Testing and Analysis of Light Gauge Steel Frame / 9 mm OSB Wood Panel Shear Walls

by

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ABSTRACT

The use of light steel gauge framed walls in the residential and commercial sector is increasing in popularity. However, our knowledge of the performance and the behaviour of structures using such walls when subjected to lateral wind and seismic loads is limited. At present, no design method for light gauge steel shear walls is contained in Canadian codes or standards. For this reason a research project on steel frame / wood panel shear walls was undertaken.

A comprehensive database of monotonic and reversed cyclic tests on steel frame / wood panel shear walls is needed to obtain different wall parameters for use in design. For this particular project 1220 × 2440 mm walls (3 wall configurations for a total of 18 specimens) composed of 1.09 mm thick 230 MPa grade steel and 9.0 mm thick OSB were tested; where the screw spacing along the perimeter of the wall was varied (75, 100 & 152 mm). The equivalent energy elastic-plastic (EEEP) analysis approach was employed to derive design values from the test results; such as stiffness, strength and a resistance factor for use with the 2005 NBCC, as well as ductility and overstrength modification factors. The resistance factor, $\phi = 0.7$, determined by Branston was confirmed. Ductility related, $R_d = 2.5$, and overstrength related, $R_o = 1.8$, seismic force modification factors are recommended for use with shear wall design values that are based on the EEEP analysis approach.

Furthermore, the test data were used to create and calibrate hysteretic models, with the Stewart element, which later were used in non-linear time history dynamic analyses. The Ruaumoko software was made use of for the modeling of two representative buildings with ten earthquake records for the Vancouver BC region. The shear wall system of a typical two-storey house and a three-storey commercial building was first designed for lateral loads and then modeled. The resulting shear deformations (rotations) obtained from the analyses were compared with the limiting parameters measured during the physical shear wall tests. It was found that the scaled ground motions caused a shear demand that did not exceed the test based deformation limits.

RÉSUMÉ

L'utilisation de murs de refend aux montants en acier formés à froid avec panneaux de revêtements en bois augmente en popularité dans le secteur résidentiel et commercial. Toutefois, nos connaissances sur les performances et le comportement de ces structures qui sont sujettes à des charges latérales dues au vent et aux secousses sismiques sont plutôt limitées. Pour le moment, il n'y a pas de méthode de conception concernant les murs de refends aux montants en acier formés à froid dans les normes et codes canadiens. C'est pour cette raison qu'un projet de recherche sur les murs aux montants en acier formés à froid avec panneaux de revêtements en bois a été entrepris.

Une importante base de données provenant d'essais monotoniques et cycliques fait sur des murs de refends bois-métal est nécessaire afin d'acquérir les différents paramètres qui seront utilisés lors de la conception de tels murs. Pour ce projet, des murs de 1220 mm × 2440 mm (3 configurations de murs pour un total de 18 spécimens) composés d'acier formés à froid de 1.09 mm en épaisseur avec une nuance de 230 MPa et des panneaux de OSB (oriented strand board) de 9.0 mm d'épaisseur ont été testés avec un espacement des vis au périmètre des murs variant de 75, 100 et 152 mm. La méthode de l'énergie équivalente élastique-plastique (EEEP) a été utilisée pour déterminer les paramètres de conception tels que la rigidité, la force, le facteur de résistance pour utilisation en conjonction avec le CNBC 2005, de plus que les facteurs de ductilité et d'écrouissage. Le facteur de résistance, $\phi = 0.7$, déterminé par Branston a été confirmé. Les facteurs de ductilité, $R_d = 2.5$, et de d'écrouissage, $R_o = 1.8$, ont été recommandés pour utilisation avec les valeurs de conception des murs de refends basés sur la méthode d'analyse EEEP.

De plus, les résultats des essais ont été utilisés pour créer et calibrer les modèles hystérétiques avec l'élément Stewart, qui par la suite ont été utilisés pour l'analyse dynamique non linéaire dans le temps. Le programme informatique Ruaumoko a été utilisé pour modeler deux édifices soumis à dix tremblements de terre pour la région de Vancouver, CB. Les murs de refends pour des édifices de deux et trois étages ont été conçus pour résister les charges latérales et ont, par la suite, été modélisés. Les

déformations en cisaillement (rotations) obtenues lors des analyses ont été comparées avec les paramètres mesurés pendant les tests réels des murs. Il a été découvert que les déformations en cisaillement causées par les secousses sismiques prédéfinies n'ont pas dépassées les limites de déformation basées sur les tests physiques des murs.

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TABLE OF CONTENTS

Abstract i

Résumé ii

Acknowledgement iv

Table of Contents v

List of Figures viii

List of Tables x

Chapter 1. Introduction 1

1.1. General Overview 1

1.2. Statement of Problem 2

1.3. Objectives 3

1.4. Scope of Study 3

1.5. Thesis Outline 4

Chapter 2. Literature Review 5

2.1. Summary of Shear Wall Testing 5

2.2. Dynamic Analyses of Shear Walls 8

2.2.1. Della Corte, Fiorino & Landolfo 8

2.2.2. Fülöp & Dubina 11

2.2.3. Ceccotti & Karacabeyli 13

2.2.4. Foliente 15

2.2.5. Summary of Past Dynamic Analysis Research Programs 17

Chapter 3. Shear Wall Testing 18

3.1. Descriptions of Tests 18

3.1.1. Shear Wall Configuration, Materials and Components 19

3.1.2. Shear Wall Fabrication 21

3.1.3. Test Set-Up 24

3.1.4. Instrumentation and Data Acquisition 25

3.1.5. Monotonic Tests 26

3.1.6. Reversed Cyclic Tests 27

3.2. Test Results / Analysis of Test Data 30
3.2.1. Data Correction 31
3.2.2. Energy Dissipation 33
3.2.3. General Test Results 35
3.3. Failure Modes 39
3.4. Ancillary Testing of Materials 42
3.4.1. OSB Wood Panels Properties 43
3.4.2. Light Gauge Steel Studs Properties 45
3.5. Wood Panel Shear Deformation & Holddown Forces 46
Chapter 4. Recommended Design Parameters 51
4.1. Introduction 51
4.2. Yield Strength and Stiffness 52
4.3. Calibration of Resistance Factor, ϕ 63
4.4. Recommended Shear and Stiffness Values for Light Gauge Steel Frame / Wood Panel Shear Walls 67
4.5. Factor of Safety 70
4.6. Capacity Based Design and Overstrength 74
4.7. 2005 NBCC and Evaluation of Force Modification Factors 77
4.7.1. Ductility-Related Force Modification Factor, R_d 77
4.7.2. Overstrength-Related Modification Factor, R_o 80
Chapter 5. Dynamic Analyses 82
5.1. Introduction 82
5.2. Hysteretic Behaviour of Walls 82
5.2.1. Stiffness Degradation 83
5.2.2. Pinching 85
5.2.3. Strength Degradation 86
5.3. Stewart Degrading Hysteresis 87
5.3.1. Experimental Data Matching 88
5.3.2. Limitations 91
5.4. Design & Base Shear Calculation 93
5.4.1. House Design 94
5.4.2. Small Commercial Building Design 97

5.5. NBCC 2005 Base Shear Calculation 100
5.5.1. Shear Wall Configuration Based on Resistance 104
5.5.2. Shear Wall Configuration Based on Stiffness 105
5.6. Dynamic Analyses 107
5.6.1. Earthquake Records 108
5.6.2. House Model 109
5.6.2.1. Pushover Analysis of Single-Storey Model 113
5.6.3. Commercial Building Model 113
5.7. Analysis Results 115
5.7.1. House Model Analysis Results 116
5.7.2. Commercial Building Model Analysis Results 120
Chapter 6. Conclusions and Recommendations 124
6.1. Conclusions 124
6.2. Recommendations 125

References 127

Appendix 'A' Reversed Cyclic Test Protocols 135
Appendix 'B' Shear Wall Test Data 139
Appendix 'C' Stewart Model 209
Appendix 'D' Base Shear Calculation 219
Appendix 'E' Ground Motion Record Accelerograms & Spectra 228
Appendix 'F' Ruaumoko Input Files for Both Building Models 239
Appendix 'G' Hystereses & Time Histories for Two- and Three-Storey Models 260

LIST OF FIGURES

Figure 1.1: Light gauge steel stud wall using platform framing technique (left: exterior view; right: interior view of side wall) 1

Figure 3.1: Typical OSB sheathed shear wall placed in test frame 18

Figure 3.2: Mill and grade stamp of OSB sheathing panels 20

Figure 3.3: Location of shear bolts and hold-downs 22

Figure 3.4: Chord studs and bottom track connection 22

Figure 3.5: Screw spacing for 75 mm / 305 mm (3"/12") schedule 23

Figure 3.6: Test frame with 1220 × 2440 mm (4'×8') wall specimen 24

Figure 3.7: Wall specimen in test frame 25

Figure 3.8: Positioning of LVDTs on wall specimen 26

Figure 3.9: Typical wall resistance curve for a monotonic test 27

Figure 3.10: CUREE ordinary ground motions protocol for shear wall tests 42-A,B,C 1220 × 2440 mm (4'×8') OSB 150/305 mm (6"/12") 29

Figure 3.11: Typical wall resistance vs. deflection curve for a reversed cyclic test 30

Figure 3.12: Deformed configuration of shear wall 32

Figure 3.13: Energy dissipation for a monotonic shear wall specimen 34

Figure 3.14: Energy dissipation for a reversed cyclic test is represented by the area enclosed by the hysteretic loops (single example loop shown in bold) 35

Figure 3.15: Screw pulling through the sheathing 39

Figure 3.16: Screw head within the OSB panel 40

Figure 3.17: Bottom corner screw failed in shear (Head still visible in the wood panel) 41

Figure 3.18: Wood bearing failure mode 41

Figure 3.19: Overall wall failure due to failure of multiple sheathing connections 42

Figure 3.20 Edgewise shear setup 43

Figure 3.21: Positioning of additional LVDTs and load cells 47

Figure 3.22: Shear deformations of OSB panel 48

Figure 3.23: North and south holddown forces 49

Figure 3.24: Shear wall uplift 50

Figure 4.1: EEEP model 53

Figure 4.2: EEEP curve for monotonic test 41A 55
Figure 4.3: EEEP curve for cyclic test 42A 56
Figure 4.4: EEEP design curve with imposed 2.5 % drift limit (Case I) 57
Figure 4.5: EEEP design curve with imposed 2.5 % drift limit (Case II) 58
Figure 4.6: Factor of safety inherent in limit states design 71
Figure 4.7: Overstrength inherent in design 75
Figure 5.1: Stiffness of one loop 84
Figure 5.2: Energy dissipation & stiffness degradation (Test 46A – 1.5 & 2.0 Δ Cycles) 85
Figure 5.3: Evolution of pinched hysteresis loops with increased displacement level 86
Figure 5.4: Strength degradation representation between two successive loops 87
Figure 5.5: Stewart degrading hysteresis 88
Figure 5.6: Superposition of Stewart model and experimental monotonic curve (Test 43A) 90
Figure 5.7: Superposition of Stewart model and experimental cyclic hysteresis (Test 44C) 91
Figure 5.8: House plan view with shear wall tributary area in grey 97
Figure 5.9: Hambro® D500 floor system 98
Figure 5.10: Commercial building plan view with shear wall tributary area in grey 100
Figure 5.11: Plan view of both structures with location of all four shear walls 103
Figure 5.12: Vancouver design spectrum vs. scaled ground motion S01 109
Figure 5.13: Ruaumoko models 110
Figure 5.14: Ruaumoko models: Two-storey shear wall 110
Figure 5.15: Pushover hysteresis vs. Stewart hysteresis 113
Figure 5.16: Ruaumoko models: Three-storey shear wall 114
Figure 5.17: Resistance vs. displacement hystereses of the two-storey shear wall under ground motion S09 117
Figure 5.18: Time history plots of the two-storey shear wall under ground motion S09 118
Figure 5.19: Resistance vs. displacement hystereses of the three-storey shear wall under ground motion S07 121
Figure 5.20: Time history plots of the three-storey shear wall under ground motion S07 122

LIST OF TABLES

Table 3.1: Light gauge steel frame / wood panel shear wall test program matrix 21

Table 3.2: CUREE Reversed Cyclic Testing Protocol (Displacement Controlled) 29

Table 3.3: Test results for monotonic tests 37

Table 3.4: Test results for reversed cyclic tests 38

Table 3.5: Experimental and CSA O86 shear properties of OSB panels 45

Table 3.6: Experimental properties of light gauge steel studs and tracks 46

Table 4.1: Lateral wall displacements at 80% of the ultimate shear force 59

Table 4.2: Design values for monotonic tests 60

Table 4.3: Design values for reversed cyclic tests (positive cycles) 61

Table 4.4: Design values for reversed cyclic tests (negative cycles) 62

Table 4.5: Resistance factor calibration for 2005 NBCC wind loads 66

Table 4.6: Nominal shear strength, S_y (kN/m), and unit elastic stiffness, k_e (kN/mm/m), for light gauge steel frame / wood panel shear walls dependent of the sheathing material 69

Table 4.7: Factor of safety inherent in design for monotonic test values 72

Table 4.8: Factor of safety inherent in design for cyclic test values 73

Table 4.9: Overstrength inherent in design for monotonic test values 76

Table 4.10: Overstrength inherent in design for cyclic test values 76

Table 4.11: Natural period for light-framed buildings 78

Table 4.12: Ductility and R_d values for monotonic tests 79

Table 4.13: Ductility and R_d values for reversed cyclic tests (avg. of both cycles) 79

Table 4.14: Overstrength-related force modification factor 81

Table 5.1: Stewart hysteresis parameters for light gauge steel frame / OSB panel (9 mm (3/8")) 89

Table 5.2: Maximum rotation of shear walls 93

Table 5.3: House dead loads 95

Table 5.4: House live and snow loads 95

Table 5.5: Specified load calculation per shear wall – house design 97

Table 5.6: Small commercial building dead loads 99

Table 5.7: Small commercial building live loads 99

Table 5.8:	Factored load calculation per shear wall – commercial building design	100
Table 5.9:	Variables for shear base calculations for both structures	102
Table 5.10:	Base shear value for design	103
Table 5.11:	Maximum shear force developed per shear wall	104
Table 5.12:	Calculation for drift limitation	107
Table 5.13:	Ground motion records used in Ruaumoko analysis	108
Table 5.14:	5% damped spectral response acceleration values (Vancouver, BC)	109
Table 5.15:	Summary of model properties	111
Table 5.16:	Maximum rotations of shear wall model for two-storey building	119
Table 5.17:	Summary of dynamic analysis results for the two-storey model	119
Table 5.18:	Maximum rotations shear wall model for three-storey building	123
Table 5.19:	Summary of results for the three-storey model	123

CHAPTER 1 INTRODUCTION

1.1 GENERAL OVERVIEW

It is anticipated that in Canada and across North America the use of light gauge steel products will increase in the years to come. In residential and small commercial structures light gauge steel can be used for wall, floor and roof framing. Figure 1.1 shows a typical light gauge steel frame house sheathed with plywood panels.

Figure 1.1: Light gauge steel stud wall using platform framing technique (left: exterior view; right: interior view of side wall) *(Branston, 2004)*

Light gauge steel walls are often used as gravity load bearing walls, but they can also be designed and used as shear walls. Shear walls transmit in-plane lateral forces due to wind or earthquake loads from the upper storey(s) to the foundation. In order to develop a resistance to these lateral forces, the walls are covered with a structural member such as plywood or oriented strand board (OSB) panels. The wood panels are fixed to the light gauge steel frame by means of screws, the size and number of which will dictate the stiffness and the shear resistance of the wall. For example, a close screw pattern (75 mm) results in an increase to both the shear strength and the stiffness of the wall compared with a wall whose sheathing is attached at 152 mm intervals. It is also necessary to attach the wall, by means of shear anchors and hold downs, to the supporting foundation or to the lower wall segments in a multi-storey building. By anchoring the walls at their ends, a structure that acts as a cantilever beam is created.

As previously stated, this type of construction is becoming more popular; presently however, in Canada there are no standards or codes to design such walls. It is for this reason that light gauge steel frame / wood panel shear walls are under intensive study at McGill University. The overall goal of the research project is to develop a design method for light gauge shear walls to be used in conjunction with the National Building Code of Canada (NBCC) *(NRCC, 2005)*. The research undertaken in the previous years at McGill University consisted mainly of physical tests of one-storey shear walls under monotonic and reversed cyclic loadings *(Boudreault, 2005; Branston, 2004; Chen, 2004)*. Different wall configurations were used for testing, for which the following were varied; fastener schedule, wall length, as well as sheathing type and thicknesses. Design parameters for wind and earthquake loadings were then developed based on the test results.

1.2 STATEMENT OF PROBLEM

In the 2005 National Building Code of Canada a procedure for the calculation of equivalent static seismic design loads is prescribed. However, the code does not list specific force modification factors (R_d and R_o) greater than 1.0 for light gauge steel frame / wood panel shear walls. In addition, the North American Specification for the Design of Cold-Formed Steel Structural Members *(CSA S136, 2002)* does not contain design information concerning shear walls. Furthermore, at this time the literature does not provide sufficient guidance, in terms of Canadian seismic design requirements, on the performance of buildings constructed with light gauge steel frame / wood panel shear walls. In contrast, the American Iron and Steel Institute (AISI) has developed a shear wall design guide *(AISI, 1998)*, a standard for the lateral design of cold-formed steel framing *(AISI, 2004)* and has been able to include shear wall design information in the IBC *(ICC, 2000, 2003)* and UBC *(ICBO, 1997)* model buildings codes. It is therefore of importance that studies be carried out in Canada to develop a method for the design of light gauge steel frame / wood panel shear walls. This method would need to incorporate Canadian limit states design philosophy, as documented in the 2005 NBCC, and account for the use of Canadian construction products. The shear wall research project, underway at McGill University since 2001, has focused on walls constructed of 11 mm OSB and 12.5 mm plywood panels. In construction it is not uncommon to use thinner sheathing, i.e. 9 mm OSB and 9.5 mm plywood panels, for which shear wall design information is not available.

1.3 OBJECTIVES

The objectives of this thesis include: i) To review the construction and testing requirements for shear walls as prescribed by Branston *(2004)* and Boudreault *(2005)*. ii) To test three configurations of light gauge steel frame / wood panel shear wall sheathed with 9 mm oriented strand board (OSB) panels. iii) To determine design values for the tested walls using the equivalent energy elastic-plastic analysis approach. iv) To calibrate hysteretic models for each of the wall configurations used in testing. v) To carry out a pushover analysis on a single-storey shear wall model to validate its applicability. vi) To create a two and three-storey building shear wall model and to carry out non-linear time history dynamic analyses using simulated and real earthquake records. vii) To evaluate the demand on the shear walls for both building models. And viii) to provide recommendations for future studies for the modeling and computer analysis of shear walls in order to expand our knowledge of light gauge steel frame / wood panel shear wall behaviour under cyclic loading conditions.

1.4 SCOPE OF STUDY

Monotonic and reversed cyclic tests were carried out on eighteen single-storey light gauge steel frame / wood panel shear walls during June and July of 2004. The wall specimens were constructed with Canadian cold-formed light gauge steel frames and 9 mm (3/8") oriented strand board sheathing (OSB) *(CSA O325, 1992)*, which was attached with screws at a spacing of 75, 100 and 152 mm (3", 4" and 6") over the panel perimeter. The resulting test data was then used to establish design parameters, seismic force modification factors and hysteretic models. The results presented and values proposed in this thesis are limited to individual 1220 × 2440 mm (4' × 8') light gauge steel frame / wood panel shear walls designed to resist lateral in-plane loading only. This research did not include the physical testing of multiple-storey shear walls nor combined vertical and lateral loading design values.

In addition, non-linear time history dynamic analyses of two representative multi-storey buildings, located in Vancouver, BC, was completed. A suite of ten earthquake ground motions from the west coast of North America was relied on to evaluate the inelastic performance of and the proposed design method for the shear wall system found in these buildings.

1.5 THESIS OUTLINE

This thesis, which consists of four main parts, is a preliminary study to evaluate shear wall performance using the design shear strength and stiffness parameters, as well as force modification factors (R_d and R_o) derived from this study. A brief review of existing cyclic tests of light framed wood and steel shear walls, followed by a more explicit review of shear wall modeling and testing using non-linear time history dynamic analysis software is found in Chapter 2. In Chapter 3, the shear wall experimental program is presented. The content of Chapter 4 focuses on the recommended design values derived from the shear wall test data. Chapter 5 consists of the calibration of hysteretic models of the tested shear walls, the design of a shear wall system for a two and a three-storey building and the modeling and analyses of shear walls in both buildings. Finally, Chapter 6 provides conclusions and recommendations for future studies on modeling and analysis of light gauge steel frame / wood panel shear walls.

CHAPTER 2 LITERATURE REVIEW

2.1 SUMMARY OF SHEAR WALL TESTING

The testing of light gauge steel frame / wood panel shear walls was started as early as in the 1970s by Tarpy at Vanderbilt University *(McCreless & Tarpy, 1978; Tarpy & Hauenstein, 1978)*. From this initial research program and subsequent studies by researchers such as Tissell *(1993)*, Serrette *et al. (1996a, 1996b, 1997a, 1997b)*, Serrette *(1997)*, Serrette and Ogunfunmi *(1996)*, NAHB *(1997)*, Selenikovich and Dolan *(1999)*, Selenikovich *et al. (2000a)* and the City of Los Angeles (CoLA) – University of California at Irvine (UCI) *(2001)* design standards were developed in the US. At present design guides and standards such as the 1997 UBC *(ICBO, 1997)*, the 1998 Shear Wall Design Guide *(AISI, 1998)*, the 2003 International Building Code *(ICC, 2003)* and the Standard for Cold-Formed Steel Framing - Lateral Design *(AISI, 2004)* are available for engineers to use in the US. No equivalent codified design standard developed for use in conjunction with the 2005 NBCC exists in Canada.

Research and testing of wood frame / wood panel shear walls has been carried out since 1929, during which year Report R896 was published by the Forest Products Laboratory *(Trayer, 1929)*. Detailed descriptions of past test programs can be found in the summary documents by van de Lindt *(2004)* and Filiatrault *(2001)*. A listing of wood shear wall test programs was also provided by Branston *(2004)*. Due to the performance of wood framed buildings in the Northridge CA earthquake in 1994 an extensive research program, under the heading "The CUREE-Caltech Woodframe Project" was undertaken. Results of this shear wall test program can be found in the work by Gatto & Uang *(2002)*.

The light gauge steel frame / wood panel shear wall research program at McGill University has been underway since 2001. To date, Zhao *(2002)*, Branston *(2004)*, Chen *(2004)* and Boudreault *(2005)* have each written a thesis on the subject. All of these researchers have presented a detailed literature review on various topics related to shear walls. For this reason only a summary of the reviews and work carried out by Zhao, Branston, Chen and Boudreault is presented herein. In addition, a comprehensive review of past research on the dynamic analyses of shear walls is provided.

Zhao *(2002)* first completed a literature review of existing shear wall test programs. The following researchers were included: McCreless and Tarpy *(1978),* Tarpy and Hauenstein *(1978),* Tarpy *(1980),* Tarpy & Girard *(1982),* Tissell *(1993),* Serrette *et al. (1996a, 1996b)* and Serrette *(1997),* Serrette & Ogunfunmi *(1996),* National Association of Home Builders (NAHB) *(1997),* Serrette *et al. (1997a, 1997b),* Gad & Duffield *(1997, 2000),* Gad *et al. (1998, 1999a, 1999b, 1999c),* Selenikovich and Dolan *(1999)* and Selenikovich et al. *(2000a)* and the City of Los Angeles (CoLA) – University of California at Irvine (UCI) *(2001).* Based on these existing shear wall tests Zhao was able to derive a ductility related R value of 2.0 for seismic design according to the 1995 NBCC *(NRCC, 1995).* Zhao was also responsible for the design of the shear wall testing frame, which is described in Chapter 3.

Branston *(2004)* presented the existing test programs of Serrette *et al. (2002)*, as well as Fülöp and Dubina *(2002, 2003).* Branston's work also included a literature review of existing North American, Australian and European shear wall test programs, and a comparison of standards for structural wood panels used in Canada and in the United States. He then carried out tests on 43 light gauge steel frame / wood panel shear walls, and proposed design parameters based on the combined data of 109 wall specimens tested by Boudreault *(2005)*, Branston *et al. (2004)* and Chen *(2004).* The shear wall test specimens were sheathed with 12.5 mm CSP and DFP, as well as 11 mm OSB panels. The design parameters for in-plane strength and stiffness were determined using the equivalent energy elastic-plastic (EEEP) method, which was originally developed by Park *(1989)* and then presented in a modified form by Foliente *(1996).* Based on the data of the 109 tests, Branston recommended a resistance factor of 0.7 for walls with a maximum aspect ratio of 2:1, and found that the specimens exhibited an approximate overstrength of 1.2.

Chen *(2004)* examined the performance of tested shear walls and developed an analytic model to theoretically calculate the resistance and lateral deflection of light / gauge steel frame wood panel shear walls of various configurations under monotonic and reversed cyclic lateral loading. He completed 46 tests, which included shear walls of different lengths and with various sheathing materials, including 12.5 mm CSP and 11 mm OSB.

Boudreault *(2005)* provided an extensive summary of reversed cyclic loading protocols available for use with shear walls. Protocols such as 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)* were discussed. Also included in his literature review were summaries of the reversed cyclic protocols used by the following researchers: Karacabeyli & Ceccotti *(1998)*, Karacabeyli *et al. (1999)*, Dinehart & Shenton III *(1998)*, Heine *(2001)*, Gatto & Uang *(2002)* and Landolfo *et al. (2004)*. Boudreault then carried out a suite of 20 shear wall tests composed of specimens sheathed with 12.5 mm CSP and DFP panels. Using the combined data of the 109 tests presented in Branston *et al. (2004)* a procedure to determine force modification factors for seismic design was presented. A value of R_d = 2.5 was recommended for the ductility-related force modification factor for walls with a maximum aspect ratio of 2:1; as well an overstrength-related force modification factor value of R_o = 1.8 was recommended. These R values are for use when designing light gauge steel frame / wood panel shear walls using the loading provisions from the 2005 NBCC and the resistance provisions as obtained through the EEEP method developed by Branston *(2004)*.

As a final chapter of his work, Boudreault reviewed the existing hysteresis models for use in dynamic analyses and commented on their applicability to shear walls. Boudreault recommended that the Stewart hysteretic element *(1987)* be used to model the shear wall experimental data. A calibration of the Stewart model was then completed for all of the shear walls tested by Boudreault, Branston and Chen.

Various wall configurations were included in the data of the 109 tests used by Boudreault, Branston and Chen. However, no test specimens constructed with an OSB sheathing thickness of 9 mm (3/8") were performed. Since OSB of this thickness is commonly used for sheathing, design parameters for walls constructed with 9 mm OSB would prove useful for structural engineers. Therefore, this thesis provides design parameters, ductility-related force modification factors, and an overstrength-related force modification factor for laterally loaded light gauge steel frame / wood panel shear walls constructed with 9 mm (3/8") OSB sheathing. All of these parameters were determined following the relevant approaches recommended by Branston and Boudreault.

2.2 DYNAMIC ANALYSES OF SHEAR WALLS

2.2.1. Della Corte, Fiorino & Landolfo (2005)

The purpose of this experimental research program was to develop reliable mathematical models of the hysteresis behaviour of cold-formed steel stud shear walls and to assess their deformation demands under earthquake ground motions. The deformation demand results were obtained using an ad-hoc mathematical model of the hysteresis response of the wall that takes into account both the non-linearity and pinching behaviour.

The authors used a numerical model, which was adapted from a steel beam-to-column connection model, to develop a hysteresis response for the walls. The hysteretic model was calibrated with the results of physical monotonic tests carried out by the authors and reversed cyclic tests by Serrette *et al. (1996a, 1997a)* as well as COLA-UCI *(2001)*. The cyclic tests were selected according to the geometry and materials of the wall specimens so that they were as close as possible to those of the monotonic tests. Neither the stiffness degradation nor the strength degradation were taken into account because of two experimental observations made by Landolfo *et al. (2004)*: 1) it is only after the peak load that the degradation starts to be noticeable; 2) the load-displacement relationship happens to be unreliable because of its instability.

The prototype structure chosen for analysis was a typical one-storey family house, the details of which are given in Landolfo *et al. (2004).* A short summary of the main characteristics is as follows:

- Stick-built construction;
- Horizontal and vertical diaphragms are made of cold-formed steel covered with structural panels;
- Single degree of freedom prototype structure with the mass at the first storey;
- Mass is assumed to be 1250 kg/m;
- Second order effects are considered;
- Relative damping ratio of 5% is used;
- The Incremental Dynamic Analysis (IDA) technique *(FEMA, 2001)* is used for the seismic analysis of the structure.

The structure was located in Italy, therefore a total of 26 earthquake records from Central Italy were chosen. The stations that recorded the earthquakes were identified as being located in a medium-high seismic region having a peak ground acceleration (PGA) of 0.25g for a 10% probability of exceedance in 50 years. Only in one case was a different PGA used: Taranta Peligna accelerogram with a PGA of 0.35g. The earthquake records were chosen to cover as much as possible the soil classifications found in Eurocode 8 *(prEN 1998-1, 2003).*

The damage-limiting value, which was defined as "the attainment of a limiting value of the inter-storey drift angle beyond which plastic deformations are so large to produce appreciable damage to the structure" *(Della Corte et al., 2005)* was set equal to 0.0035 (lateral displacement over the inter-storey height). This value corresponds to a lateral displacement of 10 mm for a 2500 mm high wall. On the other hand, the collapse-limiting value, which was defined as "the attainment of a limiting value of the inter-storey drift angle beyond which the residual safety of the structure against collapse is assumed negligible" *(Della Corte et al., 2005)*, was obtained by computation. This value was given as the minimum of $d_{u,static} / h$ and $d_{u,dyn} / h$, where $d_{u,static}$ is the lateral displacement achieved at the maximum lateral strength during the monotonic tests, $d_{u,dyn}$ is the lateral displacement achieved to the point on the IDA curve having a tangent with a slope equal to 10% of the initial one, and h is the inter-storey height. For the majority of the cases, $d_{u,static}$ was the controlling parameter as its values were smaller than the $d_{u,dyn}$ values. The numerical results from the IDA concluded that the damage of the structure was negligible under the design intensity earthquakes and that the safety of the structure against collapse was considered acceptable.

The authors defined a loading history for the reversed cyclic tests that was based on the maximum level of deformation of the structure, the cumulative plastic deformation of the structure and the number of repetitions of several discrete values of ductility demands. The new loading history was then applied to the model to obtain the capacity demand of the shear walls. In that particular case, the natural ground motion records chosen were scaled to values corresponding to the chosen ductility demand of 6. The results obtained using this loading history showed lower strength in the model.

In comparison, the up-dated numerical model overestimated the strength at the second and third cycle for each displacement value because it did not take into consideration the strength degradation of the walls. Also the model underestimated the dissipated energy for small lateral displacement and overestimated it for larger displacement. Once again, this was explained by their initial statement, saying that the model was not going to consider degradation.

The summary of their study drew the following conclusions:

- The shear walls meant to remain elastic and designed according to Eurocode 8 performed adequately during the incremental dynamic analysis.
- The safety of the structure under investigation against collapse was considered acceptable.

Therefore light gauge steel stud shear walls with structural panels can be designed in low-to-medium seismic intensity zones.

2.2.2. Fülöp & Dubina (2004a, 2004b)

Fülöp and Dubina wrote two papers (Parts 1 *(2004a)* & 2 *(2004b)*) related to the performance of light gauge steel stud walls under monotonic and cyclic performance. The first paper (Part 1) described the experimental segment of the research which mainly consisted of a test program on one-storey shear walls loaded monotonically and cyclically *(2004a)*. Two types of wall sheathing were used in the research; corrugated sheathing and OSB panels. The walls were 3.66 m (12') in length and 2.44 m (8') in height. The variation between walls with the same sheathing included the addition of gypsum board or a door opening.

In the second paper (Part 2), alternative design methods and hysteretic modeling techniques based on the results of the physical tests presented in Part 1 were introduced *(2004b)*. As a preliminary check, three different types of hysteretic models were chosen; a bilinear, a tri-linear and a non-linear model. The bilinear and the tri-linear models did not take strength degradation into account, however to overcome this limiting factor, both models were based on the stabilized envelope curve of the cyclic tests. On the other hand, the non-linear model was based on the Richard-Abbott type curve *(Richard & Abbott, 1975)*, which needed to be calibrated with relevant experimental results. To evaluate the accuracy of all three models, the Kobe-JMA earthquake records were used in non-linear dynamic analyses. The results of the three analyses were similar, that is the difference in the curves for the accelerogram multiplier vs. displacement were negligible. However, by overlapping the hysteresis of the cyclic tests and the hysteresis of the models, it was clearly shown that the non-linear model was better able to mimic the physical test results. The main characteristics of the shear walls to take into account for all hysteretic models were as follows: pinching, overstrength, and plastic deformation capacity. Therefore, the researchers suggested using both the tri-linear and non-linear hysteretic models for further studies.

A single degree of freedom system (SDOF) pin connected model was constructed with DRAIN-3DX *(Prakash et al., 1994)* and calibrated with the experimental results of Part 1.

The SDOF system was modeled with a fibre-hinge to accommodate for the hysteretic behaviour of the walls. The two columns and the beam that formed the wall frame did not contribute to the load bearing capacity of the walls. The sheathing of the walls was modeled with two pinned diagonal braces, which incorporated a fibre-hinge. The braces were of "TYPE 8" fibre-hinge beam-column elements.

Five earthquake records were chosen and scaled from 0.05 to 2 g. The normalized elastic spectra were compared to the Eurocode 8 elastic spectra for soil conditions A, B and C. Time history analyses were then carried out with the SDOF system, all the tri-linear hysteretic models, for each ground motion record and for masses varying between 2000 and 4000 kg. Damping was not considered, but to account for the second order effects, a vertical force equal to 30% of the weight applied was added to the model. The procedure used was the incremental dynamic analysis (IDA) which is also known as the dynamic pushover (DPO). The output of the IDA was in the form of curves relating a performance parameter of the structure to an intensity measure of the record. The intensity measure was a spectral acceleration (S_a) corresponding to the first mode of vibration of the structure while the performance parameter was the inter-storey drift. For all the wall configurations and ground motion records, ground motion intensity measure levels were identified based on the displacement values. Three displacement limit states corresponding to the state of the wall were considered: the displacement at elastic, yield and ultimate capacity of the walls.

The q value used in seismic design in the Eurocode 8 is the equivalent of the R_d force modification factor *(NRCC, 2005)* used in Canada for seismic design. These values are incorporated in design to reduce the forces obtained during a linear elastic analysis, in an attempt to account for the non-linear response of structures. In their research, Fülöp & Dubina have computed the q-values for all the wall configurations. In the cases involving walls with openings, the ratio of S_a at ultimate capacity and S_a at yield capacity (q_2) was taken as the q-value such that the dissipative capacity caused by the ductility could be incorporated. The q_1-value was set equal to the ratio of S_a at the elastic capacity to S_a at the yield capacity.

The analyses have shown that the overstrength effect was important in the post-elastic behaviour of the walls, and therefore an earthquake-force reduction factor could be substantiated. The results obtained for q_1 varied between 2.2 and 2.6 depending on the wall configuration. However, the q_2-values based on design force reduction caused by the ductility and the energy dissipation were of less importance (1.4 - 1.6).

2.2.3. Ceccotti & Karacabeyli (2000, 2002)

In 2000, Ceccotti and Karacabeyli wrote a report on the appropriateness of the factor *R*, the seismic force modification of the 1995 National Building Code of Canada, for nailed wood-framed shear walls *(Cecotti & Karacabeyli, 2000)*. Using the database, from Forintek Canada Corp., of wood-frame shear walls tested monotonically and cyclically, the performance parameters of the shear walls were obtained. The walls were 2.44 m (8') in height and 4.88 m (16') in length. The types of sheathing used were plywood, oriented strand board (OSB) and/or gypsum wall board (GWB).

A building was designed for the Vancouver BC region in accordance with the 1995 NBCC *(NRCC, 1995)*. The building was selected by the Wood Frame Committee of the Structural Engineering Consultants of BC. However, only one shear wall (parallel to the short dimension of the building) of the building was modeled. The shear wall was four storeys in height and was not subjected to torsional effects. The *R*-values considered in the design and analyses were as follows: Case 1: $R = 3$; Designed and analysed considering plywood only. Case 2: $R = 3$; Designed using plywood only. Analysed using plywood and gypsum wall board. Case 3: $R = 2$; Designed and analysed considering both plywood and gypsum wall board.

In order to evaluate the performance of the modeled shear wall Ceccotti and Karacabeyli *(2002)* established a near collapse criterion based on the inter-storey drift for each wall configuration. The near collapse criterion was chosen to be equal to the displacement when the post-ultimate load reached 80% of the ultimate load value. The pinching hysteresis model developed at the University of Florence *(Ceccotti & Vignoli, 1989)* was used to fit the cyclic test data. However, to ensure the reliability of the model, shake table tests were first completed on one- and two-storey shear walls. Once the hysteresis model was proven to be adequate for use, the model was implemented into DRAIN 2DX *(Powell, 1993)*, the time-history dynamic analysis program that was chosen for the research. It was found that the period of vibration calculated using the 1995 NBCC (T = 0.2 sec) was much less than that obtained in the dynamic analyses (0.65, 0.47, and 0.47s for Cases 1, 2 and 3 respectively). Nonetheless, a period of 0.2s was used for seismic design purposes.

A finite element model of the shear wall was developed, which consisted of four rigid elements per wall. At every corner of the shear wall frames, rotational spring elements were added to simulate the shear deformation of the walls. Therefore, the horizontal displacement of the frame was entirely dependent on the deformation of the spring elements. The stiffness and strength characteristics of the spring elements were obtained from the force-displacement test results. Masses were linked to each floor of the model by rigid elements because the floors were assumed to be rigid.

Twenty-eight ground motion records compatible with the Vancouver region were used for the analyses. The peak ground acceleration (PGA) of each earthquake was set to 0.05g and stepwise scaled upwards until the ultimate displacement, i.e. near collapse criterion, was reached. The acceleration corresponding to the attainment of the ultimate displacement was then identified as A_u. For the R-values assumed in design to be adequate, the PGA values given in the NBCC needed to be R times smaller than A_u.

The median values for A_u were found to be equal to 3 times and to 2 times greater than the PGA of the 1995 NBCC for Case 1 and 3, respectively, which confirmed the R-values for both design scenarios. For Case 2, most values of A_u were not greater than those found for Case 1, therefore an R value greater than 3 was not warranted.

2.2.4. Foliente (1994)

Foliente *(1994)* recommended a listing of general resistance vs. deflection hysteretic characteristics that should be taken into account for timber structures. The observed characteristics were as follows: 1) the inelastic load-displacement relationship, 2) the loss of lateral stiffness in each loading cycle, 3) the strength degradation, and 4) the pinching of loops. It was said that to accurately model the hysteretic behaviour of such walls, the strength and stiffness degradation and the pinching of the resistance vs. deflection model should resemble the behaviour of the physical tests as much as possible. Otherwise, progressive weakening or the general failure of the wall could be reached prematurely.

Short descriptions of nine hysteresis models for wood shear walls were presented. Those created by Stewart *(1987)*, Dolan *(1989)* and Ceccotti and Vignoli *(1990)* were suggested for timber joints and structural systems. However, these models were said to be limited because they relied on a complex set of force-history rules or very limited empirical relations. Hence, a general constitutive model should be preferred over models derived from specific configurations. To overcome the problem, Foliente proposed a more general non-linear model, which should comprise of a dynamic mechanical model with combinations of linear or non-linear springs, non-linear damping, linear viscous damping, and non-linear hysteretic elements. Furthermore, to avoid analytical difficulty, the system should separate the non-linear and the linear components. All the governing equations of such model were presented with the hysteretic shape properties.

Two methods of discretization of structural models are commonly used; these are the concentrated mass method and the finite element method. In the case of the concentrated mass method, single degree of freedom (SDOF) systems are often used for preliminary dynamic analyses as to understand the dynamic behaviour of the system. However, in the case of a multi-storey building, more DOF are needed and therefore a multi degree of freedom (MDOF) system is used. These models are based on the assumptions that the masses are concentrated at the floors, the floor elements are considered rigid, and the lateral deformations are independent of the axial forces in the columns. Therefore, the

MDOF system will have as many DOF as it has floor levels (lumped masses). On the other hand, a combination of plate, framing and connector elements as used by Dolan *(1989)* is a perfect example of a finite element structural model.

There are two different approaches in the evaluation of structural response to dynamic loading that are described in the document. The deterministic approach is the first and most commonly used approach, while the second one is the non-deterministic approach. The type of loading used in a dynamic analysis determines the approach. Therefore if a loading based on time is considered the analysis is said to be deterministic while an unknown (a priori) type of loading will describe the analysis as non-deterministic, which is often referred to as random vibration analysis. Recorded ground motions of past earthquakes used in dynamic analysis is considered as a deterministic approach because a structure designed based on the records of a few earthquakes may behave in a much different way under a real earthquake with varying characteristics. Therefore a large number of ground motion records is needed to estimate the structural response of a design. To represent adequately the random characteristics of natural hazards and the behaviour of a structural system, stochastic mathematical models should be used. With such a model, one can design a structure based on the level of safety desired measured in terms of probability of failure. However, all dynamic analyses done on timber structures, up to now, were based only on deterministic approaches.

It was shown that the stochastic linearization of the modified Bouc-Wen-Baber-Noori (BWBN) hysteretic model *(Baber & Noori, 1986)* was suited for both the deterministic dynamic analysis and the random vibration analysis. In summary, the random vibration analysis was shown to be adequate for use in response analyses of timber structures under natural hazard loadings. The stochastic equivalent linearization technique was sufficient to derive design response values. This method of analysis can be used for MDOF systems as long as the parameters of the hysteretic model are known.

2.2.5. *Summary of Past Dynamic Analysis Research Programs*

All the information gathered from the four research programs presented in the above literature review helped to develop the approach used for the dynamic analyses carried out for this thesis (Chap. 5). First of all, the same collapse criterion as used by Ceccotti and Karacabeyli *(2000, 2002)*, that is walls are considered to fail when the 80% post ultimate shear force is reached, was applied to the shear wall test results. Secondly, the use of a hysteretic model that takes both pinching and degradation into account was to be chosen in order to mimic the hysteresis behaviour of the walls as discussed by Foliente *(1994)*. Since the Stewart element was recommended by Boudreault *(2004)* and corresponded to the specifications listed above, it was also used in this body of research. Thirdly, concerning the modeling of the shear walls; this could be done as previously described by Fülöp and Dubina *(2004b)*, where a stick model with a bracing system that takes all lateral loads and deformation was utilized. The columns and beams of the shear walls would not contribute to the load bearing capacity of the walls *(Fülöp & Dubina, 2004b)* and the floor system in-between storeys should be designed as a rigid diaphragm *(Ceccotti and Karacabeyli, 2002*). Finally, a deterministic approach could be used since it involves less mathematical computation and because this study represents the preliminary stage in the analyses of light gauge steel frame / wood panel shear walls under lateral loading. Therefore, the use of many earthquake ground motions scaled for one specific region of interest in Canada should be used for the non-linear time history dynamic analyses.

CHAPTER 3 SHEAR WALL TESTING

3.1. DESCRIPTIONS OF TESTS

During the summer of 2004 43 light gauge steel frame / wood panel shear walls were tested under lateral in-plane loading in the Jamieson Structures Laboratory of the Department of Civil Engineering and Applied Mechanics at McGill University. Of these tests, the author was responsible for the 18 oriented strand board (OSB) sheathed walls (three configurations). The scope of study was selected such that it added to the database of test results for existing light gauge steel frame / wood panel shear walls subject to lateral earthquake and wind loading. Research by Branston *(2004)* and Chen *(2004)* included walls with 11 mm (7/16") OSB panels, whereas the tests described in this thesis were constructed of 9 mm (3/8") OSB sheathing.

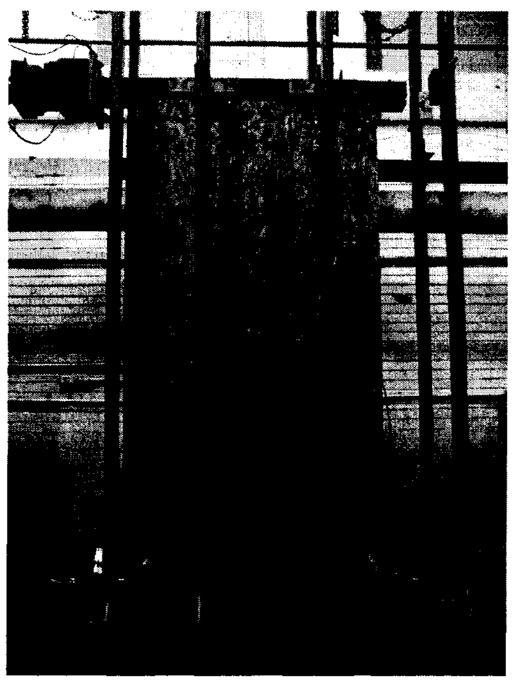


Figure 3.1: Typical OSB sheathed shear wall placed in test frame

The wall specimens were 2440 mm (8') in height and 1220 mm (4') in length. The light gauge steel frame was composed of 1.09 mm (0.043") ASTM A653 *(2002)* Grade 230 (33 ksi) steel. The OSB sheathing was attached to the steel frame with No. 8 sheathing screws at 75 mm (3"), 100 mm (4") and 152 mm (6") spacing around the panel perimeter. Figure 3.1 illustrates a typical shear wall ready for testing. Each of the three wall configurations consisted of six specimens, three of which were tested monotonically and three cyclically using the CUREE protocol for ordinary ground motions *(Krawinkler et al., 2000; ASTM E2126, 2005)*.

All wall specimens were built and tested in a similar fashion to the shear walls included in previous studies by Branston *(2004)*, Chen *(2004)* and Boudreault *(2005)*. Therefore in this Chapter, only an overview of the wall fabrication, the materials and components, the test set-up and the instrumentation used is provided. A comprehensive description of the wall components, construction sequence, instrumentation, testing protocols and data reduction procedure is provided by Branston and Boudreault, and hence is not repeated in this document.

The test data were utilized to determine design capacity, stiffness, energy absorption and ductility parameters, as well as failure modes for the three wall configurations. The design parameters were determined based on measured strengths and displacements of the walls.

3.1.1. Shear Wall Configuration, Materials and Components

The 18 shear wall specimens described herein were constructed of the following components:

- 9 mm CSA O325 *(1992)* Oriented Strand Brand (OSB) rated 2R24/W24 (face strands parallel to framing) (Figure 3.2).
- Light gauge steel ASTM A653 *(2002)* studs: nominal grade of 230 MPa (33 ksi) and thickness of 1.09 mm (0.043"). Nominal dimensions: 92.1 mm (3-5/8") web, 41.3 mm (1-5/8") flange and 12.7 mm (1/2") lip.

- Light gauge steel ASTM A653 *(2002)* tracks: nominal grade of 230 MPa (33 ksi) and thickness of 1.09 mm (0.043"). Nominal dimensions: 92.1 mm (3-5/8") web and 31.8 mm (1-1/4") flange.
- No. 10-16 x 3/4" (19.1 mm) Hex washer head self-drilling screws connecting the back-to-back chord studs. Two screws were used every 305 mm (12") along the studs. Back-to-back chord members were used to avoid compression failure of a single stud. The interior studs were spaced at 610 mm (24") o/c.
- Hold-down connectors: Industry standard Simpson Strong-Tie S/HD10 *(Simpson, 2001)*. Screws connecting the hold-down to the chord studs: 33 No. 10-16 x 3/4" (19.1 mm) long Hex washer head self-drilling screws. Anchor rod connecting the hold-down to the test frame: ASTM A307 *(2003)* 7/8" (22.2 mm) threaded rod
- Steel framing screws: No. 8 x 1/2" (12.7 mm) long wafer head self-drilling screws.
- Sheathing screws: No. 8 x 1-1/2" (38.1 mm) long Grabber SuperDrive *(SuperDrive, 2003)* bugle head self-piercing screws. The screws were placed at 12.7 mm (1/2") from the edge of the sheathing. The spacing between individual screws along the perimeter of the panel was either 75mm (3"), 100 mm (4") or 152 mm (6"). A screw spacing of 305 mm (12") was used to connect the sheathing to the inner stud.

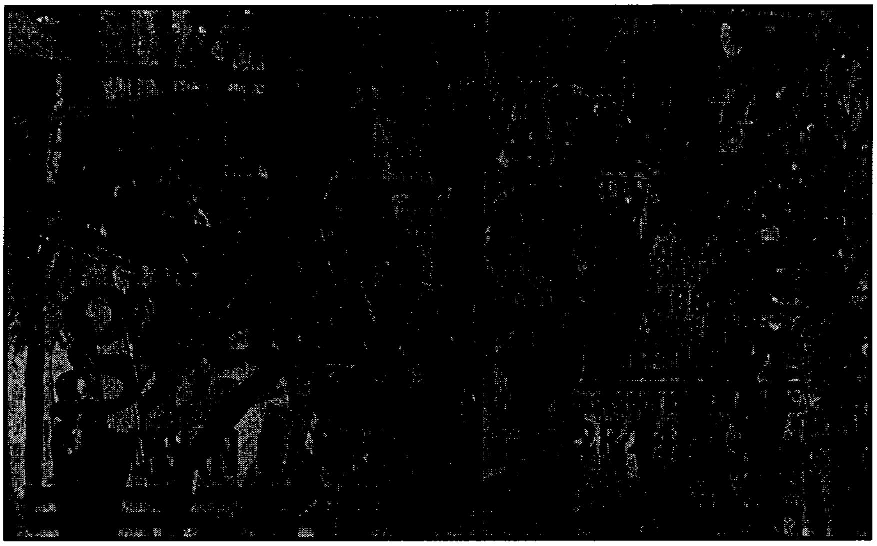


Figure 3.2: Mill and grade stamp of OSB sheathing panels

For each of the three wall configurations six tests were performed (3 monotonic and 3 cyclic) to provide a minimum level of reliability within the test data (Table 3.1). The monotonic loading protocol from Serrette *et al. (1996b)* was used, while the CUREE protocol for ordinary ground motions *(Krawinkler et al., 2000)* was used for the reversed cyclic tests. These same protocols were recommended for use by Boudreault and incorporated in the previous shear tests done at McGill University *(Boudreault, 2005; Branston, 2004; Branston et al., 2004, Chen, 2004)*. It was felt that the CUREE reversed cyclic protocol best represents the demand placed on a light framed shear wall during a design level earthquake with a probability of exceedance of 10% in 50 years.

Table 3.1: Light gauge steel frame / wood panel shear wall test program matrix

Specimen ID	**Loading Protocol**	**Length of Wall (mm)**	**Height of Wall (mm)**	**Panel Type**	**Thickness of Panel (mm)**	**Fastener Schedule (mm)**
41– A,B,C	Monotonic	1220	2440	OSB	9	152/305
42– A,B,C	CUREE	1220	2440	OSB	9	152/305
43– A,B,C	Monotonic	1220	2440	OSB	9	100/305
44– A,B,C	CUREE	1220	2440	OSB	9	100/305
45– A,B,C	Monotonic	1220	2440	OSB	9	75/305
46– A,B,C	CUREE	1220	2440	OSB	9	75/305

3.1.2. Shear Wall Fabrication

Prior to the wall fabrication, all the 1220 mm (4’) long bottom and top tracks were drilled to accommodate the shear anchors and the hold-downs. Six 3/4” ASTM A325 *(2002)* bolts were used to transfer the load from the loading beam to the top track. For the bottom track two 7/8” ASTM A307 *(2003)* threaded rods were used for the holddowns, as well as two 3/4” ASTM A325 *(2002)* for the shear anchors (Figure 3.3).

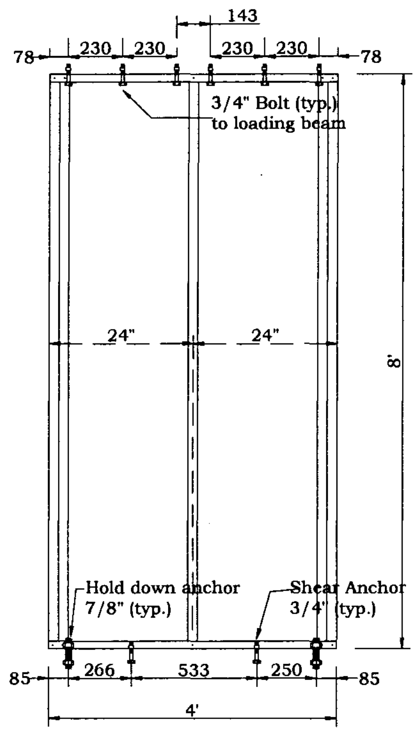


Figure 3.3: Location of shear bolts and hold-downs *(Branston, 2004)*

The back-to-back chord stud members were connected using two No. 10-16 Hex washer head self-drilling screws at 305 mm (12") on center. Hold-downs were installed at one end of the built-up members using 33 No. 10-16 x 3/4" Hex washer head screws. After fabrication of the back-to-back chord studs, the frame was assembled on the ground using the top and bottom tracks, chord studs and the intermediate stud. Figure 3.4 shows the installation of the track to stud screw fastener.

Figure 3.4: Chord studs and bottom track connection *(Branston, 2004)*

Upon completion and alignment of the frame, the wood panel was installed according to the screw schedule desired. A schematic drawing of a typical wall with a screw spacing of 75 mm (3") along the perimeter and 305 mm (12") along the intermediate stud is shown in Figure 3.5.

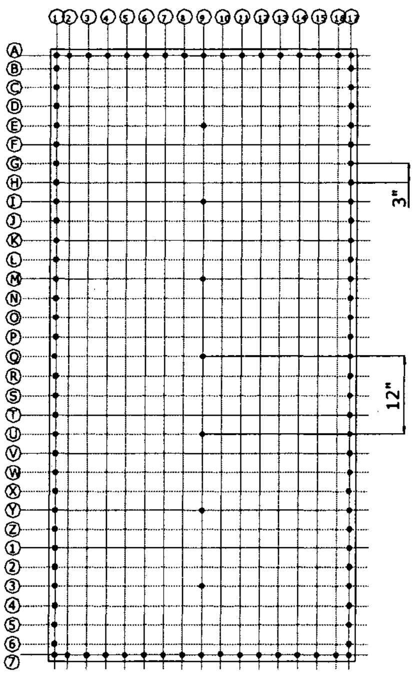


Figure 3.5: Screw spacing for 75 mm / 305 mm (3"/12") schedule *(Branston, 2004)*

Prior to installation of the wood sheathing, the moisture content was taken at five different locations using an electronic moisture meter (Delmhorst Instrument Co. RDM-2 *(Delmhorst, 2003)*) to ensure that the average moisture content was below 10%. After testing, two specimens of wood of 75 mm (3") in diameter, were taken from the wood panel. APA Test Method P-6 *(APA PRP-108, 2001)* was followed to obtain the actual moisture content at the time of testing. The moisture content (*MC*) was calculated according to the following equation:

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

where,

MC = moisture content (%)

W_w = initial weight (g)

W_d = oven-dry weight (g)

The moisture content recorded for the 18 wood panels used in the construction of the test walls varied from 4.4 to 5.6 %, well below the 10% level.

3.1.3. Test Set-Up

The walls were tested using the specially constructed test frame illustrated in Figures 3.6 & 3.7. The frame was equipped with a 250mm (± 125mm) stroke dynamic actuator and a 250 kN load cell. Lateral movement of a test wall was restrained by the vertically positioned HSS braces. Each test wall was moved into the frame, aligned vertically and in-line with respect to the load cell and loading beam, and then bolted in place. Also, two load cells were installed on the hold-down rods to measure the uplift forces. For all shear anchors a steel plate washer of 4.8 mm x 63.5 x 63.5 mm (3/16" x 2.5" x 2.5") was used.

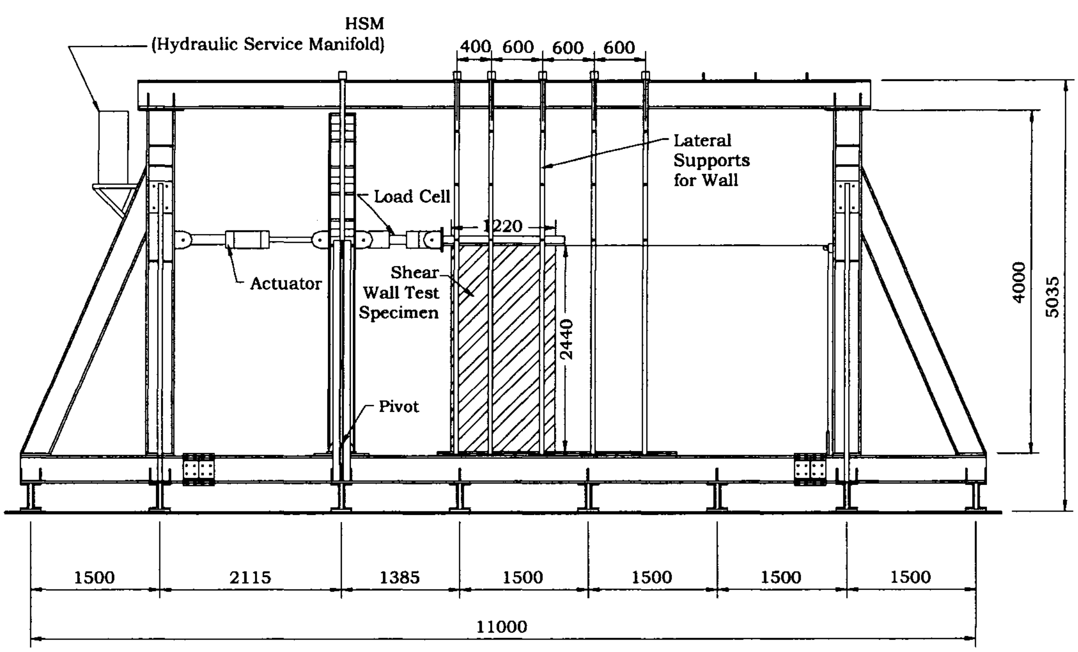


Figure 3.6: Test frame with 1220 × 2440 mm (4'×8') wall specimen

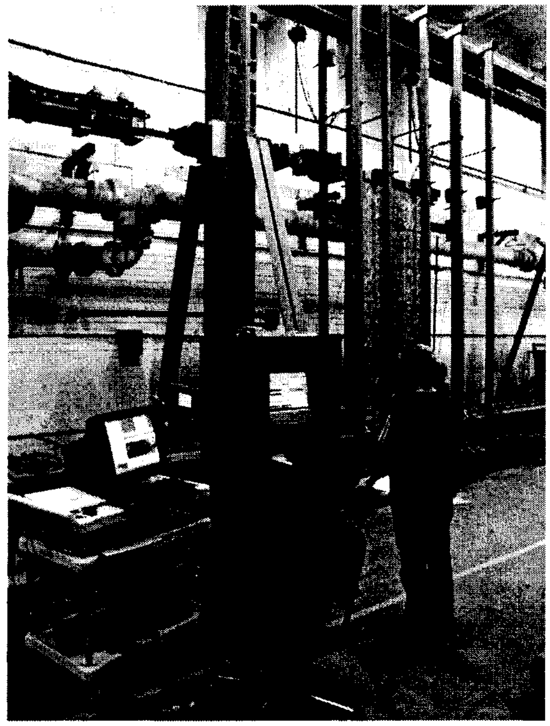


Figure 3.7: Wall specimen in test frame

All shear bolts were tightened using an electric impact wrench with a capacity of 0.4 kN-m (300 ft-lbs). The hold-downs were installed following the instructions contained in the manufacturer's literature *(Simpson, 2001)*, that is they were first secured to finger tight and then turned an additional half-turn with a wrench.

3.1.4. Instrumentation and Data Acquisition

The behaviour of each shear wall was monitored throughout testing by means of measured loads, displacements and accelerations. In all, eleven transducers (LVDTs) were directly connected to the wall specimen measuring the uplift (2 LVDTs) and slip (2 LVDTs) at bottom corners, the in-plane lateral wall displacement (1 LVDT) and the displacement of the wood panel relative to the wall frame (4 LVDTs) (Figure 3.8). Two other LVDTs were used to record any out-of-plane movement of the shear wall. In addition two LVDTs were installed to measure the shear deformation of the wood sheathing (Section 3.5).

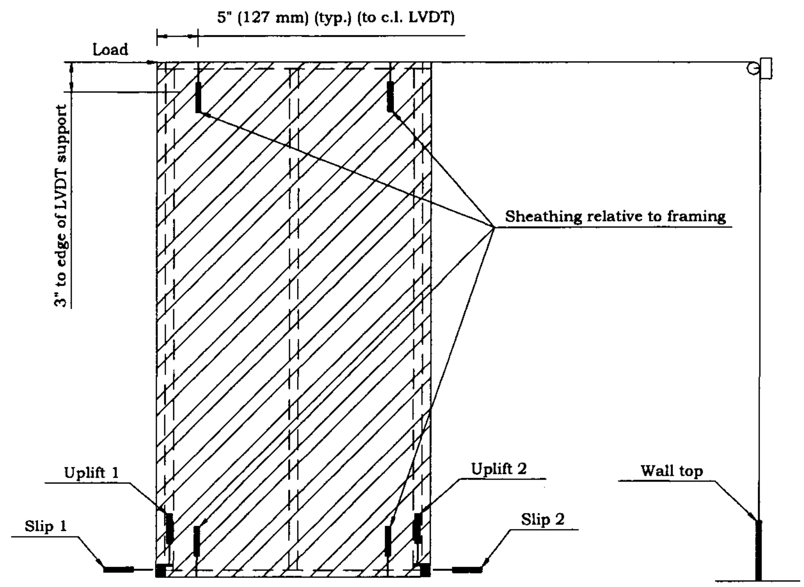


Figure 3.8: Positioning of LVDTs on wall specimen *(Branston, 2004)*

In addition to the transducers, an accelerometer and three load cells were also used to monitor the wall. Two of the load cells were installed at the hold-down rods while the other was attached to the loading beam. The accelerometer, which was attached to of the main load cell, was relied on to measure the acceleration at the top of the wall during reversed cyclic tests.

All of the measuring devices were connected to Vishay Model 5100B scanners to record data. Vishay System 5000 StrainSmart software was used to control the data acquisition system. Data for the monotonic tests were recorded at 2 scans per second and for the reversed cyclic tests at 50 scans per second.

3.1.5. Monotonic Tests

As previously stated, all monotonic tests were carried out following the Serrette *et al. (1996b)* protocol. The wall top was subjected to a constant unidirectional in-plane displacement of 7.5 mm per minute to simulate a static loading similar to a constant wind on a structure. It was also necessary to carry out the monotonic tests in order to establish the displacement amplitudes used in the reversed cyclic protocol. Twice during the testing of a wall, the permanent offset was evaluated. To do so, the wall was unloaded to zero load at 12.5 mm and at 38.0 mm, which represent a storey drift of h/200 (0.5%) and h/64 (1.5%), respectively. The test was stopped when there was a significant drop in resistance of the wall. Figure 3.9 shows a typical wall resistance vs. deflection curve for a monotonic test.

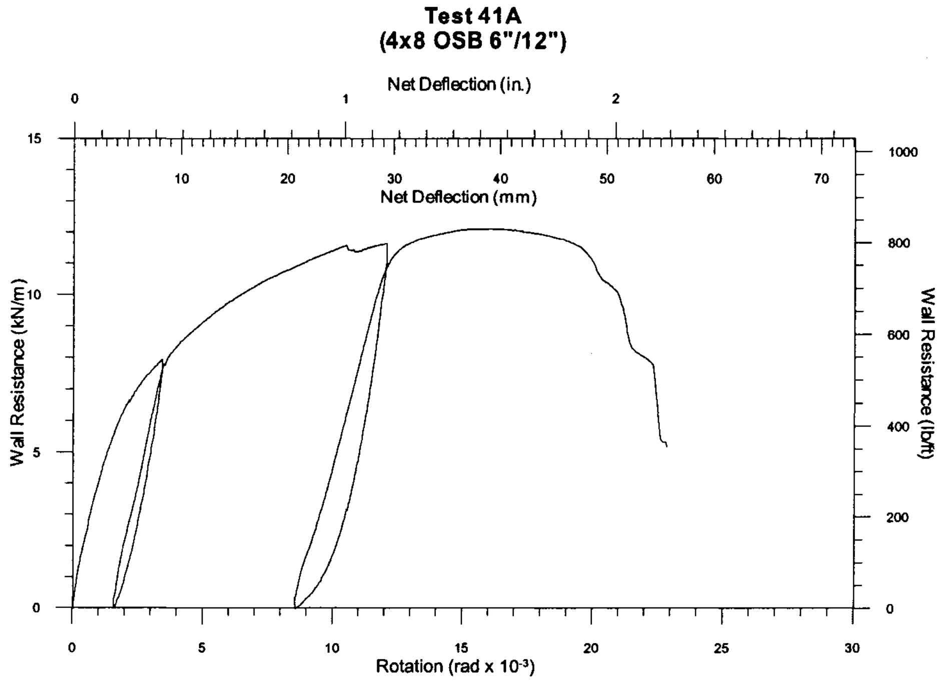


Figure 3.9: Typical wall resistance curve for a monotonic test

3.1.6. Reversed Cyclic Tests

The CUREE protocol for ordinary ground motions *(Krawinkler et al., 2000; ASTM E2126, 2005)* was chosen for the reversed cyclic tests. A study by Boudreault *(2005)* concluded that this protocol, which is based on the results of non-linear dynamic time history analyses of structures constructed of wood frame shear walls, is representative of the expected demand to be imposed on steel frame / wood panel shear walls during an earthquake. The protocol was developed to represent ordinary ground motions (not near-fault) whose probability of exceedance in 50 years is 10 %. It should be noted that the equivalent energy elastic-plastic (EEEP) analysis method (presented in Chapter 4) is not dependant upon the loading protocol imposed on the specimen, however, the loading protocol can have a significant effect on the design values obtained from the test results. The loading protocol should reflect the expected demand from a design level earthquake that may occur during the lifetime of a structure.

The CUREE reversed cyclic protocol is based on a reference displacement, which is a function of the deformation recorded during a monotonic test. The monotonic deformation capacity (Δ_m) is defined as the wall top displacement observed when the post-peak wall resistance is reduced to 80% of the ultimate shear resistance ($0.8S_u$). This $0.8S_u$ resistance level is considered to be the failure point of the test wall; that is the end of its useful load carrying capacity. The reference deformation, Δ, is then obtained by multiplying Δ_m by γ, where γ is equal to 0.6.

The protocol contains three types of cycles, the first of which is called the initiation cycles. These cycles fall within the assumed linear range of behaviour of the walls because they are of small amplitude. The second type of cycle is called the primary cycles. These are of higher amplitude than any other cycles preceding them, and hence, they enter into the non-linear range of behaviour of the wall. The last type of cycle is called trailing cycles. They are equal to 75% of the amplitude of the preceding primary cycle. As an example the complete loading history for walls with the 152/305 mm (6"/12") connection configuration, including the initiation, primary and trailing cycles, is shown in Table 3.2 and Figure 3.10. All reversed cyclic tests were carried out in displacement control at a frequency of 0.5 Hz to avoid excessive inertial effects induced by the mass of the wall and the surrounding components such as the loading beam and load cell.

Table 3.2: CUREE Reversed Cyclic Testing Protocol (Displacement Controlled)

Screw Pattern:	152"/305"	Sheathing:	OSB
	Δ=0.6*Δm	33.34	
	Target (corr.)	Actuator Input	
Displ.	(mm)	(mm)	No. Of cycles
0.050 Δ	1.667	1.988	6
0.075 Δ	2.500	2.982	1
0.056 Δ	1.875	2.236	6
0.100 Δ	3.334	3.976	1
0.075 Δ	2.500	2.982	6
0.200 Δ	6.668	7.951	1
0.150 Δ	5.001	5.964	3
0.300 Δ	10.001	11.927	1
0.225 Δ	7.501	8.945	3
0.400 Δ	13.335	15.903	1
0.300 Δ	10.001	11.927	2
0.700 Δ	23.336	27.830	1
0.525 Δ	17.502	20.873	2
1.000 Δ	33.338	39.757	1
0.750 Δ	25.003	29.818	2
1.500 Δ	50.006	59.636	1
1.125 Δ	37.505	44.727	2
2.000 Δ	66.675	79.515	1
1.500 Δ	50.006	59.636	2

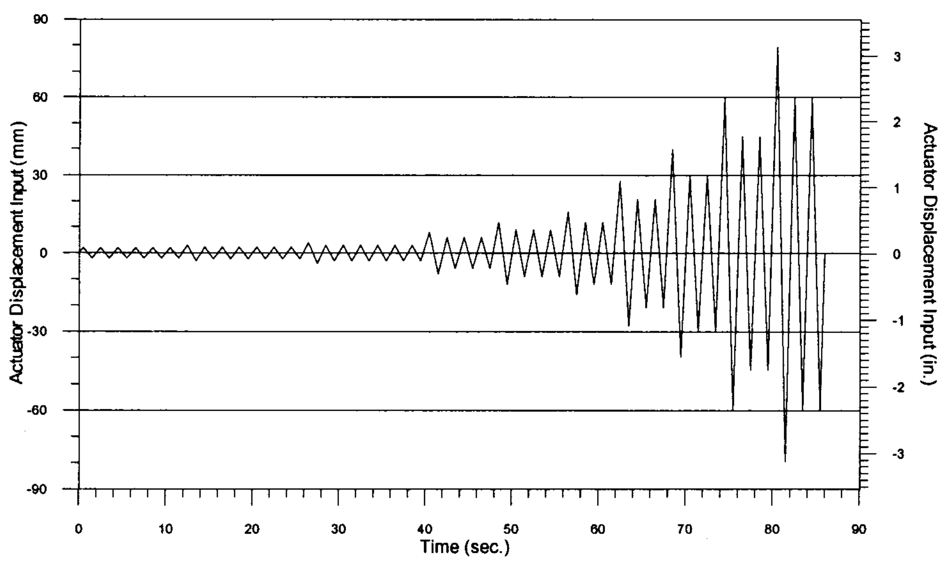


Figure 3.10: CUREE ordinary ground motions protocol for shear wall tests 42-A,B,C 1220 × 2440 mm (4'×8') OSB 152/305 mm (6"/12")

For each monotonic test of a particular configuration the average deformation capacity (Δ_m) was calculated. It is important to note that the displacements were corrected for the uplift and slip of the wall to obtain the Δ_m values. The reference deformation, Δ, was then obtained by multiplying the average Δ_m by 0.6. Once the Δ was known, the target displacements were calculated to obtain the sequence of initiation, primary and trailing cycles. Note that in Table 3.2, the displacement values in the "actuator input" column differ from the "target" column because corrections are made to account for the uplift and slip of the wall, as well as the vertical movement of the actuator. A linear relationship between the "actuator input" and "target" displacement was obtained from the monotonic test data. Figure 3.11 shows a typical wall resistance vs. deflection hysteresis obtained by using the CUREE protocol. The three reversed cyclic CUREE protocols can be found in Appendix 'A'.

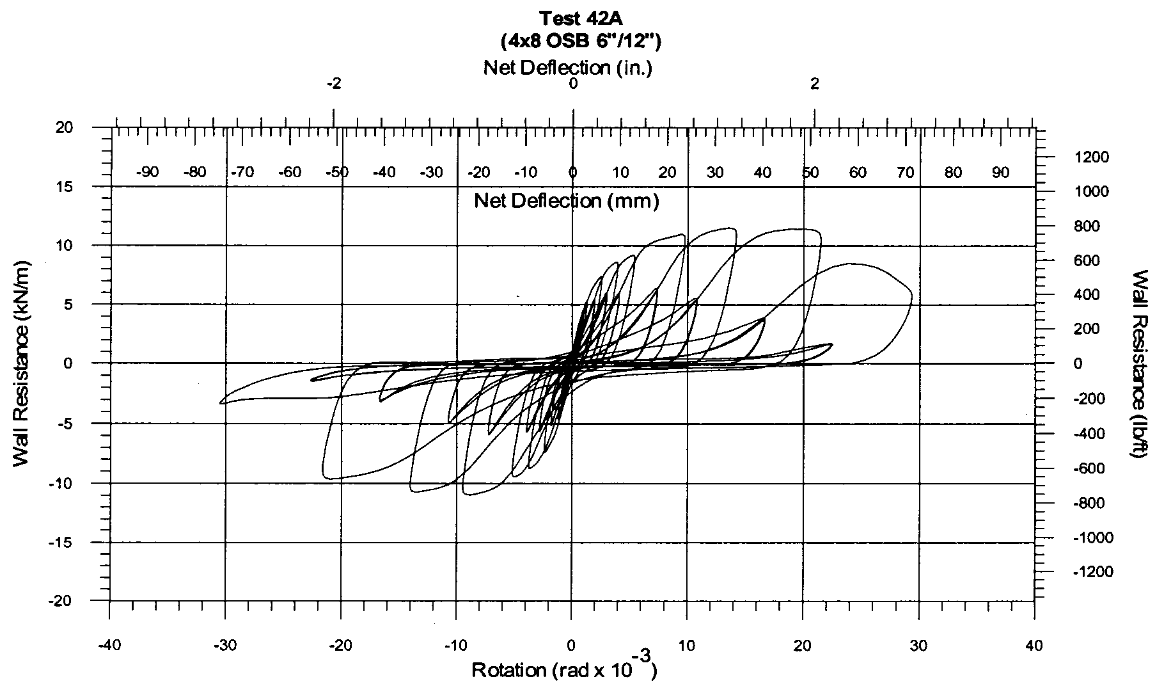


Figure 3.11: Typical wall resistance vs. deflection curve for a reversed cyclic test

3.2. TEST RESULTS / ANALYSIS OF TEST DATA

Prior to calculating the properties of each individual wall corrections needed to be made to the measured data to compensate for slip (rigid body translation) and uplift of both ends of the wall (rigid body rotation), as well as inertial effects. These modifications affected the top of the wall in-plane displacement of both cyclic and monotonic tests and the wall resistance for cyclic tests only.

3.2.1. Data Correction

The wall top displacement as measured by the LVDT did not represent the net lateral in-plane displacement because the wall also slipped and rotated (Figure 3.12). Since a gravity load was not applied to the test wall it was felt necessary to correct for the uplift and slip of the wall. With the use of the four LVDTs placed to measure the slip and the uplift at both ends of each wall specimen, it was possible to determine the net lateral in-plane displacement, as shown in Equation 3-2.

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

where,

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

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

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

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

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

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

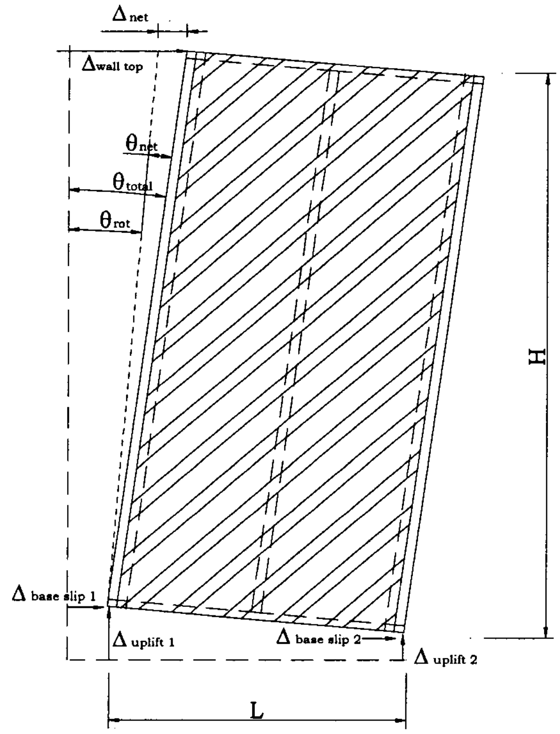


Figure 3.12: Deformed configuration of shear wall *(Branston, 2004)*

The rotation of the wall was obtained using Equation 3-3:

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

where,

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

Δ_{net} = Net lateral in-place displacement (Eq.3-2)

Since the shear flow through the wall is of interest Equation 3-4 was used to convert the force (kN) into shear (kN/m).

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

where,

S = Shear resistance of the wall, [kN/m]

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

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

However, in the case of the reversed cyclic tests the shear resistance of the wall needed to be modified because of the inertial effects. Using the measurements provided by the accelerometer and the mass of the loading beam assembly, which was equal to 200 kg, the reduced shear resistance of the wall could be obtained using Equation 3-5:

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

where,

S' = Corrected wall resistance, [kN/m]

S = Measured wall resistance (Eq.3-4)

a = Measured acceleration, [g]

m = mass, [200 kg]

3.2.2. Energy Dissipation

In terms of their ability to resist the repeated lateral loading associated with an earthquake, shear wall performance can be evaluated in part by the amount of energy that is dissipated during testing. A sufficient amount of energy should be absorbed by a shear wall to indicate that it would be able to sustain its load carrying resistance under repeated displacement cycles. In the case of the tested shear walls, the total dissipated energy can be determined from the resistance versus net deflection graphs. Numerically speaking, the energy is equal to the area within the resistance vs. deflection curve or hysteresis (Figures 3.13 and 3.14).

For both types of loading the corrected data points were used to obtain the total dissipated energy, as follows:

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

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

where,

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

$F_{i,\, i-1}$ = Corrected wall resistance at data points (i) and (i-1), [kN]

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

E = Total energy dissipated, [J]

It is important to specify that for the monotonic tests, failure was considered to have occurred when the load, in the post-peak range, decreased to 80% of the ultimate value. Hence, the energy calculation was carried out up to $\Delta_{net,\, 0.8u}$ displacement (Figure 3.13). For the reversed cyclic tests, the dissipated energy is equal to the sum of the area enclosed by every hysteretic loop for the complete loading protocol (Figure 3.14), therefore Equations 3-6 and 3-7 can be applied directly.

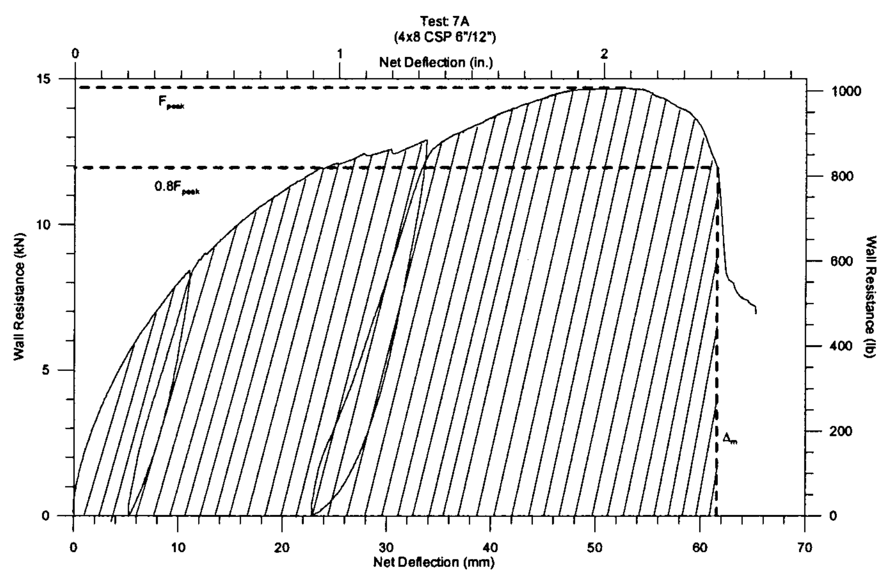


Figure 3.13: Energy dissipation for a monotonic shear wall specimen *(Branston, 2004)*

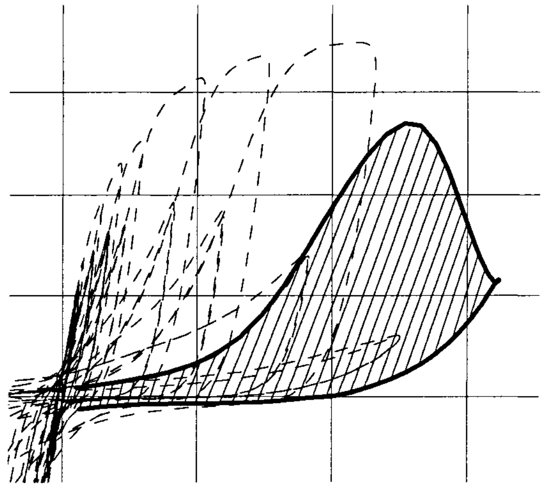


Figure 3.14: Energy dissipation for a reversed cyclic test is represented by the area enclosed by the hysteretic loops (single example loop shown in bold) *(Branston, 2004)*

3.2.3. General Test Results

Tables 3.3 and 3.4 list the test results including: maximum wall resistance (S_u), displacement at $0.4S_u$ ($\Delta_{net,\ 0.4u}$), displacement at S_u ($\Delta_{net,\ u}$), displacement at $0.8S_u$ ($\Delta_{net,\ 0.8u}$), rotation at S_u ($\theta_{net,\ u}$), rotation at $0.8S_u$ ($\theta_{net,\ 0.8u}$), energy dissipation (E) for the monotonic tests; maximum wall resistance for both positive and negative cycles ($S_u'{}_+$ and $S_u'{}_-$), displacement at $S_u'{}_+$ and $S_u'{}_-$ ($\Delta_{net,\ u+}$ and $\Delta_{net,\ u-}$), rotation at $S_u'{}_+$ and $S_u'{}_-$ ($\theta_{net,\ u+}$ and $\theta_{net,\ u-}$), and energy dissipation (E) for the reversed cyclic tests. All displacement measurements and wall resistance values (cyclic tests only) have been modified following the correction method described in Section 3.2.1. A detailed description of all shear wall test results, including graphs, test data sheets and test observations can be found in Appendix 'B'.

One can observe from the general test results that the ultimate shear wall resistance measured for the cyclic tests is lower than that obtained for the monotonic tests. This decrease in strength is due to the repetitive motion of the reversed cyclic protocols. In fact, the stiffer a wall is the more apparent the decrease in strength due to cyclic loading. Hence walls with a screw schedule of 75/305 mm (3"/12") tested cyclically exhibited an ultimate strength that was approximately 11.6 % lower than measured for walls tested monotonically. Walls with a screws spaced at a greater distance, i.e. 152/305 mm

(6"/12"), have only a 4.2 % decrease in ultimate strength. A decrease of 8% in the measured ultimate load, based on data from a combination of all three screw spacings, was recorded by Chen *(2004)*. The decrease in S_u that was observed for these 18 tests is in the same range as that obtained in previous testing of shear walls using the CUREE reversed cyclic protocol.

As the walls are loaded cyclically some damage occurs at the connections (wood being crushed), which decreases the wall resistance of the successive cycle. The same phenomenon explains the lower shear resistance of the walls during the negative cycles (S_{u-}') since the positive cycles were executed first in the protocol. Both the shear resistance and energy values increased as the screw spacing distance decreased, which was expected given the information provided by Chen *(2004)*. Another important observation is that the energy dissipation values obtained for the two testing protocols are very different. In terms of the monotonic tests the energy is equal to the area underneath the resistance vs. displacement curve, while for the cyclic tests the energy is determined based on the area enclosed by every loop in the protocol as explained in Section 3.2.2. Therefore the energy computed for a cyclic test is cumulative, and hence is much larger than a monotonic test since the loops are partially superimposed. The energy dissipation could only be directly compared if the backbone curve (Chapter 4) were used for the evaluation of cyclic test data. The values presented in Tables 3.3 & 3.4 will be further discussed in Chapter 4, which deals with the development of the recommended design values.

Table 3.3: Test results for monotonic tests

Test	Panel Type	Fastener Schedule (mm)	Maximum Wall Resistance(S_u) (kN/m)	Displacement at $0.4S_u$ ($\Delta_{net,0.4u}$) (mm)	Displacement at S_u ($\Delta_{net,u}$) (mm)	Displacement at $0.8S_u$ ($\Delta_{net,0.8u}$) (mm)	Rotation at S_u ($\theta_{net,u}$) (rad x 10^{-3})	Rotation at $0.8S_u$ ($\theta_{net,0.8u}$) (rad x 10^{-3})	Energy Dissipation (E) (Joules)
41A	OSB	152/305	12.1	3.0	39.2	51.4	16.1	21.1	633
41B	OSB	152/305	11.9	2.9	45.1	62.0	18.5	25.4	784
41C	OSB	152/305	12.0	2.2	40.6	53.5	16.7	21.9	687
AVERAGE			**12.0**	**2.7**	**41.6**	**55.6**	**17.1**	**22.8**	**701**
43A	OSB	100/305	17.7	3.6	39.4	47.7	16.2	19.6	847
43B	OSB	100/305	18.0	3.9	44.4	59.3	18.2	24.3	1066
43C	OSB	100/305	19.6	4.5	39.5	44.0	16.2	18.1	818
AVERAGE			**18.4**	**4.0**	**41.1**	**50.4**	**16.9**	**20.6**	**910**
45A	OSB	75/305	23.7	4.8	40.2	45.2	16.5	18.6	1037
45B	OSB	75/305	24.3	5.4	44.7	53.6	18.3	22.0	1266
45C	OSB	75/305	24.4	4.9	39.6	46.4	16.2	19.0	1087
AVERAGE			**24.2**	**5.0**	**41.5**	**48.4**	**17.0**	**19.8**	**1130**

Table 3.4: Test results for reversed cyclic tests

Test	Panel Type	Fastener Schedule (mm)	Maximum Wall Resistance ($S_u'_+$) (positive cycle) (kN/m)	Displacement at $S_u'_+$ ($\Delta_{net,\ u+}$) (mm)	Rotation at $S_u'_+$ ($\theta_{net,\ u+}$) (rad x 10^{-3})	Maximum Wall Resistance ($S_u'_-$) (negative cycle) (kN/m)	Displacement at $S_u'_-$ ($\Delta_{net,\ u-}$) (mm)	Rotation at $S_u'_-$ ($\theta_{net,\ u-}$) (rad x 10^{-3})	Energy Dissipation (E) (Joules)
42A	OSB	152/305	11.5	33.0	13.5	-10.9	-21.6	-8.9	3372
42B	OSB	152/305	11.1	33.6	13.8	-10.9	-22.2	-9.1	3256
42C	OSB	152/305	11.9	32.5	13.3	-11.7	-22.2	-9.1	3334
AVERAGE			**11.5**	**33.0**	**13.5**	**-11.2**	**-22.0**	**-9.0**	**3321**
44A	OSB	100/305	18.2	48.2	19.8	-16.6	-31.5	-12.9	4595
44B	OSB	100/305	16.2	42.7	17.5	-15.3	-30.3	-12.4	4168
44C	OSB	100/305	17.3	46.6	19.1	-16.4	-29.8	-12.2	4705
AVERAGE			**17.2**	**45.8**	**18.8**	**-16.1**	**-30.5**	**-12.5**	**4489**
46A	OSB	75/305	22.4	40.0	16.4	-20.5	-28.3	-11.6	5060
46B	OSB	75/305	21.5	31.1	12.8	-20.3	-32.4	-13.3	4816
46C	OSB	75/305	20.5	30.9	12.7	-20.0	-31.1	-12.7	4187
AVERAGE			**21.4**	**34.0**	**14.0**	**-20.3**	**-30.6**	**-12.5**	**4687**

3.3. FAILURE MODES

Overall failure of each of the 18 shear walls was attributed to localized failure of the sheathing to steel frame screwed connections. In no test did the chord studs fail by buckling, nor did the shear anchors or holddowns become damaged. The sheathing connection failures can be described as follows:

1. Pull-through sheathing (PT)

   During testing the screws tilted sideways under shear loads, which caused the holes in the wood sheathing to enlarge. If the damage to the wood was extensive enough then the head of the screw would pull through the sheathing (Figure 3.15). This phenomenon was even more predominant in the cyclic tests, since the tilting action was repeated back and forth, which caused greater damage to the wood panel at the connection locations.

Figure 3.15: Screw pulling through the sheathing

2. Partially pull-through sheathing (PPT)

The tilting or rocking action of the screws, as described above, was not extensive enough to result in the complete pull-through of the sheathing screws (Figure 3.16). If additional loading cycles had been applied it is likely that the pull-through failure mode would have occurred.

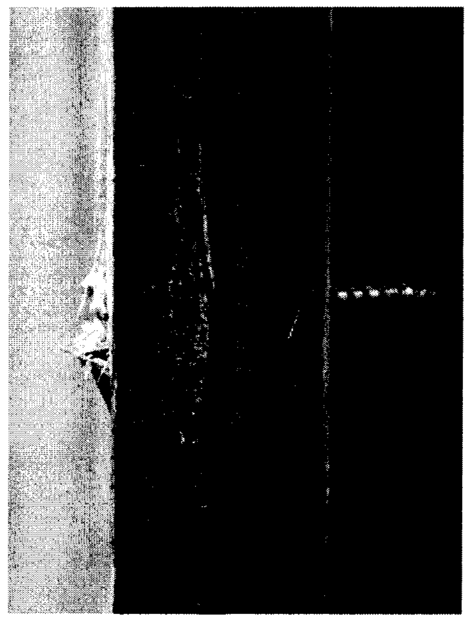


Figure 3.16: Screw head within the OSB panel

3. Fatigue shear failure (FF)

In a limited number of cases the sheathing screws failed in shear, typically at the wall corners or at the ends of the central stud member (Figure 3.17). At these particular locations the sheathing screws penetrated through two layers of steel, which limited the rocking action. Because these screws did not tilt they were required to resist a shear force instead of a tension force, which resulted in their failure.

Figure 3.17: Bottom corner screw failed in shear (Head still visible in the wood panel)

4. Wood Bearing Failure (WB)

In walls sheathed with OSB panels the wood bearing and tear-out failure modes were essentially the same. The high bearing stresses at the screw locations resulted in a tearing-out of the strands as is pictured in Figure 3.18. This failure mode was especially common at the panel corners.

Figure 3.18: Wood bearing failure mode

The overall failure of a test wall was reached when a group of adjacent screws on at least one edge of the wall failed in one or a combination of the fastener failure modes presented in this section. Figure 3.19 shows an example where all of the sheathing screws along the bottom track and a portion of the screws along the chord stud became detached from the OSB panel. This "unzipping" group of the sheathing fasteners is typical for steel frame / wood panel shear walls. The field fasteners (interior of panel) rarely exhibited any type of damage. Also, contrary to the shear walls sheathed with 9.5 mm (3/8") CSP panels *(Rokas, 2005)* the OSB sheathed walls did not exhibit any elastic shear buckling of the wood panel. Rather the OSB walls described in this thesis behaved, in terms of failure modes, in a similar fashion to those tested by Branston *(2004)*, Chen *(2004)* and Boudreault *(2005)*.

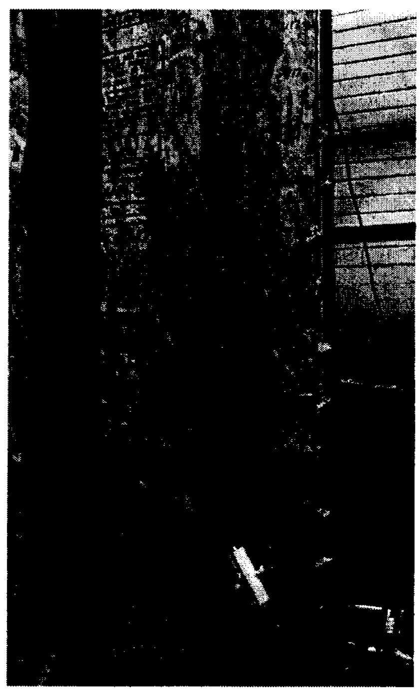


Figure 3.19: Overall wall failure due to failure of multiple sheathing connections

3.4. Ancillary Testing of Materials

Material properties of the OSB panels as well as the steel studs and tracks were measured and are summarized in this section. Information regarding the ultimate shear strength and the shear modulus for the wood panels is provided, in addition to the yield stress, the ultimate tensile stress, the modulus of elasticity (E) and the percentage of elongation of the steel members.

3.4.1. OSB Wood Panel Properties

These ancillary tests were carried out following the edgewise shear test prescribed in ASTM Standard D1037 *(1999)* Sections 130 to 136. Six OSB specimens of 254 × 90 mm (10" × 3.5") in size, three of which were aligned parallel to the grain of the outermost strands and three of which were perpendicular to the strands, were used for the tests. The specimens were clamped using a two rail loading setup as seen in Figure 3.20. Loads were applied at 0.5 mm/min using an MTS® Sintech 30/G universal testing machine to which a 150 kN load cell was attached. The shear displacement of the wood was recorded with an LVDT positioned in line with the loading rails. A Vishay Model 5100B scanner and Vishay System 5000 StrainSmart software were used for data acquisition.

