Development of Seismic Design Provisions for
Steel Sheathed Shear Walls

By
Nisreen Balh

Department of Civil Engineering and Applied Mechanics
McGill University, Montréal, Québec, Canada
January 2010

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

ABSTRACT

Seismic design provisions for cold-formed steel sheathed (CFS) shear walls are not available in the NBCC or in the CSA-S136 Standard. This limits engineers in designing with such walls in seismic zones across Canada. The objective of this research was to develop design provisions for steel sheathed shear walls constructed with CFS framing.

To develop such standards, 54 walls of various configurations were tested at McGill University in the summer of 2008. The walls varied in framing and sheathing thickness, detailing and aspect ratio. The tests carried out at McGill were used to obtain design values for Canada and to confirm the US values that are listed in the AISI S213 Lateral Design Standard.

There were two types of tests carried out; monotonic and reversed cyclic. The monotonic tests consisted of a static load simulation to eliminate any strain rate effects and the wall specimen was pushed laterally to its limits. The second type of test followed the CUREE reversed cyclic protocol where the wall was loaded laterally in both directions following a series of increasing displacement amplitudes up to failure.

Test results were incorporated with data obtained from the US to determine nominal shear resistance values, corresponding resistance factor, overstrength and ductility factors as well as seismic force modification factors. The test data was analyzed using the Equivalent Energy Elastic-Plastic (EEEP) approach which provides an equivalent bi-linear elastic plastic curve to the non linear behaviour exhibited by shear wall tests by considering the total energy dissipation. Based on the test results, a material resistance factor, ϕ, of 0.7, an overstrength value of 1.4, a ductility-related force modification factor, R_d, of 2.5 and an overstrength-related force modification factor, R_o, of 1.7 are recommended.

Dynamic analysis of multi-storey structures was carried out to validate the recommended R-values and to determine height limits. According to FEMA P695, which provides a methodology for determining the lateral performance of lateral

framing systems, the test based seismic force modification factors were shown not to provide an acceptable level of safety against collapse. Subsequent analyses resulted in a recommendation of an R_d value of 2.0 and an R_o value of 1.3. A maximum height limit of 15m is also proposed.

RÉSUMÉ

Les dispositions séismiques pour la conception de murs de refends en acier laminé à froid ne sont pas disponibles dans le Code National du Bâtiment de Canada (CNBC) et la norme CSA S136 de l'Association Canadienne de Normalisation. Cela limite la capacité des ingénieurs à concevoir de tels murs dans les zones à activité sismiques à travers le Canada. Le but de cette recherche était de développer ces dispositions de conception sismique pour les murs de contreventements en acier laminé à froid.

Pour développer de tels directives, 54 murs de configurations diverses furent testés à l'université McGill au cours de l'été 2008. Ces configurations variaient l'epaisseur des colombages et l'épaisseur des panneaux d'acier ainsi que les arrangements et espacements des connexions et la longueur de murs. Les tests menés à McGill furent utilisés pour obtenir les valeurs nécessaires à la conception au Canada et à confirmer les valeurs des États-Unis listées dans le AISI S213 Lateral Design Standard.

Deux types de tests furent menés, soit à chargement monotone et cyclique-réversible. Les tests à chargement monotone consistaient en une simulation à chargement statique afin d'éliminer tout effet de déformation et les murs furent poussés latéralement jusqu'à leurs limites. Le second type de test suivit le protocole de chargement cyclique-réversible de CUREE où les murs furent chargés latéralement vers les deux directions suivant une série de déplacements à amplitudes croissantes menées jusqu'à l'effondrement.

Les résultats des tests furent incorporés avec les données obtenues des États-Unis pour déterminer les valeurs nominales de résistance en cisaillement, les facteurs de résistance correspondants, la sur-résistance et les facteurs de ductilité ainsi que les facteurs de modification de force séismiques. Les données des tests furent analysées en utilisant la méthode équivalente de l'énergie élasto-plastique (EEEP) qui fournit une courbe équivalente bilinéaire d'élasticité plastique au comportement non-linéaire démontré par les tests des murs de contreventements

en acier laminé à froid en tenant compte le la dissipation totale d'énergie. En se basant sur les résultats de ces tests, un facteur de résistance de matériel, ϕ, de 0.7, une valeur de sur-résistance de 1.4, un facteur de modification de force relié à la ductilité, R_d, de 2.5 et un facteur de modification de force relié à la sur-résistance, R_o, de 1.7 sont recommandés.

Une analyse dynamique sur des structures à multiples étages fut menée pour valider les facteurs de modification de la charge sismique recommandés et pour déterminer les limites d'hauteur. Selon le FEMA P695, qui fournit une méthodologie pour déterminer les performances latérales des systèmes à charpente latéral, les facteurs de modification de force sismique basé sur des tests n'ont pas prouvé fournir un niveau acceptable de sécurité contre l'effondrement.

ACKNOWLEDGEMENTS

I would first like to express my sincere gratitude to my professor and supervisor Professor Colin A. Rogers. His guidance and patience throughout this research are very much appreciated.

Sincere thanks to the team that helped me carry out this research: my research partner and friend Cheryl Ong-Tone, Anthony Caruso and Gabriele Rotili. Thank you all for your tremendous help. Thank you to Kostadin Velchev for his assistance in the research as well.

I would also like to thank all the lab technicians that continually provided support in the laboratory: Marek Przykorski, Ron Sheppard, Damon Kiperchuk, and Jon Bartczak, as well as Bill Cook and Jorge Sayat for all their technical support.

This research was made possible by the financial support of NSERC, AISI, and CSSBI. I would also like to thank Bailey Metal Products Ltd., ITW Buildex, Grabber Construction Products Ltd., and Simpson Strong-Tie Co. Inc. for all the materials that have been provided.

To my family, I would like thank to each and every one of you for your support and continuous encouragement. Thank you for believing in me as I would not have been able to go through this journey without you.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1 – INTRODUCTION

1.1 General Overview

In recent years, the construction industry has increasingly been moving towards sustainable methods of construction to reduce the consumption of natural resources. The use of steel framing in low rise building construction is becoming more common as cold-formed steel is an economical, non-combustible, high quality, significantly lighter alternative to more traditional materials. Steel framing is dimensionally stable and durable. It is an emerging choice for low to medium rise structures such as schools, stacked row houses, box stores, office buildings, apartments and hotels.

Cold-formed steel (CFS) has been gaining popularity in residential and commercial buildings. There are some districts where CFS framing has rapidly increased such as in Hawaii where 40 % of residential buildings are built with steel *(Steel Framing Alliance, 2005)*. A similar increase in CFS framing can also be seen in commercial buildings such as senior care centres, multi-family residential units and hotel applications. In Canada, CFS has not gained as much popularity in construction as Canadian standards do not provide designers with sufficient design guidelines. As a result, the progression of CFS framing construction has been slow in Canada.

Along with CFS framing, wood sheathing is typically used and gypsum is often included to provide lateral resistance. The concept of using cold-formed steel sheathing, however, is relatively new. The construction process is similar to wood sheathed shear walls as this system can also be constructed using platform framing. The overall behaviour of these walls is typically attributed to the connection between the sheathing material and the framing components. The in plane forces are transferred through the shear wall which operates within a system of floors, roof and foundation, then distributed through the structure. An example of a steel sheathed shear wall structure is given in Figure 1.1.

Figure 1.1 Cold-Formed Steel Wall Construction ***(Courtesy of Jeff Ellis, Simpson Strong-Tie)***

1.2 Statement of Problem

Currently, there are no design provisions that address the seismic performance of steel sheathed shear walls in the National Building Code of Canada (NBCC) *(NRCC, 2005)* or the Canadian Standards Association (CSA) S136 Design Specification *(2007)*. The lack of such design provisions severely limits engineers in their ability to design CFS structures. There is, however, a North American Lateral Design Standard for cold-formed steel *(AISI S213, 2007)*, which is published by the American Iron and Steel Institute. The NBCC refers to the CSA S136 for cold-formed steel related design aspects. In turn, the S136 Specification refers to AISI S213 for information regarding Canadian seismic detailing and design provisions for wood sheathed and strap braced shear walls. The US design provisions found in AISI S213 are more extensive than those available for use in Canada. In addition to wood sheathed shear walls, an engineer from the US may also design steel sheathed shear walls using S213. The shear resistance values listed in the Standard were based on the results of a limited number of tests carried out by Serrette *(1997)*. In order for engineers to utilize similar lateral framing systems in Canada it is necessary that design provisions be included in

AISI S213; as well, seismic design information for steel sheathed CFS shear walls would also need to be added to the NBCC.

1.3 Objectives

The purpose of this research project is to develop a Canadian design method for steel sheathed shear walls. This method will be proposed to the AISI for inclusion in the North American Lateral Design Standard. In addition, *R*-values and height limits will be proposed to the Standard Committee for Earthquake Design (SCED) for inclusion in the NBCC as there are no seismic design provisions for CFS frame systems in the current version. The specific objectives of this research are listed below:

i) Carry out tests on single-storey cold-formed steel frame/steel sheathed shear walls constructed from various framing and sheathing thicknesses;
ii) Incorporate data with test data from the US; Yu *et al. (2007)* and Ellis *(2007)*, extract necessary information and calculate relevant design parameters;
iii) Determine a resistance factor, ϕ, for ultimate limit states design, and recommend nominal shear resistance values, factor of safety, and seismic force modification factors, R_d and R_o;
iv) Recommend appropriate detailing and capacity design methods to achieve the ductility and overstrength associated with the seismic design parameters;
v) Establish height limits based on dynamic analysis of buildings using real and synthetic ground motion records; and
vi) Verify design parameters using appropriate dynamic testing software according to the FEMA P695 methodology.

1.4 Scope and Limitations of Study

The research involved full-scale testing of steel sheathed shear walls of various configurations. Variations to the configurations involved wall size, detailing, and thicknesses of sheathing and framing. The walls varied in size from 610mm by 2440mm (2'x8') to 2440mm by 2440mm (8'x8'). Detailing differed, as well, in terms of fastener schedule, reinforcement and component thickness. The materials used for the various configurations were 0.46mm (0.018") and 0.76mm (0.030") for the sheathing and 0.84mm (0.033") and 1.09mm (0.043") for the framing elements. Recommendations from the AISI and Canadian Sheet Steel Building Institute (CSSBI) were taken into consideration. A total of 18 different wall configurations were tested with both monotonic and reversed cyclic protocols. This amounts to a total of 54 tests; 31 of which were the responsibility of the author. The equivalent energy elastic-plastic *(Park, (1989) and Foliente, (1996))* analysis approach was applied to the analysis of all tests including test data from Yu *et al.(2007)* and Ellis *(2007)*.

Seismic ductility-related, R_d, and overstrength-related, R_o, factors were determined based on test results. Non-linear dynamic time history analyses of representative buildings were run using Ruaumoko software *(Carr, 2008)*. These analyses were also used to evaluate the 'test-based' R-values and to recommend appropriate seismic height limits for buildings constructed with CFS framed steel sheathed shear walls. Structures located in Vancouver and ranging from two to seven storeys were included in the dynamic analysis phase of the study. The results were verified in accordance with the FEMA P695 methodology *(FEMA, 2009)*.

Ancillary tests included coupon tests of the framing and sheathing materials and connections tests to evaluate the shear and bearing capacity of the fasteners.

1.5 Research Outline

A general overview of the research project is given in this chapter with a brief literature review. A more detailed literature review can be found in the report by Ong-Tone *(2009)*.

The test program and test procedures are explained in Chapter 2, which includes material and component properties as well as methods for shear wall construction. Modes of failure are also discussed.

The extraction of design parameters is discussed in Chapter 3. Test data from the US is incorporated with test data from McGill University. All data is reduced in the same manner to obtain uniformity in analysis and results. Design parameters are established along with other factors and limitations.

Chapter 4 discusses in detail the design method for steel sheathed shear walls. A description of the building models is provided and the appropriate loads are summarized. Guidelines are outlined in order to provide designers with a methodology that can be followed for the design of shear walls in low to medium rise construction.

Verification of seismic modification factors of R_d and R_o is presented in Chapter 5 according to the FEMA P695 methodology. Dynamic modeling of the model representative buildings was performed using a suite of 45 ground motion records.

Finally, Chapter 6 provides conclusions for this research project. Recommendations on design parameters are presented as well as suggestions for future research.

1.6 Literature Review

In this section, information pertaining to steel sheathed shear walls is presented and valuable information from similar research is summarized. This past research provides background information for testing and analysis and offers guidelines for establishing design methods.

1.6.1 Relevant Research on Cold-Formed Steel Structures

There has been extensive research at McGill University on CFS framing with various sheathing or bracing configurations. Al-Kharat *(2005)*, Comeau *(2008)* and Velchev *(2008)* have tested single storey cross-braced CFS walls connected by either screws or welds. Zhao *(2002)*, Branston *(2004),* Boudreault *(2005)*, Chen *(2004)*, Rokas *(2005)*, Hikita *(2006)* and Blais *(2006)* have tested and analyzed single storey wood sheathed CFS shear walls. They have each provided thorough reviews of past research and existing test programs on CFS walls in different countries. Morello *(2009)* has also tested wood sheathed shear walls and analyzed the effect of the inclusion of gypsum as a sheathing material. All the tests were performed in the Jamieson Structural Laboratory at McGill University in a loading frame specifically designed by Zhao *(2002)* for CFS shear wall testing. Two loading protocols have historically been relied on to carry these shear wall tests; the first being a monotonic test where shear walls were statically loaded up to failure, the second loading protocol is the reversed cyclic test which follows the ASTM E2126 *(2007)* and the methodology provided by the Consortium of Universities for Research in Earthquake Engineering (CUREE) protocol *(Krawinkler et al., 2000)*. The CUREE protocol was initially established for wood framed shear walls but has been found to be applicable to CFS framed shear walls as well. The CUREE protocol mimics the behaviour and deformations of shear walls under seismic loading. Most of the tests have been on single-storey shear walls. Currently, there are on-going studies on multi-storey shear walls as well as dynamic shake table testing of two-storey shear walls.

Branston *(2004)* reviewed various methods for interpreting data, and the equivalent energy elastic plastic (EEEP) approach was found to be the most appropriate for the walls tested. The behaviour of shear walls is non linear and a simplified method for analysis is required. The EEEP technique provides a bilinear curve that is equivalent to the monotonic shear resistance - lateral deformation curve obtained by physical testing. It was modified and improved by Foliente *(1996)* after its first development by Park *(1989)*. Subsequently, Boudreault *(2005)* evaluated methods for modeling the hysteretic behaviour of shear walls under reversed cyclic tests. The Stewart *(1987)* hysteretic element was found to be a suitable model for the hysteretic behaviour of shear walls even though it does not account for strength degradation. The model was developed for wood sheathed-wood framed shear walls but it was deemed appropriate for CFS framed shear walls as well due to the similar behavioural characteristics of the two framing types. Boudreault *(2005)* also presented a procedure for determining test based ductility-related and overstrength- related values for use with the 2005 NBCC *(NRCC, 2005)*.

The effects of gravity loads on the design of shear walls were assessed by Hikita *(2006)*. In a limited number of shear walls tests, by Branston *(2004)*, the chord studs showed permanent deformation due to the compression forces associated with lateral loading. The design of these stud members (columns) is important in order to prevent collapse of the framing system, i.e. to maintain a framing system that continues to carry gravity loads post earthquake. The inclusion of gravity loads is critical for the design of chord studs, and as such specific design provisions were incorporated in AISI S213 for wood sheathed shear walls and strap braced walls.

With respect to steel sheathed shear walls, tests have only been carried out in the US by Serrette *(1997)*, Yu *et al. (2007)* and Ellis *(2007)*. The tests performed by Serrette *(1997)* at the Santa Clara University were limited to 2:1, 1220x2440mm (4’x8’), and 4:1, 610x2440mm (2’x8’), shear walls using 0.84mm (0.033”) CFS framing with nominal sheathing thicknesses of 0.46mm (0.018”) and 0.68mm

(0.027”). Monotonic and reversed cyclic loading protocols were utilized in these research programs. Serrette *(1997)* relied on the sequential phase displacement (SPD) protocol for the reversed cyclic tests. Yu *et al. (2007)*, at the University of North Texas, expanded the test program for steel sheathed shear walls by including specimens constructed with 0.76mm (0.030”) and 0.84mm (0.033”) nominally thick sheathing. Some tests with 0.68mm (0.027”) sheathing were carried out by Yu *et al. (2007)* to repeat those run by Serrette *(1997)*. Each test had screw configurations of 50mm (2”), 100mm (4”), and 150mm (6”) perimeter spacing. The expanded test program was to provide the AISI S213 technical committee with additional design information. However, there were inconsistencies in the data between Serrette *(1997)* and Yu *et al. (2007)* which became the basis for the tests by Ellis *(2007)*. Ellis carried out seven tests to determine the possible causes for the discrepancies among the existing test data. The use of thicker framing material for studs and tracks of 1.09mm (0.043”) was also examined with thicker sheathing materials. The cyclic tests that were carried out used the CUREE protocol which is a possible reason as to why higher shear resistances were measured compared with the SPD approach.

1.6.2 Design Standards

The 2005 NBCC *(NRCC, 2005)* and the CSA-S136 Specification *(2007)* provide no guidelines that address the seismic performance of CFS shear walls. The North American Standard for Cold-Formed Steel Framing – Lateral Design, AISI S213 *(AISI, 2007)*, addressed the design of CFS lateral force resisting systems (LFRS) for wind and seismic forces. It is applicable for use in the US, Mexico, and Canada based on the requirements of the International Building Code (IBC) *(ICC, 2003*) and the NFPA 5000 Construction and Safety Code *(NFPA, 2003)*, and the research presented by Serrette *(1995, 1996,1997)*, Tarpy *(1976-1980)*, APA-The Engineering Wood Association *(1993-2005)* and the Uniform Building Code (UBC) *(ICBO, 1997)*. It provides Allowable Strength Design (ASD) and Load and Resistance Factor Design (LRFD) information for the US and Mexico, as well as Limit States Design (LSD) provisions for Canada. The most recent version of

AISI S213 includes provisions for strap braced wall and wood sheathed shear wall structures specifically for use in Canada. The AISI S213 Specification only contains US provisions for steel sheathed CFS framed shear walls. It presents nominal shear strength values for 0.46mm (0.018") and 0.68mm (0.027") steel sheathing with 0.84mm (0.033") CFS framing. It does not list equivalent nominal shear resistances for wind, seismic, and other in-plane lateral loads for Canada.

Steel sheathed shear walls can only be designed for low seismic zones, such as Calgary, where $I_EF_aS_a(0.2)$ is less than 0.35, with a height limitation of 15m, since they fall under the category of "other cold-formed steel seismic force resisting systems (SFRS) not listed" in the section pertaining to Canada found in Table A4-1 of the AISI S213 *(2007)*. The seismic force modification factors, R_d and R_o, are equal to 1.0 which represents elastic behaviour where capacity based design is not required. For moderate and high seismic zones, such as Vancouver and Quebec, where $I_EF_aS_a(0.2)$ is greater than 0.35, the use of steel sheathed shear walls in construction is not permitted due to the lack of design information.

The AISI S213 Standard also defines a method for estimating the in-plane deformation of a shear wall that can be verified using appropriate dynamic analysis software. The 2005 NBCC provides spectral accelerations for different cities across Canada and it outlines a method for non linear analysis of shear walls using the Equivalent Static Force Procedure for regular buildings. It is a simplified and conservative method for determining the lateral earthquake force and the fundamental period, T_a, of a structure. Buildings should be checked for irregularity as prescribed by the 2005 NBCC in terms of stiffness, strength, and geometry where the Dynamic Analysis Procedure may be more appropriate for analysis.

1.6.3 Dynamic Analysis

The seismic force modification factors, R_d and R_o, determined from physical testing of shear walls can be verified according to the Federal Emergency Management Agency (FEMA) P695 document methodology *(2009)*. The FEMA P695 is a standard methodology for verifying the adequacy of seismic design and performance of structures with the intention of providing safe structures and minimizing the risk of collapse. *R*-values and height limits can be verified using collapse probability concepts including collapse fragility curves. Vamvatsikos and Cornell *(2002)* have developed a technique to obtain the collapse probability of a structure by means of Incremental Dynamic Analysis (IDA). It uses select ground motion records scaled with different factors and applied to a model building. Each model building has to be analyzed using a suitable non linear dynamic analysis software from which the inter-storey drifts can be determined. Comeau *(2008)*, Velchev *(2008)*, and Morello *(2009)* have used Ruaumoko software *(Carr, 2008)* for dynamic analysis using 45 ground motion records with different scaling factors from zero up to eight in increments of 0.20. The collapse probability is determined by the earthquake intensity that causes the model building to collapse or to reach the maximum defined inter-storey drift.

1.6.4 Ground Motion Records

A database of synthetic earthquake records has been made available by Atkinson *(2009)*. The records are compatible with the specifications for the uniform hazard spectrum (UHS) having a 2% chance of being exceeded in 50 years as described in the 2005 NBCC . Dynamic analysis of buildings requires the input of ground motion records. Since only a limited number of real earthquake records can be utilized for dynamic analysis, the database provides a valuable tool for ground motion record selection. The earthquake time histories are generated for a range of distances and magnitudes using the stochastic finite-fault method for Site Classes A, C, D and E. Each record can be scaled to match the UHS of the required city and modified to fit criteria specific to different cities.

1.7 Summary

A substantial amount of research has been carried out on CFS framed/wood sheathed shear walls, as well as braced walls. However, only a limited number of tests for steel framed/steel sheathed shear walls have been completed in the US. No equivalent data for use in Canada is available. Engineers in the US are able to utilize the AISI S213 Standard *(2007)*.

The information gathered from past research has provided valuable information that served as a basis for the test program of steel sheathed shear walls and the development of design methods at McGill. The same loading protocols used in the past (monotonic and CUREE reversed cyclic) were applied to the testing of the steel sheathed shear walls. Boudreault *(2005)* provided an extensive review of analysis methods, and Branston *(2004)* thoroughly explained the extraction of necessary information from test data and the calibration of values to determine factors for use in seismic design. The same analysis approach of data reduction using the EEEP method was used from which the seismic force modification factors, overstrength factor, ductility factor and the material resistance factor were determined.

The procedures for dynamic analysis and ground motion record selection have been examined and tested by Comeau *(2008)*, Velchev *(2008)* and Morello *(2009)*. The performance of steel sheathed shear walls was assessed by the same procedure for dynamic analysis. The 'test based' *R*-values and design method were verified according to the FEMA P695 which was also used to verify the performance of CFS framed/wood sheathed shear walls and braced walls.

CHAPTER 2 – SHEAR WALL TEST PROGRAM

2.1 Test Frame Setup and Background Information

As part of the steel sheathed shear wall research program, a total of 54 steel-sheathed single-storey shear walls were tested during the summer of 2008 in the Department of Civil Engineering and Applied Mechanics' structural laboratory at McGill University. Of these walls, 31 were the responsibility of the author while the remaining were tested by Ong-Tone *(2009)*. Platform framing techniques were used for construction where the walls were placed horizontally on the ground for assembly then erected vertically into the testing frame, which was designed and installed in 2002 (Figures 2.1 and 2.2). The testing frame is equipped with a 250kN MTS dynamic loading actuator with a ±125mm stroke. Lateral movement of the walls is resisted by means of lateral supports. A detailed review of the properties of the testing frame can be found in Zhao *(2002)*.

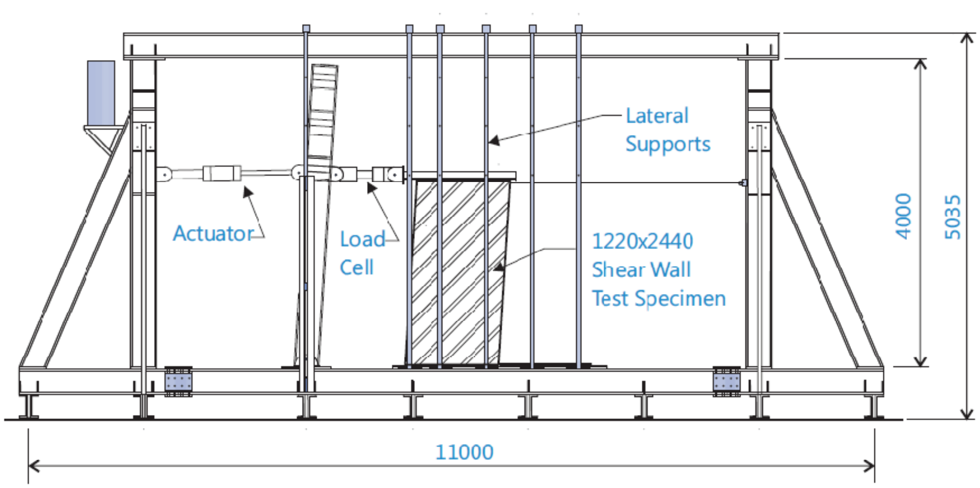


Figure 2.1 Test Frame

Figure 2.2 Wall Installation in Test Frame

2.2 Steel Frame/ Steel Panel Shear Walls Testing Program

The test specimens comprised a cold-formed steel sheathing screw connected to a cold-formed steel frame. The sheathing thickness, framing thickness (wall studs and tracks), and fastener spacing were varied as per the configurations listed in Table 2.1. Initially, the test matrix consisted of 43 shear wall specimens; complementary specimens were added to provide additional data. Overall, there were 37 1220x2440mm (4'x8') walls, 10 610x2440mm (2'x8') walls, two 1830x2440mm (6'x8') walls and five 2440x2440mm (8'x8') walls. This thesis documents the walls tested by the author; details of the remaining walls can be found in the work of Ong-Tone *(2009)*. A detailed description of each shear wall configuration can be found in Appendix A. Configuration 17 was added to determine the effects of concentrated connections at the corners of the wall with reduced fasteners in the middle. Configuration 18 was also added to observe the effects of intermediate fastener spacing of 75mm (3").

Table 2.1 Test Matrix (Nominal Dimensions)

Configuration	Sheathing Thickness (mm)	Wall Length (mm)	Wall Height (mm)	Fastener Spacing (mm)	Framing Thickness (mm)	Number of Tests and Protocol[2]
1[1]	0.46	1220	2440	150/300	1.09	3M & 2C
2[1]	0.46	1220	2440	50/300	1.09	2M & 2C
3[1]	0.46	1220	2440	150/300	0.84	2M & 3C
4	0.76	1220	2440	150/300	1.09	2M & 2C
5	0.76	1220	2440	100/300	1.09	3M & 2C
6	0.76	1220	2440	50/300	1.09	3M & 2C
7	0.76	1220	2440	100/300	0.84	1M
8[1]	0.76	610	2440	100/-	1.09	2M & 2C
9[1]	0.76	610	2440	50/-	1.09	3M[3] & 2C
10[1]	0.76	610	2440	100/-	0.84	1M
11[1]	0.76	2440	2440	100/300	1.09	2M & 2C
12	0.76	1830	2440	100/300	1.09	1M
13	0.76	1830	2440	50/300	1.09	1M
14[4]	0.76	1220	2440	50/300	0.84	4M
15[5]	0.76	1220	2440	100/300	1.09	1M
16[6]	0.76	1830	2440	100/-	1.09	1M
17[1]	0.46	1220	2440	-/300	1.09	2M
18[1]	0.46	1220	2440	75/300	1.09	1M

[1] Author's test specimens
[2] M-Monotonic, C-Cyclic
[3] Addition of bridging to Test 9M-c
[4] Various reinforcement schemes
[5] Raised hold-downs
[6] Wall with window opening

2.3 Specimen Fabrication, Test Setup and Instrumentation

This section provides a description of the materials used in construction, wall specimen fabrication, as well as the test setup and instrumentation.

