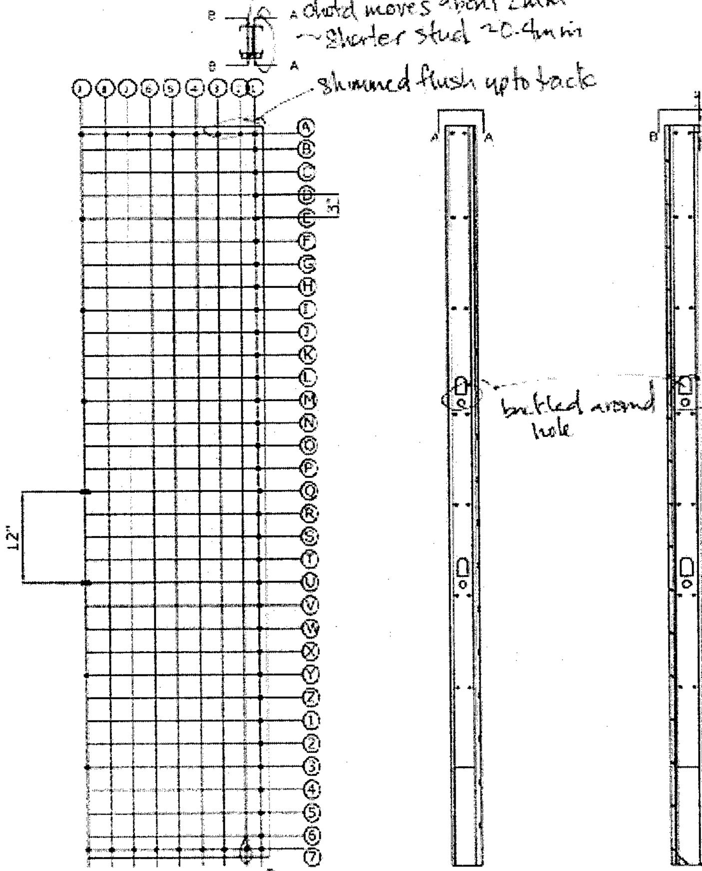
McGill

Light Gauge Steel Frame / Wood Panel Shear Wall Compression Testing

Test name : Test 24
Date tested : Feb 28 2008
Wall Size : 2' x 8'
Screw pattern : 3" / 12"
Edge Distance : 1/2"
Test mode : ☒ Axial Compression

during test
chord moves about 2mm
~ shorter stud ~0.4mm

shimmed flush up to track



DP: sheathing about 1/4" off the double chord stud (flush @ top)

No indication of bottom track flattening

Pretty sure that the rubber is causing 40 kN plateau in all the graphs

buckled around hole

* lots of movement in channels 6,7,9
→ whole wall twisted

Max Load 80.314 kN
Max Displ 14.968 mm

DP axl: not quite in

Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Figure A-24: Test 24 damage record

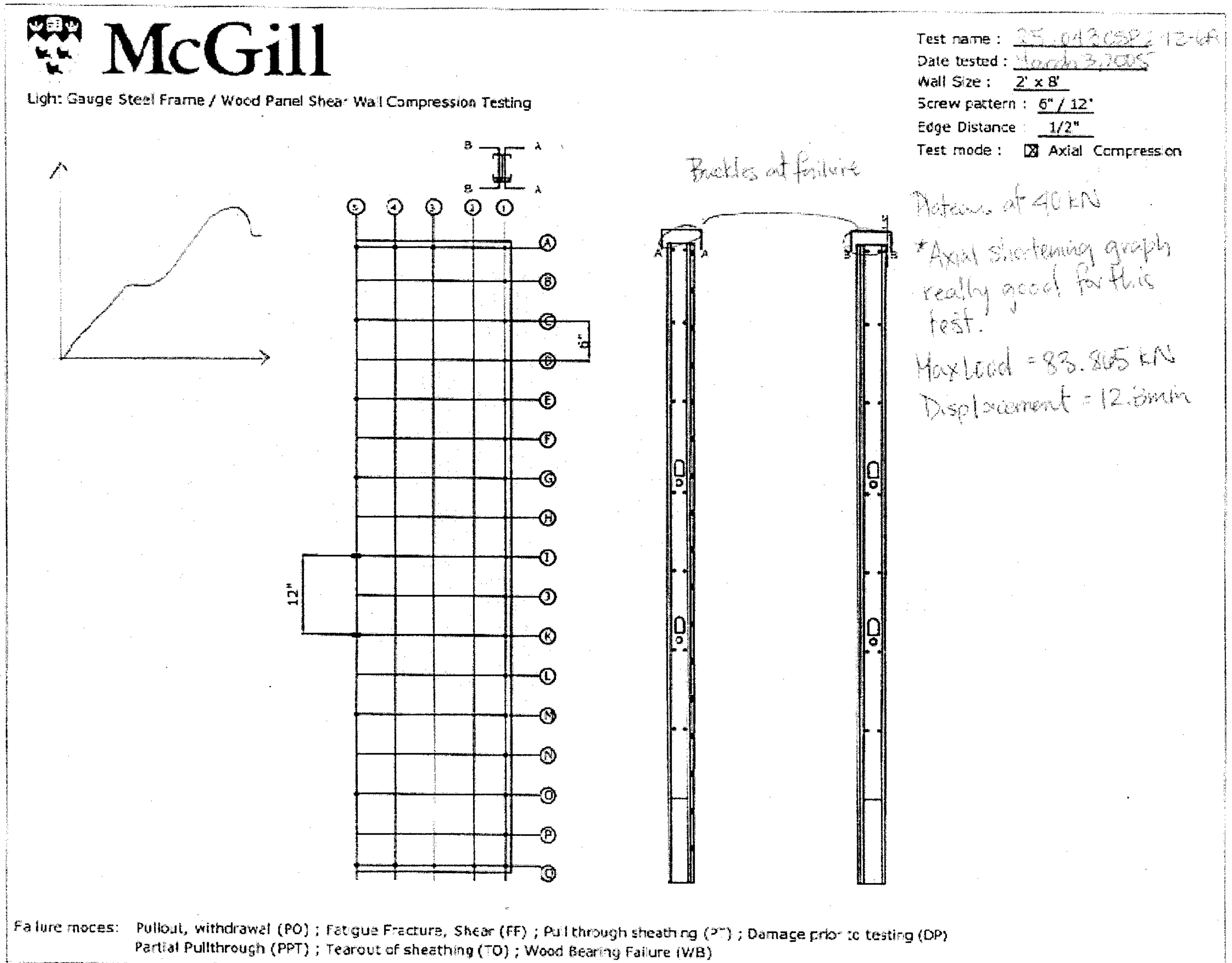


Figure A-25: Test 25 damage record

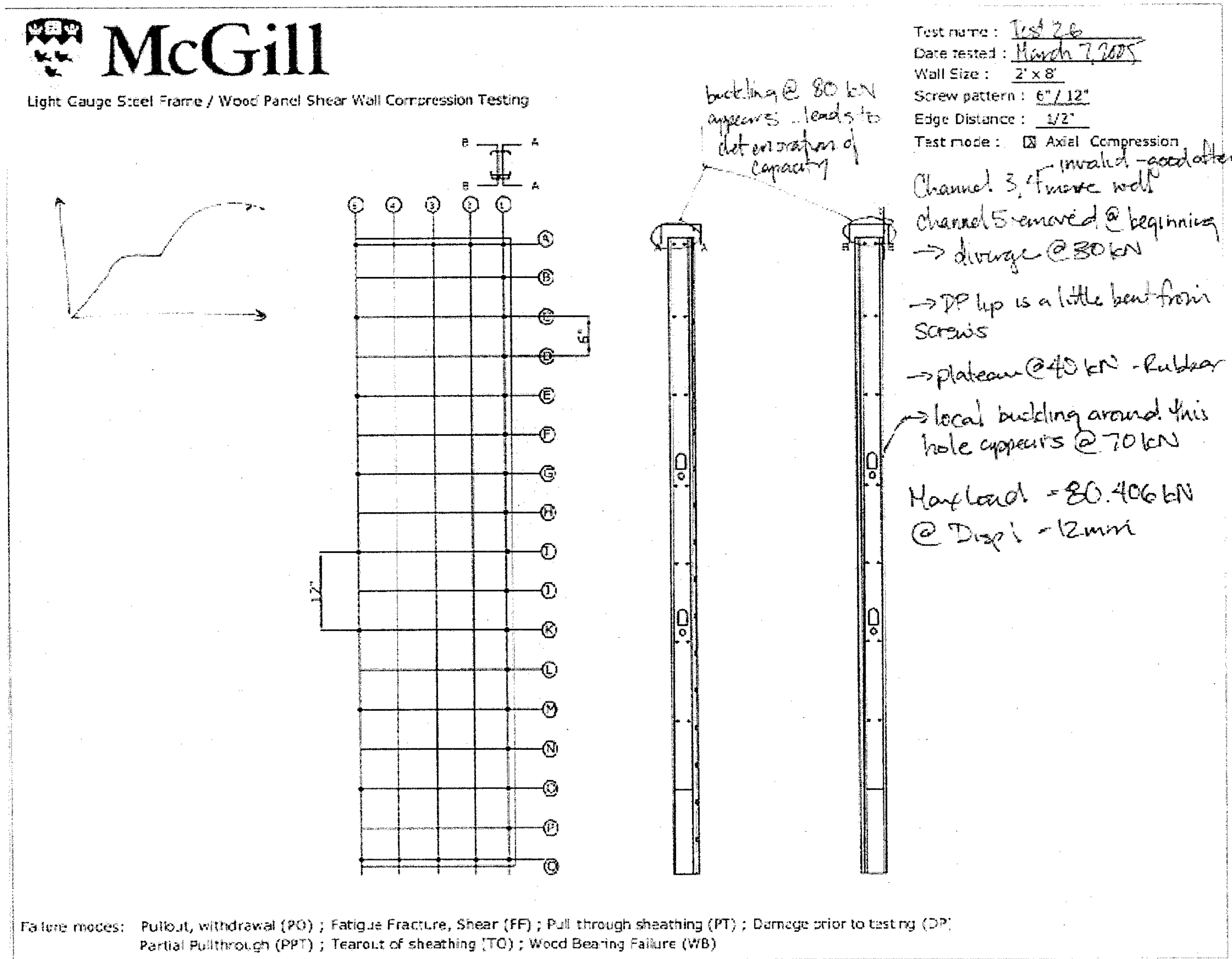


Figure A-26: Test 26 damage record

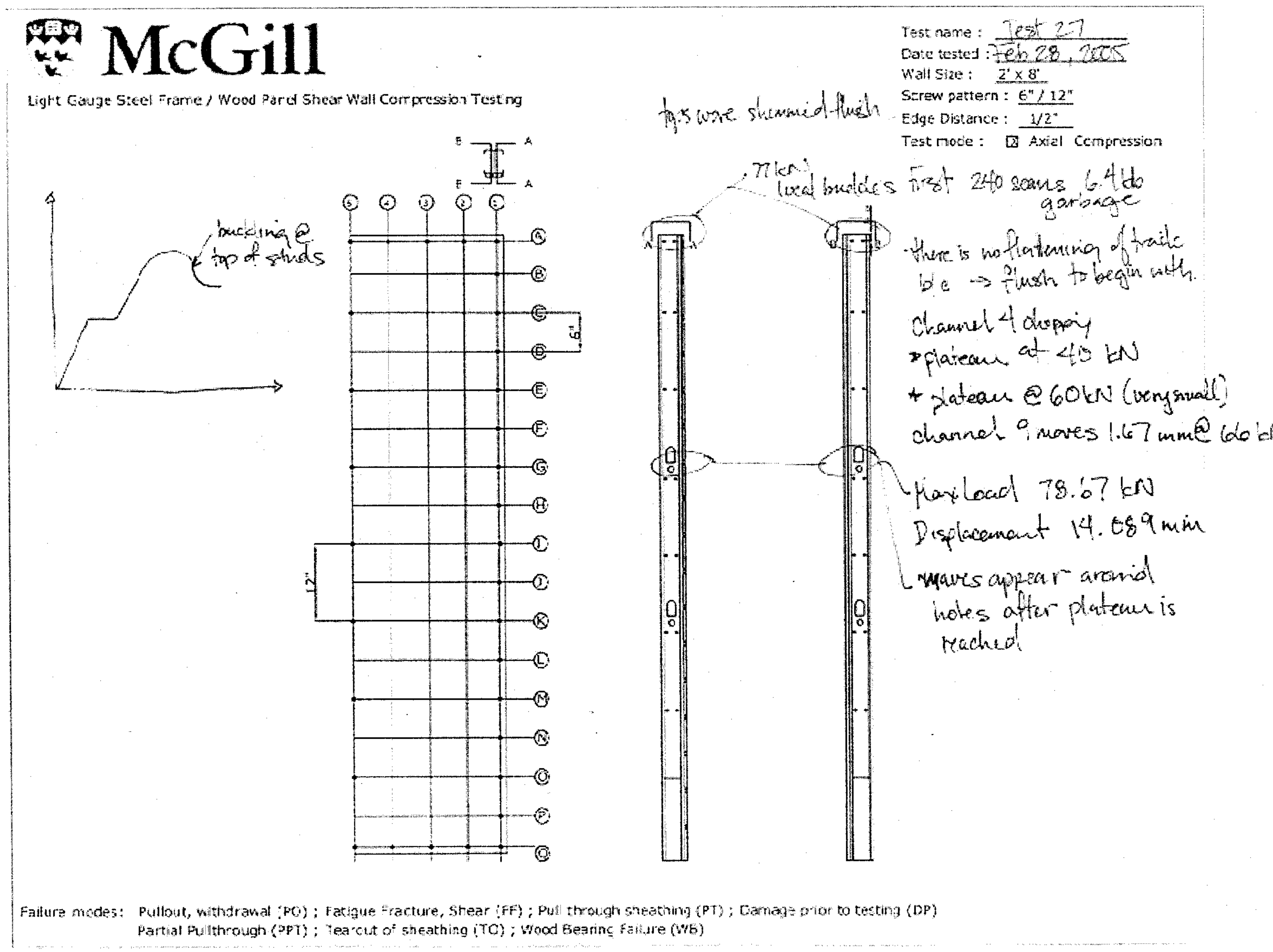


Figure A-27: Test 27 damage record

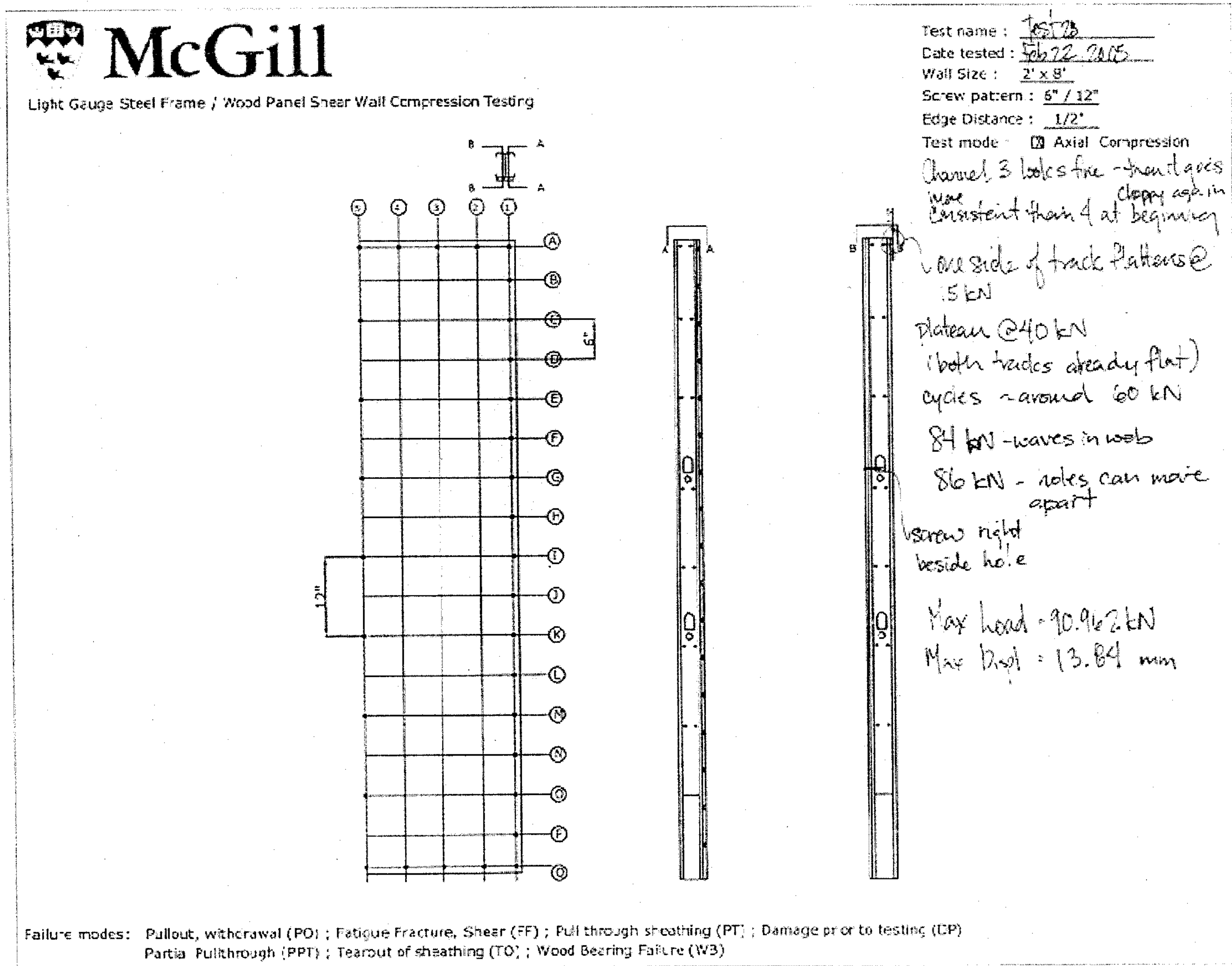


Figure A-28: Test 28 damage record

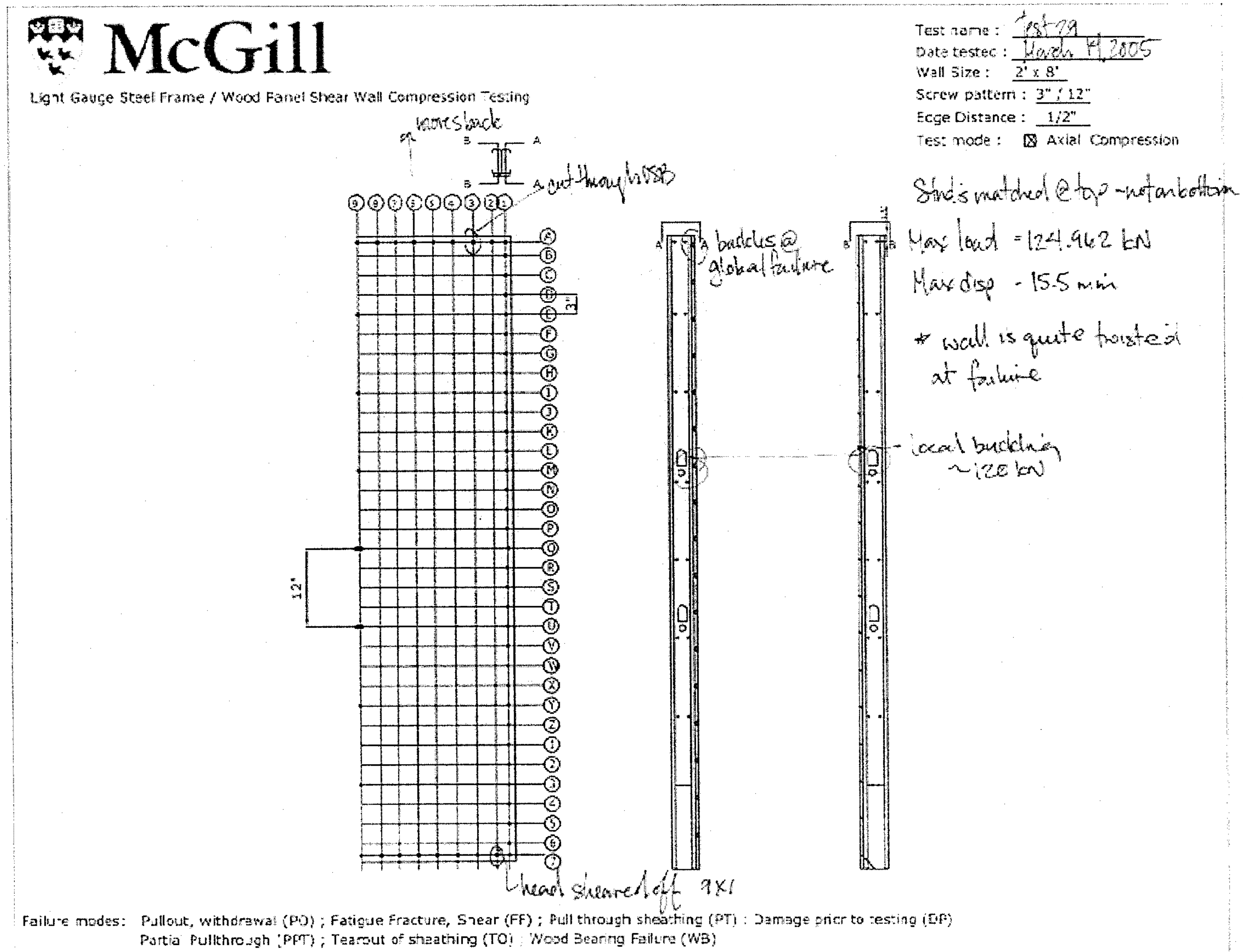
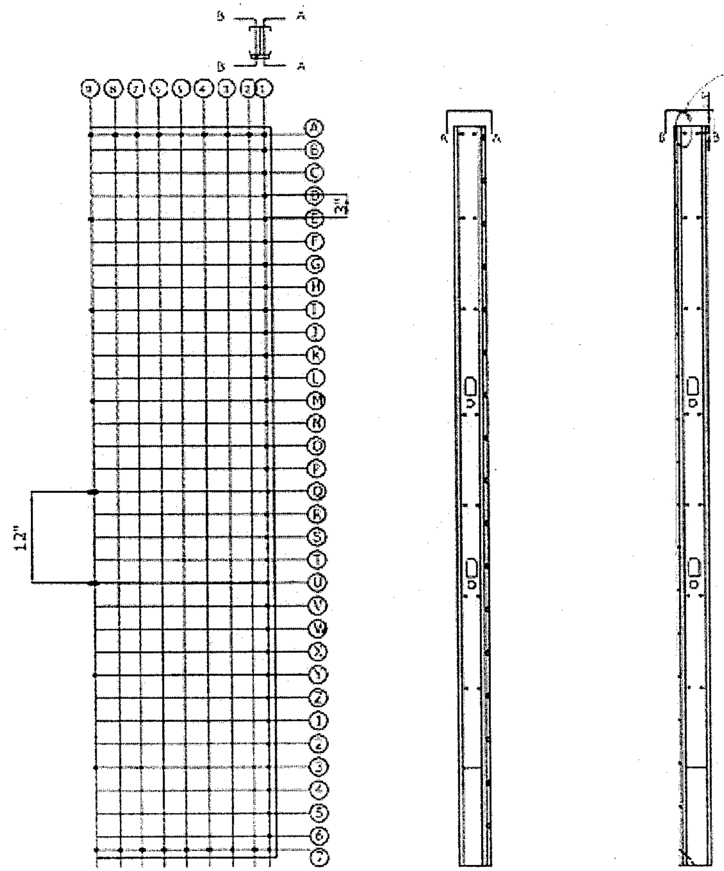


Figure A-29: Test 29 damage record

McGill

Light Gauge Steel Frame / Wood Panel Shear Wall Compression Testing

Test name : Test 30
Date tested : March 21 2005
Wall Size : 2' x 8'
Screw pattern : 3" / 12"
Edge Distance : 1/2"
Test mode : ☒ Axial Compression



- sheathing screw bent channel in - was replaced and an attempt was made to straighten it!
- channels 3 and 4 move well

40 kN plateau

local buckles @ top holes 120 kN

Max Load 124.95 kN

Deflection 15.808 mm

Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Figure A-30: Test 30 damage record

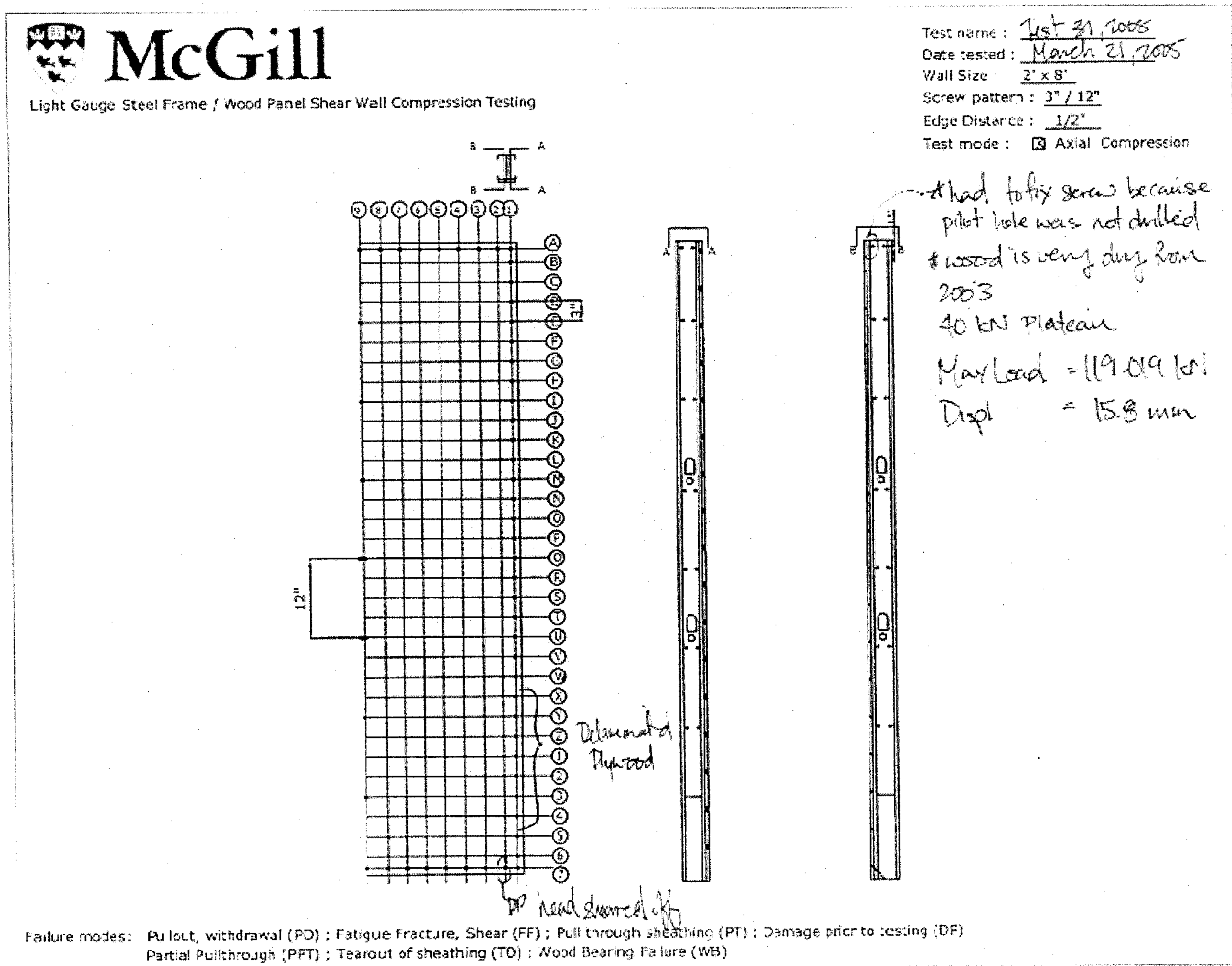


Figure A-31: Test 31 damage record

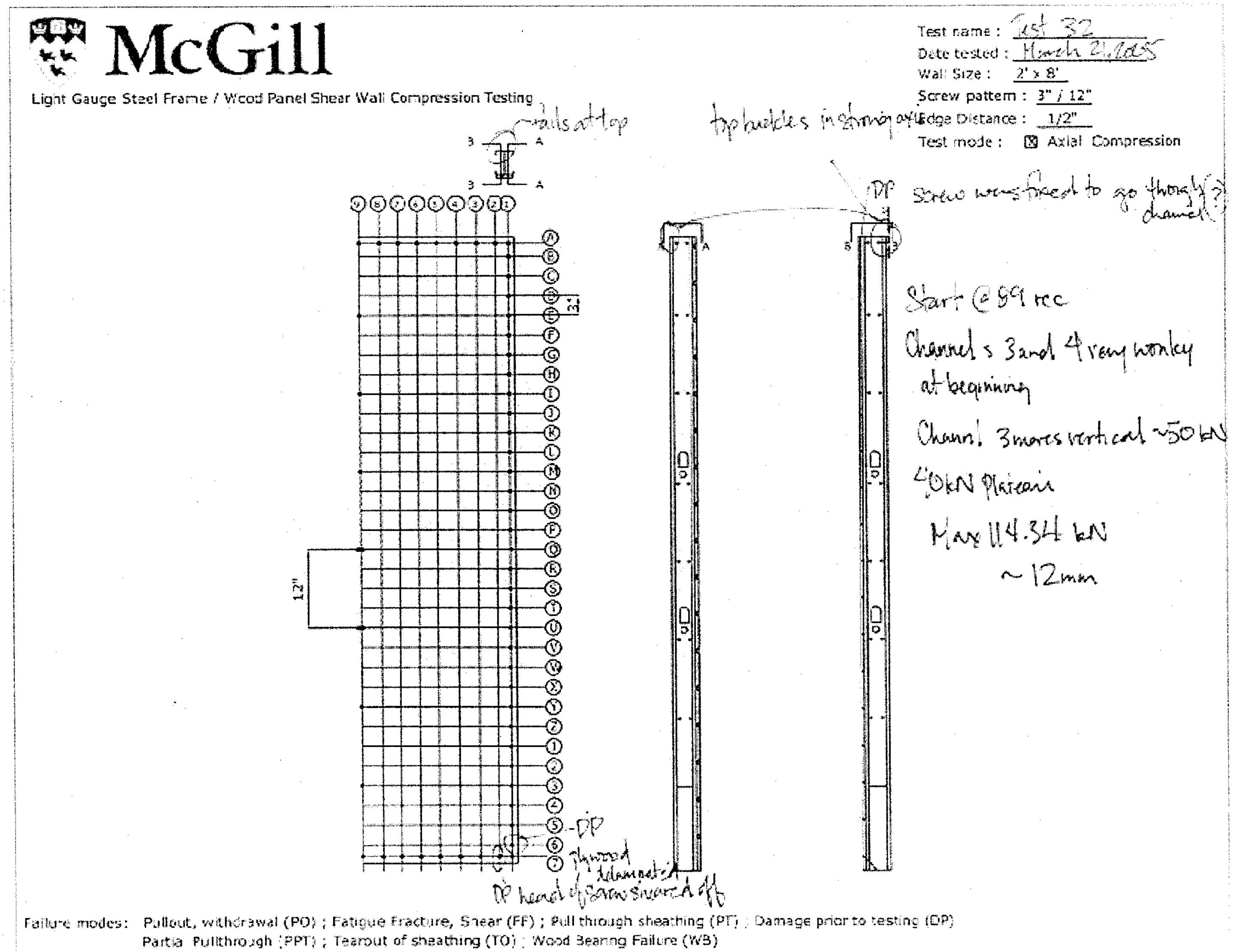
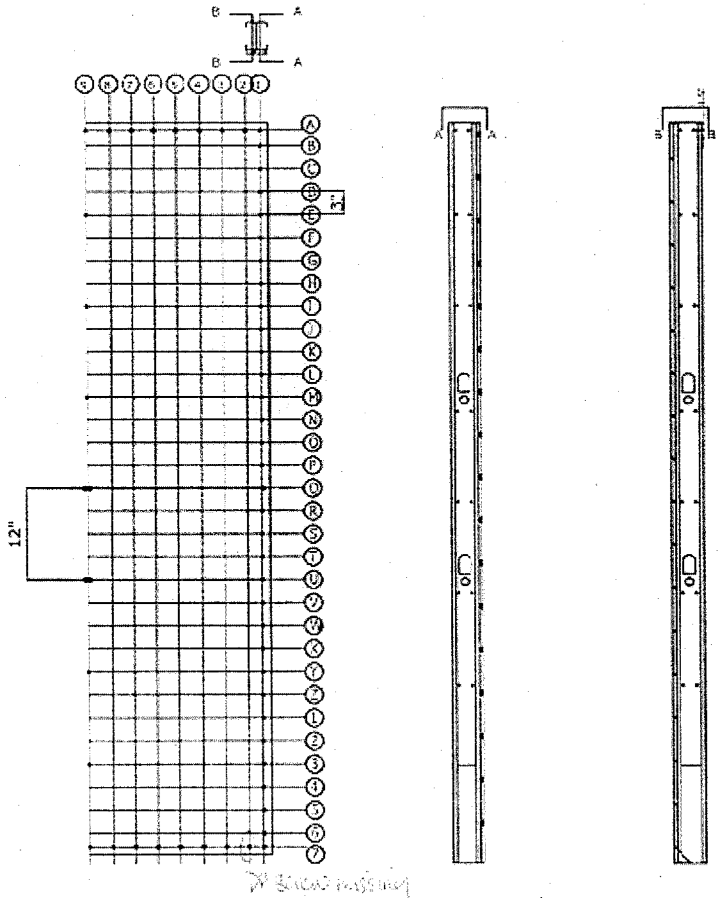


Figure A-32: Test 32 damage record

McGill

Light Gauge Steel Frame / Wood Panel Shear Wall Compression Testing

Test name : Test 33
Date tested : Mar/c 2025
Wall Size : 2' x 8'
Screw pattern : 3" / 12"
Edge Distance 1/2"
Test mode : ☒ Axial Compression



30 kN channel 9 1.22 mm
7 0.68 mm

37 kN Tracks Flatten
40 kN Plateau Rubber

channel 3 choppy at first, channel 4 fine

62 kN channel 9 2.27 mm
7 1.02 mm

93 kN pump cycles

157 kN channel 9 3.00 mm
7 2.17 mm
6 0.29 mm

Track at top widening quite a bit.

Max load 172.976 kN
Displacement 7.655 mm

Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PFT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Figure A-33: Test 33 damage record

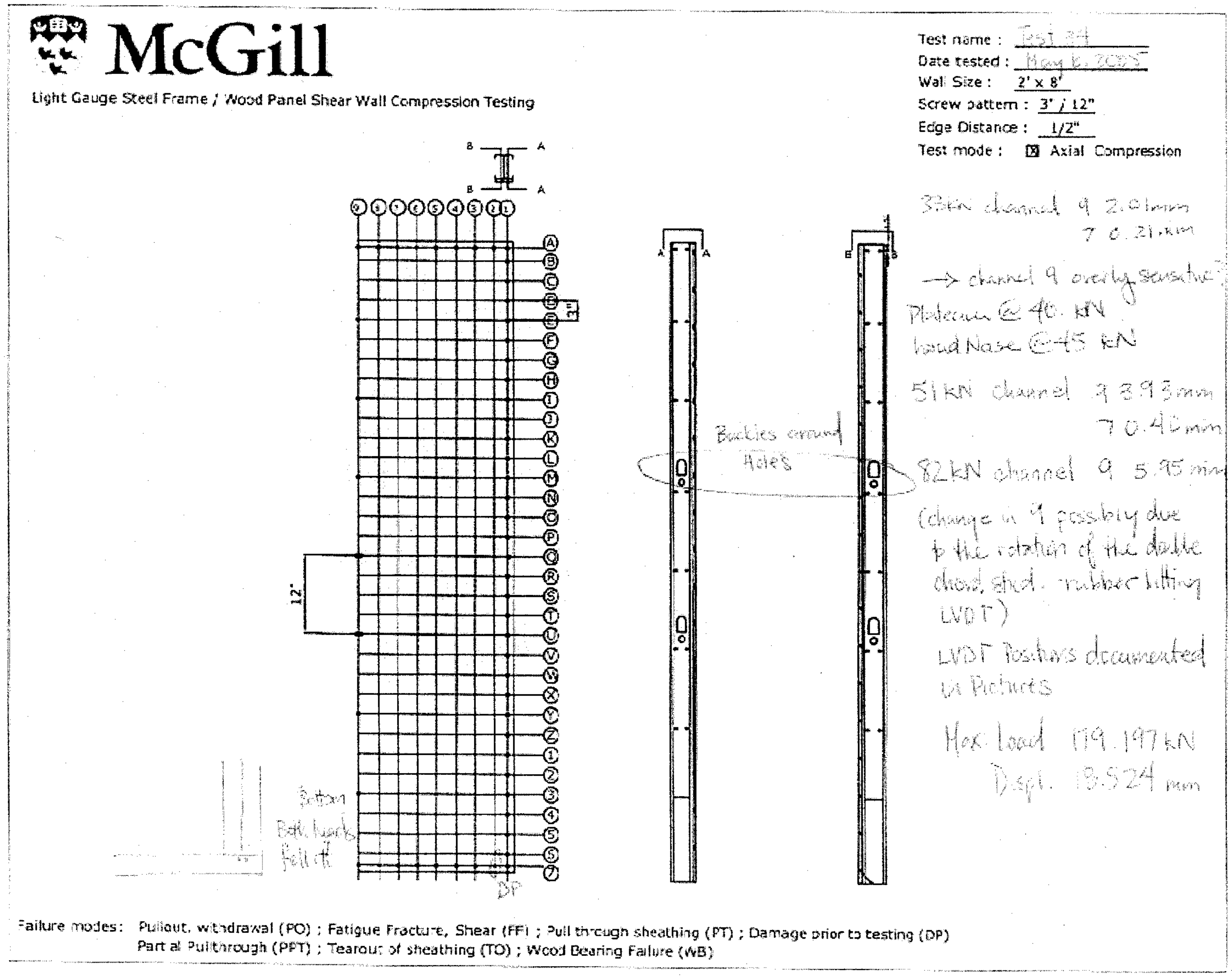
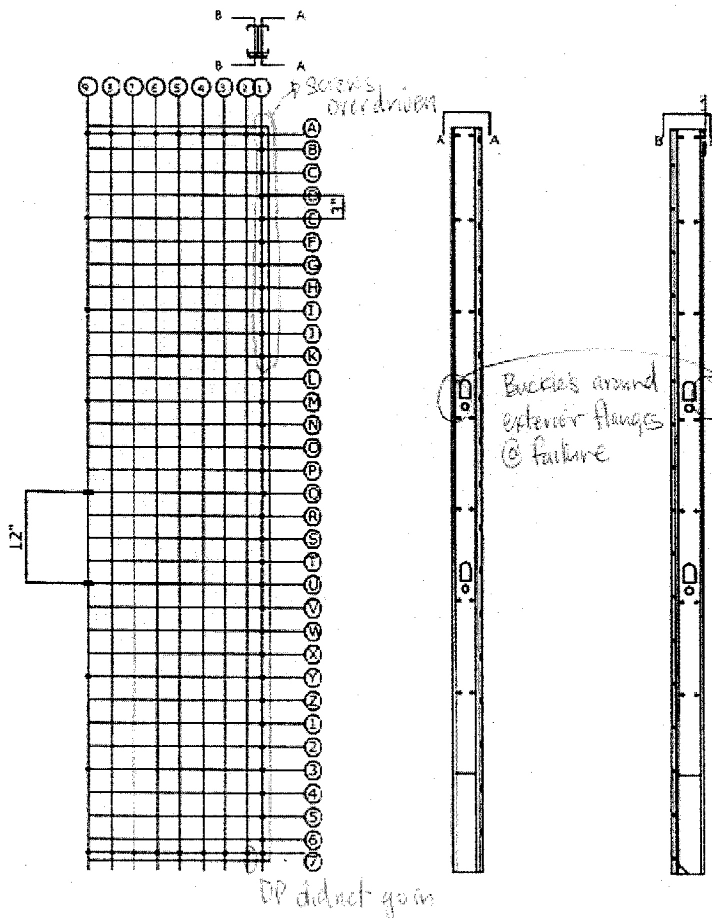


Figure A-34: Test 34 damage record

McGill

Light Gauge Steel Frame / Wood Panel Shear Wall Compression Testing

Test name : Test 35
Date tested : May 1, 2015
Wall Size : 2' x 8'
Screw pattern : 3" / 12"
Edge Distance : 1/2"
Test mode : ☒ Axial Compression



10 kN Track at top squeaks
15 kN Bottom tracks squeaky
– some of the screws in the hold-down are burnt-out

Channel 3 reads vertically @ beginning
Channel 4 is choppy @ beginning
Channel 9 1.02 mm @ 28 kN
falling forward
(32 kN both channel 3 and 4 pick-up)
42 kN plateau/rubber
channel 9 : 2.63 @ 65 kN
channels pulling away from track at top... elastic buckling
Load 172.341 kN
Displ ~18 mm

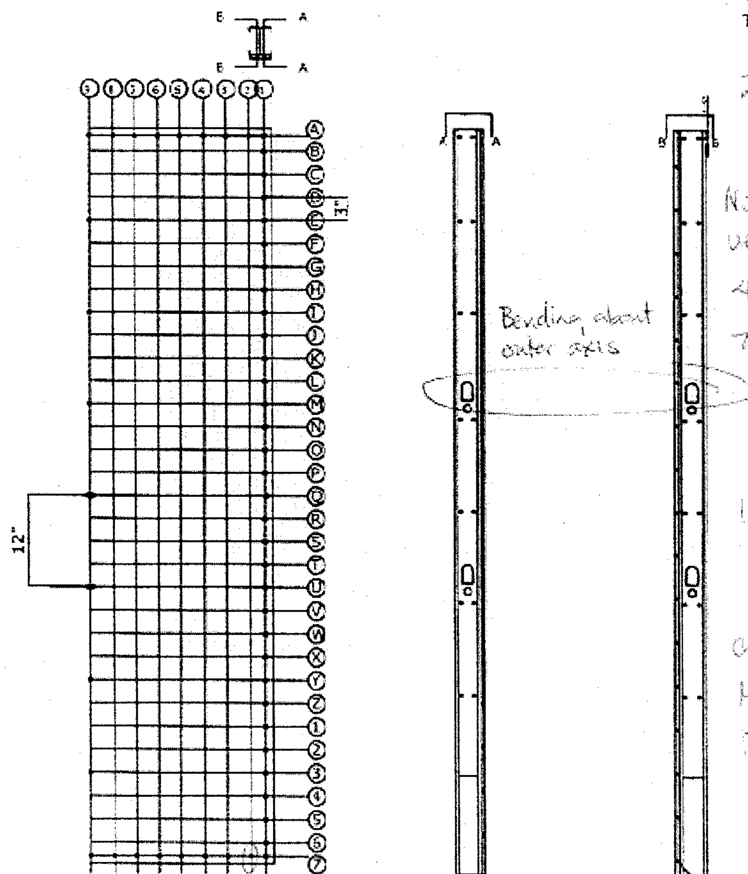
Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Figure A-35: Test 35 damage record

McGill

Light Gauge Steel Frame / Wood Panel Shear Wall Compression Testing

Test name : Test 36
Date tested : May 6, 2005
Wall Size : 2' x 8'
Screw pattern : 3" / 12"
Edge Distance : 1/2"
Test mode : ☒ Axial Compression



29 kN channel 9 1.14 mm
7 0.72 mm

No sudden collapse of tracks until 40 kN - bottom track.

40 kN Plateau - Rubber

76 kN channel 9 2.93 mm
7 0.41 mm
10 2.77 mm

157 kN channel 9 6.46 mm
10 3.32 mm
7 1.27 mm

cycled about 155 kN
Max Load 182.968 kN
Displacement 20.440 mm

* Forgot to stop [illegible] for a while, so they'll be a little bit of a delay

DP no screw, sheared off during construction

Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Figure A-36: Test 36 damage record

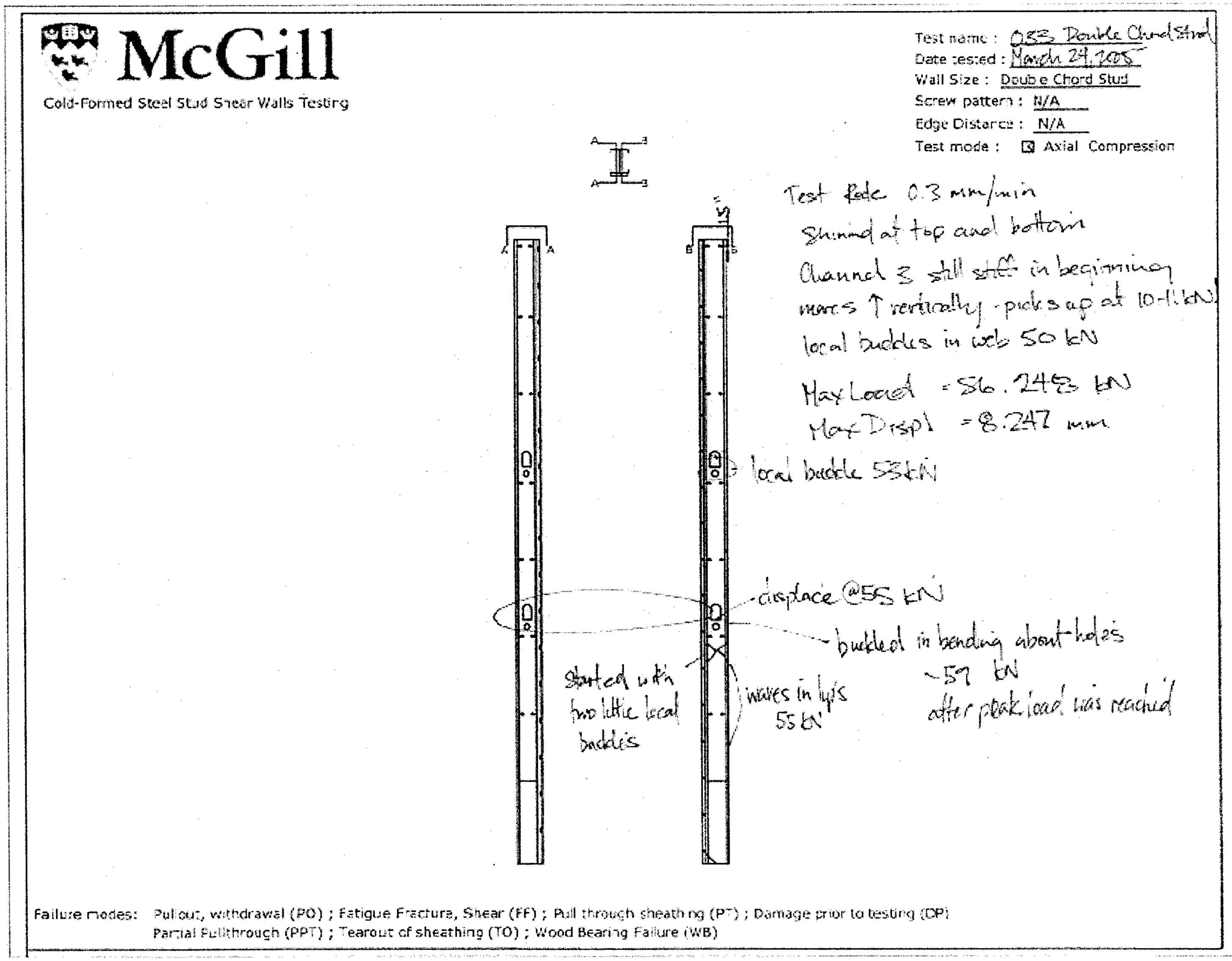


Figure A-37: Test 37 damage record

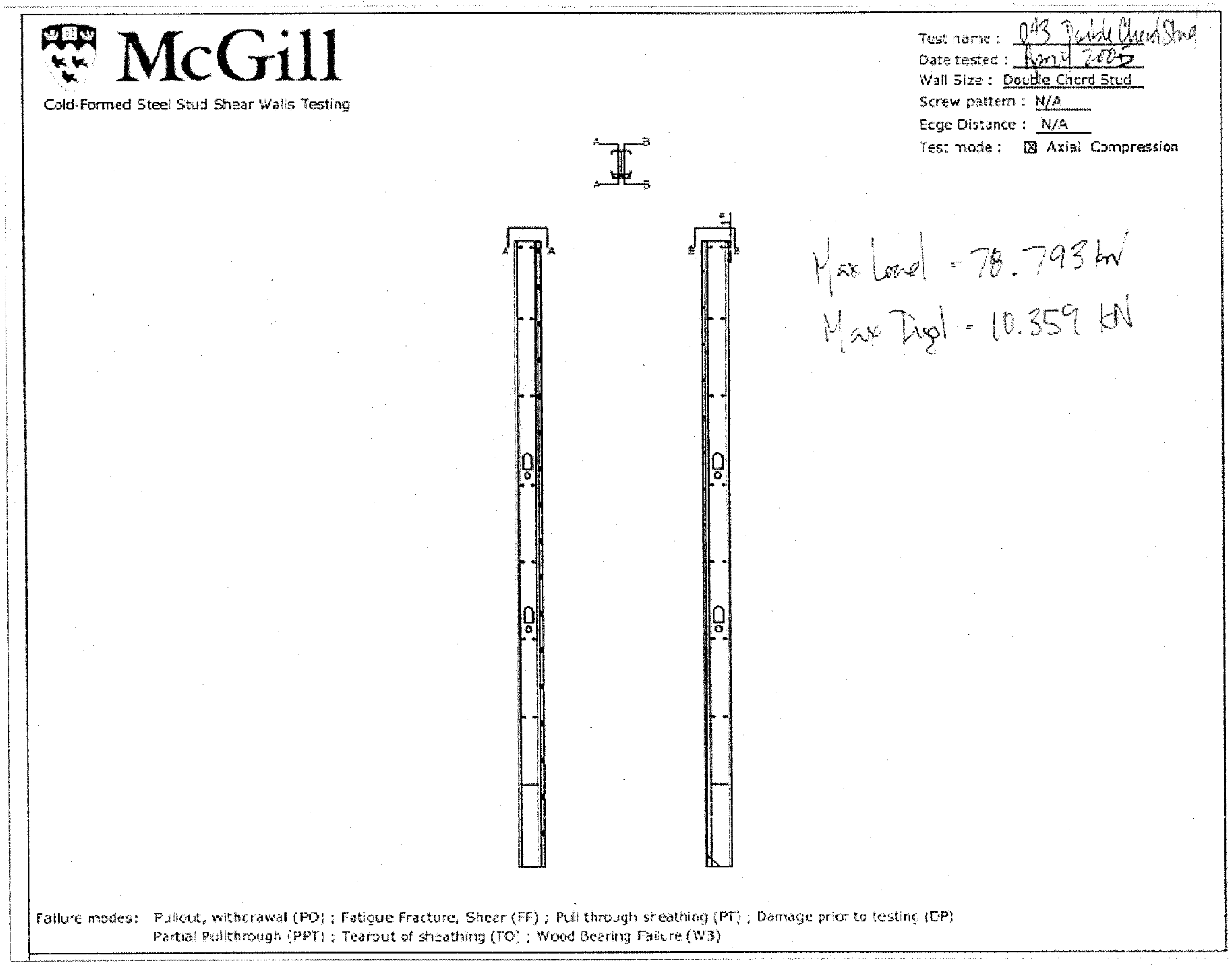
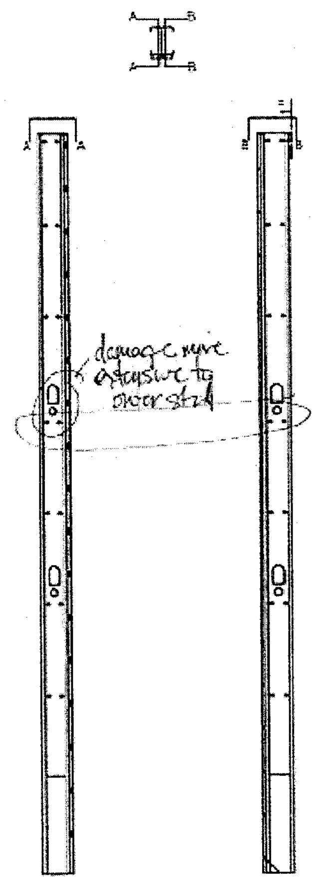


Figure A-38: Test 38 damage record

McGill

Cold-Formed Steel Stud Shear Walls Testing

Test name : C56 Double Chord Stud
Date tested : March 31, 2005
Wall Size : Double Chord Stud
Screw pattern : N/A
Edge Distance : N/A
Test mode : ☒ Axial Compression



Channels SH tacked in parallel
40 kN Plateau
cycles @ 45 kN
88 kN cycles
holes displace 107 kN
Max Load = 109.748 kN
Max Displacement = 11.764 mm
buckles and twists about hole

Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Figure A-39: Test 39 damage record

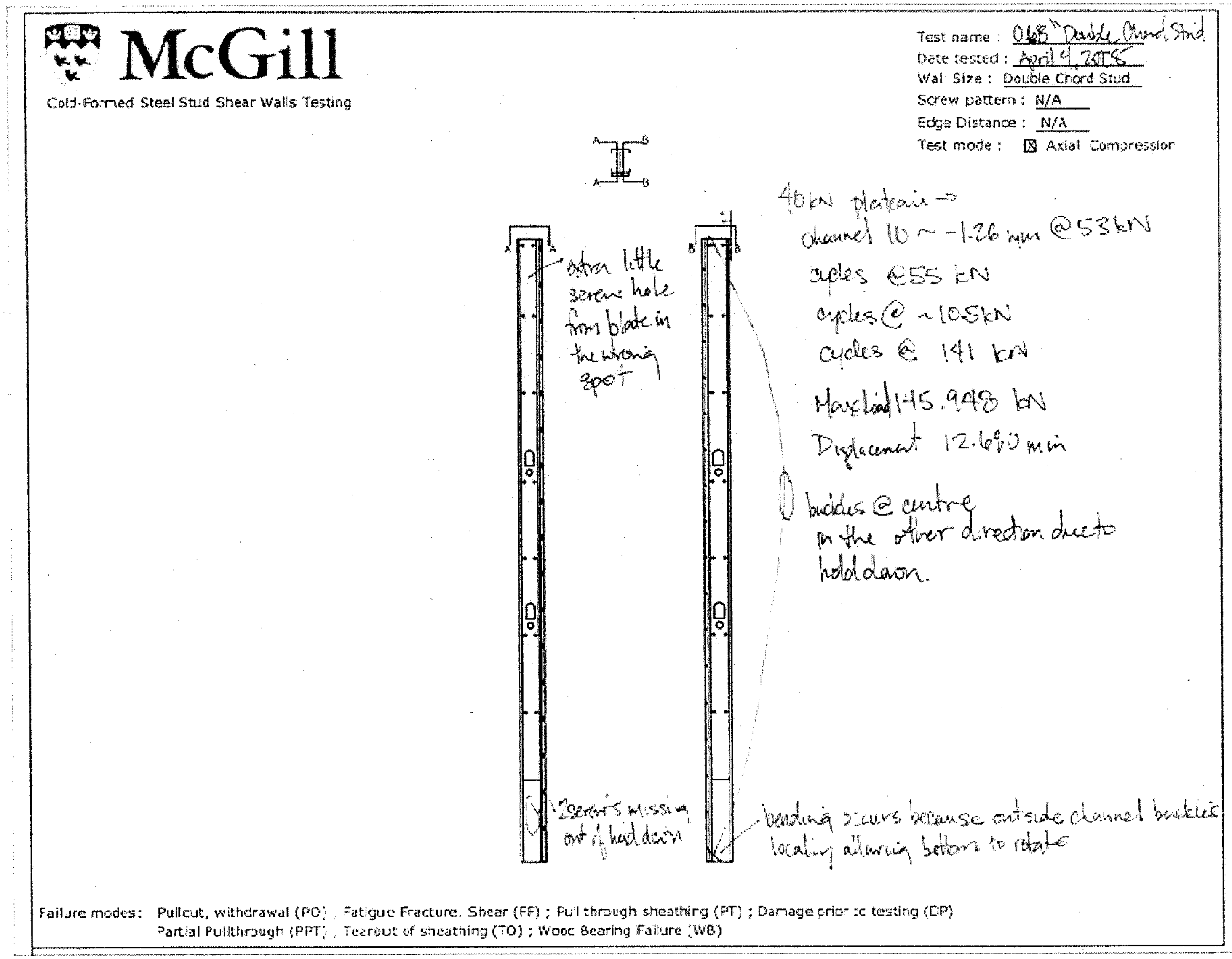


Figure A-40: Test 40 damage record

APPENDIX B

REVERSED CYCLIC TEST PROTOCOLS

Table B-1 : CUREE cyclic protocol for tests 48-A, B, C

$\Delta=0.6*\Delta_m$	47.61	Screw Pattern: 3"/12"	
		Sheathing: DFP	
Displ.	Target (corr.) mm	Actuator Input mm	No. Of cycles
0.050 Δ	2.380	3.065	6
0.075 Δ	3.571	4.597	1
0.056 Δ	2.678	3.448	6
0.100 Δ	4.761	6.129	1
0.075 Δ	3.571	4.597	6
0.200 Δ	9.521	12.258	1
0.150 Δ	7.141	9.194	3
0.300 Δ	14.282	18.387	1
0.225 Δ	10.712	13.790	3
0.400 Δ	19.043	24.516	1
0.300 Δ	14.282	18.387	2
0.700 Δ	33.325	42.904	1
0.525 Δ	24.994	32.178	2
1.000 Δ	47.607	61.291	1
0.750 Δ	35.705	45.968	2
1.500 Δ	71.410	91.936	1
1.125 Δ	53.558	68.952	2
2.000 Δ	95.214	122.581	1
1.500 Δ	71.410	91.936	2

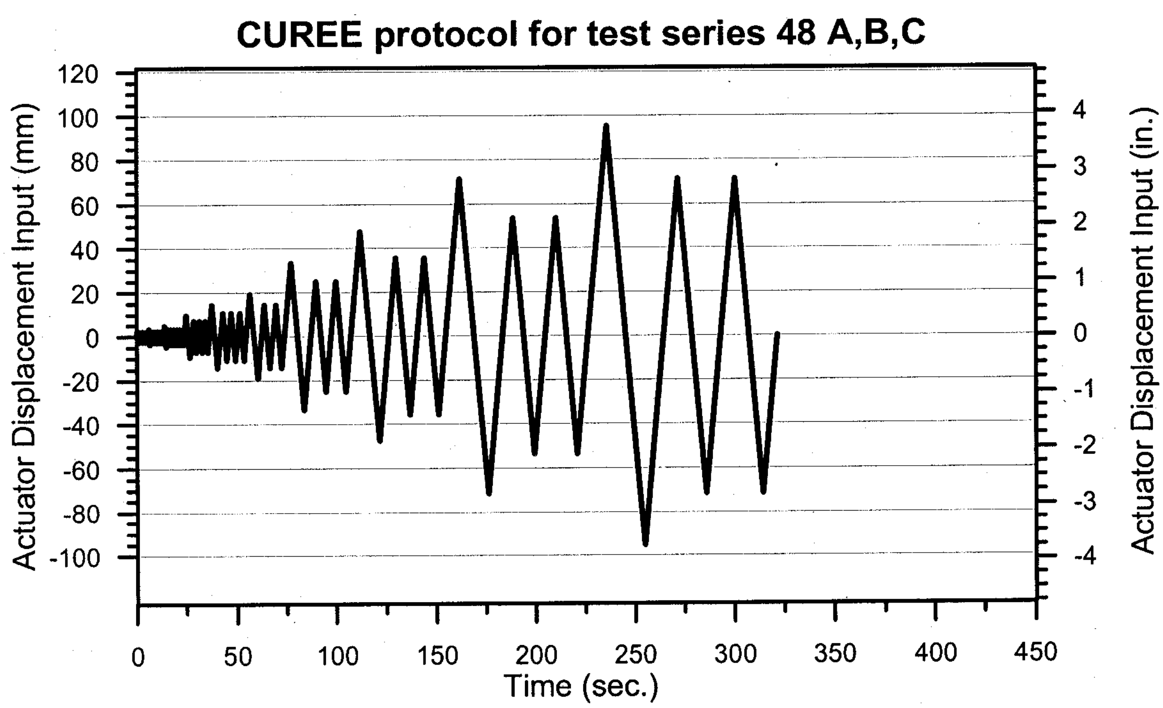


Figure B-1 : CUREE cyclic protocol for tests 48-A, B, C

Table B-2 : CUREE cyclic protocol for tests 50-A, B, C

$\Delta=0.6^*\Delta_m$	34.37	Screw Pattern: 6"/12"	
		Sheathing: OSB	
	Target (corr.)	Actuator Input	
Displ.	mm	mm	No. Of cycles
0.050 Δ	1.718	1.813	6
0.075 Δ	2.578	2.720	1
0.056 Δ	1.933	2.040	6
0.100 Δ	3.437	3.627	1
0.075 Δ	2.578	2.720	6
0.200 Δ	6.874	7.254	1
0.150 Δ	5.155	5.440	3
0.300 Δ	10.311	10.881	1
0.225 Δ	7.733	8.160	3
0.400 Δ	13.748	14.508	1
0.300 Δ	10.311	10.881	2
0.700 Δ	24.059	25.388	1
0.525 Δ	18.044	19.041	2
1.000 Δ	34.370	36.269	1
0.750 Δ	25.777	27.202	2
1.500 Δ	51.555	54.403	1
1.125 Δ	38.666	40.802	2
2.000 Δ	68.740	72.538	1
1.500 Δ	51.555	54.403	2

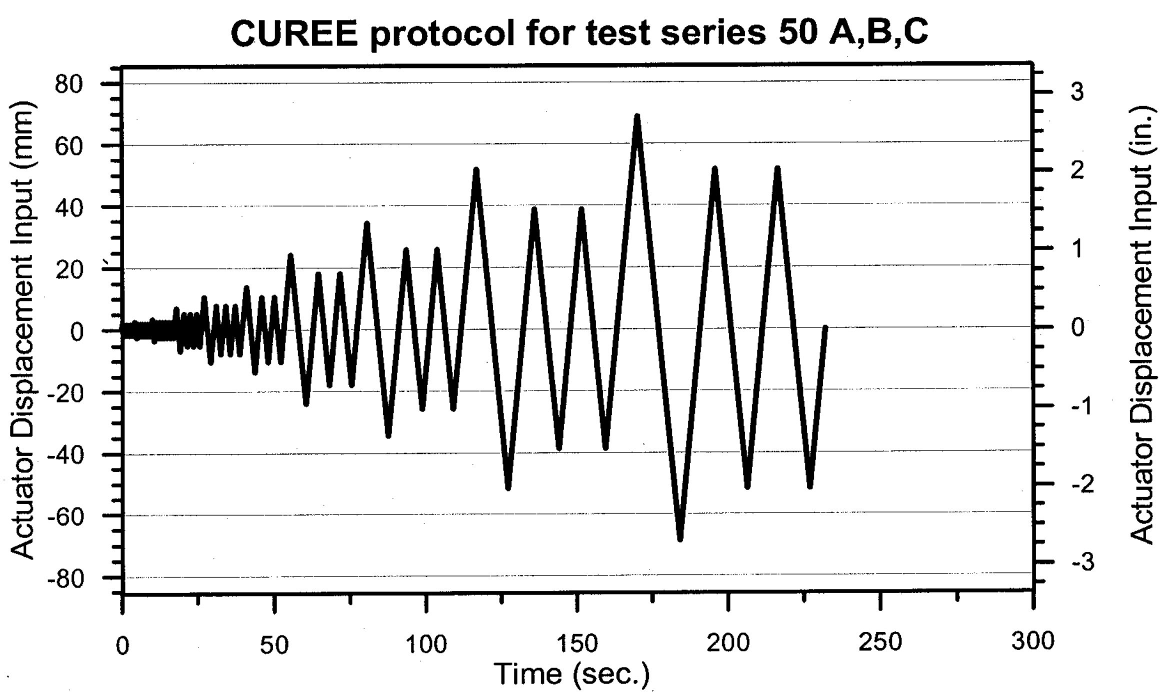


Figure B-2 : CUREE cyclic protocol for tests 50-A, B, C

Table B-3 : CUREE cyclic protocol for tests 52-A, C

$\Delta=0.6*\Delta_m$	28.82	Screw Pattern: 3"/12"	
		Sheathing: OSB	
	Target (corr.)	Actuator Input	
Displ.	mm	mm	No. Of cycles
0.050 Δ	1.441	1.531	6
0.075 Δ	2.161	2.296	1
0.056 Δ	1.621	1.722	6
0.100 Δ	2.882	3.062	1
0.075 Δ	2.161	2.296	6
0.200 Δ	5.764	6.123	1
0.150 Δ	4.323	4.592	3
0.300 Δ	8.646	9.185	1
0.225 Δ	6.484	6.888	3
0.400 Δ	11.528	12.246	1
0.300 Δ	8.646	9.185	2
0.700 Δ	20.173	21.431	1
0.525 Δ	15.130	16.073	2
1.000 Δ	28.819	30.615	1
0.750 Δ	21.614	22.961	2
1.500 Δ	43.228	45.923	1
1.125 Δ	32.421	34.442	2
2.000 Δ	57.638	61.230	1
1.500 Δ	43.228	45.923	2

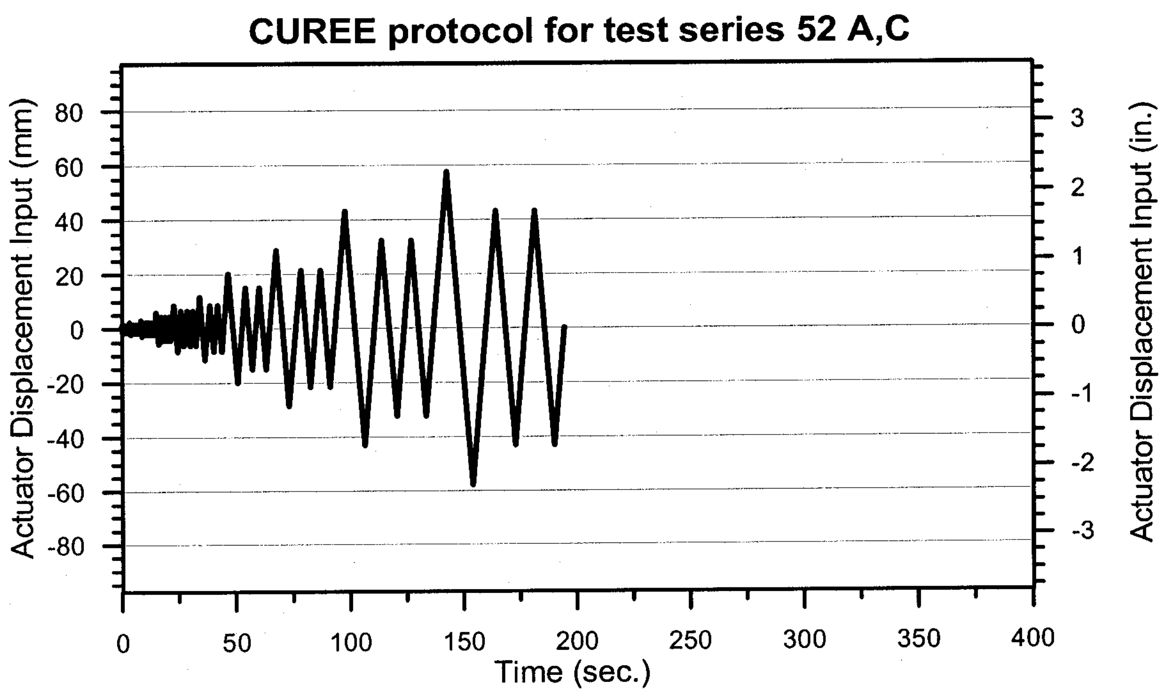


Figure B-3 : CUREE cyclic protocol for tests 52-A, C

Table B-4 : CUREE cyclic protocol for tests 54-A, B, C

Δ=0.6*Δ_m	47.03	Screw Pattern: 6"/12"	
		Sheathing: CSP	
	Target (corr.)	Actuator Input	
Displ.	mm	mm	No. Of cycles
0.050 Δ	2.351	2.486	6
0.075 Δ	3.527	3.729	1
0.056 Δ	2.645	2.796	6
0.100 Δ	4.703	4.971	1
0.075 Δ	3.527	3.729	6
0.200 Δ	9.406	9.943	1
0.150 Δ	7.054	7.457	3
0.300 Δ	14.109	14.914	1
0.225 Δ	10.582	11.186	3
0.400 Δ	18.812	19.885	1
0.300 Δ	14.109	14.914	2
0.700 Δ	32.921	34.799	1
0.525 Δ	24.691	26.100	2
1.000 Δ	47.030	49.714	1
0.750 Δ	35.272	37.285	2
1.500 Δ	70.545	74.570	1
1.125 Δ	52.908	55.928	2
2.000 Δ	94.059	99.427	1
1.500 Δ	70.545	74.570	2

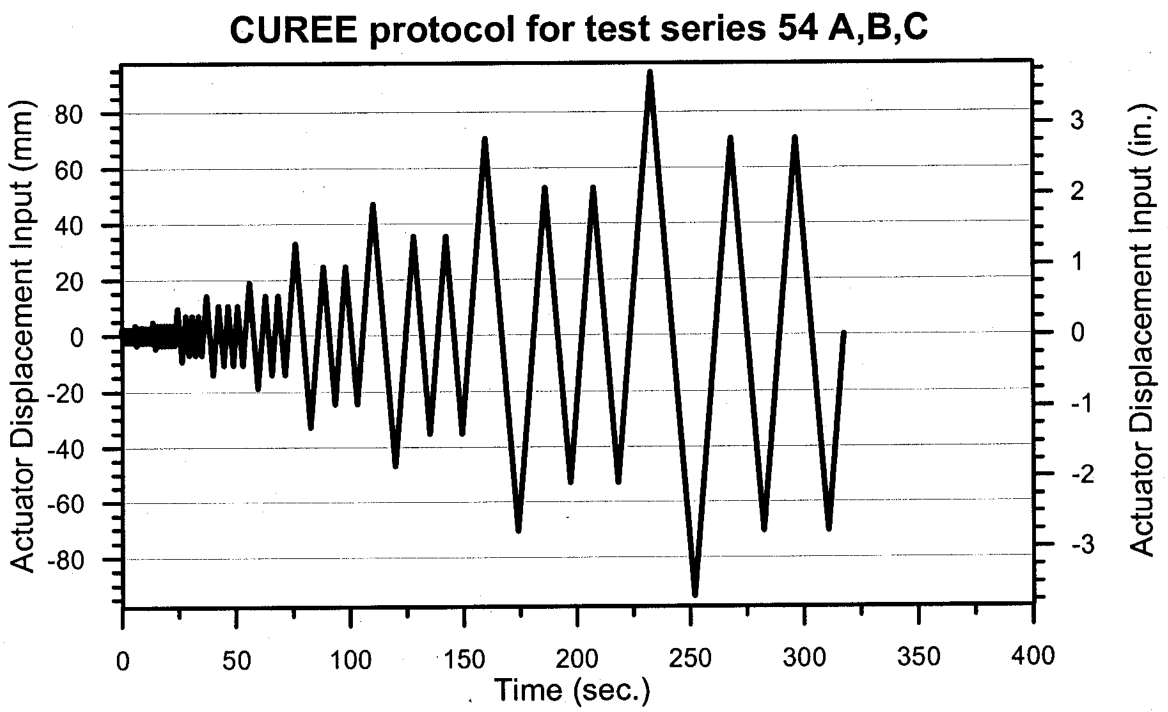


Figure B-4 : CUREE cyclic protocol for tests 54-A, B, C

Table B-5 : CUREE cyclic protocol for tests 56-A, B, C

$\Delta=0.6*\Delta_m$	49.85	Screw Pattern: 3"/12"	
		Sheathing: CSP	
	Target (corr.)	Actuator Input	
Displ.	mm	mm	No. Of cycles
0.050 Δ	2.492	2.644	6
0.075 Δ	3.739	3.966	1
0.056 Δ	2.804	2.974	6
0.100 Δ	4.985	5.288	1
0.075 Δ	3.739	3.966	6
0.200 Δ	9.970	10.575	1
0.150 Δ	7.477	7.931	3
0.300 Δ	14.954	15.863	1
0.225 Δ	11.216	11.897	3
0.400 Δ	19.939	21.151	1
0.300 Δ	14.954	15.863	2
0.700 Δ	34.894	37.014	1
0.525 Δ	26.170	27.760	2
1.000 Δ	49.848	52.877	1
0.750 Δ	37.386	39.657	2
1.500 Δ	74.772	79.315	1
1.125 Δ	56.079	59.486	2
2.000 Δ	99.697	105.753	1
1.500 Δ	74.772	79.315	2

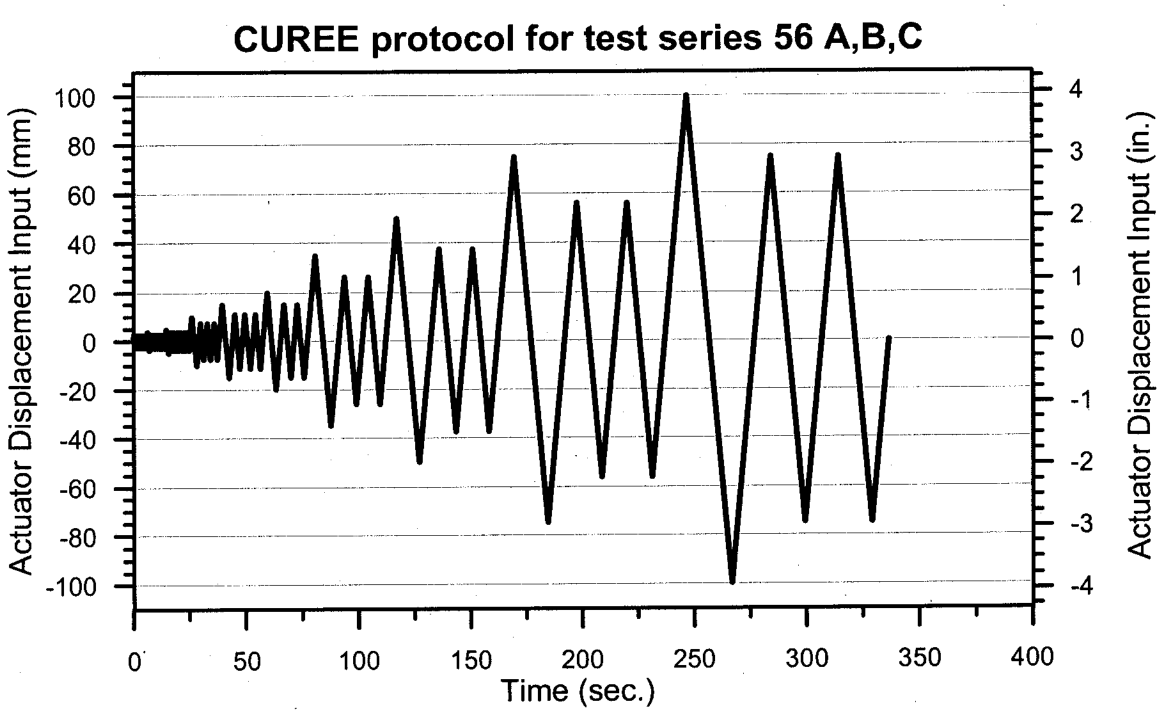


Figure B-5 : CUREE cyclic protocol for tests 56-A, B, C

APPENDIX C

SHEAR WALL TEST DATA

SUMMARY

In the summer of 2005 a total of 32 light gauge steel frame / wood panel shear walls were tested. Typically, each test group consisted of 6 specimens (3 monotonic and 3 reversed cyclic) for the reliability of data. However, in two cases the monotonic exhibited variations larger than 10% and it was necessary to complete an additional test. This appendix reports the test data and results for the 32 tests.

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

- Either 12.5 mm CSA O121 Exterior Douglas Fir Plywood (DFP) *(CSA O121, 1978)*, 12.5 mm CSA O151 Exterior Canadian Softwood Plywood (CSP) *(CSA O151, 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 the following 4 nominal grades and thicknesses: 1. 230 MPa (33 ksi) and 0.84 mm (0.033"), 2. 230 MPa (33ksi) and 1.09 mm (0.043"), 3. 340 MPa (50 ksi) and 1.37 mm (0.054") and 4. 340 MPa (50 ksi) and 1.72 mm. All studs had nominal dimensions of: 92.1 mm (3-5/8") web, 41.3 mm (1-5/8") flanges and 12.7 mm (1/2") lips. A request to the manufacturer was made to slow down the cutting speed in the manufacturing process to reduce the deformities at the ends of studs and tracks to ease the construction.
- Light gauge steel top and bottom tracks manufactured in Canada to ASTM A653 (2002) with nominal grade of 230 MPa (33 ksi) and a thickness of 1.09 mm (0.043"). The track's nominal dimensions were 92.1 mm (3-5/8") web and 30.2 mm (1-3/16") flange.
- The double chord stud (DCS) consists of two studs connected back-to-back and connected by two No. 10—16 x 19.1 mm (3/4") long Hex washer head self-drilling screws at 305 mm (12") on centre. The built-up member was used to prevent the flexural and/or local

buckling failure of a single chord stud alone. Remaining interior stud were spaced at 610 mm (24") on centre.

- Industry standard Simpson Strong-Tie S/HD10 (Simpson, 2001) hold-down connectors were attached to the DCS with 33 No. 10-16 x 19.1 mm (3/4") long Hex washer head self-drilling screws. To fasten the hold-down to the test frame ASTM A193 (2006) 22.2 mm (7/8") anchor rods were used.
- Bolts, 19.1 mm (3/4") diameter ASTM A325 (2002), were used as shear anchors
- No. 8 x 12.7 mm (1/2") long wafer head self-drilling framing screws were used to connect the track and studs.
- No. 8 x 38.1 mm (1-1/2") Grabber SuperDrive (SuperDrive, 2003) Bugle head self-piercing sheathing screws and No. 8 x 31.8 mm (1-1/4") Grabber SuperDrive (SuperDrive, 2003) Bugle head self-drilling sheathing screws were used to affix the sheathing to the light gauge steel frames. The sheathing-to-framing screws were put in 12.7 mm (1/2") away from the edge of each sheathing panel. The screw spacing / fastener schedule was 75 mm (3") or 152 mm (6") along the panel edges and 305 mm (12") in the interior.

The following were the six varying factors throughout the five different wall configurations: wood sheathing type, the loading protocol, the chord stud thickness, the fastener schedule, the sheathing-to-framing screws and gravity loading. The variation of each configuration is demonstrated in Table C-1. The complete set of testing details, recorded on individual test data sheets, is contained in the following pages with the corrected response curve (with the superimposed backbone and EEEP curves), summary design parameter table for each test and the failure mode observation record.

Specimen	Protocol	Sheathing Type	Sheathing Thickness (mm)	Chord Stud Thickness (mm)	Fastener[2] Schedule (mm)	Frame-to-Sheathing Fastener Type
47 - A,B,C[3]	Monotonic	DFP	12.5	1.37	75/305	No. 8 x 31.8 mm [4]
48 - A,B,C[3]	CUREE[1]	DFP	12.5	1.37	75/305	No. 8 x 31.8 mm [4]
49 - A,B[3],C,D	Monotonic	OSB	11	1.09	152/305	No. 8 x 38.1 mm [5]
50 - A,B,C[3]	CUREE	OSB	11	1.09	152/305	No. 8 x 38.1 mm [5]
51 - A,B[3],C	Monotonic	OSB	11	1.37	75/305	No. 8 x 38.1 mm [5]
52 - A,B,C[3]	CUREE	OSB	11	1.37	75/305	No. 8 x 38.1 mm [5]
53 - A,B,C[3]	Monotonic	CSP	12.5	1.09	152/305	No. 8 x 38.1 mm [5]
54 - A,B,C[3]	CUREE	CSP	12.5	1.09	152/305	No. 8 x 38.1 mm [5]
55 - A,B[3],C,D	Monotonic	CSP	12.5	1.37	75/305	No. 8 x 31.8 mm [4]
56 - A[3],B,C	CUREE	CSP	12.5	1.37	75/305	No. 8 x 31.8 mm [4]

[1] CUREE reversed cyclic protocol for ordinary ground motions (Krawinkler et al. 2000;

[2] Fastener schedule (ie.75/305) refers to the approximate spacing in millimetres between the sheathing to framing screws along the panel perimeter and the field spacing, respectively.

[3] Test did not include gravity loading.

[4] No. 8 x 31.8 mm (1-1/4") Grabber SuperDrive (SuperDrive, 2003) Bugle head self-drilling sheathing screws

[5] No. 8 x 38.1 mm (1-1/2") Grabber SuperDrive (SuperDrive, 2003) Bugle head self-piercing sheathing screws

Table C-1: Matrix of variables for shear wall tests

	Parameters	Units
F_u	37.93	kN
$F_{0.8u}$	30.34	kN
$F_{0.4u}$	15.17	kN
F_y	31.53	kN
K_e	1.42	kN/mm
Ductility (μ)	2.75	-
$\Delta_{net,y}$	22.16	mm
$\Delta_{net,u}$	60.96	mm
$\Delta_{net,0.8u}$	60.96	mm
$\Delta_{net,0.4u}$	10.66	mm
$Area_{Backbone}$	1572.96	J
$Area_{EEEP}$	1572.96	J
Check	OK	
R_d	**2.12**	-
S_y	**25.86**	kN/m
	2.5% Drift Limit Controls	

Test Name	47A
Date Tested	June 1, 2005
Protocol	MONOTONIC

McGill

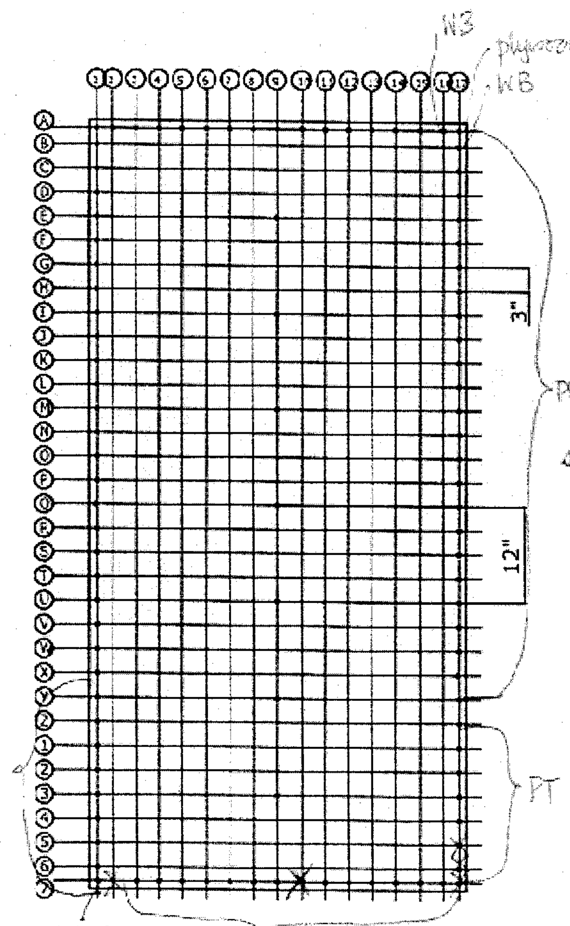
Light Gauge Steel Frame / Wood Panel Shear Wall Testing

Test name : 47A
Date tested : June 1 2005
Wall Size : 4' x 8
Screw pattern : 3" / 12'
Edge Distance : 1/2"
Test mode : ☐ Cyclic ☒ Monotonic

Moisture Content: A 22.76 → 21.02
B: 24.04 → 22.21

Max. Displacement 103.01 mm
Max Load 38.425 kN

X = sheared screw



real high compression load in steel back to back cause channels to move apart.

Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Light Gauge Steel Frame / Wood Panel Shear Walls
McGill University, Montreal

TEST: 47B

RESEARCHER: KATHERINE HIKITA **ASSISTANTS:** M. OUELLET, JIANG FAN

DATE: Built: May 31; Tested: Wednesday June 1, 2005 **TIME:** 17:00

DIMENSIONS OF WALL: 4 FT X 8 FT **PANEL ORIENTATION:** Vertical
Sheathing one side

SHEATHING:
- [ ] Plywood 15/32" APA Rated Exposure 1 (USA)
- [ ] OSB 7/16" APA Rated Exposure 1 (USA)
- [ ] Plywood (CSA 0151M) CSP 12.5mm (1/2") — CAN/PLY Exterior CSP
- [X] Plywood (CSA O121M) DFP 12.5mm (1/2") — CAN/PLY Exterior DFP
- [ ] OSB (CSA O325) 11 mm (7/16")
- [ ] Other — MFR: WELDWOOD , MILL BC 480

SCREWS Sheathing:
- [ ] No.8 gauge 1.5" self-piercing Bugle head LOX drive (Grabber Superdrive)
- [ ] No.8 gauge 1.0" self-piercing Bugle head (Flat socket head screw) (HD)
- [X] No.8 gauge 1.25" self-drilling Bugle head LOX Drive (Grabber Superdrive)
- [ ] No.9 gauge 1.0" self-piercing Bugle head (screws Y2,Y12 in track)

Framing: [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive
Hold downs: [X] No.10 gauge 0.75" self-drilling Hex washer head
Loading Beam: A325 3/4" bolts — 3 bolts [ ] 6 bolts [X] 12 bolts [ ]
Back-to-Back Chord Studs: [X] No.10 gauge 0.75" self-drilling Hex washer head (2@12" O.C.)

SHEATHING FASTENER SCHEDULE: [ ] 2"/12" [X] 3"/12" [ ] 4"/12" [ ] 6"/12" [ ] Other:

EDGE PANEL DISTANCE: [ ] 3/8" [X] 1/2" [ ] Other:

STUDS: Field / Chord
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MPa)
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MPa)
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.054" (1.37mm) 50ksi (340 MPa)
- [X] Double chord studs used
- [ ] Other

STUD SPACING:
- [ ] 12" O.C.
- [ ] 16" O.C.
- [X] 24" O.C.
- [ ] Other:

TRACK: Web: 3-5/8" inches; Flange: 1-1/4" inches — [X] 0.043" (1.09mm) 33ksi (230 MPa) [ ] Other:

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10 7/8" Anchor Rod (# of screws): 33
- [ ] UCI 18" hold down 1/2" Anchor Rod (# of screws):
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [X] Monotonic 7.5mm/min.
- [ ] Cyclic CUREE reversed cyclic (10 mm/s)
- [X] Gravity 9 kN load at both ends of wall

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral
- [X] East Frame Brace
- [X] West Frame Brace
- [X] Wood Shear

TOTAL: 9

MOISTURE CONTENT OF SHEATHING: Moisture Meter

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

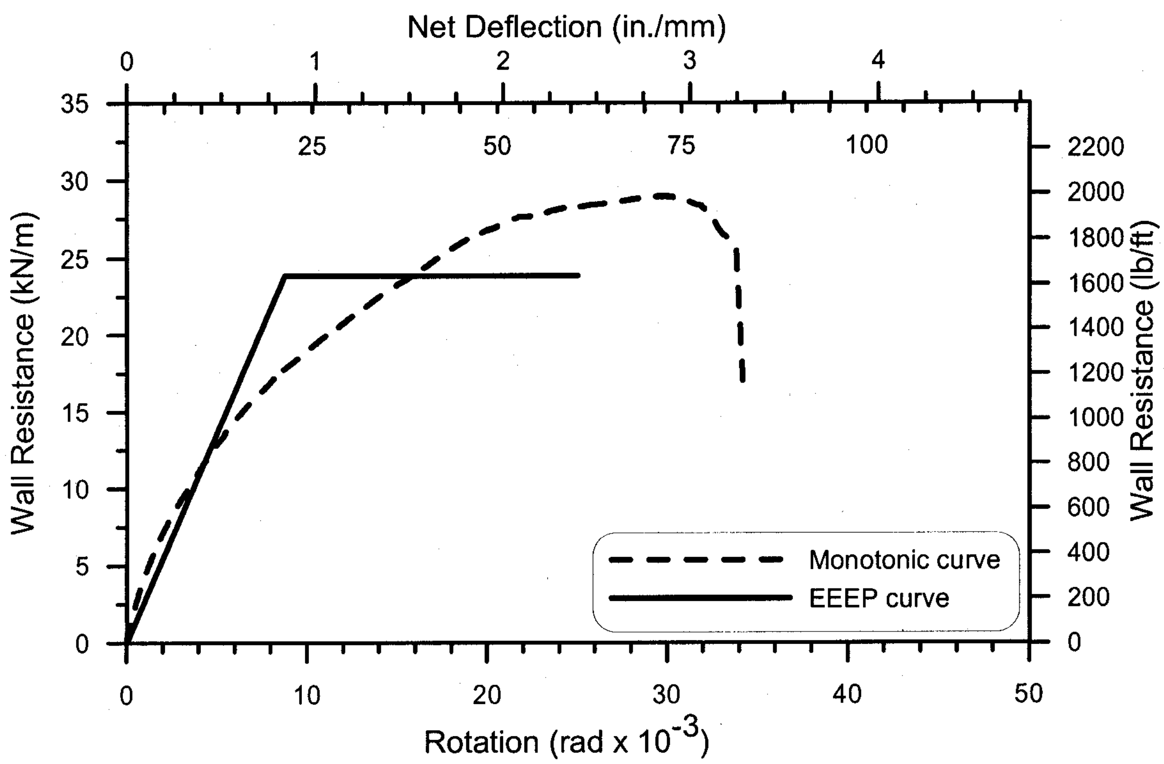
	North	North
Ww=	23.11	23.35
Wd=	21.33	21.70
m.c.=	8.35	7.60

AVG m.c. 7.97

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 50 scan/sec

COMMENTS:
-Shear anchors torqued to 77 kN
-Hold down anchors tightened to approximately 8 kN (load cells used on both hold-downs)
-Ambient temperature 23 C
-Double chord studs used
-Square plate washers (2.5"x2.5") used in all track connections
-Pilot holes drilled for screws A1,A5,A9,Q1,Q5,Q9
-Initial load set to zero at beginning of test, 0.492 mm displacement
- Offset due to Gravity Loads included in initial offset

Test 47B
(1220 x 2440 mm DFP 75/305 mm)



	Parameters	Units
F_u	35.33	kN
$F_{0.8u}$	28.26	kN
$F_{0.4u}$	14.13	kN
F_y	29.09	kN
K_e	1.37	kN/mm
Ductility (μ)	2.87	-
$\Delta_{net,y}$	21.24	mm
$\Delta_{net,u}$	60.96	mm
$\Delta_{net,0.8u}$	60.96	mm
$\Delta_{net,0.4u}$	10.31	mm
$Area_{Backbone}$	1464.53	J
$Area_{EEEP}$	1464.53	J
Check	OK	
R_d	**2.18**	-
S_y	**23.86**	kN/m
	2.5% Drift Limit Controls	

Test Name	47B
Date Tested	June 1, 2005
Protocol	MONOTONIC

McGill

Gravity load

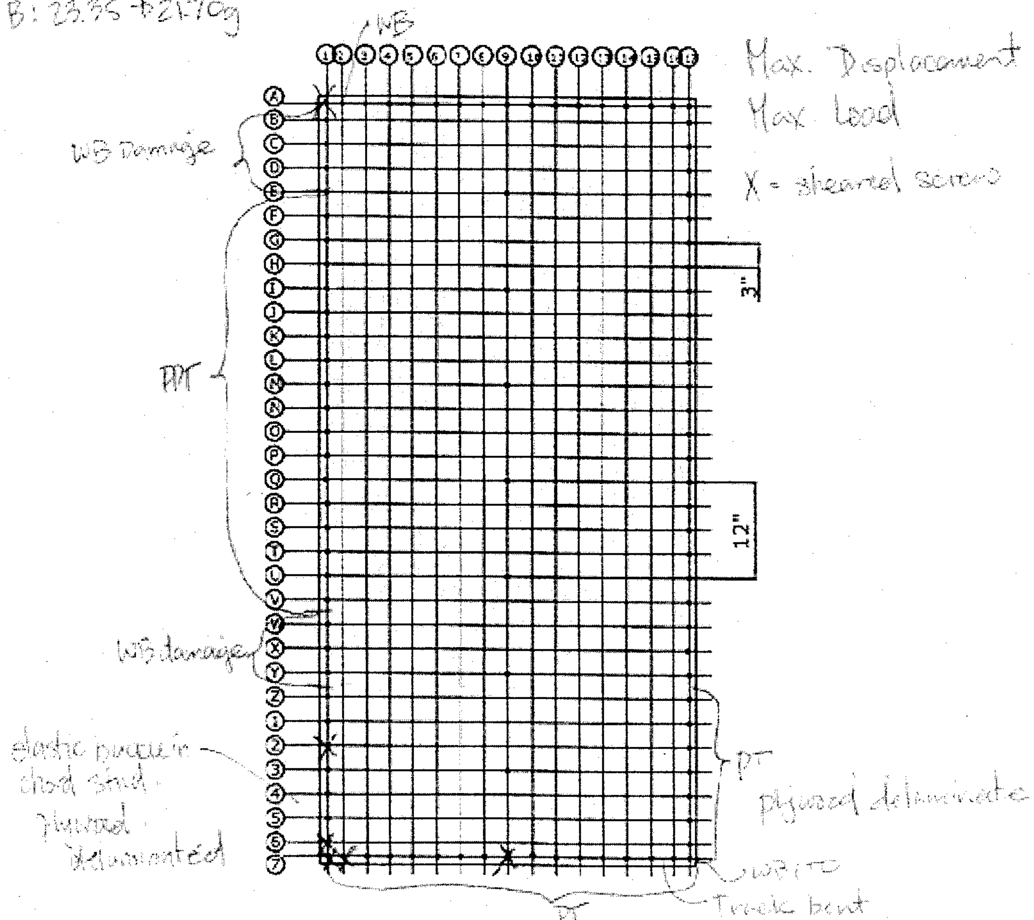
Light Gauge Steel Frame / Wood Panel Shear Wall Testing

Moisture Content: A: 23.11 → 21.33g
B: 23.35 → 21.70g

Test name : 97B
Date tested : June 2005
Wall Size : 4' x 8'
Screw pattern : 3" / 12"
Edge Distance : 1/2"
Test mode : ☒ Cyclic ☐ Monotonic

Max. Displacement 103.49mm
Max. Load 35.845kN

X = sheared screw



Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Light Gauge Steel Frame / Wood Panel Shear Walls
McGill University, Montreal

TEST:	47C		
RESEARCHER:	KATHERINE HIKITA	**ASSISTANTS:**	M. OUELLET, JIANG FAN, M. ALKHARAT D. MORELLO
DATE:	Built: May 31; Tested: Thursday June 2, 2005	**TIME:**	11:00
DIMENSIONS OF WALL:	4 FT X 8 FT	**PANEL ORIENTATION:**	Vertical Sheathing one side

SHEATHING:
- [ ] Plywood 15/32" APA Rated Exposure 1 (USA)
- [ ] OSB 7/16" APA Rated Exposure 1 (USA)
- [ ] Plywood (CSA 0151M) CSP 12.5mm (1/2") — CAN/PLY Exterior CSP
- [X] Plywood (CSA O121M) DFP 12.5mm (1/2") — CAN/PLY Exterior DFP
- [ ] OSB (CSA O325) 11 mm (7/16")
- [ ] Other — MFR: WELDWOOD , MILL BC 480

SCREWS Sheathing:
- [ ] No.8 gauge 1.5" self-piercing Bugle head LOX drive (Grabber Superdrive)
- [ ] No.8 gauge 1.0" self-piercing Bugle head (Flat socket head screw) (HD)
- [X] No.8 gauge 1.25" self-drilling Bugle head LOX Drive (Grabber Superdrive)
- [ ] No.9 gauge 1.0" self-piercing Bugle head (screws Y2,Y12 in track)

Framing: [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive

Hold downs: [X] No.10 gauge 0.75" self-drilling Hex washer head

Loading Beam: A325 3/4" bolts — 3 bolts [ ] 6 bolts [X] 12 bolts [ ]

Back-to-Back Chord Studs: [X] No.10 gauge 0.75" self-drilling Hex washer head (2@12" O.C.)