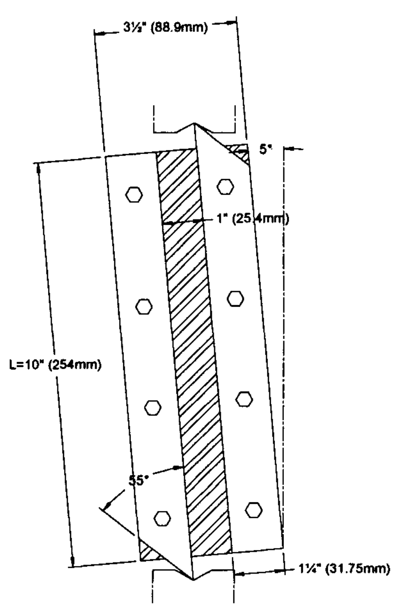


Figure 3.20 Edgewise shear setup *(Boudreault, 2005)*

The following two equations from ASTM Standard D1037 were used to obtain the ultimate shear resistance (v_p) (Eq.3-8) and the modulus of rigidity (G) (Eq.3-9) of each wood specimen.

$$v_P = \frac{P_{max}}{L \times t} \tag{3-8}$$

$$G = 1.19 \times \frac{P \times b}{L \times t \times r} \tag{3-9}$$

$$B_v = G \times t \tag{3-10}$$

where,

v_P = Edgewise shear strength, [kPa]

P_{max} = Maximum compressive load, [kN]

L = Length of the coupon test, [mm]

t = Thickness of the coupon test, [mm]

G = Modulus of rigidity, [MPa]

b = Width of member in shear, [25.4 mm]

P = Compressive load taken up to 40% of P_{max}, [N]

r = Displacement at load P, [mm]

B_v = Shear through thickness rigidity [N/mm]

It is important to note that the modulus of rigidity (G) obtained with the ASTM Standard D1037 doesn't account for the non-uniform stress distribution associated with this small-scale test unless a 1.19 factor is included (Eq3-9). According to ASTM D2719 *(1994)*, this factor provides values comparable to those that would be obtained if larger specimens had been used for the shear tests.

In Table 3.5, both the experimental and the CSA O86 *(2001)* shear values are shown. The CSA O86 is the referenced wood design standard in Canada and therefore all wood products available in the country are listed in the design code with their respective specified strength. Hence the experimental values obtained from the coupon tests should be representative of what it is listed in the CSA O86 as proven with the following Table.

Table 3.5: Experimental and CSA O86 shear properties of OSB panels

OSB 9 mm	CSA O86	Experimental Data	Exp. Corr.[1]	Difference (%)
v_P (MPa)	4.42	9.04	4.52	2.2
G (MPa)	1052	1096		4.2
B_v (N/mm)	10000	10148		1.5

[1] Load modification factor of 2 applied to experimental shear strength values to account for short duration of the test and safety;

The values shown in Table 3.5 are based on the average of the results for the parallel and perpendicular experimental data. This approach was taken because the results were very similar for the OSB specimens in the two directions, which is consistent with the behaviour observed by Boudreault *(2005)*. The edgewise shear strength (v_P) calculated from the experimental data was divided by a factor of two to account for the difference in the rate of loading of the wood specimens in the lab compared to the real life scenario, and for other safety factors as well *(Boudreault, 2005; Parasin & Stieda, 1985)*

3.4.2. Light Gauge Steel Properties

In addition to the wood panels, the light gauge steel studs and tracks were also tested to determine their material properties. Following the ASTM A370 Standard *(2002)*, five coupons were tested. The studs and tracks were rolled from the same coil of steel, hence only one set of material properties is presented. The coupons were tested at a cross-head speed of 0.5 mm/min until plastic behaviour was observed, after which the rate of loading was increased to 4 mm/min. The elongation of the coupons was measured with a 50 mm gauge length extensometer. The strain and the stress measurements were obtained by dividing the measured elongation and the applied load, respectively, by the base metal cross-section area of the coupons. Table 3.6 lists the average material properties, that is the base metal thickness, the yield stress (F_y), the ultimate stress (F_u) and the modulus of elasticity (E), as well as the percent elongation over a 50 mm gauge length and the ratio of F_u to F_y. The values shown in Table 3.6 are the static values obtained from testing.

The cross-head of the test machine was stopped for 60 seconds after yielding had been reached in the coupon, which resulted in a decrease in load due to strain rate effects. With the use of this test approach it was possible to measure the static yield and ultimate stress of the steel specimens.

Table 3.6: Experimental properties of light gauge steel studs and tracks

Specified Size and Strength	Base Metal Thickness (mm)	Yield Stress (F_y) (MPa)	Ultimate Stress (F_u) (MPa)	F_u/F_y	Modulus of Elasticity (E) (MPa)	% Elong
1.09 mm 230 MPa	1.12	264	345	1.30	198700	31.5 %

The material property requirements of the North American Specification for the Design of Cold-Formed Steel Structural Members *(AISI, 2001)* were met. This includes the ratio $F_u/F_y \geq 1.08$ and the minimum 10 % elongation over the 50 mm gauge length.

3.5. Wood Panel Shear Deformation & Holddown Forces

In addition to the data measured following the setup of instruments described in Section 3.1.4 and by Branston, new data were obtained from the two other LVDTs and two load cells that were added to the shear wall test setup. Firstly, diagonal wires attached to LVDTs were installed on all wood panels to measure the shear distortion of the sheathing (Figure 3.21). Secondly, to measure the uplift forces, a load cell was installed on both of the threaded rods used to connect the holddowns to the test frame. Only the tension forces (uplift) could be determined because of the way the load cells were attached to the holddowns.

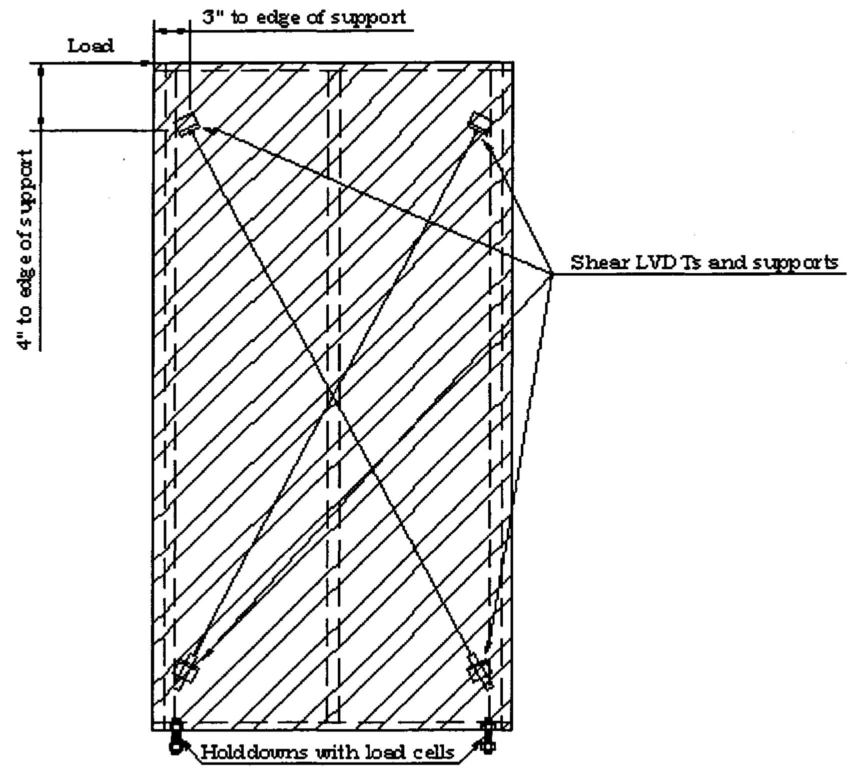


Figure 3.21: Positioning of additional LVDTs and load cells

The shear deformation of the wood sheathing alone was calculated and plotted with respect to the shear resistance of the complete wall assembly, as presented in Figure 3.22. This example graph is of wall 43C, which was tested monotonically (Section 3.1.5). The measured shear deformation of the wood panel returns through the zero position because during testing the wall was unloaded twice (Figure 3.9). Due to bearing and tilting damage to the sheathing connections during the loading phase of testing, it was necessary to pull the sheathing back through the zero position before all loads could be removed from the test wall. The third return through the zero position represents the final failure of the wall and the loss of load carrying capacity. The measured panel shear deformations are essentially linear and elastic in nature; a finding that supports the comments by Chen *(2004)* that attribute the behaviour, in terms of non-linear resistance vs. deformation and ductility, of this type of shear wall to the performance of the sheathing connections.

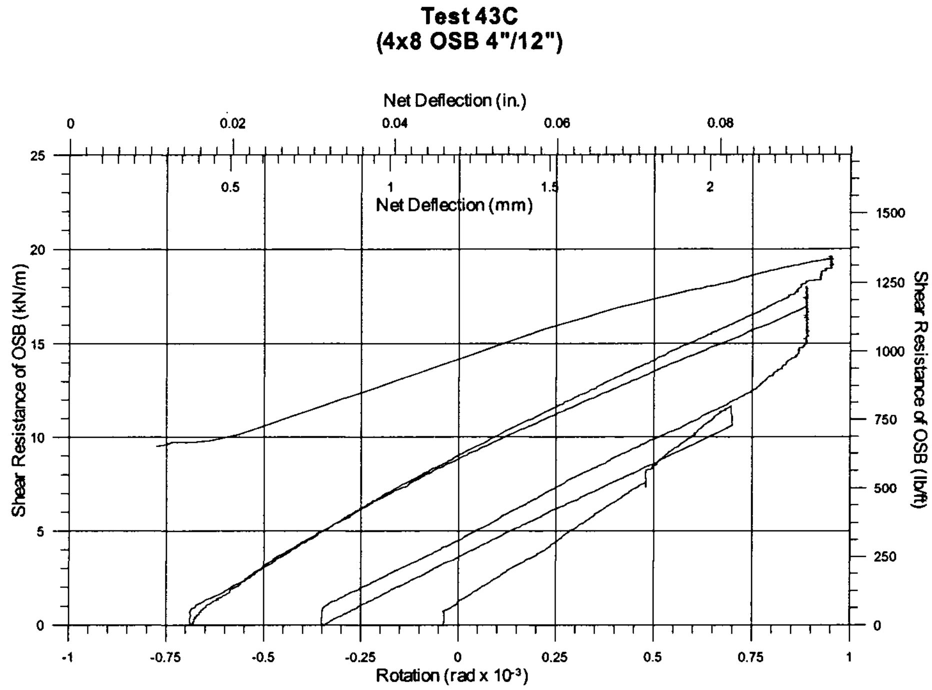


Figure 3.22: Shear deformations of OSB panel

Figure 3.23 illustrates the holddown force vs. wall rotation diagram of reversed cyclic test 44B for both the north and the south load cells. This graph shows that during some cycles both holddowns are in tension, which means that the entire wall lifts up instead of pivoting at one end (Figure 3.24). Typically, designers assume that a shear wall will pivot at one end in order to calculate uplift forces for holddown design. Figure 3.23 also includes two diagrams that show the equivalent applied force calculated from the load cell data (Eq.3-11) in comparison with the actual applied force on the wall over the duration of a reversed cyclic test. The equivalent force reaches the applied force, which was measured at the wall top, only during the last few larger cycles of the loading protocol. It is surmised that the equivalent force is not equal to the applied force during the smaller cycles in the loading protocol because the two shear anchors at the bottom of the wall (Figure 3.3) are able to resist a portion of the uplift force. Nonetheless, the holddowns should be designed for the full anticipated uplift force because during the larger cycles of a seismic event the shear anchors will not provide a significant contribution to the uplift capacity.

$$F_{eq} = 0.467 \times R_{LC} \tag{3-11}$$

where,

F_{eq} = Equivalent applied force, [kN]

0.467 = ratio of the distance between the holddowns to the height of the wall

R_{LC} = Force at load cell, [kN]

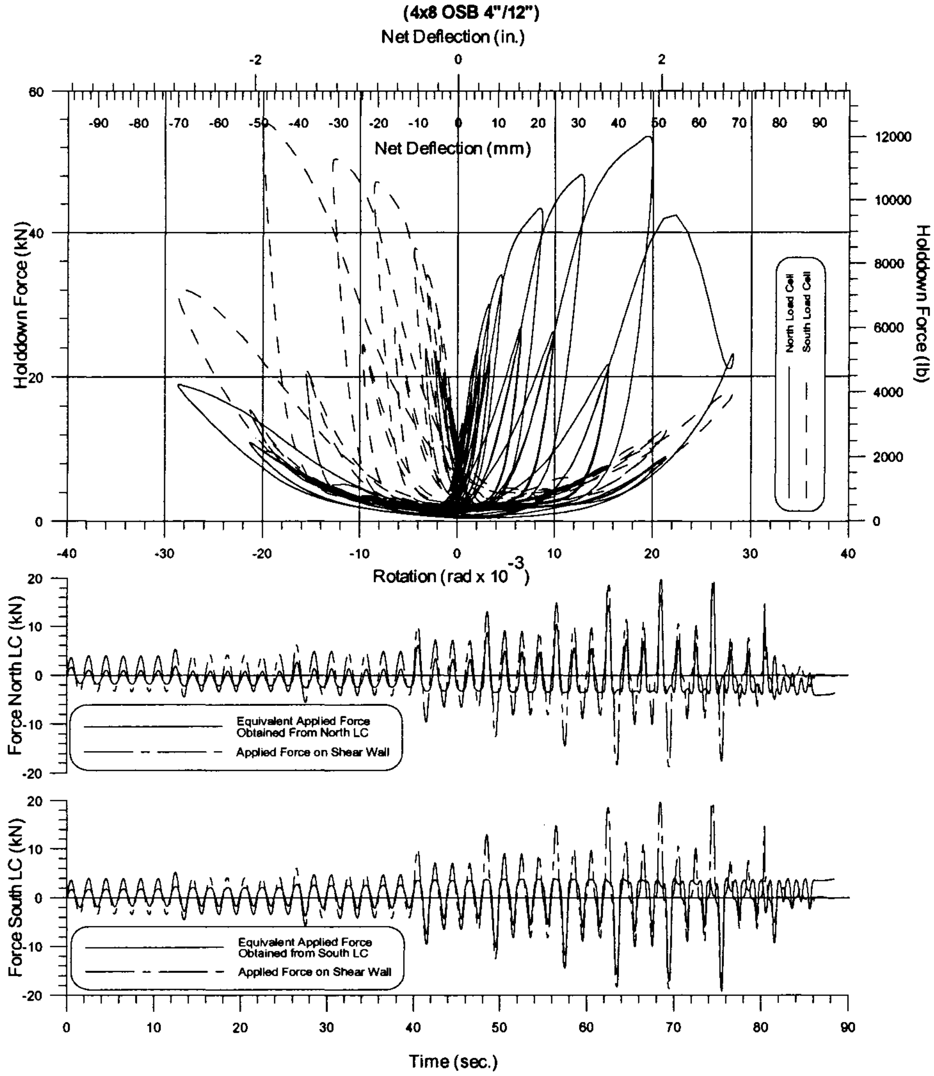
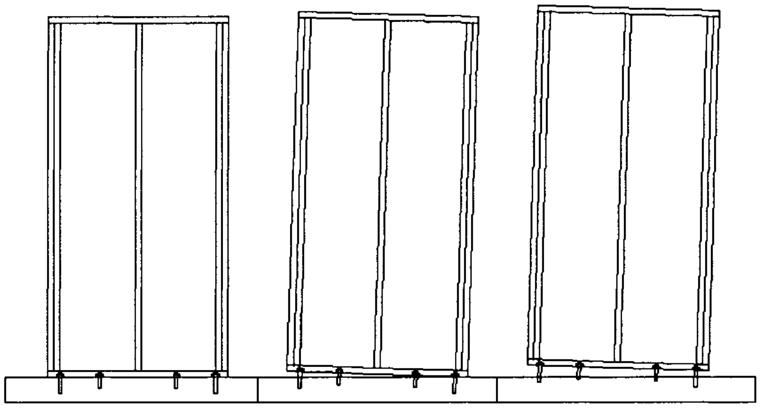


Figure 3.23: North and south holddown forces



Shear wall [a] unloaded; [b] pivoting at one end; [c] lifting up at both ends

Figure 3.24: Shear wall uplift

CHAPTER 4 RECOMMENDED DESIGN PARAMETERS

4.1. INTRODUCTION

The shear wall test program described in this thesis is a continuation of the research carried out by Boudreault *(2005)*, Branston *(2004)* and Chen *(2004)*. With this in mind, the same equivalent energy elastic-plastic (EEEP) analysis approach was therefore used to determine the recommended design parameters for the 18 wall specimens constructed with 9 mm thick OSB sheathing. As is illustrated in Figure 4.1, the measured resistance vs. deflection behaviour of a steel frame / wood panel shear wall is quite nonlinear, somewhat different from the EEEP curve. Nonetheless, the EEEP curve is assumed to represent the behaviour observed during testing based on the energy dissipation capability of a test wall. This data interpretation method was selected because it provides basic strength and stiffness information that can be used for design, it gives a measure of the ductility inherent in the shear wall, which is needed to define a test based force modification factor for seismic design, it can be applied irrespective of the loading protocol implemented, and because it has historically been used for the analysis of other structural systems that have exhibited a non-linear resistance vs. deflection behaviour *(Branston, 2004)*.

An evaluation of the correlation between the shear wall test data obtained in the previous studies at McGill University noted above and the new set of test data was carried out. Also, the 18 tests were compiled with the previous shear wall tests to obtain a larger set of data, which provides for a more comprehensive listing of recommended design parameters. Since a complete description of the EEEP analysis approach can be found in Branston *(2004)*, only a summary is provided in this Section.

4.2. YIELD STRENGTH AND STIFFNESS

In order to complement the existing database of shear wall design parameters, which were summarized by Branston *(2004)*, it was decided that the equivalent energy elastic-plastic (EEEP) approach be used to treat the new set of data. This analysis approach is based on the energy dissipated during testing, which is equal to the area under the backbone curve for the reversed cyclic tests or the area under the resistance vs. deflection curve for the monotonic tests. As is illustrated in Figure 4.1, a bi-linear curve is determined based on the area under the backbone / monotonic curve (energy) of each shear wall test. The initial linear part of the curve represents a purely elastic behaviour that continues up to the yield point. It is from this portion of the curve that the equivalent elastic stiffness (k_e) of the wall, is determined. Using 40% of the ultimate resistance ($0.4S_u$), which is considered to be a reasonable estimate of the maximum service load, and its respective displacement ($\Delta_{net,0.4u}$) the elastic stiffness can be calculated (Eq.4-5). The second linear part of the curve assumes a perfectly plastic behaviour until failure of the wall. This portion of the curve is obtained by equating the area between the EEEP curve and the experimental curve (Fig.4.1). A step by step integration is used until A_1, the area under the EEEP curve (Eq.4-1), is equal to A_2, the area above the EEEP curve (Eq.4-2). Other parameters such as the ultimate wall resistance (S_u), the 80% post-peak wall resistance ($0.8S_u$), the wall resistance at 40% of the ultimate resistance ($0.4S_u$) and their respective displacements ($\Delta_{net,\,u}$; $\Delta_{net,\,0.8u}$; $\Delta_{net,\,y}$) are found from the experimental curves. Only the yield strength (S_y) is left to compute using Equation 4-4, which is a reformulation of the area equations (Eq.4-2 and Eq.4-3).

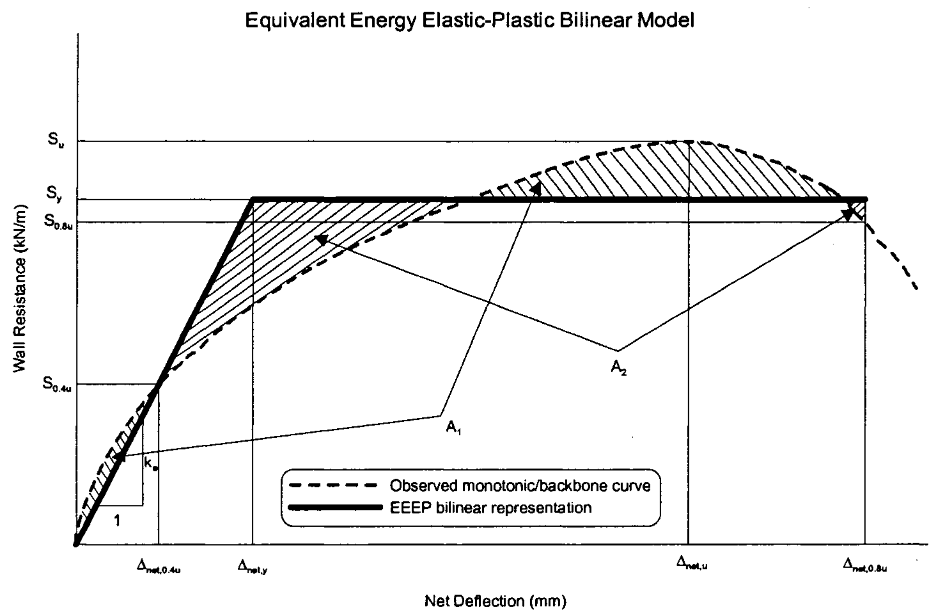


Figure 4.1: EEEP model *(Park, 1989; Salenikovich et al., 2000b; Branston, 2004)*

The mathematical derivation of the wall resistance at yield is as follows:

The area under the EEEP curve,

$$A_{EEEP} = \frac{S_y \times \Delta_{net,y}}{2} + \left[S_y \times \left(\Delta_{net,0.8u} - \Delta_{net,y}\right)\right] \tag{4-1}$$

which is set equal to the area of the backbone / monotonic curve from each test:

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

Using the definition of initial elastic stiffness ($\Delta_{net,\,y} = S_y / k_e$) and by substituting it into Equation 4-2, the following quadratic equation is obtained:

$$-\left(\frac{S_y^2}{2 \times k_e}\right) + \left(\Delta_{net,0.8u} \times S_y\right) - A = 0 \tag{4-3}$$

Solving for S_y,

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

where,

$$k_e = \frac{0.4 \times S_u}{\Delta_{net,0.4u}} \tag{4-5}$$

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

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

A = Calculated area under the backbone / monotonic curve up to failure at $\Delta_{net,\,0.8u}$.

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

$\Delta_{net,\,0.8u}$ = Displacement at post peak wall resistance $0.8S_u$.

$\Delta_{net,\,y}$ = Displacement at yield wall resistance S_y.

In addition to the yield resistance and the elastic stiffness of the wall, the ductility (μ) can be obtained from the EEEP model curve (Eq.4-7). This parameter is essential in defining the characteristics of the wall and in the latter calculation of the ductility related force modification factor.

$$K_e = \frac{S_y \times L}{\Delta_{net,y}} \tag{4-6}$$

$$\mu = \frac{\Delta_{net,0.8u}}{\Delta_{net,y}} \tag{4-7}$$

where,

K_e = Elastic stiffness, [force per unit length]

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

μ = Ductility

Figures 4.2 and 4.3 illustrate the typical monotonic and cyclic EEEP bi-linear curves in comparison with the measured monotonic and backbone curves, respectively. Figure 4.2 shows the monotonic test 41A with the two loops used to evaluate the permanent offset. These additional unloading segments were not used in the calculation of design parameters. The resistance vs. deflection hysteresis graph of reversed cyclic test 42A is shown in Figure 4.3, along with the backbone and EEEP curves.

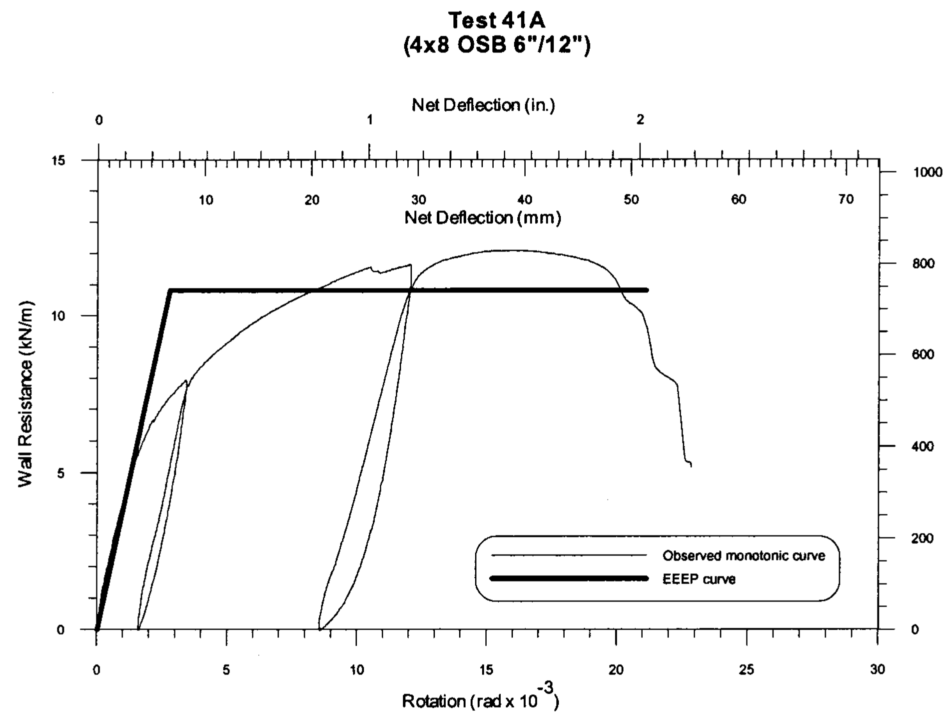


Figure 4.2: EEEP curve for monotonic test 41A

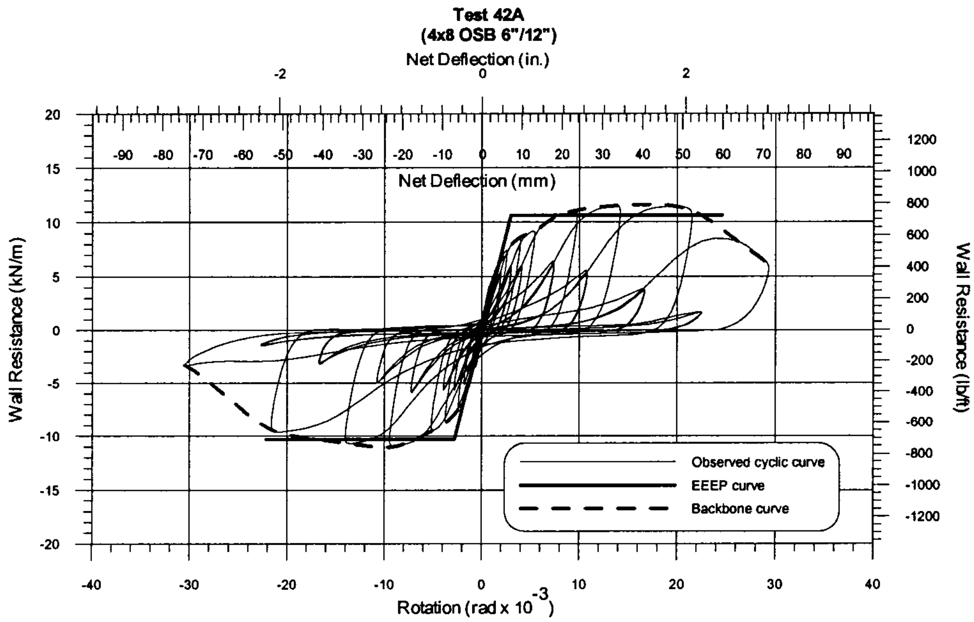


Figure 4.3: EEEP curve for cyclic test 42A

A limitation on the maximum inelastic lateral displacement of a shear wall may change the general EEEP analysis procedure described above. According to the 2005 NBCC *(NRCC, 2005)*, for seismic design the maximum acceptable inelastic inter-storey drift is equal to 2.5% of the storey height. For a 2440 mm (8') high wall this permits a maximum drift of 61 mm. In the general EEEP analysis method the equivalent energy calculation is carried out up to the post-peak displacement at 0.8 S_u ($\Delta_{net,0.8u}$). This results in two different cases in which the inelastic inter-storey drift limit may influence the calculation of design parameters for light gauge steel frame / wood panel shear walls; Case I: 61 mm < $\Delta_{net,u}$ (Fig.4.4) and Case II: $\Delta_{net,u}$ < 61 mm < $\Delta_{net,0.8u}$ (Fig.4.5). The general case is utilized when the seismic drift limitation prescribed by the 2005 National Building Code of Canada is above the failure displacement of the wall, $\Delta_{net,0.8u}$.

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

In Case I, the seismic drift limit is incorporated into the analysis approach in an attempt to preserve the structural integrity of a building during a design level earthquake. In this situation the inelastic drift limit is assumed to represent the upper bound on the useable portion of a wall's resistance vs. deflection behaviour. Hence, it is necessary to modify the general calculation procedure for the EEEP curve. The elastic part of the curve is not affected by this drift limit; only the plastic portion of the curve needs to be adjusted for the 61 mm deflection limit. As found for the general method, the areas, A_1 and A_2, are set equal to establish the value of S_y (Fig.4.4).

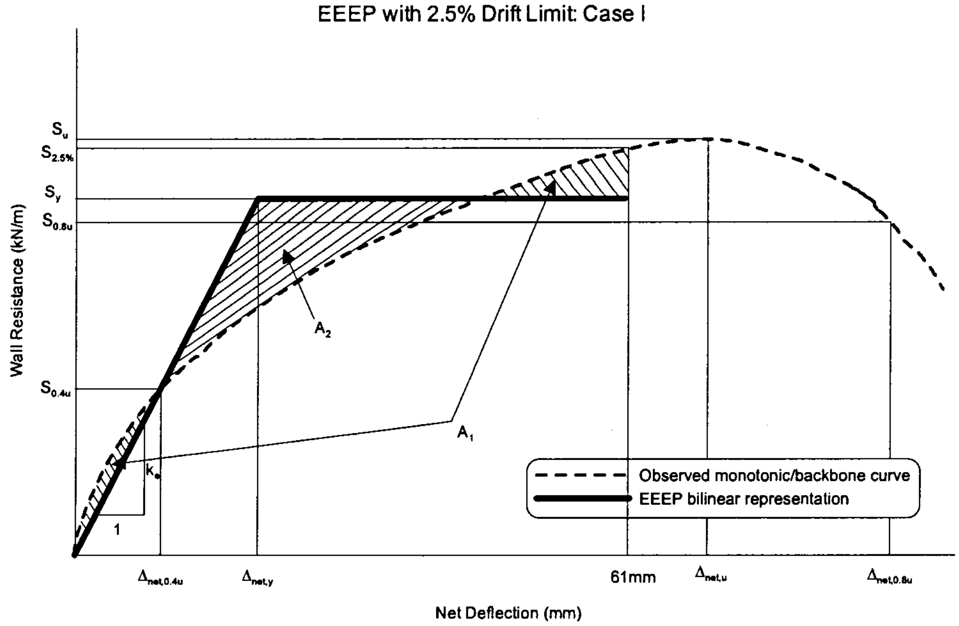


Figure 4.4: EEEP design curve with imposed 2.5 % drift limit (Case I) *(Branston, 2004)*

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

In contrast to Case I, for Case II the test wall is able to attain its ultimate shear capacity prior to reaching the 2.5% inter-storey drift limit. However, $\Delta_{net,0.8u}$ occurs at a deflection that exceeds the 61 mm associated with a 2440 mm (8') high wall. In this instance, the seismic drift limit is considered not to affect the design yield resistance of a shear wall since the test results show that the wall is able to develop its ultimate shear capacity. The resulting EEEP curve is shown in Figure 4.5, for which all values are derived as per the general approach; Equations 4-1 to 4-6.

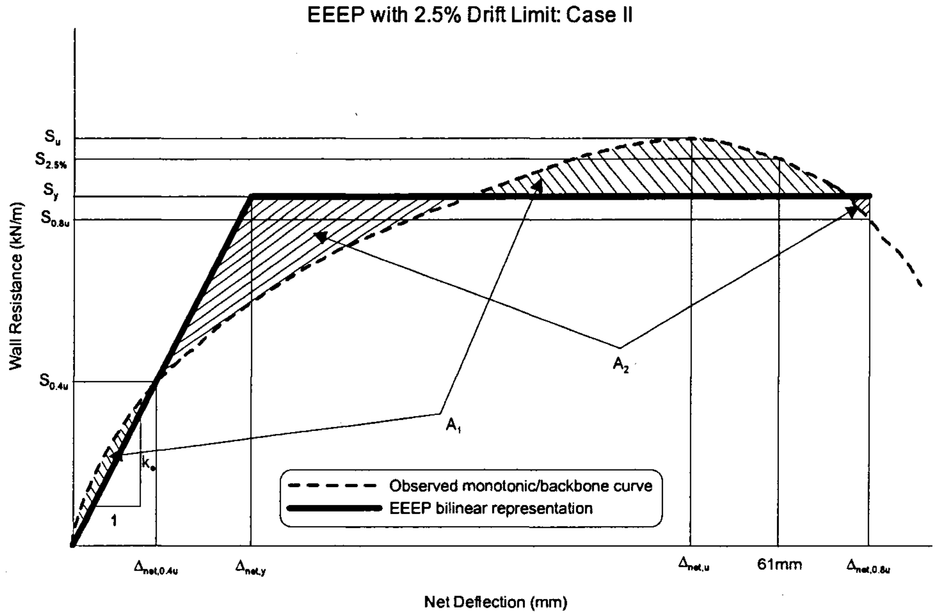


Figure 4.5: EEEP design curve with imposed 2.5 % drift limit (Case II) *(Branston, 2004)*

Based on the data recorded for the 18 tests described in this research, the average lateral displacement $\Delta_{net,0.8u}$ for all monotonic and cyclic tests was found to be below the 2.5% drift limit (61mm) (Table 4.1). Therefore, Case I and Case II did not apply to these particular tests, and hence, the EEEP general procedure was implemented to obtain the final design parameters (Tables 4.2 - 4.4).

Table 4.1: Lateral wall displacements at 80% of the ultimate shear force

Panel Type	**Fastener Schedule (mm)**	**Mono Test**	**Displacement at 0.8S$_u$ (mono) ($\Delta_{net, 0.8u}$) (mm)**	**Cyclic Test**	**Displacement at 0.8S$_u$'$_+$ (cyclic+) ($\Delta_{net, 0.8u'+}$) (mm)**	**Displacement at 0.8S$_u$'$_-$ (cyclic-) ($\Delta_{net, 0.8u'-}$) (mm)**
OSB	152/305	41A	51.4	42A	59.8	54.1
OSB	152/305	41B	62.0	42B	59.4	49.8
OSB	152/305	41C	53.5	42C	54.0	43.3
AVERAGE			**55.6**		**57.7**	**49.1**
OSB	100/305	43A	47.7	44A	53.2	51.8
OSB	100/305	43B	59.3	44B	52.9	52.7
OSB	100/305	43C	44.0	44C	58.1	51.2
AVERAGE			**50.4**		**54.7**	**51.9**
OSB	75/305	45A	45.2	46A	56.2	47.4
OSB	75/305	45B	53.6	46B	48.6	39.8
OSB	75/305	45C	46.4	46C	39.4	39.0
AVERAGE			**48.4**		**48.1**	**42.1**

Table 4.2: Design values for monotonic tests

Test	Panel Type	Fastener Schedule (mm)	Yield Load (S_y) (kN/m)	Displacement at $0.4S_u$ ($\Delta_{net,\,0.4u}$) (mm)	Displacement at S_y ($\Delta_{net,\,y}$) (mm)	Elastic Stiffness (K_e) (kN/mm)	Rotation at S_y ($\theta_{net,\,y}$) (rad x 10^{-3})	Ductility (μ)	Energy Dissipation (E) (Joules)
41A	OSB	152/305	10.8	3.0	6.8	1.95	2.8	7.60	633
41B	OSB	152/305	11.0	2.9	6.7	1.98	2.8	9.19	784
41C	OSB	152/305	11.1	2.2	5.2	2.62	2.1	10.37	687
AVERAGE			**11.0**	**2.7**	**6.2**	**2.18**	**2.6**	**9.05**	**701**
43A	OSB	100/305	15.9	3.6	8.1	2.41	3.3	5.92	847
43B	OSB	100/305	15.9	3.9	8.7	2.24	3.5	6.86	1066
43C	OSB	100/305	17.2	4.5	9.9	2.12	4.0	4.46	818
AVERAGE			**16.3**	**4.0**	**8.9**	**2.26**	**3.6**	**5.75**	**910**
45A	OSB	75/305	21.4	4.8	10.8	2.41	4.4	4.20	1037
45B	OSB	75/305	21.9	5.4	12.1	2.21	4.9	4.44	1266
45C	OSB	75/305	21.8	4.9	10.9	2.43	4.5	4.25	1087
AVERAGE			**21.7**	**5.0**	**11.3**	**2.35**	**4.6**	**4.30**	**1130**

Table 4.3: Design values for reversed cyclic tests (positive cycles)

Test	Panel Type	Fastener Schedule (mm)	Yield Load (S_{y+}) (kN/m)	Displacement at S_{y+} ($\Delta_{net,\,y+}$) (mm)	Elastic Stiffness (K_{e+}) (kN/mm)	Rotation at S_{y+} ($\theta_{net,\,y+}$) (rad x 10^{-3})	Ductility (μ)	Energy Dissipation (E^1) (Joules)
42A	OSB	152/305	10.7	7.2	1.81	2.9	8.34	730
42B	OSB	152/305	10.2	7.4	1.70	3.0	8.07	695
42C	OSB	152/305	11.0	6.7	2.01	2.7	8.07	681
AVERAGE			**10.6**	**7.1**	**1.84**	**2.9**	**8.16**	**702**
44A	OSB	100/305	17.1	11.9	1.75	4.9	4.46	984
44B	OSB	100/305	15.0	8.4	2.20	3.4	6.33	893
44C	OSB	100/305	16.0	9.0	2.16	3.7	6.45	1043
AVERAGE			**16.0**	**9.8**	**2.04**	**4.0**	**5.75**	**973**
46A	OSB	75/305	20.8	10.0	2.54	4.1	5.64	1299
46B	OSB	75/305	19.7	8.9	2.69	3.7	5.43	1060
46C	OSB	75/305	18.4	8.5	2.63	3.5	4.62	797
AVERAGE			**19.6**	**9.1**	**2.62**	**3.8**	**5.23**	**1052**

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

Table 4.4: Design values for reversed cyclic tests (negative cycles)

Test	Panel Type	Fastener Schedule (mm)	Yield Load (S_{y-}) (kN/m)	Displacement at S_{y-} ($\Delta_{net,\,y-}$) (mm)	Elastic Stiffness (K_{e-}) (kN/mm)	Rotation at S_{y-} ($\theta_{net,\,y-}$) (rad x 10^{-3})	Ductility (μ)	Energy Dissipation (E^1) (Joules)
42A	OSB	152/305	10.3	6.8	1.84	2.8	7.92	638
42B	OSB	152/305	10.0	6.7	1.83	2.7	7.46	569
42C	OSB	152/305	10.9	7.9	1.67	3.3	5.45	523
AVERAGE			**10.4**	**7.2**	**1.78**	**2.9**	**6.94**	**576**
44A	OSB	100/305	15.4	8.3	2.25	3.4	6.21	893
44B	OSB	100/305	14.4	7.7	2.27	3.2	6.83	855
44C	OSB	100/305	15.5	7.5	2.51	3.1	6.81	894
AVERAGE			**15.1**	**7.9**	**2.34**	**3.2**	**6.62**	**881**
46A	OSB	75/305	19.1	7.7	3.03	3.2	6.17	1013
46B	OSB	75/305	18.4	8.1	2.76	3.3	4.89	802
46C	OSB	75/305	18.2	7.9	2.79	3.3	4.91	776
AVERAGE			**18.6**	**7.9**	**2.86**	**3.2**	**5.32**	**864**

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

4.3. CALIBRATION OF RESISTANCE FACTOR, ϕ

The CSA S136 Standard *(2002)* for the design of cold-formed steel structures does not include an approach for the design of light gauge steel frame / wood panel shear walls subjected to in-plane lateral loading. This includes values for a nominal shear capacity, S_y, as well as a resistance factor, ϕ, calibrated according to the limit state design procedures prescribed in the 2005 edition of the National Building Code of Canada *(NRCC, 2005)*. Hence, given the design S_y values listed in Tables 4.2 to 4.4, it was necessary to calibrate a resistance factor with respect to the one in fifty years, $q_{1/50}$, NBCC factored wind load. The derivation of the calibration procedure and an explanation of why particular values were assigned to the statistical parameters needed in the calibration model were documented by Branston *(2004)*. A summary of the approach that was followed is presented in this Section.

The resistance factor for the ultimate limit state can be obtained from the following equation:

$$\phi = C_\phi (M_m F_m P_m) e^{-\beta_o \sqrt{V_m^2 + V_F^2 + C_P V_P^2 + V_S^2}} \tag{4-8}$$

where,

C_ϕ = Calibration coefficient

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

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

P_m = Mean value of professional factor for tested component

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 = Degree of freedom = n-1

n = Number of tests

e = Natural logarithmic base = 2.718...

The values for M_m, F_m, V_m, and V_F were chosen based on the recommendations of Branston *(2004)*. That is M_m = 1.05 to account for a possible 5% overstrength in the sheathing material and F_m = 1.00, assuming that the nominal thickness of the sheathing is the same as the average thickness of a large number of sheathing panels. The two other variables, V_m, and V_F, were taken has 0.11 and 0.10, respectively, to account for the coefficient of variation of 15% found for the strength distribution of the sheathing.

To obtain the professional factor, P_m, and the coefficient of variation of the professional factor, V_P, it was necessary to calculate the average wall resistance at yield, $S_{y,avg}$. The average was calculated in two different ways in order to compare the resulting resistance factor, ϕ. In the first calculation, the negative and the positive nominal shear capacities, $S_{y-,avg}$ and $S_{y+,avg}$ respectively, of a cyclic test were averaged before being added to the nominal shear capacity, $S_{y,mono,avg}$, of the monotonic shear test (Eq.4-9). In the second calculation, only the positive nominal shear value of the cyclic test was averaged with the nominal shear value of the monotonic test (Eq.4-10).

$$S_{y,avg} = \frac{S_{y,mono,avg} + \dfrac{S_{y+,avg} + S_{y-,avg}}{2}}{2} \qquad (4\text{-}9)$$

$$S_{y,avg} = \frac{S_{y,mono,avg} + S_{y+,avg}}{2} \qquad (4\text{-}10)$$

Once these values were obtained, P_m and V_P were calculated using the following equations:

$$P_m = \frac{\sum_{i=1}^{n} \left(S_y / S_{y,avg} \right)}{n} \qquad (4\text{-}11)$$

$$V_P = \frac{\sigma}{P_m} \tag{4-12}$$

where S_y is the nominal shear value of each individual test, and:

$$\sigma^2 = \frac{1}{n-1}\sum_{i=1}^{n}\left[\left(S_y \big/ S_{y,avg}\right) - P_m\right]^2 \tag{4-13}$$

The value for the calibration coefficient, C_ϕ, was obtained using the following equation:

$$C_\phi = \frac{\alpha}{\bar{S} \big/ S} \tag{4-14}$$

where,

α = Load factor for wind loads and is equal to 1.4 according to the 2005 NBCC.

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

Using the approach described by Branston *(2004)*, the mean-to-nominal ratio of the wind load and the corresponding coefficient of variation, V_S, are equal to 0.76 and 0.37, respectively. Concerning the reliability/safety factor, β_o; in the Commentary of the 2001 North American Cold-Formed Steel Specification *(AISI, 2002)*, the value varies from 2.5 to 4.0 where a value of 4.0 is used when failure at a connection is not acceptable. Therefore a value of 2.5 was proposed assuming that the walls have a built-in overstrength; that is the ultimate shear strength is 10% greater on average for the 9 mm OSB walls tested than the yield shear strength derived from the EEEP method (See Section 4.6). Given the calibration approach and the statistical values described above, the resistance factor, ϕ, was calculated for the different sheathing fastener patterns and for all 18 tests combined (Table 4.5). The values obtained are similar to those recommended by Branston *(2004)*, which shows that a ϕ value of 0.7 is appropriate for shear walls sheathed with 9 mm thick OSB panels.

Table 4.5: Resistance factor calibration for 2005 NBCC wind loads

Fastener Schedule (mm)	α	S_m/S	C_ϕ	M_m	F_m	P_m	β_o	V_m	V_F	V_S	n	C_P	V_P	ϕ
Mono/Cyclic +/-														
152/305	1.4	0.76	1.842	1.05	1.00	1.00	2.50	0.11	0.10	0.37	6	1.944	0.0336	**0.709**
100/305	1.4	0.76	1.842	1.05	1.00	1.00	2.50	0.11	0.10	0.37	6	1.944	0.0498	**0.703**
75/305	1.4	0.76	1.842	1.05	1.00	1.00	2.50	0.11	0.10	0.37	6	1.944	0.0743	**0.691**
All tests	1.4	0.76	1.842	1.05	1.00	1.00	2.50	0.11	0.10	0.37	18	1.196	0.0519	**0.707**
Mono/Cyclic positive														
152/305	1.4	0.76	1.842	1.05	1.00	1.00	2.50	0.11	0.10	0.37	6	1.944	0.0295	**0.710**
100/305	1.4	0.76	1.842	1.05	1.00	1.00	2.50	0.11	0.10	0.37	6	1.944	0.0498	**0.703**
75/305	1.4	0.76	1.842	1.05	1.00	1.00	2.50	0.11	0.10	0.37	6	1.944	0.0661	**0.695**
All tests	1.4	0.76	1.842	1.05	1.00	1.00	2.50	0.11	0.10	0.37	18	1.196	0.0477	**0.708**

It is recommended that the resistance factor, ϕ, calculated for the 2005 NBCC wind loads, be also used in seismic design. This approach is warranted because the resistance factor is found in both the equivalent static earthquake base shear (V) (Eq.4-15) and in the factored wall resistance.

$$\phi S_y \geq V = \frac{S(T)M_V I_E W}{R_d R_o} \tag{4-15}$$

where, R_o, the overstrength-related force modification factor is a function of R_ϕ, which is equal to $1/\phi$. The resistance factor, ϕ, is found on both sides of Equation 4-15, and hence, it cannot be calibrated based on seismic load factors. R_ϕ is included in the definition of R_o because seismic resistant design is based on a return period of 2500 years for the design level earthquake (probability of exceedance of 2% in 50 years) *(Mitchell et al., 2003)*. This represents a rare loading event for which a nominal resistance, in place of a factored resistance, is considered to be adequate for design. A resistance factor of ϕ = 0.7 is therefore recommended for seismic design; first of all to be consistent with the factor calibrated for wind loads, and secondly because this value was used in the calculation of R_o, as discussed in Section 4.7.2.

4.4. Recommended Shear Strength and Stiffness Values for Light Gauge Steel Frame / Wood Panel Shear Walls

Based on the results documented in Section 4.2, an average nominal shear resistance, $S_{y,\,avg.}$, and an average unit elastic stiffness, $k_{e,\,avg.}$, were computed for each wall configuration. To obtain these recommended design parameters, the monotonic strength and stiffness values were averaged with the average value of the positive and negative cycles (Eq.4-18 and 4.19).

$$S_{y,avg} = \frac{S_{y,mono} + \left(S_{y,+cyclic} + S_{y,-cyclic}\right)/2}{2} \tag{4-18}$$

$$k_{e,avg} = \frac{k_{e,mono} + \left(k_{e,+cyclic} + k_{e,-cyclic}\right)/2}{2} \tag{4-19}$$

Table 4.6 lists the average nominal shear resistance and unit elastic stiffness values for the three different light gauge steel frame / 9 mm OSB wood panel shear wall configurations. The nominal shear strength is given in kilo-Newton per metre, while the elastic stiffness is given in kilo-Newton per millimetre per metre of wall length. For comparison purposes the design values determined by Branston *(2004)* for the walls composed of 11 mm thick OSB panels are also listed.

From Table 4.6, it can be noted that the walls sheathed with the 11 mm OSB panels are able to carry slightly larger shear loads for all three connection configurations. This result was expected because the shear capacity of a wall is directly related to the bearing resistance of the sheathing connections. A thinner panel provides a smaller bearing area for the screw fasteners, which causes a decrease in the overall shear capacity of the wall. In contrast, the shear stiffness of the walls constructed with 9 mm OSB panels was higher than that measured for the walls with 11 mm OSB sheathing. A similar relationship between 9.5 mm and 12.5 mm thick Canadian Softwood Plywood sheathed walls was observed by Rokas *(2005)*. At this stage it is not possible to provide a definitive reason why the walls with thinner sheathing behave in this fashion. Some possible explanations are as follows:

The type of OSB panels used for this research and for Branston's research was not the same. The 9 mm thick panels were classified under CSA O325 *(1992)* as *2R24/W24* while the 11 mm thick panels were *1R24/2F16/W24*. For the *W24* wall rating no specific requirement concerning in-plane shear stiffness exists. With respect to fastener performance, only a minimum strength level needs to be met. It is possible that the stiffness variation exists because connection and shear stiffness parameters are not considered in the CSA O325 Standard *(1992)*. Furthermore, the CSA O86 Standard *(2001)* places no requirement on the in-plane shear stiffness for the *W24* wall rating. This Standard, however, does require a slightly higher in-plane shear stiffness for the *2R24* (9 mm) versus the *1R24/2F16* (11 mm) rating. That is, the in-plane shear must be at least v_{pf} = 0.38 MPa for the thinner sheathing compared with 0.33 MPa for the thicker.

The panels were not fabricated in the same mill or by the same company. The 11 mm sheathing was from Tembec, whereas the 9 mm panels were from Grant Forest Products. Assuming that the panels meet the requirements prescribed in CSA O325 and CSA O86 it is possible that the source / type of wood and manufacturing process are different enough to cause a variation of initial stiffness properties.

The deflections at ultimate load (S_u) and failure ($0.8S_u$) of the OSB panel shear walls included in Branston *(2004)* and those of this research were also compared. For the monotonic tests it was found that the walls with 9 mm OSB sheathing had larger deflections at ultimate and failure than the walls constructed of 11 mm panels. Also, the deflections recorded during the reversed cyclic tests at failure for the walls with 9 mm thick panels were larger than for the walls with 11 mm OSB tested by Branston. However, a comparison of the deflections measured for the cyclic tests at the ultimate load was inconclusive because smaller deflections were observed for the 9 mm OSB walls with a screw schedule of 152/305 and 75/305. Nonetheless, it appears that the relative size of the screw head to the sheathing thickness affects the initial stiffness of the shear walls, that is the k_e values listed herein. Once the walls have extended into the inelastic range the walls with thicker sheathing typically experience less shear displacement, as would be expected.

Table 4.6: Nominal shear strength, S_y (kN/m), and unit elastic stiffness, k_e (kN/mm/m), for light gauge steel frame / wood panel shear walls dependent of the sheathing material

Minimum nominal Panel thickness (mm) & Grade	Screw spacing at panel edges (mm)					
	75		100		152	
	S_y (kN/m)	k_e (kN/mm/m)	S_y (kN/m)	k_e (kN/mm/m)	S_y (kN/m)	k_e (kN/mm/m)
9 mm OSB CSA O325 2R24/W24	20.4	2.09	15.9	1.82	10.7	1.64
11 mm OSB CSA O325 1R24/2F16/W24	20.6	1.88	16.2	1.75	11.0	1.38

Notes:

(1) ϕ = 0.7 to obtain factored resistance for design.

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

(3) Tabulated values are applicable for dry service conditions (sheathing panels) and short-term load duration (K_D = 1.0) such as wind or earthquake loading. For shear walls under permanent loading, tabulated values must be multiplied by 0.565; and under standard term loads, tabulated values must be multiplied by 0.870.

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

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

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

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

(8) Studs: 3-5.8" (92.1 mm) web, 1-5/8" (41.3 mm) flange, 1/2" (12.7 mm) return lip.
Tracks: 3-5/8" (92.1 mm) web, 1-1/4" (31.8 mm) flange.

(9) The above values are for lateral loading only. It must be noted that the compression chord failure mode must be accounted for in design, including the effects of gravity loads.

The shear resistance of a given structure made of light gauge steel frame / wood panel shear walls is obtained by the summation of the shear resistances of each shear wall segment of a storey (Eq.4-20), assuming that the aspect ratio of each segment is less that 2:1 (height : length). The shear resistance of a wall is computed using the resistance factor, ϕ, the nominal shear resistance, S_y (Table 4.6), the load duration factor, K'_D, and the wall length (Eq.4-21).

$$S_r = \sum S_{rs} \tag{4-20}$$

where,

$$S_{rs} = \phi S_y K'_D L \tag{4-21}$$

S_r = Factored shear resistance, [kN]

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

$\phi = 0.7$

S_y = Nominal shear strength (Table 4.6)

K'_D = Load duration factor

= 1.0 for short term loading

= 0.565 for permanent loading

= 0.870 for standard loading

L = Length of shear wall segment [m]

4.5. FACTOR OF SAFETY

The resistance factor and the nominal shear strength values recommended for design were used to calculate the factor of safety associated with light gauge steel frame / wood panel shear walls. Two different calculation methods were implemented; the first is associated with the limit states design (LSD) approach, whereby a simple comparison of the measured ultimate shear resistance with the nominal shear capacity was carried out (Eq. 4-22) (Figure 4.6). The second approach is in terms of an allowable stress design (ASD) format where the factor for wind load is taken into account (Eq.4-23). Thus the 1.4 wind load factor defined by the 2005 National Building Code of Canada *(NRCC, 2005)* was utilized.

$$F.S.(LSD) = \frac{S_u}{S_r} \tag{4-22}$$

$$F.S.(ASD) = 1.4\frac{S_u}{S_r} \tag{4-23}$$

where,

$F.S.$ = Factor of safety for design

S_u = Ultimate wall resistance observed during test

S_r = Factored wall resistance ($\phi = 0.7$)

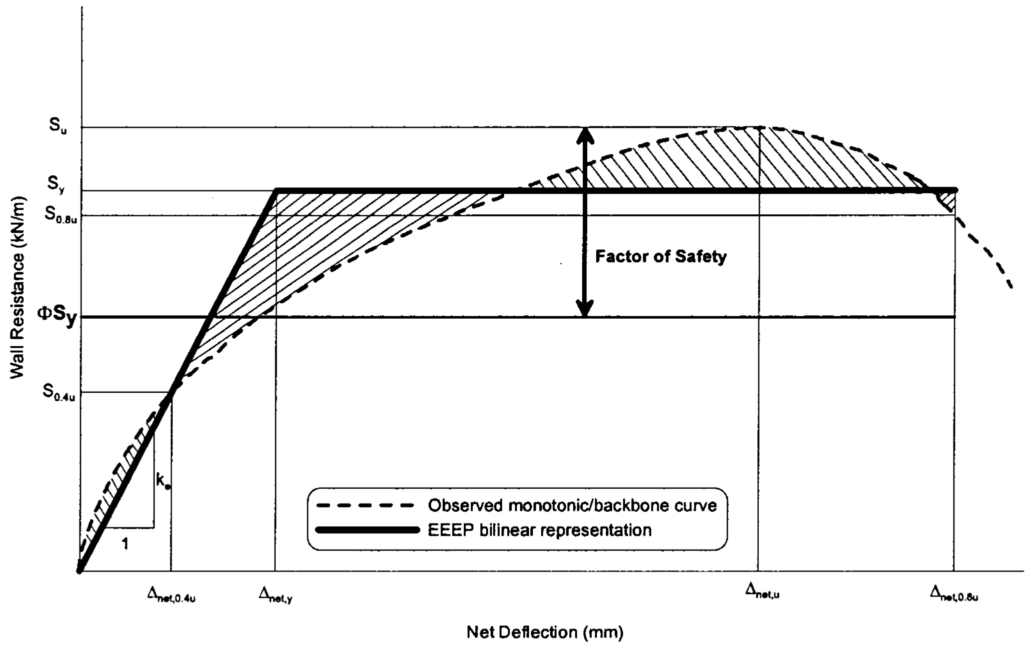


Figure 4.6: Factor of safety inherent in limit states design

According to Branston *(2004)*, the factor of safety for allowable stress design (ASD) of light gauge steel frame / wood panel shear walls should fall between 2.0 and 2.5. These values are suggested by the 2000 IBC *(ICC, 2000)* for light gauge steel frame shear walls and by the IBC 2000 Handbook *(Ghosh and Chittenden, 2001)* for wood shear walls, respectively. Tables 4.7 and 4.8 show the values computed for both the limit states design (LSD) and the allowable stress design (ASD) factor of safety. When monotonic test values are considered, the ASD factor of safety ranges from 2.22 – 2.40 with an average of 2.31, a standard deviation of 0.09 and of coefficient of variation of 3.9% (Table 4.7). For reversed cyclic tests, the ASD values range from 2.01 – 2.29 with an average of 2.14, a standard deviation of 0.09 and a coefficient of variation of 4.1% (Table 4.8). Although these results are somewhat lower than those described by Branston *(2004)*, where an ASD value of approximately 2.4 was calculated, the values fall within the suggested range of 2.0 – 2.5. Furthermore, wind loads according to the draft 2005 NBCC *(NRCC, 2005)* are now based on a return period of 50 years, which provides an added factor of safety when compared to wind loads based on the previous version of the NBCC *(NRCC, 1995)* (1 in 30 year return period).

Table 4.7: Factor of safety inherent in design for monotonic test values

Test	Panel Type	Fastener Schedule (mm)	Ultimate Resistance (S_u) (kN/m)	Yield Load (S_y) (Table 4.6) (kN/m)	Factored Resistance (S_r) ($\phi = 0.7$) (kN/m)	Factor of Safety (LSD) (S_u/S_r)	Factor of Safety (ASD) (S_u/S_r * 1.4)
41A	OSB	152/305	12.1	10.7	7.5	1.61	2.25
41B	OSB	152/305	11.9	10.7	7.5	1.58	2.22
41C	OSB	152/305	12.0	10.7	7.5	1.59	2.23
AVERAGE			**12.0**	**10.7**	**7.5**	**1.60**	**2.23**
43A	OSB	100/305	17.7	15.9	11.2	1.59	2.22
43B	OSB	100/305	18.0	15.9	11.2	1.61	2.25
43C	OSB	100/305	19.6	15.9	11.2	1.76	2.46
AVERAGE			**18.4**	**15.9**	**11.2**	**1.65**	**2.31**
45A	OSB	75/305	23.7	20.4	14.3	1.66	2.33
45B	OSB	75/305	24.3	20.4	14.3	1.70	2.39
45C	OSB	75/305	24.4	20.4	14.3	1.71	2.40
AVERAGE			**24.2**	**20.4**	**14.3**	**1.69**	**2.37**
					Average	**1.65**	**2.31**
					Std.Dev.	**0.06**	**0.09**
					CoV	**0.039**	**0.039**

Table 4.8: Factor of safety inherent in design for cyclic test values

Test	Panel Type	Fastener Schedule (mm)	Ultimate Resistance ($S_u'_+$) (positive cycle) (kN/m)	Yield Load (S_y) (Table 4.6) (kN/m)	Factored Resistance (S_r) ($\phi = 0.7$) (kN/m)	Factor of Safety (LSD) (S_u/S_r)	Factor of Safety (ASD) (S_u/S_r * 1.4)
42A	OSB	152/305	11.5	10.7	7.5	1.53	2.14
42B	OSB	152/305	11.1	10.7	7.5	1.48	2.07
42C	OSB	152/305	11.9	10.7	7.5	1.59	2.22
AVERAGE			**11.5**	**10.7**	**7.5**	**1.53**	**2.15**
44A	OSB	100/305	18.2	15.9	11.2	1.64	2.29
44B	OSB	100/305	16.2	15.9	11.2	1.45	2.03
44C	OSB	100/305	17.3	15.9	11.2	1.55	2.17
AVERAGE			**17.2**	**15.9**	**11.2**	**1.55**	**2.16**
46A	OSB	75/305	22.4	20.4	14.3	1.57	2.20
46B	OSB	75/305	21.5	20.4	14.3	1.50	2.11
46C	OSB	75/305	20.5	20.4	14.3	1.43	2.01
AVERAGE			**21.4**	**20.4**	**14.3**	**1.50**	**2.10**
					Average	**1.53**	**2.14**
					Std.Dev.	**0.06**	**0.09**
					CoV	**0.041**	**0.041**

The ultimate shear resistance (S_u) from the positive cycles was used in the calculation of the factor of safety for the cyclic tests (Table 4.8). The shear resistance of the negative cycles is indeed lower since the positive cycles were executed first. However, it was decided not to use the average of both negative and positive cycle values because a wall pushed to failure will reach the larger value.

The overall results, which include the data from Tables 4.7 and 4.8 and those of Branston *(2004)* for the factor of safety of light gauge steel frame / wood panel shear walls is as follows: 1) the average LSD factor of safety for the monotonic tests is 1.75 (SD of 0.15 and CoV of 8.56%) and for the cyclic tests is 1.68 (SD of 0.16 and CoV of 9.51%); 2) the average ASD factor of safety for the monotonic tests is 2.45 (SD of 0.21 and CoV of 8.54%) and for the cyclic tests is 2.35 (SD of 0.22 and CoV of 9.47%). The overall values also fall within the suggested range of 2.0 – 2.5.

4.6. Capacity Based Design and Overstrength

Shear walls are designed to withstand lateral loads caused by wind and earthquakes. As is discussed in Section 4.7, force modification factors greater than unity, for both ductility and overstrength, are recommended for use in the calculation of seismic loads. Hence, in terms of capacity based seismic design requirements, if these walls were selected to form the fuse element in the seismic force resisting system (SFRS), they would be expected to dissipate energy by failing in a ductile manner. Within the wall itself, it is anticipated that the sheathing to framing connections alone fail, to ensure that the steel frame is available to carry gravity loads after a design level earthquake. This presents the engineer with the problem of selecting the other components in the SFRS such that they remain essentially elastic; that is they have a capacity that exceeds the probable resistance of the shear wall. Components such as the chord studs, tracks, hold-downs, anchors rods, shear anchors, foundation, etc, are included in the SFRS.

To design the other components of the SFRS it is necessary to know the probable shear capacity of the wall. In order to estimate this capacity, the nominal shear resistance (S_y) (Table 4.6) of the wall must be multiplied by the overstrength factor (Fig.4.7). This factor can be obtained by dividing the ultimate shear wall resistance (S_u) by the nominal shear wall resistance (S_y) (Eq.4-24).

$$overstrength = \frac{S_u}{S_y} \tag{4-24}$$

where,

S_u = Ultimate shear wall resistance

S_y = Nominal shear wall resistance, Table 4.6

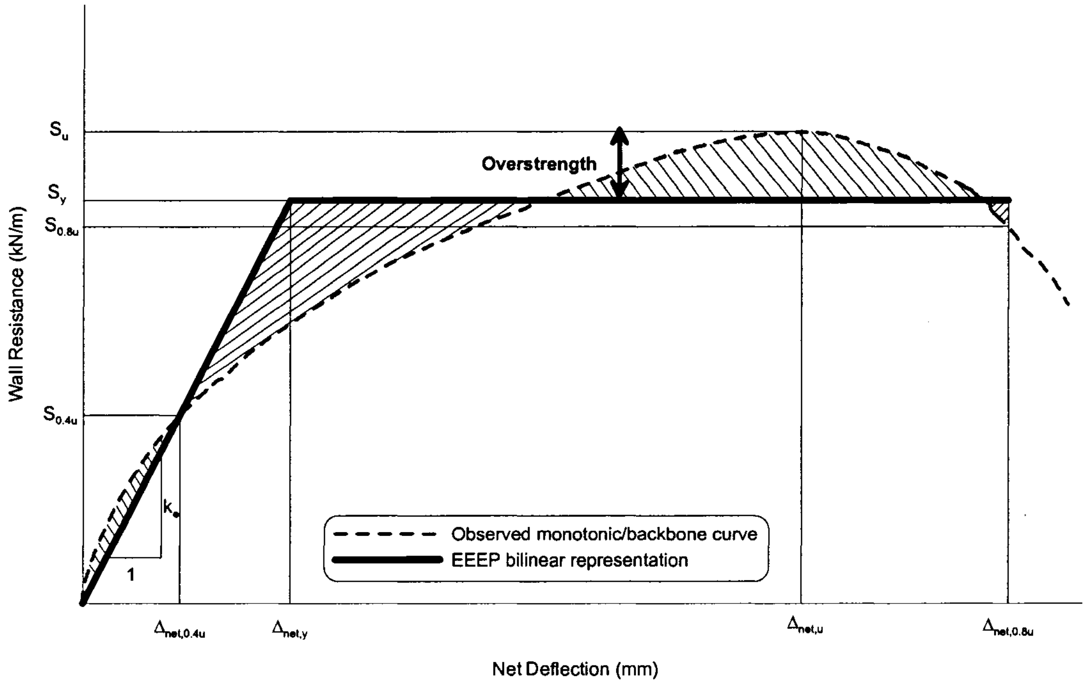


Figure 4.7: Overstrength inherent in design

The overstrength value for each test wall is listed in Table 4.9 (monotonic tests) and in Table 4.10 (cyclic tests). The overstrength factors for the monotonic tests fall between 1.11 – 1.23, with an average of 1.15 (SD of 0.043 & CoV of 3.74%). The same factors for the reversed cyclic tests fall between 1.00 – 1.11, with an average of 1.07 (SD of 0.046 & CoV of 4.26%). Both averages are within the range of overstrength factors obtained from the previous shear wall tests completed at McGill University; which were 1.08 – 1.57 and 1.04 – 1.44 for monotonic and cyclic tests, respectively *(Branston, 2004)*. To validate the overstrength value of 1.2 suggested by Branston, the data of this present research were integrated with those of the previous studies. Average values of 1.22 (SD of 0.104 and CoV of 8.53%) and 1.17 (SD of 0.111 and CoV of 9.46%) were obtained for the monotonic and cyclic tests, respectively (based on 96 tests), which show that the previously suggested value of 1.2 for overstrength is appropriate.

Table 4.9: Overstrength inherent in design for monotonic test values

Test	Panel Type	Fastener Schedule (mm)	Maximum Wall Resistance (S_u) (kN/m)	Yield Load (S_y) (Table 4.6) (kN/m)	Overstrength (S_u/S_y)
41A	OSB	152/305	12.1	10.7	1.13
41B	OSB	152/305	11.9	10.7	1.11
41C	OSB	152/305	12.0	10.7	1.12
AVERAGE			**12.0**	**10.7**	**1.12**
43A	OSB	100/305	17.7	15.9	1.11
43B	OSB	100/305	18.0	15.9	1.13
43C	OSB	100/305	19.6	15.9	1.23
AVERAGE			**18.4**	**15.9**	**1.16**
45A	OSB	75/305	23.7	20.4	1.16
45B	OSB	75/305	24.3	20.4	1.19
45C	OSB	75/305	24.4	20.4	1.20
AVERAGE			**24.2**	**20.4**	**1.19**
				Average	**1.15**
				Std. Dev.	**0.043**
				CoV	**0.0374**

Table 4.10: Overstrength inherent in design for cyclic test values

Test	Panel Type	Fastener Schedule (mm)	Maximum Wall Resistance ($S_u'_+$) (positive cycle) (kN/m)	Yield Load (S_y) (Table 4.6) (kN/m)	Overstrength (S_u/S_y)
42A	OSB	152/305	11.5	10.7	1.07
42B	OSB	152/305	11.1	10.7	1.04
42C	OSB	152/305	11.9	10.7	1.11
AVERAGE			**11.5**	**10.7**	**1.07**
44A	OSB	100/305	18.2	15.9	1.14
44B	OSB	100/305	16.2	15.9	1.02
44C	OSB	100/305	17.3	15.9	1.09
AVERAGE			**17.2**	**15.9**	**1.08**
46A	OSB	75/305	22.4	20.4	1.10
46B	OSB	75/305	21.5	20.4	1.05
46C	OSB	75/305	20.5	20.4	1.00
AVERAGE			**21.4**	**20.4**	**1.05**
				Average	**1.07**
				Std. Dev.	**0.046**
				CoV	**0.0426**

4.7. 2005 NBCC AND EVALUATION OF FORCE MODIFICATION FACTORS

In the 2005 edition of the National Building Code of Canada (NBCC), a modified base shear equation for equivalent static seismic loading was introduced (Eq.4-25). What was known as the force modification factor, R, in the previous edition of the Building Code, is now referred to as the ductility-related force modification factor, R_d. The R_d factor accounts for the ability of a structure to dissipate energy in the inelastic range of deformation under seismic loading. A second force modification factor, R_o, which represents the reserve of strength within the structure, has also been included.

$$V = \frac{S(T) M_V I_E W}{R_d R_o} \tag{4-25}$$

where,

$S(T)$ = Design spectral response acceleration

M_V = Factor for higher mode effect

I_E = Importance factor of the structure

W = Seismic weight

R_d = Ductility-related force modification factor

R_o = Overstrength-related force modification factor

Values for R_d and R_o were recommended by Boudreault *(2005)* based on the previous shear wall tests carried out at McGill University. In this Section the basis for calculation of the force modification factors is first presented, along with a comparison of the values obtained from the 18 shear wall tests described in this thesis with those recommended by Boudreault.

4.7.1. Ductility-Related Force Modification Factor, R_d

To evaluate the ductility-related force modification factor the same approach used by Boudreault *(2005)* was followed. This approach is described by Equations 4-26 to 4-28, which were originally derived by Newmark and Hall *(1982)* according to the ductility and the period of the structure. Assuming that the natural period of vibration is between 0.1 and 0.5 seconds for light-framed residential housing where shear walls are used (Table 4.11), one can determine R_d using Equation 4-27.

$$R_d = \mu \qquad \text{for T} > 0.5 \text{ sec} \tag{4-26}$$

$$R_d = \sqrt{2\mu - 1} \qquad \text{for } 0.1 < \text{T} < 0.5 \text{ sec} \tag{4-27}$$

$$R_d = 1 \qquad \text{for T} < 0.03 \text{ sec} \tag{4-28}$$

where,

μ = ductility ratio (Tables 4.2, 4.3 & 4.4)

Table 4.11: Natural period for light-framed buildings

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)*

Table 4.12: Ductility and R_d values for monotonic tests

Test	Panel Type	Fastener Schedule (mm)	Ductility (μ)	R_d
41A	OSB	152/305	7.60	3.77
41B	OSB	152/305	9.19	4.17
41C	OSB	152/305	10.37	4.44
AVERAGE			**9.05**	**4.13**
43A	OSB	100/305	5.92	3.29
43B	OSB	100/305	6.86	3.57
43C	OSB	100/305	4.46	2.81
AVERAGE			**5.75**	**3.22**
45A	OSB	75/305	4.20	2.72
45B	OSB	75/305	4.44	2.81
45C	OSB	75/305	4.25	2.74
AVERAGE			**4.30**	**2.76**
			Average	**3.37**
			Std	**0.66**
			CoV	**0.195**

Table 4.13: Ductility and R_d values for reversed cyclic tests (avg. of both cycles)

Test	Panel Type	Fastener Schedule (mm)	Ductility (μ)	R_d
42A	OSB	152/305	8.13	3.91
42B	OSB	152/305	7.77	3.81
42C	OSB	152/305	6.76	3.54
AVERAGE			**7.55**	**3.75**
44A	OSB	100/305	5.34	3.11
44B	OSB	100/305	6.58	3.49
44C	OSB	100/305	6.63	3.50
AVERAGE			**6.18**	**3.37**
46A	OSB	75/305	5.91	3.29
46B	OSB	75/305	5.16	3.05
46C	OSB	75/305	4.77	2.92
AVERAGE			**5.28**	**3.09**
			Average	**3.40**
			Std	**0.34**
			CoV	**0.099**

The average ductility-related force modification factor calculated for the monotonic tests is 3.37 (SD of 0.66 & CoV of 19.5%) (Table 4.12), while a value of 3.40 (SD of 0.34 & CoV of 9.9%) (Table 4.13) was obtained for the cyclic tests. Both values are higher than 2.5, the R_d value suggested by Boudreault *(2005)*, which was based on an evaluation of 78 shear wall tests, including both OSB and plywood sheathed walls. At this stage in the development of a design approach for light gauge steel frame / wood panel shear walls a conservative approach was considered to be appropriate, and hence a value of $R_d = 2.5$ for the ductility-related force modification factor is recommended. A single value of R_d is given, even though the test results of the OSB sheathed walls indicate that a higher value could be used.

4.7.2. Overstrength-Related Force Modification Factor, R_o

The reserve of strength within a structure depends on many factors, which is why the 2005 NBCC equation for the overstrength-related force modification factor, R_o, takes the following form *(Mitchell et al., 2003)*:

$$R_o = R_{size} \times R_\phi \times R_{yield} \times R_{sh} \times R_{mech} \tag{4-29}$$

where,

R_{size} = Overstrength coming 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

The overstrength factor related to member size, R_{size}, was chosen by Boudreault *(2005)* to be equal to 1.05 because the fastener spacing used in construction is often closer than that required by the design load calculation. R_ϕ is equal to the inverse of the resistance factor, ϕ. Given the resistance factor proposed in Section 4.3 an R_ϕ factor of 1 / 0.7 = 1.43 was obtained. The R_{yield} factor is the ratio of real yield strength to the specified yield strength. In this case the comparison is made between the nominal shear strength, S_y, and the ultimate shear resistance, S_u, of the wall as measured during testing (Tables 4.9 & 4.10).

By adding these 18 shear wall results into the database created by Boudreault *(2005)*, an overall average $R_{yield} = S_u / S_y$ is obtained and shown in Table 4.14 (R_{yield}). The average R_{yield} of all 58 monotonic walls is 1.22 (SD of 0.104 and CoV of 8.53%) and the average for the 56 cyclic walls is 1.17 (SD of 0.111 and CoV of 9.46%). Therefore, the average of both monotonic and cyclic test values (Boudreault / Blais) gives an R_{yield} of 1.20 (SD of 0.110 & CoV of 9.17%), which is slightly lower than the value suggested by Boudreault; R_{yield} of 1.22 (SD of 0.109 & CoV of 8.96%). The R_{sh} factor was chosen to be equal to 1.0 since no strain hardening occurs during lateral loading of the light gauge steel frame / wood panel shear walls. Also, for R_{mech}, a value of 1.0 was suggested because no design procedures have yet been codified for use in Canada.

Table 4.14: Overstrength-related force modification factor

Reference	**Calculation of R_o**						**Proposed**
	R_{size}	R_ϕ	R_{yield}	R_{sh}	R_{mech}	R_o	**R_o (NBCC)**
Boudreault *(2005)*	1.05	1.43	1.22	1.00	1.00	1.83	
Blais *(2005)*	1.05	1.43	1.11	1.00	1.00	1.67	**1.80**
Boudreault / Blais	1.05	1.43	1.20	1.00	1.00	1.80	

Table 4.15 summarizes all of the *R*-factors necessary to determine the overstrength related force modification factor. Results are provided for the data documented by Boudreault; that corresponding to the 18 shear walls tested for this thesis; and for the overall database of shear wall tests at McGill University. To simplify design only one value of R_o was specified, regardless of whether plywood or OSB is to be used in construction. Consequently, an R_o value based on all of the previous data from Boudreault and this research were used to recommend an overstrength-related force modification factor of R_o = 1.8, which is in agreement with the value determined by Boudreault *(2005)*.

CHAPTER 5 DYNAMIC ANALYSES

5.1. INTRODUCTION

It is difficult, time consuming and costly to carry out physical tests on a vast range of buildings and their components in order to evaluate the inelastic demand on shear walls; that is buildings with different shapes, number of storeys, wall configurations, etc. For this reason it was decided to use an analytical approach to evaluate shear wall performance, with the intent of improving the base of knowledge for light gauge steel frame / wood panel shear walls. The non-linear time history dynamic analysis program Ruaumoko *(Carr, 2000)* was chosen to model and analyse two different example structures. This Chapter contains a presentation on the hysteretic element calibration, the shear wall design and the dynamic analyses that were carried out.

5.2. HYSTERETIC BEHAVIOUR OF WALLS

It was first necessary to select and calibrate an element that simulates the load vs. deflection behaviour of a light gauge steel frame / wood panel shear wall. Boudreault *(2005)* reviewed five hysteretic models that could be relied on to represent the inelastic behaviour of these shear walls: 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 & Filiatrault [CASHEW] *(Folz & Filiatrault, 2001)* models were all evaluated in terms of their applicability to this study.

Boudreault's choice of the Stewart model is based on the fact that it was originally developed for the analysis of timber framed shear walls with nailed plywood sheathing, which behave similarly, on an overall scale, to light gauge steel frame / wood panel shear walls. The model can easily be calibrated to the shear wall test results and it accounts for the pinching and stiffness degradation that were observed during testing. Furthermore, since the Stewart model is integrated into Ruaumoko, the dynamic analysis program that was selected for use in this study, the model was considered to be the most appropriate for a light gauge steel frame / wood panel shear wall element. However, the model does not allow for strength degradation, which is the reason that the true post-ultimate behaviour cannot be modeled, rather a shear deformation limit based on test results was relied on to identify the extent of inelastic demand that can be placed on a particular shear wall.

Boudreault was also responsible for carrying out the calibration of the Stewart hysteretic model for the tests completed by Branston *(2004)*, Chen *(2004)* and himself. Hence, this Section will only describe the calibration for the new series of OSB tests, which will be added to the database created by Boudreault.

Light gauge steel frame / wood panel shear walls behave in a very complex manner, as shown by the load versus displacement hysteresis in Figure 3.11. The hysteretic behaviour cannot be modeled by a single parameter, however by using the Stewart degrading hysteresis model one can mimic the behaviour, except for the strength degradation, quite easily. The Stewart hysteresis model uses parameters that allow one to fit it to the experimental data curve. The following Sections describe the main parameters that affect light gauge steel frame / wood panel shear walls under cyclic loading: strength and stiffness degradation, as well as pinching.

5.2.1. Stiffness Degradation

The reduction in shear capacity of a wall for two successive cycles at the same displacement is referred to as the stiffness degradation of the wall. The stiffness is obtained by calculating the slope of one loop (Equation 5.1, Figure 5.1).

$$K_e = \frac{F_P^{i+} - F_P^{i-}}{\Delta_P^{i+} - \Delta_P^{i-}} \tag{5-1}$$

where,

F_p : Peak force for negative or positive i-cycle, [unit of force]

Δ_p : Corresponding displacement, [unit of length]

As the stiffness decreases with additional loading cycles being applied to the wall, the area enclosed by the hysteresis loops also decreases. This area represents the capacity of the wall to dissipate energy. A degradation of the stiffness will, hence, cause the wall to dissipate less energy as compared to the previous cycle. Once most of the connections between the wood panel and the steel frame have been subjected to bearing distortion in the wood, or have failed completely, the stiffness of the wall approaches zero. This phenomenon can be observed in the partial hysteresis of Test 46A (Figure 5.2), where the two final cycles (1.5 Δ & 2.0 Δ) are shown. The energy dissipation represented by the first loop (Area 1) is much larger than that found for the following loop (Area 2). Also a significant decrease can be observed in stiffness, K_{e1} and K_{e2}, for the two consecutive loops.

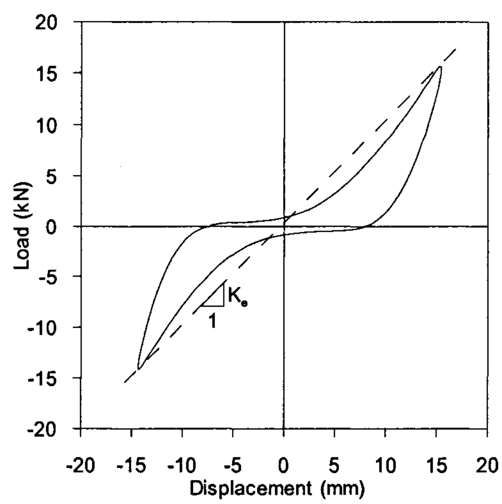


Figure 5.1: Stiffness of one loop

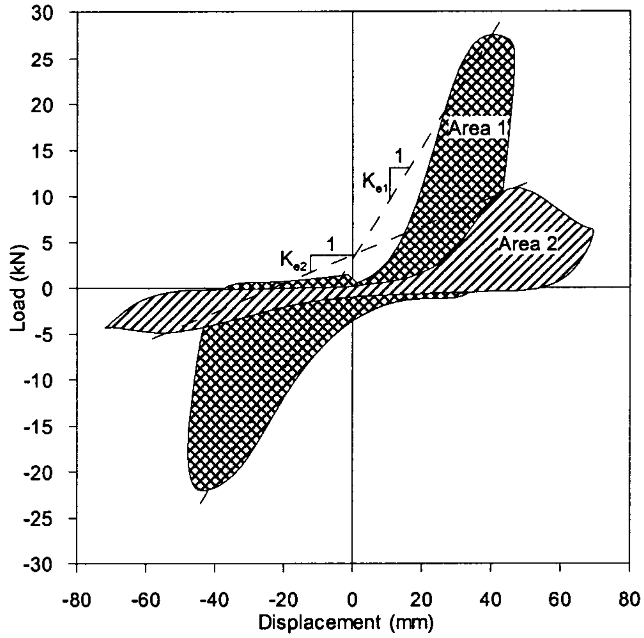


Figure 5.2: Energy dissipation & stiffness degradation (Test 46A – 1.5 & 2.0 Δ Cycles)

5.2.2. Pinching

Pinching is caused by the permanent deformation of the wood sheathing around the screw connections. During loading, the screws are placed in shear, first tilting and then bearing against the wood around them. Once permanent bearing distortion has occurred a slot is created in the wood (Figure 3.18). At this stage the connection is only able to develop a shear resistance when the screw fastener comes into contact with the edge of the slot. The inability of the connection to resist load at low displacements due to the bearing distortion caused by previous loading cycles is known as pinching. Figure 5.3 shows three hysteresis loops with different degrees of pinching during loading. At low displacement, little damage is done to the wall and therefore almost no pinching can be observed (Figure 5.3[a]). As the displacement increases, the pinching effect becomes more and more evident (Figure 5.3[b] & [c]). The effect of pinching is twofold; first of all the ability of the shear wall to dissipate energy decreases significantly, and secondly, the wall possesses no in-plane lateral stiffness near the zero displacement region.

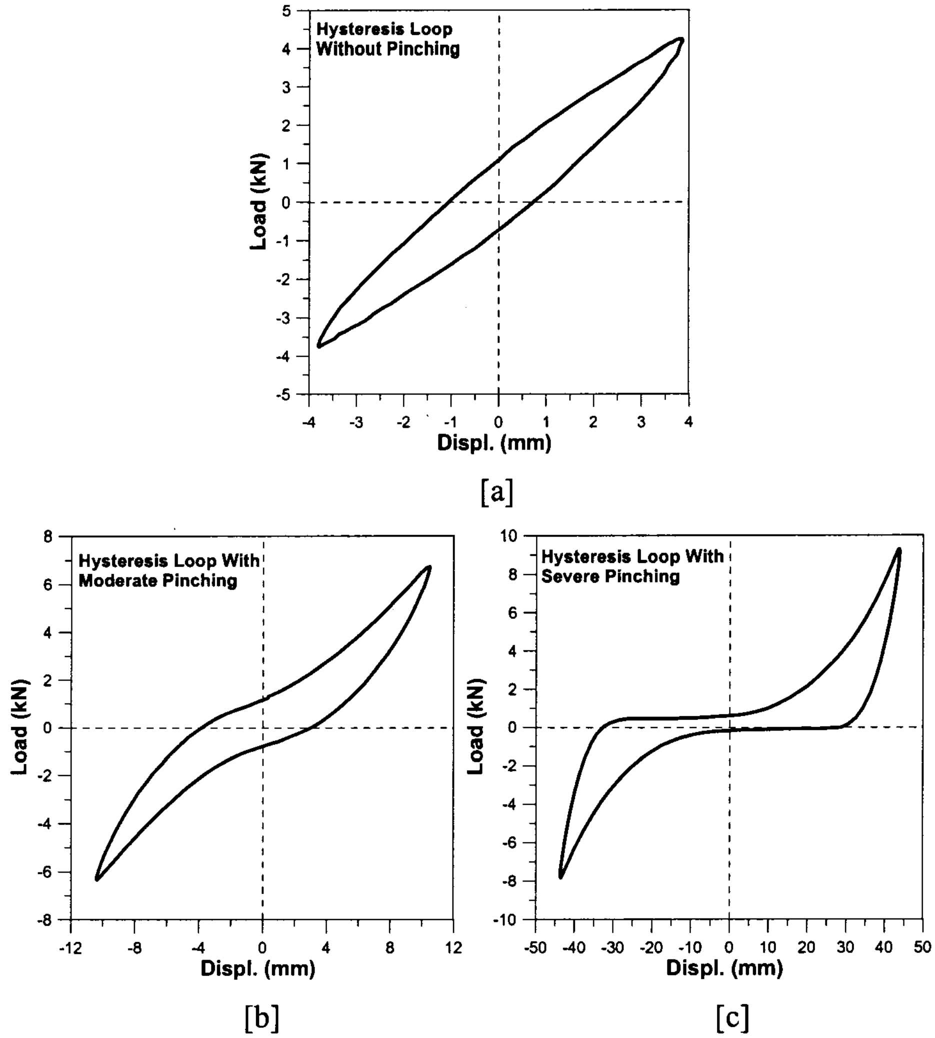


Figure 5.3: Evolution of pinched hysteresis loops with increased displacement level
(Boudreault, 2005)

5.2.3. Strength Degradation

Strength degradation can be identified when a wall, which is pushed to the same displacement level in two consecutive cycles, is not able to maintain its shear resistance. Once again, for the walls tested for this research, the damage to the wood surrounding the screw connections is responsible for this behaviour. The wood is not able to reach the same level of resistance due to the permanent bearing distortion. Figure 5.4 shows an example of strength degradation of a shear wall after two consecutive cycles at the same displacement.

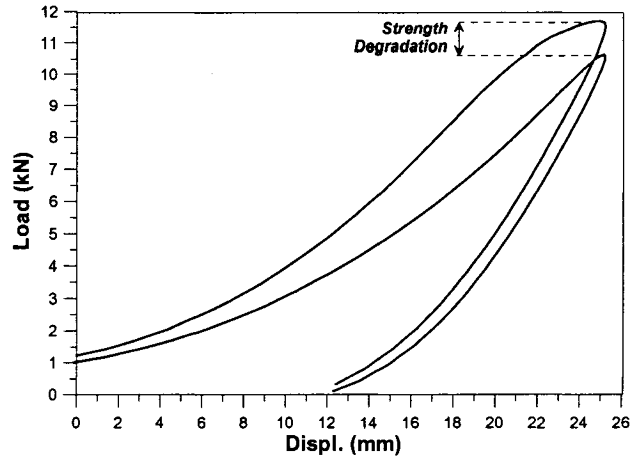


Figure 5.4: Strength degradation representation between two successive loops *(Boudreault, 2005)*

5.3. STEWART DEGRADING HYSTERESIS

The Stewart degrading hysteresis model (Figure 5.5) was initially developed for wood shear walls with nailed connections *(Stewart, 1987)*. It since has been used for other types of shear walls, such as reinforced concrete and steel shear walls *(Carr, 2000)*. It has also been used to model steel diaphragm systems whose resistance vs. displacement behaviour is highly dependent on the performance of the individual connections *(Martin, 2002; Yang, 2003)*. Hence, it is not unexpected that Boudreault *(2005)* recommended that the Stewart hysteresis element be used to model the light gauge steel frame / wood panel shear walls that are the subject of this research. The model has been formulated with parameters that account for both stiffness degradation and pinching, as well as ultimate and yield force, slackness, softening, reloading, etc. However, the model does not incorporate the effect of strength degradation on shear wall behaviour. This phenomenon is said to be less significant than the stiffness degradation and the pinching effect *(Boudreault 2005, Stewart 1987, Ceccotti & Vignoli 1989, Dolan 1989)*, and hence its exclusion from the model is not considered to be critical. All the parameters necessary for definition of the Stewart hysteresis model such that it mimics the shear wall behaviour were derived from the experimental data, as is described in Section 5.3.1.

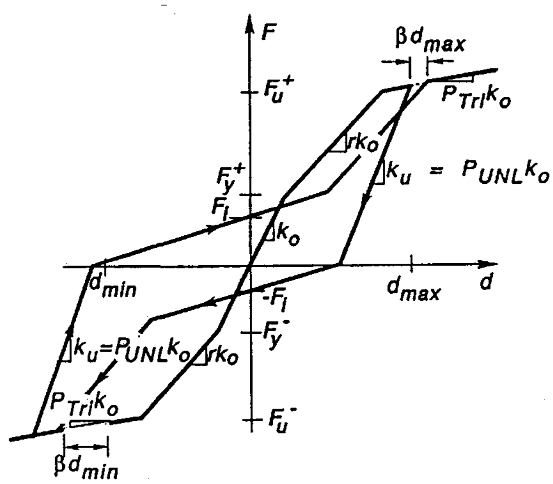


Figure 5.5: Stewart degrading hysteresis *(Carr, 2000)*

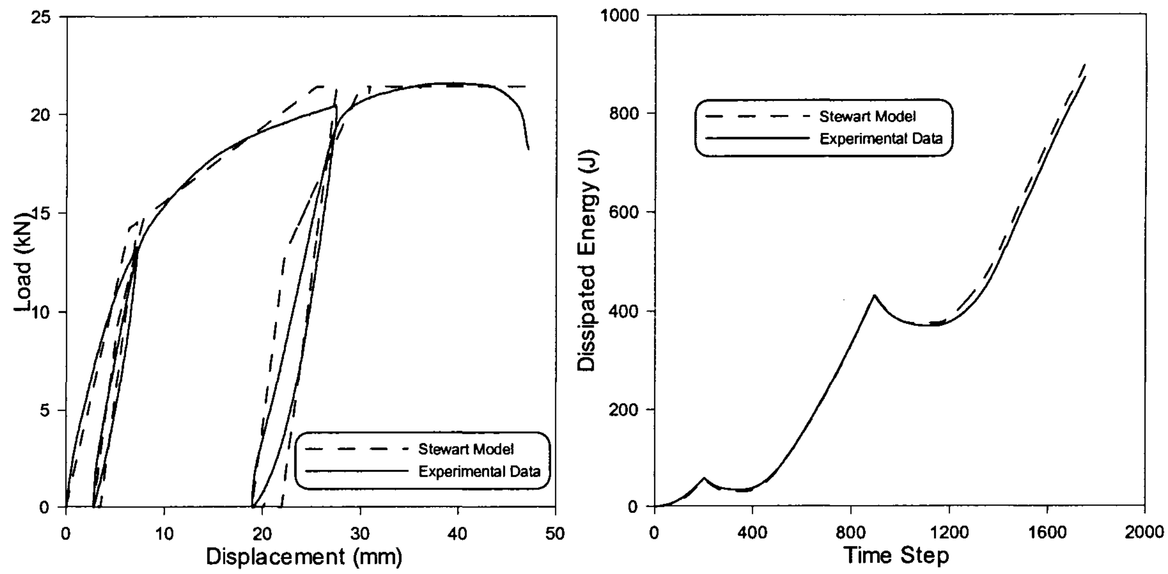
5.3.1. Experimental Data Matching

Over 30 parameters are required to accurately replicate the experimental hysteresis of a shear wall using the Stewart model. Of these parameters, seven were found by trial and error, one was calculated as described in Section 5.3.2 (F_u), and the remaining, which concerned the frame type properties, were obtained by following the Ruaumoko manual. The selection of values for the different variables was made by visual inspection and comparison of the experimental data curves and the Stewart hysteresis, as well as by an energy balance between the two curves. Table 5.1 shows the recommended parameters for the three wall configurations used in this body of research. Values for the stiffness, K_o, and the yield force, F_y, were obtained by visual inspection and not by using the wall stiffness and yield force values found with the EEEP method, as discussed in Section 4.2. The values of most parameters increase as the screw spacing decreases, with the exception of the pinch factor alpha, α, which is larger for the 152/305 mm (6"/12") screw pattern (0.52) compared with a value of 0.45 for the two other screw patterns. Also, one can note that the unloading stiffness factor, P_{UNL}, is lower for the smallest screw spacing (75 mm, (3")).

Table 5.1: Stewart hysteresis parameters for light gauge steel frame / OSB panel (9 mm (3/8"))

Tests	Wood Panel	Size (mm)	Screw Pattern (mm)	K_o (kN/mm)	r	F_y (kN)	F_u (kN)	F_i (kN)	P_{UNL}	α	β
41&42	OSB	1220×2440	152/305	1.88	0.15	9.50	14.25	1.15	1.75	0.52	1.09
43&44	OSB	1220x2440	100/305	2.22	0.17	14.23	21.41	1.70	1.75	0.45	1.09
45&46	OSB	1220x2440	75/305	2.61	0.19	18.50	27.45	2.50	1.45	0.45	1.09

The first step in matching the test results with the Stewart hysteresis model consisted of superimposing the three monotonic test curves for a particular wall configuration. Then Hysteres, a program within the non-linear time history dynamic analysis program Ruaumoko, was run using the values listed in Table 5.1 to obtain the best fit curve, based on visual inspection (Figure 5.6a). Note, in this figure only the monotonic curve for test 43A is plotted. Since a single set of parameters representing all the walls with the same configuration was needed, only one hysteresis per type of wall was created. Therefore, this hysteresis needed to fit all three monotonic curves. A perfect match was impossible to obtain because of the non-linear properties of the shear wall behaviour and because more than one test curve was considered. To assist in the selection of the parameter values, the energy dissipation was also relied on. The cumulative energy, calculated from the area under the modeled monotonic curve, was kept within 10% of that obtained from the experimental resistance vs. displacement curves (Figure 5.6b).