2.3.1 Materials

The specimens were composed from a combination of the following elements:

- 0.46mm (0.018”) 230MPa (33 ksi) nominal thickness and strength cold-formed steel sheet. Sheathing mounted vertically on one side of the steel frame (ASTM A653 *(2008)*)

- 0.76 mm (0.030”) 230MPa (33 ksi) nominal thickness and strength cold-formed steel sheet. Sheathing mounted vertically on one side of the steel frame (ASTM A653 *(2008)*)

- 0.84mm (0.033”) 230MPa (33 ksi) nominal thickness and strength cold-formed steel stud (ASTM A653 *(2008)*). Studs mounted vertically within frame at a spacing of 610mm (2’) on centre. Nominal dimensions of the steel studs were 92.1mm (3-5/8”) web and 41.3mm (1-5/8”) flange and 12.7mm (1/2”) lip.

- 1.09mm (0.043”) 230MPa (33 ksi) nominal thickness and strength cold-formed steel stud (ASTM A653 *(2008)*). Studs mounted vertically within frame at a spacing of 610mm (2’) on centre. Nominal dimensions of the steel studs were 92.1mm (3-5/8”) web and 41.3mm (1-5/8”) flange and 12.7mm (1/2”) lip.

- 0.84mm (0.033”) 230MPa (33 ksi) nominal thickness and strength cold-formed steel top and bottom tracks (ASTM A653 *(2008)*). Nominal dimensions of the steel tracks were 92.1mm (3-5/8”) web and 31.8mm (1-1/4”) flange.

- 1.09mm (0.043") 230MPa (33 ksi) nominal thickness and strength cold-formed steel top and bottom tracks (ASTM A653 *(2008)*). Nominal dimensions of the steel tracks were 92.1mm (3-5/8") web and 31.8mm (1-1/4") flange.

- Simpson Strong-Tie S/HD10S hold-down connectors. The hold-down connectors were attached to the interior base of each chord stud, 76mm (3") above the bottom track by 24- No.10 gauge 19.1mm (3/4") self-drilling Hex head washer head screws. Each hold-down connector was attached to the test frame by a 22.2mm (7/8") B7 grade threaded anchor rod (ASTM A193 *(2008)*).

- No.8 gauge 12.7mm (1/2") self-drilling wafer head Phillips drive screws (ITW Buildex) were used to connect the studs to the track and back to back chord studs.

- No.8 gauge 19.1mm (3/4") self-drilling pan head LOX drive (Grabber Superdrive) screws were used to connect the sheathing to the frame 9.5mm (3/8") from edge of the sheathing panel.

2.3.2 Specimen Fabrication

All components of the frame were prepared before assembly. All top and bottom tracks were pre-drilled to accommodate 19.1mm (3/4") A325 bolts and 22.2mm (7/8") threaded anchor rods for hold-downs. Built-up chord studs were assembled with two studs back-to-back with a hold-down installed at 75mm (3") from the base with 24- No.10 gauge 19.1mm screws (Figure 2.3).

Figure 2.3 Chord Stud Assembly

The components were assembled using the platform building technique prior to attaching the sheathing. Except for 610mm (2’) long walls, a field stud was placed at a spacing of 610mm (2’) on-centre in the 1220mm (4’) and 2440mm (8’) long walls. The frame was assembled using No.8 wafer head screws at each corner with the hold-downs facing inward (Figure 2.4). The sheathing was then placed on the frame, marked, and installed with No.8 gauge 19.1mm (3/4”) pan head screws according to the fastener schedule in Table 2.1. The sheathing was fastened around the perimeter of the wall specimen along the tracks and the chord studs at an edge distance of 9.5mm (3/8”) and along the field stud, if available (Figure 2.4). The sheathing panels were available in two sizes; 610x2440mm (2’x8’) and 1220x2440mm (4’x8’). The 610mm (2’) long walls were sheathed with a single 610x2440mm (2’x8’) sheathing panel whereas the 1220mm (4’) long walls were sheathed with a 1220x2440mm (4’x8’) sheathing panel. The longer walls measuring 2440mm (8’) in length, were sheathed with two 1220x2440mm (4’x8’) sheathing panels side by side. The panels were placed with a flush contact at the middle of the wall on a single stud. In one wall, 9M-c, a row of bridging was placed at each quarter span along the height of the wall in the stud knock-out holes. Bridge clip angles were attached to each hole in the studs for the bridging to be attached to the frame (Figure 2.5).

Figure 2.4 Frame and Sheathing Assembly

Figure 2.5 Bridging and Bridge Clip in Test 9M-c

2.3.3 Test Setup

To test the specimens after their construction, each specimen was transferred carefully from the construction area and into the test frame. Once in place, the wall was anchored into place with 19.1mm (3/4") A325 shear anchors at the base to the testing frame and at the top to the loading beam. Cut washers were used at the base with the shear anchors to minimize damage caused by bearing. At the top, cut washers were used between the loading beam and the nut, and square plate washers were used between the frame and aluminum spacer plate. A threaded anchor rod was placed at the base through each hold-down connecting it to the frame as well to transfer loads from the chord stud to the frame. The load on the wall was monitored during installation to avoid damage. Test instrumentation units were placed immediately before testing. Any damage in the test specimen prior to testing was noted at this point.

2.3.4 Instrumentation and Data Acquisition

In order to assess the performance of each test specimen, linear variable differential transformers (LVDTs) were placed on the frame, as well as load cells, and a string potentiometer. There were four LVDTs placed on each wall to measure lateral slip and uplift movement at the base of the chord stud (Figure 2.6). The LVDTs monitored any uplift movement or slip that may have occurred at the base due to the lateral applied force. In addition to the four LVDTs, a string potentiometer was attached to the top at the end of each specimen to record the lateral displacement at the top of the wall (Figure 2.6). The LVDTs and string potentiometer were positioned on small non-structural steel plates that were connected to the frame (Figure 2.7). The actuator had an internal LVDT to monitor displacement. Finally, an accelerometer was placed on the actuator's load cell to measure the acceleration in the reversed cyclic tests. In addition to displacement sensors, load cells were placed at each end of the frame beneath the anchor rods to monitor the vertical uplift forces transmitted through the chord studs.

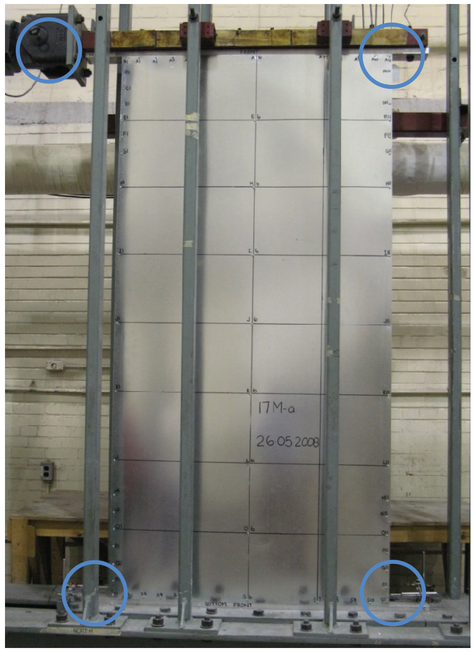


Figure 2.6 Instrumentation Locations

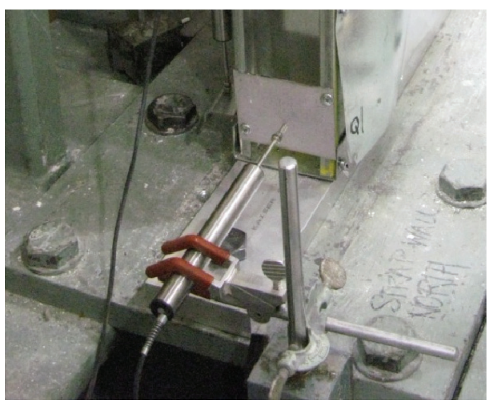


Figure 2.7 LVDT Placement on Side Plate

2.4 Testing Protocols

There were two testing protocols used for the testing of shear walls. The first type was the monotonic protocol and the second type was the CUREE reversed cyclic protocol *(Krawinkler et al., (2000), ASTM E2126 (2007))*. The CUREE cyclic protocol was dependent on the results of the monotonic test results.

2.4.1 Monotonic Testing

The first set of tests comprised of controlled lateral displacement in one direction, also known as a monotonic test protocol. Lateral displacement occurred at a constant rate of 2.5mm/min, to avoid any strain rate effects, and thus simulated static or wind loading. It is similar to the protocol used by Serrette *(1997)* and consistent with the loading used for wood sheathed shear wall and strap braced wall tests at McGill University (*Branston et al. (2006), Comeau (2008), Velchev (2008), and Morello (2009)*). Force was applied starting at zero displacement which was determined as the point at which the wall specimen did not carry any lateral load. Loading continued until the load on the specimen degraded significantly or until an approximate displacement of 100mm was reached. When the specimens were too flexible, loading was stopped at about 100mm (3.93”) because turnover would control which is well beyond the allowable drift limit of 2.5% of wall height as prescribed by the 2005 NBCC *(NRCC, 2005).* A typical relationship between resistance and displacement for a monotonic test is shown in Figure 2.8.

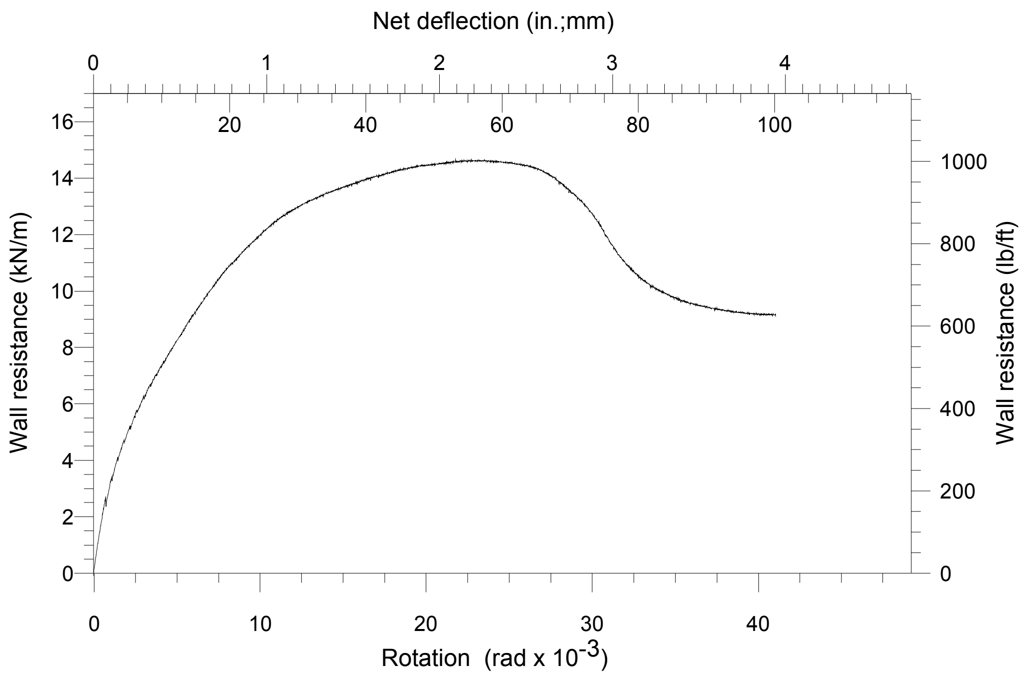


Figure 2.8 Monotonic Test Data Curve

2.4.2 Reversed Cyclic Testing

After the completion of the monotonic tests for certain configurations as listed in Table 2.1, reversed cyclic tests were performed based on the CUREE (Consortium of Universities for Research in Earthquake Engineering) ordinary ground motions protocol. The CUREE cyclic protocol for ordinary ground motions was chosen for the testing of the steel sheathed shear walls as described by Krawinkler *et al. (2000)* and ASTM E2126 *(2007).* The CUREE protocol is consistent with the protocol that was used in past research at McGill University for CFS framing with wood sheathing or strap braced walls (*Branston et al. (2006), Comeau (2008), Velchev (2008), and Morello (2009)*). The displacements for the CUREE protocol cycles are based on delta, Δ, which is defined as 60% of the average displacement corresponding to 80% of the post ultimate load reached by the monotonic tests for each configuration. The tests were run at 0.5Hz starting at 0.050Δ for 6 cycles as initiation which are well within the elastic range of the wall specimen. The initiation cycles allow the observer/author to confirm that the wall and all instrumentation are properly positioned before further loading takes place. The first primary cycle, which attempts to push the wall into the inelastic range, starts at 0.075Δ followed by a set of trailing cycles that are defined as 75% of the primary displacement. A complete cycle is defined as equal amplitude to the positive side and the negative side starting from, and returning to, the origin. The primary cycles that follow have incrementally increasing amplitude following this sequence: 0.1Δ, 0.2Δ, 0.3Δ, 0.4Δ, 0.7Δ, 1.0Δ. Primary cycles in excess of the defined sequence follow the same pattern with an increase of 0.5Δ in amplitude. When the amplitude reached 100mm, the actuator was slowed down to 0.25Hz due to deficiency in hydraulic oil supply. All loading protocols are provided in Appendix C with an example loading protocol given in Table 2.2 and a displacement time history in Figure 2.9. A typical relationship between resistance and displacement for a reversed cyclic test in the form of hysteretic curves is shown in Figure 2.10.

Table 2.2 CUREE Protocol Input Displacements for Test 11

$\Delta=0.6*\Delta_m$	31.94	Screw Pattern: 4"/12"	
		Sheathing: 0.027"	
Displ.	Actuator Input (mm)	No. Of cycles	
0.050 Δ	1.597	6	Initiation
0.075 Δ	2.396	1	Primary
0.056 Δ	1.797	6	Trailing
0.100 Δ	3.194	1	Primary
0.075 Δ	2.396	6	Trailing
0.200 Δ	6.388	1	Primary
0.150 Δ	4.791	3	Trailing
0.300 Δ	9.582	1	Primary
0.225 Δ	7.187	3	Trailing
0.400 Δ	12.776	1	Primary
0.300 Δ	9.582	2	Trailing
0.700 Δ	22.359	1	Primary
0.525 Δ	16.769	2	Trailing
1.000 Δ	31.941	1	Primary
0.750 Δ	23.956	2	Trailing
1.500 Δ	47.912	1	Primary
1.125 Δ	35.934	2	Trailing
2.000 Δ	63.882	1	Primary
1.500 Δ	47.912	2	Trailing
2.500 Δ	79.853	1	Primary
1.875 Δ	59.889	2	Trailing
3.000 Δ	95.823	1	Primary
2.250 Δ	71.867	2	Trailing
3.500 Δ	100.000	1	Primary
2.625 Δ	75.000	2	Trailing

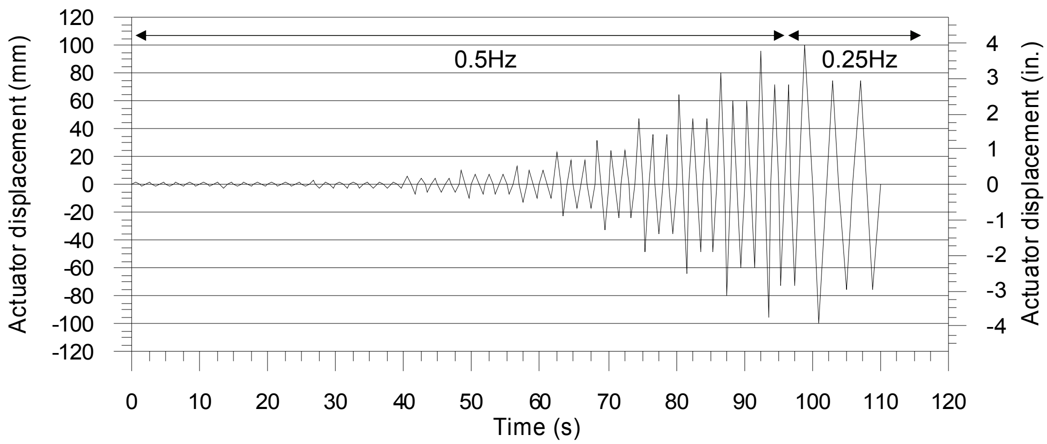


Figure 2.9 CUREE Displacement Time History for Test 11

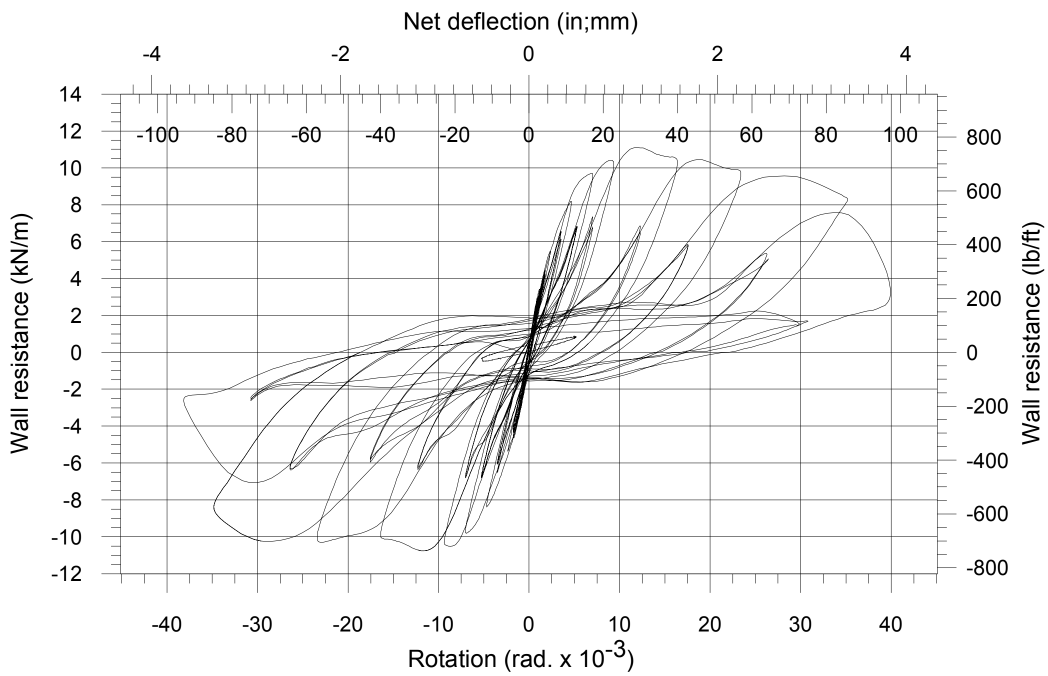


Figure 2.10 CUREE Reversed-Cyclic Test Data Curve

2.5 Observed Failure Modes

In all cases elastic shear buckling of the sheathing was first observed as the tension field action developed. This was followed by sheathing connection failures and in some cases damage to the steel frame, which was attributed to the concentrated tension field forces. The main mode of failure that took place was in the screw connections between the sheathing and the frame. However, it was not uncommon to see twisting and buckling of the chord studs and uplift damage to the tracks. This section describes each mode of failure that was observed; for each test an observation sheet is provided in Appendix B. The described failure modes did not occur independently of one another, rather combinations of these modes were observed. Furthermore, the connection failure modes usually involved multiple fasteners with failure occurring in a progressive unzipping action.

2.5.1 Connection Failure

2.5.1.1 Tilting of Sheathing Screw

Most connection failures started with tilting of the screw due to the eccentric load placed on the connector (Figure 2.11). The shear applied on the fastener also led to local bearing in the frame and sheathing which loosened the connection.

Figure 2.11 Sheathing Screw Tilting

2.5.1.2 Pull-out Failure of Sheathing Screw (PO)

As tilting occurred during testing, the connection loosened and expanded the screw hole within the frame. The fastener was fully pulled out of the frame with the application of enough force. The screw remained intact with the sheathing in some cases (Figure 2.12).

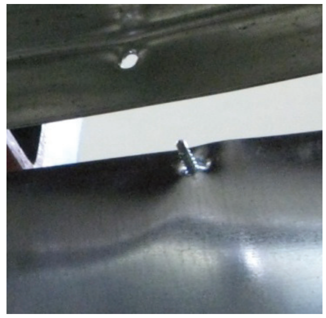


Figure 2.12 Sheathing Screw Pull-out Failure

2.5.1.3 Pull-through Sheathing Failure (PT)

The pull-through sheathing mode of failure can also be described as punching shear of the fastener through the sheathing. The fastener pulled through the sheathing mainly in the field connections of the specimens. The head of the screw penetrated completely through the sheathing but remained intact with the frame (Figure 2.13).

Figure 2.13 Screw Pull-Through Sheathing Failure

2.5.1.4 Bearing Sheathing Failure (SB)

As the wall specimen moved laterally, the sheathing moved relatively independently of the frame. Since the sheathing material was comparatively thinner, the bearing damage at the fastener led to a progressive degradation in load (Figure 2.14).

Figure 2.14 Sheathing Steel Bearing

2.5.1.5 Tear-out Sheathing Failure (TO)

Tear-out failure occurred on the perimeter of the sheathing since the screws were placed at a distance of 9.5mm (3/8”) from the panel edge. It is a severe version of bearing failure where the screw progressively tore out from the edge of the sheathing (Figure 2.15).

Figure 2.15 Screw Tear-out Failure

2.5.1.6 Screw Shear Fracture Failure

The screw shear fracture failure mode occurred in a few instances. It usually occurred at the corners of the wall where the screw was driven through three layers of steel (sheathing, track and stud) and thus was restrained from tilting. The shear fracture typically occurred just below the head of the screw (Figure 2.16).

Figure 2.16 Screw Shear Fracture Failure

2.5.2 Sheathing Failure

2.5.2.1 Shear Buckling of Sheathing

The sheathing panel showed elastic shear buckling soon after loading commenced. Tension field action also developed in a diagonal pattern across the panel in the direction of the load. Figure 2.17 is an example of a wall specimen before testing and Figure 2.18 shows the tension field action and shear buckling after a monotonic test. In the case of reversed cyclic loading, the shear buckling and tension field action were visible in both directions as represented in Figure 2.19.

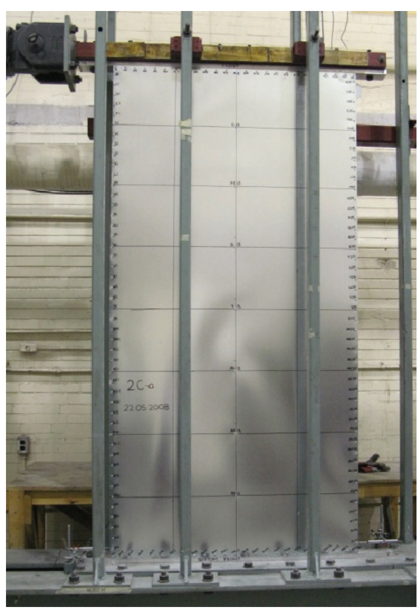


Figure 2.17 Wall Specimen before Shear Buckling and Tension Field Action

Figure 2.18 Shear Buckling and Tension Field of Sheathing in a Monotonic Test

Figure 2.19 Shear Buckling and Tension Field of Sheathing in a Reversed Cyclic Test

2.5.3 Framing

2.5.3.1 Buckling and Distortion of Framing Studs

During testing, the chord studs were observed to twist (Figure 2.20). This deformation was generally temporary in nature, however it was considered to be detrimental to the overall shear resistance and stiffness of the wall. There are two factors for this observation; firstly, the lateral load is applied at the geometric centre of the wall which does not coincide with the centre of gravity since the walls are not symmetrical. The asymmetry of the wall is due to the fact that sheathing is placed on one side of the specimen which leads to bending effects about the loading axis observed in the form of twisting of the chord studs. The second factor is the tension field action that takes place. The tension force has two components; vertical and horizontal. The vertical force is transmitted through the compression chord stud to the rigid testing frame or to the tension chord stud and to the test frame through the hold-down. The horizontal force component, however, imposes a lateral force on the chord studs in the form of twisting (torsion). The field stud showed minor bending which was attributed to the normal force caused by the sheathing tension field on one side of the wall. The screws connected to the middle stud also transmitted some of the horizontal force component which caused some local buckling.

Figure 2.20 Twisting and Local Buckling of Chord Stud

Complementary to the test program, a few walls were constructed with bridging in an attempt to minimize twisting deformations in the chord studs. The bridging stiffened the wall specimens which showed an increase in shear resistance due to a reduction in the degree of chord stud twisting. The small channel bridging members proved to be inadequate to fully support the chord studs. Even though the bridging provided for additional shear resistance, the bridging members themselves were too slender and suffered from lateral-torsional buckling failure under bending (Figure 2.21).

Figure 2.21 Flexural Buckling of Bridging in Test 9M-c

2.5.3.2 Deformation and Uplift of Tracks

The deformation of tracks was rare and usually occurred where the tension field action was highly developed. There was uplift in the track around the shear anchors as the uplift motion from the chord stud was resisted which is attributed to tension field action. The vertical component of the tension field that is developed within the sheathing panel is transmitted to the chord studs and in part through the track which results in uplift and bending (Figure 2.22).

Figure 2.22 Uplift of Bottom Track

2.5.4 Failure Modes of Short Walls

Short walls which measured 610x2440mm (2’x8’) have a high aspect ratio of 4:1. Due to their geometry, the walls were too slender which resulted in high rotations. Minimal damage was observed in the short walls because their flexible nature did not impose significant force demand on the sheathing or its connections. There was some local buckling in the chord studs that was observed during the test but diminished when the wall returned to its original position. Only a few fasteners failed at the corners where the tension field developed the most.

2.5.5 Failure Modes of Long Walls

The 2440x2440mm (8’x8’) walls consisted of two sheathing panels side by side. The perimeter connections of each panel at midspan of the wall were fastened to a single middle field stud. The tension field action was observed in both sheathing panels where it spanned across each panel independently (Figure 2.23). The middle stud was not affected by the loading as it behaved as both a tension and compression member and the forces transmitted through this stud are counteracted by one another.