SHEATHING FASTENER SCHEDULE: [ ] 2"/12" [X] 3"/12" [ ] 4"/12" [ ] 6"/12" [ ] Other:

EDGE PANEL DISTANCE: [ ] 3/8" [X] 1/2" [ ] Other:

STUDS: Field / Chord
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MPa)
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MPa)
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.054" (1.37mm) 50ksi (340 MPa)
- [X] Double chord studs used
- [ ] Other

STUD SPACING:
- [ ] 12" O.C.
- [ ] 16" O.C.
- [X] 24" O.C.
- [ ] Other:

TRACK: Web: 3-5/8" inches; Flange: 1-1/4" inches — [X] 0.043" (1.09mm) 33ksi (230 MPa) [ ] Other:

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10 7/8" Anchor Rod — (# of screws): 33
- [ ] UCI 18" hold down 1/2" Anchor Rod — (# of screws):
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [X] Monotonic — 7.5mm/min.
- [ ] Cyclic — CUREE reversed cyclic (10 mm/s)
- [ ] Gravity — 9 kN load at both ends of wall

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral
- [X] East Frame Brace
- [X] West Frame Brace
- [X] Wood Shear

TOTAL: 9

MOISTURE CONTENT OF SHEATHING: Moisture Meter

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

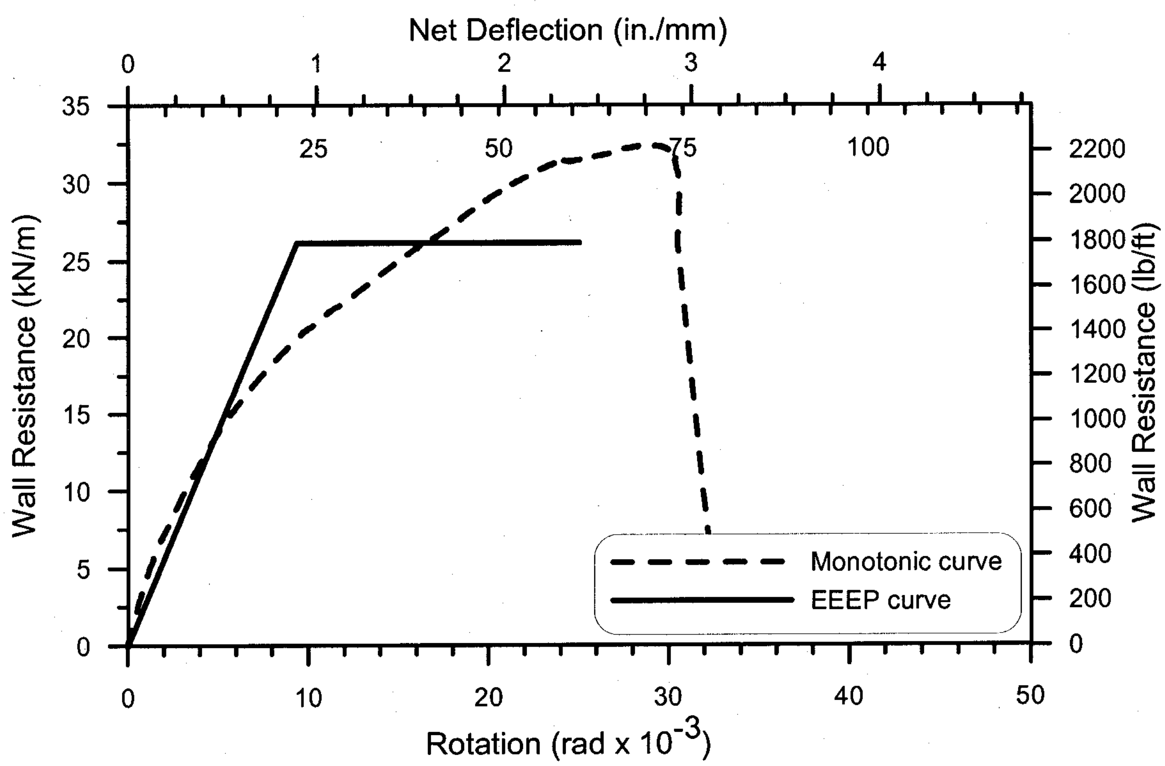
	North	North
Ww=	27.02	24.98
Wd=	25.05	23.12
m.c.=	7.86	8.04

AVG m.c. 7.95

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 50 scan/sec

COMMENTS:
- -Shear anchors torqued to 77 kN
- -Hold down anchors tightened to approximately 8 kN (load cells used on both hold-downs)
- -Ambient temperature 23 C
- -Double chord studs used
- -Square plate washers (2.5"x2.5") used in all track connections
- -Pilot holes drilled for screws A1,A5,A9,Q1,Q5,Q9
- -Initial load set to zero at beginning of test, 0.152 mm displacement
- - No gravity loads included

Test 47C
(1220 x 2440 mm DFP 75/305 mm)



	Parameters	Units
F_u	39.50	kN
$F_{0.8u}$	31.60	kN
$F_{0.4u}$	15.80	kN
F_y	31.87	kN
K_e	1.40	kN/mm
Ductility (μ)	2.69	-
$\Delta_{net,y}$	22.69	mm
$\Delta_{net,u}$	60.96	mm
$\Delta_{net,0.8u}$	60.96	mm
$\Delta_{net,0.4u}$	11.25	mm
$Area_{Backbone}$	1581.44	J
$Area_{EEEP}$	1581.44	J
Check	OK	
R_d	**2.09**	-
S_y	**26.14**	kN/m
	2.5% Drift Limit Controls	

Test Name	47C
Date Tested	June 2, 2005
Protocol	MONOTONIC

McGill *No gravity loads

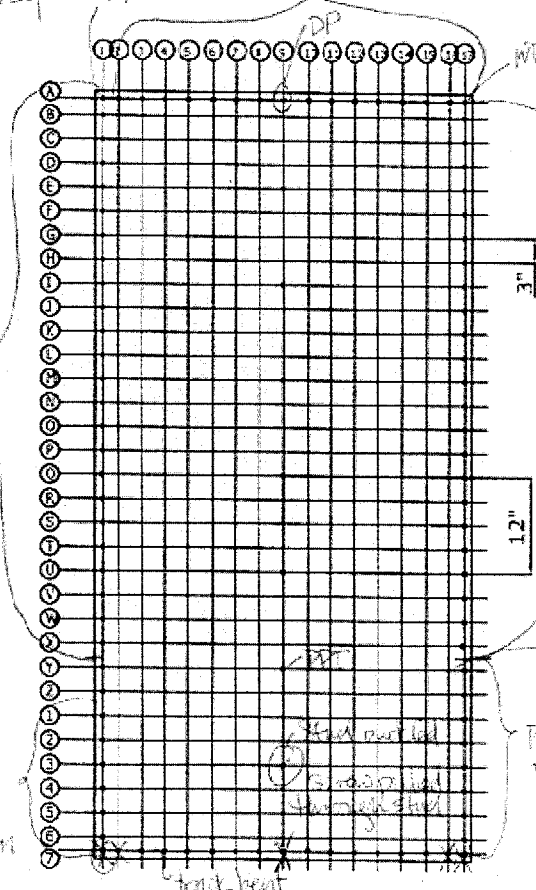
Light Gauge Steel Frame / Wood Panel Shear Wall Testing

Moisture Content: A: 27.02 → 25.05g
B: 24.98 → 23.12g

Test name : 47C
Date tested : June 2, 2005
Wall Size : 4' x 8'
Screw pattern : 3" / 12"
Edge Distance : 1/2"
Test mode : ☐ Cyclic ☒ Monotonic

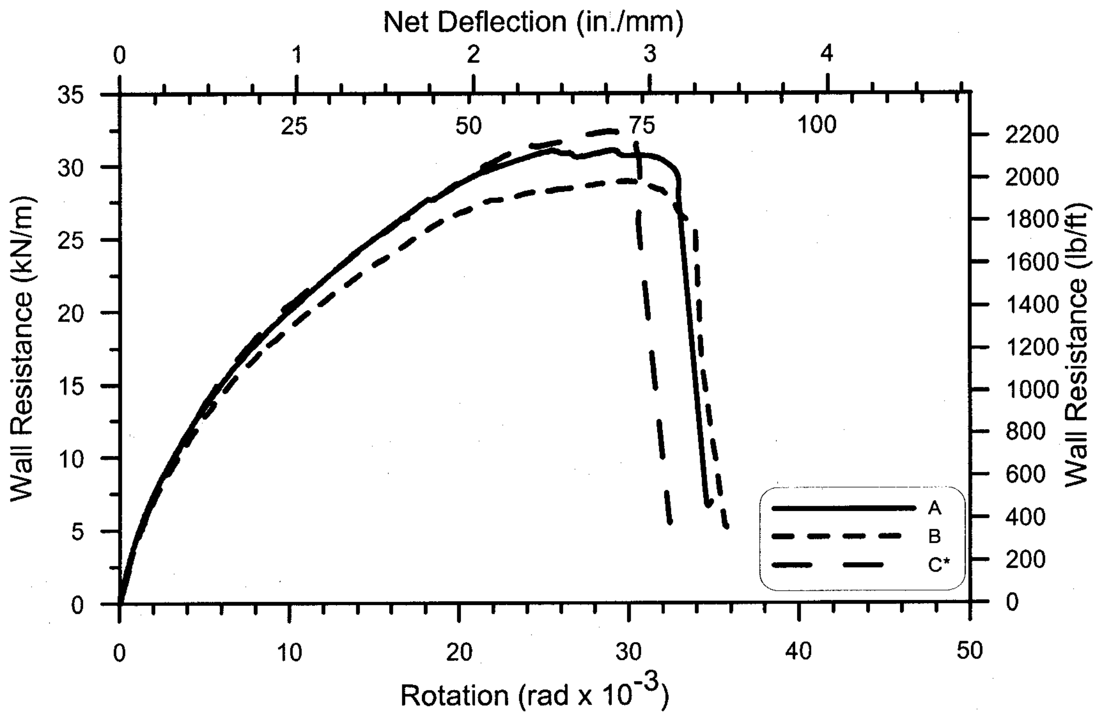
Max. Displacement 103.29mm
Max. Load 39.5kN

X = sheared screw
- south shear, north shear popped off at final big bang
- plywood bowed out from frame.

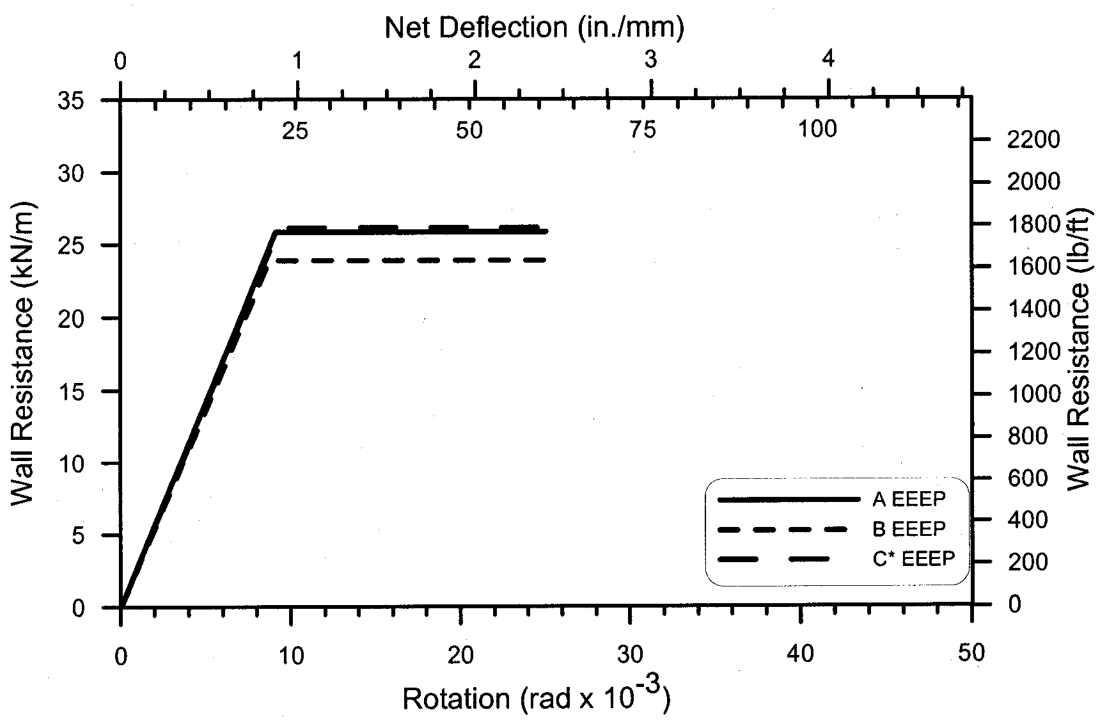


Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) , Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Test 47-A,B,C
(1220 x 2440 mm DFP 75/305 mm)



Test 47-A,B,C EEEP
(1220 x 2440 mm DFP 75/305 mm)



Light Gauge Steel Frame / Wood Panel Shear Walls
McGill University, Montreal

TEST: 48A

RESEARCHER: KATHERINE HIKITA **ASSISTANTS:** M. OUELLET, JIANG FAN, M. ALKHARAT, D. MORELLO

DATE: Built: May 31; Tested: Friday June 3, 2005 **TIME:** 11:00

DIMENSIONS OF WALL: 4 FT X 8 FT **PANEL ORIENTATION:** Vertical; Sheathing one side

SHEATHING:
- [ ] Plywood 15/32" APA Rated Exposure 1 (USA)
- [ ] OSB 7/16" APA Rated Exposure 1 (USA)
- [ ] Plywood (CSA 0151M) CSP 12.5mm (1/2") — CAN/PLY Exterior CSP
- [X] Plywood (CSA O121M) DFP 12.5mm (1/2") — CAN/PLY Exterior DFP
- [ ] OSB (CSA O325) 11 mm (7/16")
- [ ] Other — MFR: WELDWOOD , MILL BC 480

SCREWS Sheathing:
- [ ] No.8 gauge 1.5" self-piercing Bugle head LOX drive (Grabber Superdrive)
- [ ] No.8 gauge 1.0" self-piercing Bugle head (Flat socket head screw) (HD)
- [X] No.8 gauge 1.25" self-drilling Bugle head LOX Drive (Grabber Superdrive)
- [ ] No.9 gauge 1.0" self-piercing Bugle head (screws Y2,Y12 in track)

Framing: [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive

Hold downs: [X] No.10 gauge 0.75" self-drilling Hex washer head

Loading Beam: A325 3/4" bolts — 3 bolts [ ] 6 bolts [X] 12 bolts [ ]

Back-to-Back Chord Studs: [X] No.10 gauge 0.75" self-drilling Hex washer head (2@12" O.C.)

SHEATHING FASTENER SCHEDULE: [ ] 2"/12" [X] 3"/12" [ ] 4"/12" [ ] 6"/12" [ ] Other:

EDGE PANEL DISTANCE: [ ] 3/8" [X] 1/2" [ ] Other:

STUDS: Field / Chord
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MPa)
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MPa)
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.054" (1.37mm) 50ksi (340 MPa)
- [X] Double chord studs used
- [ ] Other

STUD SPACING:
- [ ] 12" O.C.
- [ ] 16" O.C.
- [X] 24" O.C.
- [ ] Other:

TRACK: Web: 3-5/8" inches; Flange: 1-1/4" inches — [X] 0.043" (1.09mm) 33ksi (230 MPa) [ ] Other:

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10 7/8" Anchor Rod — (# of screws): 33
- [ ] UCI 18" hold down 1/2" Anchor Rod — (# of screws):
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [ ] Monotonic — 7.5mm/min.
- [X] Cyclic — CUREE reversed cyclic (10 mm/s)
- [X] Gravity — 9 kN load at both ends of wall

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral
- [X] East Frame Brace
- [X] West Frame Brace
- [X] Wood Shear

TOTAL: 9

MOISTURE CONTENT OF SHEATHING: Moisture Meter

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

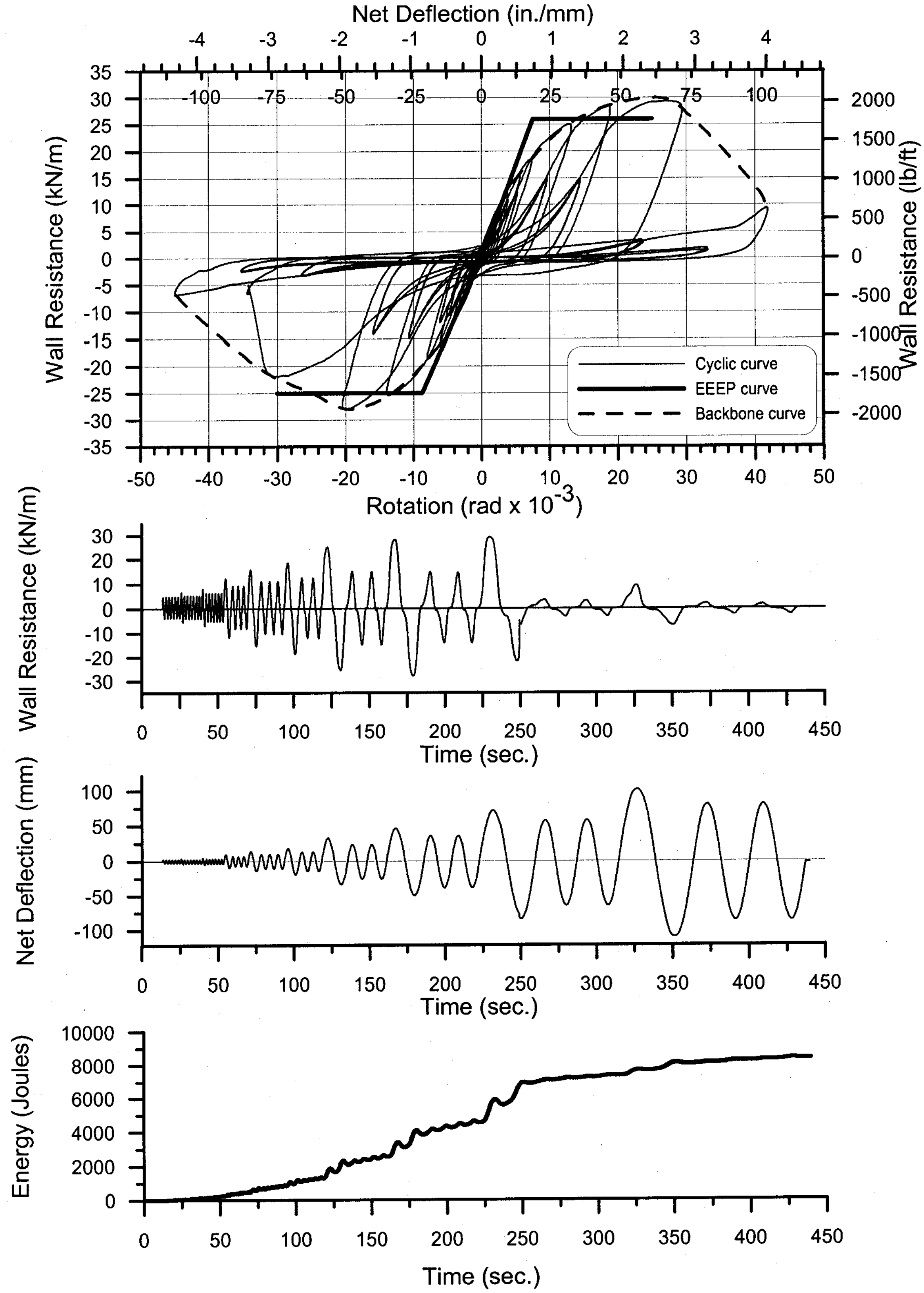
Ww=	26.31	25.42
Wd=	24.65	23.70
m.c.=	6.73	7.26
	North	North

AVG m.c. 7.00

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 50 scan/sec

COMMENTS:
- -Shear anchors torqued to 77 kN
- -Hold down anchors tightened to approximately 8 kN (load cells used on both hold-downs)
- -Ambient temperature 23 C
- -Double chord studs used
- -Square plate washers (2.5"x2.5") used in all track connections
- -Pilot holes drilled for screws A1,A5,A9,Q1,Q5,Q9
- -Initial load set to zero at beginning of test, -0.333 mm displacement
- - gravity loads included

Test 48A
(1220 x 2440 mm DFP 75/305 mm)



	Negative	Positive	Units
F_u	-34.12	35.67	kN
$F_{0.8u}$	-27.29	28.54	kN
$F_{0.4u}$	-13.65	14.27	kN
F_y	-30.49	31.65	kN
K_e	1.44	1.72	kN/mm
Ductility (μ)	3.45	3.31	-
$\Delta_{net,y}$	-21.23	18.41	mm
$\Delta_{net,u}$	-49.09	60.96	mm
$\Delta_{net,0.8u}$	-73.20	60.96	mm
$\Delta_{net,0.4u}$	-9.50	8.30	mm
$Area_{Backbone}$	1908.46	1638.04	kN-mm
$Area_{EEEP}$	1908.46	1638.04	kN-mm
Check	OK	OK	
R_d	**2.43**	**2.37**	-
S_y	**-25.01**	**25.96**	kN/m
		2.5% Drift Limit Controls	

Test Name	48A
Date Tested	June 3, 2005
Protocol	CUREE ORDINARY GROUND MOTIONS

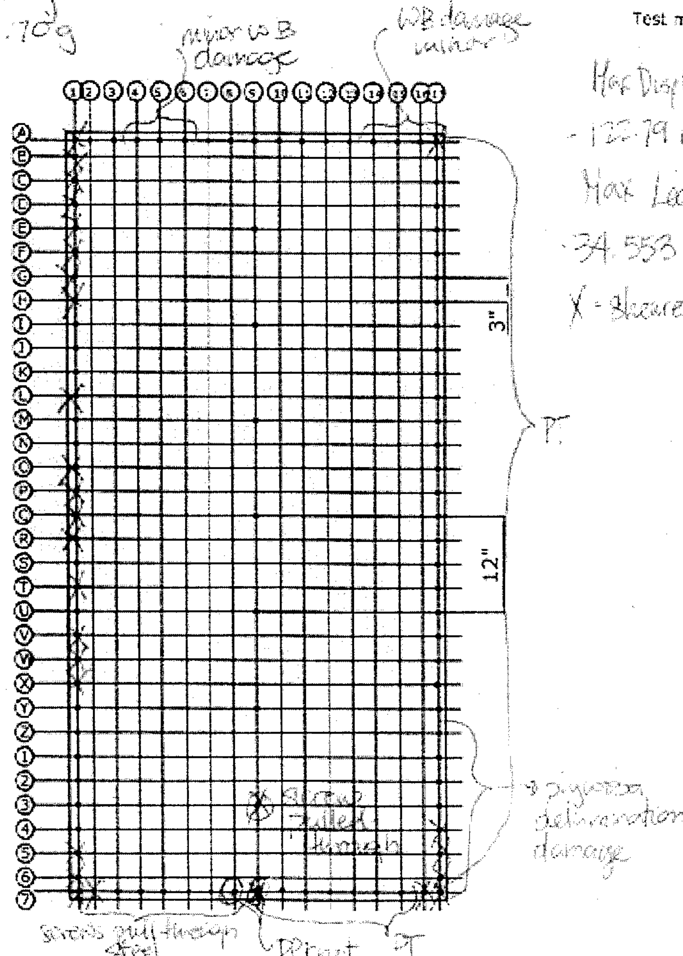
McGill Gravity loads Included

Light Gauge Steel Frame / Wood Panel Shear Wall Testing

Moisture Content: A 26.31g → 24.65g
B 25.42g → 23.70g

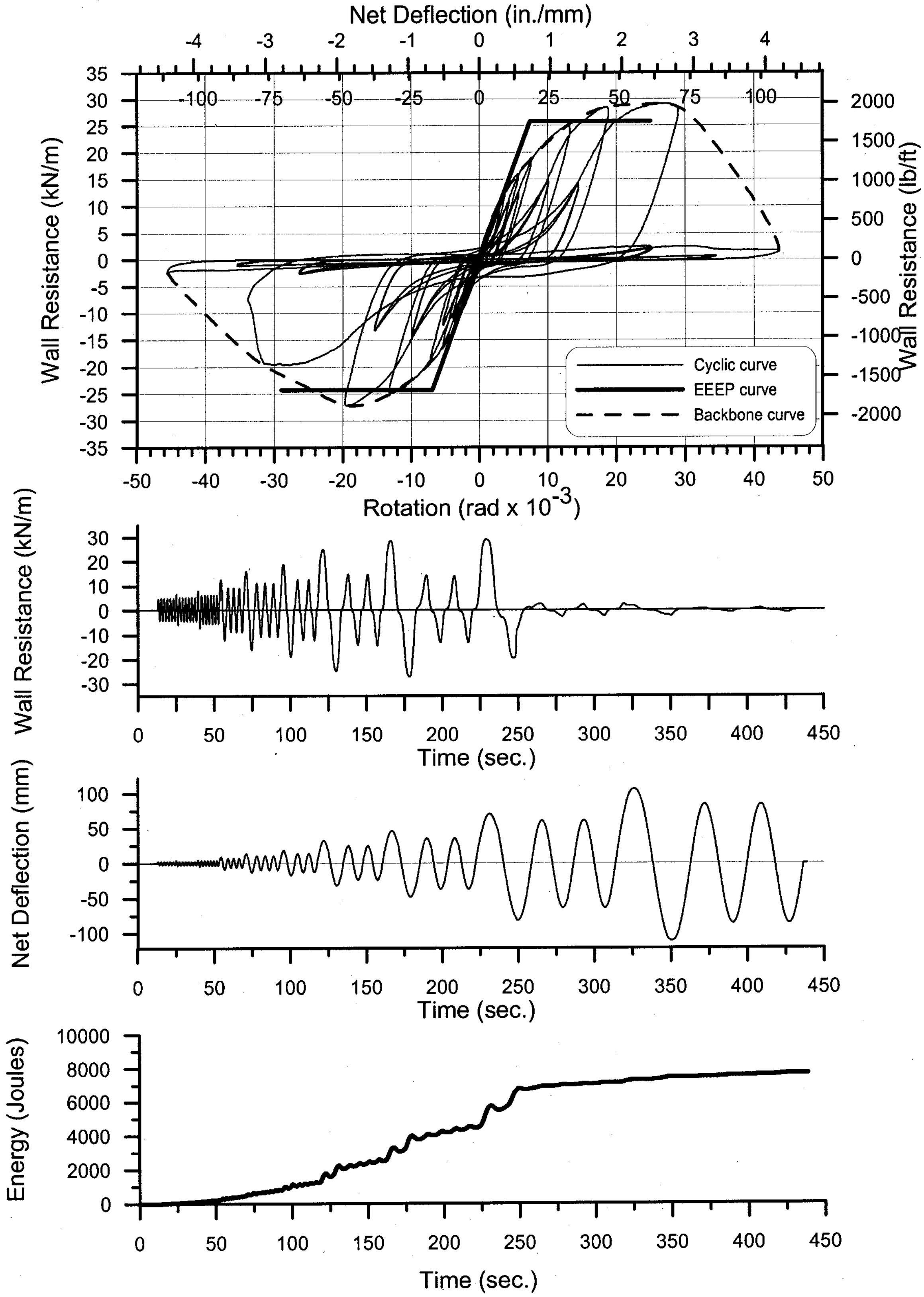
Test name : 48A
Date tested : June 3, 2005
Wall Size : 4' x 8'
Screw pattern : 3" / 12"
Edge Distance : 1/2"
Test mode : ☒ Cyclic ☐ Monotonic

Max Displacement
-122.79 mm 122.76 mm
Max Load
-34.553 kN 36.239 kN
X = Sheared screw



Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Test 48B
(1220 x 2440 mm DFP 75/305 mm)



	Negative	Positive	Units
F_u	-33.16	35.53	kN
$F_{0.8u}$	-26.53	28.43	kN
$F_{0.4u}$	-13.26	14.21	kN
F_y	-29.58	31.39	kN
K_e	1.77	1.73	kN/mm
Ductility (μ)	4.22	3.37	-
$\Delta_{net,y}$	-16.73	18.11	mm
$\Delta_{net,u}$	-47.25	60.96	mm
$\Delta_{net,0.8u}$	-70.60	60.96	mm
$\Delta_{net,0.4u}$	-7.50	8.20	mm
$Area_{Backbone}$	1841.05	1629.16	kN-mm
$Area_{EEEP}$	1841.05	1629.16	kN-mm
Check	OK	OK	
R_d	**2.73**	**2.39**	-
S_y	**-24.26**	**25.74**	kN/m
		2.5% Drift Limit Controls	

Test Name	48B
Date Tested	June 3, 2005
Protocol	CUREE ORDINARY GROUND MOTIONS

McGill

Light Gauge Steel Frame / Wood Panel Shear Wall Testing

Test name : 48B
Date tested : June 8, 2005
Wall Size : 4 x 8'
Screw pattern : 3" / 12"
Edge Distance : 1/2"
Test mode : ☒ Cyclic ☐ Monotonic

Moisture Content A: 24.40g → 22.68g
B: 25.16g → 23.43g

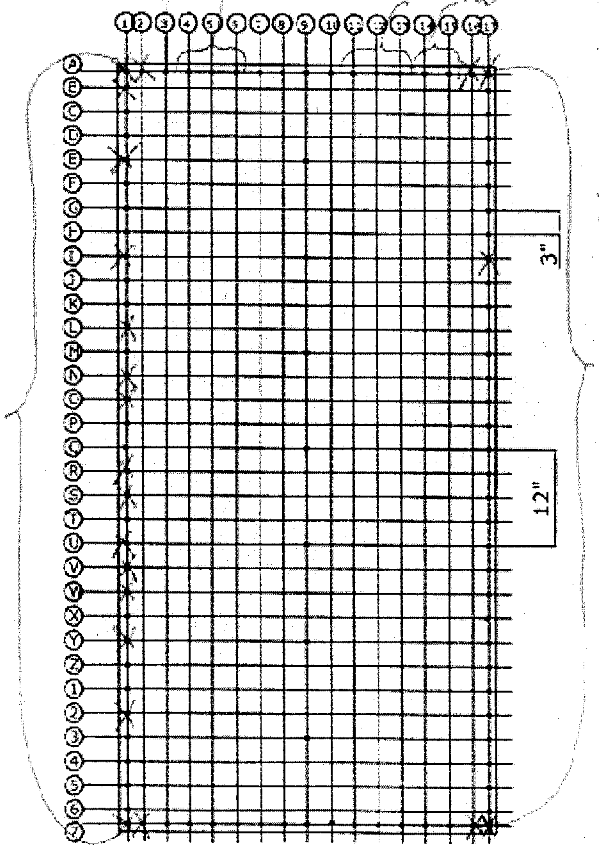
PPT

screws pulled through wood

WB WB

Displacements (mm)
-122.78 122.76
Loads (kN)
-33.590 36.104

X = sheared screw



PT WOOD DELAMINATED.

PT wood delaminated

Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Light Gauge Steel Frame / Wood Panel Shear Walls
McGill University, Montreal

TEST: 48C

RESEARCHER: KATHERINE HIKITA **ASSISTANTS:** M. OUELLET, JIANG FAN, M. ALKHARAT, D. MORELLO

DATE: Built: May 31; Tested: Friday June 3, 2005 **TIME:** 15:00

DIMENSIONS OF WALL: 4 FT X 8 FT **PANEL ORIENTATION:** Vertical; Sheathing one side

SHEATHING:
- [ ] Plywood 15/32" APA Rated Exposure 1 (USA)
- [ ] OSB 7/16" APA Rated Exposure 1 (USA)
- [ ] Plywood (CSA 0151M) CSP 12.5mm (1/2") — CAN/PLY Exterior CSP
- [X] Plywood (CSA O121M) DFP 12.5mm (1/2") — CAN/PLY Exterior DFP
- [ ] OSB (CSA O325) 11 mm (7/16")
- [ ] Other — MFR: WELDWOOD , MILL BC 480

SCREWS

Sheathing:
- [ ] No.8 gauge 1.5" self-piercing Bugle head LOX drive (Grabber Superdrive)
- [ ] No.8 gauge 1.0" self-piercing Bugle head (Flat socket head screw) (HD)
- [X] No.8 gauge 1.25" self-drilling Bugle head LOX Drive (Grabber Superdrive)
- [ ] No.9 gauge 1.0" self-piercing Bugle head (screws Y2,Y12 in track)

Framing: [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive

Hold downs: [X] No.10 gauge 0.75" self-drilling Hex washer head

Loading Beam: A325 3/4" bolts — 3 bolts [ ] 6 bolts [X] 12 bolts [ ]

Back-to-Back Chord Studs: [X] No.10 gauge 0.75" self-drilling Hex washer head (2@12" O.C.)

SHEATHING FASTENER SCHEDULE: [ ] 2"/12" [X] 3"/12" [ ] 4"/12" [ ] 6"/12" [ ] Other:

EDGE PANEL DISTANCE: [ ] 3/8" [X] 1/2" [ ] Other:

STUDS: Field / Chord
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MPa)
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MPa)
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.054" (1.37mm) 50ksi (340 MPa)
- [X] Double chord studs used
- [ ] Other

STUD SPACING:
- [ ] 12" O.C.
- [ ] 16" O.C.
- [X] 24" O.C.
- [ ] Other:

TRACK: Web: 3-5/8" inches; Flange: 1-1/4" inches
- [X] 0.043" (1.09mm) 33ksi (230 MPa)
- [ ] Other:

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10 7/8" Anchor Rod (# of screws): 33
- [ ] UCI 18" hold down 1/2" Anchor Rod (# of screws):
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [ ] Monotonic — 7.5mm/min.
- [X] Cyclic — CUREE reversed cyclic (10 mm/s)
- [ ] Gravity — 9 kN load at both ends of wall

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral
- [X] East Frame Brace
- [X] West Frame Brace
- [X] Wood Shear

TOTAL: 9

MOISTURE CONTENT OF SHEATHING: Moisture Meter

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

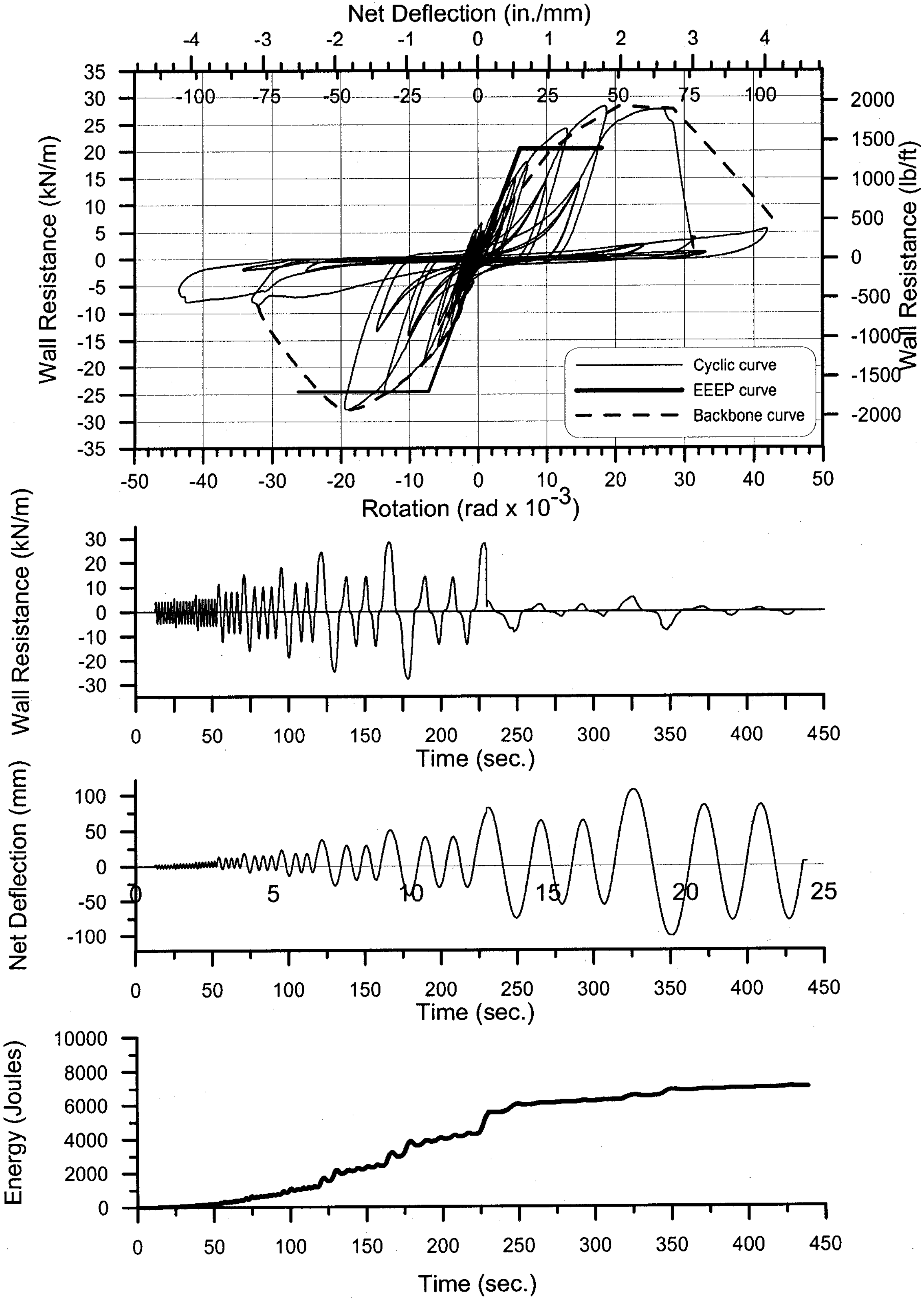
	North	North
Ww=	25.06	24.72
Wd=	23.45	23.02
m.c.=	6.87	7.38

AVG m.c. 7.13

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 50 scan/sec

COMMENTS:
- -Shear anchors torqued to 77 kN
- -Hold down anchors tightened to approximately 8 kN (load cells used on both hold-downs)
- -Ambient temperature 23 C
- -Double chord studs used
- -Square plate washers (2.5"x2.5") used in all track connections
- -Pilot holes drilled for screws A1,A5,A9,Q1,Q5,Q9
- -Initial load set to zero at beginning of test, 0.843 mm displacement
- - No gravity loads included

Test 48C
(1220 x 2440 mm DFP 75/305 mm)



	Negative	Positive	Units
F_u	-33.95	34.57	kN
$F_{0.8u}$	-27.16	27.65	kN
$F_{0.4u}$	-13.58	13.83	kN
F_y	-29.83	31.43	kN
K_e	1.70	1.19	kN/mm
Ductility (μ)	3.64	2.98	-
$\Delta_{net,y}$	-17.58	26.37	mm
$\Delta_{net,u}$	-46.68	50.05	mm
$\Delta_{net,0.8u}$	-63.90	78.60	mm
$\Delta_{net,0.4u}$	-8.00	11.60	mm
$Area_{Backbone}$	1644.08	2056.20	kN-mm
$Area_{EEEP}$	1644.08	2056.20	kN-mm
Check	OK	OK	
R_d	**2.50**	**2.23**	-
S_y	**-24.47**	**25.78**	kN/m

Test Name	48C
Date Tested	June 3, 2005
Protocol	CUREE ORDINARY GROUND MOTIONS

McGill

* No Gooch Loads

Light Gauge Steel Frame / Wood Panel Shear Wall Testing

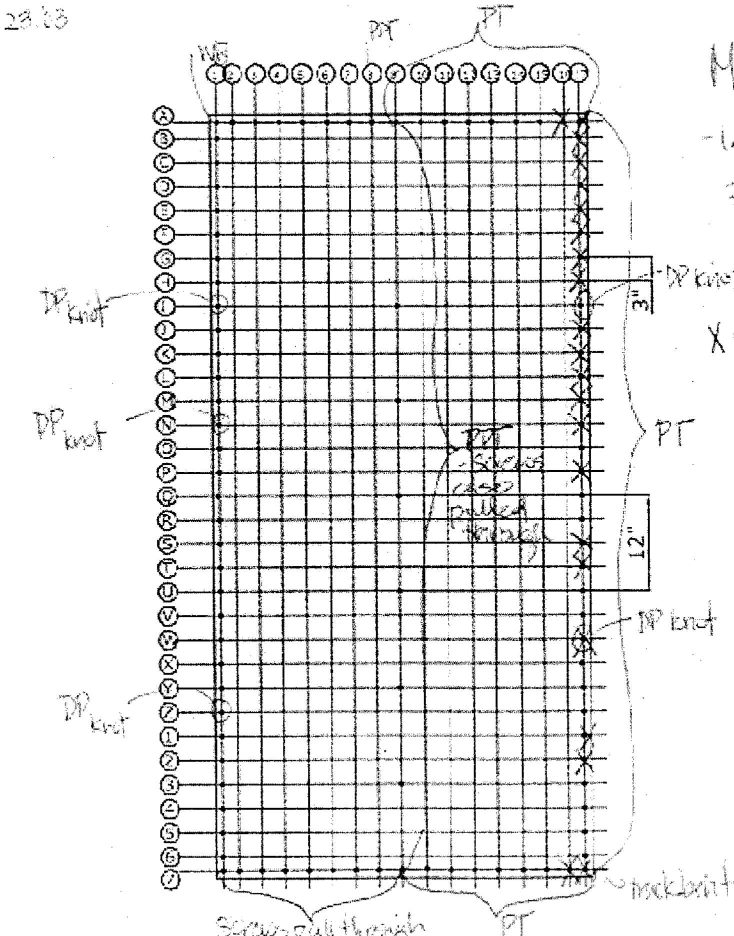
Moisture Content: A 25.06 23.15
B 24.72 23.63

Test name: 48C
Date tested: June 3 2015
Wall Size: 4' x 8'
Screw pattern: 3" / 12"
Edge Distance: 1/2"
Test mode: ☒ Cyclic ☐ Monotonic

Max Loads / Displacement

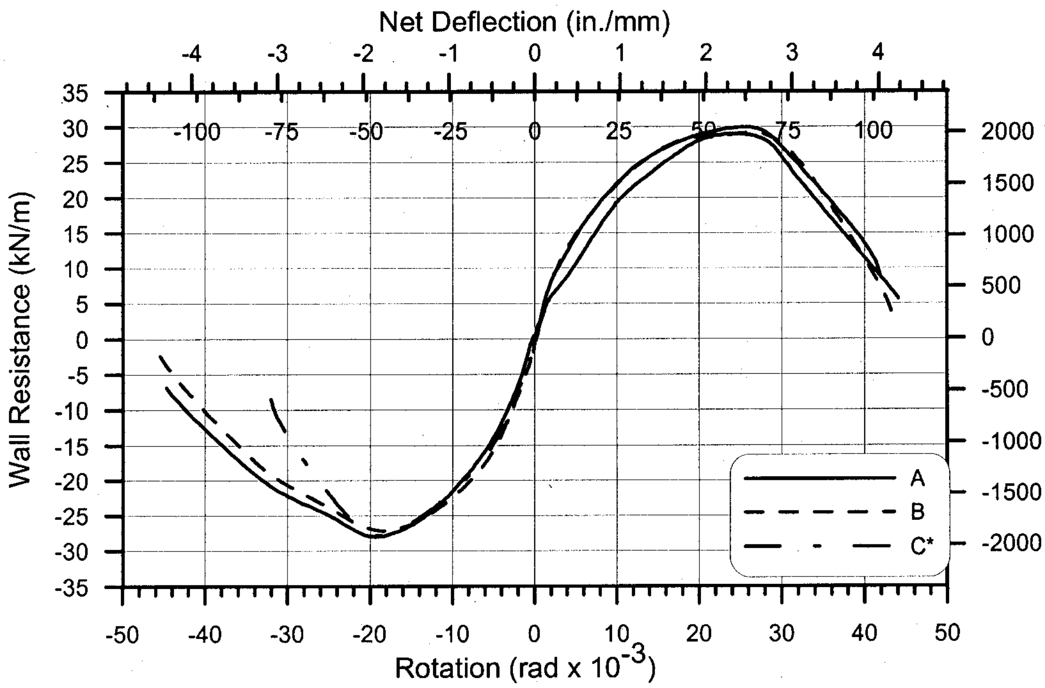
-122.79 122.76 mm
34.040 34.633 kN

X = screw sheared.

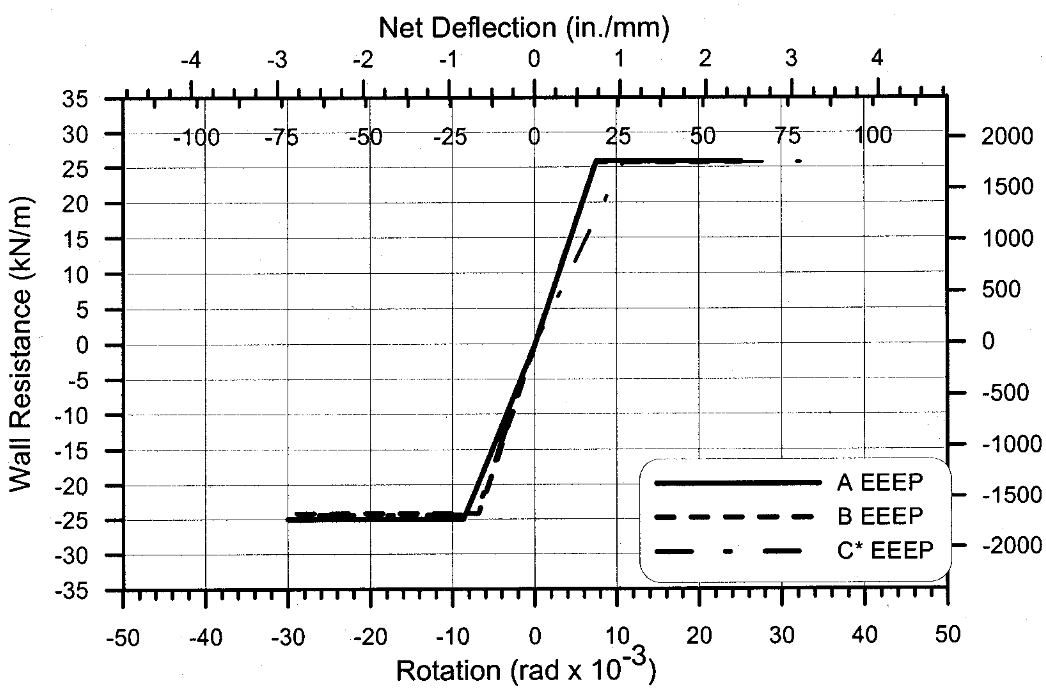


Failure modes: Pull out, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Test 48-A,B,C Backbone
(1220 x 2440 mm DFP 75/305 mm)



Test 48-A,B,C EEEP
(1220 x 2440 mm DFP 75/305 mm)



Light Gauge Steel Frame / Wood Panel Shear Walls
McGill University, Montreal

TEST: 49A

RESEARCHER: KATHERINE HIKITA **ASSISTANTS:** M. OUELLET, D. MORELLO, R. CARBONNEAU

DATE: Built: May 11; Tested: Friday May 13, 2005 **TIME:** 12:00

DIMENSIONS OF WALL: 4 FT X 8 FT **PANEL ORIENTATION:** Vertical, Sheathing one side

SHEATHING:
- [ ] Plywood 15/32" APA Rated Exposure 1 (USA)
- [ ] OSB 7/16" APA Rated Exposure 1 (USA)
- [ ] Plywood (CSA 0151M) CSP 12.5mm (1/2") — CAN/PLY Exterior CSP
- [ ] Plywood (CSA O121M) DFP 12.5mm (1/2") — CAN/PLY Exterior DFP
- [X] OSB (CSA O325) 11 mm (7/16")
- [ ] Other — MFR: Grant Forest Products Corp. Englehart, ON

SCREWS
Sheathing:
- [X] No.8 gauge 1.5" self-piercing Bugle head LOX drive (Grabber Superdrive)
- [ ] No.8 gauge 1.0" self-piercing Bugle head (Flat socket head screw) (HD)
- [ ] No.8 gauge 1.25" self-drilling Bugle head LOX Drive (Grabber Supe
- [ ] No.9 gauge 1.0" self-piercing Bugle head (screws Y2,Y12 in track)

Framing:
- [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive

Hold downs:
- [X] No.10 gauge 0.75" self-drilling Hex washer head

Loading Beam: A325 3/4" bolts — 3 bolts [ ] 6 bolts [X] 12 bolts [ ]

Back-to-Back Chord Studs:
- [X] No.10 gauge 0.75" self-drilling Hex washer head (2@12" O.C.)

SHEATHING FASTENER SCHEDULE: [ ] 2"/12" [ ] 3"/12" [ ] 4"/12" [X] 6"/12" [ ] Other:

EDGE PANEL DISTANCE: [ ] 3/8" [X] 1/2" [ ] Other:

STUDS: Field, Chord
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MPa)
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MPa)
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.054" (1.37mm) 50ksi (340 MPa)
- [X] Double chord studs used
- [ ] Other

STUD SPACING:
- [ ] 12" O.C.
- [ ] 16" O.C.
- [X] 24" O.C.
- [ ] Other:

TRACK: Web: 3-5/8" inches; Flange: 1-1/4" inches
- [X] 0.043" (1.09mm) 33ksi (230 MPa)
- [ ] Other:

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10 7/8" Anchor Rod (# of screws): 33
- [ ] UCI 18" hold down 1/2" Anchor Rod (# of screws):
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [X] Monotonic 7.5mm/min.
- [ ] Cyclic CUREE reversed cyclic (10 mm/s)
- [X] Gravity 9 kN load at both ends of wall

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral
- [X] East Frame Brace
- [X] West Frame Brace
- [X] Wood Shear

TOTAL: 9

MOISTURE CONTENT OF SHEATHING: Moisture Meter

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

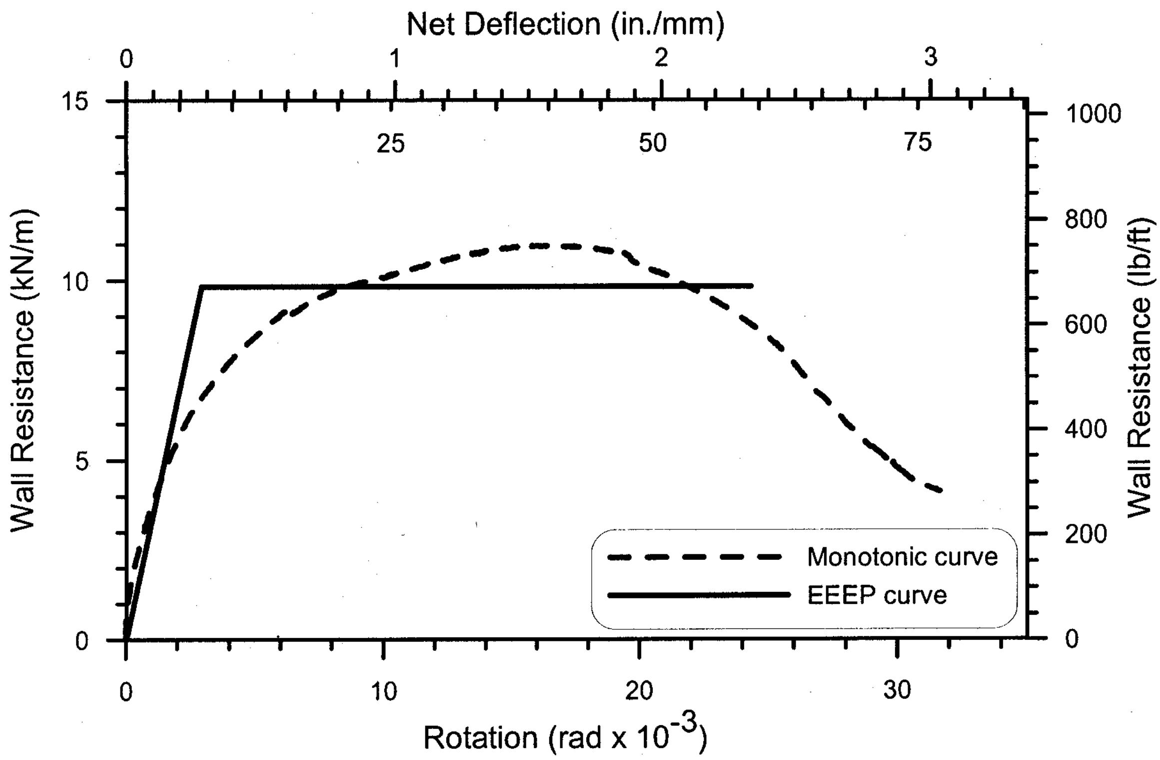
Ww=	28.61	29.94
Wd=	27.56	28.84
m.c.=	3.81	3.81
	North	North

AVG m.c. 3.81

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 50 scan/sec

COMMENTS:
- -Shear anchors torqued to 77 kN
- -Hold down anchors 1/2 turn from finger tight (load cells used on both hold-downs)
- -Ambient temperature 23 C
- -Double chord studs used
- -Square plate washers (2.5"x2.5") used in all track connections
- -Pilot holes drilled for screws A1,A5,A9,Q1,Q5,Q9
- -Initial load set to zero at beginning of test, displacement 0.007 mm
- -Offset after gravity loads engaged -0.128 mm
- - extra hole between A6 and A7 due to mismeasurement of screw spacing

Test 49A
(1220 x 2440 mm OSB 152/305 mm)



	Parameters	Units
F_u	13.37	kN
$F_{0.8u}$	10.70	kN
$F_{0.4u}$	5.35	kN
F_y	11.99	kN
K_e	1.69	kN/mm
Ductility (μ)	8.36	-
$\Delta_{net,y}$	7.09	mm
$\Delta_{net,u}$	38.41	mm
$\Delta_{net,0.8u}$	59.27	mm
$\Delta_{net,0.4u}$	3.16	mm
$Area_{Backbone}$	668.13	J
$Area_{EEEP}$	668.13	J
Check	OK	
R_d	**3.96**	-
S_y	**9.83**	kN/m

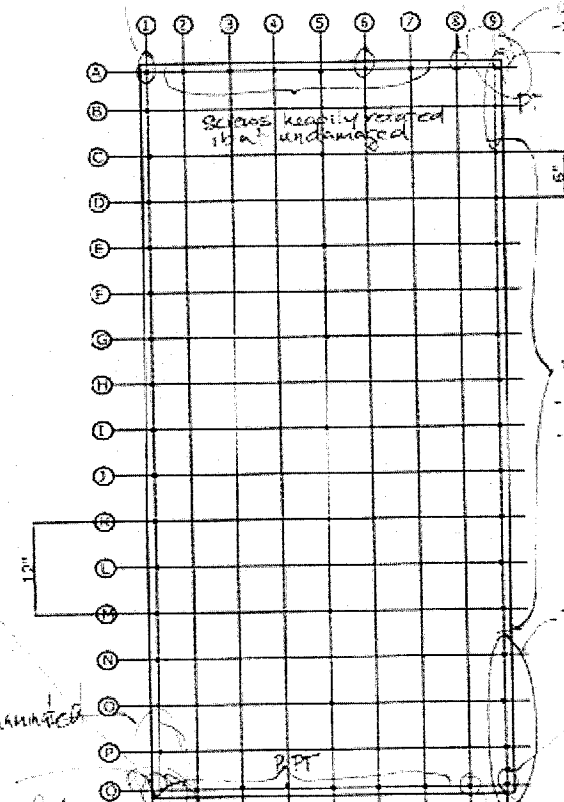
Test Name	49A
Date Tested	June 2, 2005
Protocol	MONOTONIC

McGill

Cold-Formed Steel Stud Shear Walls Testing

Moisture Content A 28.61 27.58
B 29.94 28.84

Test name : 49A
Date tested : May 13, 2005
Wall Size : 4' x 8'
Screw pattern : 6" / 12"
Edge Distance : 1/2"
Test mode : ☐ Cyclic ☒ Monotonic



TO

DP 1st Ply Removed

PT

3.6 kN
->corner turned out, broken
- corner broken
17.5mm ->North Corner Closed

PT

Screws heavily rotated but undamaged

- wall installed "backwards"
- hold downs tightened to 8 kN South, 10 kN North
- one of the teflon braces spun

Max Load = 13.648 kN

PPT
- plywood delaminated
- ultimate failure ductile -> loosing capacity

PT

track-stud connection closed

plywood delaminated

PPT
Plywood highly feathered

PPT

13.6kN corner delaminated

PT

Screw 57mm pull through tear out

track-stud connection never closed

Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP) ;
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Light Gauge Steel Frame / Wood Panel Shear Walls
McGill University, Montreal

TEST: 49B

RESEARCHER: KATHERINE HIKITA **ASSISTANTS:** M. OUELLET, D. MORELLO, R. CARBONNEAU

DATE: Built: May 11; Tested: Friday May 13, 2005 **TIME:** 17:00

DIMENSIONS OF WALL: 4 FT X 8 FT **PANEL ORIENTATION:** Vertical; Sheathing one side

SHEATHING:
- [ ] Plywood 15/32" APA Rated Exposure 1 (USA)
- [ ] OSB 7/16" APA Rated Exposure 1 (USA)
- [ ] Plywood (CSA 0151M) CSP 12.5mm (1/2") — CAN/PLY Exterior CSP
- [ ] Plywood (CSA O121M) DFP 12.5mm (1/2") — CAN/PLY Exterior DFP
- [X] OSB (CSA O325) 11 mm (7/16")
- [ ] Other — MFR: Grant Forest Products Corp. Englehart, ON

SCREWS

Sheathing:
- [X] No.8 gauge 1.5" self-piercing Bugle head LOX drive (Grabber Superdrive)
- [ ] No.8 gauge 1.0" self-piercing Bugle head (Flat socket head screw) (HD)
- [ ] No.8 gauge 1.25" self-drilling Bugle head LOX Drive (Grabber Super
- [ ] No.9 gauge 1.0" self-piercing Bugle head (screws Y2,Y12 in track)

Framing:
- [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive

Hold downs:
- [X] No.10 gauge 0.75" self-drilling Hex washer head

Loading Beam: A325 3/4" bolts — 3 bolts [ ] 6 bolts [X] 12 bolts [ ]

Back-to-Back Chord Studs:
- [X] No.10 gauge 0.75" self-drilling Hex washer head (2@12" O.C.)

SHEATHING FASTENER SCHEDULE: [ ] 2"/12" [ ] 3"/12" [ ] 4"/12" [X] 6"/12" [ ] Other:

EDGE PANEL DISTANCE: [ ] 3/8" [X] 1/2" [ ] Other:

STUDS: Field, Chord
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MPa)
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MPa)
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.054" (1.37mm) 50ksi (340 MPa)
- [X] Double chord studs used
- [ ] Other

STUD SPACING:
- [ ] 12" O.C.
- [ ] 16" O.C.
- [X] 24" O.C.
- [ ] Other:

TRACK: Web: 3-5/8" inches; Flange: 1-1/4" inches
- [X] 0.043" (1.09mm) 33ksi (230 MPa)
- [ ] Other:

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10 7/8" Anchor Rod (# of screws): 33
- [ ] UCI 18" hold down 1/2" Anchor Rod (# of screws):
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [X] Monotonic — 7.5mm/min.
- [ ] Cyclic — CUREE reversed cyclic (10 mm/s)
- [ ] Gravity — 9 kN load at both ends of wall

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral
- [X] East Frame Brace
- [X] West Frame Brace
- [X] Wood Shear

TOTAL: 9

MOISTURE CONTENT OF SHEATHING: Moisture Meter

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

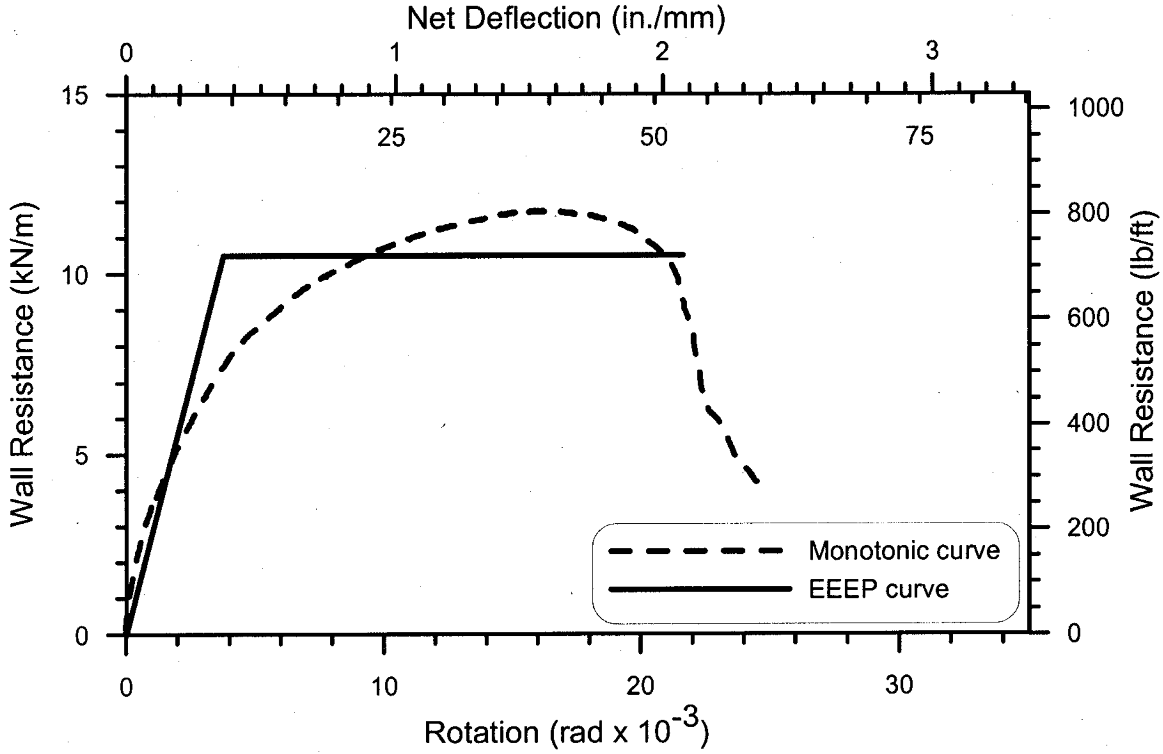
	North	North
Ww=	30.02	28.52
Wd=	28.98	27.56
m.c.=	3.59	3.48

AVG m.c. 3.54

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 50 scan/sec

COMMENTS:
- -Shear anchors torqued to 77 kN
- -Hold down anchors tightened to approximately 8 kN (load cells used on both hold-downs)
- -Ambient temperature 23 C
- -Double chord studs used
- -Square plate washers (2.5"x2.5") used in all track connections
- -Pilot holes drilled for screws A1,A5,A9,Q1,Q5,Q9
- -Initial load set to zero at beginning of test, displacement 0.267 mm
- -No gravity loadsÉTest 49B comparison to 49A to determine effects of gravity loads
- - extra hole between A6 and A7 due to mismeasurement of screw spacing

Test 49B
(1220 x 2440 mm OSB 152/305 mm)



	Parameters	Units
F_u	14.32	kN
$F_{0.8u}$	11.46	kN
$F_{0.4u}$	5.73	kN
F_y	12.82	kN
K_e	1.40	kN/mm
Ductility (μ)	5.77	-
$\Delta_{net,y}$	9.13	mm
$\Delta_{net,u}$	40.61	mm
$\Delta_{net,0.8u}$	52.69	mm
$\Delta_{net,0.4u}$	4.08	mm
$Area_{Backbone}$	616.67	J
$Area_{EEEP}$	616.67	J
Check	OK	
R_d	**3.25**	-
S_y	**10.51**	kN/m

Test Name	49B
Date Tested	May 13, 2005
Protocol	MONOTONIC

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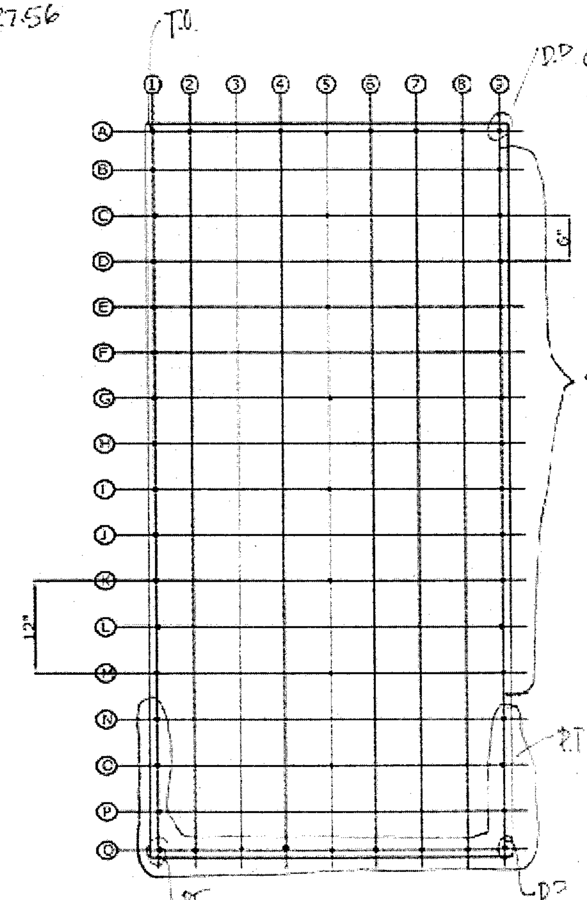
McGill *No gravity loads

Cold-Formed Steel Stud Shear Walls Testing

Moisture Content A: 30.02 28.93
B: 28.52 27.56

Test name : 49B
Date tested : May 13, 2005
Wall Size : 4' x 8'
Screw pattern : 6" / 12"
Edge Distance : 1/2"
Test mode : ☐ Cyclic ☒ Monotonic



D.P. corner smashed up

31.9 mm → All three damaged corners torn out

Max load 14.34 kN
Disp L 70.069 mm

slight PPT in comparison to the other side

Suspect south shear reading to be inadmissible → nuts in LVDT - not tight - checked after and that was not the case.

PT. / T.O. OSB Disintegrates - feathers out.

Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Light Gauge Steel Frame / Wood Panel Shear Walls
McGill University, Montreal

TEST: 49C

RESEARCHER: KATHERINE HIKITA **ASSISTANTS:** M. OUELLET, D. MORELLO, R. CARBONNEAU, JIANG FAN

DATE: Built: May 13; Tested: Friday May 16, 2005 **TIME:** 17:00

DIMENSIONS OF WALL: 4 FT X 8 FT **PANEL ORIENTATION:** Vertical, Sheathing one side

SHEATHING:
- [ ] Plywood 15/32" APA Rated Exposure 1 (USA)
- [ ] OSB 7/16" APA Rated Exposure 1 (USA)
- [ ] Plywood (CSA 0151M) CSP 12.5mm (1/2") — CAN/PLY Exterior CSP
- [ ] Plywood (CSA O121M) DFP 12.5mm (1/2") — CAN/PLY Exterior DFP
- [X] OSB (CSA O325) 11 mm (7/16")
- [ ] Other — MFR: Grant Forest Products Corp. Englehart, ON

SCREWS Sheathing:
- [X] No.8 gauge 1.5" self-piercing Bugle head LOX drive (Grabber Superdrive)
- [ ] No.8 gauge 1.0" self-piercing Bugle head (Flat socket head screw) (HD)
- [ ] No.8 gauge 1.25" self-drilling Bugle head LOX Drive (Grabber Super
- [ ] No.9 gauge 1.0" self-piercing Bugle head (screws Y2,Y12 in track)

Framing: [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive

Hold downs: [X] No.10 gauge 0.75" self-drilling Hex washer head

Loading Beam: A325 3/4" bolts — 3 bolts [ ] 6 bolts [X] 12 bolts [ ]

Back-to-Back Chord Studs: [X] No.10 gauge 0.75" self-drilling Hex washer head (2@12" O.C.)

SHEATHING FASTENER SCHEDULE: [ ] 2"/12" [ ] 3"/12" [ ] 4"/12" [X] 6"/12" [ ] Other:

EDGE PANEL DISTANCE: [ ] 3/8" [X] 1/2" [ ] Other:

STUDS: Field / Chord
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MPa)
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MPa)
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.054" (1.37mm) 50ksi (340 MPa)
- [X] Double chord studs used
- [ ] Other

STUD SPACING: [ ] 12" O.C. [ ] 16" O.C. [X] 24" O.C. [ ] Other:

TRACK: Web: 3-5/8" inches; Flange: 1-1/4" inches — [X] 0.043" (1.09mm) 33ksi (230 MPa) [ ] Other:

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10 7/8" Anchor Rod (# of screws): 33
- [ ] UCI 18" hold down 1/2" Anchor Rod (# of screws):
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [X] Monotonic — 7.5mm/min.
- [ ] Cyclic — CUREE reversed cyclic (10 mm/s)
- [X] Gravity — 9 kN load at both ends of wall

LVDT MEASUREMENTS:
- [X] Actuator LVDT [X] North Uplift [X] East Frame Brace
- [X] North Slip [X] South Uplift [X] West Frame Brace
- [X] South Slip [X] Top of Wall Lateral [X] Wood Shear

TOTAL: 9

MOISTURE CONTENT OF SHEATHING: Moisture Meter

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

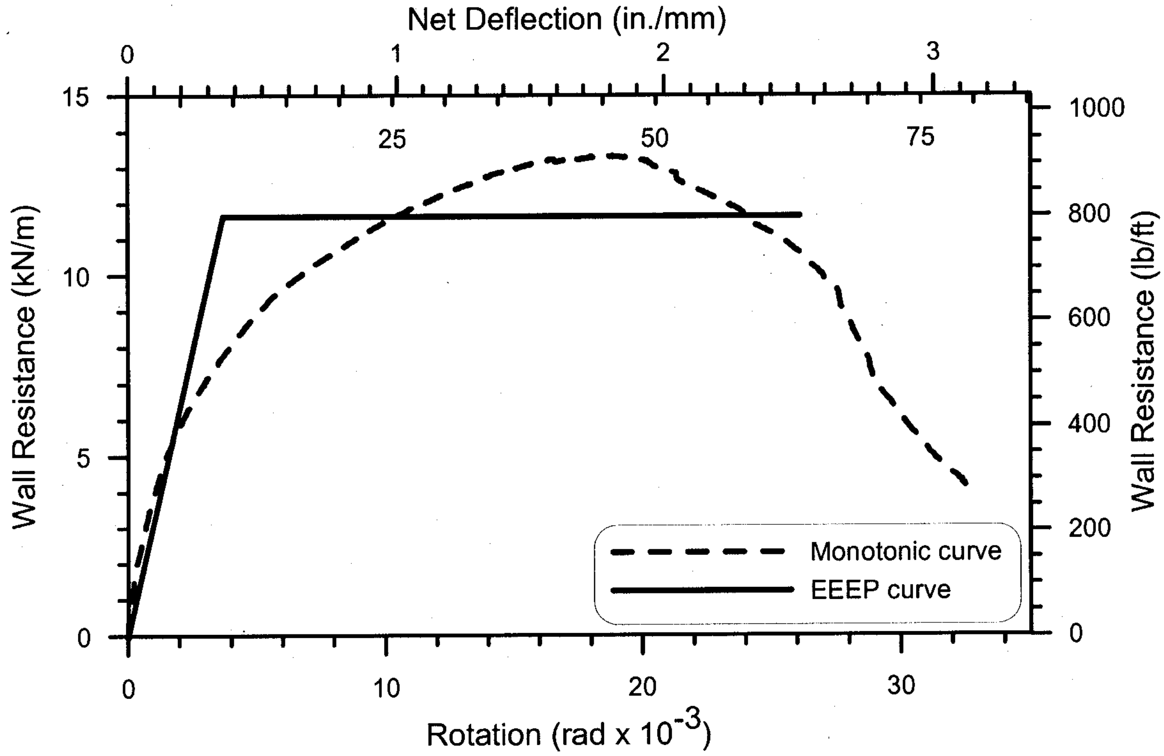
	North	North
Ww=	28.78	27.30
Wd=	27.75	26.50
m.c.=	3.71	3.02

AVG m.c. 3.37

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 50 scan/sec

COMMENTS:
- -Shear anchors torqued to 77 kN
- -Hold down anchors tightened to approximately 8 kN (load cells used on both hold-downs)
- -Ambient temperature 23 C
- -Double chord studs used
- -Square plate washers (2.5"x2.5") used in all track connections
- -Pilot holes drilled for screws A1,A5,A9,Q1,Q5,Q9
- -Initial load set to zero at beginning of test, displacement 0.172 mm
- - Gravity loads included in test
- -Offset after gravity loads engaged 0.1 mm

Test 49C
(1220 x 2440 mm OSB 152/305 mm)



	Parameters	Units
F_u	16.24	kN
$F_{0.8u}$	12.99	kN
$F_{0.4u}$	6.50	kN
F_y	14.18	kN
K_e	1.58	kN/mm
Ductility (μ)	7.07	-
$\Delta_{net,y}$	9.00	mm
$\Delta_{net,u}$	45.96	mm
$\Delta_{net,0.8u}$	63.61	mm
$\Delta_{net,0.4u}$	4.12	mm
$Area_{Backbone}$	838.36	J
$Area_{EEEP}$	838.36	J
Check	OK	
R_d	**3.63**	-
S_y	**11.63**	kN/m

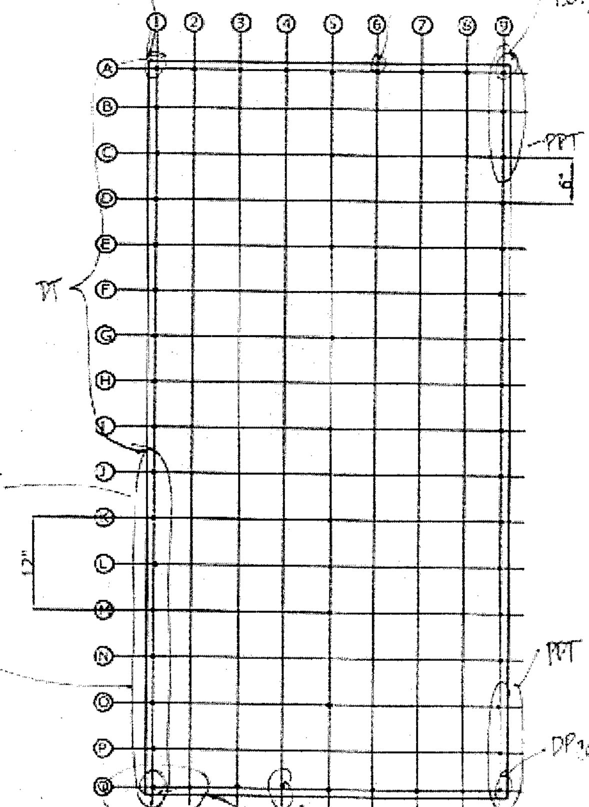
Test Name	49C
Date Tested	May 16, 2005
Protocol	MONOTONIC

McGill Gravity loads included

Cold-Formed Steel Stud Shear Walls Testing

Moisture Content: A 28.78 27.75
B 27.53 26.50

Test name: 49C
Date tested: May 16, 2005
Wall Size: 4' x 8'
Screw pattern: 6" / 12"
Edge Distance: 1/2"
Test mode: ☐ Cyclic ☒ Monotonic



* double chord studs were bent by screws for shear LVDT (Top) holders - hammered back in place & put bolts+washers in first

Max load 16.564 kN

Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Light Gauge Steel Frame / Wood Panel Shear Walls
McGill University, Montreal

TEST: 49D

RESEARCHER: KATHERINE HIKITA **ASSISTANTS:** M. OUELLET, JIANG FAN

DATE: Built: May 11; Tested: Wednesday May 18, 2005 **TIME:** 13:00

DIMENSIONS OF WALL: 4 FT X 8 FT **PANEL ORIENTATION:** Vertical
Sheathing one side

SHEATHING:
- [ ] Plywood 15/32" APA Rated Exposure 1 (USA)
- [ ] OSB 7/16" APA Rated Exposure 1 (USA)
- [ ] Plywood (CSA 0151M) CSP 12.5mm (1/2") CAN/PLY Exterior
- [ ] Plywood (CSA O121M) DFP 12.5mm (1/2") CAN/PLY Exterior DFP
- [X] OSB (CSA O325) 11 mm (7/16")
- [ ] Other MFR: Grant Forest Products Corp. Englehart, ON

SCREWS Sheathing:
- [X] No.8 gauge 1.5" self-piercing Bugle head LOX drive (Grabber Superdrive)
- [ ] No.8 gauge 1.0" self-piercing Bugle head (Flat socket head screw) (HD)
- [ ] No.8 gauge 1.25" self-drilling Bugle head LOX Drive (Grabber Superdrive)
- [ ] No.9 gauge 1.0" self-piercing Bugle head (screws Y2,Y12 in track)

Framing: [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive
Hold downs: [X] No.10 gauge 0.75" self-drilling Hex washer head
Loading Beam: A325 3/4" bolts 3 bolts [ ] 6 bolts [X] 12 bolts [ ]
Back-to-Back
Chord Studs: [X] No.10 gauge 0.75" self-drilling Hex washer head (2@12" O.C.)

SHEATHING FASTENER SCHEDULE: [ ] 2"/12" [ ] 3"/12" [ ] 4"/12" [X] 6"/12" [ ] Other:

EDGE PANEL DISTANCE: [ ] 3/8" [X] 1/2" [ ] Other:

STUDS: Field
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MPa)

Chord
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MPa)
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.054" (1.37mm) 50ksi (340 MPa)
- [X] Double chord studs used
- [ ] Other

STUD SPACING:
- [ ] 12" O.C.
- [ ] 16" O.C.
- [X] 24" O.C.
- [ ] Other:

TRACK: Web: 3-5/8" inches Flange: 1-1/4" inches
- [X] 0.043" (1.09mm) 33ksi (230 MPa)
- [ ] Other:

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10 7/8" Anchor Rod (# of screws): 33
- [ ] UCI 18" hold down 1/2" Anchor Rod (# of screws):
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [X] Monotonic 7.5mm/min.
- [ ] Cyclic CUREE reversed cyclic (10 mm/s)
- [X] Gravity 9 kN load at both ends of wall

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral
- [X] East Frame Brace
- [X] West Frame Brace
- [X] Wood Shear

TOTAL: 9

MOISTURE CONTENT OF SHEATHING: Moisture Meter

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

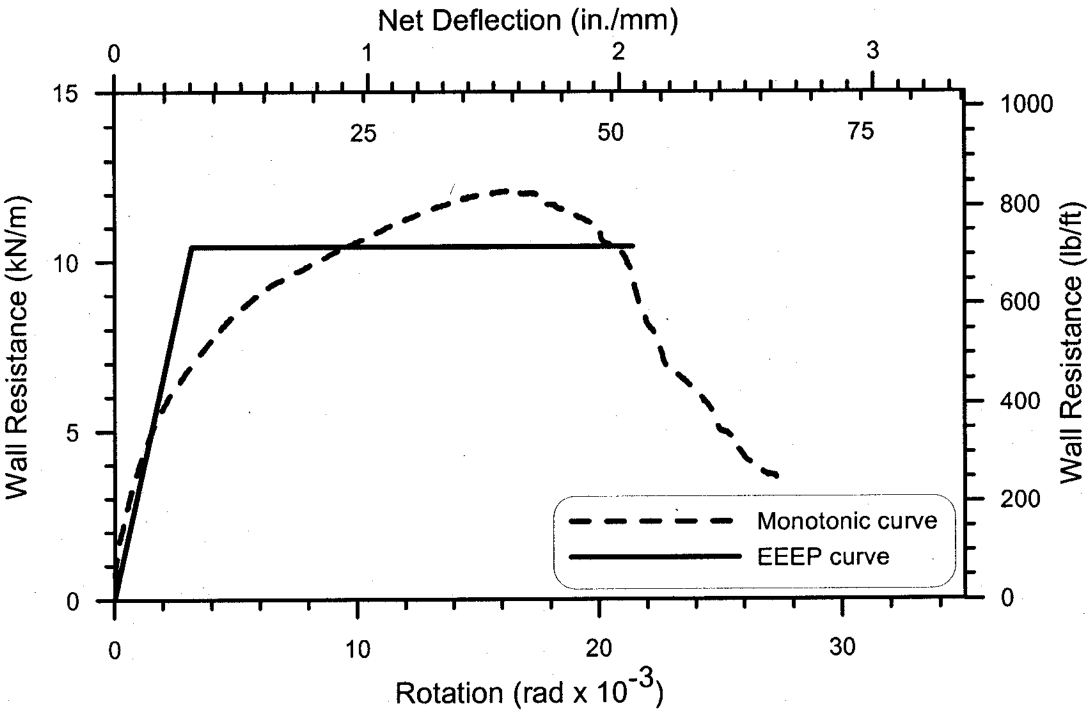
	North	North
Ww=	28.94	26.15
Wd=	27.92	25.08
m.c.=	3.65	4.27

AVG m.c. 3.96

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 50 scan/sec

COMMENTS:
- -Shear anchors torqued to 77 kN
- -Hold down anchors tightened to approximately 8 kN (load cells used on both hold-downs)
- -Ambient temperature 23 C
- -Double chord studs used
- -Square plate washers (2.5"x2.5") used in all track connections
- -Pilot holes drilled for screws A1,A5,A9,Q1,Q5,Q9
- -Initial load set to zero at beginning of test, displacement -0.236 mm
- - Displacment after gravity loads were engaged -0.438 mm

Test 49D
(1220 x 2440 mm OSB 152/305 mm)



	Parameters	Units
F_u	14.79	kN
$F_{0.8u}$	11.83	kN
$F_{0.4u}$	5.92	kN
F_y	12.73	kN
K_e	1.64	kN/mm
Ductility (μ)	6.73	-
$\Delta_{net,y}$	7.74	mm
$\Delta_{net,u}$	40.32	mm
$\Delta_{net,0.8u}$	52.08	mm
$\Delta_{net,0.4u}$	3.60	mm
$Area_{Backbone}$	613.74	J
$Area_{EEEP}$	613.74	J
Check	OK	
R_d	**3.53**	-
S_y	**10.44**	kN/m

Test Name	49D
Date Tested	May 18, 2005
Protocol	MONOTONIC

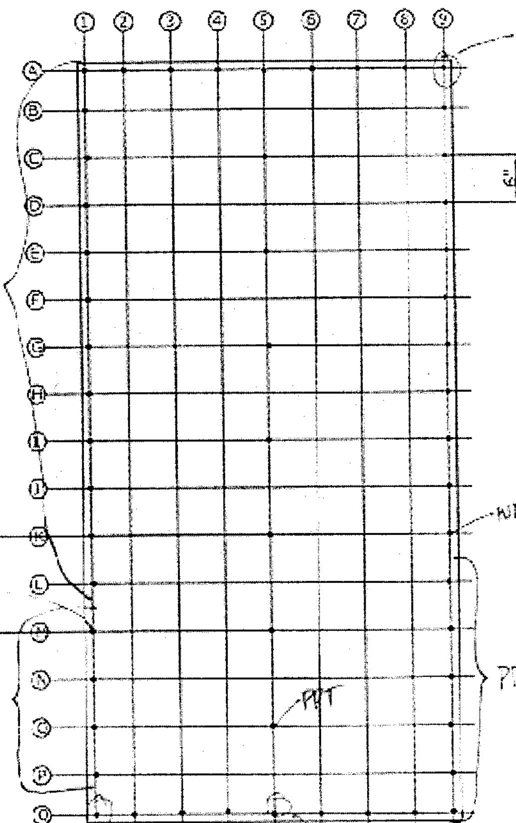
McGill

Cold-Formed Steel Stud Shear Walls Testing

Moisture Content A 28.94 27.92
B 26.15 25.08

Test name : Test 49 D
Date tested : May 18 2005
Wall Size : 4' x 8'
Screw pattern : 6" / 12"
Edge Distance : 1/2"
Test mode : ☐ Cyclic ☒ Monotonic

Max load 15.072 kN
Max Displ 74.06 mm



T.O. WB

PPT due to slight rotation

WB damage

PPT WB

WB, TO

PT

PPT

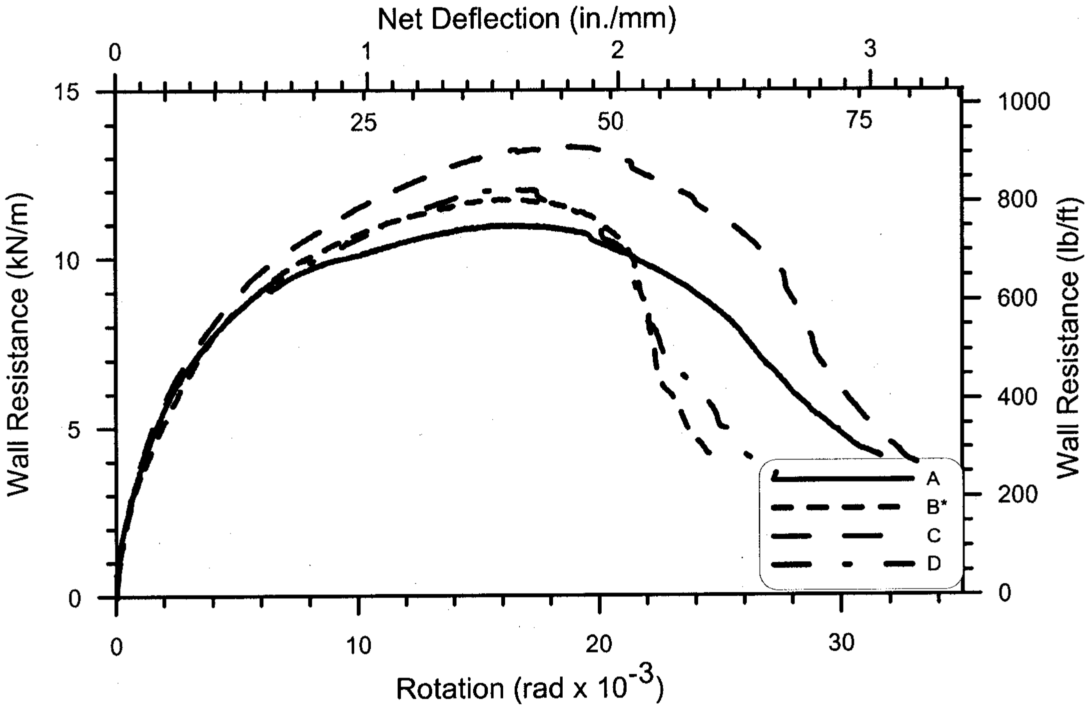
TO/WB

PT

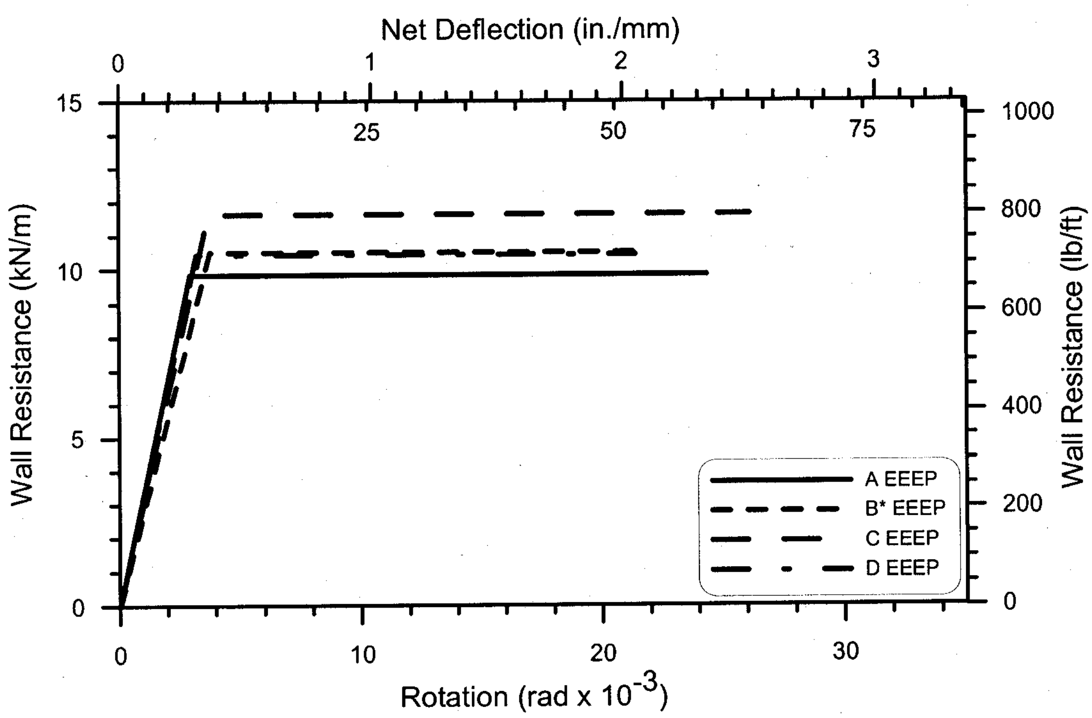
DP middle stud deformed at bottom

Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DF)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Test 49-A,B,C,D
(1220 x 2440 mm OSB 152/305 mm)



Test 49-A,B,C,D EEEP
(1220 x 2440 mm OSB 152/305 mm)



Light Gauge Steel Frame / Wood Panel Shear Walls
McGill University, Montreal

TEST: 50A

RESEARCHER: KATHERINE HIKITA **ASSISTANTS:** M. OUELLET, JIANG FAN, M. ALKHARAT

DATE: Built: May 11; Tested: Thursday May 19, 2005 **TIME:** 12:00

DIMENSIONS OF WALL: 4 FT X 8 FT **PANEL ORIENTATION:** Vertical
Sheathing one side

SHEATHING:
- [ ] Plywood 15/32" APA Rated Exposure 1 (USA)
- [ ] OSB 7/16" APA Rated Exposure 1 (USA)
- [ ] Plywood (CSA 0151M) CSP 12.5mm (1/2") CAN/PLY Exterior CSP
- [ ] Plywood (CSA O121M) DFP 12.5mm (1/2") CAN/PLY Exterior DFP
- [X] OSB (CSA O325) 11 mm (7/16")
- [ ] Other MFR: Grant Forest Products Corp. Englehart, ON

SCREWS Sheathing:
- [X] No.8 gauge 1.5" self-piercing Bugle head LOX drive (Grabber Superdrive)
- [ ] No.8 gauge 1.0" self-piercing Bugle head (Flat socket head screw) (HD)
- [ ] No.8 gauge 1.25" self-drilling Bugle head LOX Drive (Grabber !
- [ ] No.9 gauge 1.0" self-piercing Bugle head (screws Y2,Y12 in track)

Framing: [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive
Hold downs: [X] No.10 gauge 0.75" self-drilling Hex washer head
Loading Beam: A325 3/4" bolts 3 bolts [ ] 6 bolts [X] 12 bolts [ ]
Back-to-Back Chord Studs: [X] No.10 gauge 0.75" self-drilling Hex washer head (2@12" O.C.)