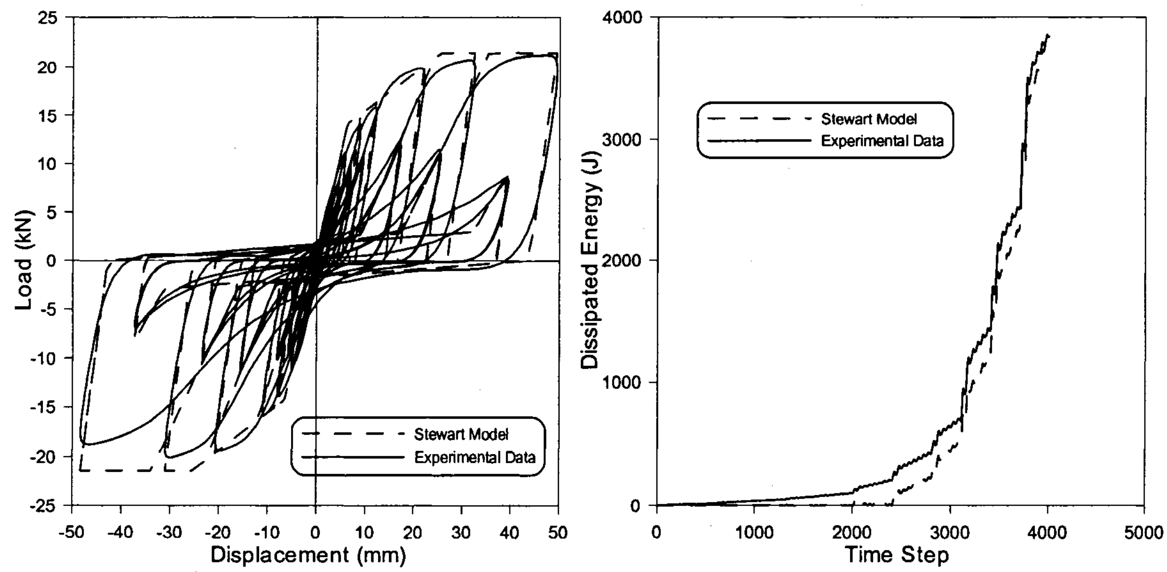


[a] Resistance vs. displacement

[b] Cumulative energy dissipation

Figure 5.6: Superposition of Stewart model and experimental monotonic curve (Test 43A)

The same process was applied to the cyclic test data. That is, the experimental set of data for the three shear walls built with the same screw schedule were superimposed to find the best combination of parameters. In addition, the parameter values needed to be adequate for the monotonic tests. Therefore, the parameters obtained from the modeling of the monotonic curves were used and then modified as necessary to calibrate the cyclic test models. As a final recommendation, only one set of parameters per wall configuration, which independently represent the monotonic and cyclic behaviour, were provided (Table 5.1). This matching process for the cyclic tests also relied on visual inspection and a cummulative energy dissipation check, as described for the monotonic tests. Figure 5.7 [a] shows the hysteresis behaviour of a typical 1220 × 2440 m (4' × 8') OSB shear wall with the Stewart hysteresis model, while Figure 5.7 [b] illustrates the cumulative dissipated energy of both the test and model hystereses. Figures that show the superposition of test and modeled hystereses can be found in Appendix 'C'.



[a] Resistance vs. displacement

[b] Cumulative energy dissipation

Figure 5.7: Superposition of Stewart model and experimental cyclic hysteresis (Test 44C)

5.3.2. Limitations

Even with good representation of all the experimental hystereses as provided by the parameter values listed in Table 5.1, the Stewart model has some limitations. First of all, the movement of the wall from positive to negative displacements is defined as symmetric according to the model. In contrast, the experimental hystereses show that a shear wall first pushed in the positive direction to a certain displacement will not reach the same resistance level in the following negative displacement excursion because some damage has occurred during the initial segment of the displacement cycle. This phenomenon cannot be accounted for in the model. However, as shown in Section 4.4, the ultimate shear resistance, S_u, was calculated based on both the negative and positive segments of the reversed cyclic test hysteresis. This average value, which is lower than the one expected in the positive loading segment alone, was used to define the maximum wall resistance of the model. In this fashion, and by incorporating the cumulative energy balance, it was possible to account for the discrepancy between the resistance level reached during the positive and negative cycles of the physical shear wall test.

A second limitation of the model is due to the fact that it does not include a parameter that indicates whether or not the wall has reached failure. That is, the model will deform indefinitely with the maximum shear resistance maintaining its ultimate value. This is in contrast to the tested walls, which exhibited a decrease in capacity once the lateral displacement became large. In Section 3.1.6, it is explained that the test wall was considered to have reached failure when the post ultimate load decreased to 80% of the ultimate (peak) load.

To account for this shortcoming in the subsequent analyses of building models using Ruaumoko (Section 5.4) the maximum rotation that a shear wall can undergo based on the 80% post ultimate load was defined (Eq. 5-1). This equation is similar to those used for average shear stiffness and resistance of the wall in Section 4.4. The results of the dynamic analyses were then compared with this limit (Table 5.2) to establish whether the wall remained within its useable performance range.

$$\theta_{0.8u,avg} = \frac{\theta_{0.8u,mono,avg} + \dfrac{\theta_{0.8u+,avg} + \theta_{0.8u-,avg}}{2}}{2} \tag{5-1}$$

where,

$$\theta_{0.8u,mono,avg} = \frac{\sum\left[\theta_{net,0.8u}\right]_{mono}}{n}, \text{ [rad]} \tag{5-2}$$

$$\theta_{0.8u+,avg} = \frac{\sum\left[\theta_{net,0.8u+}\right]_{cyclic}}{n}, \text{ [rad]} \tag{5-3}$$

$$\theta_{0.8u-,avg} = \frac{\sum\left[\theta_{net,0.8u-}\right]_{cyclic}}{n}, \text{ [rad]} \tag{5-4}$$

$\theta_{net,0.8u}$ = Rotation at 80% of ultimate shear force (after peak load), [rad]

n = number of walls with same configuration and protocol

Table 5.2: Maximum rotation of shear walls

OSB 9mm (3/8")	
Screw Pattern (mm)	**Maximum Rotation (10^{-3} rad)**
75/305	19.1
100/305	21.2
152/305	22.3

One other important limitation is due to the values that were recommended for the stiffness and the yield force parameters of the model (Table 5.1). These values were chosen to fit the monotonic test curve and the cyclic hystereses loops as precisely as possible. The stiffness and strength values recommended for modeling are not those calculated in Chapter 4 using the equivalent energy elastic-plastic (EEEP) approach, and hence should not be used for design. Rather, these values should only be incorporated in the non-linear time history dynamic analysis of buildings. It is important to make this point clear such that no errors are made while interpreting the output results from the analyses presented in Section 5.4.

Finally, the modeling parameters derived in this Chapter are appropriate for the three wall configurations of 9 mm OSB included in this research. If another wall configuration is to be analyzed, one would need to consult the recommendations by Boudreault *(2005)*, which cover shear walls with 12.5 mm plywood and 11 mm OSB sheathing. If the wall configuration is different from what is found herein and in the work by Boudreault, then additional testing would be required to establish the correct model parameters.

5.4. DESIGN AND BASE SHEAR CALCULATION

Typically light gauge steel frame / wood panel shear walls are used in buildings of relatively small proportions. That is residential structures, condominiums and small commercial buildings. Hence, to evaluate these shear walls under earthquake loading, it was decided to design the seismic force resisting system (SFRS) of two representative buildings located in Vancouver, BC. Their design was carried out following the 2005

National Building Code of Canada *(NRCC, 2005)* and the shear wall design values tabulated in Chapter 4 of this thesis. The first structure is a traditional two-storey Canadian house of approximately 167.2 m^2 (1800 ft^2) and the second structure is a small three-storey commercial building of about 174.2 m^2 (1875 ft^2). The design of both structures, as well as the respective base shear calculation, is presented in the following Sections. Note that the design of the shear wall system was done without consideration of the lateral wind loads applied to the building, which in some cases could control the selection of framing, sheathing and connection components.

5.4.1. House Design

According to the Canadian Home Builder's Association (CHBA), the average Canadian house has an area of 167 m^2 (1800 ft^2), which is equivalent to a 7.2 m x 11.6 m two-storey house (83.5 m^2 / storey) *(CHBA, 2003)*. To keep the SFRS design as simple as possible, it was decided to specify a symmetric rectangular shape house with a flat roof. The entire wall and floor structure is to be made of light gauge steel framing with OSB panels used for sheathing and flooring. It was also assumed that two 1220 mm (4') long shear walls would be symmetrically placed along each face of the building (Figure 5.8).

The approximate specified dead loads for the roof, wall and floor were found in the load tables included in the Handbook of Steel Construction, 8th Edition *(CISC, 2004)*. Table 5.3 summarizes the respective weights of the structural and non-structural components of the house. Table 5.4 lists the live loads for both the roof and the floor. To obtain the live load due to the snow accumulation on the roof, Equation 5-5 was used. As noted previously the snow (S_s) and rain (S_r) loads were those for Vancouver, BC. The basic roof snow load factor (C_b) was taken as equal to 0.8 as suggested by the NBCC since the amount of snow on the roof is usually lower than on the ground. The wind exposure factor (C_w) was chosen to be 1.0 because in suburbs, houses are about the same height and are a few metres apart, which leaves their roofs well exposed to wind. As the roof chosen for this specific design had no slope, a value of 1.0 for the slope factor (C_s) and for the accumulation factor (C_a) was used.

Table 5.3: House dead loads

Roof	Description	Load (kPa)
Sheathing	3/4" (19.1 mm) plywood	0.10
Insulation	Glass fibre blown (100 mm)	0.04
Ceiling	Gypsum (12.5mm)	0.10
Joists	Light gauge steel: 600 mm apart	0.12
Roofing	3-ply + gravel	0.27
Other fixtures		0.03
Total load for roof		**0.66**
Floor		
Interior partitions		1.00
Flooring	Hard wood (25 mm)	0.20
Sheathing	3/4" (19.1 mm) OSB	0.09
Joists	Light gauge steel: 400 mm apart	0.15
Ceiling	Gypsum (12.5 mm)	0.10
Other fixtures		0.03
Total load for floor		**1.57**

Table 5.4: House live and snow loads (NBCC)

Roof	Description	Load (kPa)
Snow	Vancouver region	1.64
Total load for roof		**1.64**
Floor		
House – live		1.90
Total load for floor		**1.90**

The snow load equation as prescribed by the 2005 NBCC accounts for the geographical location, exposure, roof shape and slope:

$$S = I_s \times \left[S_s \times \left(C_b \times C_w \times C_s \times C_a\right) + S_r\right] \tag{5-5}$$

where,

S = Snow load, [kPa]

I_s = 1.0, importance factor for snow load (2005 NBCC)

S_s = 1.8 kPa, snow load for Vancouver (1/50yr) (2005 NBCC)

$C_b = 0.8$, basic roof snow load factor (2005 NBCC)

$C_w = 1.0$, wind exposure factor (2005 NBCC)

$C_s = 1.0$, roof slope factor (flat roof) (2005 NBCC)

$C_a = 1.0$, accumulation factor (2005 NBCC)

$S_r = 0.2$ kPa, rain load for Vancouver (1/50yr) (2005 NBCC)

Also according to the 2005 NBCC, the structural components of a building must be designed with the most unfavourable effect of the seven load combinations shown in Table 4.1.3.2 and requirements of Article 4.1.3.2 of the Code. Since it is the shear wall element subjected to earthquake loading that is the main subject of this study, load case 6 of the NBCC was considered to be applicable (Eq.5-6).

$$1.0D + 1.0E + 0.5L + 0.25S \tag{5-6}$$

where,

D = Specified dead load, [kN]

E = Specified earthquake load, [kN]

L = Specified live load, [kN]

S = Specified snow load, [kN]

In order to obtain the factored loads used in design, the specified loads given in Tables 5.3 and 5.4 would need to be multiplied by their respective tributary areas (T.A.) and the appropriate factors, which are shown in Equation (Eq.5-6). However, for the modeling that was carried out it was assumed that only the dead load and 25% of the specified snow load contributed to the seismic force in the building (Eq. 5-7). In some situations it is considered appropriate to include a proportion of the specified live load to account for the permanent components of the live load in the seismic weight, such as partition walls. Nonetheless, for these analyses the forces on the shear walls did not include any contribution from the live load on the buildings. Furthermore, it was assumed that the floor and roof diaphragm structures acted in a rigid fashion, and hence only the shear walls of the building were modeled. For this reason the seismic weight at each storey was based on the area tributary to an individual shear wall, i.e. one quarter of the floor or roof area, even though the full gravity effects of these loads could have been supported by other parts of the structure. The total loads at each storey, which are necessary for the calculation of equivalent static seismic base shear forces and for the dynamic analyses, are shown in Table 5.5.

$$1.0D + 0.25S \tag{5-7}$$

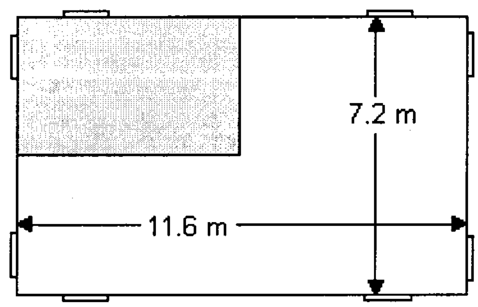


Figure 5.8: House plan view with shear wall tributary area in grey

Table 5.5: Specified load calculation per shear wall – house design

Storey	Type of load	Specified load (kPa) (Table 5.3 & 5.4)	T.A. for one wall (m^2)	Specified load (kN)	1.0D+0.25S (kN)	Total load (kN) (Eq.5-7)
1st storey	Dead	1.57	20.9	32.8	32.8	32.8
Roof	Dead	0.66	20.9	13.8	13.8	22.3
	Snow	1.64	20.9	34.2	8.56	

5.4.2. Small Commercial Building Design

A small commercial building was included in the scope of study in order to examine the seismic performance of a structure with more than two floors and with higher lateral loads than a residential building. The floors of this building consisted of a concrete slab, which added to the seismic weight of the building, and hence the base shear. A three-storey structure of 7.62 m × 7.62 m (25' × 25') with light gauge steel frame / wood panel walls and a Hambro® floor system *(Canam Group, 2004)* was chosen (Figure 5.9). The rest of the structure was kept as simple as possible by using a symmetrical square shape with a flat roof.

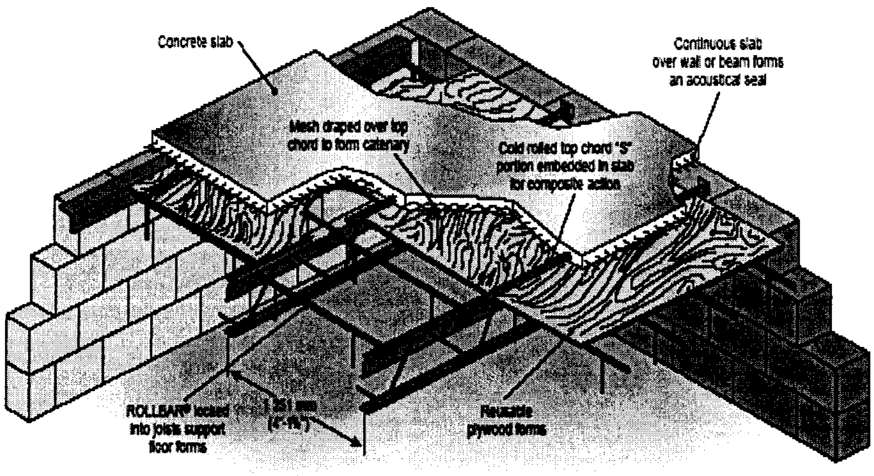


Figure 5.9: Hambro® D500 floor system *(Canam Group, 2004)*

Both the Handbook of Steel Construction, 8th Edition *(CISC, 2004)*, and the Hambro D500 document *(Canam Group, 2004)* were used to approximate the specified dead loads for the roof and floors. Table 5.6 summarizes the respective weights of the structural and non-structural components of the commercial building. Table 5.7 lists the live loads for both the roof and the floors. To approximate the snow accumulation on the roof of the building Equation 5-5 was used. The results for the snow load as well as those for the occupancy live load are shown in Table 5.7.

Table 5.6: Small commercial building dead loads

Roof	Description	Load (kPa)
Sheathing	3/4" (19.1 mm) plywood	0.10
Insulation	Glass fibre blown (100 mm)	0.04
Ceiling	Gypsum (12.5mm)	0.10
Joists	Light gauge steel: 600 mm apart	0.12
Sprinklers		0.03
Roofing	3-ply + gravel	0.27
Other fixtures		0.03
Total load for roof		**0.69**
Floor		
Interior partitions		0.72
Flooring	Hard wood (25 mm)	0.19
Concrete slab	3" thick (75 mm)	1.77
Acoustic tile	12 mm	0.04
Joists	1251 mm apart	0.12
Other fixtures		0.03
Total load for floor		**2.87**

Table 5.7: Small commercial building live loads (NBCC)

Roof	Description	Load (kPa)
Snow	Vancouver region	1.64
Total load for roof		**1.64**
Floor		
Commercial structure	On ground	4.80
	Above ground	2.40
Total load for floor		**2.40**

Using Equation 5-7, the loads for each floor were computed. Table 5.8 summarizes the specified loads (dead and live), the tributary area for a given shear wall and the total loads per floor per shear wall. The calculation of these loads was based on the same philosophy as outlined for the house in Section 5.4.1. These values will later be used in the dynamic analyses and for the calculation of NBCC equivalent static loads.

Table 5.8: Factored load calculation per shear wall – commercial building design

Storey	Type of load	Specified load (kPa) (Tables 5.6 & 5.7)	T.A. for one wall (m^2)	Specified load (kN)	1.0D+0.25S (kN)	Total load (kN) (Eq.5-7)
1^{st} storey	Dead	2.87	14.5	41.7	41.7	41.7
2^{nd} storey	Dead	2.87	14.5	41.7	41.7	41.7
Roof	Dead	0.69	14.5	10.0	10.0	16.0
	Snow	1.64	14.5	23.81	5.95	

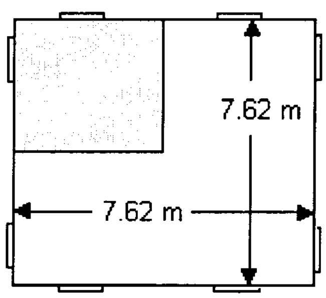


Figure 5.10: Commercial building plan view with shear wall tributary area in grey

5.5. NBCC 2005 BASE SHEAR CALCULATION

The 2005 NBCC seismic provisions are based on a uniform hazard spectrum with a 2% in 50 year probability of exceedance. This is one of the major changes from the 1995 to the 2005 version of the code, where previously a 10% in 50 year probability of exceedance was used for the determination of seismic hazard. The other major changes to the seismic provisions are as follows: "updated hazard in spectral format, period-dependent site factors, delineation of effects of overstrength and ductility, modified period calculation formulae, explicit recognition of higher mode effects, rational treatment of irregularities, triggers for special provisions incorporated directly in classification of structural systems, and placing dynamic analysis as the normal "default" method of analysis for use in seismic design" *(Heidebrecht, 2003).*

The overstrength and the ability of a structure to perform in the inelastic range of behaviour are two aspects on which engineers base their design for seismic actions in Canada. Equation 4-25 shows the 2005 NBCC design base shear (V) in which the factors R_d and R_o are used to reduce the elastic shear force. These two force reduction factors must be known prior to the shear force calculation in order to design the structure. In the case of light gauge steel frame / wood panel shear walls, the ductility-related ($R_d = 2.5$) and overstrength-related ($R_o = 1.8$) force modification factors were derived from full scale tests as described in Chapter 4.

$$V = \frac{S(T)M_v I_E W}{R_d R_o} \tag{4-25}$$

However, the result of the base shear equation is bounded by the following two equations:

$$V \geq \frac{S(2.0)M_V I_E W}{R_d R_o} \tag{5-8}$$

$$V \leq \frac{2}{3}\frac{S(0.2)I_E W}{R_d R_o} \tag{5-9}$$

where,

$S(T)$ = Design spectral response acceleration

$T = 0.05 \times h_n^{3/4}$ (for shear walls) (5-10)

T = Period of the structure, [s]

h_n = Height of the structure, [m]

M_V = Factor for higher mode effects

I_E = Importance factor of the structure

W = Seismic weight, [kN]

R_d = Ductility-related force modification factor

R_o = Overstrength-related force modification factor

From the period the spectral response acceleration is interpolated between Tables 4.1.8.4.B & 4.1.8.4.C of the 2005 NBCC. A site class E was chosen for both designs because Vancouver is mainly considered as a soft soil region. The factor for higher mode effects, M_v, depends on the $S_a(0.2)$ / $S_a(2.0)$ ratio, the type of lateral resisting system and the period of the structure. From Table 4.1.8.11 of the 2005 NBCC, M_v was chosen to be 1.0 since the period of both structures is below 1.0 s regardless of the $S_a(0.2)$ / $S_a(2.0)$. The importance factor, I_E, is equal to 1.0 as both structures fall in the normal importance category. The seismic weight is equal to the dead load of the structure plus 25% of the snow load, which is equal to 220.8 kN and 397.3 kN for the house and the commercial structure respectively. All the variables described with their corresponding values are summarized in Table 5.9.

Table 5.9: Variables for shear base calculations for both structures

Variables of Eq.4-25	**Residential Building**	**Commercial Building**
R_d	2.5	2.5
R_o	1.8	1.8
I_E	1.0	1.0
M_v	1.0	1.0
W	220.8 kN, W_1 = 131.3 kN W_2 = 89.5 kN	397.3 kN, W_1, W_2 = 166.7 kN W_3 = 63.9 kN
T	0.17 sec.	0.25 sec
h_n	5.19 m	8.52 m
$S(T)$	0.891 ($F_aS_a(0.2)$)	0.891 ($F_vS_a(0.25)$)
$S_a(0.2)$	0.94	
$S_a(0.5)$	0.64	
$S_a(1.0)$	0.33	
$S_a(2.0)$	0.17	

The design base shear is computed with equations 4-25, 5-8 and 5-9. In both cases the base shear given by Equation 5-9 is smaller than that of Equation 4-25. Therefore a value of 29.2 kN and of 52.4 kN are used as the total base shear for the house and the small commercial structure respectively (Table 5.10). Note that the minimum base shear (Eq. 5-8) requirement is met for both buildings. The base shear force acts at the centre of mass of the structures which happen to match the centre of gravity due to the symmetry of the structures. However, torsional effects should be considered as prescribed by the 2005 NBCC. To account for torsional effects, the base shear force was applied at a distance $0.1D_n$ from the centre of mass of the storey. Therefore, using statics, the largest lateral force induced on one shear wall was found, as follows:

$$V_{tor} = V \times \left(\frac{0.1D_n + 0.5D_n}{D_n} \right) \div 2 = 0.3 \times V \quad (5\text{-}11)$$

where,

V_{tor} = Base shear induces to one shear wall with torsional effects, [kN]

V = Base shear, [kN] (Table 5.10)

D_n = Plan dimension of the building perpendicular to earthquake load, [m]

The results of Equation 5-11 are shown in Table 5.10. The V_{tor} values found (one per building) represent the total base shear acting on a single shear wall. Figure 5.11 shows both structures with their respective shear walls in grey.

Table 5.10: Base shear value for design

Base shear calculation	**Residential Building V (kN)**	**Commercial Building V (kN)**
Equation 4-25	43.7	78.6
Equation 5-8	15.2	26.7
Equation 5-9	29.2	52.4
Design Base Shear, V	**29.2**	**52.4**
V_{tor}	**8.76**	**15.7**

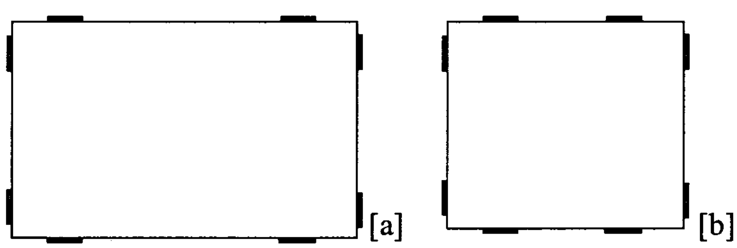


Figure 5.11: Plan view of both structures with location of all four shear walls: [a] house, and; [b] commercial building

The base shear force, V_{tor}, values were then distributed over the height of the buildings (at every storey) according to Equation 5-12.

$$F_x = \frac{(V_{tor} - F_t) W_x h_x}{\sum_{i=1}^{n=2} W_i h_i} \qquad (5\text{-}12)$$

where,

F_x = Expected force on one shear wall at storey x, [kN]

V_{tor} = Base shear induces to one shear wall with torsion effects, [kN] (Table 5.10)

F_t = 0 since T < 0.7s, [kN]

W_x = Seismic weight at storey x, [kN] (Table 5.9)

h_x = height of storey x, [m]

$W_i h_i$ = Seismic weight times storey height for storey i, [kNm]

In Table 5.11, F_x values, which represent the applied seismic load at the floor and roof levels, and the S_x values, which are the cumulative shear transferred down through the SFRS, are provided. To view the complete calculation of these values as well as the calculation of base shear design values refer to Appendix 'D'.

Table 5.11: Maximum shear force developed per shear wall

	Residential Building		**Commercial Building**	
Storey	***F_x* (Eq.5-12) (kN)**	**S_f per wall (kN/m)**	***F_x* (Eq.5-12) (kN)**	**S_f per wall (kN/m)**
1st Floor	5.46	7.18	6.55	12.8
2nd Floor	3.30	2.70	6.65	7.46
3rd Floor			2.45	2.01

5.5.1. Shear Wall Configuration Based on Resistance

The maximum seismic shear forces obtained for the house using the 2005 NBCC approach (S_{f1} = 7.18 kN/m and S_{f2} = 2.70 kN/m) (Table 5.11) are smaller than the lowest factored resistance value obtained for the shear walls tested in this body of research (S_r =

7.5 kN/m) (Table 4.6). Therefore, all the shear walls to be used in the construction of the two-storey residential structure were specified to consist of 9 mm thick OSB sheathing with a screw spacing of 152 mm (6") along the perimeter of the panels. Walls with a greater shear resistance were needed for the commercial building due to the elevated seismic shear forces. In this case a shear wall with 9 mm thick OSB panels connected to the framing using a screw spacing of 75/305 mm (3"/12") was necessary to ensure that S_{f1}, S_{f2}, and S_{f3} were below the shear resistance (S_r = 14.3 kN/m) (Table 4.6).

Other wall configurations, as detailed by Branston *(2004)*, could have been specified for these buildings. It was decided, however, to use the wall configurations tested for this research project since they possessed adequate capacity. Hence, 9 mm thick OSB panels were used for the shear walls of both structures. In addition, other wall configurations could have been chosen for the upper storey of the three storey building, i.e. walls with a screw spacing of 152 or 100 mm (6" or 4"), because of the lower applied shear forces. Owing to the preliminary nature of the dynamic analyses documented in this thesis, in terms of simplicity it was decided to use only one wall configuration per model. By taking this approach the lower storey of the SFRS will always control the design. As well, it is reasonable to assume that the greatest inelastic demand, as obtained from the dynamic analyses (Section 5.7), will also be placed on the lower storey. In the future it is recommended that additional non-linear time history analyses be carried out on buildings in which the screw spacing is increased for the upper storeys.

5.5.2. Shear Wall Configuration Based on Stiffness

Once the shear wall configuration had been selected based on resistance, it was necessary to evaluate the deflection of the SFRS and to compare it with the drift limit as defined by Section 4.1.8.13 of the 2005 NBCC. Note that only the inelastic seismic drift limit of 2.5% of the storey height was considered for these calculations. Elastic deflections based on service wind loads were not evaluated.

The elastic displacement (Δ_e) is the maximum lateral displacement of a storey when a static force is applied at a distance $0.1D_n$ from the centre of mass of the storey, i.e. accidental torsion effects are accounted for. Therefore, the shear force presented in Table 5.11 can be used directly to calculate the elastic displacement because it already includes the torsion effects. The elastic displacement can be obtained with Equation 5-13.

$$\Delta_{e,n} = {S_{f,n}}/{k_e} \tag{5-13}$$

where,

$\Delta_{e,n}$ = Elastic displacement at storey n, [mm]

$S_{f,n}$ = Factor shear resistance at storey n, Table 5.11, [kN/m]

k_e = Stiffness of wall, [kN/mm/m] (Table 4.6)

To obtain more realistic values of the anticipated inelastic storey deflection (Δ_{max}) under seismic loads, the elastic displacement (Δ_e) is multiplied by R_dR_o/I_E (Eq.5-14). The largest interstorey drift must be kept under 0.025 times the storey height (Eq.5-15) *(NRCC, 2005)*. The results are shown in Table 5.12 for both types of building and for all storeys.

$$\Delta_{\max,n} = \frac{R_d R_o}{I_E} \times \Delta_{e,n} \tag{5-14}$$

where,

$\Delta_{\max,n}$ = Anticipated inelastic deflection of storey n, [mm]

R_d = 2.5, Ductility-related force modification factor, Chapter 4.7.1

R_o = 1.8, Overstrength-related force modification factor, Chapter 4.7.2 (Table 4.14)

I_E = 1.0, Importance factor of the structure, Table 4.1.8.5 of the 2005 NBCC

$\Delta_{e,n}$ = Elastic displacement at storey n, [mm] (Eq.5-13)

$$\Delta_{drift\,\lim,n} = 0.025 \times h_s \tag{5-15}$$

where,

$\Delta_{driftlim,n}$ = Drift limit, section 4.1.8.13. 3 of the 2005 NBCC, [mm]

h_s = height of storey s, [m]

Table 5.12: Calculation for drift limitation

Drift Limitation	Residential Building	Commercial Building
$\Delta_{e,1}$	4.4 mm	6.1 mm
$\Delta_{e,2}$	1.6 mm	3.6 mm
$\Delta_{e,3}$		1.0 mm
$\Delta_{max,1}$	19.8 mm	27.5 mm
$\Delta_{max,2}$	7.2 mm	16.2 mm
$\Delta_{max,3}$		4.5 mm
$\Delta_{driftlim,1}$	68.8 mm	71.0 mm
$\Delta_{driftlim,2}$	61.0 mm	72.3 mm
$\Delta_{driftlim,3}$		69.8 mm

As can be seen in Table 5.12, $\Delta_{driftlim,n}$ is always larger than $\Delta_{max,n}$, which indicates that the wall configurations chosen for the two buildings are adequate based on the inelastic seismic drift limit. In summary, a design that includes four shear walls per storey parallel to the earthquake load made with 9 mm OSB panels and with a screw spacing of 152/305 mm (6"/12") is adequate for the residential building. In the case of the small commercial building, the same number of walls is needed, except that the shear loads are greater. Therefore 9 mm OSB wood panel shear walls with a screw spacing of 75/305 mm (3"/12") was chosen and proven to be adequate for this structure.

5.6. Dynamic Analyses

This Section describes the non-linear time history dynamic analyses that were carried out for the two representative buildings. A two-storey shear wall model with Stewart hysteretic elements *(Stewart, 1987)* was used for the house design in addition to a three-storey shear wall model for the small commercial building design. As noted previously, the floor and roof diaphragms were considered rigid, and hence, were not included in the model. Ruaumoko *(Carr, 2000)* was used for the dynamic analyses with ten earthquake ground motion records from the west coast of North America. The earthquake records were scaled to the spectral acceleration curve from the 2005 NBCC for the Vancouver, BC, Canada region.

5.6.1. Earthquake Records

The earthquake ground motion records of interest were those that would represent possible seismic events on the west coast of Canada. A total of ten records, four simulated and six real earthquake records from California and Washington States, were obtained for the study (Table 5.13). Each earthquake record was scaled such that its acceleration spectrum matched the 5% damped 2% in 50 year probability of exceedance spectrum from the 2005 NBCC for the Vancouver region (Table 5.14 & Figure 5.12). The scaling was made by visual inspection (Figure 5.12) for each earthquake listed. The peak ground acceleration as provided by the 2005 NBCC is 0.46 g for Vancouver. The scaled ground motion records resulted in peak ground accelerations in the range of 0.26-0.53 g. All the scaled earthquake spectra are shown in Appendix 'E'.

Table 5.13: Ground motion records used in Ruaumoko analysis

No.	**Event**	**Magn.**	**Station**	**deg**	**PGA (g)**	**SF**	**Time Step (sec)**
S01	Simulated (Trial #1)	M_w 6.5		-	0.53	1.0	0.01
S02	Simulated (Trial #4)	M_w 6.5		-	0.43	1.1	0.01
S03	Simulated (Trial #1)	M_w 7.2		-	0.28	1.1	0.01
S04	Simulated (Trial #2)	M_w 7.2		-	0.31	1.2	0.01
S05	Apr. 24, 1984 Morgan Hill	M_w 6.1	San Ysidro, Gilroy #6	90	0.26	0.9	0.005
S06	Jan. 17, 1994 Northridge	M_w 6.7	Castaic, Old Ridge Rt.	90	0.34	0.6	0.02
S07	Jan. 17, 1994 Northridge	M_w 6.7	Castaic, Old Ridge Rt.	0	0.26	0.5	0.02
S08	Oct. 18, 1989 Loma Prieta	M_w 7.0	Stanford Univ.	0	0.29	1.0	0.005
S09	Oct. 18, 1989 Loma Prieta	M_w 7.0	Presidio	90	0.26	1.3	0.02
S10	Apr. 13, 1949 West.Wash.	M_w 7.1	Olympia, Test Lab	86	0.42	1.5	0.02

Table 5.14: 5% damped spectral response acceleration values (Vancouver, BC)

T(sec)	$S_a(T)$
0.2	0.94
0.5	0.64
1.0	0.33
2.0	0.17
4.0	0.085

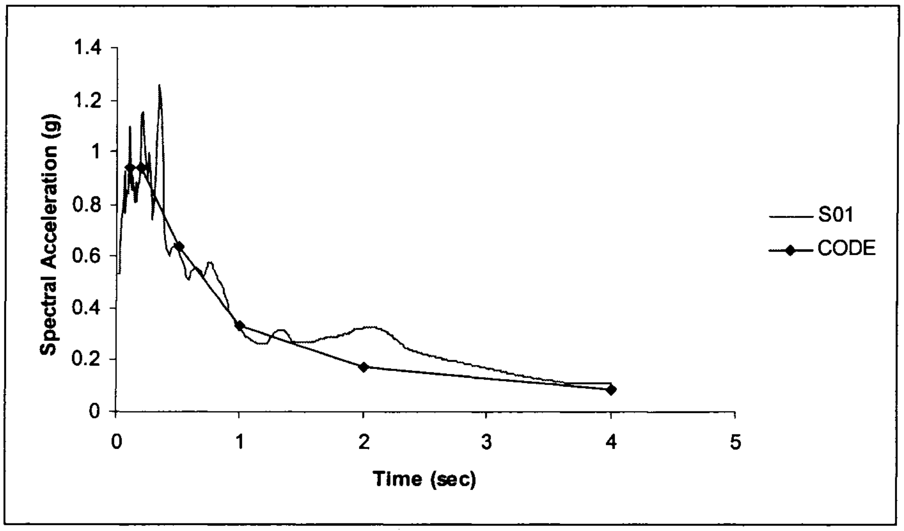


Figure 5.12: Vancouver 5% damped spectral response acceleration vs. scaled ground motion S01

5.6.2. House Model

A diagonally braced structural model was created to replace the OSB sheathed shear wall (Figure 5.13). The single-storey model was first used in a pushover analysis with which the definition of model parameters and the use of a brace frame for analysis were verified (Section 5.6.2.1). The two sets of diagonal braces (elements 4, 5, 12 & 13) were defined as Stewart elements, while simple linear elastic beam (elements 2, 7 & 10), column (elements 1, 3, 9 & 11) and floor connector elements (elements 6 & 8) were used elsewhere (Figure 5.14). Lumped weights were applied to the model at nodes 5 & 8 (1st storey) and at nodes 6 & 7 (roof). The weights considered were equal to 100% of the dead load on the 1st floor and equal to 100% of the dead load plus 25% of the snow load on the roof over the tributary area to the shear wall, i.e. one quarter of the floor or roof area (Tables 5.5 & 5.8). It is important to note that although the term "weight" is used herein, Ruaumoko converts the applied weights into masses automatically.

An inelastic Newmark constant average acceleration (non-linear dynamic time-history) analysis was chosen to test the model with a time step interval of 0.005 sec or 0.01 sec depending on the ground motion data file (Table 5.13). A Rayleigh damping of 5% was assumed for the 1st and 2nd mode of vibration of the structure.

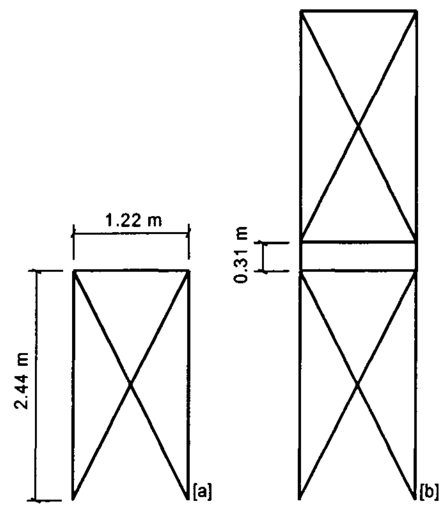


Figure 5.13: Ruaumoko models: [a] One-storey shear wall
[b] Two-storey shear wall with floor

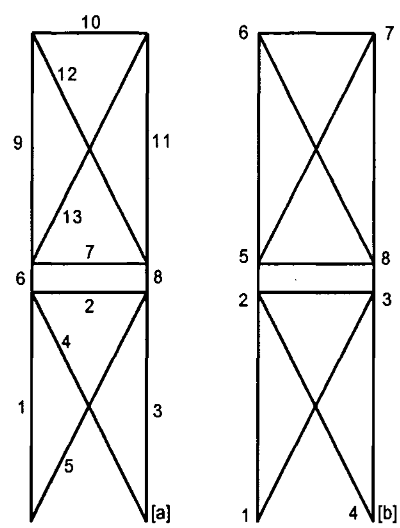


Figure 5.14: Ruaumoko models: Two-storey shear wall:
[a] Element numbers, and [b] Node numbers

Each element was defined as a spring with appropriate values of stiffness and strength. Since the braces were responsible for providing the hysteretic behaviour of the entire wall as it was tested, they were given the values of the hysteresis models derived in Section 5.2 (Table 5.1). All the hysteretic parameters of the wall were adjusted to account for the oblique position of the braces, since these parameters represented the horizontal resistance of the wall. In addition, the hysteretic parameters were divided by two because the braces worked equally as well in tension as in compression (Table 5.15). The beam and column elements were given a stiffness of 1×10^6 kN/m to ensure that the forces and the deformations of the wall passed through the fuse elements; i.e. the brace elements.

Table 5.15: Summary of model properties

Braces
Spring member – Stewart hysteresis

	Unit	*House*	*Commercial building*
Length	(m)	2.728	2.999
FU	(kN)	15.92	33.80
FX	(kN)	10.61	22.78
FI	(kN)	1.29	3.08
KX	(kN/m)	4700	7915
RF		0.15	0.19
PTRI		0	0
PUNL		1.75	1.45
GAP+	(m)	0	0
GAP-	(m)	0	0
BETA		1.09	1.09
ALPHA		0.52	0.45

Columns
Spring member – Elastic

	Unit	*House*	*Commercial building*
Length	(m)	2.44	2.74
KX	(kN/m)	1×10^6	1×10^6

Beams
Spring member – Elastic

	Unit	*House*	*Commercial building*
Length	(m)	1.22	1.22
KX	(kN/m)	1×10^6	1×10^6

Floor connectors
Spring member – Elastic

	Unit	*House*	*Commercial building*
Length	(m)	0.31	0.15
KX	(kN/m)	254559	254559

In the case of the two-storey model, a gap of 0.31 m between the upper and lower walls was left to represent a floor (Figure 5.13[b]). The two vertical linear elastic elements (elements 6 & 8) joining the walls together through the floor gap were given the axial stiffness of the holddown threaded rods used in testing. These two elements were free to deform in the vertical axis only because the floor, in this case through its thickness, was considered to be a rigid diaphragm. The nodes at the end of these elements (nodes 2 & 5, and 3 & 8) were constrained such that their horizontal movement was equal, i.e. the floor gap of the model did not distort in shear but could move horizontally or vertically as a rigid body. All connections between the elements were pins allowing for rotation as well as horizontal and vertical displacements, except at the lower ends (nodes 1 & 4) where the pins were only allowed to rotate.

The lumped weights used in the models were taken from the previous calculations described in Section 5.4 (Table 5.5). For practical reasons, since dynamic tests of two-storey shear walls will hopefully be carried out on a shake table in future studies, the weights given in Table 5.5 were rounded to the nearest kilo Newton, thus keeping the total weight of the structure as close as possible to the one calculated. The floor weight in the model was taken as 33 kN, which was placed at the bottom corners of the second storey wall segment (nodes 5 & 8) (16.5 kN / node), while the roof weight was taken as 22 kN, which was located at the extremities of the wall top (nodes 6 & 7) (11 kN / node). The total weight applied on the model is therefore 55 kN, which is approximately equal to the total of 55.1 kN listed in Table 5.4.

In all, four column, three beam, two link and four brace elements were used to model the two-storey shear wall. All node and element coordinates / parameters were included in a text file (Appendix 'F') created for later use in Ruaumoko. All the data in the input text files were expressed in kilo-Newtons (kN), meters (m), seconds (sec.) or a combination of the three. The other information that varied for each input file were parameters specific to the ground motion record, such as the length (time) and the scale (increment of time) of the earthquake record, as well as the name of the record to be used for the analysis.

5.6.2.1. Pushover analysis of single-storey model

It was decided to first build a one-storey version of a typical 152/305 mm (6"/12") screw schedule wall (Figure 5.13[a]) to evaluate the model in terms of its ability to replicate the monotonic behaviour of the tested shear walls. A push-over evaluation was carried out which resulted in a shear resistance vs. deflection diagram identical to the calibrated Stewart hysteresis (Figure 5.15). This indicates that the cross-braced model was able to replicate the resistance vs. deflection behaviour of the calibrated Stewart hysteretic elements (Section 5.3), and hence, represent the actual performance of the shear walls that were tested.

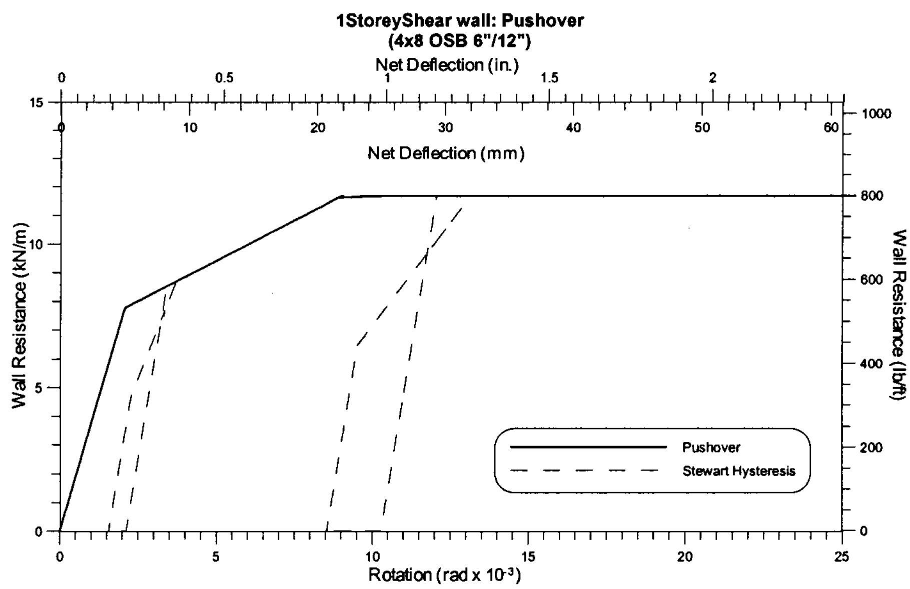


Figure 5.15: Pushover hysteresis vs. Stewart hysteresis

5.6.3. Commercial Building Model

The three-storey shear wall model of the commercial building was essentially the same as that used for the house. However, it was not possible to simply add one more floor and storey (shear wall) to the two-storey model because of the type of floor that was chosen in the preliminary design in Section 5.4.2. The Hambro® system is typically composed of a 300 mm (12") deep truss which is placed underneath a 75 mm (3") thick concrete slab. It was assumed that a W150 x 22 beam is placed above each shear wall to support the concrete slab. Furthermore, the slab was positioned on top of the bottom flange of the W-section, which caused the truss members to hang 300 mm (12") below the top of the shear wall. This will decrease the clear height of the storey to 2.14 m (7'), which is not considered sufficient. Hence, the wall height was increased by 300 mm (12") resulting in a 2.74 m (9') high storey (Figure 2). The floor depth was taken as the depth of the W-beam that is 150 mm (6").

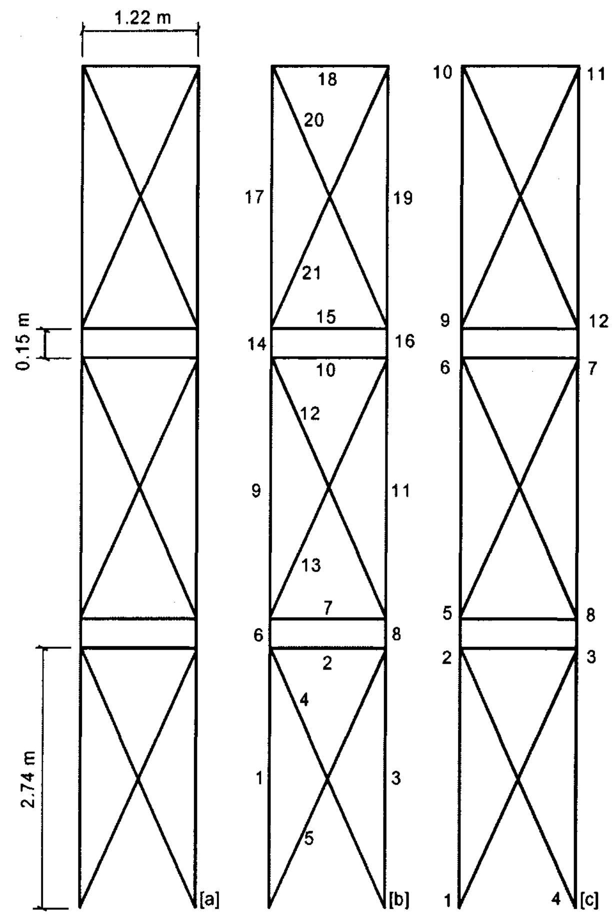


Figure 5.16: Ruaumoko models: Three-storey shear wall:
[a] Dimensions, [b] Element numbers, and [c] Node numbers

The various properties of the shear walls obtained from the 1.22 m × 2.44 m (4' × 8') test results were assumed to apply for the 2.74 m (9') tall walls as well. It was also assumed that for this type of construction the same threaded rods were used to connect one storey to the next as described in the previous two-storey model. Therefore, the stiffness used for the beams, columns and link elements were the same as previously discussed in

Section 5.6.2. The properties of the braces were modified since the screw schedule was 75/305 mm (3"/12") and their slope was changed. All the other elements were given the same properties as those of the previous model (Table 5.15).

Based on the calculations presented in Section 5.4.2, the weights on the walls were as follows: 42 kN (21 kN / node) on the first storey and second storey, and 16 kN (8 kN / node) on the roof. Once again the weights on all storeys were distributed to the two corners of the modeled walls. However, contrary to the two-storey model, the weights were set directly on top of the wall (nodes 2 & 3, and 6 & 7) instead of at the bottom of the upper storey wall (nodes 5 & 8, and 9 & 12). These nodes were chosen because the weight originating from the slab was applied at the bottom of the W-section beam, which is located close to the top of the wall. The weight on the roof was positioned at nodes 10 & 11. The total weight applied to the model was 100 kN, which is approximately equal to the total of 99.4 kN listed in Table 5.8. The weights considered were equal to 100% of the dead load on the 1st and 2nd floors and equal to 100% of the dead load plus 25% of the snow load on the roof over the area tributary to the shear wall, i.e. one quarter of the floor or roof area (Table 5.8).

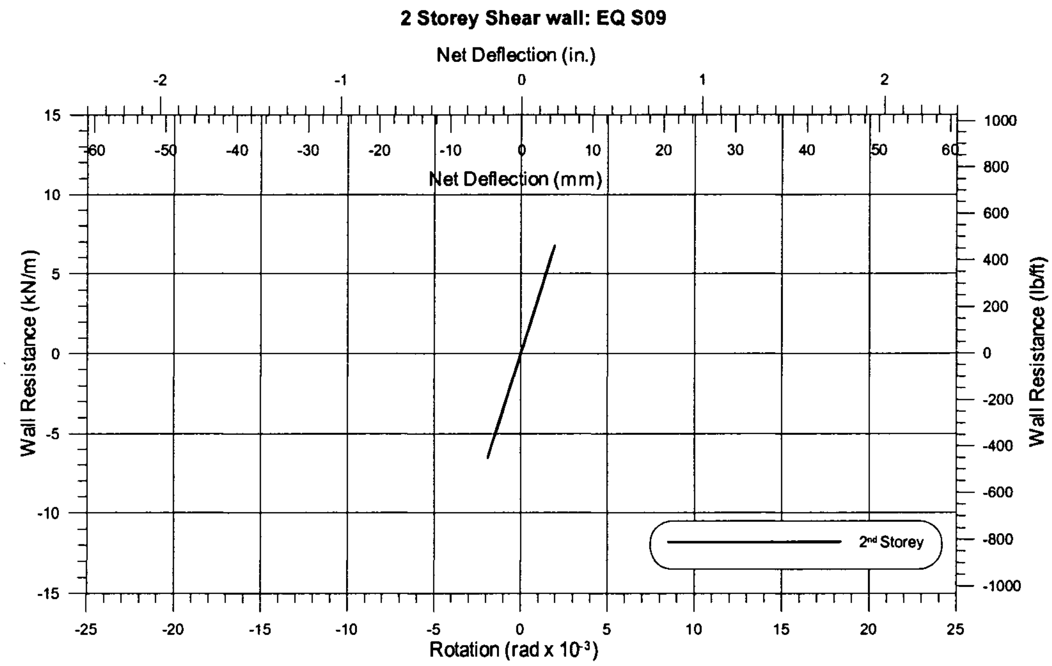
In all, six column (elements 1, 3, 9, 11, 17 & 19), five beam (elements 2, 7, 10, 15 & 18), four link (elements 6, 8, 14 & 16) and six brace elements (elements 4, 5, 12, 13, 20 & 21) for a total of twenty-one elements were used to model the three-storey shear wall for the commercial building (Figure 5.16[b]). All twelve nodes were defined as pins, with only the two lower nodes (nodes 1 & 4) not able to move laterally or vertically (Figure 5.16[c]). The properties of the link elements were defined as in the house model. The three-storey shear wall input text files can be found in Appendix 'F'.

5.7. ANALYSIS RESULTS

The Dynaplot program *(Carr, 2000)* was used with the output files from the non-linear time history dynamic analyses to obtain the displacement and force time histories for all storeys and for at least one brace per storey. It was not necessary to create a file for each brace in a particular storey since the two braces provided the same contribution to the SFRS, albeit with opposite signs, resulting in the same axial load vs. displacement behaviour. From the geometry of the model, the axial forces in the brace were converted to the shear forces in the direction of loading. Similarly, the nodal displacements were converted to rotations to compare with the results obtained from the experimental tests described in Chapter 3.

5.7.1. House Model Analysis Results

Using the wall resistance obtained from the axial forces in the braces and the nodal displacements, wall resistance vs. rotation curves, as well as time history plots, were created for each storey in the model and for each of the ten ground motion records. Figure 5.17 shows a typical hysteresis for both the upper and lower storeys of the shear wall model. The results for the ground motion S09 represented the worst case scenario, based on maximum wall deflection, obtained for the house model given the suite of ground motions that were used. As can be seen, the wall segments did not experience a rotation that exceeded the limit suggested by the full-scale shear wall tests (Table 5.2). Figure 5.18 shows the wall resistance and the rotation with respect to time. The wall rotation remained within the deformation limit for the entire time history, however a permanent post earthquake deformation resulted. Resistance vs. rotation and time history plots for the remaining ground motion records can be found in Appendix 'G'.



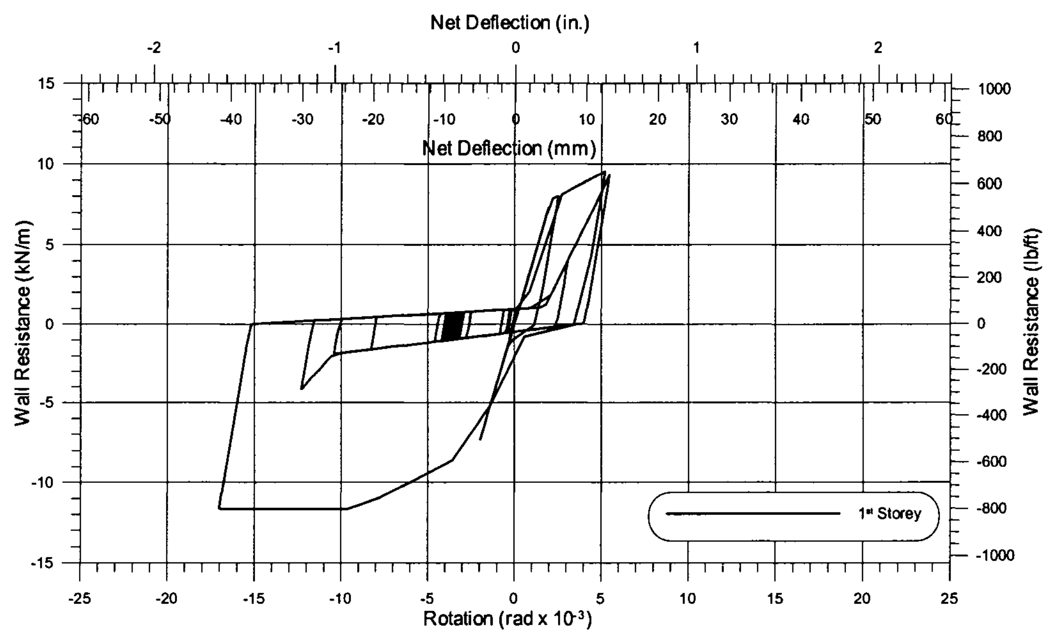


Figure 5.17: Resistance vs. displacement hystereses of the two-storey shear wall under ground motion S09

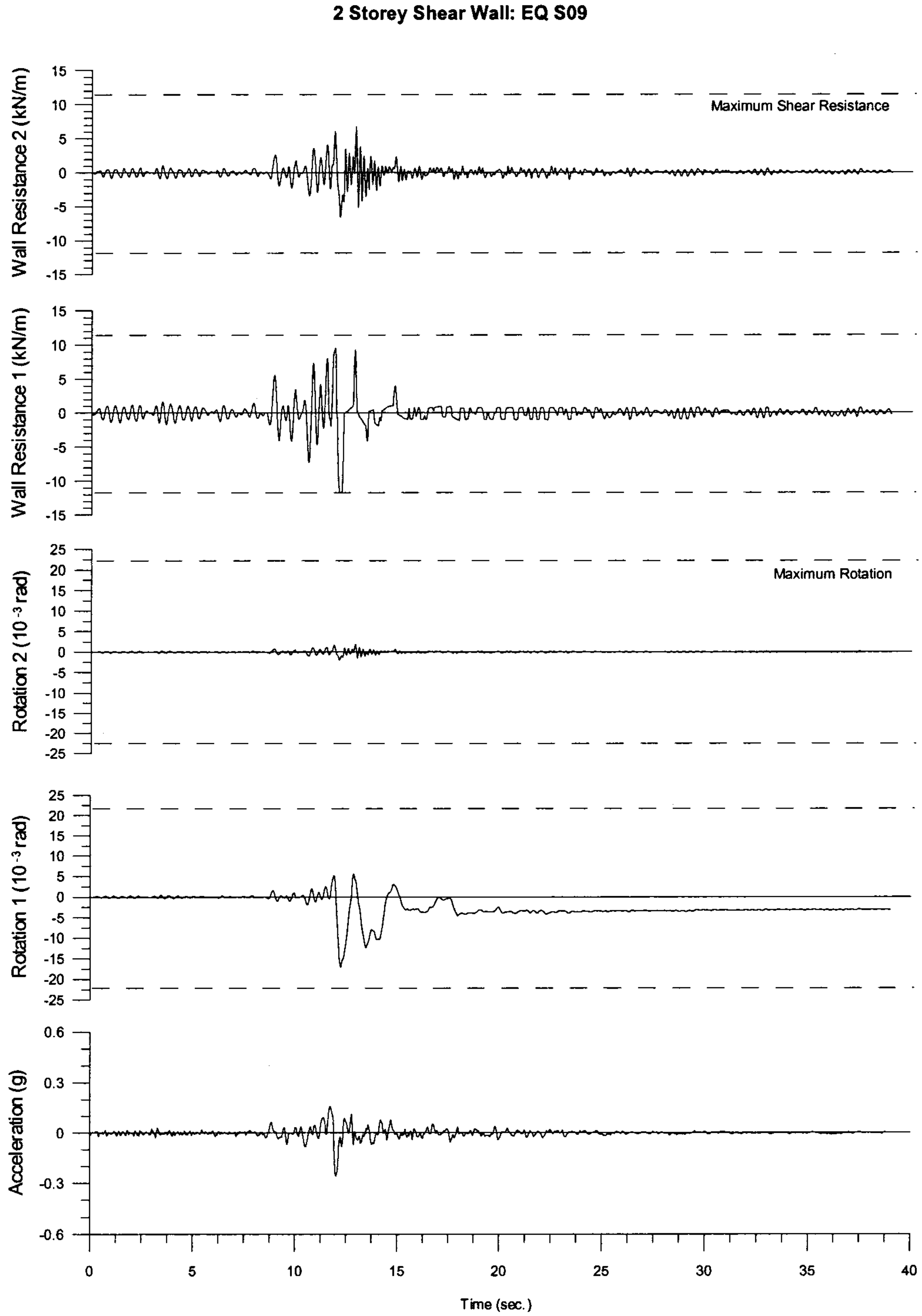


Figure 5.18: Time history plots of the two-storey shear wall under ground motion S09

For each of the analyses that were run the maximum rotation of the shear wall model was recorded and tabulated (Table 5.16). In all cases the 1st storey exhibited the largest deformations, with the majority of results being less than 15×10^{-3} rad. The shear rotation measured during the simulated ground motion S09 was noticeably larger, but remained below the test based limit, as indicated earlier.

Table 5.16: Maximum rotations of shear wall model for two-storey building

Ground Motion	**Max rotation (10^{-3} rad)**	
	1st storey	**2nd storey**
S01	11.3	2.42
S02	15.3	2.50
S03	11.9	**2.79**
S04	11.3	2.09
S05	12.4	2.22
S06	11.2	2.56
S07	12.7	2.65
S08	12.9	2.56
S09	**17.0**	1.95
S10	12.3	2.33

The values presented in Table 5.17 summarize the largest shear resistance and rotation for both the 1st and 2nd storeys, independent of the ground motion record. The limiting parameters are also shown for the wall configuration chosen in design, that is; 9 mm OSB sheathing with a screw spacing of 152/305 mm (6"/12").

Table 5.17: Summary of dynamic analysis results for the two-storey model

Two-Storey Model 9 mm OSB sheathing Screw Schedule: 152/305 mm	**Maximum Shear Resistance (Ruaumoko) (kN/m)**	**Maximum Rotation (Ruaumoko) (10^{-3} rad)**
1st storey	11.7	17.0
2nd storey	8.08	2.79
Allowable Values (experimental data)	**11.7**	**22.3**

5.7.2. Commercial Building Model Analysis Results

As for the two-storey model, wall resistance vs. rotation curves and time history plots, were created for each storey of the commercial building model and for each earthquake ground motion record. Typical hystereses are shown in Figure 5.19 for all three storeys of the model when subjected to ground motion S07, which represented the worst case scenario in terms of the storey shear rotation. Nevertheless, none of the shear wall segments experienced a rotation that exceeded the limit suggested by the full-scale shear wall tests (Table 5.2). The wall resistance and the rotation time history plots for ground motion S07 are shown in Figure 5.20. As can be seen, the wall rotation remained within the deformation limit for the entire time history, however a permanent post earthquake deformation resulted in the ground storey, as found for the house model. Resistance vs. rotation and time history plots for the remaining ground motion records can be found in Appendix 'G'.

The values presented in Table 5.19 summarize the largest shear resistance and rotation for the 1st, 2nd and 3rd storeys, inclusive of all earthquake records used. The limiting strength and stiffness parameters are also shown for the wall configuration chosen in design, that is; 9 mm OSB sheathing with a screw spacing of 75/305 mm (3"/12"). However, the most dominant effect was on the first storey as expected and as cited previously. If a different wall configuration had been chosen for the upper two storeys, i.e. with less strength and stiffness, the findings may have been different.

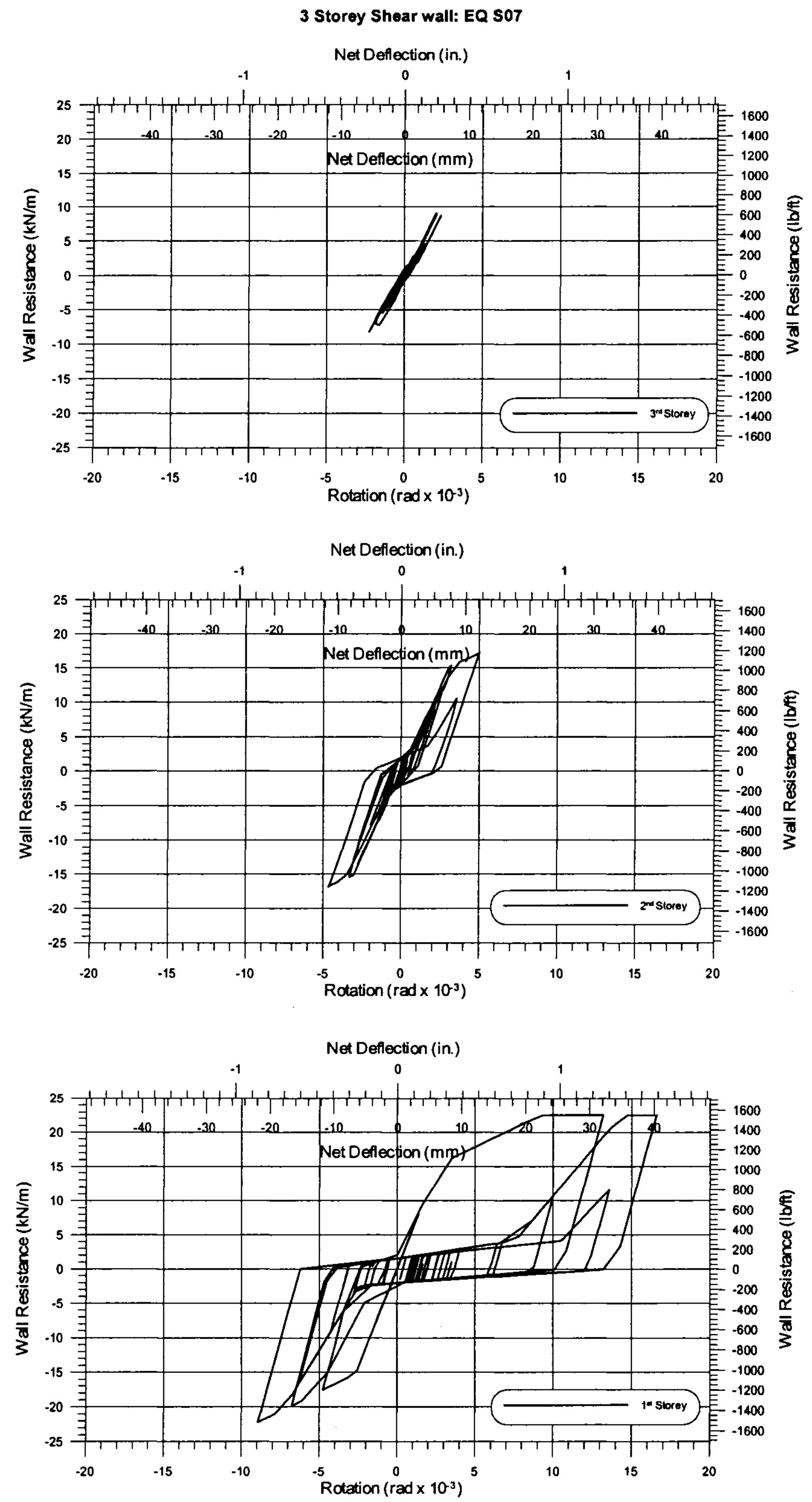


Figure 5.19: Resistance vs. displacement hystereses of the three-storey shear wall under ground motion S07

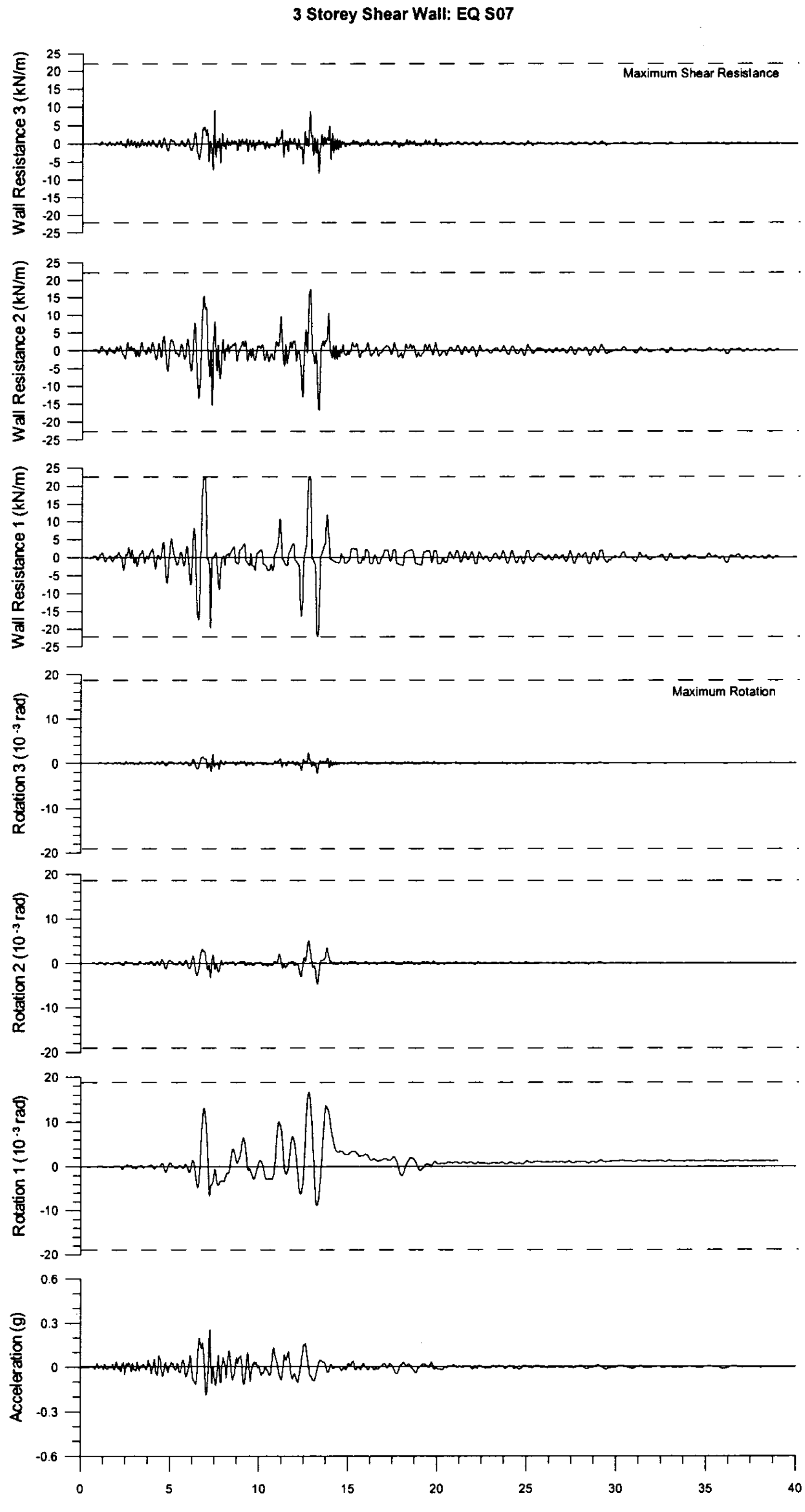


Figure 5.20: Time history plots of the three-storey shear wall under ground motion S07

Table 5.18: Maximum rotations shear wall model for three-storey building

Ground Motion	**Max rotation (10^{-3} rad)**		
	1st storey	**2nd storey**	**3rd storey**
S01	12.6	4.92	2.29
S02	10.5	4.99	2.26
S03	12.6	4.31	1.99
S04	6.92	3.65	2.10
S05	12.5	3.43	1.99
S06	10.6	3.88	2.22
S07	**16.6**	**5.05**	**2.35**
S08	11.9	4.81	2.24
S09	16.4	3.34	1.87
S10	9.48	4.62	2.20

Table 5.19: Summary of results for the three-storey model

Three-Storey Model 9 mm OSB sheathing Screw Schedule: 75/305 mm	**Maximum Shear Resistance (Ruaumoko) (kN/m)**	**Maximum Rotation (Ruaumoko) (10^{-3} rad)**
1st storey	22.5	16.6
2nd storey	17.3	5.05
3rd storey	9.02	2.35
Allowable Values (experimental data)	**22.5**	**19.2**

CHAPTER 6 CONCLUSIONS & RECOMMENDATIONS

6.1 CONCLUSIONS

In the summer of 2004 the testing of an additional eighteen light gauge steel frame / wood panel shear walls (three configurations) sheathed with 9 mm OSB panels was completed under the scope of the shear wall research program at McGill University. These tests are an addition to the database of sixteen wall configurations created by Boudreault *(2005)*, Branston *(2004)* and Chen *(2004)* in the previous year, and the three configurations developed by Rokas *(2005)* in the summer of 2004. The construction method and testing procedures used herein matched those of the tests carried out in 2003 *(Branston et al., 2004)*. The data obtained from the tests were used in combination with the equivalent energy elastic-plastic (EEEP) analysis approach to derive the design values for the walls, including: shear stiffness, shear strength, resistance factor, maximum allowable rotation, ductility and force modification factors. A resistance factor, $\phi = 0.7$, and force modification factors, $R_d = 2.5$ and $R_o = 1.8$, are recommended for design based on the test results.

Following Boudreault's work, the parameters for the Stewart degrading hysteretic element *(Stewart, 1987)* were obtained for the three new wall configurations. A total of nineteen calibrated steel frame / wood panel shear wall hysteresis models have now been developed, whose parameters can be used to model any type of structure, residential or commercial, for dynamic analyses.

Using Ruaumoko *(Carr, 2000)* a single-storey shear wall / braced frame model, which incorporated the Stewart hysteretic elements as lateral braces, was first developed. This model was relied on to validate the applicability of the model with respect to its ability to replicate the load vs. resistance behaviour of the tested shear walls. A pushover analysis was carried out using the single-storey shear wall model, which showed that the model was accurate, and hence, could be used for the dynamic analyses.

Following this, a preliminary analytical study involving non-linear time history dynamic analyses of representative wall models was completed. Shear wall models for a two-storey house and a three-storey commercial building were created. The design of the SFRS of these buildings, located in Vancouver BC, was first carried out following the 2005 NBCC seismic loading provisions and the recommended design parameters; R_d and R_o as well as design strengths and stiffnesses, obtained from this study. This limited study showed that the two and three-storey shear walls were adequate under the ten scaled ground motion records chosen in terms of not exceeding the deformation limits obtained from the physical test results. This finding confirms that the recommended design method for light gauge steel frame / wood panel shear walls, including force modification factors, is valid on a preliminary basis.

6.2 RECOMMENDATIONS

This research forms an introductory analytical study of light gauge steel frame /wood panel shear walls. Only two simple, symmetric buildings were modeled under ground motion records scaled for one region of the country. The need for more complex models situated at various high risk earthquake regions across Canada is necessary. There is also an important need for background knowledge on the behaviour of the inter-storey connection and how it should be modeled. Moreover, the scope of modeling should include different wall lengths and configurations, as well as the non-structural components of buildings, such as gypsum board, veneer, insulation layer, brick, etc. As for the loads used in the models, partial live load should be accounted for because some live loads can be considered permanent, e.g. partition walls. This will increase the seismic weight acting on the shear walls, and therefore, may increase the displacement demand. It is important to take all these considerations into account in future studies to increase the level of understanding concerning the behaviour of light gauge steel / wood panel shear walls.

Further study is also warranted with respect to the dynamic testing of shear walls. It is recommended that multi-storey shear wall systems be subjected to ground motion records using a shake table setup. The results of the present study, namely expected lateral loads and deformations, can be used in the design of the testing apparatus.

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APPENDIX 'A'
REVERSED CYCLIC TEST PROTOCOLS

Table A.1: CUREE cyclic protocol for tests 42-A,B,C

$\Delta=0.6*\Delta_m$	33.34	Screw Pattern:	6"/12"
		Sheathing:	OSB

	Target (corr.)	Actuator Input	
Displ.	mm	mm	No. Of cycles
0.050 D	1.667	1.988	6
0.075 D	2.500	2.982	1
0.056 D	1.875	2.236	6
0.100 D	3.334	3.976	1
0.075 D	2.500	2.982	6
0.200 D	6.668	7.951	1
0.150 D	5.001	5.964	3
0.300 D	10.001	11.927	1
0.225 D	7.501	8.945	3
0.400 D	13.335	15.903	1
0.300 D	10.001	11.927	2
0.700 D	23.336	27.830	1
0.525 D	17.502	20.873	2
1.000 D	33.338	39.757	1
0.750 D	25.003	29.818	2
1.500 D	50.006	59.636	1
1.125 D	37.505	44.727	2
2.000 D	66.675	79.515	1
1.500 D	50.006	59.636	2

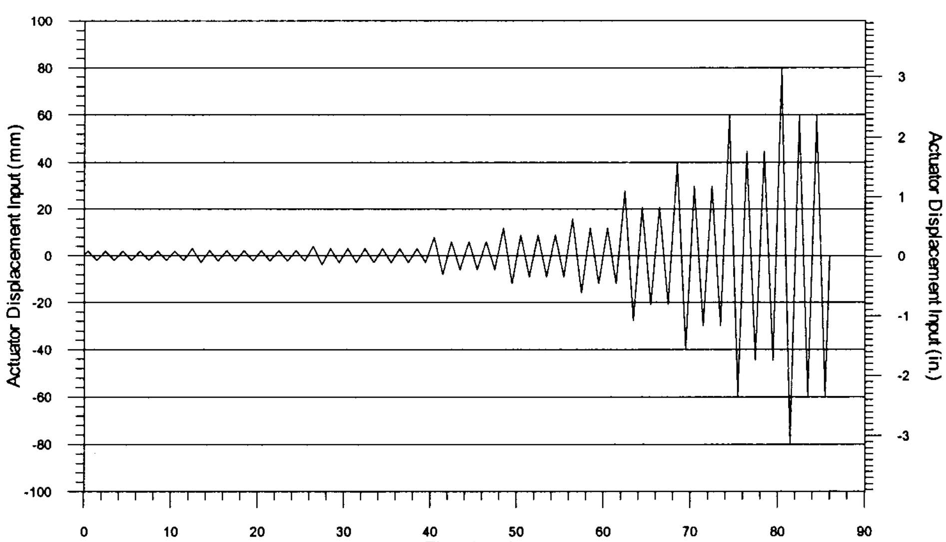


Figure A.1: CUREE cyclic protocol for tests 42-A,B,C

Table A.2: CUREE cyclic protocol for tests 44-A,B,C

$\Delta=0.6*\Delta_m$	30.19	Screw Pattern:	4"/12"
		Sheathing:	OSB

	Target (corr.)	Actuator Input	
Displ.	mm	mm	No. Of cycles
0.050 D	1.510	1.961	6
0.075 D	2.264	2.941	1
0.056 D	1.698	2.206	6
0.100 D	3.019	3.921	1
0.075 D	2.264	2.941	6
0.200 D	6.038	7.843	1
0.150 D	4.529	5.882	3
0.300 D	9.058	11.764	1
0.225 D	6.793	8.823	3
0.400 D	12.077	15.686	1
0.300 D	9.058	11.764	2
0.700 D	21.135	27.450	1
0.525 D	15.851	20.588	2
1.000 D	30.192	39.215	1
0.750 D	22.644	29.411	2
1.500 D	45.288	58.822	1
1.125 D	33.966	44.116	2
2.000 D	60.384	78.429	1
1.500 D	45.288	58.822	2

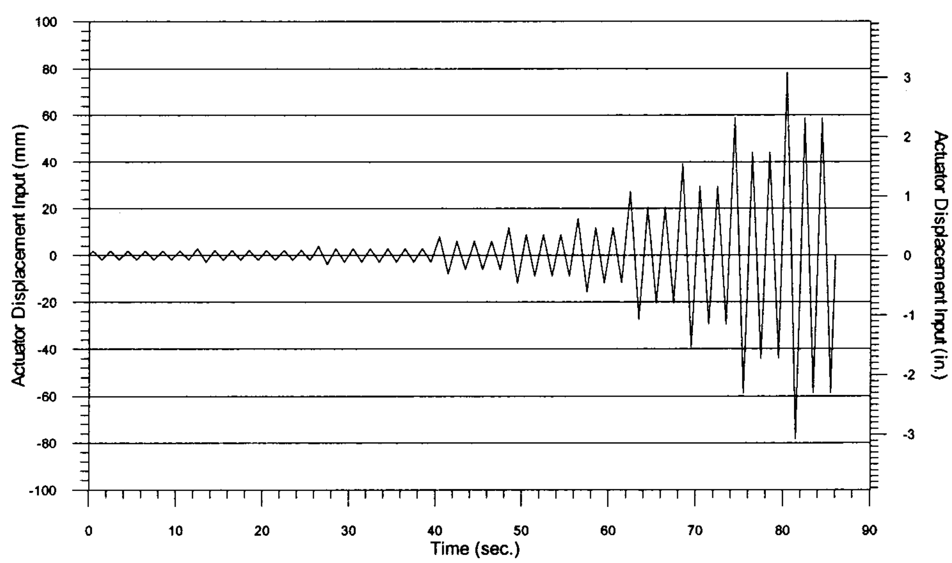


Figure A.2: CUREE cyclic protocol for tests 44-A,B,C

Table A.3: CUREE cyclic protocol for tests 46-A,B,C

$\Delta=0.6*\Delta_m$	29.03	Screw Pattern:	3"/12"
		Sheathing:	OSB

	Target (corr.)	Actuator Input	
Displ.	mm	mm	No. Of cycles
0.050 D	1.451	2.022	6
0.075 D	2.177	3.033	1
0.056 D	1.633	2.275	6
0.100 D	2.903	4.044	1
0.075 D	2.177	3.033	6
0.200 D	5.806	8.088	1
0.150 D	4.354	6.066	3
0.300 D	8.709	12.132	1
0.225 D	6.531	9.099	3
0.400 D	11.611	16.176	1
0.300 D	8.709	12.132	2
0.700 D	20.320	28.307	1
0.525 D	15.240	21.230	2
1.000 D	29.029	40.439	1
0.750 D	21.771	30.329	2
1.500 D	43.543	60.658	1
1.125 D	32.657	45.494	2
2.000 D	58.057	80.878	1
1.500 D	43.543	60.658	2

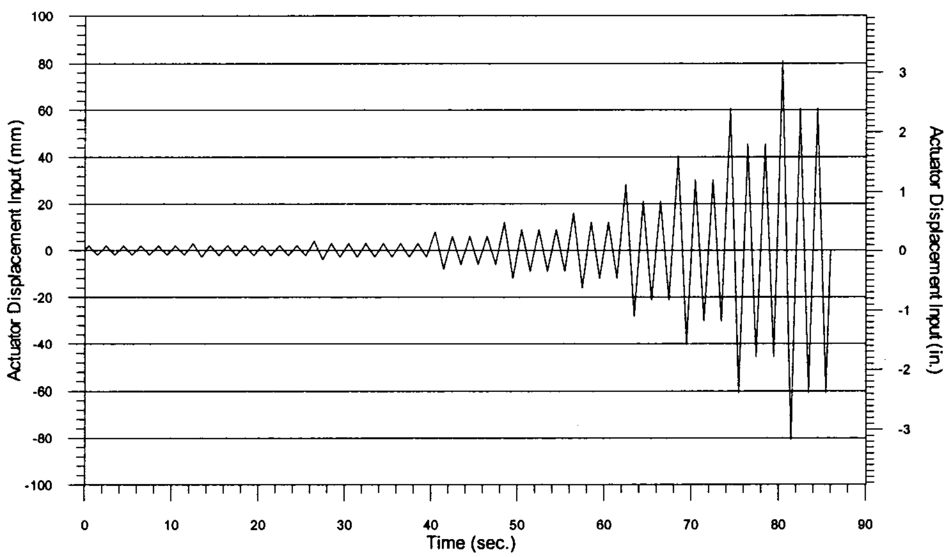


Figure A.3: CUREE cyclic protocol for tests 46-A,B,C

APPENDIX 'B'
SHEAR WALL TEST DATA

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 41A

RESEARCHER: Caroline Blais **ASSISTANTS:** A. Frattini

DATE: 15-Jun-04 **TIME:** 13:30

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)
- [ ] Plywood (CSA 0151M) CSP 12.5mm (1/2")
- [ ] Plywood (CSA 0151M) CSP 9mm (3/8")
- [X] OSB (CSA 0325) 9mm (3/8") Grant Forest Products
- [ ] Other MFR:

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.9 gauge 1.0" self-piercing Bugle head (HD = near hold-down (1 screw 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:
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip Thickness: 0.043" (1.09 mm) 33ksi (230 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 [X] T=0.043" (1.09 mm) 33ksi (230 Mpa)
Flange: 1-1/4" inches [ ] 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
- [ ] Cyclic

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] Sheathing at Corners of Panels
- [X] Panel She **TOTAL:** [ ]

MOISTURE CONTENT OF SHEATHING: Moisture Meter
Wood: Grant Forest Products Nth Sth
Temp.: C
AVG: #DIV/0!

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

	North	South
Ww=	24.78	24.38
Wd=	23.71	23.32
m.c.=	4.51	4.55

AVG m.c. 4.53

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

COMMENTS:
-Shear anchors torqued for 10 s with impact wrench
-North hold down anchor 3/4 turn from finger tight, south 1/2 turn
(load cells used on both hold-downs)
-Ambient temperature 20 C
-Double chord studs used
-Square plate washers (2.5"x2.5") + circular ones in both corners of top track
-Initial load set to zero at beginning of test, displacement -0.003 mm

Test 41A
(4x8 OSB 6"/12")

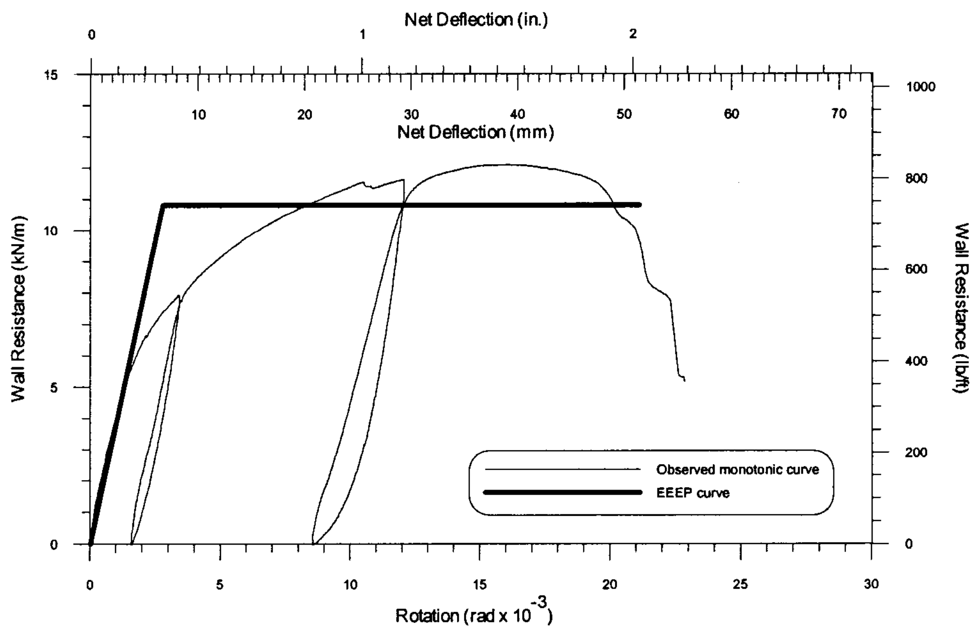


Figure B.1: Monotonic and EEEP curves for test 41-A

Table B.1: Parameters for monotonic test 41-A

	Parameters	Units
F_u	14.75	kN
$F_{0.8u}$	11.80	kN
$F_{0.4u}$	5.90	kN
F_y	13.17	kN
K_e	1.95	kN/mm
Ductility (μ)	7.60	-
$\Delta_{net,y}$	6.76	mm
$\Delta_{net,u}$	39.19	mm
$\Delta_{net,0.8u}$	51.43	mm
$\Delta_{net,0.4u}$	3.03	mm
$Area_{Backbone}$	632.72	J
$Area_{EEEP}$	632.72	J
Check	**OK**	
R_d	**3.77**	-
S_y	**10.80**	kN/m

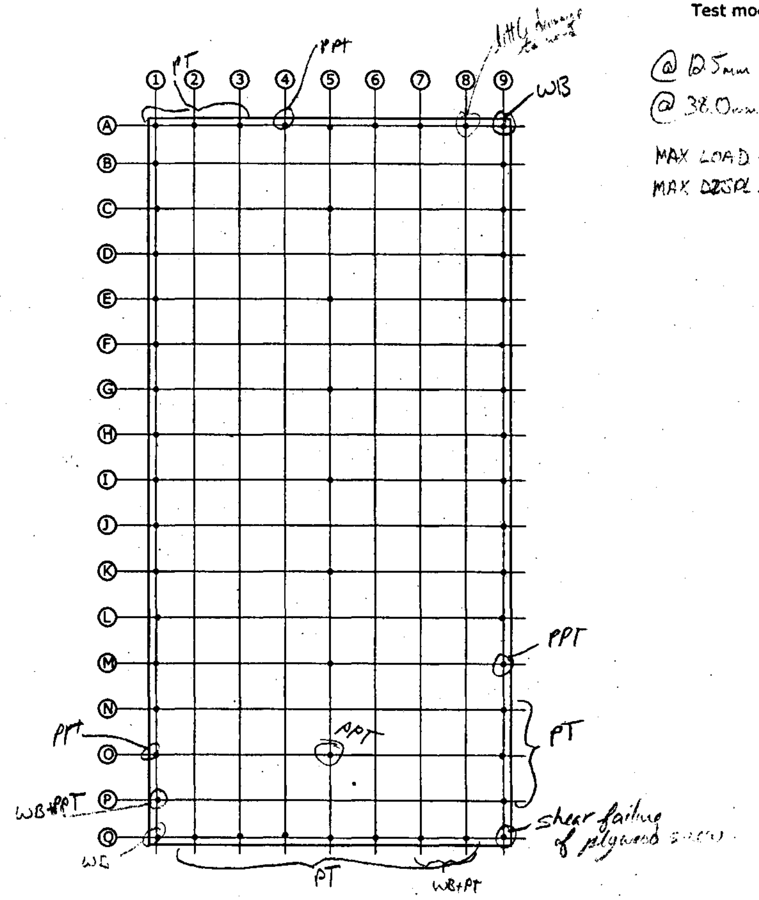
McGill

Light Gauge Steel Frame / Wood Panel Shear Wall Testing

Test name : 41A
Date tested : June 15, 2005
Wall Size : 4' x 8'
Screw pattern : 6" / 12"
Edge Distance : 1/2"
Test mode : ☐ Cyclic ☒ Monotonic

@ 12.5mm ⇒ 9745 N
@ 38.0mm ⇒ 14259 N

MAX LOAD → 14823 N
MAX DISPL. ⇒ 67.23mm



bottom left: little buckling of track near holddown

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)

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 41B

RESEARCHER: Caroline Blais **ASSISTANTS:** A. Frattini

DATE: 15-Jun-04 **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")
- [ ] Plywood (CSA 0151M) CSP 9mm (3/8")
- [X] OSB (CSA 0325) 9mm (3/8") — Grant Forest Products
- [ ] Other — MFR:

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.9 gauge 1.0" self-piercing Bugle head (HD = near hold-down (1 screw 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:
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip Thickness: 0.043" (1.09 mm) 33ksi (230 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] T=0.043" (1.09 mm) 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
- [ ] Cyclic

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] Panel She
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral
- [X] East Frame Brace
- [X] West Frame Brace
- [X] Sheathing at Corners of Panels

TOTAL: [ ]

MOISTURE CONTENT OF SHEATHING: Moisture Meter
Wood:
Temp.: C
Nth / Sth
AVG: #DIV/0!

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

	North	South
Ww=	25.25	24.64
Wd=	24.09	23.55
m.c.=	4.82	4.63

AVG m.c. 4.72

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

COMMENTS:
-Shear anchors torqued for 10 s with impact wrench
-North hold down anchor 2/3 turn from finger tight, south 3/4 turn
(load cells used on both hold-downs)
-Ambient temperature 20 C
-Double chord studs used
-Square plate washers (2.5"x2.5") + circular ones in both corners of top track
-Initial load set to zero at beginning of test, displacement -0.102 mm

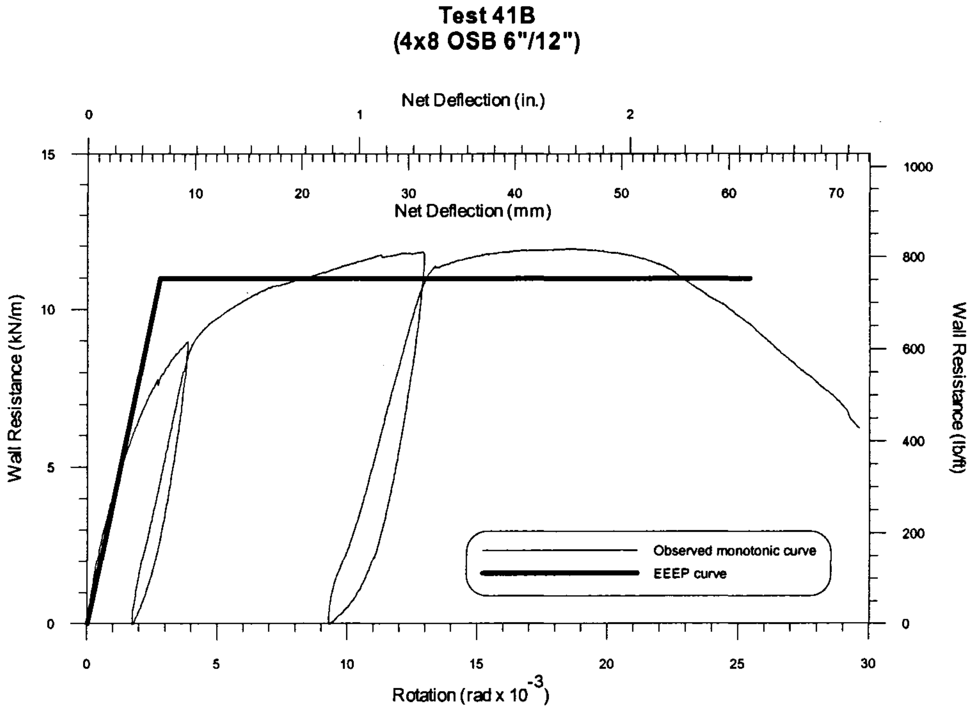


Figure B.2: Monotonic and EEEP curves for test 41-B

Table B.2: Parameters for monotonic test 41-B

	Parameters	Units
F_u	14.54	kN
$F_{0.8u}$	11.63	kN
$F_{0.4u}$	5.81	kN
F_y	13.37	kN
K_e	1.98	kN/mm
Ductility (μ)	9.19	-
$\Delta_{net,y}$	6.74	mm
$\Delta_{net,u}$	45.08	mm
$\Delta_{net,0.8u}$	61.97	mm
$\Delta_{net,0.4u}$	2.93	mm
$Area_{Backbone}$	783.70	J
$Area_{EEEP}$	783.70	J
Check	**OK**	
R_d	**4.17**	-
S_y	**10.97**	kN/m

McGill

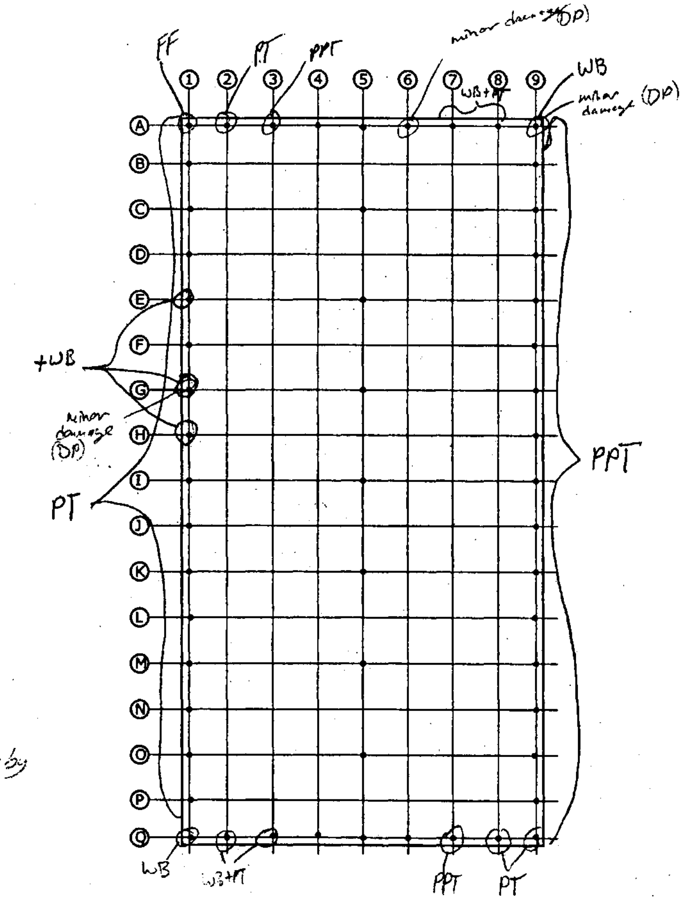
Light Gauge Steel Frame / Wood Panel Shear Wall Testing

Test name : 41B
Date tested : June 15, 2004
Wall Size : 4' x 8'
Screw pattern : 6" / 12"
Edge Distance : 1/2"
Test mode : ☐ Cyclic ☒ Monotonic

@ 12.5mm ⇒ 10022N
@ 38.0mm ⇒ 14488N
MAX LOAD ⇒ 14596N
MAX DISPL ⇒

slipping @ about 9mm

Did not break drastically.
Stopped taking reading @ ≈85mm



bottom left corner
buckling of track nearby
hold down

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)

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 41C

RESEARCHER: Caroline Blais **ASSISTANTS:** A. Frattini

DATE: 16-Jun-04 **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")
- [ ] Plywood (CSA O151M) CSP 9mm (3/8")
- [X] OSB (CSA 0325) 9mm (3/8") — Grant Forest Products
- [ ] Other — MFR:

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.9 gauge 1.0" self-piercing Bugle head (HD = near hold-down (1 screw 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:
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip Thickness: 0.043" (1.09 mm) 33ksi (230 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] T=0.043" (1.09 mm) 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
- [ ] Cyclic

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] Panel She
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral
- [X] East Frame Brace
- [X] West Frame Brace
- [X] Sheathing at Corners of Panels

TOTAL: [ ]

MOISTURE CONTENT OF SHEATHING: Moisture Meter
Wood:
Temp.: C
Nth Sth
AVG: #DIV/0!