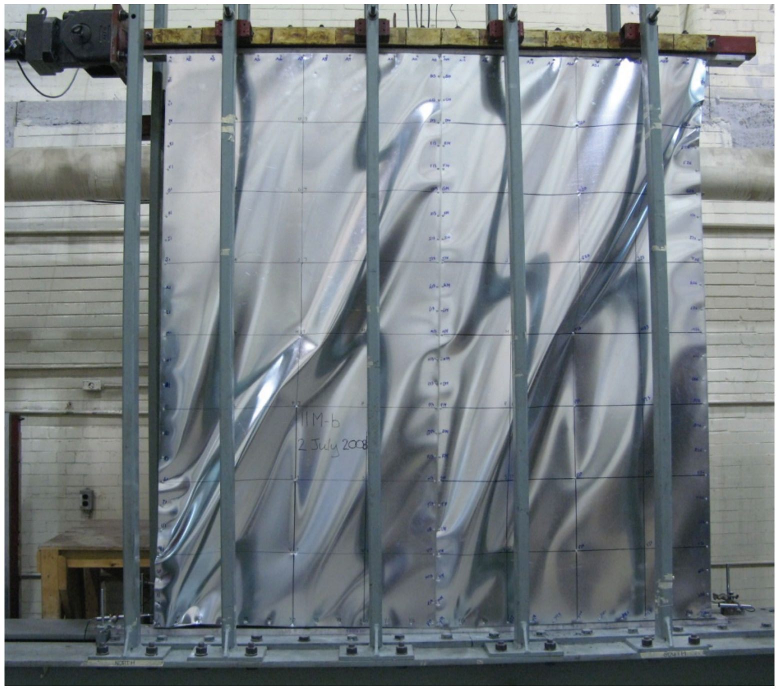


Figure 2.23 Tension Field of a Monotonic Long Wall

2.6 Data Reduction

2.6.1 Lateral Displacement

The net lateral displacement was taken as the total measured wall top displacement, Δ_{top}, (Equation (2-1)). In addition, the rotation of the wall is given by Equation (2-2):

$$\Delta_{net} = \Delta_{top} \tag{2-1}$$

$$\theta_{net} = \frac{\Delta_{top}}{H} \tag{2-2}$$

where,

θ_{net} = Net rotation of wall (radians)

Δ_{net} = Net lateral displacement (mm)

Δ_{top} = Top wall lateral displacement as measured (mm)

H = Height of wall (mm)

2.6.2 Energy Dissipation

It was also necessary to calculate the energy dissipated by the wall under loading. Graphically, energy is idealized as the area below the resistance-displacement curve (Figure 2.24).

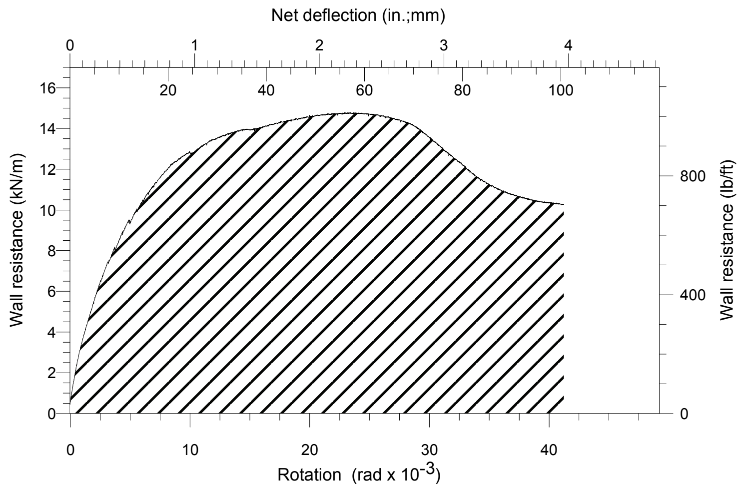


Figure 2.24 Energy as Area Below Resistance-Displacement Curve

The area was calculated using an incremental approach following Equation (2-3):

$$E_i = \frac{F_i + F_{i-1}}{2} \times (\Delta_{top,i} - \Delta_{top,i-1}) \qquad (2\text{-}3)$$

where,

E_i = Energy between two consecutive points

F_i = Corrected shear force between two consecutive data points

$\Delta_{top,i}$ = Measured wall top displacement

The cumulative energy dissipation, E_{total}, can be calculated by the summation of each increment of energy as defined by Equation (2-4):

$$E_{total} = \sum E_i \qquad (2\text{-}4)$$

2.7 Test Results

The summarized results obtained from all monotonic and reversed cyclic tests are listed in Tables 2.3, 2.4 and 2.5 and are graphically presented in Figures 2.25 and 2.26. For monotonic tests, the results include maximum wall resistance, S_u, wall resistance at 40% of S_u, $0.4S_u$, and wall resistance at 80% of S_u, $0.8S_u$, as well as their corresponding displacements $\Delta_{net,u}$, $\Delta_{net,0.4u}$, and $\Delta_{net,0.8u}$, respectively. In addition, the rotation at S_u, θ_u, rotation at 40% of S_u, $\theta_{0.4u}$, rotation at 80% of S_u, $\theta_{0.8u}$, and the total energy dissipated, E, by each test specimen are listed. For reversed cyclic tests, the results include maximum wall resistances for the positive and negative cycles, $S_u'_+$ and $S_u'_-$, wall resistance at 40% of S_u, $0.4S_u'_+$ and $0.4S_u'_-$, and wall resistance at 80% of S_u, $0.8S_u'_+$ and $0.8S_u'_-$, as well as their corresponding displacements, $\Delta_{net,u+}$, $\Delta_{net,u-}$, $\Delta_{net,0.4u+}$, $\Delta_{net,0.4u-}$, $\Delta_{net,0.8u+}$, and $\Delta_{net,0.8u-}$, respectively. The corresponding rotations, θ_{u+}, θ_{u-}, $\theta_{0.4u+}$, $\theta_{0.4u-}$, $\theta_{0.8u+}$, and $\theta_{0.8u-}$, respectively and the total energy dissipated, E are also included in the results.

The displacement at 40% peak load point, $\Delta_{net,0.4u}$, represents the common service load level, which the 2005 NBCC defines as 0.2% of the storey height. This is equivalent to a displacement of 4.9mm (0.192") since all the specimens were 2440mm (8') in height. The drift limit of 0.2% is a serviceability criterion to guarantee functionality of non-structural elements within a structure. The walls displayed a drift less than 4.9mm at $0.4S_u$ except for the 610mm (2') long walls. The displacement at 80% peak load, post-ultimate, $\Delta_{net,0.8u}$,is defined as the maximum usable displacement, or displacement at failure. The maximum inelastic drift limit defined in the 2005 NBCC is 2.5% which is equivalent to 61mm (2.4").

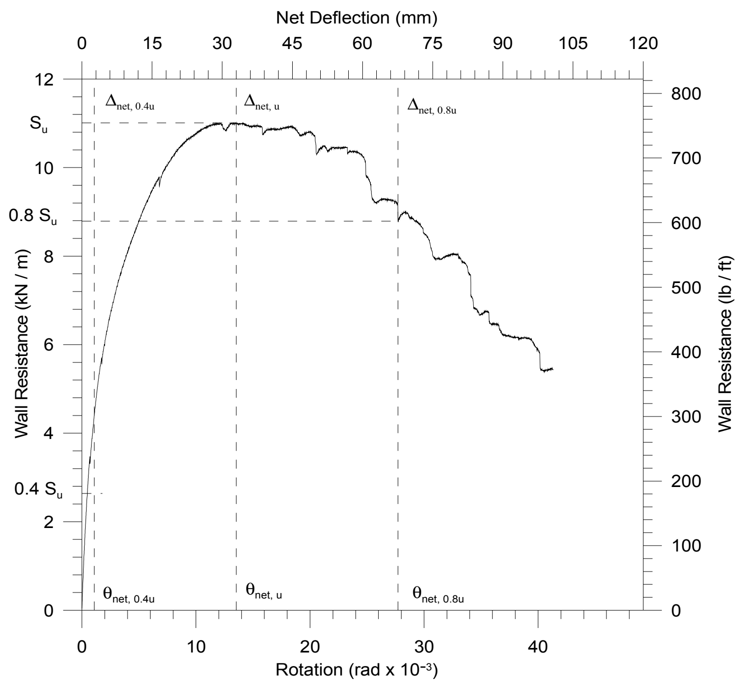


Figure 2.25 Parameters of Monotonic Tests *(Ong-Tone, 2009)*

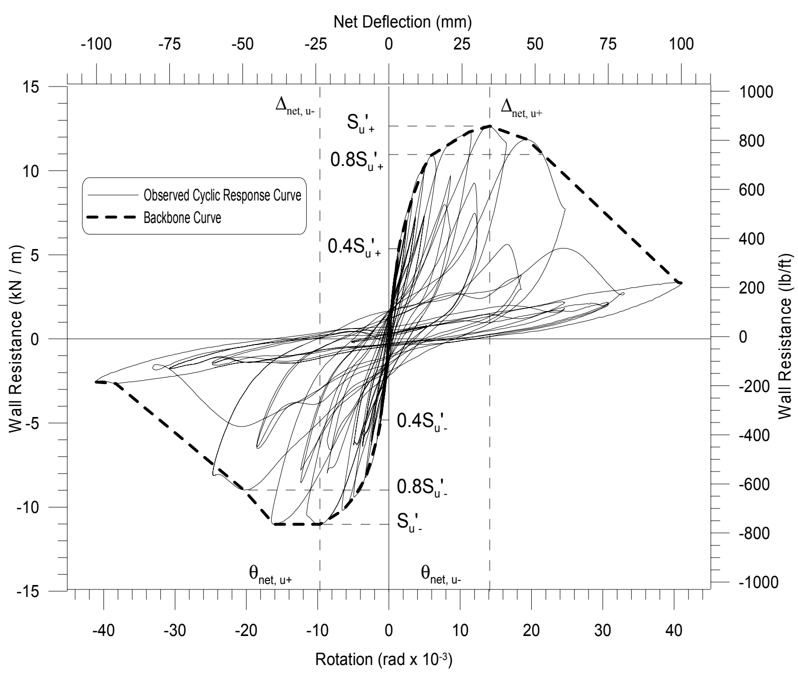


Figure 2.26 Parameters of Reversed Cyclic Tests *(Ong-Tone, 2009)*

Table 2.3 Test Data Summary – Monotonic Tests

Test Specimen	Maximum Wall Resistance S_u (kN/m)	Displacement at S_u $\Delta_{net,u}$ (mm)	Displacement at $0.4S_u$ $\Delta_{net, 0.4u}$ (mm)	Displacement at $0.8S_u$ $\Delta_{net, 0.8u}$ (mm)	Rotation at S_u $\theta_{net,u}$ (rad)	Rotation at $0.4S_u$ $\theta_{net,0.4u}$ (rad)	Rotation at $0.8S_u$ $\theta_{net,0.8u}$ (rad)	Energy Dissipation, E (Joules)
1M-a	6.50	33.13	3.30	72.99	0.01359	0.00135	0.02993	631
1M-b	6.63	26.34	2.81	37.02	0.01080	0.00115	0.01518	411
1M-c	6.41	19.69	2.04	35.73	0.00808	0.00084	0.01465	581
2M-a	10.10	31.54	4.46	90.42	0.01294	0.00183	0.03708	1047
2M-b	9.81	64.24	3.52	100.00	0.02635	0.00144	0.04101	1305
3M-a	5.44	39.48	2.84	57.56	0.01619	0.00116	0.02361	523
3M-b	5.58	31.72	3.16	60.23	0.01301	0.00130	0.02470	527
8M-a	12.66	59.01	5.56	100.00	0.02420	0.00228	0.04101	748
8M-b	13.02	65.32	4.97	100.00	0.02679	0.00204	0.04101	792
9M-a	14.67	53.17	6.68	75.85	0.02181	0.00274	0.03111	694
9M-b	14.78	55.88	5.41	81.84	0.02292	0.00222	0.03356	742
9M-c	18.31	88.53	7.28	100.00	0.03631	0.00299	0.04101	1120
10M-a	10.53	44.18	4.20	100.00	0.01812	0.00172	0.04101	638
11M-a	15.25	28.66	2.97	55.26	0.01175	0.00122	0.02266	2547
11M-b	15.41	25.84	3.68	50.96	0.01060	0.00151	0.02090	2708
17M-a	8.20	25.34	3.13	39.69	0.01039	0.00128	0.01628	355
17M-b	7.30	22.49	5.47	30.65	0.00922	0.00224	0.01257	283
18M-a	9.15	33.21	3.18	64.27	0.01362	0.00130	0.02636	770

Table 2.4 Test Data Summary – Positive Cycles Reversed Cyclic Tests

Test Specimen	Maximum Wall Resistance $S_u'_+$ (kN/m)	Displacement at $S_u'_+$, $\Delta_{net,u+}$ (mm)	Displacement at $0.4S_u'_+$, $\Delta_{net,\ 0.4u+}$ (mm)	Displacement at $0.8S_u'_+$, $\Delta_{net,\ 0.8u+}$ (mm)	Rotation at $S_u'_+$, $\theta_{net,u+}$ (rad)	Rotation at $0.4S_u'_+$, $\theta_{net,0.4u+}$ (rad)	Rotation at $0.8S_u'_+$, $\theta_{net,0.8u+}$ (rad)	Energy Dissipation, E (Joules)
1C-a	6.09	34.55	2.70	51.40	0.01417	0.00111	0.02108	2554
1C-b	6.37	19.34	3.10	40.20	0.00793	0.00127	0.01649	2418
2C-a	11.11	29.00	4.40	81.20	0.01189	0.00180	0.03330	5807
2C-b	10.76	29.52	4.20	95.90	0.01211	0.00172	0.03933	6098
3C-a	6.04	50.34	3.30	68.60	0.02064	0.00135	0.02813	2934
3C-c	5.91	28.98	2.60	55.30	0.01188	0.00107	0.02268	2805
8C-a	13.78	76.27	6.00	90.70	0.03128	0.00246	0.03720	3468
8C-b	13.68	71.92	5.30	89.90	0.02949	0.00217	0.03687	3960
9C-a	16.17	55.20	8.10	99.40	0.02264	0.00332	0.04076	5480
9C-b	16.04	57.03	7.80	99.90	0.02339	0.00320	0.04097	4857
11C-a	16.12	26.04	3.20	52.00	0.01068	0.00131	0.02133	18912
11C-b	16.19	27.81	2.70	48.90	0.01141	0.00111	0.02005	21268

Table 2.5 Test Data Summary – Negative Cycles Reversed Cyclic Tests

Test Specimen	Maximum Wall Resistance S_u'- (kN/m)	Displacement at S_u'-, $\Delta_{net,u-}$ (mm)	Displacement at 0.4S_u'-, $\Delta_{net,\ 0.4u-}$ (mm)	Displacement at 0.8S_u'-, $\Delta_{net,\ 0.8u-}$ (mm)	Rotation at S_u'-, $\theta_{net,u-}$ (rad)	Rotation at 0.4S_u'-, $\theta_{net,0.4u-}$ (rad)	Rotation at 0.8S_u'-, $\theta_{net,0.8u-}$ (rad)	Energy Dissipation, E (Joules)
1C-a	-6.55	-22.59	-3.10	-40.20	-0.00926	-0.00127	-0.01649	2554
1C-b	-6.11	-19.70	-2.90	-34.60	-0.00808	-0.00119	-0.01419	2418
2C-a	-10.76	-28.21	-4.00	-84.80	-0.01157	-0.00164	-0.03478	5807
2C-b	-10.65	-38.57	-3.80	-87.90	-0.01582	-0.00156	-0.03605	6098
3C-a	-5.49	-43.71	-3.10	-56.90	-0.01793	-0.00127	-0.02333	2934
3C-c	-6.27	-19.35	-3.60	-44.30	-0.00794	-0.00148	-0.01817	2805
8C-a	-13.94	-76.25	-5.40	-87.90	-0.03127	-0.00221	-0.03605	3468
8C-b	-12.98	-53.63	-6.10	-100.00	-0.02199	-0.00250	-0.04101	3960
9C-a	-15.67	-77.89	-9.20	-100.00	-0.03194	-0.00377	-0.04101	5480
9C-b	-15.42	-55.31	-5.90	-100.00	-0.02268	-0.00242	-0.04101	4857
11C-a	-16.17	-29.35	-3.70	-60.10	-0.01204	-0.00152	-0.02465	18912
11C-b	-15.80	-26.97	-2.90	-49.30	-0.01106	-0.00119	-0.02022	21268

2.8 Comparison of Shear Walls

The test results were examined to determine the effects of each detailing factor such as screw spacing, length, sheathing and framing thickness, and the use of bridging. The walls tested at McGill University were compared with each other and with the results of the US data *(Serrette (1997), Yu et al. (2007))*. To expand the comparison of observations, and to include all tests within the test program, Ong-Tone *(2009)* provided comparisons of some configurations and compared the effects of various reinforcement details.

2.8.1 Comparison of Shear Wall Configurations

The test specimens for each wall configuration performed similarly and provided similar results. The monotonic and cyclic behaviour were similar and a summary of all measured results can be found in Appendix C. The positive cycles of a reversed cyclic test performed better than the negative cycles in terms of capacity because the wall was first displaced in the positive direction. The wall's ability to carry shear is decreased as it becomes damaged when it is pushed into the inelastic cycles in the positive direction.

2.8.1.1 Effect of Screw Spacing

A smaller fastener spacing resulted in higher shear resistance as in Tests 2M-a,b,c with a spacing of 50mm (2"). Tests 1M-a,b,c had a spacing of 150mm (6") and displayed lower strengths as illustrated in Figure 2.27. A spacing of 75mm (3") was also evaluated with wall 18M-a, which performed as expected providing an intermediate shear capacity. Figure 2.27 illustrates the results of all the test specimens with 0.46mm (0.018") sheathing and 1.09mm (0.043") framing. Configuration 17 was designed to determine the effects of varying fastener spacing along the edge of the sheathing where screws were closer in spacing at each corner and the spacing was lengthened progressively. It was observed in tests 1M-a,b and 2M-a,b that the tension field mostly occurred from corner to corner of the wall specimen and that the fasteners at mid-height were virtually undamaged. Therefore, a panel perimeter spacing of 50mm (2") was used in the corners and progressively increased to 300mm (12”) at mid-height with the same number of fasteners as Configuration 1 (See Appendix A). Even though Tests 17M-a,b resulted in higher resistances than Tests 1M-a,b,c, they did not exhibit ductile behaviour that was observed in the other tests which indicates that the placement of fasteners affects the performance and stiffness of shear walls since the fasteners are not uniformly spaced. The corner spacing in Tests 17M-a,b was 50mm (2”) but the shear walls did not reach similar resistances to that of Tests 2M-a,b which had a 50mm (2”) fastener spacing all around the edge which indicates that all screws are necessary for load resistance.

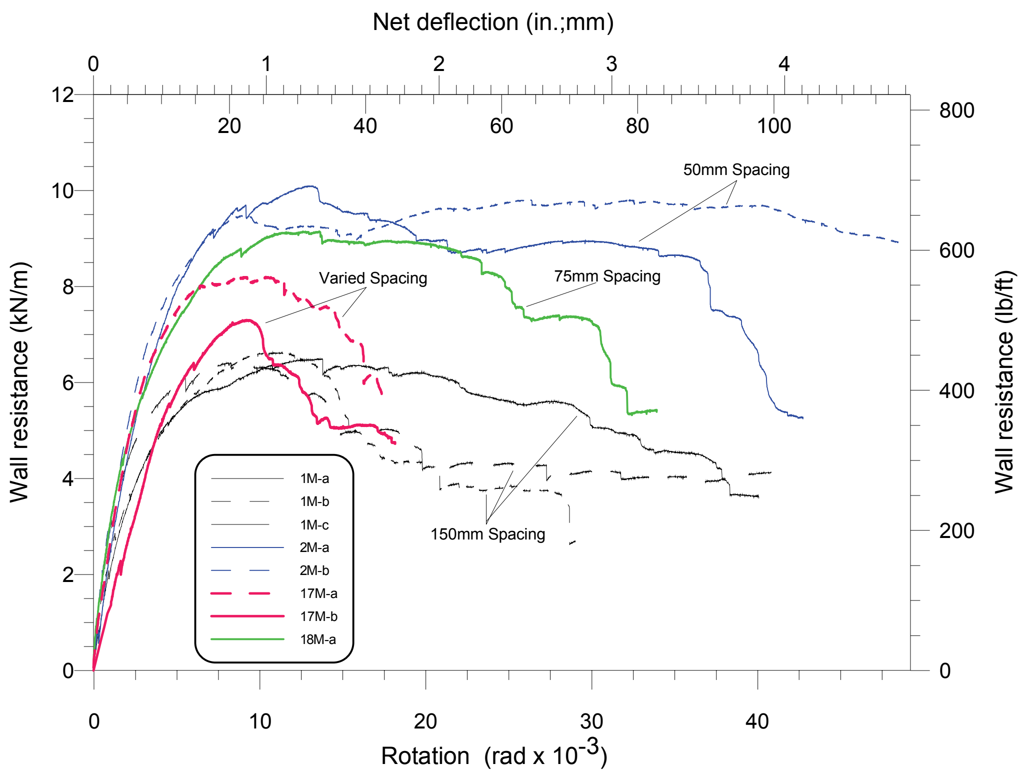


Figure 2.27 Comparison of Fastener Spacing: Wall Resistance vs. Displacement of Tests 1M-a,b,c, Tests 2M-a,b, Tests 17M-a,b and Test 18M-a

A similar observation can be drawn with respect to test specimens with 0.76mm (0.030”) sheathing and 1.09mm (0.043”) framing (Figure 2.28). Tests 8M-a,b had a screw spacing of 100mm (4”) and did not perform as well as Tests 9M-a,b that had a screw spacing of 50mm (2”).

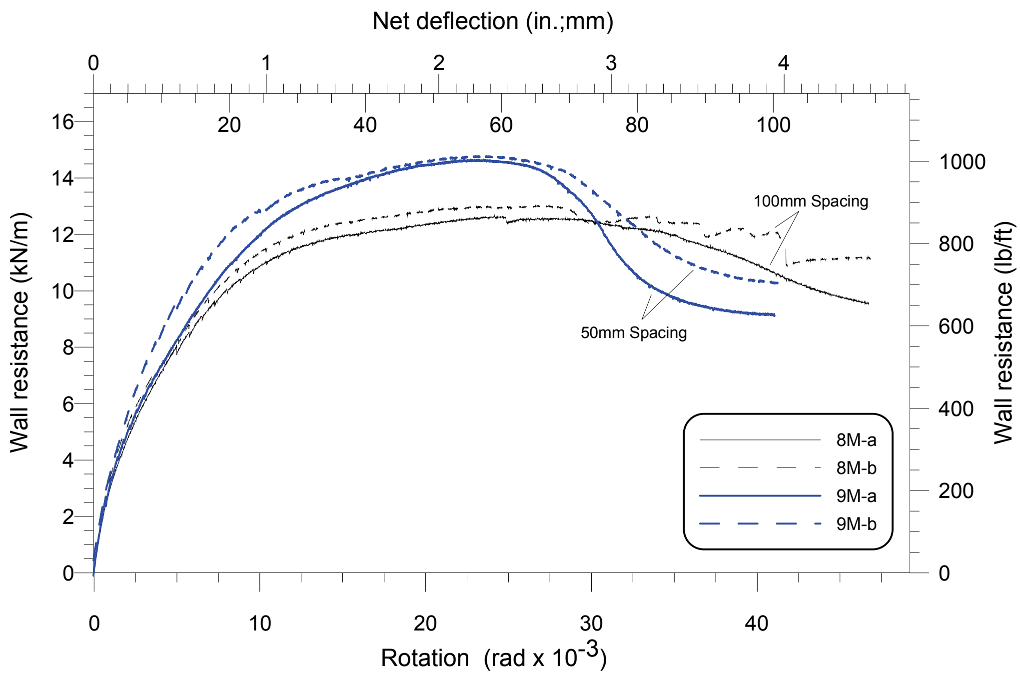


Figure 2.28 Comparison of Fastener Spacing: Wall Resistance vs. Displacement of Tests 8M-a,b and Tests 9M-a,b

2.8.1.2 Effect of Wall Length

Figure 2.29 compares Tests 5M-a,b, Tests 8M-a,b, Tests 11M-a,b and Test 12M-a which were constructed using the same specifications of 100mm (4”) fastener spacing, 1.09mm (0.043”) framing thickness, and 0.76mm (0.030”) sheathing. The only variation is the length of the specimens where Configuration 8 is 610mm (2’) in length, Configuration 5 is 1220mm (4’) in length, Configuration 12 is 1630mm (6’) in length and Configuration 11 is 2440mm (8’) in length. It was initially assumed that the wall length would not affect the shear resistance (normalized to length) of the specimens but, contrary to expectation, the longer walls exhibited higher capacities. It was expected that the 610mm (2’) long walls would not perform as well as the longer walls due to their high aspect ratio rendering them too slender. The short walls rotated when pushed laterally which did not allow for the development of strength. The longer walls were able to reach similar resistance levels because their rotation was limited.

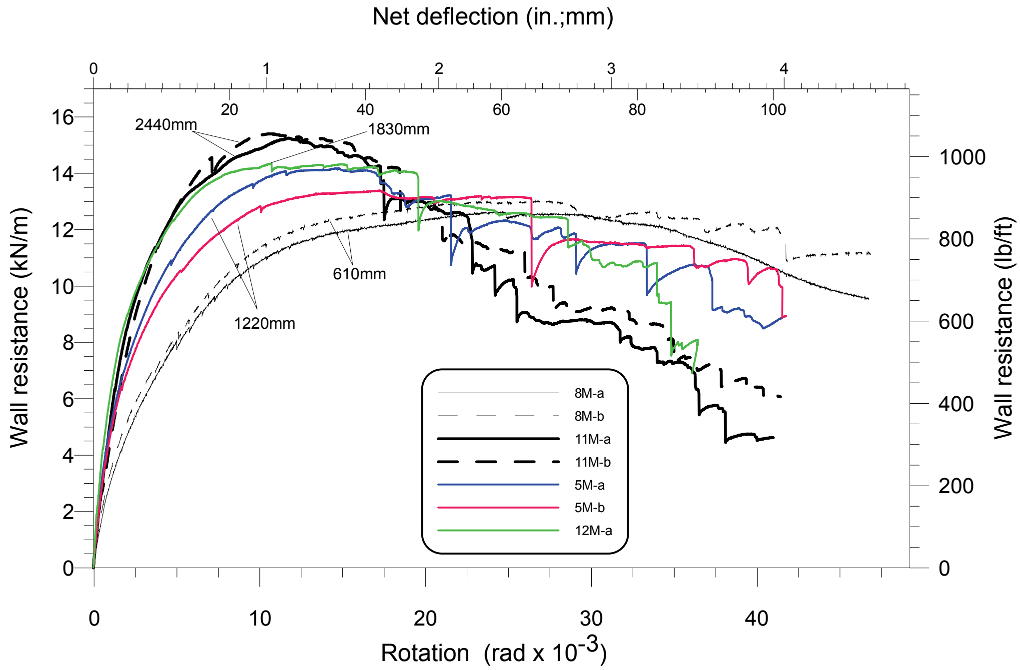


Figure 2.29 Comparison of Wall Lengths: Wall Resistance vs. Displacement of Tests 8M-a,b, Tests 5M-a,b, Tests 11M-a,b, and Test 12M-a

2.8.1.3 Effect of Framing Thickness

As part of the test program, the effect of framing thickness was examined. The variation of framing thickness was examined with sheathing thickness of 0.46mm (0.018”) and 0.76mm (0.030”). Figure 2.30 presents the results of the use of thinner 0.46mm (0.018”) sheathing with 1.09mm (0.043”) framing in Tests 1M-a,b,c and with 0.84mm (0.033”) framing in Tests 3M-a,b. Figure 2.31 presents the results of the use of 0.76mm(0.030”) sheathing with 1.09mm (0.043”) framing in Tests 8M-a,b and with 0.84mm (0.033”) framing in Test 10M-a. In both graphs, a decrease in capacity of approximately 15% was observed with the thinner 0.84mm (0.033”) framing. When the thickness of the framing and sheathing were close in value, the measured response was affected.