SHEATHING FASTENER SCHEDULE: [ ] 2"/12" [ ] 3"/12" [ ] 4"/12" [X] 6"/12" [ ] Other:

EDGE PANEL DISTANCE: [ ] 3/8" [X] 1/2" [ ] Other:

STUDS: Field / Chord
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MPa)
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MPa)
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.054" (1.37mm) 50ksi (340 MPa)
- [X] Double chord studs used
- [ ] Other

STUD SPACING:
- [ ] 12" O.C.
- [ ] 16" O.C.
- [X] 24" O.C. [ ] Other:

TRACK: Web: 3-5/8" inches; Flange: 1-1/4" inches; [X] 0.043" (1.09mm) 33ksi (230 MPa) [ ] Other:

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10 7/8" Anchor Rod (# of screws): 33
- [ ] UCI 18" hold down 1/2" Anchor Rod (# of screws):
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [ ] Monotonic 7.5mm/min.
- [X] Cyclic CUREE reversed cyclic (10 mm/s)
- [X] Gravity 9 kN load at both ends of the wall

10 m

LVDT MEASUREMENTS:
- [X] Actuator LVDT [X] North Uplift [X] East Frame Brace
- [X] North Slip [X] South Uplift [X] West Frame Brace
- [X] South Slip [X] Top of Wall Lateral [X] Wood Shear

TOTAL: 9

MOISTURE CONTENT OF SHEATHING: Moisture Meter

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

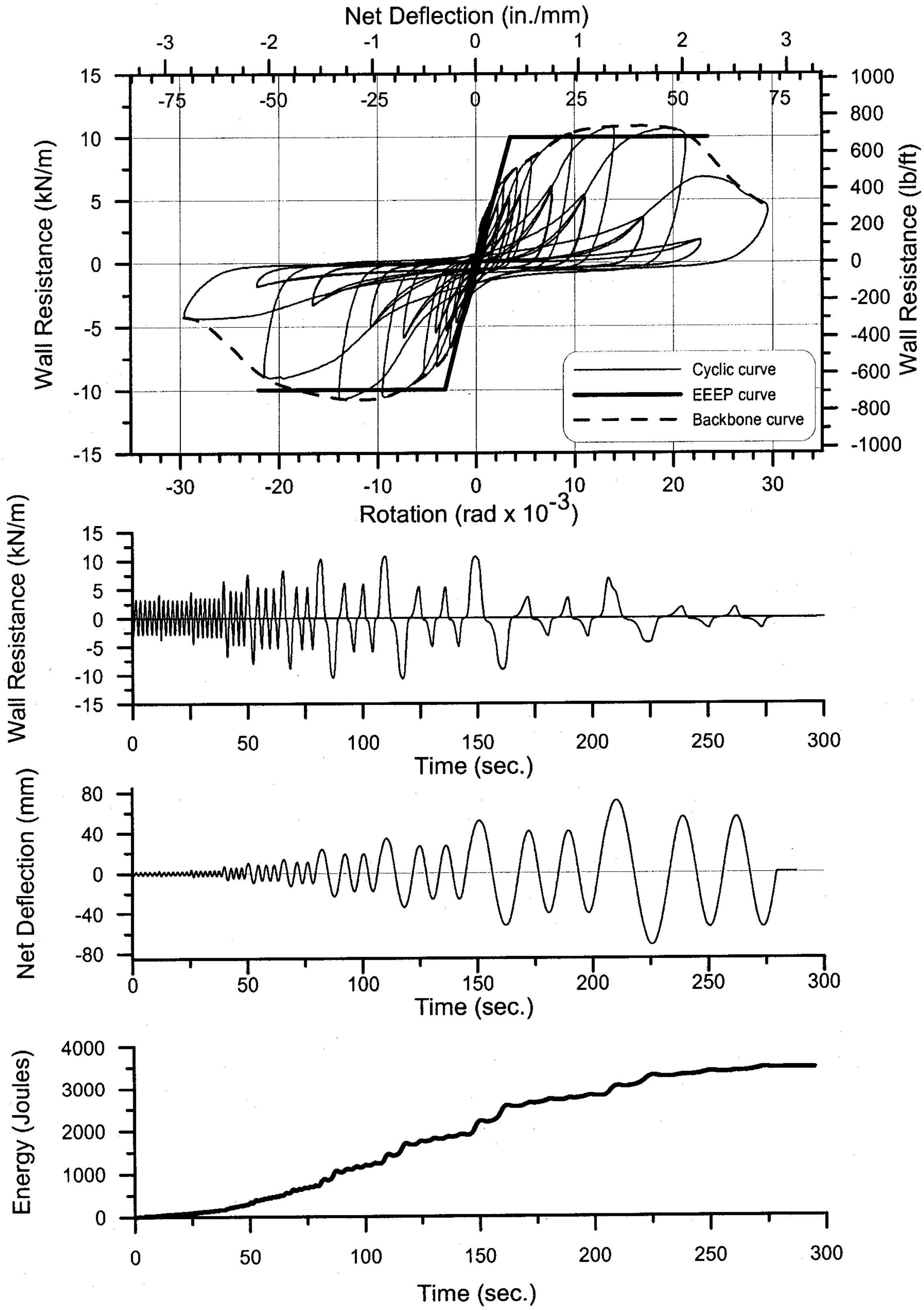
Ww=	27.08	30.68
Wd=	26.13	29.61
m.c.=	3.64	3.61
	North	North

AVG m.c. 3.62

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 50 scan/sec

COMMENTS:
- -Shear anchors torqued to 77 kN
- -Hold down anchors tightened to approximately 8 kN (load cells used on both hold-downs)
- -Ambient temperature 23 C
- -Double chord studs used
- -Square plate washers (2.5"x2.5") used in all track connections

Test 50A
(1220 x 2440 mm OSB 152/305 mm)



	Negative	Positive	Units
F_u	-13.05	13.13	kN
$F_{0.8u}$	-10.44	10.50	kN
$F_{0.4u}$	-5.22	5.25	kN
F_y	-12.13	12.14	kN
K_e	1.58	1.42	kN/mm
Ductility (μ)	7.01	6.68	-
$\Delta_{net,y}$	-7.67	8.55	mm
$\Delta_{net,u}$	-33.01	33.24	mm
$\Delta_{net,0.8u}$	-53.80	57.10	mm
$\Delta_{net,0.4u}$	-3.30	3.70	mm
$Area_{Backbone}$	606.15	641.20	kN-mm
$Area_{EEEP}$	606.15	641.20	kN-mm
Check	OK	OK	
R_d	**3.61**	**3.51**	-
S_y	**-9.95**	**9.96**	kN/m

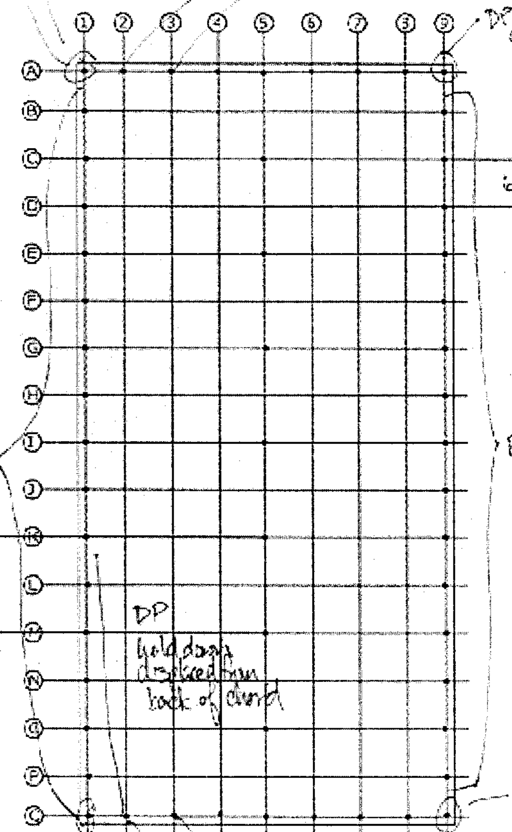
Test Name	50A
Date Tested	May 19, 2005
Protocol	CUREE ORDINARY GROUND MOTIONS

McGill

Cold-Formed Steel Stud Shear Walls Testing

MOISTURE CONTENT A 27.08 26.13
B 30.68 29.61

Test name : 50 A
Date tested : May 19, 2005
Wall Size : 4' x 8'
Screw pattern : 6" / 12"
Edge Distance : 1/2"
Test mode : ☐ Cyclic ☒ Monotonic



Max Load 13.411
Max Displ 77.025

Hydraulic Jacks Gravity

	Min	Max
North	8.4565	9.6013
South	8.6986	9.3122

Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Light Gauge Steel Frame / Wood Panel Shear Walls
McGill University, Montreal

TEST: 50B

RESEARCHER: KATHERINE HIKITA **ASSISTANTS:** M. OUELLET, D. MORELLO, R. CARBONNEAU

DATE: Built: May 11; Tested: Friday May 20, 2005 **TIME:** 11:25

DIMENSIONS OF WALL: 4 FT X 8 FT **PANEL ORIENTATION:** Vertical, Sheathing one side

SHEATHING:
- [ ] Plywood 15/32" APA Rated Exposure 1 (USA)
- [ ] OSB 7/16" APA Rated Exposure 1 (USA)
- [ ] Plywood (CSA 0151M) CSP 12.5mm (1/2") — CAN/PLY Exterior CSP
- [ ] Plywood (CSA O121M) DFP 12.5mm (1/2") — CAN/PLY Exterior DFP
- [X] OSB (CSA O325) 11 mm (7/16")
- [ ] Other — MFR: Grant Forest Products Corp. Englehart, ON

SCREWS

Sheathing:
- [X] No.8 gauge 1.5" self-piercing Bugle head LOX drive (Grabber Superdrive)
- [ ] No.8 gauge 1.0" self-piercing Bugle head (Flat socket head screw) (HD)
- [ ] No.8 gauge 1.25" self-piercing Bugle head LOX Drive (Grabber Supe
- [ ] No.9 gauge 1.0" self-piercing Bugle head (screws Y2,Y12 in track)

Framing:
- [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive

Hold downs:
- [X] No.10 gauge 0.75" self-drilling Hex washer head

Loading Beam: A325 3/4" bolts — 3 bolts [ ] 6 bolts [X] 12 bolts [ ]

Back-to-Back Chord Studs:
- [X] No.10 gauge 0.75" self-drilling Hex washer head (2@12" O.C.)

SHEATHING FASTENER SCHEDULE: [ ] 2"/12" [ ] 3"/12" [ ] 4"/12" [X] 6"/12" [ ] Other:

EDGE PANEL DISTANCE: [ ] 3/8" [X] 1/2" [ ] Other:

STUDS: Field / Chord
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.054" (1.37mm) 50ksi (340 MF
- [X] Double chord studs used
- [ ] Other

STUD SPACING:
- [ ] 12" O.C.
- [ ] 16" O.C.
- [X] 24" O.C.
- [ ] Other:

TRACK: Web: 3-5/8" inches; Flange: 1-1/4" inches
- [X] 0.043" (1.09mm) 33ksi (230 MPa)
- [ ] Other:

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10 7/8" Anchor Rod (# of screws): 33
- [ ] UCI 18" hold down 1/2" Anchor Rod (# of screws):
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [ ] Monotonic — 7.5mm/min.
- [X] Cyclic — CUREE reversed cyclic (10 mm/s)
- [X] Gravity — 9 kN load at both ends of the wall, 10 m

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral
- [X] East Frame Brace
- [X] West Frame Brace
- [X] Wood Shear

TOTAL: 9

MOISTURE CONTENT OF SHEATHING: Moisture Meter

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

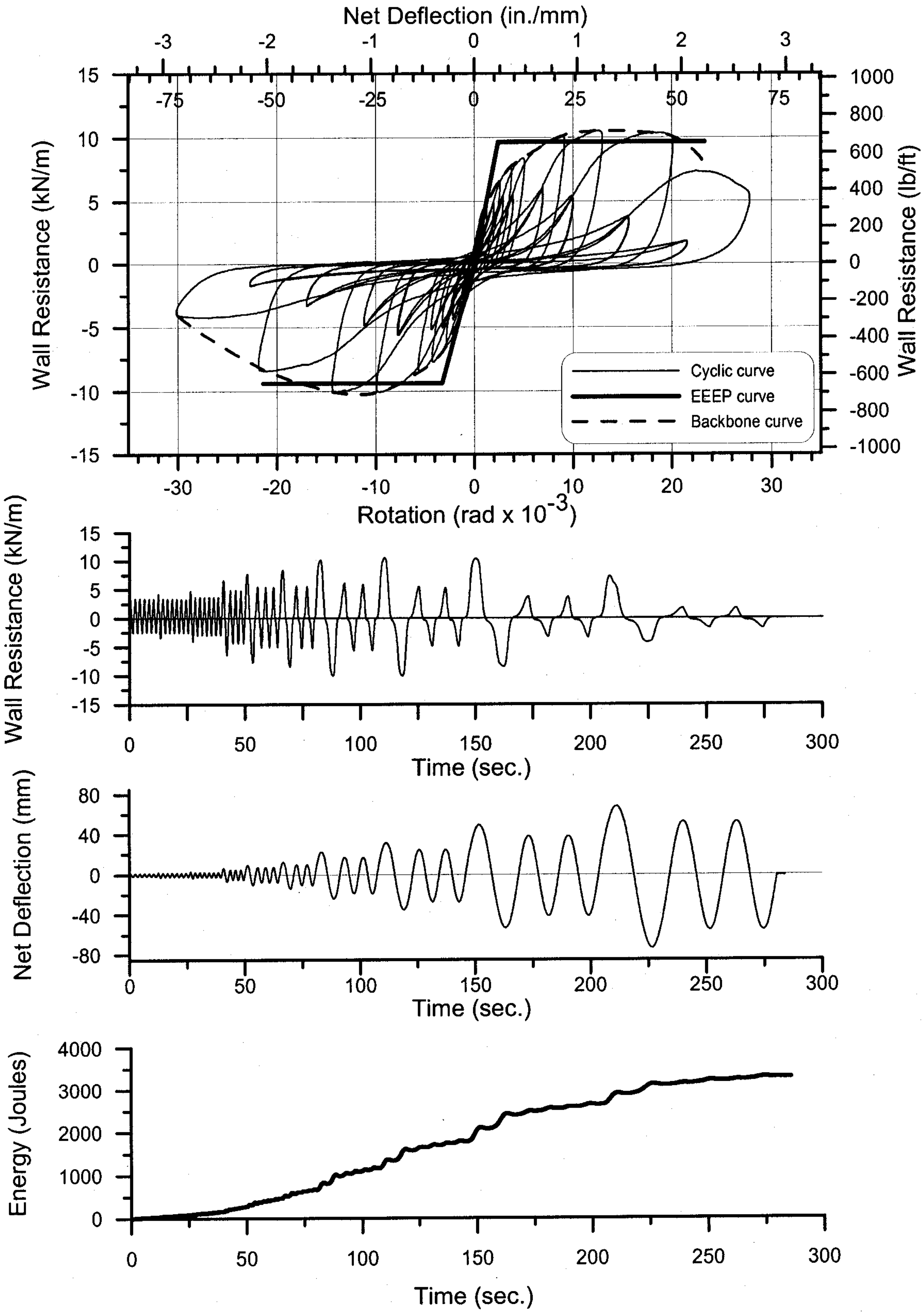
	North	North
Ww=	29.01	30.44
Wd=	27.97	29.39
m.c.=	3.72	3.57

AVG m.c. 3.65

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 50 scan/sec

COMMENTS:
- -Shear anchors torqued to 77 kN
- -Hold down anchors tightened to approximately 8 kN (load cells used on both hold-downs)
- -Ambient temperature 23 C
- -Double chord studs used
- -Square plate washers (2.5"x2.5") used in all track connections
- -Pilot holes drilled for screws A1,A5,A9,Q1,Q5,Q9
- -Initial load set to zero at beginning of test, displacement 0.267 mm
- - Gravity loads engaged

Test 50B
(1220 x 2440 mm OSB 152/305 mm)



	Negative	Positive	Units
F_u	-12.27	12.79	kN
$F_{0.8u}$	-9.82	10.23	kN
$F_{0.4u}$	-4.91	5.12	kN
F_y	-11.43	11.79	kN
K_e	1.44	1.97	kN/mm
Ductility (μ)	6.59	9.49	-
$\Delta_{net,y}$	-7.92	5.99	mm
$\Delta_{net,u}$	-23.43	30.84	mm
$\Delta_{net,0.8u}$	-52.20	56.80	mm
$\Delta_{net,0.4u}$	-3.40	2.60	mm
$Area_{Backbone}$	551.60	634.10	kN-mm
$Area_{EEEP}$	551.60	634.10	kN-mm
Check	OK	OK	
R_d	**3.49**	**4.24**	-
S_y	**-9.38**	**9.67**	kN/m

Test Name	50B
Date Tested	May 20, 2005
Protocol	CUREE ORDINARY GROUND MOTIONS

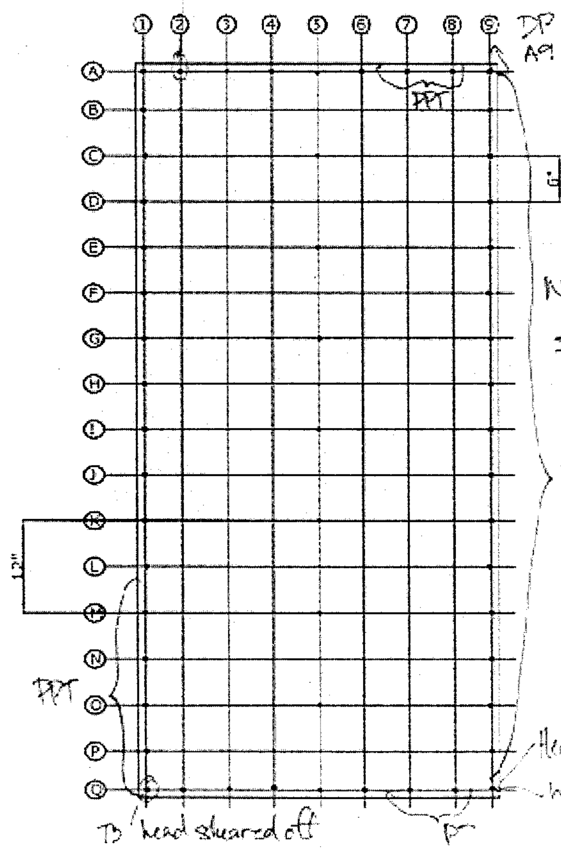
McGill

Cold-Formed Steel Stud Shear Walls Testing

50B Ⓐ 29.01 27.97
50B Ⓑ 30.44 29.39

Test name : 50B
Date tested : May 20, 20XX
Wall Size : 4' x 8'
Screw pattern : 6" / 12"
Edge Distance : 1/2"
Test mode : ☐ Cyclic ☒ Monotonic

Some damage to the exterior of OSB



Max Load 13.037 kN
Max Displ. 77.025 mm

Gravity Loads

	MIN	MAX
NORTH	8.369	9.655
SOUTH	8.681	9.281

PT more damage to the OSB towards the top

Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Light Gauge Steel Frame / Wood Panel Shear Walls
McGill University, Montreal

TEST: 50C

RESEARCHER: KATHERINE HIKITA **ASSISTANTS:** M. OUELLET, JIANG FAN, M. ALKHARAT

DATE: Built: May 11; Tested: Thursday May 26, 2005 **TIME:** 11:25

DIMENSIONS OF WALL: 4 FT X 8 FT **PANEL ORIENTATION:** Vertical, Sheathing one side

SHEATHING:
- [ ] Plywood 15/32" APA Rated Exposure 1 (USA)
- [ ] OSB 7/16" APA Rated Exposure 1 (USA)
- [ ] Plywood (CSA 0151M) CSP 12.5mm (1/2") — CAN/PLY Exterior CSP
- [ ] Plywood (CSA O121M) DFP 12.5mm (1/2") — CAN/PLY Exterior DFP
- [X] OSB (CSA O325) 11 mm (7/16")
- [ ] Other — MFR: Grant Forest Products Corp. Englehart, ON

SCREWS
- Sheathing:
  - [X] No.8 gauge 1.5" self-piercing Bugle head LOX drive (Grabber Superdrive)
  - [ ] No.8 gauge 1.0" self-piercing Bugle head (Flat socket head screw) (HD)
  - [ ] No.8 gauge 1.25" self-piercing Bugle head LOX Drive (Grabber Superdrive)
  - [ ] No.9 gauge 1.0" self-piercing Bugle head (screws Y2,Y12 in track)
- Framing: [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive
- Hold downs: [X] No.10 gauge 0.75" self-drilling Hex washer head
- Loading Beam: A325 3/4" bolts — 3 bolts [ ] 6 bolts [X] 12 bolts [ ]
- Back-to-Back Chord Studs: [X] No.10 gauge 0.75" self-drilling Hex washer head (2@12" O.C.)

SHEATHING FASTENER SCHEDULE: [ ] 2"/12" [ ] 3"/12" [ ] 4"/12" [X] 6"/12" [ ] Other:

EDGE PANEL DISTANCE: [ ] 3/8" [X] 1/2" [ ] Other:

STUDS: Field, Chord
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.054" (1.37mm) 50ksi (340 MF
- [X] Double chord studs used
- [ ] Other

STUD SPACING: [ ] 12" O.C. [ ] 16" O.C. [X] 24" O.C. [ ] Other:

TRACK: Web: 3-5/8" inches; Flange: 1-1/4" inches — [X] 0.043" (1.09mm) 33ksi (230 MPa) [ ] Other:

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10 7/8" Anchor Rod — (# of screws): 33
- [ ] UCI 18" hold down 1/2" Anchor Rod — (# of screws):
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [ ] Monotonic — 7.5mm/min.
- [X] Cyclic — CUREE reversed cyclic (10 mm/s)
- [X] Gravity — 9 kN load at both ends of the wall
- 10 m

LVDT MEASUREMENTS:
- [X] Actuator LVDT [X] North Uplift [X] East Frame Brace
- [X] North Slip [X] South Uplift [X] West Frame Brace
- [X] South Slip [X] Top of Wall Lateral [X] Wood Shear

TOTAL: 9

MOISTURE CONTENT OF SHEATHING: Moisture Meter

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

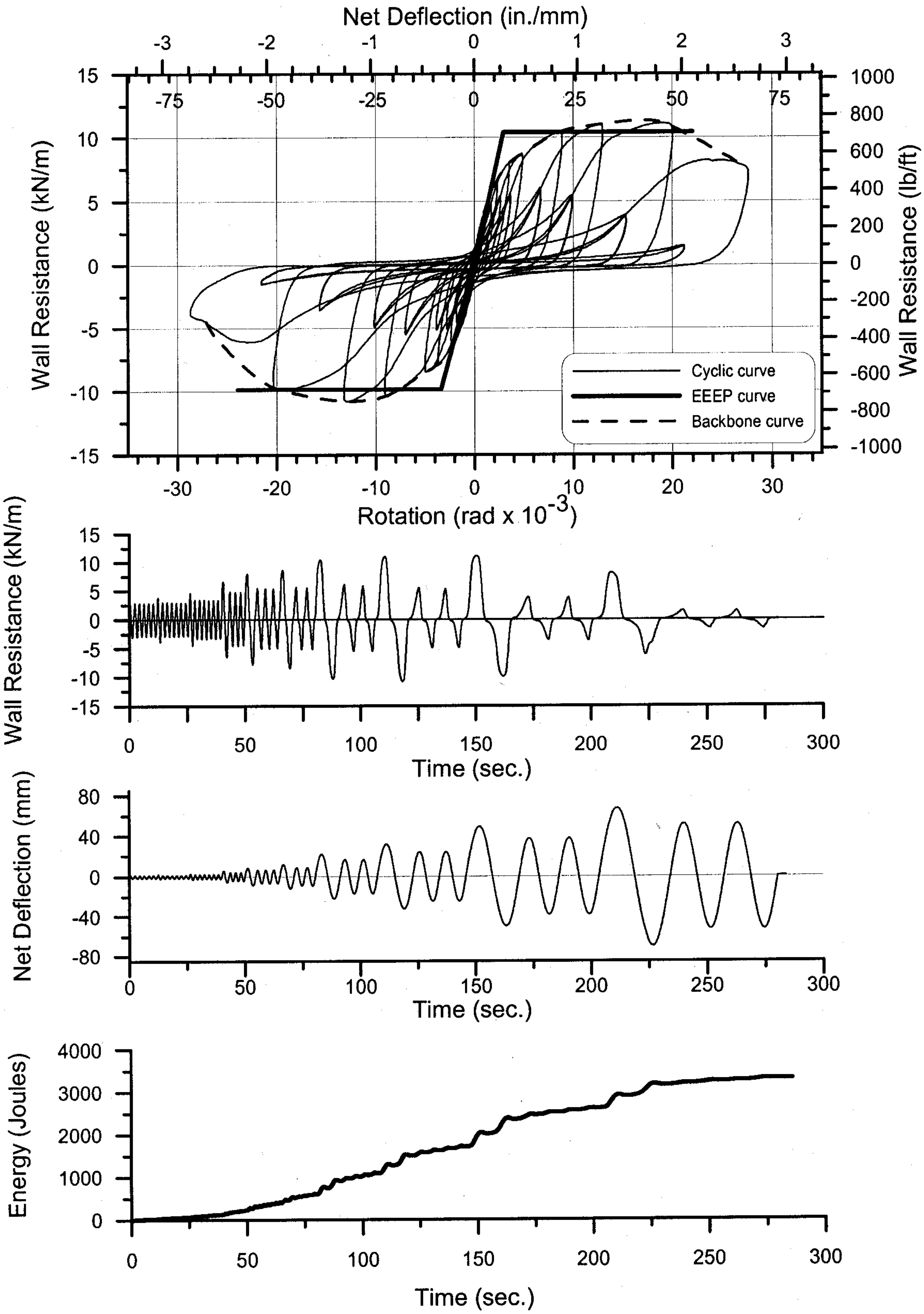
	North	North
Ww=	30.44	27.42
Wd=	29.34	26.47
m.c.=	3.75	3.59

AVG m.c. 3.67

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 50 scan/sec

COMMENTS:
- -Shear anchors torqued to 77 kN
- -Hold down anchors tightened to approximately 8 kN (load cells used on both hold-downs)
- -Ambient temperature 23 C
- -Double chord studs used
- -Square plate washers (2.5"x2.5") used in all track connections
- -Pilot holes drilled for screws A1,A5,A9,Q1,Q5,Q9
- -Initial load set to zero at beginning of test, displacement -0.632 mm
- - Screw in A1 head sheared off during testing

Test 50C
(1220 x 2440 mm OSB 152/305 mm)



	Negative	Positive	Units
F_u	-13.15	13.55	kN
$F_{0.8u}$	-10.52	10.84	kN
$F_{0.4u}$	-5.26	5.42	kN
F_y	-12.02	12.69	kN
K_e	1.46	1.75	kN/mm
Ductility (μ)	7.09	7.40	-
$\Delta_{net,y}$	-8.23	7.26	mm
$\Delta_{net,u}$	-31.22	46.66	mm
$\Delta_{net,0.8u}$	-58.30	53.70	mm
$\Delta_{net,0.4u}$	-3.60	3.10	mm
$Area_{Backbone}$	651.40	635.33	kN-mm
$Area_{EEEP}$	651.40	635.33	kN-mm
Check	OK	OK	
R_d	**3.63**	**3.71**	-
S_y	**-9.86**	**10.41**	kN/m

Test Name	50C
Date Tested	May 26, 2005
Protocol	CUREE ORDINARY GROUND MOTIONS

McGill

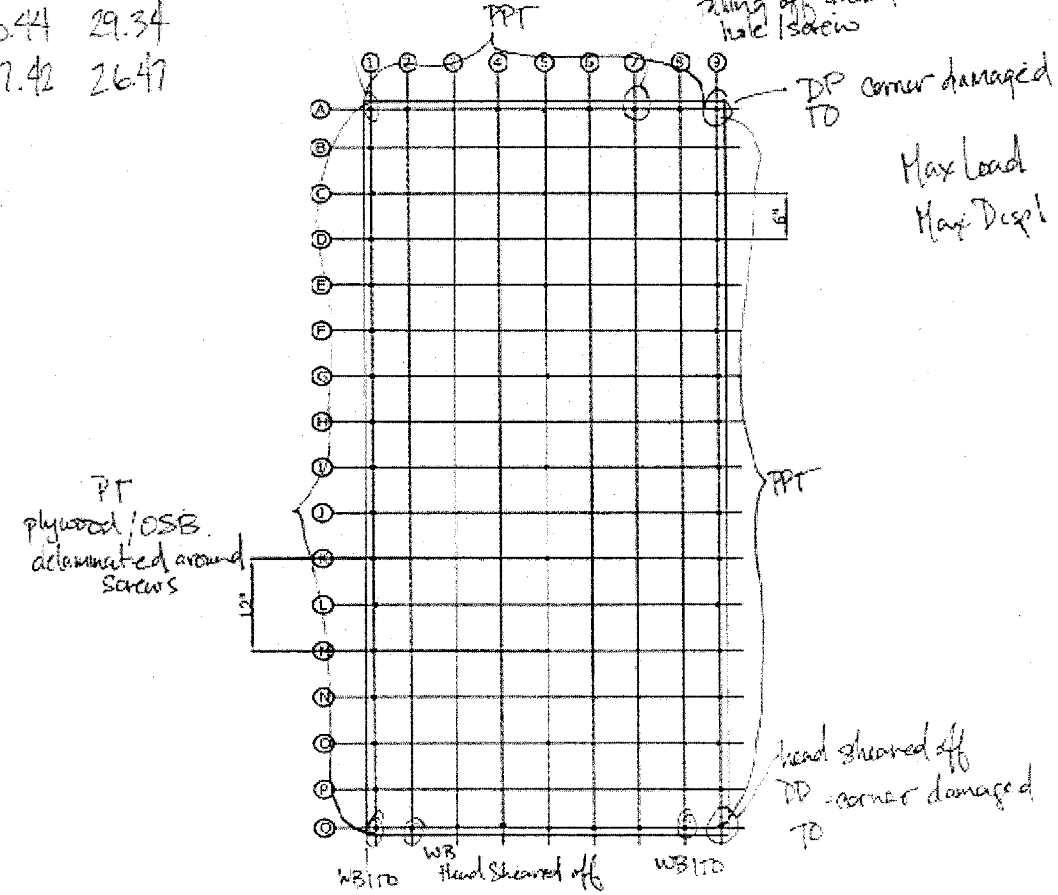
*No gravity loads

Cold-Formed Steel Stud Shear Walls Testing

Test name : 50 C
Date tested : May 26 2005
Wall Size : 4' x 8'
Screw pattern : 6" / 12"
Edge Distance : 1/2"
Test mode : ☐ Cyclic ☒ Monotonic

MOISTURE CONTENT

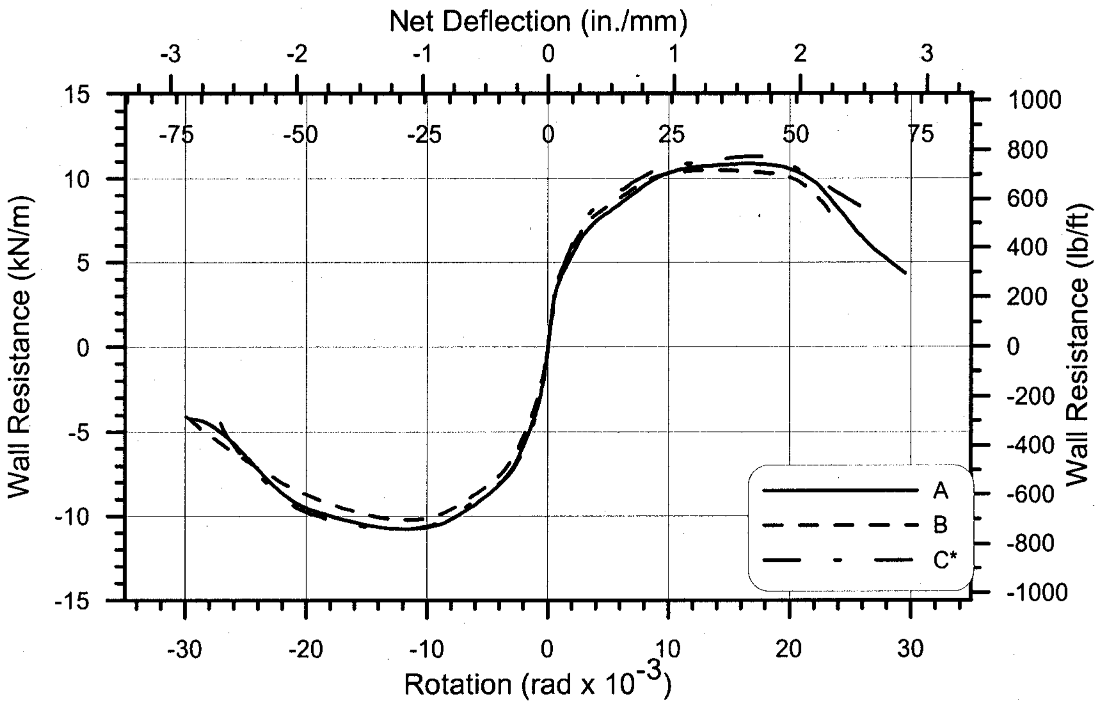
50C A	30.44	29.34
50C B	27.42	26.47



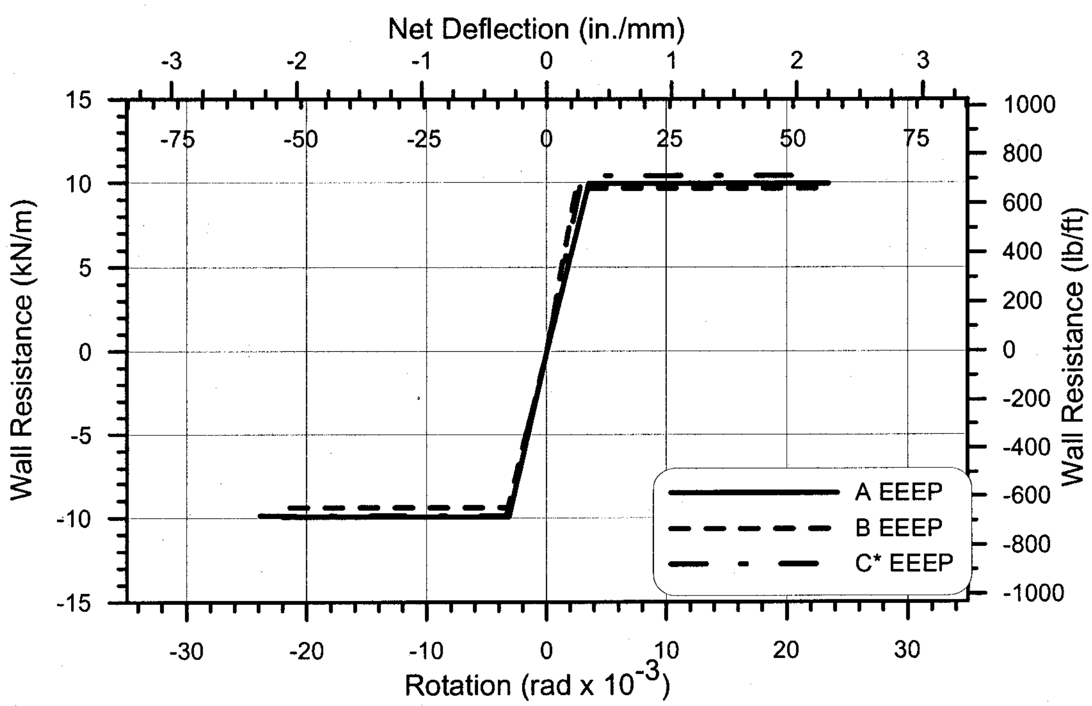
Max Load 13.579 kN
Max Displ = 77.018 mm

Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pull through (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Test 50-A,B,C Backbone
(1220 x 2440 mm OSB 152/305 mm)



Test 50-A,B,C EEEP
(1220 x 2440 mm OSB 152/305 mm)



Light Gauge Steel Frame / Wood Panel Shear Walls
McGill University, Montreal

TEST: 51A

RESEARCHER: KATHERINE HIKITA **ASSISTANTS:** M. OUELLET, JIANG FAN, M. ALKHARAT

DATE: Built: May 16; Tested: Friday May 17, 2005 **TIME:** 16:00

DIMENSIONS OF WALL: 4 FT X 8 FT **PANEL ORIENTATION:** Vertical
Sheathing one side

SHEATHING:
- [ ] Plywood 15/32" APA Rated Exposure 1 (USA)
- [ ] OSB 7/16" APA Rated Exposure 1 (USA)
- [ ] Plywood (CSA 0151M) CSP 12.5mm (1/2") — CAN/PLY Exterior CSP
- [ ] Plywood (CSA O121M) DFP 12.5mm (1/2") — CAN/PLY Exterior DFP
- [X] OSB (CSA O325) 11 mm (7/16")
- [ ] Other — MFR: Grant Forest Products Corp. Englehart, ON

SCREWS Sheathing:
- [X] No.8 gauge 1.5" self-piercing Bugle head LOX drive (Grabber Superdrive)
- [ ] No.8 gauge 1.0" self-piercing Bugle head (Flat socket head screw) (HD)
- [ ] No.8 gauge 1.25" self-drilling Bugle head LOX Drive (Grabber Super
- [ ] No.9 gauge 1.0" self-piercing Bugle head (screws Y2,Y12 in track)

Framing: [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive
Hold downs: [X] No.10 gauge 0.75" self-drilling Hex washer head
Loading Beam: A325 3/4" bolts — 3 bolts [ ] 6 bolts [X] 12 bolts [ ]
Back-to-Back Chord Studs: [X] No.10 gauge 0.75" self-drilling Hex washer head (2@12" O.C.)

SHEATHING FASTENER SCHEDULE: [ ] 2"/12" [X] 3"/12" [ ] 4"/12" [ ] 6"/12" [ ] Other:

EDGE PANEL DISTANCE: [ ] 3/8" [X] 1/2" [ ] Other:

STUDS: Field / Chord
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.054" (1.37mm) 50ksi (340 MF
- [X] Double chord studs used
- [ ] Other

STUD SPACING:
- [ ] 12" O.C.
- [ ] 16" O.C.
- [X] 24" O.C.
- [ ] Other:

TRACK: Web: 3-5/8" inches; Flange: 1-1/4" inches
- [X] 0.043" (1.09mm) 33ksi (230 MPa)
- [ ] Other:

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10 7/8" Anchor Rod (# of screws): 33
- [ ] UCI 18" hold down 1/2" Anchor Rod (# of screws):
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [X] Monotonic — 7.5mm/min.
- [ ] Cyclic — CUREE reversed cyclic (10 mm/s)
- [X] Gravity — 9 kN load at both ends of the wall

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral
- [X] East Frame Brace
- [X] West Frame Brace
- [X] Wood Shear

TOTAL: 9

MOISTURE CONTENT OF SHEATHING: Moisture Meter

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

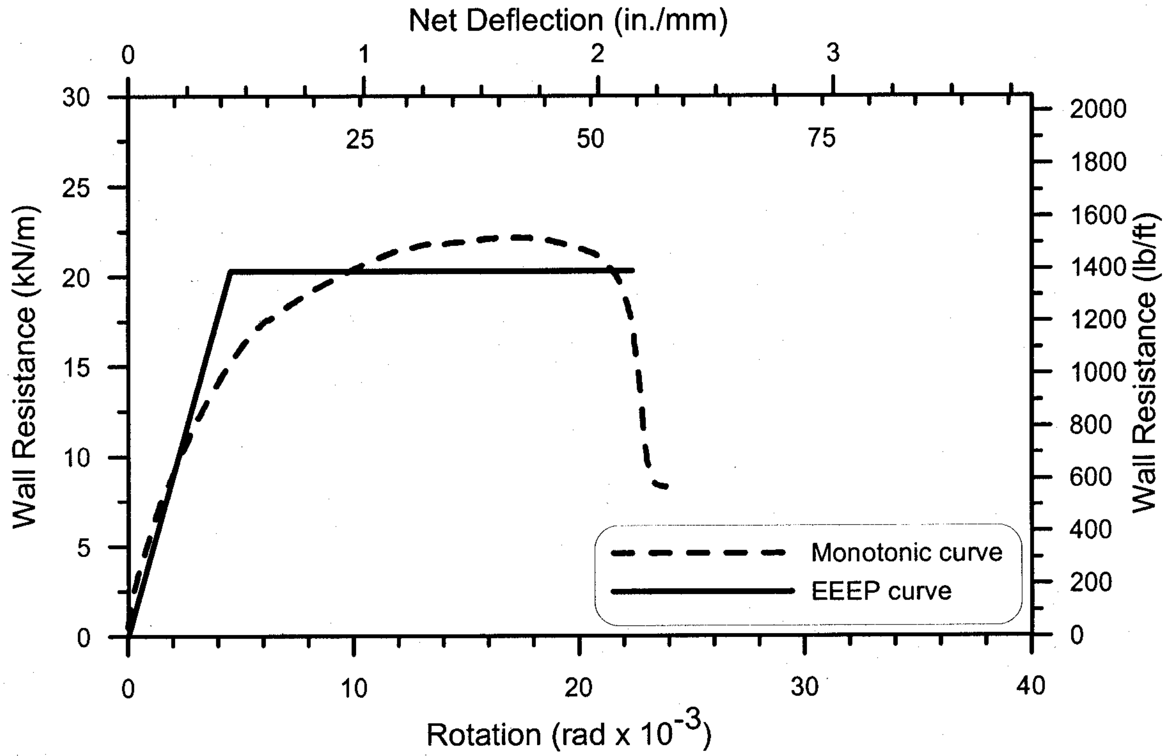
	North	North
Ww=	29.41	30.13
Wd=	28.36	29.06
m.c.=	3.70	3.68

AVG m.c. 3.69

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 50 scan/sec

COMMENTS:
-Shear anchors torqued to 77 kN
-Hold down anchors tightened to approximately 8 kN (load cells used on both hold-downs)
-Ambient temperature 23 C
-Double chord studs used
-Square plate washers (2.5"x2.5") used in all track connections
-Pilot holes drilled for screws A1,A5,A9,Q1,Q5,Q9
-Initial load set to zero at beginning of test, displacement -0.129 mm
- Offset after gravity loads are engaged -1.721 mm
- Screws A5, A4 and A2 were replaced during installation

Test 51A
(1220 x 2440 mm OSB 75/305 mm)



	Parameters	Units
F_u	27.03	kN
$F_{0.8u}$	21.62	kN
$F_{0.4u}$	10.81	kN
F_y	24.71	kN
K_e	2.25	kN/mm
Ductility (μ)	4.96	-
$\Delta_{net,y}$	10.98	mm
$\Delta_{net,u}$	42.01	mm
$\Delta_{net,0.8u}$	54.42	mm
$\Delta_{net,0.4u}$	4.80	mm
$Area_{Backbone}$	1209.08	J
$Area_{EEEP}$	1209.08	J
Check	OK	
R_d	**2.99**	-
S_y	**20.27**	kN/m

Test Name	51A
Date Tested	May 17, 2005
Protocol	MONOTONIC

McGill Gravity load included

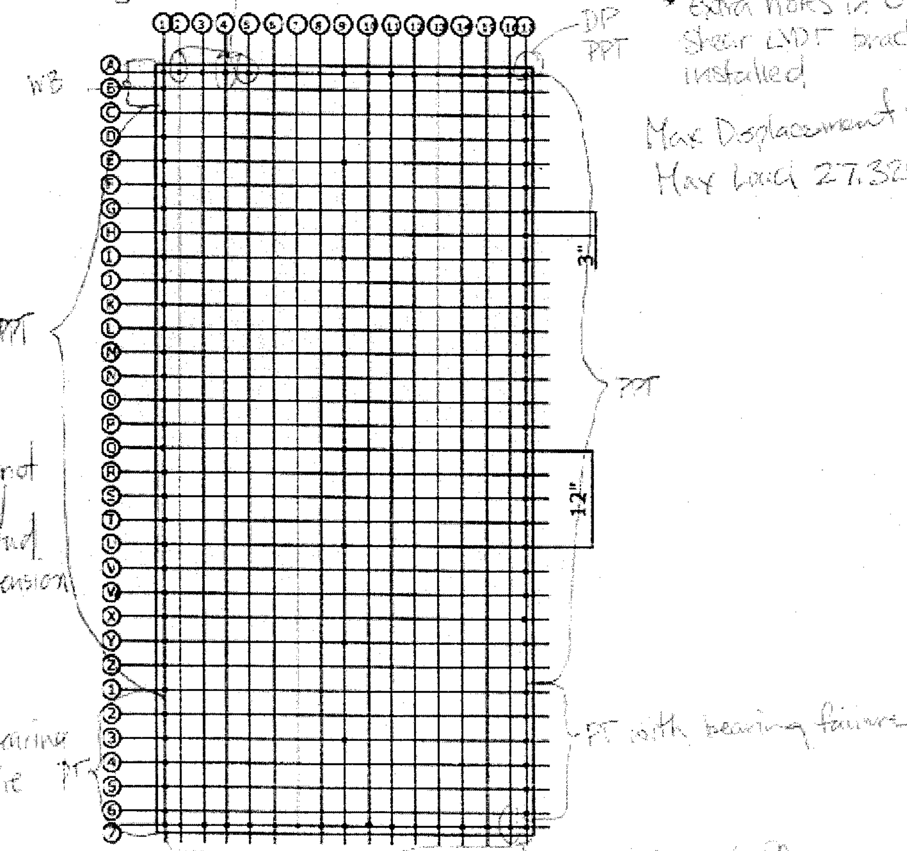
Light Gauge Steel Frame / Wood Panel Shear Wall Testing

Moisture Content A: 29.41 → 28.36g
B: 30.13 → 29.06g

Test name : 51A
Date tested : May 17, 2005
Wall Size : 4' x 8'
Screw pattern : 3" / 12"
Edge Distance : 1/2"
Test mode : ☐ Cyclic ☒ Monotonic

* extra holes in OSB where Shear LVDT brackets were installed

Max Displacement: 69.467 mm
Max Load 27.329 kN



Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Light Gauge Steel Frame / Wood Panel Shear Walls
McGill University, Montreal

TEST: 51B

RESEARCHER: KATHERINE HIKITA **ASSISTANTS:** M. OUELLET, JIANG FAN, M. ALKHARAT

DATE: Built: May 11; Tested: Friday May 18, 2005 **TIME:** 10:00

DIMENSIONS OF WALL: 4 FT X 8 FT **PANEL ORIENTATION:** Vertical
Sheathing one side

SHEATHING:
- [ ] Plywood 15/32" APA Rated Exposure 1 (USA)
- [ ] OSB 7/16" APA Rated Exposure 1 (USA)
- [ ] Plywood (CSA 0151M) CSP 12.5mm (1/2") CAN/PLY Exterior CSP
- [ ] Plywood (CSA O121M) DFP 12.5mm (1/2") CAN/PLY Exterior DFP
- [X] OSB (CSA O325) 11 mm (7/16")
- [ ] Other MFR: Grant Forest Products Corp. Englehart, ON

SCREWS Sheathing:
- [X] No.8 gauge 1.5" self-piercing Bugle head LOX drive (Grabber Superdrive)
- [ ] No.8 gauge 1.0" self-piercing Bugle head (Flat socket head screw) (HD)
- [ ] No.8 gauge 1.25" self-drilling Bugle head LOX Drive (Grabber Super
- [ ] No.9 gauge 1.0" self-piercing Bugle head (screws Y2,Y12 in track)

Framing: [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive
Hold downs: [X] No.10 gauge 0.75" self-drilling Hex washer head
Loading Beam: A325 3/4" bolts 3 bolts [ ] 6 bolts [X] 12 bolts [ ]
Back-to-Back Chord Studs: [X] No.10 gauge 0.75" self-drilling Hex washer head (2@12" O.C.)

SHEATHING FASTENER SCHEDULE: [ ] 2"/12" [X] 3"/12" [ ] 4"/12" [ ] 6"/12" [ ] Other:

EDGE PANEL DISTANCE: [ ] 3/8" [X] 1/2" [ ] Other:

STUDS: Field / Chord
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.054" (1.37mm) 50ksi (340 MF
- [X] Double chord studs used
- [ ] Other

STUD SPACING:
- [ ] 12" O.C.
- [ ] 16" O.C.
- [X] 24" O.C. [ ] Other:

TRACK: Web: 3-5/8" inches Flange: 1-1/4" inches [X] 0.043" (1.09mm) 33ksi (230 MPa) [ ] Other:

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10 7/8" Anchor Rod (# of screws): 33
- [ ] UCI 18" hold down 1/2" Anchor Rod (# of screws):
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [X] Monotonic 7.5mm/min.
- [ ] Cyclic CUREE reversed cyclic (10 mm/s)
- [ ] Gravity 9 kN load at both ends of the wall

LVDT MEASUREMENTS:
- [X] Actuator LVDT [X] North Uplift [X] East Frame Brace
- [X] North Slip [X] South Uplift [X] West Frame Brace
- [X] South Slip [X] Top of Wall Lateral [X] Wood Shear

TOTAL: 9

MOISTURE CONTENT OF SHEATHING: Moisture Meter

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

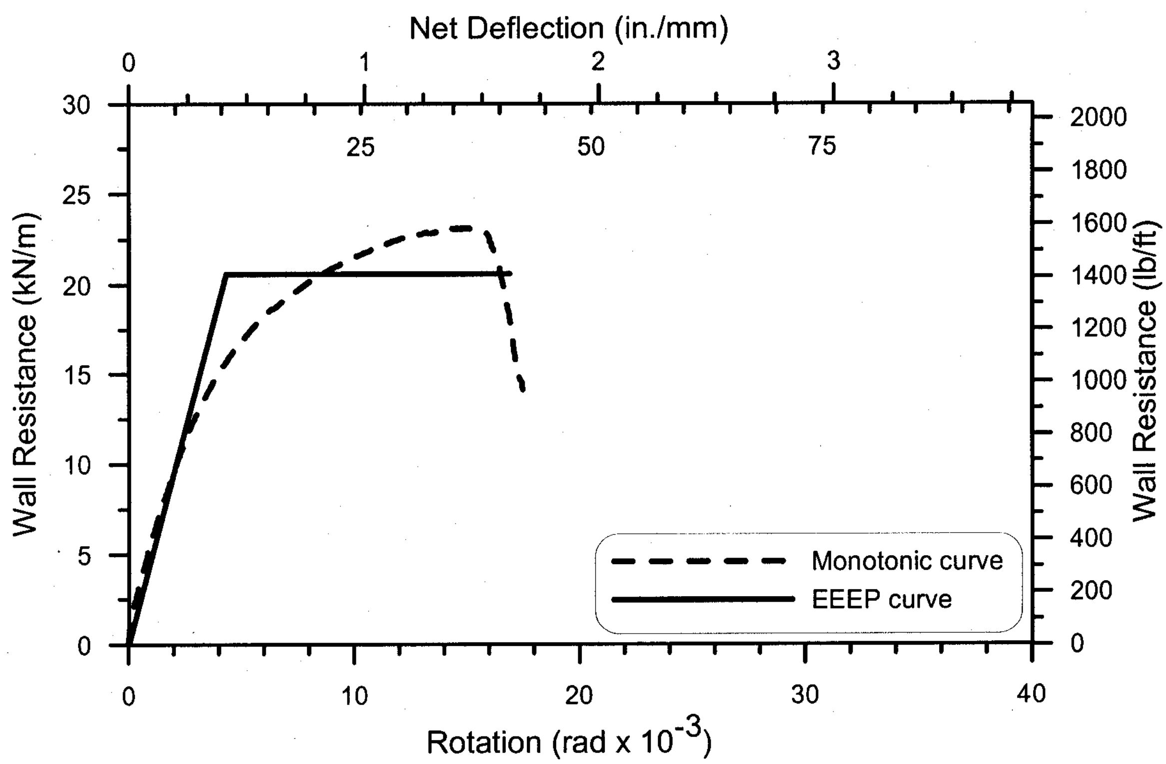
Ww=	26.66	26.58
Wd=	25.76	25.66
m.c.=	3.49	3.59
	North	North

AVG m.c. 3.54

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 50 scan/sec

COMMENTS:
-Shear anchors torqued to 77 kN
-Hold down anchors tightened to approximately 8 kN (load cells used on both hold-downs)
-Ambient temperature 23 C
-Double chord studs used
-Square plate washers (2.5"x2.5") used in all track connections
-Pilot holes drilled for screws A1,A5,A9,Q1,Q5,Q9
-Initial load set to zero at beginning of test, displacement -0.516 mm
- Gravity loads not included
- Screws A5, A4 and A2 were replaced during installation

Test 51B
(1220 x 2440 mm OSB 75/305 mm)



	Parameters	Units
F_u	28.18	kN
$F_{0.8u}$	22.54	kN
$F_{0.4u}$	11.27	kN
F_y	25.11	kN
K_e	2.41	kN/mm
Ductility (μ)	3.94	-
$\Delta_{net,y}$	10.43	mm
$\Delta_{net,u}$	36.66	mm
$\Delta_{net,0.8u}$	41.12	mm
$\Delta_{net,0.4u}$	4.68	mm
$Area_{Backbone}$	901.76	J
$Area_{EEEP}$	901.76	J
Check	OK	
R_d	**2.62**	-
S_y	**20.60**	kN/m

Test Name	51B
Date Tested	May 18, 2005
Protocol	MONOTONIC

McGill

(Gravity Loads not included)

Light Gauge Steel Frame / Wood Panel Shear Wall Testing

Moisture Content A: 26.66 → 25.76

B: 26.58 → 25.66

Test name : 5B

Date tested : May 18, 2005

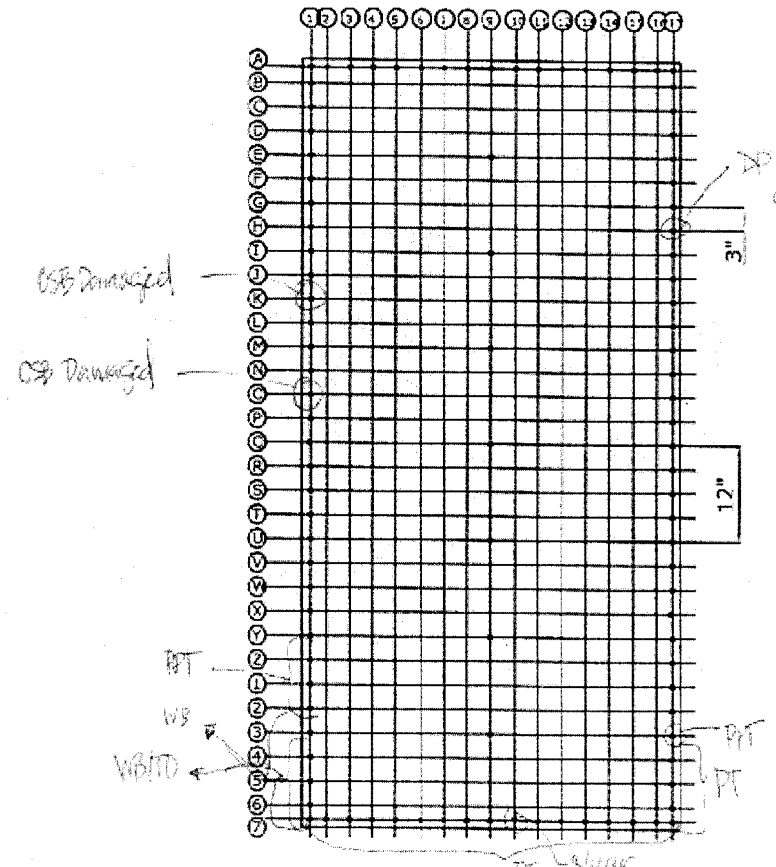
Wall Size : 4' x 8'

Screw pattern : 3" / 12"

Edge Distance : 1/2"

Test mode : ☐ Cyclic ☒ Monotonic

Max Load 28.186 kN



Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Light Gauge Steel Frame / Wood Panel Shear Walls
McGill University, Montreal

TEST: 51C

RESEARCHER: KATHERINE HIKITA **ASSISTANTS:** M. OUELLET, D. MORELLO
R. CARBONNEAU

DATE: Built: May 11; Tested: Wednesday May 18, 2005 **TIME:** 16:00

DIMENSIONS OF WALL: 4 FT X 8 FT **PANEL ORIENTATION:** Vertical
Sheathing one side

SHEATHING:
[ ] Plywood 15/32" APA Rated Exposure 1 (USA)
[ ] OSB 7/16" APA Rated Exposure 1 (USA)
[ ] Plywood (CSA 0151M) CSP 12.5mm (1/2") CAN/PLY Exterior CSP
[ ] Plywood (CSA O121M) DFP 12.5mm (1/2") CAN/PLY Exterior DFP
[X] OSB (CSA O325) 11 mm (7/16")
[ ] Other MFR: Grant Forest Products Corp. Englehart, ON

SCREWS Sheathing:
[X] No.8 gauge 1.5" self-piercing Bugle head LOX drive (Grabber Superdrive)
[ ] No.8 gauge 1.0" self-piercing Bugle head (Flat socket head screw) (HD)
[ ] No.8 gague 1.25" self-drilling Bugle head LOX Drive (Grabber Super
[ ] No.9 gauge 1.0" self-piercing Bugle head (screws Y2,Y12 in track)

Framing: [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive
Hold downs: [X] No.10 gauge 0.75" self-drilling Hex washer head
Loading Beam: A325 3/4" bolts 3 bolts [ ] 6 bolts [X] 12 bolts [ ]
Back-to-Back
Chord Studs: [X] No.10 gauge 0.75" self-drilling Hex washer head (2@12" O.C.)

SHEATHING FASTENER SCHEDULE: [ ] 2"/12" [X] 3"/12" [ ] 4"/12" [ ] 6"/12" [ ] Other:

EDGE PANEL DISTANCE: [ ] 3/8" [X] 1/2" [ ] Other:

STUDS: Field / Chord
[X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
[ ] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
[X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.054" (1.37mm) 50ksi (340 MF
[X] Double chord studs used
[ ] Other

STUD SPACING:
[ ] 12" O.C.
[ ] 16" O.C.
[X] 24" O.C. [ ] Other:

TRACK: Web: 3-5/8" inches Flange: 1-1/4" inches [X] 0.043" (1.09mm) 33ksi (230 MPa) [ ] Other:

HOLD DOWNS:
[X] Simpson Strong-Tie S/HD10 7/8" Anchor Rod (# of screws): 33
[ ] UCI 18" hold down 1/2" Anchor Rod (# of screws):
[ ] Other

TEST PROTOCOL AND DESCRIPTION:
[X] Monotonic 7.5mm/min.
[ ] Cyclic CUREE reversed cyclic (10 mm/s)
[X] Gravity 9 kN load at both ends of the wall

LVDT MEASUREMENTS:
[X] Actuator LVDT [X] North Uplift [X] East Frame Brace
[X] North Slip [X] South Uplift [X] West Frame Brace
[X] South Slip [X] Top of Wall Lateral [X] Wood Shear
TOTAL: 9

MOISTURE CONTENT OF SHEATHING: Moisture Meter

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

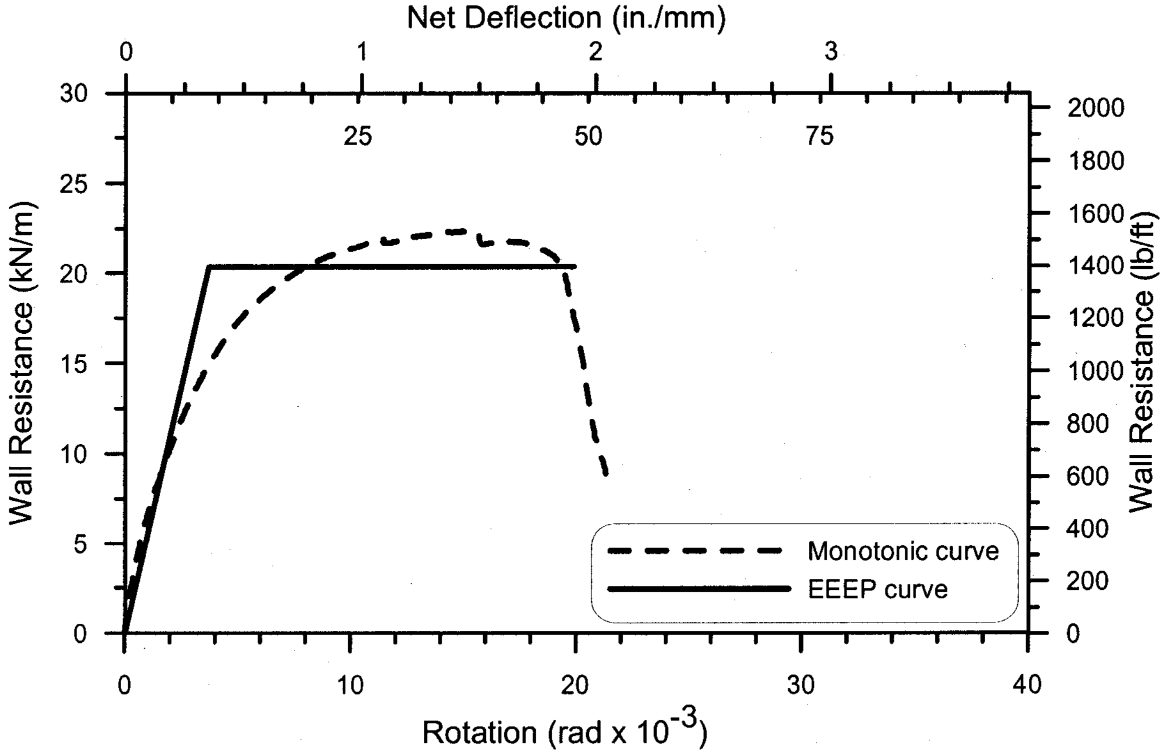
Ww=	26.15	31.40
Wd=	25.22	30.29
m.c.=	3.69	3.66
	North	North

AVG m.c. 3.68

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 50 scan/sec

COMMENTS:
-Shear anchors torqued to 77 kN
-Hold down anchors tightened to approximately 8 kN (load cells used on both hold-downs)
-Ambient temperature 23 C
-Double chord studs used
-Square plate washers (2.5"x2.5") used in all track connections
-Pilot holes drilled for screws A1,A5,A9,Q1,Q5,Q9
-Initial load set to zero at beginning of test, displacement -0.180 mm
-Intial engagement displacement after gravity loads are applied 0.062 mm

Test 51C
(1220 x 2440 mm OSB 75/305 mm)



	Parameters	Units
F_u	27.25	kN
$F_{0.8u}$	21.80	kN
$F_{0.4u}$	10.90	kN
F_y	24.83	kN
K_e	2.78	kN/mm
Ductility (μ)	5.43	-
$\Delta_{net,y}$	8.93	mm
$\Delta_{net,u}$	36.86	mm
$\Delta_{net,0.8u}$	48.43	mm
$\Delta_{net,0.4u}$	3.92	mm
$Area_{Backbone}$	1091.77	J
$Area_{EEEP}$	1091.77	J
Check	OK	
R_d	**3.14**	-
S_y	**20.37**	kN/m

Test Name	51C
Date Tested	May 18, 2005
Protocol	MONOTONIC

McGill Gravity loading included

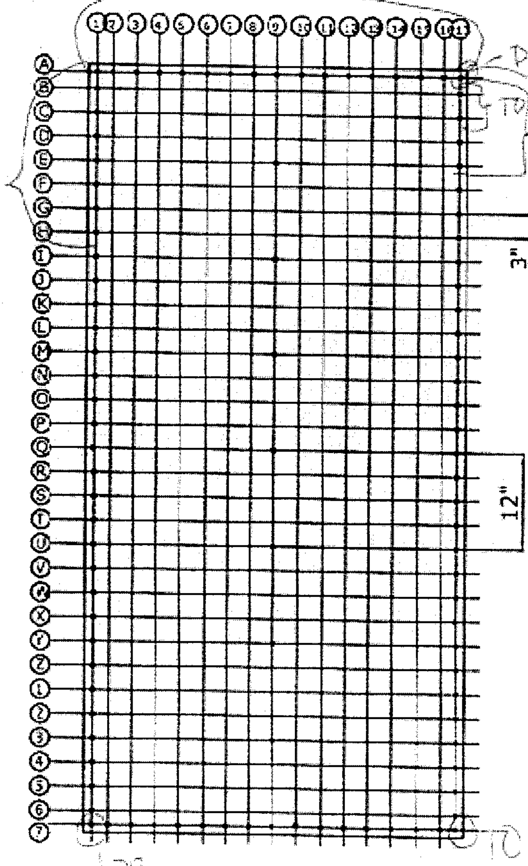
Light Gauge Steel Frame / Wood Panel Shear Wall Testing

Moisture Content: A 26.15 → 25.22g
B 31.40 → 30.29g

DP top ply's of OSB damaged due to difficulties installing screws → should reset tapping

TO, WB - OSB destroyed around screws

Test name :	51C
Date tested :	May 15 2005
Wall Size :	4' x 8'
Screw pattern :	3" / 12"
Edge Distance :	1/2"
Test mode :	☐ Cyclic ☒ Monotonic



TO/WB

DP to corner screw sheared

TO

WB/TO
plywood splintered

DP

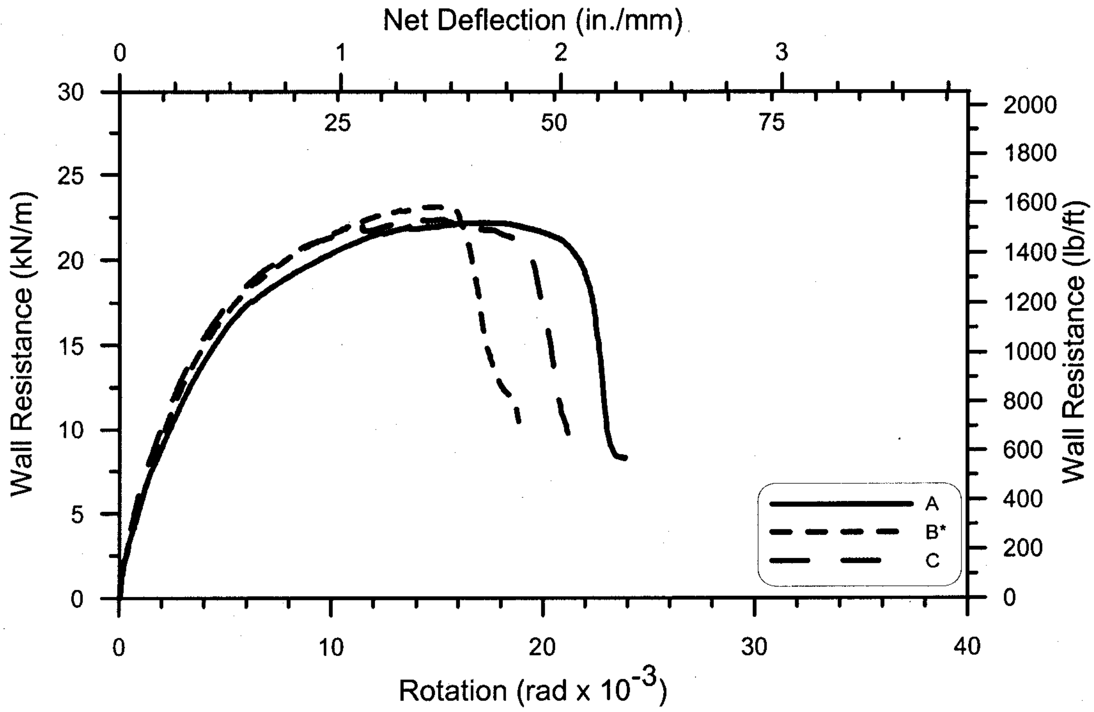
TO

Max load 27.539 kN

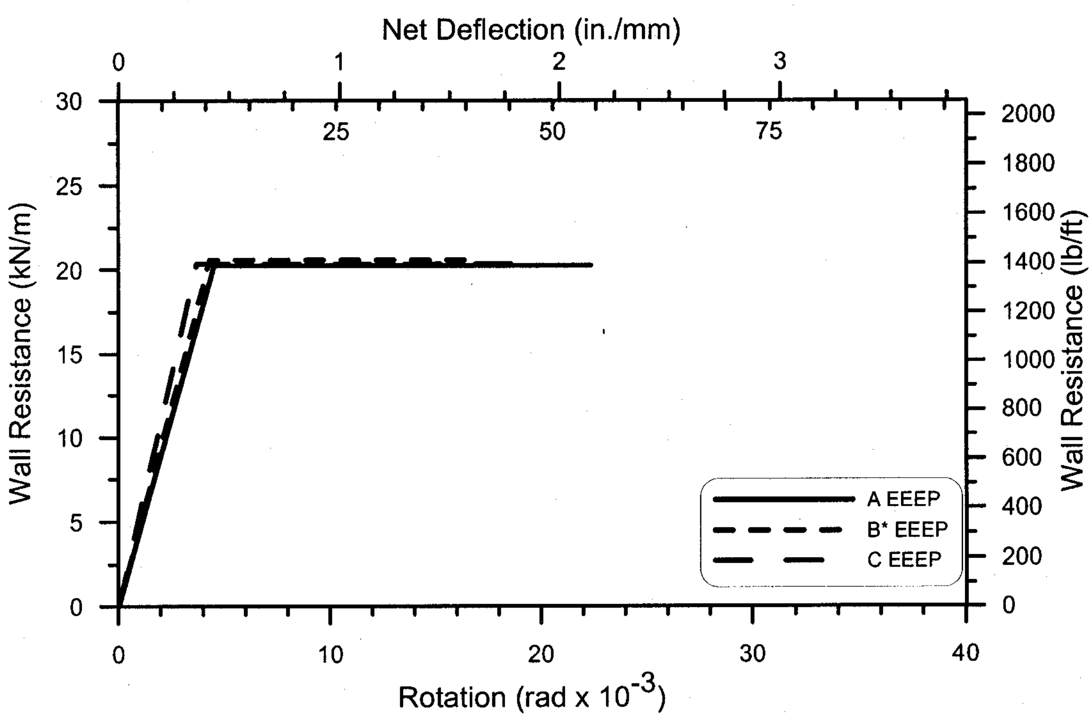
Max Displacement 65.691 mm

Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Test 51-A,B,C
(1220 x 2440 mm OSB 75/305 mm)



Test 51-A,B,C EEEP
(1220 x 2440 mm OSB 75/305 mm)



Light Gauge Steel Frame / Wood Panel Shear Walls
McGill University, Montreal

TEST: 52A

RESEARCHER: KATHERINE HIKITA **ASSISTANTS:** M. OUELLET, JIANG FAN, M. ALKHARAT

DATE: Built: May 11; Tested: Thursday May 19, 2005 **TIME:** 16:00

DIMENSIONS OF WALL: 4 FT X 8 FT **PANEL ORIENTATION:** Vertical
Sheathing one side

SHEATHING:
- [ ] Plywood 15/32" APA Rated Exposure 1 (USA)
- [ ] OSB 7/16" APA Rated Exposure 1 (USA)
- [ ] Plywood (CSA 0151M) CSP 12.5mm (1/2") CAN/PLY Exterior CSP
- [ ] Plywood (CSA O121M) DFP 12.5mm (1/2") CAN/PLY Exterior DFP
- [X] OSB (CSA O325) 11 mm (7/16")
- [ ] Other MFR: Grant Forest Products Corp. Englehart, ON

SCREWS Sheathing:
- [X] No.8 gauge 1.5" self-piercing Bugle head LOX drive (Grabber Superdrive)
- [ ] No.8 gauge 1.0" self-piercing Bugle head (Flat socket head screw) (HD)
- [ ] No.8 gauge 1.25" self-drilling Bugle head LOX Drive (Grabber S
- [ ] No.9 gauge 1.0" self-piercing Bugle head (screws Y2,Y12 in track)

Framing: [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive
Hold downs: [X] No.10 gauge 0.75" self-drilling Hex washer head
Loading Beam: A325 3/4" bolts 3 bolts [ ] 6 bolts [X] 12 bolts [ ]
Back-to-Back Chord Studs: [X] No.10 gauge 0.75" self-drilling Hex washer head (2@12" O.C.)

SHEATHING FASTENER SCHEDULE: [ ] 2"/12" [X] 3"/12" [ ] 4"/12" [ ] 6"/12" [ ] Other:

EDGE PANEL DISTANCE: [ ] 3/8" [X] 1/2" [ ] Other:

STUDS: Field / Chord
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.054" (1.37mm) 50ksi (340 MF
- [X] Double chord studs used
- [ ] Other

STUD SPACING:
- [ ] 12" O.C.
- [ ] 16" O.C.
- [X] 24" O.C. [ ] Other:

TRACK: Web: 3-5/8" inches; Flange: 1-1/4" inches [X] 0.043" (1.09mm) 33ksi (230 MPa) [ ] Other:

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10 7/8" Anchor Rod (# of screws): 33
- [ ] UCI 18" hold down 1/2" Anchor Rod (# of screws):
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [ ] Monotonic 7.5mm/min.
- [X] Cyclic CUREE reversed cyclic (10 mm/s)
- [X] Gravity 9 kN load at both ends of wall

LVDT MEASUREMENTS:
- [X] Actuator LVDT [X] North Uplift [X] East Frame Brace
- [X] North Slip [X] South Uplift [X] West Frame Brace
- [X] South Slip [X] Top of Wall Lateral [X] Wood Shear

TOTAL: 9

MOISTURE CONTENT OF SHEATHING: Moisture Meter

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

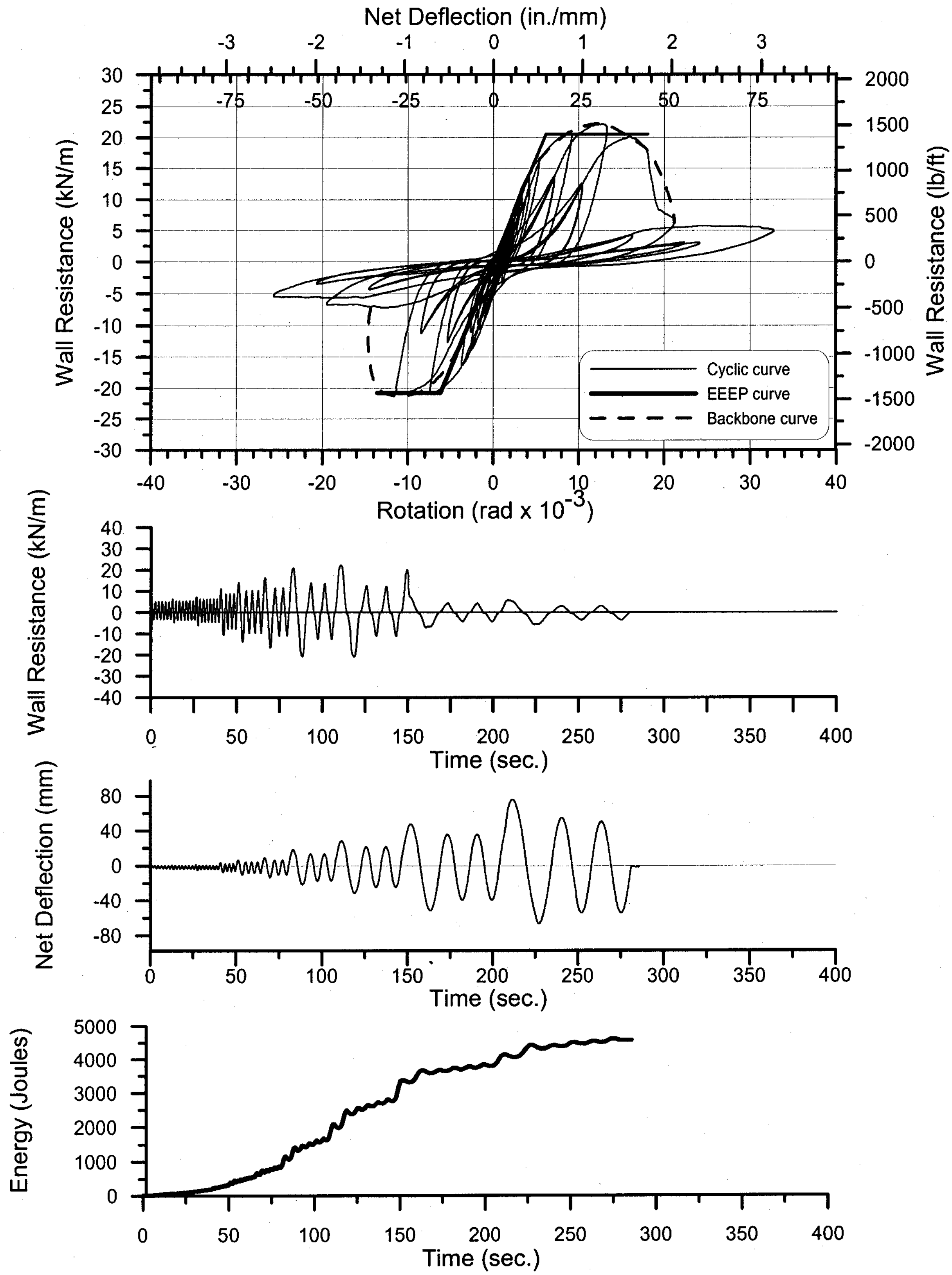
Ww=	26.54	28.49
Wd=	25.59	27.43
m.c.=	3.71	3.86
	North	North

AVG m.c. 3.79

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 50 scan/sec

COMMENTS:
-Shear anchors torqued to 77 kN
-Hold down anchors tightened to approximately 8 kN (load cells used on both hold-downs)
-Ambient temperature 23 C
-Double chord studs used
-Square plate washers (2.5"x2.5") used in all track connections
-Pilot holes drilled for screws A1,A5,A9,Q1,Q5,Q9
-Initial load set to zero at beginning of test, displacement -1.027 mm
-Intial engagement displacement after gravity loads are applied -1.42 mm
- Screw in A1 head sheared off during testing

Test 52A
(1220 x 2440 mm OSB 75/305 mm)



	Negative	Positive	Units
Fu	-25.39	27.04	kN
F0.8u	-20.31	21.63	kN
F0.4u	-10.16	10.82	kN
Fy	-25.33	25.06	kN
Ke	1.69	1.66	kN/mm
Ductility (μ)	2.22	2.92	-
Δnet,y	-14.96	15.06	mm
Δnet,u	-30.82	30.79	mm
Δnet,0.8u	-33.20	44.00	mm
Δnet,0.4u	-6.00	6.50	mm
AreaBackbor	651.32	914.02	kN-mm
AreaEEEP	651.32	914.02	kN-mm
Check	OK	OK	
Rd	**1.85**	**2.20**	-
Sy	**-20.77**	**20.56**	kN/m

Test Name	52A
Date Tested	May 19, 2005
Protocol	CUREE ORDINARY GROUND MOTIONS

McGill

Light Gauge Steel Frame / Wood Panel Shear Wall Testing

Test name : 52A
Date tested : May 19, 2005
Wall Size : 4' x 8'
Screw pattern : 3" / 12"
Edge Distance : 1/2"
Test mode : ☒ Cyclic ☐ Monotonic

Moisture Content: A: 26.54 → 25.59g
B: 28.99 → 27.43g

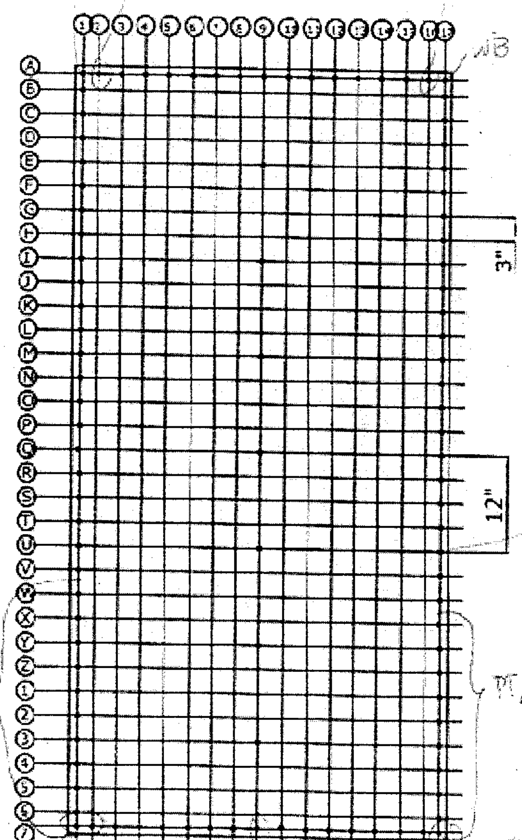
WB, PPO when driven

9&10 PT

WB

North and South Side is no good at end of test because they hit panel

Max Load 27.11 kN check.



buckled lip on outer chord around U1.

PT/WB

PT/WB - ply's destroyed around screw

Screw sheared off · Screw sheared off

Screw sheared

Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Light Gauge Steel Frame / Wood Panel Shear Walls
McGill University, Montreal

TEST: 52B

RESEARCHER: KATHERINE HIKITA **ASSISTANTS:** M. OUELLET, JIANG FAN, M. ALKHARAT, D. MORELLO

DATE: Built: June 2; Tested: Thursday June 2, 2005 **TIME:** 16:00

DIMENSIONS OF WALL: 4 FT X 8 FT **PANEL ORIENTATION:** Vertical, Sheathing one side

SHEATHING:
- [ ] Plywood 15/32" APA Rated Exposure 1 (USA)
- [ ] OSB 7/16" APA Rated Exposure 1 (USA)
- [ ] Plywood (CSA 0151M) CSP 12.5mm (1/2") CAN/PLY Exterior CSP
- [ ] Plywood (CSA O121M) DFP 12.5mm (1/2") CAN/PLY Exterior DFP
- [X] OSB (CSA O325) 11 mm (7/16")
- [ ] Other MFR: Grant Forest Products Corp. Englehart, ON

SCREWS Sheathing:
- [ ] No.8 gauge 1.5" self-piercing Bugle head LOX drive (Grabber Superdrive)
- [ ] No.8 gauge 1.0" self-piercing Bugle head (Flat socket head screw) (HD)
- [X] No.8 gauge 1.25" self-drilling Bugle head LOX Drive (Grabber Superdrive)
- [ ] No.9 gauge 1.0" self-piercing Bugle head (screws Y2,Y12 in track)

Framing:
- [ ] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive

Hold downs:
- [X] No.10 gauge 0.75" self-drilling Hex washer head

Loading Beam: A325 3/4" bolts 3 bolts [ ] 6 bolts [X] 12 bolts [ ]

Back-to-Back Chord Studs:
- [X] No.10 gauge 0.75" self-drilling Hex washer head (2@12" O.C.)