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

	North	South
Ww=	27.98	26.20
Wd=	26.85	25.07
m.c.=	4.21	4.51

AVG m.c. 4.36

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

COMMENTS:
-Shear anchors torqued for 10 s with impact wrench
-North hold down anchor 1/2 turn from finger tight, south 1/2 turn
(load cells used on both hold-downs)
-Ambient temperature 18 C
-Double chord studs used
-Square plate washers (2.5"x2.5") + circular ones in both corners of top track
-Initial load set to zero at beginning of test, displacement -0.353 mm

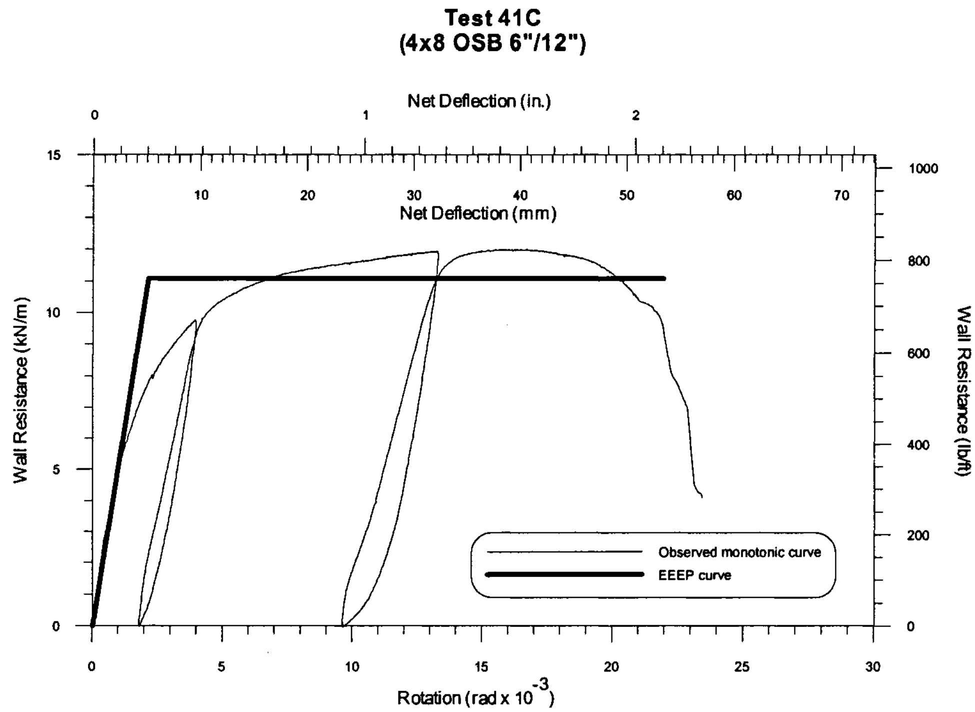


Figure B.3: Monotonic and EEEP curves for test 41-C

Table B.3: Parameters for monotonic test 41-C

	Parameters	Units
F_u	14.63	kN
$F_{0.8u}$	11.70	kN
$F_{0.4u}$	5.85	kN
F_y	13.51	kN
K_e	2.62	kN/mm
Ductility (μ)	10.37	-
$\Delta_{net,y}$	5.16	mm
$\Delta_{net,u}$	40.64	mm
$\Delta_{net,0.8u}$	53.47	mm
$\Delta_{net,0.4u}$	2.23	mm
$Area_{Backbone}$	687.38	J
$Area_{EEEP}$	687.38	J
Check	**OK**	
R_d	**4.44**	-
S_y	**11.08**	kN/m

McGill

Light Gauge Steel Frame / Wood Panel Shear Wall Testing

Test name : 4/C

Date tested : June 16, 2004

Wall Size : 4' x 8'

Screw pattern : 6" / 12"

Edge Distance : 1/2"

Test mode : ☐ Cyclic ☒ Monotonic

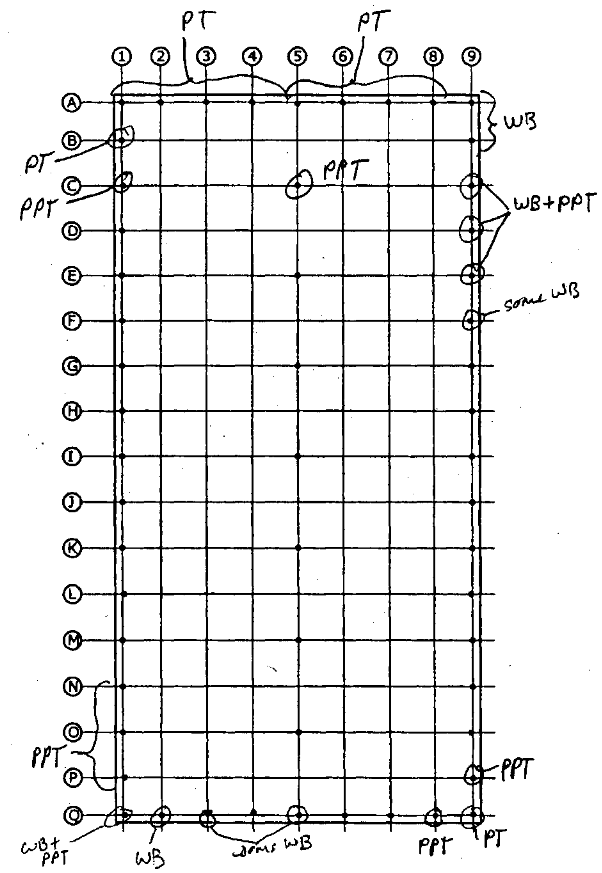
@ 6.5mm ⇒ 11989 N

@ 38.0mm ⇒ 14624 N

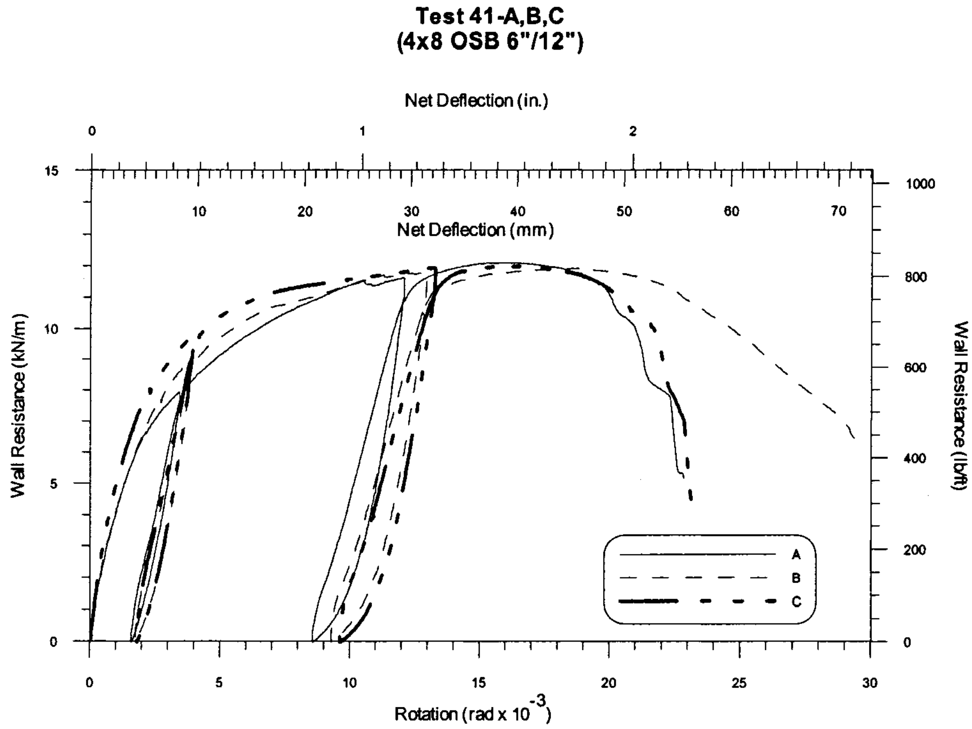
MAX LOAD ⇒ 14676N

MAX DISP. ⇒ 63.621mm

slipping @ 7.4mm



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)



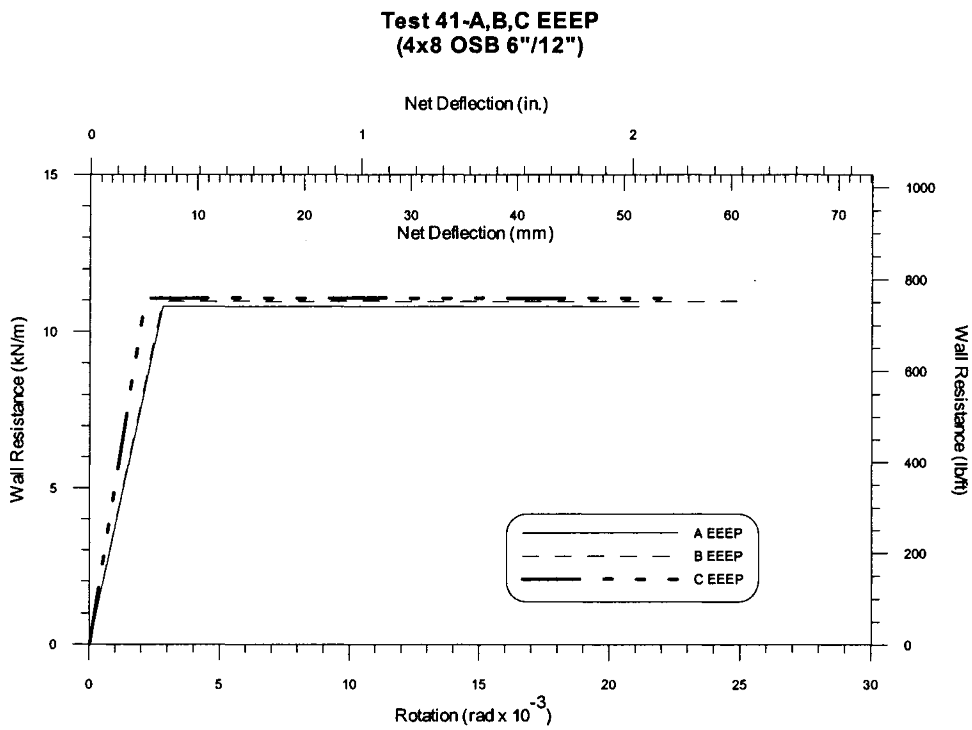


Figure B.3: Superposition of monotonic and EEEP curves for tests 41-A,B,C

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 43A

RESEARCHER: Caroline Blais **ASSISTANTS:** A. Frattini

DATE: 14-Jun-04 **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")
- [ ] Plywood (CSA 0151M) CSP 9mm (3/8")
- [X] OSB (CSA 0325) 9mm (3/8") — Grant Forest Products
- [ ] Other — MFR:

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.9 gauge 1.0" self-piercing Bugle head (HD = near hold-down (1 screw 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" [X] 4"/12" [X] 6"/12" [ ] Other:

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

STUDS:
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip Thickness: 0.043" (1.09 mm) 33ksi (230 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] T=0.043" (1.09 mm) 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
- [ ] Cyclic

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] Sheathing at Corners of Panels
- [X] Panel Sh¢ — **TOTAL:** [ ]

MOISTURE CONTENT OF SHEATHING: Moisture Meter
Wood: Nth Sth
Temp.: C
AVG: #DIV/0!

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

	North	South		
Ww=	25.13	25.04		
Wd=	24.01	23.94		
m.c.=	4.66	4.59		

AVG m.c. 4.63

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

COMMENTS:
-Shear anchors torqued for 10 s with impact wrench
-North hold down anchor 1/2 turn from finger tight, south 1/2 turn
(load cells used on both hold-downs)
-Ambient temperature 20 C
-Double chord studs used
-Top left corner of plywood was touching the upper beam by 1 or 2mm
-Square plate washers (2.5"x2.5") + circular ones in both corners of top track
-Initial load set to zero at beginning of test, displacement 0.521 mm

Test 43A
(4x8 OSB 4"/12")

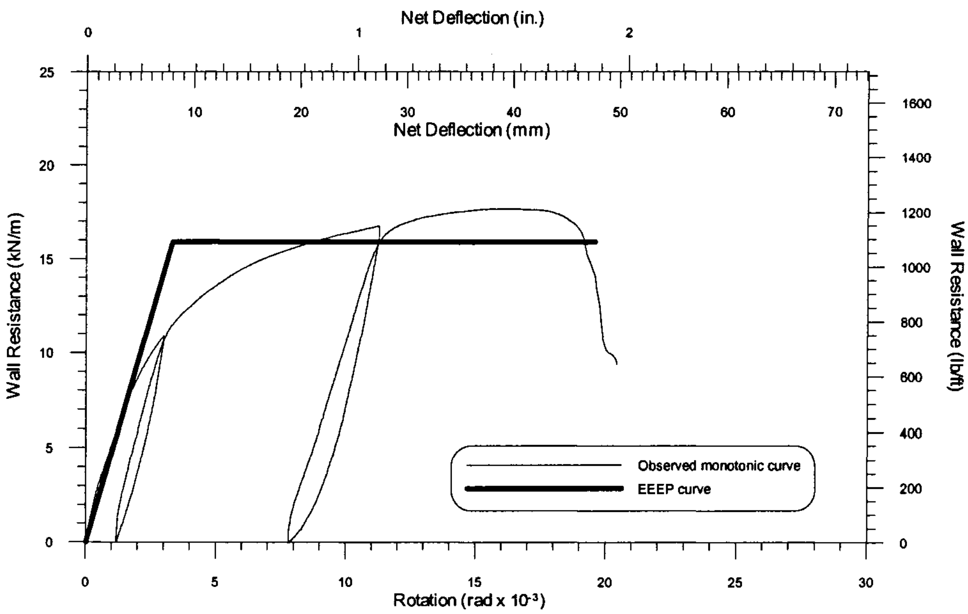


Figure B.4: Monotonic and EEEP curves for test 43-A

Table B.4: Parameters for monotonic test 43-A

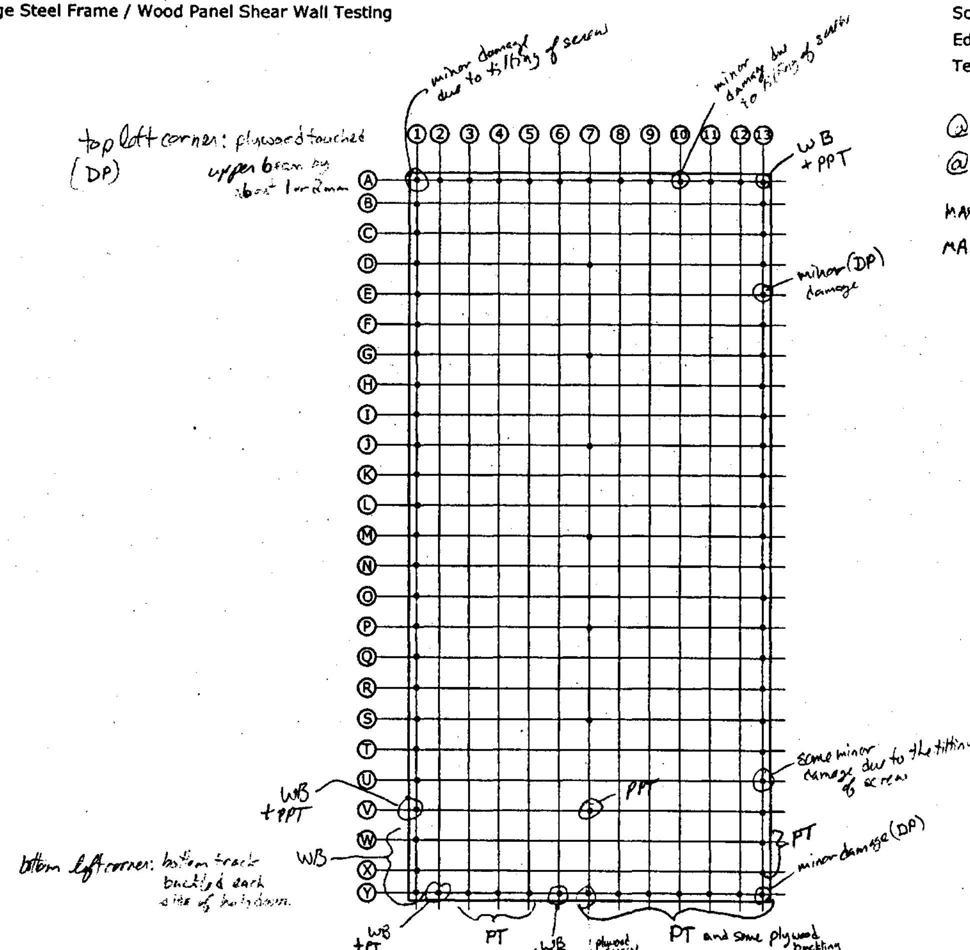
	Parameters	Units
F_u	21.59	kN
$F_{0.8u}$	17.27	kN
$F_{0.4u}$	8.63	kN
F_y	19.40	kN
K_e	2.41	kN/mm
Ductility (μ)	5.92	-
$\Delta_{net,y}$	8.06	mm
$\Delta_{net,u}$	39.40	mm
$\Delta_{net,0.8u}$	47.71	mm
$\Delta_{net,0.4u}$	3.59	mm
$Area_{Backbone}$	847.44	J
$Area_{EEEP}$	847.44	J
Check	**OK**	
R_d	**3.29**	-
S_y	**15.91**	kN/m

McGill

Light Gauge Steel Frame / Wood Panel Shear Wall Testing

Test name : 43 A
Date tested : June 14, 2004
Wall Size : 4' x 8'
Screw pattern : 4" / 12"
Edge Distance : 1/2"
Test mode : ☐ Cyclic ☒ Monotonic

@ 12.5mm ⇒ 13 354 N
@ 38.0mm ⇒ 20502 N
MAX LOAD : 21 630 N
MAX DISPL. : 64.449 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) ; No Damage (ND)

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 43B

RESEARCHER: Caroline Blais **ASSISTANTS:** A. Frattini

DATE: 14-Jun-04 **TIME:** 14:45

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")
- [ ] Plywood (CSA 0151M) CSP 9mm (3/8")
- [X] OSB (CSA 0325) 9mm (3/8") — Grant Forest Products
- [ ] Other — MFR:

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.9 gauge 1.0" self-piercing Bugle head (HD = near hold-down (1 screw 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" [X] 4"/12" [ ] 6"/12" [ ] Other:

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

STUDS:
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip Thickness: 0.043" (1.09 mm) 33ksi (230 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] T=0.043" (1.09 mm) 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
- [ ] Cyclic

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] Panel She
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral
- [X] East Frame Brace
- [X] West Frame Brace
- [X] Sheathing at Corners of Panels

TOTAL: [ ]

MOISTURE CONTENT OF SHEATHING: Moisture Meter
Wood:
Temp.: C
Nth Sth
AVG: #DIV/0!

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

	North	South		
Ww=	24.03	24.58		
Wd=	23.04	23.54		
m.c.=	4.30	4.42		

AVG m.c. 4.36

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

COMMENTS:
-Shear anchors torqued for 10 s with impact wrench
-North hold down anchor 1/2 turn from finger tight, south 1/2 turn
(load cells used on both hold-downs)
-Ambient temperature 20 C
-Double chord studs used
-Square plate washers (2.5"x2.5") + circular ones in both corners of top track
-Initial load set to zero at beginning of test, displacement 0.949 mm

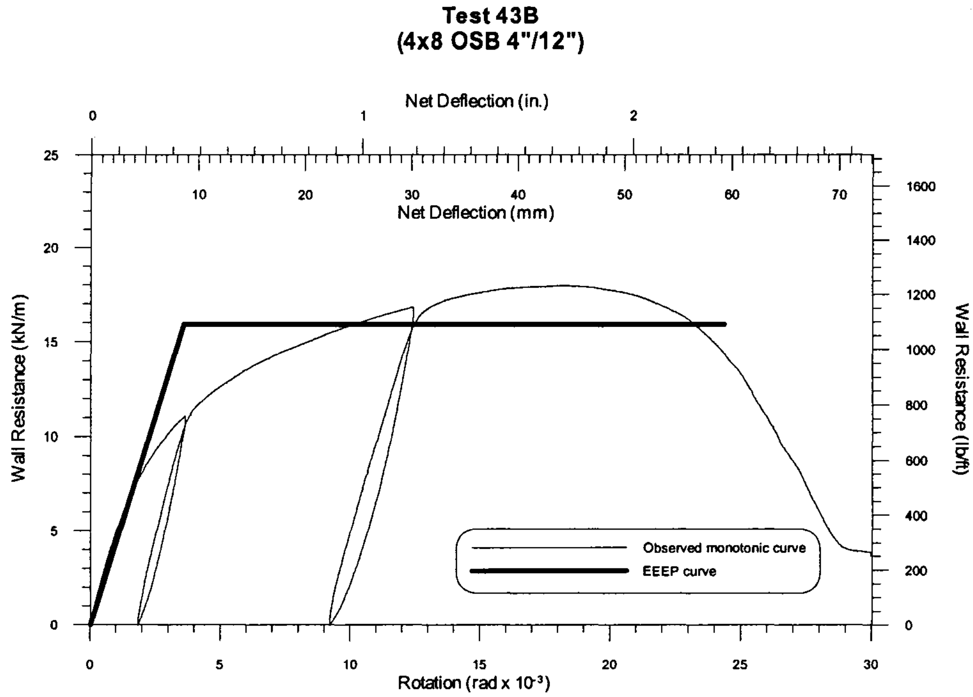


Figure B.5: Monotonic and EEEP curves for test 43-B

Table B.5: Parameters for monotonic test 43-B

	Parameters	Units
F_u	21.91	kN
$F_{0.8u}$	17.53	kN
$F_{0.4u}$	8.77	kN
F_y	19.39	kN
K_e	2.24	kN/mm
Ductility (μ)	6.86	-
$\Delta_{net,y}$	8.65	mm
$\Delta_{net,u}$	44.44	mm
$\Delta_{net,0.8u}$	59.30	mm
$\Delta_{net,0.4u}$	3.91	mm
$Area_{Backbone}$	1065.93	J
$Area_{EEEP}$	1065.93	J
Check	OK	
R_d	**3.57**	-
S_y	**15.90**	kN/m

McGill

Light Gauge Steel Frame / Wood Panel Shear Wall Testing

Test name : 43B

Date tested : June 14, 2004

Wall Size : 4' x 8'

Screw pattern : 4" / 12"

Edge Distance : 1/2"

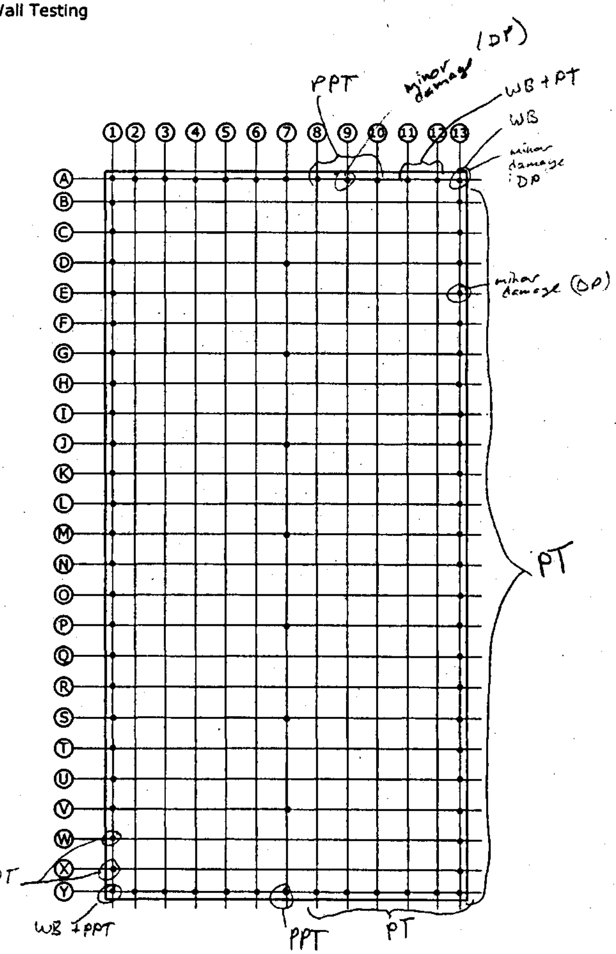
Test mode : ☐ Cyclic ☒ Monotonic

@ 12.5mm => 13587 N

@ 38.0mm => 20588 N

MAX LOAD => 21970 N

MAX DISPL. => 80.00 mm



Bottom left corner
Buckling of bottom track on both side of the holddown

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) ; No Damage (ND)

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 43C

RESEARCHER: Caroline Blais **ASSISTANTS:** A. Frattini

DATE: 15-Jun-04 **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")
- [ ] Plywood (CSA 0151M) CSP 9mm (3/8")
- [X] OSB (CSA 0325) 9mm (3/8") — Grant Forest Products
- [ ] Other — MFR:

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.9 gauge 1.0" self-piercing Bugle head (HD = near hold-down (1 screw 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" [X] 4"/12" [ ] 6"/12" [ ] Other:

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

STUDS:
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip Thickness: 0.043" (1.09 mm) 33ksi (230 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] T=0.043" (1.09 mm) 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
- [ ] Cyclic

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] Sheathing at Corners of Panels
- [X] Panel She **TOTAL:** [ ]

MOISTURE CONTENT OF SHEATHING: Moisture Meter
Wood:
Temp.: C
Nth Sth
AVG: #DIV/0!

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

	North	South
Ww=	24.71	25.05
Wd=	23.60	23.90
m.c.=	4.70	4.81

AVG m.c. 4.76

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

COMMENTS:
- -Shear anchors torqued for 10 s with impact wrench
- -North hold down anchor 3/4 turn from finger tight, south 1/2 turn
  (load cells used on both hold-downs)
- -Ambient temperature 20 C
- -Double chord studs used
- -Square plate washers (2.5"x2.5") + circular ones in both corners of top track
- -Initial load set to zero at beginning of test, displacement -0.335 mm

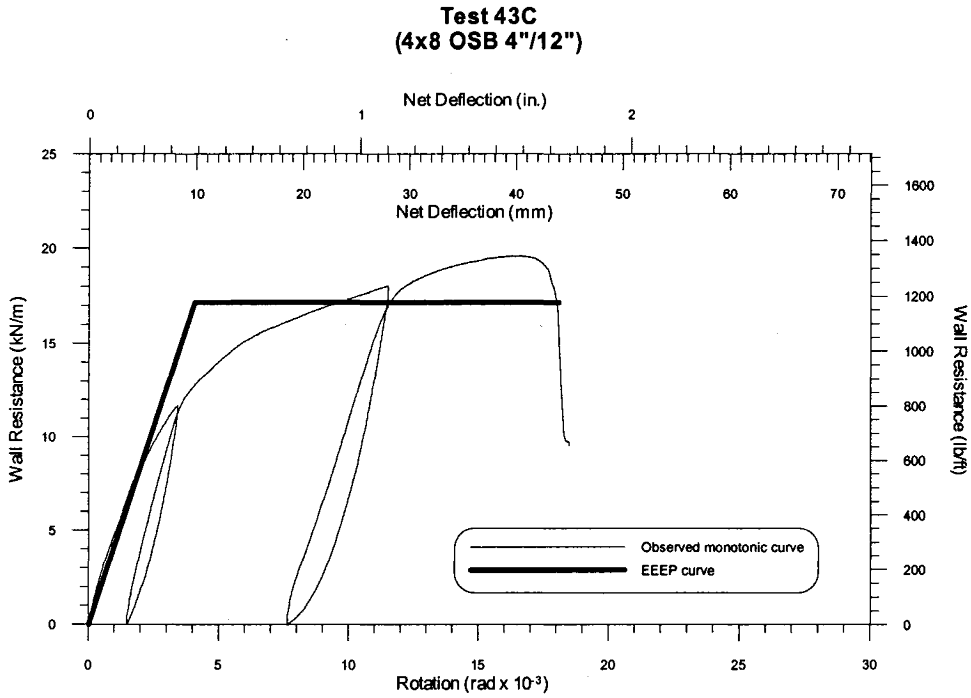


Figure B.6: Monotonic and EEEP curves for test 43-C

Table B.6: Parameters for monotonic test 43-C

	Parameters	Units
F_u	23.90	kN
$F_{0.8u}$	19.12	kN
$F_{0.4u}$	9.56	kN
F_y	20.91	kN
K_e	2.12	kN/mm
Ductility (μ)	4.46	-
$\Delta_{net,y}$	9.87	mm
$\Delta_{net,u}$	39.51	mm
$\Delta_{net,0.8u}$	44.04	mm
$\Delta_{net,0.4u}$	4.51	mm
$Area_{Backbone}$	817.87	J
$Area_{EEEP}$	817.87	J
Check	**OK**	
R_d	**2.81**	-
S_y	**17.15**	kN/m

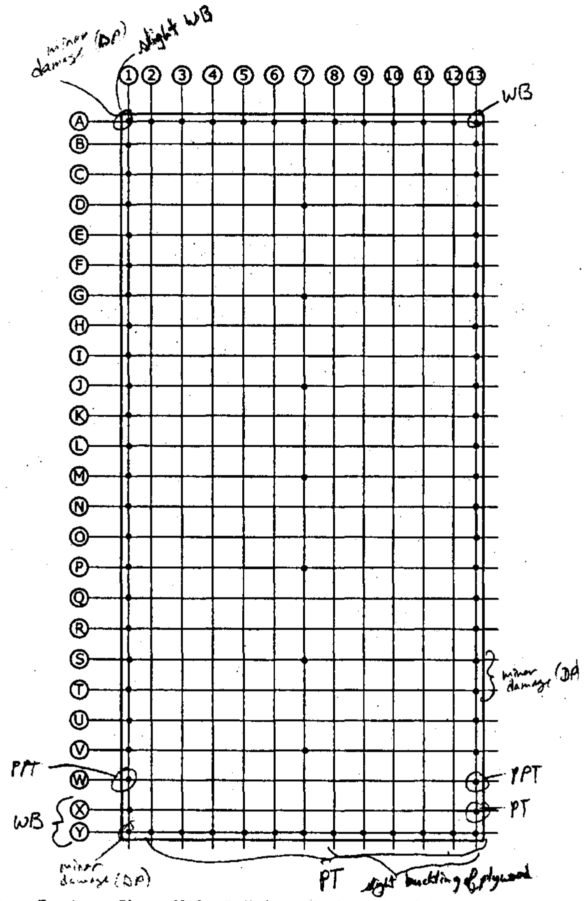
McGill

Light Gauge Steel Frame / Wood Panel Shear Wall Testing

Test name : 43C
Date tested : June 15, 2004
Wall Size : 4' x 8'
Screw pattern : 4" / 12"
Edge Distance : 1/2"
Test mode : ☐ Cyclic ☒ Monotonic

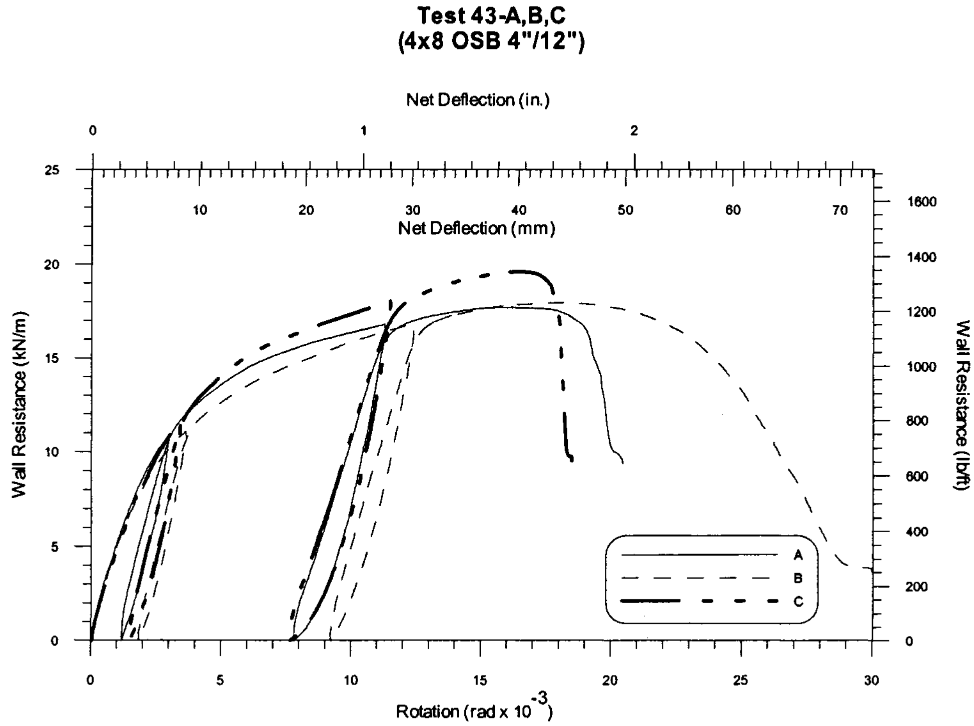
@ 12.5mm => 14320 N
@ 38.0mm => 22 057 N
MAX LOAD => 23969 N
MAX DISPL. => @ 59mm

Slipping of wall @ about 6.4mm



Bottom left: Buckling of bottom track on both sides of holddown.

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) ; No Damage (ND)



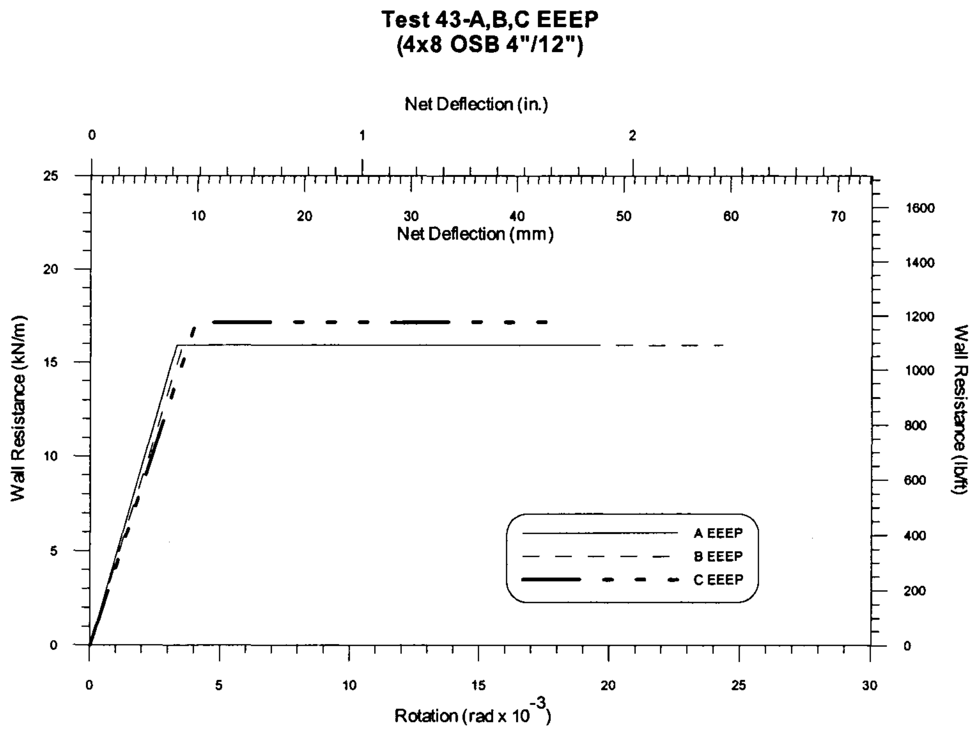


Figure B.3: Superposition of monotonic and EEEP curves for tests 43-A,B,C

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 45A

RESEARCHER: Caroline Blais **ASSISTANTS:** A. Frattini

DATE: 11-Jun-04 **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")
- [ ] Plywood (CSA 0151M) CSP 9mm (3/8")
- [X] OSB (CSA 0325) 9mm (3/8") Grant Forest Products
- [ ] Other MFR:

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.9 gauge 1.0" self-piercing Bugle head (HD = near hold-down (1 screw 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:
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip Thickness: 0.043" (1.09 mm) 33ksi (230 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] T=0.043" (1.09 mm) 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
- [ ] Cyclic

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] Sheathing at Corners of Panels
[X] Panel She		**TOTAL:** [ ]

MOISTURE CONTENT OF SHEATHING: Moisture Meter
Wood:
Temp.: C
Nth Sth
AVG: #DIV/0!

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

	North	South		
Ww=	24.89	22.00		
Wd=	23.83	21.06		
m.c.=	4.45	4.46		

AVG m.c. 4.46

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

COMMENTS:
-Shear anchors torqued for 10 s with impact wrench
-North hold down anchor 1/2 turn from finger tight, south 1/2 turn
(load cells used on both hold-downs)
-Ambient temperature 16 C
-Double chord studs used
-Square plate washers (2.5"x2.5") + circular ones in both corners of top track
-Initial load set to zero at beginning of test, displacement -0.060 mm

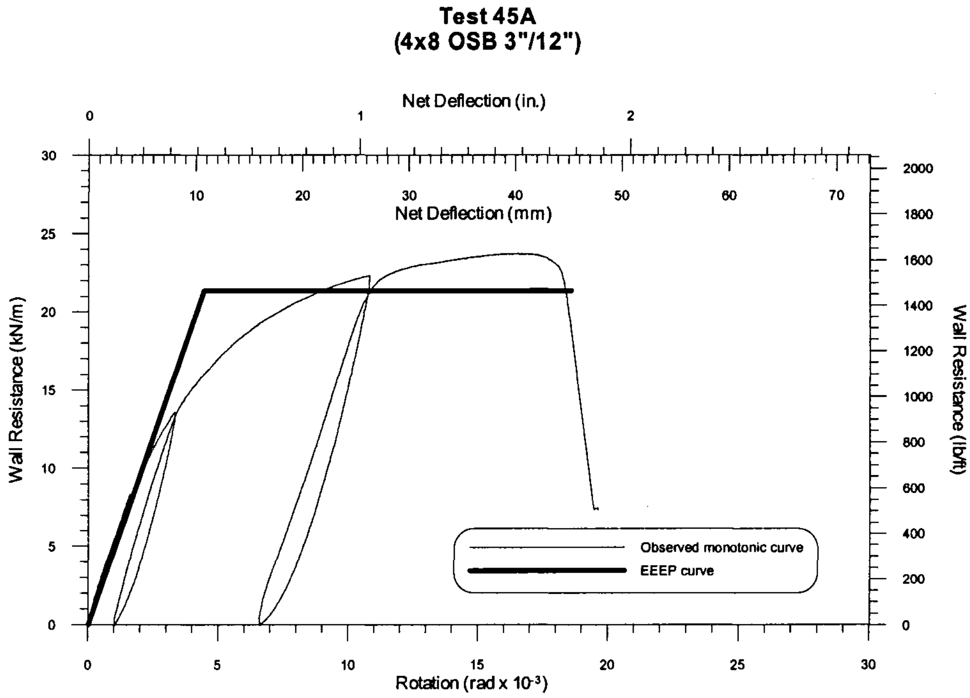


Figure B.7: Monotonic and EEEP curves for test 45-A

Table B.7: Parameters for monotonic test 45-A

	Parameters	Units
F_u	28.93	kN
$F_{0.8u}$	23.15	kN
$F_{0.4u}$	11.57	kN
F_y	26.03	kN
K_e	2.41	kN/mm
Ductility (μ)	4.20	-
$\Delta_{net,y}$	10.78	mm
$\Delta_{net,u}$	40.18	mm
$\Delta_{net,0.8u}$	45.25	mm
$\Delta_{net,0.4u}$	4.79	mm
Area$_{Backbone}$	1037.47	J
Area$_{EEEP}$	1037.47	J
Check	**OK**	
R_d	**2.72**	-
S_y	**21.35**	kN/m

McGill

Light Gauge Steel Frame / Wood Panel Shear Wall Testing

Test name : 45A

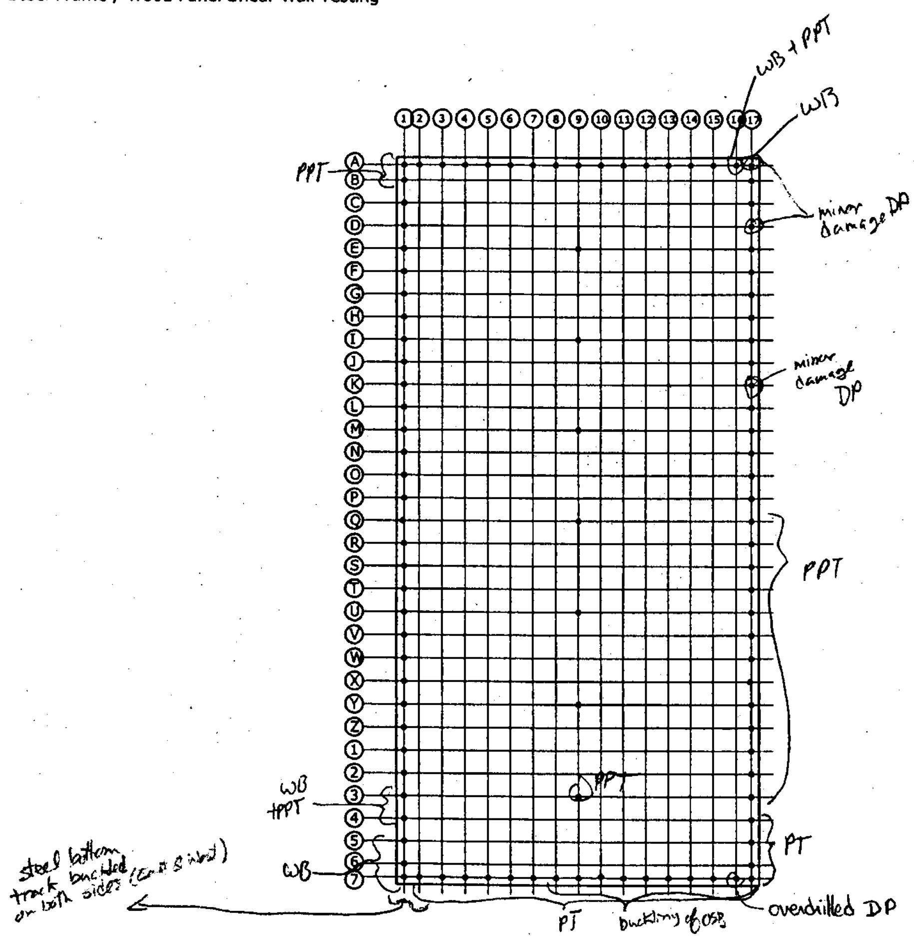
Date tested : June 11, 2004

Wall Size : 4' x 8'

Screw pattern : 3" / 12"

Edge Distance : 1/2"

Test mode : ☐ Cyclic ☒ Monotonic



@ 12.5mm => 16,630 N

@ 38.0mm => 27301 N

MAX DISPL. 63.666 mm

MAX LOAD. 28986 N

@ 6mm => some slipping (bottom track probably)

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)

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 45B

RESEARCHER: Caroline Blais **ASSISTANTS:** A. Frattini, W. Lim

DATE: 10-Jun-04 **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")
- [ ] Plywood (CSA O151M) CSP 9mm (3/8")
- [X] OSB (CSA 0325) 9mm (3/8") — Grant Forest Products
- [ ] Other — MFR:

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.9 gauge 1.0" self-piercing Bugle head (HD = near hold-down (1 screw 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:
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip Thickness: 0.043" (1.09 mm) 33ksi (230 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] T=0.043" (1.09 mm) 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
- [ ] Cyclic

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] Sheathing at Corners of Panels
[X] Panel She		TOTAL: [ ]

MOISTURE CONTENT OF SHEATHING: Moisture Meter
Wood: Nth / Sth
Temp.: C
AVG: #DIV/0!

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

	North	South
Ww=	22.40	24.19
Wd=	21.44	23.17
m.c.=	4.48	4.40

AVG m.c. 4.44

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

COMMENTS:
-Shear anchors torqued for 10 s with impact wrench
-North hold down anchor 1/2 turn from finger tight, south 1/2 turn
(load cells used on both hold-downs)
-Ambient temperature 16 C
-Double chord studs used
-Square plate washers (2.5"x2.5") + circular ones in both corners of top track
-Initial load set to zero at beginning of test, displacement 0.295 mm

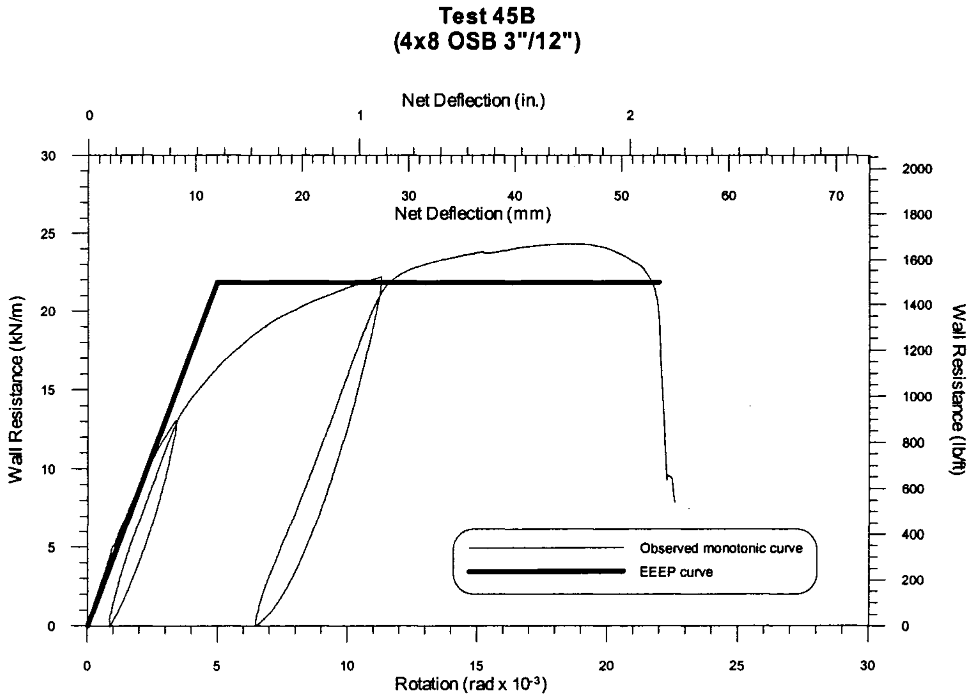


Figure B.8: Monotonic and EEEP curves for test 45-B

Table B.8: Parameters for monotonic test 45-B

	Parameters	Units
F_u	29.65	kN
$F_{0.8u}$	23.72	kN
$F_{0.4u}$	11.86	kN
F_y	26.64	kN
K_e	2.21	kN/mm
Ductility (μ)	4.44	-
$\Delta_{net,y}$	12.06	mm
$\Delta_{net,u}$	44.66	mm
$\Delta_{net,0.8u}$	53.55	mm
$\Delta_{net,0.4u}$	5.37	mm
$Area_{Backbone}$	1265.96	J
$Area_{EEEP}$	1265.96	J
Check	OK	
R_d	**2.81**	-
S_y	**21.85**	kN/m

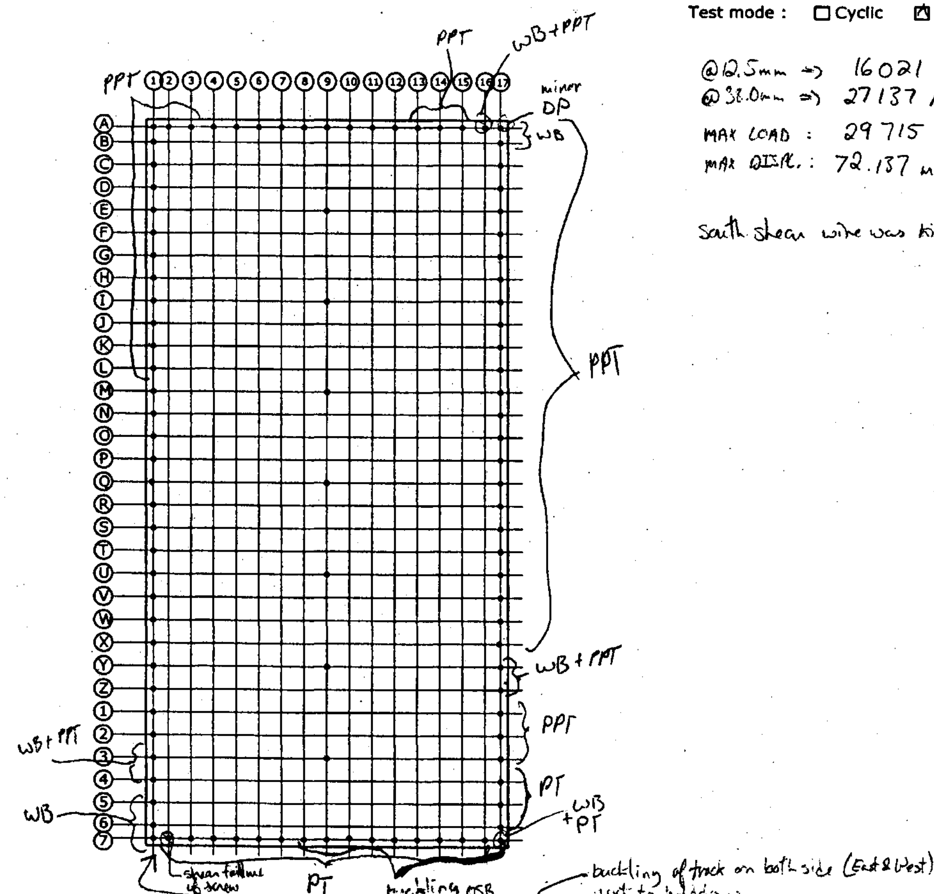
McGill

Light Gauge Steel Frame / Wood Panel Shear Wall Testing

Test name : 45B
Date tested : June 11, 2004
Wall Size : 4' x 8'
Screw pattern : 3" / 12"
Edge Distance : 1/2"
Test mode : ☐ Cyclic ☒ Monotonic

@12.5mm ⇒ 16021 N
@38.0mm ⇒ 27137 N
MAX LOAD : 29715 N
MAX DISP. : 72.137 mm

South shear wire was kinked



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)

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 45C

RESEARCHER: Caroline Blais **ASSISTANTS:** K. Hikita, A. Frattini

DATE: 10-Jun-04 **TIME:** 11:30

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)
- [ ] Plywood (CSA 0151M) CSP 12.5mm (1/2")
- [ ] Plywood (CSA 0151M) CSP 9mm (3/8")
- [X] OSB (CSA 0325) 9mm (3/8") — Grant Forest Products
- [ ] Other — MFR:

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.9 gauge 1.0" self-piercing Bugle head (HD = near hold-down (1 screw 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:
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip Thickness: 0.043" (1.09 mm) 33ksi (230 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 — [X] T=0.043" (1.09 mm) 33ksi (230 Mpa)
- Flange: 1-1/4" inches — [ ] 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
- [ ] Cyclic

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] Panel She
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral
- [X] East Frame Brace
- [X] West Frame Brace
- [X] Sheathing at Corners of Panels

TOTAL: [ ]

MOISTURE CONTENT OF SHEATHING: Moisture Meter
Wood:
Temp.: C
Nth Sth
AVG: #DIV/0!

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

	North	South		
Ww=	26.21	25.14		
Wd=	25.05	24.06		
m.c.=	4.63	4.49		

AVG m.c. 4.56

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

COMMENTS:
- -Shear anchors torqued for 10 s with impact wrench
- -North hold down anchor 1/2 turn from finger tight, south 1/2 turn
- (load cells used on both hold-downs)
- -Ambient temperature 20 C
- -Double chord studs used
- -Square plate washers (2.5"x2.5") + circular ones in both corners of top track

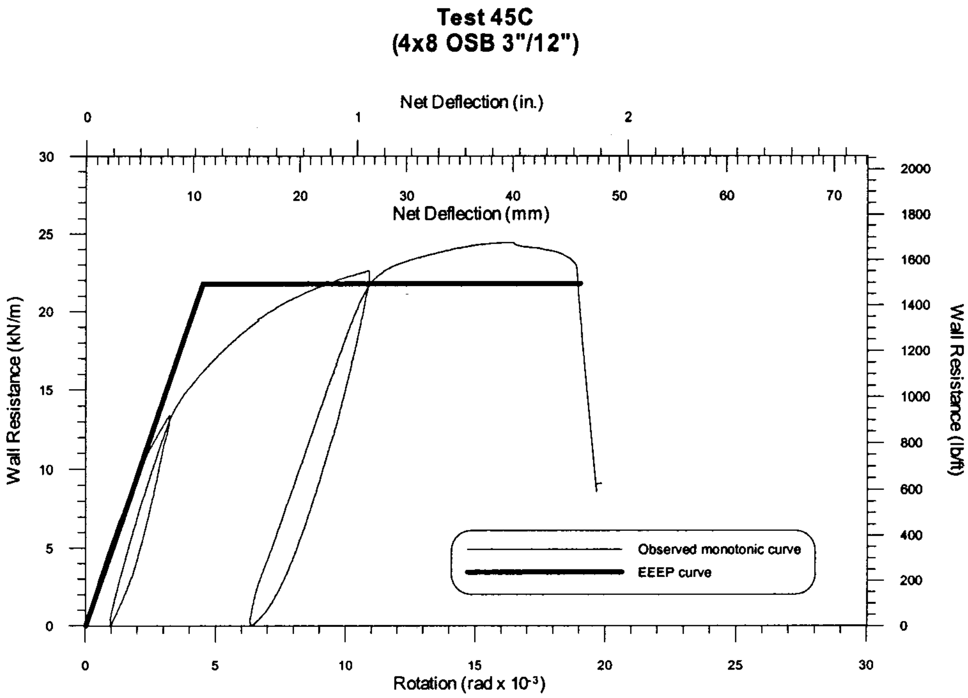


Figure B.9: Monotonic and EEEP curves for test 45-C

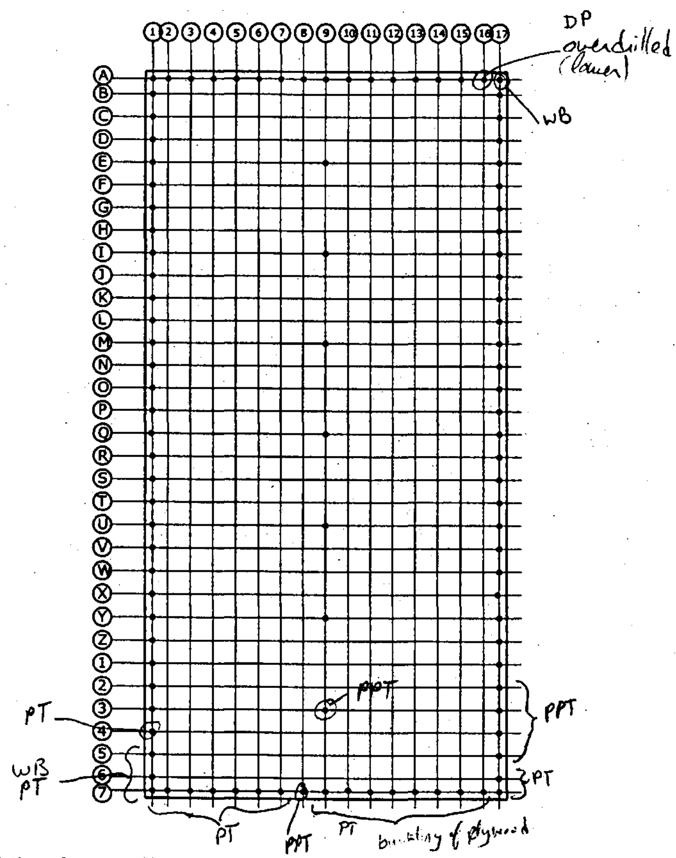
Table B.9: Parameters for monotonic test 45-C

	Parameters	Units
F_u	29.80	kN
$F_{0.8u}$	23.84	kN
$F_{0.4u}$	11.92	kN
F_y	26.56	kN
K_e	2.43	kN/mm
Ductility (μ)	4.25	-
$\Delta_{net,y}$	10.92	mm
$\Delta_{net,u}$	39.61	mm
$\Delta_{net,0.8u}$	46.37	mm
$\Delta_{net,0.4u}$	4.90	mm
$Area_{Backbone}$	1086.70	J
$Area_{EEEP}$	1086.70	J
Check	**OK**	
R_d	**2.74**	-
S_y	**21.79**	kN/m

McGill

Light Gauge Steel Frame / Wood Panel Shear Wall Testing

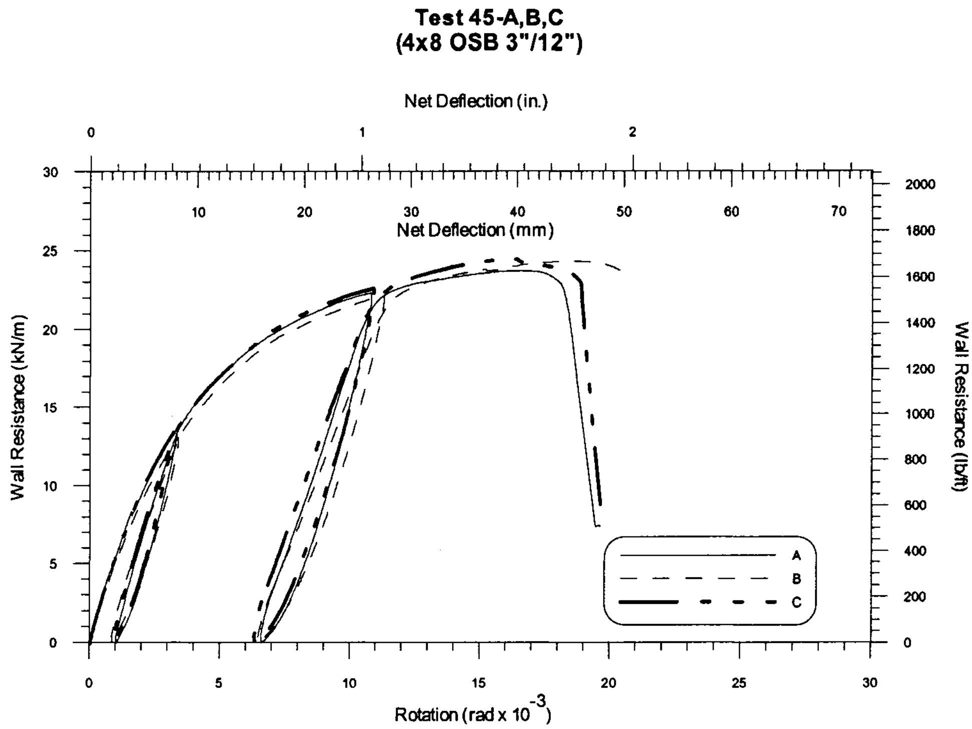
Test name : 45C
Date tested : June 10, 2004
Wall Size : 4' x 8'
Screw pattern : 3" / 12"
Edge Distance : 1/2"
Test mode : ☐ Cyclic ☒ Monotonic



@ 0.5mm ⇒ 16390N
@ 38.0mm ⇒ 27656N
max load ⇒ 29871N
max disp. ⇒ 65,559mm

south shear wire is kinked

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)



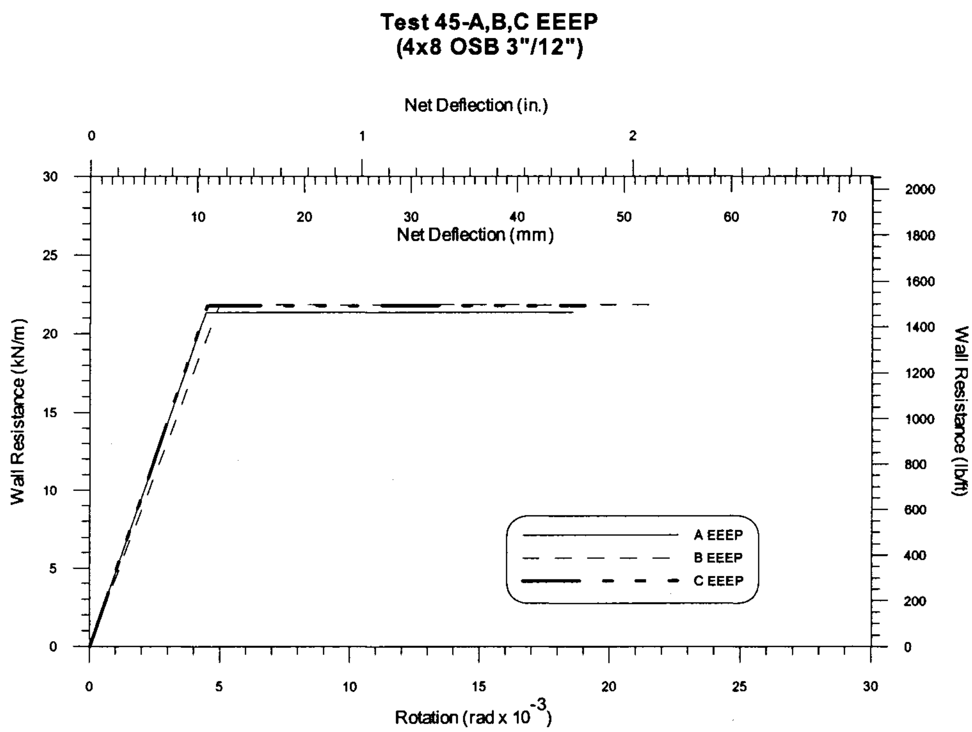


Figure B.3: Superposition of monotonic and EEEP curves for tests 45-A,B,C

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 42A

RESEARCHER: Caroline Blais **ASSISTANTS:** A. Frattini

DATE: 17-Jun-04 **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")
- [ ] Plywood (CSA 0151M) CSP 9mm (3/8")
- [X] OSB (CSA 0325) 9mm (3/8") — Grant Forest Products
- [ ] Other — MFR:

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.9 gauge 1.0" self-piercing Bugle head (HD = near hold-down (1 screw 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:
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip Thickness: 0.043" (1.09 mm) 33ksi (230 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] T=0.043" (1.09 mm) 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
- [X] Cyclic

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] Panel She
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral
- [X] East Frame Brace
- [X] West Frame Brace
- [X] Sheathing at Corners of Panels

TOTAL: [ ]

MOISTURE CONTENT OF SHEATHING: Moisture Meter
Wood:
Temp.: C
Nth / Sth
AVG: #DIV/0!

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

	North	South		
Ww=	21.58	23.88		
Wd=	20.55	22.77		
m.c.=	5.01	4.87		

AVG m.c. 4.94

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

COMMENTS:
-Shear anchors torqued for 10 s with impact wrench
-North hold down anchor 1/2 turn from finger tight, south 1/2 turn
(load cells used on both hold-downs)
-Ambient temperature 20 C
-Double chord studs used
-Square plate washers (2.5"x2.5") + circular ones in both corners of top track
-Initial load set to zero at beginning of test, displacement -0.520 mm

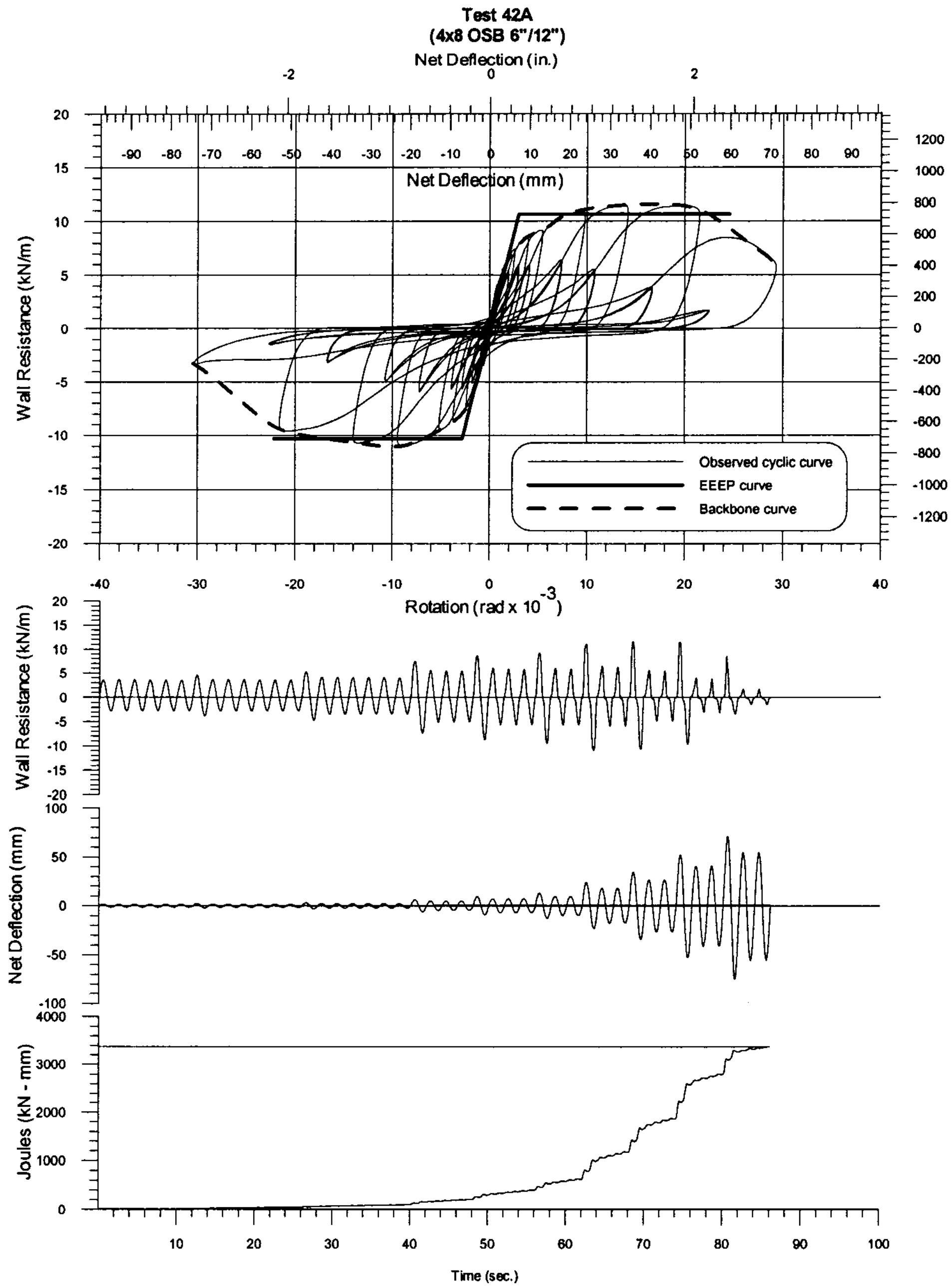


Figure B.10: Cyclic, EEEP and backbone curves for test 42-A

Table B.10: Parameters for cyclic test 42-A

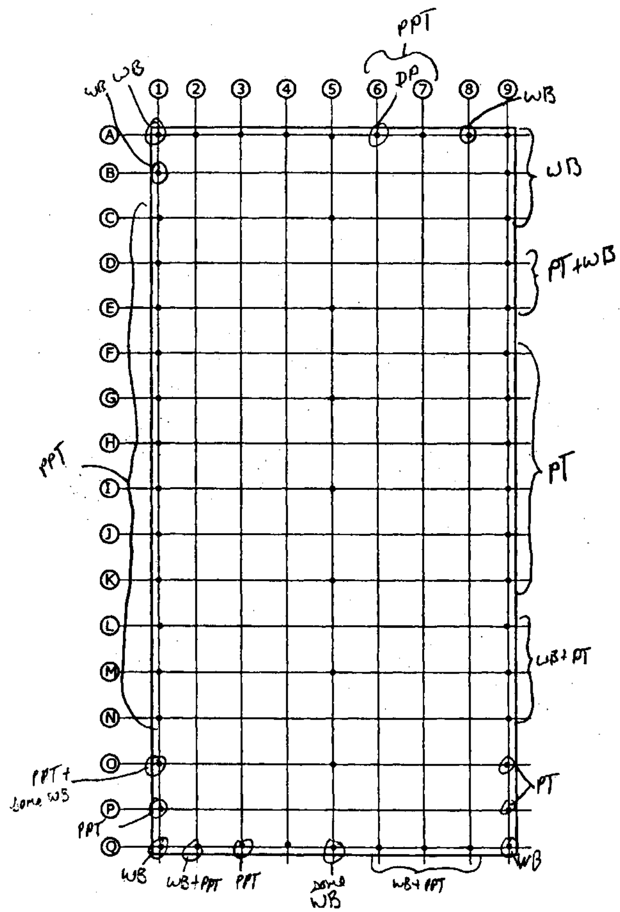
	Negative	Positive	Units
F_u	-13.36	14.03	kN
$F_{0.8u}$	-10.69	11.23	kN
$F_{0.4u}$	-5.34	5.61	kN
F_y	-12.59	12.99	kN
K_e	1.84	1.81	kN/mm
Ductility (μ)	7.92	8.34	-
$\Delta_{net,y}$	-6.83	7.17	mm
$\Delta_{net,u}$	-21.64	33.00	mm
$\Delta_{net,0.8u}$	-54.10	59.80	mm
$\Delta_{net,0.4u}$	-2.90	3.10	mm
$Area_{Backbone}$	638.12	730.20	kN-mm
$Area_{EEEP}$	638.12	730.20	kN-mm
Check	OK	OK	
R_d	**3.85**	**3.96**	-
S_y	**-10.33**	**10.65**	kN/m

McGill

Light Gauge Steel Frame / Wood Panel Shear Wall Testing

Test name : 42A
Date tested : June 17, 2004
Wall Size : 4' x 8'
Screw pattern : 6" / 12"
Edge Distance : 1/2"
Test mode : ☒ Cyclic ☐ Monotonic

MAX LOAD: 14 198N



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)

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 42B

RESEARCHER: Caroline Blais **ASSISTANTS:** A. Frattini

DATE: 18-Jun-04 **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")
- [ ] Plywood (CSA O151M) CSP 9mm (3/8")
- [X] OSB (CSA O325) 9mm (3/8") Grant Forest Products
- [ ] Other MFR:

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.9 gauge 1.0" self-piercing Bugle head (HD = near hold-down (1 screw 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:
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip Thickness: 0.043" (1.09 mm) 33ksi (230 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 [X] T=0.043" (1.09 mm) 33ksi (230 Mpa)
Flange: 1-1/4" inches [ ] 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
- [X] Cyclic

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] Sheathing at Corners of Panels
- [X] Panel She **TOTAL:** [ ]

MOISTURE CONTENT OF SHEATHING: Moisture Meter
Wood:
Temp.: C
Nth Sth
AVG: #DIV/0!

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

	North	South
Ww=	26.06	26.20
Wd=	24.81	24.99
m.c.=	5.04	4.84

AVG m.c. 4.94

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

COMMENTS:
-Shear anchors torqued for 10 s with impact wrench
-North hold down anchor 1/2 turn from finger tight, south 1/2 turn
(load cells used on both hold-downs)
-Ambient temperature 18 C
-Double chord studs used
-Square plate washers (2.5"x2.5") + circular ones in both corners of top track
-Initial load set to zero at beginning of test, displacement 0.517 mm

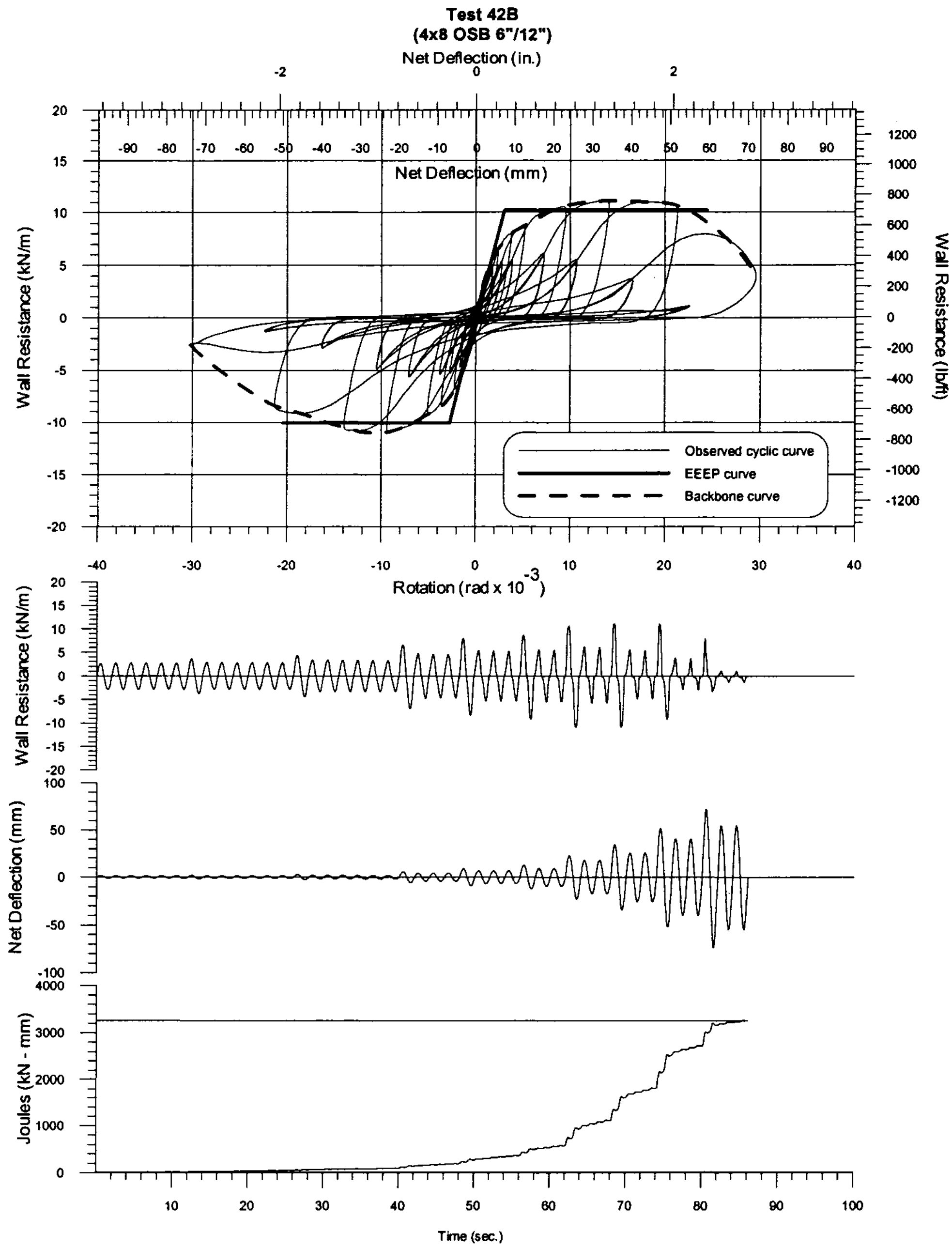


Figure B.11: Cyclic, EEEP and backbone curves for test 42-B

Table B.11: Parameters for cyclic test 42-B

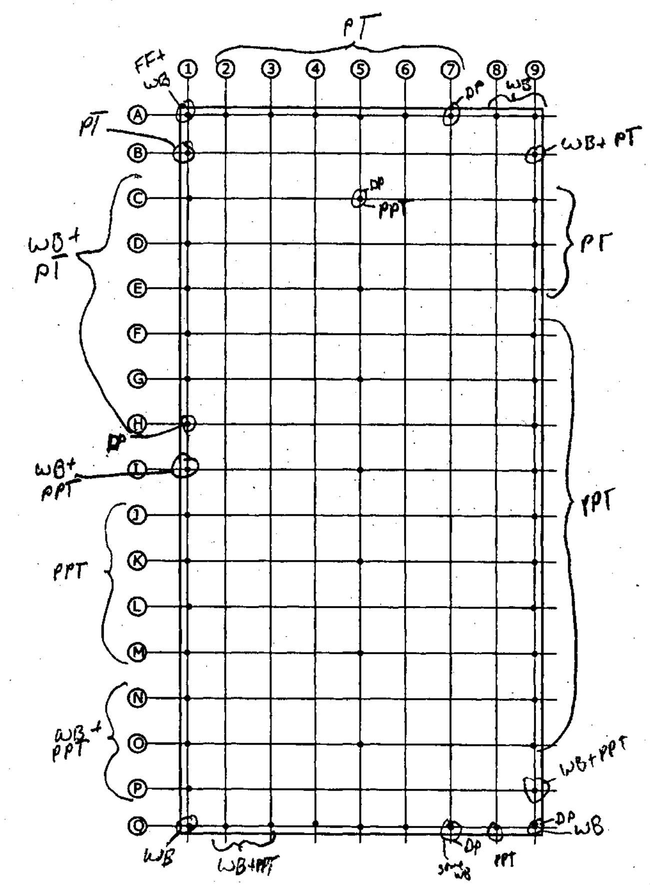
	Negative	Positive	Units
F_u	-13.29	13.57	kN
$F_{0.8u}$	-10.63	10.85	kN
$F_{0.4u}$	-5.31	5.43	kN
F_y	-12.24	12.48	kN
K_e	1.83	1.70	kN/mm
Ductility (μ)	7.46	8.07	-
$\Delta_{net,y}$	-6.68	7.36	mm
$\Delta_{net,u}$	-22.19	33.62	mm
$\Delta_{net,0.8u}$	-49.80	59.40	mm
$\Delta_{net,0.4u}$	-2.90	3.20	mm
$Area_{Backbone}$	568.56	695.13	kN-mm
$Area_{EEEP}$	568.56	695.13	kN-mm
Check	OK	OK	
R_d	**3.73**	**3.89**	-
S_y	**-10.04**	**10.23**	kN/m

McGill

Light Gauge Steel Frame / Wood Panel Shear Wall Testing

Test name : 42B
Date tested : June 18, 2004
Wall Size : 4' x 8'
Screw pattern : 6" / 12"
Edge Distance : 1/2"
Test mode : ☒ Cyclic ☐ Monotonic

MAX LOAD: 13 767 N



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)

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 42C

RESEARCHER: Caroline Blais **ASSISTANTS:** A. Frattini

DATE: 18-Jun-04 **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")
- [ ] Plywood (CSA 0151M) CSP 9mm (3/8")
- [X] OSB (CSA 0325) 9mm (3/8") — Grant Forest Products
- [ ] Other — MFR:

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.9 gauge 1.0" self-piercing Bugle head (HD = near hold-down (1 screw 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:
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip Thickness: 0.043" (1.09 mm) 33ksi (230 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] T=0.043" (1.09 mm) 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
- [X] Cyclic

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] Panel Shi
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral
- [X] East Frame Brace
- [X] West Frame Brace
- [X] Sheathing at Corners of Panels

TOTAL: [ ]

MOISTURE CONTENT OF SHEATHING: Moisture Meter
Wood:
Temp.: C
Nth Sth
AVG: #DIV/0!

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

	North	South
Ww=	23.71	22.55
Wd=	22.56	21.42
m.c.=	5.10	5.28

AVG m.c. 5.19

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

COMMENTS:
-Shear anchors torqued for 10 s with impact wrench
-North hold down anchor 1/2 turn from finger tight, south 1/2 turn
(load cells used on both hold-downs)
-Ambient temperature 18 C
-Double chord studs used
-Square plate washers (2.5"x2.5") + circular ones in both corners of top track
-Initial load set to zero at beginning of test, displacement 0.349 mm

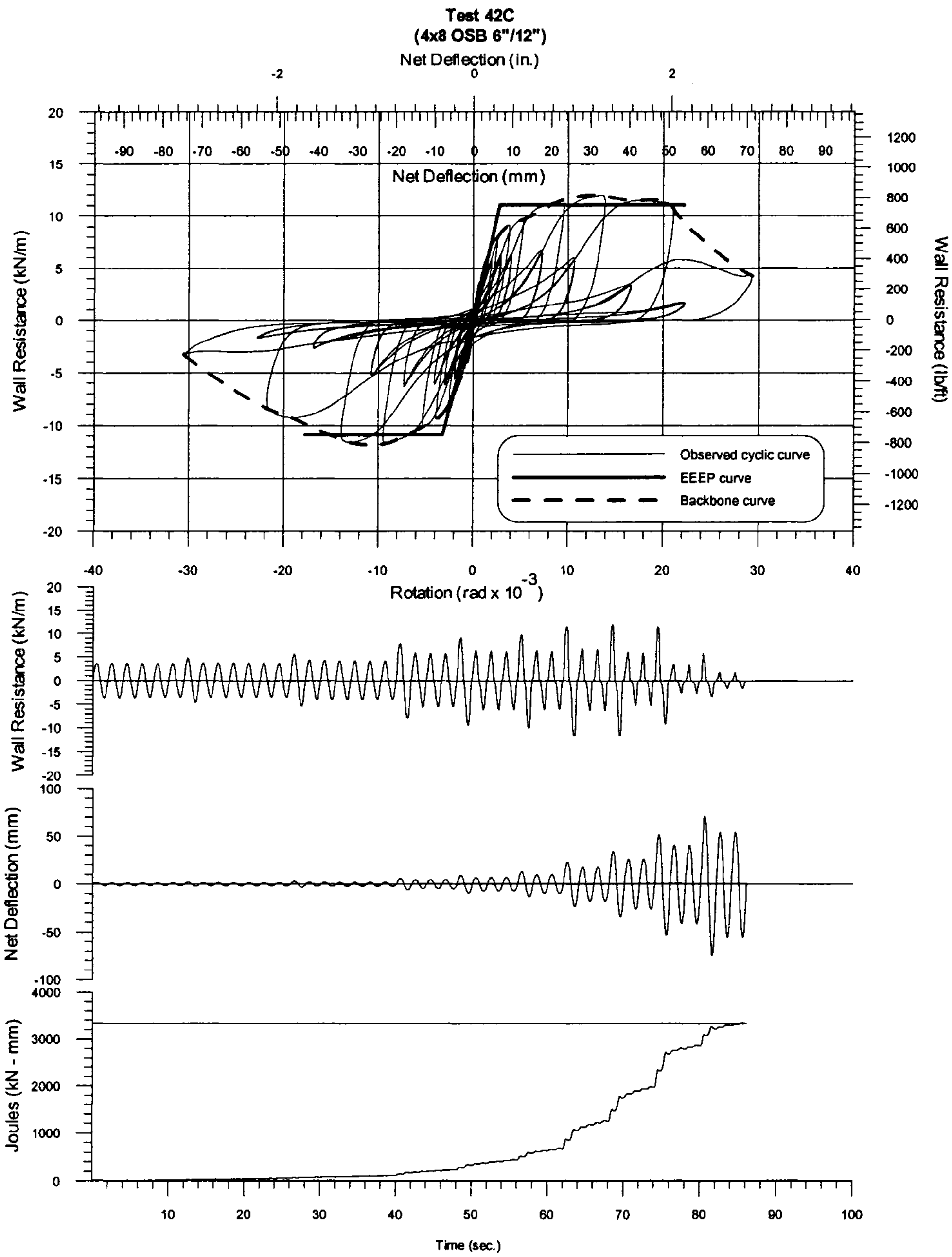


Figure B.12: Cyclic, EEEP and backbone curves for test 42-C

Table B.12: Parameters for cyclic test 42-C

	Negative	Positive	Units
F_u	-14.23	14.57	kN
$F_{0.8u}$	-11.38	11.66	kN
$F_{0.4u}$	-5.69	5.83	kN
F_y	-13.29	13.45	kN
K_e	1.67	2.01	kN/mm
Ductility (μ)	5.45	8.07	-
$\Delta_{net,y}$	-7.94	6.69	mm
$\Delta_{net,u}$	-22.22	32.50	mm
$\Delta_{net,0.8u}$	-43.30	54.00	mm
$\Delta_{net,0.4u}$	-3.40	2.90	mm
$Area_{Backbone}$	522.68	681.33	kN-mm
$Area_{EEEP}$	522.68	681.33	kN-mm
Check	OK	OK	
R_d	**3.15**	**3.89**	-
S_y	**-10.90**	**11.03**	kN/m

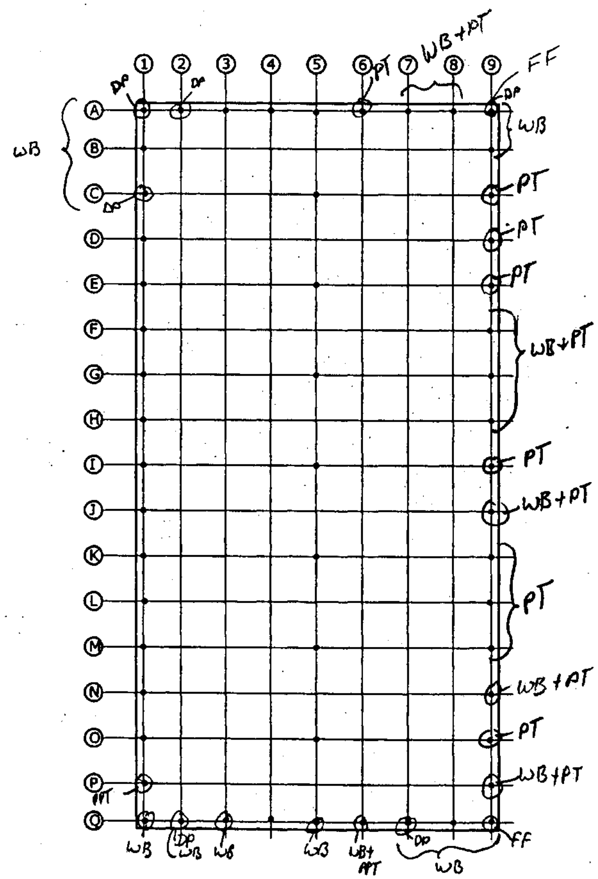
McGill

Light Gauge Steel Frame / Wood Panel Shear Wall Testing

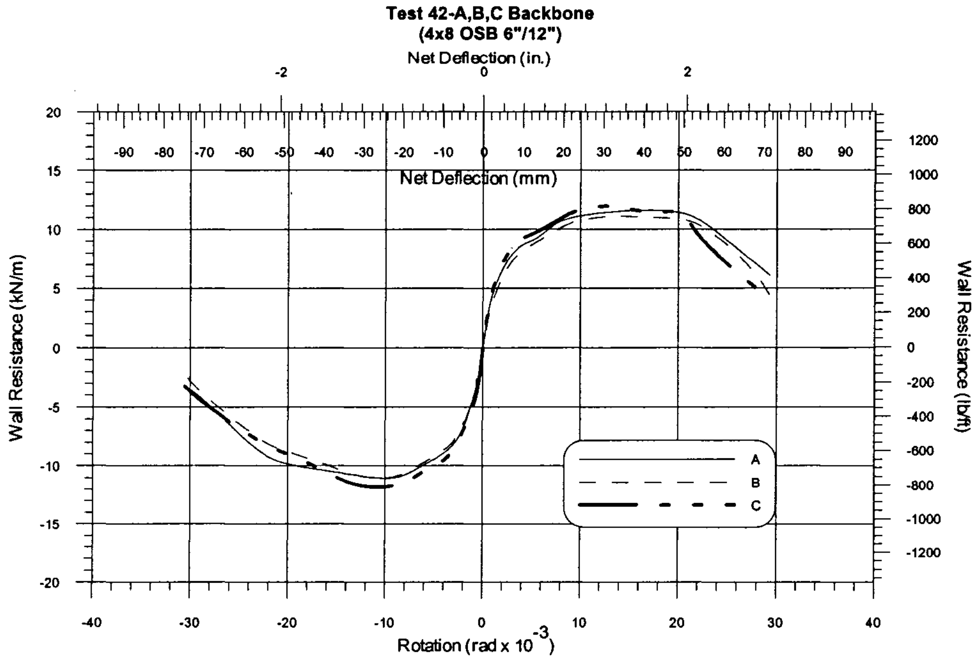
Test name : 42C
Date tested : June 18, 2004
Wall Size : 4' x 8'
Screw pattern : 6" / 12"
Edge Distance : 1/2"
Test mode : ☒ Cyclic ☐ Monotonic

MAX LOAD: 14742N

→ Right brace was not tight enough



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)



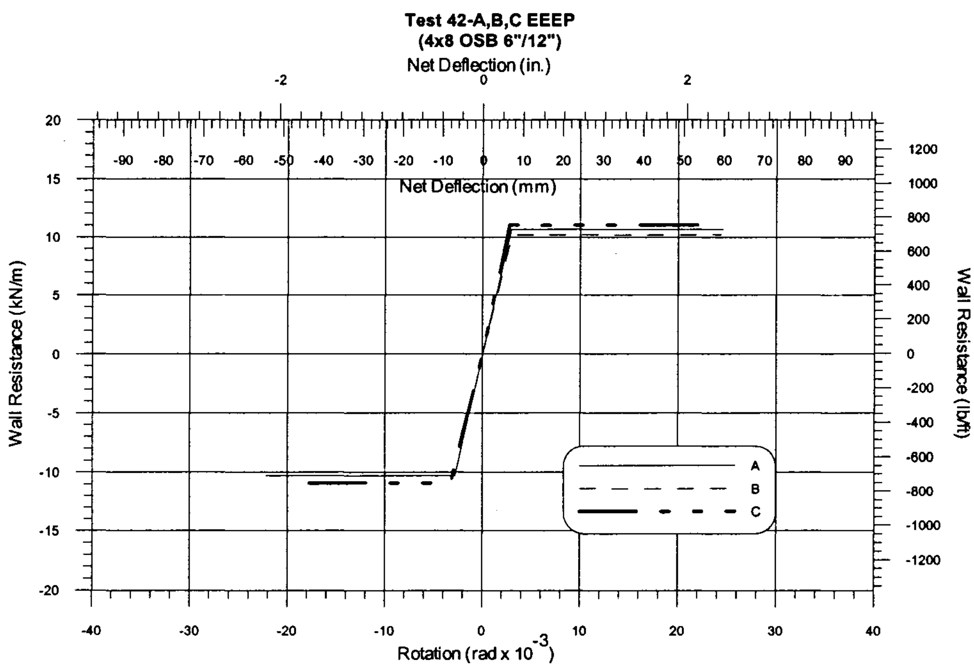


Figure B.3: Superposition of backbone and EEEP curves for tests 42-A,B,C

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 44A

RESEARCHER: Caroline Blais **ASSISTANTS:** A. Frattini

DATE: 18-Jun-04 **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")
- [ ] Plywood (CSA O151M) CSP 9mm (3/8")
- [X] OSB (CSA O325) 9mm (3/8") — Grant Forest Products
- [ ] Other — MFR:

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.9 gauge 1.0" self-piercing Bugle head (HD = near hold-down (1 screw 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 [X] 6 bolts [ ] 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" [X] 4"/12" [ ] 6"/12" [ ] Other:

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

STUDS:
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip Thickness: 0.043" (1.09 mm) 33ksi (230 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] T=0.043" (1.09 mm) 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
- [X] Cyclic

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] Panel She
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral
- [X] East Frame Brace
- [X] West Frame Brace
- [X] Sheathing at Corners of Panels

TOTAL:

MOISTURE CONTENT OF SHEATHING: Moisture Meter
Wood:
Temp.: C
Nth Sth
AVG: #DIV/0!