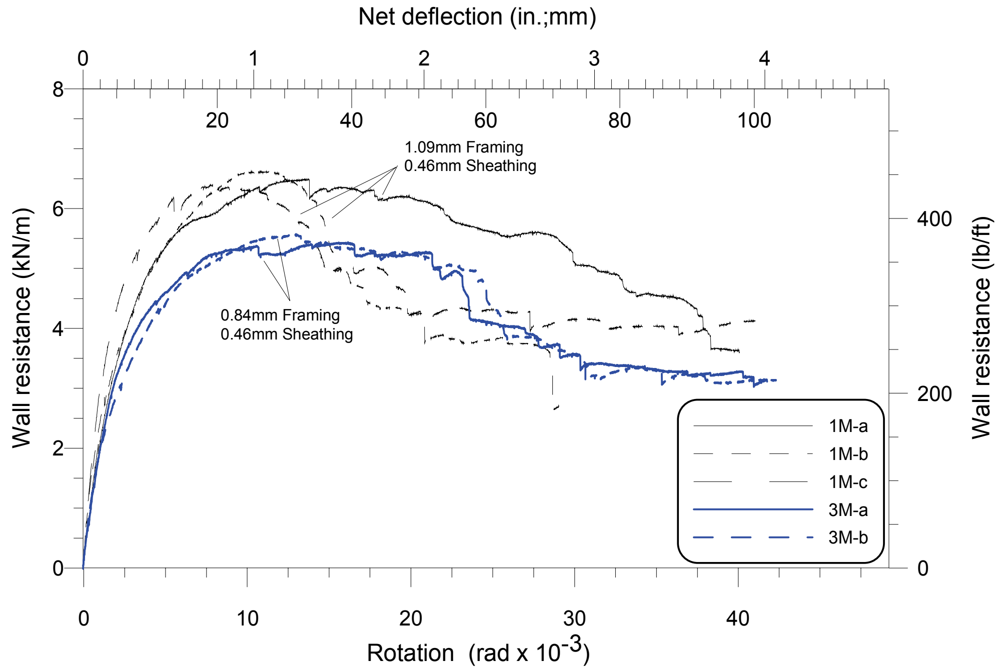


Figure 2.30 Comparison of Framing Thickness: Wall Resistance vs. Displacement of Tests 1M-a,b,c and Tests 3M-a,b

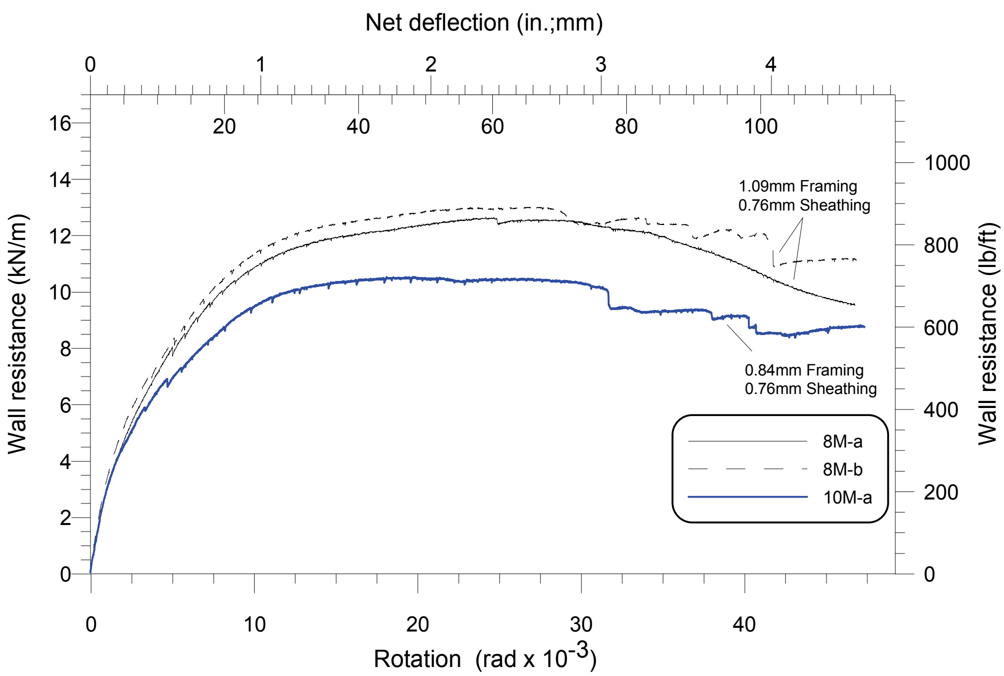


Figure 2.31 Comparison of Framing Thickness: Wall Resistance vs. Displacement of Tests 8M-a,b and Test 10M-a

2.8.1.4 Effect of Sheathing Thickness

As expected, an increase in shear resistance was observed when a thicker sheathing was used. Figure 2.32 illustrates the results of the use of 0.46mm (0.018”) sheathing in Tests 2M-a,b and 0.76mm (0.030”) sheathing in Tests 6M-a,b. Both configurations were constructed using 1.09mm (0.043”) framing thickness and 50mm fastener spacing and were 1220mm (4’) in length. The use of thicker sheathing significantly increased the capacity since the individual sheathing connection resistance was higher.

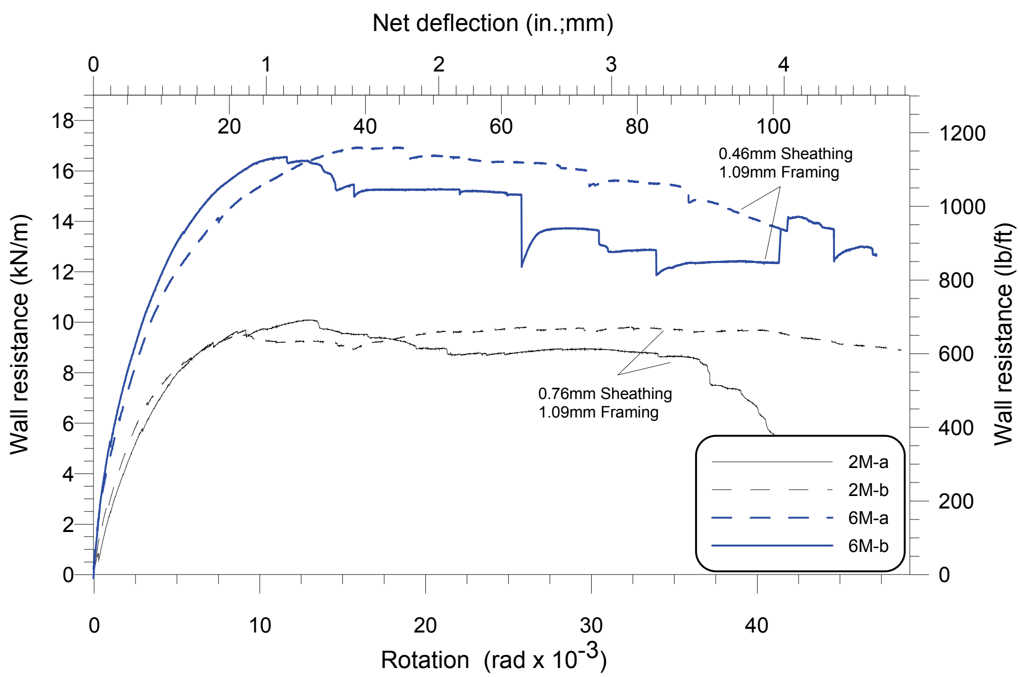


Figure 2.32 Comparison of Sheathing Thickness: Wall Resistance vs. Displacement of Tests 2M-a,b and Tests 6M-a,b

2.8.1.5 Effect of Bridging

The use of bridging was examined in Test 9M-c and compared with 9M-a and 9M-b which were all constructed using 1.09mm (0.043”) framing, 0.76 (0.030”) sheathing, and were 610x2440mm (2’x8’) in size (Figure 2.33). Ong-Tone *(2009)* also examined the effects of bridging in Configuration 5 (1.09mm (0.043”) framing, 0.76mm (0.030”) sheathing, 100mm (4”) fastener spacing, 1220x2440mm (4’x8’) in size) and Configuration 6 (1.09mm (0.043”) framing, 0.76mm (0.030”) sheathing, 50mm (2”) fastener spacing, 1220x2440mm (4’x8’) in size) (Figures 2.34 and 2.35). Three rows of bridging were installed to minimize twisting of the chord studs. It was observed that the bridging was successful at reducing damage in the chord studs which led to an increase in shear resistance. The corner fasteners, which contribute to tension field action, were able to participate more effectively in resisting the applied loads.

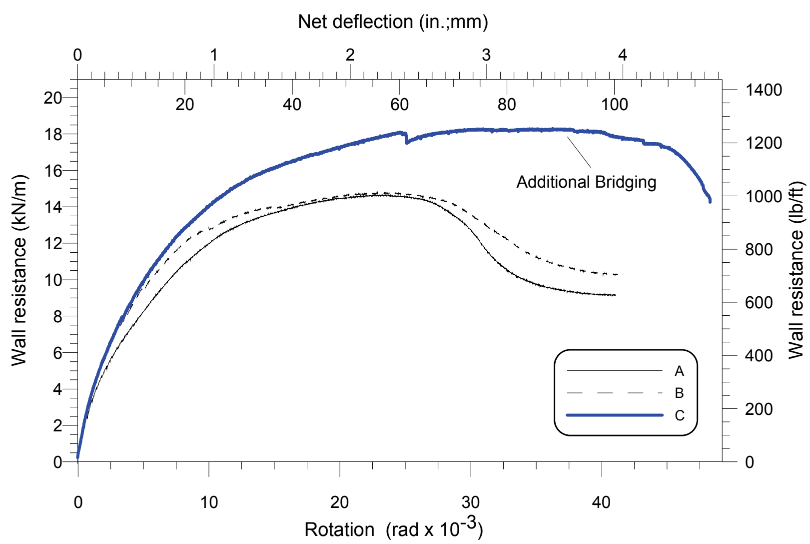


Figure 2.33 Comparison of Reinforcement: Wall Resistance vs. Displacement of Tests 9M-a,b,c

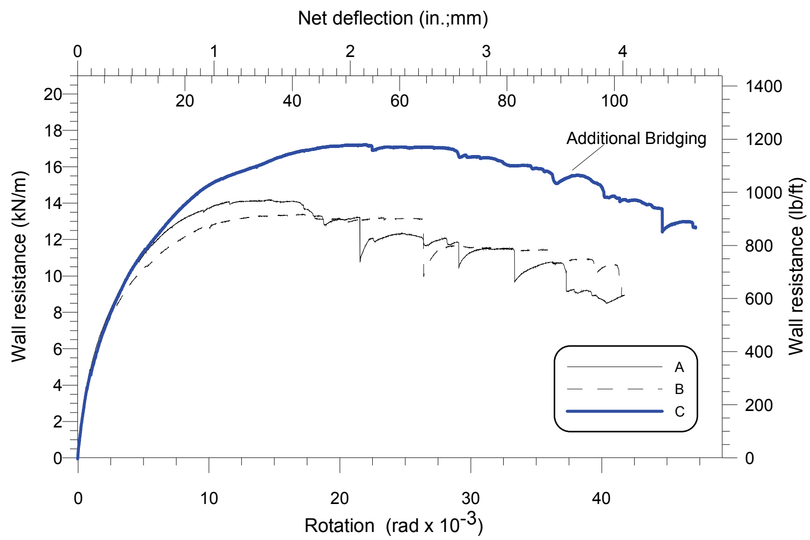


Figure 2.34 Comparison of Reinforcement: Wall Resistance vs. Displacement of Tests 5M-a,b,c

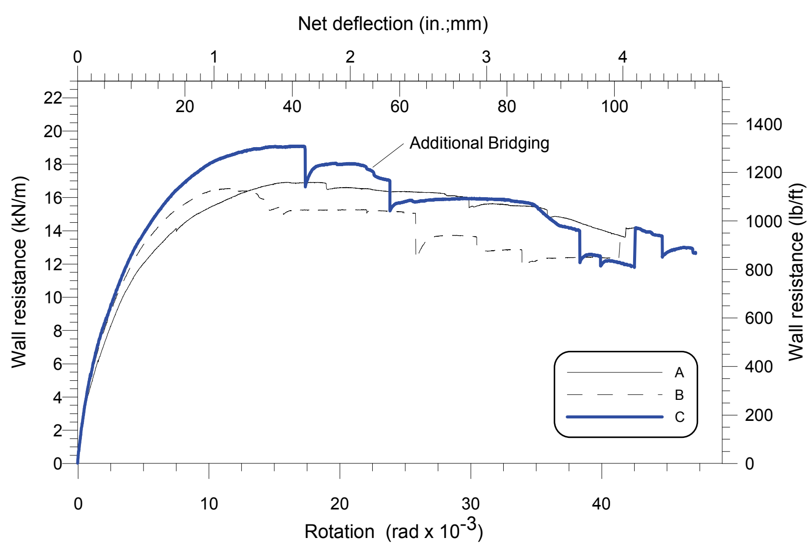


Figure 2.35 Comparison of Reinforcement: Wall Resistance vs. Displacement of Tests 6M-a,b,c

2.8.2 Comparison with US Shear Walls

Initially, the test program was to consist of test walls with 0.68mm (0.027") sheathing to compare with the tests by Serrette *(1997)* but the thickness of 0.68mm (0.027") was found to be unavailable in the market and, therefore, the test program proceeded with 0.76mm (0.030") sheathing. Table 2.6 contains a comparison of these test specimens; all walls are 610x2440mm (2'x8') in size. Even though wall 10M-a was constructed with a sheathing thickness of 0.76mm (0.030"), it had a lower ultimate shear resistance than Serrette's *(1997)* AISI 13,14 and AISI F1,F2 (Table 2.6) which had a nominal sheathing thickness of 0.68mm (0.027"). A possible explanation for the discrepancy is that the materials used for Serrette's tests were thicker than the nominal values listed. The measured base metal thickness of the sheathing for the McGill walls was 0.76 mm (see Section 2.9).

Table 2.6 Average Ultimate Shear Resistances and Displacements of Configuration 10 and AISI-13,14, F1,F2

Test Specimen	Average S_u (kN/m)	Average Displacement at S_u (mm)	Nominal Sheathing Thickness (mm)	Protocol
10M-a	10.53	44.18	0.76	Monotonic
AISI 13,14	14.45	51.55	0.68	Monotonic
AISI F1, F2	14.71	45.72	0.68	SPD Cyclic

Serrette *(1997)* also tested shear walls with light framing of 0.84mm (0.033") and 0.46mm (0.018") sheathing with a fastener spacing of 150mm (6"). Two of the walls were 610x2440mm (2'x8') in size, AISI 11,12, and two other walls were 1220mmx2440mm (4'x8') in size, AISI 15,16; all these walls were tested monotonically. Two 1220x2440mm (4'x8') walls were tested using the SPD reversed cyclic protocol (AISI D1, D2). All six specimens were similar to the Configuration 3 shear walls tested at McGill University. Once again, the tests by Serrette had higher ultimate shear resistances compared with the tests of Configuration 3 (Table 2.7). The difference in strength is probably due to a sheathing that was thicker than the nominal value. The measured base metal thickness of the sheathing for the McGill walls was 0.46 mm (see Section 2.9).

The displacements of the tests were comparable except for the shorter walls. The larger displacements at peak load of walls AISI 11,12 were likely a result of the greater flexibility of these high aspect ratio walls.

Table 2.7 Average Ultimate Shear Resistances and Displacements of Configuration 3 and AISI-11,12,15,16, D1,D2

Test Specimen	Average S_u (kN/m)	Average Displacement at S_u (mm)	Length (mm)
3M-a,b	5.51	35.60	1220
3C-a,c	5.93	35.58	1220
AISI 11,12	7.17	51.82	610
AISI 15,16	7.05	32.97	1220
AISI D1,D2	5.72	25.40	1220

Configuration 11 was similar to the Y5 tests by Yu *et al. (2007)*. The shear walls had a framing thickness of 1.09mm (0.043”) and a sheathing thickness of 0.76mm (0.030”) with a fastener spacing of 100mm (4”). Configuration 11 measured 2440x2440mm (8’x8’) in size whereas Y5 tests measured 1220x2440mm (4’x8’) in size. Configurations 5, 12, and 15 by Ong-Tone *(2009)* had the same specifications as Configuration 11 except Configuration 5 was 1220mm (4’) in length, Configuration 12 was 1830mm (6’) in length, and Configuration 15 was 1220mm (4’) in length with raised hold-downs. Configuration 11 tests resulted in slightly higher ultimate shear resistance because as mentioned, the wall length had an effect on the performance of the shear walls. The other configurations had similar ultimate resistances but the corresponding displacements were smaller than tests Y5. It was found that the sheathing thickness used by Yu *et al. (2007)* was actually 0.73mm (0.0286”) which is thinner than the nominal value. Also of note, the size of the anchors used by Yu *et al.* for the hold-downs was 12.7mm (1/2”), whereas 22.2mm (7/8”) threaded rods were used for the McGill tests and by Serrette; this may have contributed to the larger displacements at S_u. A comparison of ultimate shear resistance and displacements for the different configurations are given in Table 2.8. It should be noted that the values listed in Table 2.8 for Y5 tests are obtained from Yu *et al. (2007)* values and not from the analysis of this data by Velchev *(2009).*

Table 2.8 Average Ultimate Shear Resistances and Displacements of Configuration 11 and Y5 Tests

Test Specimen	Average S_u (kN/m)	Average Displacement at S_u (mm)	Length (mm)
11M-a,b	15.33	27.25	2440
11C-a,c	16.07	27.54	2440
5M-a,b	13.79	39.08	1220
5C-a,b	14.34	30.08	1220
12M-a	14.35	26.09	1830
15M-a[1]	13.79	35.93	1220
Y5M1,M2	13.99	66.5	1220
Y5C1,C2	14.80	51.05	1220

[1] Raised hold-downs

2.9 Ancillary Testing of Materials

Coupons from the framing and sheathing materials were tested to confirm thickness and mechanical properties. Members of a particular thickness were all obtained from the same coil, Grade 230MPa (33ksi) as specified by ASTM A653 *(2008)*. Three samples were tested for each thickness (two stud/track thicknesses of 0.84 (0.033”) and 1.09mm (0.043”), and two sheathing thicknesses of 0.46 (0.018”) and 0.76mm (0.030”)). Coupons were tested according to ASTM A370 *(2006)* requirements. The coupons were tested under tension loading at a cross-head movement rate of 0.5mm/min within the elastic range and then increased to 4mm/min past the yield point. A 50mm (2”) extensometer was attached to each coupon to measure elongation.

After the completion of the tensile coupon tests, the zinc coating was removed using 25% hydrochloric acid solution to measure the true thickness of the specimens in order to calculate material properties. It was found that the coating thickness is negligible compared to the base metal thickness and, therefore, the capacity was not affected by the coating.

The measured base metal thickness of the framing was greater than that specified by the manufacturer. In addition, a higher yield stress was measured in comparison to the minimum specified. The coupons exhibited the typical stress-strain relationship of steel; linear within the elastic range, with a plateau past yielding followed by strain hardening before ultimate failure. It can be seen that the relationship of F_u/F_y is greater than 1.08 which is the minimum required by CSA-S136 *(2007)* and the observed elongation over a 50mm (2") gauge length is well over the minimum specified of 10%. A summary of the coupon tests is given in Table 2.9.

Table 2.9 Summary of Material Properties

Coupon	Specimen (mm)	Member	Base Metal Thickness (mm)	Yield Stress, F_y (MPa)	Tensile Stress, F_u (MPa)	F_u / F_y	Elongation %
A	0.84	Stud/track	0.87	342	391	1.14	31.0
B	1.09	Stud/track	1.14	346	496	1.43	31.3
C	0.46	Sheathing	0.46	300	395	1.32	26.2
D	0.76	Sheathing	0.76	284	373	1.32	34.9

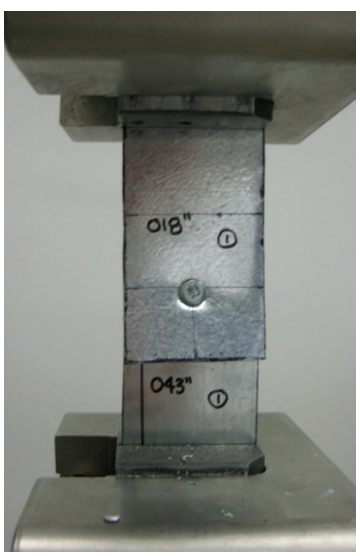
The ratio of measured yield stress to nominal yield stress, R_y, is listed as 1.5 for 230MPa (33ksi) materials in AISI S213 *(2007)*. Similarly, a value of 1.2 is listed for the measured tensile stress to nominal tensile stress ratio, R_t, for 230MPa (33ksi) materials *(AISI S213, 2007)*. The results obtained from the coupon tests had similar values for R_y and R_t as listed in the AISI S213 (Table 2.10) except for the R_y values of the sheathing, which were less than 1.5. As well, the R_t value for the 1.09mm (0.043") thick steel was much higher than 1.2.

Table 2.10 R_t and R_y Values of Studs/Tracks and Sheathing

Member	Thickness (mm)	R_y	R_t
Stud / Track	0.84	1.50	1.26
Stud / Track	1.09	1.50	1.60
Sheathing	0.46	1.31	1.28
Sheathing	0.76	1.23	1.20

2.10 Screw Connection Testing

Connection tests were carried out to determine the shear resistance of the sheathing fasteners. In all test specimens, No. 8x19.1mm (3/4") flat pan head drilling screws (LOX drive) were used (Figure 2.36). The bearing/tilting capacity of the screw connection was determined for the different framing-sheathing variations that were used. Four samples were tested for a framing thickness of 1.09mm (0.043") with sheathing thicknesses of 0.46mm (0.018") and 0.76mm (0.030"), and a framing thickness of 0.84mm (0.033") with a sheathing thickness of 0.46mm (0.018") and 0.76mm (0.030"). The shear capacity of the screws themselves was approximated by testing representative fasteners with 2.46mm (0.097") thick steel plates. A summary of the screw connection tests is provided in Table 2.11. The nominal resistance values were obtained following the procedure outlined in Clause E.4.3.1 of the CSA-S136 *(2007)* for connection shear resistance through bearing and tilting. The nominal resistance values are lower than the average values obtained through lab testing because the CSA-S136 Standard is more conservative since it is applicable for a variety of screw types. A comparison with the manufacturer's data would have been more appropriate but it was unavailable.



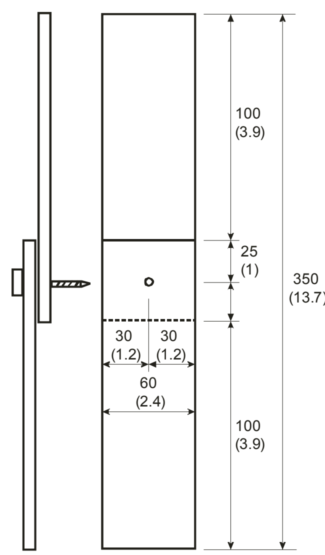


Figure 2.36 Screw Connection Setup and Schematic *(Velchev, 2008)*

Table 2.11 Screw Connection Shear Resistance Summary

Test	Nominal Sheathing Thickness	Nominal Framing Thickness	Maximum Resistance (kN)	Average Resistance (kN)	Nominal Resistance (kN)
Bearing/Tilting Resistance					
1	0.46mm (0.018”)	0.84mm (0.033”)	1.92	2.05	1.56
2			1.94		
3			1.82		
4			2.6		
5			1.98		
1		1.09mm (0.043”)	1.79	2.11	1.56
2			2.29		
3			1.86		
4			2.36		
5			2.25		
1	0.76mm (0.030”)	0.84mm (0.033”)	2.77	2.80	2.27
2			2.65		
3			2.87		
4			2.74		
5			3.00		
1		1.09mm (0.043”)	4.01	4.01	2.43
2			3.94		
3			4.10		
4			4.25		
5			3.73		
Shear Capacity					
1	2.46mm (0.097”)	2.46mm (0.097”)	5.97	5.68	-
2			5.69		
3			5.82		
4			5.16		
5			5.78		

CHAPTER 3 – INTERPRETATION OF TEST RESULTS AND PRESCRIPTIVE DESIGN

3.1 Introduction

The results obtained from testing were highly nonlinear. In order to simplify the test results for designers, Branston *(2004)* found that the Equivalent Energy Elastic Plastic (EEEP) method *(Park, 1989 and Foliente, 1996)* was appropriate for the analysis of shear walls. The EEEP method provided a bilinear elastic-plastic curve that is similar to model behaviour of steel materials. This method is also consistent with the analysis method for wood sheathed shear walls tested at McGill by Branston *et al. (2004)*. Due to the amount of data obtained from testing, an Excel™ Macro program was created to automate the analysis process with minimal manual manipulation. A brief overview of the method is given below and an elaborate explanation of the program procedure is explained in Appendix L. The analysis also provides parameters that will be used in the design procedure.

3.2 EEEP Concept

The EEEP method simplifies test results by means of a bilinear elastic-plastic curve. The basis for this method is the energy dissipated by the test specimen up to 80% of the post-peak load, which is considered to be the ultimate failure. The energy provided by the EEEP must be equal to the energy dissipated in a test. Graphically, the area under the observed (monotonic or backbone) and EEEP curves represents the energy dissipated and is equated with the assumption that A1 and A2 are equal as shown in Figure 3.1.