SHEATHING FASTENER SCHEDULE: [ ] 2"/12" [X] 3"/12" [ ] 4"/12" [ ] 6"/12" [ ] Other:

EDGE PANEL DISTANCE: [ ] 3/8" [X] 1/2" [ ] Other:

STUDS: Field / Chord
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.054" (1.37mm) 50ksi (340 MF
- [X] Double chord studs used
- [ ] Other

STUD SPACING:
- [ ] 12" O.C.
- [ ] 16" O.C.
- [X] 24" O.C.
- [ ] Other:

TRACK: Web: 3-5/8" inches; Flange: 1-1/4" inches
- [X] 0.043" (1.09mm) 33ksi (230 MPa)
- [ ] Other:

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10 7/8" Anchor Rod (# of screws): 33
- [ ] UCI 18" hold down 1/2" Anchor Rod (# of screws):
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [ ] Monotonic 7.5mm/min.
- [X] Cyclic CUREE reversed cyclic (10 mm/s)
- [X] Gravity 9 kN load at both ends of wall

10 m

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral
- [X] East Frame Brace
- [X] West Frame Brace
- [X] Wood Shear

TOTAL: 9

MOISTURE CONTENT OF SHEATHING: Moisture Meter

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

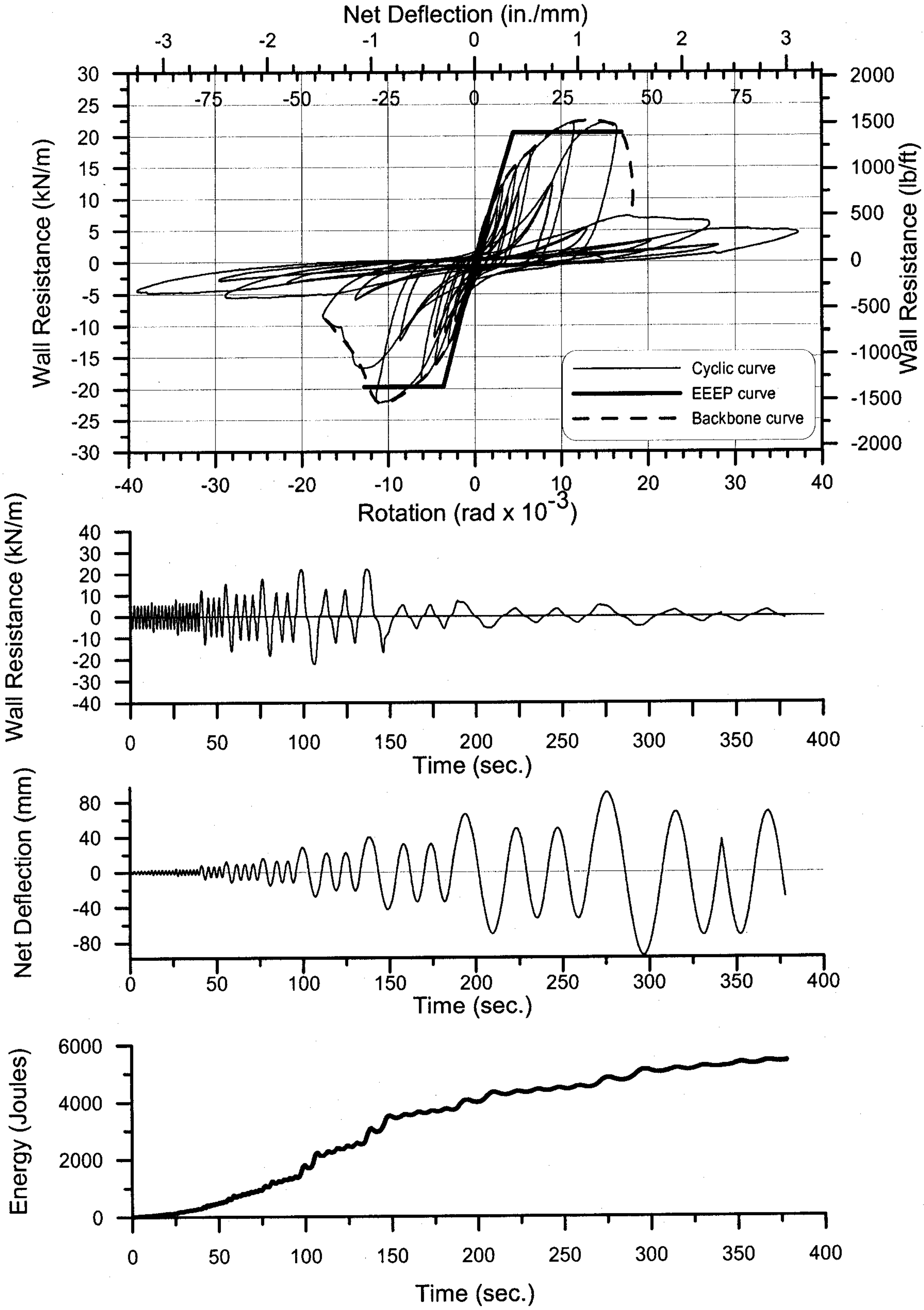
	North	North
Ww=	29.92	29.90
Wd=	28.81	28.84
m.c.=	3.85	3.68

AVG m.c. 3.76

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 50 scan/sec

COMMENTS:
-Shear anchors torqued to 77 kN
-Hold down anchors tightened to approximately 8 kN (load cells used on both hold-downs)
-Ambient temperature 23 C
-Double chord studs used
-Square plate washers (2.5"x2.5") used in all track connections
-Pilot holes drilled for screws A1,A5,A9,Q1,Q5,Q9
-Initial load set to zero at beginning of test, displacement 0.121 mm
-Displacement includes gravity loads
- Self Tapping screws used

Test 52B
(1220 x 2440 mm OSB 75/305 mm)



	Negative	Positive	Units
F_u	-27.10	26.97	kN
$F_{0.8u}$	-21.68	21.57	kN
$F_{0.4u}$	-10.84	10.79	kN
F_y	-23.99	25.00	kN
K_e	2.71	2.30	kN/mm
Ductility (μ)	3.52	3.81	-
$\Delta_{net,y}$	-8.85	10.89	mm
$\Delta_{net,u}$	-27.10	37.11	mm
$\Delta_{net,0.8u}$	-31.20	41.50	mm
$\Delta_{net,0.4u}$	-4.00	4.70	mm
$Area_{Backbone}$	642.27	901.26	kN-mm
$Area_{EEEP}$	642.27	901.26	kN-mm
Check	OK	OK	
R_d	**2.46**	**2.57**	-
S_y	**-19.68**	**20.50**	kN/m

Test Name	52B
Date Tested	June 2, 2005
Protocol	CUREE ORDINARY GROUND MOTIONS

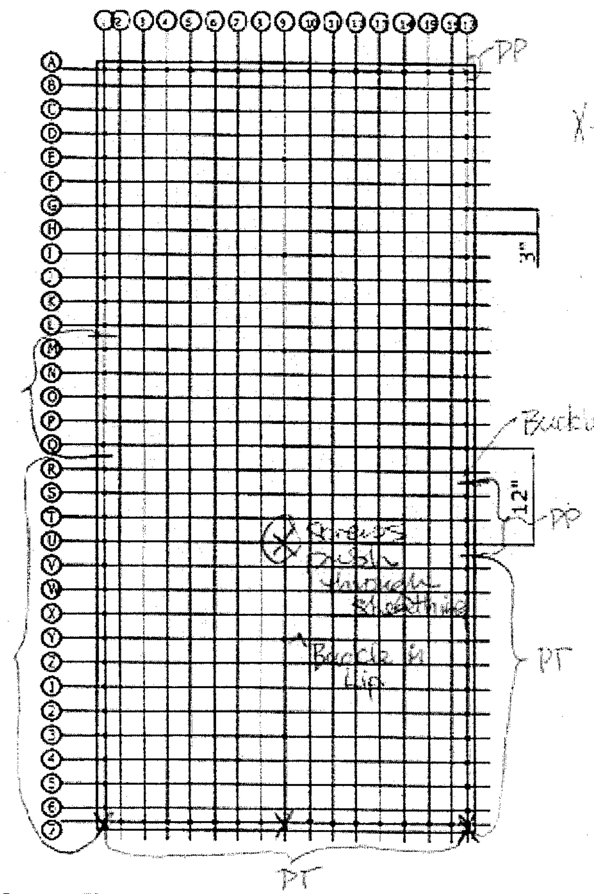
McGill + gravity loads
1.85" screws used

Light Gauge Steel Frame / Wood Panel Shear Wall Testing

Moisture Content: A: 29.92 → 28.81g
B: 29.90 → 28.84g

Test name: 52-B
Date tested: June 2, 2005
Wall Size: 4' x 8'
Screw pattern: 3" / 12"
Edge Distance: 1/2"
Test mode: ☒ Cyclic ☐ Monotonic

Max Load
+27.2 kN −27.3 kN
X - sheared screws



Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Light Gauge Steel Frame / Wood Panel Shear Walls
McGill University, Montreal

TEST: 52C

RESEARCHER: KATHERINE HIKITA **ASSISTANTS:** M. OUELLET, JIANG FAN, M. ALKHARAT
D. MORELLO

DATE: Built: June 2; Tested: Thursday June 2, 2005 **TIME:** 16:00

DIMENSIONS OF WALL: 4 FT X 8 FT **PANEL ORIENTATION:** Vertical
Sheathing one side

SHEATHING:
- [ ] Plywood 15/32" APA Rated Exposure 1 (USA)
- [ ] OSB 7/16" APA Rated Exposure 1 (USA)
- [ ] Plywood (CSA 0151M) CSP 12.5mm (1/2") CAN/PLY Exterior CSP
- [ ] Plywood (CSA O121M) DFP 12.5mm (1/2") CAN/PLY Exterior DFP
- [X] OSB (CSA O325) 11 mm (7/16")
- [ ] Other MFR: Grant Forest Products Corp. Englehart, ON

SCREWS Sheathing:
- [ ] No.8 gauge 1.5" self-piercing Bugle head LOX drive (Grabber Superdrive)
- [ ] No.8 gauge 1.0" self-piercing Bugle head (Flat socket head screw) (HD)
- [X] No. 8 gauge 1.25" self-drilling Bugle head LOX Drive (Grabber Superdrive)
- [ ] No.9 gauge 1.0" self-piercing Bugle head (screws Y2,Y12 in track)

Framing:
- [ ] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive

Hold downs:
- [X] No.10 gauge 0.75" self-drilling Hex washer head

Loading Beam: A325 3/4" bolts 3 bolts [ ] 6 bolts [X] 12 bolts [ ]

Back-to-Back Chord Studs:
- [X] No.10 gauge 0.75" self-drilling Hex washer head (2@12" O.C.)

SHEATHING FASTENER SCHEDULE: [ ] 2"/12" [X] 3"/12" [ ] 4"/12" [ ] 6"/12" [ ] Other:

EDGE PANEL DISTANCE: [ ] 3/8" [X] 1/2" [ ] Other:

STUDS: Field / Chord
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.054" (1.37mm) 50ksi (340 MF
- [X] Double chord studs used
- [ ] Other

STUD SPACING:
- [ ] 12" O.C.
- [ ] 16" O.C.
- [X] 24" O.C.
- [ ] Other:

TRACK: Web: 3-5/8" inches; Flange: 1-1/4" inches
- [X] 0.043" (1.09mm) 33ksi (230 MPa)
- [ ] Other:

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10 7/8" Anchor Rod (# of screws): 33
- [ ] UCI 18" hold down 1/2" Anchor Rod (# of screws):
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [ ] Monotonic 7.5mm/min.
- [X] Cyclic CUREE reversed cyclic (10 mm/s)
- [ ] Gravity 9 kN load at both ends of wall

10 m

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral
- [X] East Frame Brace
- [X] West Frame Brace
- [X] Wood Shear

TOTAL: 9

MOISTURE CONTENT OF SHEATHING: Moisture Meter

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

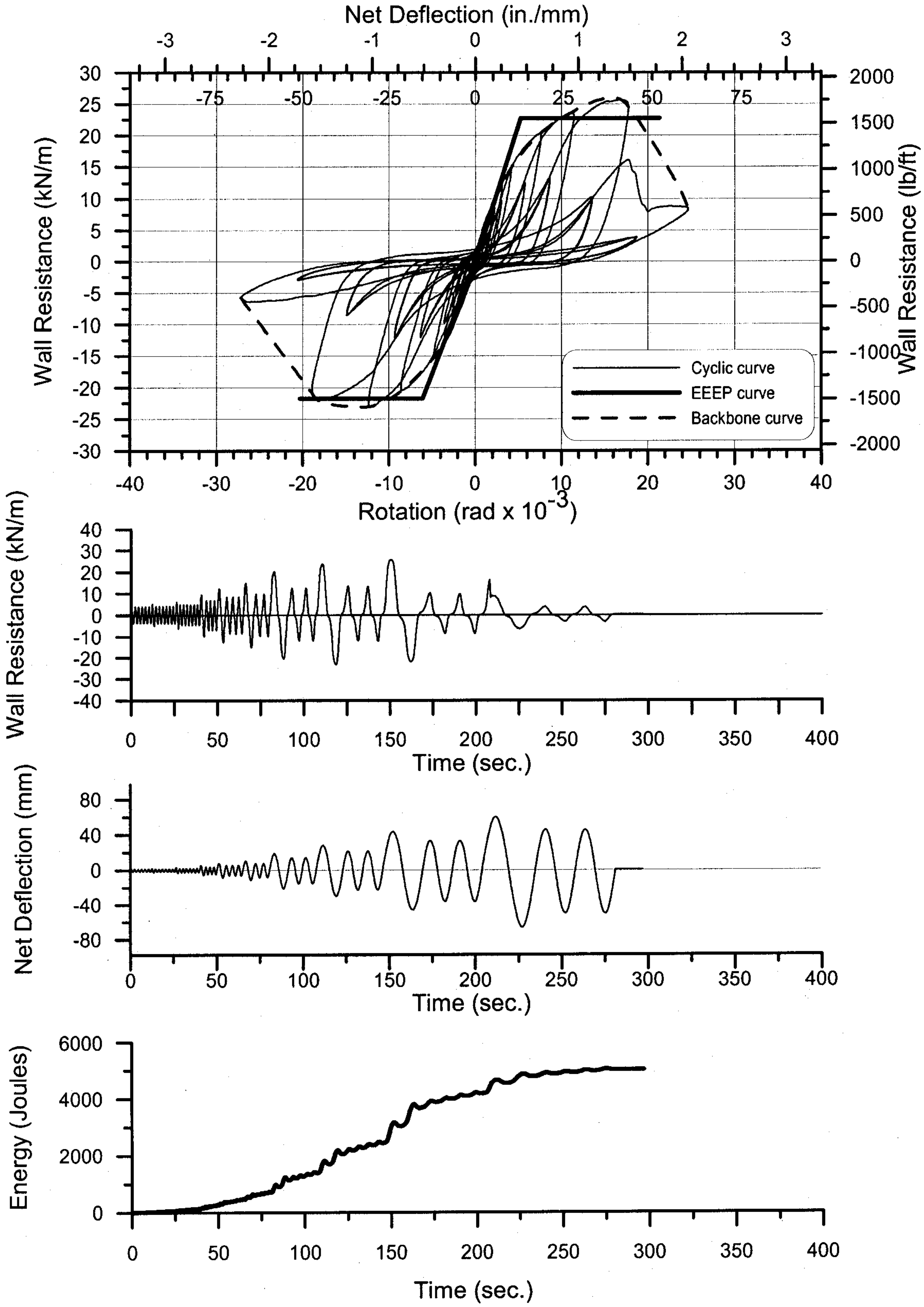
	North	North
Ww=	26.20	29.03
Wd=	25.12	27.90
m.c.=	4.30	4.05

AVG m.c. 4.17

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 50 scan/sec

COMMENTS:
-Shear anchors torqued to 77 kN
-Hold down anchors tightened to approximately 8 kN (load cells used on both hold-downs)
-Ambient temperature 23 C
-Double chord studs used
-Square plate washers (2.5"x2.5") used in all track connections
-Pilot holes drilled for screws A1,A5,A9,Q1,Q5,Q9
-Initial load set to zero at beginning of test, displacement 0.121 mm
-No gravity loads
- Self Tapping screws used

Test 52C
(1220 x 2440 mm OSB 75/305 mm)



	Negative	Positive	Units
F_u	-28.03	31.26	kN
$F_{0.8u}$	-22.43	25.01	kN
$F_{0.4u}$	-11.21	12.50	kN
F_y	-26.53	27.67	kN
K_e	1.78	2.16	kN/mm
Ductility (μ)	3.33	4.06	-
$\Delta_{net,y}$	-14.90	12.84	mm
$\Delta_{net,u}$	-29.54	41.25	mm
$\Delta_{net,0.8u}$	-49.60	52.10	mm
$\Delta_{net,0.4u}$	-6.30	5.80	mm
$Area_{Backbone}$	1118.07	1263.97	kN-mm
$Area_{EEEP}$	1118.07	1263.97	kN-mm
Check	OK	OK	
R_d	**2.38**	**2.67**	-
S_y	**-21.76**	**22.69**	kN/m

Test Name	52C
Date Tested	June 2, 2005
Protocol	CUREE ORDINARY GROUND MOTIONS

McGill *No gravity loads

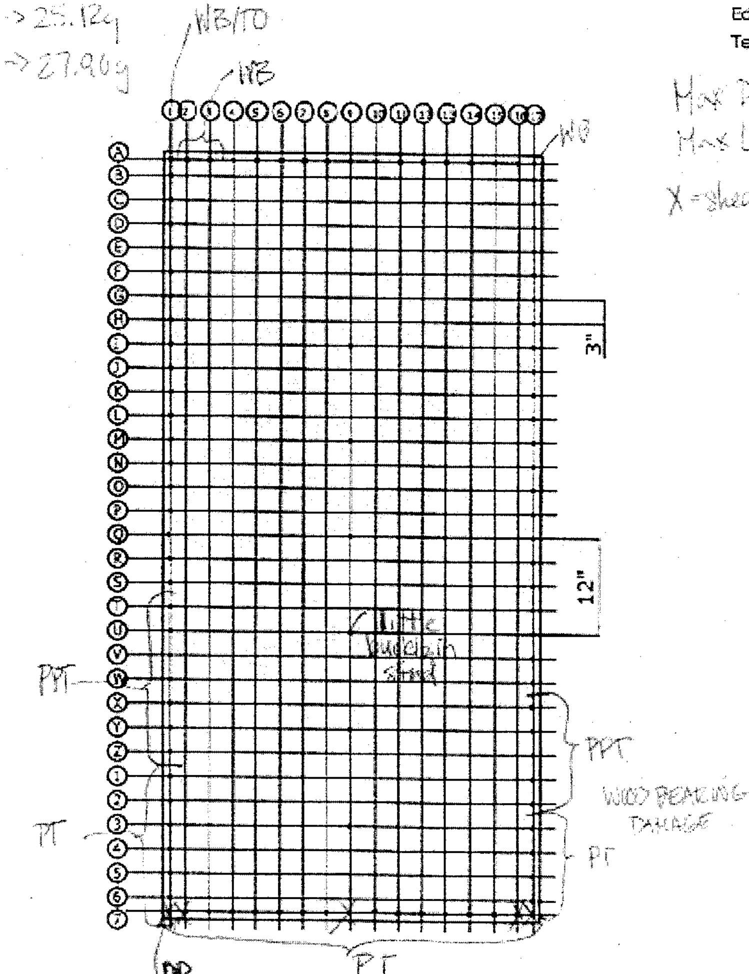
Light Gauge Steel Frame / Wood Panel Shear Wall Testing

Test name : 52C
Date tested : June 2, 2005
Wall Size : 4' x 8'
Screw pattern : 3" / 12'
Edge Distance : 1/2"
Test mode : ☒ Cyclic ☐ Monotonic

Moisture Content A: 26.20g -> 25.12g
B: 29.03g -> 27.90g

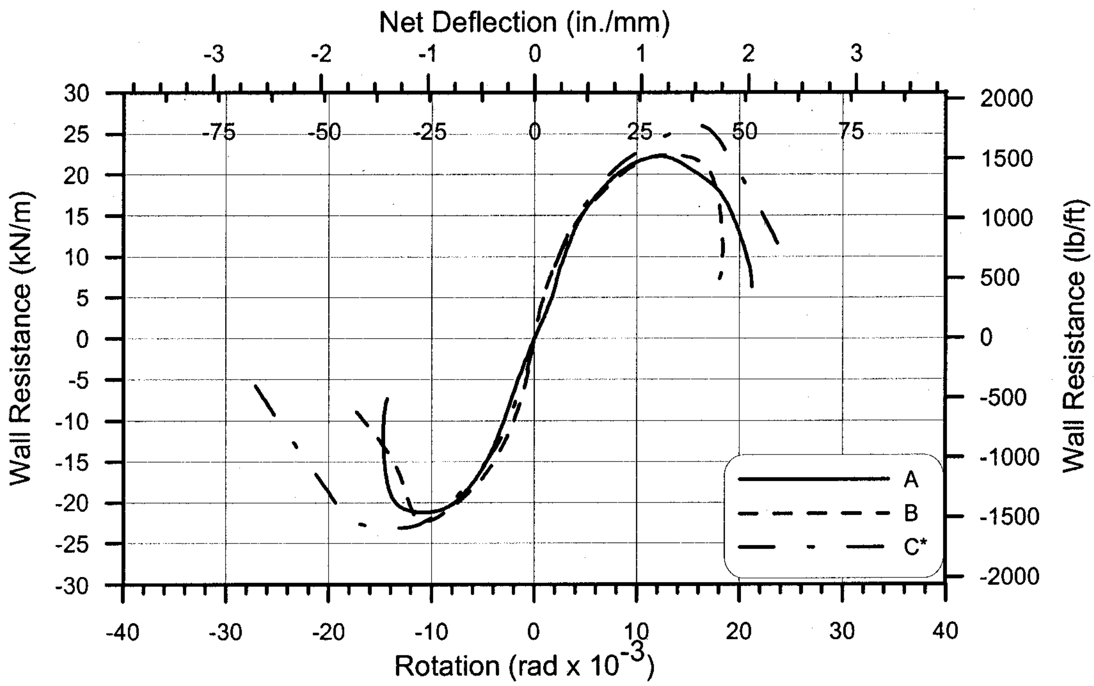
Max Displacement 77.232 mm
Max Load 31.313 kN

X = sheared screw

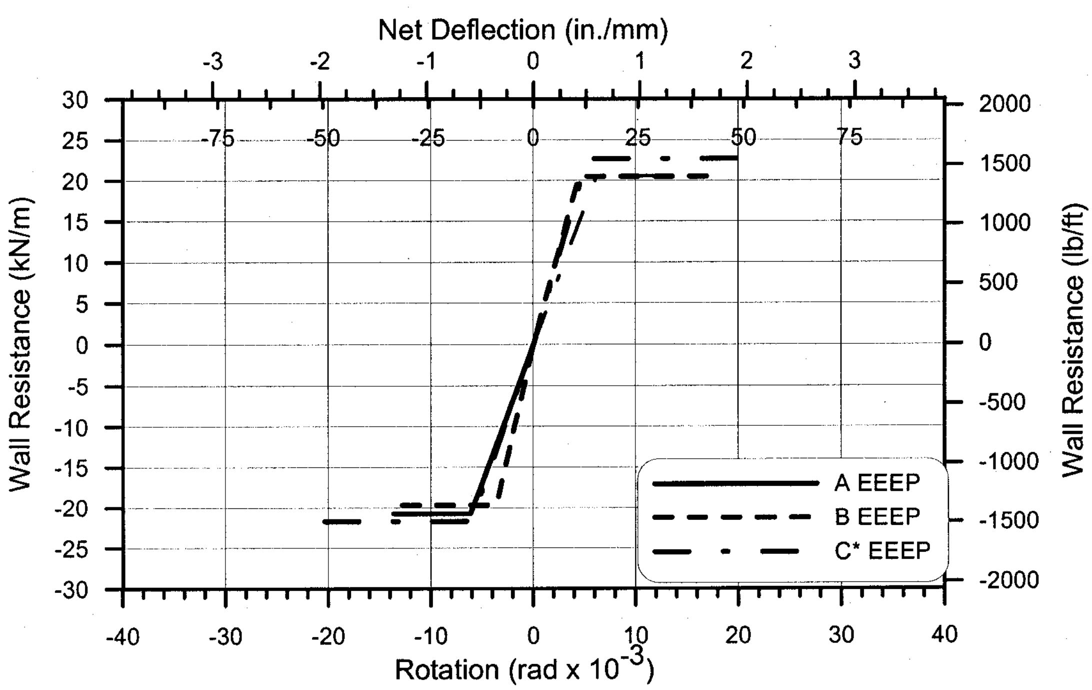


Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TC) ; Wood Bearing Failure (WB)

Test 52-A,B,C Backbone
(1220 x 2440 mm OSB 75/305 mm)



Test 52-A,B,C EEEP
(1220 x 2440 mm OSB 75/305 mm)



Light Gauge Steel Frame / Wood Panel Shear Walls
McGill University, Montreal

TEST: 53A

RESEARCHER: KATHERINE HIKITA **ASSISTANTS:** M. OUELLET, JIANG FAN

DATE: Built: May 11; Tested: Thursday May 12, 2005 **TIME:** 16:00

DIMENSIONS OF WALL: 4 FT X 8 FT **PANEL ORIENTATION:** Vertical
Sheathing one side

SHEATHING:
- [ ] Plywood 15/32" APA Rated Exposure 1 (USA)
- [ ] OSB 7/16" APA Rated Exposure 1 (USA)
- [X] Plywood (CSA 0151M) CSP 12.5mm (1/2") — CAN/PLY Exterior CSP
- [ ] Plywood (CSA O121M) DFP 12.5mm (1/2") — CAN/PLY Exterior DFP
- [ ] OSB (CSA O325) 11 mm (7/16")
- [ ] Other — MFR: ALBERTA PLYWOOD, MILL AB 244 - Weatherproof Bond

SCREWS Sheathing:
- [X] No.8 gauge 1.5" self-piercing Bugle head LOX drive (Grabber Superdrive)
- [ ] No.8 gauge 1.0" self-piercing Bugle head (Flat socket head screw) (HD)
- [ ] No.8 gauge 1.25" self-drilling Bugle head LOX Drive (Grabber Superdrive)
- [ ] No.9 gauge 1.0" self-piercing Bugle head (screws Y2,Y12 in track)

Framing:
- [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive

Hold downs:
- [X] No.10 gauge 0.75" self-drilling Hex washer head

Loading Beam: A325 3/4" bolts — 3 bolts [ ] 6 bolts [X] 12 bolts [ ]

Back-to-Back Chord Studs:
- [X] No.10 gauge 0.75" self-drilling Hex washer head (2@12" O.C.)

SHEATHING FASTENER SCHEDULE: [ ] 2"/12" [ ] 3"/12" [ ] 4"/12" [X] 6"/12" [ ] Other:

EDGE PANEL DISTANCE: [ ] 3/8" [X] 1/2" [ ] Other:

STUDS: Field / Chord
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.054" (1.37mm) 50ksi (340 MF
- [X] Double chord studs used
- [ ] Other

STUD SPACING:
- [ ] 12" O.C.
- [ ] 16" O.C.
- [X] 24" O.C.
- [ ] Other:

TRACK: Web: 3-5/8" inches; Flange: 1-1/4" inches
- [X] 0.043" (1.09mm) 33ksi (230 MPa)
- [ ] Other:

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10 7/8" Anchor Rod (# of screws): 33
- [ ] UCI 18" hold down 1/2" Anchor Rod (# of screws):
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [X] Monotonic — 7.5mm/min.
- [ ] Cyclic — CUREE reversed cyclic (10 mm/s)
- [X] Gravity — 9 kN load at both ends of wall

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral
- [X] East Frame Brace
- [X] West Frame Brace
- [X] Wood Shear

TOTAL: 9

MOISTURE CONTENT OF SHEATHING: Moisture Meter

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

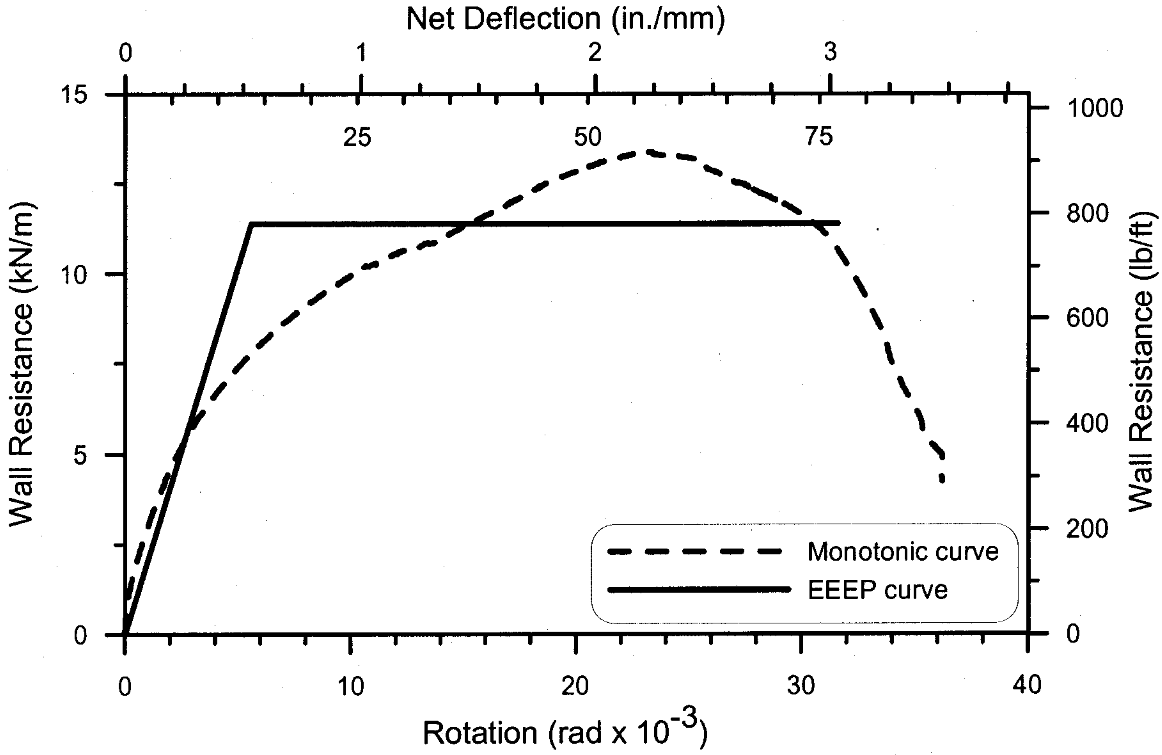
Ww=	21.51	21.77
Wd=	20.41	20.70
m.c.=	5.39	5.17
	North	North

AVG m.c. 5.28

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 50 scan/sec

COMMENTS:
- -Shear anchors torqued to 77 kN
- -Hold down anchors 1/2 turn from finger tight (load cells used on both hold-downs)
- -Ambient temperature 23 C
- -Double chord studs used
- -Square plate washers (2.5"x2.5") used in all track connections
- -Pilot holes drilled for screws A1,A5,A9,Q1,Q5,Q9
- -Initial load set to zero at beginning of test, displacement -1.67 mm
- - Gravity loads engaged
- - Extra hole between A6 and A7 due to mismeasurement of screw spacing

Test 53A
(1220 x 2440 mm CSP 152/305 mm)



	Parameters	Units
F_u	16.32	kN
$F_{0.8u}$	13.06	kN
$F_{0.4u}$	6.53	kN
F_y	13.87	kN
K_e	1.02	kN/mm
Ductility (μ)	5.66	-
$\Delta_{net,y}$	13.61	mm
$\Delta_{net,u}$	57.06	mm
$\Delta_{net,0.8u}$	76.99	mm
$\Delta_{net,0.4u}$	6.41	mm
$Area_{Backbone}$	973.42	J
$Area_{EEEP}$	973.42	J
Check	OK	
R_d	**3.21**	-
S_y	**11.38**	kN/m

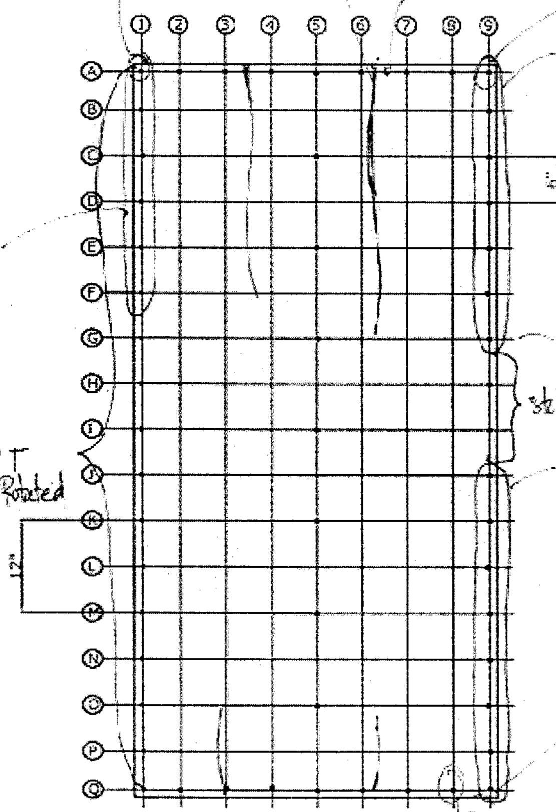
Test Name	53A
Date Tested	May 12, 2005
Protocol	MONOTONIC

McGill

Cold-Formed Steel Stud Shear Walls Testing

Moisture Content: 53A 21.51 20.4
53B 21.77 20.70

Test name: 53A
Date tested: May 12, 2008
Wall Size: 4' x 8'
Screw pattern: 6" / 12"
Edge Distance: 1/2"
Test mode: ☐ Cyclic ☒ Monotonic



T.O.

DP crack in first ply

DP extra hole

P.O. Screw sheared

- Shear LVDT screws hit double chord stud at top, but then removed
- West brace knocked reading will be a bit high should be steady
- wall installed "backwards"
- wall top is negative

Max Load = 16.745 kN

Max Displ = 98.562 mm

PPT

Some delamination of plywood

still hanging on a bit

Screws withdraw P.O. heavily rotated
ultimate failure this side lets go

PPT
Heavily Rotated

Track deformed

GRADE STAMP:
ALBERTA PLYWOOD
BOND

P.O. uplift visible

Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Light Gauge Steel Frame / Wood Panel Shear Walls
McGill University, Montreal

TEST: 53B

RESEARCHER: KATHERINE HIKITA **ASSISTANTS:** M. OUELLET, D. MORELLO, R. CARBONNEAU

DATE: Built: May 11; Tested: Thursday May 16, 2005 **TIME:** 16:00

DIMENSIONS OF WALL: 4 FT X 8 FT **PANEL ORIENTATION:** Vertical; Sheathing one side

SHEATHING:
- [ ] Plywood 15/32" APA Rated Exposure 1 (USA)
- [ ] OSB 7/16" APA Rated Exposure 1 (USA)
- [X] Plywood (CSA 0151M) CSP 12.5mm (1/2") — CAN/PLY Exterior CSP
- [ ] Plywood (CSA O121M) DFP 12.5mm (1/2") — CAN/PLY Exterior DFP
- [ ] OSB (CSA O325) 11 mm (7/16")
- [ ] Other — MFR: ALBERTA PLYWOOD, MILL AB 244 - Weatherproof Bond

SCREWS

Sheathing:
- [X] No.8 gauge 1.5" self-piercing Bugle head LOX drive (Grabber Superdrive)
- [ ] No.8 gauge 1.0" self-piercing Bugle head (Flat socket head screw) (HD)
- [ ] No.8 gauge 1.25" self-drilling Bugle head LOX Drive (Grabber Super
- [ ] No.9 gauge 1.0" self-piercing Bugle head (screws Y2,Y12 in track)

Framing: [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive

Hold downs: [X] No.10 gauge 0.75" self-drilling Hex washer head

Loading Beam: A325 3/4" bolts — 3 bolts [ ] 6 bolts [X] 12 bolts [ ]

Back-to-Back Chord Studs: [X] No.10 gauge 0.75" self-drilling Hex washer head (2@12" O.C.)

SHEATHING FASTENER SCHEDULE: [ ] 2"/12" [ ] 3"/12" [ ] 4"/12" [X] 6"/12" [ ] Other:

EDGE PANEL DISTANCE: [ ] 3/8" [X] 1/2" [ ] Other:

STUDS:
- Field [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- Chord [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.054" (1.37mm) 50ksi (340 MF
- [X] Double chord studs used
- [ ] Other

STUD SPACING: [ ] 12" O.C. [ ] 16" O.C. [X] 24" O.C. [ ] Other:

TRACK: Web: 3-5/8" inches; Flange: 1-1/4" inches; [X] 0.043" (1.09mm) 33ksi (230 MPa) [ ] Other:

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10 7/8" Anchor Rod (# of screws): 33
- [ ] UCI 18" hold down 1/2" Anchor Rod (# of screws):
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [X] Monotonic — 7.5mm/min.
- [ ] Cyclic — CUREE reversed cyclic (10 mm/s)
- [X] Gravity — 9 kN load at both ends of wall

LVDT MEASUREMENTS:
- [X] Actuator LVDT [X] North Uplift [X] East Frame Brace
- [X] North Slip [X] South Uplift [X] West Frame Brace
- [X] South Slip [X] Top of Wall Lateral [X] Wood Shear

TOTAL: 9

MOISTURE CONTENT OF SHEATHING: Moisture Meter

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

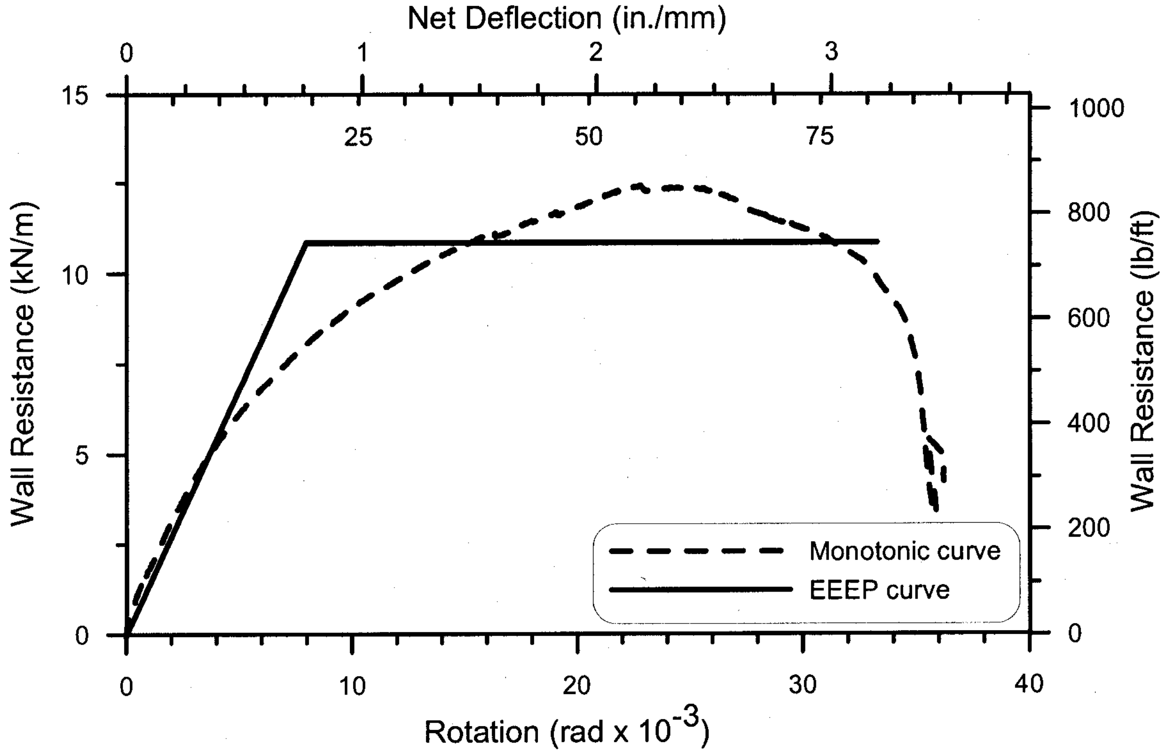
	North	North
Ww=	22.24	22.71
Wd=	20.97	21.52
m.c.=	6.06	5.53

AVG m.c. 5.79

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 50 scan/sec

COMMENTS:
- -Shear anchors torqued to 77 kN
- -Hold down anchors tightened to approximately 8 kN (load cells used on both hold-downs)
- -Ambient temperature 23 C
- -Double chord studs used
- -Square plate washers (2.5"x2.5") used in all track connections
- -Pilot holes drilled for screws A1,A5,A9,Q1,Q5,Q9
- -Initial load set to zero at beginning of test, displacement 0.6 mm
- -Offset after gravity loads engaged 0.62 mm

Test 53B
(1220 x 2440 mm CSP 152/305 mm)



	Parameters	Units
F_u	15.13	kN
$F_{0.8u}$	12.11	kN
$F_{0.4u}$	6.05	kN
F_y	13.23	kN
K_e	0.68	kN/mm
Ductility (μ)	4.19	-
$\Delta_{net,y}$	19.35	mm
$\Delta_{net,u}$	55.71	mm
$\Delta_{net,0.8u}$	81.06	mm
$\Delta_{net,0.4u}$	8.85	mm
$Area_{Backbone}$	944.69	J
$Area_{EEEP}$	944.69	J
Check	OK	
R_d	**2.72**	-
S_y	**10.85**	kN/m

Test Name	53B
Date Tested	May 16, 2005
Protocol	MONOTONIC

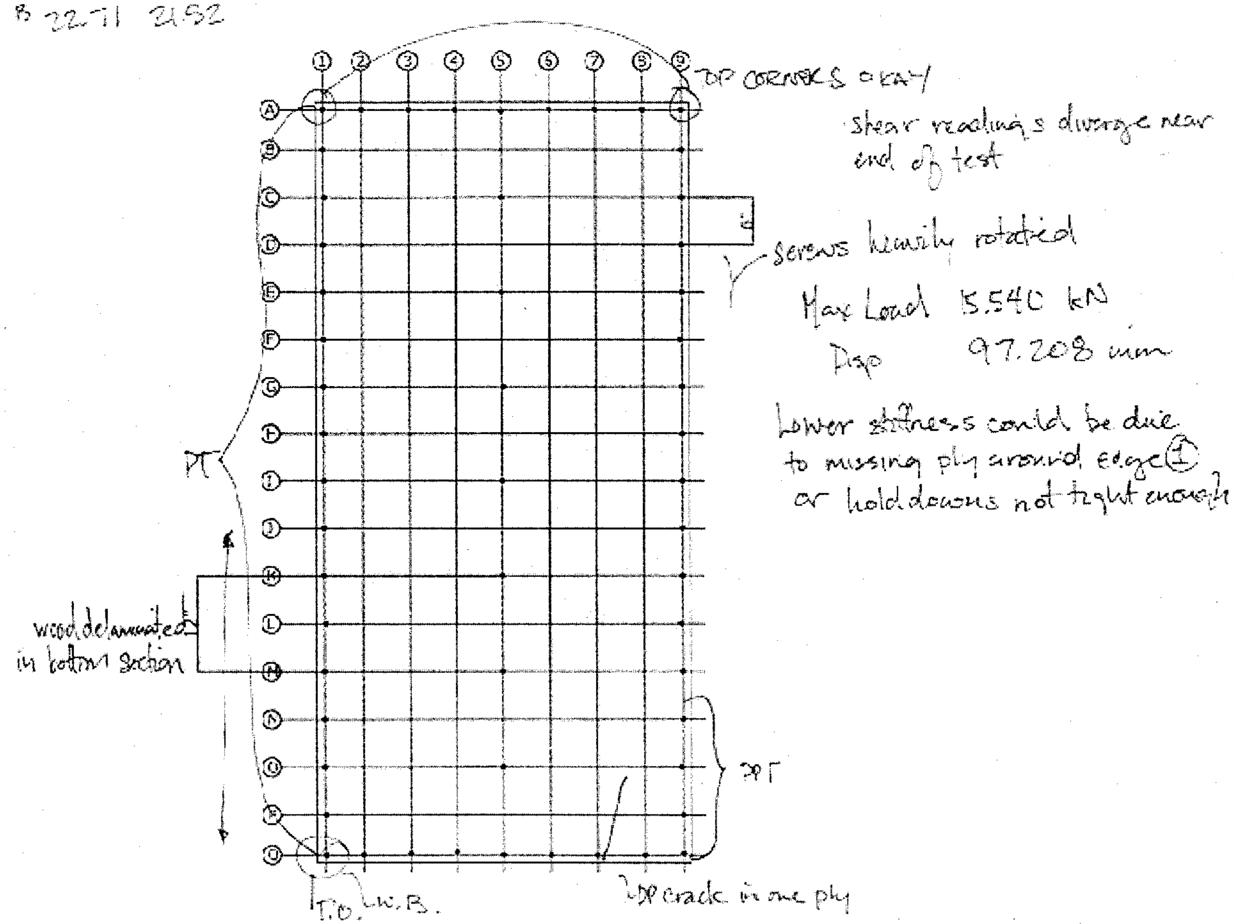
McGill

Gravity loads included

Cold-Formed Steel Stud Shear Walls Testing

Moisture Content A 22.24 20.97
B 22.71 21.52

Test name : S3B
Date tested : May 2005
Wall Size : 4' x 8'
Screw pattern : 6" / 12"
Edge Distance : 1/2"
Test mode : ☐ Cyclic ☒ Monotonic



Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DF)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Light Gauge Steel Frame / Wood Panel Shear Walls
McGill University, Montreal

TEST: 53C

RESEARCHER: KATHERINE HIKITA **ASSISTANTS:** M. OUELLET, JIANG FAN, M. ALKHARAT

DATE: Built: May 16; Tested: Friday May 17, 2005 **TIME:** 16:00

DIMENSIONS OF WALL: 4 FT X 8 FT **PANEL ORIENTATION:** Vertical, Sheathing one side

SHEATHING:
- [ ] Plywood 15/32" APA Rated Exposure 1 (USA)
- [ ] OSB 7/16" APA Rated Exposure 1 (USA)
- [X] Plywood (CSA 0151M) CSP 12.5mm (1/2") — CAN/PLY Exterior CSP
- [ ] Plywood (CSA O121M) DFP 12.5mm (1/2") — CAN/PLY Exterior DFP
- [ ] OSB (CSA O325) 11 mm (7/16")
- [ ] Other — MFR: ALBERTA PLYWOOD, MILL AB 244 - Weatherproof Bond

SCREWS Sheathing:
- [X] No.8 gauge 1.5" self-piercing Bugle head LOX drive (Grabber Superdrive)
- [ ] No.8 gauge 1.0" self-piercing Bugle head (Flat socket head screw) (HD)
- [ ] No.8 gauge 1.25" self-piercing Bugle head LOX Drive (Grabber Supe
- [ ] No.9 gauge 1.0" self-piercing Bugle head (screws Y2,Y12 in track)

Framing: [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive

Hold downs: [X] No.10 gauge 0.75" self-drilling Hex washer head

Loading Beam: A325 3/4" bolts — 3 bolts [ ] 6 bolts [X] 12 bolts [ ]

Back-to-Back Chord Studs: [X] No.10 gauge 0.75" self-drilling Hex washer head (2@12" O.C.)

SHEATHING FASTENER SCHEDULE: [ ] 2"/12" [ ] 3"/12" [ ] 4"/12" [X] 6"/12" [ ] Other:

EDGE PANEL DISTANCE: [ ] 3/8" [X] 1/2" [ ] Other:

STUDS: Field / Chord
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.054" (1.37mm) 50ksi (340 MF
- [X] Double chord studs used
- [ ] Other

STUD SPACING:
- [ ] 12" O.C.
- [ ] 16" O.C.
- [X] 24" O.C.
- [ ] Other:

TRACK: Web: 3-5/8" inches; Flange: 1-1/4" inches — [X] 0.043" (1.09mm) 33ksi (230 MPa) [ ] Other:

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10 7/8" Anchor Rod (# of screws): 33
- [ ] UCI 18" hold down 1/2" Anchor Rod (# of screws):
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [X] Monotonic — 7.5mm/min.
- [ ] Cyclic — CUREE reversed cyclic (10 mm/s)
- [ ] Gravity — 9 kN load at both ends of wall

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral
- [X] East Frame Brace
- [X] West Frame Brace
- [X] Wood Shear

TOTAL: 9

MOISTURE CONTENT OF SHEATHING: Moisture Meter

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

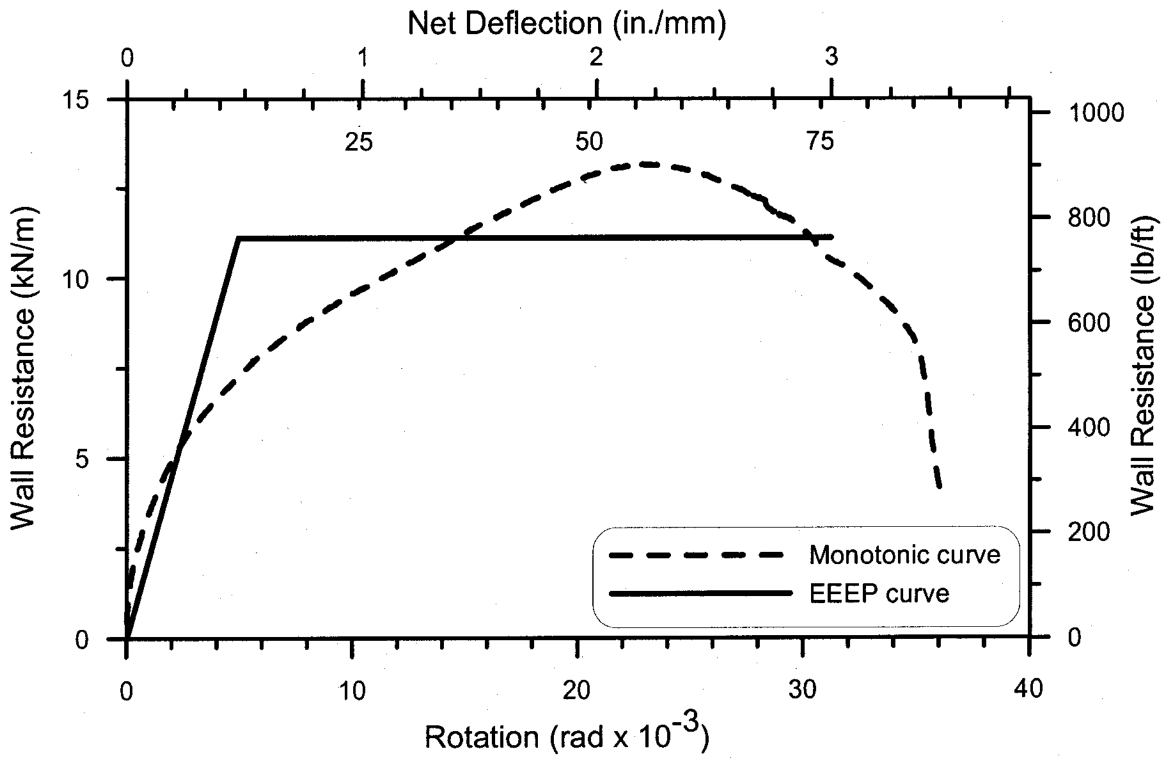
	North	North
Ww=	21.27	21.34
Wd=	20.13	20.23
m.c.=	5.66	5.49

AVG m.c. 5.58

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 50 scan/sec

COMMENTS:
- -Shear anchors torqued to 77 kN
- -Hold down anchors tightened to approximately 8 kN (load cells used on both hold-downs)
- -Ambient temperature 23 C
- -Double chord studs used
- -Square plate washers (2.5"x2.5") used in all track connections
- -Pilot holes drilled for screws A1,A5,A9,Q1,Q5,Q9
- -Initial load set to zero at beginning of test, displacement -0.602 mm
- - No Gravity Loads engaged

Test 53C
(1220 x 2440 mm CSP 152/305 mm)



	Parameters	Units
F_u	16.05	kN
$F_{0.8u}$	12.84	kN
$F_{0.4u}$	6.42	kN
F_y	13.56	kN
K_e	1.13	kN/mm
Ductility (μ)	6.33	-
$\Delta_{net,y}$	12.03	mm
$\Delta_{net,u}$	55.84	mm
$\Delta_{net,0.8u}$	76.12	mm
$\Delta_{net,0.4u}$	5.69	mm
$Area_{Backbone}$	950.91	J
$Area_{EEEP}$	950.91	J
Check	OK	
R_d	**3.41**	-
S_y	**11.12**	kN/m

Test Name	53C
Date Tested	May 17, 2005
Protocol	MONOTONIC

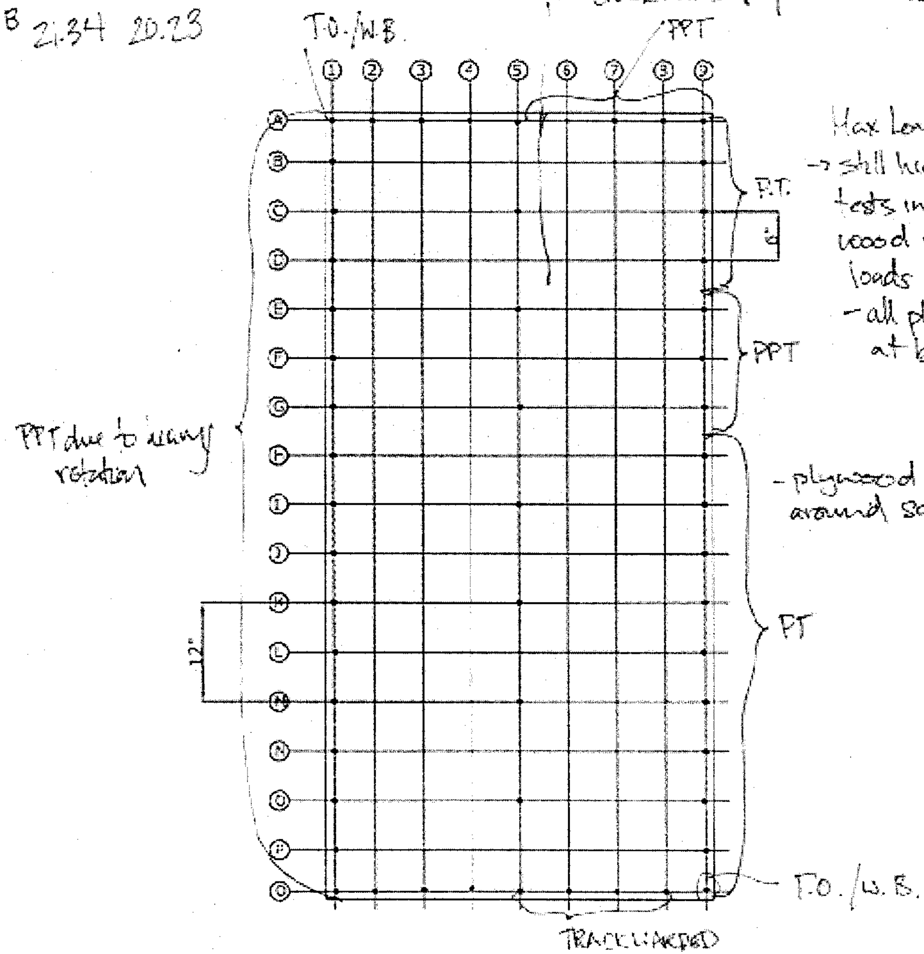
McGill

*No gravity load

Cold-Formed Steel Stud Shear Walls Testing

Moisture Content A 21.27 20.13
B 21.34 20.23

Test name : 53C
Date tested : May 17 2005
Wall Size : 4' x 8'
Screw pattern : 6" / 12"
Edge Distance : 1/2"
Test mode : ☐ Cyclic ☒ Monotonic

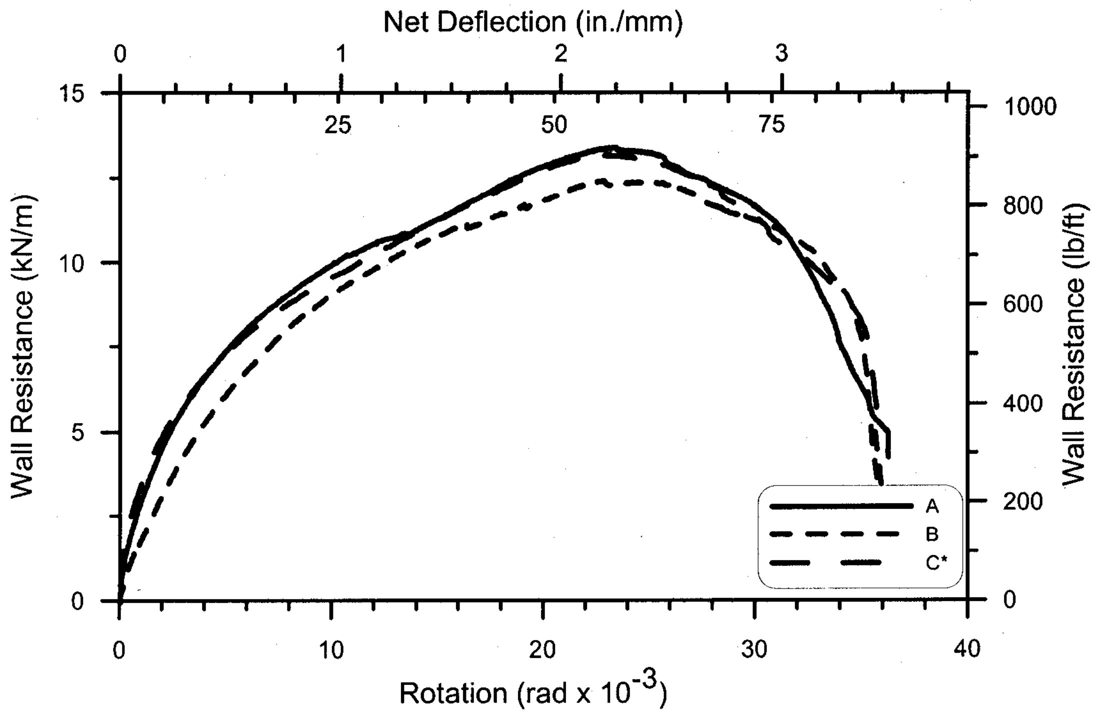


Max load : 16.038 kN
→ still higher ductility than last tests indicating difference in wood not the effect of gravity loads
- all plys all present at edges at beginning

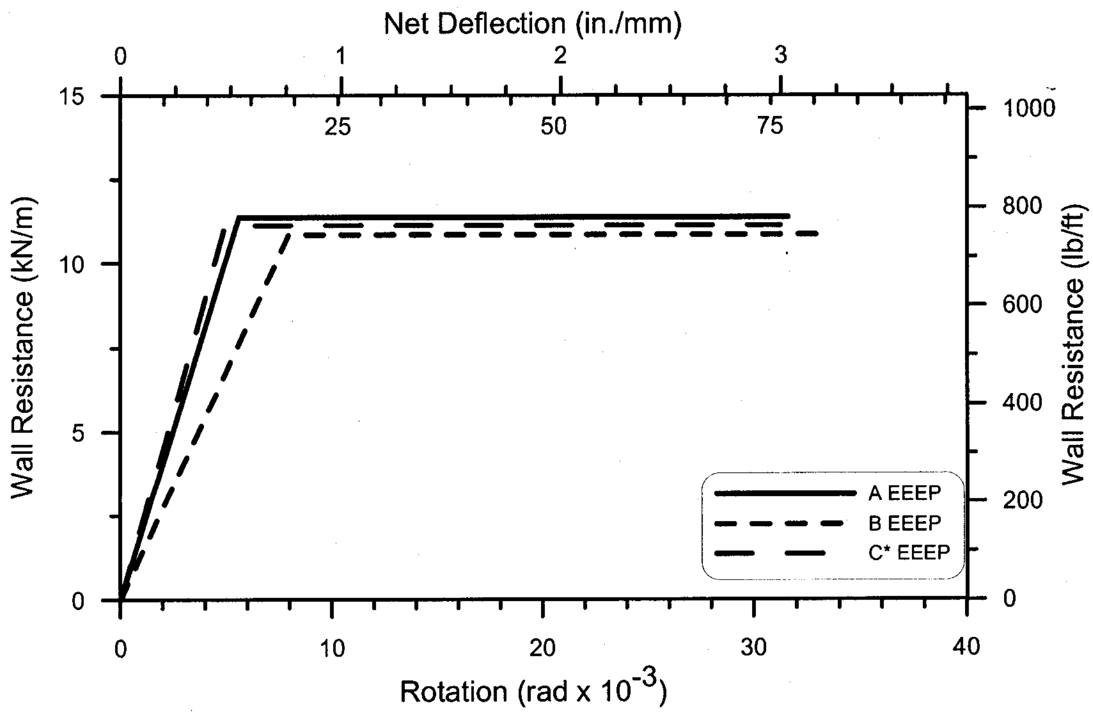
- plywood delaminated around screw heads

Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Test 53-A,B,C
(1220 x 2440 mm CSP 152/305 mm)



Test 53-A,B,C EEEP
(1220 x 2440 mm CSP 152/305 mm)



Light Gauge Steel Frame / Wood Panel Shear Walls
McGill University, Montreal

TEST: 54A

RESEARCHER: KATHERINE HIKITA **ASSISTANTS:** M. OUELLET, D. MORELLO, R. CARBONNEAU

DATE: Built: May 11; Tested: Friday May 20, 2005 **TIME:** 14:00

DIMENSIONS OF WALL: 4 FT X 8 FT **PANEL ORIENTATION:** Vertical, Sheathing one side

SHEATHING:
- [ ] Plywood 15/32" APA Rated Exposure 1 (USA)
- [ ] OSB 7/16" APA Rated Exposure 1 (USA)
- [x] Plywood (CSA 0151M) CSP 12.5mm (1/2") — CAN/PLY Exterior CSP
- [ ] Plywood (CSA O121M) DFP 12.5mm (1/2") — CAN/PLY Exterior DFP
- [ ] OSB (CSA O325) 11 mm (7/16")
- [ ] Other — MFR: ALBERTA PLYWOOD, MILL AB 244 - Weatherproof Bond

SCREWS

Sheathing:
- [x] No.8 gauge 1.5" self-piercing Bugle head LOX drive (Grabber Superdrive)
- [ ] No.8 gauge 1.0" self-piercing Bugle head (Flat socket head screw) (HD)
- [ ] No.8 gauge 1.25" self-drilling Bugle head LOX Drive (Grabber Super
- [ ] No.9 gauge 1.0" self-piercing Bugle head (screws Y2,Y12 in track)

Framing:
- [x] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive

Hold downs:
- [x] No.10 gauge 0.75" self-drilling Hex washer head

Loading Beam: A325 3/4" bolts — 3 bolts [ ] 6 bolts [X] 12 bolts [ ]

Back-to-Back Chord Studs:
- [x] No.10 gauge 0.75" self-drilling Hex washer head (2@12" O.C.)

SHEATHING FASTENER SCHEDULE: [ ] 2"/12" [ ] 3"/12" [ ] 4"/12" [X] 6"/12" [ ] Other:

EDGE PANEL DISTANCE: [ ] 3/8" [X] 1/2" [ ] Other:

STUDS: Field, Chord
- [x] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [x] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.054" (1.37mm) 50ksi (340 MF
- [x] Double chord studs used
- [ ] Other

STUD SPACING:
- [ ] 12" O.C.
- [ ] 16" O.C.
- [x] 24" O.C.
- [ ] Other:

TRACK: Web: 3-5/8" inches; Flange: 1-1/4" inches
- [x] 0.043" (1.09mm) 33ksi (230 MPa)
- [ ] Other:

HOLD DOWNS:
- [x] Simpson Strong-Tie S/HD10 7/8" Anchor Rod — (# of screws): 33
- [ ] UCI 18" hold down 1/2" Anchor Rod — (# of screws):
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [ ] Monotonic — 7.5mm/min.
- [x] Cyclic — CUREE reversed cyclic (10 mm/s)
- [x] Gravity — 9 kN load at both ends of wall, 10 m

LVDT MEASUREMENTS:
- [x] Actuator LVDT
- [x] North Slip
- [x] South Slip
- [x] North Uplift
- [x] South Uplift
- [x] Top of Wall Lateral
- [x] East Frame Brace
- [x] West Frame Brace
- [x] Wood Shear

TOTAL: 9

MOISTURE CONTENT OF SHEATHING: Moisture Meter

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

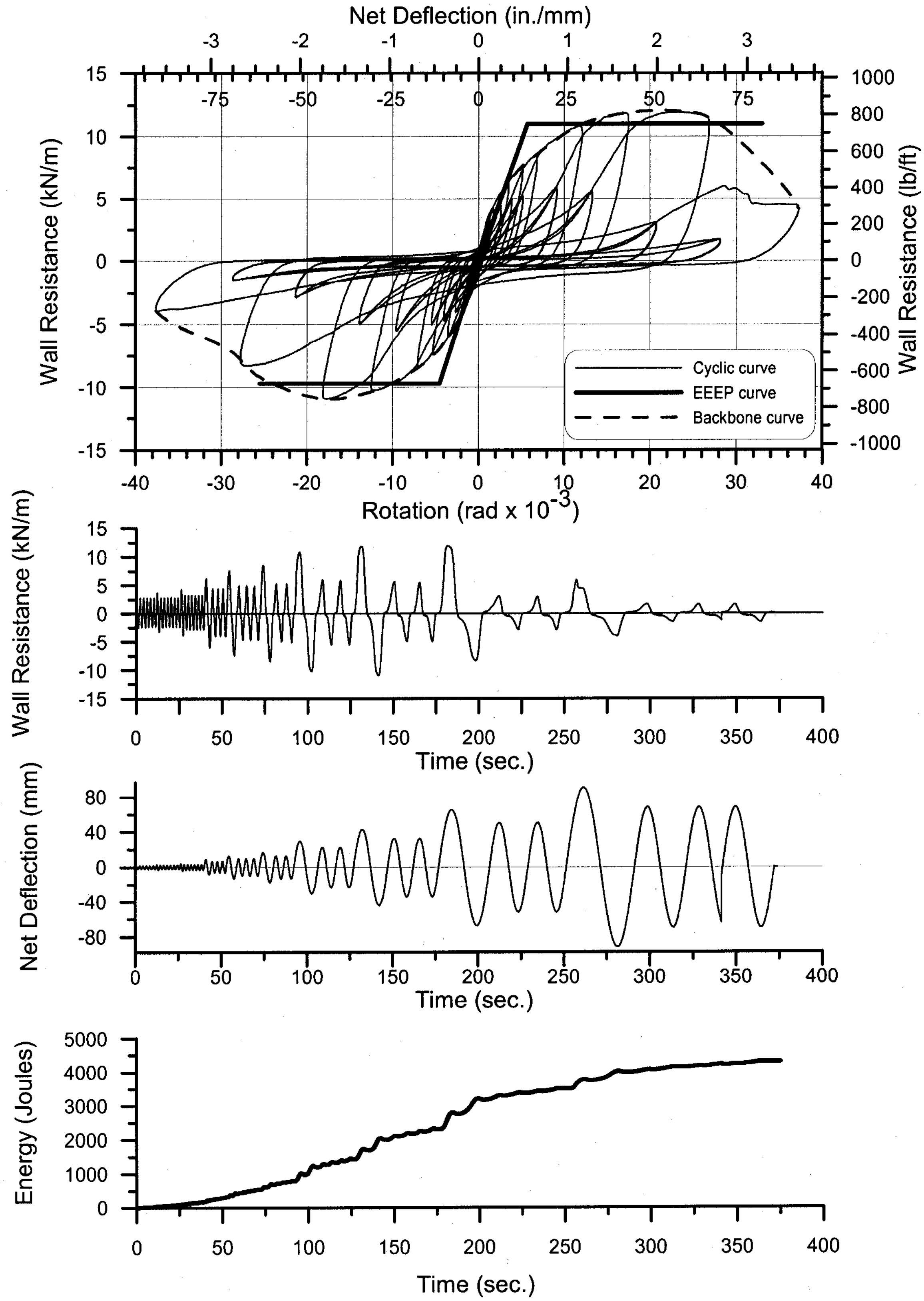
	North	North
Ww=	20.62	20.50
Wd=	19.53	19.40
m.c.=	5.58	5.67

AVG m.c. 5.63

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 50 scan/sec

COMMENTS:
- -Shear anchors torqued to 77 kN
- -Hold down anchors tightened to approximately 8 kN (load cells used on both hold-downs)
- -Ambient temperature 23 C
- -Double chord studs used
- -Square plate washers (2.5"x2.5") used in all track connections
- -Pilot holes drilled for screws A1,A5,A9,Q1,Q5,Q9
- -Initial load set to zero at beginning of test, displacement -0.023 mm
- -Intial engagement displacement after gravity loads are applied 0.129 mm

Test 54A
(1220 x 2440 mm CSP 152/305 mm)



	Negative	Positive	Units
F_u	-13.33	14.57	kN
$F_{0.8u}$	-10.67	11.65	kN
$F_{0.4u}$	-5.33	5.83	kN
F_y	-11.85	13.39	kN
K_e	1.07	0.96	kN/mm
Ductility (μ)	5.60	5.74	-
$\Delta_{net,y}$	-11.11	14.02	mm
$\Delta_{net,u}$	-43.43	56.06	mm
$\Delta_{net,0.8u}$	-62.20	80.40	mm
$\Delta_{net,0.4u}$	-5.00	6.10	mm
$Area_{Backbone}$	671.08	982.59	kN-mm
$Area_{EEEP}$	671.08	982.59	kN-mm
Check	OK	OK	
R_d	**3.19**	**3.24**	-
S_y	**-9.72**	**10.98**	kN/m

Test Name	54A
Date Tested	May 20, 2005
Protocol	CUREE ORDINARY GROUND MOTIONS

McGill

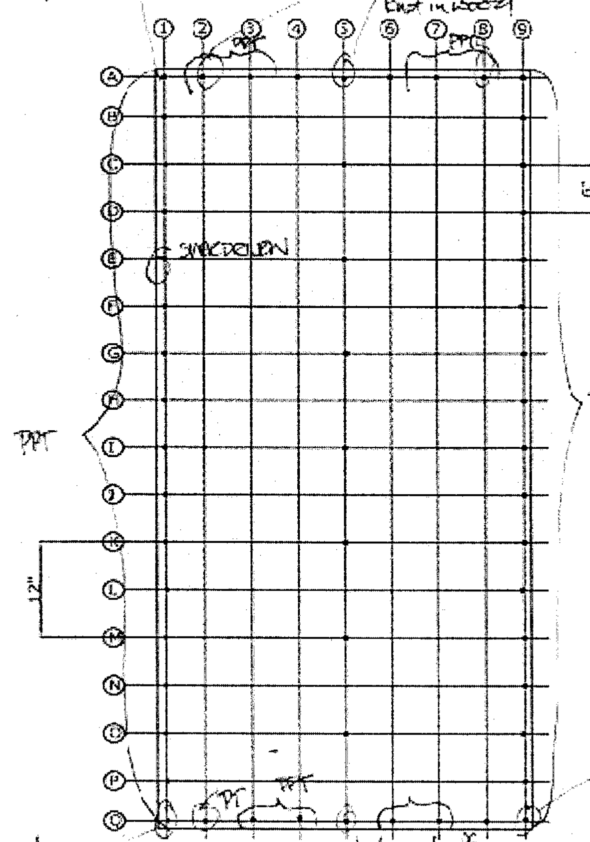
Cold-Formed Steel Stud Shear Walls Testing

Moisture Content: A 20.62 19.53
B 20.50 19.40

Test name: 54 A
Date tested: May 2005
Wall Size: 4' x 8'
Screw pattern: 6" / 12"
Edge Distance: 1/2"
Test mode: ☐ Cyclic ☒ Monotonic

Maxload 15.014 kN
Displacement 99.592 mm
Hold downs ✓
Gravity Loads

	Min	Max
North	8.561	9.561
South	8.676	9.2987



Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP) ; Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Light Gauge Steel Frame / Wood Panel Shear Walls
McGill University, Montreal

TEST: 54B

RESEARCHER: KATHERINE HIKITA **ASSISTANTS:** M. OUELLET, JIANG FAN, M. ALKHARAT

DATE: Built: May 11; Tested: Thursday May 26, 2005 **TIME:** 14:00

DIMENSIONS OF WALL: 4 FT X 8 FT **PANEL ORIENTATION:** Vertical, Sheathing one side

SHEATHING:
- [ ] Plywood 15/32" APA Rated Exposure 1 (USA)
- [ ] OSB 7/16" APA Rated Exposure 1 (USA)
- [x] Plywood (CSA 0151M) CSP 12.5mm (1/2") — CAN/PLY Exterior CSP
- [ ] Plywood (CSA O121M) DFP 12.5mm (1/2") — CAN/PLY Exterior DFP
- [ ] OSB (CSA O325) 11 mm (7/16")
- [ ] Other — MFR: ALBERTA PLYWOOD, MILL AB 244 - Weatherproof Bond

SCREWS

Sheathing:
- [X] No.8 gauge 1.5" self-piercing Bugle head LOX drive (Grabber Superdrive)
- [ ] No.8 gauge 1.0" self-piercing Bugle head (Flat socket head screw) (HD)
- [ ] No.8 gauge 1.25" self-drilling Bugle head LOX Drive (Grabber Super
- [ ] No.9 gauge 1.0" self-piercing Bugle head (screws Y2,Y12 in track)

Framing:
- [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive

Hold downs:
- [X] No.10 gauge 0.75" self-drilling Hex washer head

Loading Beam: A325 3/4" bolts — 3 bolts [ ] 6 bolts [X] 12 bolts [ ]

Back-to-Back Chord Studs:
- [X] No.10 gauge 0.75" self-drilling Hex washer head (2@12" O.C.)

SHEATHING FASTENER SCHEDULE: [ ] 2"/12" [ ] 3"/12" [ ] 4"/12" [X] 6"/12" [ ] Other:

EDGE PANEL DISTANCE: [ ] 3/8" [X] 1/2" [ ] Other:

STUDS:

Field:
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI

Chord:
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.054" (1.37mm) 50ksi (340 MF
- [X] Double chord studs used
- [ ] Other

STUD SPACING:
- [ ] 12" O.C.
- [ ] 16" O.C.
- [X] 24" O.C.
- [ ] Other:

TRACK: Web: 3-5/8" inches; Flange: 1-1/4" inches
- [X] 0.043" (1.09mm) 33ksi (230 MPa)
- [ ] Other:

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10 7/8" Anchor Rod (# of screws): 33
- [ ] UCI 18" hold down 1/2" Anchor Rod (# of screws):
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [ ] Monotonic — 7.5mm/min.
- [X] Cyclic — CUREE reversed cyclic (10 mm/s)
- [X] Gravity — 9 kN load at both ends of wall, 10 m

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral
- [X] East Frame Brace
- [X] West Frame Brace
- [X] Wood Shear

TOTAL: 9

MOISTURE CONTENT OF SHEATHING: Moisture Meter

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

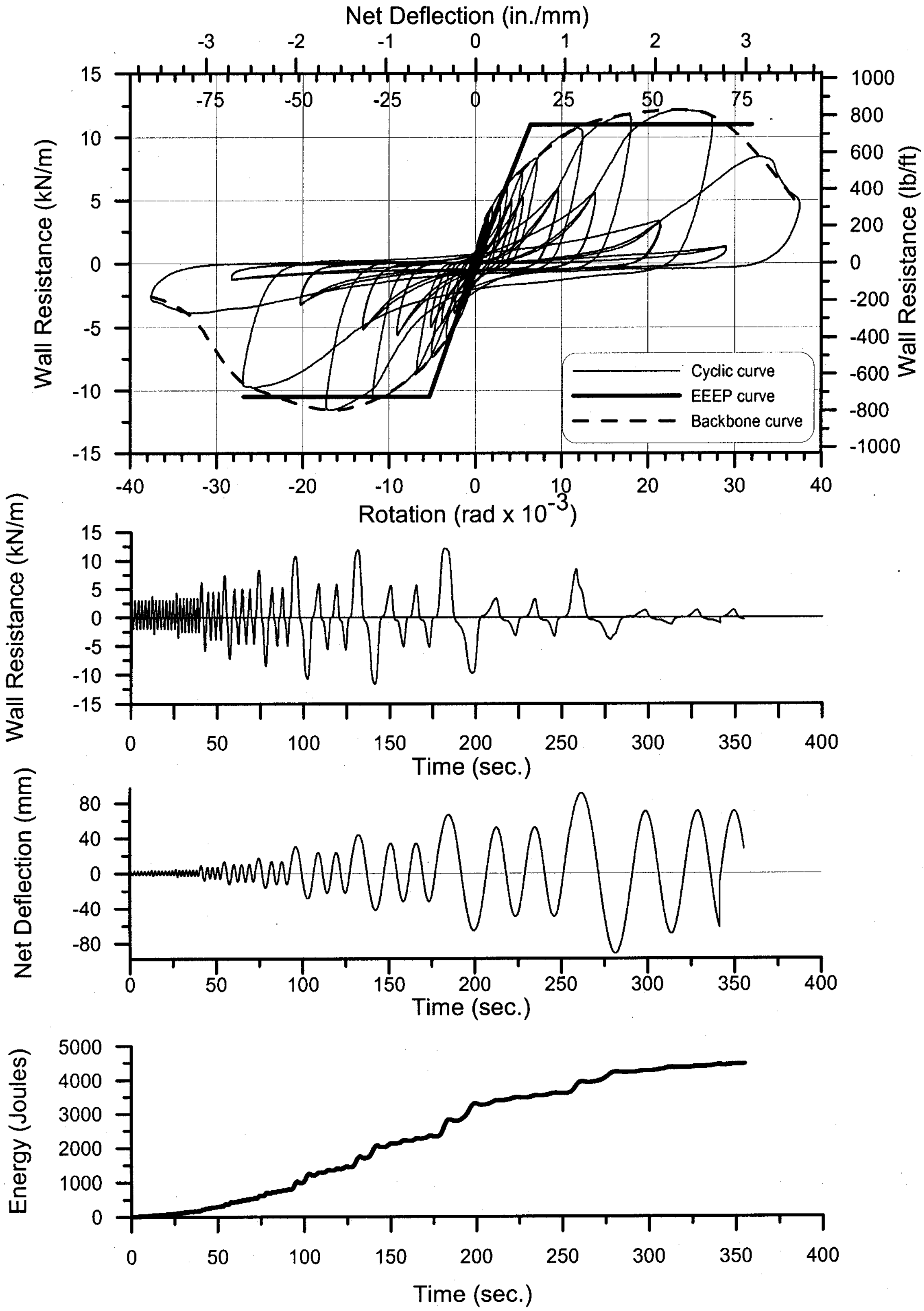
	North	North
Ww=	21.81	21.94
Wd=	20.07	20.75
m.c.=	8.70	5.73

AVG m.c. 7.22

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 50 scan/sec

COMMENTS:
- -Shear anchors torqued to 77 kN
- -Hold down anchors tightened to approximately 8 kN (load cells used on both hold-downs)
- -Ambient temperature 23 C
- -Double chord studs used
- -Square plate washers (2.5"x2.5") used in all track connections
- -Pilot holes drilled for screws A1,A5,A9,Q1,Q5,Q9
- -Initial load set to zero at beginning of test, displacement -0.339 mm
- -Intial engagement displacement after gravity loads are applied -0.315 mm

Test 54B
(1220 x 2440 mm CSP 152/305 mm)



	Negative	Positive	Units
F_u	-14.09	14.82	kN
$F_{0.8u}$	-11.28	11.86	kN
$F_{0.4u}$	-5.64	5.93	kN
F_y	-12.81	13.45	kN
K_e	0.99	0.86	kN/mm
Ductility (μ)	5.06	4.98	-
$\Delta_{net,y}$	-12.95	15.65	mm
$\Delta_{net,u}$	-40.49	57.42	mm
$\Delta_{net,0.8u}$	-65.50	77.90	mm
$\Delta_{net,0.4u}$	-5.70	6.90	mm
$Area_{Backbone}$	756.05	942.26	kN-mm
$Area_{EEEP}$	756.05	942.26	kN-mm
Check	OK	OK	
R_d	**3.02**	**2.99**	-
S_y	**-10.51**	**11.03**	kN/m

Test Name	54B
Date Tested	May 26, 2005
Protocol	CUREE ORDINARY GROUND MOTIONS

McGill

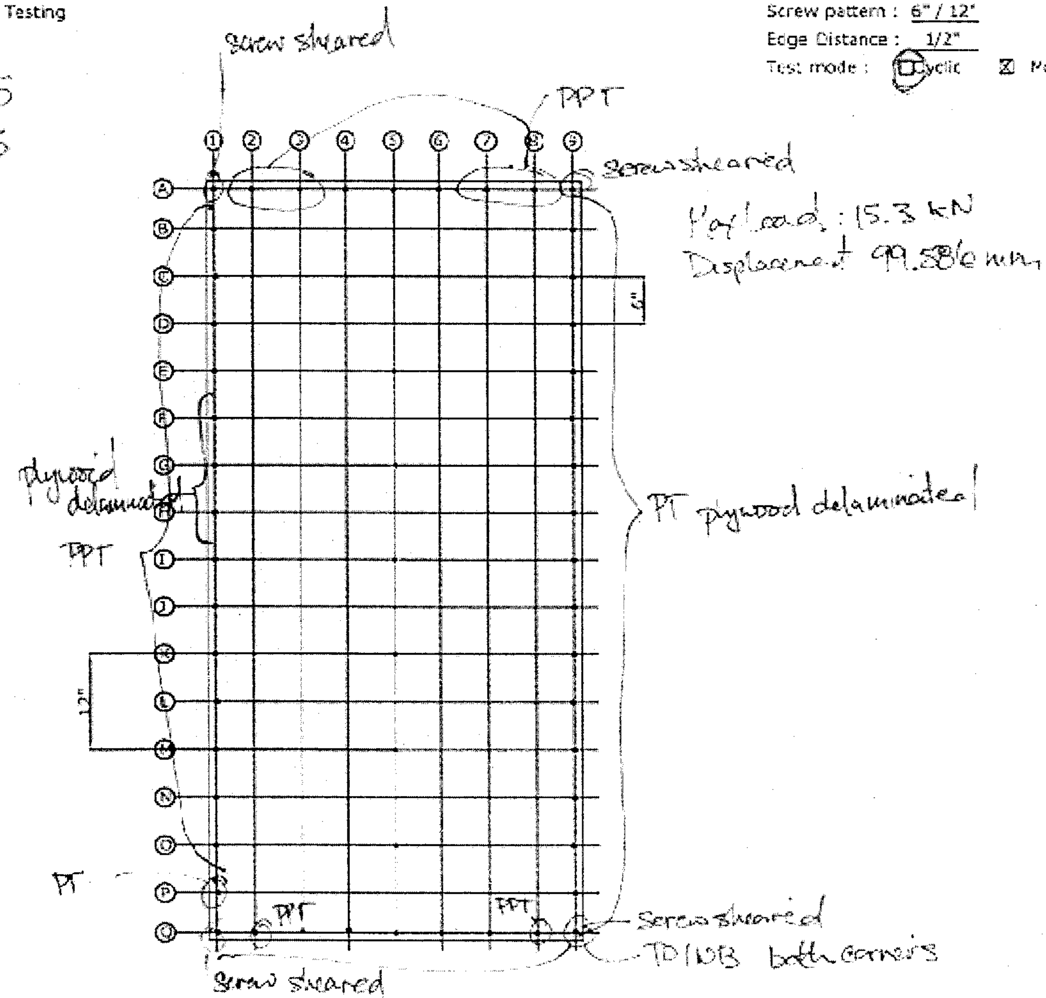
Cold-Formed Steel Stud Shear Walls Testing

Test name: Test 54B
Date tested: May 28, 2005
Wall Size: 4' x 8'
Screw pattern: 6" / 12"
Edge Distance: 1/2"
Test mode: ☐ Cyclic ☒ Monotonic

HARDNESS CONTENT

54B A 21.81 20.65

54B B 21.94 26.75



Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Light Gauge Steel Frame / Wood Panel Shear Walls
McGill University, Montreal

TEST: 54C

RESEARCHER: KATHERINE HIKITA **ASSISTANTS:** M. OUELLET, JIANG FAN, M. ALKHARAT

DATE: Built: May 25; Tested: Thursday May 26, 2005 **TIME:** 14:00

DIMENSIONS OF WALL: 4 FT X 8 FT **PANEL ORIENTATION:** Vertical, Sheathing one side

SHEATHING:
- [ ] Plywood 15/32" APA Rated Exposure 1 (USA)
- [ ] OSB 7/16" APA Rated Exposure 1 (USA)
- [x] Plywood (CSA 0151M) CSP 12.5mm (1/2") — CAN/PLY Exterior CSP
- [ ] Plywood (CSA O121M) DFP 12.5mm (1/2") — CAN/PLY Exterior DFP
- [ ] OSB (CSA O325) 11 mm (7/16")
- [ ] Other — MFR: ALBERTA PLYWOOD, MILL AB 244 - Weatherproof Bond

SCREWS Sheathing:
- [x] No.8 gauge 1.5" self-piercing Bugle head LOX drive (Grabber Superdrive)
- [ ] No.8 gauge 1.0" self-piercing Bugle head (Flat socket head screw) (HD)
- [ ] No.8 gauge 1.25" self-drilling Bugle head LOX Drive (Grabber Super
- [ ] No.9 gauge 1.0" self-piercing Bugle head (screws Y2,Y12 in track)

Framing:
- [x] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive

Hold downs:
- [x] No.10 gauge 0.75" self-drilling Hex washer head

Loading Beam: A325 3/4" bolts — 3 bolts [ ] 6 bolts [X] 12 bolts [ ]

Back-to-Back Chord Studs:
- [x] No.10 gauge 0.75" self-drilling Hex washer head (2@12" O.C.)

SHEATHING FASTENER SCHEDULE: [ ] 2"/12" [ ] 3"/12" [ ] 4"/12" [X] 6"/12" [ ] Other:

EDGE PANEL DISTANCE: [ ] 3/8" [X] 1/2" [ ] Other:

STUDS: Field, Chord
- [x] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [x] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.054" (1.37mm) 50ksi (340 MF
- [x] Double chord studs used
- [ ] Other

STUD SPACING:
- [ ] 12" O.C.
- [ ] 16" O.C.
- [x] 24" O.C.
- [ ] Other:

TRACK: Web: 3-5/8" inches; Flange: 1-1/4" inches
- [x] 0.043" (1.09mm) 33ksi (230 MPa)
- [ ] Other:

HOLD DOWNS:
- [x] Simpson Strong-Tie S/HD10 7/8" Anchor Rod (# of screws): 33
- [ ] UCI 18" hold down 1/2" Anchor Rod (# of screws):
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [ ] Monotonic 7.5mm/min.
- [x] Cyclic CUREE reversed cyclic (10 mm/s)
- [ ] Gravity 9 kN loads at both ends of the wall

LVDT MEASUREMENTS:
- [x] Actuator LVDT
- [x] North Slip
- [x] South Slip
- [x] North Uplift
- [x] South Uplift
- [x] Top of Wall Lateral
- [x] East Frame Brace
- [x] West Frame Brace
- [x] Wood Shear

TOTAL: 9

MOISTURE CONTENT OF SHEATHING: Moisture Meter

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

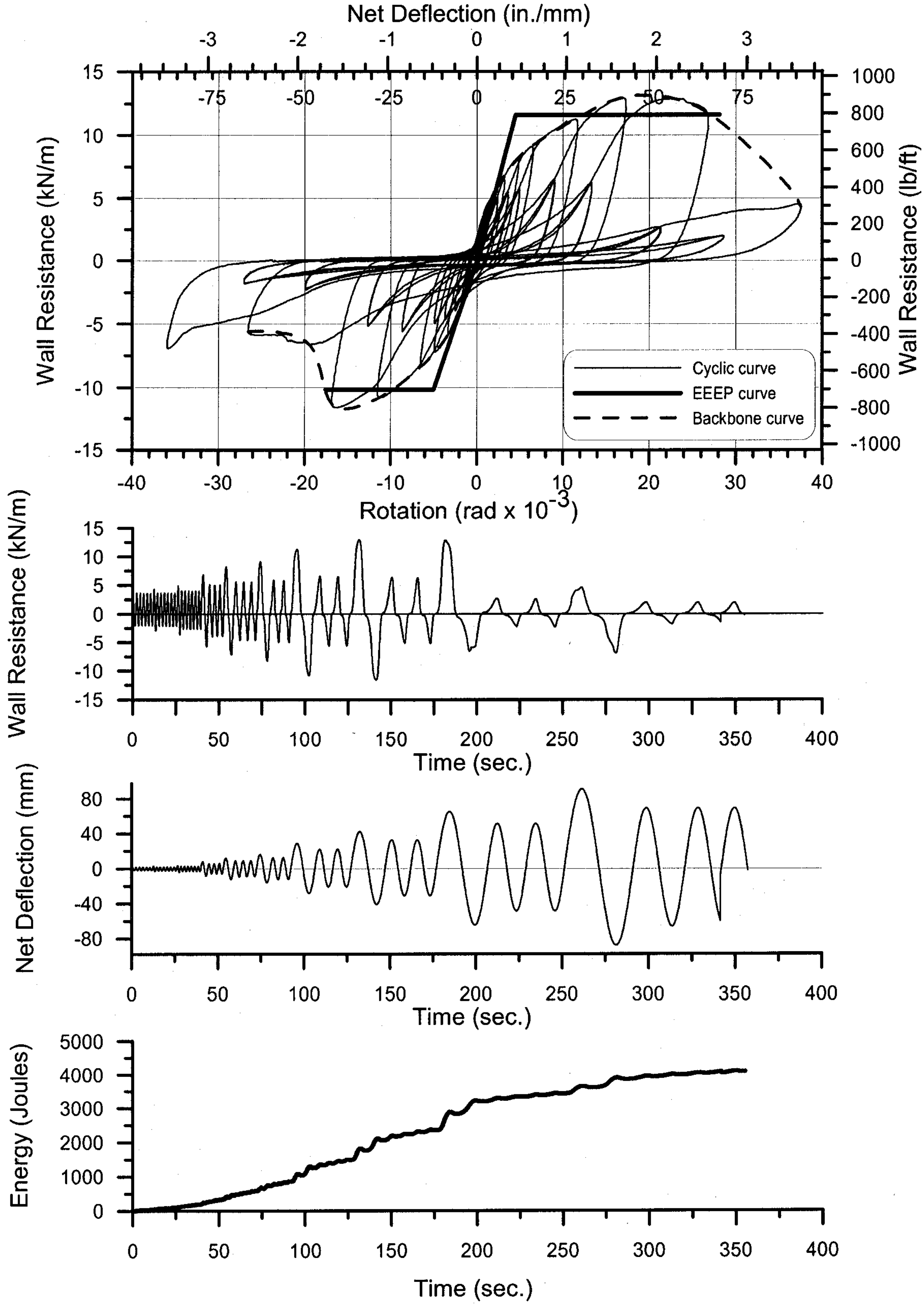
Ww=	23.42	23.67
Wd=	22.28	22.29
m.c.=	5.12	6.19
	North	North

AVG m.c. 5.65

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 50 scan/sec

COMMENTS:
-Shear anchors torqued to 77 kN
-Hold down anchors tightened to approximately 8 kN (load cells used on both hold-downs)
-Ambient temperature 23 C
-Double chord studs used
-Square plate washers (2.5"x2.5") used in all track connections
-Pilot holes drilled for screws A1,A5,A9,Q1,Q5,Q9
-Initial load set to zero at beginning of test, displacement -0.641 mm
- No gravity loads engaged

Test 54C
(1220 x 2440 mm CSP 152/305 mm)



	Negative	Positive	Units
F_u	-14.13	15.80	kN
$F_{0.8u}$	-11.31	12.64	kN
$F_{0.4u}$	-5.65	6.32	kN
F_y	-12.40	14.17	kN
K_e	1.01	1.29	kN/mm
Ductility (μ)	3.49	6.23	-
$\Delta_{net,y}$	-12.28	10.99	mm
$\Delta_{net,u}$	-39.97	41.04	mm
$\Delta_{net,0.8u}$	-42.90	68.50	mm
$\Delta_{net,0.4u}$	-5.60	4.90	mm
$Area_{Backbone}$	455.74	893.01	kN-mm
$Area_{EEEP}$	455.74	893.01	kN-mm
Check	OK	OK	
R_d	**2.45**	**3.39**	-
S_y	**-10.17**	**11.63**	kN/m

Test Name	54C
Date Tested	May 26, 2005
Protocol	CUREE ORDINARY GROUND MOTIONS

McGill

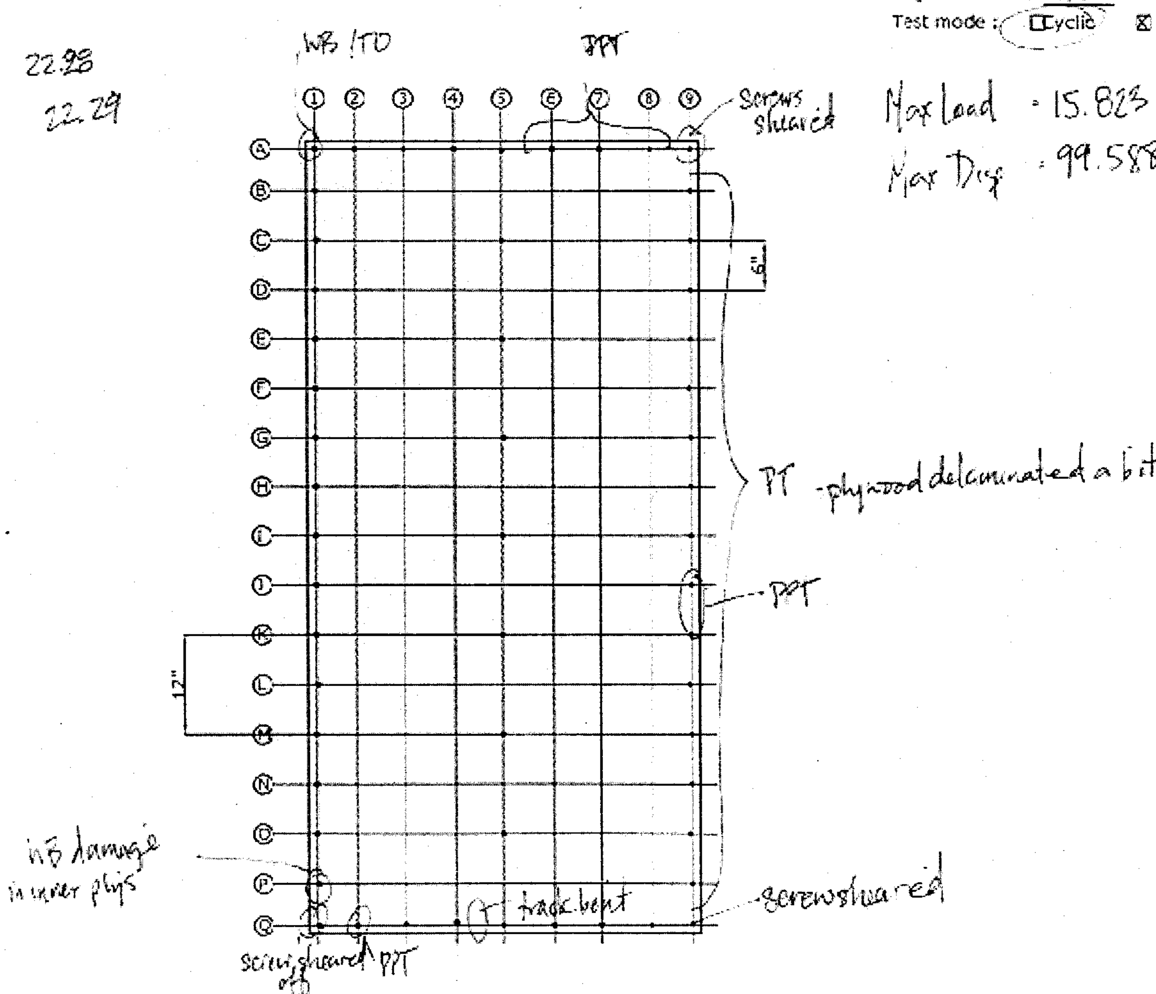
Cold-Formed Steel Stud Shear Walls Testing

Test name : Test 54C
Date tested : Nov 26, 2005
Wall Size : 4' x 8'
Screw pattern : 6" / 12"
Edge Distance : 1/2"
Test mode : ☐ Cyclic ☒ Monotonic

Moisture Content

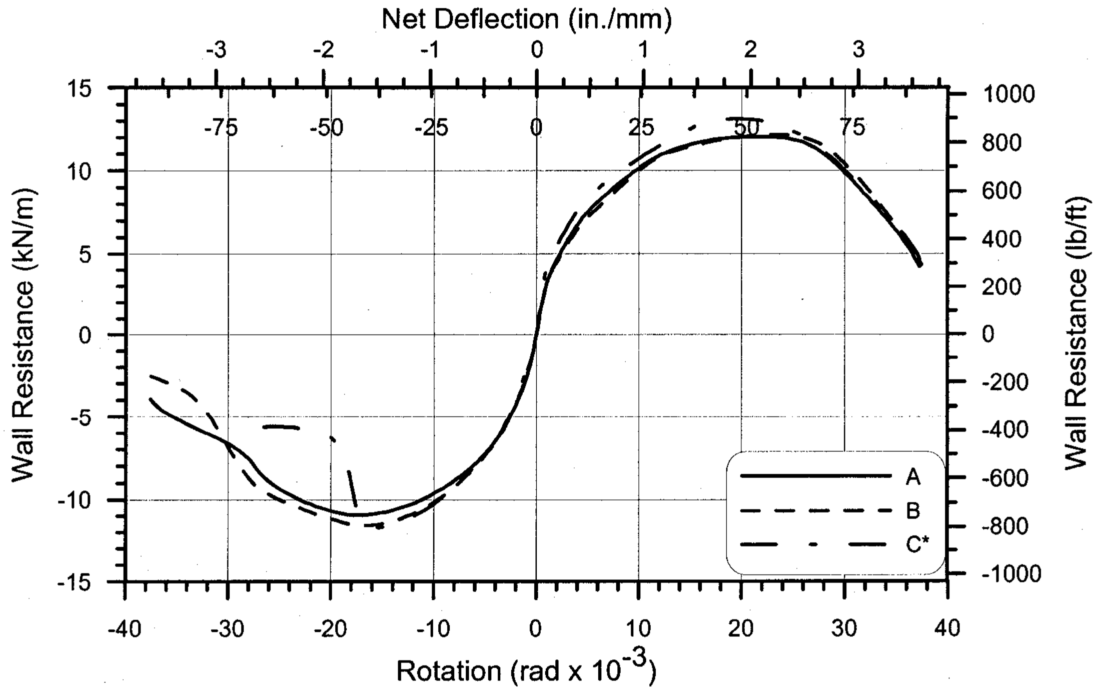
54C A 23.42 22.98
54C B 23.67 22.29

Max Load = 15.823 kN
Max Disp = 99.588 mm

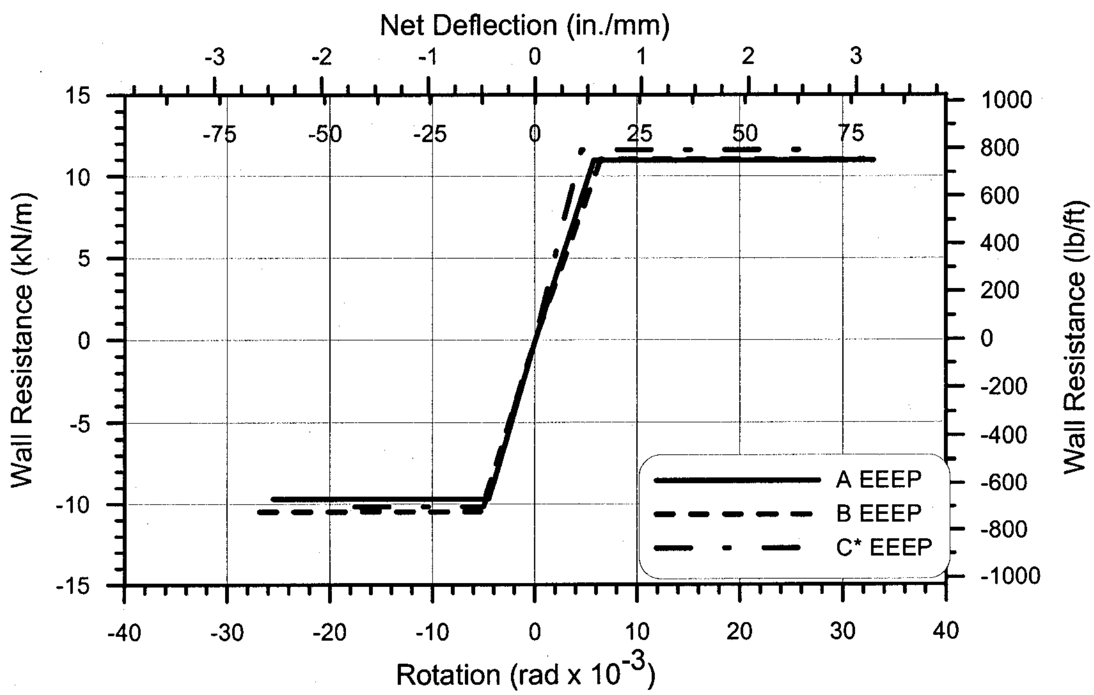


Failure modes: Pull out, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP) ; Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Test 54-A,B,C Backbone
(1220 x 2440 mm CSP 152/305 mm)



Test 54-A,B,C EEEP
(1220 x 2440 mm CSP 152/305 mm)



Light Gauge Steel Frame / Wood Panel Shear Walls
McGill University, Montreal

TEST: 55A

RESEARCHER: KATHERINE HIKITA **ASSISTANTS:** M. OUELLET, JIANG FAN

DATE: Built: May 19; Tested: Wednesday May 25, 2005 **TIME:** 16:00

DIMENSIONS OF WALL: 4 FT X 8 FT **PANEL ORIENTATION:** Vertical
Sheathing one side

SHEATHING:
- [ ] Plywood 15/32" APA Rated Exposure 1 (USA)
- [ ] OSB 7/16" APA Rated Exposure 1 (USA)
- [X] Plywood (CSA 0151M) CSP 12.5mm (1/2") CAN/PLY Exterior CSP
- [ ] Plywood (CSA O121M) DFP 12.5mm (1/2") CAN/PLY Exterior DFP
- [ ] OSB (CSA O325) 11 mm (7/16")
- [ ] Other MFR: ALBERTA PLYWOOD, MILL AB 244 - Weatherproof Bond

SCREWS Sheathing:
- [ ] No.8 gauge 1.5" self-piercing Bugle head LOX drive (Grabber Superdrive)
- [ ] No.8 gauge 1.0" self-piercing Bugle head (Flat socket head screw) (HD)
- [X] No. 8 gauge 1.25" self-drilling Bugle head LOX Drive (Grabber Superdrive)
- [ ] No.9 gauge 1.0" self-piercing Bugle head (screws Y2,Y12 in track)

Framing:
- [ ] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive

Hold downs:
- [X] No.10 gauge 0.75" self-drilling Hex washer head

Loading Beam: A325 3/4" bolts — 3 bolts [ ] 6 bolts [X] 12 bolts [ ]

Back-to-Back Chord Studs:
- [X] No.10 gauge 0.75" self-drilling Hex washer head (2@12" O.C.)