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

	North	South
Ww=	26.64	21.85
Wd=	25.26	20.65
m.c.=	5.46	5.81

AVG m.c. 5.64

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

COMMENTS:
-Shear anchors torqued for 10 s with impact wrench
-North hold down anchor 1/2 turn from finger tight, south 1/2 turn
(load cells used on both hold-downs)
-Ambient temperature 20 C
-Double chord studs used
-Square plate washers (2.5"x2.5") + circular ones in both corners of top track
-Initial load set to zero at beginning of test, displacement 0.160 mm

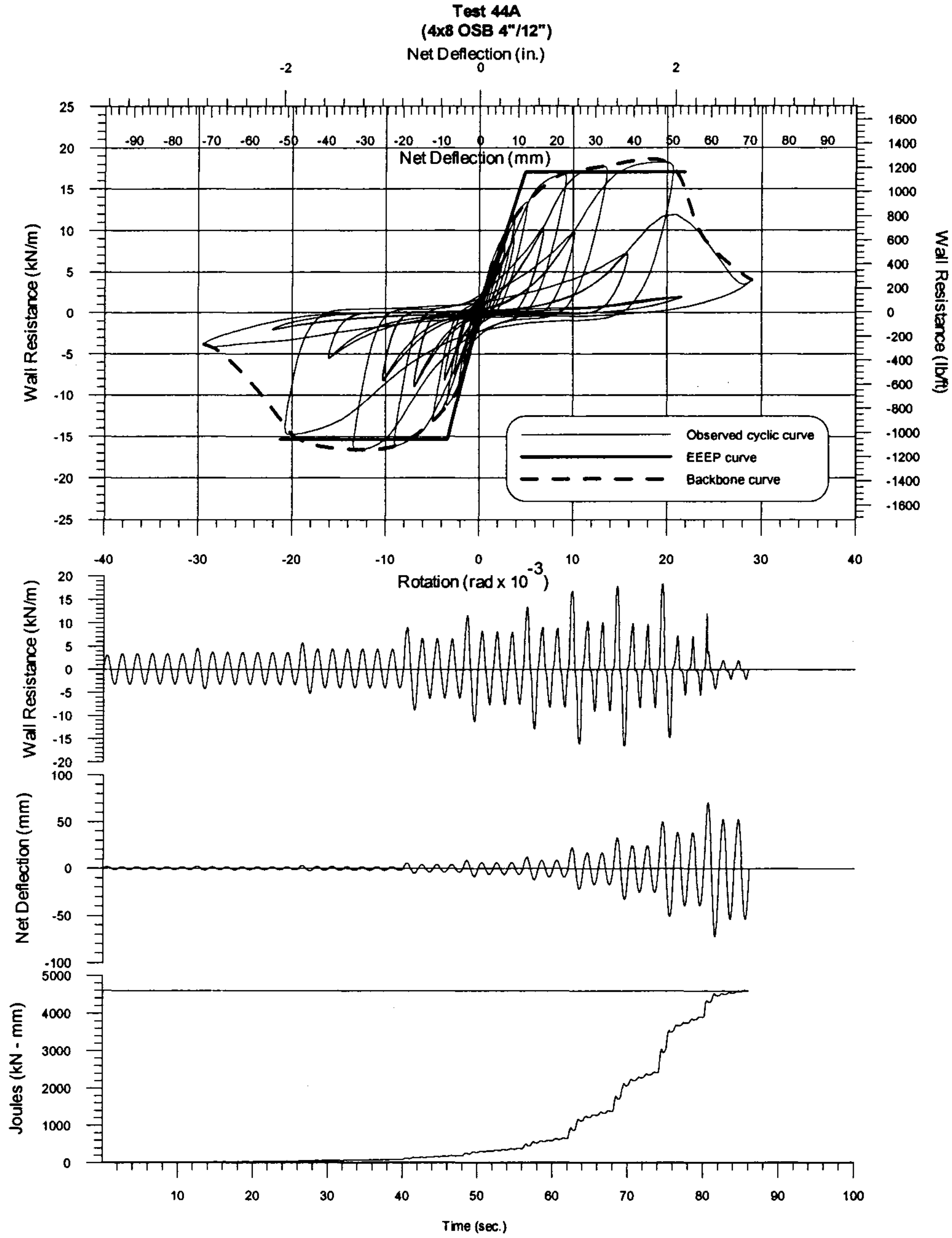


Figure B.13: Cyclic, EEEP and backbone curves for test 44-A

Table B.13: Parameters for cyclic test 44-A

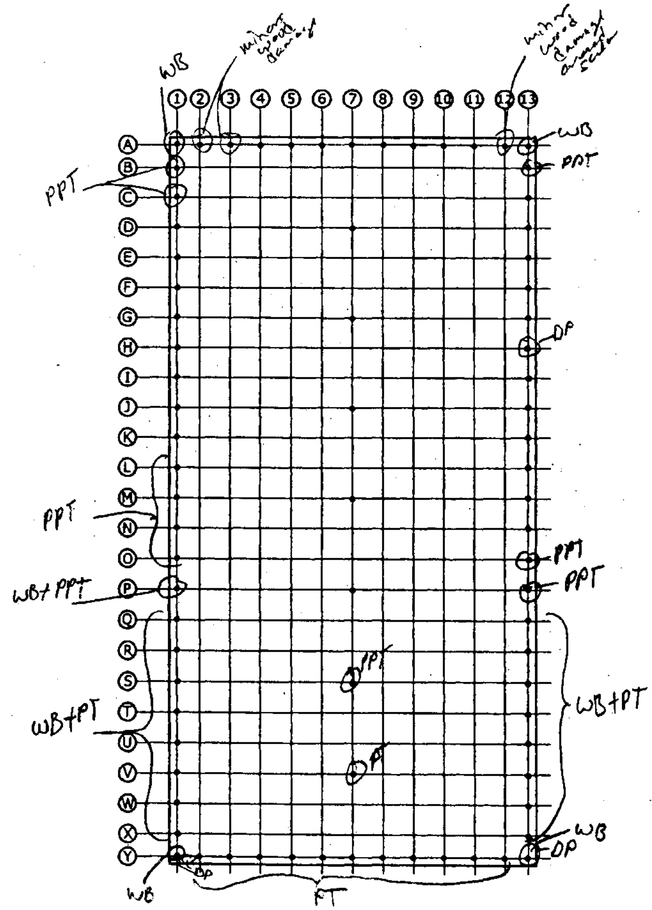
	Negative	Positive	Units
F_u	-20.23	22.26	kN
$F_{0.8u}$	-16.19	17.81	kN
$F_{0.4u}$	-8.09	8.90	kN
F_y	-18.75	20.82	kN
K_e	2.25	1.75	kN/mm
Ductility (μ)	6.21	4.46	-
$\Delta_{net,y}$	-8.34	11.93	mm
$\Delta_{net,u}$	-31.45	48.17	mm
$\Delta_{net,0.8u}$	-51.80	53.20	mm
$\Delta_{net,0.4u}$	-3.60	5.10	mm
$Area_{Backbone}$	892.86	983.67	kN-mm
$Area_{EEEP}$	892.86	983.67	kN-mm
Check	OK	OK	
R_d	**3.38**	**2.81**	-
S_y	**-15.38**	**17.08**	kN/m

McGill

Light Gauge Steel Frame / Wood Panel Shear Wall Testing

Test name : 44A
Date tested : June 18
Wall Size : 4' x 8'
Screw pattern : 4" / 12"
Edge Distance : 1/2"
Test mode : ☒ Cyclic ☐ Monotonic

MAX LOAD: 22516 N



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) ; No Damage (ND)

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 44B

RESEARCHER: Caroline Blais **ASSISTANTS:** A. Frattini

DATE: 21-Jun-04 **TIME:** 10:15

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")
- [ ] Plywood (CSA O151M) CSP 9mm (3/8")
- [X] OSB (CSA 0325) 9mm (3/8") — Grant Forest Products
- [ ] Other — MFR:

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.9 gauge 1.0" self-piercing Bugle head (HD = near hold-down (1 screw 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" [X] 4"/12" [ ] 6"/12" [ ] Other:

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

STUDS:
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip Thickness: 0.043" (1.09 mm) 33ksi (230 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] T=0.043" (1.09 mm) 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
- [X] Cyclic

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] Panel She
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral
- [X] East Frame Brace
- [X] West Frame Brace
- [X] Sheathing at Corners of Panels

TOTAL: [ ]

MOISTURE CONTENT OF SHEATHING: Moisture Meter
Wood:
Temp.: C
Nth Sth
AVG: #DIV/0!

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

	North	South		
Ww=	23.71	23.26		
Wd=	22.69	22.26		
m.c.=	4.50	4.49		

AVG m.c. 4.49

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

COMMENTS:
-Shear anchors torqued for 10 s with impact wrench
-North hold down anchor 1/2 turn from finger tight, south 1/2 turn
(load cells used on both hold-downs)
-Ambient temperature 18 C
-Double chord studs used
-Square plate washers (2.5"x2.5") + circular ones in both corners of top track
-Initial load set to zero at beginning of test, displacement 0.197 mm

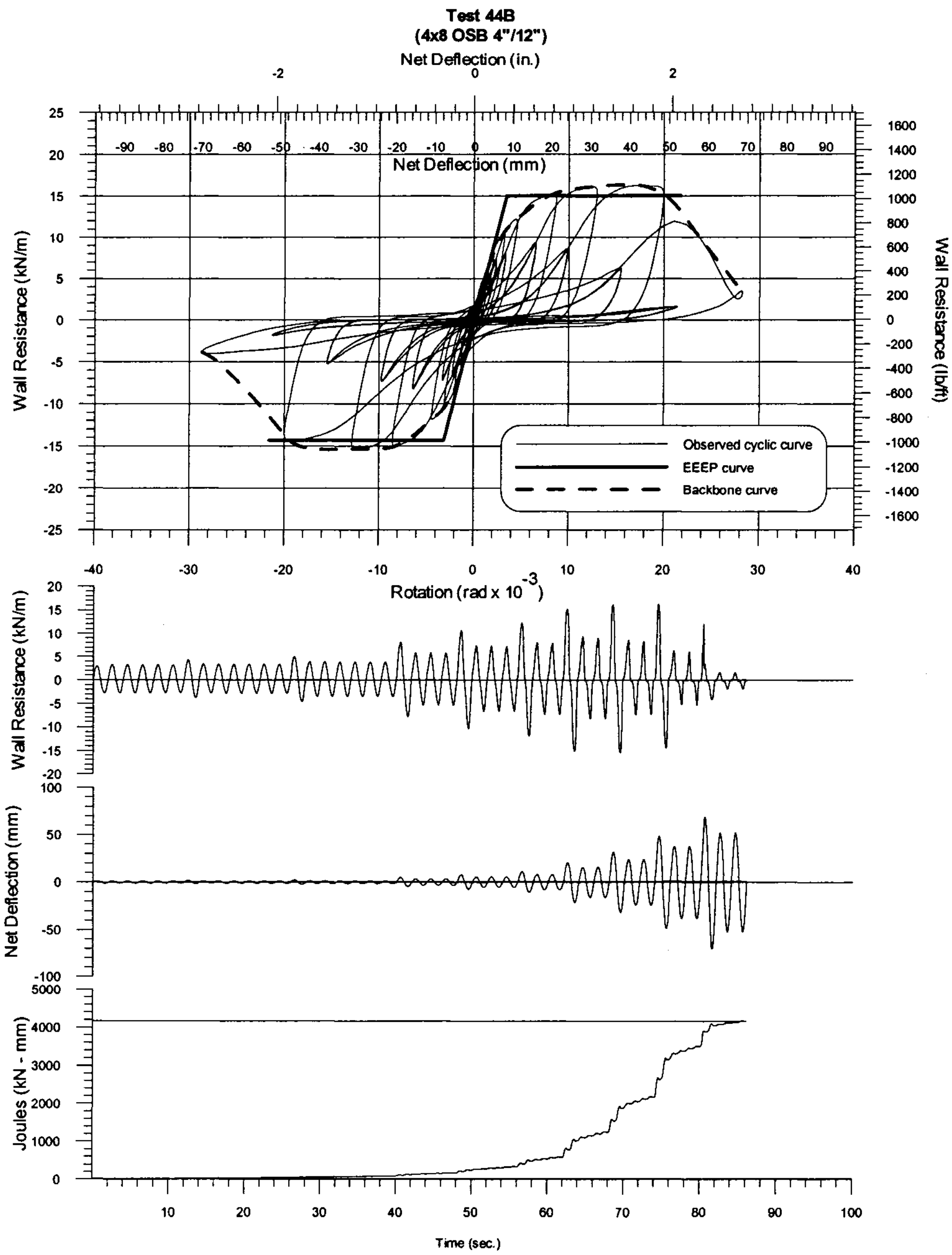


Figure B.14: Cyclic, EEEP and backbone curves for test 44-B

Table B.14: Parameters for cyclic test 44-B

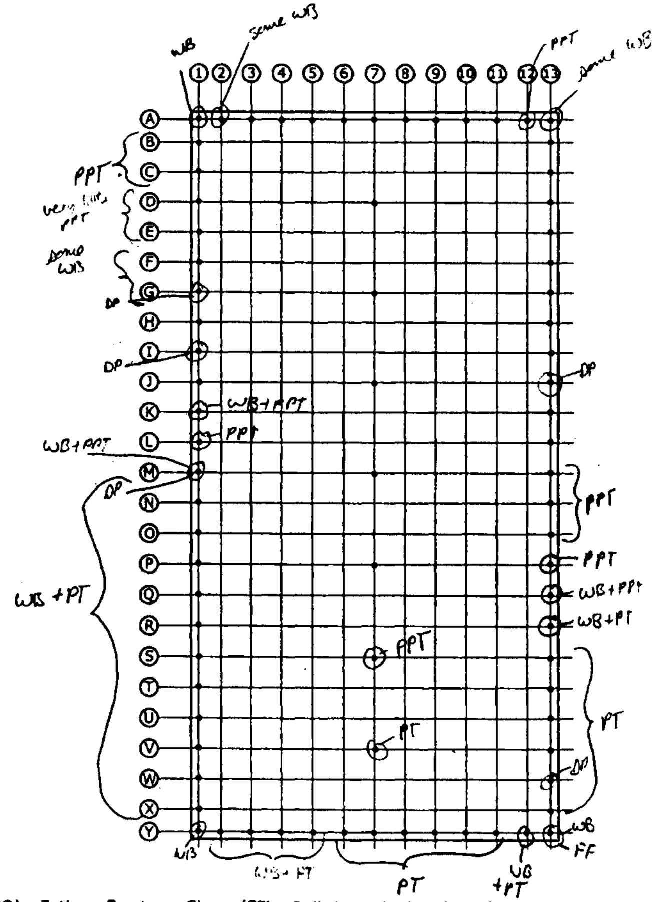
	Negative	Positive	Units
F_u	-18.71	19.76	kN
$F_{0.8u}$	-14.97	15.81	kN
$F_{0.4u}$	-7.49	7.90	kN
F_y	-17.49	18.33	kN
K_e	2.27	2.20	kN/mm
Ductility (μ)	6.83	6.33	-
$\Delta_{net,y}$	-7.71	8.35	mm
$\Delta_{net,u}$	-30.28	42.73	mm
$\Delta_{net,0.8u}$	-52.70	52.90	mm
$\Delta_{net,0.4u}$	-3.30	3.60	mm
$Area_{Backbone}$	854.51	893.23	kN-mm
$Area_{EEEP}$	854.51	893.23	kN-mm
Check	OK	OK	
R_d	**3.56**	**3.42**	-
S_y	**-14.35**	**15.04**	kN/m

McGill

Light Gauge Steel Frame / Wood Panel Shear Wall Testing

Test name : 44C
Date tested : [illegible]
Wall Size : 4' x 8'
Screw pattern : 4" / 12"
Edge Distance : 1/2"
Test mode : ☒ Cyclic ☐ Monotonic

MAX LOAD 19958 N



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) ; No Damage (ND)

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 44C

RESEARCHER: Caroline Blais **ASSISTANTS:** A. Frattini

DATE: 21-Jun-04 **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")
- [ ] Plywood (CSA O151M) CSP 9mm (3/8")
- [X] OSB (CSA 0325) 9mm (3/8") — Grant Forest Products
- [ ] Other — MFR:

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.9 gauge 1.0" self-piercing Bugle head (HD = near hold-down (1 screw 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" [X] 4"/12" [ ] 6"/12" [ ] Other:

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

STUDS:
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip Thickness: 0.043" (1.09 mm) 33ksi (230 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] T=0.043" (1.09 mm) 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
- [X] Cyclic

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] Panel She
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral
- [X] East Frame Brace
- [X] West Frame Brace
- [X] Sheathing at Corners of Panels
- TOTAL: [ ]

MOISTURE CONTENT OF SHEATHING: Moisture Meter
Wood:
Temp.: C
Nth | Sth
AVG: #DIV/0!

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

	North	South		
Ww=	24.98	25.69		
Wd=	23.89	24.58		
m.c.=	4.56	4.52		

AVG m.c. 4.54

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

COMMENTS:
- -Shear anchors torqued for 10 s with impact wrench
- -North hold down anchor 1/2 turn from finger tight, south 1/2 turn (load cells used on both hold-downs)
- -Ambient temperature 18 C
- -Double chord studs used
- -Square plate washers (2.5"x2.5") + circular ones in both corners of top track
- -Initial load set to zero at beginning of test, displacement 0.366 mm

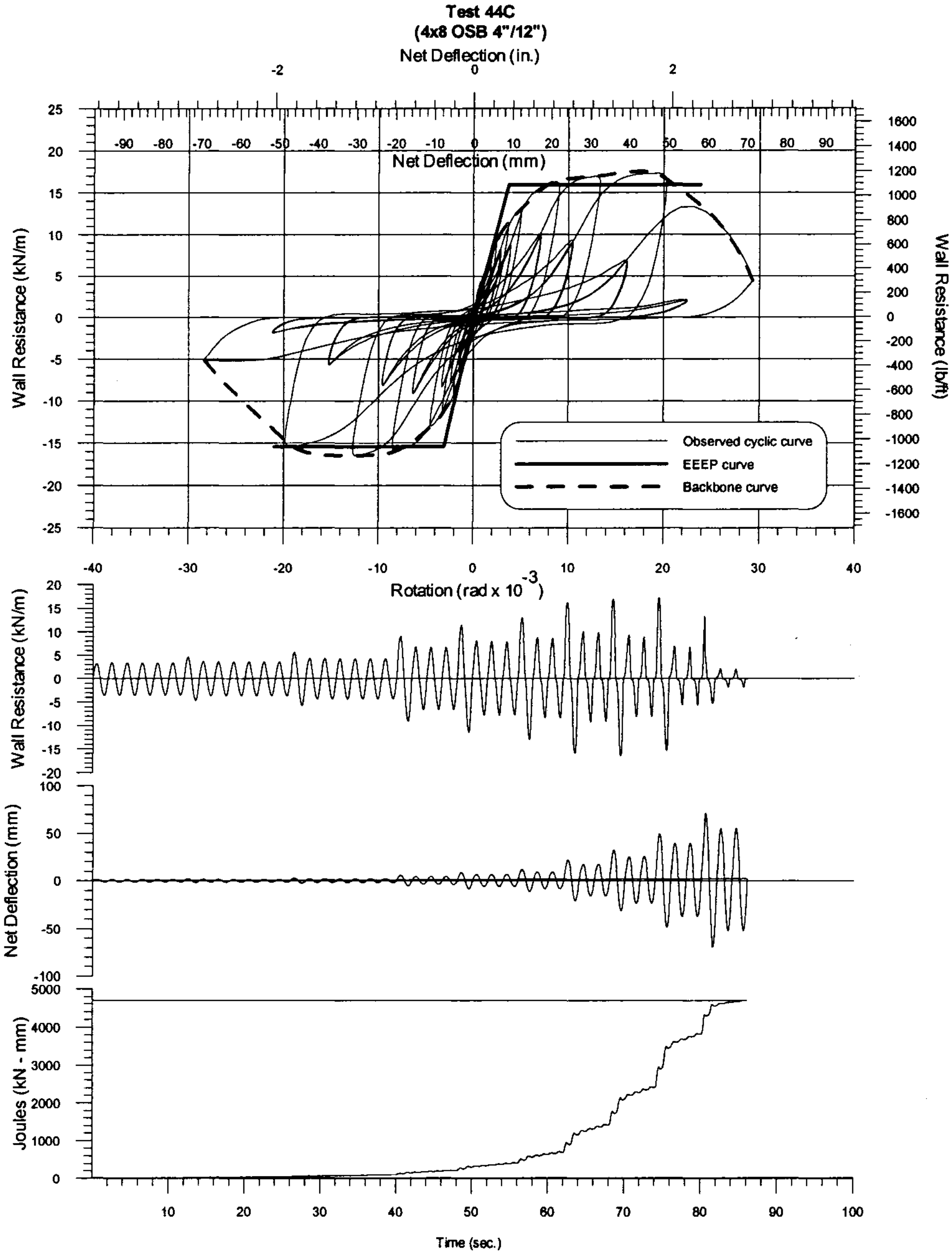


Figure B.15: Cyclic, EEEP and backbone curves for test 44-C

Table B.15: Parameters for cyclic test 44-C

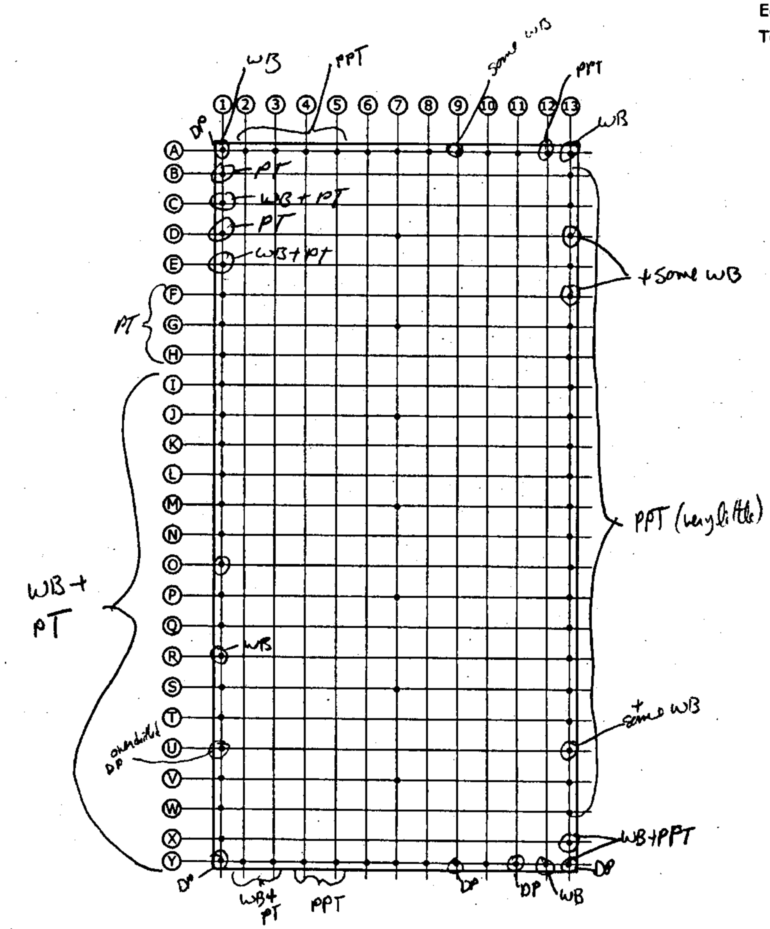
	Negative	Positive	Units
F_u	-20.06	21.05	kN
$F_{0.8u}$	-16.05	16.84	kN
$F_{0.4u}$	-8.02	8.42	kN
F_y	-18.85	19.46	kN
K_e	2.51	2.16	kN/mm
Ductility (μ)	6.81	6.45	-
$\Delta_{net,y}$	-7.52	9.01	mm
$\Delta_{net,u}$	-29.77	46.60	mm
$\Delta_{net,0.8u}$	-51.20	58.10	mm
$\Delta_{net,0.4u}$	-3.20	3.90	mm
$Area_{Backbone}$	894.43	1042.82	kN-mm
$Area_{EEEP}$	894.43	1042.82	kN-mm
Check	OK	OK	
R_d	**3.55**	**3.45**	-
S_y	**-15.46**	**15.96**	kN/m

McGill

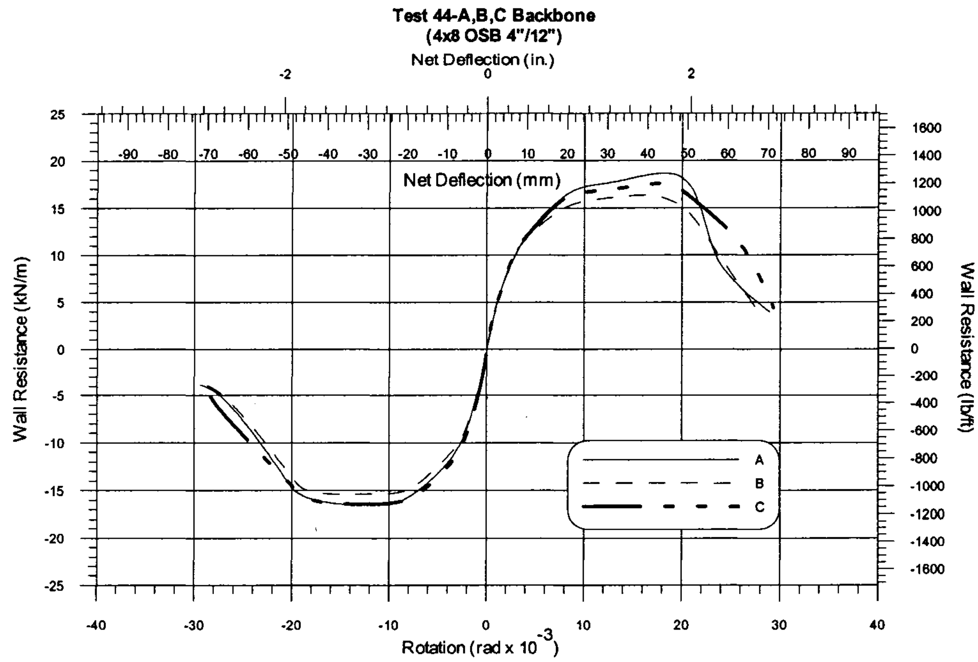
Light Gauge Steel Frame / Wood Panel Shear Wall Testing

Test name : 44C
Date tested : June 21, 2004
Wall Size : 4' x 8'
Screw pattern : 4" / 12"
Edge Distance : 1/2"
Test mode : ☒ Cyclic ☐ Monotonic

MAX LOAD : 21315N



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) ; No Damage (ND)



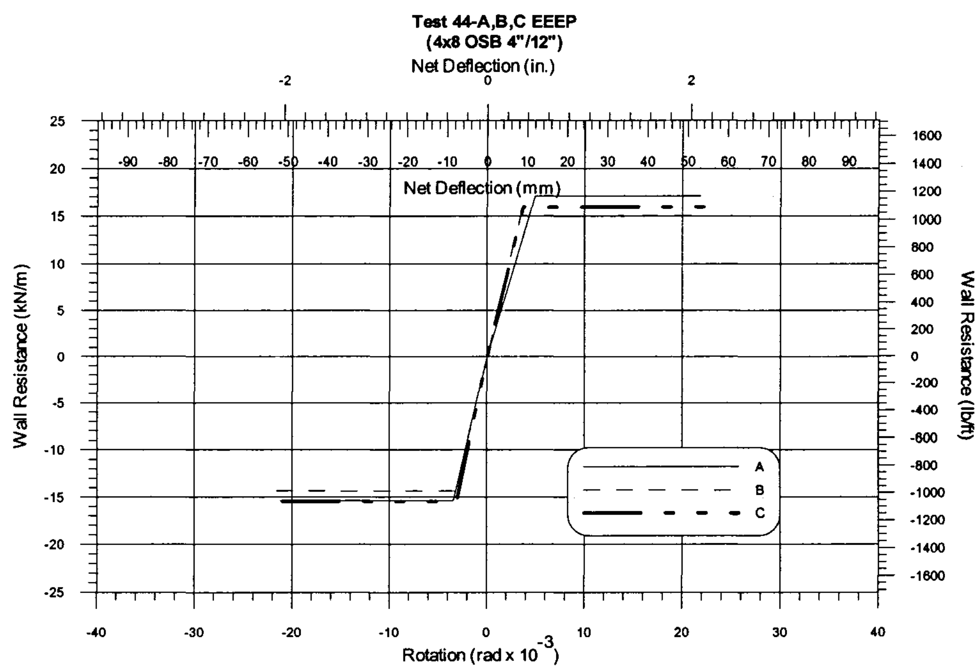


Figure B.3: Superposition of backbone and EEEP curves for tests 44-A,B,C

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 46A

RESEARCHER: Caroline Blais **ASSISTANTS:** A. Frattini

DATE: 17-Jun-04 **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")
- [ ] Plywood (CSA O151M) CSP 9mm (3/8")
- [X] OSB (CSA O325) 9mm (3/8") — Grant Forest Products
- [ ] Other — MFR:

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.9 gauge 1.0" self-piercing Bugle head (HD = near hold-down (1 screw 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:
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip Thickness: 0.043" (1.09 mm) 33ksi (230 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 [X] T=0.043" (1.09 mm) 33ksi (230 Mpa)
- Flange: 1-1/4" inches [ ] 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
- [X] Cyclic

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] Sheathing at Corners of Panels
- [X] Panel She
- TOTAL: [ ]

MOISTURE CONTENT OF SHEATHING: Moisture Meter
Wood:
Temp.: C
Nth Sth
AVG: #DIV/0!

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

	North	South		
Ww=	25.71	25.88		
Wd=	24.44	24.59		
m.c.=	5.20	5.25		

AVG m.c. 5.22

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

COMMENTS:
-Shear anchors torqued for 10 s with impact wrench
-North hold down anchor 1/2 turn from finger tight, south 1/2 turn
(load cells used on both hold-downs)
-Ambient temperature 18 C
-Double chord studs used
-Square plate washers (2.5"x2.5") + circular ones in both corners of top track
-Initial load set to zero at beginning of test, displacement 0.139 mm

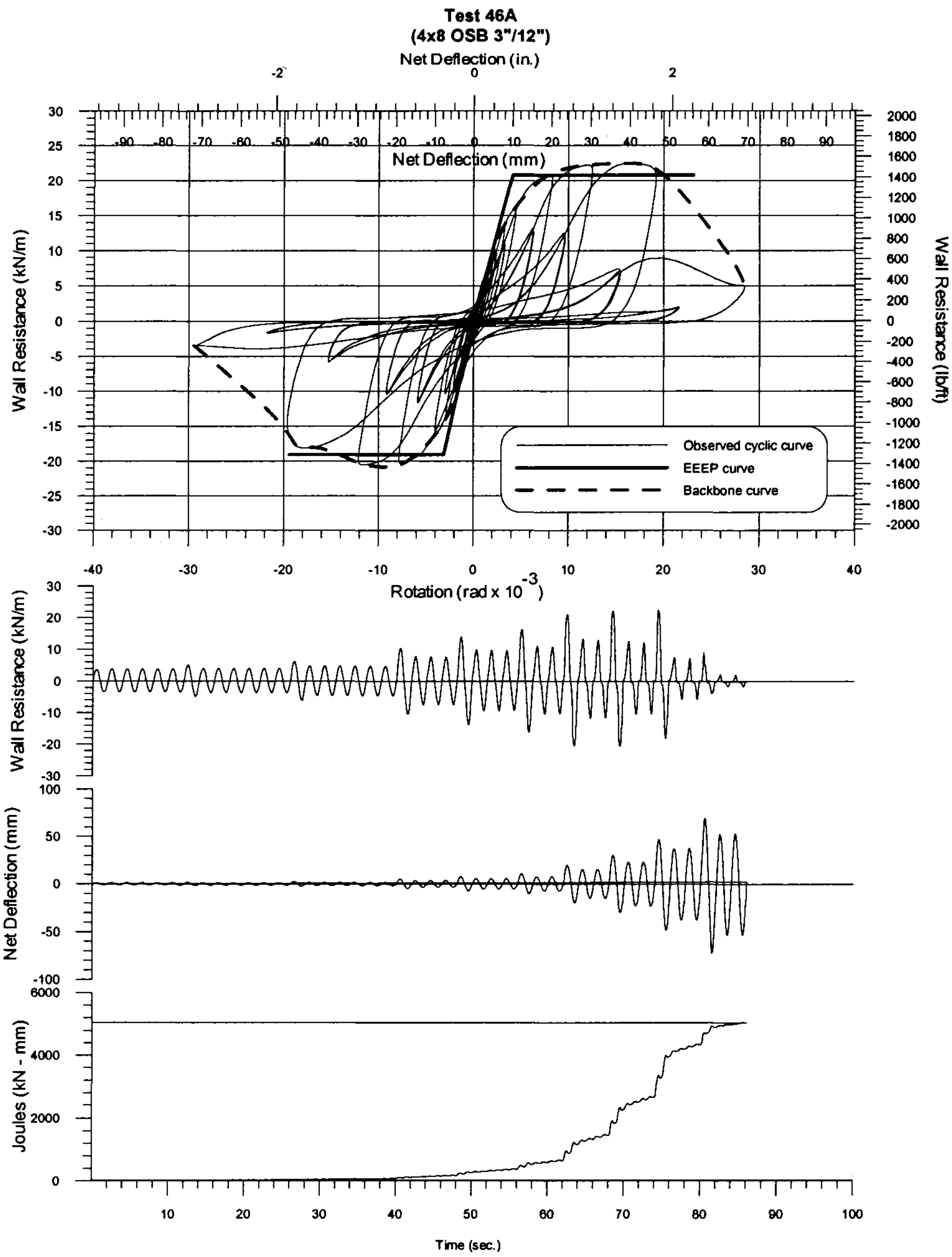


Figure B.16: Cyclic, EEEP and backbone curves for test 46-A

Table B.16: Parameters for cyclic test 46-A

	Negative	Positive	Units
F_u	-24.96	27.35	kN
$F_{0.8u}$	-19.97	21.88	kN
$F_{0.4u}$	-9.99	10.94	kN
F_y	-23.25	25.36	kN
K_e	3.03	2.54	kN/mm
Ductility (μ)	6.17	5.64	-
$\Delta_{net,y}$	-7.69	9.97	mm
$\Delta_{net,u}$	-28.26	40.03	mm
$\Delta_{net,0.8u}$	-47.40	56.20	mm
$\Delta_{net,0.4u}$	-3.30	4.30	mm
$Area_{Backbone}$	1012.90	1299.07	kN-mm
$Area_{EEEP}$	1012.90	1299.07	kN-mm
Check	OK	OK	
R_d	**3.37**	**3.21**	-
S_y	**-19.07**	**20.80**	kN/m

McGill

Light Gauge Steel Frame / Wood Panel Shear Wall Testing

Test name : 46 A

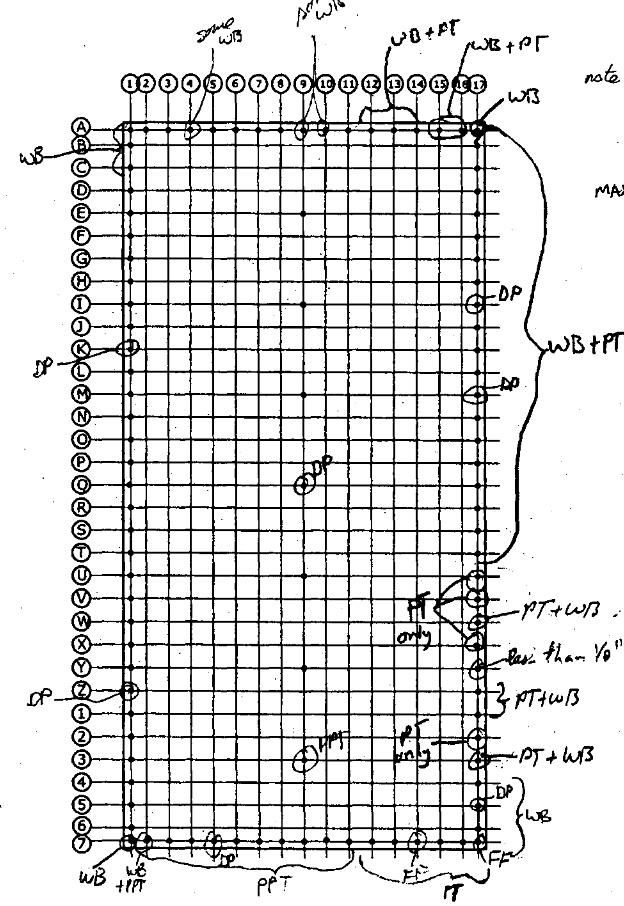
Date tested : June 17, 2004

Wall Size : 4' x 8'

Screw pattern : 3" / 12"

Edge Distance : 1/2"

Test mode : ☒ Cyclic ☐ Monotonic



note ⇒ Plywood slightly touched the beam @ the second shear bolt from the left. (TOP)

MAX LOAD ⇒ 27478N

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)

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 46B

RESEARCHER: Caroline Blais **ASSISTANTS:** A. Frattini

DATE: 17-Jun-04 **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")
[ ] Plywood (CSA 0151M) CSP 9mm (3/8")
[X] OSB (CSA 0325) 9mm (3/8") Grant Forest Products
[ ] Other MFR:

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.9 gauge 1.0" self-piercing Bugle head (HD = near hold-down (1 screw 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:
[X] 3-5/8"Wx1-5/8"Fx1/2"Lip Thickness: 0.043" (1.09 mm) 33ksi (230 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 [X] T=0.043" (1.09 mm) 33ksi (230 Mpa)
Flange: 1-1/4" inches [ ] 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
[X] Cyclic

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] Sheathing at Corners of Panels
[X] Panel She		**TOTAL:** [ ]

MOISTURE CONTENT OF SHEATHING: Moisture Meter
Wood:
Temp.: C
Nth Sth
AVG: #DIV/0!

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

	North	South		
Ww=	24.06	19.36		
Wd=	22.91	18.42		
m.c.=	5.02	5.10		

AVG m.c. 5.06

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

COMMENTS:
-Shear anchors torqued for 10 s with impact wrench
-North hold down anchor 1/2 turn from finger tight, south 1/2 turn
(load cells used on both hold-downs)
-Ambient temperature 18 C
-Double chord studs used
-Square plate washers (2.5"x2.5") + circular ones in both corners of top track
-Initial load set to zero at beginning of test, displacement 0.756 mm

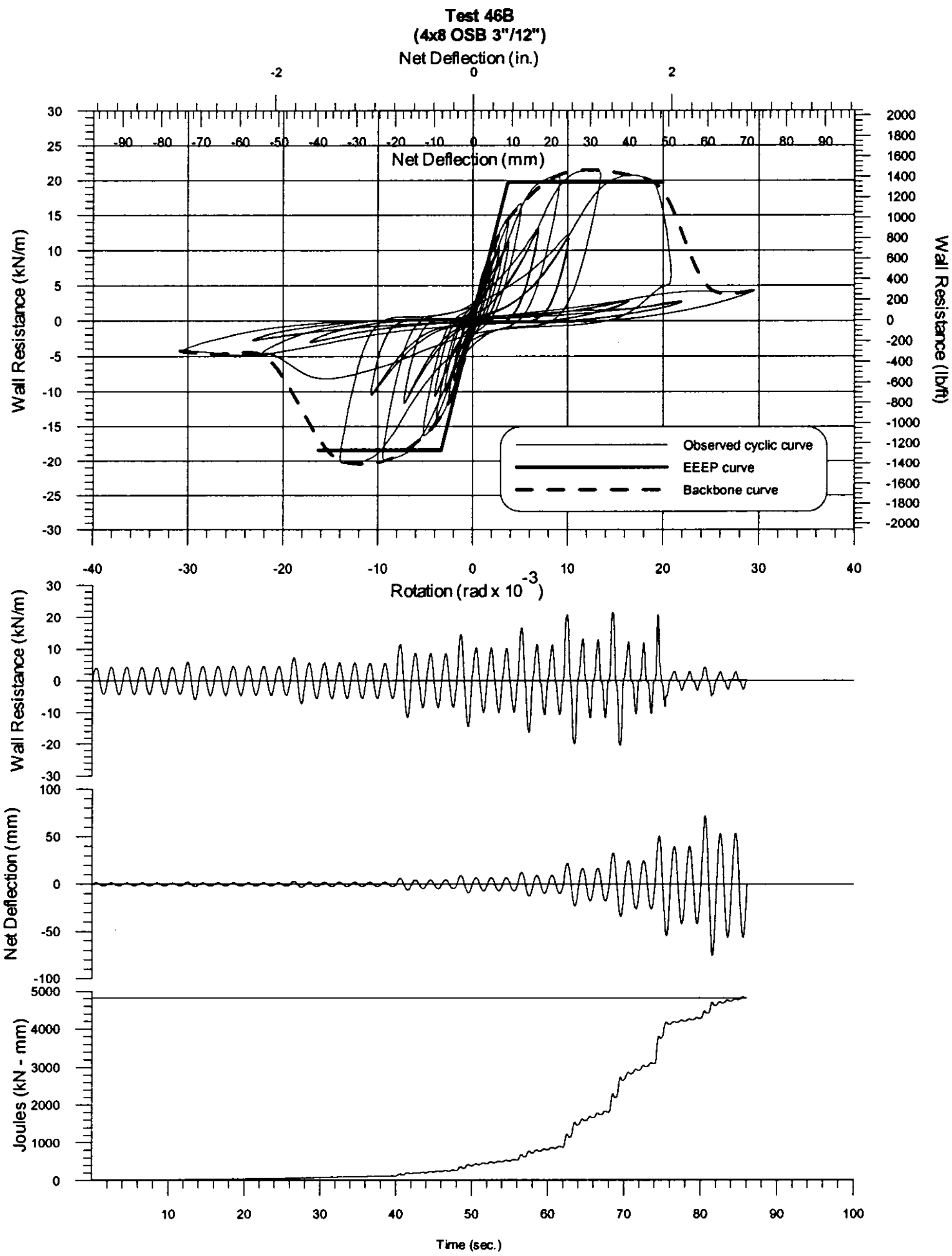


Figure B.17: Cyclic, EEEP and backbone curves for test 46-B

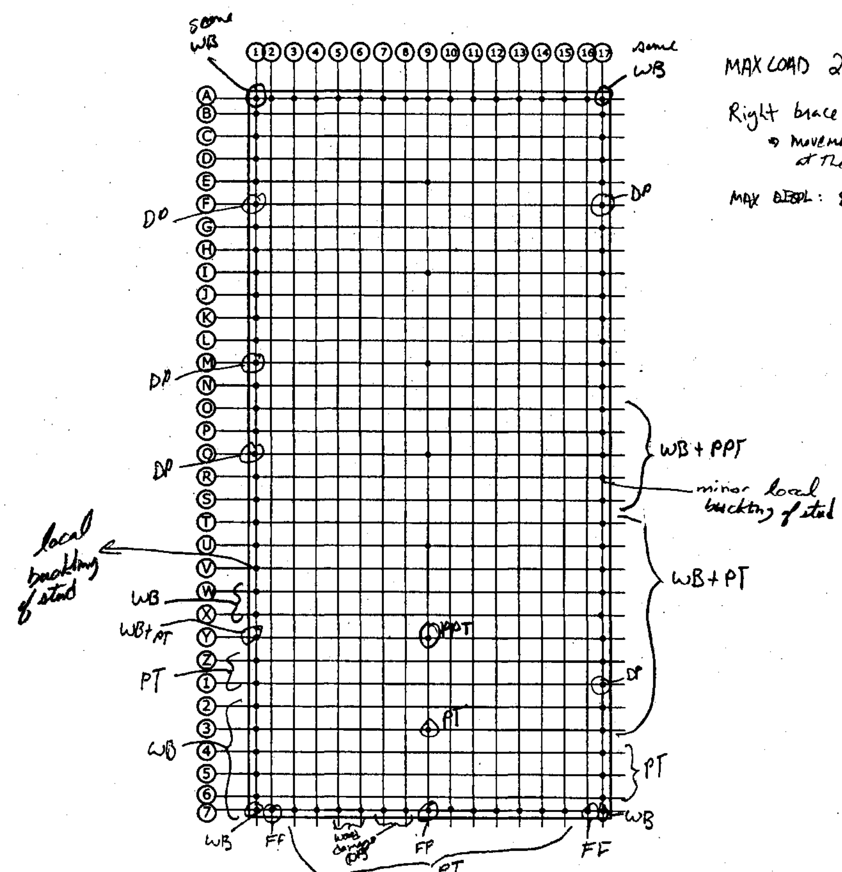
Table B.17: Parameters for cyclic test 46-B

	Negative	Positive	Units
F_u	-24.81	26.18	kN
$F_{0.8u}$	-19.85	20.95	kN
$F_{0.4u}$	-9.92	10.47	kN
F_y	-22.45	24.02	kN
K_e	2.76	2.69	kN/mm
Ductility (μ)	4.89	5.43	-
$\Delta_{net,y}$	-8.14	8.94	mm
$\Delta_{net,u}$	-32.37	31.14	mm
$\Delta_{net,0.8u}$	-39.80	48.60	mm
$\Delta_{net,0.4u}$	-3.60	3.90	mm
$Area_{Backbone}$	802.03	1059.93	kN-mm
$Area_{EEEP}$	802.03	1059.93	kN-mm
Check	OK	OK	
R_d	**2.96**	**3.14**	-
S_y	**-18.41**	**19.70**	kN/m

McGill

Light Gauge Steel Frame / Wood Panel Shear Wall Testing

Test name : 46B
Date tested : June 17, 2004
Wall Size : 4' x 8'
Screw pattern : 3" / 12"
Edge Distance : 1/2"
Test mode : ☒ Cyclic ☐ Monotonic



MAX LOAD 26 389 N

Right brace wasn't tight
⇒ movement of wall (back and forth) at the top

MAX DISPL : 80.856 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)

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 46C

RESEARCHER: Caroline Blais **ASSISTANTS:** A. Frattini

DATE: 20-Jun-04 **TIME:** 15:30

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)
- [ ] Plywood (CSA 0151M) CSP 12.5mm (1/2")
- [ ] Plywood (CSA 0151M) CSP 9mm (3/8")
- [X] OSB (CSA 0325) 9mm (3/8") — Grant Forest Products
- [ ] Other — MFR:

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.9 gauge 1.0" self-piercing Bugle head (HD = near hold-down (1 screw 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:
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip Thickness: 0.043" (1.09 mm) 33ksi (230 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 — [X] T=0.043" (1.09 mm) 33ksi (230 Mpa)
- Flange: 1-1/4" inches — [ ] 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
- [X] Cyclic

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] Sheathing at Corners of Panels
[X] Panel She		TOTAL: [ ]

MOISTURE CONTENT OF SHEATHING: Moisture Meter
Wood:
Temp.: C
Nth / Sth
AVG: #DIV/0!

OVEN DRIED ACCORDING TO APA TEST METHOD P-6

	North	South		
Ww=	26.06	24.98		
Wd=	24.80	23.75		
m.c.=	5.08	5.18		

AVG m.c. 5.13

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

COMMENTS:
-Shear anchors torqued for 10 s with impact wrench
-North hold down anchor 1/2 turn from finger tight, south 1/2 turn
(load cells used on both hold-downs)
-Ambient temperature 20 C
-Double chord studs used
-Square plate washers (2.5"x2.5") + circular ones in both corners of top track
-Initial load set to zero at beginning of test, displacement -0.879 mm

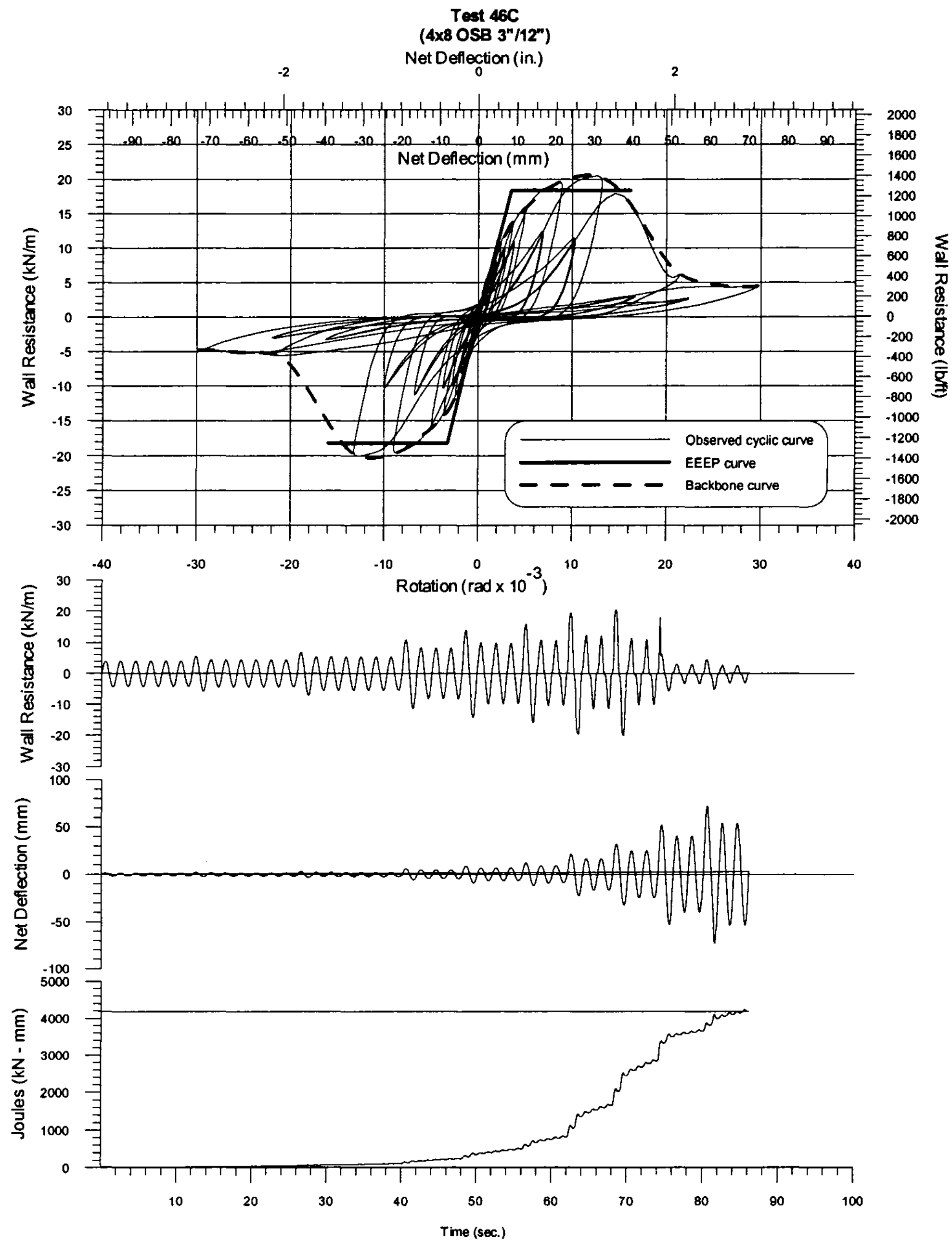


Figure B.18: Cyclic, EEEP and backbone curves for test 46-C

Table B.18: Parameters for cyclic test 46-C

	Negative	Positive	Units
F_u	-24.43	24.95	kN
$F_{0.8u}$	-19.55	19.96	kN
$F_{0.4u}$	-9.77	9.98	kN
F_y	-22.16	22.41	kN
K_e	2.79	2.63	kN/mm
Ductility (μ)	4.91	4.62	-
$\Delta_{net,y}$	-7.94	8.53	mm
$\Delta_{net,u}$	-31.08	30.95	mm
$\Delta_{net,0.8u}$	-39.00	39.40	mm
$\Delta_{net,0.4u}$	-3.50	3.80	mm
$Area_{Backbone}$	776.26	787.21	kN-mm
$Area_{EEEP}$	776.26	787.21	kN-mm
Check	OK	OK	
R_d	**2.97**	**2.87**	-
S_y	**-18.17**	**18.38**	kN/m

McGill

Light Gauge Steel Frame / Wood Panel Shear Wall Testing

Test name : 46C

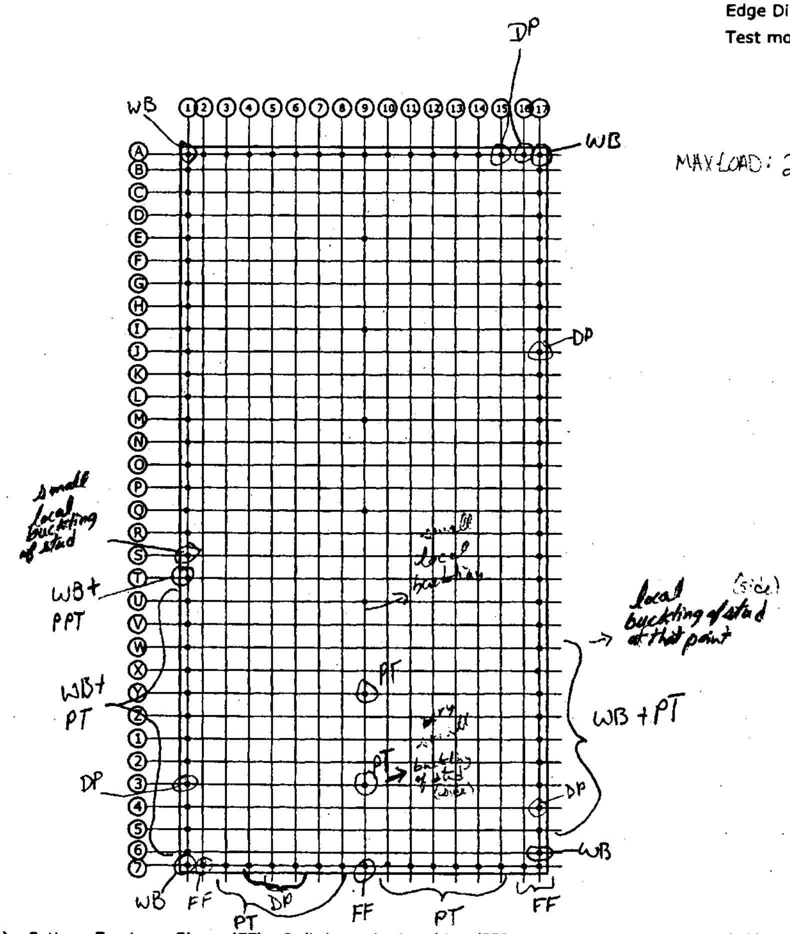
Date tested : June 16, 2004

Wall Size : 4' x 8'

Screw pattern : 3" / 12"

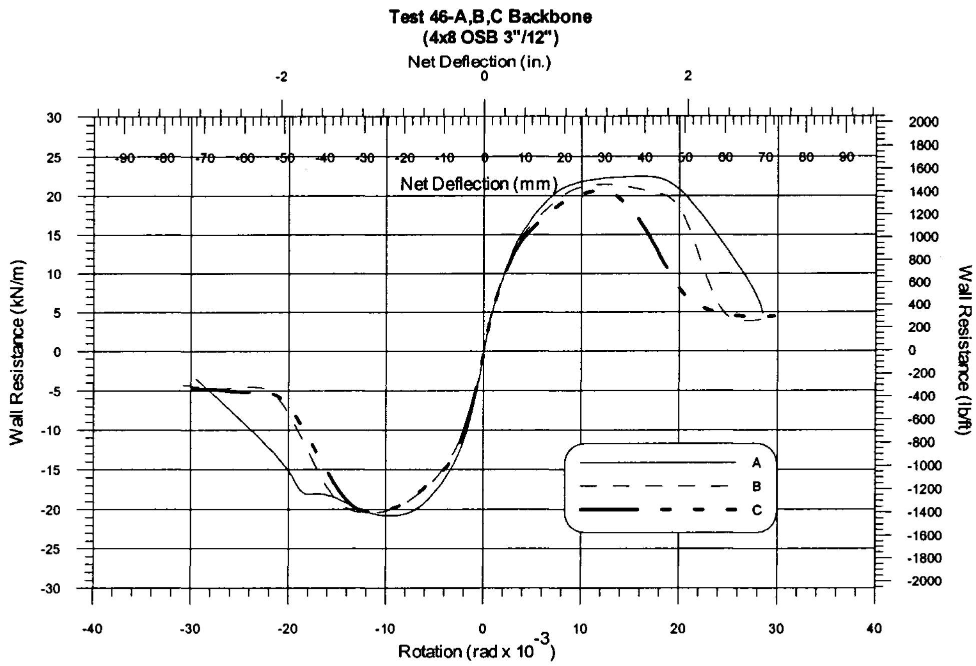
Edge Distance : 1/2"

Test mode : ☒ Cyclic ☐ Monotonic



MAX LOAD: 25210 N bottom fasteners gave up & then the left side

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)



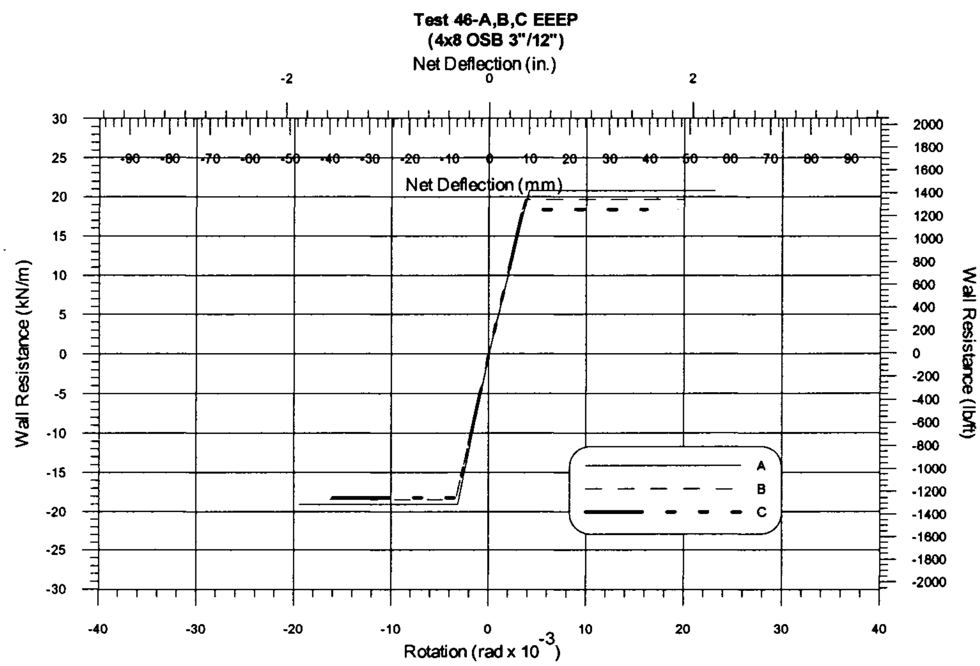


Figure B.3: Superposition of backbone and EEEP curves for tests 46-A,B,C

APPENDIX 'C'
STEWART MODEL

Monotonic Tests:

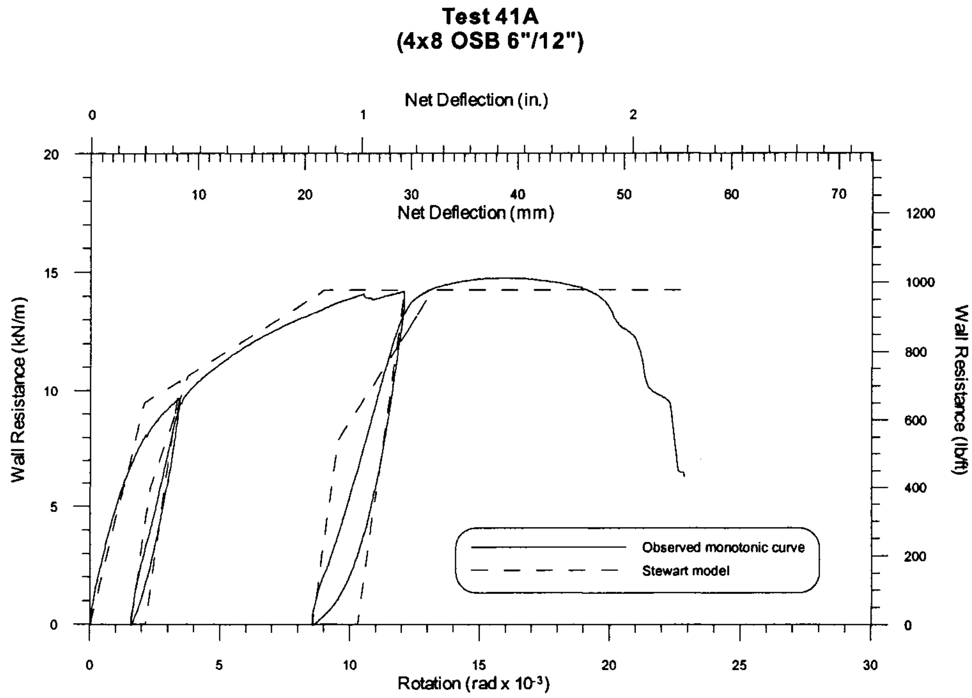


Figure C.1: Superposition of Stewart model and experimental monotonic curve-Test 41A

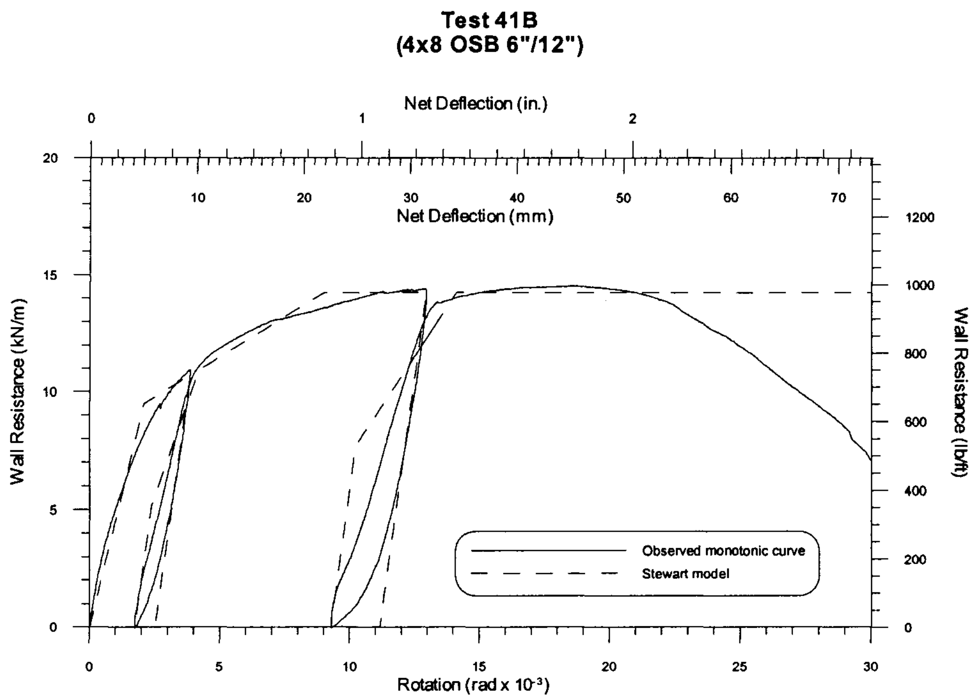


Figure C.2: Superposition of Stewart model and experimental monotonic curve-Test 41B

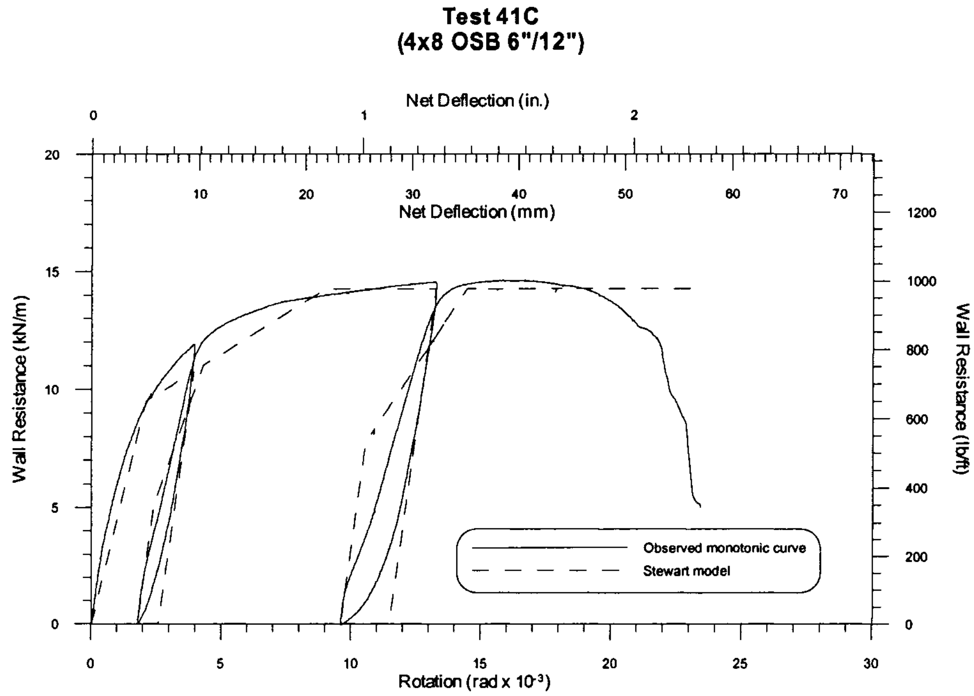


Figure C.3: Superposition of Stewart model and experimental monotonic curve-Test 41C

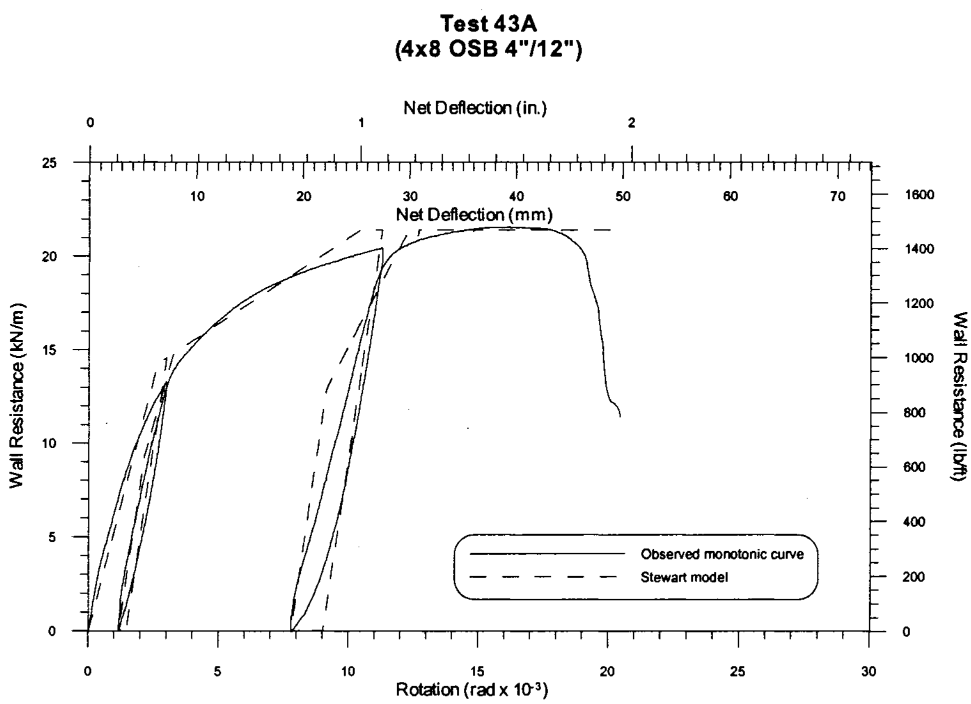


Figure C.4: Superposition of Stewart model and experimental monotonic curve-Test 43A

Test 43B
(4x8 OSB 4"/12")

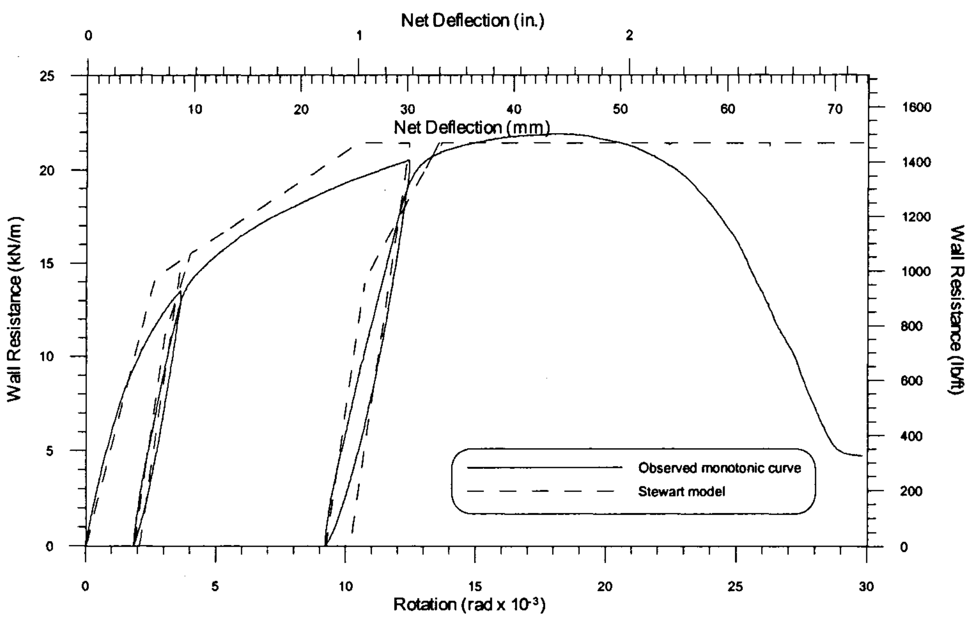


Figure C.5: Superposition of Stewart model and experimental monotonic curve-Test 43B

Test 43C
(4x8 OSB 4"/12")

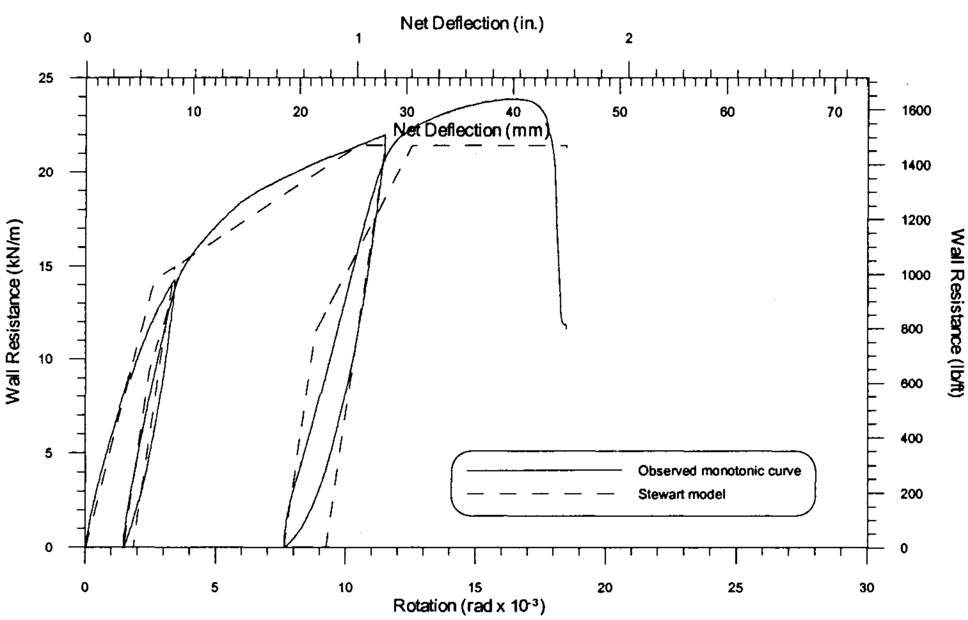


Figure C.6: Superposition of Stewart model and experimental monotonic curve-Test 43C

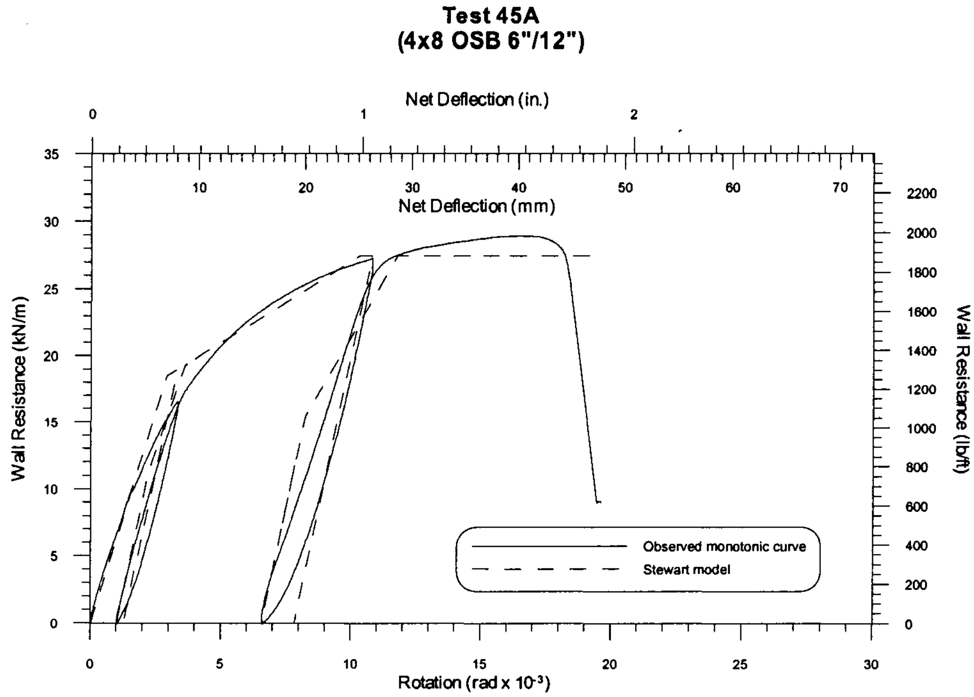


Figure C.7: Superposition of Stewart model and experimental monotonic curve-Test 45A

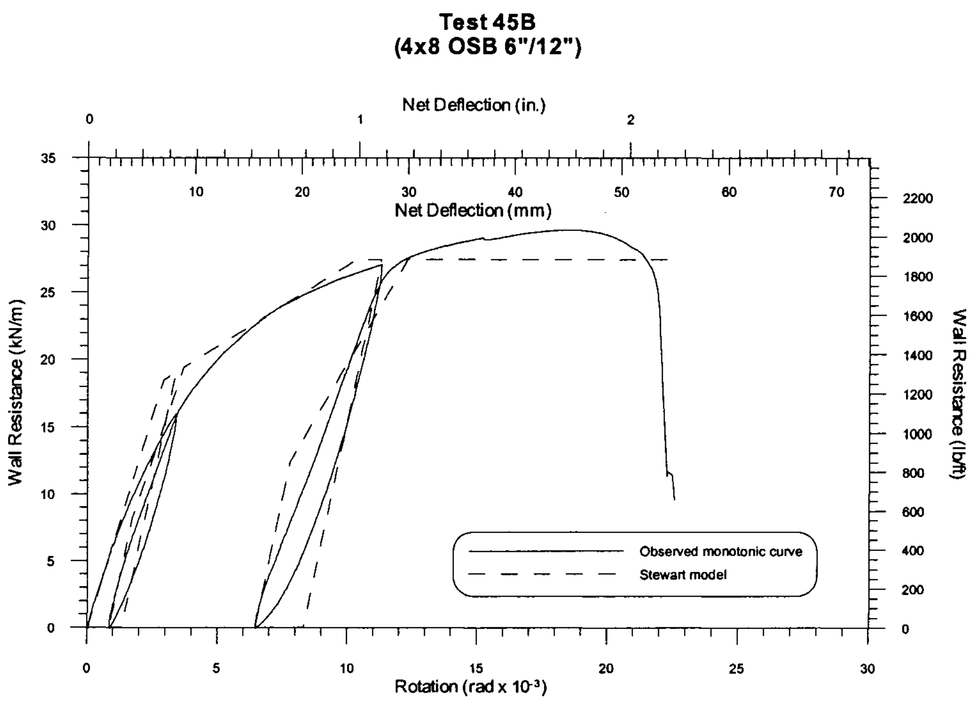


Figure C.8: Superposition of Stewart model and experimental monotonic curve-Test 45B

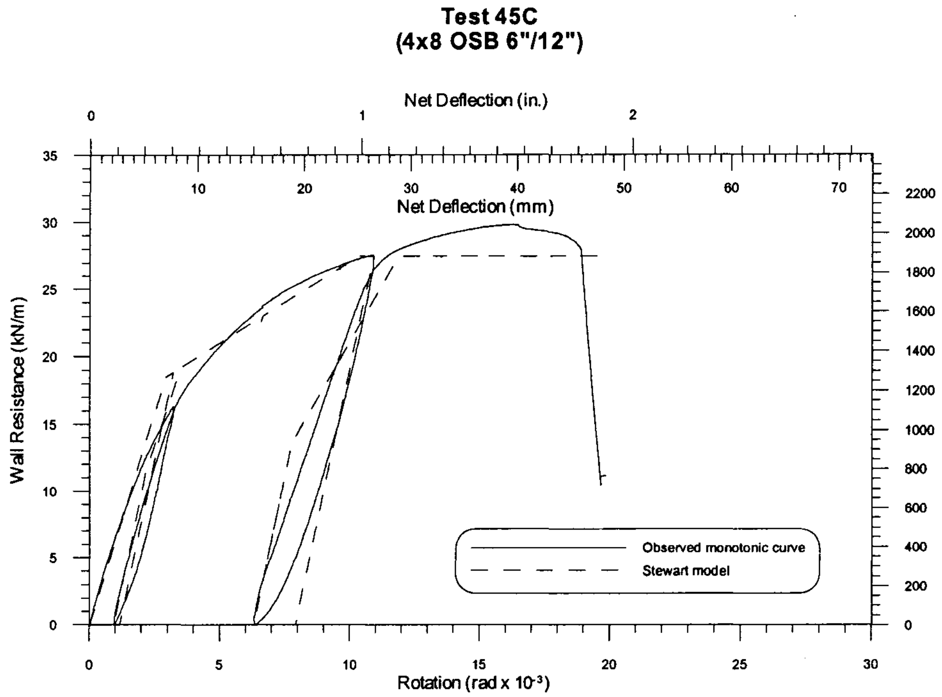


Figure C.9: Superposition of Stewart model and experimental monotonic curve-Test 45C

<u>Cyclic Tests:</u>

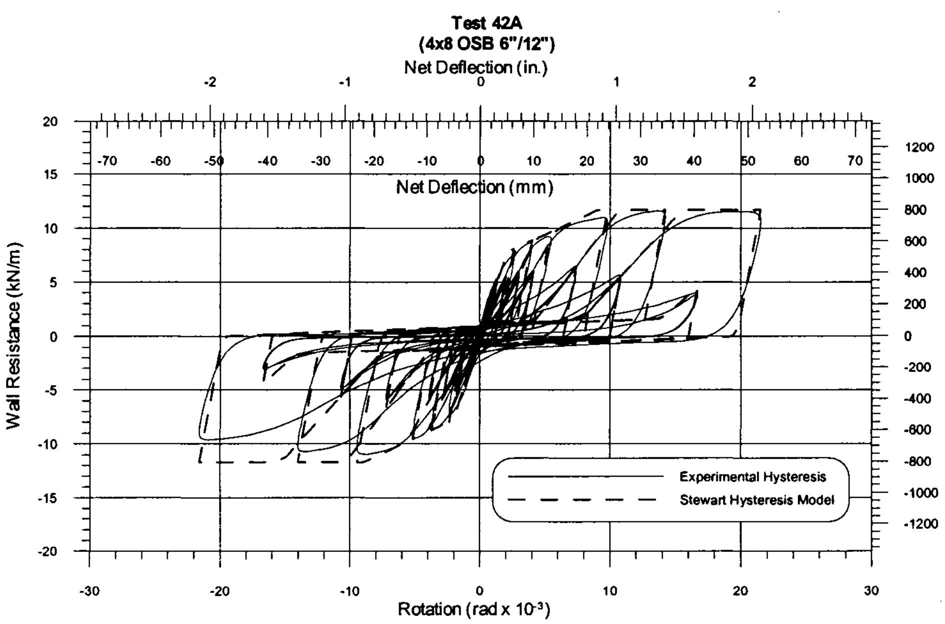


Figure C.10: Superposition of Stewart model and experimental cyclic hysteresis-Test 42A

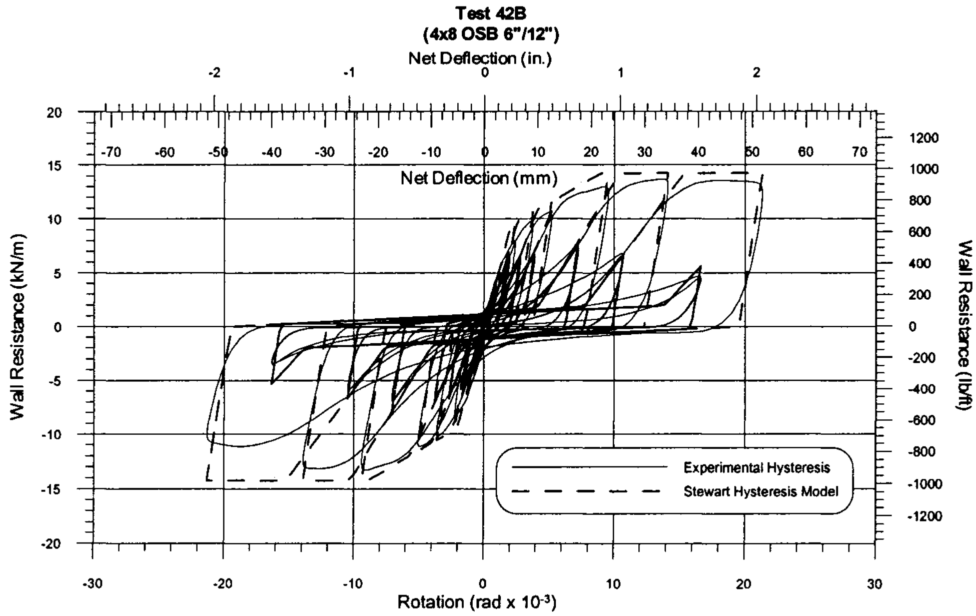


Figure C.11: Superposition of Stewart model and experimental cyclic hysteresis-Test 42A

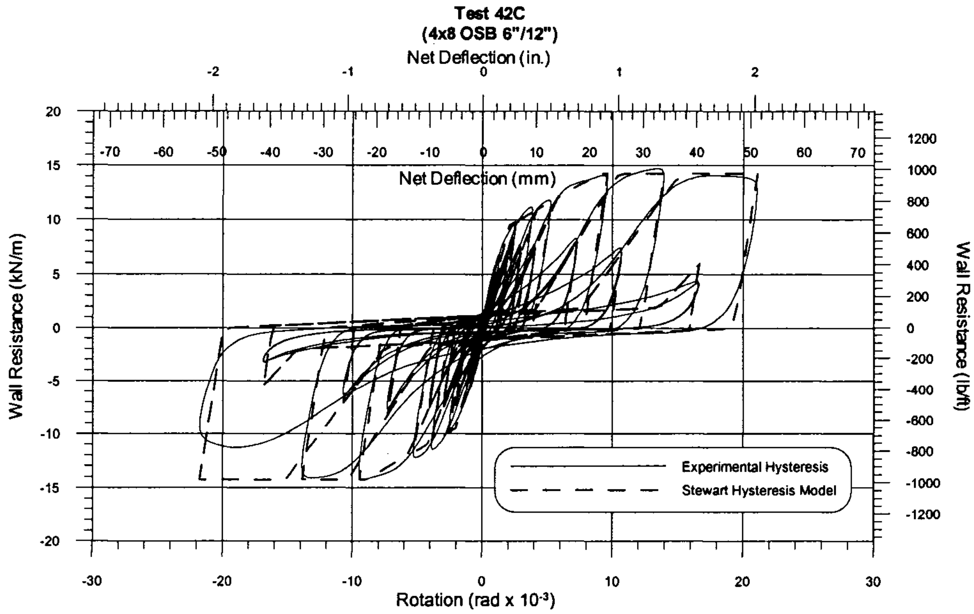


Figure C.12: Superposition of Stewart model and experimental cyclic hysteresis-Test 42C

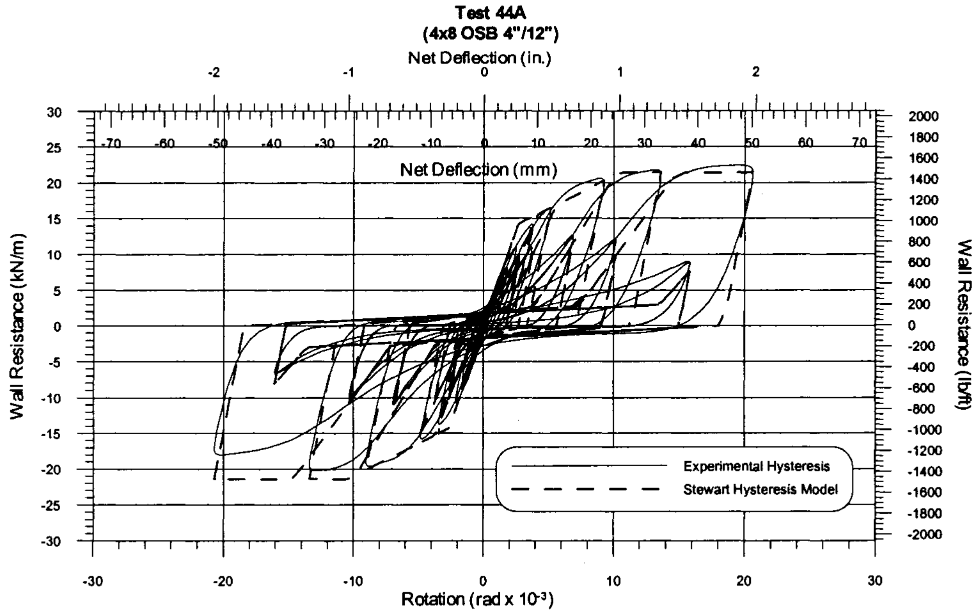


Figure C.13: Superposition of Stewart model and experimental cyclic hysteresis-Test 44A

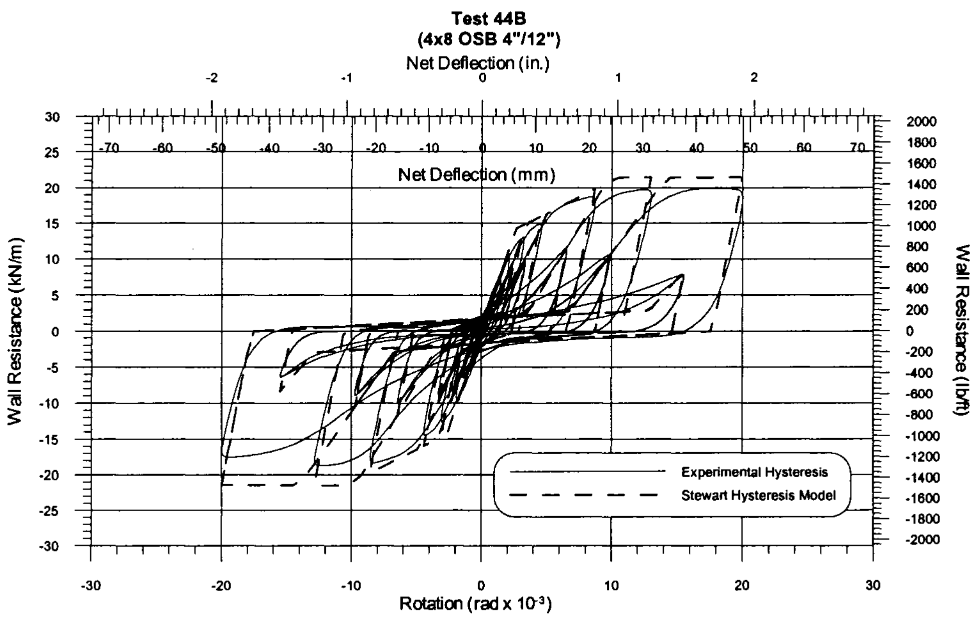


Figure C.14: Superposition of Stewart model and experimental cyclic hysteresis-Test 44B

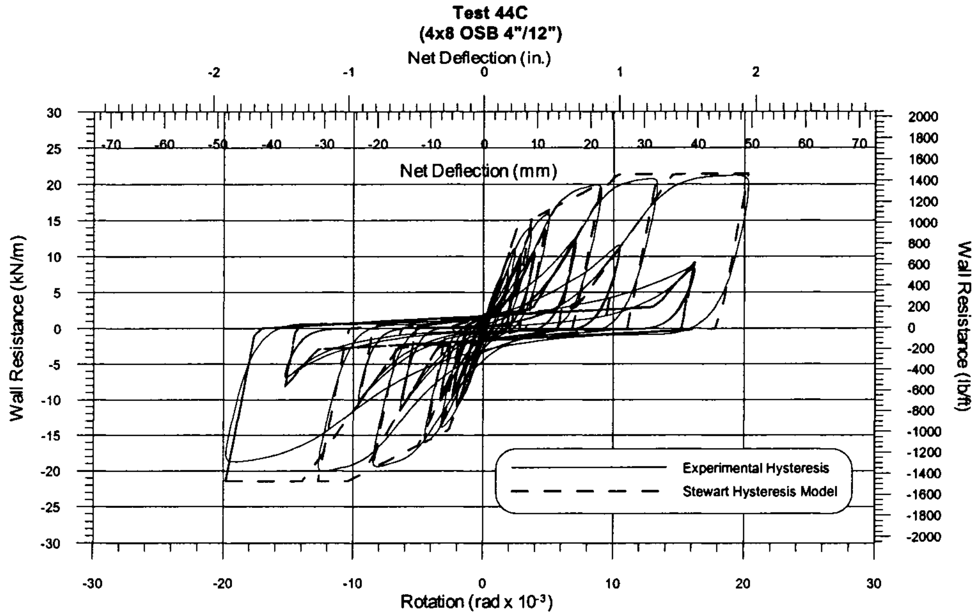


Figure C.15: Superposition of Stewart model and experimental cyclic hysteresis-Test 44C

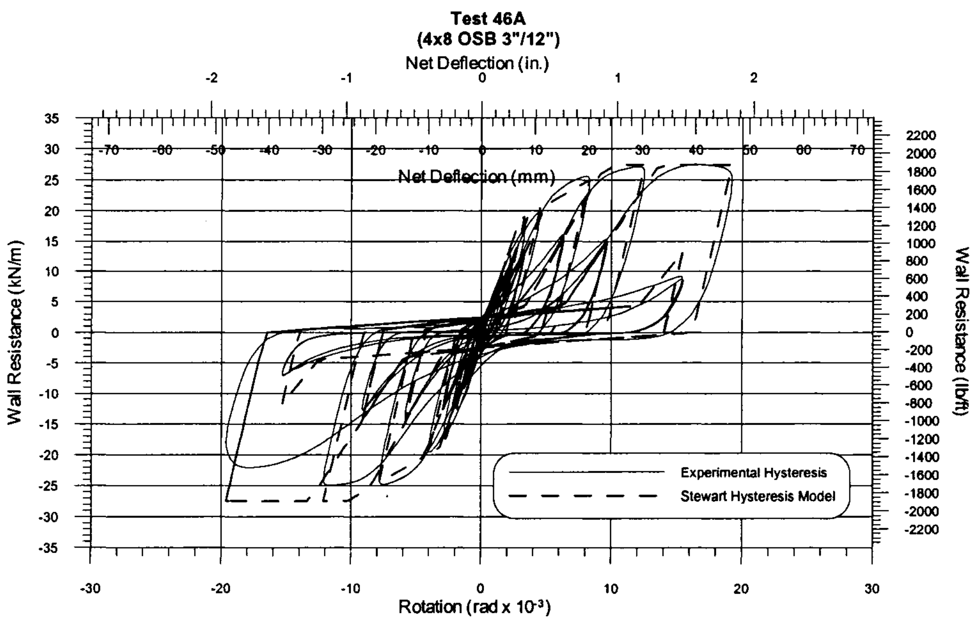


Figure C.16: Superposition of Stewart model and experimental cyclic hysteresis-Test 46A

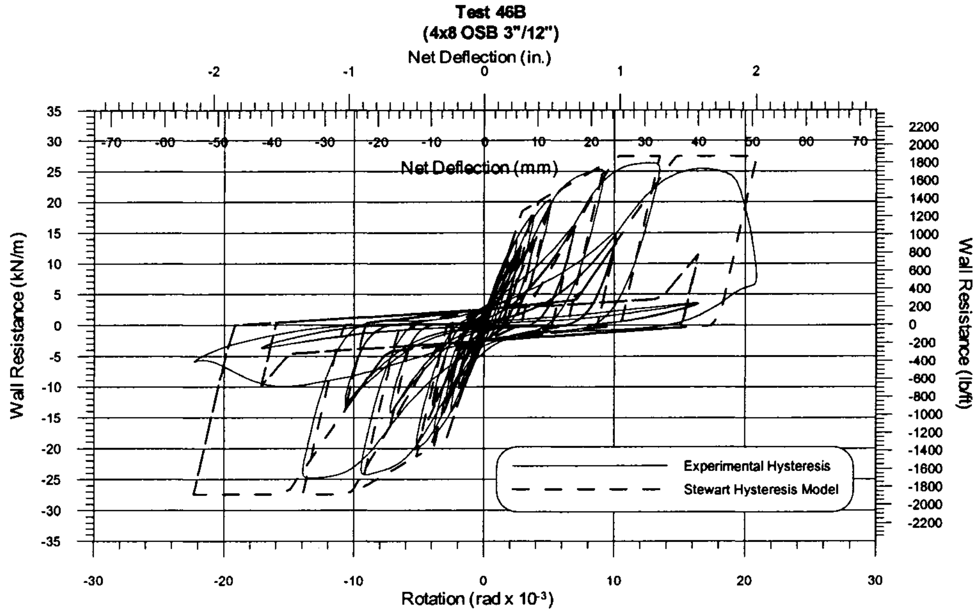


Figure C.17: Superposition of Stewart model and experimental cyclic hysteresis-Test 46B

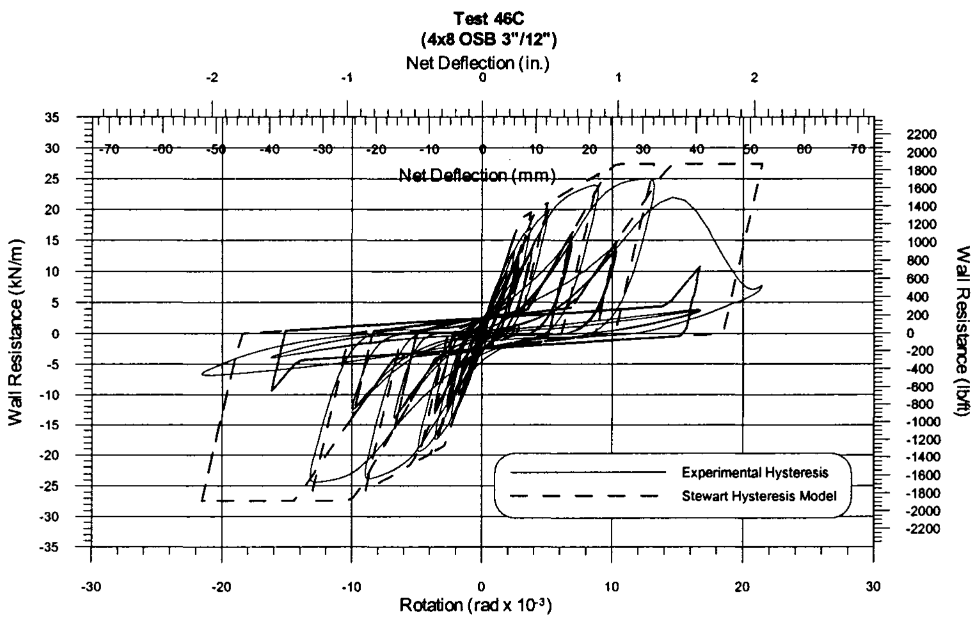


Figure C.18: Superposition of Stewart model and experimental cyclic hysteresis-Test 46C

APPENDIX 'D'
BASE SHEAR CALCULATION

Base Shear Calculation for Two-Storey Model

According to the Canadian Home Builder's Association (CHBA), the average Canadian house size is about 167.2m^2 (1800ft^2) which is equivalent to a 7.2 m per 11.6 m two-storey house (83.6m^2/storey).