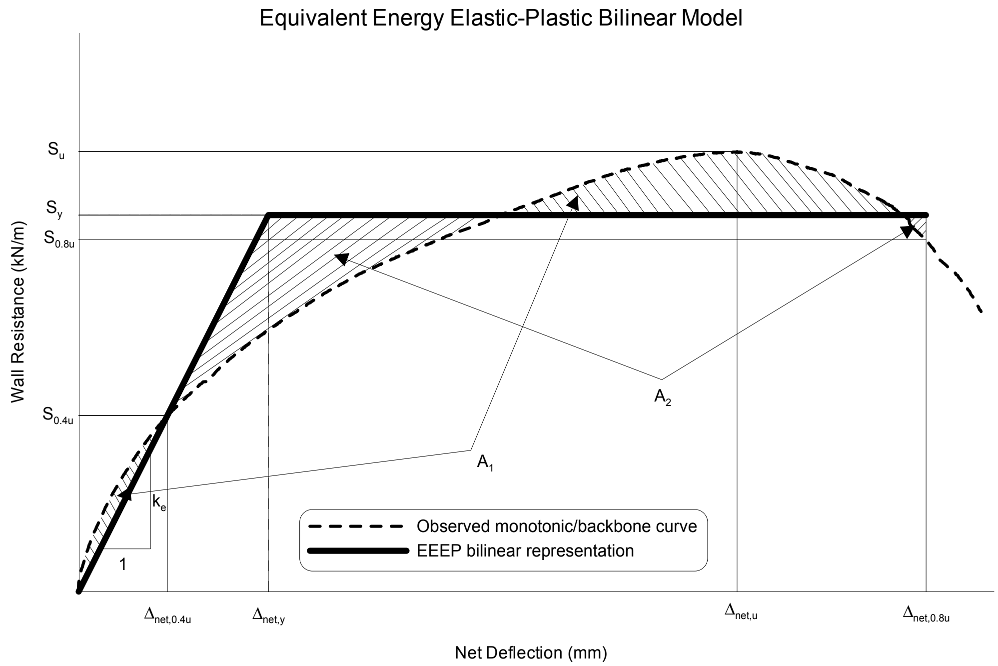


Figure 3.1 EEEP Model ***(Branston, 2004)***

There were three possible outcomes of the EEEP procedure depending on the test results.

a) If the 80% post-peak load was reached at a displacement greater than 100mm (4”), the ultimate displacement was set to 100mm (4”)
b) In some cases, the lateral drift was well beyond 100mm (4”) before a significant decrease in load capacity was observed. If the 80% post-peak load was lower than the last reached load, then displacement at 80% post-peak load was determined as the last reached displacement or 100mm (4”) if the last displacement was greater than 100mm (4”)
c) If neither of the above scenarios occured, then 80% of the post-peak load and the corresponding displacement were located.

Some parameters were required from the test data to obtain the EEEP curve. One of the important points was the yield wall resistance, S_y, from which nominal strengths were determined. The yield wall resistance is the point at which the bilinear curve transforms from elastic to plastic behaviour. The corresponding displacement, $\Delta_{net,y}$, for the yield wall resistance represents the elastic deflection. The end displacement of the EEEP curve was determined as the displacement reached at 80% of ultimate post-peak load, $\Delta_{net,0.8u}$. The elastic stiffness, k_e, was another parameter of significance and was determined using 40% of the ultimate load, $0.4S_u$, which is considered to be within the elastic range of the wall specimen (Equation (3-1)). The yield wall resistance was determined from the elastic stiffness and end displacement as given in Equation 3-2. The corresponding yield displacement, $\Delta_{net,y}$, was then determined from the elastic stiffness and yield displacement as presented in Figure 3-1 and calculated using Equation 3-3. As for the cumulative energy dissipated during a test, it was the area below the resistance-displacement curve. The energy dissipated by a wall specimen was only considered up to 80% of the post-peak load reached. Finally, the ductility, μ, was calculated in order to measure the ductile behaviour of each wall during seismic activity (Equation (3-4)). Ductility is measured by comparing the displacement at 80% post-peak load with the displacement at yield.

$$k_e = \frac{0.4S_u}{\Delta_{net,0.4u}} \tag{3-1}$$

$$S_y = \frac{-\Delta_{net,0.8u} \pm \sqrt{{\Delta_{net,0.8u}}^2 - \frac{2A}{k_e}}}{-\frac{1}{k_e}} \tag{3-2}$$

$$\Delta_{net,y} = \frac{S_y}{k_e} \tag{3-3}$$

$$\mu = \frac{\Delta_{net,0.8u}}{\Delta_{net,y}} \tag{3-4}$$

where,

S_y = Yield wall resistance (kN/m)

S_u = Ultimate wall resistance (kN/m)

A = Area under observed curve up to 80% load ($\Delta_{net,0.8u}$)

k_e = Unit elastic stiffness ((kN/m)/mm)

$\Delta_{net,0.8u}$ = Displacement at $0.8S_u$ (post-peak)

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

μ = ductility of shear wall

An example displaying the EEEP result for a monotonic test is given in Figure 3.2. The procedure for reversed-cyclic test analysis is similar to that of a monotonic but requires some user input. The observed curve for a cyclic test is in the form of hysteretic loops. A backbone curve must be created that embodies the hysteretic curves, which is determined using the maxima of the hysteretic loops of both the positive and negative regions. However, the positive and negative regions should be treated separately as they can be considered to be independent in behaviour. Once a backbone curve was obtained, the curve was analyzed in the same manner as that of the monotonic test with the backbone curve as a simulated nonlinear curve. An example displaying the EEEP result for a reversed-cyclic test for the positive and negative regions is given in Figure 3.3. A summary of EEEP results is provided in Tables 3.1, 3.2 and 3.3 with details of the Macro created for EEEP provided in Appendix L. Table 3.1 provides a summary of the monotonic tests and Table 3.2 provides a summary of the positive cycles of the reversed cyclic tests, and Table 3.3 provides a summary of the negative cycles of the reversed cyclic tests. In these tables, the yield resistance, S_y, and its corresponding displacement, $\Delta_{net,y}$, the elastic stiffness, k_e, ductility, μ, and cumulative energy are given for each test specimen.

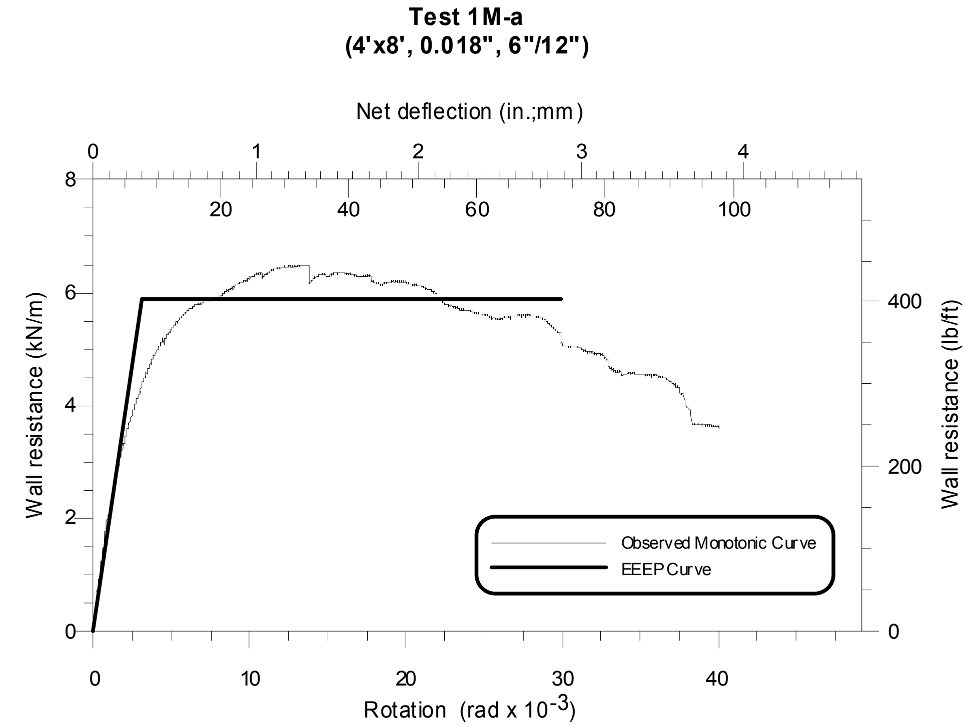


Figure 3.2 EEEP Curve for an Observed Monotonic Test (Test 1M-a)

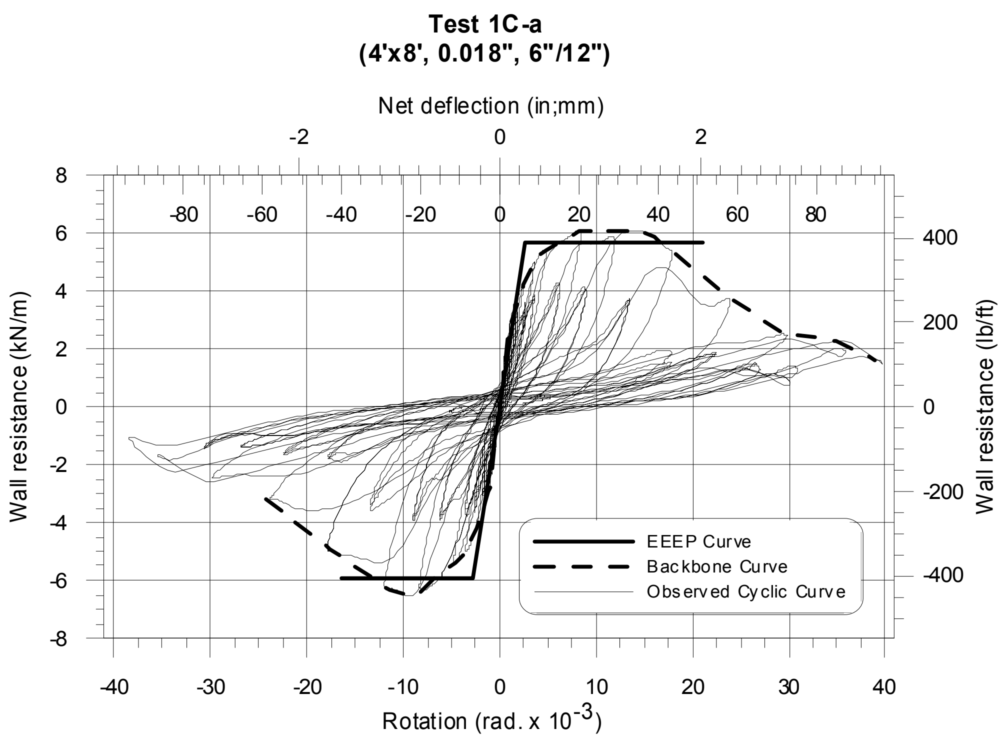


Figure 3.3 EEEP Curves for an Observed Reversed-Cyclic Test (Test 1C-a)

Table 3.1 Design Values from Monotonic Tests

Test Specimen	Yield Wall Resistance 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/m)/mm)	Rotation at $0.4S_u$ $\theta_{net,0.4u}$ (rad)	Rotation at S_y $\theta_{net,y}$ (rad)	Ductility μ	Energy Dissipation E (Joules)
1M-a	5.86	3.30	7.45	0.79	0.00135	0.00306	9.79	496
1M-b	5.85	2.81	6.20	0.94	0.00115	0.00254	5.97	242
1M-c	5.83	2.04	4.64	1.25	0.00084	0.00190	7.70	238
2M-a	9.00	4.46	9.94	0.90	0.00183	0.00408	9.10	937
2M-b	9.36	3.52	8.40	1.12	0.00144	0.00345	11.91	1094
3M-a	5.04	2.84	6.58	0.76	0.00116	0.00270	8.75	333
3M-b	5.04	3.16	7.15	0.71	0.00130	0.00293	8.43	348
8M-a	11.60	5.56	12.73	0.92	0.00228	0.00522	7.86	662
8M-b	12.01	4.97	11.45	1.05	0.00204	0.00470	8.73	690
9M-a	13.16	6.68	14.98	0.89	0.00274	0.00614	5.06	548
9M-b	13.40	5.41	12.26	1.10	0.00222	0.00503	6.67	619
9M-c	16.77	7.28	16.67	1.00	0.00299	0.00684	6.00	937
10M-a	9.60	4.20	9.56	1.00	0.00172	0.00392	10.46	557
11M-a	13.61	2.97	6.63	2.05	0.00122	0.00272	8.34	1724
11M-b	14.10	3.68	8.42	1.67	0.00151	0.00345	6.05	1607
17M-a	7.55	3.13	7.20	1.05	0.00128	0.00295	5.51	332
17M-b	6.61	5.47	12.38	0.53	0.00224	0.00508	2.48	197
18M-a	8.38	3.18	7.29	1.15	0.00130	0.00299	8.82	620

Table 3.2 Design Values from Reversed Cyclic Tests – Positive Cycles

Test Specimen	Yield Wall Resistance 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/m)/mm)	Rotation at $0.4S_{u+}$ $\theta_{net,0.4u+}$ (rad)	Rotation at S_{y+} $\theta_{net,y+}$ (rad)	Ductility μ	Energy Dissipation[1] E (Joules)
1C-a	5.68	2.70	6.29	0.90	0.00111	0.00258	8.18	334
1C-b	5.76	3.10	7.00	0.82	0.00127	0.00287	5.74	258
2C-a	9.98	4.40	9.88	1.01	0.00180	0.00405	8.22	928
2C-b	10.00	4.20	9.76	1.03	0.00172	0.00400	9.83	1109
3C-a	5.63	3.30	7.70	0.73	0.00135	0.00316	8.91	445
3C-c	5.58	2.60	6.13	0.91	0.00107	0.00251	9.02	355
8C-a	12.40	6.00	13.50	0.92	0.00246	0.00554	6.72	634
8C-b	12.54	5.30	12.15	1.03	0.00217	0.00498	7.40	641
9C-a	15.15	8.10	18.96	0.80	0.00332	0.00778	5.24	831
9C-b	14.88	7.80	18.08	0.82	0.00320	0.00741	5.52	824
11C-a	14.81	3.20	7.35	2.01	0.00131	0.00301	7.08	1745
11C-b	14.96	2.70	6.24	2.40	0.00111	0.00256	7.84	1670

[1] Energy Calculation based on area below backbone curve

Table 3.3 Design Values from Reversed Cyclic Tests – Negative Cycles

Test Specimen	Yield Wall Resistance 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/m)/mm)	Rotation at $0.4S_{u-}$ $\theta_{net,0.4u-}$ (rad)	Rotation at S_{y-} $\theta_{net,y-}$ (rad)	Ductility μ	Energy Dissipation[1] E (Joules)
1C-a	-5.90	-3.10	-6.99	0.84	-0.00127	-0.00287	5.75	264
1C-b	-5.52	-2.90	-6.56	0.84	-0.00119	-0.00269	5.28	211
2C-a	-10.15	-4.00	-9.43	1.07	-0.00164	-0.00387	8.99	991
2C-b	-10.02	-3.80	-8.94	1.12	-0.00156	-0.00367	9.83	1019
3C-a	-5.11	-3.10	-7.22	0.71	-0.00127	-0.00296	7.88	332
3C-c	-5.70	-3.60	-8.19	0.70	-0.00148	-0.00336	5.41	279
8C-a	-12.39	-5.40	-11.99	1.03	-0.00221	-0.00492	7.33	619
8C-b	-12.02	-6.10	-14.12	0.85	-0.00250	-0.00579	7.08	681
9C-a	-14.59	-9.20	-21.41	0.69	-0.00377	-0.00878	4.67	794
9C-b	-12.84	-5.90	-12.28	1.05	-0.00242	-0.00504	8.14	734
11C-a	-14.73	-3.70	-8.43	1.75	-0.00152	-0.00346	7.13	2008
11C-b	-14.46	-2.90	-6.63	2.18	-0.00119	-0.00272	7.43	1621

[1] Energy Calculation based on area below backbone curve

3.3 Limit States Design Procedure

The data from tests by the author, Ong-Tone *(2009)*, Yu *et al. (2007)* and Ellis *(2007)* were combined to develop a limit states design procedure for use with the 2005 NBCC and consistent with what has been done for wood sheathed shear walls in Canada. The test results from Serrette *(1997)* were not utilized because the measured material properties of the framing and sheathing were not available. The US data (Yu *et al.* and Ellis) was analyzed by Velchev *(2009)* and was incorporated with the test data from McGill University to compare results and to obtain uniform values.

All tests were analyzed using the EEEP method to obtain uniformity in analysis. There were a total of 73 tests from the US and McGill that were analyzed of which 36 were monotonic tests, and 37 were reversed cyclic tests. Additional tests were carried out; however these were excluded from the analysis because they had modifications that could not be used for analysis due to their variation from the basic wall configurations. All modified walls with additional reinforcement around corner edges or bridging were also excluded. The short walls, 610x2440mm (2'x8'), were only considered to compare the effect of length on shear resistance, i.e. the AISI S213 specified shear resistance reduction factor for high aspect ratio shear walls.

Table 3.4 Material Properties

Component	Nominal Thickness	Measured Base Thickness		Yield Stress	Tensile Stress	Reference
	mils	mm	in	MPa	MPa	
Sheathing	18	0.46	0.018	300	395	McGill
	27	0.61	0.024	347	399	Yu *et al.*
	30	0.73	0.029	337	383	Yu *et al.*
		0.76	0.030	307	385	Ellis
		0.76	0.030	284	373	McGill
	33	0.91	0.036	299	371	Yu *et al.*

Table 3.5 Description of Groups and Tests

Group	Nominal Framing (mils)	Nominal Sheathing (mils)	Fastener Spacing (mm)	Protocol	Test Name
1	33	18	150/300	monotonic	3M-a, 3M-b
				cyclic	3C-a, 3C-c
2		27	50/300	monotonic	Y7M1, Y7M2
				cyclic	Y7C1, Y7C2
3			100/300	monotonic	Y8M1, Y8M2
				cyclic	Y8C1, Y8C2
4			150/300	monotonic	Y9M1, Y9M2
				cyclic	Y9C1, Y9C2
5	43	18	50/300	monotonic	2M-a, 2M-b
				cyclic	2C-a, 2C-b
6			150/300	monotonic	1M-a, 1M-b, 1M-c
				cyclic	1C-a, 1C-b
7		30	50/300	monotonic	Y4M1, Y4M2 6M-a, 6M-b, 13M-a
				cyclic	Y4C1, 6C-a, 6C-b
8			100/300	monotonic	Y5M1, Y5M2, 5M-a, 5M-b, 11M-a, 11M-b, 12M-a, 15M-a
				cyclic	Y5C1, 5C-a, 5C-b, 11C-a, 11C-b, E114, E115, E116, E117, E118, E119, E120
9			150/300	monotonic	Y6M1, Y6M2 4M-a, 4M-b
				cyclic	Y6C1, Y6C2 4C-a, 4C-b
10		33	50/300	monotonic	Y1M1, Y1M2
				cyclic	Y1C1, Y1C2
11			100/300	monotonic	Y2M1, Y2M2
				cyclic	Y2C1, Y2C2
12			150/300	monotonic	Y3M1, Y3M2
				cyclic	Y3C1, Y3C2

In both the US and McGill tests, the walls had the same nominal sizes although the coupon tests that were carried out showed that the measured material properties were different in thickness, yield stress, and tensile stress (Table 3.4). The tests were grouped based on nominal values of framing thickness, sheathing thickness, and the fastener spacing schedule for a total of 12 groups (Table 3.5). The minimum specified yield stress is 230MPa (33ksi) and the minimum specified tensile stress is 310MPa (45ksi) as per ASTM A653 *(2008).*

3.3.1 Calibration of Resistance Factor

In limit states design, the factored resistance of any structural element must have sufficient strength and stability to resist the combined effects of loads applied to it (Equation (3-5)). The combined effects of loads are based on the most critical load combination as defined in Clause 4.1.3.2 of the 2005 NBCC *(NRCC, 2005).*

$$\phi R \geq \sum \alpha S \qquad (3\text{-}5)$$

where,

ϕ= Resistance factor of structural element

R= Nominal resistance of structural member

α= Load factor

S= Effect of particular specified load

The North American Specification for the Design of Cold-Formed Steel Structural Members (CSA-S136) *(2007)* defines a method for determining the resistance factor of CFS materials for ultimate limit states design (Equation (3-6)).

$$\phi = C_\phi (M_m F_m P_m) e^{-\beta_o \sqrt{V_M{}^2 + V_F{}^2 + C_P V_P{}^2 + V_S{}^2}} \qquad (3\text{-}6)$$

where,

C_ϕ= Calibration coefficient

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

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

P_m = Mean value of professional factor for tested component

V_m = Coefficient of variation of material factor

V_F = Coefficient of variation of fabrication factor

e = Natural logarithmic base = 2.718…

V_P = Coefficient of variation of the assembly resistance

V_S = Coefficient of variation of the load effect

β_o =Target reliability index, 2.5 for structural members

C_p = Correction factor for sample size

= $(1+1/n)m/(m-2)$ for $n \geq 4$,

= 5.7 for n=3

where,

n = Number of tests (sample size)

m = Degrees of freedom = n-1

The CSA-S136 Standard *(2007)* lists values for the mean value, M_m, and its coefficient of variation, V_M, for the material factor and the mean value, F_m, and its corresponding coefficient of variation, V_F, for the fabrication factor. The variables are based on statistical analysis of the materials used and their type of failure. For this analysis, four types of failure were considered and are listed together with their corresponding factors in Table 3.6. The connection failures considered were the shear failure of the screw and tilting and bearing failure. The frame failure modes considered were the buckling of the compression chord stud, and the deformation of the track due to uplift.

Table 3.6 Statistical Data for the Determination of Resistance Factor *(CSA-S136,2007)*

Type of Component	M_m	V_M	F_m	V_F
1.Connection: Shear Strength of Screw Connection	1.10	0.10	1.00	0.10
2.Connection: Tilting and Bearing Strength of Screw Connection	1.10	0.08	1.00	0.05
3.Wall Studs: Wind loads considering Compression of Chord Stud	1.10	0.10	1.00	0.05
4.Tracks: Structural Members not listed	1.00	0.10	1.00	0.05

Branston *(2004)* was able to calculate the coefficient of calibration, C_ϕ, based on documented wind load statistics. Branston *(2004)* used a load factor, α, of 1.4, with a mean value to nominal value, $\bar{S}/S$, of 0.76 for wind loads and a coefficient of variation, V_S, of 0.37. The wind load factor of 1.4 was proposed to and included in the 2005 NBCC *(NRCC, 2005)*. The calibration coefficient, C_ϕ, was then calculated using Equation (3-7) using the aforementioned values for a result of 1.842.

$$C_\phi = \frac{\alpha}{\bar{S}/S} \tag{3-7}$$

For structural members, the CSA-S136 Standard *(2007)* lists a value of 2.5 for the reliability factor, β_o, which is a factor describing the probability of failure. The professional factor, P_m, is calculated based on the yield shear resistance, S_y, to average yield shear resistance, $S_{y,avg}$, ratio for all tests in a sample, and divided by the sample size of each configuration, n (Equation (3-8)). The average yield shear resistance, $S_{y,avg}$, is based on the average of both the monotonic and cyclic test values (Equation (3-9)). The monotonic and cyclic tests are given the same weight regardless of the number of tests carried out for each type of protocol. In addition, the positive and negative yield shear resistances of the cyclic tests were considered as part of a conservative approach. The negative region of the cyclic tests usually resulted in lower yield shear resistances since the walls were pushed into the inelastic region on the positive cycles before returning to the negative cycles.

$$P_m = \frac{\sum_{i=1}^{n}\left(S_y / S_{y,avg}\right)_i}{n} \tag{3-8}$$

$$S_{y,avg} = \frac{S_{y,mono,avg} + \frac{S_{y+,avg} + S_{y-,avg}}{2}}{2} \tag{3-9}$$

where,

$S_{y,mono,avg}$= average shear resistance of monotonic tests of a specific configuration

$S_{y+,avg}$= average shear resistance of the positive cyclic tests of a specific configuration

$S_{y-,avg}$= average shear resistance of the negative cyclic tests of a specific configuration

The coefficient of variation, V_P, related to the professional factor, P_m, can be calculated using Equation (3-10)

$$V_P = \frac{\sigma}{P_m} \tag{3-10}$$

where,

$$\sigma^2 = \frac{1}{n-1}\sum_{i=1}^{n}\left[\left(S_y/S_{y,avg}\right)_i - P_m\right]^2 \tag{3-11}$$

Tables 3.7, 3.8, 3.9, and 3.10 summarize all the factors that contribute to the resistance factor, ϕ, based on the failure mode as described in Table 3.6. The resistance factor calculated was very consistent in all cases with an average of 0.74. Therefore, the resistance factor, ϕ, is recommended to be 0.70 which is slightly more conservative than the value calculated and it is also consistent with the findings of Ong-Tone *(2009).* This factor is applied to steel sheathed shear walls in design and analysis.