SHEATHING FASTENER SCHEDULE: [ ] 2"/12" [X] 3"/12" [ ] 4"/12" [ ] 6"/12" [ ] Other:

EDGE PANEL DISTANCE: [ ] 3/8" [X] 1/2" [ ] Other:

STUDS: Field / Chord
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.054" (1.37mm) 50ksi (340 MF
- [X] Double chord studs used
- [ ] Other

STUD SPACING:
- [ ] 12" O.C.
- [ ] 16" O.C.
- [X] 24" O.C.
- [ ] Other:

TRACK: Web: 3-5/8" inches; Flange: 1-1/4" inches
- [X] 0.043" (1.09mm) 33ksi (230 MPa)
- [ ] Other:

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10 7/8" Anchor Rod (# of screws): 33
- [ ] UCI 18" hold down 1/2" Anchor Rod (# of screws):
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [X] Monotonic 7.5mm/min.
- [ ] Cyclic CUREE reversed cyclic (10 mm/s)
- [X] Gravity 9 kN load at both ends of wall

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral
- [X] East Frame Brace
- [X] West Frame Brace
- [X] Wood Shear

TOTAL: 9

MOISTURE CONTENT OF SHEATHING: Moisture Meter

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

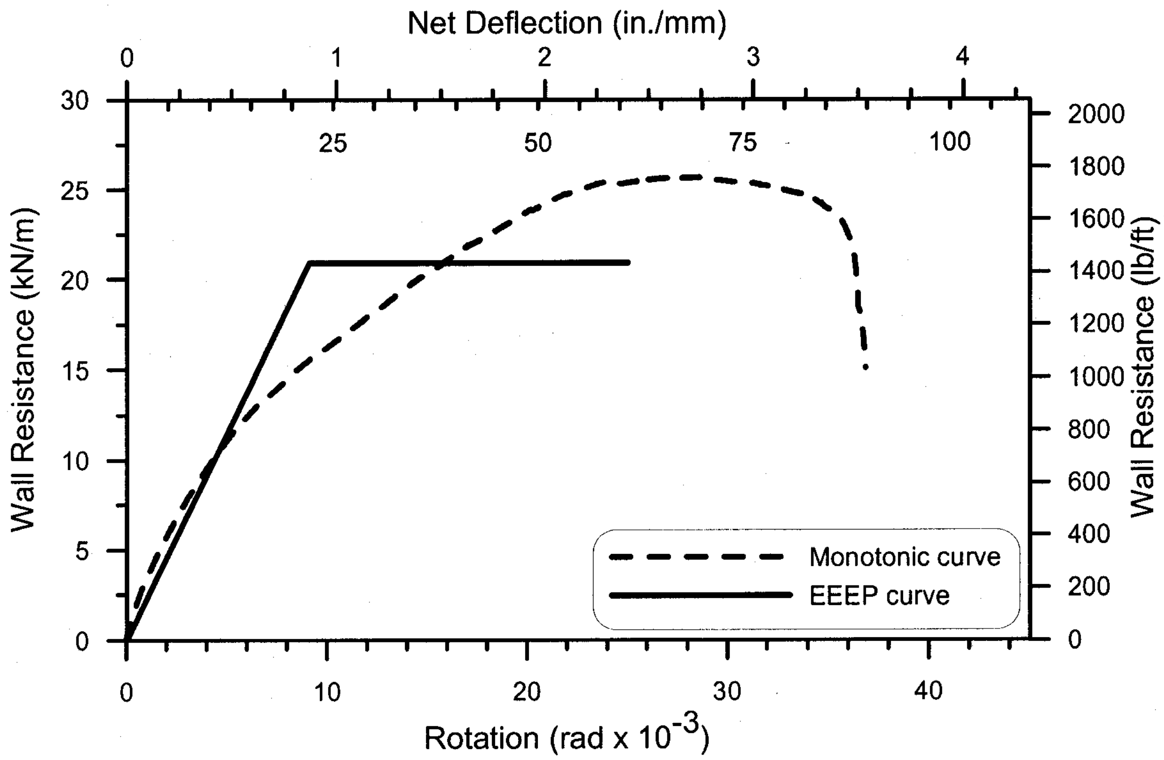
	North	North
Ww=	22.83	21.97
Wd=	21.97	20.87
m.c.=	3.91	5.27

AVG m.c. 4.59

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 50 scan/sec

COMMENTS:
-Shear anchors torqued to 77 kN
-Hold down anchors tightened to approximately 8 kN (load cells used on both hold-downs)
-Ambient temperature 23 C
-Double chord studs used
-Square plate washers (2.5"x2.5") used in all track connections
-Pilot holes drilled for screws A1,A5,A9,Q1,Q5,Q9
-Initial load set to zero at beginning of test, displacement 0.07 mm
-Gravity Loads engaged, offset is 0.096 mm
- Servo Valve made high pitch noise near end of test before breaking

Test 55A
(1220 x 2440 mm CSP 75/305 mm)



	Parameters	Units
F_u	31.31	kN
$F_{0.8u}$	25.05	kN
$F_{0.4u}$	12.52	kN
F_y	25.49	kN
K_e	1.15	kN/mm
Ductility (μ)	2.76	-
$\Delta_{net,y}$	22.08	mm
$\Delta_{net,u}$	60.96	mm
$\Delta_{net,0.8u}$	60.96	mm
$\Delta_{net,0.4u}$	10.85	mm
$Area_{Backbone}$	1272.25	J
$Area_{EEEP}$	1272.25	J
Check	OK	
R_d	**2.13**	-
S_y	**20.90**	kN/m
	2.5% Drift Limit Controls	

Test Name	55A
Date Tested	May 25, 2005
Protocol	MONOTONIC

McGill

*facing out (frame in sight)

Light Gauge Steel Frame / Wood Panel Shear Wall Testing

Test name : 55A
Date tested : May 25, 2005
Wall Size : 4' x 8'
Screw pattern : 3" / 12"
Edge Distance : 1/2"
Test mode : ☐ Cyclic ☒ Monotonic

Moisture Content A: 22.83 → 21.71g
B: 21.97 → 20.87g

WB/TO

Screw missing in frame / 1 wafer head inside

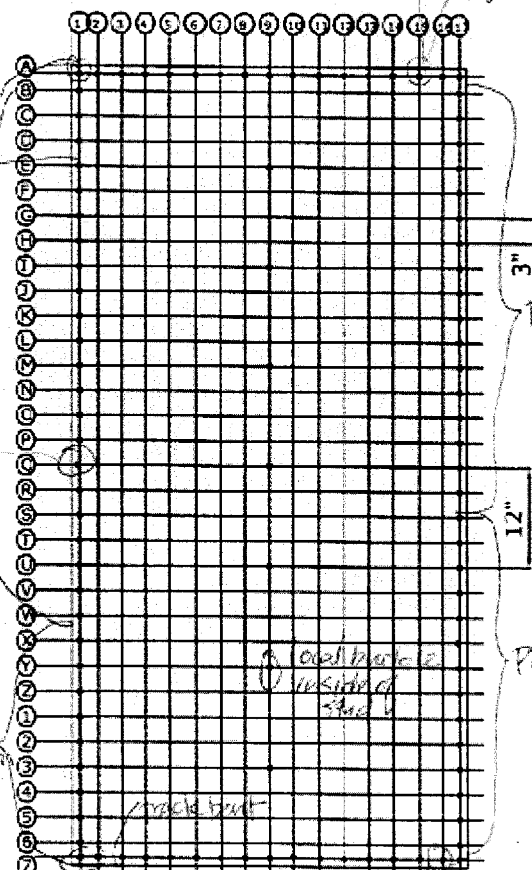
Track Bent

South plate gouged out so it sits flat against bottom (may not have been flat before)

Peak load -31.81kN

Servo-valve made a high pitch noise at the end of test

- new self tapping screws used



WB damage to middle ply

PPT due to rotation

plywood delamination

PT plywood delamination

PPT

PT - Plywood tearout & delamination

dead load inside of stud

track bent

PP screws missing in very bottom

screw head sheared

screws sheared

PT

Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Light Gauge Steel Frame / Wood Panel Shear Walls
McGill University, Montreal

TEST: 55B

RESEARCHER: KATHERINE HIKITA **ASSISTANTS:** M. OUELLET, JIANG FAN; D. MORELLO, R. CARBONNEAU

DATE: Built: May 19; Tested: Wednesday May 25, 2005 **TIME:** 16:00

DIMENSIONS OF WALL: 4 FT X 8 FT **PANEL ORIENTATION:** Vertical; Sheathing one side

SHEATHING:
- [ ] Plywood 15/32" APA Rated Exposure 1 (USA)
- [ ] OSB 7/16" APA Rated Exposure 1 (USA)
- [X] Plywood (CSA 0151M) CSP 12.5mm (1/2") — CAN/PLY Exterior CSP
- [ ] Plywood (CSA O121M) DFP 12.5mm (1/2") — CAN/PLY Exterior DFP
- [ ] OSB (CSA O325) 11 mm (7/16")
- [ ] Other — MFR: ALBERTA PLYWOOD, MILL AB 244 - Weatherproof Bond

SCREWS

Sheathing:
- [ ] No.8 gauge 1.5" self-piercing Bugle head LOX drive (Grabber Superdrive)
- [ ] No.8 gauge 1.0" self-piercing Bugle head (Flat socket head screw) (HD)
- [X] No. 8 gauge 1.25" self-drilling Bugle head LOX Drive (Grabber Superdrive)
- [ ] No.9 gauge 1.0" self-piercing Bugle head (screws Y2,Y12 in track)

Framing:
- [ ] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive

Hold downs:
- [X] No.10 gauge 0.75" self-drilling Hex washer head

Loading Beam: A325 3/4" bolts — 3 bolts [ ] 6 bolts [X] 12 bolts [ ]

Back-to-Back Chord Studs:
- [X] No.10 gauge 0.75" self-drilling Hex washer head (2@12" O.C.)

SHEATHING FASTENER SCHEDULE: [ ] 2"/12" [X] 3"/12" [ ] 4"/12" [ ] 6"/12" [ ] Other:

EDGE PANEL DISTANCE: [ ] 3/8" [X] 1/2" [ ] Other:

STUDS: Field / Chord
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.054" (1.37mm) 50ksi (340 MF
- [X] Double chord studs used
- [ ] Other

STUD SPACING:
- [ ] 12" O.C.
- [ ] 16" O.C.
- [X] 24" O.C.
- [ ] Other:

TRACK: Web: 3-5/8" inches; Flange: 1-1/4" inches
- [X] 0.043" (1.09mm) 33ksi (230 MPa)
- [ ] Other:

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10 7/8" Anchor Rod — (# of screws): 33
- [ ] UCI 18" hold down 1/2" Anchor Rod — (# of screws):
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [X] Monotonic — 7.5mm/min.
- [ ] Cyclic — CUREE reversed cyclic (10 mm/s)
- [ ] Gravity — 9 kN load at both ends of wall

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral
- [X] East Frame Brace
- [X] West Frame Brace
- [X] Wood Shear

TOTAL: 9

MOISTURE CONTENT OF SHEATHING: Moisture Meter

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

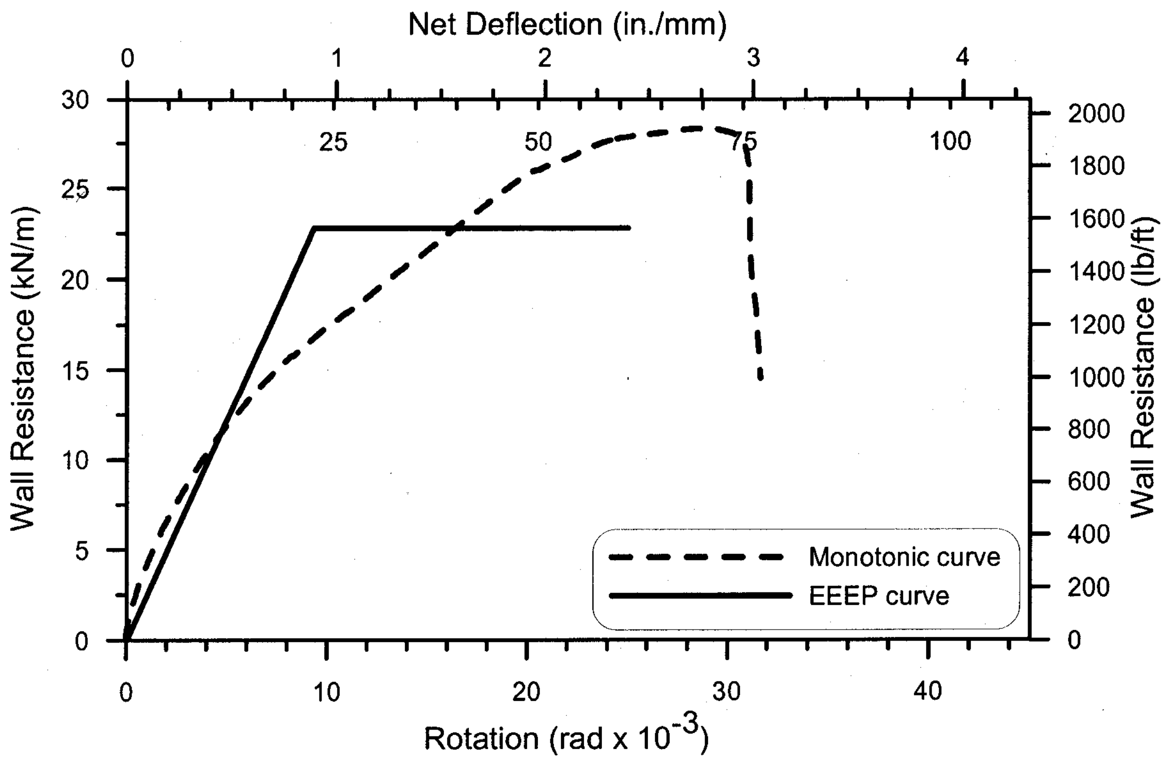
	North	North
Ww=	20.80	22.04
Wd=	19.68	20.85
m.c.=	5.69	5.71

AVG m.c. 5.70

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 50 scan/sec

COMMENTS:
- -Shear anchors torqued to 77 kN
- -Hold down anchors tightened to approximately 8 kN (load cells used on both hold-downs)
- -Ambient temperature 23 C
- -Double chord studs used
- -Square plate washers (2.5"x2.5") used in all track connections
- -Pilot holes drilled for screws A1,A5,A9,Q1,Q5,Q9
- -Initial load set to zero at beginning of test, displacement -0.337 mm
- -No gravity loads

Test 55B
(1220 x 2440 mm CSP 75/305 mm)



	Parameters	Units
F_u	34.58	kN
$F_{0.8u}$	27.66	kN
$F_{0.4u}$	13.83	kN
F_y	27.81	kN
K_e	1.23	kN/mm
Ductility (μ)	2.69	-
$\Delta_{net,y}$	22.69	mm
$\Delta_{net,u}$	60.96	mm
$\Delta_{net,0.8u}$	60.96	mm
$\Delta_{net,0.4u}$	11.29	mm
$Area_{Backbone}$	1379.91	J
$Area_{EEEP}$	1379.91	J
Check	OK	
R_d	**2.09**	-
S_y	**22.81**	kN/m
	2.5% Drift Limit Controls	

Test Name	55B
Date Tested	May 25, 2005
Protocol	MONOTONIC

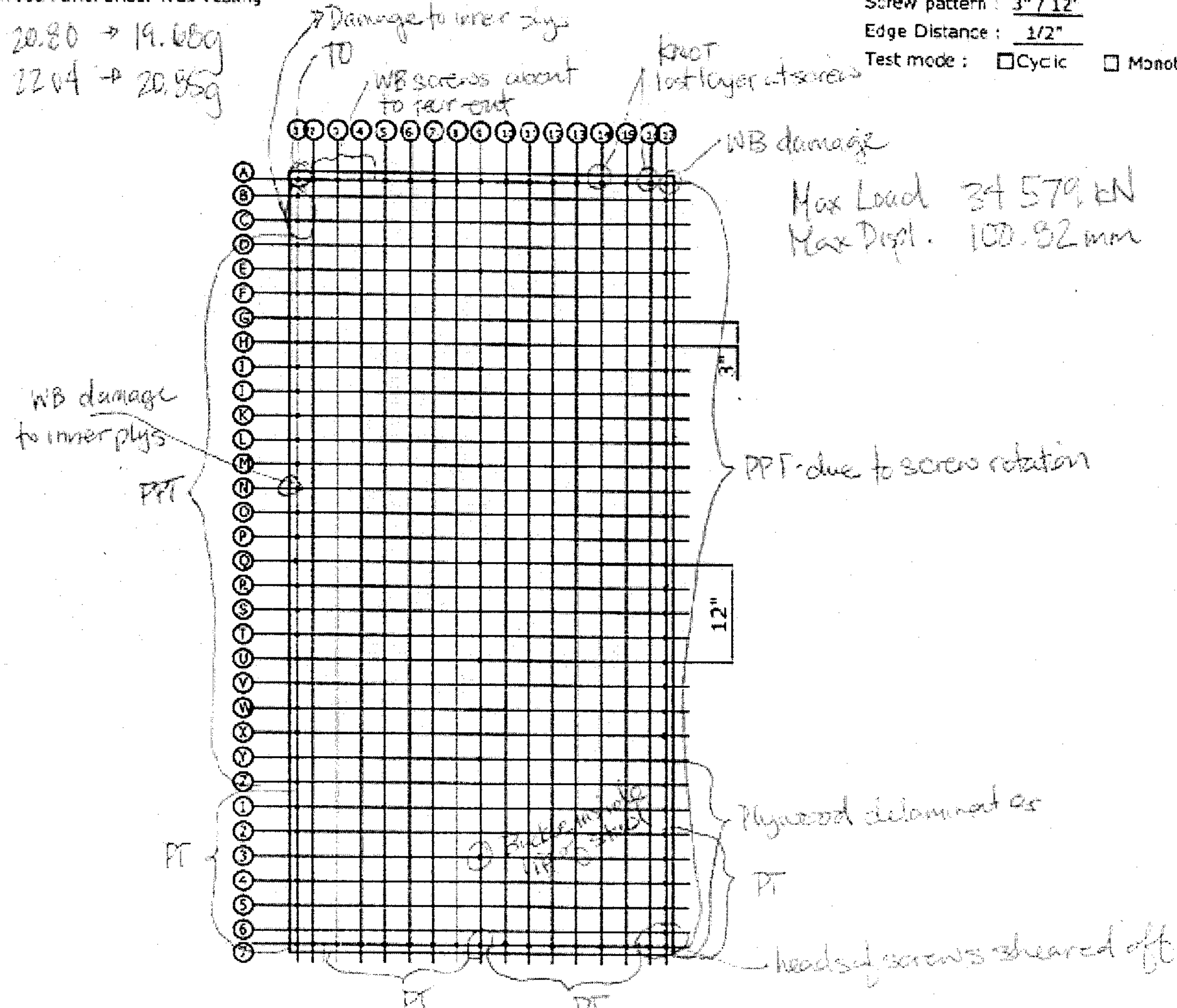
McGill

No Gravity loads
New self tapping screws

Light Gauge Steel Frame / Wood Panel Shear Wall Testing

Test name : 53B
Date tested : May 25 2005
Wall Size : 4' x 8'
Screw pattern : 3" / 12"
Edge Distance : 1/2"
Test mode : ☐ Cyclic ☐ Monotonic

Moisture Content A: 20.80 → 19.68g
B: 22.04 → 20.95g



Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DF)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Light Gauge Steel Frame / Wood Panel Shear Walls
McGill University, Montreal

TEST: 55C

RESEARCHER: KATHERINE HIKITA **ASSISTANTS:** M. OUELLET, JIANG FAN, M. ALKHARAT

DATE: Built: May 25; Tested: Thursday May 26, 2005 **TIME:** 10:30

DIMENSIONS OF WALL: 4 FT X 8 FT **PANEL ORIENTATION:** Vertical; Sheathing one side

SHEATHING:
- [ ] Plywood 15/32" APA Rated Exposure 1 (USA)
- [ ] OSB 7/16" APA Rated Exposure 1 (USA)
- [X] Plywood (CSA 0151M) CSP 12.5mm (1/2") — CAN/PLY Exterior CSP
- [ ] Plywood (CSA O121M) DFP 12.5mm (1/2") — CAN/PLY Exterior DFP
- [ ] OSB (CSA O325) 11 mm (7/16")
- [ ] Other — MFR: ALBERTA PLYWOOD, MILL AB 244 - Weatherproof Bond

SCREWS

Sheathing:
- [X] No.8 gauge 1.5" self-piercing Bugle head LOX drive (Grabber Superdrive)
- [ ] No.8 gauge 1.0" self-piercing Bugle head (Flat socket head screw) (HD)
- [X] No. 8 gauge 1.25" self-drilling Bugle head LOX Drive (Grabber Superdrive)
- [ ] No.9 gauge 1.0" self-piercing Bugle head (screws Y2,Y12 in track)

Framing:
- [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive

Hold downs:
- [X] No.10 gauge 0.75" self-drilling Hex washer head

Loading Beam: A325 3/4" bolts — 3 bolts [ ] 6 bolts [X] 12 bolts [ ]

Back-to-Back Chord Studs:
- [X] No.10 gauge 0.75" self-drilling Hex washer head (2@12" O.C.)

SHEATHING FASTENER SCHEDULE: [ ] 2"/12" [X] 3"/12" [ ] 4"/12" [ ] 6"/12" [ ] Other:

EDGE PANEL DISTANCE: [ ] 3/8" [X] 1/2" [ ] Other:

STUDS: Field / Chord
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.054" (1.37mm) 50ksi (340 MF
- [X] Double chord studs used
- [ ] Other

STUD SPACING:
- [ ] 12" O.C.
- [ ] 16" O.C.
- [X] 24" O.C.
- [ ] Other:

TRACK: Web: 3-5/8" inches; Flange: 1-1/4" inches
- [X] 0.043" (1.09mm) 33ksi (230 MPa)
- [ ] Other:

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10 7/8" Anchor Rod — (# of screws): 33
- [ ] UCI 18" hold down 1/2" Anchor Rod — (# of screws):
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [X] Monotonic — 7.5mm/min.
- [ ] Cyclic — CUREE reversed cyclic (10 mm/s)
- [X] Gravity — 9 kN load at both ends of wall

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral
- [X] East Frame Brace
- [X] West Frame Brace
- [X] Wood Shear

TOTAL: 9

MOISTURE CONTENT OF SHEATHING: Moisture Meter

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

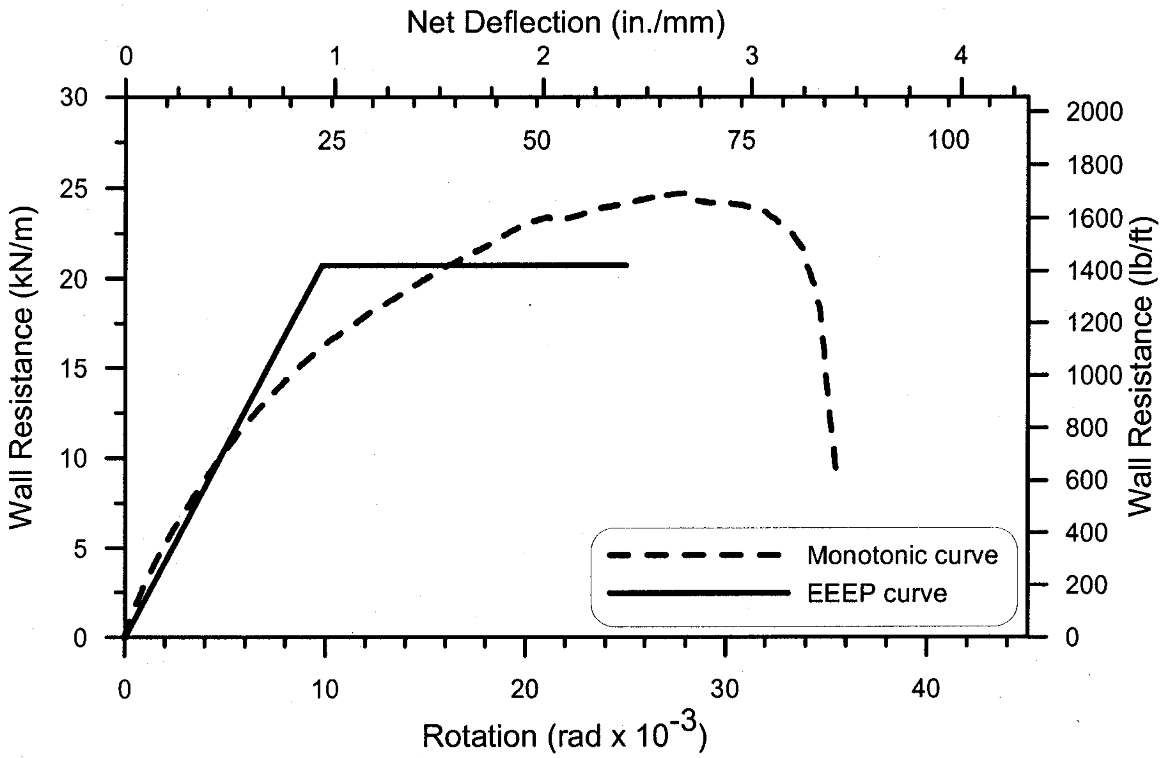
	North	North
Ww=	21.13	22.10
Wd=	20.10	21.02
m.c.=	5.12	5.14

AVG m.c. 5.13

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 50 scan/sec

COMMENTS:
- -Shear anchors torqued to 77 kN
- -Hold down anchors tightened to approximately 8 kN (load cells used on both hold-downs)
- -Ambient temperature 23 C
- -Double chord studs used
- -Square plate washers (2.5"x2.5") used in all track connections
- -Pilot holes drilled for screws A1,A5,A9,Q1,Q5,Q9
- -Initial load set to zero at beginning of test, displacement 0.588 mm
- - Offset due to Gravity Loads 0.54 mm
- - Mixture of self-piercing and self-drilling screws used to attach sheathing (Difficulties installing self-piercing)

Test 55C
(1220 x 2440 mm CSP 75/305 mm)



	Parameters	Units
F_u	30.12	kN
$F_{0.8u}$	24.10	kN
$F_{0.4u}$	12.05	kN
F_y	25.28	kN
K_e	1.06	kN/mm
Ductility (μ)	2.55	-
$\Delta_{net,y}$	23.88	mm
$\Delta_{net,u}$	60.96	mm
$\Delta_{net,0.8u}$	60.96	mm
$\Delta_{net,0.4u}$	11.38	mm
Area$_{Backbone}$	1239.39	J
Area$_{EEEP}$	1239.39	J
Check	OK	
R_d	**2.03**	-
S_y	**20.74**	kN/m
	2.5% Drift Limit Controls	

Test Name	55C
Date Tested	May 26, 2005
Protocol	MONOTONIC

McGill Gravity loads included - wood facing is

Light Gauge Steel Frame / Wood Panel Shear Wall Testing

Moisture Content: A: 21.13→20.10g
B: 22.10→21.02g

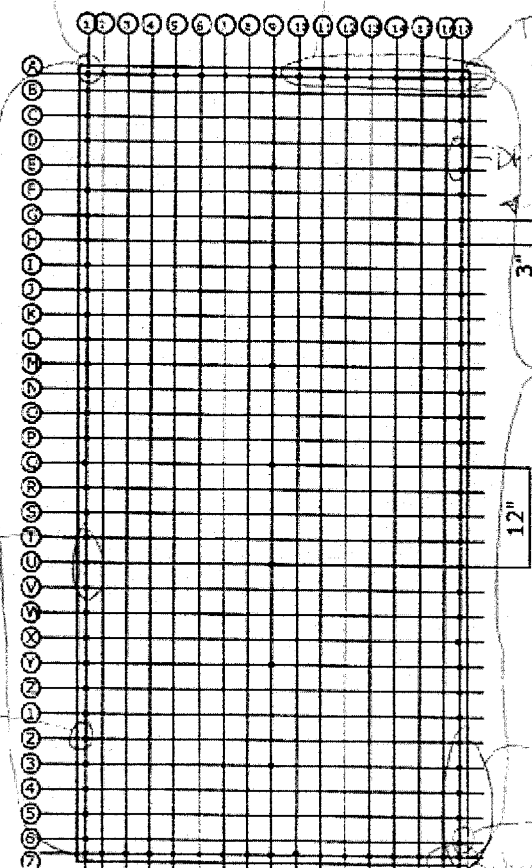
Test name : 55C
Date tested : May 26 2005
Wall Size : 4' x 8'
Screw pattern : 3" / 12"
Edge Distance : 1/2"
Test mode : ☐ Cyclic ☑ Monotonic

- Stud (interior) doesn't buckle because the lip is in tension

PPT

WB damage

TO/WB



DP ply fell off

small buckles in stud (waves-lips) around screws along this lip/edge

3"

Max Load 30.631 kN
Max Disp 100.76 mm

Plywood Delaminated
PT

PPT

12"

DP damage from screws in plywood

* LVDT plate hit wood at bottom

bump in stud around screw

DP ply fell off

screw head sheared off

heads of screws sheared off

Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Light Gauge Steel Frame / Wood Panel Shear Walls
McGill University, Montreal

TEST: 55D

RESEARCHER: KATHERINE HIKITA **ASSISTANTS:** M. OUELLET, D. MORELLO; R. CARBONNEAU, JIANG FAN

DATE: Built: May 25; Tested: Friday May 27, 2005 **TIME:** 10:30

DIMENSIONS OF WALL: 4 FT X 8 FT **PANEL ORIENTATION:** Vertical; Sheathing one side

SHEATHING:
- [ ] Plywood 15/32" APA Rated Exposure 1 (USA)
- [ ] OSB 7/16" APA Rated Exposure 1 (USA)
- [X] Plywood (CSA 0151M) CSP 12.5mm (1/2") — CAN/PLY Exterior CSP
- [ ] Plywood (CSA O121M) DFP 12.5mm (1/2") — CAN/PLY Exterior DFP
- [ ] OSB (CSA O325) 11 mm (7/16")
- [ ] Other — MFR: ALBERTA PLYWOOD, MILL AB 244 - Weatherproof Bond

SCREWS Sheathing:
- [ ] No.8 gauge 1.5" self-piercing Bugle head LOX drive (Grabber Superdrive)
- [ ] No.8 gauge 1.0" self-piercing Bugle head (Flat socket head screw) (HD)
- [X] No. 8 gauge 1.25" self-drilling Bugle head LOX Drive (Grabber Superdrive)
- [ ] No.9 gauge 1.0" self-piercing Bugle head (screws Y2,Y12 in track)

Framing:
- [ ] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive

Hold downs:
- [X] No.10 gauge 0.75" self-drilling Hex washer head

Loading Beam: A325 3/4" bolts — 3 bolts [ ] 6 bolts [X] 12 bolts [ ]

Back-to-Back Chord Studs:
- [X] No.10 gauge 0.75" self-drilling Hex washer head (2@12" O.C.)

SHEATHING FASTENER SCHEDULE: [ ] 2"/12" [X] 3"/12" [ ] 4"/12" [ ] 6"/12" [ ] Other:

EDGE PANEL DISTANCE: [ ] 3/8" [X] 1/2" [ ] Other:

STUDS: Field / Chord
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.054" (1.37mm) 50ksi (340 MF
- [X] Double chord studs used
- [ ] Other

STUD SPACING:
- [ ] 12" O.C.
- [ ] 16" O.C.
- [X] 24" O.C.
- [ ] Other:

TRACK: Web: 3-5/8" inches; Flange: 1-1/4" inches
- [X] 0.043" (1.09mm) 33ksi (230 MPa)
- [ ] Other:

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10 7/8" Anchor Rod (# of screws): 33
- [ ] UCI 18" hold down 1/2" Anchor Rod (# of screws):
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [X] Monotonic 7.5mm/min.
- [ ] Cyclic CUREE reversed cyclic (10 mm/s)
- [X] Gravity 9 kN load at both ends of wall

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral
- [X] East Frame Brace
- [X] West Frame Brace
- [X] Wood Shear

TOTAL: 9

MOISTURE CONTENT OF SHEATHING: Moisture Meter

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

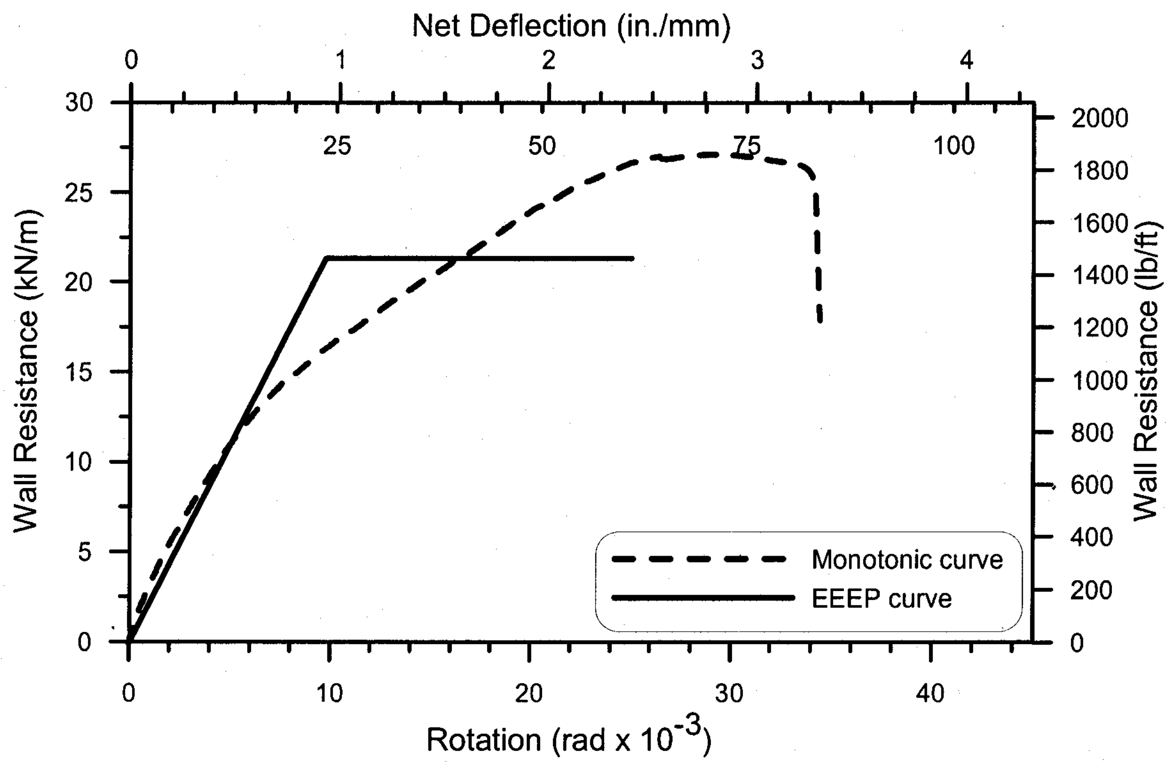
	North	North
Ww=	20.95	22.13
Wd=	19.95	21.05
m.c.=	5.01	5.13

AVG m.c. 5.07

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 50 scan/sec

COMMENTS:
- -Shear anchors torqued to 77 kN
- -Hold down anchors tightened to approximately 8 kN (load cells used on both hold-downs)
- -Ambient temperature 23 C
- -Double chord studs used
- -Square plate washers (2.5"x2.5") used in all track connections
- -Pilot holes drilled for screws A1,A5,A9,Q1,Q5,Q9
- -Initial load set to zero at beginning of test, displacement 1.832 mm
- - Offset due to Gravity Loads included in initial offset

Test 55D
(1220 x 2440 mm CSP 75/305 mm)



	Parameters	Units
F_u	33.07	kN
$F_{0.8u}$	26.46	kN
$F_{0.4u}$	13.23	kN
F_y	26.00	kN
K_e	1.10	kN/mm
Ductility (μ)	2.57	-
$\Delta_{net,y}$	23.70	mm
$\Delta_{net,u}$	60.96	mm
$\Delta_{net,0.8u}$	60.96	mm
$\Delta_{net,0.4u}$	12.06	mm
$Area_{Backbone}$	1277.07	J
$Area_{EEEP}$	1277.07	J
Check	OK	
R_d	**2.04**	-
S_y	**21.33**	kN/m
	2.5% Drift Limit Controls	

Test Name	55D
Date Tested	May 27, 2005
Protocol	MONOTONIC

McGill Gravity loads included

Light Gauge Steel Frame / Wood Panel Shear Wall Testing

DFP moisture Content 25.39 → 23.54g

Moisture Content: A 20.95 → 19.95g

B 22.13 → 21.05g

Test name : SS2

Date tested : May 27, 2005

Wall Size : 4' x 8'

Screw pattern : 3" / 12"

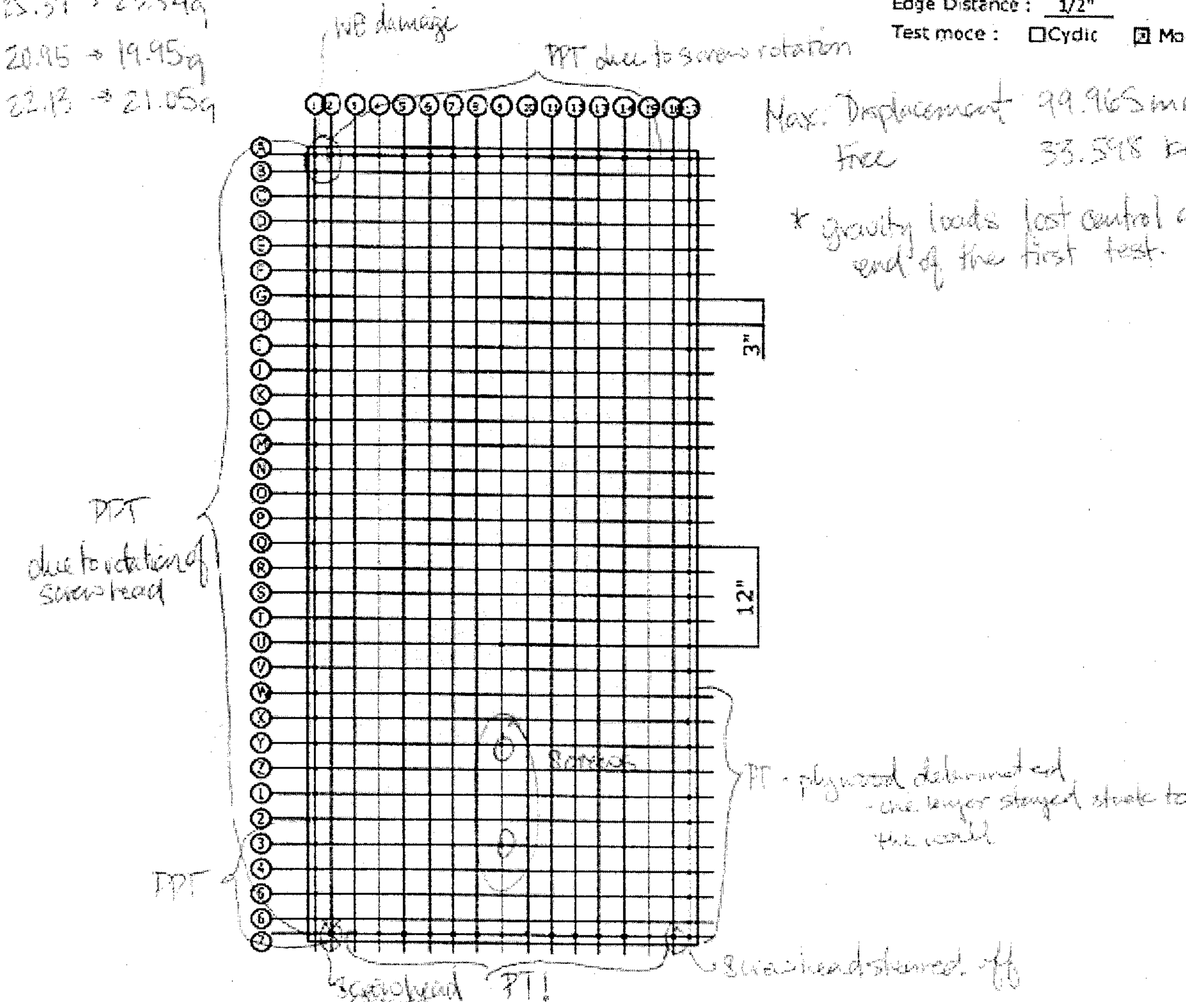
Edge Distance : 1/2"

Test mode : ☐ Cyclic ☒ Monotonic

Max. Displacement 99.965 mm

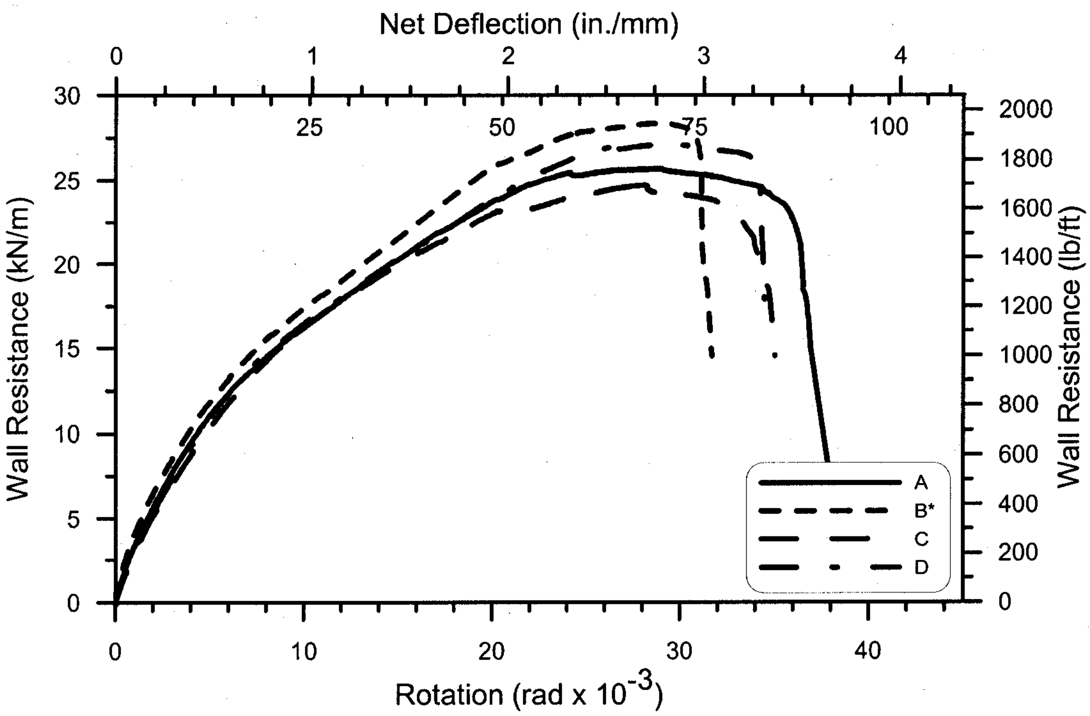
Force 33.598 kN

* gravity loads lost control at end of the first test.

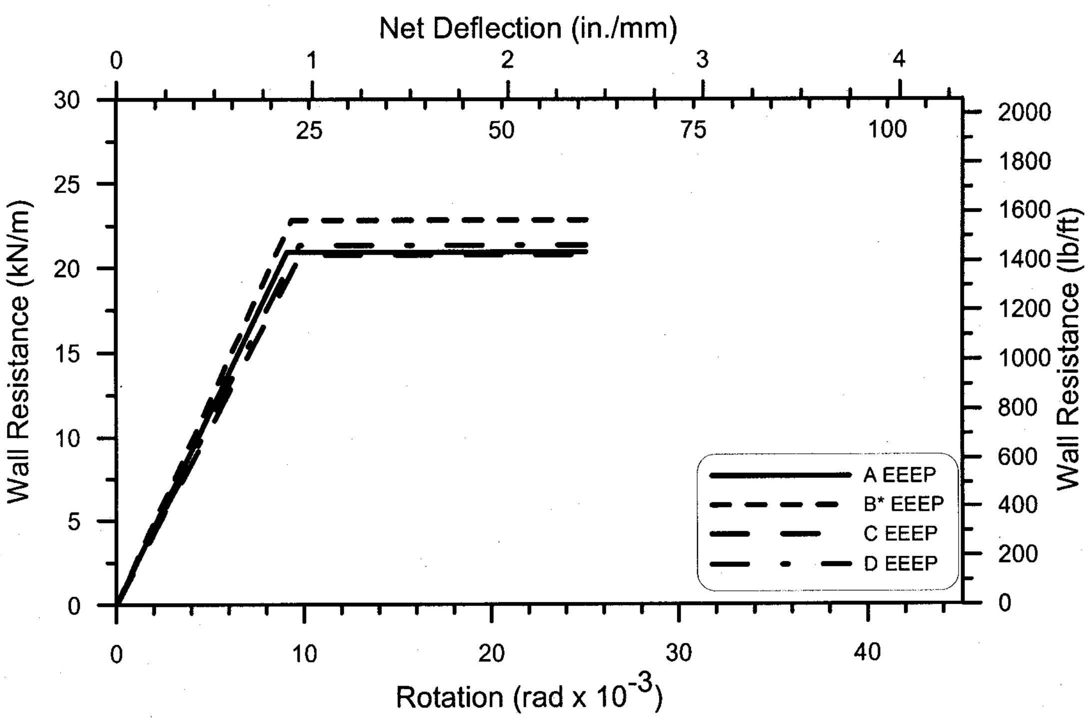


Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Test 55-A,B,C,D
(1220 x 2440 mm CSP 75/305 mm)



Test 55-A,B,C,D EEEP
(1220 x 2440 mm CSP 75/305 mm)



Light Gauge Steel Frame / Wood Panel Shear Walls
McGill University, Montreal

TEST: 56A

RESEARCHER: KATHERINE HIKITA **ASSISTANTS:** M. OUELLET, C. ROGERS

DATE: Built: May 25; Tested: Sunday May 29, 2005 **TIME:** 15:00

DIMENSIONS OF WALL: 4 FT X 8 FT **PANEL ORIENTATION:** Vertical
Sheathing one side

SHEATHING:
- [ ] Plywood 15/32" APA Rated Exposure 1 (USA)
- [ ] OSB 7/16" APA Rated Exposure 1 (USA)
- [X] Plywood (CSA 0151M) CSP 12.5mm (1/2") — CAN/PLY Exterior CSP
- [ ] Plywood (CSA O121M) DFP 12.5mm (1/2") — CAN/PLY Exterior DFP
- [ ] OSB (CSA O325) 11 mm (7/16")
- [ ] Other — MFR: ALBERTA PLYWOOD, MILL AB 244 - Weatherproof Bond

SCREWS Sheathing:
- [ ] No.8 gauge 1.5" self-piercing Bugle head LOX drive (Grabber Superdrive)
- [ ] No.8 gauge 1.0" self-piercing Bugle head (Flat socket head screw) (HD)
- [X] No. 8 gauge 1.25" self-drilling Bugle head LOX Drive (Grabber Superdrive)
- [ ] No.9 gauge 1.0" self-piercing Bugle head (screws Y2,Y12 in track)

Framing:
- [ ] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive

Hold downs:
- [X] No.10 gauge 0.75" self-drilling Hex washer head

Loading Beam: A325 3/4" bolts — 3 bolts [ ] 6 bolts [X] 12 bolts [ ]

Back-to-Back Chord Studs:
- [X] No.10 gauge 0.75" self-drilling Hex washer head (2@12" O.C.)

SHEATHING FASTENER SCHEDULE: [ ] 2"/12" [X] 3"/12" [ ] 4"/12" [ ] 6"/12" [ ] Other:

EDGE PANEL DISTANCE: [ ] 3/8" [X] 1/2" [ ] Other:

STUDS: Field / Chord
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.054" (1.37mm) 50ksi (340 MF
- [X] Double chord studs used
- [ ] Other

STUD SPACING:
- [ ] 12" O.C.
- [ ] 16" O.C.
- [X] 24" O.C.
- [ ] Other:

TRACK: Web: 3-5/8" inches; Flange: 1-1/4" inches
- [X] 0.043" (1.09mm) 33ksi (230 MPa)
- [ ] Other:

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10 7/8" Anchor Rod (# of screws): 33
- [ ] UCI 18" hold down 1/2" Anchor Rod (# of screws):
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [ ] Monotonic — 7.5mm/min.
- [X] Cyclic — CUREE reversed cyclic (10 mm/s)
- [ ] Gravity — 9 kN load at both ends of wall

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral
- [X] East Frame Brace
- [X] West Frame Brace
- [X] Wood Shear

TOTAL: 9

MOISTURE CONTENT OF SHEATHING: Moisture Meter

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

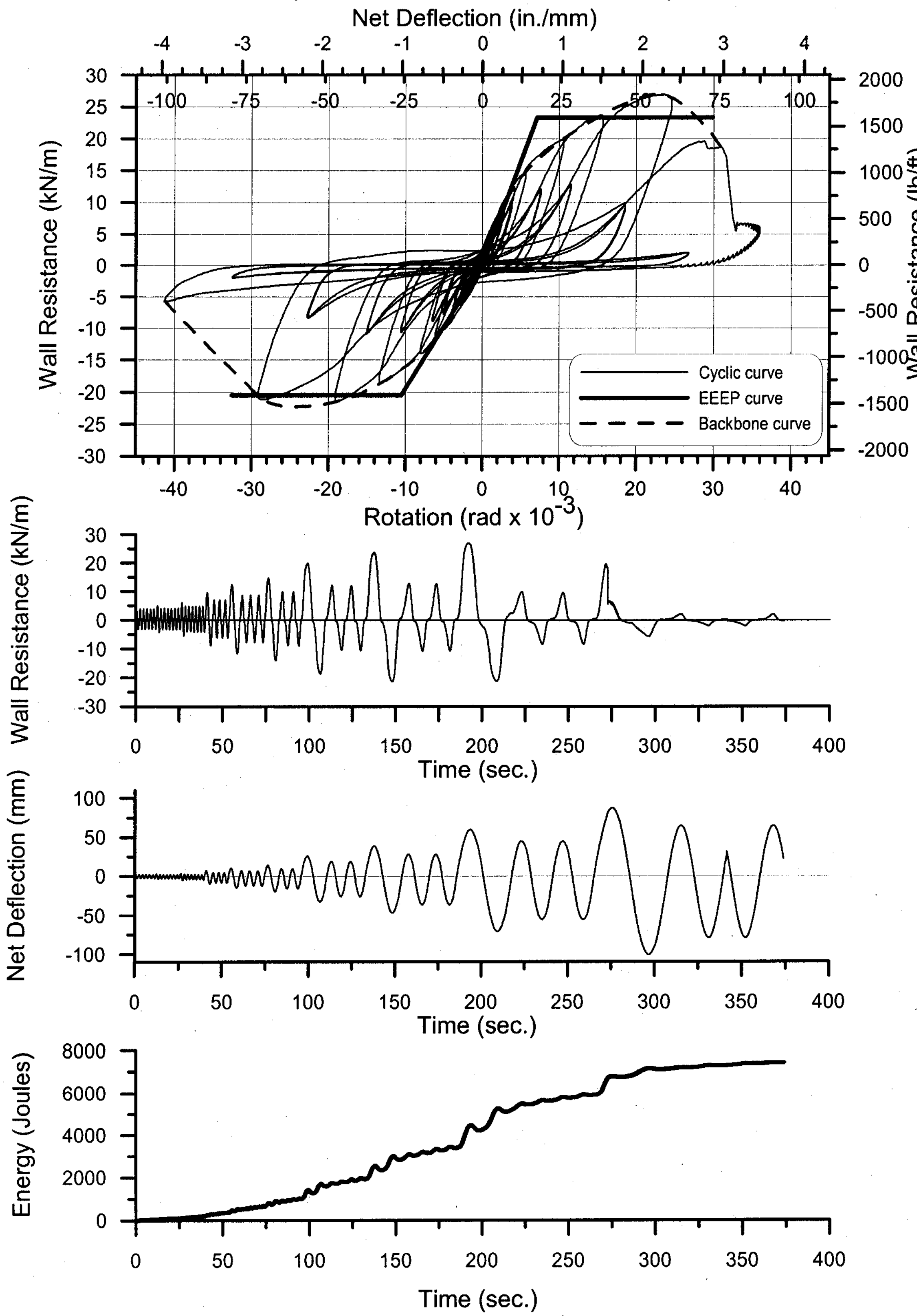
	North	North
Ww=	23.12	23.73
Wd=	21.91	22.50
m.c.=	5.52	5.47

AVG m.c. 5.49

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 50 scan/sec

COMMENTS:
- -Shear anchors torqued to 77 kN
- -Hold down anchors tightened to approximately 8 kN (load cells used on both hold-downs)
- -Ambient temperature 23 C
- -Double chord studs used
- -Square plate washers (2.5"x2.5") used in all track connections
- -Pilot holes drilled for screws A1,A5,A9,Q1,Q5,Q9
- -Initial load set to zero at beginning of test, displacement -0.418 mm (includes gravity load force)
- - No gravity loads

Test 56A
(1220 x 2440 mm CSP 75/305 mm)



	Negative	Positive	Units
F_u	-26.18	32.80	kN
$F_{0.8u}$	-20.95	26.24	kN
$F_{0.4u}$	-10.47	13.12	kN
F_y	-24.98	28.46	kN
K_e	0.98	1.64	kN/mm
Ductility (μ)	3.11	4.19	-
$\Delta_{net,y}$	-25.52	17.35	mm
$\Delta_{net,u}$	-45.98	57.13	mm
$\Delta_{net,0.8u}$	-79.30	72.80	mm
$\Delta_{net,0.4u}$	-10.70	8.00	mm
$Area_{Backbone}$	1662.28	1825.23	kN-mm
$Area_{EEEP}$	1662.28	1825.23	kN-mm
Check	OK	OK	
R_d	**2.28**	**2.72**	-
S_y	**-20.49**	**23.35**	kN/m

Test Name	56A
Date Tested	May 29, 2005
Protocol	CUREE ORDINARY GROUND MOTIONS

McGill

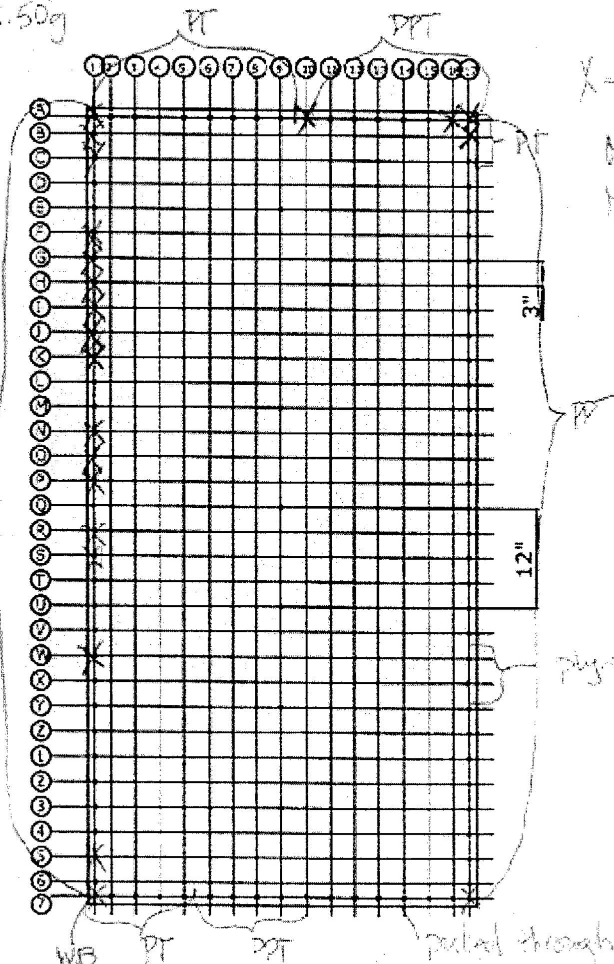
Light Gauge Steel Frame / Wood Panel Shear Wall Testing

Moisture Contents A: 23.12g → 21.91g
B 23.73g → 22.50g

Test name : 56 A
Date tested : May 29, 2008
Wall Size : 4' x 8'
Screw pattern : 3" / 12"
Edge Distance : 1/2"
Test mode : ☒ Cyclic ☐ Monotonic

X = screws w/ sheared heads
Max. Displacement 105.92mm
Max. load 33.286 kN

plywood
delaminated
damaged



plywood delaminated/damaged

pulled through steel

Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Light Gauge Steel Frame / Wood Panel Shear Walls
McGill University, Montreal

TEST: 56B

RESEARCHER: KATHERINE HIKITA **ASSISTANTS:** M. OUELLET, D. MORELLO
R. CARBONNEAU, JIANG FAN

DATE: Built: May 25; Tested: Sunday May 29, 2005 **TIME:** 13:00

DIMENSIONS OF WALL: 4 FT X 8 FT **PANEL ORIENTATION:** Vertical
Sheathing one side

SHEATHING:
- [ ] Plywood 15/32" APA Rated Exposure 1 (USA)
- [ ] OSB 7/16" APA Rated Exposure 1 (USA)
- [X] Plywood (CSA 0151M) CSP 12.5mm (1/2") — CAN/PLY Exterior CSP
- [ ] Plywood (CSA O121M) DFP 12.5mm (1/2") — CAN/PLY Exterior DFP
- [ ] OSB (CSA O325) 11 mm (7/16")
- [ ] Other — MFR: ALBERTA PLYWOOD, MILL AB 244 - Weatherproof Bond

SCREWS Sheathing:
- [ ] No.8 gauge 1.5" self-piercing Bugle head LOX drive (Grabber Superdrive)
- [ ] No.8 gauge 1.0" self-piercing Bugle head (Flat socket head screw) (HD)
- [X] No. 8 gauge 1.25" self-drilling Bugle head LOX Drive (Grabber Superdrive)
- [ ] No.9 gauge 1.0" self-piercing Bugle head (screws Y2,Y12 in track)

Framing:
- [ ] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive

Hold downs:
- [X] No.10 gauge 0.75" self-drilling Hex washer head

Loading Beam: A325 3/4" bolts — 3 bolts [ ] 6 bolts [X] 12 bolts [ ]

Back-to-Back Chord Studs:
- [X] No.10 gauge 0.75" self-drilling Hex washer head (2@12" O.C.)

SHEATHING FASTENER SCHEDULE: [ ] 2"/12" [X] 3"/12" [ ] 4"/12" [ ] 6"/12" [ ] Other:

EDGE PANEL DISTANCE: [ ] 3/8" [X] 1/2" [ ] Other:

STUDS: Field / Chord
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.054" (1.37mm) 50ksi (340 MF
- [X] Double chord studs used
- [ ] Other

STUD SPACING:
- [ ] 12" O.C.
- [ ] 16" O.C.
- [X] 24" O.C.
- [ ] Other:

TRACK: Web: 3-5/8" inches; Flange: 1-1/4" inches
- [X] 0.043" (1.09mm) 33ksi (230 MPa)
- [ ] Other:

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10 7/8" Anchor Rod — (# of screws): 33
- [ ] UCI 18" hold down 1/2" Anchor Rod — (# of screws):
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [ ] Monotonic — 7.5mm/min.
- [X] Cyclic — CUREE reversed cyclic (10 mm/s)
- [X] Gravity — 9 kN load at both ends of walls

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral
- [X] East Frame Brace
- [X] West Frame Brace
- [X] Wood Shear

TOTAL: 9

MOISTURE CONTENT OF SHEATHING: Moisture Meter

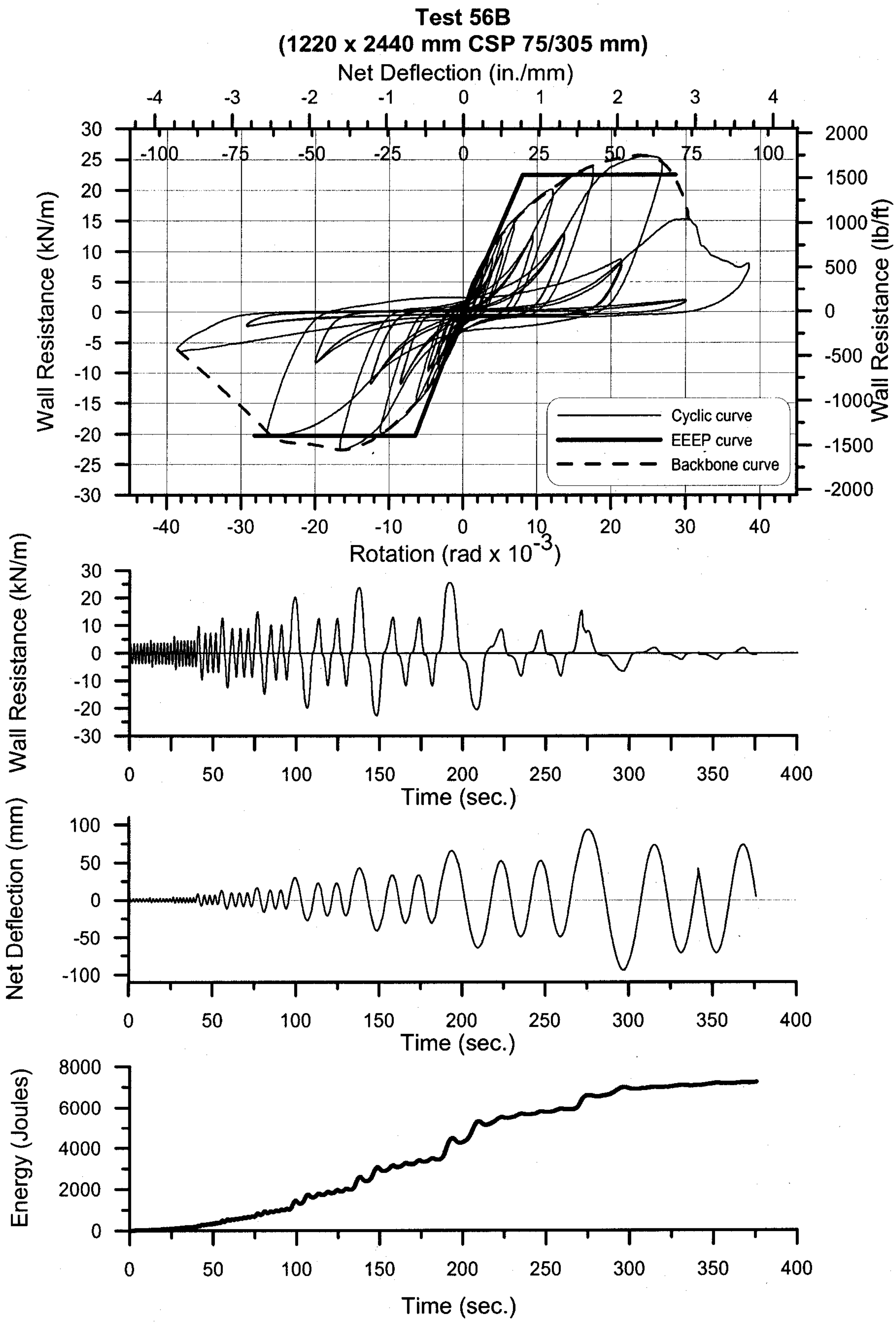
OVEN DRIED ACCORDING TO APA TEST METHOD P-6

Ww=	22.50	23.16
Wd=	21.19	21.81
m.c.=	6.18	6.19
	North	North

AVG m.c. 6.19

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 50 scan/sec

COMMENTS:
-Shear anchors torqued to 77 kN
-Hold down anchors tightened to approximately 8 kN (load cells used on both hold-downs)
-Ambient temperature 23 C
-Double chord studs used
-Square plate washers (2.5"x2.5") used in all track connections
-Pilot holes drilled for screws A1,A5,A9,Q1,Q5,Q9
-Initial load set to zero at beginning of test, displacement 0.781 mm (includes gravity load force)
- Gravity loads included



Test 56B
(1220 x 2440 mm CSP 75/305 mm)
Net Deflection (in./mm)
Wall Resistance (kN/m)
Wall Resistance (lb/ft)
Rotation (rad x 10^-3)
Cyclic curve
EEEP curve
Backbone curve
Time (sec.)
Net Deflection (mm)
Energy (Joules)

	Negative	Positive	Units
F_u	-27.56	31.16	kN
$F_{0.8u}$	-22.05	24.93	kN
$F_{0.4u}$	-11.02	12.46	kN
F_y	-24.68	27.45	kN
K_e	1.57	1.40	kN/mm
Ductility (μ)	4.38	3.56	-
$\Delta_{net,y}$	-15.67	19.60	mm
$\Delta_{net,u}$	-39.98	60.20	mm
$\Delta_{net,0.8u}$	-68.70	69.80	mm
$\Delta_{net,0.4u}$	-7.00	8.90	mm
$Area_{Backbone}$	1502.15	1647.09	kN-mm
$Area_{EEEP}$	1502.15	1647.09	kN-mm
Check	OK	OK	
R_d	**2.79**	**2.47**	-
S_y	**-20.24**	**22.52**	kN/m

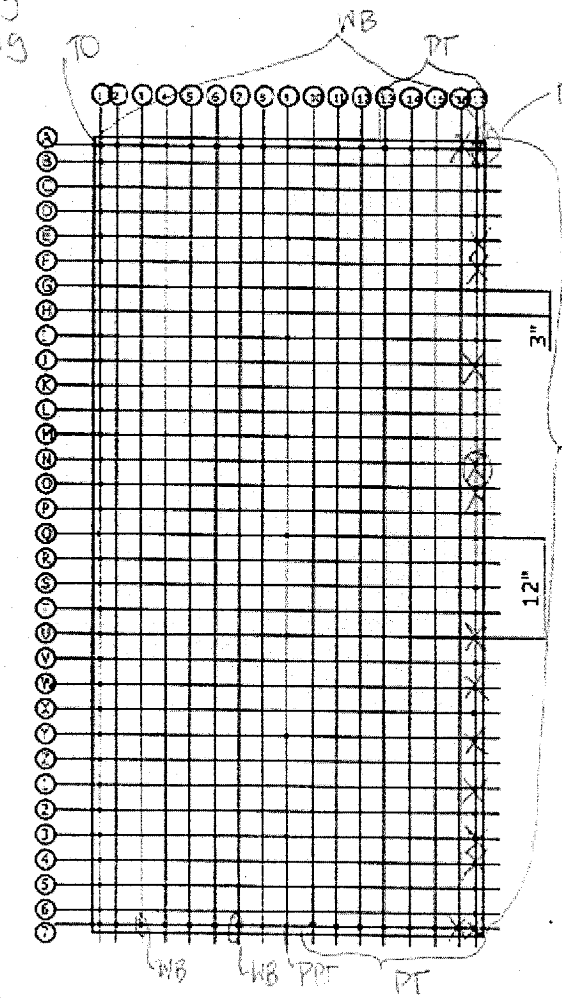
Test Name	56B
Date Tested	May 29, 2005
Protocol	CUREE ORDINARY GROUND MOTIONS

McGill Gravity loads included

Light Gauge Steel Frame / Wood Panel Shear Wall Testing

Moisture Content: A 22.59g→21.19g
B 23.16g→21.81g

Test name : 56B
Date tested : May 29, 2005
Wall Size : 4' x 8'
Screw pattern : 3" / 12"
Edge Distance : 1/2"
Test mode : ☒ Cyclic ☐ Monotonic



DP ply splintered off

X = screw sheared

Many of the new self-tapping 1.25" screws broke during test.

Max. Displacement 105.92 mm
Max. Load 31.654 kN

Watch next test to determine point at which failure occurs for screws.

Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DF)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

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Light Gauge Steel Frame / Wood Panel Shear Walls
McGill University, Montreal

TEST: 56C

RESEARCHER: KATHERINE HIKITA **ASSISTANTS:** M. OUELLET, D. MORELLO, R. CARBONNEAU, JIANG FAN

DATE: Built: May 25; Tested: Wednesday June 1, 2005 **TIME:** 13:00

DIMENSIONS OF WALL: 4 FT X 8 FT **PANEL ORIENTATION:** Vertical, Sheathing one side

SHEATHING:
- [ ] Plywood 15/32" APA Rated Exposure 1 (USA)
- [ ] OSB 7/16" APA Rated Exposure 1 (USA)
- [X] Plywood (CSA 0151M) CSP 12.5mm (1/2") — CAN/PLY Exterior CSP
- [ ] Plywood (CSA O121M) DFP 12.5mm (1/2") — CAN/PLY Exterior DFP
- [ ] OSB (CSA O325) 11 mm (7/16")
- [ ] Other — MFR: ALBERTA PLYWOOD, MILL AB 244 - Weatherproof Bond

SCREWS Sheathing:
- [ ] No.8 gauge 1.5" self-piercing Bugle head LOX drive (Grabber Superdrive)
- [ ] No.8 gauge 1.0" self-piercing Bugle head (Flat socket head screw) (HD)
- [X] No. 8 gauge 1.25" self-drilling Bugle head LOX Drive (Grabber Superdrive)
- [ ] No.9 gauge 1.0" self-piercing Bugle head (screws Y2,Y12 in track)

Framing:
- [ ] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive

Hold downs:
- [X] No.10 gauge 0.75" self-drilling Hex washer head

Loading Beam: A325 3/4" bolts — 3 bolts [ ] 6 bolts [X] 12 bolts [ ]

Back-to-Back Chord Studs:
- [X] No.10 gauge 0.75" self-drilling Hex washer head (2@12" O.C.)

SHEATHING FASTENER SCHEDULE: [ ] 2"/12" [X] 3"/12" [ ] 4"/12" [ ] 6"/12" [ ] Other:

EDGE PANEL DISTANCE: [ ] 3/8" [X] 1/2" [ ] Other:

STUDS: Field / Chord
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.043" (1.09 mm) 33ksi (230 MI
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip : Thickness 0.054" (1.37mm) 50ksi (340 MF
- [X] Double chord studs used
- [ ] Other

STUD SPACING:
- [ ] 12" O.C.
- [ ] 16" O.C.
- [X] 24" O.C.
- [ ] Other:

TRACK: Web: 3-5/8" inches; Flange: 1-1/4" inches
- [X] 0.043" (1.09mm) 33ksi (230 MPa)
- [ ] Other:

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10 7/8" Anchor Rod (# of screws): 33
- [ ] UCI 18" hold down 1/2" Anchor Rod (# of screws):
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [ ] Monotonic — 7.5mm/min.
- [X] Cyclic — CUREE reversed cyclic (10 mm/s)
- [X] Gravity — 9 kN gravity load at both ends of wall

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral
- [X] East Frame Brace
- [X] West Frame Brace
- [X] Wood Shear

TOTAL: 9

MOISTURE CONTENT OF SHEATHING: Moisture Meter

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

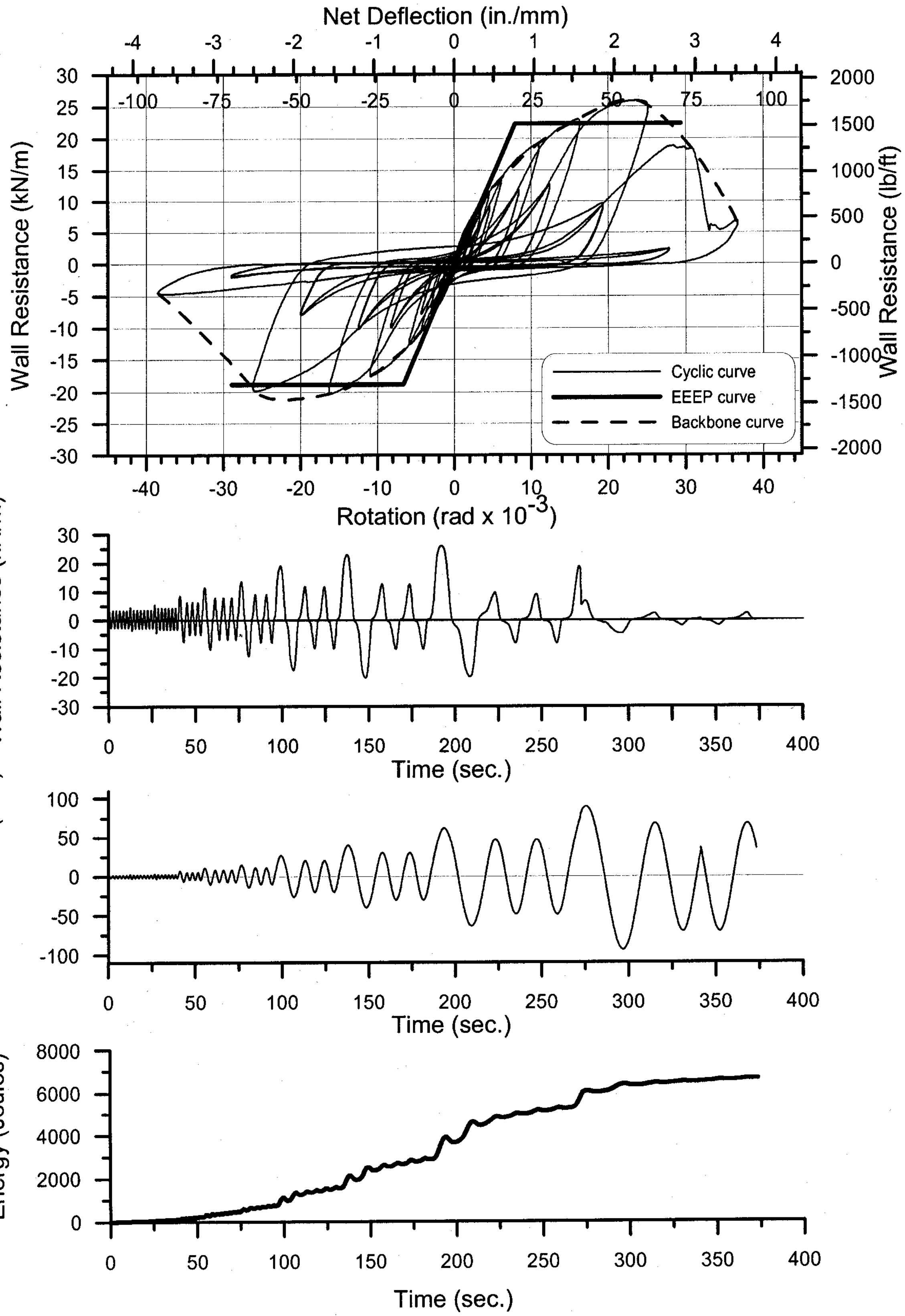
	North	North
Ww=	21.47	22.12
Wd=	20.29	20.93
m.c.=	5.82	5.69

AVG m.c. 5.75

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 50 scan/sec

COMMENTS:
- -Shear anchors torqued to 77 kN
- -Hold down anchors tightened to approximately 8 kN (load cells used on both hold-downs)
- -Ambient temperature 23 C
- -Double chord studs used
- -Square plate washers (2.5"x2.5") used in all track connections
- -Pilot holes drilled for screws A1,A5,A9,Q1,Q5,Q9
- -Initial load set to zero at beginning of test, displacement 20.789 mm (includes gravity load force)

Test 56C
(1220 x 2440 mm CSP 75/305 mm)



	Negative	Positive	Units
F_u	-24.65	31.52	kN
$F_{0.8u}$	-19.72	25.22	kN
$F_{0.4u}$	-9.86	12.61	kN
F_y	-22.95	27.18	kN
K_e	1.43	1.40	kN/mm
Ductility (μ)	4.39	3.69	-
$\Delta_{net,y}$	-16.06	19.40	mm
$\Delta_{net,u}$	-39.13	57.60	mm
$\Delta_{net,0.8u}$	-70.50	71.60	mm
$\Delta_{net,0.4u}$	-6.90	9.00	mm
$Area_{Backbone}$	1433.47	1682.48	kN-mm
$Area_{EEEP}$	1433.47	1682.48	kN-mm
Check	OK	OK	
R_d	**2.79**	**2.53**	-
S_y	**-18.82**	**22.29**	kN/m

Test Name	56C
Date Tested	June 1, 2005
Protocol	CUREE ORDINARY GROUND MOTIONS

McGill

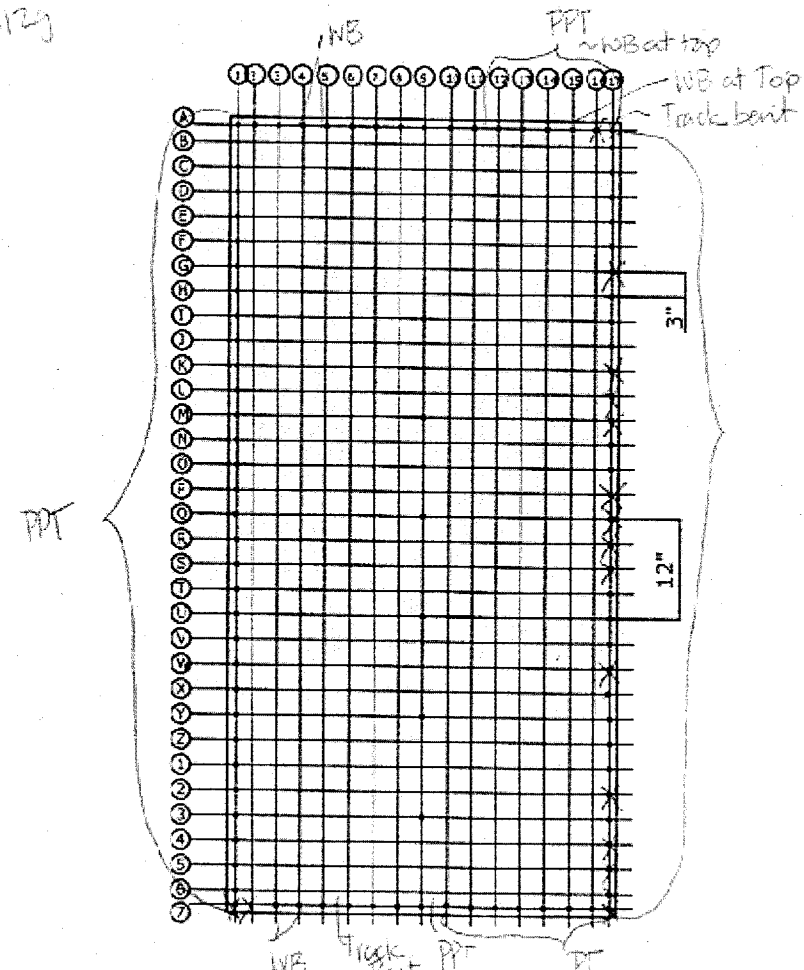
Light Gauge Steel Frame / Wood Panel Shear Wall Testing

Moisture Content: A: 21.97g
B: 22.12g

Test name : 5f0 C
Date tested : June 1 2005
Wall Size : 4' x 8
Screw pattern : 3" / 12"
Edge Distance : 1/2"
Test mode : ☒ Cyclic ☐ Monotonic

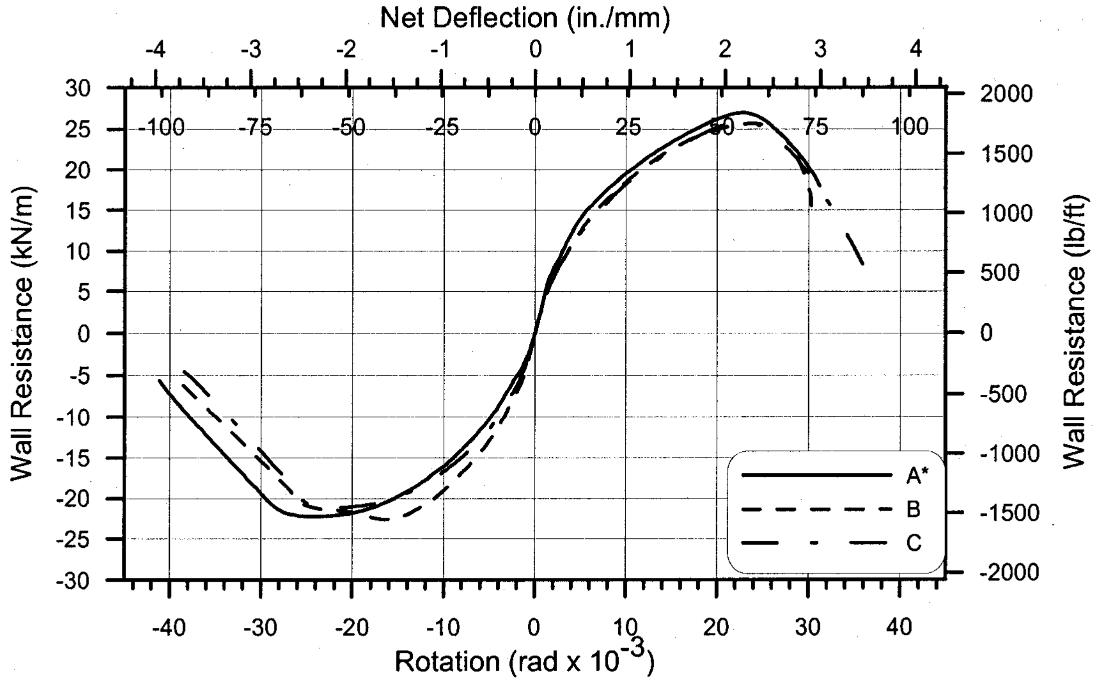
Max. Displacement 105.92 mm
Max. load 31.61 kN

X - Sheared screw.