Base shear calculation for a two-storey house in Vancouver, BC with two shear walls per exterior wall per storey. (Following the NBCC 2005 edition)

$$V = \frac{S(T)M_v I_E W}{R_d R_o} \geq \frac{S(2.0)M_v I_E W}{R_d R_o}$$

Where,

$R_d = 2.5$ Ductility modification factor (Branston 2004)
$R_o = 1.8$ Overstrength modification factor (Boudreault 2005)
$I_E = 1.0$ Importance factor (Table 4.1.8.5, NBCC 2005)
$M_v = 1.0$ Factor for higher mode effect (Table 4.1.8.11, NBCC 2005)

$$W = \sum_{i=1}^{n=2} W_i$$

W_1 = Dead load of 1st storey

$$W_1 = 1.57kPa \times 83.6m^2$$

$$W_1 = 131.3kN$$

W_2 = Dead load of roof + 25% snow load

$$W_2 = 0.66kPa \times 83.6m^2 + 25\% \times 1.64kPa \times 83.6m^2$$

$$W_2 = 89.5kN$$

$$W = 131.3kN + 89.5kN$$

$$W = 220.8kN$$

Period of the structure – NBCC 2005 – for shear wall structures

$$T = 0.05(h_n)^{3/4}$$

$$T = 0.05(2.44m \times 2 + 0.31m)^{3/4}$$

$$T = 0.17\sec$$

For $T < 0.2$sec and a class E soil we have,

$$S(T) = F_a S_a(0.2)$$

$S_a(0.2) = 0.94$ For Vancouver, BC, NBCC 2005
$F_a = 0.948$ Table 4.1.8.4.B in NBCC 2005

$$S(0.2) = 0.948 \times 0.94$$

$$S(0.2) = 0.891$$

$$V = \frac{0.891 \times 1.0 \times 1.0 \times 220.8kN}{2.5 \times 1.8}$$
$$V = 43.7kN$$

Except that V should not be less than $\frac{S(2.0)M_V I_E W}{R_d R_o}$ and need not be taken greater than $\frac{2}{3}\frac{S(0.2)I_E W}{R_d R_o}$ for a SFRS with R_d equal to or greater than 1.5.

For $T = 2.0$ sec and a class E soil we have,

$S(T) = F_v S_a(2.0)$

$S_a(1.0) = 0.33$	For Vancouver, BC, NBCC 2005
$S_a(2.0) = 0.17$	For Vancouver, BC, NBCC 2005
$F_v = 1.840$	Table 4.1.8.4.B, NBCC 2005

$$S(2.0) = 1.840 \times 0.17$$
$$S(2.0) = 0.31$$

$$\frac{S(2.0)M_V I_E W}{R_d R_o} = \frac{0.31 \times 1.0 \times 1.0 \times 220.8kN}{2.5 \times 1.8} = 15.2kN$$

and

$$\frac{2}{3}\frac{S(0.2)I_E W}{R_d R_o} = \frac{2}{3}\frac{0.891 \times 1.0 \times 220.8kN}{2.5 \times 1.8} = 29.2kN$$

Therefore, V is equal to 29.2 kN.

However, to account for torsional effects, the base shear force (V) was applied at a distance $0.1D_n$ from the centre of mass of the storey. Using statics, the largest lateral force induced on one shear wall is found as follows:

$$V_{tor} = V \times \left(\frac{0.1D_n + 0.5D_n}{D_n}\right) \div 2 = 0.3 \times V$$

where,

V_{tor} = Base shear induces to one shear wall with torsional effects, [kN]
V = Base shear, [kN] (29.2 kN)
D_n = Plan dimension of the building perpendicular to earthquake load, [m]

Therefore,

$$V_{tor} = 0.3 \times V = 0.3 \times 29.2 = 8.76kN$$

The base shear force (with torsion) should be distributed over the height of the structure as follows:

Since $T < 0.7\text{sec}$, $F_t = 0$ and

$$F_x = \frac{(V - F_t)W_x h_x}{\sum_{i=1}^{n=2} W_i h_i}$$

$$F_1 = \frac{(8.76kN - 0kN) \times 131.3kN \times (2.44m + 0.31m)}{131.3kN \times (2.44m + 0.31m) + 89.5kN(2.44m)}$$

$$F_1 = 5.46kN$$

$$F_2 = \frac{(8.76kN - 0kN) \times 89.5kN \times 2.44m}{131.3kN \times (2.44m + 0.31m) + 89.5kN(2.44m)}$$

$$F_2 = 3.30kN$$

Maximum shear resistance developed for one shear wall on the 1st and 2nd storey respectively;

1st storey

$$S_1 = \frac{F_1 + F_2}{SWlength}$$

$$S_1 = \frac{5.46kN + 3.30kN}{1.22m}$$

$$S_1 = 7.2kN/m$$

2nd storey

$$S_2 = \frac{F_2}{SWlength}$$

$$S_2 = \frac{3.30kN}{1.22m}$$

$$S_2 = 2.7kN/m$$

Design shear resistance and stiffness of shear walls from test results

Sheathing Type	Sheathing Thickness (mm)	Fastener Schedule (mm)	$S_{y,avg}$ (kN/m)	S_r (kN/m)	$k_{e,avg}$ (kN/mm/m wall length)
			1220x2440mm (4'x8') wall	$S_r = \phi \times S_{y,avg}$ ($\phi = 0.7$)	1220x2440mm (4'x8') wall
OSB	9	152/305	10.7	7.5	1.64
OSB	9	150/305	15.9	11.1	1.82
OSB	9	75/305	20.4	14.3	2.09

The maximum shear resistances obtained using the NBCC 2005 approach (S_1 = 7.2 kN/m and S_2 = 2.7 kN/m) is smaller than the lowest design value for the shear walls tested in this body of research (S_r = 7.5 kN/m) therefore the eight shear walls that will be used in this house will be made of a 9.5 mm thick OSB panel with a screw spacing of 6 inches along the perimeter of the wall.

Deflection check

The elastic displacement, Δ_e, is the maximum displacement of a storey. Then the anticipated deflection can be calculated using R_dR_o/I_E times the elastic displacement.

Calculation for drift limitation:

1st Storey	2nd Storey
$\Delta_e = S/K_e$	
$\Delta_{e,1} = 7.2kN/m / 1.64kN/mm/m = 4.4mm$	$\Delta_{e,2} = 2.7kN/m / 1.64kN/mm/m = 1.6mm$
$\Delta_{\max} = \frac{R_dR_o}{I_E} \times \Delta_e$	
$\Delta_{\max,1} = \frac{2.5 \times 1.8}{1.0} \times 4.4mm = 19.8mm$	$\Delta_{\max,2} = \frac{2.5 \times 1.8}{1.0} \times 1.6mm = 7.2mm$

The largest interstorey deflection for a storey should be equal or less than $0.025h_s$, where h_s is the height of the storey.

$\Delta_{\max,1} \le 0.025 \times (2440 + 310)mm = 68.8mm$	$\Delta_{\max,2} \le 0.025 \times (2440)mm = 61.0mm$

Therefore a total of eight 1.22m x 2.44m shear walls per storey that is, two shear walls per exterior wall per storey, with a screw schedule of 152/305 mm (6"/12") is adequate for the modeled house.

Base Shear Calculation for Two-Storey Model

Commerce size is about 174.2m^2 (1875ft^2) which is equivalent to a 7.62 m per 7.62 m three-storey building (58.1m^2/storey).

Base shear calculation for a three-storey building in Vancouver, BC with two shear walls per exterior wall per storey. (Following the NBCC 2005 edition)

$$V = \frac{S(T)M_vI_EW}{R_dR_o} \ge \frac{S(2.0)M_vI_EW}{R_dR_o}$$

Where,

$R_d = 2.5$	Ductility modification factor (Branston 2004)
$R_o = 1.8$	Overstrength modification factor (Boudreault 2005)
$I_E = 1.0$	Importance factor (Table 4.1.8.5, NBCC 2005)

$M_v = 1.0$ Factor for higher mode effect (Table 4.1.8.11, NBCC 2005)

$$W = \sum_{i=1}^{n=2} W_i$$

W_1 = Dead load of 1st storey

$$W_1 = 2.87kPa \times 58.1m^2$$

$$W_1 = 166.7kN$$

W_2 = Dead load of 2nd storey

$$W_2 = 2.87kPa \times 58.1m^2$$

$$W_2 = 166.7kN$$

W_3 = Dead load of roof + 25% snow load

$$W_3 = 0.69kPa \times 58.1m^2 + 25\% \times 1.64kPa \times 58.1m^2$$

$$W_3 = 63.9kN$$

$$W = 166.7kN + 166.7kN + 63.9kN$$

$$W = 397.3kN$$

Period of the structure – NBCC 2005 – for shear wall structures

$$T = 0.05(h_n)^{3/4}$$

$$T = 0.05(2.74m \times 3 + 2 * 0.15m)^{3/4}$$

$$T = 0.25\sec$$

For $T < 0.2$sec and a class E soil we have,

$$S(T) = F_a S_a(0.2)$$

$S_a(0.2) = 0.94$	For Vancouver, BC, NBCC 2005
$F_a = 0.948$	Table 4.1.8.4.B in NBCC 2005

$$S(0.2) = 0.948 \times 0.94$$

$$S(0.2) = 0.891$$

For $T = 0.5$sec and a class E soil we have,

$S(T) = F_v S_a(0.5)$ or $= F_a S_a(0.2)$, whichever is smaller

$S_a(0.2) = 0.94$	For Vancouver, BC, NBCC 2005
$S_a(0.5) = 0.64$	For Vancouver, BC, NBCC 2005
$S_a(1.0) = 0.33$	For Vancouver, BC, NBCC 2005
$F_a = 0.948$	Table 4.1.8.4.B, NBCC 2005
$F_v = 1.840$	Table 4.1.8.4.B, NBCC 2005

$$S(0.5) = 1.840 \times 0.64$$

$$S(0.5) = 1.178$$

Therefore S(T) at T = 0.25s:

$$S(0.25) = 0.891$$

$$V = \frac{0.891 \times 1.0 \times 1.0 \times 397.3kN}{2.5 \times 1.8}$$
$$V = 78.7kN$$

Except that V should not be less than $\frac{S(2.0)M_V I_E W}{R_d R_o}$ and need not be taken greater than $\frac{2}{3}\frac{S(0.2)I_E W}{R_d R_o}$ for a SFRS with R_d equal to or greater than 1.5.

For $T = 2.0$ sec and a class E soil we have,

$$S(T) = F_v S_a(2.0)$$

$S_a(1.0) = 0.33$	For Vancouver, BC, NBCC 2005
$S_a(2.0) = 0.17$	For Vancouver, BC, NBCC 2005
$F_v = 1.840$	Table 4.1.8.4.B, NBCC 2005

$$S(2.0) = 1.840 \times 0.17$$
$$S(2.0) = 0.31$$

$$\frac{S(2.0)M_V I_E W}{R_d R_o} = \frac{0.31 \times 1.0 \times 1.0 \times 397.3kN}{2.5 \times 1.8} = 27.6kN$$

and

$$\frac{2}{3}\frac{S(0.2)I_E W}{R_d R_o} = \frac{2}{3}\frac{0.891 \times 1.0 \times 397.3kN}{2.5 \times 1.8} = 52.4kN$$

Therefore, V is equal to 52.4 *kN*.

However, to account for torsional effects, the base shear force (V) was applied at a distance $0.1D_n$ from the centre of mass of the storey. Using statics, the largest lateral force induced on one shear wall is found as follows:

$$V_{tor} = V \times \left(\frac{0.1D_n + 0.5D_n}{D_n}\right) \div 2 = 0.3 \times V$$

where,

V_{tor} = Base shear induces to one shear wall with torsional effects, [kN]
V = Base shear, [kN] (29.2 kN)
D_n = Plan dimension of the building perpendicular to earthquake load, [m]

Therefore,

$$V_{tor} = 0.3 \times V = 0.3 \times 52.4 = 15.72kN$$

The base shear force (with torsion) should be distributed over the height of the structure as follows:

Since $T < 0.7\text{sec}$, $F_t = 0$ and

$$F_x = \frac{(V - F_t)W_x h_x}{\sum_{i=1}^{n=2} W_i h_i}$$

$$F_1 = \frac{(15.72kN - 0kN)\times 166.7kN \times (2.74m + 0.10m)}{166.7kN(2.74+0.10)m + 166.7kN(2.74+0.15)m + 63.9kN(2.74+0.05)m}$$

$$F_1 = 6.55kN$$

$$F_2 = \frac{(15.72kN - 0kN)\times 166.7kN \times (2.74m + 0.15m)}{166.7kN(2.74+0.10)m + 166.7kN(2.74+0.15)m + 63.9kN(2.74+0.05)m}$$

$$F_2 = 6.65kN$$

$$F_3 = \frac{(15.72kN - 0kN)\times 63.9kN \times (2.74m + 0.05m)}{166.7kN(2.74+0.10)m + 166.7kN(2.74+0.15)m + 63.9kN(2.74+0.05)m}$$

$$F_3 = 2.45kN$$

Maximum shear resistance developed for one shear wall on the 1st and 2nd storey respectively;

1st storey

$$S_1 = \frac{F_1 + F_2 + F_3}{SWlength}$$

$$S_1 = \frac{(6.55 + 6.65 + 2.45)kN}{1.22m}$$

$$S_1 = 12.8kN/m$$

2nd storey

$$S_2 = \frac{F_2 + F_3}{SWlength}$$

$$S_2 = \frac{(6.65 + 2.45)kN}{1.22m}$$

$$S_2 = 7.46kN/m$$

3rd storey

$$S_3 = \frac{F_3}{SWlength}$$

$$S_3 = \frac{2.45kN}{1.22m}$$

$$S_3 = 2.01kN/m$$

Design shear resistance and stiffness of shear walls from test results

Sheathing Type	Sheathing Thickness (mm)	Fastener Schedule (mm)	$S_{y,avg}$ (kN/m)	S_r (kN/m)	$k_{e,avg}$ (kN/mm/m wall length)
			1220x2440mm (4'x8') wall	$S_r = \phi \times S_{y,avg}$ ($\phi = 0.7$)	1220x2440mm (4'x8') wall
OSB	9.5	152/305	10.7	7.5	1.64
OSB	9.5	100/305	15.9	11.1	1.82
OSB	9.5	75/305	20.4	14.3	2.09

The maximum shear resistances obtained using the NBCC 2005 approach ($S_1 = 12.8$ kN/m, $S_2 = 7.46$ kN/m and $S_2 = 2.01$ kN/m) is smaller than the highest design value for the shear walls tested in this body of research ($S_r = 14.3$ kN/m) therefore the eight shear walls that will be used in this house will be made of a 9.5mm thick OSB panel with a screw spacing of 3 inches along the perimeter of the wall.

Deflection check

The elastic displacement, Δ_e, is the maximum displacement of a storey. Then the anticipated deflection can be calculated using R_dR_o/I_E times the elastic displacement.

Calculation for drift limitation:

	$\Delta_e = S / K_e$
1st Storey	$\Delta_{e,1} = 12.8kN/m \big/ 2.09kN/mm/m = 6.1mm$
2nd Storey	$\Delta_{e,2} = 7.5kN/m \big/ 2.09kN/mm/m = 3.6mm$
3rd Storey	$\Delta_{e,3} = 2.0kN/m \big/ 2.09kN/mm/m = 1.0mm$

	$\Delta_{\max} = \dfrac{R_d R_o}{I_E} \times \Delta_e$
1st Storey	$\Delta_{\max,1} = \dfrac{2.5 \times 1.8}{1.0} \times 6.1mm = 27.5mm$
2nd Storey	$\Delta_{\max,2} = \dfrac{2.5 \times 1.8}{1.0} \times 3.6mm = 16.2mm$
3rd Storey	$\Delta_{\max,3} = \dfrac{2.5 \times 1.8}{1.0} \times 1.0mm = 4.5mm$

The largest interstorey deflection for a storey should be equal or less than $0.025h_s$, where h_s is the height of the storey.

1st Storey	$\Delta_{\max,1} \leq 0.025 \times (2740 + 100)mm = 71.0mm$
2nd Storey	$\Delta_{\max,2} \leq 0.025 \times (2740 + 150)mm = 72.3mm$
3rd Storey	$\Delta_{\max,3} \leq 0.025 \times (2740 + 50)mm = 69.8mm$

Therefore a total of eight 1.22m x 2.44m shear walls per storey that is, two shear walls per exterior wall per storey, with a screw schedule of 75/305 mm (3"/12") is adequate for the modeled building.

APPENDIX 'E'
GROUND MOTION RECORD ACCELEROGRAMS & SPECTRA

Ground Motion Record S01

No	Event	Magn.	R	Station	deg	PGA(g)	PHV(m/s)	td(s)	Sf
S01	Simulated (Trial #1)	M_w 6.5	30		-	0.53	0.57	8.52	1

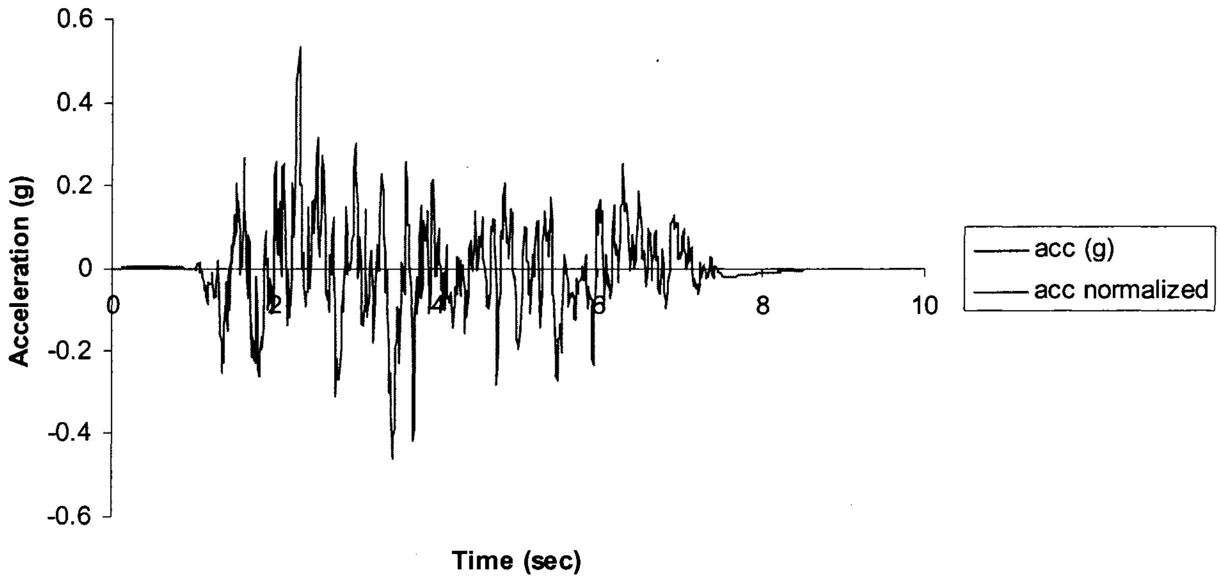


Figure E.1: Accelerogram for ground motion record S01

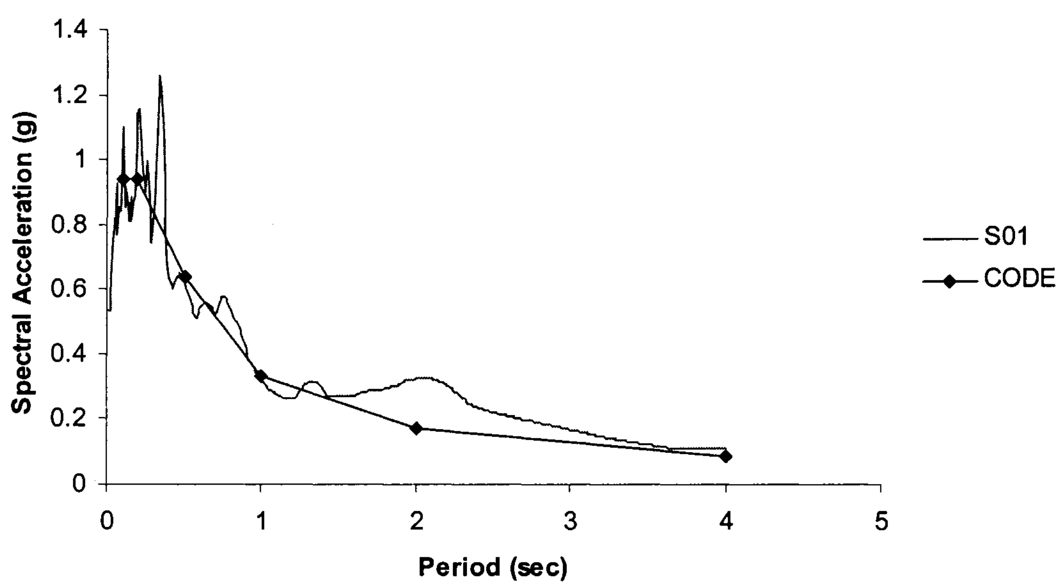


Figure E.2: 2005 NBCC Vancouver 5% damped spectral response acceleration : ground motion record S01 scaling

Ground Motion Record S02

No	Event	Magn.	R	Station	deg	PGA(g)	PHV(m/s)	td(s)	Sf
S02	Simulated (Trial #4)	M_w 6.5	30		-	0.43	0.31	8.52	1.1

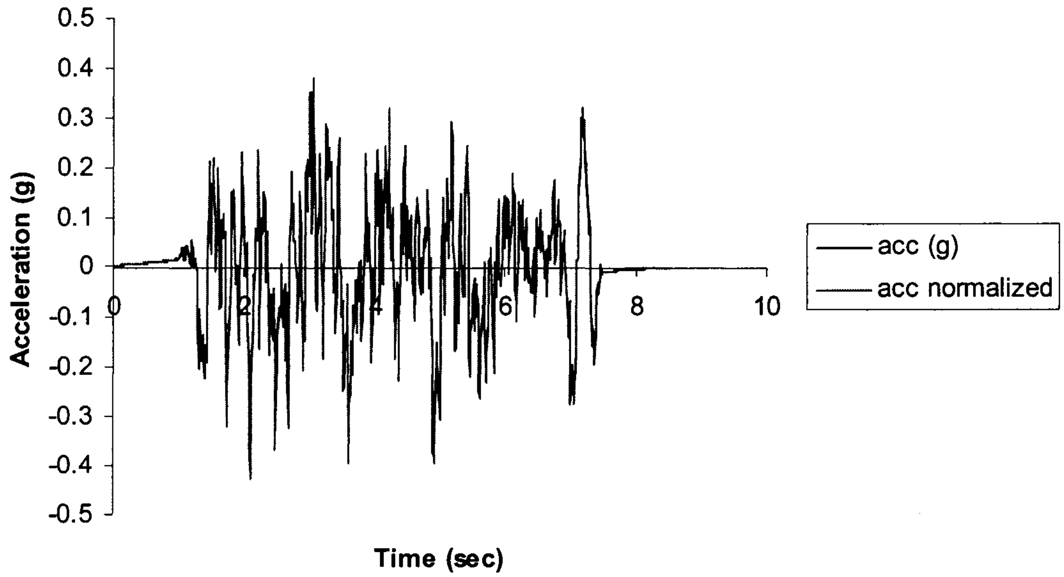


Figure E.3: Accelerogram for ground motion record S02

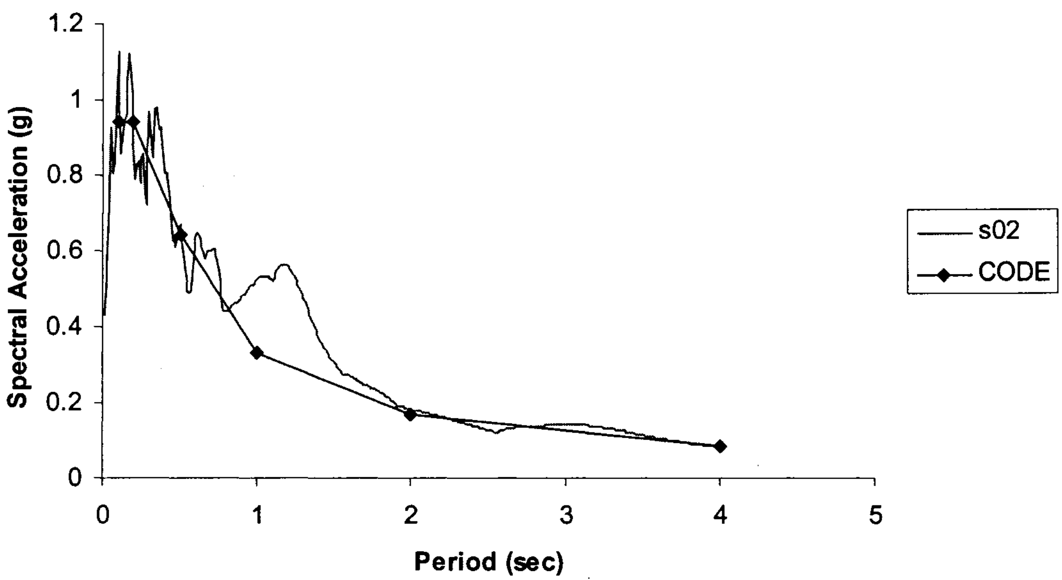


Figure E.4: 2005 NBCC Vancouver 5% damped spectral response acceleration : ground motion record S02 scaling

Ground Motion Record S03

No	Event	Magn.	R	Station	deg	PGA(g)	PHV(m/s)	td(s)	Sf
S03	Simulated (Trial #1)	M_w 7.2	70		-	0.28	0.3	18.17	1.1

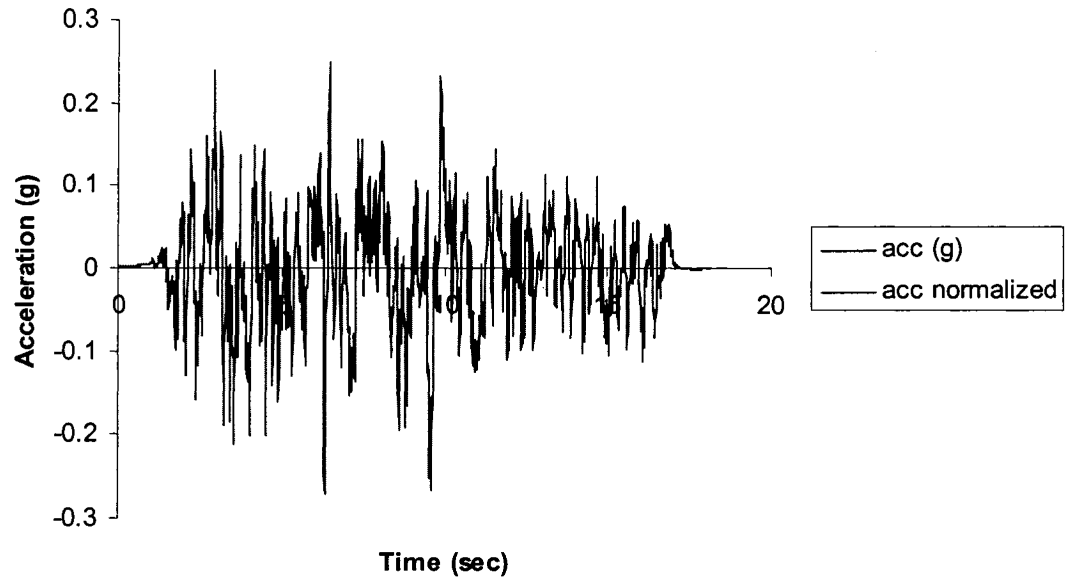


Figure E.5: Accelerogram for ground motion record S03

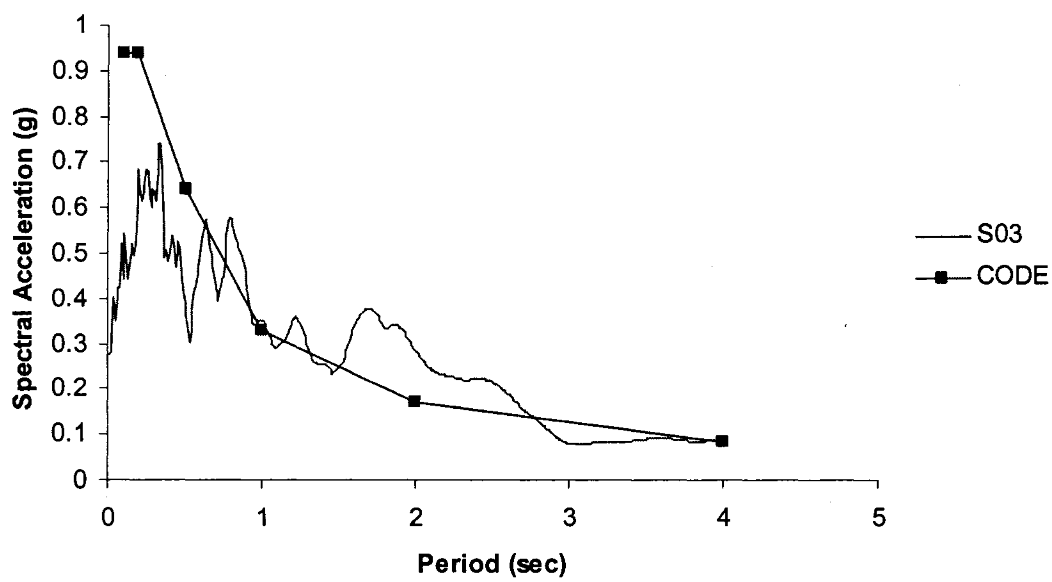


Figure E.6: 2005 NBCC Vancouver 5% damped spectral response acceleration : ground motion record S03 scaling

Ground Motion Record S04

No	Event	Magn.	R	Station	deg	PGA(g)	PHV(m/s)	td(s)	Sf
S04	Simulated (Trial #2)	M_w 7.2	70		-	0.31	0.24	18.17	1.2

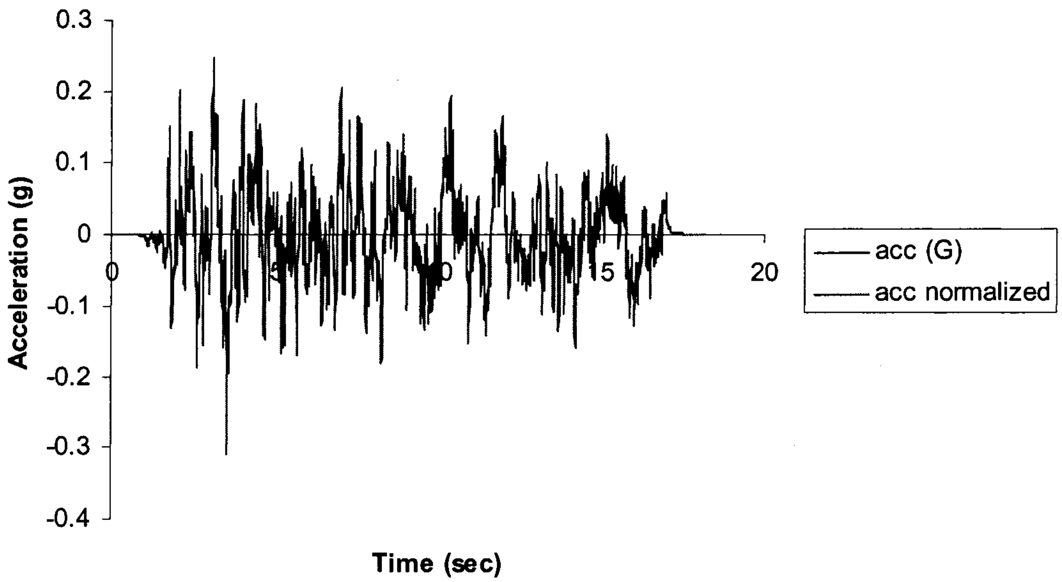


Figure E.7: Accelerogram for ground motion record S04

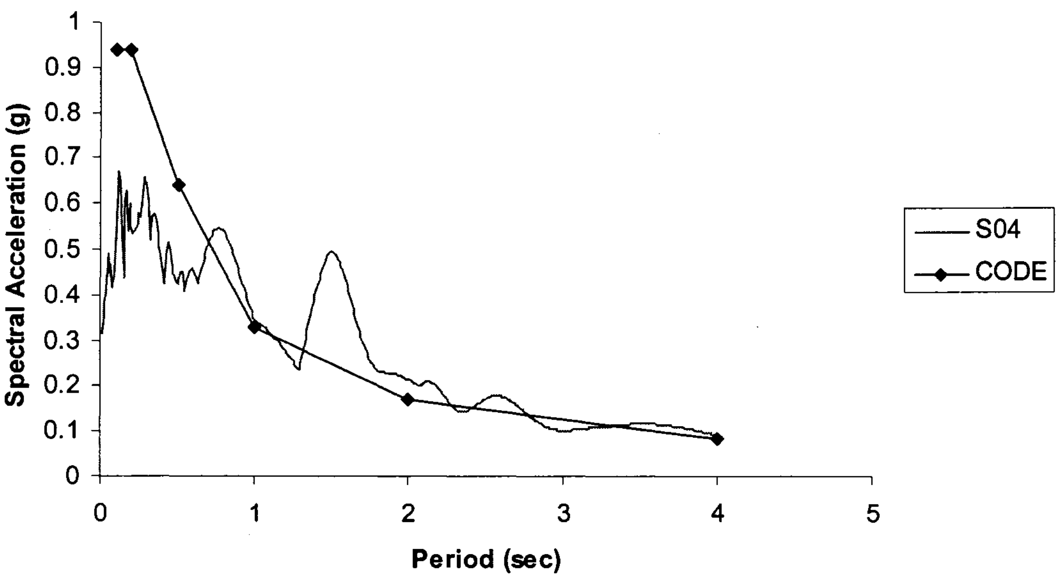


Figure E.8: 2005 NBCC Vancouver 5% damped spectral response acceleration : ground motion record S04 scaling

Ground Motion Record S05

No	Event	Magn.	R	Station	deg	PGA(g)	PHV(m/s)	td(s)	Sf
S05	Apr. 24, 1984 Morgan Hill	M_w 6.1	38	San Ysidro, Gilroy #6	90	0.26	0.37	15	0.9

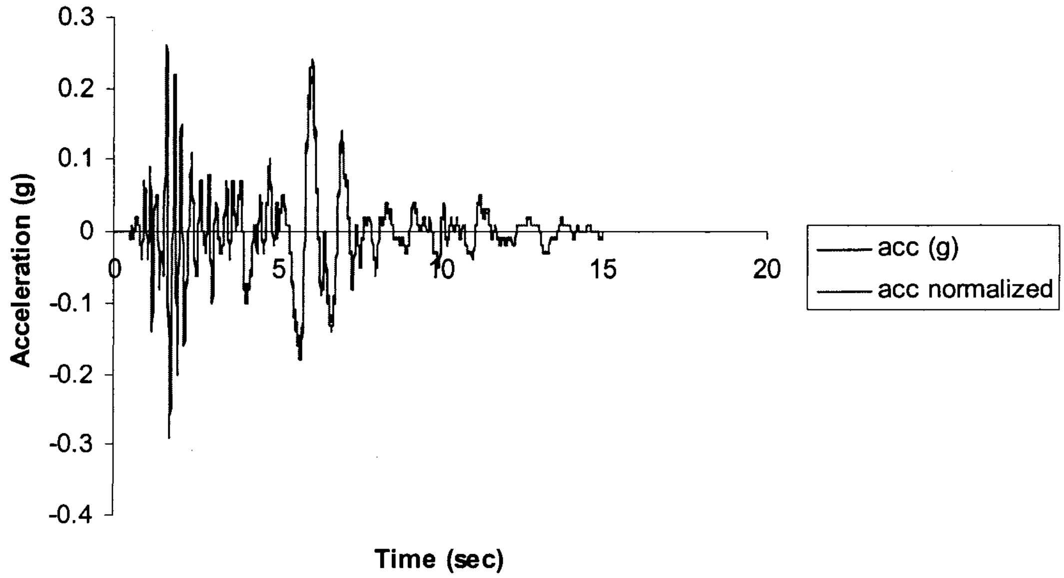


Figure E.9: Accelerogram for ground motion record S05

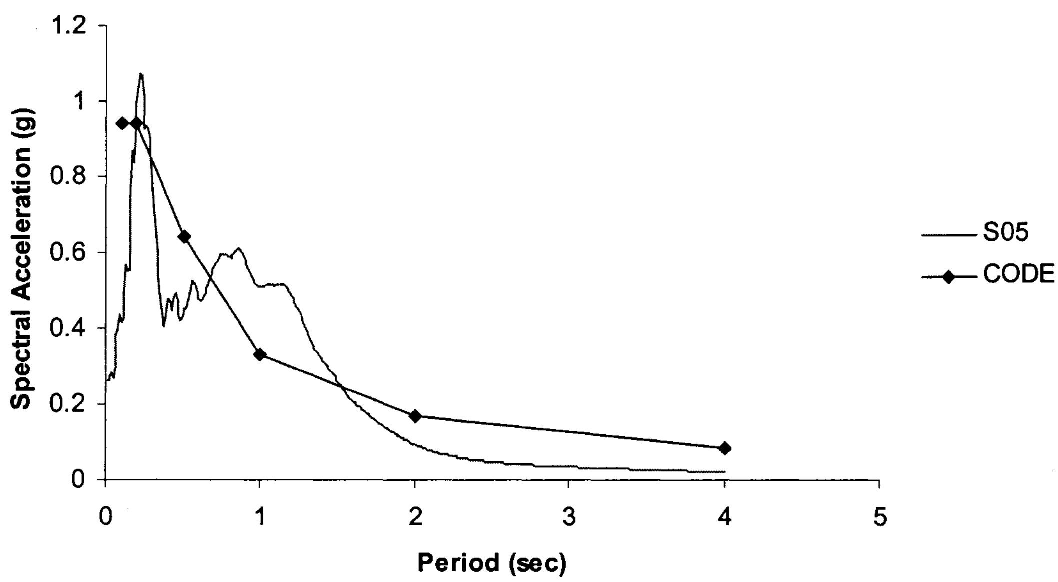


Figure E.10: 2005 NBCC Vancouver 5% damped spectral response acceleration : ground motion record S05 scaling

Ground Motion Record S06

No	Event	Magn.	R	Station	deg	PGA(g)	PHV(m/s)	td(s)	Sf
S06	Jan. 17, 1994 Northridge	M_w 6.7	44	Castaic, Old Ridge Rd	90	0.34	0.52	25	0.6

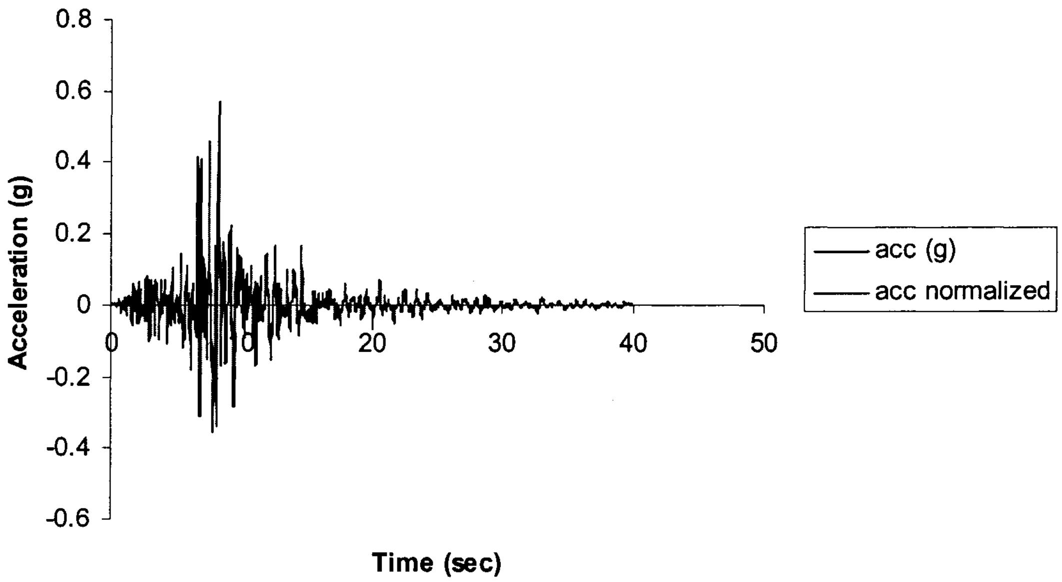


Figure E.11: Accelerogram for ground motion record S06

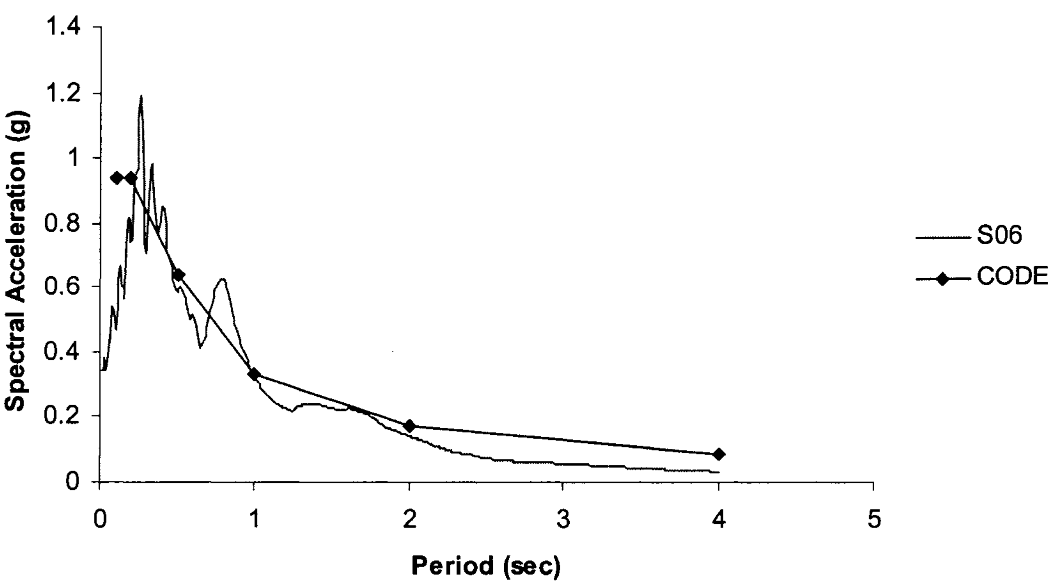


Figure E.12: 2005 NBCC Vancouver 5% damped spectral response acceleration : ground motion record S06 scaling

Ground Motion Record S07

No	Event	Magn.	R	Station	deg	PGA(g)	PHV(m/s)	td(s)	Sf
S07	Jan. 17, 1994 Northridge	M_w 6.7	44	Castaic, Old Ridge Rd	0	0.26	0.53	40	0.5

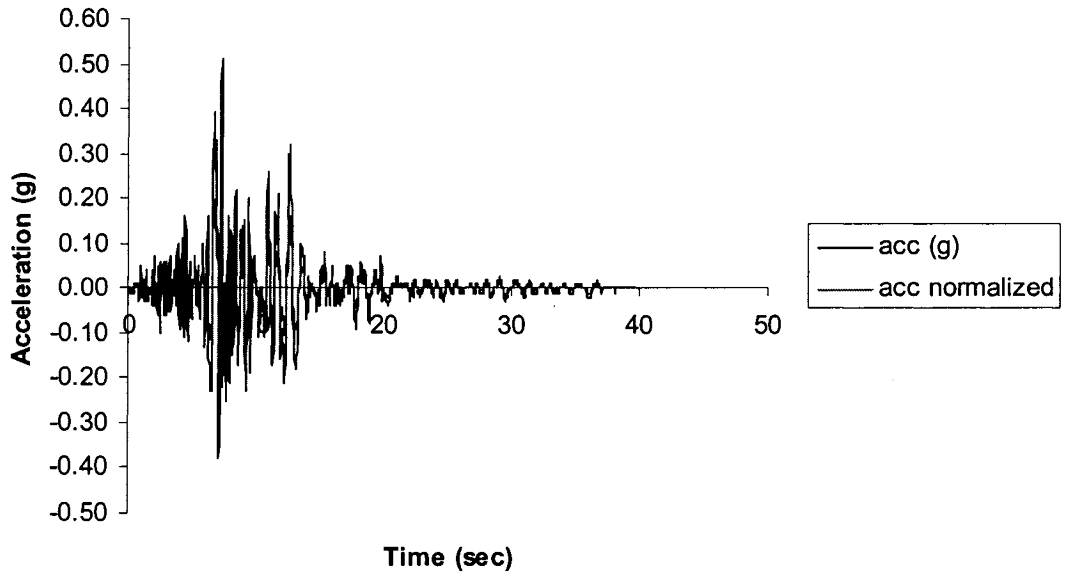


Figure E.13: Accelerogram for ground motion record S07

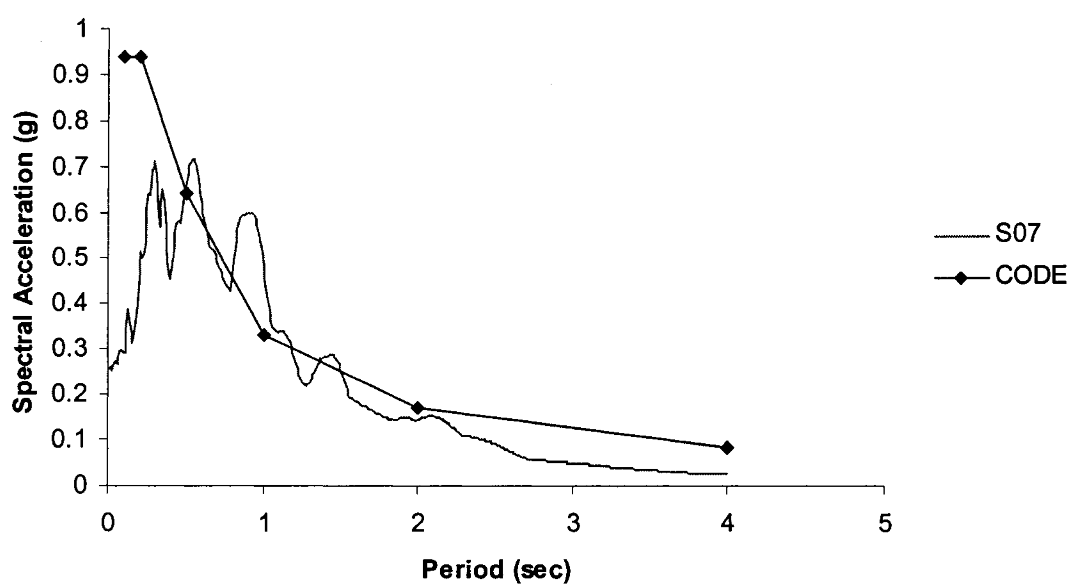


Figure E.14: 2005 NBCC Vancouver 5% damped spectral response acceleration : ground motion record S07 scaling

Ground Motion Record S08

No	Event	Magn.	R	Station	deg	PGA(g)	PHV(m/s)	td(s)	Sf
S08	Oct. 18, 1989 Loma Prieta	M_w 7.0	54	Stanford Univ.	0	0.29	0.28	39.6	1

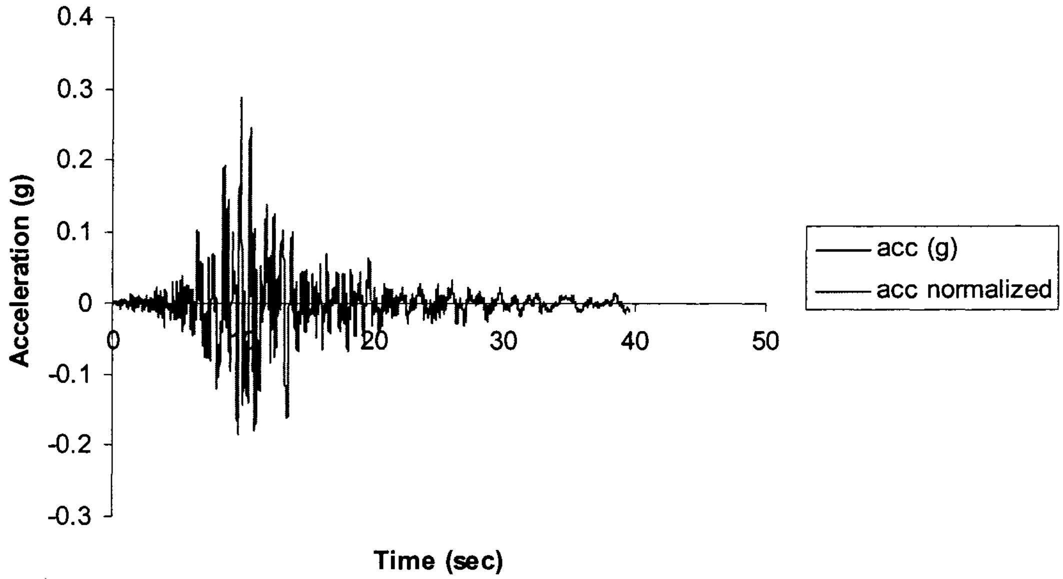


Figure E.15: Accelerogram for ground motion record S08

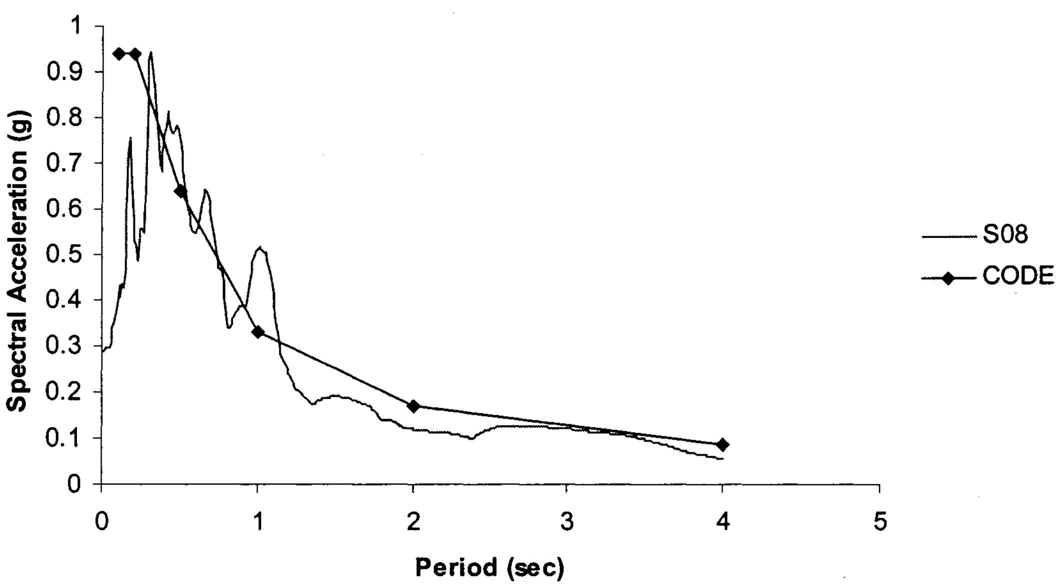


Figure E.16: 2005 NBCC Vancouver 5% damped spectral response acceleration : ground motion record S08 scaling

Ground Motion Record S09

No	Event	Magn.	R	Station	deg	PGA(g)	PHV(m/s)	td(s)	Sf
S09	Oct. 18, 1989 Loma Prieta	M_w 7.0	100	Presidio	90	0.26	0.34	40	1.3

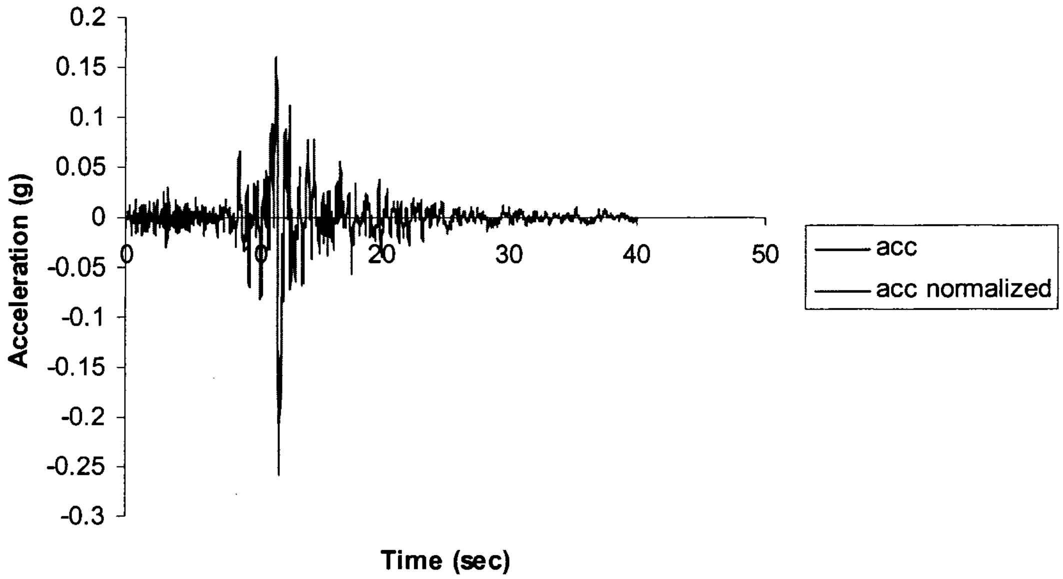


Figure E.17: Accelerogram for ground motion record S09

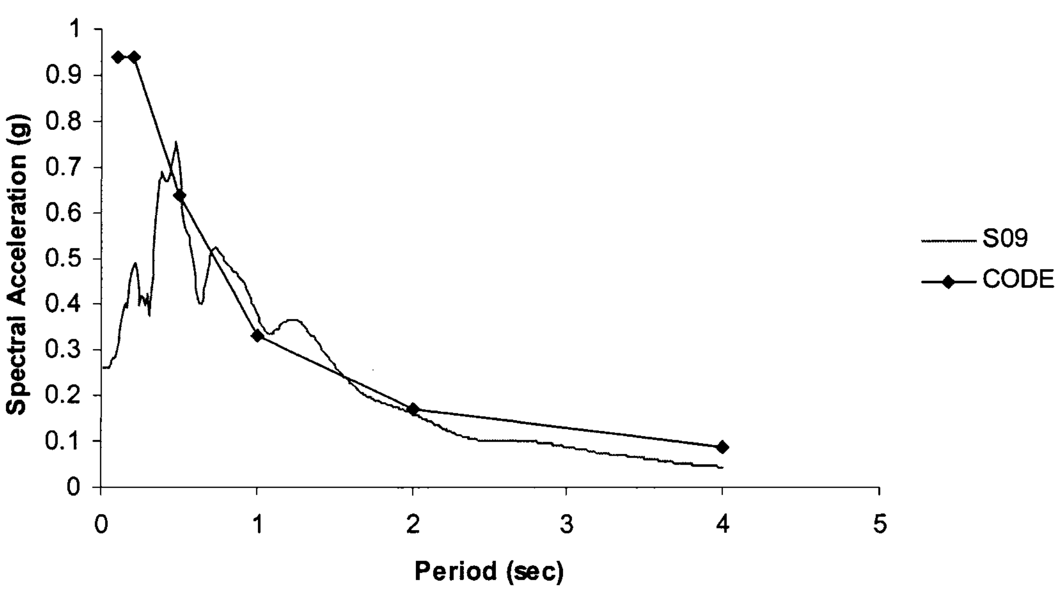


Figure E.18: 2005 NBCC Vancouver 5% damped spectral response acceleration : ground motion record S09 scaling

Ground Motion Record S10

No	Event	Magn.	R	Station	deg	PGA(g)	PHV(m/s)	td(s)	Sf
S10	Apr. 13, 1949 West.Wash.	M_w 7.1	76	Olympia, Test Lab	86	0.42	0.17	30	1.5

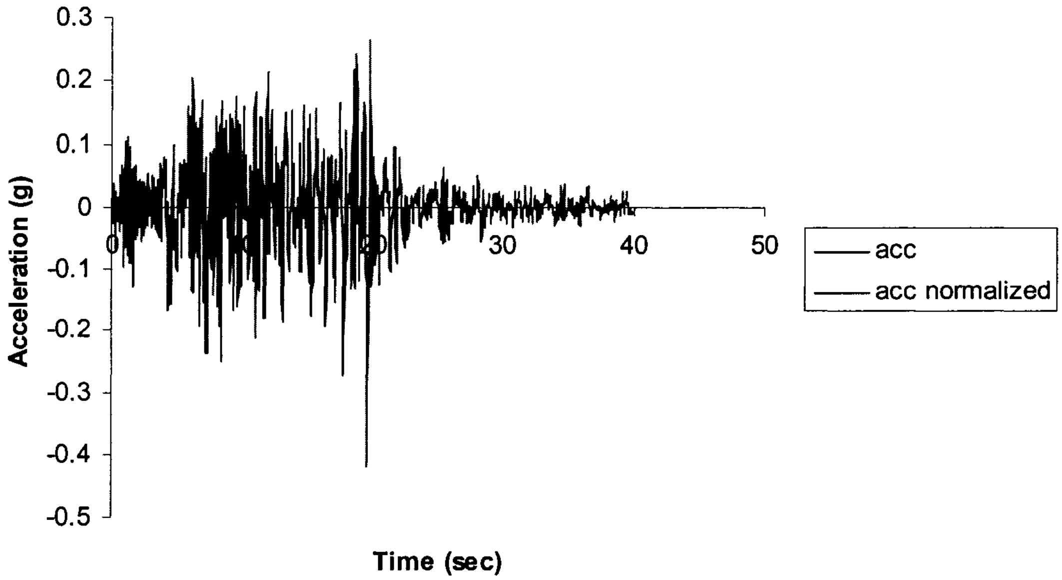


Figure E.19: Accelerogram for ground motion record S10

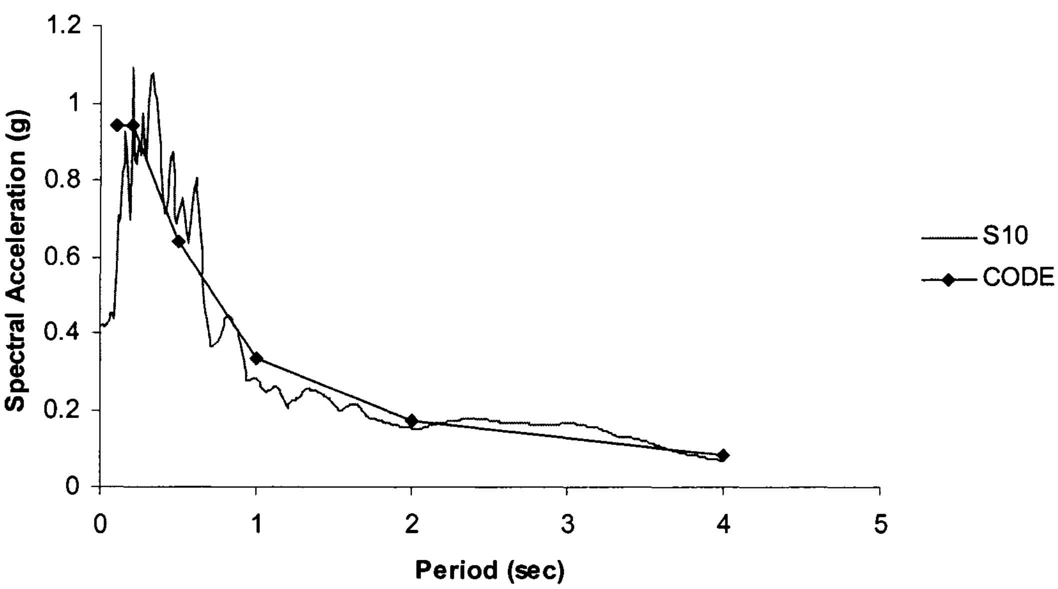


Figure E.20: 2005 NBCC Vancouver 5% damped spectral response acceleration : ground motion record S10 scaling

APPENDIX 'F'
RUAUMOKO INPUT FILES FOR BOTH BUILDING MODELS

Two-Storey Shear Wall

```
Shear wall 152/305 95mm OSB           ! Units kN, m and s
2 0 1 0 0 0 0 0 0                     ! Principal Analysis Options
8 13 4 2 1 2 9.81 5 5 0.01 8.52 1     ! Frame Control Parameters
1 1 1 0 1                             ! Output Intervals and Plotting Control Parameters
0 0                                   ! Iteration Control

NODES
1 0    0    1 1 1  0 0 0  3
2 0    2.44 0 0 1  0 0 0  0
3 1.22 2.44 0 0 1  0 0 0  3
4 1.22 0    1 1 1  0 0 0  3
5 0    2.75 0 0 1  2 0 0  3
6 0    5.19 0 0 1  0 0 0  0
7 1.22 5.19 0 0 1  0 0 0  3
8 1.22 2.75 0 0 1  3 0 0  3

ELEMENTS
1  1 1 2 0 0 3
2  2 2 3 0 0 3
3  1 3 4 0 0 3
4  3 2 4 0 0 0
5  3 3 1 0 0 3
6  4 2 5 0 0 3
7  2 5 8 0 0 3
8  4 8 3 0 0 3
9  1 5 6 0 0 3
10 2 6 7 0 0 3
11 1 7 8 0 0 3
12 3 6 8 0 0 0
13 3 7 5 0 0 3

PROPS                                 ! Member Properties Tables
1 SPRING                              ! columns
  1 0 0 0 1000000
2 SPRING                              ! beam
  1 0 0 0 1000000
3 SPRING                              ! brace: 152/305 95mm OSB Blais Table 5.1
1 9 0 0 4700 0 0 0 0.15               ! Itype 1, Ihyst = Wayne Stewart, Ilos = No Strength Degradation
                                      (Not available for Stewart),IDAMG , Kx, Ky, GJ, WGT, RF
  10.61 -10.61                        ! FX+ FX-
  15.92 1.29 0.0 1.75 0 0 1.09 0.52 0 ! FU  FI  PTRI  PUNL  GAP+  GAP-  BETA  ALPHA
4 SPRING
  1 0 0 0 254559                      ! Floor components

WEIGHT
1 0
2 0
3 0
4 0
5 16.5
6 11.0
7 11.0
8 16.5

LOAD
1 0 0 0
2 0 0 0
3 0 0 0
4 0 0 0
5 0 0 0
6 0 0 0
7 0 0 0
8 0 0 0

EQUAKE s01.dat
3 1 0.01 1 8.52 0 0 1.0

START
```

Figure F.1: Ruaumoko input file for two-storey model under ground motion record S01

```
Shear wall 152/305 95mm OSB             ! Units kN, m and s
2 0 1 0 0 0 0 0 0                       ! Principal Analysis Options
8 13 4 2 1 2 9.81 5 5 0.01 8.52 1       ! Frame Control Parameters
1 1 1 0 1                               ! Output Intervals and Plotting Control Parameters
0 0                                     ! Iteration Control

NODES
1 0    0    1 1 1  0 0 0  3
2 0    2.44 0 0 1  0 0 0  0
3 1.22 2.44 0 0 1  0 0 0  3
4 1.22 0    1 1 1  0 0 0  3
5 0    2.75 0 0 1  2 0 0  3
6 0    5.19 0 0 1  0 0 0  0
7 1.22 5.19 0 0 1  0 0 0  3
8 1.22 2.75 0 0 1  3 0 0  3

ELEMENTS
1  1 1 2 0 0 3
2  2 2 3 0 0 3
3  1 3 4 0 0 3
4  3 2 4 0 0 0
5  3 3 1 0 0 3
6  4 2 5 0 0 3
7  2 5 8 0 0 3
8  4 8 3 0 0 3
9  1 5 6 0 0 3
10 2 6 7 0 0 3
11 1 7 8 0 0 3
12 3 6 8 0 0 0
13 3 7 5 0 0 3

PROPS                                   ! Member Properties Tables
1 SPRING                                ! columns
  1 0 0 0 1000000
2 SPRING                                ! beam
  1 0 0 0 1000000
3 SPRING                                ! brace: 152/305 95mm OSB Blais Table 5.1
  1 9 0 0 4700 0 0 0 0.15               ! Itype 1, Ihyst = Wayne Stewart, Ilos = No Strength Degradation
                                        (Not available for Stewart),IDAMG , Kx, Ky, GJ, WGT, RF
  10.61 -10.61                          ! FX+  FX-
  15.92 1.29 0.0 1.75 0 0 1.09 0.52 0 ! FU  FI  PTRI  PUNL  GAP+  GAP-  BETA  ALPHA
4 SPRING
  1 0 0 0 254559                        ! Floor components

WEIGHT
1 0
2 0
3 0
4 0
5 16.5
6 11.0
7 11.0
8 16.5

LOAD
1 0 0 0
2 0 0 0
3 0 0 0
4 0 0 0
5 0 0 0
6 0 0 0
7 0 0 0
8 0 0 0

EQUAKE s02.dat
3 1 0.01 1 8.52 0 0 1.0

START
```

Figure F.2: Ruaumoko input file for two-storey model under ground motion record S02

```
Shear wall 152/305 95mm OSB          ! Units kN, m and s
2 0 1 0 0 0 0 0 0                    ! Principal Analysis Options
8 13 4 2 1 2 9.81 5 5 0.01 18.17 1   ! Frame Control Parameters
1 1 1 0 1                            ! Output Intervals and Plotting Control Parameters
0 0                                  ! Iteration Control

NODES
1 0    0    1 1 1  0 0 0  3
2 0    2.44 0 0 1  0 0 0  0
3 1.22 2.44 0 0 1  0 0 0  3
4 1.22 0    1 1 1  0 0 0  3
5 0    2.75 0 0 1  2 0 0  3
6 0    5.19 0 0 1  0 0 0  0
7 1.22 5.19 0 0 1  0 0 0  3
8 1.22 2.75 0 0 1  3 0 0  3

ELEMENTS
1  1 1 2 0 0 3
2  2 2 3 0 0 3
3  1 3 4 0 0 3
4  3 2 4 0 0 0
5  3 3 1 0 0 3
6  4 2 5 0 0 3
7  2 5 8 0 0 3
8  4 8 3 0 0 3
9  1 5 6 0 0 3
10 2 6 7 0 0 3
11 1 7 8 0 0 3
12 3 6 8 0 0 0
13 3 7 5 0 0 3

PROPS                                ! Member Properties Tables
1 SPRING                             ! columns
  1 0 0 0 1000000
2 SPRING                             ! beam
  1 0 0 0 1000000
3 SPRING                             ! brace: 152/305 95mm OSB Blais Table 5.1
  1 9 0 0 4700 0 0 0 0.15            ! Itype 1, Ihyst = Wayne Stewart, Ilos = No Strength Degradation
                                     (Not available for Stewart),IDAMG , Kx, Ky, GJ, WGT, RF
  10.61 -10.61                       ! FX+ FX-
  15.92 1.29 0.0 1.75 0 0 1.09 0.52 0 ! FU  FI  PTRI  PUNL  GAP+  GAP-  BETA  ALPHA
4 SPRING
  1 0 0 0 254559                     ! Floor components

WEIGHT
1 0
2 0
3 0
4 0
5 16.5
6 11.0
7 11.0
8 16.5

LOAD
1 0 0 0
2 0 0 0
3 0 0 0
4 0 0 0
5 0 0 0
6 0 0 0
7 0 0 0
8 0 0 0

EQUAKE s03.dat
3 1 0.01 1 18.17 0 0 1.0

START
```

Figure F.3: Ruaumoko input file for two-storey model under ground motion record S03

```
Shear wall 152/305 95mm OSB            ! Units kN, m and s
2 0 1 0 0 0 0 0 0                      ! Principal Analysis Options
8 13 4 2 1 2 9.81 5 5 0.01 18.17 1     ! Frame Control Parameters
1 1 1 0 1                              ! Output Intervals and Plotting Control Parameters
0 0                                    ! Iteration Control

NODES
1 0    0    1 1 1  0 0 0  3
2 0    2.44 0 0 1  0 0 0  0
3 1.22 2.44 0 0 1  0 0 0  3
4 1.22 0    1 1 1  0 0 0  3
5 0    2.75 0 0 1  2 0 0  3
6 0    5.19 0 0 1  0 0 0  0
7 1.22 5.19 0 0 1  0 0 0  3
8 1.22 2.75 0 0 1  3 0 0  3

ELEMENTS
1  1 1 2 0 0 3
2  2 2 3 0 0 3
3  1 3 4 0 0 3
4  3 2 4 0 0 0
5  3 3 1 0 0 3
6  4 2 5 0 0 3
7  2 5 8 0 0 3
8  4 8 3 0 0 3
9  1 5 6 0 0 3
10 2 6 7 0 0 3
11 1 7 8 0 0 3
12 3 6 8 0 0 0
13 3 7 5 0 0 3

PROPS                                  ! Member Properties Tables
1 SPRING                               ! columns
  1 0 0 0 1000000
2 SPRING                               ! beam
  1 0 0 0 1000000
3 SPRING                               ! brace: 152/305 95mm OSB Blais Table 5.1
  1 9 0 0 4700 0 0 0 0.15              ! Itype 1, Ihyst = Wayne Stewart, Ilos = No Strength Degradation
                                       (Not available for Stewart),IDAMG , Kx, Ky, GJ, WGT, RF
  10.61 -10.61                         ! FX+  FX-
  15.92 1.29 0.0 1.75 0 0 1.09 0.52 0 ! FU  FI  PTRI  PUNL  GAP+  GAP-  BETA  ALPHA
4 SPRING
  1 0 0 0 254559                       ! Floor components

WEIGHT
1 0
2 0
3 0
4 0
5 16.5
6 11.0
7 11.0
8 16.5

LOAD
1 0 0 0
2 0 0 0
3 0 0 0
4 0 0 0
5 0 0 0
6 0 0 0
7 0 0 0
8 0 0 0

EQUAKE s04.dat
3 1 0.01 1 18.17 0 0 1.0

START
```

Figure F.4: Ruaumoko input file for two-storey model under ground motion record S04

```
Shear wall 152/305 95mm OSB          ! Units kN, m and s
2 0 1 0 0 0 0 0 0                    ! Principal Analysis Options
8 13 4 2 1 2 9.81 5 5 0.005 29.975 1 ! Frame Control Parameters
1 1 1 0 1                            ! Output Intervals and Plotting Control Parameters
0 0                                  ! Iteration Control

NODES
1 0    0    1 1 1  0 0 0  3
2 0    2.44 0 0 1  0 0 0  0
3 1.22 2.44 0 0 1  0 0 0  3
4 1.22 0    1 1 1  0 0 0  3
5 0    2.75 0 0 1  2 0 0  3
6 0    5.19 0 0 1  0 0 0  0
7 1.22 5.19 0 0 1  0 0 0  3
8 1.22 2.75 0 0 1  3 0 0  3

ELEMENTS
1  1 1 2 0 0 3
2  2 2 3 0 0 3
3  1 3 4 0 0 3
4  3 2 4 0 0 0
5  3 3 1 0 0 3
6  4 2 5 0 0 3
7  2 5 8 0 0 3
8  4 8 3 0 0 3
9  1 5 6 0 0 3
10 2 6 7 0 0 3
11 1 7 8 0 0 3
12 3 6 8 0 0 0
13 3 7 5 0 0 3

PROPS                                ! Member Properties Tables
1 SPRING                             ! columns
  1 0 0 0 1000000
2 SPRING                             ! beam
  1 0 0 0 1000000
3 SPRING                             ! brace: 152/305 95mm OSB Blais Table 5.1
  1 9 0 0 4700 0 0 0 0.15            ! Itype 1, Ihyst = Wayne Stewart, Ilos = No Strength Degradation
                                     (Not available for Stewart),IDAMG , Kx, Ky, GJ, WGT, RF
  10.61 -10.61                       ! FX+  FX-
  15.92 1.29 0.0 1.75 0 0 1.09 0.52 0 ! FU  FI  PTRI  PUNL  GAP+  GAP-  BETA  ALPHA
4 SPRING
  1 0 0 0 254559                     ! Floor components

WEIGHT
1 0
2 0
3 0
4 0
5 16.5
6 11.0
7 11.0
8 16.5

LOAD
1 0 0 0
2 0 0 0
3 0 0 0
4 0 0 0
5 0 0 0
6 0 0 0
7 0 0 0
8 0 0 0

EQUAKE s05.dat
3 1 0.005 1 29.975 0 0 1.0

START
```

Figure F.5: Ruaumoko input file for two-storey model under ground motion record S05

```
Shear wall 152/305 95mm OSB           ! Units kN, m and s
2 0 1 0 0 0 0 0 0                     ! Principal Analysis Options
8 13 4 2 1 2 9.81 5 5 0.02 39.98 1    ! Frame Control Parameters
1 1 1 0 1                             ! Output Intervals and Plotting Control Parameters
0 0                                   ! Iteration Control

NODES
1 0    0    1 1 1  0 0 0  3
2 0    2.44 0 0 1  0 0 0  0
3 1.22 2.44 0 0 1  0 0 0  3
4 1.22 0    1 1 1  0 0 0  3
5 0    2.75 0 0 1  2 0 0  3
6 0    5.19 0 0 1  0 0 0  0
7 1.22 5.19 0 0 1  0 0 0  3
8 1.22 2.75 0 0 1  3 0 0  3

ELEMENTS
1  1 1 2 0 0 3
2  2 2 3 0 0 3
3  1 3 4 0 0 3
4  3 2 4 0 0 0
5  3 3 1 0 0 3
6  4 2 5 0 0 3
7  2 5 8 0 0 3
8  4 8 3 0 0 3
9  1 5 6 0 0 3
10 2 6 7 0 0 3
11 1 7 8 0 0 3
12 3 6 8 0 0 0
13 3 7 5 0 0 3

PROPS                                 ! Member Properties Tables
1 SPRING                              ! columns
  1 0 0 0 1000000
2 SPRING                              ! beam
  1 0 0 0 1000000
3 SPRING                              ! brace: 152/305 95mm OSB Blais Table 5.1
  1 9 0 0 4700 0 0 0 0.15             ! Itype 1, Ihyst = Wayne Stewart, Ilos = No Strength Degradation
                                      (Not available for Stewart),IDAMG , Kx, Ky, GJ, WGT, RF
  10.61 -10.61                        ! FX+  FX-
  15.92 1.29 0.0 1.75 0 0 1.09 0.52 0 ! FU  FI  PTRI  PUNL  GAP+  GAP-  BETA  ALPHA
4 SPRING
  1 0 0 0 254559                      ! Floor components

WEIGHT
1 0
2 0
3 0
4 0
5 16.5
6 11.0
7 11.0
8 16.5

LOAD
1 0 0 0
2 0 0 0
3 0 0 0
4 0 0 0
5 0 0 0
6 0 0 0
7 0 0 0
8 0 0 0

EQUAKE s06.dat
3 1 0.02 1 39.98 0 0 1.0

START
```

Figure F.6: Ruaumoko input file for two-storey model under ground motion record S06

```
Shear wall 152/305 95mm OSB             ! Units kN, m and s
2 0 1 0 0 0 0 0 0                       ! Principal Analysis Options
8 13 4 2 1 2 9.81 5 5 0.02 39.98 1      ! Frame Control Parameters
1 1 1 0 1                               ! Output Intervals and Plotting Control Parameters
0 0                                     ! Iteration Control

NODES
1 0    0    1 1 1  0 0 0  3
2 0    2.44 0 0 1  0 0 0  0
3 1.22 2.44 0 0 1  0 0 0  3
4 1.22 0    1 1 1  0 0 0  3
5 0    2.75 0 0 1  2 0 0  3
6 0    5.19 0 0 1  0 0 0  0
7 1.22 5.19 0 0 1  0 0 0  3
8 1.22 2.75 0 0 1  3 0 0  3

ELEMENTS
1  1 1 2 0 0 3
2  2 2 3 0 0 3
3  1 3 4 0 0 3
4  3 2 4 0 0 0
5  3 3 1 0 0 3
6  4 2 5 0 0 3
7  2 5 8 0 0 3
8  4 8 3 0 0 3
9  1 5 6 0 0 3
10 2 6 7 0 0 3
11 1 7 8 0 0 3
12 3 6 8 0 0 0
13 3 7 5 0 0 3

PROPS                                   ! Member Properties Tables
1 SPRING                                ! columns
  1 0 0 0 1000000
2 SPRING                                ! beam
  1 0 0 0 1000000
3 SPRING                                ! brace: 152/305 95mm OSB Blais Table 5.1
  1 9 0 0 4700 0 0 0 0.15               ! Itype 1, Ihyst = Wayne Stewart, Ilos = No Strength Degradation
                                        (Not available for Stewart),IDAMG , Kx, Ky, GJ, WGT, RF
  10.61 -10.61                          ! FX+  FX-
  15.92 1.29 0.0 1.75 0 0 1.09 0.52 0 ! FU  FI  PTRI  PUNL  GAP+  GAP-  BETA  ALPHA
4 SPRING
  1 0 0 0 254559                        ! Floor components

WEIGHT
1 0
2 0
3 0
4 0
5 16.5
6 11.0
7 11.0
8 16.5

LOAD
1 0 0 0
2 0 0 0
3 0 0 0
4 0 0 0
5 0 0 0
6 0 0 0
7 0 0 0
8 0 0 0

EQUAKE s07.dat
3 1 0.02 1 39.98 0 0 1.0

START
```

Figure F.7: Ruaumoko input file for two-storey model under ground motion record S07

```
Shear wall 152/305 95mm OSB                ! Units kN, m and s
2 0 1 0 0 0 0 0 0                          ! Principal Analysis Options
8 13 4 2 1 2 9.81 5 5 0.005 39.6 1         ! Frame Control Parameters
1 1 1 0 1                                  ! Output Intervals and Plotting Control Parameters
0 0                                        ! Iteration Control

NODES
1 0    0    1 1 1  0 0 0  3
2 0    2.44 0 0 1  0 0 0  0
3 1.22 2.44 0 0 1  0 0 0  3
4 1.22 0    1 1 1  0 0 0  3
5 0    2.75 0 0 1  2 0 0  3
6 0    5.19 0 0 1  0 0 0  0
7 1.22 5.19 0 0 1  0 0 0  3
8 1.22 2.75 0 0 1  3 0 0  3

ELEMENTS
1  1 1 2 0 0 3
2  2 2 3 0 0 3
3  1 3 4 0 0 3
4  3 2 4 0 0 0
5  3 3 1 0 0 3
6  4 2 5 0 0 3
7  2 5 8 0 0 3
8  4 8 3 0 0 3
9  1 5 6 0 0 3
10 2 6 7 0 0 3
11 1 7 8 0 0 3
12 3 6 8 0 0 0
13 3 7 5 0 0 3

PROPS                                      ! Member Properties Tables
1 SPRING                                   ! columns
  1 0 0 0 1000000
2 SPRING                                   ! beam
  1 0 0 0 1000000
3 SPRING                                   ! brace: 152/305 95mm OSB Blais Table 5.1
  1 9 0 0 4700 0 0 0 0.15                  ! Itype 1, Ihyst = Wayne Stewart, Ilos = No Strength Degradation
                                           (Not available for Stewart),IDAMG , Kx, Ky, GJ, WGT, RF
  10.61 -10.61                             ! FX+  FX-
  15.92 1.29 0.0 1.75 0 0 1.09 0.52 0 ! FU  FI  PTRI  PUNL  GAP+  GAP-  BETA  ALPHA
4 SPRING
  1 0 0 0 254559                           ! Floor components

WEIGHT
1 0
2 0
3 0
4 0
5 16.5
6 11.0
7 11.0
8 16.5

LOAD
1 0 0 0
2 0 0 0
3 0 0 0
4 0 0 0
5 0 0 0
6 0 0 0
7 0 0 0
8 0 0 0

EQUAKE s08.dat
3 1 0.005 1 39.6 0 0 1.0

START
```

Figure F.8: Ruaumoko input file for two-storey model under ground motion record S08

```
Shear wall 152/305 95mm OSB              ! Units kN, m and s
2 0 1 0 0 0 0 0 0                        ! Principal Analysis Options
8 13 4 2 1 2 9.81 5 5 0.02 39.98 1       ! Frame Control Parameters
1 1 1 0 1                                ! Output Intervals and Plotting Control Parameters
0 0                                      ! Iteration Control

NODES
1 0    0    1 1 1  0 0 0  3
2 0    2.44 0 0 1  0 0 0  0
3 1.22 2.44 0 0 1  0 0 0  3
4 1.22 0    1 1 1  0 0 0  3
5 0    2.75 0 0 1  2 0 0  3
6 0    5.19 0 0 1  0 0 0  0
7 1.22 5.19 0 0 1  0 0 0  3
8 1.22 2.75 0 0 1  3 0 0  3

ELEMENTS
1  1 1 2 0 0 3
2  2 2 3 0 0 3
3  1 3 4 0 0 3
4  3 2 4 0 0 0
5  3 3 1 0 0 3
6  4 2 5 0 0 3
7  2 5 8 0 0 3
8  4 8 3 0 0 3
9  1 5 6 0 0 3
10 2 6 7 0 0 3
11 1 7 8 0 0 3
12 3 6 8 0 0 0
13 3 7 5 0 0 3

PROPS                                    ! Member Properties Tables
1 SPRING                                 ! columns
  1 0 0 0 1000000
2 SPRING                                 ! beam
  1 0 0 0 1000000
3 SPRING                                 ! brace: 152/305 95mm OSB Blais Table 5.1
  1 9 0 0 4700 0 0 0 0.15                ! Itype 1, Ihyst = Wayne Stewart, Ilos = No Strength Degradation
                                         (Not available for Stewart),IDAMG , Kx, Ky, GJ, WGT, RF
  10.61 -10.61                           ! FX+  FX-
  15.92 1.29 0.0 1.75 0 0 1.09 0.52 0 ! FU  FI  PTRI  PUNL  GAP+  GAP-  BETA  ALPHA
4 SPRING
  1 0 0 0 254559                         ! Floor components

WEIGHT
1 0
2 0
3 0
4 0
5 16.5
6 11.0
7 11.0
8 16.5

LOAD
1 0 0 0
2 0 0 0
3 0 0 0
4 0 0 0
5 0 0 0
6 0 0 0
7 0 0 0
8 0 0 0

EQUAKE s09.dat
3 1 0.02 1 39.98 0 0 1.0

START
```

Figure F.9: Ruaumoko input file for two-storey model under ground motion record S09

```
Shear wall 152/305 95mm OSB             ! Units kN, m and s
2 0 1 0 0 0 0 0 0                       ! Principal Analysis Options
8 13 4 2 1 2 9.81 5 5 0.02 89.04 1      ! Frame Control Parameters
1 1 1 0 1                               ! Output Intervals and Plotting Control Parameters
0 0                                     ! Iteration Control

NODES
1 0    0    1 1 1  0 0 0  3
2 0    2.44 0 0 1  0 0 0  0
3 1.22 2.44 0 0 1  0 0 0  3
4 1.22 0    1 1 1  0 0 0  3
5 0    2.75 0 0 1  2 0 0  3
6 0    5.19 0 0 1  0 0 0  0
7 1.22 5.19 0 0 1  0 0 0  3
8 1.22 2.75 0 0 1  3 0 0  3

ELEMENTS
1  1 1 2 0 0 3
2  2 2 3 0 0 3
3  1 3 4 0 0 3
4  3 2 4 0 0 0
5  3 3 1 0 0 3
6  4 2 5 0 0 3
7  2 5 8 0 0 3
8  4 8 3 0 0 3
9  1 5 6 0 0 3
10 2 6 7 0 0 3
11 1 7 8 0 0 3
12 3 6 8 0 0 0
13 3 7 5 0 0 3

PROPS                                   ! Member Properties Tables
1 SPRING                                ! columns
  1 0 0 0 1000000
2 SPRING                                ! beam
  1 0 0 0 1000000
3 SPRING                                ! brace: 152/305 95mm OSB Blais Table 5.1
  1 9 0 0 4700 0 0 0 0.15               ! Itype 1, Ihyst = Wayne Stewart, Ilos = No Strength Degradation
                                        (Not available for Stewart),IDAMG , Kx, Ky, GJ, WGT, RF
  10.61 -10.61                          ! FX+  FX-
  15.92 1.29 0.0 1.75 0 0 1.09 0.52 0 ! FU  FI  PTRI  PUNL  GAP+  GAP-  BETA  ALPHA
4 SPRING
  1 0 0 0 254559                        ! Floor components

WEIGHT
1 0
2 0
3 0
4 0
5 16.5
6 11.0
7 11.0
8 16.5

LOAD
1 0 0 0
2 0 0 0
3 0 0 0
4 0 0 0
5 0 0 0
6 0 0 0
7 0 0 0
8 0 0 0

EQUAKE s10.dat
3 1 0.02 1 89.04 0 0 1.0

START
```
Figure F.10: Ruaumoko input file for two-storey model under ground motion record S10

Three-Storey Shear Wall

```
Shear wall 152/305 95mm OSB 3 storey  ! Units kN, m and s
2 0 1 0 0 0 0 0 0                     ! Principal Analysis Options
12 21 4 3 1 2 9.81 5 5 0.01 8.52 1    ! Frame Control Parameters
1 1 1 0 1                             ! Output Intervals and Plotting Control Parameters
0 0                                   ! Iteration Control

NODES
1  0    0    1 1 1  0 0 0  3
2  0    2.74 0 0 1  0 0 0  0
3  1.22 2.74 0 0 1  0 0 0  3
4  1.22 0    1 1 1  0 0 0  3
5  0    2.89 0 0 1  2 0 0  3
6  0    5.63 0 0 1  0 0 0  0
7  1.22 5.63 0 0 1  0 0 0  3
8  1.22 2.89 0 0 1  3 0 0  3
9  0    5.78 0 0 1  6 0 0  3
10 0    8.52 0 0 1  0 0 0  0
11 1.22 8.52 0 0 1  0 0 0  3
12 1.22 5.78 0 0 1  7 0 0  3

ELEMENTS
1  1 1 2 0 0 3
2  2 2 3 0 0 3
3  1 3 4 0 0 3
4  3 2 4 0 0 0
5  3 3 1 0 0 3
6  4 2 5 0 0 3
7  2 5 8 0 0 3
8  4 8 3 0 0 3
9  1 5 6 0 0 3
10 2 6 7 0 0 3
11 1 7 8 0 0 3
12 3 6 8 0 0 0
13 3 7 5 0 0 3
14 4 6 9 0 0 3
15 2 9 12 0 0 3
16 4 12 7 0 0 3
17 1 9 10 0 0 3
18 2 10 11 0 0 3
19 1 11 12 0 0 3
20 3 10 12 0 0 0
21 3 11 9 0 0 3

PROPS                                 ! Member Properties Tables
1 SPRING                              ! columns
  1 0 0 0 1000000
2 SPRING                              ! beam
  1 0 0 0 1000000
3 SPRING                              ! brace: 75/305 9.5mm OSB Blais Table 5.1
  1 9 0 0 7915 0 0 0 0.19             ! Itype 1, Ihyst = Wayne Stewart, Ilos = No Strength Degradation
                                      (Not available for Stewart),IDAMG , Kx, Ky, GJ, WGT, RF
  22.78 -22.78                        ! FX+  FX-
  33.80 3.08 0.0 1.45 0 0 1.09 0.45 0 ! FU  FI  PTRI  PUNL  GAP+  GAP-  BETA  ALPHA Loop
4 SPRING
  1 0 0 0 254559                      ! Floor components

WEIGHT
1 0
2 21
3 21
4 0
5 0
6 21
7 21
8 0
9 0
10 8
11 8
12 0

LOAD
1 0 0 0
2 0 0 0
3 0 0 0
4 0 0 0
5 0 0 0
6 0 0 0
7 0 0 0
8 0 0 0
9 0 0 0
10 0 0 0
11 0 0 0
12 0 0 0

EQUAKE s01.dat
3 1 0.01 1 8.52 0 0 1.0

START
```