Table 3.7 Resistance Factor Calibration for Type 1: Shear Strength of Screw Connection

Configuration	α	$\bar{S}/S$	C_ϕ	M_m	F_m	P_m	β_o	V_M	V_F	V_S	n	C_p	V_p	ϕ
1	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.10	0.10	0.37	4	3.75	0.06	0.73
2	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.10	0.10	0.37	4	3.75	0.06	0.72
3	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.10	0.10	0.37	4	3.75	0.02	0.75
4	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.10	0.10	0.37	4	3.75	0.02	0.75
5	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.10	0.10	0.37	4	3.75	0.05	0.73
6	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.10	0.10	0.37	5	2.40	0.02	0.75
7	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.10	0.10	0.37	8	1.58	0.06	0.74
8	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.10	0.10	0.37	20	1.17	0.07	0.74
9	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.10	0.10	0.37	8	1.58	0.06	0.74
10	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.10	0.10	0.37	4	3.75	0.03	0.75
11	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.10	0.10	0.37	4	3.75	0.04	0.74
12	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.10	0.10	0.37	4	3.75	0.03	0.75
Average	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.10	0.10	0.37	73	1.04	0.05	0.75

Table 3.8 Resistance Factor Calibration for Type 2: Tilting and Bearing of Screw

Configuration	α	$\overline{S}/S$	C_ϕ	M_m	F_m	P_m	β_o	V_M	V_F	V_S	n	C_p	V_p	ϕ
1	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.08	0.05	0.37	4	3.75	0.06	0.75
2	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.08	0.05	0.37	4	3.75	0.06	0.75
3	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.08	0.05	0.37	4	3.75	0.02	0.78
4	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.08	0.05	0.37	4	3.75	0.02	0.78
5	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.08	0.05	0.37	4	3.75	0.05	0.75
6	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.08	0.05	0.37	5	2.4	0.02	0.78
7	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.08	0.05	0.37	8	1.58	0.06	0.77
8	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.08	0.05	0.37	20	1.17	0.07	0.76
9	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.08	0.05	0.37	8	1.58	0.06	0.76
10	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.08	0.05	0.37	4	3.75	0.03	0.77
11	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.08	0.05	0.37	4	3.75	0.04	0.77
12	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.08	0.05	0.37	4	3.75	0.03	0.77
Average	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.08	0.05	0.37	73	1.04	0.05	0.77

Table 3.9 Resistance Factor Calibration for Type 3: Compression Chord Stud

Configuration	α	$\overline{S}/S$	C_ϕ	M_m	F_m	P_m	β_o	V_M	V_F	V_S	n	C_p	V_p	ϕ
1	1.4	0.76	1.842	1.00	1.00	1.00	2.50	0.10	0.05	0.37	4	3.75	0.06	0.68
2	1.4	0.76	1.842	1.00	1.00	1.00	2.50	0.10	0.05	0.37	4	3.75	0.06	0.67
3	1.4	0.76	1.842	1.00	1.00	1.00	2.50	0.10	0.05	0.37	4	3.75	0.02	0.70
4	1.4	0.76	1.842	1.00	1.00	1.00	2.50	0.10	0.05	0.37	4	3.75	0.02	0.70
5	1.4	0.76	1.842	1.00	1.00	1.00	2.50	0.10	0.05	0.37	4	3.75	0.05	0.68
6	1.4	0.76	1.842	1.00	1.00	1.00	2.50	0.10	0.05	0.37	5	2.40	0.02	0.70
7	1.4	0.76	1.842	1.00	1.00	1.00	2.50	0.10	0.05	0.37	8	1.58	0.06	0.69
8	1.4	0.76	1.842	1.00	1.00	1.00	2.50	0.10	0.05	0.37	20	1.17	0.07	0.69
9	1.4	0.76	1.842	1.00	1.00	1.00	2.50	0.10	0.05	0.37	8	1.58	0.06	0.69
10	1.4	0.76	1.842	1.00	1.00	1.00	2.50	0.10	0.05	0.37	4	3.75	0.03	0.69
11	1.4	0.76	1.842	1.00	1.00	1.00	2.50	0.10	0.05	0.37	4	3.75	0.04	0.69
12	1.4	0.76	1.842	1.00	1.00	1.00	2.50	0.10	0.05	0.37	4	3.75	0.03	0.69
Average	1.4	0.76	1.842	1.00	1.00	1.00	2.50	0.10	0.05	0.37	73	1.04	0.05	0.69

Table 3.10 Resistance Factor Calibration for Type 4: Uplift of Track

Configuration	α	$\bar{S}/S$	C_ϕ	M_m	F_m	P_m	β_o	V_M	V_F	V_S	n	C_p	V_p	ϕ
1	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.10	0.05	0.37	4	3.75	0.06	0.74
2	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.10	0.05	0.37	4	3.75	0.06	0.74
3	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.10	0.05	0.37	4	3.75	0.02	0.77
4	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.10	0.05	0.37	4	3.75	0.02	0.77
5	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.10	0.05	0.37	4	3.75	0.05	0.74
6	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.10	0.05	0.37	5	2.40	0.02	0.77
7	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.10	0.05	0.37	8	1.58	0.06	0.76
8	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.10	0.05	0.37	20	1.17	0.07	0.76
9	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.10	0.05	0.37	8	1.58	0.06	0.76
10	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.10	0.05	0.37	4	3.75	0.03	0.76
11	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.10	0.05	0.37	4	3.75	0.04	0.76
12	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.10	0.05	0.37	4	3.75	0.03	0.76
Average	1.4	0.76	1.842	1.10	1.00	1.00	2.50	0.10	0.05	0.37	73	1.04	0.05	0.76

3.3.3 Nominal Shear Wall Resistance

The measured material properties of the test components were higher than the minimum specified values (Section 3.3). The ASTM A653 Specification *(2008)* states that a material with a yield stress of 230MPa (33ksi) should have a corresponding tensile stress of 310MPa (45ksi). Table 3.4 summarizes the measured material properties and shows that the tensile stresses are much higher than the minimum specified. The resistance values calculated using the EEEP approach (Section 3.2) are influenced by the overstrength of the steel compared with the minimum specified. To address this, it was proposed to reduce the shear resistance of the wall specimens to provide values that correspond to the minimum specified properties. The connection resistance for bearing in the CSA-S136 Standard is based on the thickness of the material and its tensile stress. Since the overall shear wall resistance was found to be directly dependent on the sheathing connections a procedure was adopted to adjust the calculated EEEP S_y values by the measured-to-nominal thickness ratio and the measured-to-nominal tensile stress ratio of the sheathing. The modification of the shear resistance values for thickness and tensile stress to obtain nominal resistance values is provided in Appendix D.

The proposed nominal shear resistance values for CFS frame/steel sheathed shear walls are listed in Table 3.11. The values for a fastener spacing of 75mm (3") are interpolated from the data provided by the other fastener spacings. The nominal shear resistance values represent lower bound values for lateral loading of unblocked walls. A factor accounting for the increase in resistance provided by blocking can be used, however the effects of full blocking have not been thoroughly examined. Moreover, chord studs must be designed to avoid compression failure of these column members. An aspect ratio of 4:1 is permissible for shear walls consisting of 0.76mm (0.030") sheathing with 1.09mm (0.043") framing, and for 0.84mm (0.033") sheathing with 1.09mm (0.043") framing.

Table 3.11 Proposed Nominal Shear Resistance, S_y, for CFS Frame/Steel Sheathed Shear Walls[1,2,8] (kN/m)

Assembly Description	Max. Aspect Ratio (h/w)[3]	Fastener Spacing[4] at Panel Edges (mm(in))				Designation Thickness[5,6] of Stud, Track, and Blocking (mils)	Required Sheathing Screw Size[7]
		150(6)	100(4)	75(3)	50(2)		
0.46 mm (0.018") steel sheet, one side	2:1	4.13	-	-	-	33	8
		4.53	6.03	6.78	7.53	43	8
0.68 mm (0.027") steel sheet, one side	2:1	6.48	7.17	7.94	8.69	33	8
0.76 mm (0.030") steel sheet, one side	4:1	8.89	10.58	11.56	12.54	43	8
0.84 mm (0.033") steel sheet, one side	4:1	10.69	12.01	12.97	13.93	43	8

1 Nominal resistance is to be multiplied by the resistance factor, ϕ, to obtain factored resistance

2 Sheathing will be connected vertically to the steel frame

3 Nominal shear resistances are to be multiplied by 2w/h for aspect ratios greater than 2:1 but no greater than 4:1

4 Field screws to be spaced at 300mm on centre

5 Wall stud and track shall be of ASTM A653 grade 230MPa with a minimum uncoated base thickness of 0.84mm (0.033") for members with a designation thickness of 33mils, and ASTM A653 grade 230MPA with a minimum uncoated base thickness of 1.09mm (0.043") for members with a designation thickness of 43mils

6 Substitution of wall stud or track is not permitted

7 Minimum No.8x12.7mm (1/2") sheathing screws shall be used

8 Tabulated nominal shear resistances are applicable for lateral loading only

3.3.2.1 Verification of Shear Resistance Reduction for High Aspect Ratio Walls

Some short walls measuring 610x2440mm (2'x8') for an aspect ratio of 4:1 were tested by Yu *et al. (2007)* and at McGill University. The purpose of these specimens was to verify whether walls with higher aspect ratios can be utilized in design. The AISI S213 Standard *(2007)* states that for walls with an aspect ratio greater than 2:1 but no greater than 4:1, the shear resistance for design can be obtained by multiplying the listed nominal shear resistance by two times the ratio of width to height (2*w*/*h*). To verify the applicability of this allowance, the nominal shear resistances tabulated in Table 3.11 were multiplied by 2*w*/*h* and were compared with test results of 610x2440mm (2'x8') shear walls. The shear resistances were obtained using the EEEP method for the short walls (Tables 3.1, 3.2, and 3.3) and were reduced based on thickness and tensile stress. Yu *et al. (2007)* tested a number of short walls consisting of 1.09mm (0.043") framing with 0.76mm (0.030") and 0.84mm (0.033") sheathing for 50mm (2"), 100mm (4"), and 150mm (6") fastener spacing. Similarly, at McGill University, short walls consisting of 0.76mm (0.030") sheathing on 1.09mm (0.043") framing for 50mm (2") and 100mm (4") fastener spacing were tested.

It was found that the test-based resistances of the short walls that were calibrated for thickness and tensile stress resulted in higher shear strength values than the nominal resistance values modified using the 2*w*/*h* factor (Table 3.12). However, even though the short wall tests reached higher resistances, they had to be pushed to large displacements to reach those load levels. A comparison of the drifts, Δ_d, that are presented in Figure 3.4 for the 610mm (2') long walls and in Figure 3.5 for the 1220mm (4') long walls was made. The drift, Δ_d, is determined as the displacement reached at the equivalent resistance level for the 610mm (2') and 1220mm (4') long walls. It was found that the drifts, Δ_d, for the 610mm (2') long walls were less than the drifts for the 1220mm (4') long walls (Table 3.13). These values show that the reduction factor of 2*w*/*h* is applicable because if the short walls reach the modified resistance level, they will perform adequately as they would reach similar drifts as the longer walls. Therefore, higher aspect ratios not greater than 4:1 are permissible for shear walls consisting of 0.76mm (0.030”) or 0.84mm (0.033”) sheathing with 1.09mm (0.043”) framing for a fastener spacing of 50mm (2”), 100mm (4”), or 150mm (6”). The 1220mm (4’) shear walls with 0.46mm (0.018”) had low capacities and, therefore, shorter 610mm (2’) walls were not tested and the use of higher aspect ratios could not be verified. A shear resistance reduction for 610mm (2’) shear walls with 0.68mm (0.027”) sheathing could potentially be used. However, no short walls were tested as 0.68mm (0.027”) sheathing was difficult to obtain.

Table 3.12 Verification of Shear Resistance Reduction for High Aspect Ratio Walls

Group	Framing (mils)	Sheathing (mils)	Fastener Spacing (mm)	Test	S_y (kN/m)	$S_{y,red}$ (kN/m)	$S_{y,red,avg}$ (kN/m)	$S_{y,red,avg}$ (kN/m)	S_y nominal[1] (kN/m)	S_y*2w/h (kN/m)
7	43	30	50	9M-a	13.15	10.93	11.64	12.02	12.54	6.27
				9M-b	13.40	11.14				
				Y13M1	15.29	12.36				
				Y13M2	15.02	12.14				
				9C-a	14.87	12.36	12.39			
				9C-b	13.86	11.52				
				Y13C1	15.82	12.79				
				Y13C2	15.93	12.88				
8			100	8M-a	11.60	9.64	10.03	10.32	10.58	5.29
				8M-b	12.00	9.97				
				Y14M1	12.55	10.15				
				Y14M2	12.82	10.37				
				8C-a	12.39	10.30	10.61			
				8C-b	12.28	10.21				
				Y14C1	13.98	11.30				
				Y14C2	13.15	10.63				
9			150	Y15M1	11.57	9.35	9.22	9.34	8.89	4.45
				Y15M2	11.25	9.09				
				Y15C1	11.69	9.46	9.46			
				Y15C2	11.71	9.47				
10		33	50	Y10M1	17.29	13.32	13.51	13.57	13.93	6.97
				Y10M2	17.78	13.69				
				Y10C1	18.75	14.44	13.63			
				Y10C2	16.65	12.82				
11			100	Y11M1	14.96	11.52	11.43	11.88	12.01	6.01
				Y11M2	14.71	11.33				
				Y11C1	15.81	12.18	12.33			
				Y11C2	16.21	12.49				
12			150	Y12M1	14.23	10.96	10.18	10.65	10.69	5.35
				Y12M2	12.20	9.40				
				Y12C1	14.47	11.15	11.12			
				Y12C2	14.40	11.09				

[1] **Nominal resistance values from Table 3.11**

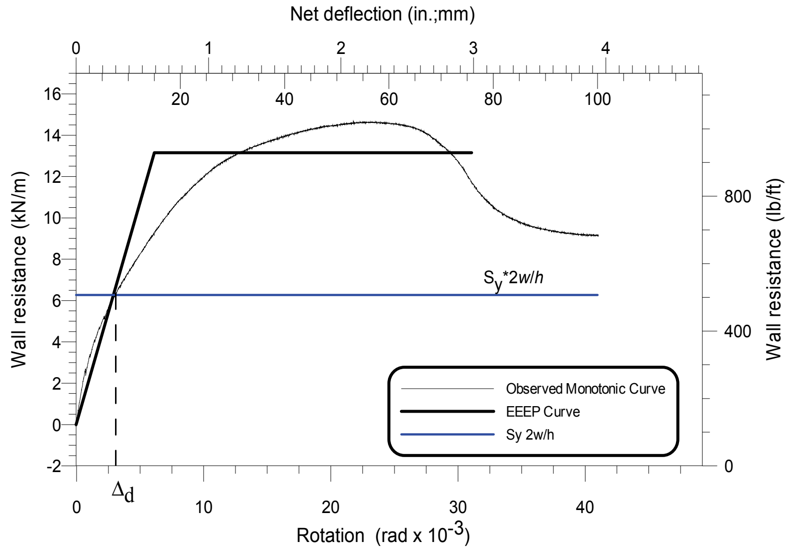


Figure 3.4 Drift, Δ_d, for Short Wall at Reduced Resistance

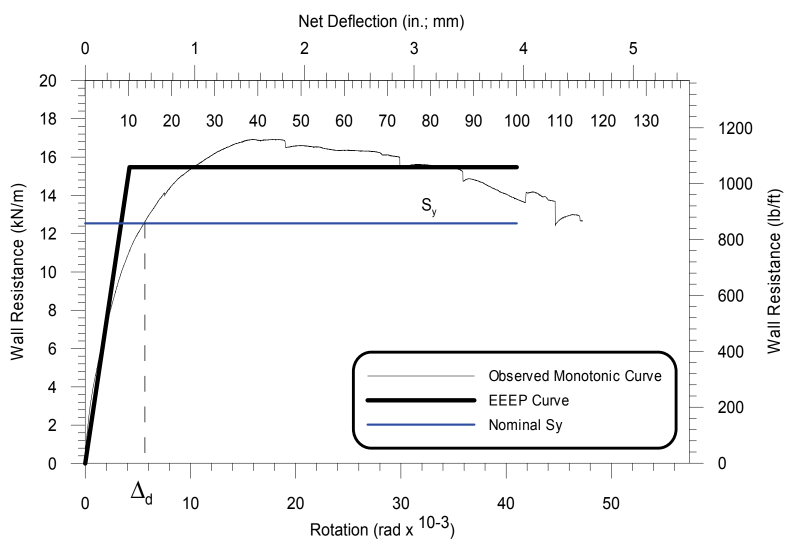


Figure 3.5 Drift, Δ_d, for 1220mm (4') Long Wall at Nominal Resistance

Table 3.13 Average Drift Values, Δ_d

Group	Nominal Framing (mils)	Nominal Sheathing (mils)	Fastener Spacing (mm)	Average Drift, Δ_d, for 610mm Long Walls (mm)	Average Drift, Δ_d, for 1220mm Long Walls (mm)
7	43	30	50	13	19
8			100	10	17
9			150	14	18
10		33	50	19	21
11			100	14	21
12			150	13	20

3.3.3 Factor of Safety

The factor of safety is the ratio of the ultimate shear resistance to the factored resistance of a shear wall as illustrated in Figure 3.6 and as calculated according to Equation (3-12). The difference in ultimate resistance of the shear walls in the positive and negative regions of the reversed cyclic tests was small and was considered negligible. When the walls were pushed to the same displacements in the negative region, the walls had already undergone damage from being initially pushed to the positive region, and in turn resulting in slightly lower ultimate resistance values. However, the degradation caused by the positive cycles was not significant and a decision was made to account for both the positive and negative values of the reversed cyclic tests. The ultimate resistance of each monotonic and reversed cyclic test used to calculate the factor of safety was not reduced for thickness and tensile stress. The factored resistance was obtained by multiplying the nominal shear resistance values tabulated in Table 3.11 with the recommended load resistance factor, ϕ, of 0.7.

$$F.S. = \frac{S_u}{S_r} \tag{3-12}$$

where,

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

S_u = Ultimate wall shear resistance observed during test

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

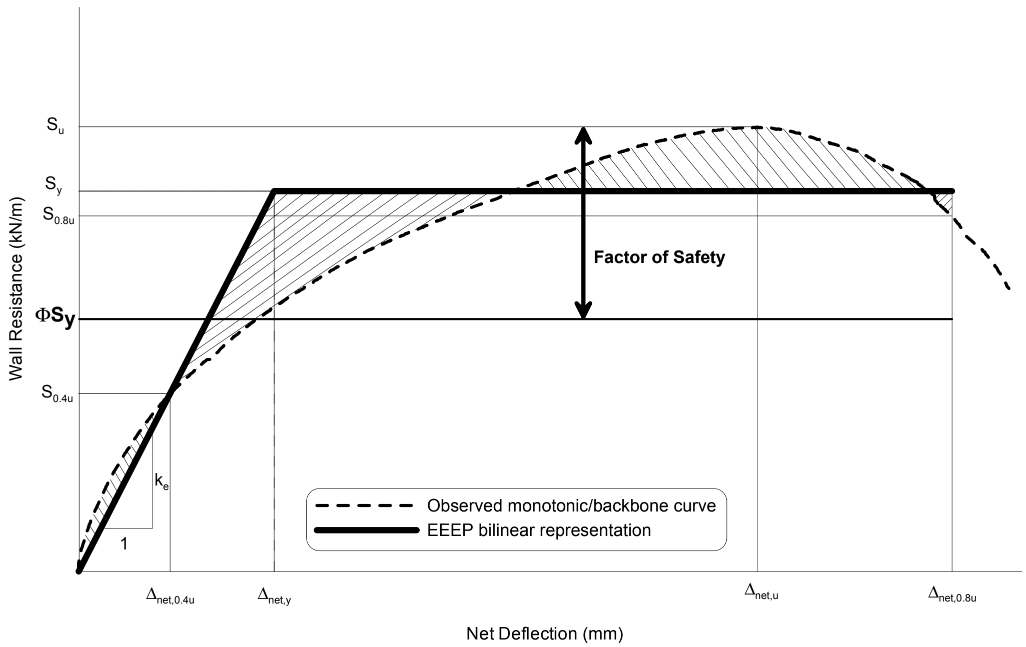


Figure 3.6 Factor of Safety Relationship with Ultimate and Factored Resistance *(Branston, 2004)*

The factor of safety was calculated using results for the 1220mm (4’), 1630mm (6’) and 2440mm (8’) long walls from both monotonic and reversed cyclic tests. The monotonic tests resulted in a mean factor of safety of 1.97 with a standard deviation of 0.085 and a coefficient of variation of 0.7% (Table 3.14). The reversed cyclic tests yielded a slightly higher factor of safety with a mean of 2.03, a standard deviation of 0.07 and a coefficient of variation of 0.5% (Table 3.15). In limit states design (LSD), where factored loads are compared with factored resistances, an average factor of safety of 2.00 was determined for monotonic and reversed cyclic tests. In addition, for allowable stress design (ASD), the factor of safety is amplified by the factor defined by the 2005 NBCC for wind loading of 1.4 for an average amplified factor of safety of 2.8 (Table 3.14 and 3.15). The factor of safety is applicable for wind loading; more specifically for lateral loading only and does not take into account the effects of gravity loads. For seismic loading, however, the capacity based design approach is used to account for the inelastic response of the structure using the seismic force modification factors, R_d and R_o as discussed in Section 3.3.4 .

Table 3.14 Factor of Safety for the Monotonic Test Specimens

Group	Test Name	Ultimate Resistance S_u (kN/m)	Nominal Resistance S_y (kN/m)[1]	Factored Resistance S_r (ϕ=0.7) (kN/m)	Factor of Safety (LSD) S_u/S_r		Factor of Safety (ASD) $1.4xS_u/S_r$	
					test	average	test	average
1	3M-a	5.44	4.13	2.89	1.88	1.90	2.63	2.66
	3M-b	5.58			1.93		2.70	
2	Y7M1	12.50	8.69	6.08	2.06	2.01	2.88	2.81
	Y7M2	11.91			1.96		2.74	
3	Y8M1	9.99	7.17	5.02	1.99	1.99	2.79	2.78
	Y8M2	9.96			1.98		2.78	
4	Y9M1	9.40	6.48	4.54	2.07	2.01	2.90	2.82
	Y9M2	8.85			1.95		2.73	
5	2M-a	10.10	7.53	5.27	1.91	1.89	2.68	2.64
	2M-b	9.81			1.86		2.60	
6	1M-a	6.50	4.53	3.17	2.05	2.05	2.87	2.87
	1M-b	6.63			2.09		2.92	
	1M-c	6.41			2.02		2.83	
7	Y4M1	15.74	12.54	8.78	1.79	1.89	2.51	2.64
	Y4M2	15.04			1.71		2.40	
	6M-a	16.93			1.93		2.70	
	6M-b	16.55			1.89		2.64	
	13M-a	18.53			2.11		2.96	
8	Y5M1	13.71	10.58	7.41	1.85	1.93	2.59	2.70
	Y5M2	14.26			1.93		2.70	
	5M-a	14.19			1.92		2.68	
	5M-b	13.39			1.81		2.53	
	11M-a	15.25			2.06		2.88	
	11M-b	15.41			2.08		2.91	
	12M-a	14.35			1.94		2.71	
	15M-a	13.79			1.86		2.61	
9	Y6M1	11.69	8.89	6.23	1.88	1.81	2.63	2.54
	Y6M2	11.48			1.84		2.58	
	4M-a	11.01			1.77		2.48	
	4M-b	10.98			1.76		2.47	
10	Y1M1	19.22	13.93	9.75	1.97	2.01	2.76	2.82
	Y1M2	20.07			2.06		2.88	
11	Y2M1	17.12	12.01	8.41	2.04	2.06	2.85	2.89
	Y2M2	17.57			2.09		2.93	
12	Y3M1	14.93	10.69	7.48	2.00	2.09	2.79	2.93
	Y3M2	16.40			2.19		3.07	
					Average	**1.97**		**2.76**
					STD.DEV.	**0.0854**		**0.1195**
					CoV.	**0.0073**		**0.0143**

[1] **Nominal values from Table 3.11**

Table 3.15 Factor of Safety for the Reversed Cyclic Test Specimens

<table>
<tr><th rowspan="2">Group</th><th rowspan="2">Test Name</th><th rowspan="2">Ultimate Resistance S_u (kN/m)</th><th rowspan="2">Nominal Resistance S_y (kN/m)[1]</th><th rowspan="2">Factored Resistance S_r (ϕ=0.7) (kN/m)</th><th colspan="2">Factor of Safety (LSD) S_u/S_r</th><th colspan="2">Factor of Safety (ASD) $1.4xS_u/S_r$</th></tr>
<tr><th>test</th><th>average</th><th>test</th><th>average</th></tr>
<tr><td rowspan="2">1</td><td>3C-a</td><td>5.76</td><td rowspan="2">4.13</td><td rowspan="2">2.89</td><td>1.99</td><td rowspan="2">2.05</td><td>2.79</td><td rowspan="2">2.87</td></tr>
<tr><td>3C-c</td><td>6.09</td><td>2.10</td><td>2.95</td></tr>
<tr><td rowspan="2">2</td><td>Y7C1</td><td>11.71</td><td rowspan="2">8.69</td><td rowspan="2">6.08</td><td>1.93</td><td rowspan="2">2.03</td><td>2.70</td><td rowspan="2">2.84</td></tr>
<tr><td>Y7C2</td><td>12.95</td><td>2.13</td><td>2.98</td></tr>
<tr><td rowspan="2">3</td><td>Y8C1</td><td>10.59</td><td rowspan="2">7.17</td><td rowspan="2">5.02</td><td>2.11</td><td rowspan="2">2.06</td><td>2.95</td><td rowspan="2">2.89</td></tr>
<tr><td>Y8C2</td><td>10.13</td><td>2.02</td><td>2.82</td></tr>
<tr><td rowspan="2">4</td><td>Y9C1</td><td>9.54</td><td rowspan="2">6.48</td><td rowspan="2">4.54</td><td>2.10</td><td rowspan="2">2.08</td><td>2.94</td><td rowspan="2">2.91</td></tr>
<tr><td>Y9C2</td><td>9.34</td><td>2.06</td><td>2.88</td></tr>
<tr><td rowspan="2">5</td><td>2C-a</td><td>10.93</td><td rowspan="2">7.53</td><td rowspan="2">5.27</td><td>2.07</td><td rowspan="2">2.05</td><td>2.90</td><td rowspan="2">2.87</td></tr>
<tr><td>2C-b</td><td>10.70</td><td>2.03</td><td>2.84</td></tr>
<tr><td rowspan="2">6</td><td>1C-a</td><td>6.32</td><td rowspan="2">4.53</td><td rowspan="2">3.17</td><td>1.99</td><td rowspan="2">1.98</td><td>2.79</td><td rowspan="2">2.77</td></tr>
<tr><td>1C-b</td><td>6.24</td><td>1.97</td><td>2.75</td></tr>
<tr><td rowspan="3">7</td><td>Y4C1</td><td>15.56</td><td rowspan="3">12.54</td><td rowspan="3">8.78</td><td>1.77</td><td rowspan="3">1.90</td><td>2.48</td><td rowspan="3">2.67</td></tr>
<tr><td>6C-a</td><td>17.11</td><td>1.95</td><td>2.73</td></tr>
<tr><td>6C-b</td><td>17.48</td><td>1.99</td><td>2.79</td></tr>
<tr><td rowspan="12">8</td><td>Y5C1</td><td>15.19</td><td rowspan="12">10.58</td><td rowspan="12">7.41</td><td>2.05</td><td rowspan="12">1.90</td><td>2.87</td><td rowspan="12">2.66</td></tr>
<tr><td>5C-a</td><td>14.47</td><td>1.95</td><td>2.73</td></tr>
<tr><td>5C-b</td><td>14.21</td><td>1.92</td><td>2.69</td></tr>
<tr><td>11C-a</td><td>16.15</td><td>2.18</td><td>3.05</td></tr>
<tr><td>11C-b</td><td>15.99</td><td>2.16</td><td>3.02</td></tr>
<tr><td>E114</td><td>14.18</td><td>1.91</td><td>2.68</td></tr>
<tr><td>E115</td><td>14.01</td><td>1.89</td><td>2.65</td></tr>
<tr><td>E116</td><td>12.77</td><td>1.72</td><td>2.41</td></tr>
<tr><td>E117</td><td>13.61</td><td>1.84</td><td>2.57</td></tr>
<tr><td>E118</td><td>12.47</td><td>1.68</td><td>2.36</td></tr>
<tr><td>E119</td><td>13.08</td><td>1.77</td><td>2.47</td></tr>
<tr><td>E120</td><td>12.86</td><td>1.74</td><td>2.43</td></tr>
<tr><td rowspan="4">9</td><td>Y6C1</td><td>13.15</td><td rowspan="4">8.89</td><td rowspan="4">6.23</td><td>2.11</td><td rowspan="4">2.04</td><td>2.96</td><td rowspan="4">2.85</td></tr>
<tr><td>Y6C2</td><td>13.44</td><td>2.16</td><td>3.02</td></tr>
<tr><td>4C-a</td><td>11.84</td><td>1.90</td><td>2.66</td></tr>
<tr><td>4C-b</td><td>12.29</td><td>1.97</td><td>2.76</td></tr>
<tr><td rowspan="2">10</td><td>Y1C1</td><td>20.41</td><td rowspan="2">13.93</td><td rowspan="2">9.75</td><td>2.09</td><td rowspan="2">2.02</td><td>2.93</td><td rowspan="2">2.83</td></tr>
<tr><td>Y1C2</td><td>18.98</td><td>1.95</td><td>2.72</td></tr>
<tr><td rowspan="2">11</td><td>Y2C1</td><td>17.32</td><td rowspan="2">12.01</td><td rowspan="2">8.41</td><td>2.06</td><td rowspan="2">2.10</td><td>2.88</td><td rowspan="2">2.94</td></tr>
<tr><td>Y2C2</td><td>17.97</td><td>2.14</td><td>2.99</td></tr>
<tr><td rowspan="2">12</td><td>Y3C1</td><td>16.24</td><td rowspan="2">10.69</td><td rowspan="2">7.48</td><td>2.17</td><td rowspan="2">2.13</td><td>3.04</td><td rowspan="2">2.98</td></tr>
<tr><td>Y3C2</td><td>15.64</td><td>2.09</td><td>2.93</td></tr>
<tr><td></td><td></td><td></td><td></td><td></td><td>Average</td><td>2.03</td><td></td><td>2.84</td></tr>
<tr><td></td><td></td><td></td><td></td><td></td><td>STD.DEV.</td><td>0.0702</td><td></td><td>0.0983</td></tr>
<tr><td></td><td></td><td></td><td></td><td></td><td>CoV.</td><td>0.0049</td><td></td><td>0.0097</td></tr>
</table>

[1] Nominal values from Table 3.11

3.3.4 Capacity Based Design

The AISI S213 Standard requires that the design of structures for seismic resistance follows the capacity based design method. The method is based on the selection of an element that dissipates energy by means of inelastic deformations. However, the chosen element is designed to be ductile in the case of failure. The energy dissipating element, or “fuse”, exhibits inelastic behaviour while all other elements in the seismic force resisting system are designed to remain elastic and are expected to be able to resist corresponding applied loads.