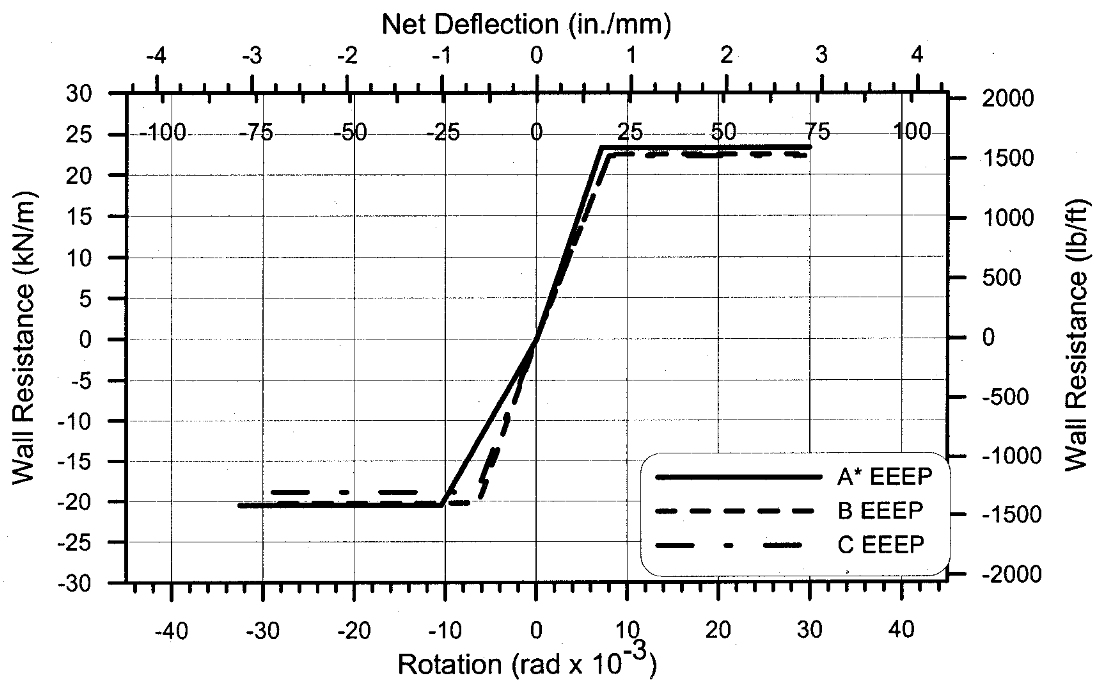


Failure modes: Pullout, withdrawal (PO) ; Fatigue Fracture, Shear (FF) ; Pull through sheathing (PT) ; Damage prior to testing (DP)
Partial Pullthrough (PPT) ; Tearout of sheathing (TO) ; Wood Bearing Failure (WB)

Test 56-A,B,C Backbone
(1220 x 2440 mm CSP 75/305 mm)



Test 56-A,B,C EEEP
(1220 x 2440 mm CSP 75/305 mm)



APPENDIX D

STEWART MODEL

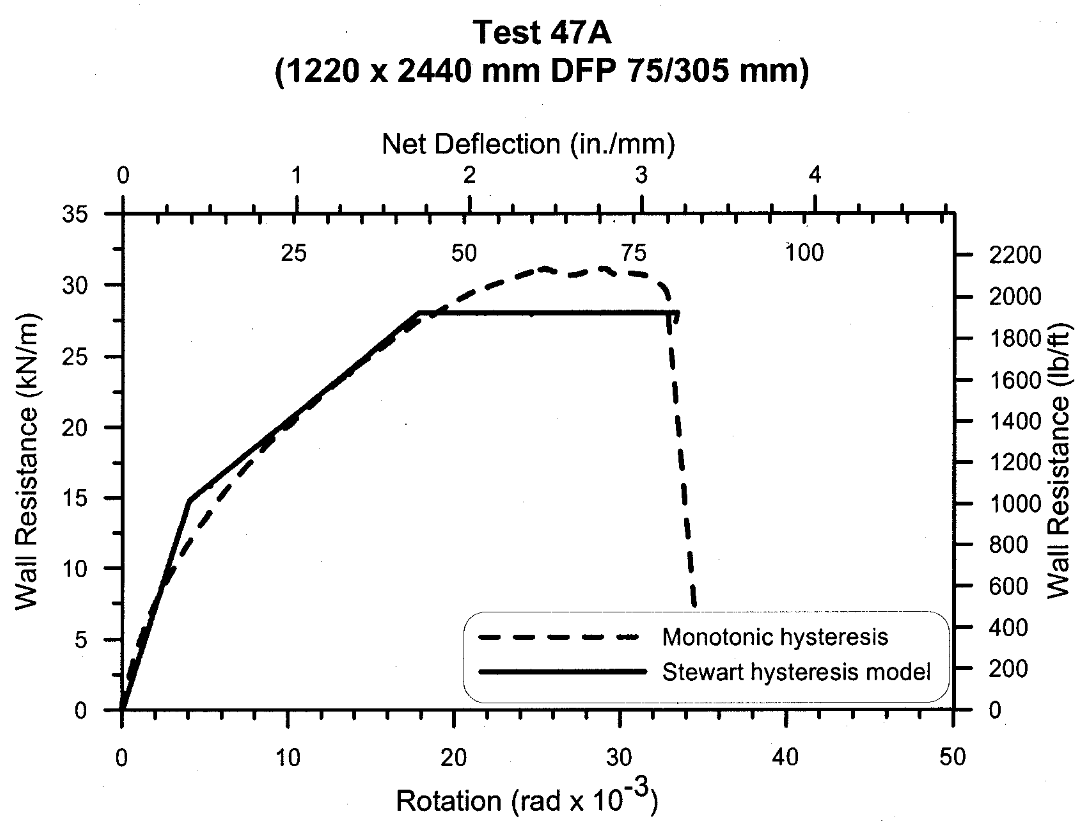


Figure D-1 : Superimposed Experimental and Modeled Hystereses Test 47A

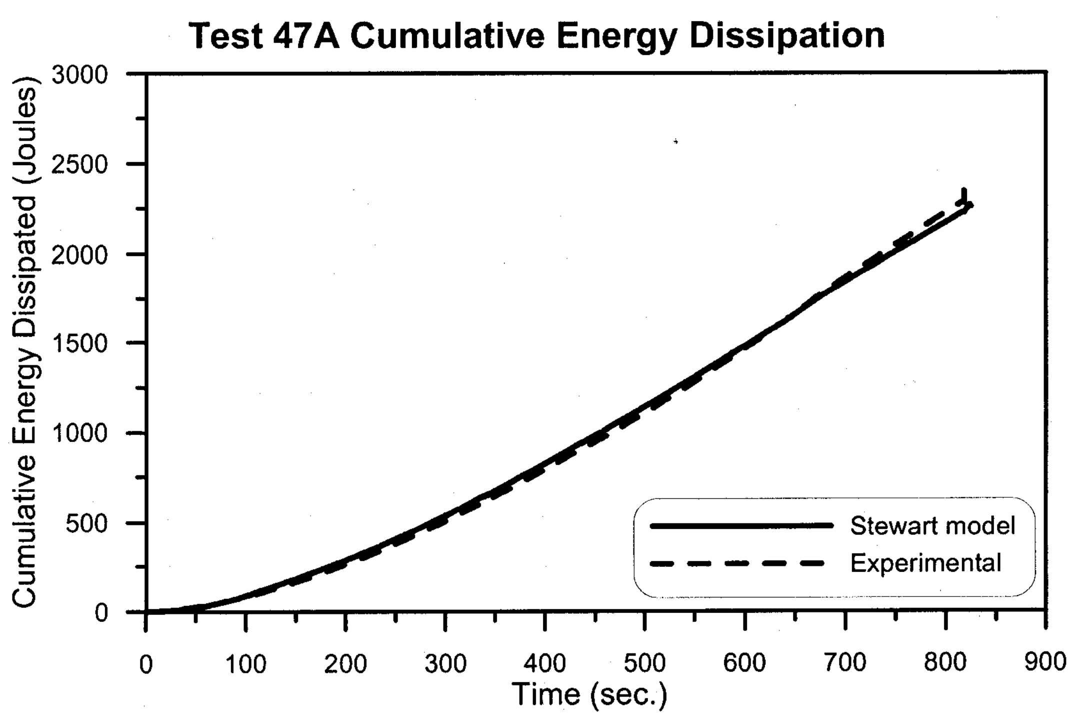


Figure D-2 : Cumulative Energy Dissipated from Experimental Test and Stewart Model Hystereses

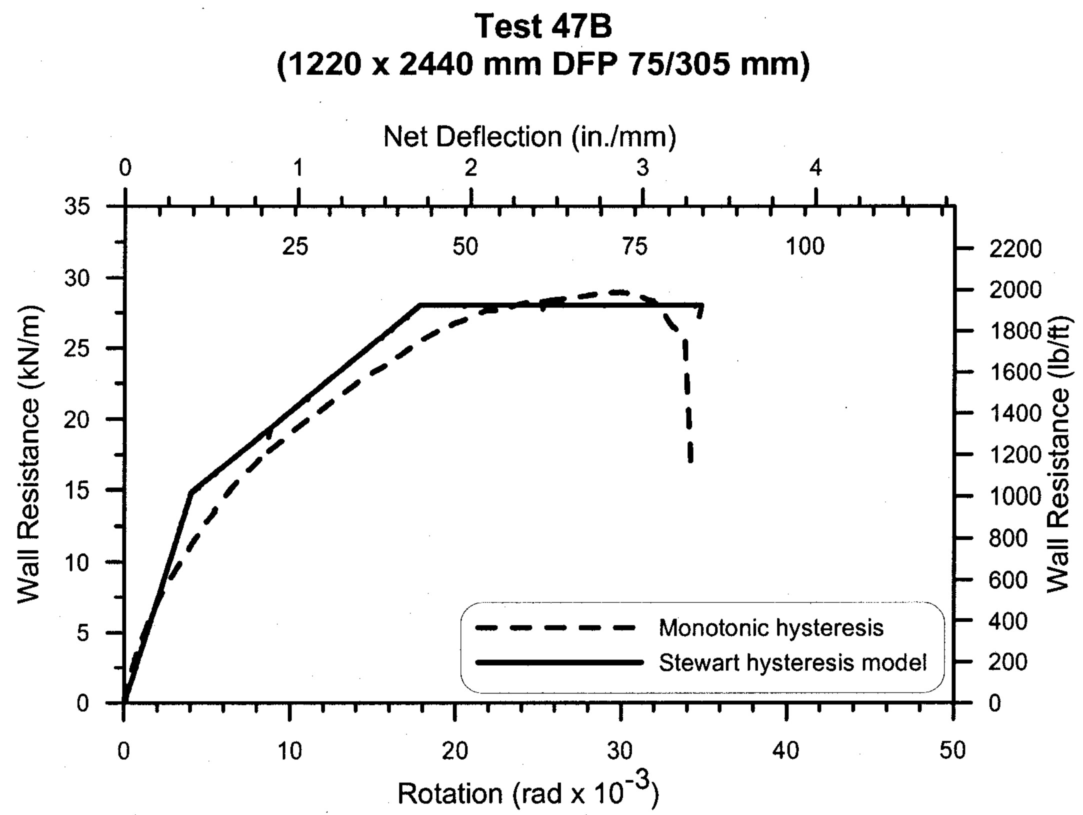


Figure D-3 : Superimposed Experimental and Modeled Hystereses Test 47B

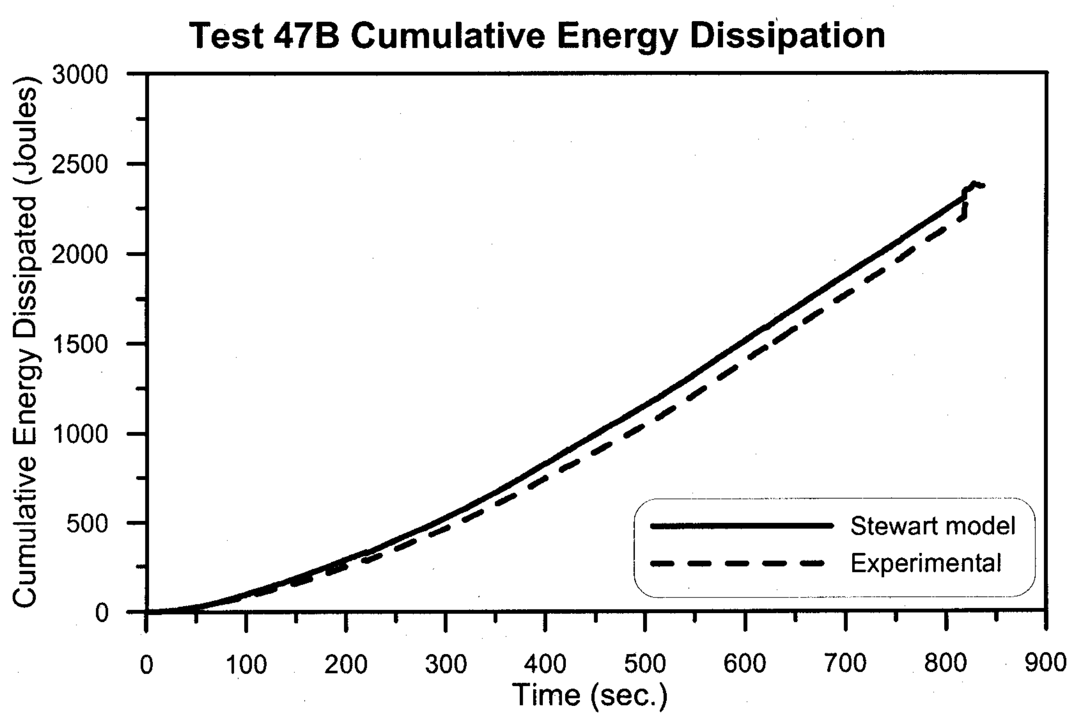


Figure D-4 : Cumulative Energy Dissipated from Experimental Test and Stewart Model Hystereses

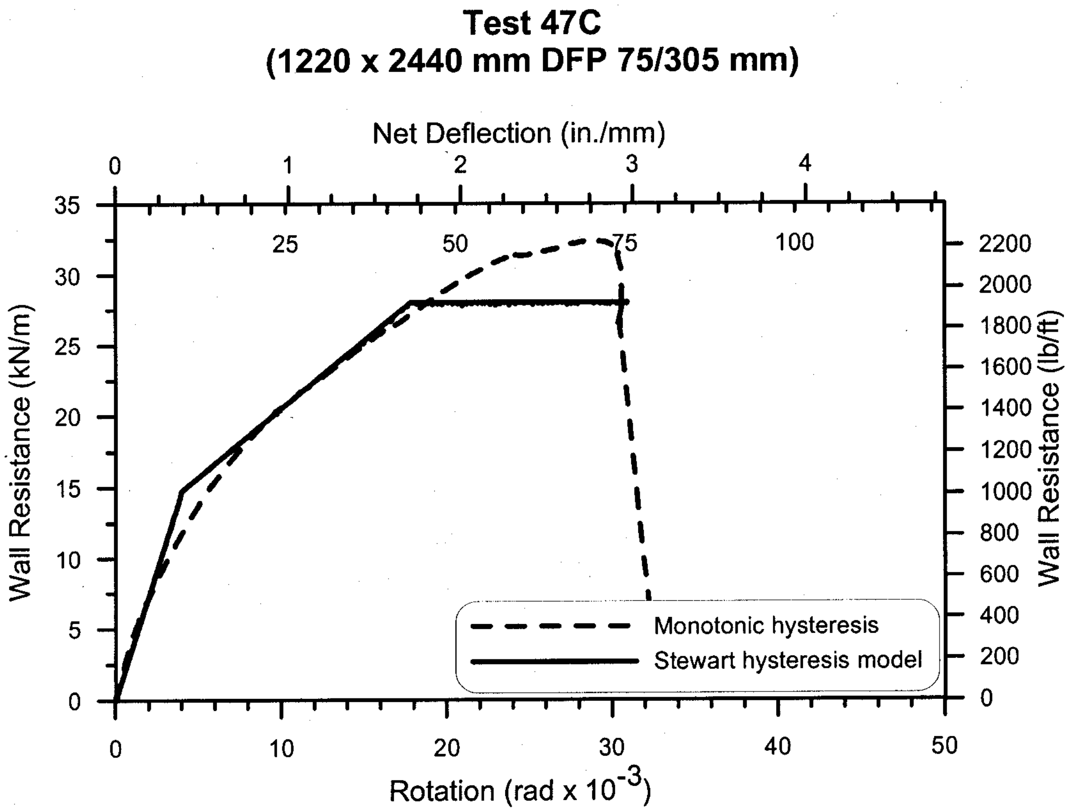


Figure D-5 : Superimposed Experimental and Modeled Hystereses Test 47C

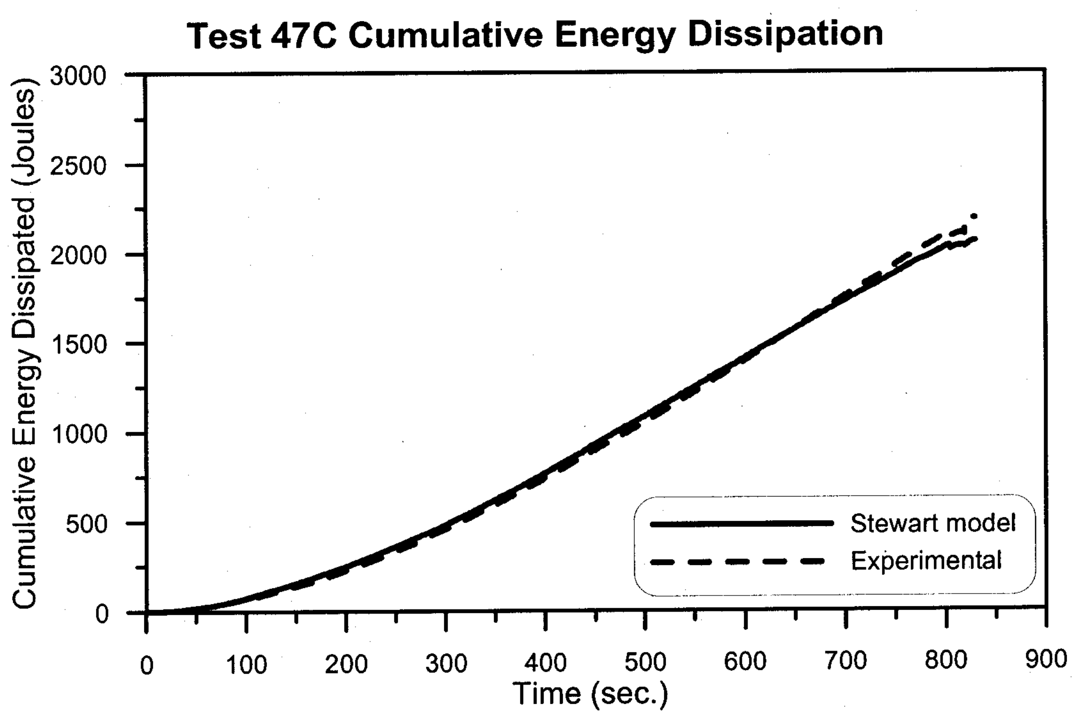


Figure D-6 : Cumulative Energy Dissipated from Experimental Test and Stewart Model Hystereses

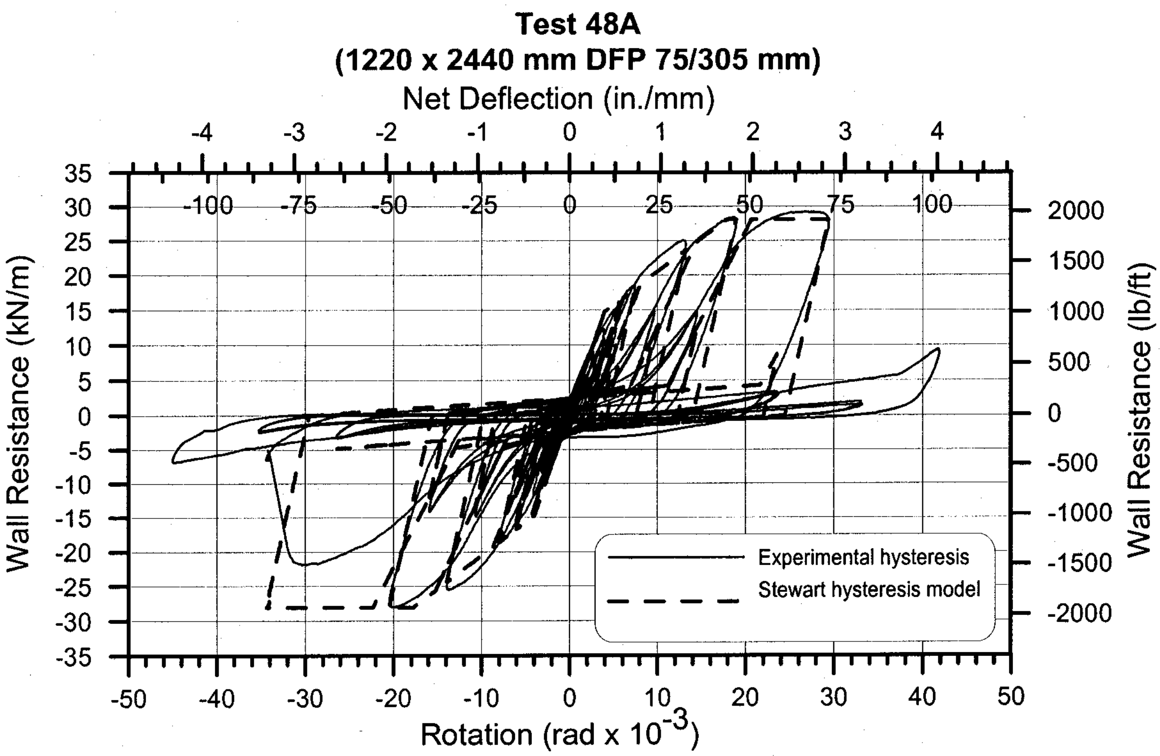


Figure D-7 : Superimposed Experimental and Modeled Hystereses Test 48A

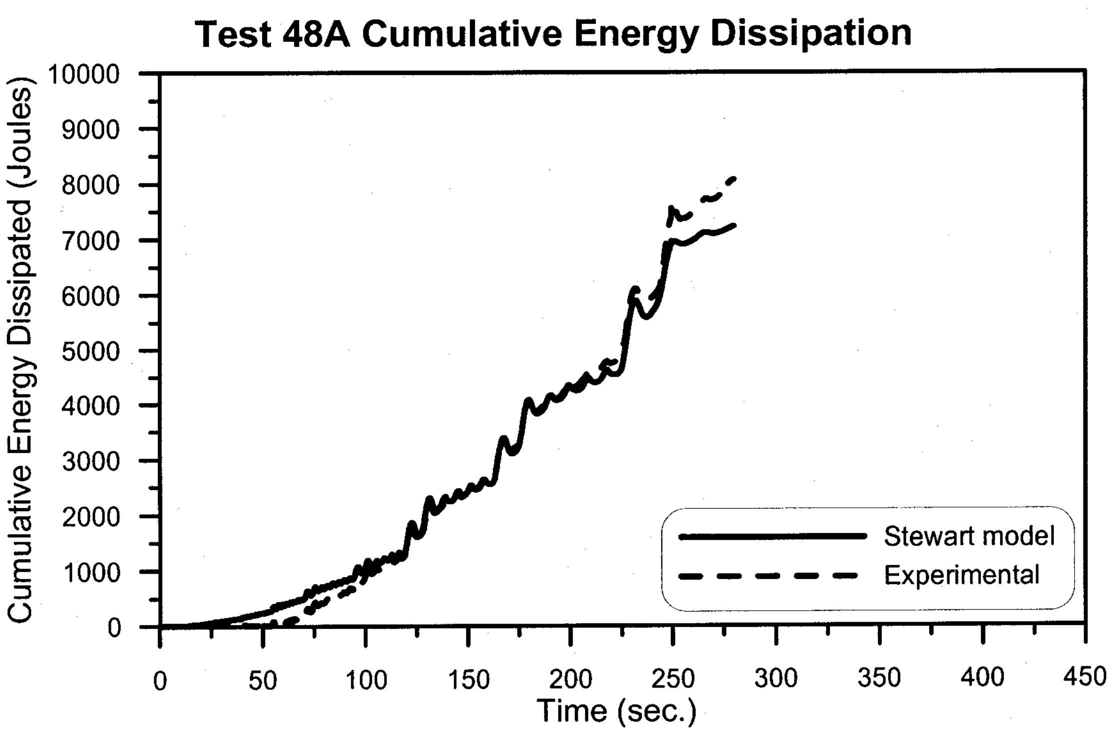


Figure D-8 : Cumulative Energy Dissipated from Experimental Test and Stewart Model Hystereses

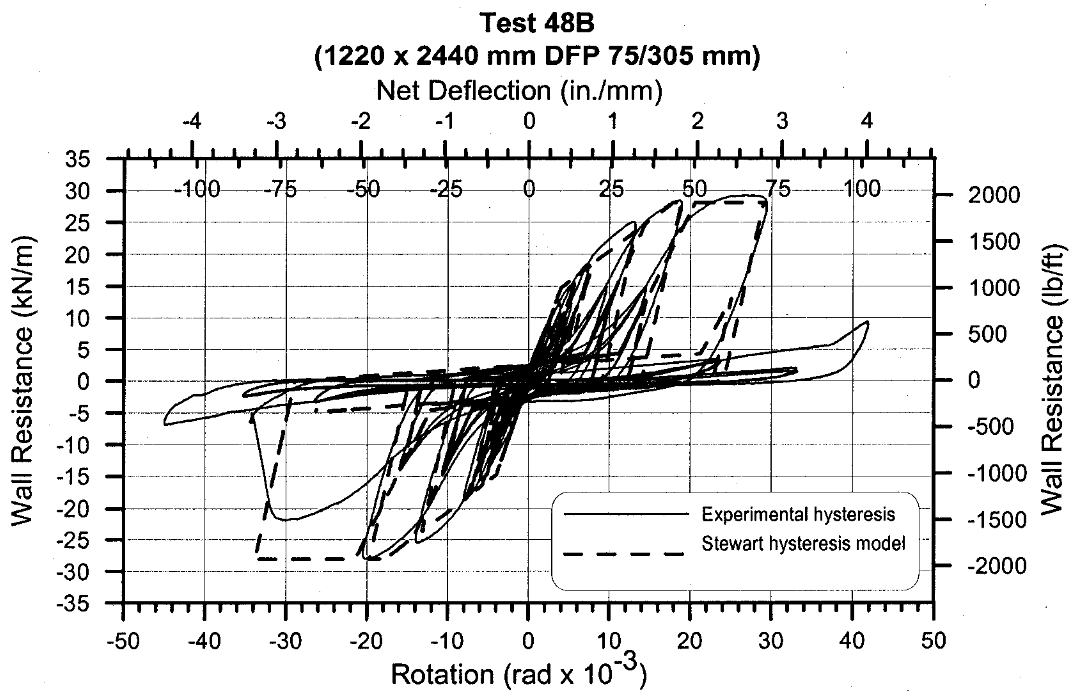


Figure D-9 : Superimposed Experimental and Modeled Hystereses Test 48B

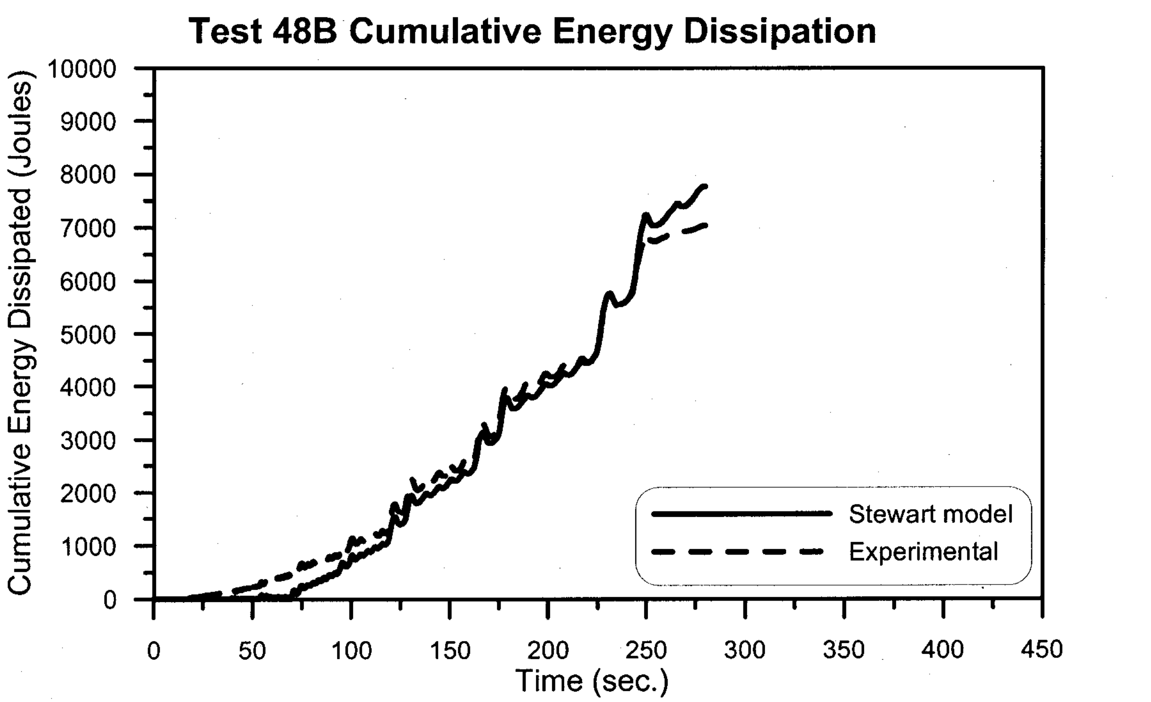


Figure D-10 : Cumulative Energy Dissipated from Experimental Test and Stewart Model Hystereses

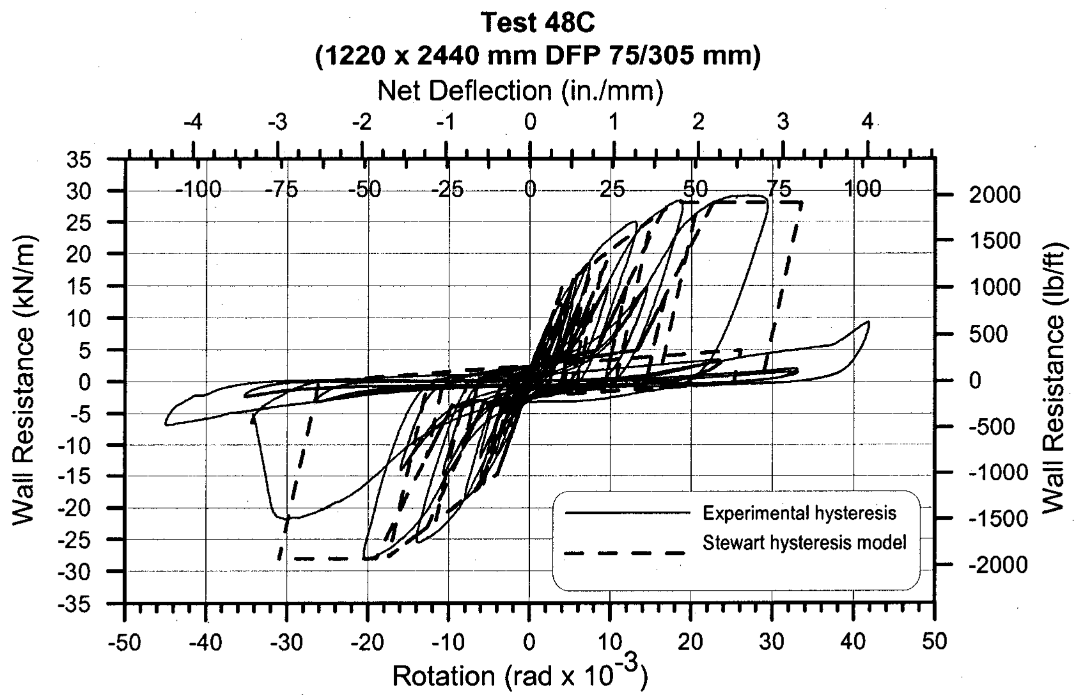


Figure D-11 : Superimposed Experimental and Modeled Hystereses Test 48C

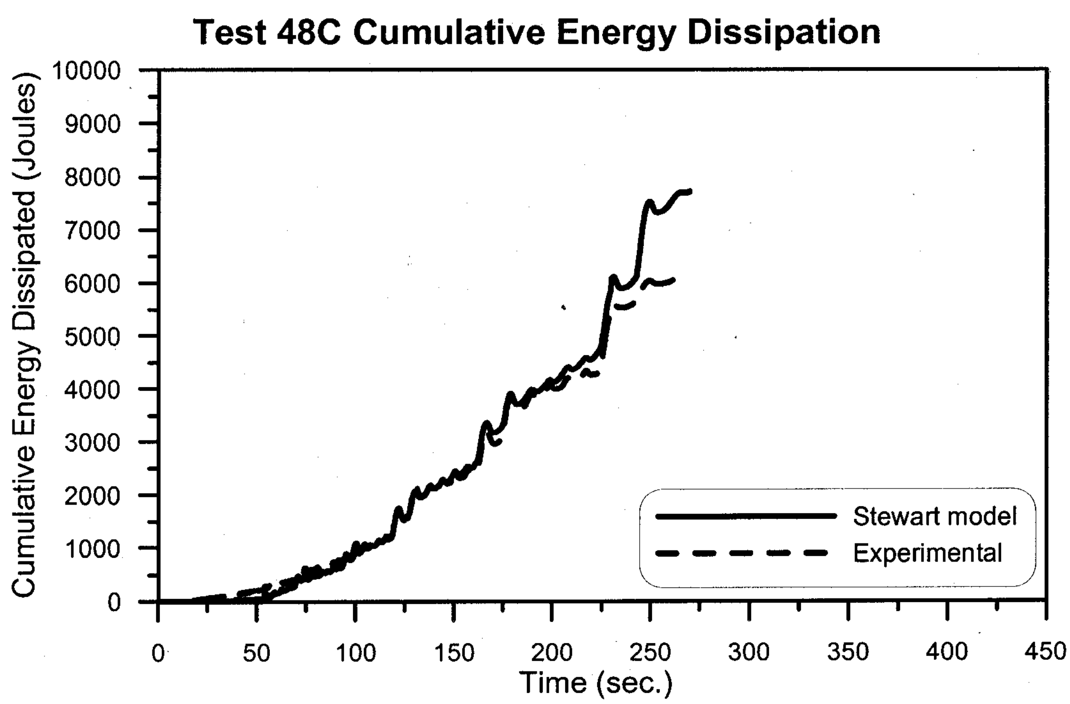


Figure D-12 : Cumulative Energy Dissipated from Experimental Test and Stewart Model Hystereses

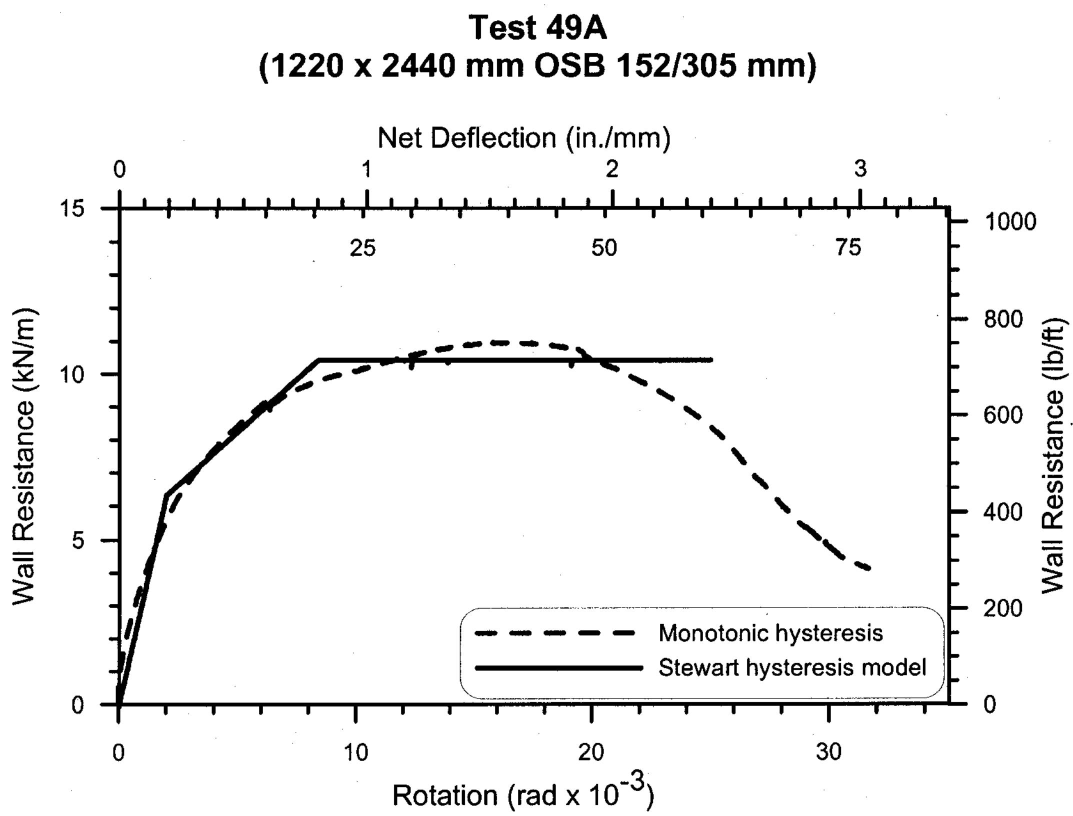


Figure D-13 : Superimposed Experimental and Modeled Hystereses Test 49A

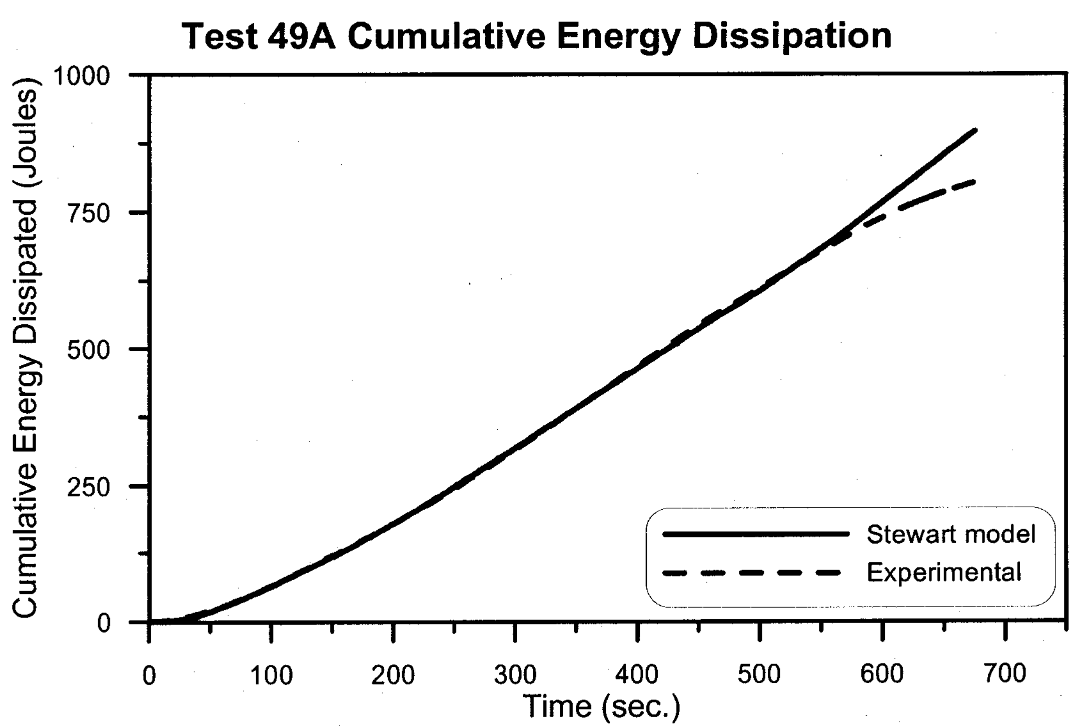


Figure D-14 : Cumulative Energy Dissipated from Experimental Test and Stewart Model Hystereses

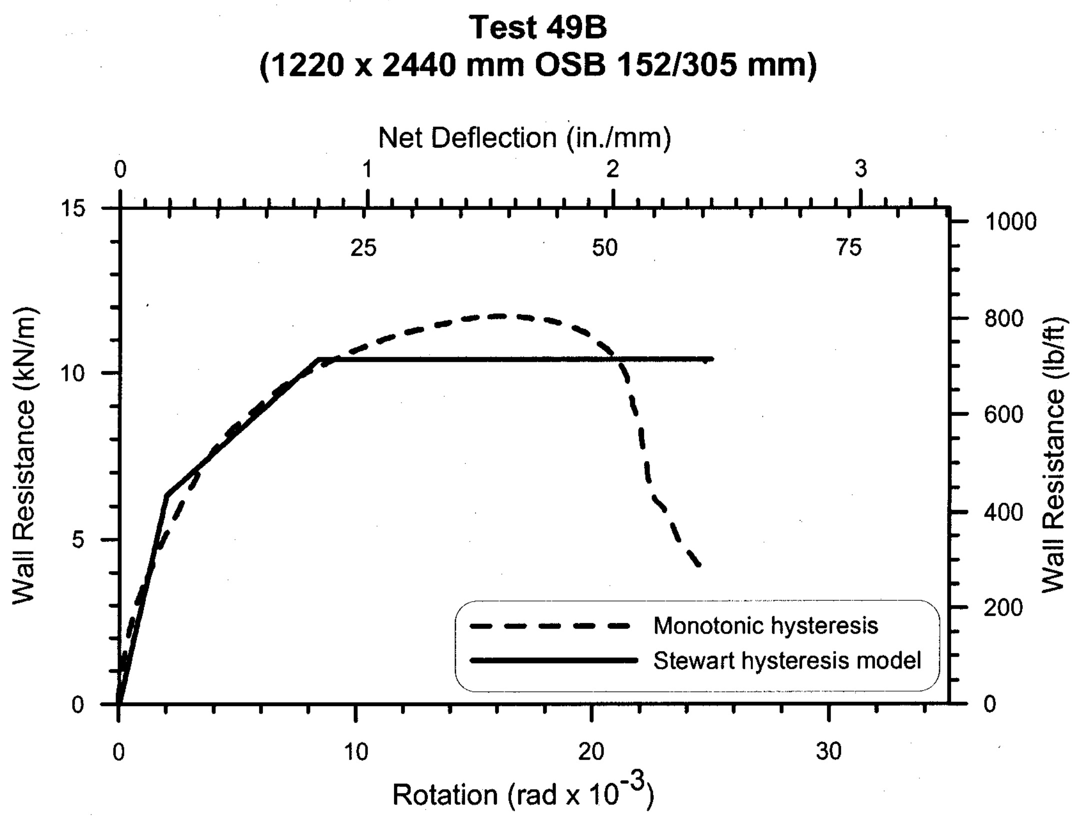


Figure D-15 : Superimposed Experimental and Modeled Hystereses Test 49B

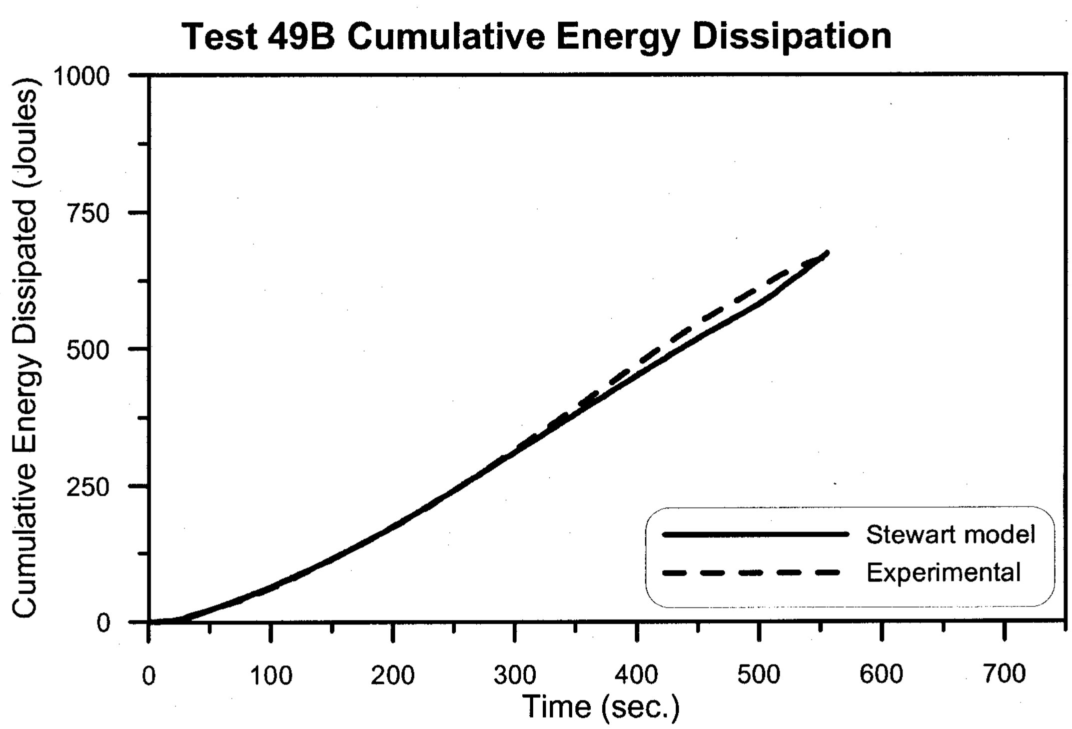


Figure D-16 : Cumulative Energy Dissipated from Experimental Test and Stewart Model Hystereses

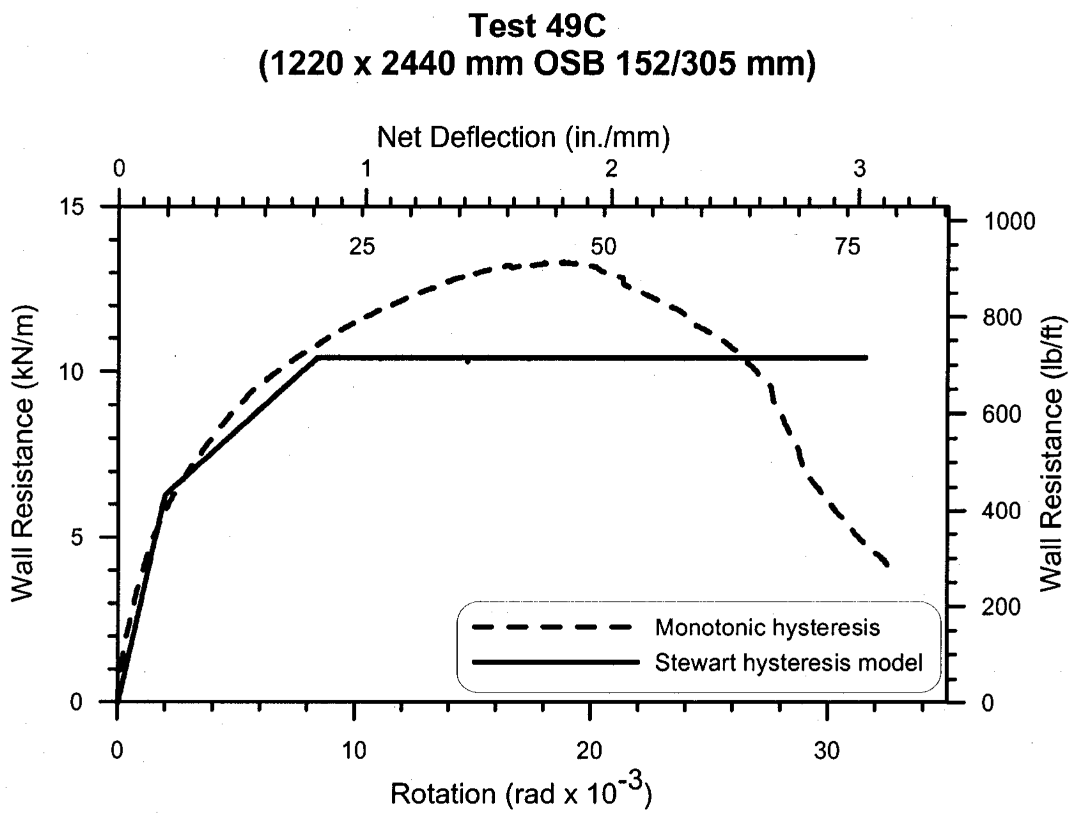


Figure D-17 : Superimposed Experimental and Modeled Hystereses Test 49C

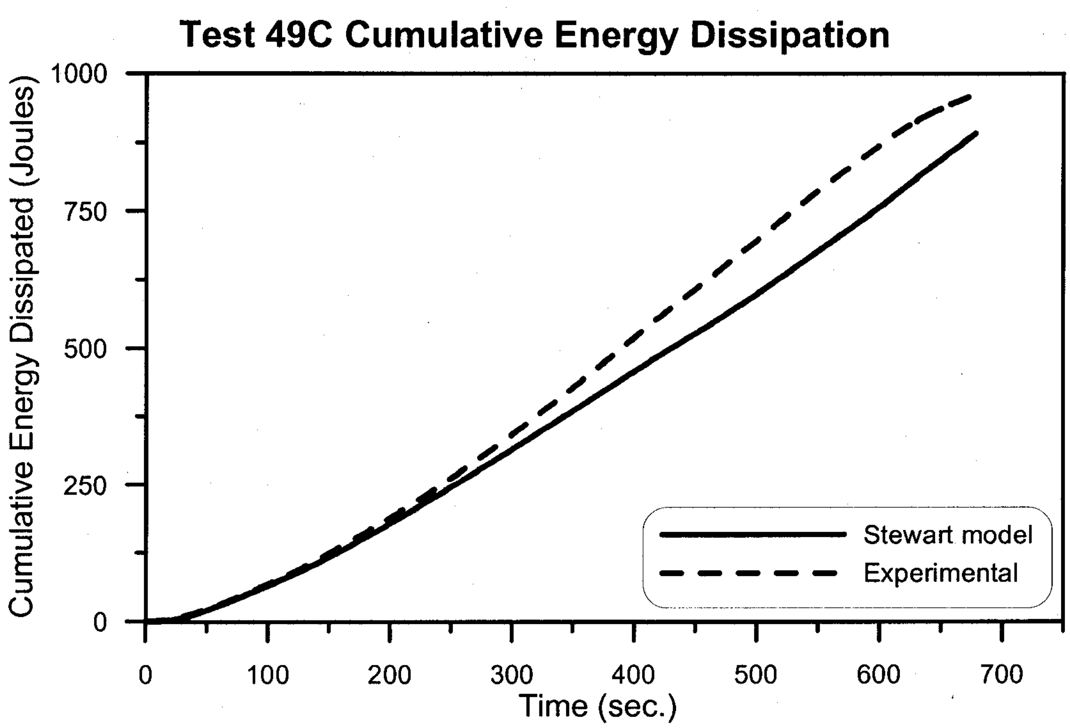


Figure D-18 : Cumulative Energy Dissipated from Experimental Test and Stewart Model Hystereses

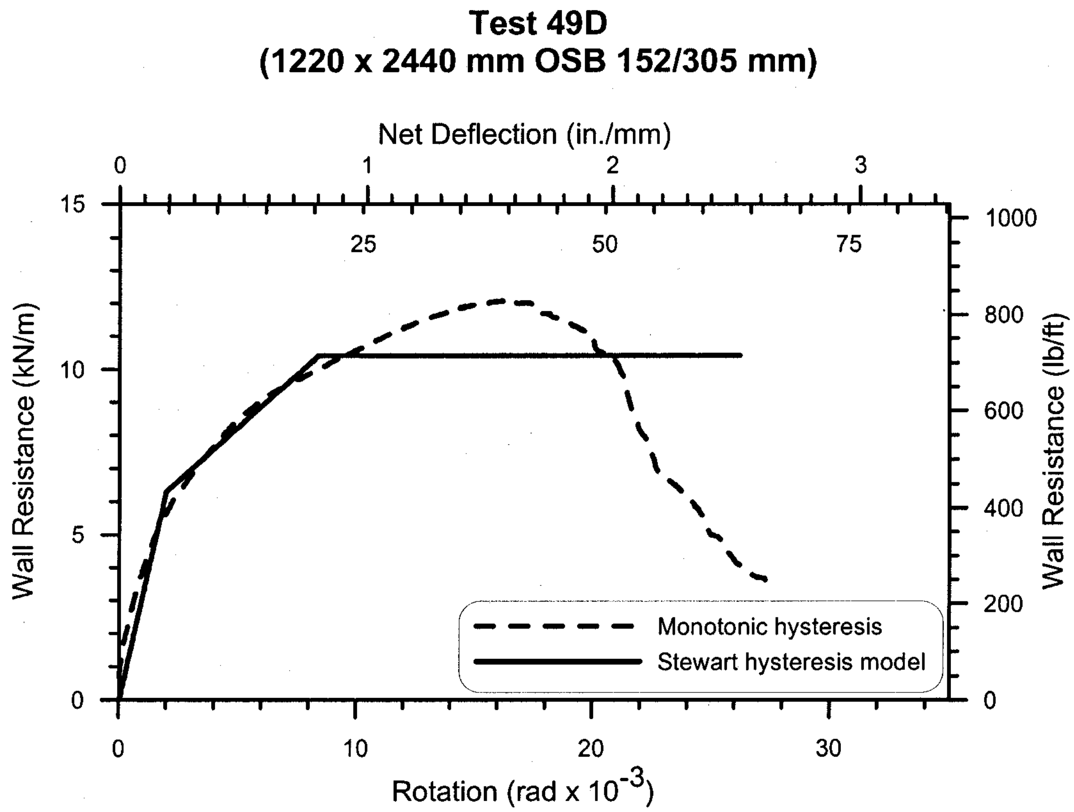


Figure D-19 : Superimposed Experimental and Modeled Hystereses Test 49D

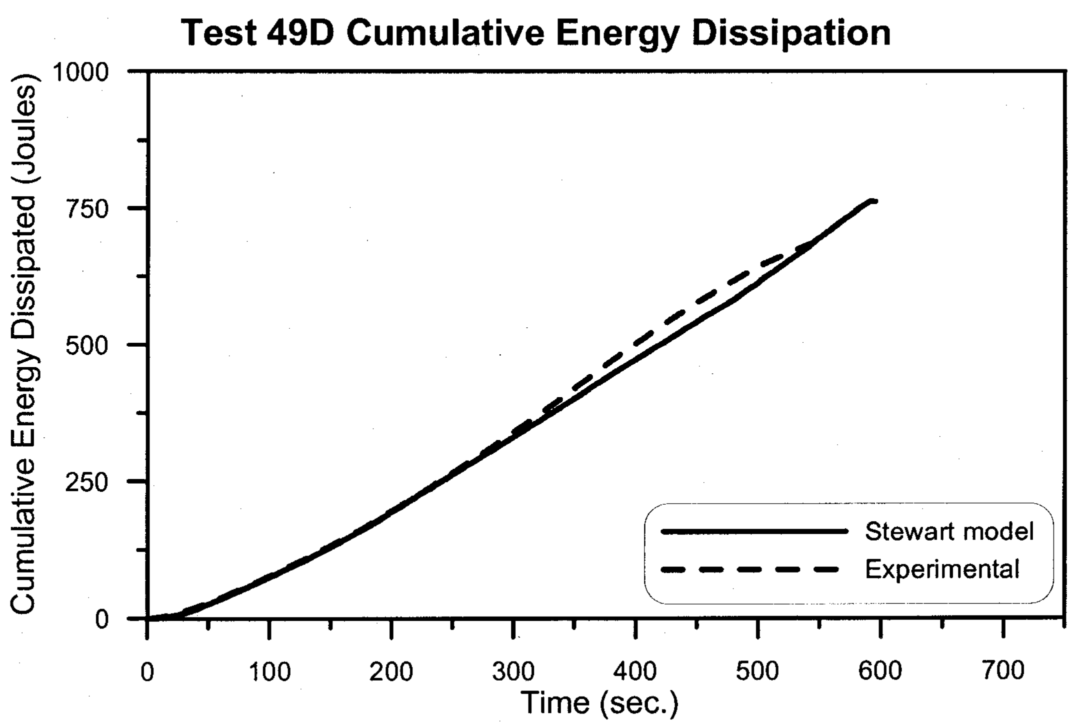


Figure D-20 : Cumulative Energy Dissipated from Experimental Test and Stewart Model Hystereses

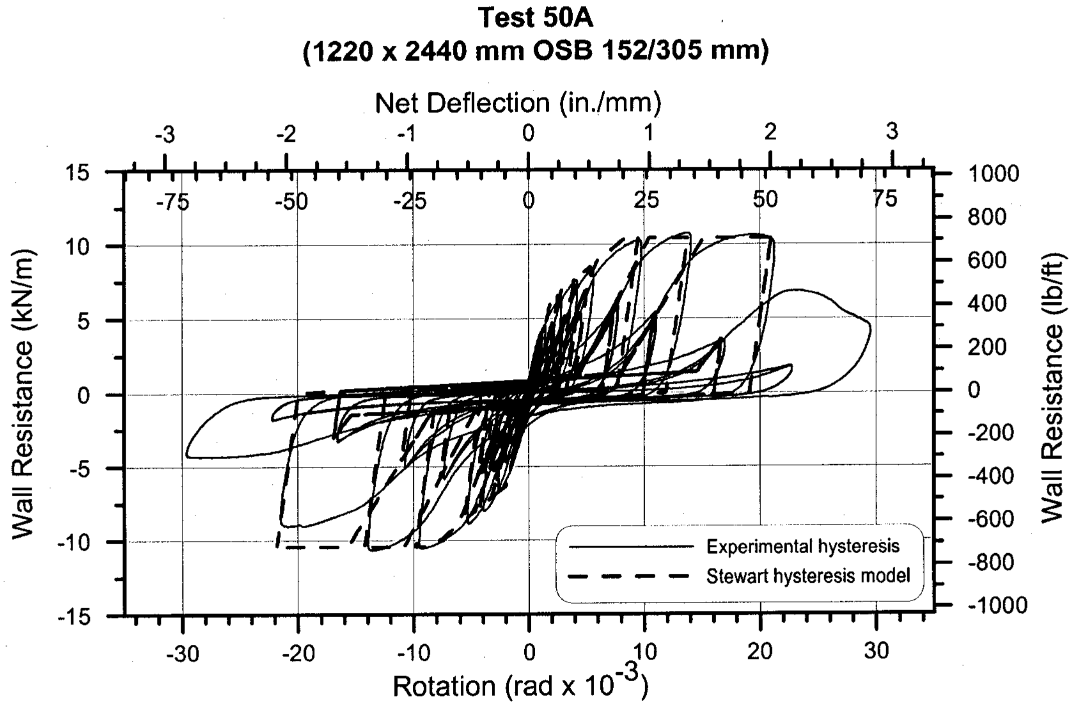


Figure D-21 : Superimposed Experimental and Modeled Hystereses Test 50A

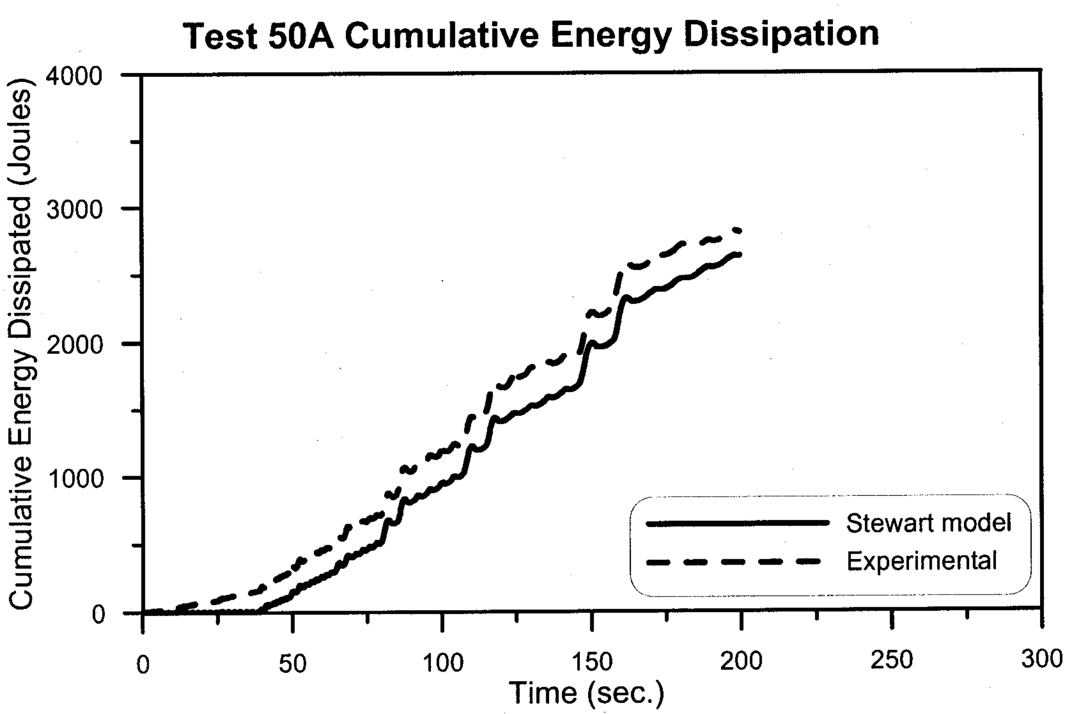


Figure D-22 : Cumulative Energy Dissipated from Experimental Test and Stewart Model Hystereses

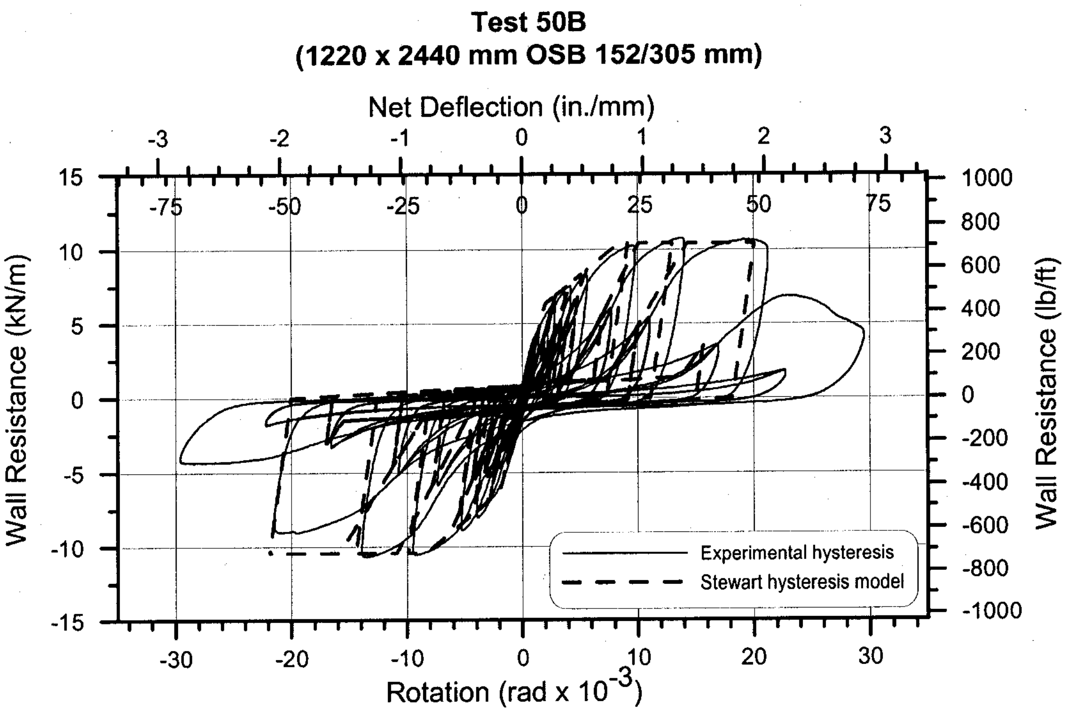


Figure D-23 : Superimposed Experimental and Modeled Hystereses Test 50B

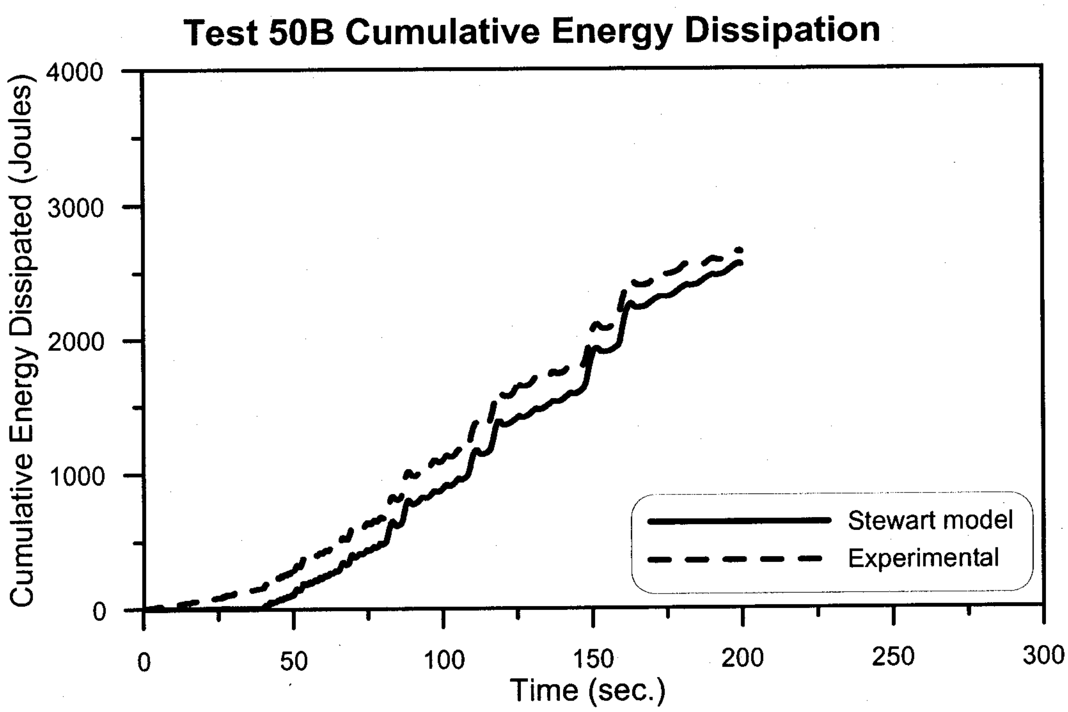


Figure D-24 : Cumulative Energy Dissipated from Experimental Test and Stewart Model Hystereses

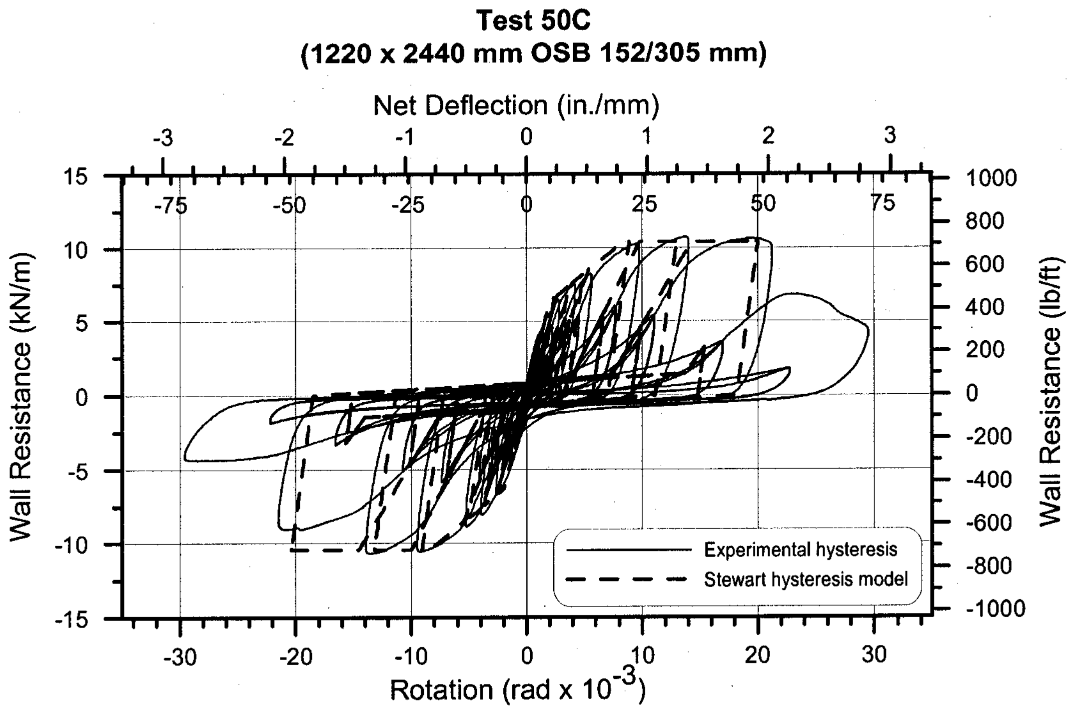


Figure D-25 : Superimposed Experimental and Modeled Hystereses Test 50C

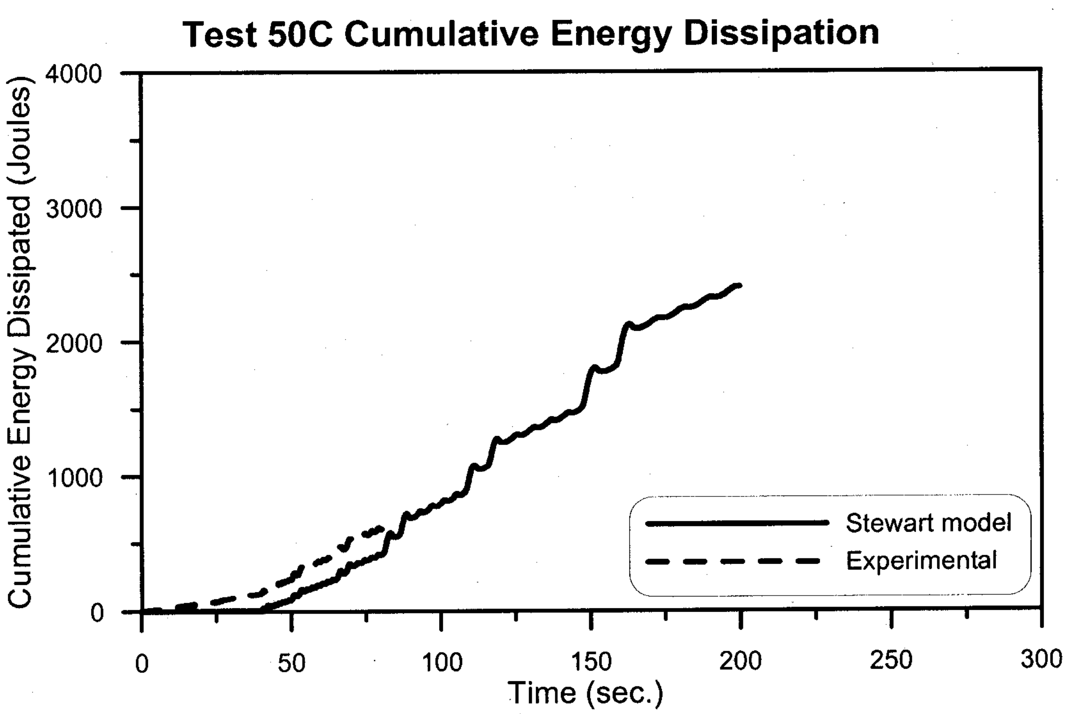


Figure D-26 : Cumulative Energy Dissipated from Experimental Test and Stewart Model Hystereses

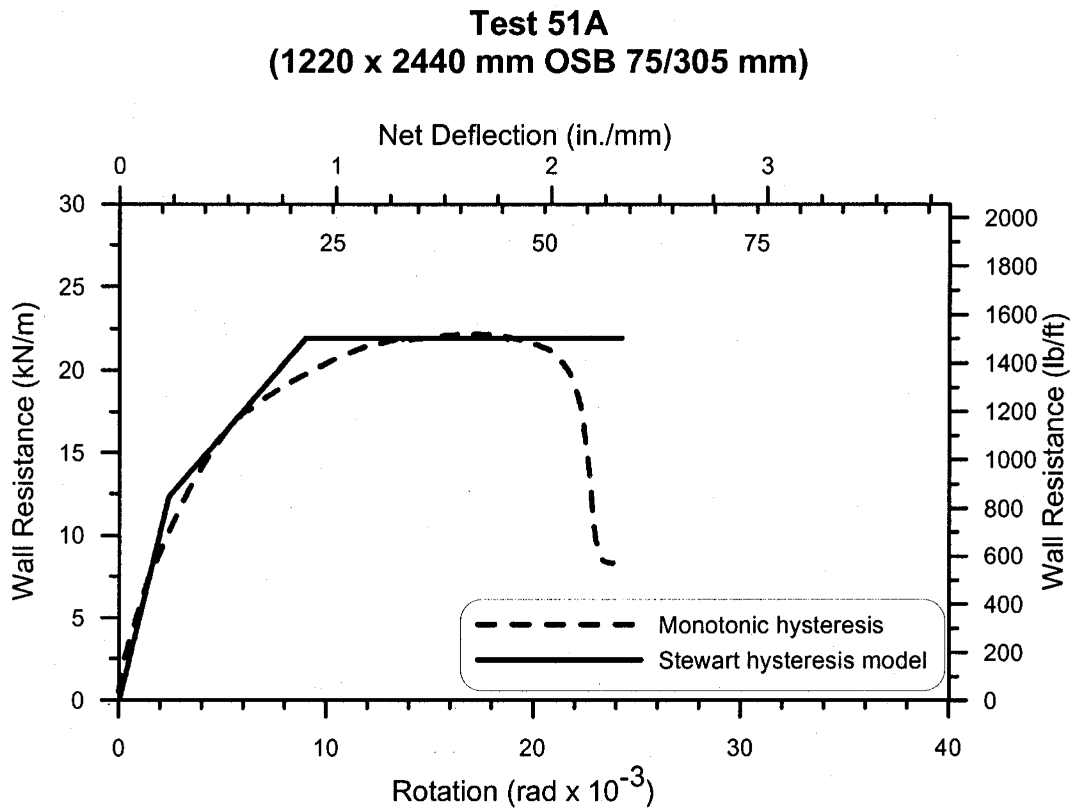


Figure D-27 : Superimposed Experimental and Modeled Hystereses Test 51A

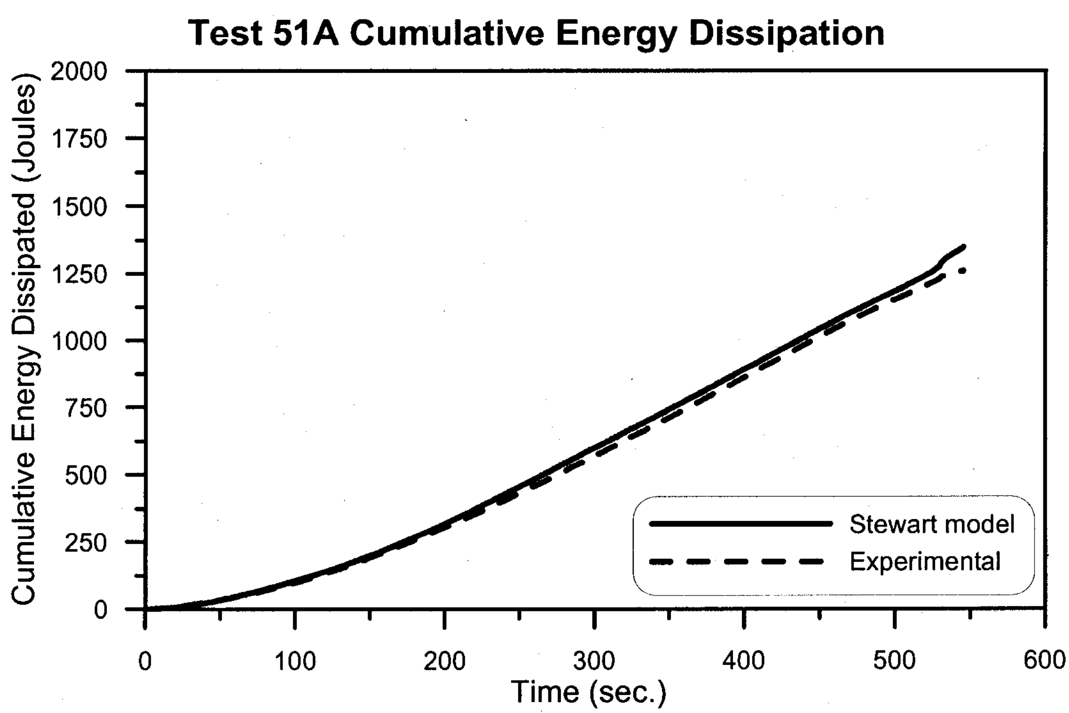


Figure D-28 : Cumulative Energy Dissipated from Experimental Test and Stewart Model Hystereses

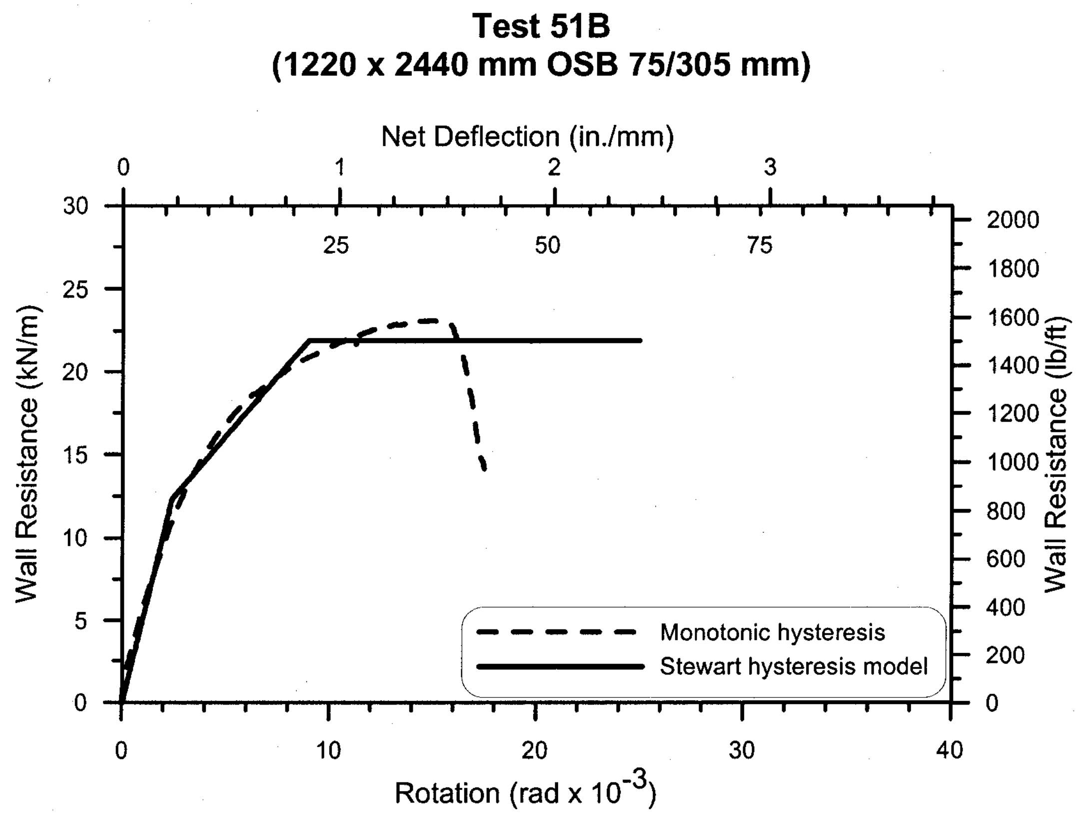


Figure D-29 : Superimposed Experimental and Modeled Hystereses Test 51B

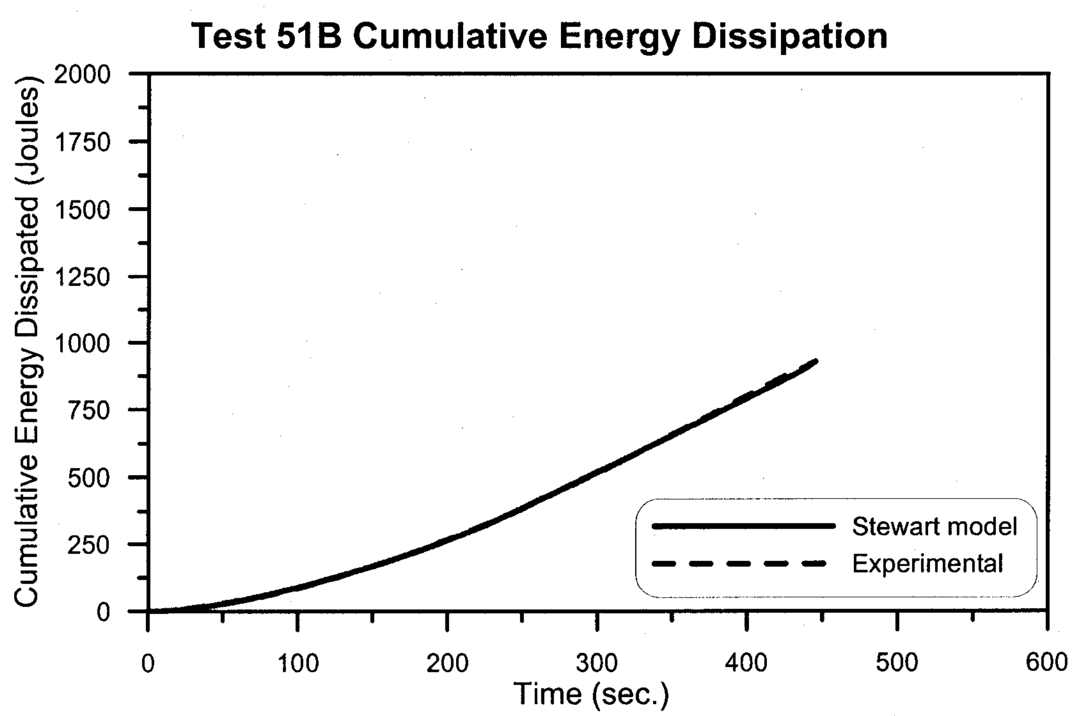


Figure D-30 : Cumulative Energy Dissipated from Experimental Test and Stewart Model Hystereses

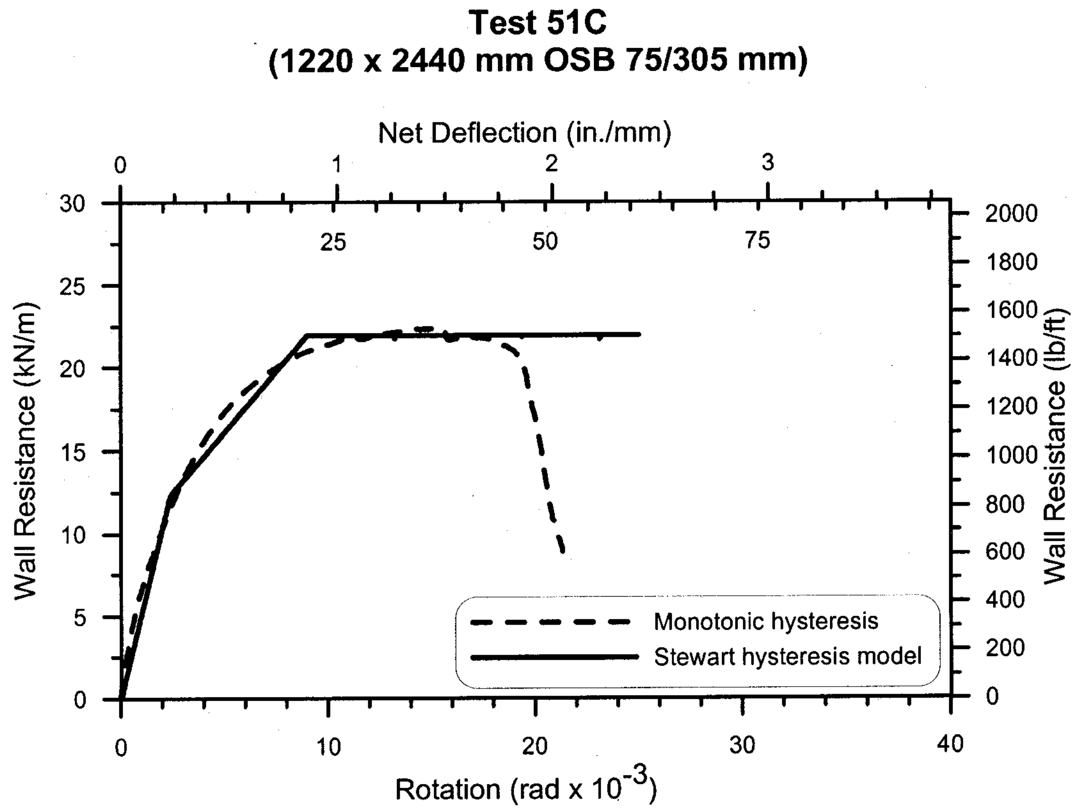


Figure D-31 : Superimposed Experimental and Modeled Hystereses Test 51C

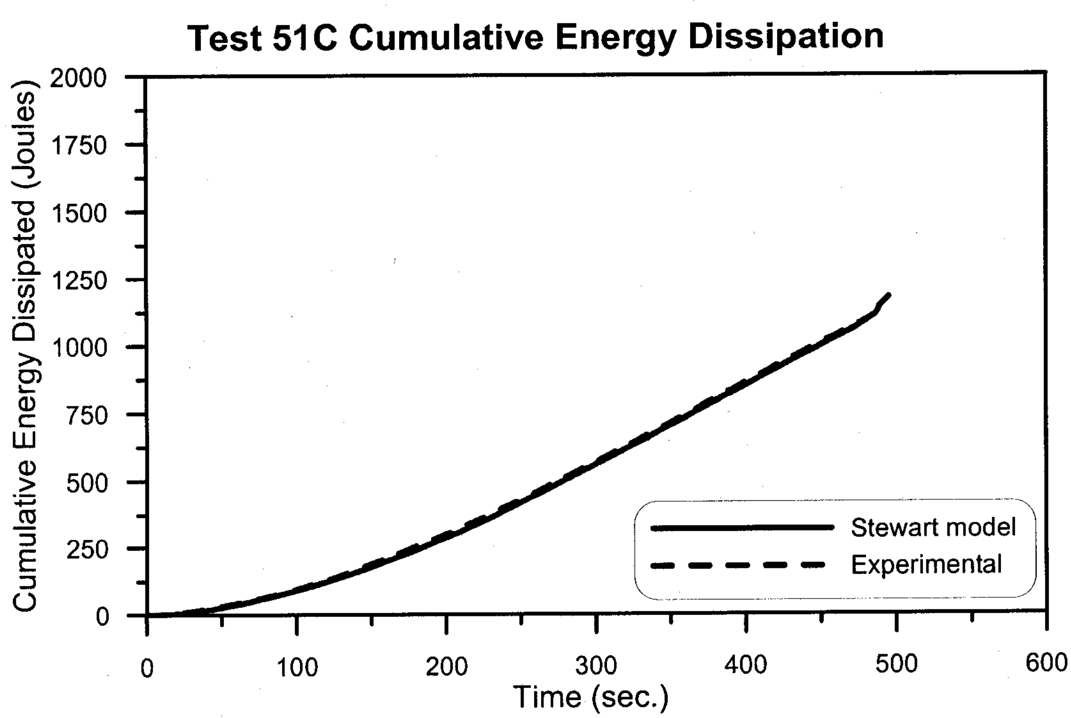


Figure D-32 : Cumulative Energy Dissipated from Experimental Test and Stewart Model Hystereses

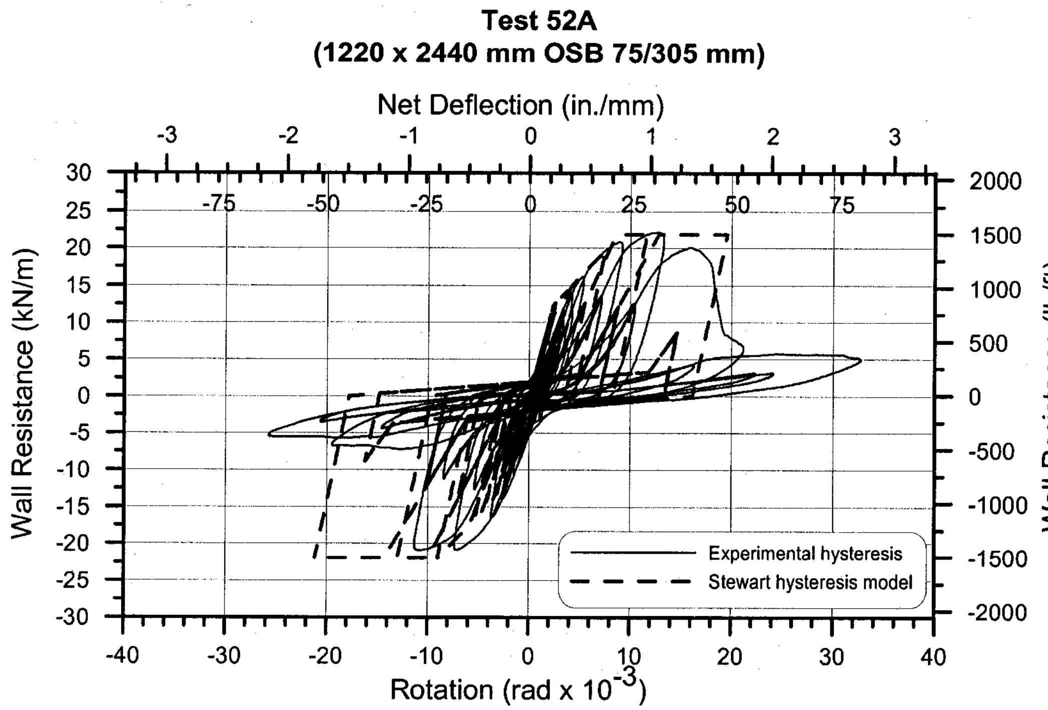


Figure D-33 : Superimposed Experimental and Modeled Hystereses Test 52A

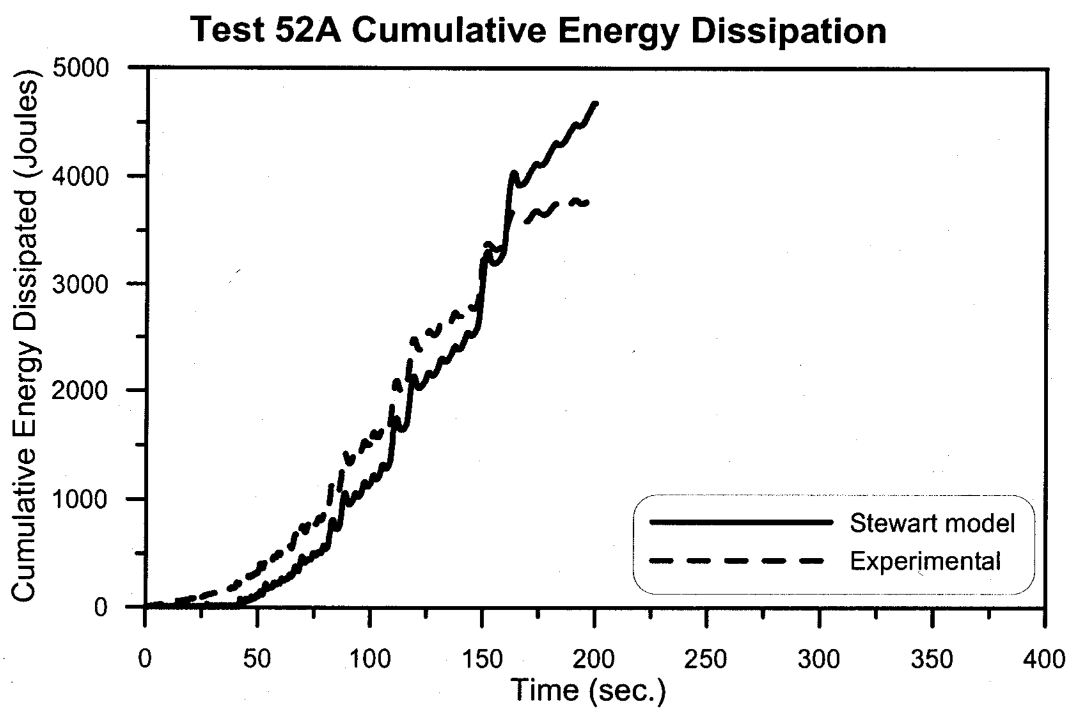


Figure D-34 : Cumulative Energy Dissipated from Experimental Test and Stewart Model Hystereses