Figure F.11: Ruaumoko input file for three-storey model under ground motion record S01

```
Shear wall 152/305 95mm OSB 3 storey  ! Units kN, m and s
2 0 1 0 0 0 0 0 0                     ! Principal Analysis Options
12 21 4 3 1 2 9.81 5 5 0.01 8.52 1    ! Frame Control Parameters
1 1 1 0 1                             ! Output Intervals and Plotting Control Parameters
0 0                                   ! Iteration Control

NODES
1  0    0    1 1 1  0 0 0  3
2  0    2.74 0 0 1  0 0 0  0
3  1.22 2.74 0 0 1  0 0 0  3
4  1.22 0    1 1 1  0 0 0  3
5  0    2.89 0 0 1  2 0 0  3
6  0    5.63 0 0 1  0 0 0  0
7  1.22 5.63 0 0 1  0 0 0  3
8  1.22 2.89 0 0 1  3 0 0  3
9  0    5.78 0 0 1  6 0 0  3
10 0    8.52 0 0 1  0 0 0  0
11 1.22 8.52 0 0 1  0 0 0  3
12 1.22 5.78 0 0 1  7 0 0  3

ELEMENTS
1  1 1 2 0 0 3
2  2 2 3 0 0 3
3  1 3 4 0 0 3
4  3 2 4 0 0 0
5  3 3 1 0 0 3
6  4 2 5 0 0 3
7  2 5 8 0 0 3
8  4 8 3 0 0 3
9  1 5 6 0 0 3
10 2 6 7 0 0 3
11 1 7 8 0 0 3
12 3 6 8 0 0 0
13 3 7 5 0 0 3
14 4 6 9 0 0 3
15 2 9 12 0 0 3
16 4 12 7 0 0 3
17 1 9 10 0 0 3
18 2 10 11 0 0 3
19 1 11 12 0 0 3
20 3 10 12 0 0 0
21 3 11 9 0 0 3

PROPS                                 ! Member Properties Tables
1 SPRING                              ! columns
  1 0 0 0 1000000
2 SPRING                              ! beam
  1 0 0 0 1000000
3 SPRING                              ! brace: 75/305 9.5mm OSB Blais Table 5.1
  1 9 0 0 7915 0 0 0 0.19             ! Itype 1, Ihyst = Wayne Stewart, Ilos = No Strength Degradation
                                      (Not available for Stewart),IDAMG , Kx, Ky, GJ, WGT, RF
  22.78 -22.78                        ! FX+  FX-
  33.80 3.08 0.0 1.45 0 0 1.09 0.45 0 ! FU  FI  PTRI  PUNL  GAP+  GAP-  BETA  ALPHA Loop
4 SPRING
  1 0 0 0 254559                      ! Floor components

WEIGHT
1 0
2 21
3 21
4 0
5 0
6 21
7 21
8 0
9 0
10 8
11 8
12 0

LOAD
1 0 0 0
2 0 0 0
3 0 0 0
4 0 0 0
5 0 0 0
6 0 0 0
7 0 0 0
8 0 0 0
9 0 0 0
10 0 0 0
11 0 0 0
12 0 0 0

EQUAKE s02.dat
3 1 0.01 1 8.52 0 0 1.0

START
```

Figure F.12: Ruaumoko input file for three-storey model under ground motion record S02

```
Shear wall 152/305 95mm OSB 3 storey  ! Units kN, m and s
2 0 1 0 0 0 0 0 0                     ! Principal Analysis Options
12 21 4 3 1 2 9.81 5 5 0.01 18.17 1   ! Frame Control Parameters
1 1 1 0 1                             ! Output Intervals and Plotting Control Parameters
0 0                                   ! Iteration Control

NODES
1  0    0    1 1 1  0 0 0  3
2  0    2.74 0 0 1  0 0 0  0
3  1.22 2.74 0 0 1  0 0 0  3
4  1.22 0    1 1 1  0 0 0  3
5  0    2.89 0 0 1  2 0 0  3
6  0    5.63 0 0 1  0 0 0  0
7  1.22 5.63 0 0 1  0 0 0  3
8  1.22 2.89 0 0 1  3 0 0  3
9  0    5.78 0 0 1  6 0 0  3
10 0    8.52 0 0 1  0 0 0  0
11 1.22 8.52 0 0 1  0 0 0  3
12 1.22 5.78 0 0 1  7 0 0  3

ELEMENTS
1  1 1 2 0 0 3
2  2 2 3 0 0 3
3  1 3 4 0 0 3
4  3 2 4 0 0 0
5  3 3 1 0 0 3
6  4 2 5 0 0 3
7  2 5 8 0 0 3
8  4 8 3 0 0 3
9  1 5 6 0 0 3
10 2 6 7 0 0 3
11 1 7 8 0 0 3
12 3 6 8 0 0 0
13 3 7 5 0 0 3
14 4 6 9 0 0 3
15 2 9 12 0 0 3
16 4 12 7 0 0 3
17 1 9 10 0 0 3
18 2 10 11 0 0 3
19 1 11 12 0 0 3
20 3 10 12 0 0 0
21 3 11 9 0 0 3

PROPS                                 ! Member Properties Tables
1 SPRING                              ! columns
  1 0 0 0 1000000
2 SPRING                              ! beam
  1 0 0 0 1000000
3 SPRING                              ! brace: 75/305 9.5mm OSB Blais Table 5.1
  1 9 0 0 7915 0 0 0 0.19             ! Itype 1, Ihyst = Wayne Stewart, Ilos = No Strength Degradation
                                      (Not available for Stewart),IDAMG , Kx, Ky, GJ, WGT, RF
  22.78 -22.78                        ! FX+  FX-
  33.80 3.08 0.0 1.45 0 0 1.09 0.45 0 ! FU  FI  PTRI  PUNL  GAP+  GAP-  BETA  ALPHA Loop
4 SPRING
  1 0 0 0 254559                      ! Floor components

WEIGHT
1 0
2 21
3 21
4 0
5 0
6 21
7 21
8 0
9 0
10 8
11 8
12 0

LOAD
1 0 0 0
2 0 0 0
3 0 0 0
4 0 0 0
5 0 0 0
6 0 0 0
7 0 0 0
8 0 0 0
9 0 0 0
10 0 0 0
11 0 0 0
12 0 0 0

EQUAKE s03.dat
3 1 0.01 1 18.17 0 0 1.0

START
```

Figure F.13: Ruaumoko input file for three-storey model under ground motion record S03

```
Shear wall 152/305 95mm OSB 3 storey  ! Units kN, m and s
2 0 1 0 0 0 0 0 0                     ! Principal Analysis Options
12 21 4 3 1 2 9.81 5 5 0.01 18.17 1   ! Frame Control Parameters
1 1 1 0 1                             ! Output Intervals and Plotting Control Parameters
0 0                                   ! Iteration Control

NODES
1  0    0    1 1 1  0 0 0  3
2  0    2.74 0 0 1  0 0 0  0
3  1.22 2.74 0 0 1  0 0 0  3
4  1.22 0    1 1 1  0 0 0  3
5  0    2.89 0 0 1  2 0 0  3
6  0    5.63 0 0 1  0 0 0  0
7  1.22 5.63 0 0 1  0 0 0  3
8  1.22 2.89 0 0 1  3 0 0  3
9  0    5.78 0 0 1  6 0 0  3
10 0    8.52 0 0 1  0 0 0  0
11 1.22 8.52 0 0 1  0 0 0  3
12 1.22 5.78 0 0 1  7 0 0  3

ELEMENTS
1  1 1 2 0 0 3
2  2 2 3 0 0 3
3  1 3 4 0 0 3
4  3 2 4 0 0 0
5  3 3 1 0 0 3
6  4 2 5 0 0 3
7  2 5 8 0 0 3
8  4 8 3 0 0 3
9  1 5 6 0 0 3
10 2 6 7 0 0 3
11 1 7 8 0 0 3
12 3 6 8 0 0 0
13 3 7 5 0 0 3
14 4 6 9 0 0 3
15 2 9 12 0 0 3
16 4 12 7 0 0 3
17 1 9 10 0 0 3
18 2 10 11 0 0 3
19 1 11 12 0 0 3
20 3 10 12 0 0 0
21 3 11 9 0 0 3

PROPS                                 ! Member Properties Tables
1 SPRING                              ! columns
  1 0 0 0 1000000
2 SPRING                              ! beam
  1 0 0 0 1000000
3 SPRING                              ! brace: 75/305 9.5mm OSB Blais Table 5.1
  1 9 0 0 7915 0 0 0 0.19             ! Itype 1, Ihyst = Wayne Stewart, Ilos = No Strength Degradation
                                      (Not available for Stewart),IDAMG , Kx, Ky, GJ, WGT, RF
  22.78 -22.78                        ! FX+ FX-
  33.80 3.08 0.0 1.45 0 0 1.09 0.45 0 ! FU FI PTRI PUNL GAP+ GAP- BETA ALPHA Loop
4 SPRING
  1 0 0 0 254559                      ! Floor components

WEIGHT
1 0
2 21
3 21
4 0
5 0
6 21
7 21
8 0
9 0
10 8
11 8
12 0

LOAD
1 0 0 0
2 0 0 0
3 0 0 0
4 0 0 0
5 0 0 0
6 0 0 0
7 0 0 0
8 0 0 0
9 0 0 0
10 0 0 0
11 0 0 0
12 0 0 0

EQUAKE s04.dat
3 1 0.01 1 18.17 0 0 1.0

START
```

Figure F.14: Ruaumoko input file for three-storey model under ground motion record S04

```
Shear wall 152/305 95mm OSB 3 storey  ! Units kN, m and s
2 0 1 0 0 0 0 0 0                     ! Principal Analysis Options
12 21 4 3 1 2 9.81 5 5 0.005 29.975 1 ! Frame Control Parameters
1 1 1 0 1                             ! Output Intervals and Plotting Control Parameters
0 0                                   ! Iteration Control

NODES
1  0    0    1 1 1  0 0 0  3
2  0    2.74 0 0 1  0 0 0  0
3  1.22 2.74 0 0 1  0 0 0  3
4  1.22 0    1 1 1  0 0 0  3
5  0    2.89 0 0 1  2 0 0  3
6  0    5.63 0 0 1  0 0 0  0
7  1.22 5.63 0 0 1  0 0 0  3
8  1.22 2.89 0 0 1  3 0 0  3
9  0    5.78 0 0 1  6 0 0  3
10 0    8.52 0 0 1  0 0 0  0
11 1.22 8.52 0 0 1  0 0 0  3
12 1.22 5.78 0 0 1  7 0 0  3

ELEMENTS
1  1 1 2 0 0 3
2  2 2 3 0 0 3
3  1 3 4 0 0 3
4  3 2 4 0 0 0
5  3 3 1 0 0 3
6  4 2 5 0 0 3
7  2 5 8 0 0 3
8  4 8 3 0 0 3
9  1 5 6 0 0 3
10 2 6 7 0 0 3
11 1 7 8 0 0 3
12 3 6 8 0 0 0
13 3 7 5 0 0 3
14 4 6 9 0 0 3
15 2 9 12 0 0 3
16 4 12 7 0 0 3
17 1 9 10 0 0 3
18 2 10 11 0 0 3
19 1 11 12 0 0 3
20 3 10 12 0 0 0
21 3 11 9 0 0 3

PROPS                                 ! Member Properties Tables
1 SPRING                              ! columns
  1 0 0 0 1000000
2 SPRING                              ! beam
  1 0 0 0 1000000
3 SPRING                              ! brace: 75/305 9.5mm OSB Blais Table 5.1
  1 9 0 0 7915 0 0 0 0.19             ! Itype 1, Ihyst = Wayne Stewart, Ilos = No Strength Degradation
                                      (Not available for Stewart),IDAMG , Kx, Ky, GJ, WGT, RF
  22.78 -22.78                        ! FX+ FX-
  33.80 3.08 0.0 1.45 0 0 1.09 0.45 0 ! FU FI PTRI PUNL GAP+ GAP- BETA ALPHA Loop
4 SPRING
  1 0 0 0 254559                      ! Floor components

WEIGHT
1 0
2 21
3 21
4 0
5 0
6 21
7 21
8 0
9 0
10 8
11 8
12 0

LOAD
1 0 0 0
2 0 0 0
3 0 0 0
4 0 0 0
5 0 0 0
6 0 0 0
7 0 0 0
8 0 0 0
9 0 0 0
10 0 0 0
11 0 0 0
12 0 0 0

EQUAKE s05.dat
3 1 0.005 1 29.975 0 0 1.0

START
```

Figure F.15: Ruaumoko input file for three-storey model under ground motion record S05

```
Shear wall 152/305 95mm OSB 3 storey ! Units kN, m and s
2 0 1 0 0 0 0 0 0                    ! Principal Analysis Options
12 21 4 3 1 2 9.81 5 5 0.02 39.98 1  ! Frame Control Parameters
1 1 1 0 1                            ! Output Intervals and Plotting Control Parameters
0 0                                  ! Iteration Control

NODES
1  0    0    1 1 1  0 0 0  3
2  0    2.74 0 0 1  0 0 0  0
3  1.22 2.74 0 0 1  0 0 0  3
4  1.22 0    1 1 1  0 0 0  3
5  0    2.89 0 0 1  2 0 0  3
6  0    5.63 0 0 1  0 0 0  0
7  1.22 5.63 0 0 1  0 0 0  3
8  1.22 2.89 0 0 1  3 0 0  3
9  0    5.78 0 0 1  6 0 0  3
10 0    8.52 0 0 1  0 0 0  0
11 1.22 8.52 0 0 1  0 0 0  3
12 1.22 5.78 0 0 1  7 0 0  3

ELEMENTS
1  1 1 2 0 0 3
2  2 2 3 0 0 3
3  1 3 4 0 0 3
4  3 2 4 0 0 0
5  3 3 1 0 0 3
6  4 2 5 0 0 3
7  2 5 8 0 0 3
8  4 8 3 0 0 3
9  1 5 6 0 0 3
10 2 6 7 0 0 3
11 1 7 8 0 0 3
12 3 6 8 0 0 0
13 3 7 5 0 0 3
14 4 6 9 0 0 3
15 2 9 12 0 0 3
16 4 12 7 0 0 3
17 1 9 10 0 0 3
18 2 10 11 0 0 3
19 1 11 12 0 0 3
20 3 10 12 0 0 0
21 3 11 9 0 0 3

PROPS                                ! Member Properties Tables
1 SPRING                             ! columns
  1 0 0 0 1000000
2 SPRING                             ! beam
  1 0 0 0 1000000
3 SPRING                             ! brace: 75/305 9.5mm OSB Blais Table 5.1
  1 9 0 0 7915 0 0 0 0.19            ! Itype 1, Ihyst = Wayne Stewart, Ilos = No Strength Degradation
                                     (Not available for Stewart),IDAMG , Kx, Ky, GJ, WGT, RF
  22.78 -22.78                       ! FX+ FX-
  33.80 3.08 0.0 1.45 0 0 1.09 0.45 0 ! FU FI PTRI PUNL GAP+ GAP- BETA ALPHA Loop
4 SPRING
  1 0 0 0 254559                     ! Floor components

WEIGHT
1 0
2 21
3 21
4 0
5 0
6 21
7 21
8 0
9 0
10 8
11 8
12 0

LOAD
1 0 0 0
2 0 0 0
3 0 0 0
4 0 0 0
5 0 0 0
6 0 0 0
7 0 0 0
8 0 0 0
9 0 0 0
10 0 0 0
11 0 0 0
12 0 0 0

EQUAKE s06.dat
3 1 0.02 1 39.98 0 0 1.0

START
```

Figure F.16: Ruaumoko input file for three-storey model under ground motion record S06

```
Shear wall 152/305 95mm OSB 3 storey  ! Units kN, m and s
2 0 1 0 0 0 0 0 0                     ! Principal Analysis Options
12 21 4 3 1 2 9.81 5 5 0.02 39.98 1   ! Frame Control Parameters
1 1 1 0 1                             ! Output Intervals and Plotting Control Parameters
0 0                                   ! Iteration Control

NODES
1  0    0    1 1 1  0 0 0  3
2  0    2.74 0 0 1  0 0 0  0
3  1.22 2.74 0 0 1  0 0 0  3
4  1.22 0    1 1 1  0 0 0  3
5  0    2.89 0 0 1  2 0 0  3
6  0    5.63 0 0 1  0 0 0  0
7  1.22 5.63 0 0 1  0 0 0  3
8  1.22 2.89 0 0 1  3 0 0  3
9  0    5.78 0 0 1  6 0 0  3
10 0    8.52 0 0 1  0 0 0  0
11 1.22 8.52 0 0 1  0 0 0  3
12 1.22 5.78 0 0 1  7 0 0  3

ELEMENTS
1  1 1 2 0 0 3
2  2 2 3 0 0 3
3  1 3 4 0 0 3
4  3 2 4 0 0 0
5  3 3 1 0 0 3
6  4 2 5 0 0 3
7  2 5 8 0 0 3
8  4 8 3 0 0 3
9  1 5 6 0 0 3
10 2 6 7 0 0 3
11 1 7 8 0 0 3
12 3 6 8 0 0 0
13 3 7 5 0 0 3
14 4 6 9 0 0 3
15 2 9 12 0 0 3
16 4 12 7 0 0 3
17 1 9 10 0 0 3
18 2 10 11 0 0 3
19 1 11 12 0 0 3
20 3 10 12 0 0 0
21 3 11 9 0 0 3

PROPS                                 ! Member Properties Tables
1 SPRING                              ! columns
  1 0 0 0 1000000
2 SPRING                              ! beam
  1 0 0 0 1000000
3 SPRING                              ! brace: 75/305 9.5mm OSB Blais Table 5.1
  1 9 0 0 7915 0 0 0 0.19             ! Itype 1, Ihyst = Wayne Stewart, Ilos = No Strength Degradation
                                      (Not available for Stewart),IDAMG , Kx, Ky, GJ, WGT, RF
  22.78 -22.78                        ! FX+  FX-
  33.80 3.08 0.0 1.45 0 0 1.09 0.45 0 ! FU  FI  PTRI  PUNL  GAP+  GAP-  BETA  ALPHA Loop
4 SPRING
  1 0 0 0 254559                      ! Floor components

WEIGHT
1 0
2 21
3 21
4 0
5 0
6 21
7 21
8 0
9 0
10 8
11 8
12 0

LOAD
1 0 0 0
2 0 0 0
3 0 0 0
4 0 0 0
5 0 0 0
6 0 0 0
7 0 0 0
8 0 0 0
9 0 0 0
10 0 0 0
11 0 0 0
12 0 0 0

EQUAKE s07.dat
3 1 0.02 1 39.98 0 0 1.0

START
```

Figure F.17: Ruaumoko input file for three-storey model under ground motion record S07

```
Shear wall 152/305 95mm OSB 3 storey  ! Units kN, m and s
2 0 1 0 0 0 0 0 0                     ! Principal Analysis Options
12 21 4 3 1 2 9.81 5 5 0.005 39.6 1   ! Frame Control Parameters
1 1 1 0 1                             ! Output Intervals and Plotting Control Parameters
0 0                                   ! Iteration Control

NODES
1  0    0    1 1 1  0 0 0  3
2  0    2.74 0 0 1  0 0 0  0
3  1.22 2.74 0 0 1  0 0 0  3
4  1.22 0    1 1 1  0 0 0  3
5  0    2.89 0 0 1  2 0 0  3
6  0    5.63 0 0 1  0 0 0  0
7  1.22 5.63 0 0 1  0 0 0  3
8  1.22 2.89 0 0 1  3 0 0  3
9  0    5.78 0 0 1  6 0 0  3
10 0    8.52 0 0 1  0 0 0  0
11 1.22 8.52 0 0 1  0 0 0  3
12 1.22 5.78 0 0 1  7 0 0  3

ELEMENTS
1  1 1 2 0 0 3
2  2 2 3 0 0 3
3  1 3 4 0 0 3
4  3 2 4 0 0 0
5  3 3 1 0 0 3
6  4 2 5 0 0 3
7  2 5 8 0 0 3
8  4 8 3 0 0 3
9  1 5 6 0 0 3
10 2 6 7 0 0 3
11 1 7 8 0 0 3
12 3 6 8 0 0 0
13 3 7 5 0 0 3
14 4 6 9 0 0 3
15 2 9 12 0 0 3
16 4 12 7 0 0 3
17 1 9 10 0 0 3
18 2 10 11 0 0 3
19 1 11 12 0 0 3
20 3 10 12 0 0 0
21 3 11 9 0 0 3

PROPS                                 ! Member Properties Tables
1 SPRING                              ! columns
  1 0 0 0 1000000
2 SPRING                              ! beam
  1 0 0 0 1000000
3 SPRING                              ! brace: 75/305 9.5mm OSB Blais Table 5.1
  1 9 0 0 7915 0 0 0 0.19             ! Itype 1, Ihyst = Wayne Stewart, Ilos = No Strength Degradation
                                      (Not available for Stewart),IDAMG , Kx, Ky, GJ, WGT, RF
  22.78 -22.78                        ! FX+  FX-
  33.80 3.08 0.0 1.45 0 0 1.09 0.45 0 ! FU  FI  PTRI  PUNL  GAP+  GAP-  BETA  ALPHA Loop
4 SPRING
  1 0 0 0 254559                      ! Floor components

WEIGHT
1 0
2 21
3 21
4 0
5 0
6 21
7 21
8 0
9 0
10 8
11 8
12 0

LOAD
1 0 0 0
2 0 0 0
3 0 0 0
4 0 0 0
5 0 0 0
6 0 0 0
7 0 0 0
8 0 0 0
9 0 0 0
10 0 0 0
11 0 0 0
12 0 0 0

EQUAKE s08.dat
3 1 0.005 1 39.6 0 0 1.0

START
```

Figure F.18: Ruaumoko input file for three-storey model under ground motion record S08

```
Shear wall 152/305 95mm OSB 3 storey  ! Units kN, m and s
2 0 1 0 0 0 0 0 0                     ! Principal Analysis Options
12 21 4 3 1 2 9.81 5 5 0.02 39.98 1   ! Frame Control Parameters
1 1 1 0 1                             ! Output Intervals and Plotting Control Parameters
0 0                                   ! Iteration Control

NODES
1  0    0    1 1 1  0 0 0  3
2  0    2.74 0 0 1  0 0 0  0
3  1.22 2.74 0 0 1  0 0 0  3
4  1.22 0    1 1 1  0 0 0  3
5  0    2.89 0 0 1  2 0 0  3
6  0    5.63 0 0 1  0 0 0  0
7  1.22 5.63 0 0 1  0 0 0  3
8  1.22 2.89 0 0 1  3 0 0  3
9  0    5.78 0 0 1  6 0 0  3
10 0    8.52 0 0 1  0 0 0  0
11 1.22 8.52 0 0 1  0 0 0  3
12 1.22 5.78 0 0 1  7 0 0  3

ELEMENTS
1  1 1 2 0 0 3
2  2 2 3 0 0 3
3  1 3 4 0 0 3
4  3 2 4 0 0 0
5  3 3 1 0 0 3
6  4 2 5 0 0 3
7  2 5 8 0 0 3
8  4 8 3 0 0 3
9  1 5 6 0 0 3
10 2 6 7 0 0 3
11 1 7 8 0 0 3
12 3 6 8 0 0 0
13 3 7 5 0 0 3
14 4 6 9 0 0 3
15 2 9 12 0 0 3
16 4 12 7 0 0 3
17 1 9 10 0 0 3
18 2 10 11 0 0 3
19 1 11 12 0 0 3
20 3 10 12 0 0 0
21 3 11 9 0 0 3

PROPS                                 ! Member Properties Tables
1 SPRING                              ! columns
  1 0 0 0 1000000
2 SPRING                              ! beam
  1 0 0 0 1000000
3 SPRING                              ! brace: 75/305 9.5mm OSB Blais Table 5.1
  1 9 0 0 7915 0 0 0 0.19             ! Itype 1, Ihyst = Wayne Stewart, Ilos = No Strength Degradation
                                      (Not available for Stewart),IDAMG , Kx, Ky, GJ, WGT, RF
  22.78 -22.78                        ! FX+  FX-
  33.80 3.08 0.0 1.45 0 0 1.09 0.45 0 ! FU  FI  PTRI  PUNL  GAP+  GAP-  BETA  ALPHA Loop
4 SPRING
  1 0 0 0 254559                      ! Floor components

WEIGHT
1 0
2 21
3 21
4 0
5 0
6 21
7 21
8 0
9 0
10 8
11 8
12 0

LOAD
1 0 0 0
2 0 0 0
3 0 0 0
4 0 0 0
5 0 0 0
6 0 0 0
7 0 0 0
8 0 0 0
9 0 0 0
10 0 0 0
11 0 0 0
12 0 0 0

EQUAKE s09.dat
3 1 0.02 1 39.98 0 0 1.0

START
```

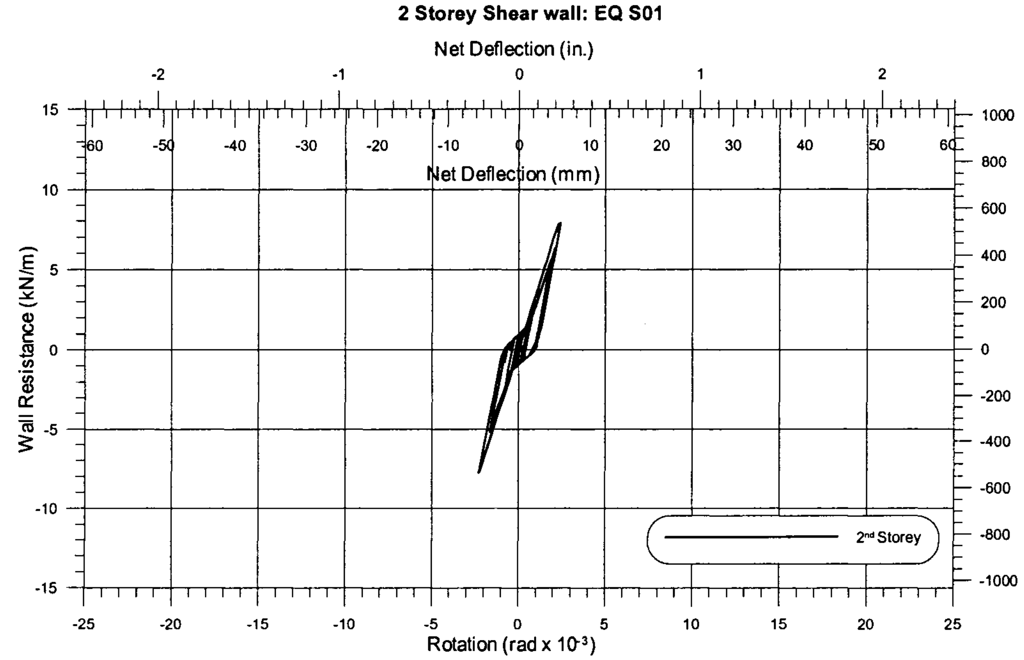
Figure F.19: Ruaumoko input file for three-storey model under ground motion record S09

```
Shear wall 152/305 95mm OSB 3 storey  ! Units kN, m and s
2 0 1 0 0 0 0 0 0                     ! Principal Analysis Options
12 21 4 3 1 2 9.81 5 5 0.02 89.04 1   ! Frame Control Parameters
1 1 1 0 1                             ! Output Intervals and Plotting Control Parameters
0 0                                   ! Iteration Control

NODES
1  0    0    1 1 1  0 0 0  3
2  0    2.74 0 0 1  0 0 0  0
3  1.22 2.74 0 0 1  0 0 0  3
4  1.22 0    1 1 1  0 0 0  3
5  0    2.89 0 0 1  2 0 0  3
6  0    5.63 0 0 1  0 0 0  0
7  1.22 5.63 0 0 1  0 0 0  3
8  1.22 2.89 0 0 1  3 0 0  3
9  0    5.78 0 0 1  6 0 0  3
10 0    8.52 0 0 1  0 0 0  0
11 1.22 8.52 0 0 1  0 0 0  3
12 1.22 5.78 0 0 1  7 0 0  3

ELEMENTS
1  1 1 2 0 0 3
2  2 2 3 0 0 3
3  1 3 4 0 0 3
4  3 2 4 0 0 0
5  3 3 1 0 0 3
6  4 2 5 0 0 3
7  2 5 8 0 0 3
8  4 8 3 0 0 3
9  1 5 6 0 0 3
10 2 6 7 0 0 3
11 1 7 8 0 0 3
12 3 6 8 0 0 0
13 3 7 5 0 0 3
14 4 6 9 0 0 3
15 2 9 12 0 0 3
16 4 12 7 0 0 3
17 1 9 10 0 0 3
18 2 10 11 0 0 3
19 1 11 12 0 0 3
20 3 10 12 0 0 0
21 3 11 9 0 0 3

PROPS                                 ! Member Properties Tables
1 SPRING                              ! columns
  1 0 0 0 1000000
2 SPRING                              ! beam
  1 0 0 0 1000000
3 SPRING                              ! brace: 75/305 9.5mm OSB Blais Table 5.1
  1 9 0 0 7915 0 0 0 0.19             ! Itype 1, Ihyst = Wayne Stewart, Ilos = No Strength Degradation
                                      (Not available for Stewart),IDAMG , Kx, Ky, GJ, WGT, RF
  22.78 -22.78                        ! FX+  FX-
  33.80 3.08 0.0 1.45 0 0 1.09 0.45 0 ! FU  FI  PTRI  PUNL  GAP+  GAP-  BETA  ALPHA Loop
4 SPRING
  1 0 0 0 254559                      ! Floor components

WEIGHT
1 0
2 21
3 21
4 0
5 0
6 21
7 21
8 0
9 0
10 8
11 8
12 0

LOAD
1 0 0 0
2 0 0 0
3 0 0 0
4 0 0 0
5 0 0 0
6 0 0 0
7 0 0 0
8 0 0 0
9 0 0 0
10 0 0 0
11 0 0 0
12 0 0 0

EQUAKE s10.dat
3 1 0.02 1 89.04 0 0 1.0

START
```

Figure F.20: Ruaumoko input file for three-storey model under ground motion record S10

APPENDIX 'G'
HYSTERESES AND TIME HISTORIES FOR TWO- AND THREE-STOREY MODELS

2-Storey Model



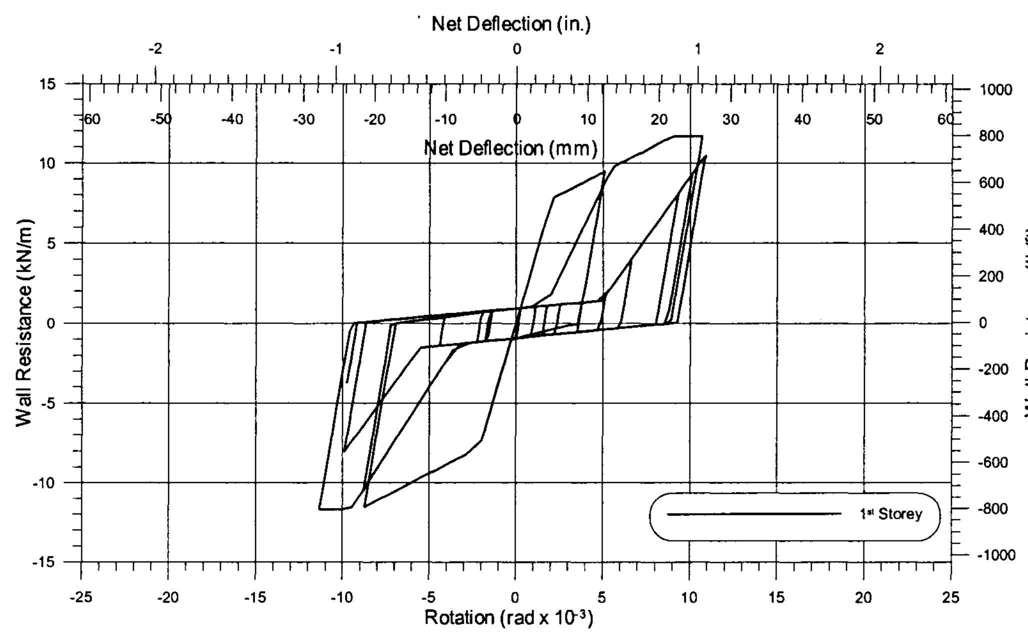


Figure G.1: Resistance vs. displacement hystereses of the two-storey shear wall under ground motion record S01

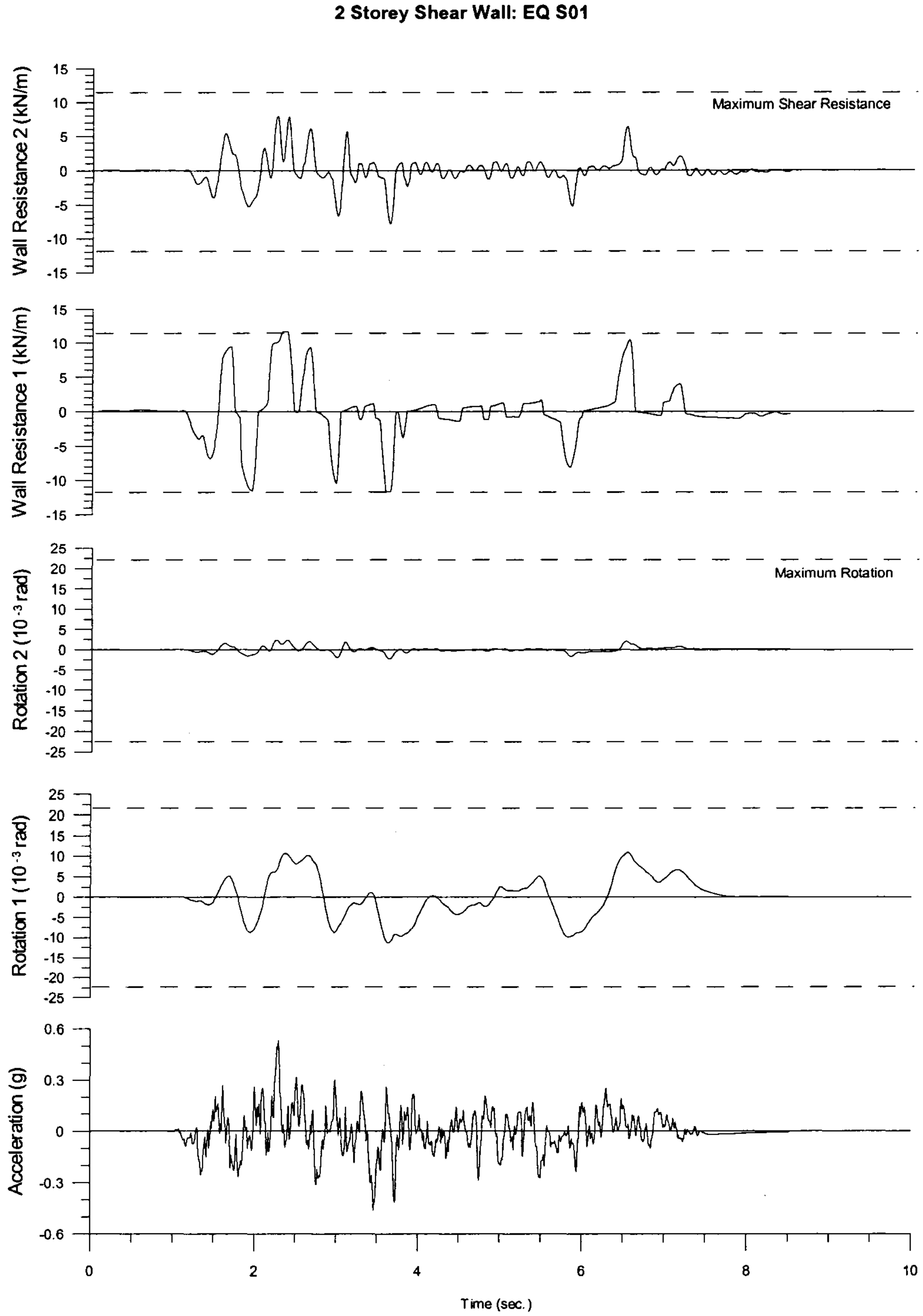
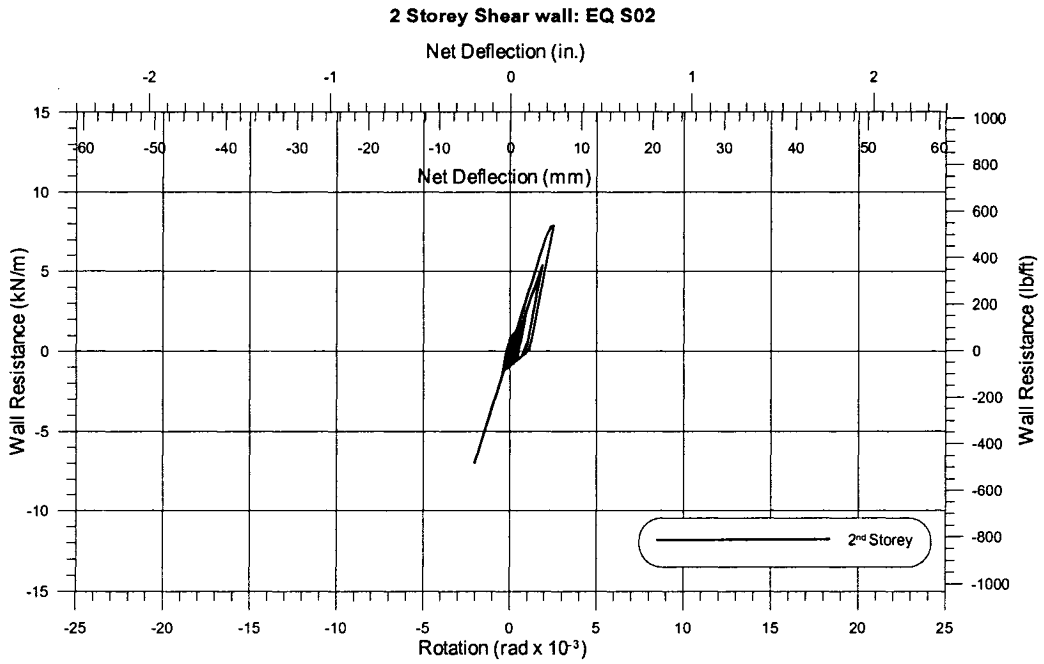


Figure G.2: Resistance and displacement time histories of the two-storey shear wall under ground motion record S01



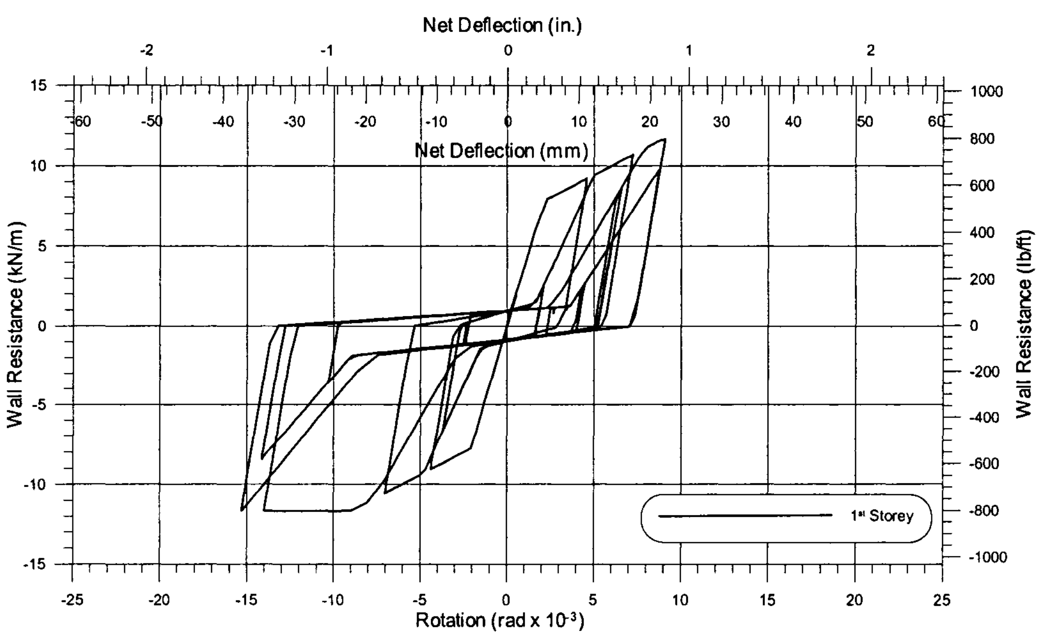


Figure G.3: Resistance vs. displacement hystereses of the two-storey shear wall under ground motion record S02

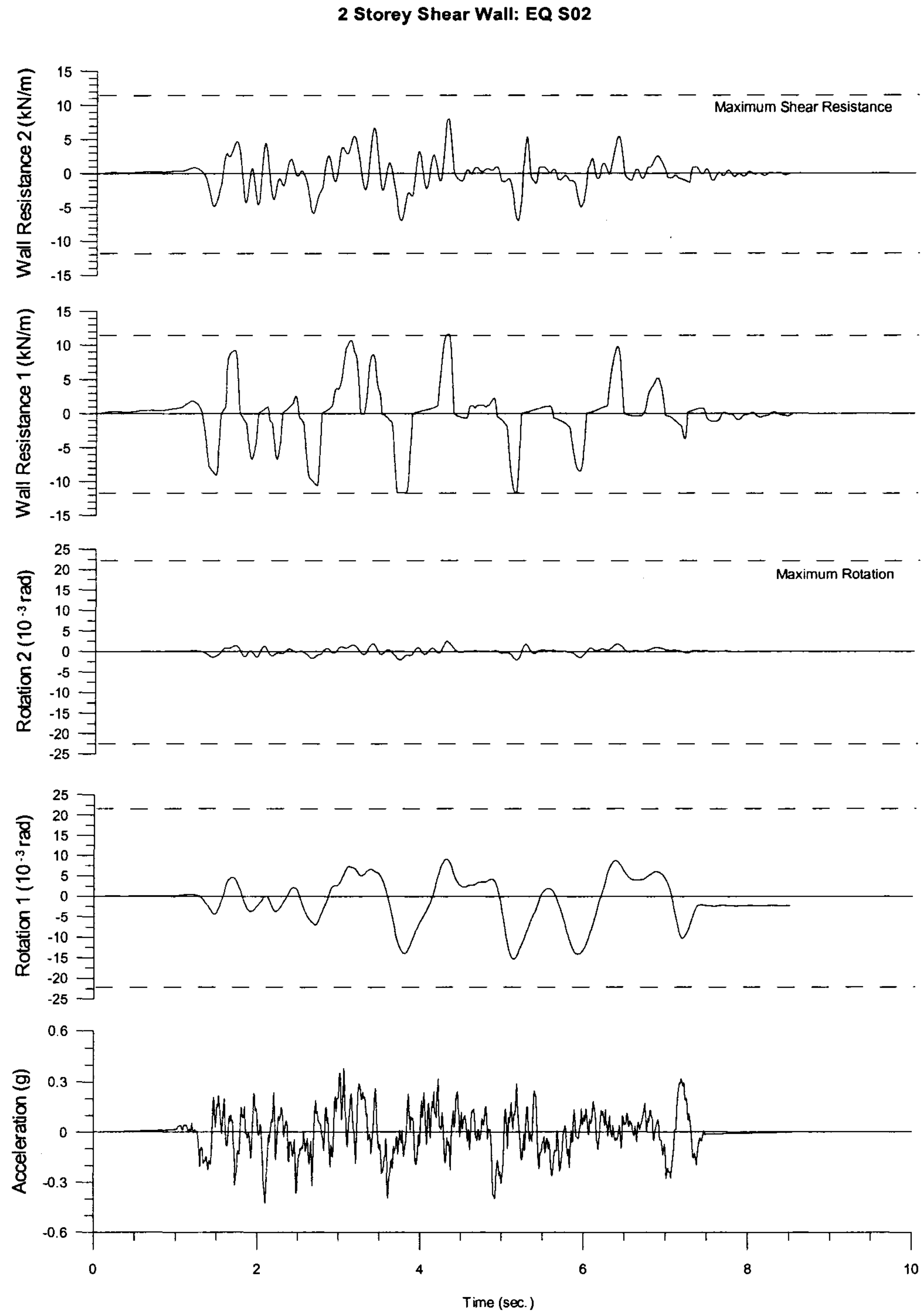
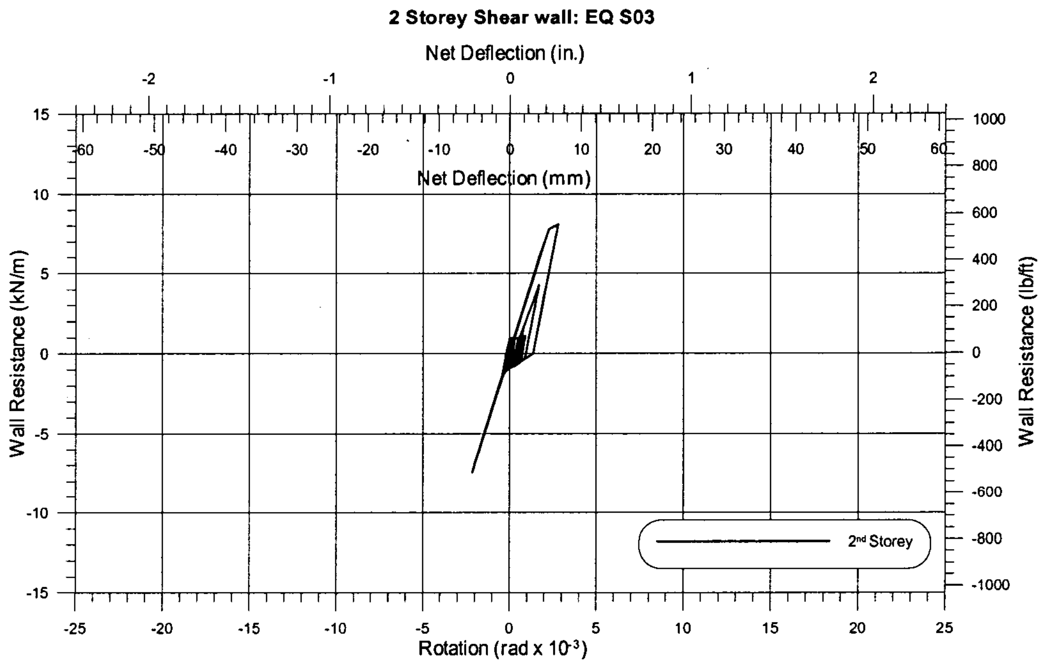


Figure G.4: Resistance and displacement time histories of the two-storey shear wall under ground motion record S02



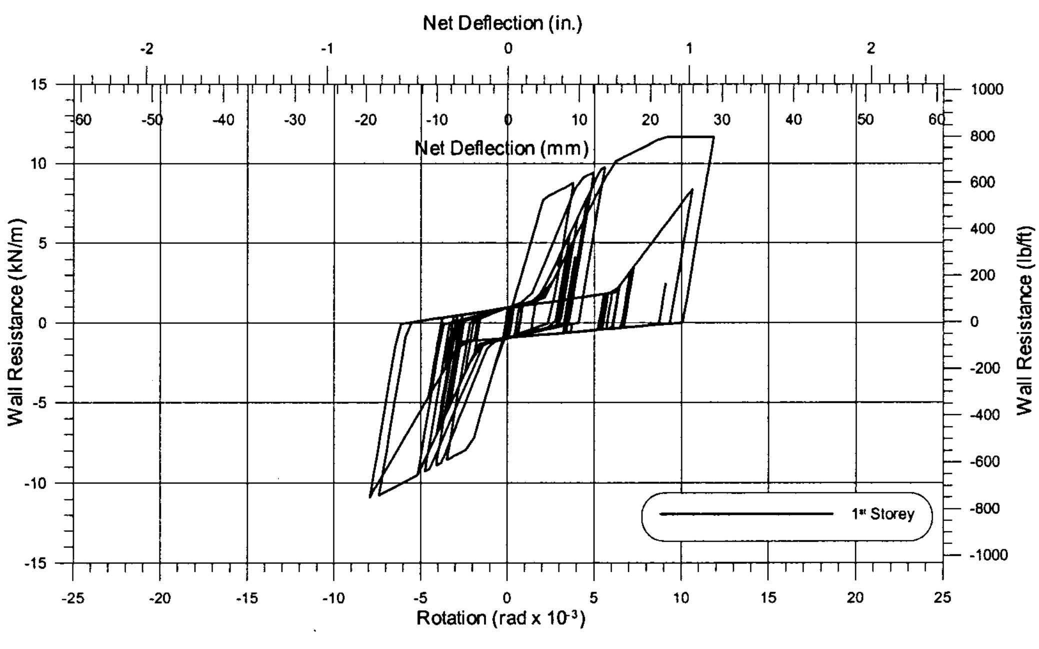


Figure G.5: Resistance vs. displacement hystereses of the two-storey shear wall under ground motion record S03

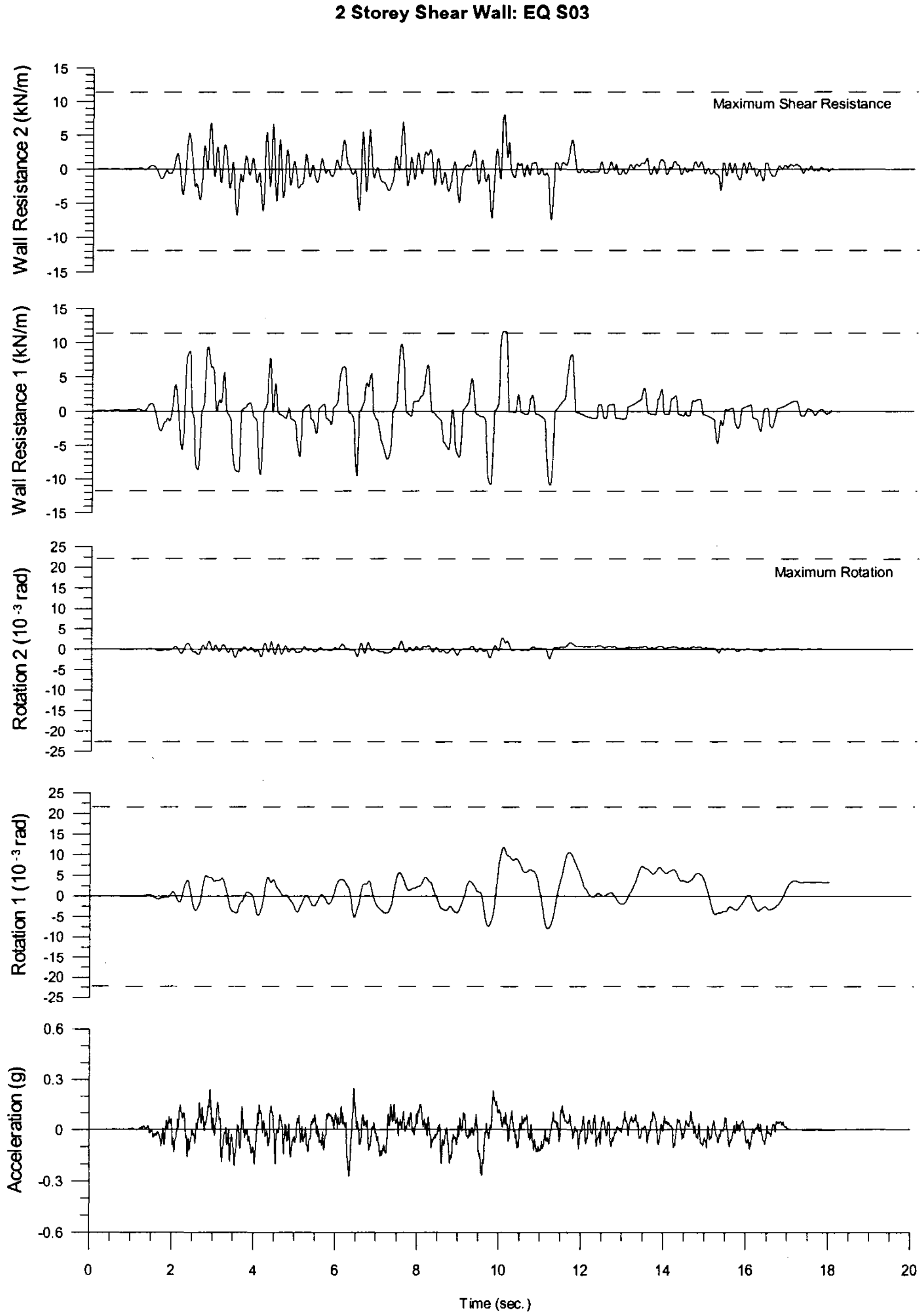
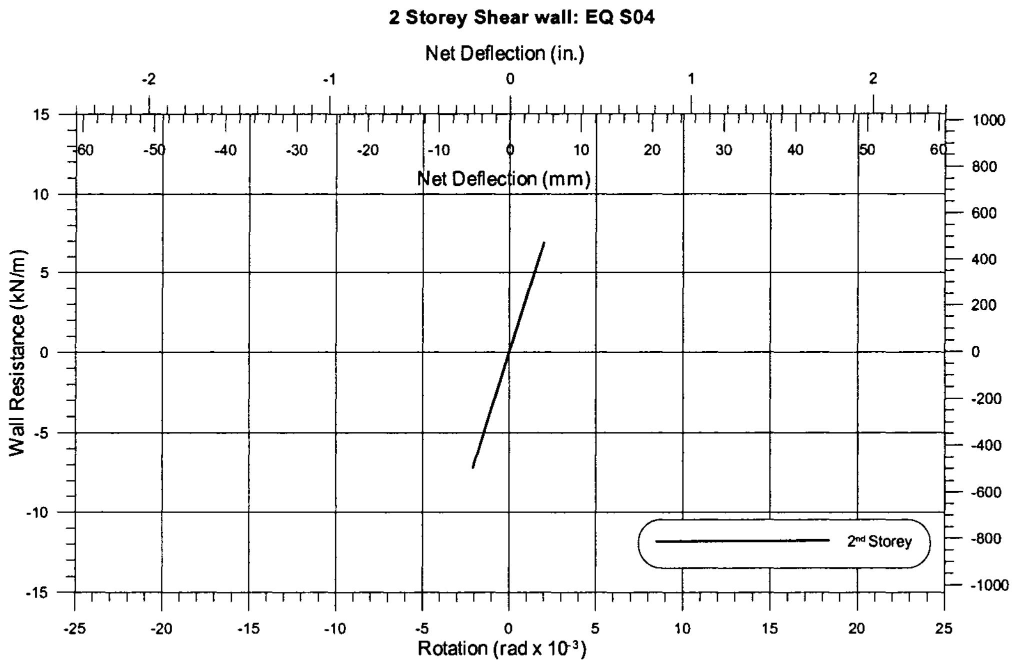


Figure G.6: Resistance and displacement time histories of the two-storey shear wall under ground motion record S03



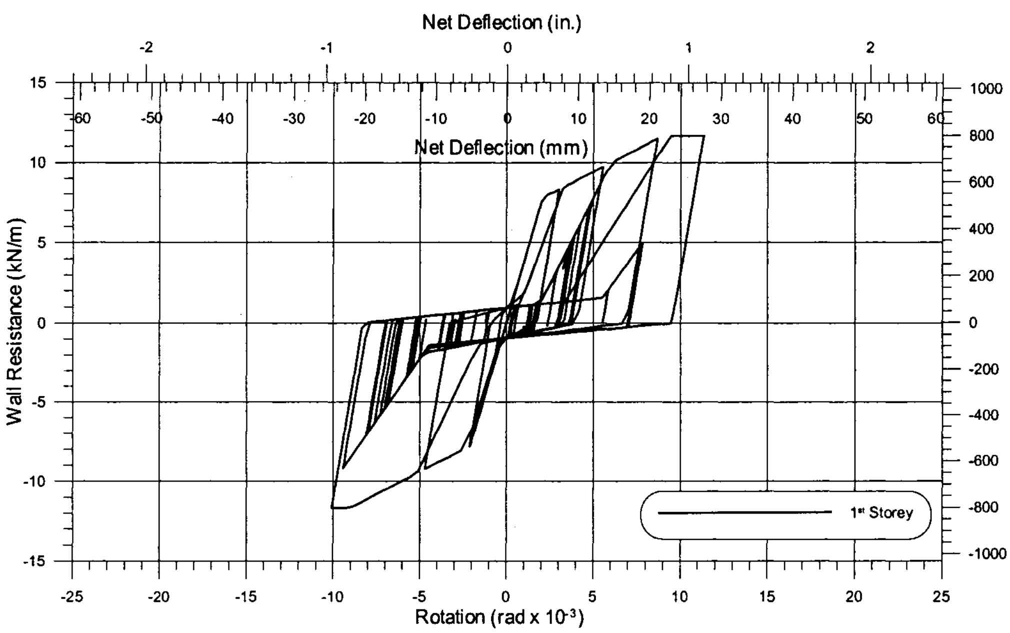


Figure G.7: Resistance vs. displacement hystereses of the two-storey shear wall under ground motion record S04

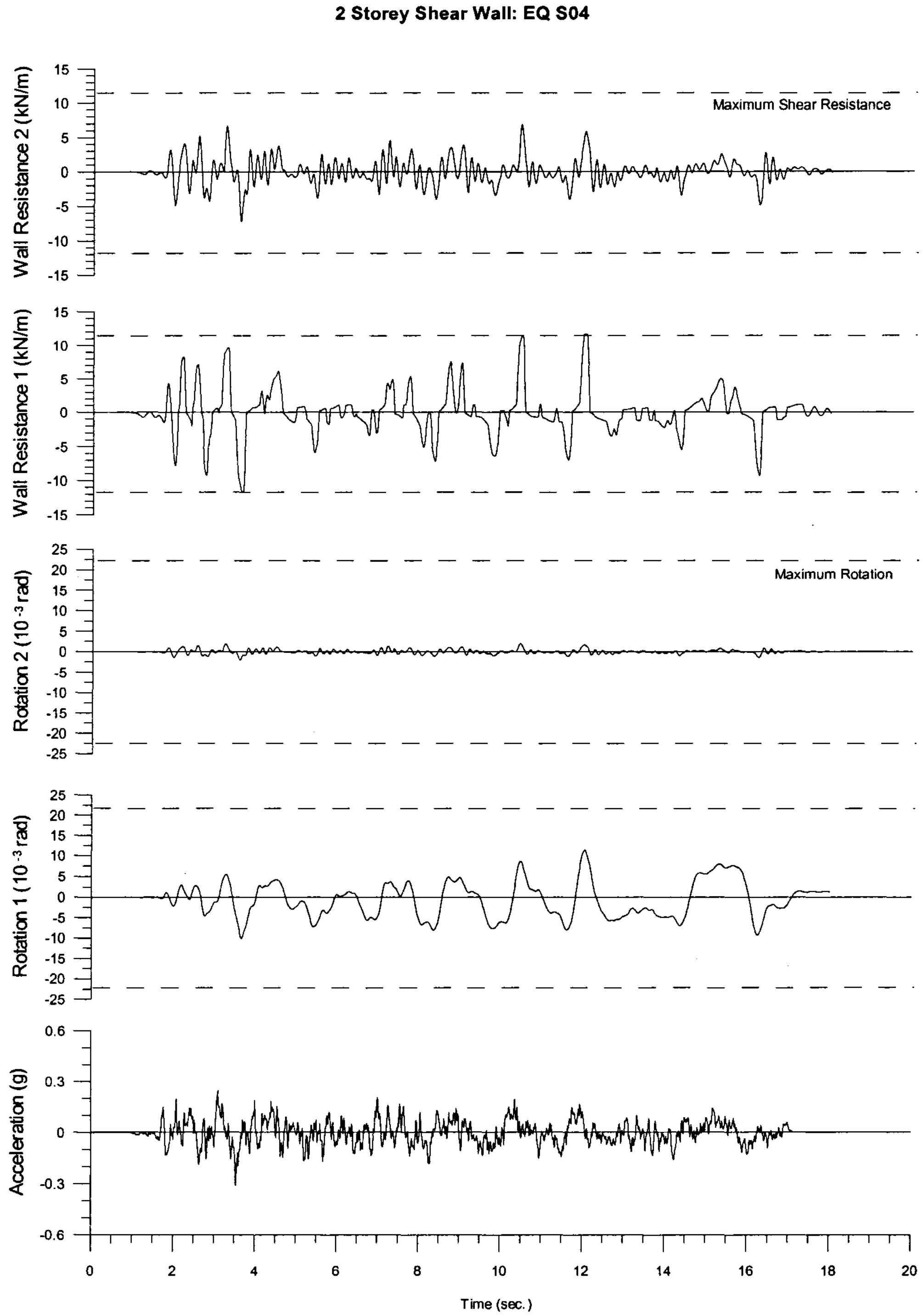
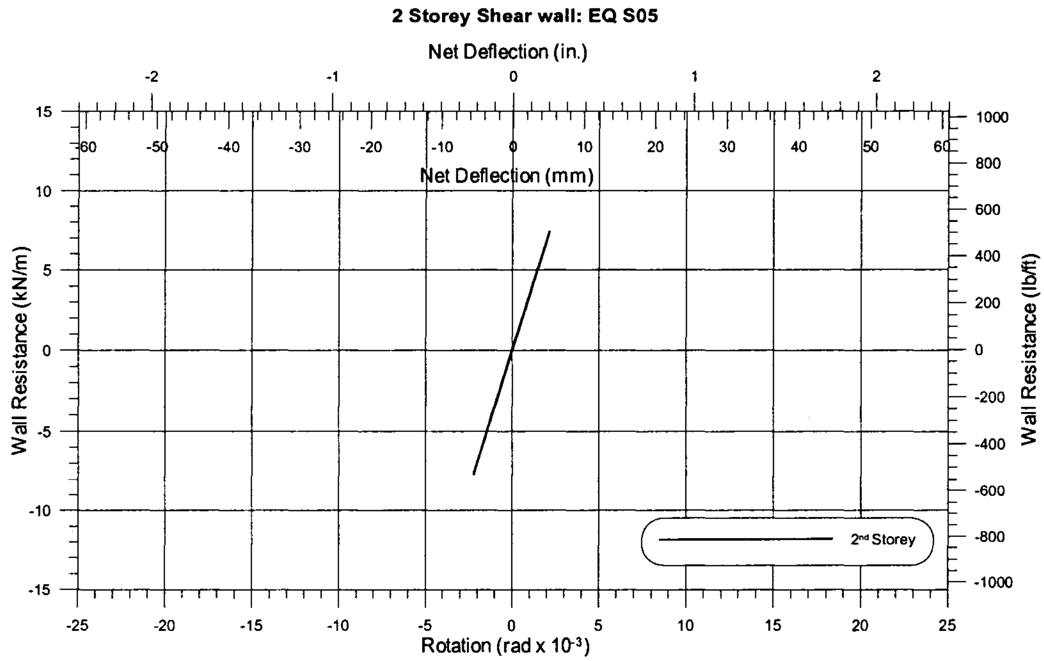


Figure G.8: Resistance and displacement time histories of the two-storey shear wall under ground motion record S04



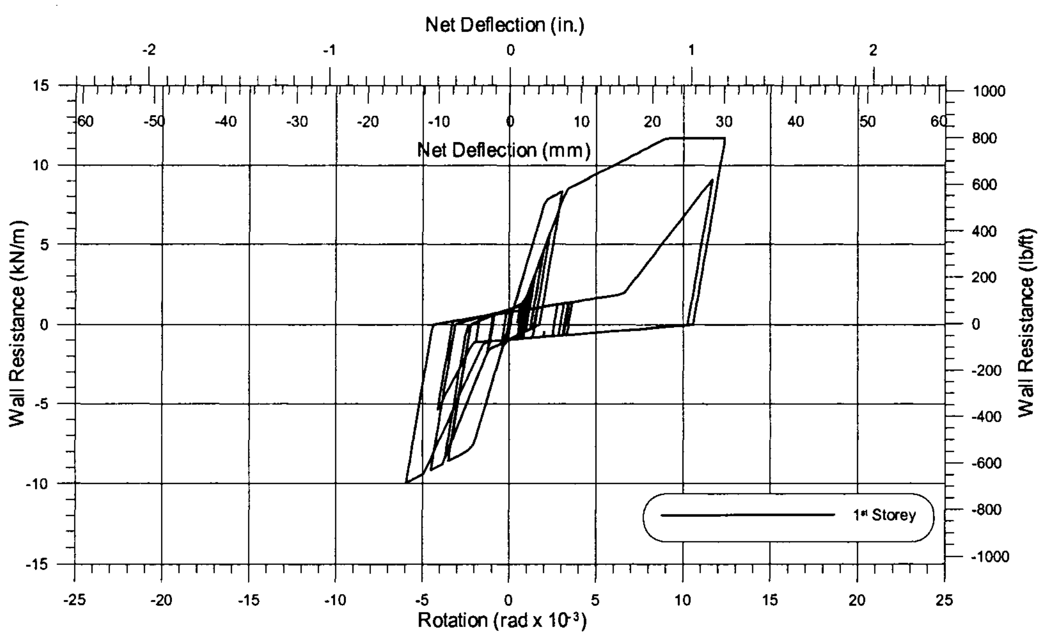


Figure G.9: Resistance vs. displacement hystereses of the two-storey shear wall under ground motion record S05

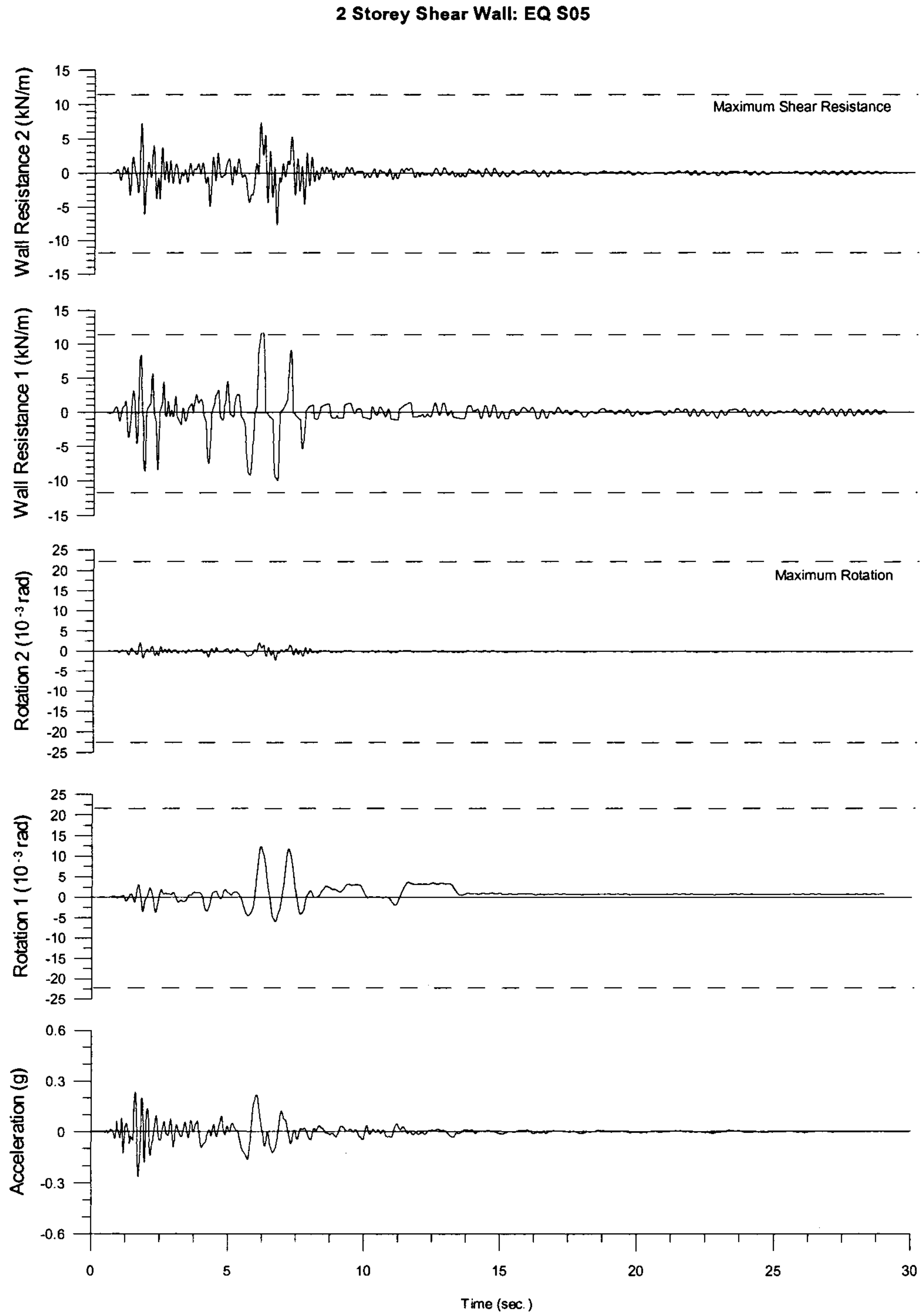


Figure G.10: Resistance and displacement time histories of the two-storey shear wall under ground motion record S05

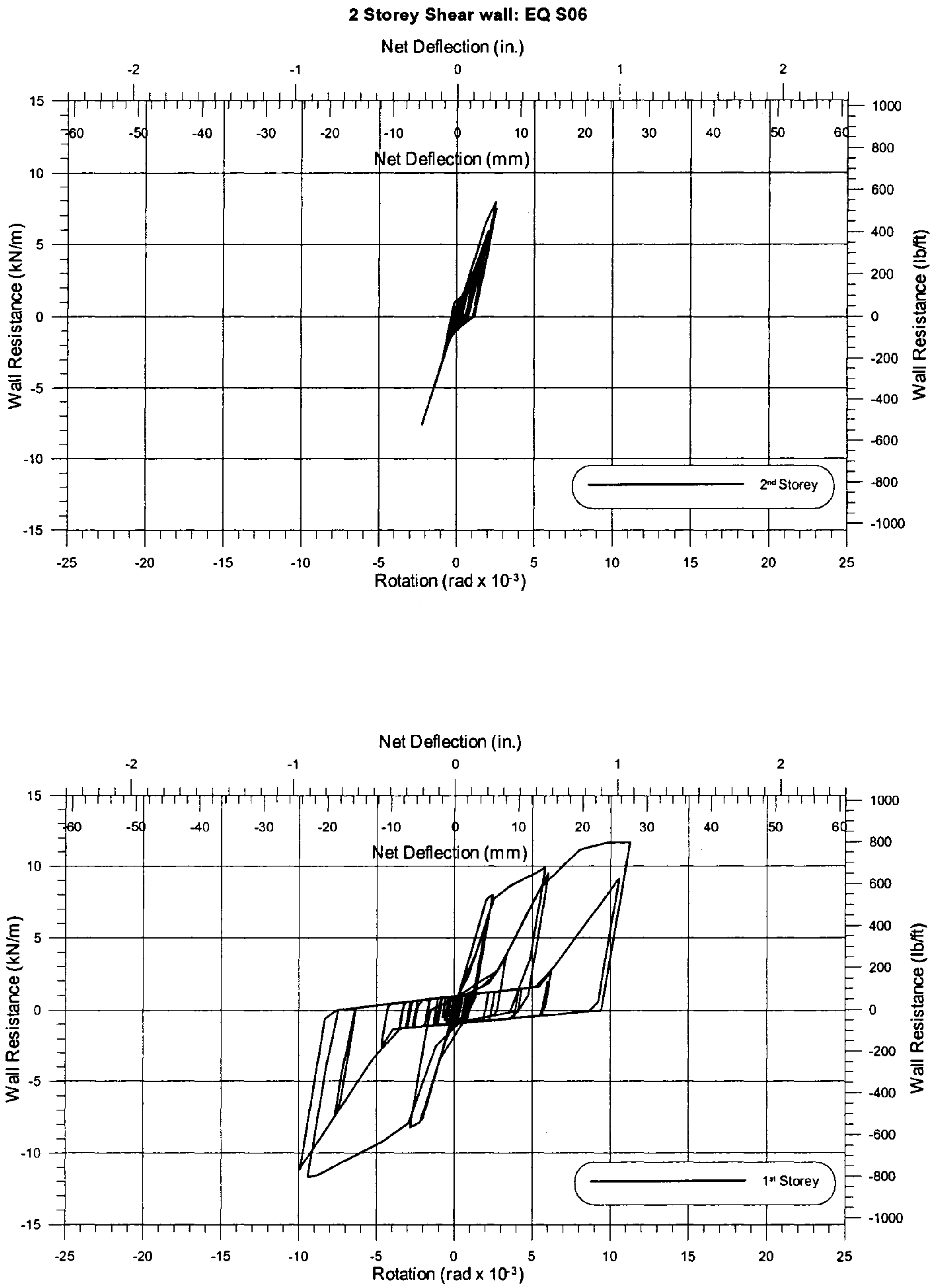


Figure G.11: Resistance vs. displacement hystereses of the two-storey shear wall under ground motion record S06

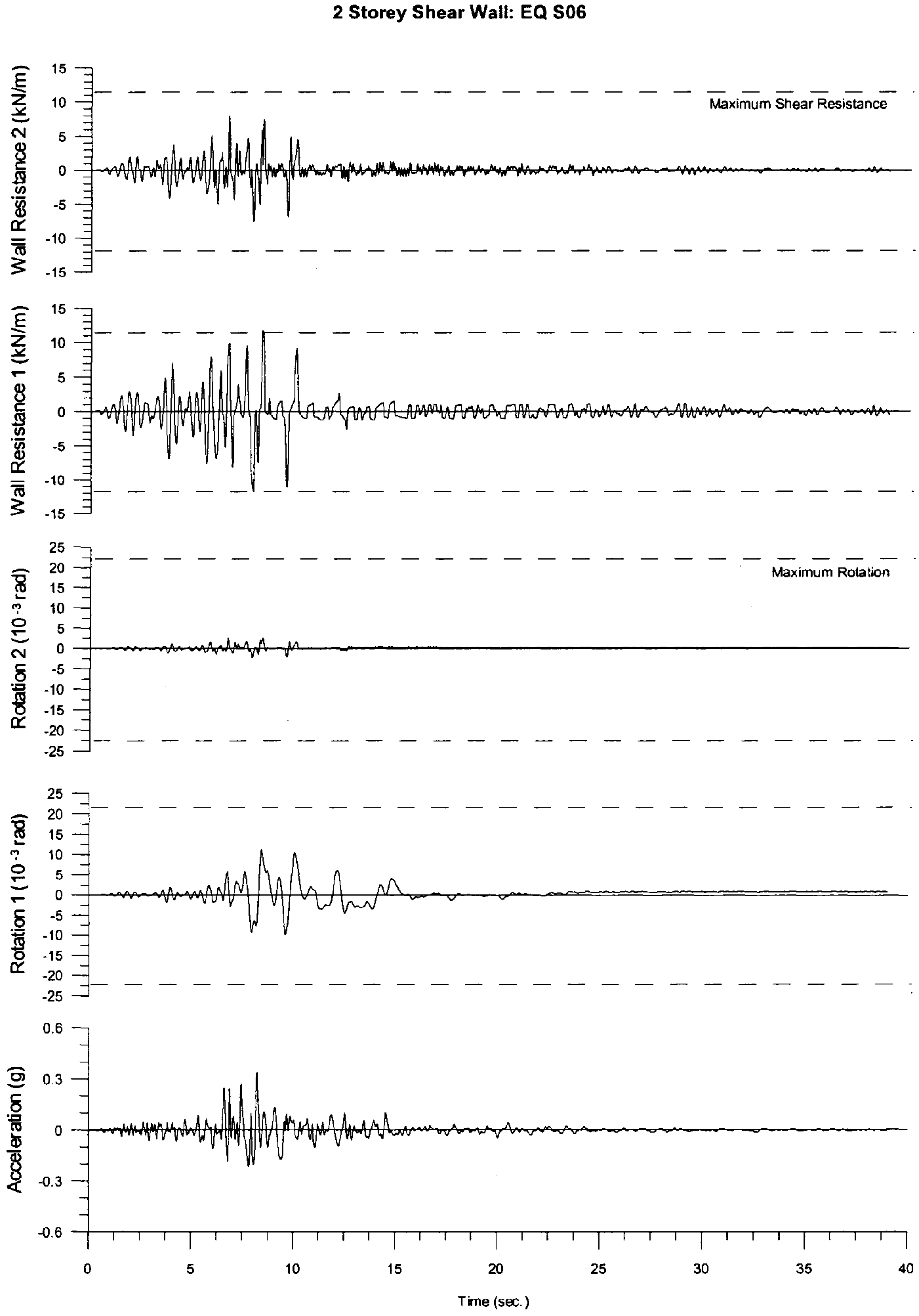
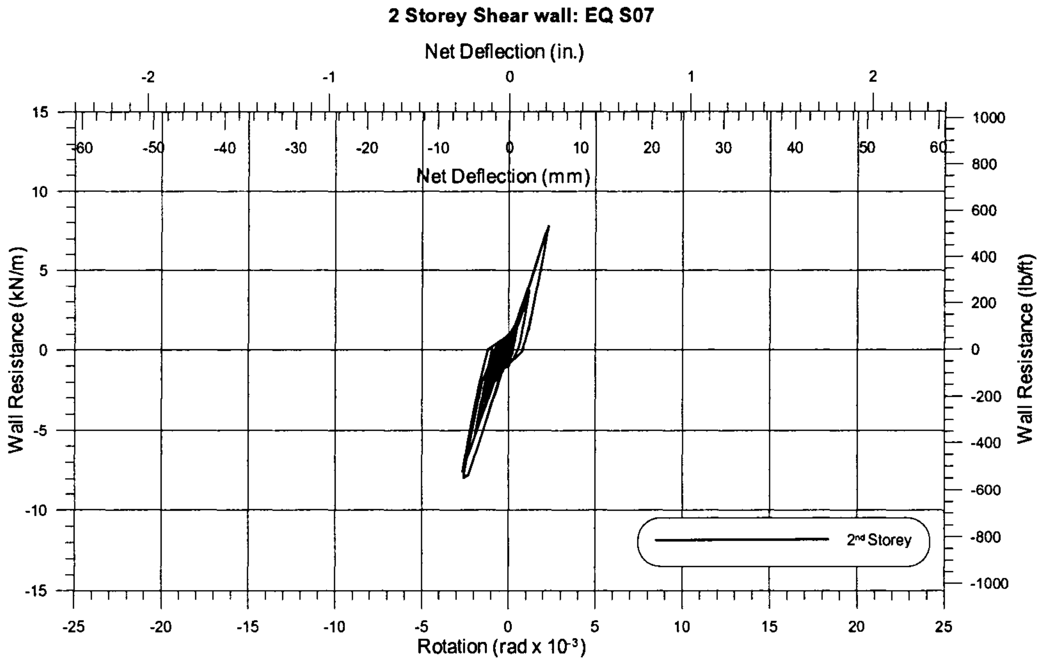


Figure G.12: Resistance and displacement time histories of the two-storey shear wall under ground motion record S06



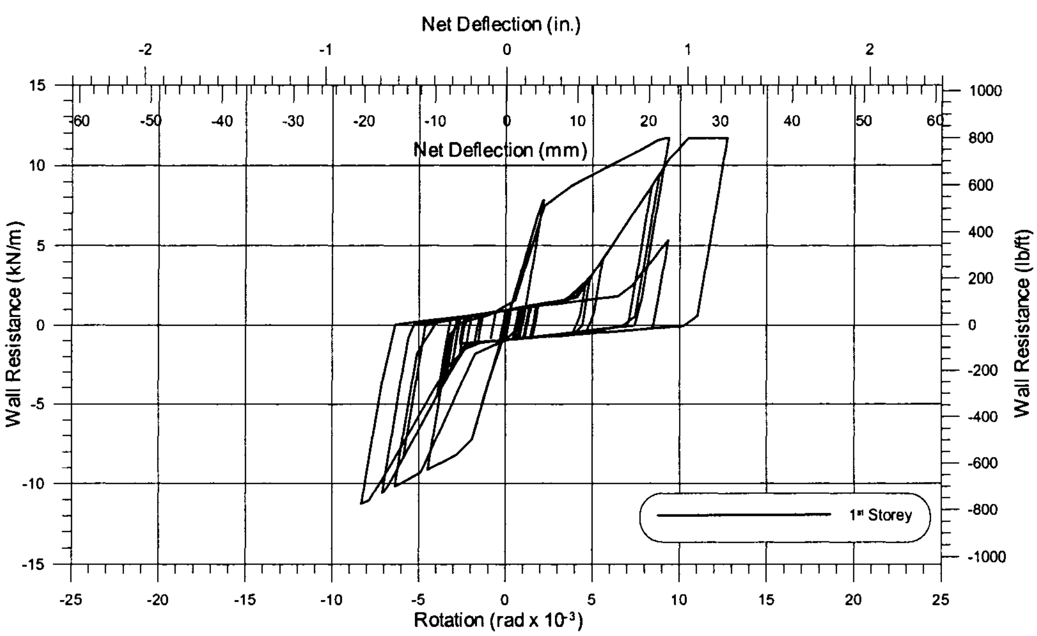


Figure G.13: Resistance vs. displacement hystereses of the two-storey shear wall under ground motion record S07

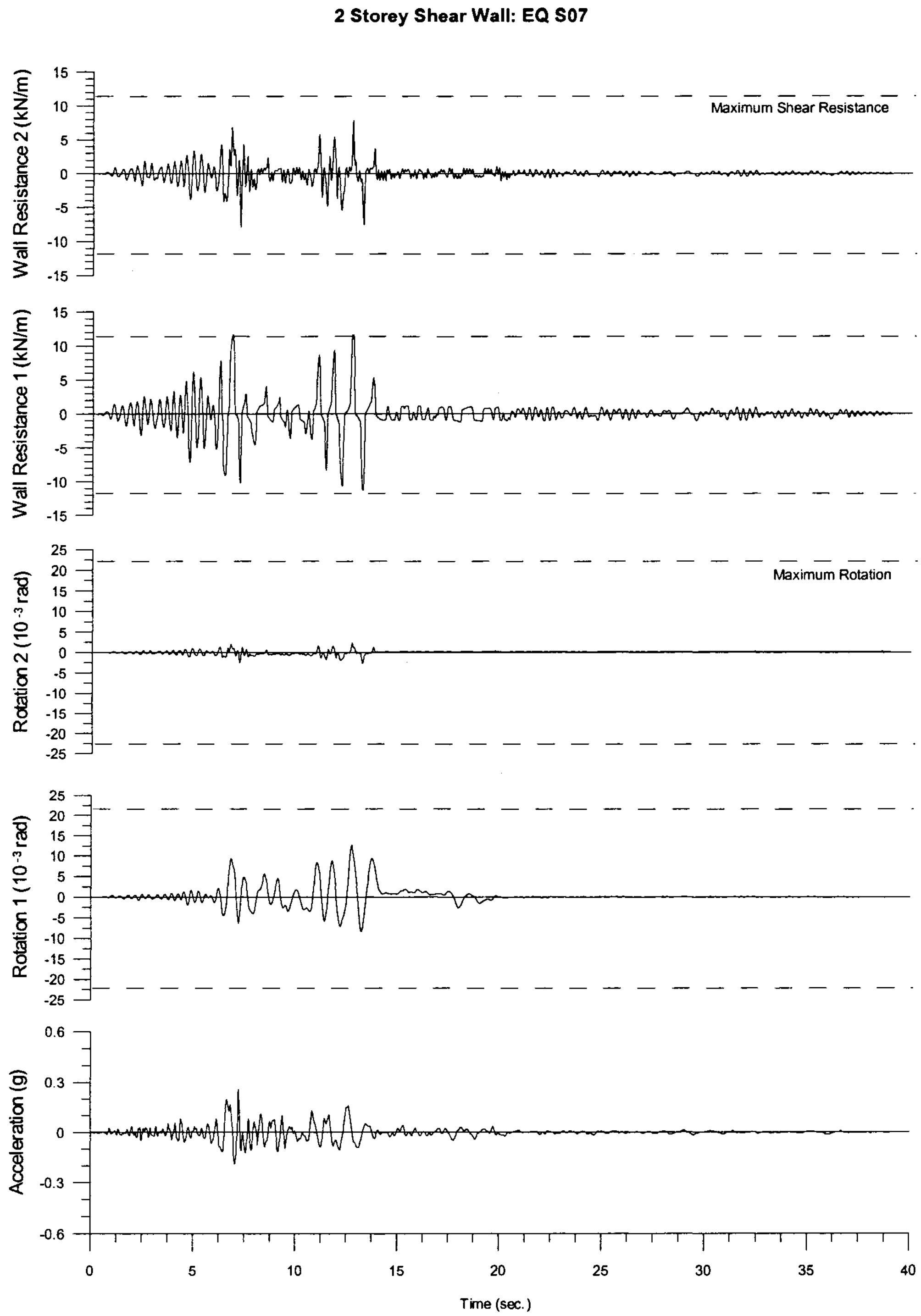
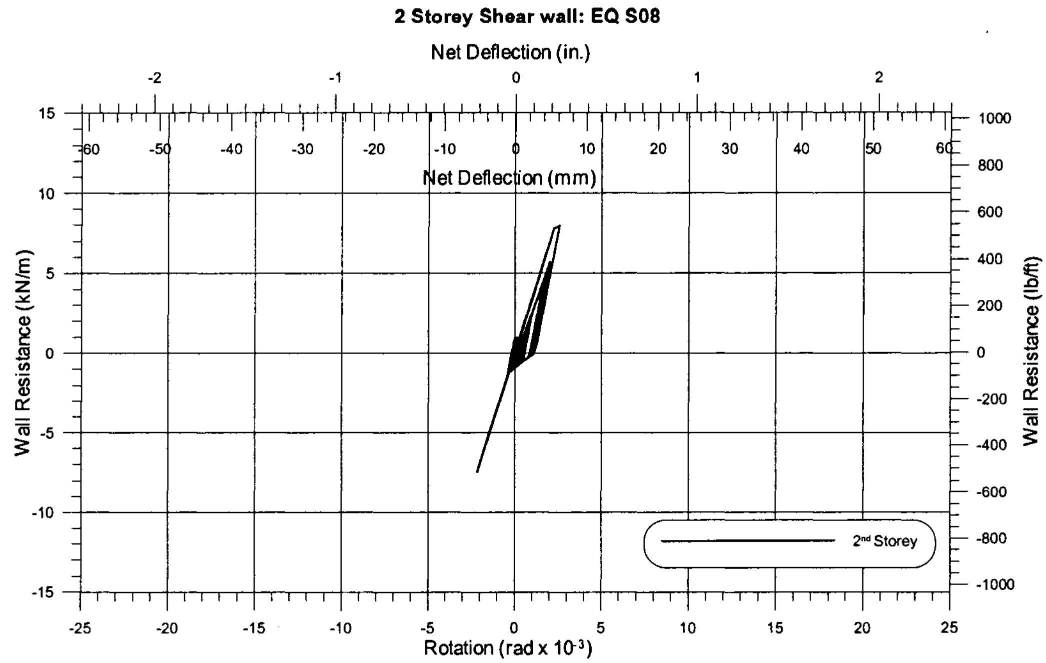


Figure G.14: Resistance and displacement time histories of the two-storey shear wall under ground motion record S07



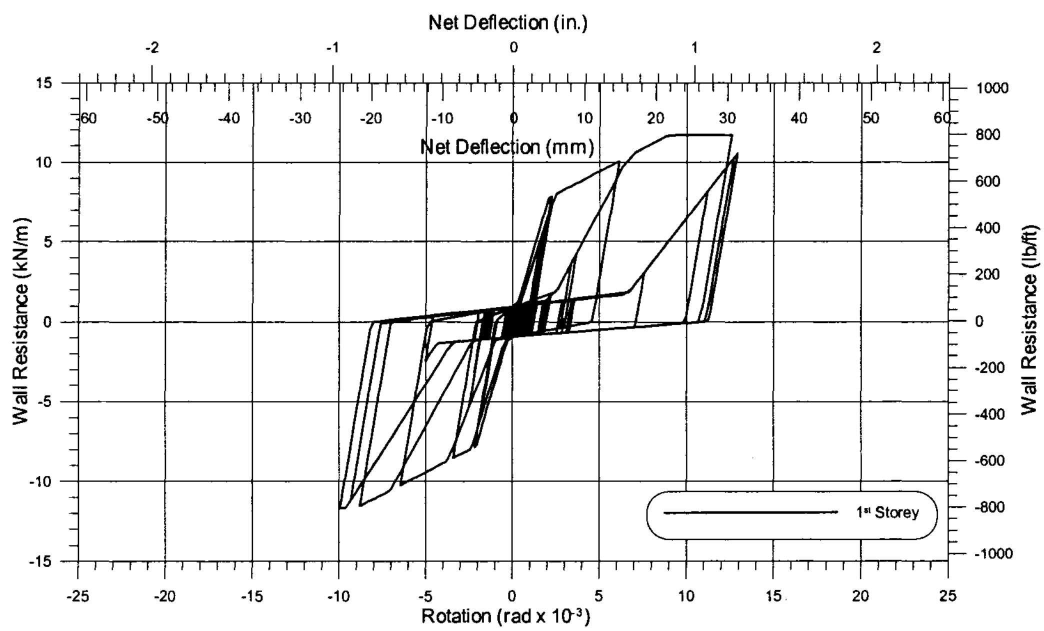


Figure G.13: Resistance vs. displacement hystereses of the two-storey shear wall under ground motion record S08

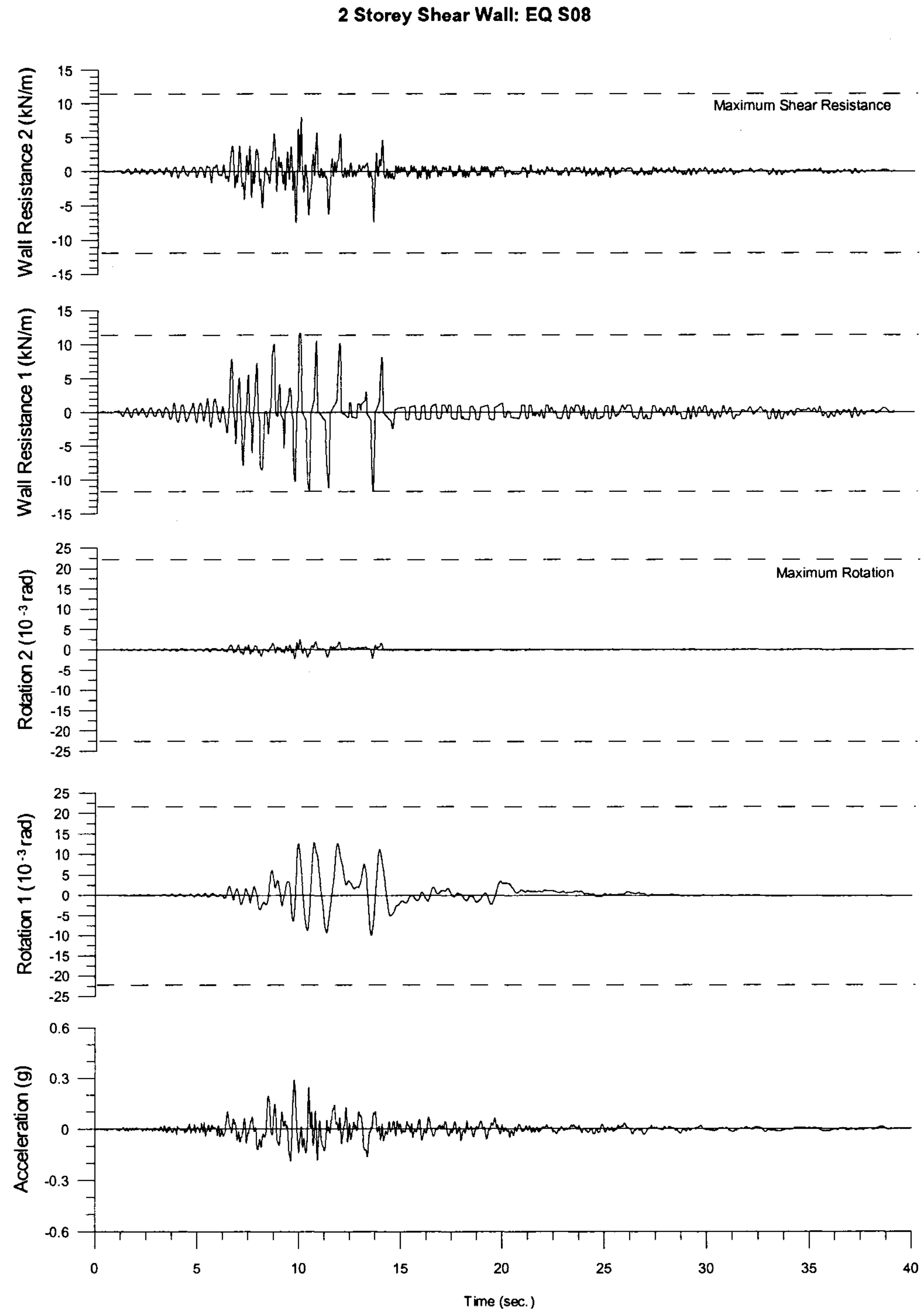
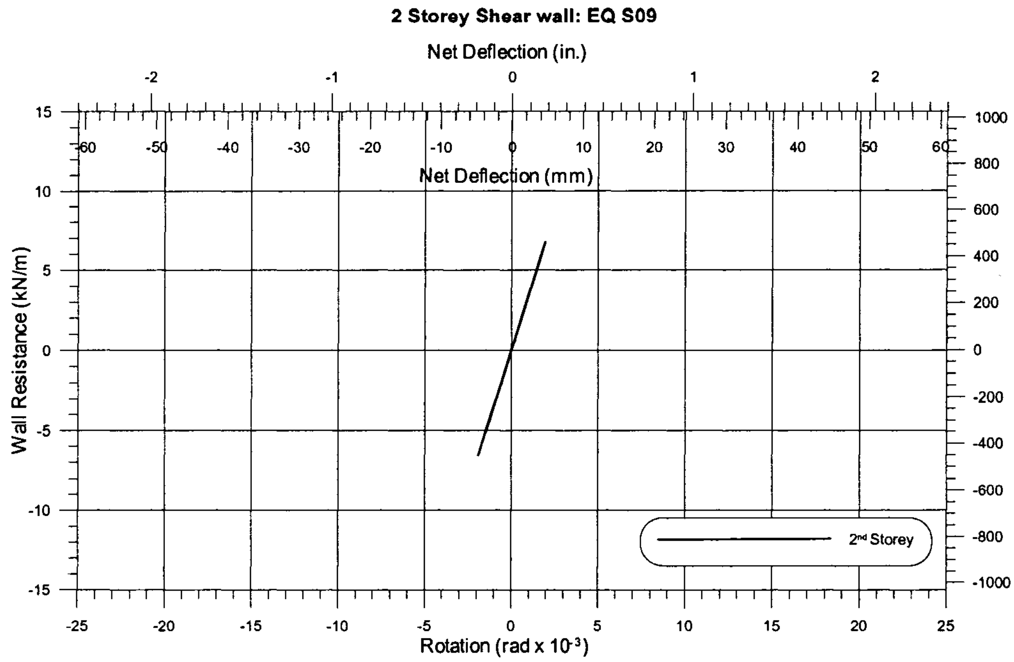