In the case of steel sheathed shear walls, the energy dissipating element is the connection between the sheathing and framing. The ductile behaviour is exhibited through bearing deformation at the sheathing connections. All other elements within the shear wall such as hold-downs, anchors, tracks, field and chord studs, and fasteners are expected to retain their strengths throughout the duration of seismic activity. It should be noted that the walls may exhibit brittle behaviour due to loss of ductility if the fasteners fail due to fracture or if the compression chord studs fail due to buckling.

An overstrength factor is applied to approximate the probable capacity of a shear wall. It is based on the assumption that during design level seismic activity the shear wall will reach its ultimate capacity when pushed to inelastic displacements. The structural elements are designed using the overstrength factor to resist the estimated capacity of the shear wall and to ensure that they do not themselves exhibit inelastic behaviour.

The overstrength factor is determined by the ratio of ultimate to nominal resistance as depicted in Figure 3.7. The overstrength is calculated in a similar manner to the factor of safety where the ultimate resistance used is not calibrated for thickness and tensile stress and accounts for both positive and negative values of the reversed cyclic tests as well as the monotonic tests (Equation (3-13)). Only the results for the 1220mm (4’) and longer walls were used to determine the overstrength factor.

$$overstrength = \frac{S_u}{S_y} \tag{3-13}$$

where,

S_u = Ultimate wall resistance observed during test

S_y = Nominal yield wall resistance

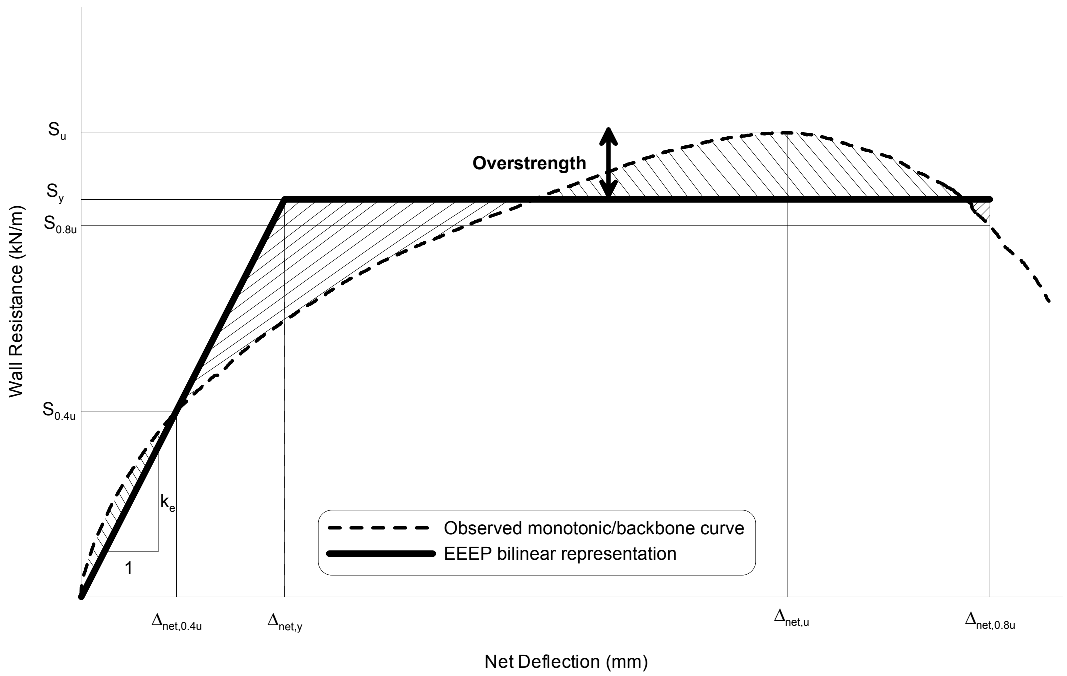


Figure 3.7 Overstrength Relationship with Ultimate and Factored Resistance
(Branston, 2004)

The monotonic tests have a mean overstrength factor of 1.38, a standard deviation of 0.06 and a coefficient of variation of 3.6 % (Table 3.16). The reversed cyclic tests have a mean overstrength factor of 1.42, a standard deviation of 0.05 and a coefficient of variation of 2.4 % (Table 3.17). Therefore, it is recommended to use an average overstrength factor of 1.40 for steel sheathed shear walls in the design of structural elements such as chord studs.

Table 3.16 Overstrength Design Values for Monotonic Tests

Group	Test Name	Ultimate Resistance S_u (kN/m)	Nominal Resistance S_y (kN/m)[1]	Overstrength S_u/S_y	
				test	average
1	3M-a 3M-b	5.44 5.58	4.13	1.32 1.35	1.33
2	Y7M1 Y7M2	12.50 11.91	8.69	1.44 1.37	1.40
3	Y8M1 Y8M2	9.99 9.96	7.17	1.39 1.39	1.39
4	Y9M1 Y9M2	9.40 8.85	6.48	1.45 1.37	1.41
5	2M-a 2M-b	10.10 9.81	7.53	1.34 1.30	1.32
6	1M-a 1M-b 1M-c	6.50 6.63 6.41	4.53	1.43 1.46 1.41	1.44
7	Y4M1 Y4M2 6M-a 6M-b 13M-a	15.74 15.04 16.93 16.55 18.53	12.54	1.26 1.20 1.35 1.32 1.48	1.32
8	Y5M1 Y5M2 5M-a 5M-b 11M-a 11M-b 12M-a 15M-a	13.71 14.26 14.19 13.39 15.25 15.41 14.35 13.79	10.58	1.30 1.35 1.34 1.27 1.44 1.46 1.36 1.30	1.35
9	Y6M1 Y6M2 4M-a 4M-b	11.69 11.48 11.01 10.98	8.89	1.31 1.29 1.24 1.23	1.27
10	Y1M1 Y1M2	19.22 20.07	13.93	1.38 1.44	1.41
11	Y2M1 Y2M2	17.12 17.57	12.01	1.43 1.46	1.44
12	Y3M1 Y3M2	14.93 16.40	10.69	1.40 1.53	1.47
				Average	**1.38**
				STD.DEV.	**0.0598**
				CoV.	**0.0036**

[1] Nominal values from Table 3.11

Table 3.17 Overstrength Design Values for Reversed Cyclic Tests

Group	Test Name	Ultimate Resistance S_u (kN/m)	Nominal Resistance S_y (kN/m)[1]	Overstrength S_u/S_y	
				test	average
1	3C-a 3C-c	5.76 6.09	4.13	1.39 1.47	1.43
2	Y7C1 Y7C2	11.71 12.95	8.69	1.35 1.49	1.42
3	Y8C1 Y8C2	10.59 10.13	7.17	1.48 1.41	1.44
4	Y9C1 Y9C2	9.54 9.34	6.48	1.47 1.44	1.46
5	2C-a 2C-b	10.93 10.70	7.53	1.45 1.42	1.44
6	1C-a 1C-b	6.32 6.24	4.53	1.39 1.38	1.39
7	Y4C1 6C-a 6C-b	15.56 17.11 17.48	12.54	1.24 1.36 1.39	1.33
8	Y5C1 5C-a 5C-b 11C-a 11C-b E114 E115 E116 E117 E118 E119 E120	15.19 14.47 14.21 16.15 15.99 14.18 14.01 12.77 13.61 12.47 13.08 12.86	10.58	1.44 1.37 1.34 1.53 1.51 1.34 1.32 1.21 1.29 1.18 1.24 1.22	1.33
9	Y6C1 Y6C2 4C-a 4C-b	13.15 13.44 11.84 12.29	8.89	1.48 1.51 1.33 1.38	1.43
10	Y1C1 Y1C2	20.41 18.98	13.93	1.46 1.36	1.41
11	Y2C1 Y2C2	17.32 17.97	12.01	1.44 1.50	1.47
12	Y3C1 Y3C2	16.24 15.64	10.69	1.52 1.46	1.49
				Average	**1.42**
				STD.DEV.	**0.0491**
				CoV.	**0.0024**

[1] **Nominal values from Table 3.11**

3.3.5 Seismic Force Resistance Factor Calibration

The base shear force, V, used for seismic design as defined by the equivalent static force method in Clause 4.1.8.11 of the 2005 NBCC *(NRCC, 2005)* can be calculated using Equation (3-14). There are two factors related to seismic design: the ductility-related force modification factor, R_d, and the overstrength-related force modification factor, R_o.

$$V = \frac{S(T_a)M_v I_E W}{R_d R_o} \tag{3-14}$$

where,

$S(T_a)$ = Design spectral acceleration
T_a = Fundamental lateral period of vibration of the building
M_v = Factor accounting for higher mode effects
I_E = Earthquake importance factor of structure (1.0 for normal buildings)
W = Weight of structure (dead load plus 25% snow load)
R_d = Ductility-related force modification factor
R_o = Overstrength-related force modification factor

3.3.5.1 Ductility-Related Force Modification Factor, R_d

The ductility-related force modification factor is a measure of the "fuse" element's ability to dissipate energy through inelastic deformation which, as previously mentioned, is an important aspect in seismic design. A relationship between ductility and the ductility-related force modification factor, R_d, was derived by Newmark and Hall *(1982)* based on the natural period of the structure as given in Equations (3-15), (3-16) and (3-17).

$$R_d = \mu \qquad \text{for T > 0.5s} \tag{3-15}$$

$$R_d = \sqrt{2\mu - 1} \qquad \text{for 0.1s < T < 0.5s} \tag{3-16}$$

$$R_d = 1 \qquad \text{for T < 0.03s} \tag{3-17}$$

where,

R_d = Ductility-related force modification factor

μ = Ductility of shear wall

T= Natural period of structure

Boudreault *(2005)* found that many light framed structures have a natural period less than 0.5 seconds. Therefore, the same assumption for low natural periods was used to determine the R_d value for steel sheathed shear walls and Equation (3-16) was used with the ductility values obtained from test results. Only walls with a length of 1220mm (4') or longer were considered. The short, 610x2440mm (2'x8'), shear walls were excluded because they had low ductility values due to high rotations.

Miranda and Bertero *(1994)* demonstrated that the ductility ratio is dependent on the loading protocol used for testing where reversed cyclic tests have higher ductility values than monotonic tests. Contrary to their findings, the monotonic tests of steel sheathed shear walls have a higher average ductility value than the reversed cyclic tests of approximately 4% which is not high enough to be a considerable difference (Tables 3.18 and 3.19). The average R_d accounting for both monotonic and reversed cyclic tests is 2.87. It is, therefore, recommended to use a conservative value of 2.5 for R_d which is consistent with the R_d used for the design of wood sheathed shear walls by Morello *(2009)* and as stated in AISI S213 *(2007)*.

Table 3.18 Ductility, μ, and R_d Values for Monotonic Tests

Group	Test Name	Ductility (μ)[1]	Ductility-Related Force Modification Factor (R_d)	
			test	average
1	3M-a	8.75	4.06	4.02
	3M-b	8.43	3.98	
2	Y7M1	3.05	2.26	2.20
	Y7M2	2.77	2.13	
3	Y8M1	2.92	2.20	2.40
	Y8M2	3.87	2.60	
4	Y9M1	2.56	2.03	2.42
	Y9M2	4.45	2.81	
5	2M-a	9.10	4.15	4.46
	2M-b	11.91	4.78	
6	1M-a	9.79	4.31	3.80
	1M-b	5.97	3.31	
	1M-c	7.70	3.79	
7	Y4M1	2.93	2.20	3.19
	Y4M2	2.61	2.05	
	6M-a	9.75	4.30	
	6M-b	7.63	3.78	
	13M-a	7.02	3.61	
8	Y5M1	2.41	1.95	3.47
	Y5M2	2.19	1.84	
	5M-a	7.61	3.77	
	5M-b	9.18	4.17	
	11M-a	8.34	3.96	
	11M-b	6.05	3.33	
	12M-a	13.78	5.15	
	15M-a	6.97	3.60	
9	Y6M1	2.69	2.09	3.36
	Y6M2	2.67	2.08	
	4M-a	11.19	4.62	
	4M-b	11.17	4.62	
10	Y1M1	2.33	1.91	1.73
	Y1M2	1.71	1.55	
11	Y2M1	3.07	2.27	2.20
	Y2M2	2.79	2.14	
12	Y3M1	2.20	1.85	1.95
	Y3M2	2.61	2.05	
			Average	**2.93**
			STD.DEV.	**0.90**
			CoV.	**0.8043**

[1] Ductility values obtained from Table 3.1

Table 3.19 Ductility, μ, and R_d Values for Reversed Cyclic Tests

Group	Test Name	Ductility (μ)[1]	Ductility-Related Force Modification Factor (R_d)	
			test	average
1	3C-a 3C-c	8.40 7.22	3.97 3.66	3.82
2	Y7C1 Y7C2	2.72 2.41	2.11 1.95	2.03
3	Y8C1 Y8C2	3.21 4.07	2.33 2.67	2.50
4	Y9C1 Y9C2	3.76 3.80	2.55 2.57	2.56
5	2C-a 2C-b	8.61 9.83	4.03 4.32	4.17
6	1C-a 1C-b	6.97 5.51	3.60 3.17	3.38
7	Y4C1 6C-a 6C-b	2.96 7.73 5.16	2.22 3.80 3.05	3.02
8	Y5C1 5C-a 5C-b 11C-a 11C-b E114 E115 E116 E117 E118 E119 E120	3.03 6.58 6.90 7.11 7.64 3.54 5.75 4.12 3.46 4.98 5.04 3.69	2.25 3.49 3.58 3.63 3.78 2.46 3.24 2.69 2.43 2.99 3.01 2.52	3.01
9	Y6C1 Y6C2 4C-a 4C-b	3.04 3.25 7.25 5.46	2.25 2.34 3.67 3.15	2.85
10	Y1C1 Y1C2	4.05 2.89	2.66 2.18	2.42
11	Y2C1 Y2C2	2.63 2.39	2.06 1.94	2.00
12	Y3C1 Y3C2	2.36 2.49	1.93 2.00	1.96
			Average	**2.81**
			STD.DEV.	**0.71**
			CoV.	**0.5066**

[1] Ductility values obtained from Tables 3.2 and 3.3

3.3.5.2 Overstrength-Related Force Modification Factor, R_o

As mentioned for limit states design, the factored resistance is required to be greater than the factored applied loads based on the critical load case provided by the 2005 NBCC (*NRCC, 2005*). However, the factored applied loads are often overestimated to achieve conservative values for design. Conversely, in capacity based design, for the energy dissipating element to deform inelastically, the factored loads should not be overestimated. Therefore, an overstrength factor is used in seismic design. Mitchell *et al. (2003)* proposed a formula for calculating the overstrength-related force modification factor as given in Equation (3-18).

$$R_o = R_{size} R_{\phi} R_{yield} R_{sh} R_{mech} \tag{3-18}$$

where,

R_{size} = overstrength due to restricted choices for sizes of components
$R_{\phi} = 1/\phi$, ($\phi = 0.7$)
R_{yield} = ratio of test yield strength to minimum specified yield strength
R_{sh} = overstrength due to development of strain hardening
R_{mech} = overstrength due to collapse mechanism

The formula includes five factors from which overstrength is expected. The size factor, R_{size}, for which a value of 1.05 is used, is considered because there are limitations on component sizes that are available which restricts designers in their choice of sizes for members. The second factor, R_{ϕ}, is used to consider nominal load values and not the factored loads as given in limit states design. The R_{ϕ} value is taken as the inverse of the material resistance factor, ϕ, which was recommended to be 0.7. The value for R_{yield} is taken as the average overstrength factor calculated for monotonic and reversed cyclic from Tables 3.16 and 3.17 which is 1.40.

The factor due to development of strain hardening, R_{sh}, is taken to be equal to unity because shear walls are not affected by steel's ability to undergo strain hardening. Finally, the overstrength resulting from the collapse mechanism, R_{mech}, is also taken as unity because the collapse mechanism for steel sheathed shear walls has not been established. A summary of the overstrength factors are given in Table 3.20.

The calculated overstrength-related force modification factor, R_o, was equal to 2.10 which was high when compared with other systems. A conservative value of 1.7 is recommended which is consistent with the R_o value for wood sheathed shear walls given in AISI S213 *(2007)*.

Table 3.20 Factors for the Calculation of the Overstrength-Related Force Modification Factor, R_o

	R_{size}	R_{ϕ}	R_{yield}	R_{sh}	R_{mech}	R_o
All Groups	1.05	1.43	1.40	1.00	1.00	2.10

3.3.6 Inelastic Drift Limit

The 2005 NBCC defines an inelastic drift limit of 2.5%. Upon examination of the test results, the monotonic tests exhibited higher drifts than the reversed cyclic tests (Table 3.21 and 3.22). An average drift limit of 2.56% including the monotonic and reversed cyclic tests was calculated based on a height of 2440mm (8'). Only the 1220mm (4') and longer walls were considered to determine the drift limit. The drift limit is the ratio of maximum displacement to height where the maximum displacement was taken as the displacement reached at 80% of the post-peak load. The average drift limit is higher than the value defined in the 2005 NBCC. For a more conservative value, a drift limit of 2% is proposed for steel sheathed shear walls.

Table 3.21 Drift Limit of Monotonic Tests

Group	Test Name	$\Delta_{0.8u}$ [1] (mm)	% Drift	
			test	average
1	3M-a	57.56	2.36	2.41
	3M-b	60.23	2.47	
2	Y7M1	62.68	2.57	2.60
	Y7M2	64.27	2.63	
3	Y8M1	53.95	2.21	2.70
	Y8M2	77.72	3.19	
4	Y9M1	58.03	2.38	2.77
	Y9M2	76.92	3.15	
5	2M-a	90.42	3.71	3.90
	2M-b	100.00	4.10	
6	1M-a	72.99	2.99	1.99
	1M-b	37.02	1.52	
	1M-c	35.73	1.46	
7	Y4M1	100.00	4.10	3.33
	Y4M2	84.23	3.45	
	6M-a	100.00	4.10	
	6M-b	62.99	2.58	
	13M-a	58.67	2.40	
8	Y5M1	72.41	2.97	2.56
	Y5M2	78.61	3.22	
	5M-a	52.60	2.16	
	5M-b	64.45	2.64	
	11M-a	55.26	2.26	
	11M-b	50.96	2.09	
	12M-a	69.81	2.86	
	15M-a	56.49	2.32	
9	Y6M1	79.33	3.25	2.99
	Y6M2	81.68	3.35	
	4M-a	67.57	2.77	
	4M-b	62.97	2.58	
10	Y1M1	71.06	2.91	2.47
	Y1M2	49.56	2.03	
11	Y2M1	58.98	2.42	2.58
	Y2M2	67.01	2.75	
12	Y3M1	58.11	2.38	2.18
	Y3M2	48.46	1.99	
			Average	**2.71**
			STD.DEV.	**0.5107**
			CoV.	**0.2609**

[1] Maximum drift displacements from Table 3.1

Table 3.22 Drift Limit of Reversed Cyclic Tests

Group	Test Name	$\Delta_{0.8u}$ (mm)	% Drift test	% Drift average
1	3C-a	62.75	2.57	2.31
	3C-c	49.80	2.04	
2	Y7C1	56.00	2.30	2.44
	Y7C2	63.10	2.59	
3	Y8C1	51.35	2.10	2.33
	Y8C2	62.50	2.56	
4	Y9C1	54.65	2.24	2.26
	Y9C2	55.55	2.28	
5	2C-a	83.00	3.40	3.58
	2C-b	91.90	3.77	
6	1C-a	45.80	1.88	1.70
	1C-b	37.40	1.53	
7	Y4C1	68.85	2.82	2.80
	6C-a	79.30	3.25	
	6C-b	56.70	2.32	
8	Y5C1	66.90	2.74	2.07
	5C-a	53.80	2.20	
	5C-b	59.50	2.44	
	11C-a	56.05	2.30	
	11C-b	49.10	2.01	
	E114	47.75	1.96	
	E115	47.15	1.93	
	E116	51.25	2.10	
	E117	51.40	2.11	
	E118	53.35	2.19	
	E119	37.75	1.55	
	E120	32.20	1.32	
9	Y6C1	65.70	2.69	2.51
	Y6C2	82.75	3.39	
	4C-a	51.10	2.09	
	4C-b	45.90	1.88	
10	Y1C1	60.55	2.48	2.49
	Y1C2	61.10	2.50	
11	Y2C1	57.30	2.35	2.26
	Y2C2	53.05	2.17	
12	Y3C1	52.30	2.14	2.07
	Y3C2	48.60	1.99	
			Average	**2.40**
			STD.DEV.	**0.4611**
			CoV.	**0.2126**

[1] Maximum drift displacements from Tables 3.2 and 3.3

CHAPTER 4 – DESIGN PROCEDURE

The NBCC currently does not have guidelines for the seismic design of steel sheathed CFS shear walls. A design procedure is outlined in this chapter for steel sheathed shear walls after having determined the pertinent parameters and factors in Chapter 3. The parameters and factors were verified through dynamic analysis of the model buildings that were designed herein.

4.1 Selection of Model Building

A model building was selected to be designed with the tested shear walls in order to verify the test-based seismic force modification factors recommended for steel sheathed shear walls. The building layout is provided by the NEESWood Project *(Cobeen et al., 2007)* (Figure 4.1), which has also been used in past research at McGill University by Comeau *(2008)*, Velchev *(2008)* and Morello *(2009)* in dynamic analysis. The NEESWood model was also selected as it is representative of low-rise to medium-rise residential buildings in Canada.

4.2 Description of Design

The design of the model building was carried out for Vancouver, British Columbia; located in a high seismicity zone on very dense soil to soft rock Site Class C. The buildings that were designed were two, three, four, five, six and seven storeys in height. The first storey is 3.66m (12') in height, while all other storeys are 3.05m (10') (Figure 4.2). The typical floor layout of the building is as given in Figure 4.1 with a footprint of 18.10m x 12.14m for a total floor area of 220m^2.

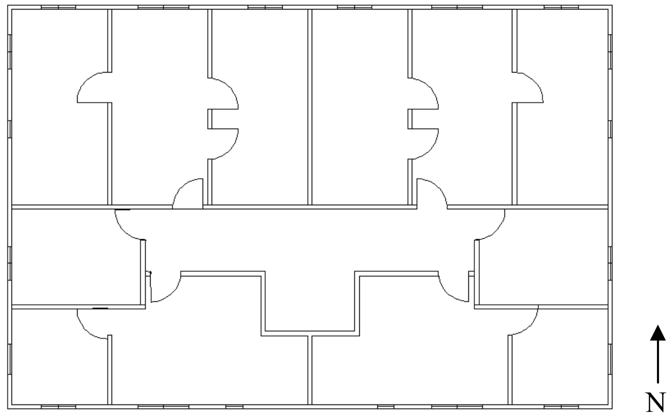


Figure 4.1 NEESWood Project Floor Layout *(Cobeen et al., 2007)*

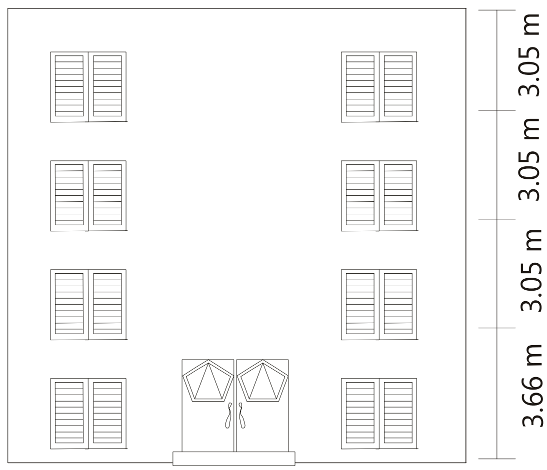


Figure 4.2 Elevation View of the Four Storey Model Building

4.2.1 Design Loads

As mentioned in Chapter 3, structures are to be designed to resist the factored applied loads based on the critical load case defined by the 2005 NBCC (NRCC, 2005). The load case that was deemed to be critical for the design of steel sheathed shear walls combines the effects of dead loads, earthquake load, live loads and snow loads (Equation (4-1)). A summary of all applied loads is given in Table 4.1.

$$W_f = 1.0D + 1.0E + 0.5L + 0.25S \tag{4-1}$$

where,

D = Specified dead load

E = Specified earthquake load

L = Specified live load

S = Specified snow load

4.2.1.1 Dead Loads

The dead load applied on the building was calculated using the weight of the floors and other elements within the building. The interior floor was chosen from the Canam group and the specified dead loads were determined for the Hambro D500 concrete type floor system (Figure 4.3). The specified dead load of the other elements were taken from the Handbook of Steel Construction *(CISC, 2004)*.