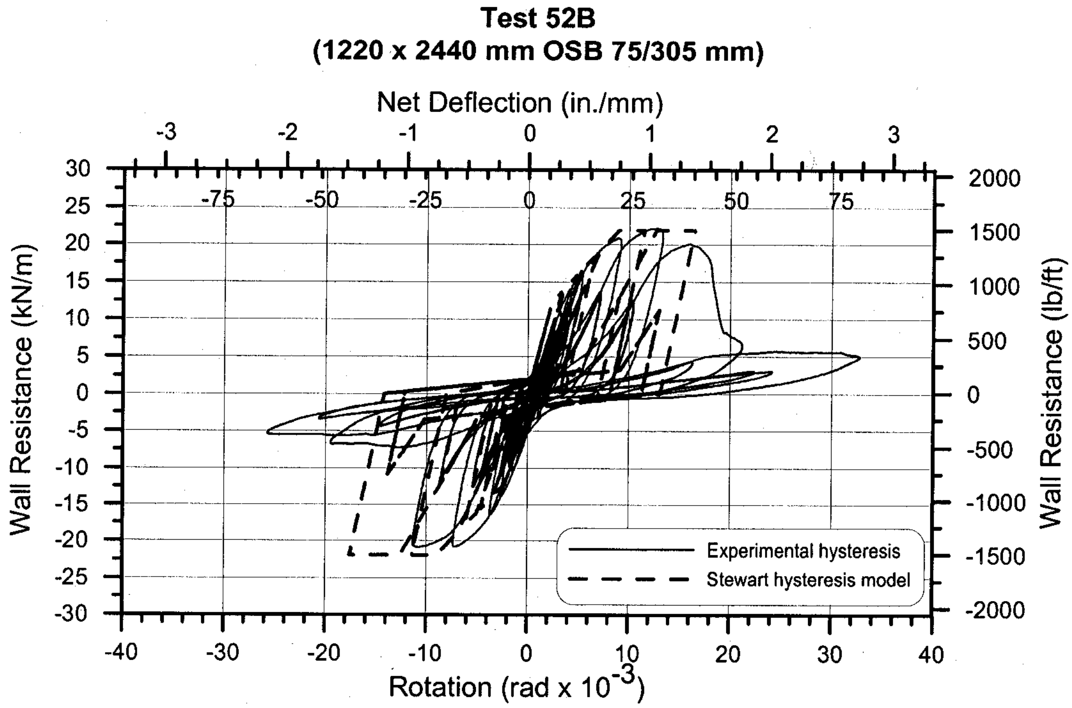


Figure D-35 : Superimposed Experimental and Modeled Hystereses Test 52B

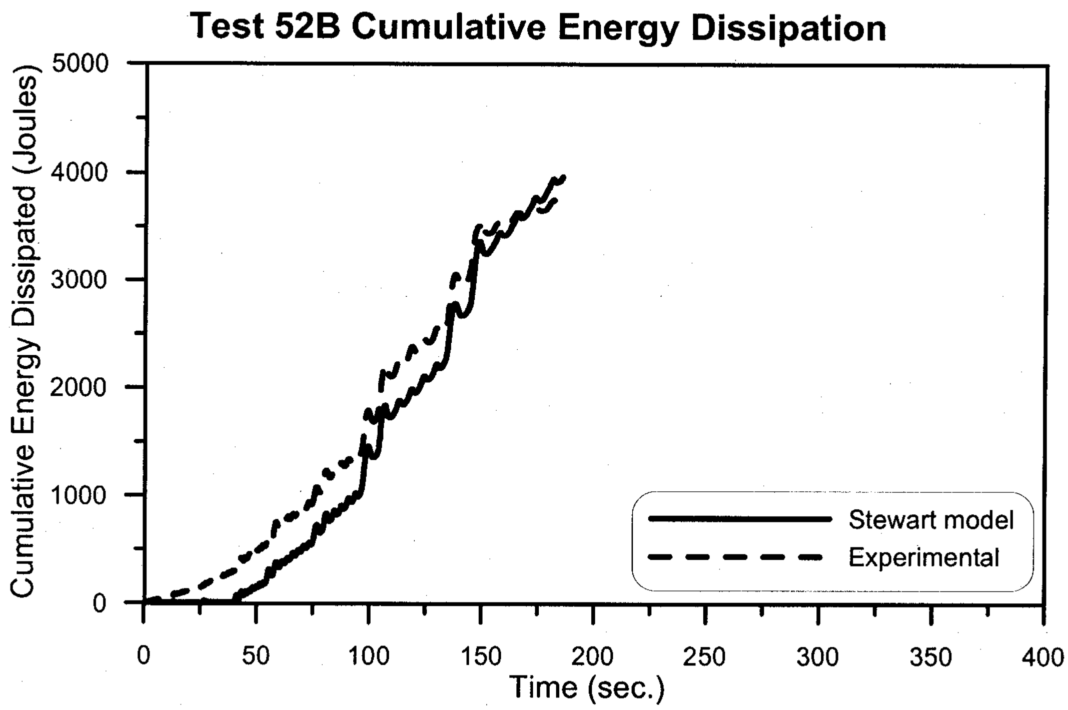


Figure D-36 : Cumulative Energy Dissipated from Experimental Test and Stewart Model Hystereses

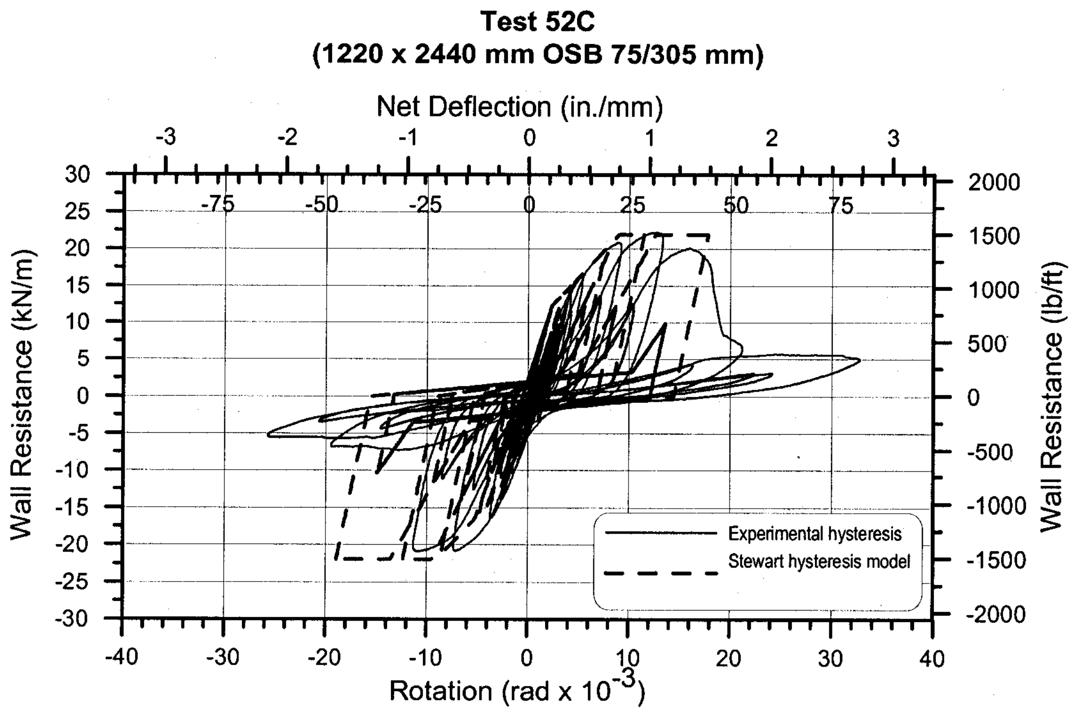


Figure D-37 : Superimposed Experimental and Modeled Hystereses Test 52C

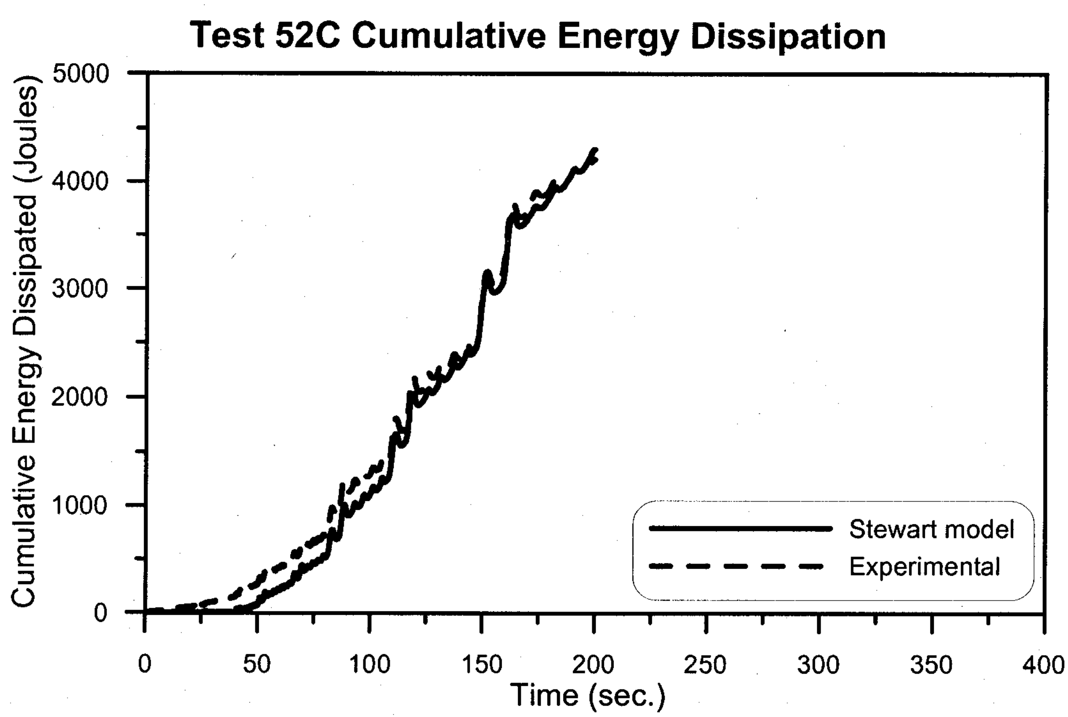


Figure D-38 : Cumulative Energy Dissipated from Experimental Test and Stewart Model Hystereses

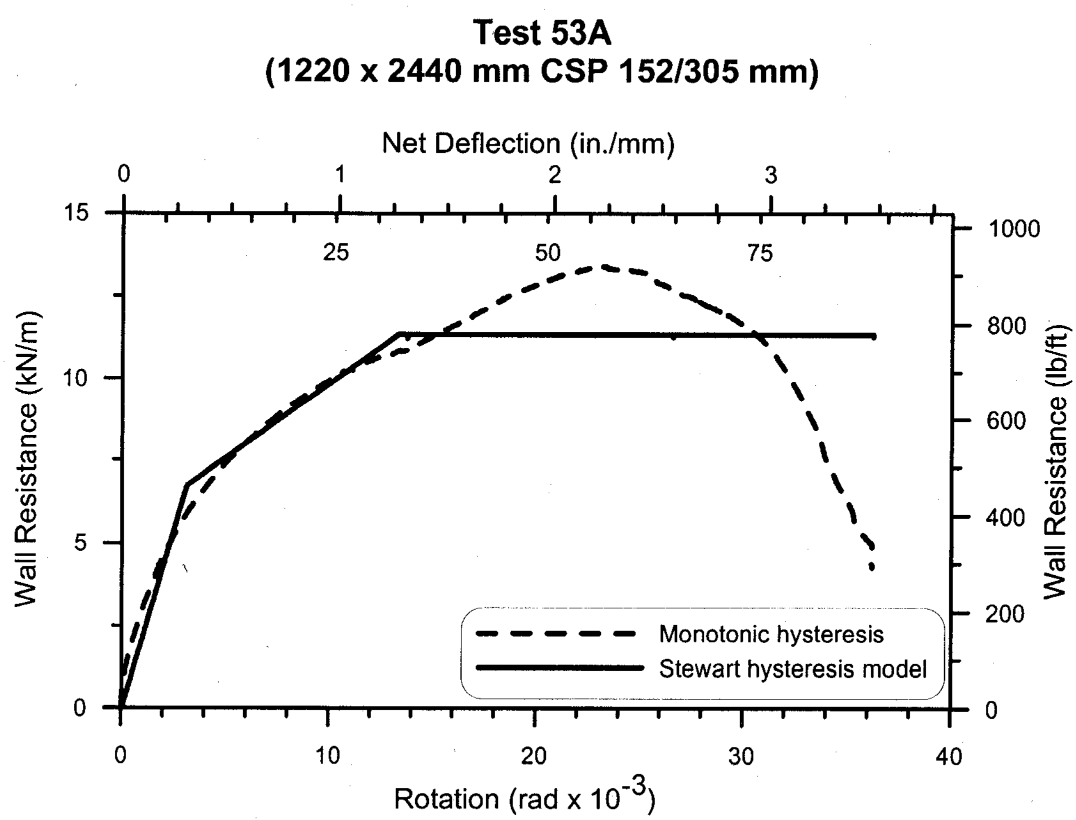


Figure D-39 : Superimposed Experimental and Modeled Hystereses Test 53A

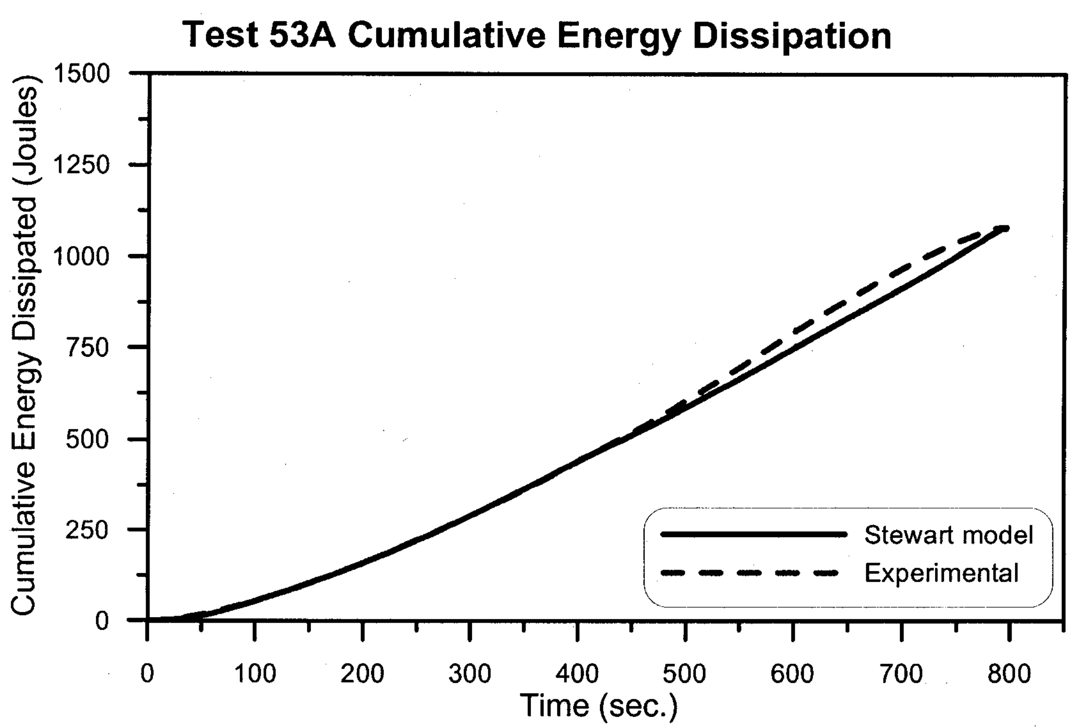


Figure D-40 : Cumulative Energy Dissipated from Experimental Test and Stewart Model Hystereses

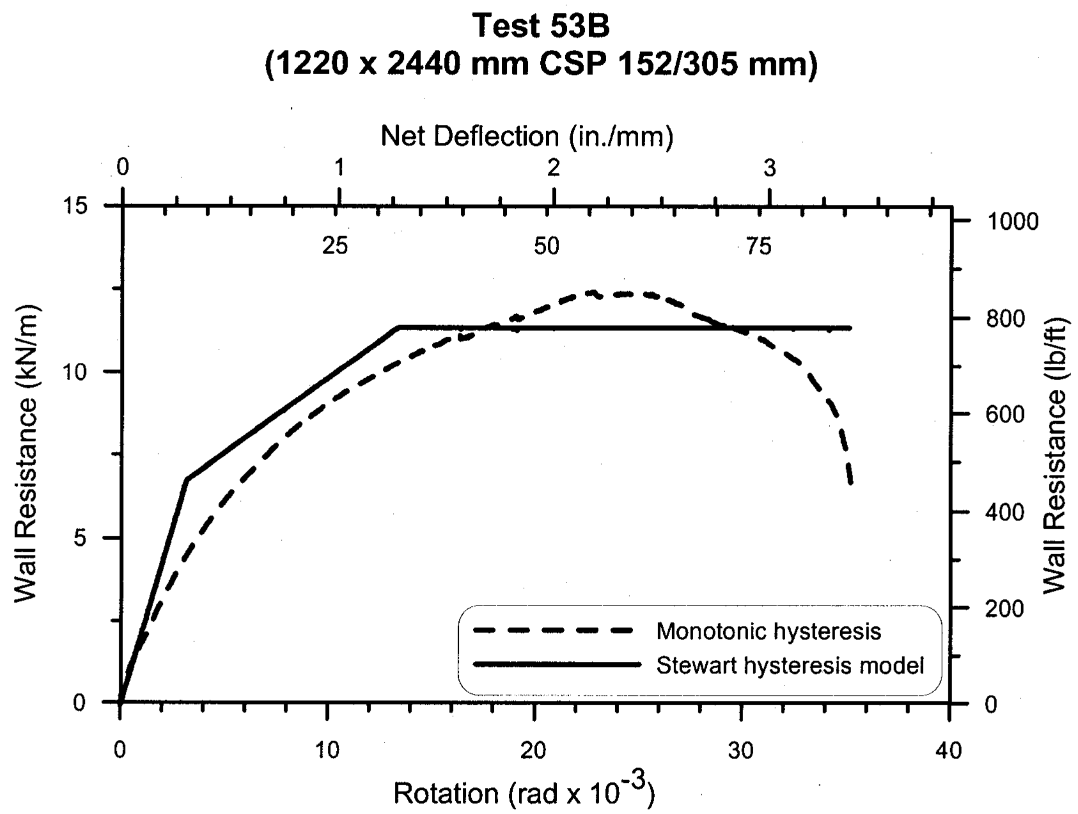


Figure D-41 : Superimposed Experimental and Modeled Hystereses Test 53B

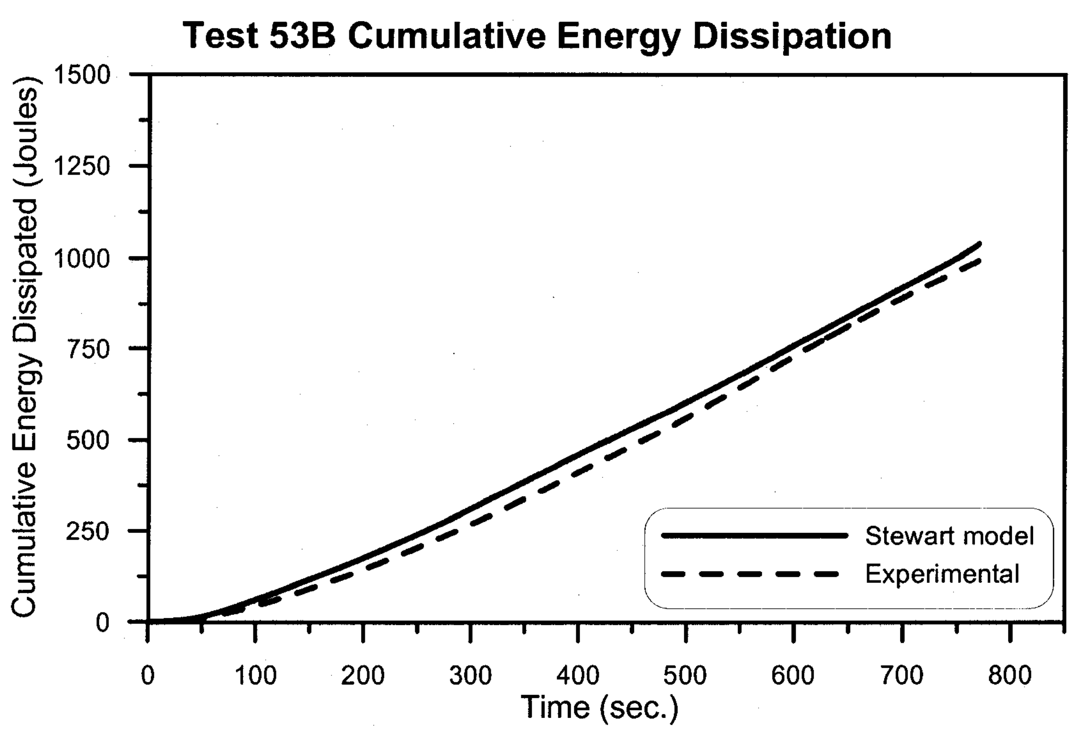


Figure D-42 : Cumulative Energy Dissipated from Experimental Test and Stewart Model Hystereses

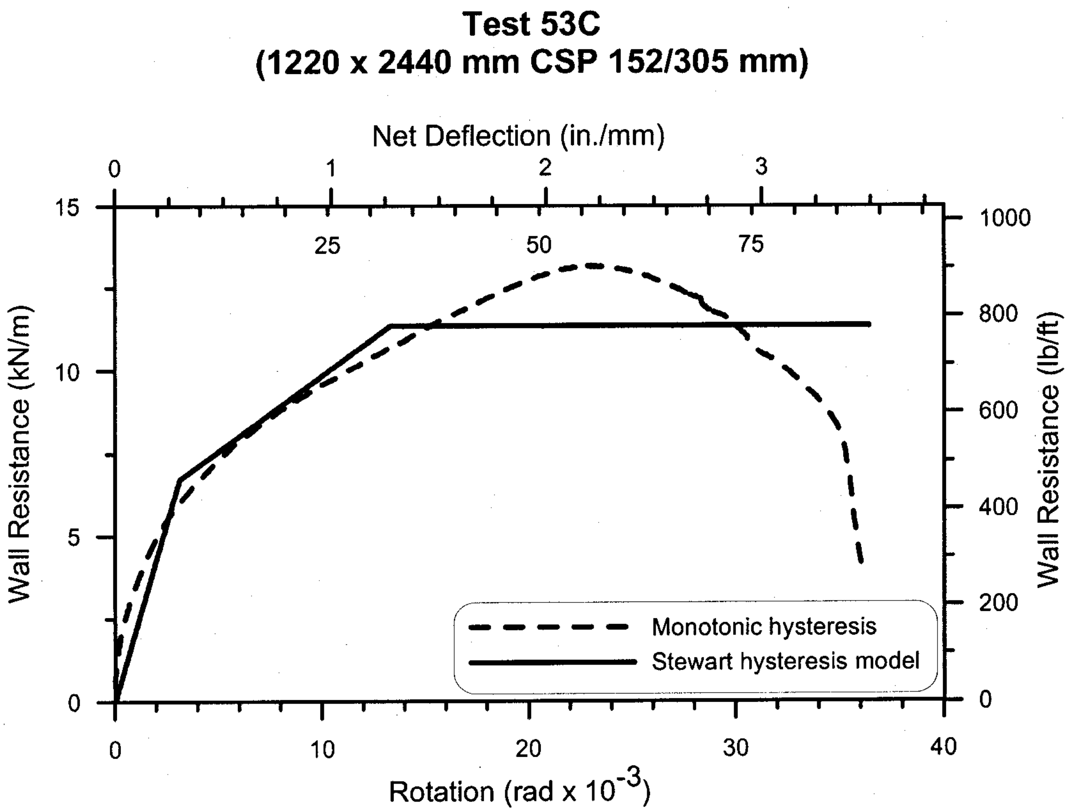


Figure D-43 : Superimposed Experimental and Modeled Hystereses Test 53C

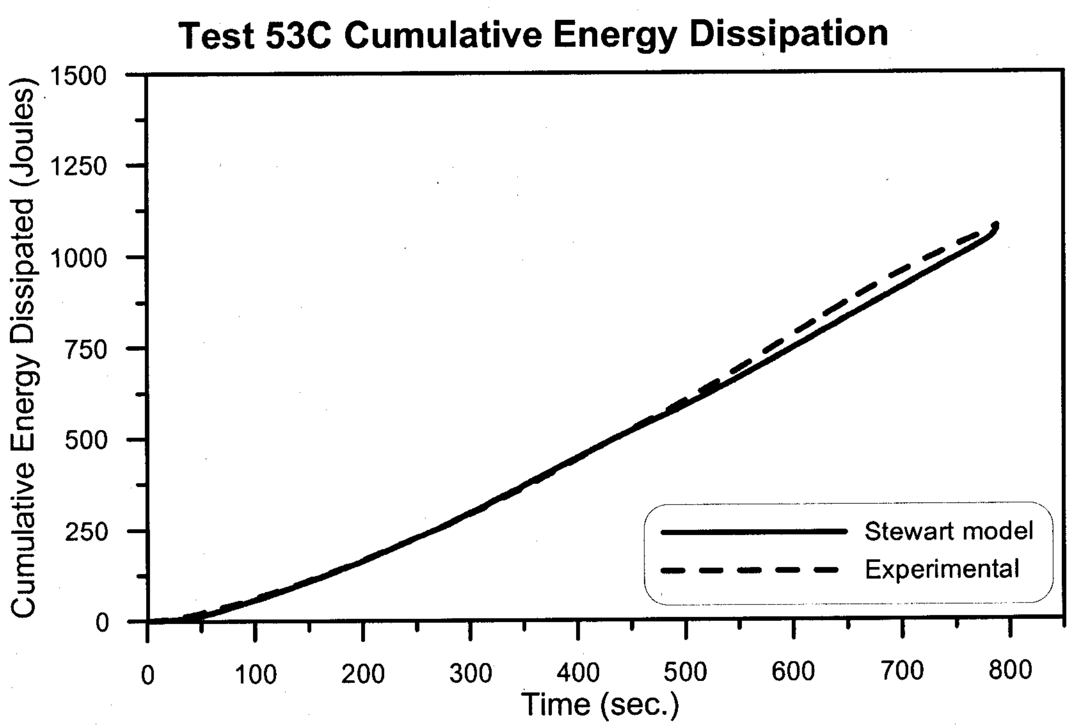


Figure D-44 : Cumulative Energy Dissipated from Experimental Test and Stewart Model Hystereses

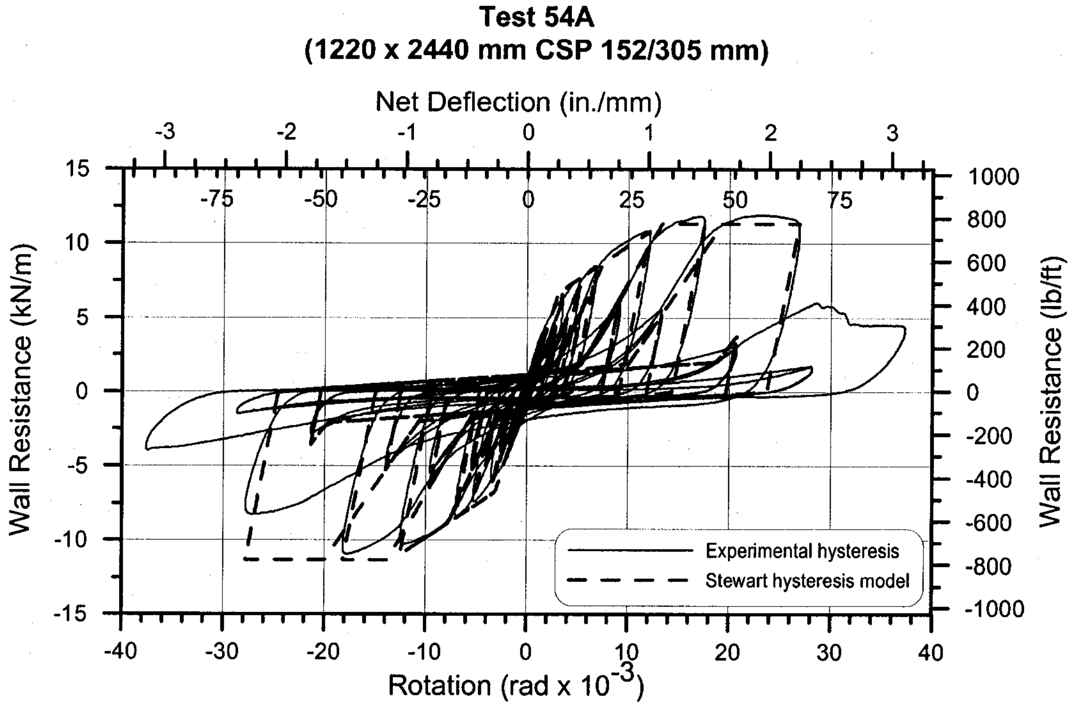


Figure D-45 : Superimposed Experimental and Modeled Hystereses Test 54A

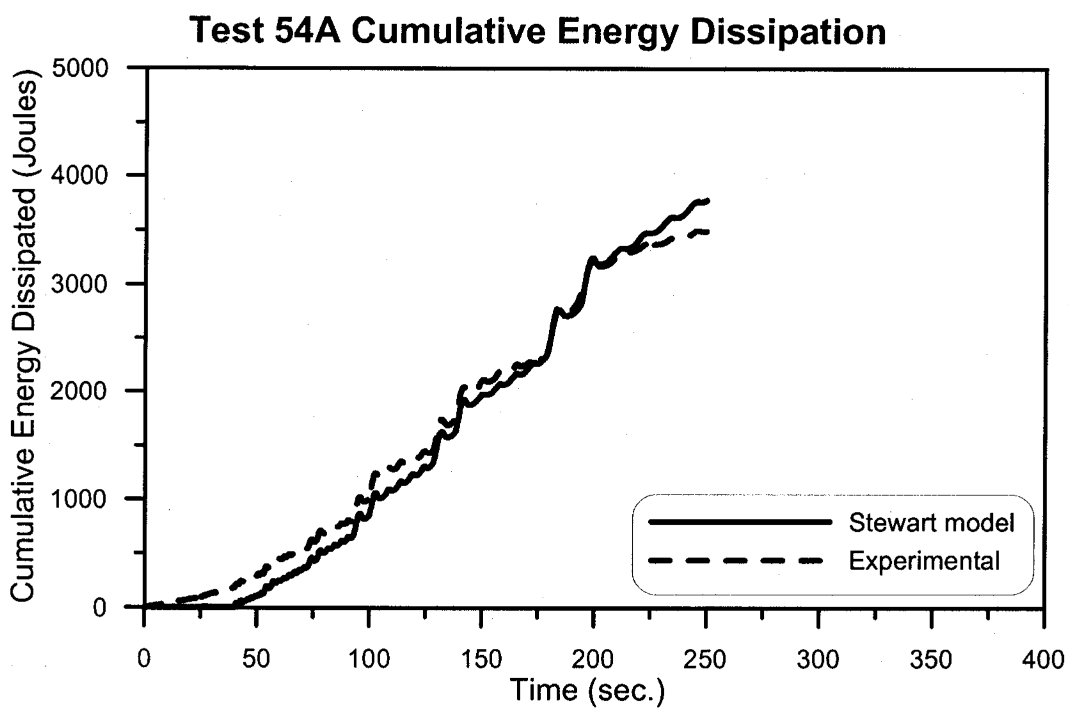


Figure D-46 : Cumulative Energy Dissipated from Experimental Test and Stewart Model Hystereses

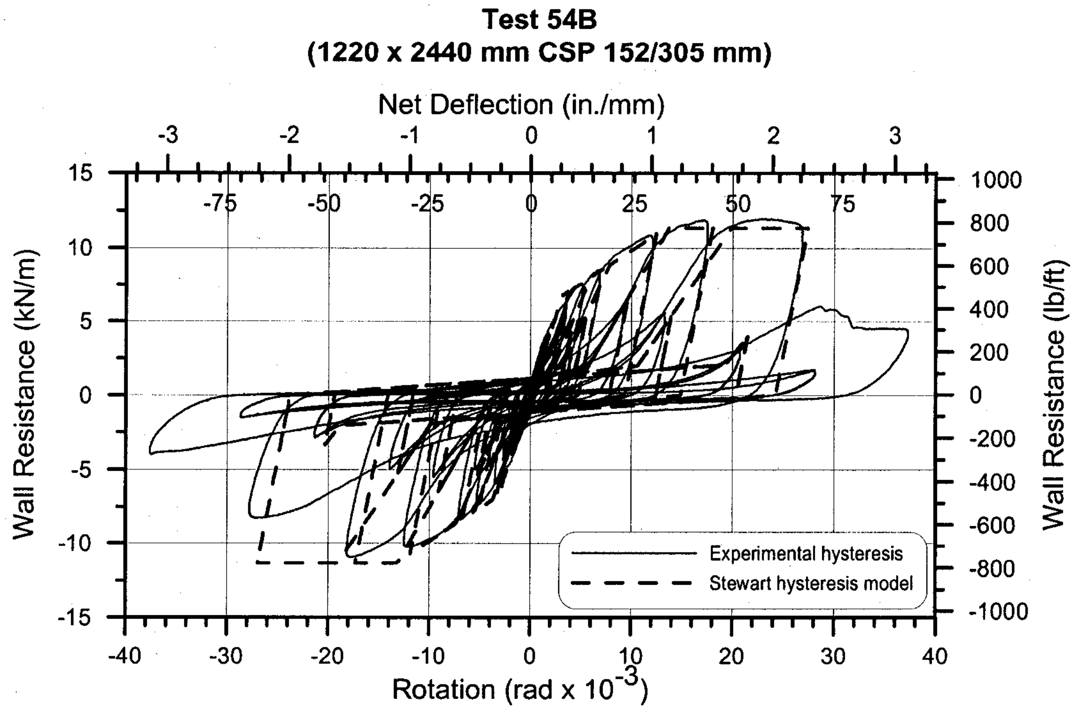


Figure D-47 : Superimposed Experimental and Modeled Hystereses Test 54B

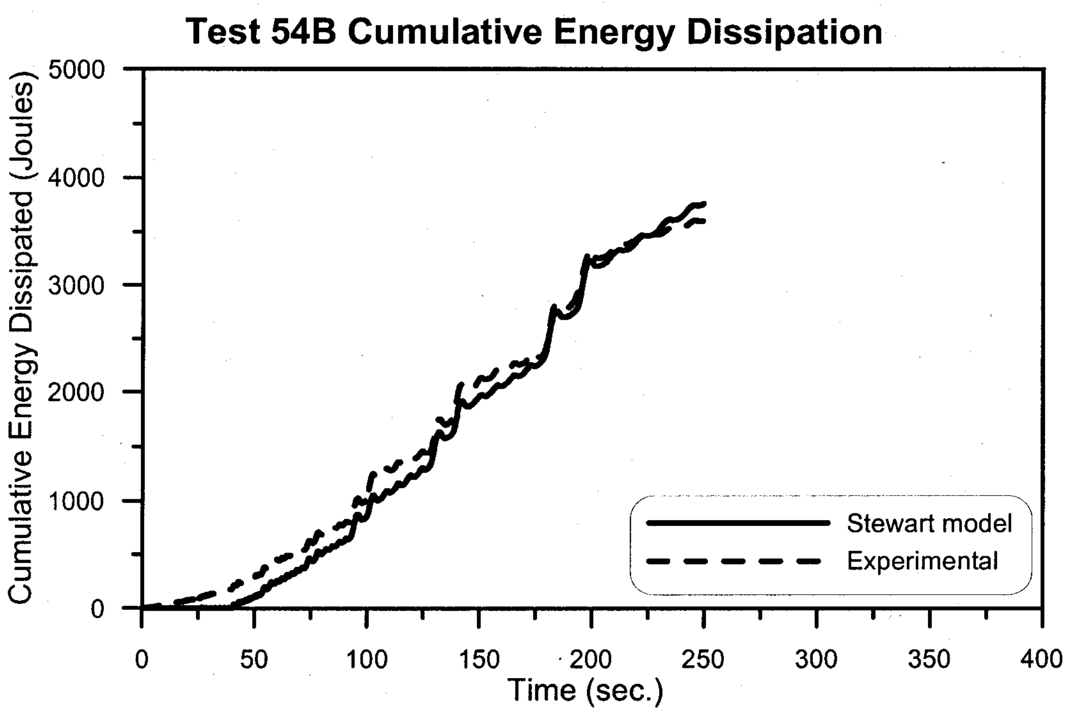


Figure D-48 : Cumulative Energy Dissipated from Experimental Test and Stewart Model Hystereses

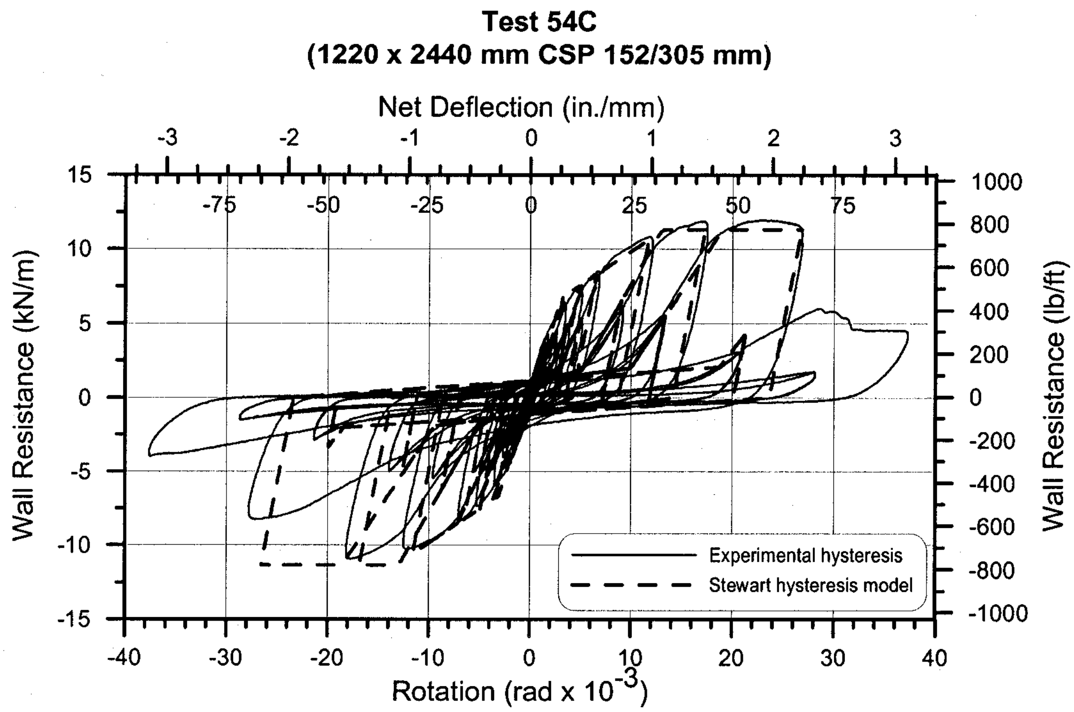


Figure D-49 : Superimposed Experimental and Modeled Hystereses Test 54C

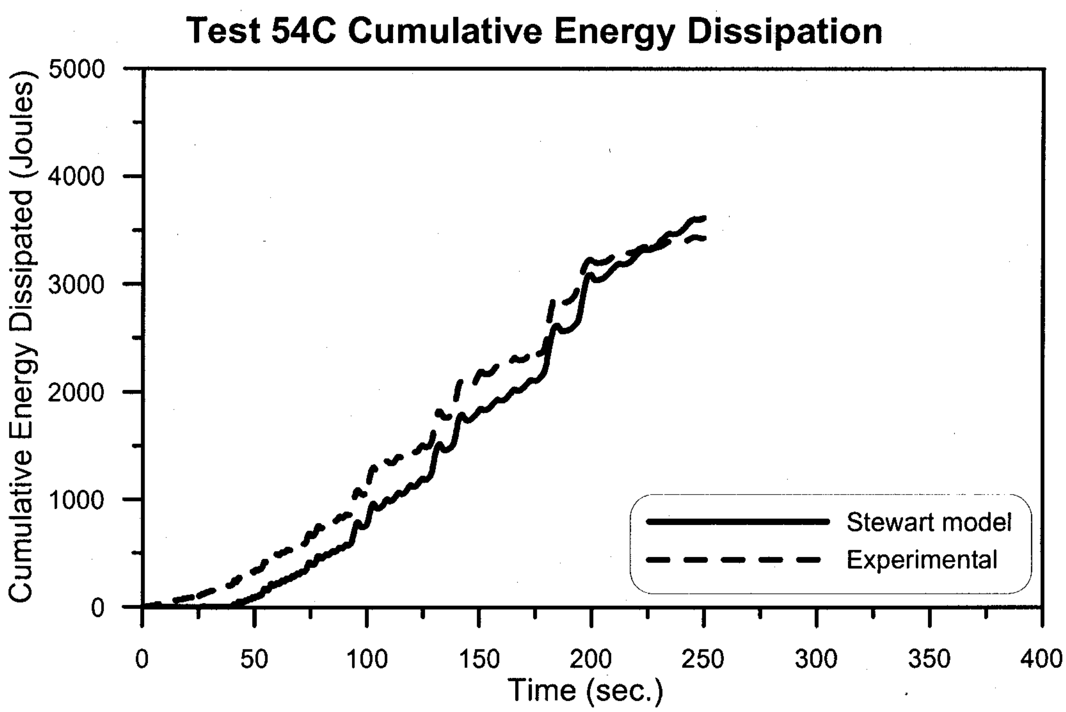


Figure D-50 : Cumulative Energy Dissipated from Experimental Test and Stewart Model Hystereses

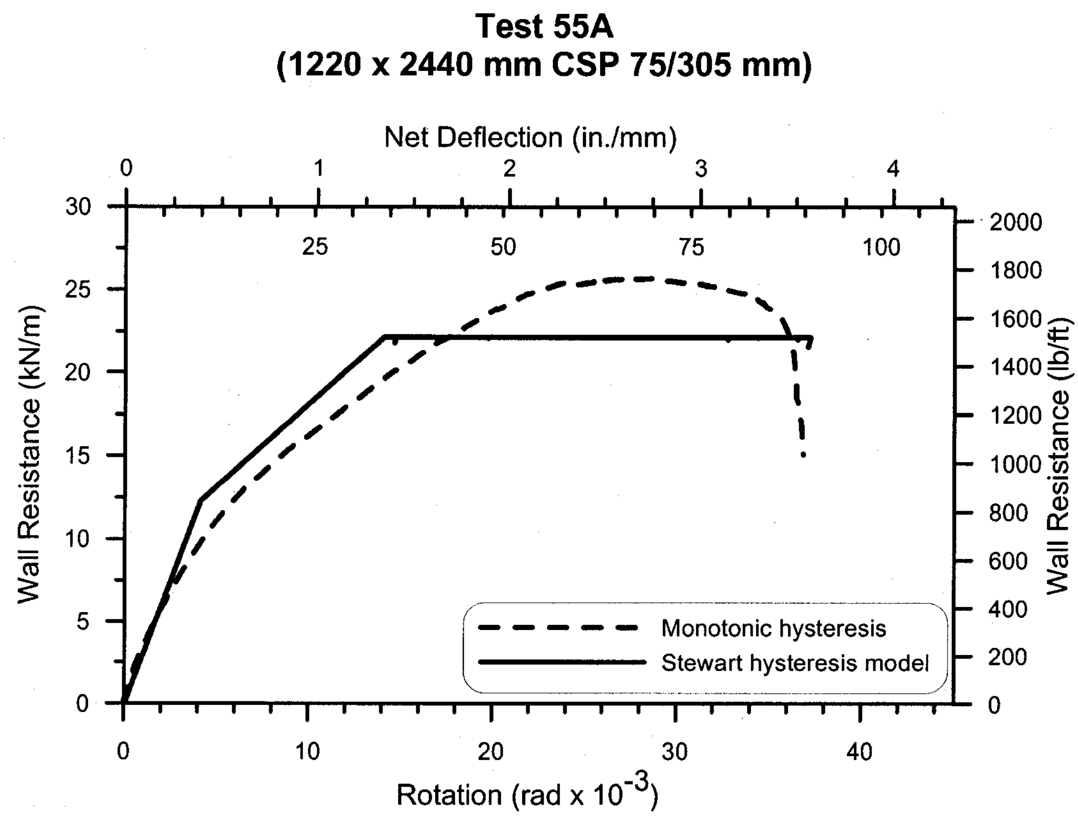


Figure D-51 : Superimposed Experimental and Modeled Hystereses Test 55A

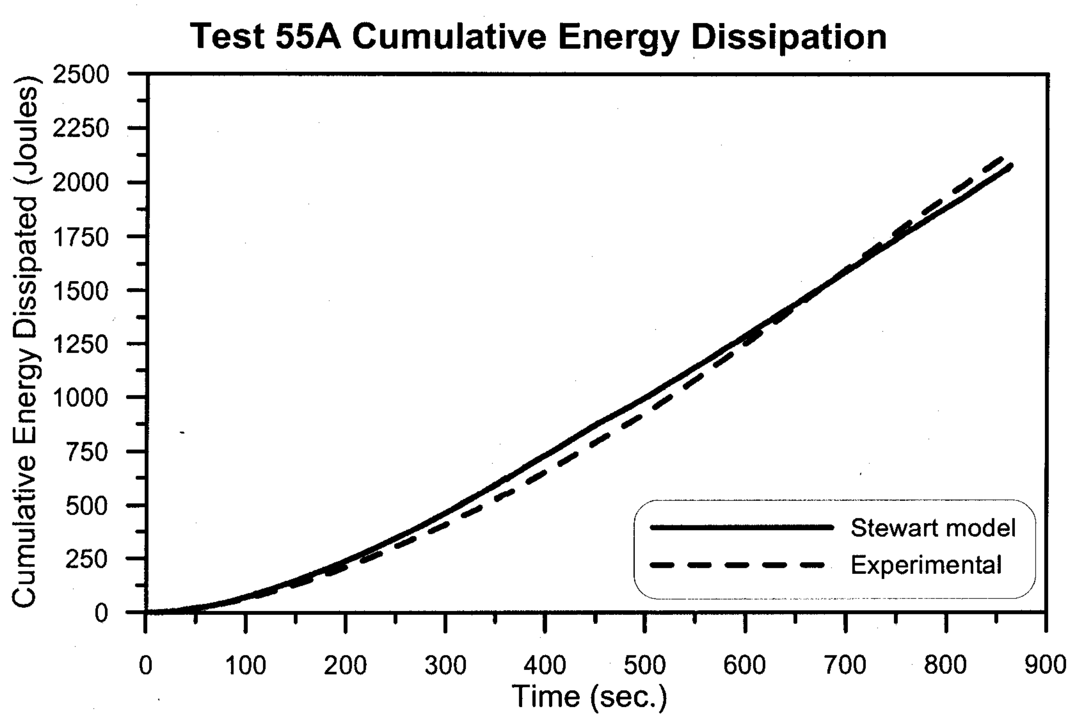


Figure D-52 : Cumulative Energy Dissipated from Experimental Test and Stewart Model Hystereses

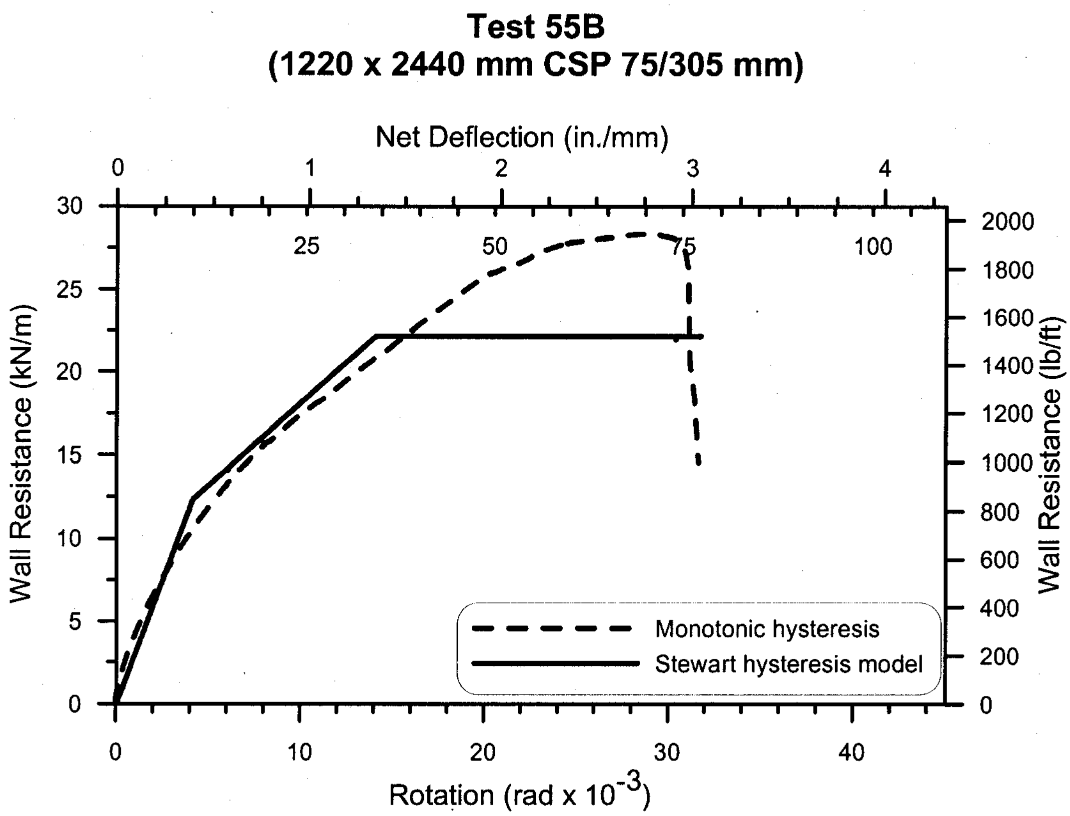


Figure D-53 : Superimposed Experimental and Modeled Hystereses Test 55B

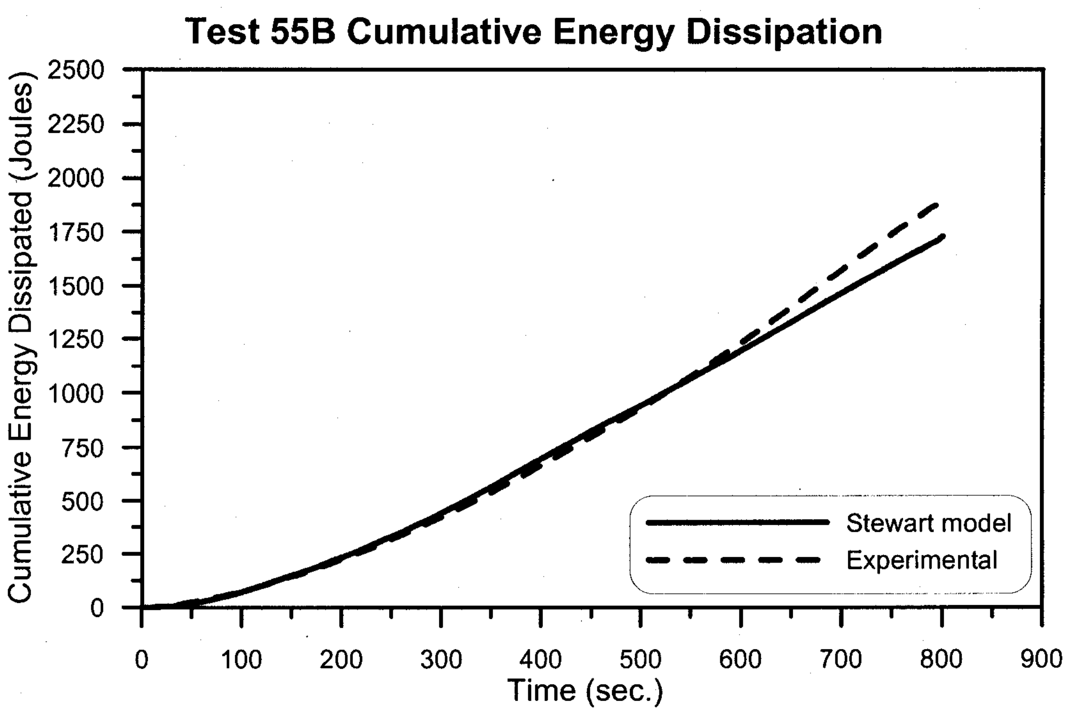


Figure D-54 : Cumulative Energy Dissipated from Experimental Test and Stewart Model Hystereses

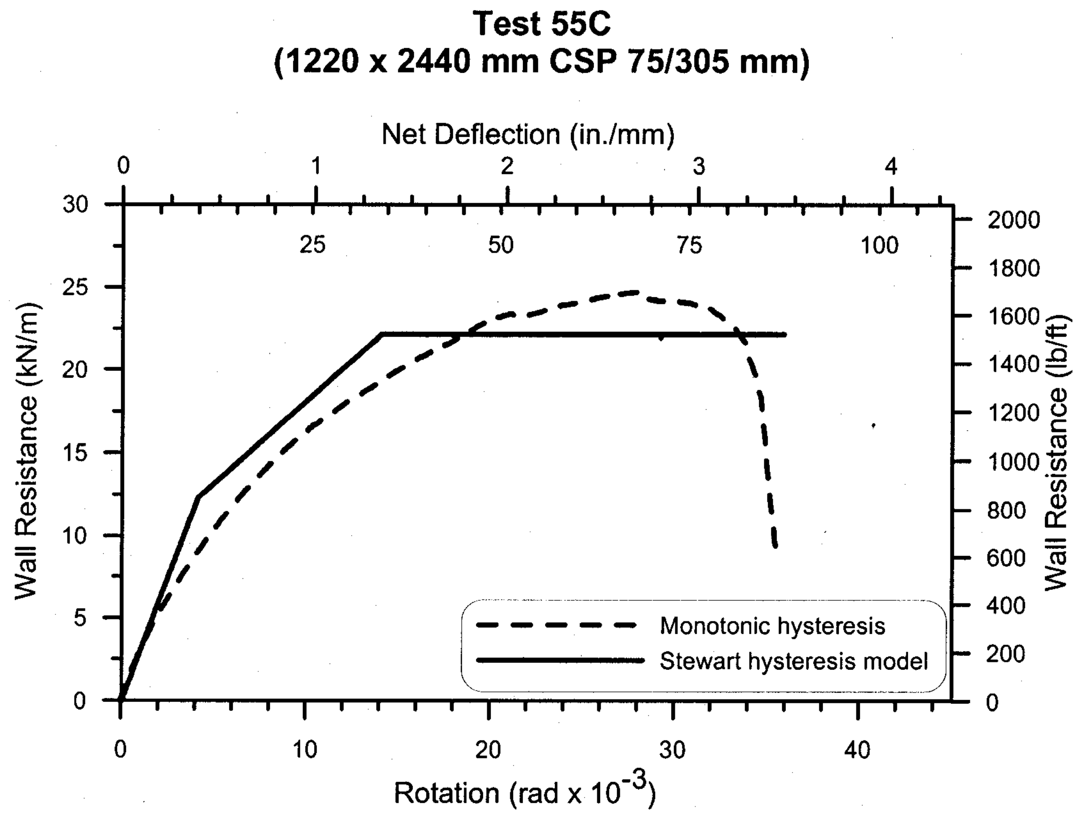


Figure D-55 : Superimposed Experimental and Modeled Hystereses Test 55C

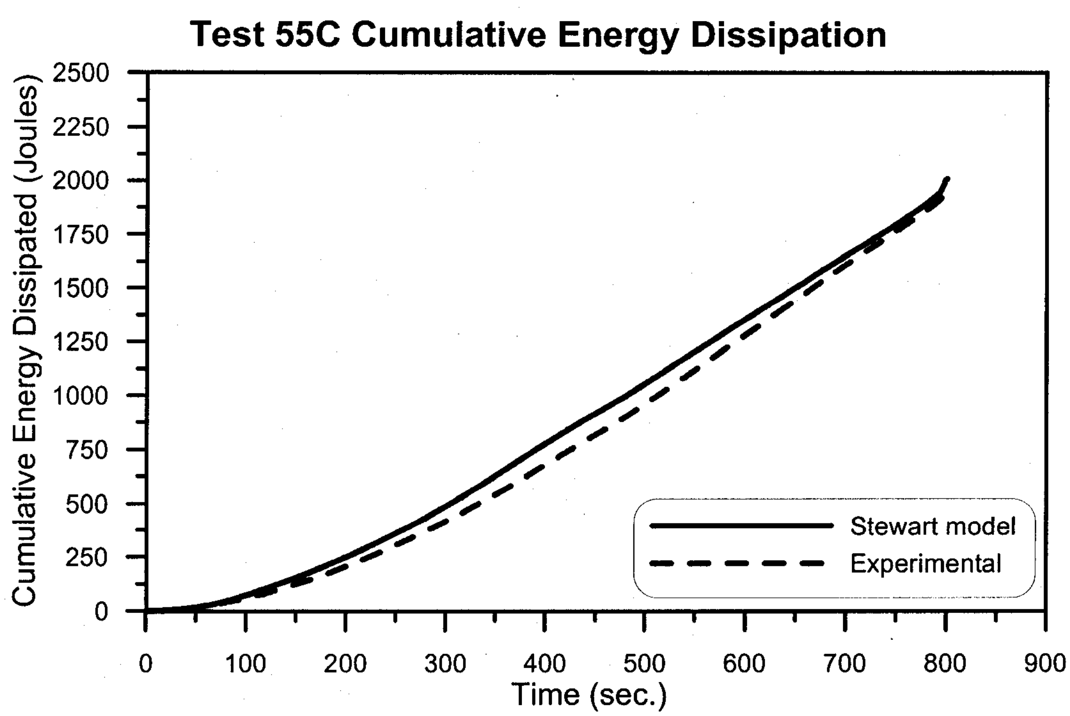


Figure D-56 : Cumulative Energy Dissipated from Experimental Test and Stewart Model Hystereses

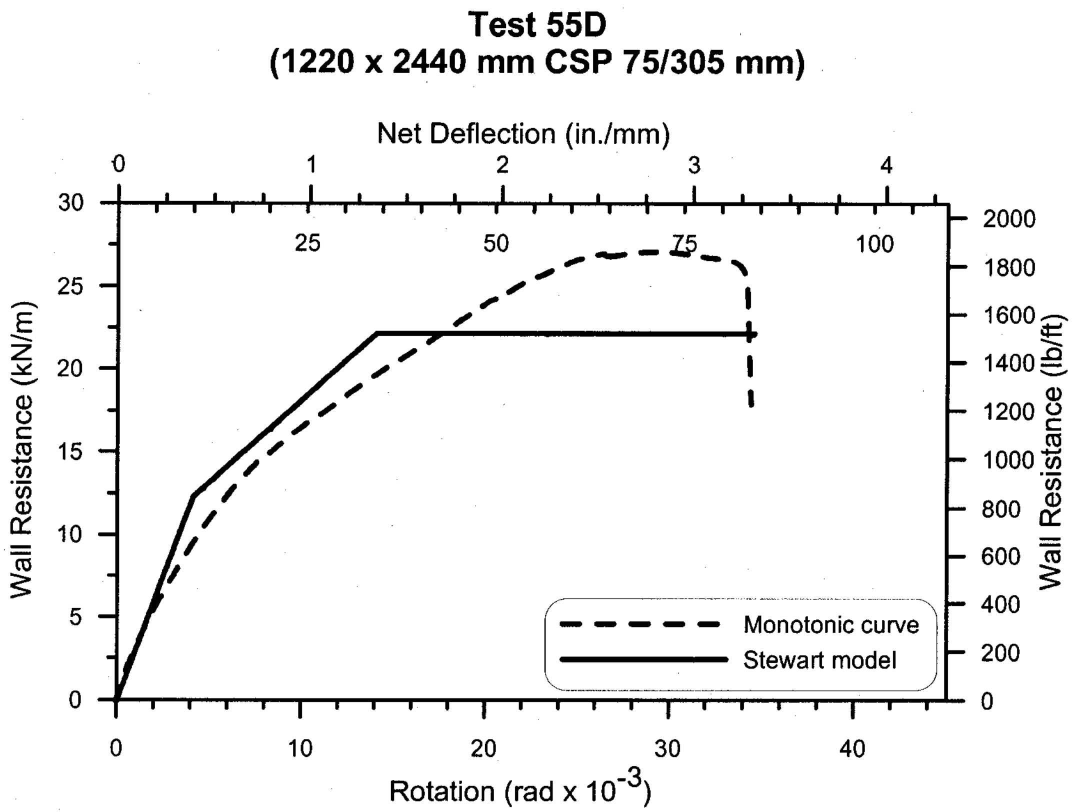


Figure D-57 : Superimposed Experimental and Modeled Hystereses Test 55D

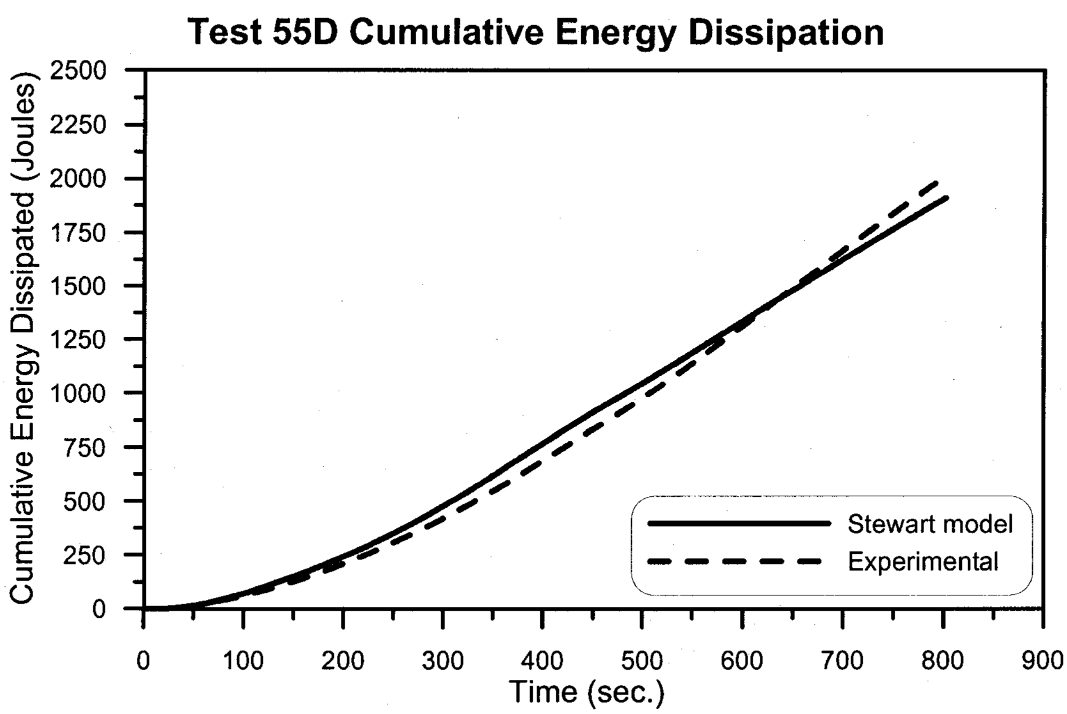


Figure D-58 : Cumulative Energy Dissipated from Experimental Test and Stewart Model Hystereses

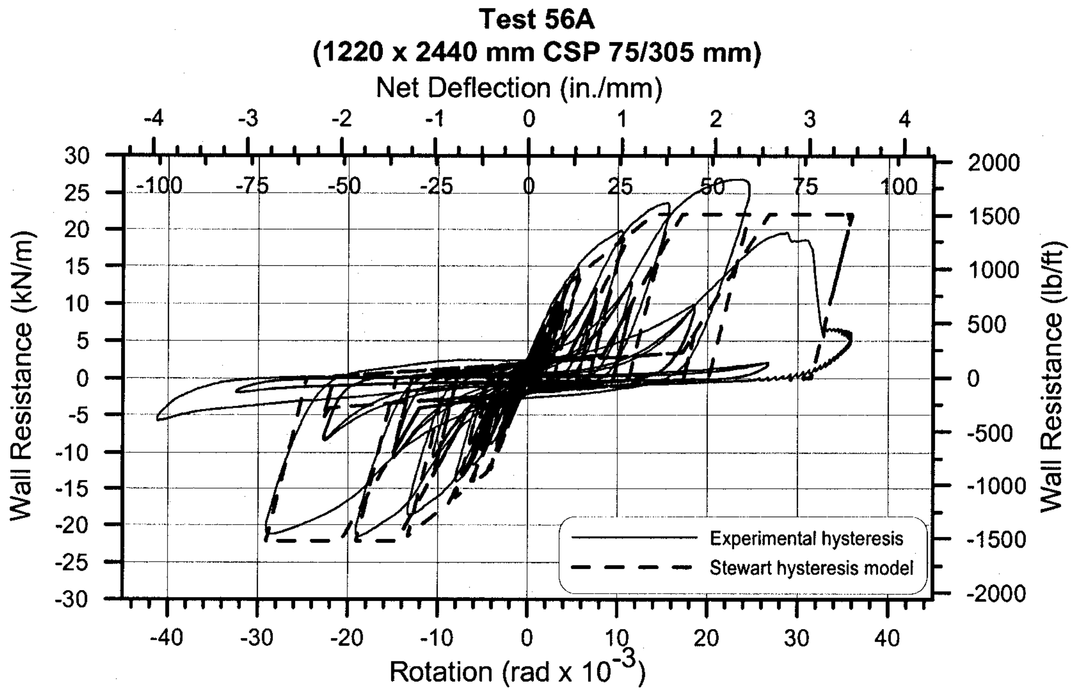


Figure D-59 : Superimposed Experimental and Modeled Hystereses Test 56A

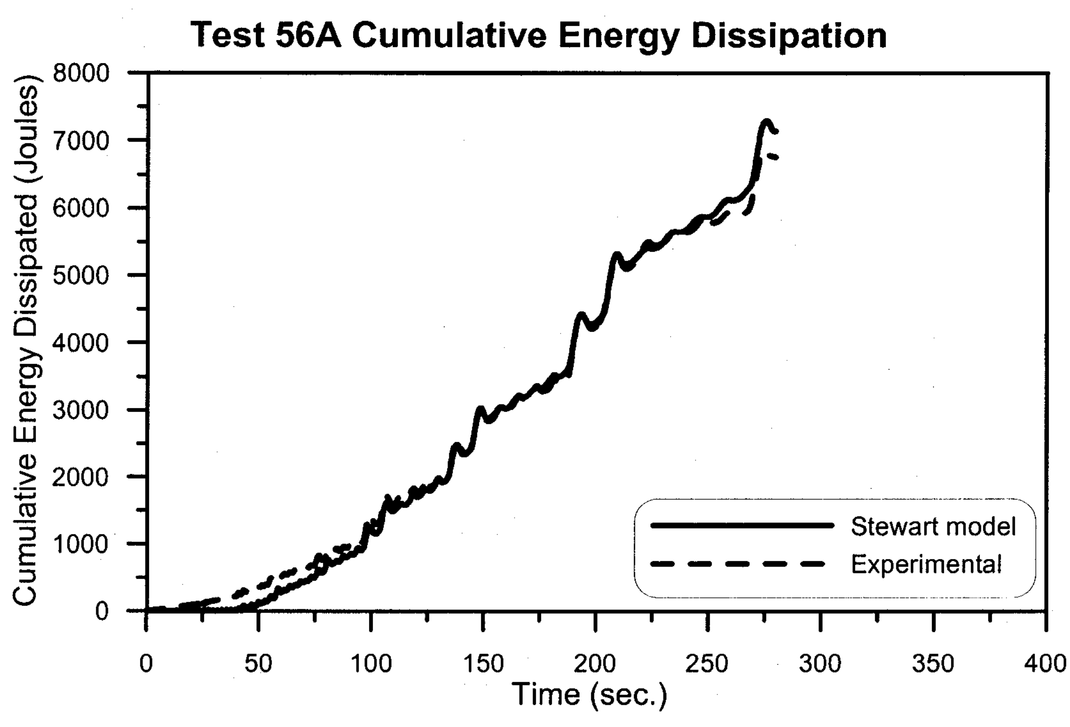


Figure D-60 : Cumulative Energy Dissipated from Experimental Test and Stewart Model Hystereses

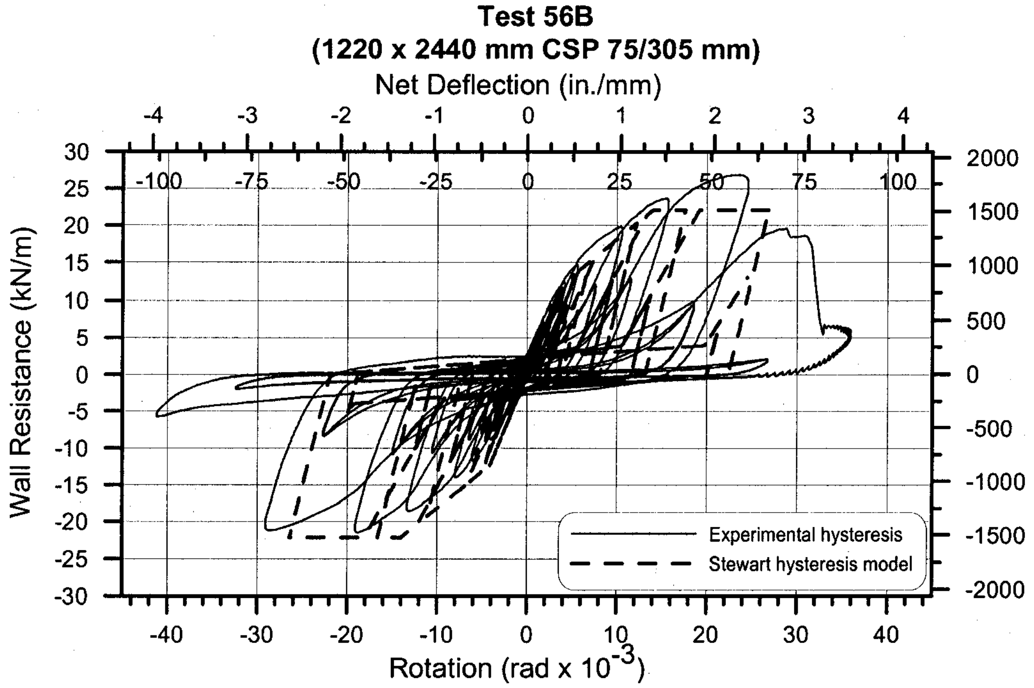


Figure D-61 : Superimposed Experimental and Modeled Hystereses Test 56B

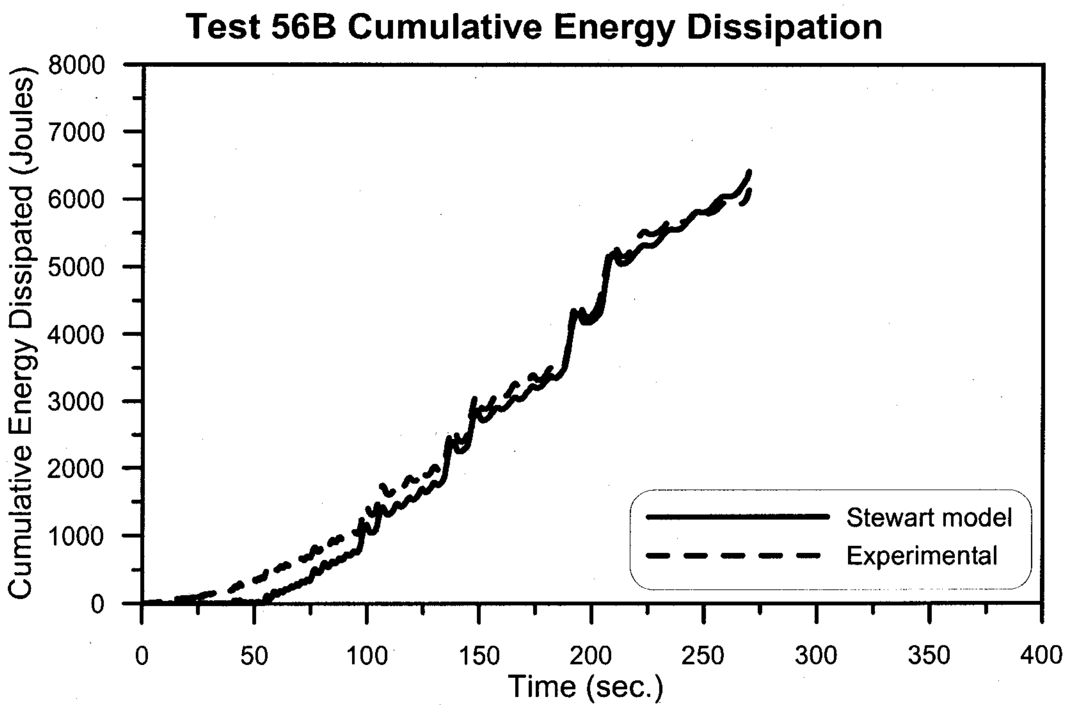


Figure D-62 : Cumulative Energy Dissipated from Experimental Test and Stewart Model Hystereses

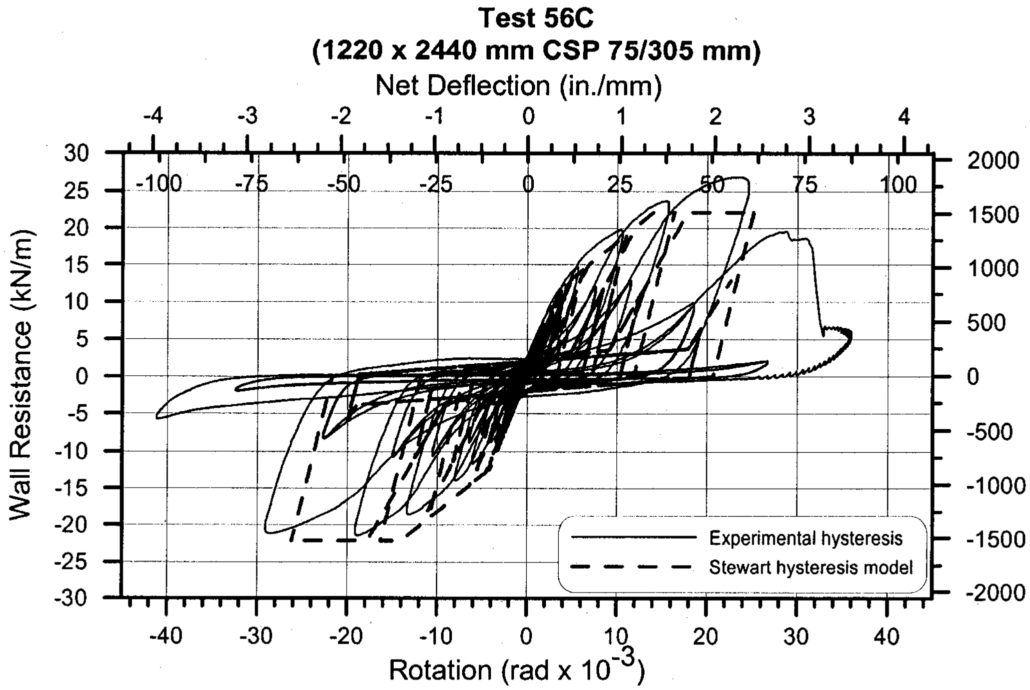


Figure D-63 : Superimposed Experimental and Modeled Hystereses Test 56C

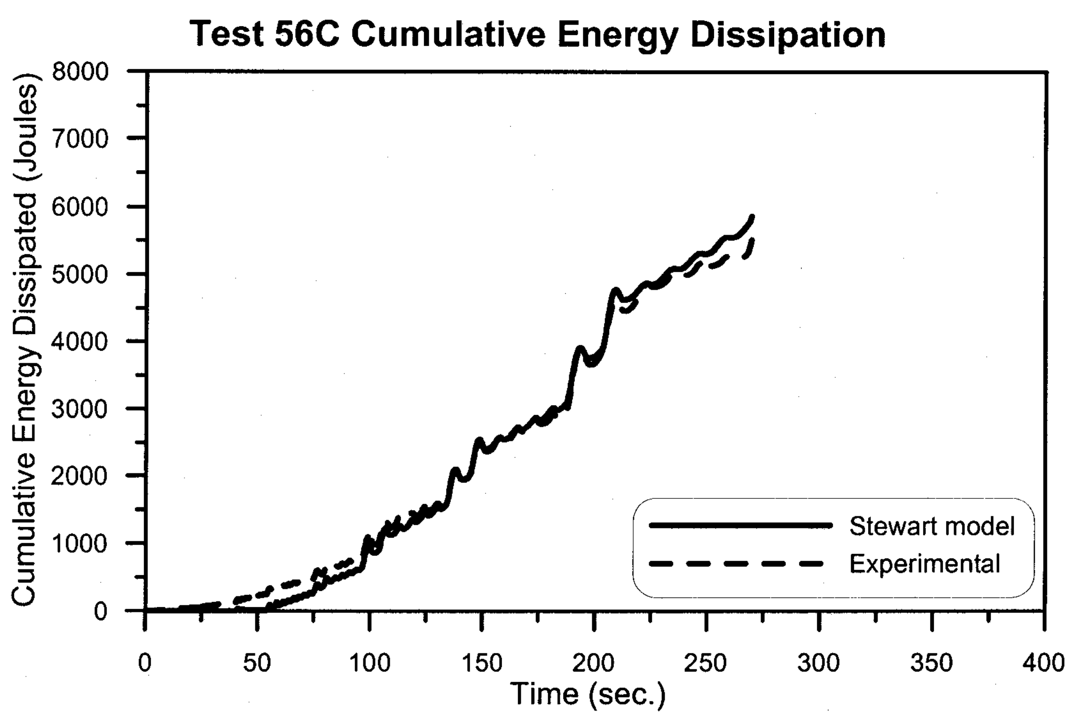
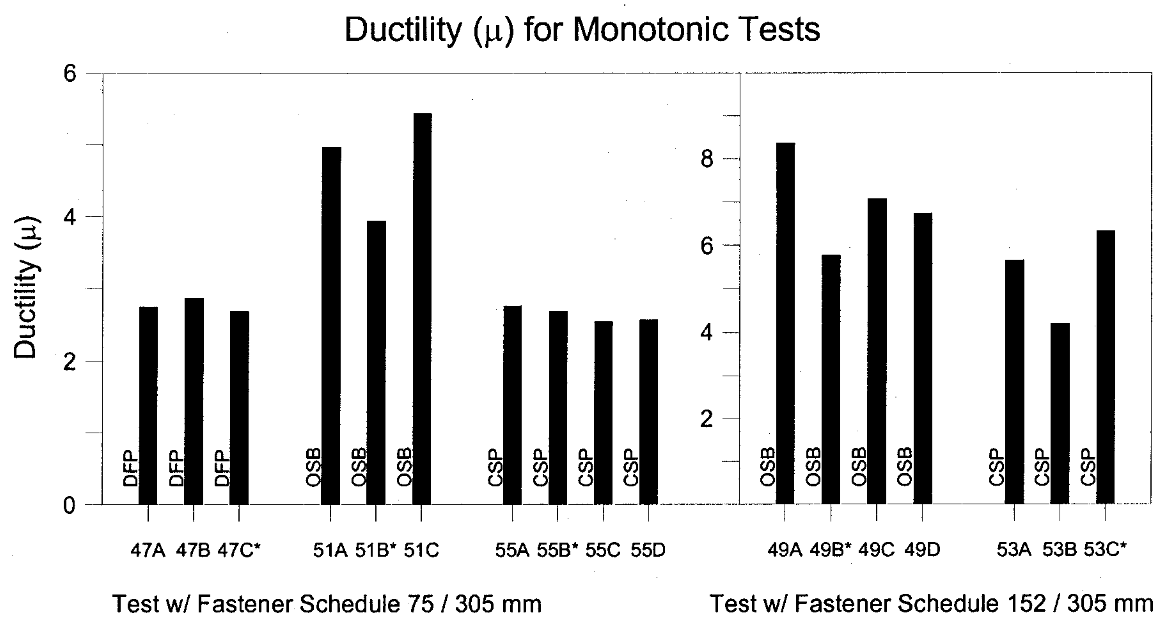


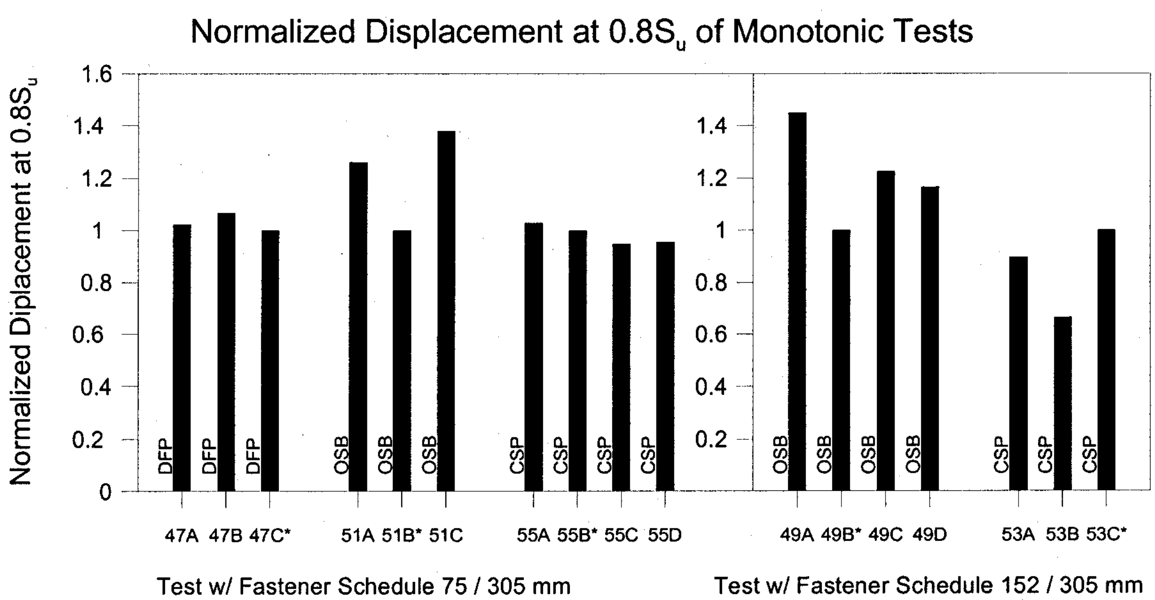
Figure D-64 : Cumulative Energy Dissipated from Experimental Test and Stewart Model Hystereses

APPENDIX E

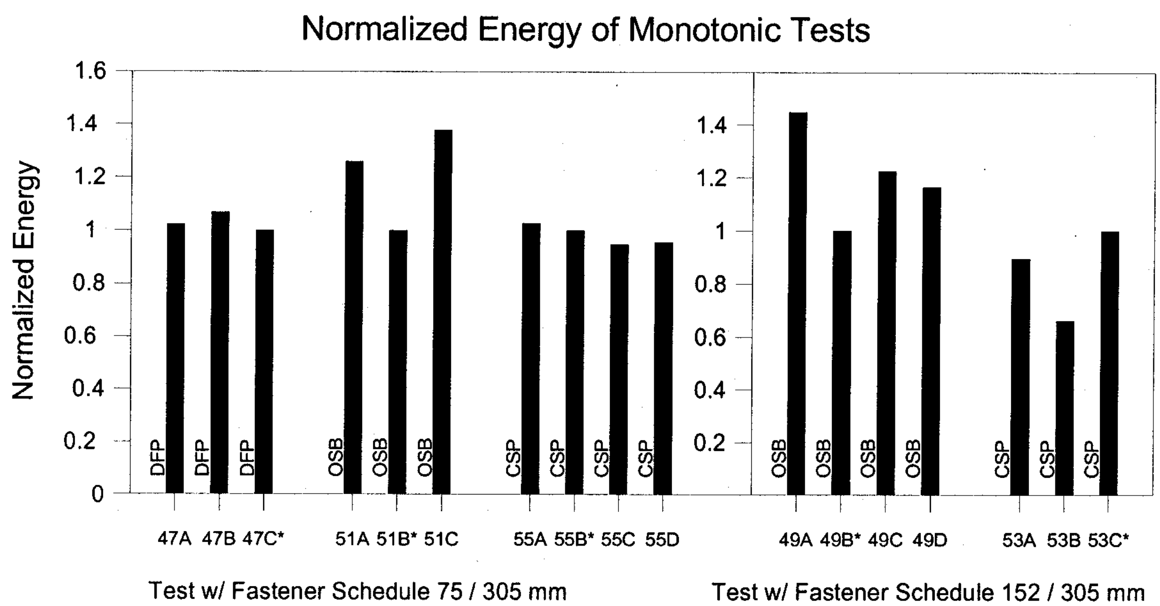
BAR GRAPHS OF SHEAR WALL RESULTS

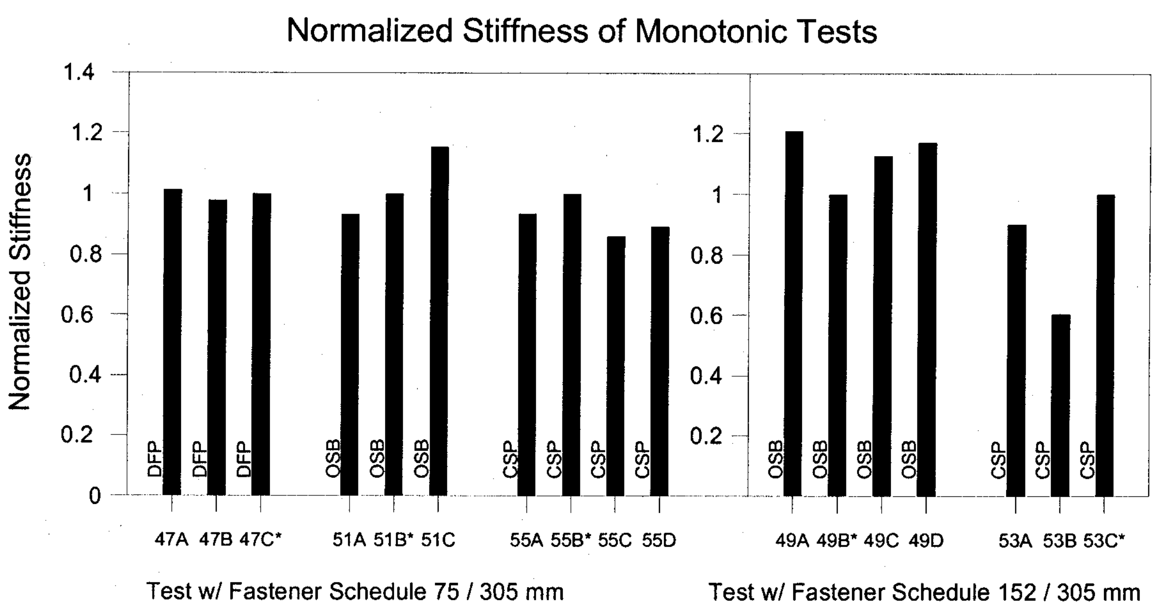


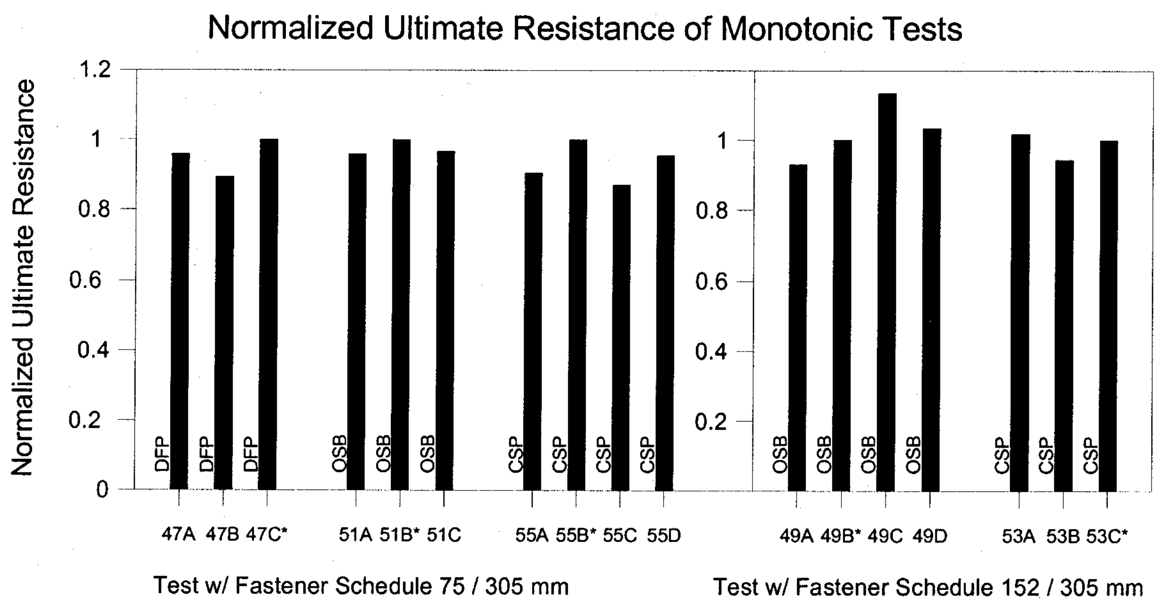
Ductility (μ) for Monotonic Tests
Ductility (μ)
DFP DFP DFP OSB OSB OSB CSP CSP CSP CSP OSB OSB OSB OSB CSP CSP CSP
47A 47B 47C* 51A 51B* 51C 55A 55B* 55C 55D 49A 49B* 49C 49D 53A 53B 53C*
Test w/ Fastener Schedule 75 / 305 mm
Test w/ Fastener Schedule 152 / 305 mm