Figure G.16: Resistance and displacement time histories of the two-storey shear wall under ground motion record S08



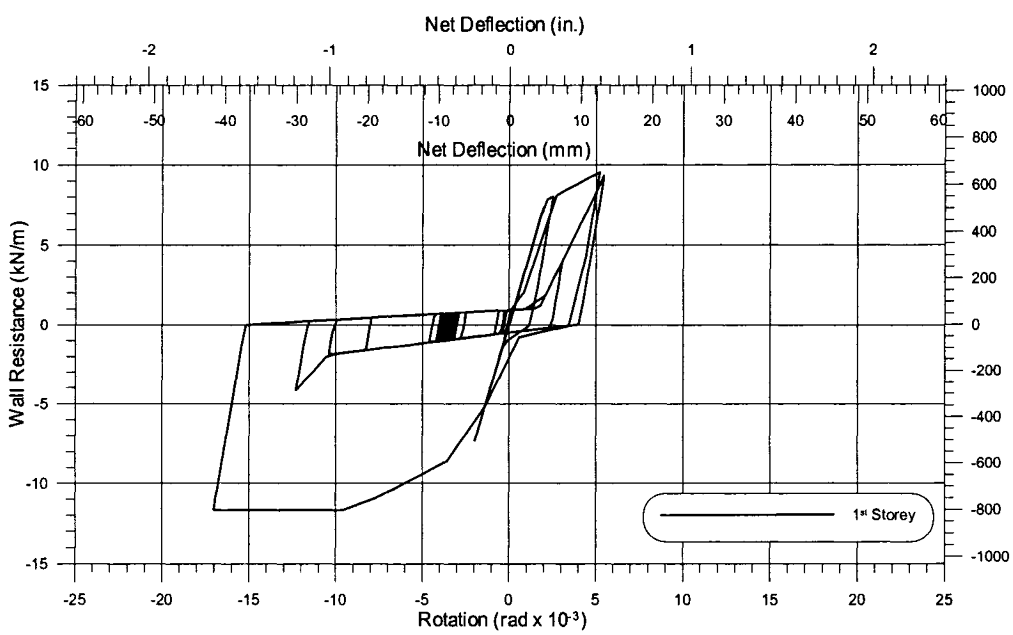


Figure G.17: Resistance vs. displacement hystereses of the two-storey shear wall under ground motion record S09

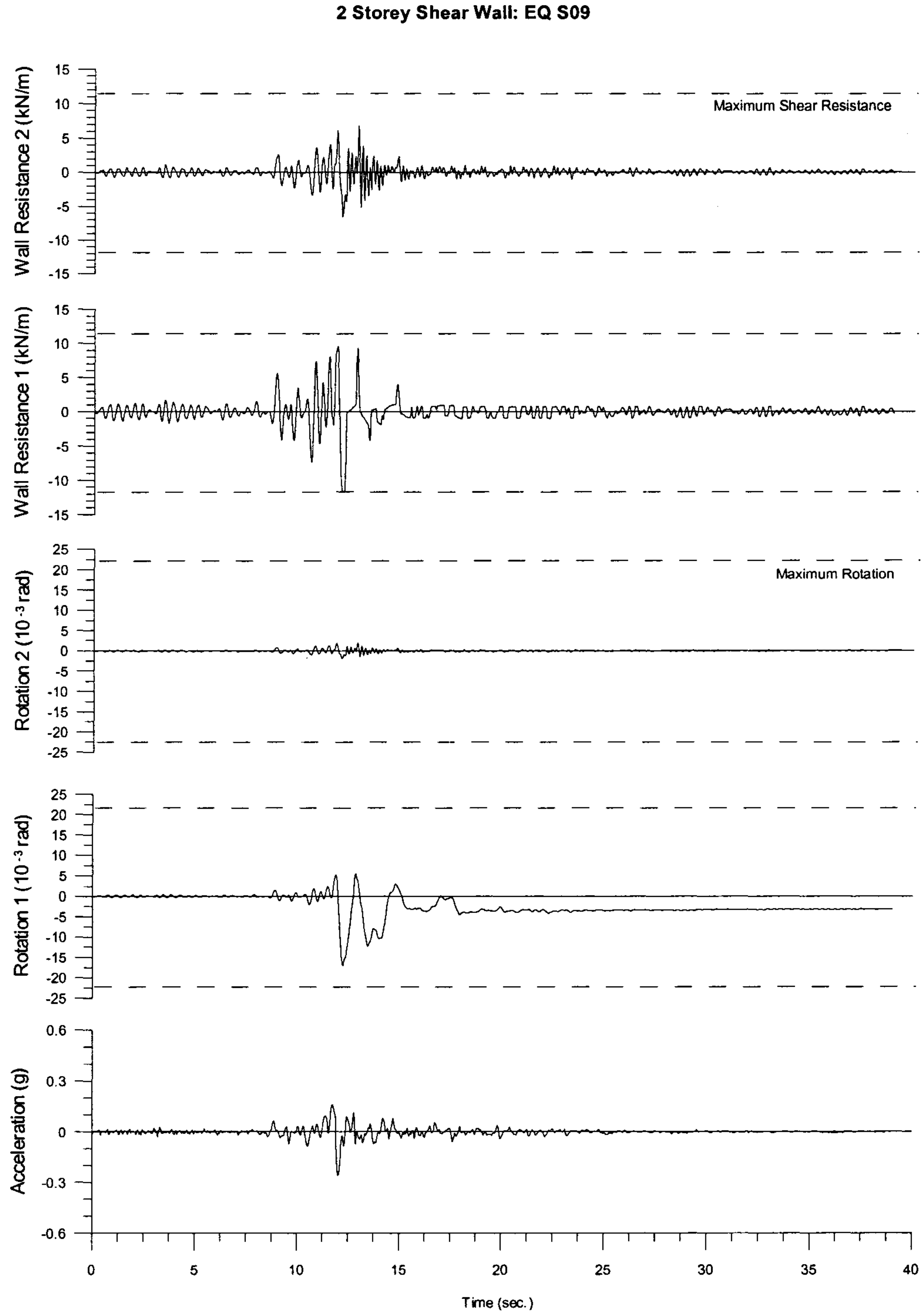
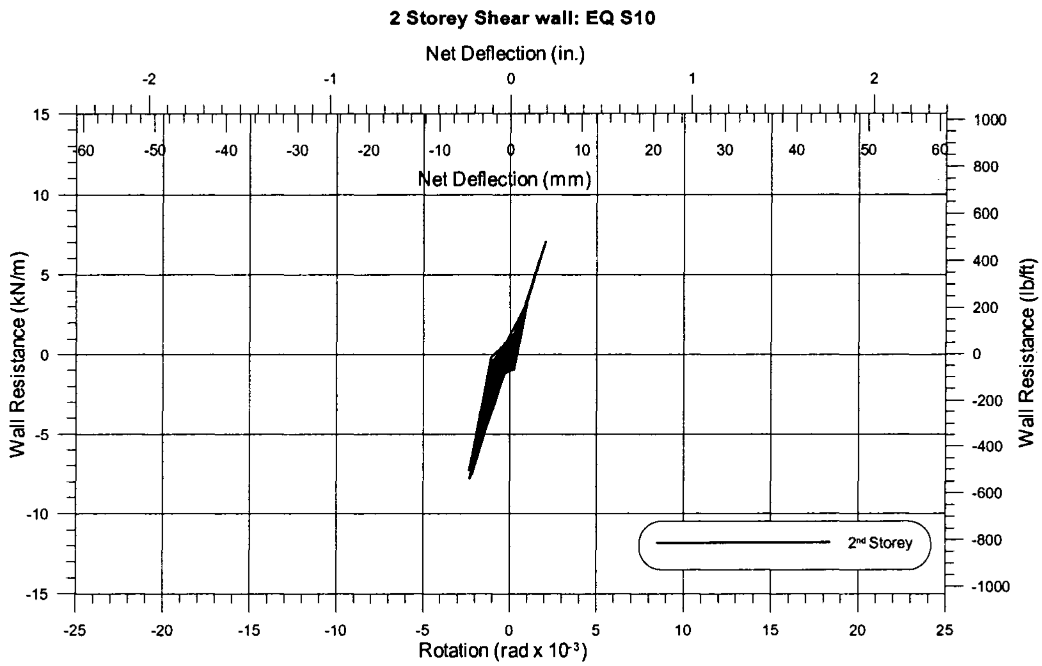


Figure G.18: Resistance and displacement time histories of the two-storey shear wall under ground motion record S09



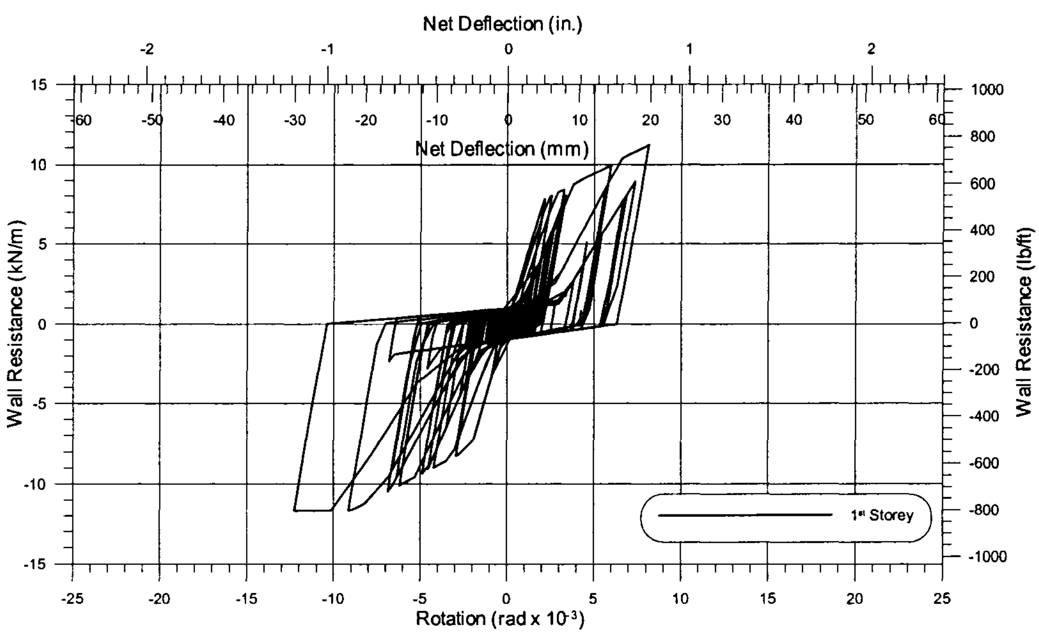


Figure G.19: Resistance vs. displacement hystereses of the two-storey shear wall under ground motion record S10

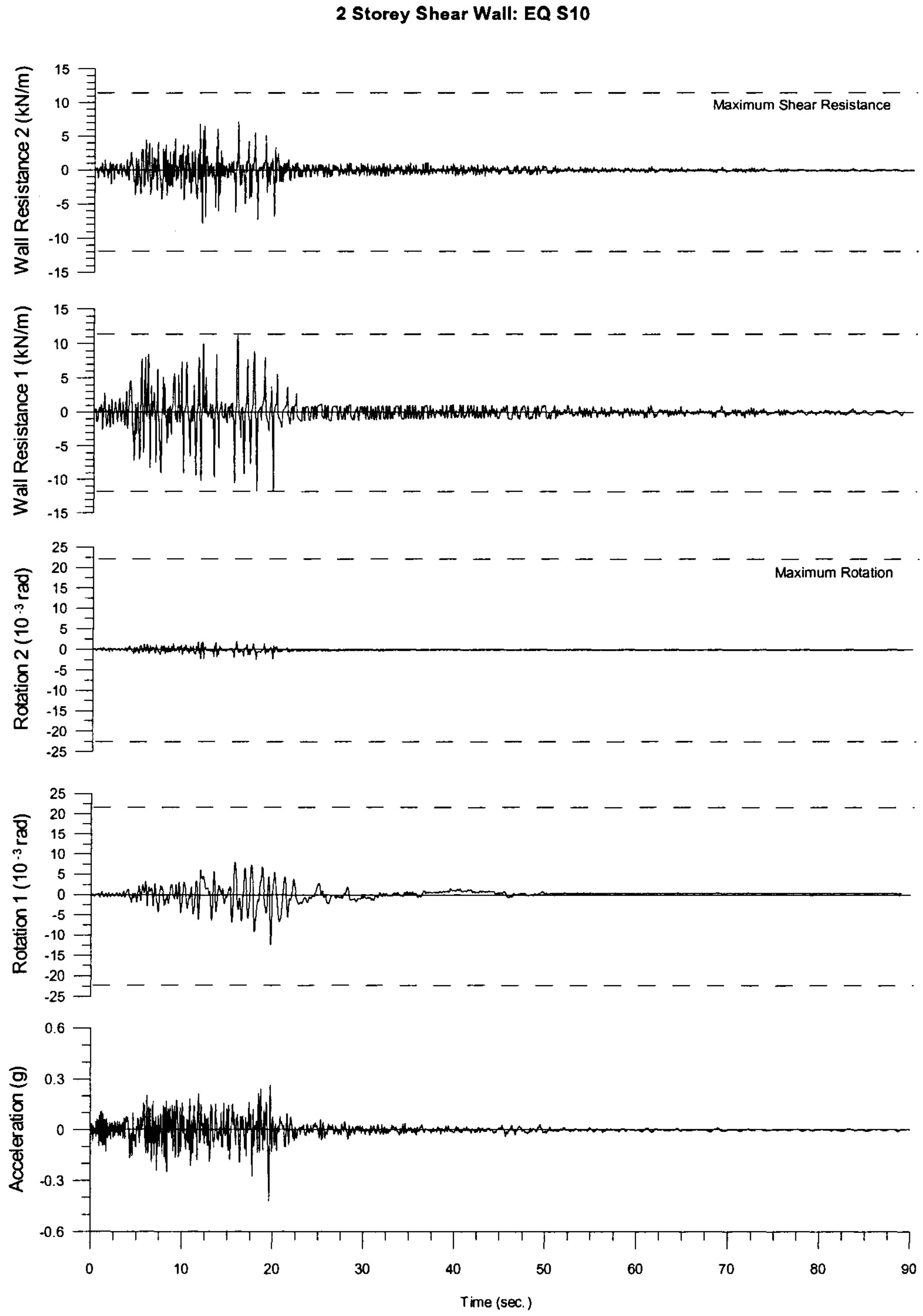


Figure G.20: Resistance and displacement time histories of the two-storey shear wall under ground motion record S10

3-Storey Model

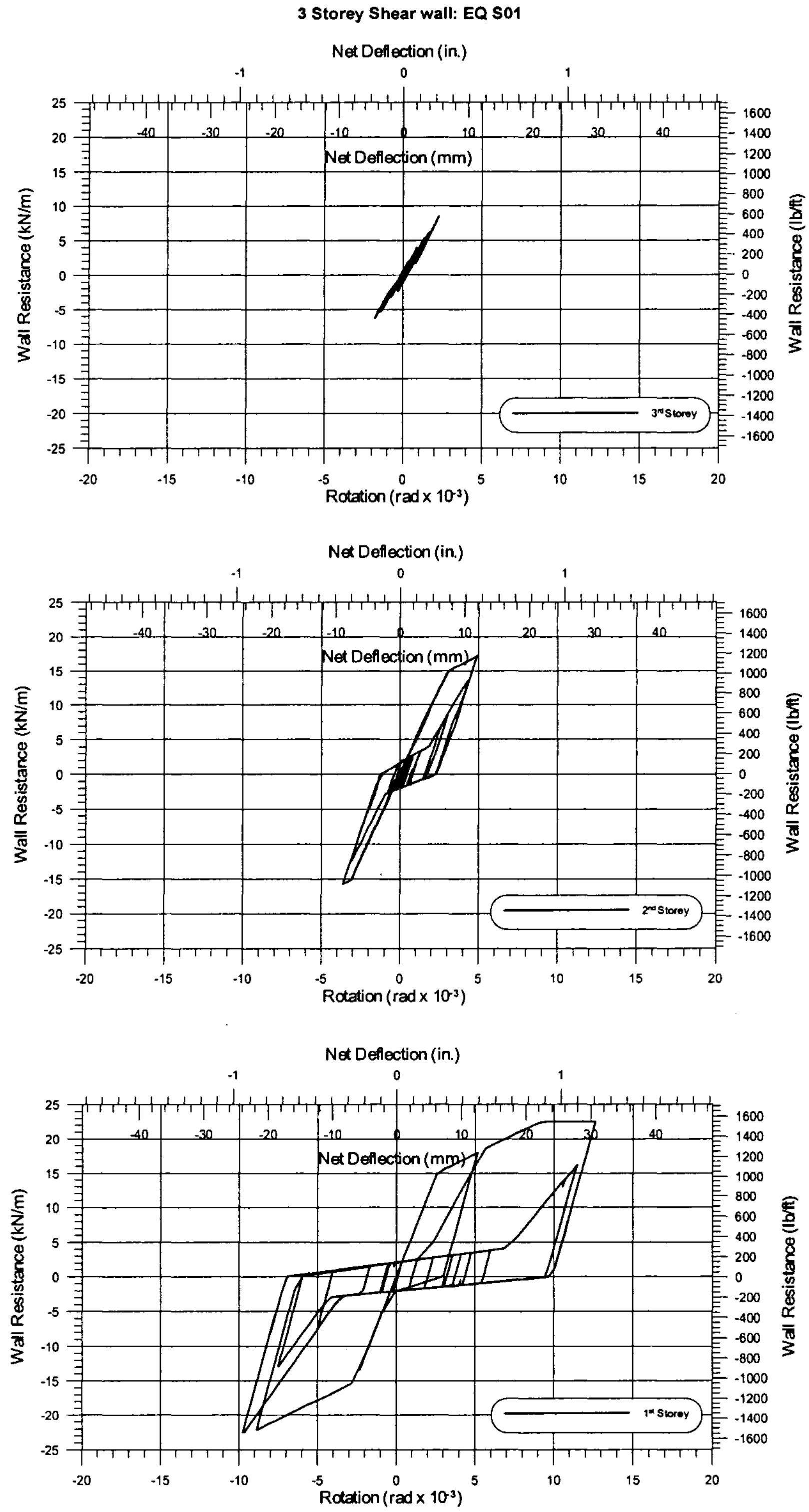


Figure G.21: Resistance vs. displacement hystereses of the three-storey shear wall under ground motion record S01

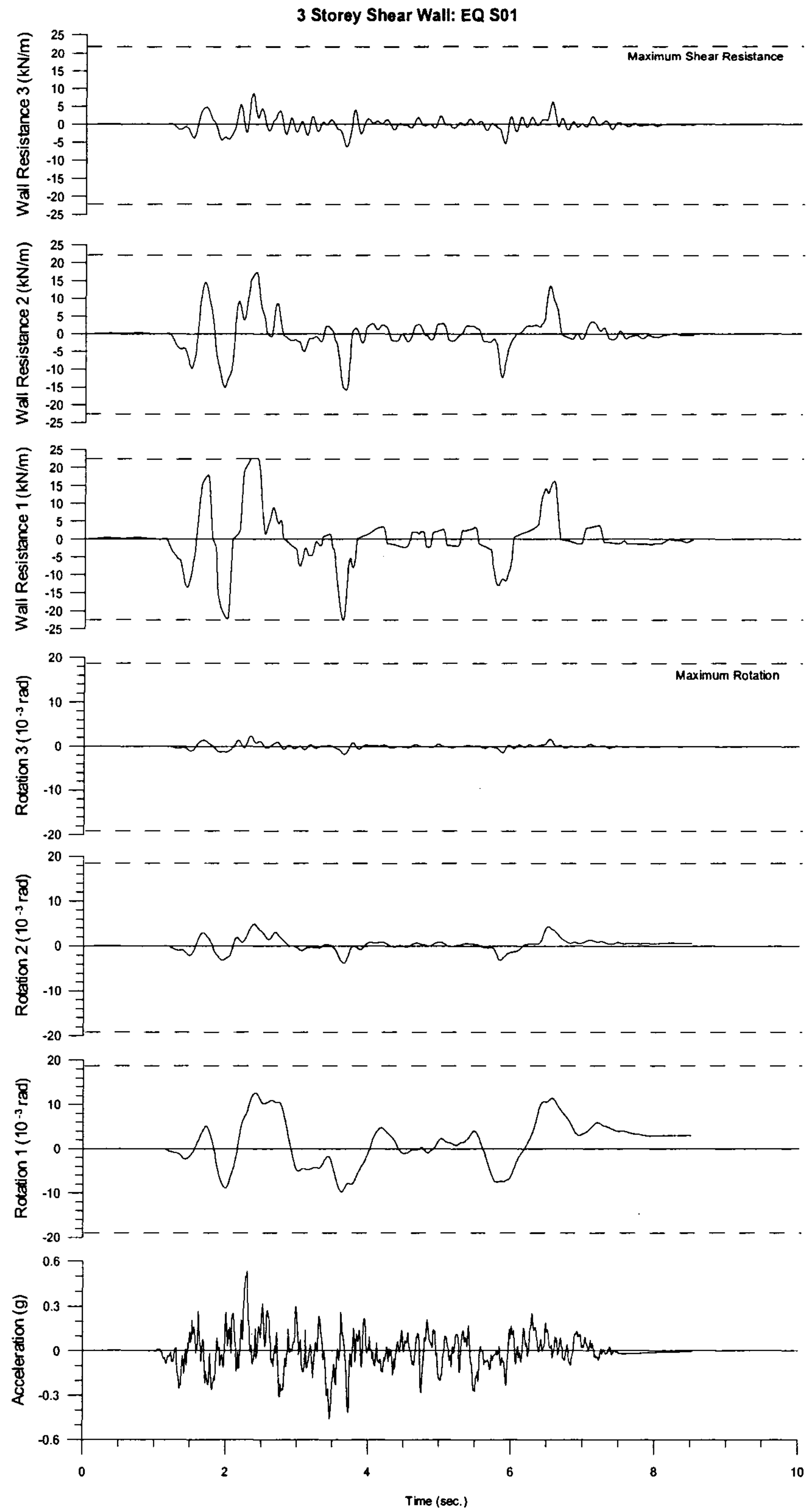


Figure G.22: Resistance and displacement time histories of the three-storey shear wall under ground motion record S01

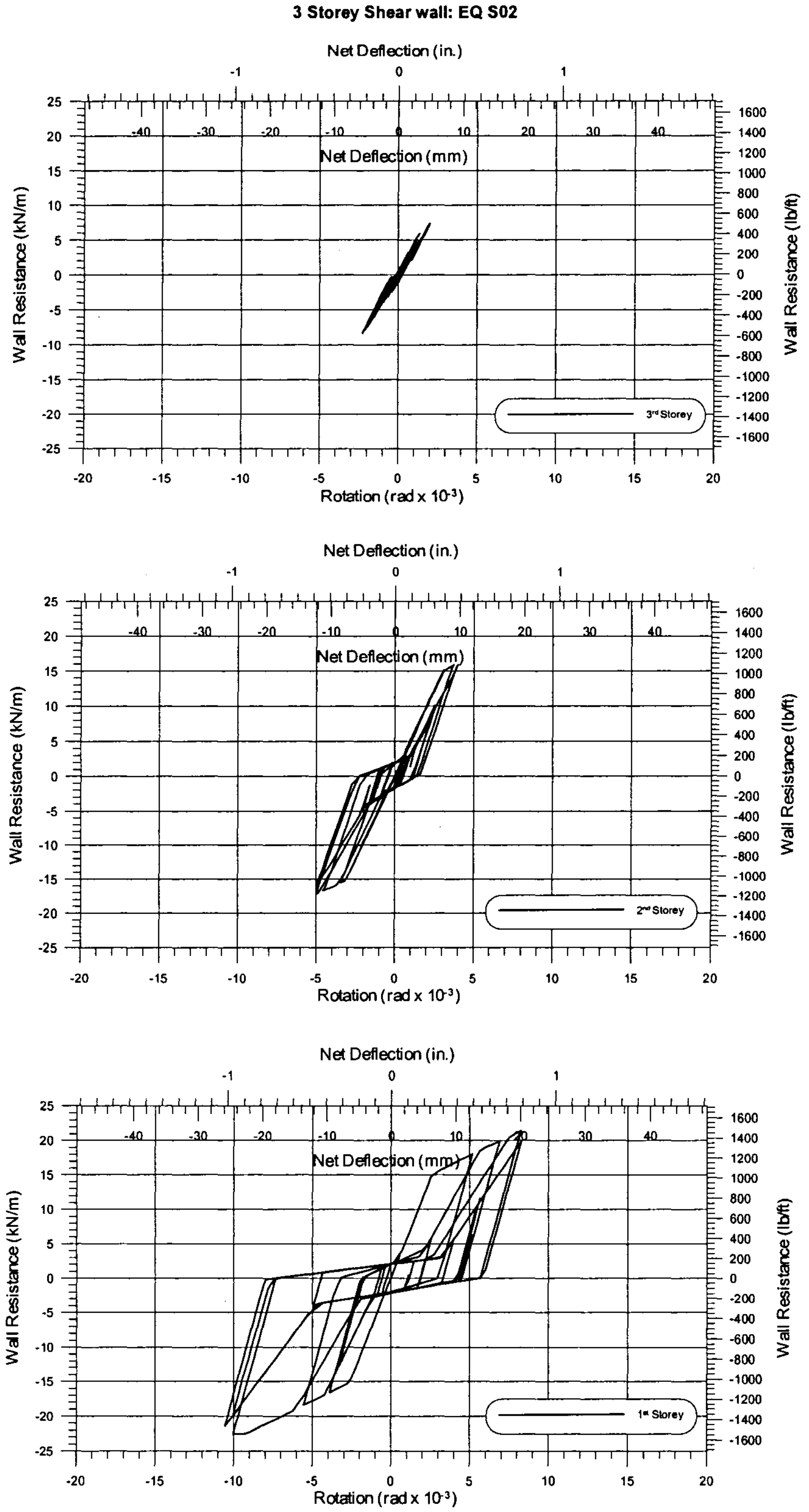


Figure G.23: Resistance vs. displacement hystereses of the three-storey shear wall under ground motion record S02

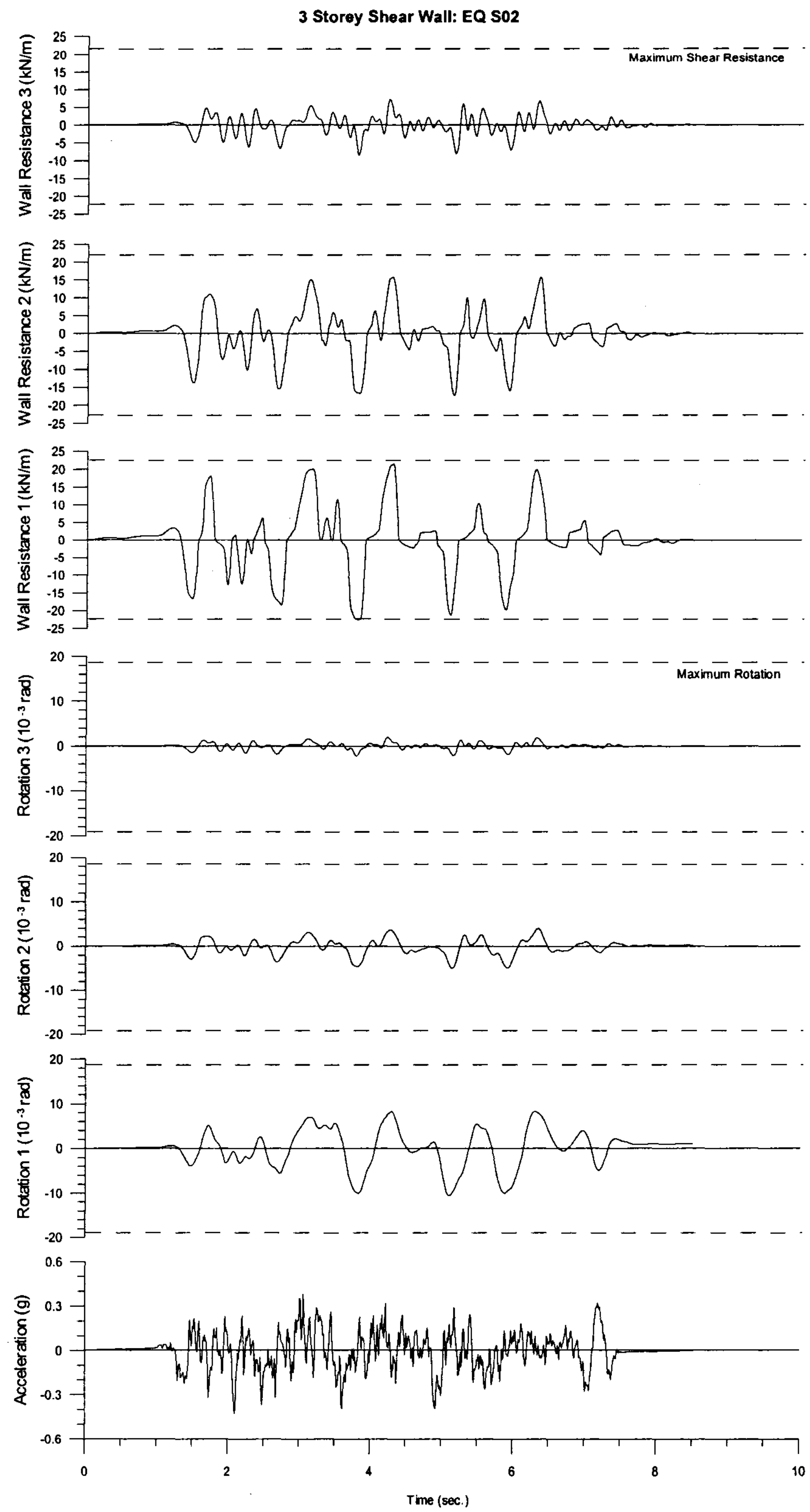


Figure G.24: Resistance and displacement time histories of the three-storey shear wall under ground motion record S02

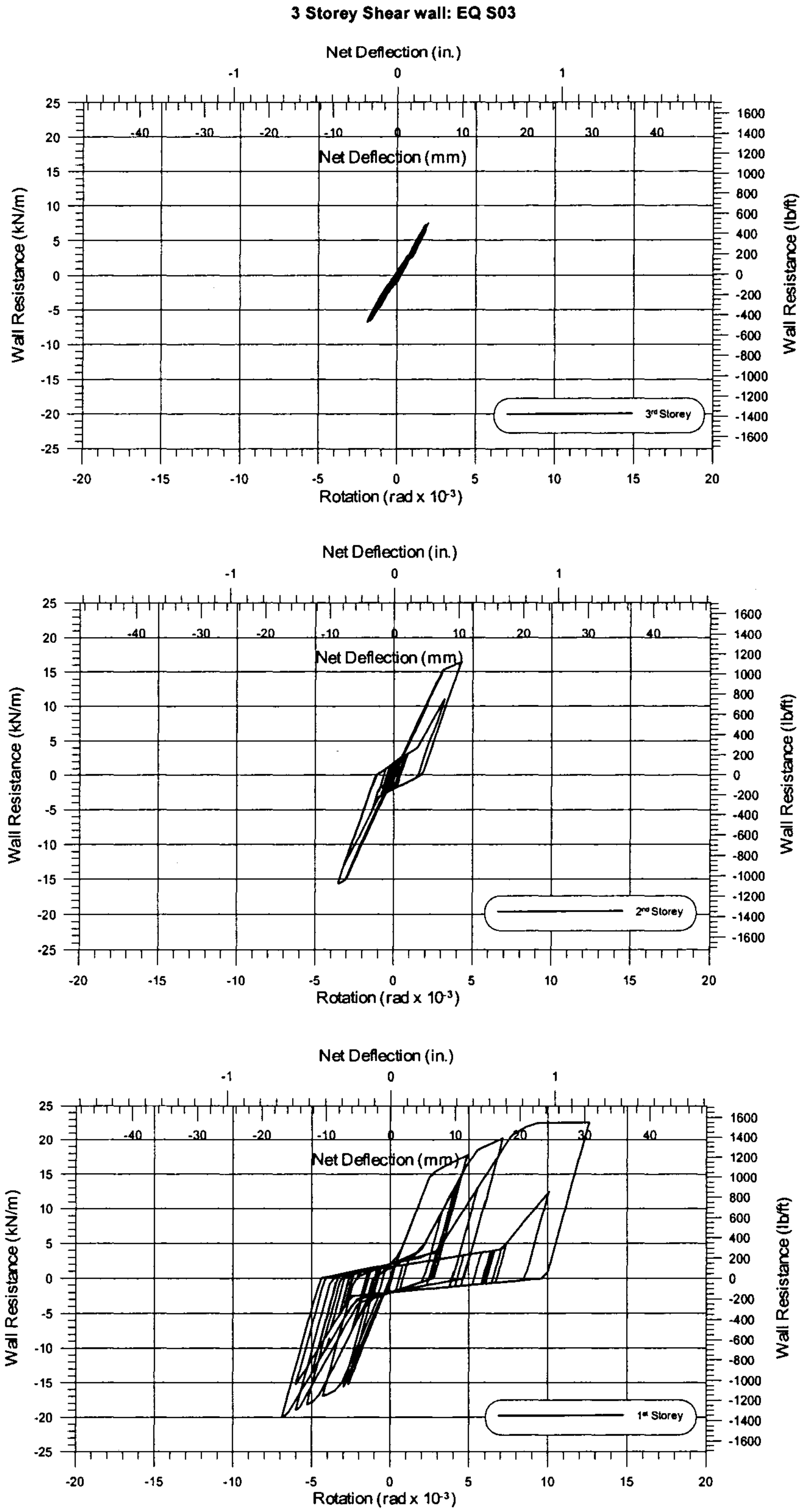


Figure G.25: Resistance vs. displacement hystereses of the three-storey shear wall under ground motion record S03

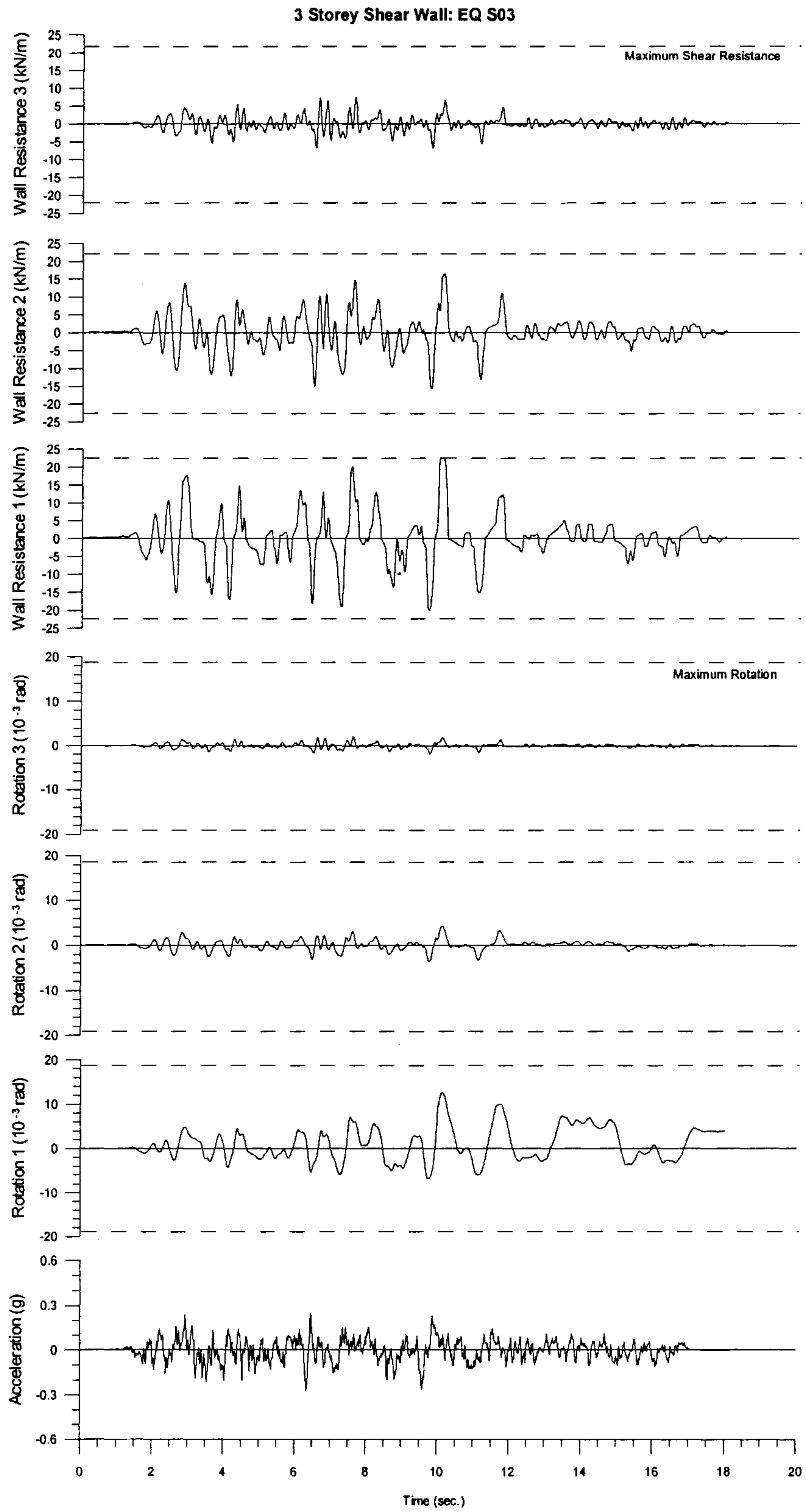


Figure G.26: Resistance and displacement time histories of the three-storey shear wall under ground motion record S03

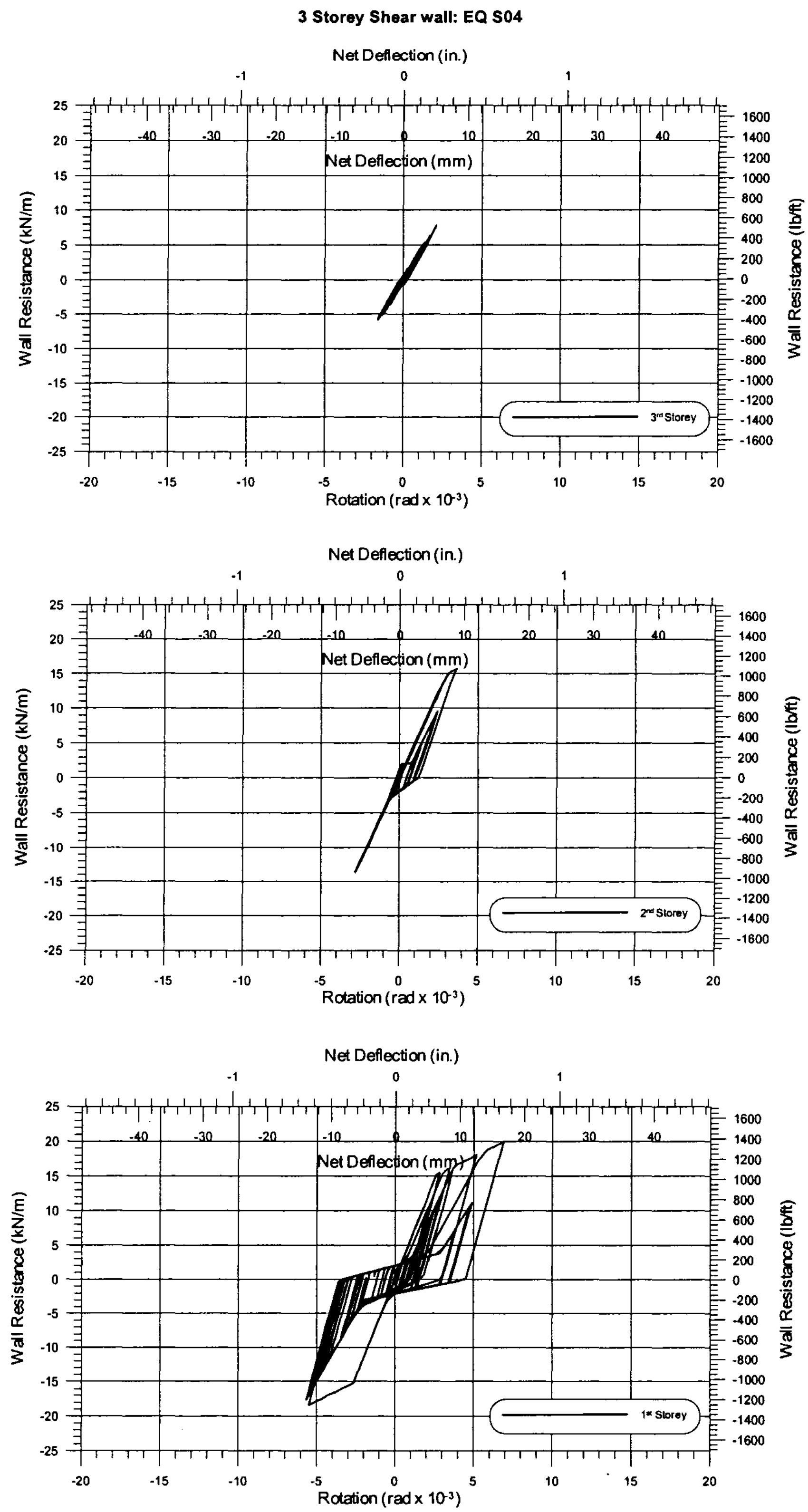


Figure G.27: Resistance vs. displacement hystereses of the three-storey shear wall under ground motion record S04

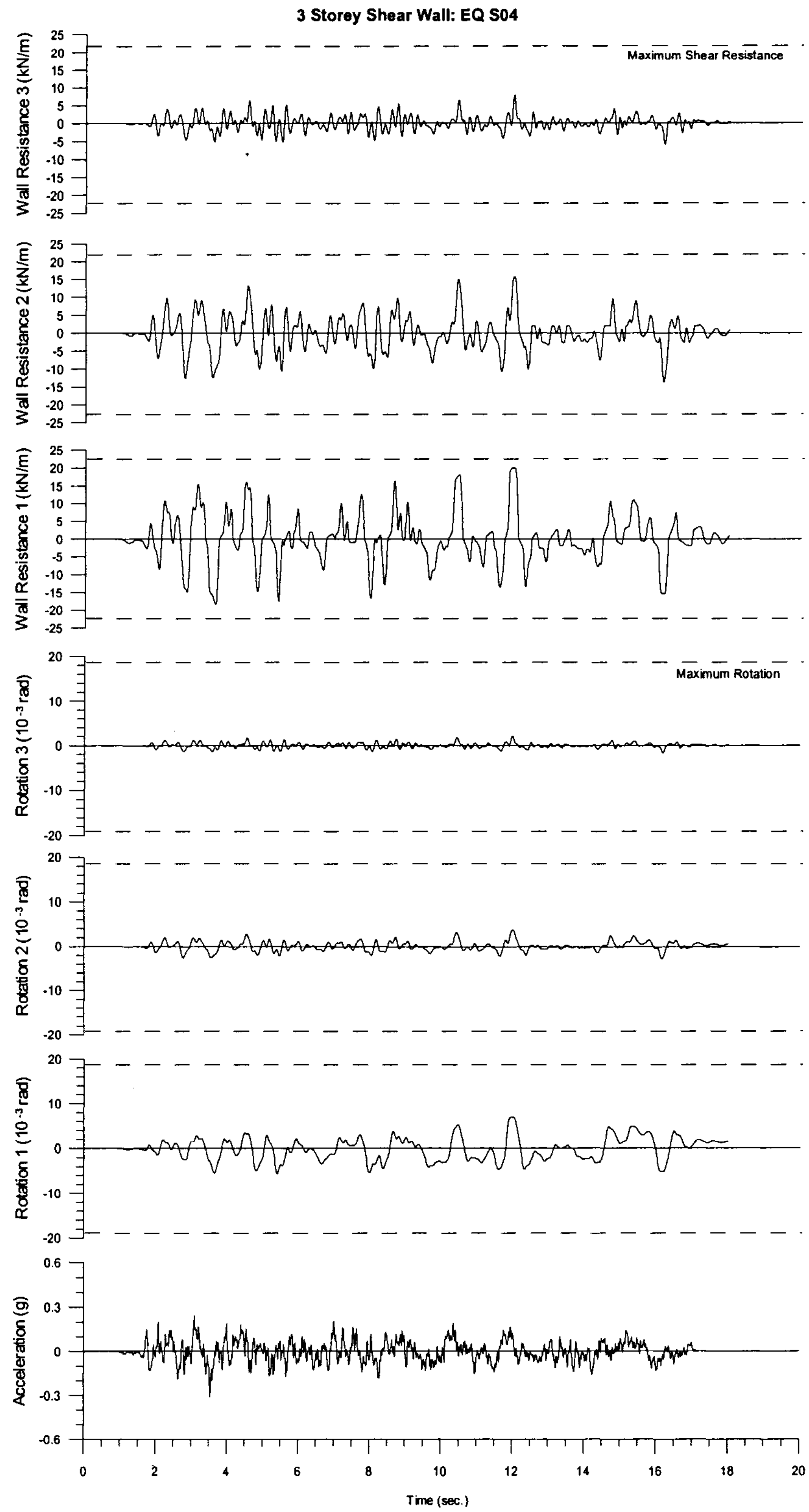


Figure G.28: Resistance and displacement time histories of the three-storey shear wall under ground motion record S04

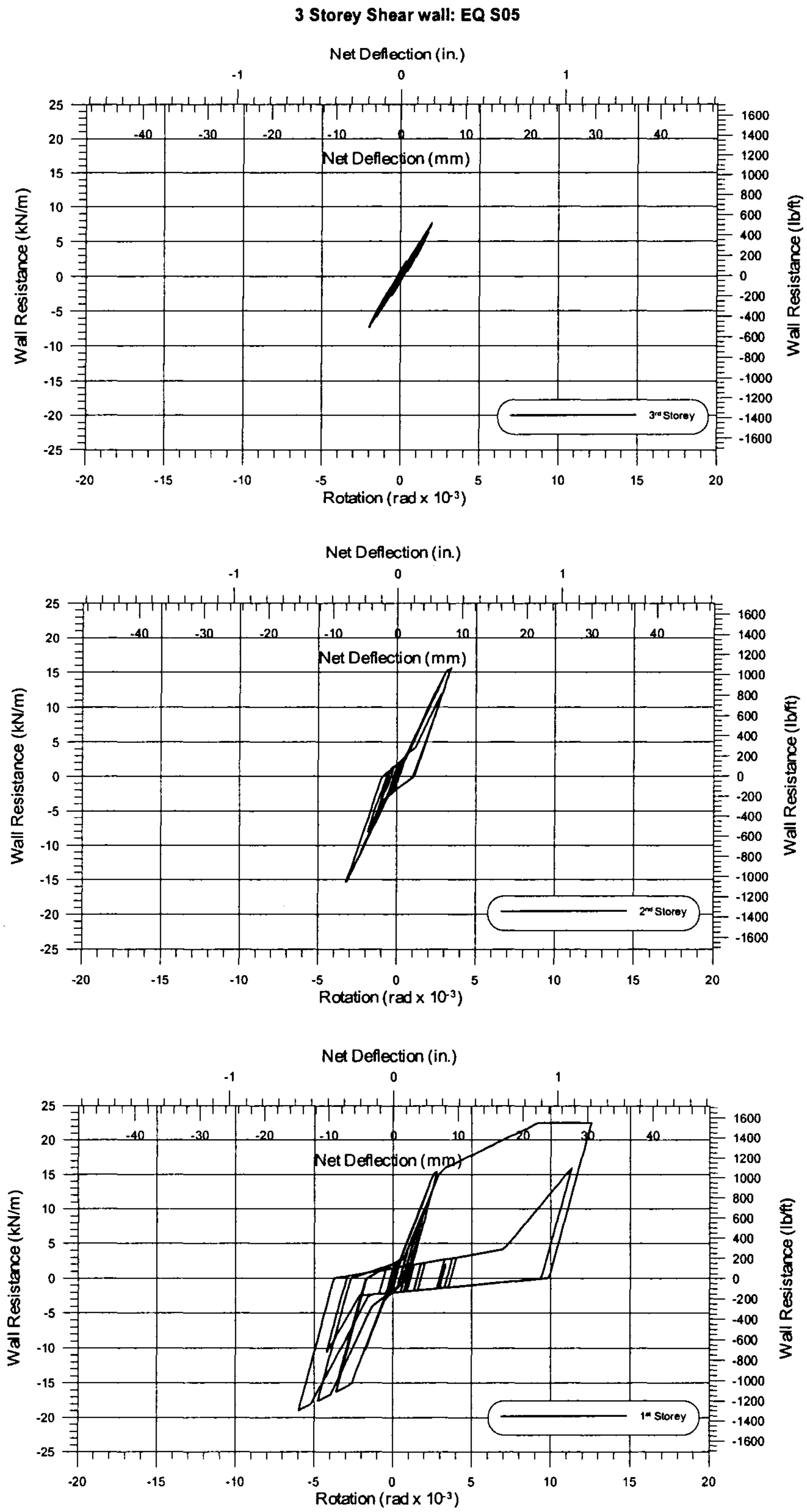


Figure G.29: Resistance vs. displacement hystereses of the three-storey shear wall under ground motion record S05

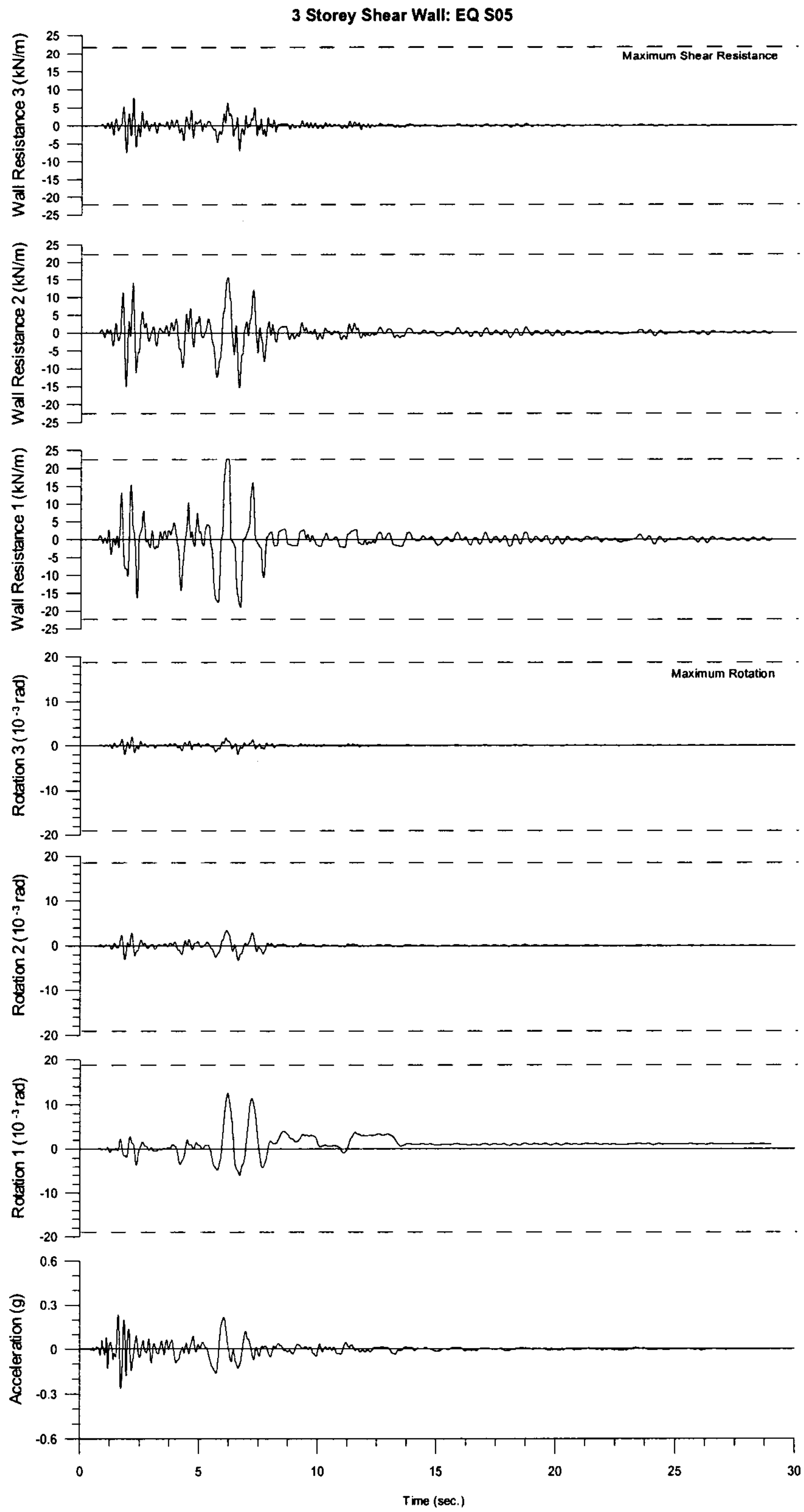


Figure G.30: Resistance and displacement time histories of the three-storey shear wall under ground motion record S05

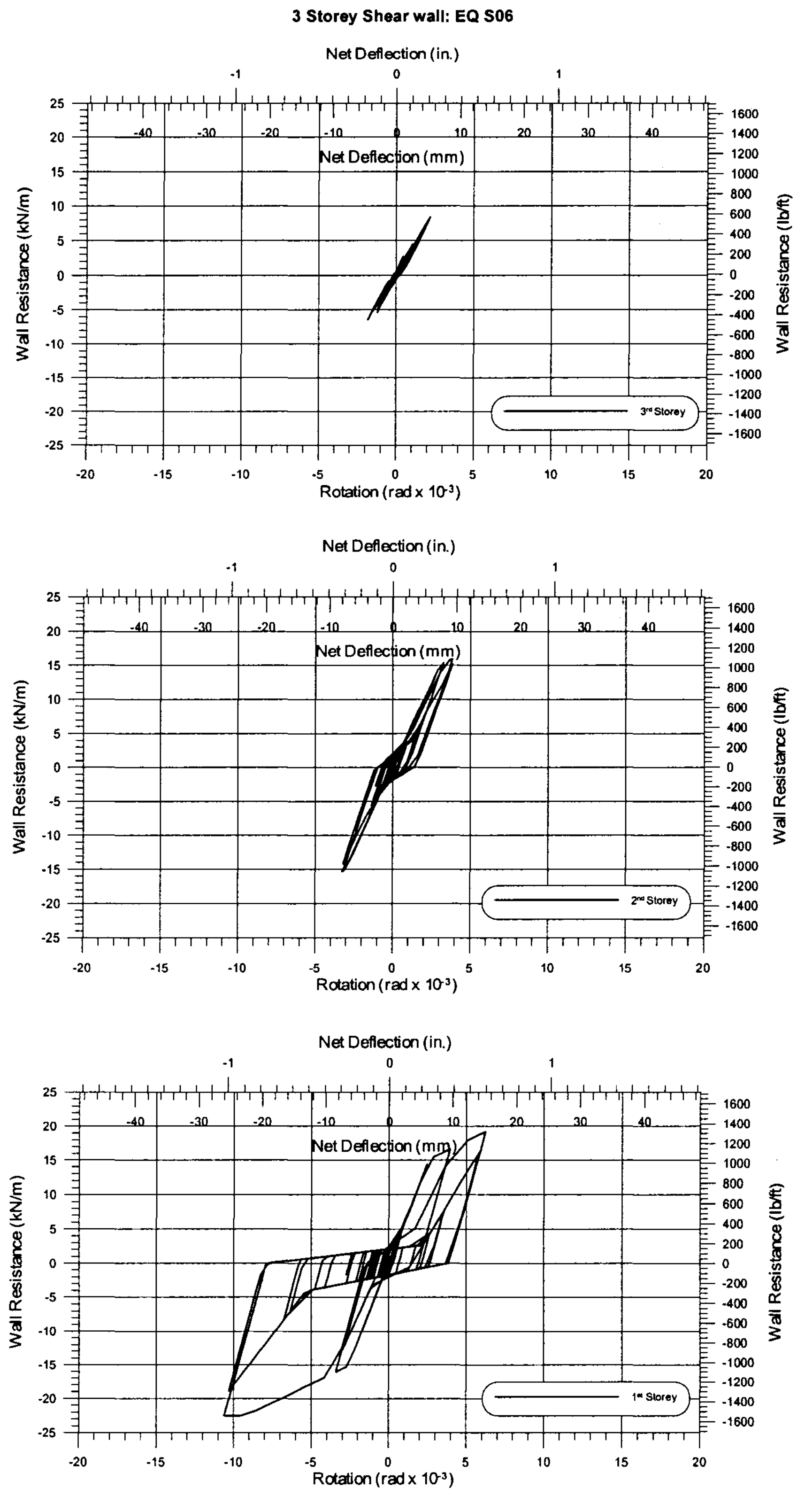


Figure G.31: Resistance vs. displacement hystereses of the three-storey shear wall under ground motion record S06

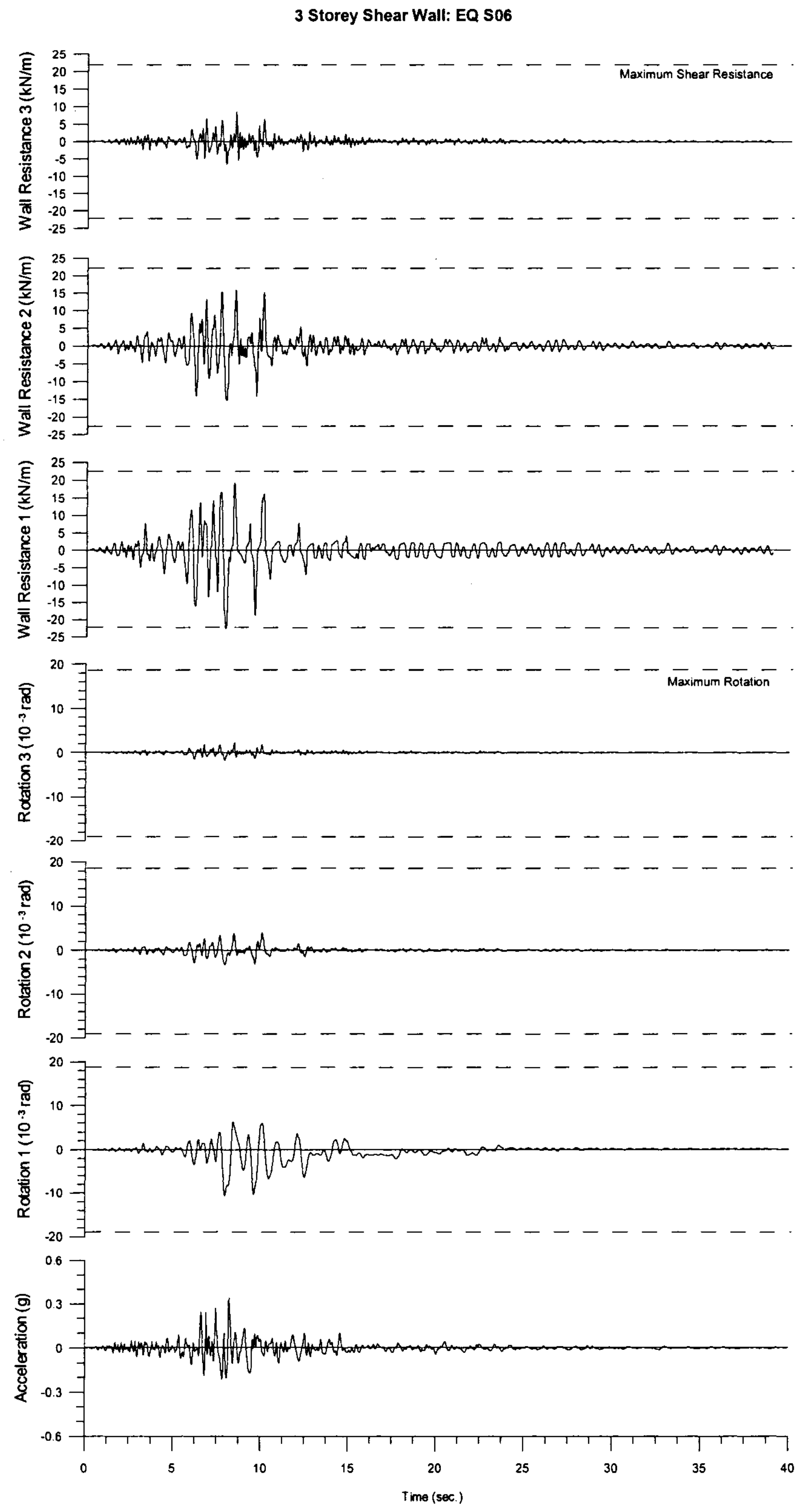


Figure G.32: Resistance and displacement time histories of the three-storey shear wall under ground motion record S06

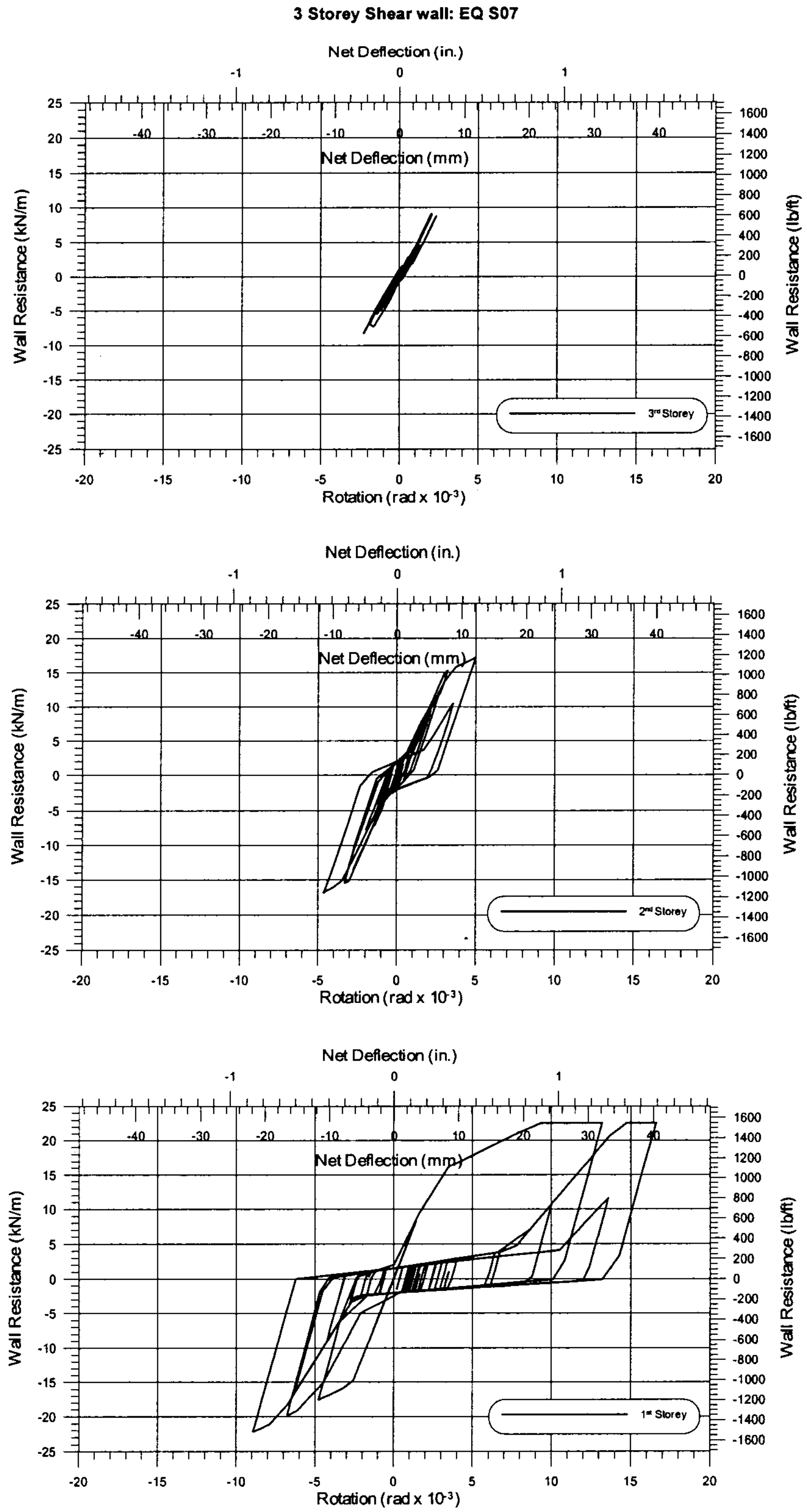


Figure G.33: Resistance vs. displacement hystereses of the three-storey shear wall under ground motion record S07

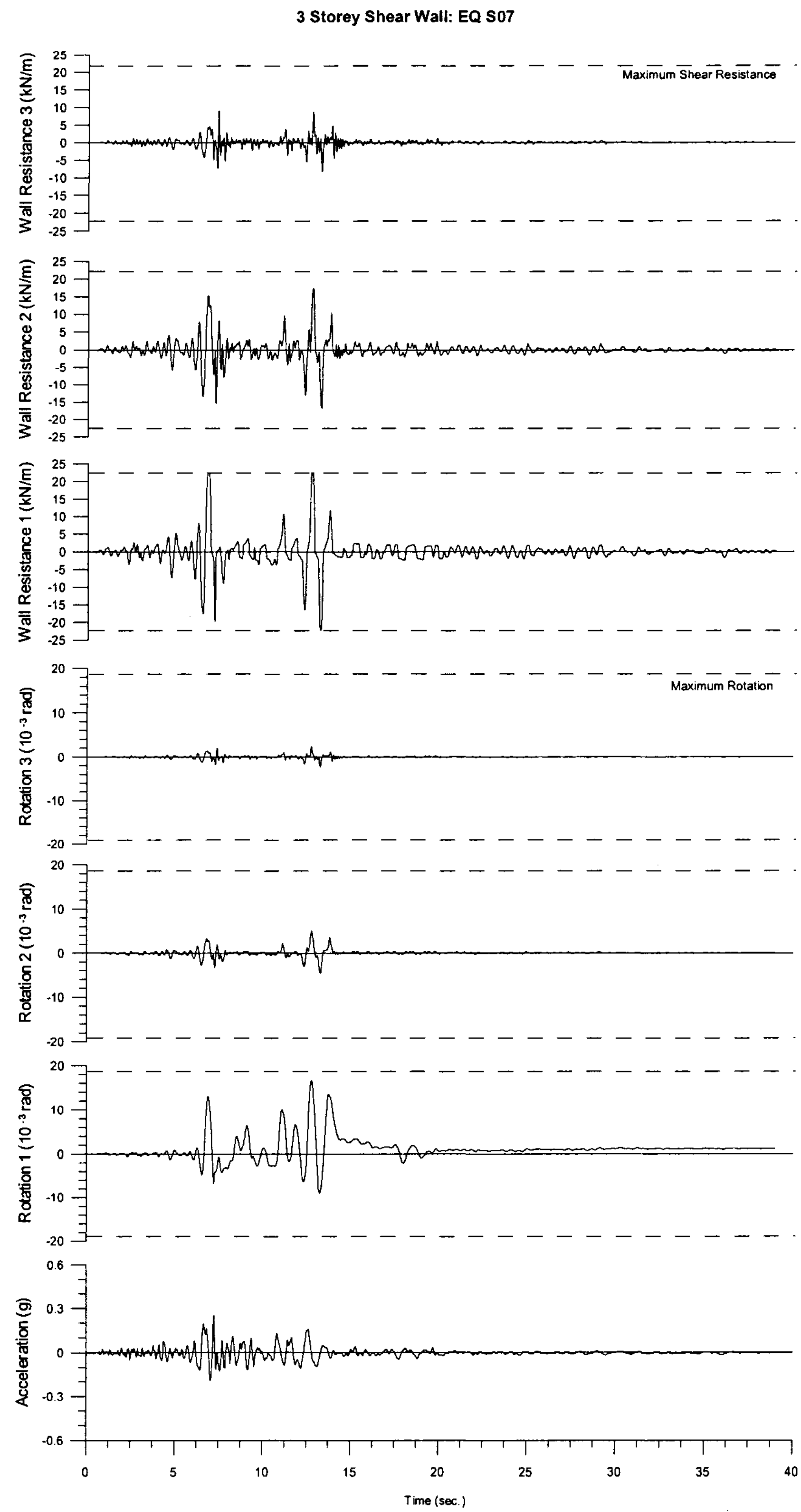


Figure G.34: Resistance and displacement time histories of the three-storey shear wall under ground motion record S07

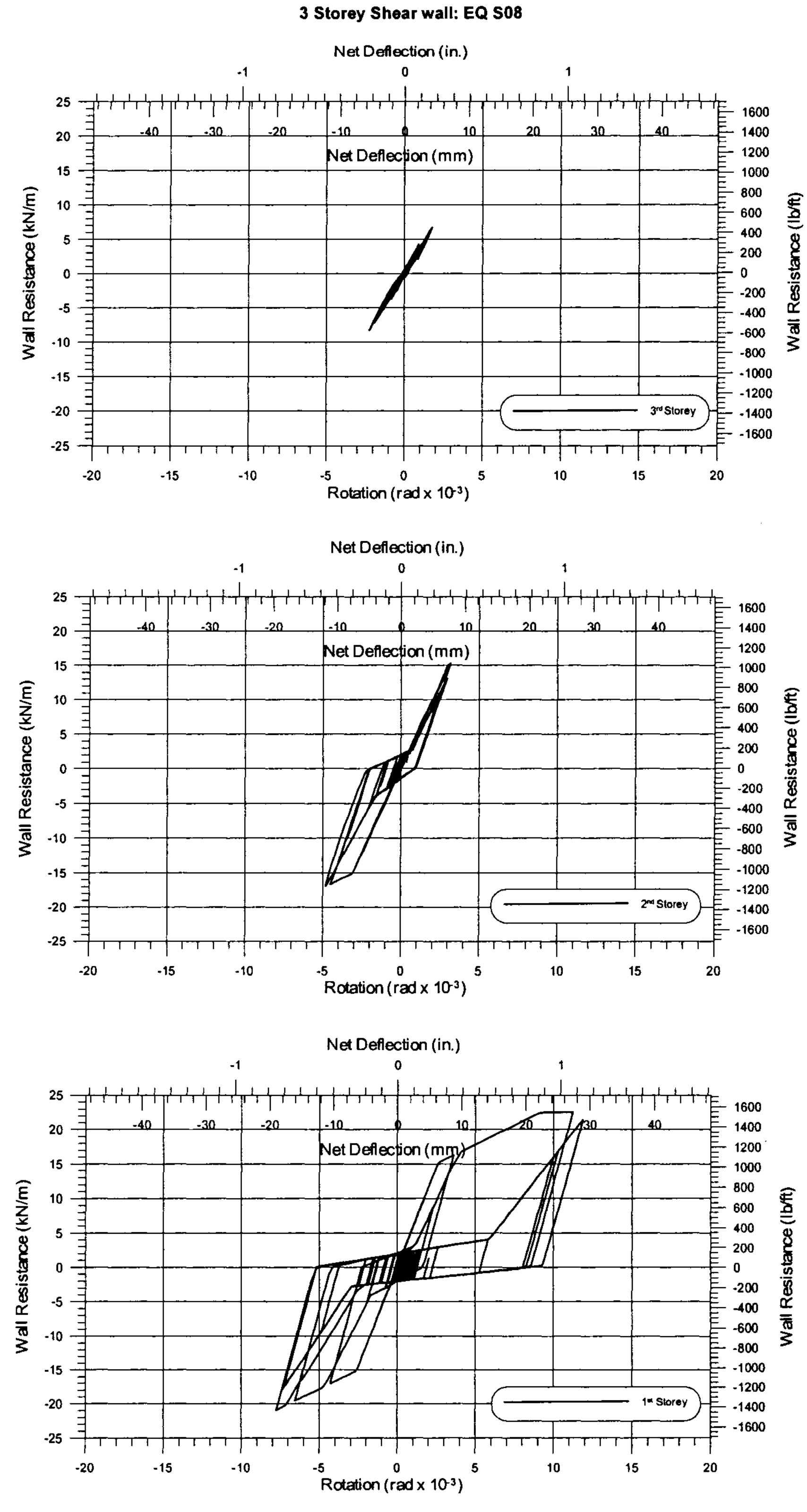


Figure G.35: Resistance vs. displacement hystereses of the three-storey shear wall under ground motion record S08

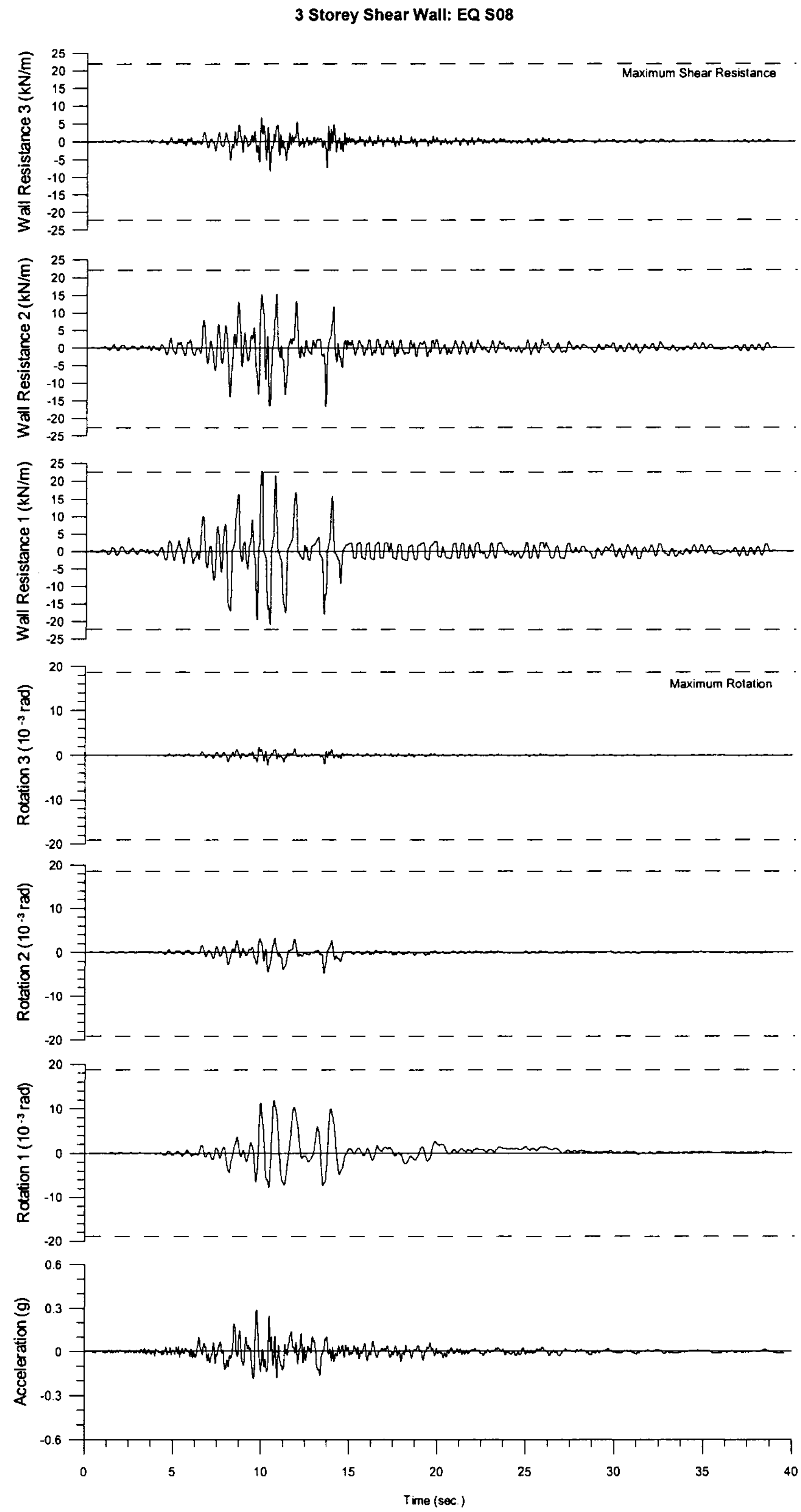


Figure G.36: Resistance and displacement time histories of the three-storey shear wall under ground motion record S08

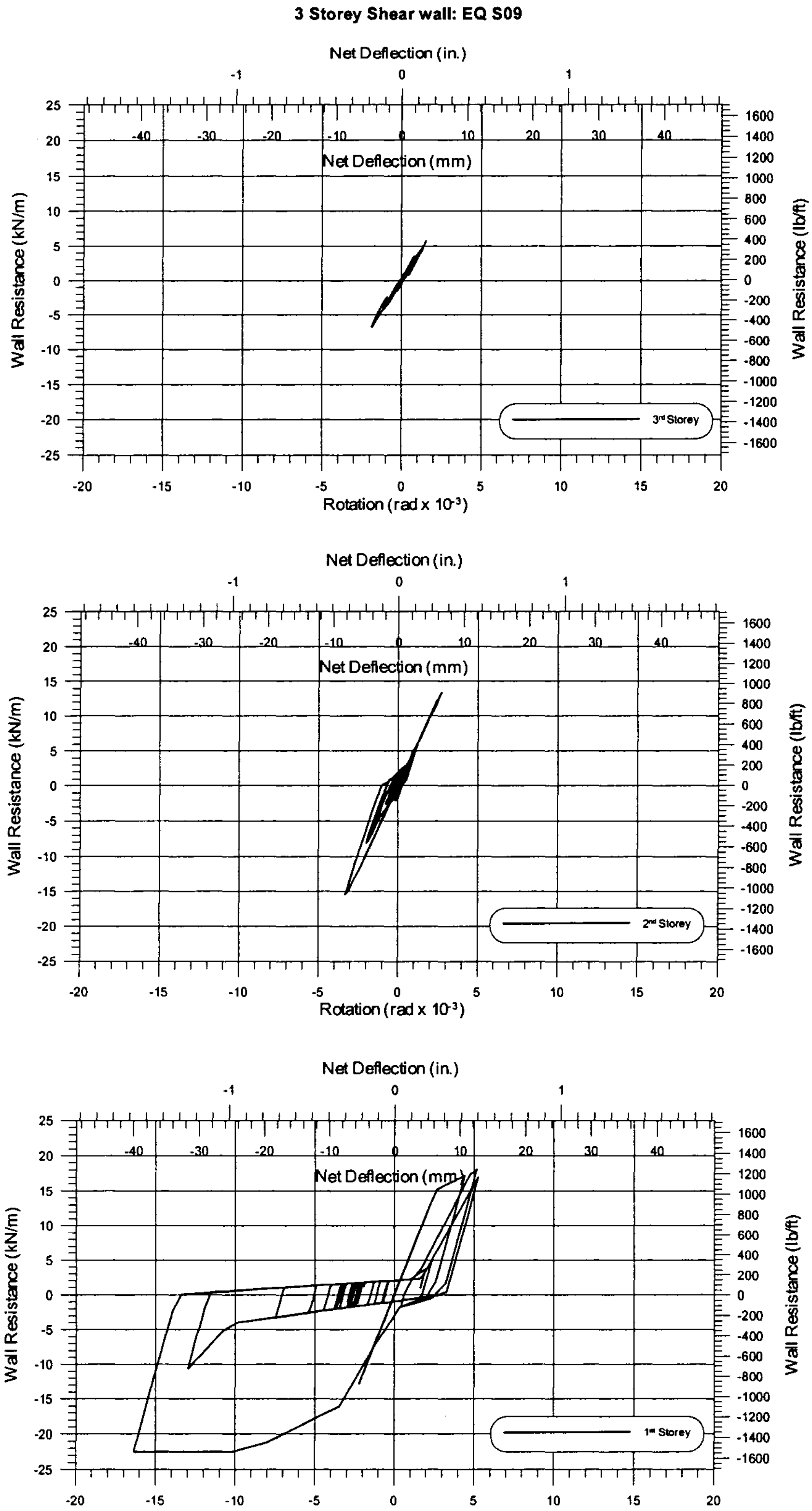


Figure G.37: Resistance vs. displacement hystereses of the three-storey shear wall under ground motion record S09

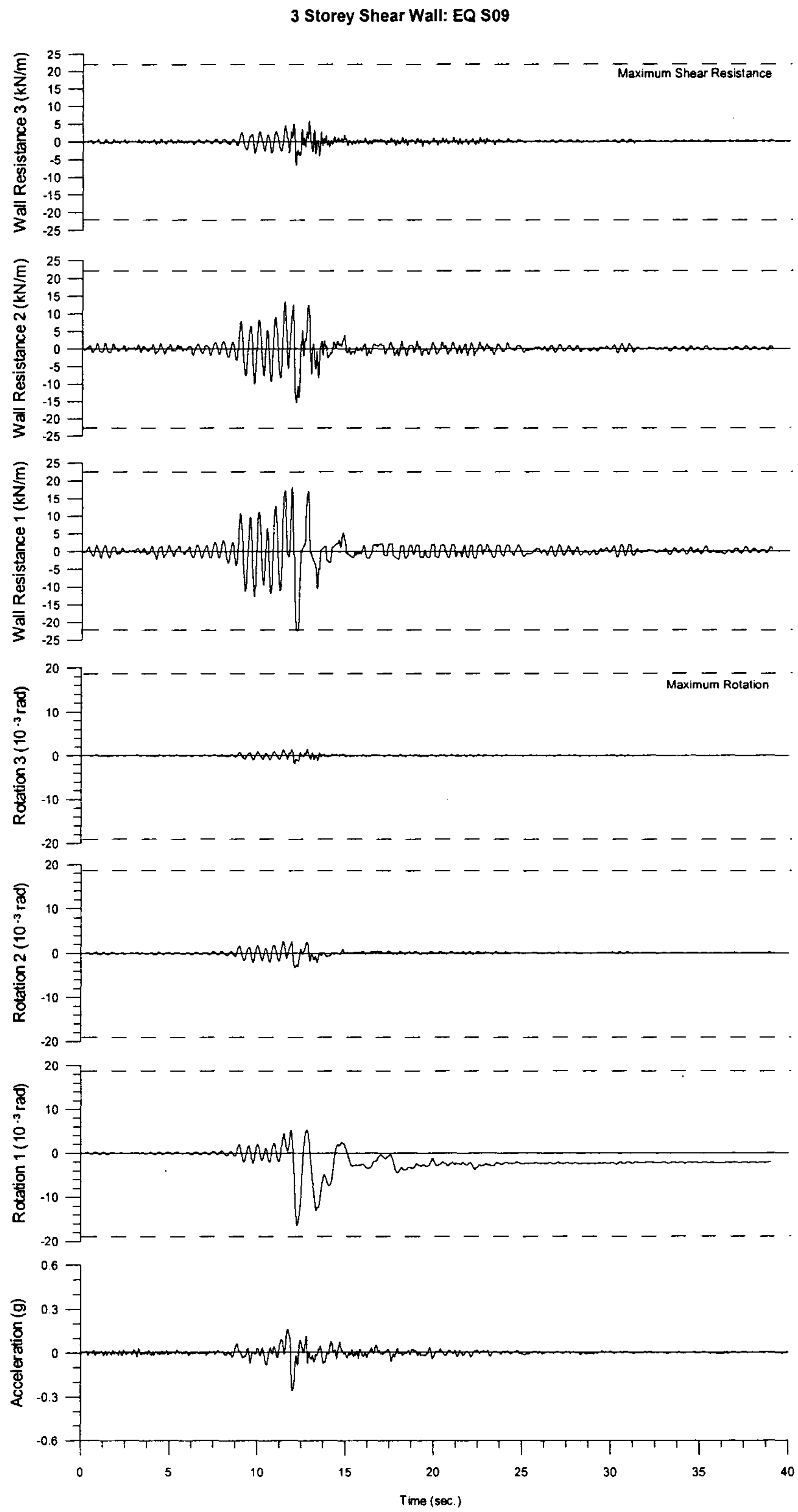


Figure G.38: Resistance and displacement time histories of the three-storey shear wall under ground motion record S09

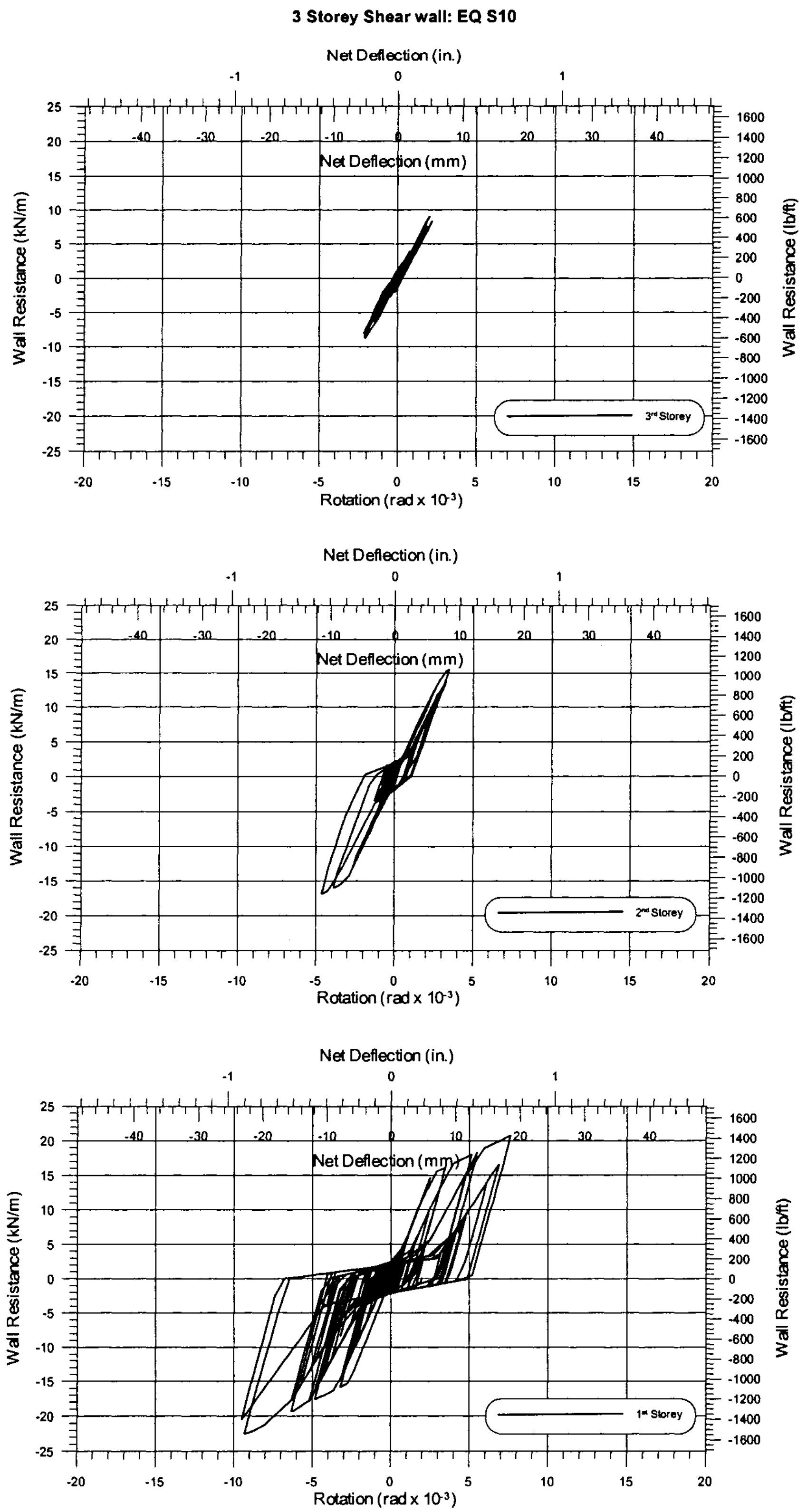


Figure G.39: Resistance vs. displacement hystereses of the three-storey shear wall under ground motion record S10

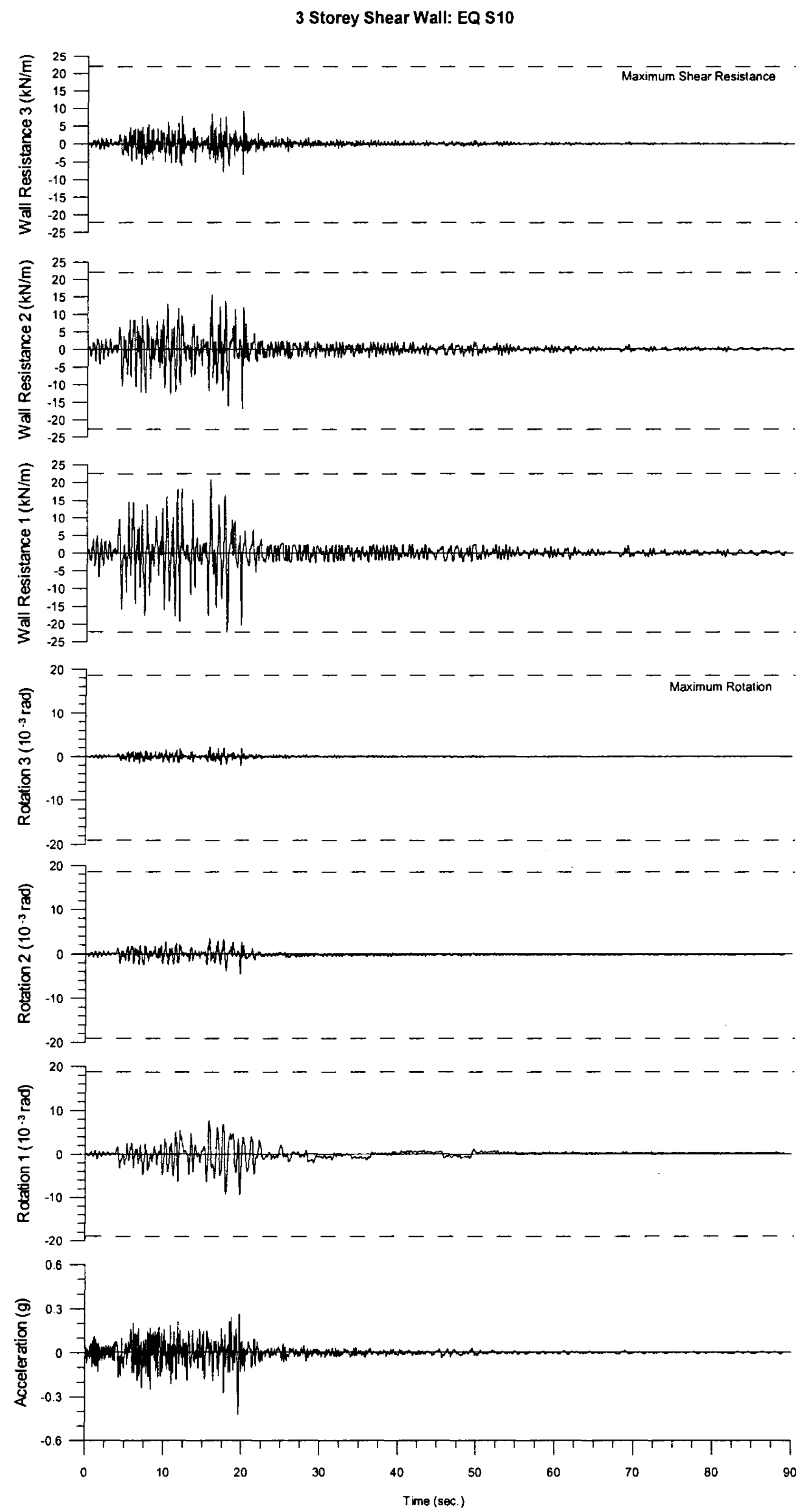


Figure G.40: Resistance and displacement time histories of the three-storey shear wall under ground motion record S10