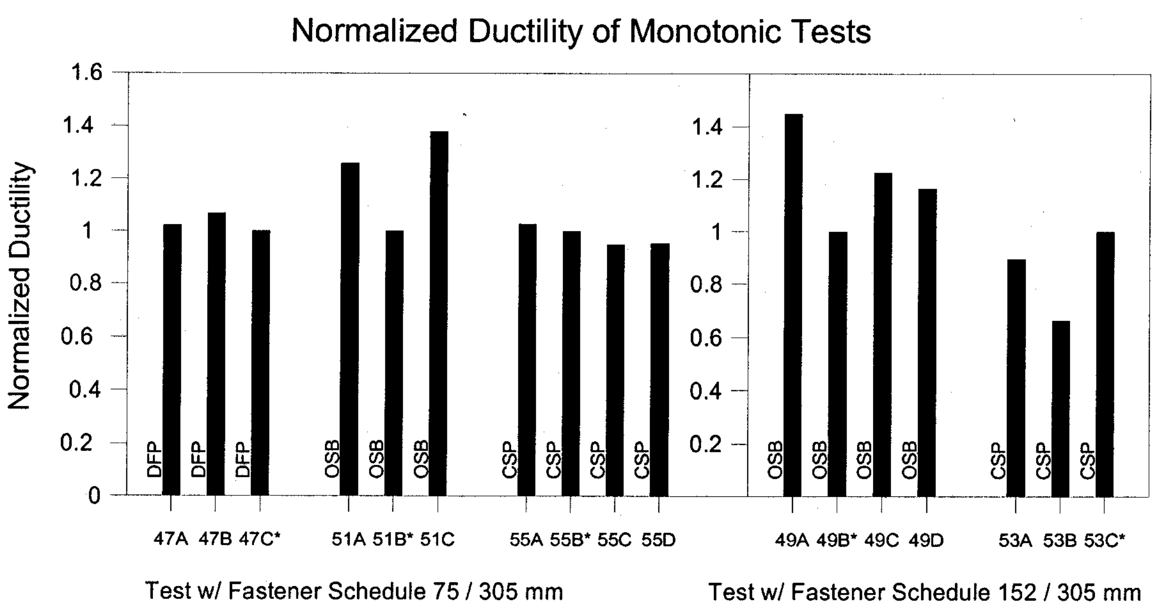
Normalized Displacement at 0.8Su of Monotonic Tests
Normalized Diplacement at 0.8Su
DFP DFP DFP OSB OSB OSB CSP CSP CSP CSP OSB OSB OSB OSB CSP CSP CSP
47A 47B 47C* 51A 51B* 51C 55A 55B* 55C 55D 49A 49B* 49C 49D 53A 53B 53C*
Test w/ Fastener Schedule 75 / 305 mm
Test w/ Fastener Schedule 152 / 305 mm


Normalized Energy of Monotonic Tests
Normalized Energy
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
DFP
OSB
CSP
47A 47B 47C*
51A 51B* 51C
55A 55B* 55C 55D
49A 49B* 49C 49D
53A 53B 53C*
Test w/ Fastener Schedule 75 / 305 mm
Test w/ Fastener Schedule 152 / 305 mm

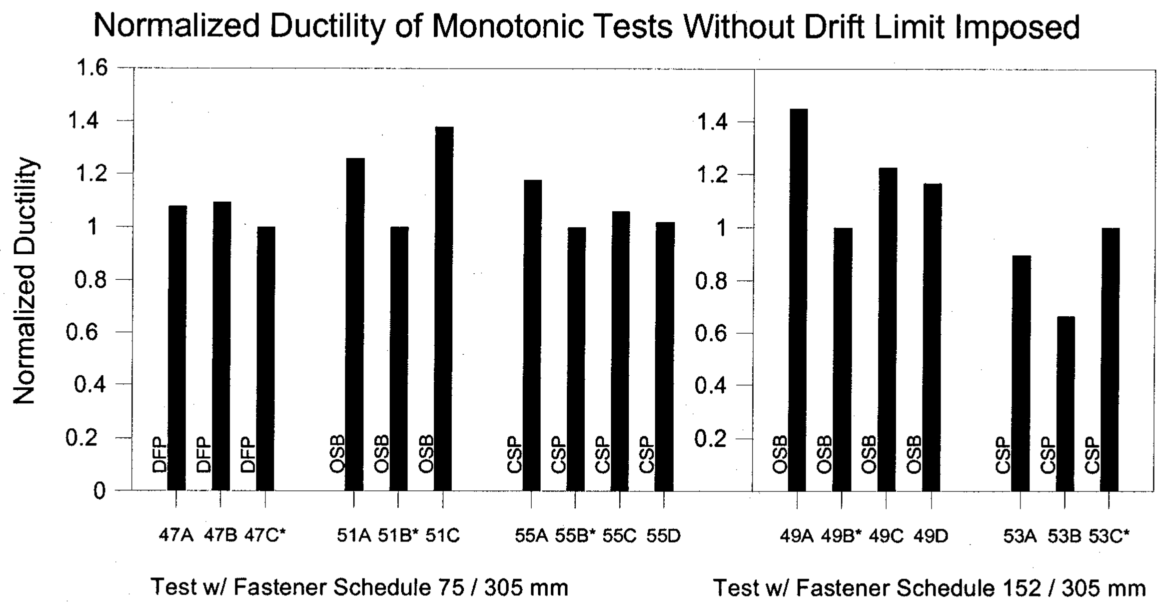

Normalized Stiffness of Monotonic Tests
Normalized Stiffness
1.4
1.2
1
0.8
0.6
0.4
0.2
0
DFP
OSB
CSP
47A 47B 47C*
51A 51B* 51C
55A 55B* 55C 55D
49A 49B* 49C 49D
53A 53B 53C*
Test w/ Fastener Schedule 75 / 305 mm
Test w/ Fastener Schedule 152 / 305 mm



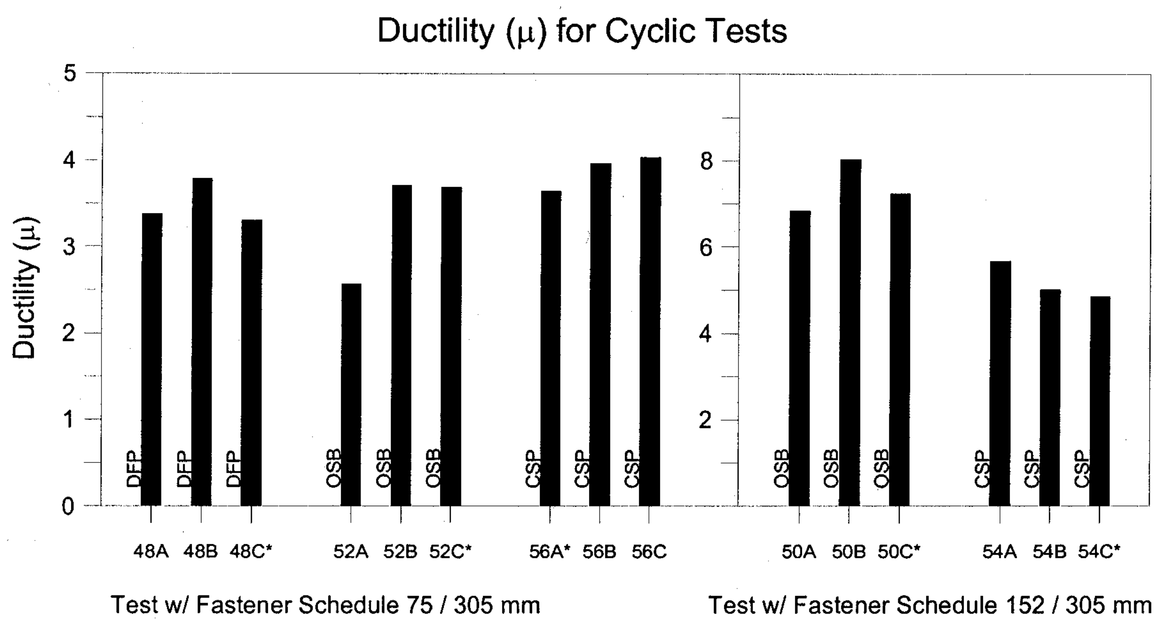
Normalized Ultimate Resistance of Monotonic Tests
Normalized Ultimate Resistance
1.2
1
0.8
0.6
0.4
0.2
0
DFP
OSB
CSP
47A 47B 47C*
51A 51B* 51C
55A 55B* 55C 55D
49A 49B* 49C 49D
53A 53B 53C*
Test w/ Fastener Schedule 75 / 305 mm
Test w/ Fastener Schedule 152 / 305 mm



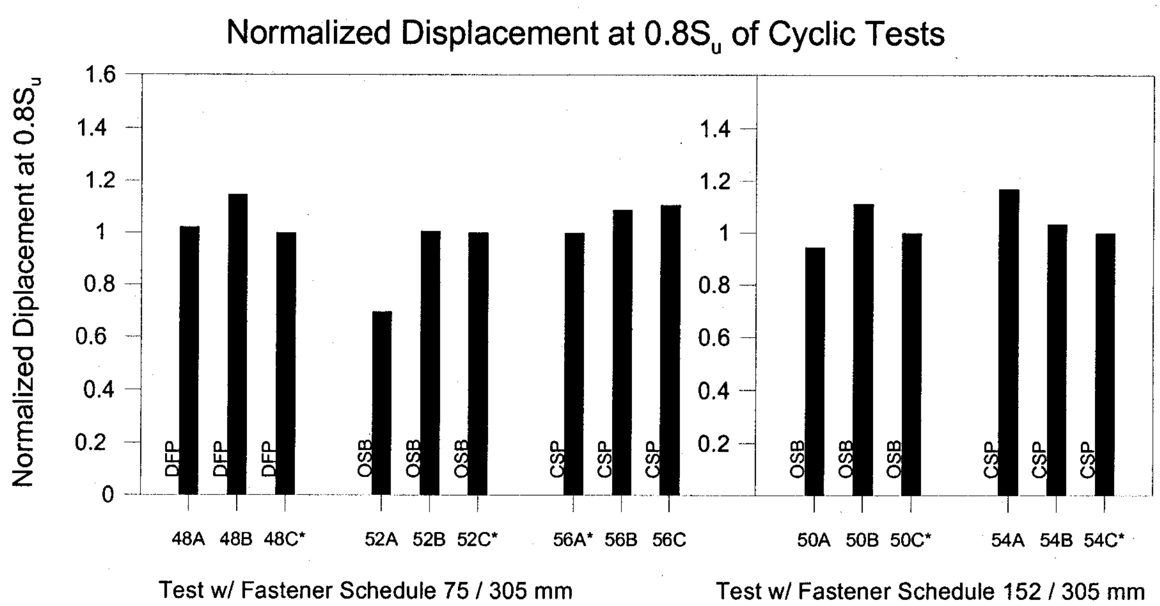
Normalized Ductility of Monotonic Tests
Normalized Ductility
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
DFP
OSB
CSP
47A 47B 47C*
51A 51B* 51C
55A 55B* 55C 55D
49A 49B* 49C 49D
53A 53B 53C*
Test w/ Fastener Schedule 75 / 305 mm
Test w/ Fastener Schedule 152 / 305 mm

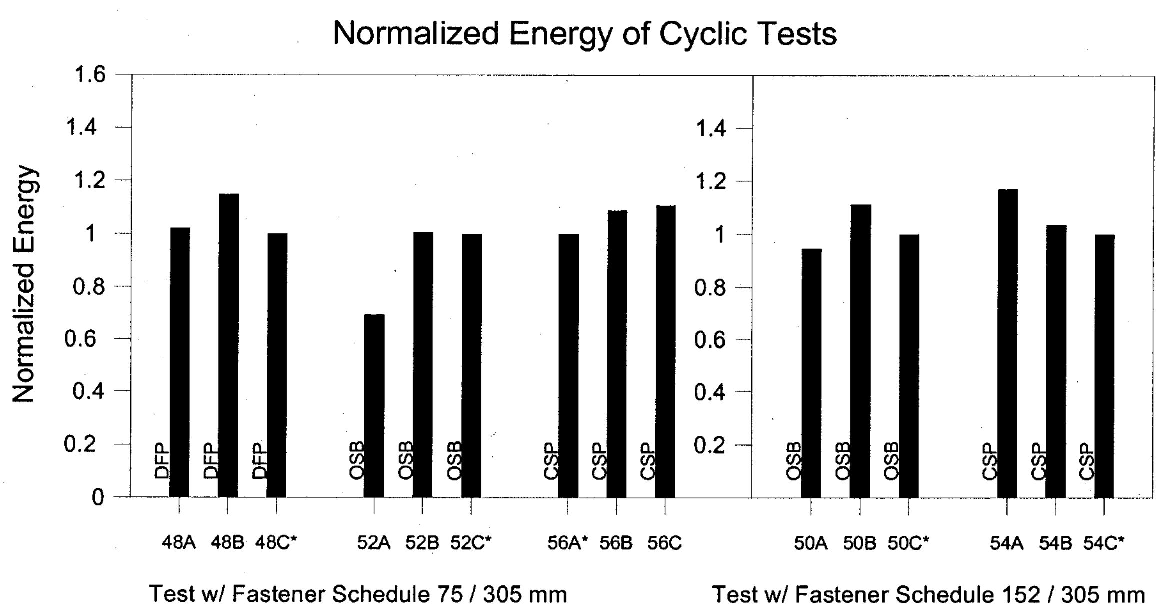


Normalized Ductility of Monotonic Tests Without Drift Limit Imposed
Normalized Ductility
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
DFP
OSB
CSP
47A 47B 47C*
51A 51B* 51C
55A 55B* 55C 55D
49A 49B* 49C 49D
53A 53B 53C*
Test w/ Fastener Schedule 75 / 305 mm
Test w/ Fastener Schedule 152 / 305 mm

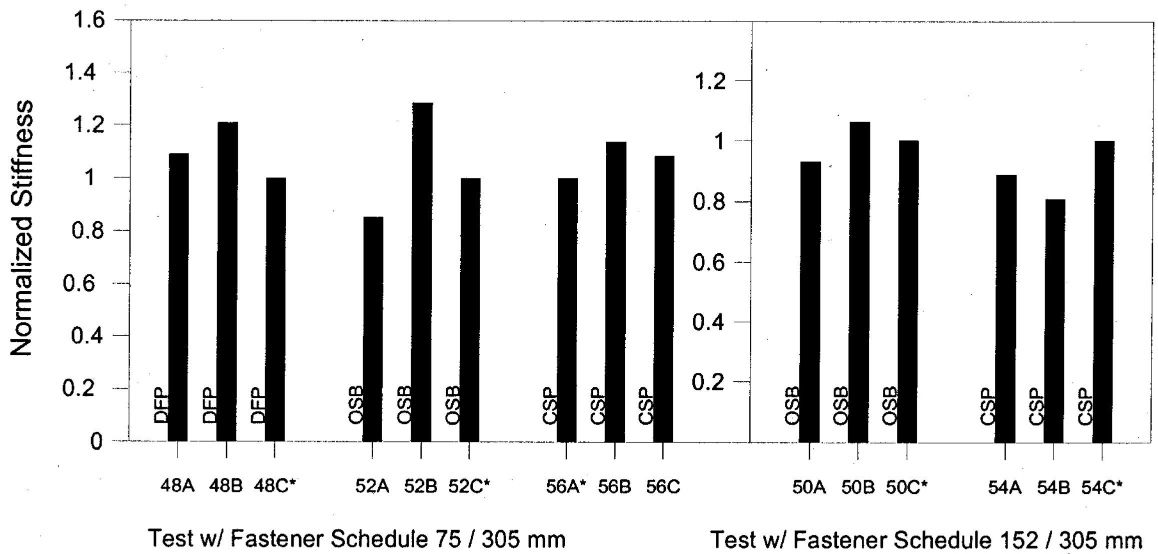


Ductility (μ) for Cyclic Tests
Ductility (μ)
5
4
3
2
1
0
8
6
4
2
DFP
OSB
CSP
48A 48B 48C*
52A 52B 52C*
56A* 56B 56C
50A 50B 50C*
54A 54B 54C*
Test w/ Fastener Schedule 75 / 305 mm
Test w/ Fastener Schedule 152 / 305 mm

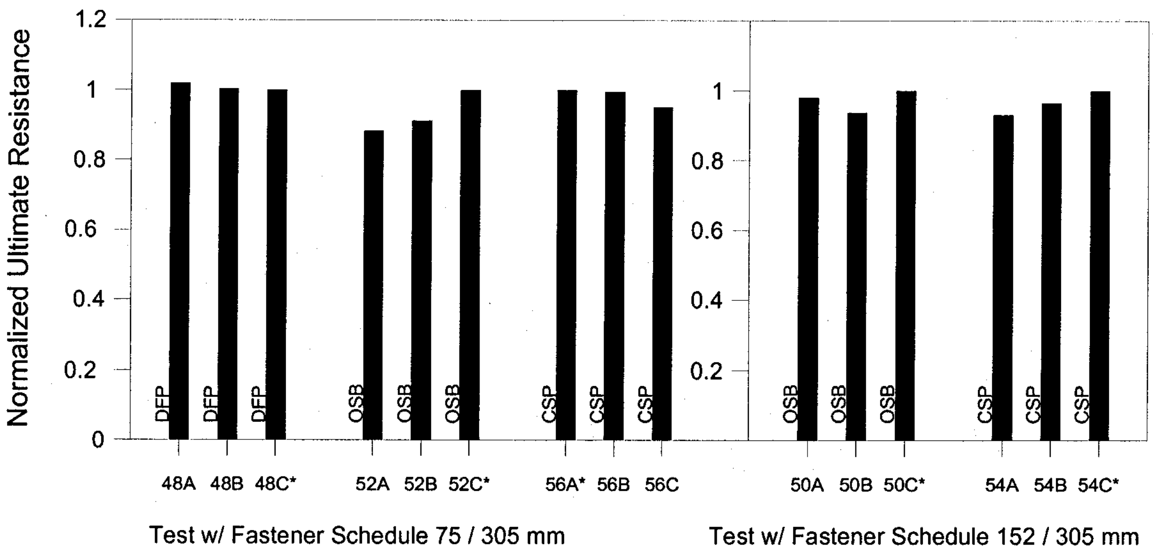

Normalized Displacement at 0.8Su of Cyclic Tests
Normalized Diplacement at 0.8Su
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
DFP
OSB
CSP
48A 48B 48C*
52A 52B 52C*
56A* 56B 56C
50A 50B 50C*
54A 54B 54C*
Test w/ Fastener Schedule 75 / 305 mm
Test w/ Fastener Schedule 152 / 305 mm

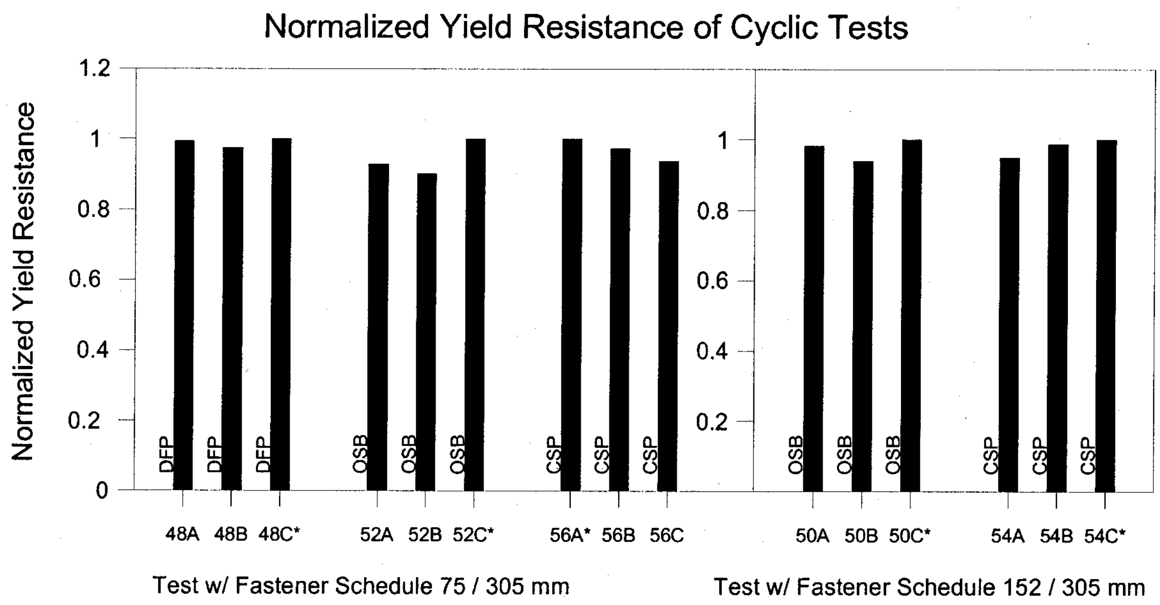

Normalized Energy of Cyclic Tests
Normalized Energy
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
DFP
OSB
CSP
48A 48B 48C*
52A 52B 52C*
56A* 56B 56C
50A 50B 50C*
54A 54B 54C*
Test w/ Fastener Schedule 75 / 305 mm
Test w/ Fastener Schedule 152 / 305 mm

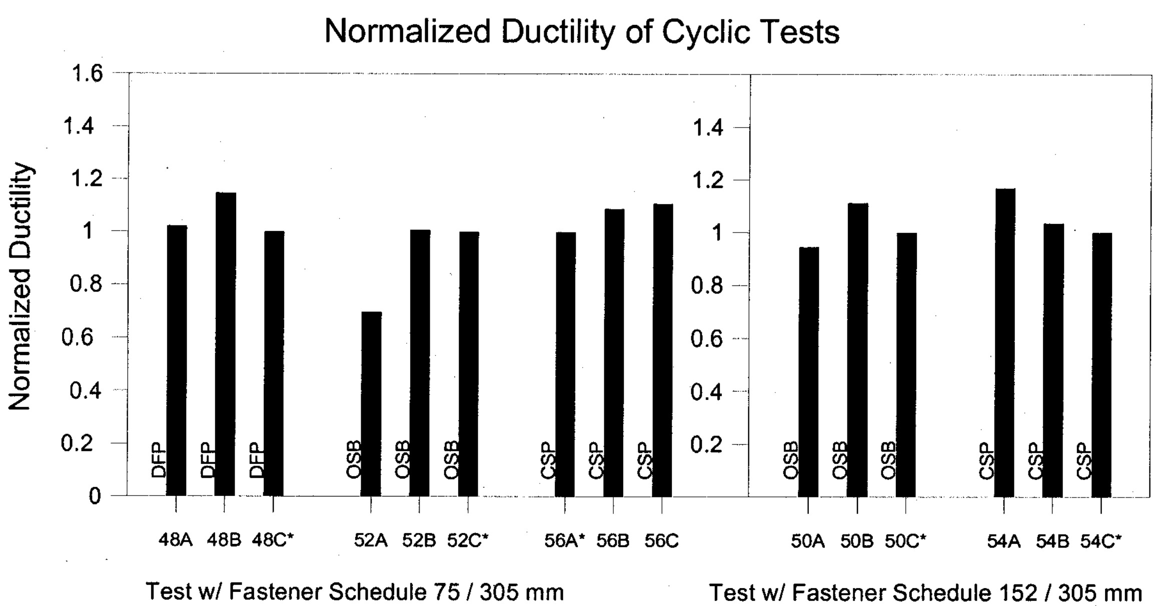
Normalized Stiffness of Cyclic Tests

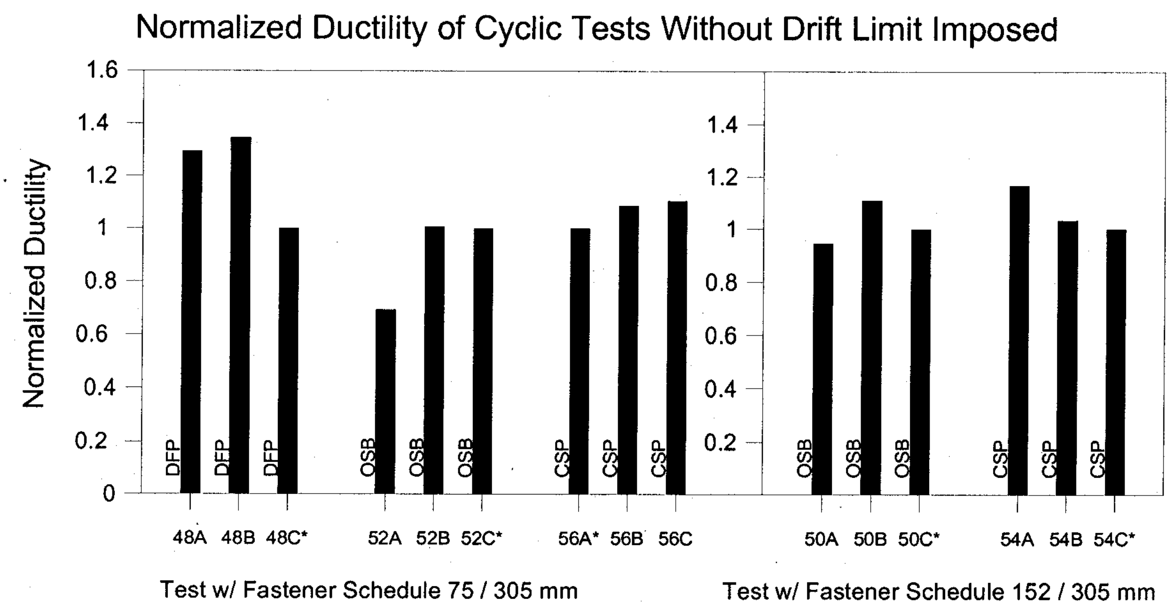


Normalized Ultimate Resistance of Cyclic Tests




Normalized Yield Resistance of Cyclic Tests
Normalized Yield Resistance
1.2
1
0.8
0.6
0.4
0.2
0
DFP
OSB
CSP
48A 48B 48C*
52A 52B 52C*
56A* 56B 56C
50A 50B 50C*
54A 54B 54C*
Test w/ Fastener Schedule 75 / 305 mm
Test w/ Fastener Schedule 152 / 305 mm


Normalized Ductility of Cyclic Tests
Normalized Ductility
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
DFP
OSB
CSP
48A 48B 48C*
52A 52B 52C*
56A* 56B 56C
50A 50B 50C*
54A 54B 54C*
Test w/ Fastener Schedule 75 / 305 mm
Test w/ Fastener Schedule 152 / 305 mm



Normalized Ductility of Cyclic Tests Without Drift Limit Imposed
Normalized Ductility
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
DFP
OSB
CSP
48A 48B 48C*
52A 52B 52C*
56A* 56B 56C
50A 50B 50C*
54A 54B 54C*
Test w/ Fastener Schedule 75 / 305 mm
Test w/ Fastener Schedule 152 / 305 mm

APPENDIX F

SAMPLE CALCULATION OF CHORD STUD CAPACITY

The axial load capacity of an end chord is the minimum of the capacity of a hold-down connection, the capacity of the back-to-back stud connections, as well as the tension and compression capacity of the stud (or studs) calculated in accordance with the *North American Specification for the Design of Cold-formed Steel Structural Members* (CSA, 2001). The composite action of the studs and wood sheathing is neglected, however, the wood sheathing is assumed to act as the lateral brace in the minor axis of the built-up chord section at an interval of *2s* for the largest spacing of 152 mm (6'). Calculations for the built-up chord section are first shown for the case when no web perforations exist, and then the case where webs are perforated. The end conditions and unbraced lengths recommended in Chapter 3 for the sheathed DCS are applied in this calculation. Furthermore, the nominal cross section size is used, but the measured material properties (thickness, strength and E) are applied.

The sizes of a chord are shown in Figure A-F-1&2, and the longitudinal screw spacing along the axis of the stud is 305 mm (12").

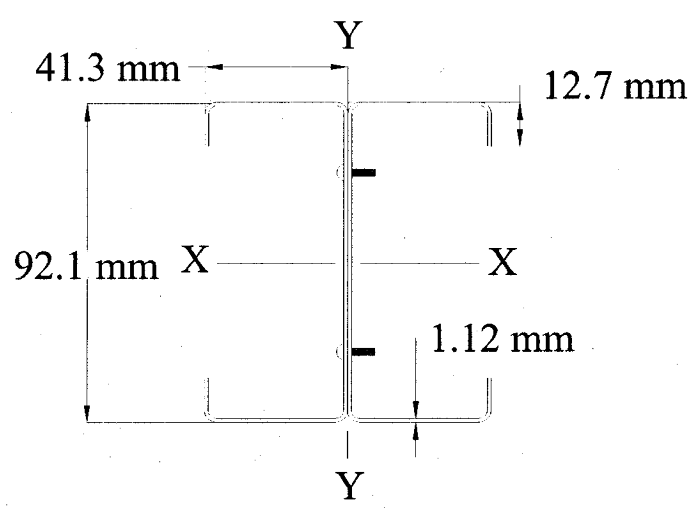


Figure A-F-1 Cross-Section of Chord Studs

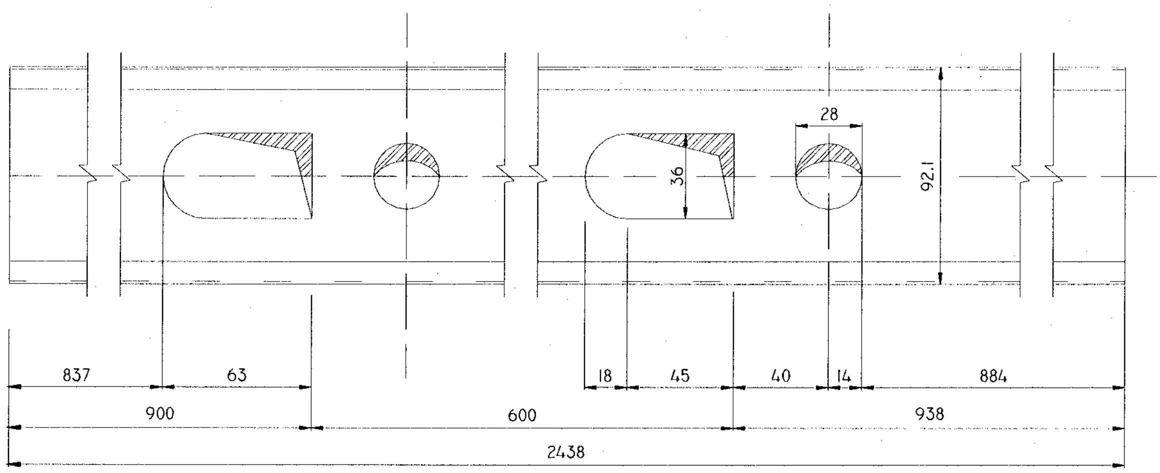


Figure A-F-2 Stud Dimensions and Hole Locations (*Chen, 2004*)

The dimensions of a chord are shown in Figure A-F-1&2. The inside bend radius of the corners is assumed to be $2 \times 1.12 = 2.24mm$ and hence $r = 2.5 \times 1.12 = 2.8mm$. The mechanical properties:

$F_y = 246MPa;\ F_u = 321MPa;\ F_u / F_y = 1.30;\ E = 220000MPa;$

$Elongation = 32.4\%$

Web slenderness ratio: $w/t = (92.1 - 2(3 \times 1.12))/1.12 = 76.23 < 500$;

Flange slenderness ratio: $w/t = (41.3 - 2(3 \times 1.12))/1.12 = 30.88 < 60$;

Lip slenderness ratio: $w/t = (12.7 - 3 \times 1.12)/1.12 = 8.34 < 60$.

$Ag = 2 \times 213.71 = 427.42mm^2$; $I_x = 2 \times 287811.5 = 575623mm^4$; $R_x = 36.07mm$

$I_y = 2 \times 91119.5 = 182239mm^4$; $R_y = 20.65mm$

Distance between centroid of single stud and web centerline is: $\bar{x} = 12.691mm$;

1) Calculate P_n **(CSA S136-01 Clause 4 (a), 4.5)**

Wall studs without perforation:

$k = 0.9$;

$k \times L_x / R_x = 0.9 \times 2438.4 / 36.7 = 59.80 < 200$;

$$r_i = \sqrt{\frac{91119.5 - 213.71 \times 13.63^2}{213.71}} = 15.51;$$

$$\frac{a}{r_i} = \frac{12 \times 25.4}{15.51} = 19.65 < 0.5 \times 59.80 = 29.90;$$

$$(kL/R)_m = \sqrt{(59.80)^2 + (19.65)^2} = 62.95;$$

for studs with connectors at 12 inch spacings

$k \times L_y / R_y = 0.9 \times 304.8 / 20.65 = 13.28 < 200$;

$$F_e = \frac{\pi^2 E}{(kL/R)_m{}^2} = \frac{\pi^2 \times 220000}{62.95^2} = 547.9MPa;$$

$$\lambda_c = \sqrt{\frac{f_y}{f_e}} = \sqrt{\frac{246}{547.9}} = 0.670 < 1.5; \quad F_n = \left(0.658^{\lambda_c^2}\right) \bullet f_y = 203.86MPa.$$

Check the effective width of the webs:

$$F_{cr} = k\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 = 4 \times \frac{\pi^2 \times 220000}{12 \times (1 - 0.3^2)} \times \left(\frac{1}{76.23}\right)^2 = 136.9MPa;$$

$$\lambda = \sqrt{\frac{f_n}{F_{cr}}} = \sqrt{\frac{203.86}{136.9}} = 1.220 > 0.673; \; \rho = \frac{(1 - 0.22/1.220)}{1.220} = 0.672;$$

$b = pw = 0.672 \times 85.38 = 57.3mm$;

Check the effective width of the flanges:

$$S = 1.28\sqrt{\frac{E}{f_n}} = 1.28\sqrt{\frac{220000}{203.86}} = 42.0; \; w/t = 30.88 > 0.328S = 13.8;$$

$$I_a = 399 \times 1.12^4 \times \left[\frac{30.9}{42.0} - 0.328\right]^3 = 42.55mm^4 \ldots$$

$$< 1.12^4 \times \left[115 \times \frac{30.9}{42.0} + 5\right] = 141.0mm^4;$$

$$n = \left[0.582 - \frac{30.9}{4 \times 42.0}\right] = 0.40 > \frac{1}{3};$$

$$I_s = \frac{1.12 \times 9.34^3}{12} = 76.0mm^4; \quad R_I = \frac{I_s}{I_a} = \frac{76.0}{42.6} = 1.78;$$

$D/w = 12.7/34.58 = 0.367 > 0.25$ and < 0.8;

$$k = (4.82 - \frac{5 \times 12.7}{34.58})(1.0)^{0.402} + 0.43 = 3.41 < 4;$$

$$F_{cr} = k\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 = 3.41 \times \frac{\pi^2 \times 220000}{12 \times (1 - 0.3^2)} \times \left(\frac{1}{30.9}\right)^2 = 710.1MPa$$

$$\lambda = \sqrt{\frac{f_n}{F_{cr}}} = \sqrt{\frac{203.86}{710.1}} = 0.536 < 0.673; \; b = w = 34.58mm;$$

Check the effective width of the lips:

$$F_{cr} = k\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 = 3.41 \times \frac{\pi^2 \times 220000}{12 \times (1 - 0.3^2)} \times \left(\frac{1}{8.34}\right)^2 = 1229.2MPa$$

$$\lambda = \sqrt{\frac{f_n}{F_{cr}}} = \sqrt{\frac{203.86}{1229.2}} = 0.407 < 0.673; \; ds' = w = 9.34mm$$

$$ds = 1.0 \times 9.34 = 9.34mm.$$

$$A_e = 427.42 - 2 \times 1.12 \times (85.38 - 57.3) - 4 \times 1.12 \times (34.58 - 34.58) - 4 \times 1.12 \times (9.34 - 9.34) = 3$$

$$P_n = A_e - F_n = 364.5 \times 203.86 / 1000 = 74.3kN$$

Wall studs with perforation:

The requirement of Clause D4(a)-(1) ~ (5) are all satisfied in this case.

Check the effective width of the webs:

$$w = (92.1 - 36)/2 - 3 \times 1.12 = 24.69mm$$

$$F_{cr} = k\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 = 0.43 \times \frac{\pi^2 \times 220000}{12 \times (1-0.3^2)} \times \left(\frac{1.12}{24.69}\right)^2 = 175.9MPa$$

$$\lambda = \sqrt{\frac{f_n}{F_{cr}}} = \sqrt{\frac{203.86}{175.9}} = 1.08 > 0.673\,;\ \rho = \frac{(1-0.22/1.08)}{1.08} = 0.737\,;$$

$$b = pw = 0.737 \times 24.69 = 18.2mm$$

Check the effective width of the flanges:

Same as previous calculation, b = 34.58 mm.

Check the effective width of the lips:

Same as previous calculation, ds = 9.34 mm.

Net area $= 427.42 - 2 \times (36 \times 1.12) = 346.8mm^2$

Net effective area (A_e)...

$$= 346.8 - 4 \times 1.12 \times (24.69 - 18.2) - 4 \times 1.12 \times (34.58 - 34.58) - 4 \times 1.12 \times (9.34 - 9.34) = 317.2n$$

$$P_n = A_e \times F_n = (317.2 \times 203.86)/1000 = 64.7kN$$

The maximum shear load at the bottom of the studs with the perforation at 837 mm (2.75') from the bottom can be determined using similar triangles:

$$\left(\frac{64.7}{1601.4}\right) = \left(\frac{Max.Load}{2438.4}\right) \rightarrow Max.Load = \left(\frac{64.7 \times 2438.4}{1601.4}\right) = 98.5kN.$$

The capacity of the full section controls the maximum shear load on the DCS.

With the incorporation of gravity loads the maximum allowable shear load is reduced. Therefore the maximum allowable shear load a 1220 x 2440 mm (4' x 8') shear wall system is able to resist with a 14.8 kN/m distributed gravity load imposed is:

For 4 feet walls:

$$S = (74.3 - ((1.220/2) \times 14.8)) \times \left(\frac{1.22}{2.4384} \right) \times (1/1.22) = 26.8 kN/m$$

Therefore the maximum allowable shear load resistance of the a 1220 x 2440 mm (4' x 8') shear wall constructed with this particular set of cold-formed studs was 26.8 kN/m when combined with a 14.8 kN/m distributed load.

In this thesis, gravity loads were applied to most shear walls, so the axial loads on the double chord studs (DCS) were produced by the sheathing connections and the gravity loads. The shear flow along the screw lines on the chord studs cause the axial forces to increase in a triangular fashion, with the maximum force at the bottom of the end studs and zero at the top (Figure A-F-3). The axial loads created by the gravity loads are constant along the DCS. The distance from the edge of the bottom hole in the web of the stud to the lower end of the chord measured 837 mm. Assuming that the chord stud will fail when the force at the hole location reaches 64.7 kN, it can be hypothesised that the maximum load at the bottom of the studs would be 98.5 kN (Figure A-F-3) under this lateral loading scenario. However, the full section capacity is not able to reach a load of 98.5 kN, so the full section controls the allowable resistance with a capacity of 74.3kN.

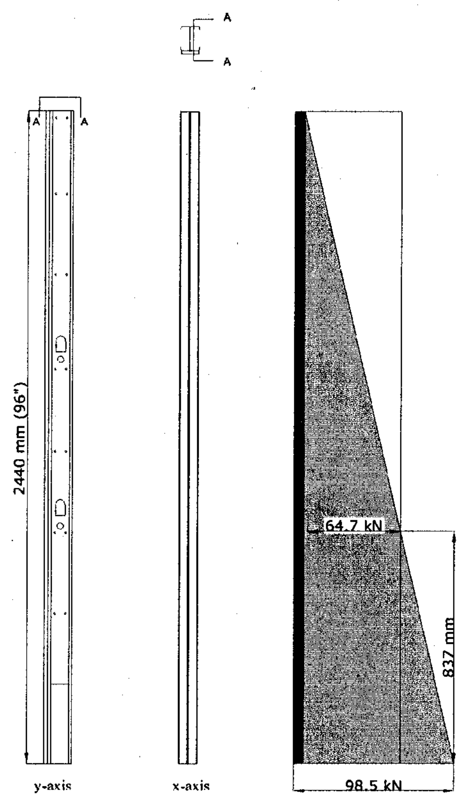


Figure A-F-3 Axial Force Diagram of an End Stud

Tension capacity of the ends:

For sections full sections:

$T_{studs} = A_g \times f_y = 427.42 \times 246/1000 = 105.1kN$

For sections with holes:

$T_{studs} = A_n \times f_y = (427.42 - 2 \times 1.12 \times 36) \times 246/1000 = 85.3kN$

2) Capacity of a hold-down: (This value is from the manufacture's website *Simpson-Strongtie*.)

$T_{hd} = 129kN$

3) Capacity of the stud connections for studs at panel joints:

(The shear value for single screw is from the manufacturer's website *Grabber*)

$V = 9 \times 2 \times 1206 \times 4.44822 / 1000 = 96.6kN$.

Conclusion:

Failure of a chord stud will occur when either the bottom of the stud reaches the full capacity (without holes) 74.3 kN, or when the force at the bottom hole location reaches 64.7 kN (corresponding force at the bottom end of the stud is 98.5 kN). From this, it is assumed that the full section of the DCS controls the axial capacity. However, the bottom of the stud is reinforced by the hold-down connector, and hence the chord's true capacity, although difficult to determine, is certainly higher than 74.3 kN. In this case it is plausible that the force at the chord stud end could reach 95.3 kN, assuming a triangular axial force distribution, and then failure could take place at the perforated section.

In summary, the capacity of the chord stud using the all steel method is 64.7 kN at the perforation section and 74.3 kN for the full-section. The effective length factors $K_x = K_y = 0.9$ and $K_t = 0.65$ and buckling length $L_x = 2.4384m$ $L_y = L_t = 0.152m$ are drawn from recommendations made in Chapter 3. These material properties and measurements are based on the ancillary testing of this study to make a comparison between the nominal and actual strengths of the member. Also, these reported values are intended to be used for capacity based design and therefore do not include resistance factors. In practical design, resistance factors must be incorporated into the determination of the factored resistance with nominal values, but for capacity based design unfactored resistances are used with nominal values.

APPENDIX G

STATISTICAL RESULTS OF CHORD STUD CAPACITY EVALUATION METHODS

Table G-1 : Comparison of predicted ultimate load based on nominal and actual properties using Method 1 with experimental loads

Test	Ultimate load (kN)			Expt/Pred(Nominal)	Expt/Pred(Measurec
	Predicted		Experimental		
	Nominal Properties	Measured Properties			
1. 043OSB1-12-3A	64.8	71.8	94.3	1.45	1.31
2. 043OSB1-12-3B	64.8	71.8	83.0	1.28	1.16
3. 043OSB2-12-3A	64.8	71.8	91.5	1.41	1.27
4. 043OSB2-12-3B	64.8	71.8	83.3	1.29	1.16
5. 043OSB1-24-3A	64.8	71.8	82.0	1.27	1.14
6. 043OSB1-24-3B	64.8	71.8	84.2	1.30	1.17
7. 043OSB2-24-3A	64.8	71.8	78.4	1.21	1.09
8. 043OSB2-24-3B	64.8	71.8	84.0	1.30	1.17
9. 043OSB1-12-6A[1]	64.8	71.8	80.8	1.25	1.13
10. 043OSB1-12-6B	64.8	71.8	74.0	1.14	1.03
11. 043OSB2-12-6A	64.8	71.8	78.1	1.20	1.09
12. 043OSB2-12-6B	64.8	71.8	77.2	1.19	1.08
13. 043OSB1-24-6A	64.8	71.8	85.8	1.32	1.19
14. 043OSB1-24-6B	64.8	71.8	70.4	1.09	0.98
15. 043OSB2-24-6A	64.8	71.8	85.6	1.32	1.19
16. 043OSB2-24-6B	64.8	71.8	85.7	1.32	1.19
17. 033OSB1-12-3A	45.7	55.7	60.3	1.32	1.08
18. 033OSB1-12-3B	45.7	55.7	62.4	1.37	1.12
19. 033OSB1-12-6A	45.7	55.7	62.6	1.37	1.12
20. 033OSB1-12-6B	45.7	55.7	62.3	1.36	1.12
21. 043CSP1-12-3A	64.8	71.8	82.6	1.27	1.15
22. 043CSP1-12-3B	64.8	71.8	89.8	1.39	1.25
23. 043CSP2-12-3A	64.8	71.8	84.9	1.31	1.18
24. 043CSP2-12-3B	64.8	71.8	80.3	1.24	1.12
25. 043CSP1-12-6A	64.8	71.8	83.9	1.29	1.17
26. 043CSP1-12-6B	64.8	71.8	80.4	1.24	1.12
27. 043CSP2-12-6A	64.8	71.8	78.7	1.21	1.10
28. 043CSP2-12-6B	64.8	71.8	91.0	1.40	1.27
29. 054OSB1-12-3A	109.4	104.4	125.0	1.14	1.20
30. 054OSB1-12-3B	109.4	104.4	125.0	1.14	1.20
31. 054CSP1-12-3A	109.4	104.4	119.0	1.09	1.14
32. 054CSP1-12-3B	109.4	104.4	114.3	1.05	1.10
33. 068OSB1-12-3A	144.9	145.4	173.0	1.19	1.19
34. 068OSB1-12-3B	144.9	145.4	179.2	1.24	1.23
35. 068CSP1-12-3A	144.9	145.4	172.3	1.19	1.19
36. 068CSP1-12-3B	144.9	145.4	183.0	1.26	1.26
			Average	1.26	1.16
			Standard Deviation	0.0964	0.0696
			CoV	0.0764	0.0602

[1] Two extra fastners in web-to-web connection at 19 mm (0.75") from the top

Table G-2 : Comparison of predicted ultimate load based on nominal and actual properties using Method 2 with experimental loads

Test	Ultimate load (kN)			Expt/Pred(Nominal)	Expt/Pred(Measurec
	Predicted		Experimental		
	Nominal Properties	Measured Properties			
1. 043OSB1-12-3A	67.1	74.3	94.3	1.41	1.27
2. 043OSB1-12-3B	67.1	74.3	83.0	1.24	1.12
3. 043OSB2-12-3A	67.1	74.3	91.5	1.36	1.23
4. 043OSB2-12-3B	67.1	74.3	83.3	1.24	1.12
5. 043OSB1-24-3A	67.1	74.3	82.0	1.22	1.10
6. 043OSB1-24-3B	67.1	74.3	84.2	1.26	1.13
7. 043OSB2-24-3A	67.1	74.3	78.4	1.17	1.05
8. 043OSB2-24-3B	67.1	74.3	84.0	1.25	1.13
9. 043OSB1-12-6A[1]	67.1	74.3	80.8	1.20	1.09
10. 043OSB1-12-6B	67.1	74.3	74.0	1.10	1.00
11. 043OSB2-12-6A	67.1	74.3	78.1	1.16	1.05
12. 043OSB2-12-6B	67.1	74.3	77.2	1.15	1.04
13. 043OSB1-24-6A	67.1	74.3	85.8	1.28	1.15
14. 043OSB1-24-6B	67.1	74.3	70.4	1.05	0.95
15. 043OSB2-24-6A	67.1	74.3	85.6	1.28	1.15
16. 043OSB2-24-6B	67.1	74.3	85.7	1.28	1.15
17. 033OSB1-12-3A	47.1	57.6	60.3	1.28	1.05
18. 033OSB1-12-3B	47.1	57.6	62.4	1.33	1.08
19. 033OSB1-12-6A	47.1	57.6	62.6	1.33	1.09
20. 033OSB1-12-6B	47.1	57.6	62.3	1.32	1.08
21. 043CSP1-12-3A	67.1	74.3	82.6	1.23	1.11
22. 043CSP1-12-3B	67.1	74.3	89.8	1.34	1.21
23. 043CSP2-12-3A	67.1	74.3	84.9	1.27	1.14
24. 043CSP2-12-3B	67.1	74.3	80.3	1.20	1.08
25. 043CSP1-12-6A	67.1	74.3	83.9	1.25	1.13
26. 043CSP1-12-6B	67.1	74.3	80.4	1.20	1.08
27. 043CSP2-12-6A	67.1	74.3	78.7	1.17	1.06
28. 043CSP2-12-6B	67.1	74.3	91.0	1.36	1.22
29. 054OSB1-12-3A	115.2	109.0	125.0	1.08	1.15
30. 054OSB1-12-3B	115.2	109.0	125.0	1.08	1.15
31. 054CSP1-12-3A	115.2	109.0	119.0	1.03	1.09
32. 054CSP1-12-3B	115.2	109.0	114.3	0.99	1.05
33. 068OSB1-12-3A	152.8	152.0	173.0	1.13	1.14
34. 068OSB1-12-3B	152.8	152.0	179.2	1.17	1.18
35. 068CSP1-12-3A	152.8	152.0	172.3	1.13	1.13
36. 068CSP1-12-3B	152.8	152.0	183.0	1.20	1.20
			Average	1.22	1.12
			Standard Deviation	0.099	0.0662
			CoV	0.0811	0.0593

[1] Two extra fastners in web-to-web connection at 19 mm (0.75") from the tor

Table G-3 : Comparison of predicted ultimate load based on nominal and actual properties using Method 3 with experimental loads

Test	Ultimate load (kN)			Expt/Pred(Nominal)	Expt/Pred(Measured
	Predicted		Experimental		
	Nominal Properties	Measured Properties			
1. 043OSB1-12-3A	69.2	76.7	94.3	1.36	1.23
2. 043OSB1-12-3B	69.2	76.7	83.0	1.20	1.08
3. 043OSB2-12-3A	69.2	76.7	91.5	1.32	1.19
4. 043OSB2-12-3B	69.2	76.7	83.3	1.20	1.09
5. 043OSB1-24-3A	69.2	76.7	82.0	1.18	1.07
6. 043OSB1-24-3B	69.2	76.7	84.2	1.22	1.10
7. 043OSB2-24-3A	69.2	76.7	78.4	1.13	1.02
8. 043OSB2-24-3B	69.2	76.7	84.0	1.21	1.10
9. 043OSB1-12-6A[1]	69.2	76.7	80.8	1.17	1.05
10. 043OSB1-12-6B	69.2	76.7	74.0	1.07	0.97
11. 043OSB2-12-6A	69.2	76.7	78.1	1.13	1.02
12. 043OSB2-12-6B	69.2	76.7	77.2	1.12	1.01
13. 043OSB1-24-6A	69.2	76.7	85.8	1.24	1.12
14. 043OSB1-24-6B	69.2	76.7	70.4	1.02	0.92
15. 043OSB2-24-6A	69.2	76.7	85.6	1.24	1.12
16. 043OSB2-24-6B	69.2	76.7	85.7	1.24	1.12
17. 033OSB1-12-3A	48.3	59.4	60.3	1.25	1.01
18. 033OSB1-12-3B	48.3	59.4	62.4	1.29	1.05
19. 033OSB1-12-6A	48.3	59.4	62.6	1.30	1.05
20. 033OSB1-12-6B	48.3	59.4	62.3	1.29	1.05
21. 043CSP1-12-3A	69.2	76.7	82.6	1.19	1.08
22. 043CSP1-12-3B	69.2	76.7	89.8	1.30	1.17
23. 043CSP2-12-3A	69.2	76.7	84.9	1.23	1.11
24. 043CSP2-12-3B	69.2	76.7	80.3	1.16	1.05
25. 043CSP1-12-6A	69.2	76.7	83.9	1.21	1.09
26. 043CSP1-12-6B	69.2	76.7	80.4	1.16	1.05
27. 043CSP2-12-6A	69.2	76.7	78.7	1.14	1.03
28. 043CSP2-12-6B	69.2	76.7	91.0	1.31	1.19
29. 054OSB1-12-3A	120.7	113.3	125.0	1.04	1.10
30. 054OSB1-12-3B	120.7	113.3	125.0	1.04	1.10
31. 054CSP1-12-3A	120.7	113.3	119.0	0.99	1.05
32. 054CSP1-12-3B	120.7	113.3	114.3	0.95	1.01
33. 068OSB1-12-3A	160.1	158.1	173.0	1.08	1.09
34. 068OSB1-12-3B	160.1	158.1	179.2	1.12	1.13
35. 068CSP1-12-3A	160.1	158.1	172.3	1.08	1.09
36. 068CSP1-12-3B	160.1	158.1	183.0	1.14	1.16
			Average	1.17	1.08
			Standard Deviation	0.101	0.0635
			CoV	0.0856	0.0588

[1] Two extra fastners in web-to-web connection at 19 mm (0.75") from the top

Table G-4 : Comparison of predicted ultimate load based on nominal and actual properties using Method 4 with experimental loads

Test	Ultimate load (kN)			Expt/Pred(Nominal)	Expt/Pred(Measurec
	Predicted		Experimental		
	Nominal Properties	Measured Properties			
1. 043OSB1-12-3A	61.0	67.6	94.3	1.55	1.39
2. 043OSB1-12-3B	61.0	67.6	83.0	1.36	1.23
3. 043OSB2-12-3A	61.0	67.6	91.5	1.50	1.35
4. 043OSB2-12-3B	61.0	67.6	83.3	1.37	1.23
5. 043OSB1-24-3A	61.0	67.6	82.0	1.34	1.21
6. 043OSB1-24-3B	61.0	67.6	84.2	1.38	1.25
7. 043OSB2-24-3A	61.0	67.6	78.4	1.29	1.16
8. 043OSB2-24-3B	61.0	67.6	84.0	1.38	1.24
9. 043OSB1-12-6A[1]	61.0	67.6	80.8	1.33	1.20
10. 043OSB1-12-6B	61.0	67.6	74.0	1.21	1.09
11. 043OSB2-12-6A	61.0	67.6	78.1	1.28	1.15
12. 043OSB2-12-6B	61.0	67.6	77.2	1.27	1.14
13. 043OSB1-24-6A	61.0	67.6	85.8	1.41	1.27
14. 043OSB1-24-6B	61.0	67.6	70.4	1.16	1.04
15. 043OSB2-24-6A	61.0	67.6	85.6	1.40	1.27
16. 043OSB2-24-6B	61.0	67.6	85.7	1.41	1.27
17. 033OSB1-12-3A	43.4	52.5	60.3	1.39	1.15
18. 033OSB1-12-3B	43.4	52.5	62.4	1.44	1.19
19. 033OSB1-12-6A	43.4	52.5	62.6	1.44	1.19
20. 033OSB1-12-6B	43.4	52.5	62.3	1.44	1.19
21. 043CSP1-12-3A	61.0	67.6	82.6	1.35	1.22
22. 043CSP1-12-3B	61.0	67.6	89.8	1.47	1.33
23. 043CSP2-12-3A	61.0	67.6	84.9	1.39	1.26
24. 043CSP2-12-3B	61.0	67.6	80.3	1.32	1.19
25. 043CSP1-12-6A	61.0	67.6	83.9	1.38	1.24
26. 043CSP1-12-6B	61.0	67.6	80.4	1.32	1.19
27. 043CSP2-12-6A	61.0	67.6	78.7	1.29	1.16
28. 043CSP2-12-6B	61.0	67.6	91.0	1.49	1.35
29. 054OSB1-12-3A	99.7	96.6	125.0	1.25	1.29
30. 054OSB1-12-3B	99.7	96.6	125.0	1.25	1.29
31. 054CSP1-12-3A	99.7	96.6	119.0	1.19	1.23
32. 054CSP1-12-3B	99.7	96.6	114.3	1.15	1.18
33. 068OSB1-12-3A	131.5	133.9	173.0	1.32	1.29
34. 068OSB1-12-3B	131.5	133.9	179.2	1.36	1.34
35. 068CSP1-12-3A	131.5	133.9	172.3	1.31	1.29
36. 068CSP1-12-3B	131.5	133.9	183.0	1.39	1.37
			Average	1.35	1.23
			Standard Deviation	0.093	0.0776
			CoV	0.0690	0.0628

[1] Two extra fastners in web-to-web connection at 19 mm (0.75") from the top

Table G-5 : Comparison of predicted ultimate load based on nominal and actual properties using Method 5 with experimental loads

Test	Ultimate load (kN)			Expt/Pred(Nominal)	Expt/Pred(Measurec
	Predicted		Experimental		
	Nominal Properties	Measured Properties			
1. 043OSB1-12-3A	67.3	74.6	94.3	1.40	1.26
2. 043OSB1-12-3B	67.3	74.6	83.0	1.23	1.11
3. 043OSB2-12-3A	67.3	74.6	91.5	1.36	1.23
4. 043OSB2-12-3B	67.3	74.6	83.3	1.24	1.12
5. 043OSB1-24-3A	67.3	74.6	82.0	1.22	1.10
6. 043OSB1-24-3B	67.3	74.6	84.2	1.25	1.13
7. 043OSB2-24-3A	67.3	74.6	78.4	1.16	1.05
8. 043OSB2-24-3B	67.3	74.6	84.0	1.25	1.13
9. 043OSB1-12-6A[1]	67.3	74.6	80.8	1.20	1.08
10. 043OSB1-12-6B	67.3	74.6	74.0	1.10	0.99
11. 043OSB2-12-6A	67.3	74.6	78.1	1.16	1.05
12. 043OSB2-12-6B	67.3	74.6	77.2	1.15	1.03
13. 043OSB1-24-6A	67.3	74.6	85.8	1.27	1.15
14. 043OSB1-24-6B	67.3	74.6	70.4	1.05	0.94
15. 043OSB2-24-6A	67.3	74.6	85.6	1.27	1.15
16. 043OSB2-24-6B	67.3	74.6	85.7	1.27	1.15
17. 033OSB1-12-3A	47.2	57.9	60.3	1.28	1.04
18. 033OSB1-12-3B	47.2	57.9	62.4	1.32	1.08
19. 033OSB1-12-6A	47.2	57.9	62.6	1.33	1.08
20. 033OSB1-12-6B	47.2	57.9	62.3	1.32	1.08
21. 043CSP1-12-3A	67.3	74.6	82.6	1.23	1.11
22. 043CSP1-12-3B	67.3	74.6	89.8	1.33	1.20
23. 043CSP2-12-3A	67.3	74.6	84.9	1.26	1.14
24. 043CSP2-12-3B	67.3	74.6	80.3	1.19	1.08
25. 043CSP1-12-6A	67.3	74.6	83.9	1.25	1.12
26. 043CSP1-12-6B	67.3	74.6	80.4	1.19	1.08
27. 043CSP2-12-6A	67.3	74.6	78.7	1.17	1.05
28. 043CSP2-12-6B	67.3	74.6	91.0	1.35	1.22
29. 054OSB1-12-3A	115.8	109.4	125.0	1.08	1.14
30. 054OSB1-12-3B	115.8	109.4	125.0	1.08	1.14
31. 054CSP1-12-3A	115.8	109.4	119.0	1.03	1.09
32. 054CSP1-12-3B	115.8	109.4	114.3	0.99	1.04
33. 068OSB1-12-3A	153.3	152.4	173.0	1.13	1.14
34. 068OSB1-12-3B	153.3	152.4	179.2	1.17	1.18
35. 068CSP1-12-3A	153.3	152.4	172.3	1.12	1.13
36. 068CSP1-12-3B	153.3	152.4	183.0	1.19	1.20
			Average	1.21	1.11
			Standard Deviation	0.098	0.0661
			CoV	0.0813	0.0595

[1] Two extra fastners in web-to-web connection at 19 mm (0.75") from the top

Table G-6 : Comparison of predicted ultimate load based on nominal and actual properties using Method 6 with experimental loads

Test	Ultimate load (kN)			Expt/Pred(Nominal)	Expt/Pred(Measurec
	Predicted		Experimental		
	Nominal Properties	Measured Properties			
1. 043OSB1-12-3A	56.2	62.4	94.3	1.68	1.51
2. 043OSB1-12-3B	56.2	62.4	83.0	1.48	1.33
3. 043OSB2-12-3A	56.2	62.4	91.5	1.63	1.47
4. 043OSB2-12-3B	56.2	62.4	83.3	1.48	1.33
5. 043OSB1-24-3A	56.2	62.4	82.0	1.46	1.31
6. 043OSB1-24-3B	56.2	62.4	84.2	1.50	1.35
7. 043OSB2-24-3A	56.2	62.4	78.4	1.39	1.26
8. 043OSB2-24-3B	56.2	62.4	84.0	1.49	1.34
9. 043OSB1-12-6A[1]	56.2	62.4	80.8	1.44	1.29
10. 043OSB1-12-6B	56.2	62.4	74.0	1.32	1.19
11. 043OSB2-12-6A	56.2	62.4	78.1	1.39	1.25
12. 043OSB2-12-6B	56.2	62.4	77.2	1.37	1.24
13. 043OSB1-24-6A	56.2	62.4	85.8	1.53	1.37
14. 043OSB1-24-6B	56.2	62.4	70.4	1.25	1.13
15. 043OSB2-24-6A	56.2	62.4	85.6	1.52	1.37
16. 043OSB2-24-6B	56.2	62.4	85.7	1.52	1.37
17. 033OSB1-12-3A	40.5	48.0	60.3	1.49	1.26
18. 033OSB1-12-3B	40.5	48.0	62.4	1.54	1.30
19. 033OSB1-12-6A	40.5	48.0	62.6	1.54	1.30
20. 033OSB1-12-6B	40.5	48.0	62.3	1.54	1.30
21. 043CSP1-12-3A	56.2	62.4	82.6	1.47	1.32
22. 043CSP1-12-3B	56.2	62.4	89.8	1.60	1.44
23. 043CSP2-12-3A	56.2	62.4	84.9	1.51	1.36
24. 043CSP2-12-3B	56.2	62.4	80.3	1.43	1.29
25. 043CSP1-12-6A	56.2	62.4	83.9	1.49	1.34
26. 043CSP1-12-6B	56.2	62.4	80.4	1.43	1.29
27. 043CSP2-12-6A	56.2	62.4	78.7	1.40	1.26
28. 043CSP2-12-6B	56.2	62.4	91.0	1.62	1.46
29. 054OSB1-12-3A	88.3	87.3	125.0	1.41	1.43
30. 054OSB1-12-3B	88.3	87.3	125.0	1.41	1.43
31. 054CSP1-12-3A	88.3	87.3	119.0	1.35	1.36
32. 054CSP1-12-3B	88.3	87.3	114.3	1.29	1.31
33. 068OSB1-12-3A	116.1	120.5	173.0	1.49	1.43
34. 068OSB1-12-3B	116.1	120.5	179.2	1.54	1.49
35. 068CSP1-12-3A	116.1	120.5	172.3	1.48	1.43
36. 068CSP1-12-3B	116.1	120.5	183.0	1.58	1.52
			Average	1.47	1.35
			Standard Deviation	0.093	0.0896
			CoV	0.0633	0.0666

[1] Two extra fastners in web-to-web connection at 19 mm (0.75") from the tor

Table G-7 : Comparison of predicted ultimate load based on nominal and actual properties using Method 7 with experimental loads

Test	Ultimate load (kN) Predicted Nominal Properties	Ultimate load (kN) Predicted Measured Properties	Ultimate load (kN) Experimental	Expt/Pred(Nominal)	Expt/Pred(Measured
1. 043OSB1-12-3A	64.8	71.8	94.3	1.45	1.31
2. 043OSB1-12-3B	64.8	71.8	83.0	1.28	1.16
3. 043OSB2-12-3A	64.8	71.8	91.5	1.41	1.27
4. 043OSB2-12-3B	64.8	71.8	83.3	1.29	1.16
5. 043OSB1-24-3A	64.8	71.8	82.0	1.27	1.14
6. 043OSB1-24-3B	64.8	71.8	84.2	1.30	1.17
7. 043OSB2-24-3A	64.8	71.8	78.4	1.21	1.09
8. 043OSB2-24-3B	64.8	71.8	84.0	1.30	1.17
9. 043OSB1-12-6A[1]	64.8	71.8	80.8	1.25	1.13
10. 043OSB1-12-6B	64.8	71.8	74.0	1.14	1.03
11. 043OSB2-12-6A	64.8	71.8	78.1	1.20	1.09
12. 043OSB2-12-6B	64.8	71.8	77.2	1.19	1.08
13. 043OSB1-24-6A	64.8	71.8	85.8	1.32	1.19
14. 043OSB1-24-6B	64.8	71.8	70.4	1.09	0.98
15. 043OSB2-24-6A	64.8	71.8	85.6	1.32	1.19
16. 043OSB2-24-6B	64.8	71.8	85.7	1.32	1.19
17. 033OSB1-12-3A	45.7	55.7	60.3	1.32	1.08
18. 033OSB1-12-3B	45.7	55.7	62.4	1.37	1.12
19. 033OSB1-12-6A	45.7	55.7	62.6	1.37	1.12
20. 033OSB1-12-6B	45.7	55.7	62.3	1.36	1.12
21. 043CSP1-12-3A	64.8	71.8	82.6	1.27	1.15
22. 043CSP1-12-3B	64.8	71.8	89.8	1.39	1.25
23. 043CSP2-12-3A	64.8	71.8	84.9	1.31	1.18
24. 043CSP2-12-3B	64.8	71.8	80.3	1.24	1.12
25. 043CSP1-12-6A	64.8	71.8	83.9	1.29	1.17
26. 043CSP1-12-6B	64.8	71.8	80.4	1.24	1.12
27. 043CSP2-12-6A	64.8	71.8	78.7	1.21	1.10
28. 043CSP2-12-6B	64.8	71.8	91.0	1.40	1.27
29. 054OSB1-12-3A	109.4	104.4	125.0	1.14	1.20
30. 054OSB1-12-3B	109.4	104.4	125.0	1.14	1.20
31. 054CSP1-12-3A	109.4	104.4	119.0	1.09	1.14
32. 054CSP1-12-3B	109.4	104.4	114.3	1.05	1.10
33. 068OSB1-12-3A	144.9	145.4	173.0	1.19	1.19
34. 068OSB1-12-3B	144.9	145.4	179.2	1.24	1.23
35. 068CSP1-12-3A	144.9	145.4	172.3	1.19	1.19
36. 068CSP1-12-3B	144.9	145.4	183.0	1.26	1.26
			Average	1.26	1.16
			Standard Deviation	0.096	0.0696
			CoV	0.0764	0.0602

[1] Two extra fastners in web-to-web connection at 19 mm (0.75") from the top

Table G-8 : Comparison of predicted ultimate load based on nominal and actual properties using Method 8 with experimental loads

Test	Ultimate load (kN)			Expt/Pred(Nominal)	Expt/Pred(Measurec
	Predicted		Experimental		
	Nominal Properties	Measured Properties			
1. 043OSB1-12-3A	68.2	75.5	94.3	1.38	1.25
2. 043OSB1-12-3B	68.2	75.5	83.0	1.22	1.10
3. 043OSB2-12-3A	68.2	75.5	91.5	1.34	1.21
4. 043OSB2-12-3B	68.2	75.5	83.3	1.22	1.10
5. 043OSB1-24-3A	68.2	75.5	82.0	1.20	1.09
6. 043OSB1-24-3B	68.2	75.5	84.2	1.24	1.12
7. 043OSB2-24-3A	68.2	75.5	78.4	1.15	1.04
8. 043OSB2-24-3B	68.2	75.5	84.0	1.23	1.11
9. 043OSB1-12-6A[1]	68.2	75.5	80.8	1.19	1.07
10. 043OSB1-12-6B	68.2	75.5	74.0	1.09	0.98
11. 043OSB2-12-6A	68.2	75.5	78.1	1.15	1.03
12. 043OSB2-12-6B	68.2	75.5	77.2	1.13	1.02
13. 043OSB1-24-6A	68.2	75.5	85.8	1.26	1.14
14. 043OSB1-24-6B	68.2	75.5	70.4	1.03	0.93
15. 043OSB2-24-6A	68.2	75.5	85.6	1.26	1.13
16. 043OSB2-24-6B	68.2	75.5	85.7	1.26	1.13
17. 033OSB1-12-3A	47.1	57.6	60.3	1.28	1.05
18. 033OSB1-12-3B	47.1	57.6	62.4	1.33	1.08
19. 033OSB1-12-6A	47.1	57.6	62.6	1.33	1.09
20. 033OSB1-12-6B	47.1	57.6	62.3	1.32	1.08
21. 043CSP1-12-3A	68.2	75.5	82.6	1.21	1.09
22. 043CSP1-12-3B	68.2	75.5	89.8	1.32	1.19
23. 043CSP2-12-3A	68.2	75.5	84.9	1.25	1.12
24. 043CSP2-12-3B	68.2	75.5	80.3	1.18	1.06
25. 043CSP1-12-6A	68.2	75.5	83.9	1.23	1.11
26. 043CSP1-12-6B	68.2	75.5	80.4	1.18	1.06
27. 043CSP2-12-6A	68.2	75.5	78.7	1.15	1.04
28. 043CSP2-12-6B	68.2	75.5	91.0	1.33	1.20
29. 054OSB1-12-3A	118.0	111.2	125.0	1.06	1.12
30. 054OSB1-12-3B	118.0	111.2	125.0	1.06	1.12
31. 054CSP1-12-3A	118.0	111.2	119.0	1.01	1.07
32. 054CSP1-12-3B	118.0	111.2	114.3	0.97	1.03
33. 068OSB1-12-3A	156.6	155.2	173.0	1.10	1.11
34. 068OSB1-12-3B	156.6	155.2	179.2	1.14	1.15
35. 068CSP1-12-3A	156.6	155.2	172.3	1.10	1.11
36. 068CSP1-12-3B	156.6	155.2	183.0	1.17	1.18
			Average	1.20	1.10
			Standard Deviation	0.102	0.0638
			CoV	0.0852	0.0581

[1] Two extra fastners in web-to-web connection at 19 mm (0.75") from the tor

Table G-9 : Comparison of predicted ultimate load based on nominal and actual properties using Method 9 with experimental loads

Test	Ultimate load (kN)			Expt/Pred(Nominal)	Expt/Pred(Measurec
	Predicted		Experimental		
	Nominal Properties	Measured Properties			
1. 043OSB1-12-3A	70.3	77.9	94.3	1.34	1.21
2. 043OSB1-12-3B	70.3	77.9	83.0	1.18	1.07
3. 043OSB2-12-3A	70.3	77.9	91.5	1.30	1.18
4. 043OSB2-12-3B	70.3	77.9	83.3	1.19	1.07
5. 043OSB1-24-3A	70.3	77.9	82.0	1.17	1.05
6. 043OSB1-24-3B	70.3	77.9	84.2	1.20	1.08
7. 043OSB2-24-3A	70.3	77.9	78.4	1.11	1.01
8. 043OSB2-24-3B	70.3	77.9	84.0	1.19	1.08
9. 043OSB1-12-6A[1]	70.3	77.9	80.8	1.15	1.04
10. 043OSB1-12-6B	70.3	77.9	74.0	1.05	0.95
11. 043OSB2-12-6A	70.3	77.9	78.1	1.11	1.00
12. 043OSB2-12-6B	70.3	77.9	77.2	1.10	0.99
13. 043OSB1-24-6A	70.3	77.9	85.8	1.22	1.10
14. 043OSB1-24-6B	70.3	77.9	70.4	1.00	0.90
15. 043OSB2-24-6A	70.3	77.9	85.6	1.22	1.10
16. 043OSB2-24-6B	70.3	77.9	85.7	1.22	1.10
17. 033OSB1-12-3A	48.3	59.4	60.3	1.25	1.01
18. 033OSB1-12-3B	48.3	59.4	62.4	1.29	1.05
19. 033OSB1-12-6A	48.3	59.4	62.6	1.30	1.05
20. 033OSB1-12-6B	48.3	59.4	62.3	1.29	1.05
21. 043CSP1-12-3A	70.3	77.9	82.6	1.17	1.06
22. 043CSP1-12-3B	70.3	77.9	89.8	1.28	1.15
23. 043CSP2-12-3A	70.3	77.9	84.9	1.21	1.09
24. 043CSP2-12-3B	70.3	77.9	80.3	1.14	1.03
25. 043CSP1-12-6A	70.3	77.9	83.9	1.19	1.08
26. 043CSP1-12-6B	70.3	77.9	80.4	1.14	1.03
27. 043CSP2-12-6A	70.3	77.9	78.7	1.12	1.01
28. 043CSP2-12-6B	70.3	77.9	91.0	1.29	1.17
29. 054OSB1-12-3A	123.6	115.6	125.0	1.01	1.08
30. 054OSB1-12-3B	123.6	115.6	125.0	1.01	1.08
31. 054CSP1-12-3A	123.6	115.6	119.0	0.96	1.03
32. 054CSP1-12-3B	123.6	115.6	114.3	0.92	0.99
33. 068OSB1-12-3A	164.2	161.5	173.0	1.05	1.07
34. 068OSB1-12-3B	164.2	161.5	179.2	1.09	1.11
35. 068CSP1-12-3A	164.2	161.5	172.3	1.05	1.07
36. 068CSP1-12-3B	164.2	161.5	183.0	1.11	1.13
			Average	1.16	1.06
			Standard Deviation	0.104	0.0614
			CoV	0.0899	0.0577

[1] Two extra fastners in web-to-web connection at 19 mm (0.75") from the top

Table G-10

Reliability/Saftey Factors for Prediction Method 1 (with respect to actual sections)

Stud Thickness (mm)	α	$\overline{Q}/Q$	C_Φ	Φ	M_m	F_m	P_m	V_m	V_f	V_s	n	C_p	V_p	β_o
0.84	1.5	0.76	1.42	0.8	1.1	1	1.11	0.1	0.1	0.21	4	3.75	0.07	2.74
1.09	1.5	0.76	1.42	0.8	1.1	1	1.15	0.1	0.1	0.21	24	1.14	0.07	3.08
1.32	1.5	0.76	1.42	0.8	1.1	1	1.16	0.1	0.1	0.21	4	3.75	0.07	2.89
1.57	1.5	0.76	1.42	0.8	1.1	1	1.22	0.1	0.1	0.21	4	3.75	0.07	3.07
AVG	1.5	0.76	1.42	0.8	1.1	1	1.16	0.1	0.1	0.21	36	1.09	0.07	3.12

Table G-11

Reliability/Saftey Factors for Prediction Method 2 (with respect to nominal sections)

Stud Thickness	α	$\overline{Q}/Q$	C_Φ	Φ	M_m	F_m	P_m	V_m	V_f	V_s	n	C_p	V_p	β_o
0.84	1.5	0.76	1.42	0.8	1.1	1	1.31	0.1	0.1	0.21	4	3.75	0.07	3.32
1.09	1.5	0.76	1.42	0.8	1.1	1	1.24	0.1	0.1	0.21	24	1.14	0.07	3.36
1.32	1.5	0.76	1.42	0.8	1.1	1	1.05	0.1	0.1	0.21	4	3.75	0.07	2.54
1.57	1.5	0.76	1.42	0.8	1.1	1	1.16	0.1	0.1	0.21	4	3.75	0.07	2.89
AVG	1.5	0.76	1.42	0.8	1.1	1	1.22	0.1	0.1	0.21	36	1.09	0.07	3.31

Table G-12

Reliability/Saftey Factors for Prediction Method 2 (with respect to actual sections)

Stud Thickness	α	$\overline{Q}/Q$	C_Φ	Φ	M_m	F_m	P_m	V_m	V_f	V_s	n	C_p	V_p	β_o
0.84	1.5	0.76	1.42	0.8	1.1	1	1.07	0.1	0.1	0.21	4	3.75	0.07	2.61
1.09	1.5	0.76	1.42	0.8	1.1	1	1.12	0.1	0.1	0.21	24	1.14	0.07	2.98
1.32	1.5	0.76	1.42	0.8	1.1	1	1.11	0.1	0.1	0.21	4	3.75	0.07	2.74
1.57	1.5	0.76	1.42	0.8	1.1	1	1.16	0.1	0.1	0.21	4	3.75	0.07	2.89
AVG	1.5	0.76	1.42	0.8	1.1	1	1.12	0.1	0.1	0.21	36	1.09	0.07	2.99

Table G-13

Reliability/Saftey Factors for Prediction Method 3 (with respect to nominal sections)

Stud Thickness (mm)	α	$\overline{Q}/Q$	C_Φ	Φ	M_m	F_m	P_m	V_m	V_f	V_s	n	C_p	V_p	β_o
0.84	1.5	0.76	1.42	0.8	1.1	1	1.28	0.1	0.1	0.21	4	3.75	0.07	3.24
1.09	1.5	0.76	1.42	0.8	1.1	1	1.20	0.1	0.1	0.21	24	1.14	0.07	3.24
1.32	1.5	0.76	1.42	0.8	1.1	1	1.00	0.1	0.1	0.21	4	3.75	0.07	2.37
1.57	1.5	0.76	1.42	0.8	1.1	1	1.10	0.1	0.1	0.21	4	3.75	0.07	2.70
AVG	1.5	0.76	1.42	0.8	1.1	1	1.17	0.1	0.1	0.21	36	1.09	0.09	3.06

Table G-14

Reliability/Saftey Factors for Prediction Method 3 (with respect to actual sections)

Stud Thickness (mm)	α	$\overline{Q}/Q$	C_Φ	Φ	M_m	F_m	P_m	V_m	V_f	V_s	n	C_p	V_p	β_o
0.84	1.5	0.76	1.42	0.8	1.1	1	1.04	0.1	0.1	0.21	4	3.75	0.07	2.51
1.09	1.5	0.76	1.42	0.8	1.1	1	1.08	0.1	0.1	0.21	24	1.14	0.07	2.84
1.32	1.5	0.76	1.42	0.8	1.1	1	1.07	0.1	0.1	0.21	4	3.75	0.07	2.61
1.57	1.5	0.76	1.42	0.8	1.1	1	1.12	0.1	0.1	0.21	4	3.75	0.07	2.77
AVG	1.5	0.76	1.42	0.8	1.1	1	1.08	0.1	0.1	0.21	36	1.09	0.09	2.77

Table G-15

Reliability/Saftey Factors for Prediction Method 4 (with respect to nominal sections)

Stud Thickness (mm)	α	$\overline{Q}/Q$	C_Φ	Φ	M_m	F_m	P_m	V_m	V_f	V_s	n	C_p	V_p	β_o
0.84	1.5	0.76	1.42	0.8	1.1	1	1.43	0.1	0.1	0.21	4	3.75	0.07	3.63
1.09	1.5	0.76	1.42	0.8	1.1	1	1.36	0.1	0.1	0.21	24	1.14	0.07	3.71
1.32	1.5	0.76	1.42	0.8	1.1	1	1.21	0.1	0.1	0.21	4	3.75	0.07	3.04
1.57	1.5	0.76	1.42	0.8	1.1	1	1.35	0.1	0.1	0.21	4	3.75	0.07	3.43
AVG	1.5	0.76	1.42	0.8	1.1	1	1.35	0.1	0.1	0.21	36	1.09	0.07	3.68

Table G-16

Reliability/Saftey Factors for Prediction Method 4 (with respect to actual sections)

Stud Thickness (mm)	α	$\overline{Q}/Q$	C_Φ	Φ	M_m	F_m	P_m	V_m	V_f	V_s	n	C_p	V_p	β_o
0.84	1.5	0.76	1.42	0.8	1.1	1	1.18	0.1	0.1	0.21	4	3.75	0.07	2.95
1.09	1.5	0.76	1.42	0.8	1.1	1	1.23	0.1	0.1	0.21	24	1.14	0.07	3.33
1.32	1.5	0.76	1.42	0.8	1.1	1	1.25	0.1	0.1	0.21	4	3.75	0.07	3.16
1.57	1.5	0.76	1.42	0.8	1.1	1	1.32	0.1	0.1	0.21	4	3.75	0.07	3.35
AVG	1.5	0.76	1.42	0.8	1.1	1	1.23	0.1	0.1	0.21	36	1.09	0.07	3.34

Table G-17

Reliability/Saftey Factors for Prediction Method 5 (with respect to nominal sections)

Stud Thickness (mm)	α	$\overline{Q}/Q$	C_Φ	Φ	M_m	F_m	P_m	V_m	V_f	V_s	n	C_p	V_p	β_o
0.84	1.5	0.76	1.42	0.8	1.1	1	1.31	0.1	0.1	0.21	4	3.75	0.07	3.32
1.09	1.5	0.76	1.42	0.8	1.1	1	1.23	0.1	0.1	0.21	24	1.14	0.07	3.33
1.32	1.5	0.76	1.42	0.8	1.1	1	1.04	0.1	0.1	0.21	4	3.75	0.07	2.51
1.57	1.5	0.76	1.42	0.8	1.1	1	1.15	0.1	0.1	0.21	4	3.75	0.07	2.86
AVG	1.5	0.76	1.42	0.8	1.1	1	1.21	0.1	0.1	0.21	36	1.09	0.08	3.22

Table G-18

Reliability/Saftey Factors for Prediction Method 5 (with respect to actual sections)

Stud Thickness (mm)	α	$\overline{Q}/Q$	C_Φ	Φ	M_m	F_m	P_m	V_m	V_f	V_s	n	C_p	V_p	β_o
0.84	1.5	0.76	1.42	0.8	1.1	1	1.07	0.1	0.1	0.21	4	3.75	0.07	2.61
1.09	1.5	0.76	1.42	0.8	1.1	1	1.11	0.1	0.1	0.21	24	1.14	0.07	2.94
1.32	1.5	0.76	1.42	0.8	1.1	1	1.1	0.1	0.1	0.21	4	3.75	0.07	2.70
1.57	1.5	0.76	1.42	0.8	1.1	1	1.16	0.1	0.1	0.21	4	3.75	0.07	2.89
AVG	1.5	0.76	1.42	0.8	1.1	1	1.11	0.1	0.1	0.21	36	1.09	0.07	2.95

Table G-19

Reliability/Saftey Factors for Prediction Method 6 (with respect to nominal sections)

Stud Thickness (mm)	α	$\bar{Q}/Q$	C_Φ	Φ	M_m	F_m	P_m	V_m	V_f	V_s	n	C_p	V_p	β_o
0.84	1.5	0.76	1.42	0.8	1.1	1	1.53	0.1	0.1	0.21	4	3.75	0.07	3.87
1.09	1.5	0.76	1.42	0.8	1.1	1	1.48	0.1	0.1	0.21	24	1.14	0.07	4.04
1.32	1.5	0.76	1.42	0.8	1.1	1	1.37	0.1	0.1	0.21	4	3.75	0.07	3.48
1.57	1.5	0.76	1.42	0.8	1.1	1	1.52	0.1	0.1	0.21	4	3.75	0.07	3.85
AVG	1.5	0.76	1.42	0.8	1.1	1	1.47	0.1	0.1	0.21	36	1.09	0.07	4.02

Table G-20

Reliability/Saftey Factors for Prediction Method 6 (with respect to actual sections)

Stud Thickness (mm)	α	$\bar{Q}/Q$	C_Φ	Φ	M_m	F_m	P_m	V_m	V_f	V_s	n	C_p	V_p	β_o
0.84	1.5	0.76	1.42	0.8	1.1	1	1.29	0.1	0.1	0.21	4	3.75	0.07	3.27
1.09	1.5	0.76	1.42	0.8	1.1	1	1.33	0.1	0.1	0.21	24	1.14	0.07	3.63
1.32	1.5	0.76	1.42	0.8	1.1	1	1.38	0.1	0.1	0.21	4	3.75	0.07	3.51
1.57	1.5	0.76	1.42	0.8	1.1	1	1.47	0.1	0.1	0.21	4	3.75	0.07	3.73
AVG	1.5	0.76	1.42	0.8	1.1	1	1.35	0.1	0.1	0.21	36	1.09	0.07	3.69

Table G-21

Reliability/Saftey Factors for Prediction Method 7 (with respect to nominal sections)

Stud Thickness (mm)	α	$\bar{Q}/Q$	C_Φ	Φ	M_m	F_m	P_m	V_m	V_f	V_s	n	C_p	V_p	β_o
0.84	1.5	0.76	1.42	0.8	1.1	1	1.35	0.1	0.1	0.21	4	3.75	0.07	3.43
1.09	1.5	0.76	1.42	0.8	1.1	1	1.28	0.1	0.1	0.21	24	1.14	0.07	3.48
1.32	1.5	0.76	1.42	0.8	1.1	1	1.1	0.1	0.1	0.21	4	3.75	0.07	2.70
1.57	1.5	0.76	1.42	0.8	1.1	1	1.22	0.1	0.1	0.21	4	3.75	0.07	3.07
AVG	1.5	0.76	1.42	0.8	1.1	1	1.26	0.1	0.1	0.21	36	1.09	0.08	3.39

Table G-22

Reliability/Saftey Factors for Prediction Method 7 (with respect to actual sections)

Stud Thickness (mm)	α	$\bar{Q}/Q$	C_Φ	Φ	M_m	F_m	P_m	V_m	V_f	V_s	n	C_p	V_p	β_o
0.84	1.5	0.76	1.42	0.8	1.1	1	1.11	0.1	0.1	0.21	4	3.75	0.07	2.74
1.09	1.5	0.76	1.42	0.8	1.1	1	1.15	0.1	0.1	0.21	24	1.14	0.07	3.08
1.32	1.5	0.76	1.42	0.8	1.1	1	1.16	0.1	0.1	0.21	4	3.75	0.07	2.89
1.57	1.5	0.76	1.42	0.8	1.1	1	1.22	0.1	0.1	0.21	4	3.75	0.07	3.07
AVG	1.5	0.76	1.42	0.8	1.1	1	1.16	0.1	0.1	0.21	36	1.09	0.07	3.12

Table G-23

Reliability/Saftey Factors for Prediction Method 8 (with respect to nominal sections)

Stud Thickness (mm)	α	$\bar{Q}/Q$	C_Φ	Φ	M_m	F_m	P_m	V_m	V_f	V_s	n	C_p	V_p	β_o
0.84	1.5	0.76	1.42	0.8	1.1	1	1.31	0.1	0.1	0.21	4	3.75	0.07	3.32
1.09	1.5	0.76	1.42	0.8	1.1	1	1.22	0.1	0.1	0.21	24	1.14	0.07	3.30
1.32	1.5	0.76	1.42	0.8	1.1	1	1.02	0.1	0.1	0.21	4	3.75	0.07	2.44
1.57	1.5	0.76	1.42	0.8	1.1	1	1.13	0.1	0.1	0.21	4	3.75	0.07	2.80
AVG	1.5	0.76	1.42	0.8	1.1	1	1.2	0.1	0.1	0.21	36	1.09	0.09	3.15

Table G-24

Reliability/Saftey Factors for Prediction Method 8 (with respect to actual sections)

Stud Thickness (mm)	α	$\overline{Q}/Q$	C_Φ	Φ	M_m	F_m	P_m	V_m	V_f	V_s	n	C_p	V_p	β_o
0.84	1.5	0.76	1.42	0.8	1.1	1	1.07	0.1	0.1	0.21	4	3.75	0.07	2.61
1.09	1.5	0.76	1.42	0.8	1.1	1	1.10	0.1	0.1	0.21	24	1.14	0.07	2.91
1.32	1.5	0.76	1.42	0.8	1.1	1	1.09	0.1	0.1	0.21	4	3.75	0.07	2.67
1.57	1.5	0.76	1.42	0.8	1.1	1	1.14	0.1	0.1	0.21	4	3.75	0.07	2.83
AVG	1.5	0.76	1.42	0.8	1.1	1	1.10	0.1	0.1	0.21	36	1.09	0.09	2.84

Table G-25

Reliability/Saftey Factors for Prediction Method 9 (with respect to nominal sections)

Stud Thickness (mm)	α	$\overline{Q}/Q$	C_Φ	Φ	M_m	F_m	P_m	V_m	V_f	V_s	n	C_p	V_p	β_o
0.84	1.5	0.76	1.42	0.8	1.1	1	1.28	0.1	0.1	0.21	4	3.75	0.07	3.24
1.09	1.5	0.76	1.42	0.8	1.1	1	1.18	0.1	0.1	0.21	24	1.14	0.07	3.17
1.32	1.5	0.76	1.42	0.8	1.1	1	0.98	0.1	0.1	0.21	4	3.75	0.07	2.30
1.57	1.5	0.76	1.42	0.8	1.1	1	1.08	0.1	0.1	0.21	4	3.75	0.07	2.64
AVG	1.5	0.76	1.42	0.8	1.1	1	1.16	0.1	0.1	0.21	36	1.09	0.09	3.03

Table G-26

Reliability/Saftey Factors for Prediction Method 9 (with respect to actual sections)

Stud Thickness (mm)	α	$\overline{Q}/Q$	C_Φ	Φ	M_m	F_m	P_m	V_m	V_f	V_s	n	C_p	V_p	β_o
0.84	1.5	0.76	1.42	0.8	1.1	1	1.04	0.1	0.1	0.21	4	3.75	0.07	2.51
1.09	1.5	0.76	1.42	0.8	1.1	1	1.06	0.1	0.1	0.21	24	1.14	0.07	2.77
1.32	1.5	0.76	1.42	0.8	1.1	1	1.05	0.1	0.1	0.21	4	3.75	0.07	2.54
1.57	1.5	0.76	1.42	0.8	1.1	1	1.1	0.1	0.1	0.21	4	3.75	0.07	2.70
AVG	1.5	0.76	1.42	0.8	1.1	1	1.06	0.1	0.1	0.21	36	1.09	0.09	2.70

Table G-27

Summary of Average Reliability/Safety Factors (with respect to nominal sections)

Method	α	$\overline{Q}/Q$	C_Φ	Φ	M_m	F_m	P_m	V_m	V_f	V_s	n	C_p	V_p	β_o
1	1.5	0.76	1.42	0.8	1.1	1	1.26	0.1	0.1	0.21	36	1.09	0.08	3.38
2	1.5	0.76	1.42	0.8	1.1	1	1.22	0.1	0.1	0.21	36	1.09	0.07	3.31
3	1.5	0.76	1.42	0.8	1.1	1	1.17	0.1	0.1	0.21	36	1.09	0.0899	3.06
4	1.5	0.76	1.42	0.8	1.1	1	1.35	0.1	0.1	0.21	36	1.09	0.07	3.68
5	1.5	0.76	1.42	0.8	1.1	1	1.21	0.1	0.1	0.21	36	1.09	0.08	3.22
6	1.5	0.76	1.42	0.8	1.1	1	1.47	0.1	0.1	0.21	36	1.09	0.07	4.02
7	1.5	0.76	1.42	0.8	1.1	1	1.26	0.1	0.1	0.21	36	1.09	0.08	3.39
8	1.5	0.76	1.42	0.8	1.1	1	1.20	0.1	0.1	0.21	36	1.09	0.09	3.15
9	1.5	0.76	1.42	0.8	1.1	1	1.16	0.1	0.1	0.21	36	1.09	0.0899	3.03

Table G-28

Summary of Average Reliability/Safety Factors (with respect to actual sections)

Method	α	$\overline{Q}/Q$	C_Φ	Φ	M_m	F_m	P_m	V_m	V_f	V_s	n	C_p	V_p	β_o
1	1.5	0.76	1.42	0.8	1.1	1	1.16	0.1	0.1	0.21	36	1.09	0.065	3.12
2	1.5	0.76	1.42	0.8	1.1	1	1.12	0.1	0.1	0.21	36	1.09	0.07	2.99
3	1.5	0.76	1.42	0.8	1.1	1	1.08	0.1	0.1	0.21	36	1.09	0.0877	2.77
4	1.5	0.76	1.42	0.8	1.1	1	1.23	0.1	0.1	0.21	36	1.09	0.07	3.34
5	1.5	0.76	1.42	0.8	1.1	1	1.11	0.1	0.1	0.21	36	1.09	0.07	2.95
6	1.5	0.76	1.42	0.8	1.1	1	1.35	0.1	0.1	0.21	36	1.09	0.07	3.69
7	1.5	0.76	1.42	0.8	1.1	1	1.16	0.1	0.1	0.21	36	1.09	0.07	3.12
8	1.5	0.76	1.42	0.8	1.1	1	1.1	0.1	0.1	0.21	36	1.09	0.09	2.84
9	1.5	0.76	1.42	0.8	1.1	1	1.06	0.1	0.1	0.21	36	1.09	0.0877	2.70