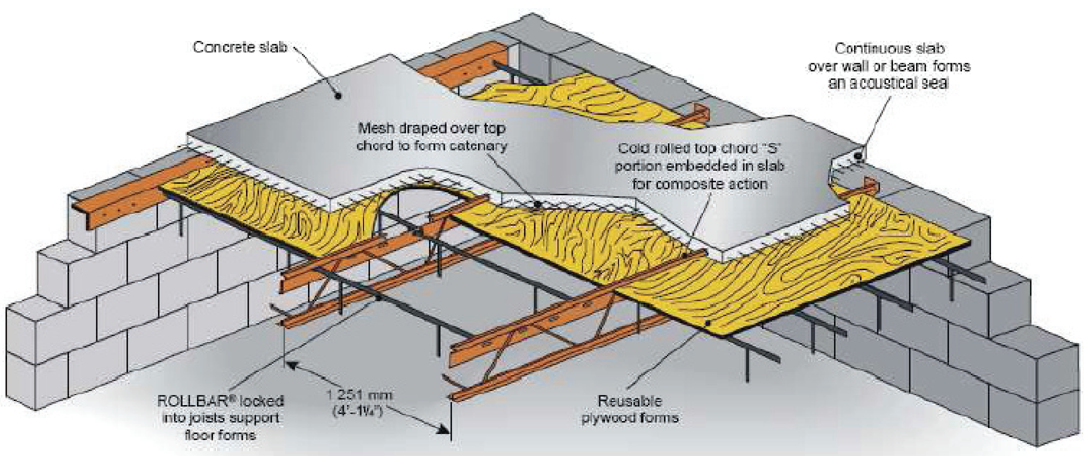


Figure 4.3 Hambro D500 Floor System *(Canam,2004)*

4.2.1.2 Snow Loads

The snow load was determined as prescribed by Clause 4.1.6.2 of the 2005 NBCC using the parameters for Vancouver as provided in Equation (4-2).

$$S = I_s\left[S_s\left(C_b C_w C_s C_a\right) + S_r\right] \tag{4-2}$$

where,

I_s = Importance factor for snow load, 1.0

S_s = 1/50 year ground snow load, 1.8kPa

C_b = Basic roof snow load factor, 0.8

C_w = Wind exposure factor, 1.0

C_s = Roof slope factor, 1.0

C_a = Shape factor, 1.0

S_r = 1/50 year associated rain load, 0.2kPa

4.2.1.3 Live Loads

The model building has more than one type of occupancy within a floor; mainly residential units and corridors. The live load was then determined based on the combination of different occupancy loads based on their respective areas. For residential type occupancy, a live load of 1.9kPa was used with an occupancy of 81.5%; for corridors and stairwells, a live load of 4.8kPa was used with an occupancy of 18.5%. The load combination provided an average live load of 2.44kPa.

Table 4.1 Description of Loads

Location	Description	Load (kPa)
Dead Loads		
Roof	Sheathing - 19mm (3/4”) plywood	0.10
	Insulation - 100mm blown fibre glass	0.04
	Ceiling - 12.5mm gypsum	0.10
	Joists - cold-formed steel at 600mm o/c	0.12
	Sprinkler system	0.03
	Roofing - 3ply+gravel	0.27
	Mechanical	0.03
	***D*=**	**0.69**
Floor	Walls - interior and exterior	0.72
	Flooring	0.19
	Concrete slab - Hambro	1.77
	Acoustic tile - 12mm	0.04
	Joists - cold-formed steel at 600mm o/c	0.12
	Mechanical	0.03
	***D*=**	**2.87**
Snow Loads		
Roof	***S*=**	**1.64**
Live Loads		
Floor	Residential (81.5% occupancy)	1.9
	Corridors and Stairwells (18.5% occupancy)	4.8
	***L*=**	**2.44**

4.3 Evaluation of Design Base Shear Force

The Equivalent Static Force Procedure was used to design the lateral system of the buildings as outlined in the 2005 NBCC Cl.4.1.8.11. The base shear force and its components are, therefore, calculated by Equation (4-3).

$$V = \frac{S(T_a)M_v I_E W}{R_d R_o} \tag{4-3}$$

where V cannot be less than

$$V = \frac{S(2.0)M_v I_E W}{R_d R_o} \tag{4-4}$$

and V should not exceed

$$V = \frac{2}{3}\frac{S(0.2)M_v I_E W}{R_d R_o} \tag{4-5}$$

where,

$S(T_a)$ = Design spectral acceleration

T_a = Fundamental lateral period of vibration of the building

M_v = Factor accounting for higher mode effects

I_E = Earthquake importance factor of structure (1.0 for normal buildings)

W = Weight of structure (dead load plus 25% snow load)

R_d = Ductility-related force modification factor

R_o = Overstrength-related force modification factor

To calculate the base shear force, some parameters had to first be determined. The weight of the structure is taken as the dead load and 25% of the snow load from Table 4.1. The snow load was only included in the weight of the uppermost storey. To determine the design spectral acceleration, S_a, the natural period of each building was calculated according to Equation (4-6) which is applicable to shear walls (Table 4.2). The NBCC allows a natural period of up to $2T_a$ if verification by means of dynamic analysis is possible. The values for the design spectral acceleration were interpolated based on the uniform hazard spectrum (UHS) for Vancouver as given in Table 4.3 and Figure 4.4.

$$T_a = 0.05 h_n^{3/4} \tag{4-6}$$

where,

T_a = Fundamental lateral period of vibration of the building, (s)

h_n= total height of building, (m)

For periods greater than one second, which was the case for the seven storey building, a factor accounting for higher mode effects, M_v, was included. The higher mode factor was interpolated based on the values given in Table 4.1.8.11 of the 2005 NBCC *(NRCC, 2005)* for shear walls. An importance factor is included in the calculation of the base shear force and was taken as unity for normal buildings. Finally, the R_d and R_o factors were 2.5 and 1.7, respectively, as recommended in Chapter 3.

Table 4.2 Natural Period and Spectral Acceleration of Model Buildings

Storeys	Height (m)	NBCC T_a (s)	Design Period, $2T_a$ (s)	$S_a(2T)$	M_v	Ruaumoko Period, T (s)
2	6.71	0.208	0.417	0.72	1.0	0.435
3	9.76	0.276	0.552	0.61	1.0	0.681
4	12.81	0.339	0.677	0.53	1.0	0.821
5	15.86	0.397	0.795	0.46	1.0	1.007
6	18.91	0.453	0.907	0.39	1.0	1.181
7	21.96	0.507	1.014	0.33	1.0029	1.374

Table 4.3 Uniform Hazard Spectrum for Vancouver as given in the 2005 NBCC

T(s)	S(Ta)
0.2	0.94
0.5	0.64
1.0	0.33
2.0	0.17

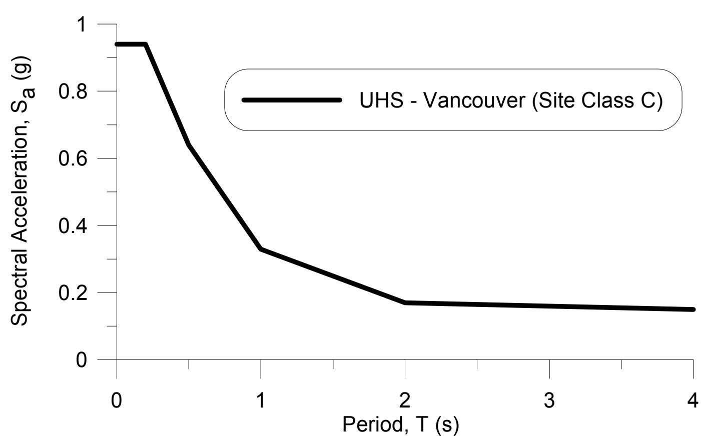


Figure 4.4 Uniform Hazard Spectrum for Vancouver

The acceleration- and velocity-based factors, F_a and F_v, for Site Class C were equal to 1.0 as per Tables 4.1.8.4.B and 4.1.8.4.C of the 2005 NBCC. Based on the limits for the calculation of base shear force, the design base shear force for each building was calculated and is presented in Table 4.4.

Table 4.4 Determination of the Design Base Shear Force

Storeys	V	V_{min}	V_{max}	V_{design}
2	148.4	34.9	128.6	128.6
3	214.9	60.1	221.6	214.9
4	266.2	85.3	314.6	266.2
5	297.4	110.6	407.6	297.4
6	309.8	135.8	500.6	309.8
7	311.3	161.5	595.3	311.3

The design base shear force was distributed to each storey level according to Equation (4-7).

$$F_x = \frac{(V - F_t)W_x h_x}{\sum_{i=1}^{n} W_i h_i} \tag{4-7}$$

where,

F_x = base shear applied at each storey

W_x = seismic weight at storey under consideration

h_x = height of storey under consideration

F_t = additional load at roof level

$= 0.07T_aV \leq 0.25V$

$= 0$ for $T_a<0.7s$

$\sum_{i=1}^{n} W_i h_i$ = sum of all seismic weight multiplied by each storey height

In addition to the base shear force, a lateral notional load (0.5% gravity) and torsional effects were included. Notional loads were calculated based on the gravity load applied on the area of a given storey (Table 4.5)

Table 4.5 Notional Loads

Notional Loads (kN)	
Roof	0.005(*D*+0.25*S*)*A* = 1.21
Floor	0.005(*D*+*L*)*A* = 4.49

The torsional effects are based on the eccentricity within the building and the dimensions of its layout (Equation (4-8)). It was assumed that the building is symmetric with its centre of rigidity coinciding with its centre of mass, therefore, the eccentricity, e_x, was taken as zero. Since the shear walls are distributed along the layout of the building with difference eccentricities, the accidental torsional force was taken as 10% of the base shear force (Equation (4-9)) which assumes the maximum eccentricity at each end of the building's layout (Figure 4.5).

$$T_x = F_x(e_x + 0.10D_{nx}) \tag{4-8}$$

$$F_{tor} = \frac{T_x}{D_{nx}} = \frac{F_x(e_x + 0.10D_{nx})}{D_{nx}} = 0.1F_x \tag{4-9}$$

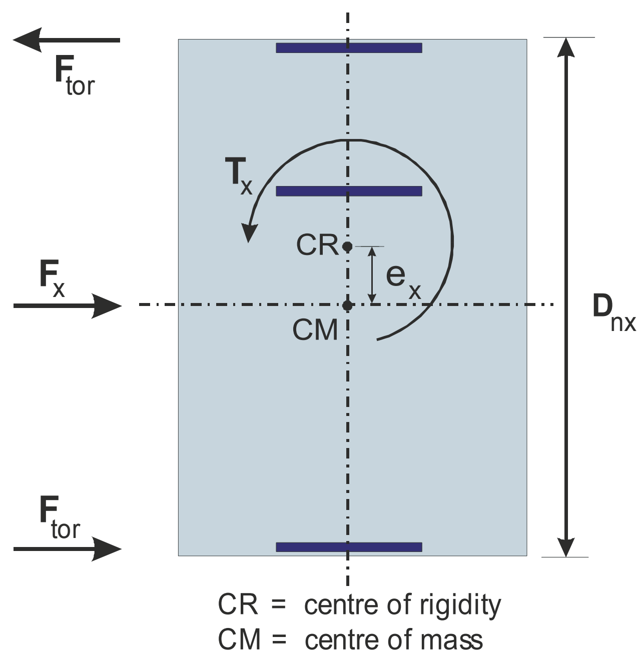


Figure 4.5 Torsional Effects *(Velchev, 2008)*

The four storey building is used as an example for calculations and design throughout the text. The values and details for all other model buildings are given in Appendix G.

A summary of seismic weights for the four-storey building is given in Table 4.6. The distribution of the design base shear and its components for the four-storey building is given in Table 4.7. The portion, F_t, used in the calculation of the base shear distribution was taken as zero for the four-storey building since the period of vibration was less than 0.7s.

Table 4.6 Seismic Weight Distribution for Four-Storey Building

Level	Storey Height (m)	Area (m^2)	Dead (kPa)	Snow (kPa)	Live (kPa)	Seismic Weight (kN)	Cumulative Seismic Weight (kN)
Roof	-	220	0.69	1.64	-	241.71	241.71
4	3.05	220	2.87	-	2.44	630.64	872.34
3	3.05	220	2.87	-	2.44	630.64	1502.98
2	3.05	220	2.87	-	2.44	630.64	2133.62
1	3.66	220	2.87	-	2.44	-	2133.62

Table 4.7 Design Base Shear Distribution for Four-Storey Building

Storey	W_i (kN)	h_i (m)	W_i x h_i	F_x (kN)	T_x (kN)	N_x (kN)	Vf_x (kN)
Roof	241.7	12.81	3096	52.2	5.22	1.21	58.6
4	630.6	9.76	6155	103.7	10.37	4.49	118.6
3	630.6	6.71	4232	71.3	7.13	4.49	83.0
2	630.6	3.66	2308	38.9	3.89	4.49	47.3
1	-	-	-	-	-	-	-
Σ			15791	266.2			307.5

4.4 Design of Model and Selection of Shear Wall

After having calculated the distributed shear force, it was necessary to determine the size, configuration and number of shear walls required for each storey to resist the applied loads. The resistance of the seismic force resisting system (SFRS) is the sum of the resistance of all the individual components that contribute to shear resistance (Equation (4-10)). The design shear resistance of a shear wall was calculated based on its length and using the nominal strength of the given shear wall from Table 3.11 factored with the resistance factor, ϕ, of 0.7 (Equation (4-11)). It was assumed that wall segments in each storey were of equal length.

$$S_r = \sum S_{rs} \tag{4-10}$$

$$S_{rs} = \phi S_y L \tag{4-11}$$

where,

S_r = Factored shear resistance of shear wall

S_{rs} =Factored shear resistance of shear wall segment

$\phi = 0.7$

S_y = Nominal shear strength for shear wall segment

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

The seismic design procedure was carried out for the North-South direction of the model building because it was assumed that the floors consist of a rigid floor system and that the effects of seismic loading are the same in both loading directions. In the N-S direction, there is approximately 45.5m (150’) of wall length available for the placement of shear walls. The available wall length accounts for windows and doors to be placed. Therefore, a maximum of approximately 37 1220mm (4’) long shear walls can be placed in any given storey. However, it is not desirable to have the maximum number of shear walls as it limits the size and location of open space. In addition, fewer shear walls is more economical. Therefore, the design approach was based on minimizing the number of shear walls.

The storey with the highest shear force was designed first, which was the bottom storey. In low to medium rise structures, it is unlikely to use framing members with a thickness less than 1.09mm (0.043"), consequently, only shear walls with 1.09mm (0.043") framing were included in the design. It is preferable to use the same sheathing throughout the building while only varying the fastener spacing to avoid confusion at the construction site.

For the four-storey building, a sheathing thickness of 0.84mm (0.033") was selected. Initially, it was desirable to maintain approximately the same number of shear walls on each storey to simplify modeling. In past research, a single shear wall bay from the building was modeled, therefore, it was important to have the same number of shear walls on each storey *(Comeau (2008), Velchev (2008), Morello (2009))*. At the bottom storey, a fastener spacing of 50mm (2") was selected to minimize the number of shear walls. The fastener spacing was gradually increased up to 150mm (6") at the uppermost storey (Table 4.8). However, it was difficult to obtain the same number of shear wall segments in all storeys even with the varied fastener spacing due to the decrease in shear force distribution at the higher storey levels.

Designers should not hesitate to use the 75mm fastener spacing however it was not used in design because its nominal strength was interpolated from other values and would be complicated to model in Hysteres *(Carr, 2008)* due to lack of test data. Therefore, the design approach only considered fastener spacings for which experimental test data is available. The majority of walls tested were 1220mm (4') in length, therefore, shear wall segments of 1220mm (4') in length were used in the design of buildings. As well, it is common to obtain coils of steel that are 1220mm (4') in width.

Table 4.8 Initial Design of Four-Storey Building

Level	Shear (kN)	Framing (mm)	Sheathing (mm)	Fastener Spacing (mm)	Nominal Strength S_y (kN/m)	Design Strength S_r (kN/m)	Min Length Required (m)	Required # walls (1220mm long)	Rounded # walls (1220mm long)
4	58.6	1.09	0.84	150	10.69	7.48	7.83	6.42	7
3	177.2	1.09	0.84	150	10.69	7.48	23.69	19.42	20
2	260.2	1.09	0.84	100	12.01	8.41	30.94	25.36	26
1	307.5	1.09	0.84	50	13.94	9.75	31.52	25.84	26

4.4.1 Building Irregularity

After a preliminary verification of the design using dynamic analysis, which is discussed in Chapter 5, it was deemed necessary to consider the irregularity of each building as prescribed by the NBCC. Even though the design approach indicated that the capacity was sufficient by the number of shear walls, there were large drifts obtained during dynamic analysis that were attributed to the considerable change in shear wall length from one storey to another. There were three main types of irregularity that were considered which were related to stiffness, geometry and capacity. However, the Equivalent Static Force Procedure still applied for analysis as the buildings met the conditions of Cl.4.1.8.7 of the 2005 NBCC where the building height was less than 60m and the fundamental lateral period was less than two seconds. The NBCC describes the applicable types of irregularity as:

1. Type 1: Vertical Stiffness Irregularity occurs when the lateral stiffness in a storey is less than 70% of that of an adjacent storey or less than 80% of the average stiffness of three storeys above or below.
2. Type 3: Vertical Geometry Irregularity occurs when the horizontal dimension of the (SFRS), or shear wall in this case, is more than 130% of that of an adjacent storey.
3. Type 6: Discontinuity in Capacity – Weak Storey occurs when the shear strength of a storey is less than the storey above.

As a result, the design approach was adjusted to account for irregularity. The number of shear wall segments was increased to meet the length criterion even though the shear resistance was sufficient with fewer wall segments. For the stiffness criterion to be met, the fastener spacing was decreased to reduce the difference in stiffness from one storey to another. Finally, in some cases, the bottom storey had a lower strength capacity due to the change in height from 3.66m (12') to 3.05m (10') in the storeys above in which case the number of shear wall segments was increased. The modified design for the four-storey building is presented in Table 4.9.

Table 4.9 Design of Four-Storey Building Adjusted for Irregularity

Level	Shear (kN)	Framing (mm)	Sheathing (mm)	Fastener Spacing (mm)	Nominal Strength S_y (kN/m)	Design Strength S_r (kN/m)	Min Length Required (m)	Required # walls (1220mm long)	Rounded # walls[1] (1220 mm long)
4	58.6	1.09	0.84	50	13.94	9.75	6.01	4.93	14
3	177.2	1.09	0.84	50	13.94	9.75	18.17	14.89	17
2	260.2	1.09	0.84	50	13.94	9.75	26.67	21.86	22
1	307.5	1.09	0.84	50	13.94	9.75	31.52	25.84	26

[1] Number of walls accounts for building irregularity

4.5 Capacity Based Design of Chord Studs

The compression force applied on the chord stud results from two components: the compression force due to the lateral shear force moment couple and the gravity load carried by the tributary area of the chord stud. The full storey height was used in calculating the compression load because the lateral force is applied on to the top of the rigid floor. Therefore, the full storey height was more appropriate to calculate the overturning moment caused by the lateral force. An overstrength factor of 1.40 was applied to the compression force component due to shear as determined in Chapter 3. As for the gravity load, it was assumed that all the walls within the building shared the gravity load, including those perpendicular to the loading direction as well as non shear walls.

After calculating the compression force that is applied on the chord stud, the size and number of chord studs was determined. The capacity of the double chord stud (DCS) was calculated as prescribed by CSA-S136 following the procedure provided by Hikita *(2006)*. The approach used for the design of chord studs of steel sheathed shear walls was more conservative and was decided that the effective length factor for the chord studs would be 1.0 instead of 0.9 as used by Hikita *(2006)*. In addition, the weak axis of the double chord stud was assumed to be braced by means of three rows of bridging which reduced the unbraced length to one quarter of the height. A summary of the nominal compression capacity of double chord studs for each thickness is given in Table 4.10.

Table 4.10 Nominal Capacity of Double Chord Studs[1]

Nominal Thickness		Area	Compression Capacity, P_n
in	**mm**	**mm^2**	**kN**
0.043”	1.09	417	56.6
0.054”	1.37	541	100.0
0.068”	1.73	670	128.8
0.097”	2.46	923	176.4

[1] Nominal dimensions of stud: 92.1mm (3-5/8”) web, 41.3mm (1-5/8”) flange, and 12.7mm (1/2”) lip

The minimum number of studs used was two which is equivalent to one double chord stud. The maximum number, however, was set to four studs because as the number of chord studs increases the stiffness of the shear wall increases as well causing the SFRS to be rigid. Additionally, an upper limit was placed on the number of chord studs used in design as it is inefficient to have several studs connected; instead, a thicker chord stud would be preferable. However, six studs were required (or three double chord studs) to be placed in the seven-storey building to obtain sufficient capacity to resist the applied gravity and lateral loads. In a building of such height it would not be ideal to use CFS compression members as HSS members would be more efficient. To be consistent with the design approach of the shorter buildings, a higher number of studs were used to illustrate the general design approach of compression members. The thickness of chord stud was selected based on minimizing the number of chord studs required to resist the applied loads.

The number of shear walls on each storey was different which affects the mode in which loads are transferred. Shear walls transfer both shear and gravity loads from one storey to another. However, since the shear walls do not align due to the varying number of shear walls on each storey, the load path was not as direct. A conservative scenario was assumed where both the shear and gravity loads are transferred from one storey to the other because it was found that the gravity component was a fraction of the shear load (Figure 4.6).

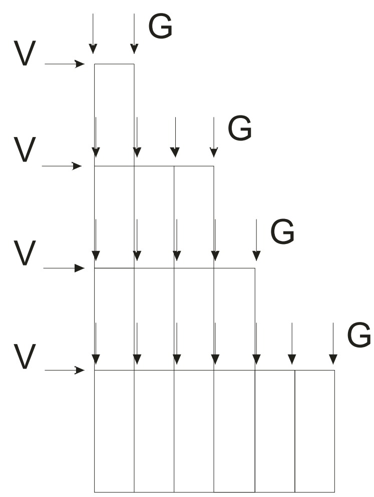


Figure 4.6 Shear Wall Load Distribution Schematic

To calculate the compression force on each stud, its tributary area had to be determined. Based on the total length of walls available in the model building on a given storey and assuming a stud spacing of 610mm (2'), the tributary area for each stud was calculated. Realistically, a stud spacing of 300mm (12") would be used in a building but 610mm (2') was chosen as a conservative approach. The larger stud spacing would result in a larger tributary area and, therefore, a larger compression force due to gravity. A stud spacing of 610mm (2') also coincides with the stud spacing used in shear wall testing. The available wall length was measured geometrically in the N-S and E-W directions with allocated space for doors and windows as given in Figure 4.1 for an approximate length of 90m. The resulting tributary area for a single stud was estimated to be 1.48m^2 (15.9ft^2). It was also assumed that the tributary area for each chord stud did not increase with an increased number of chord studs.

The load case chosen for analysis of seismic loads does not apply a large factor to live loads (See Equation (4-1)). The live load component of the load was not the controlling load as a result. Realistically, the live load component would play an important role in gravity design as there would be wind loads as well. Only one load case was considered for the design of the model building where seismic loads were included. Therefore, to maximize the live load component, a live load reduction factor was not applied to determine the compression due to gravity in the design of chord studs.

The design of double chord studs of the four-storey model building is summarized in Table 4.11 where the compression force due to shear and gravity are calculated using Equations (4-12) and (4-13). The compression force due to shear was calculated based on the nominal shear resistance of the shear wall segments and amplified using the overstrength factor of 1.4.

$$C_s = \frac{S_y h}{b} b \cdot overstrength \qquad (4\text{-}12)$$

$$C_g = (D + 0.5L + 0.25S) \cdot T.A._{stud} \qquad (4\text{-}13)$$

where,

C_s = compression force due to shear (kN)

C_g = compression force due to gravity (kN)

S_y = nominal shear resistance of wall segment (kN/m)

h = height of full storey (m)

b = width of shear wall segment (m)

overstrength = overstrength factor (R_{yield} = 1.4) (See Section 3.3.5.2)

$T.A._{stud}$ = tributary area of stud

Table 4.11 Design of Double Chord Studs of Four-Storey Building

Storey	Compression – shear (kN)	Compression – gravity (kN)	Compression – total (kN)	DCS Thickness (mm)	# DCS	DCS Pn (kN)	Area DCS (mm^2)
4	59.50	1.63	61	1.37	1	100.0	541
3	59.50	6.07	127	1.73	1	128.8	670
2	59.50	6.07	192	1.73	1.5	193.2	1006
1	71.40	6.07	270	2.46	2	352.8	1846

4.6 Estimation of Inelastic Drift

The AISI S213 Standard provides an equation for estimating the elastic drift of CFS frame shear walls (Equation (4-14)). The inelastic drift, Δ_{mx}, is calculated by multiplying the elastic drift by the ductility and overstrength force modification factors (Equation (4-15)). The estimated inelastic drift was compared with the drift limit of 2% proposed in Chapter 3. In all cases, the inelastic drift was less than the maximum allowable drift limit for steel sheathed shear walls (Table 4.12).

$$\Delta = \frac{2vh^3}{3E_s A_c b} + \omega_1 \omega_2 \frac{vh}{\rho G t_{sheathing}} + \omega_1^{5/4} \omega_2 \omega_3 \omega_4 \left(\frac{v}{\beta} \right) + \frac{h}{b} \delta_v \tag{4-14}$$

$$\Delta_{mx} = R_d R_o \Delta \tag{4-15}$$

where,

A_c = Gross cross-sectional area of chord member (mm^2)

b = width of the shear wall (mm)

E_s = Modulus of Elasticity of steel, 203000 MPa

G = Shear modulus of sheathing material, 78000 MPa

h = wall height (mm)

s = maximum fastener spacing at panel edges (mm)

$t_{sheathing}$ = nominal panel thickness (mm)

t_{stud} = framing designation thickness (mm)

v = shear demand (V/b) (N/mm)

V = total lateral load applied to the shear wall (N)

β = 1.45 ($t_{sheathing}$ /0.457) for sheet steel (N/mm$^{1.5}$)

δ_v = vertical deformation of anchorage/attachment details (mm)

ρ = 0.075($t_{sheathing}$ /0.457) for sheet steel

ω_1 = s/152.4 (for s in mm)

ω_2 = 0.838/ t_{stud}

$$\omega_3 = \sqrt{\frac{\left(h/b\right)}{2}}$$

$$\omega_4 = \sqrt{\frac{227.5}{F_y}}$$ for F_y in MPa for sheet steel

Δ_{mx} = factored inelastic drift

The inter-storey drift was based on the shear wall height as opposed to the full storey height that included a 300mm (12”) rigid floor. During dynamic testing of two-storey wood-sheathed shear walls on a shake table by Morello *(2009)*, it was observed that the floor did not undergo any significant shear deformations (*Shamim et al., 2010*). It was assumed that the rigid floor in buildings using steel sheathed shear walls would perform in a similar manner to the wood sheathed shear wall buildings.

The area of chord stud used to determine the elastic drift was the area determined by the design of double chord studs. The value used for the deformation of anchorage was obtained from Simpson Strong-Tie *(2008)*; a maximum deflection of 2.44mm (0.096”) for the S/HD10S hold-downs used for all shear wall tests was used.

4.7 P-Δ Effects

The stability factor, θ_x, at each level is calculated as given in Equation (4-16). The stability factor is defined by the NBCC as the additional load due to second order effects. The stability factor was checked in all model buildings and was found to be less than 10% in all cases (Table 4.12). Therefore, it was not necessary to include P-Δ effects in design.

$$\theta_x = \frac{\sum_{i=1}^{n} W_i}{R_o \sum_{i=1}^{n} F_i} \frac{\Delta_{mx}}{h_s} \tag{4-16}$$

where,

θ_x = Stability factor of storey under consideration

W_i = Seismic weight of storey

Δ_{mx} = Factored inelastic drift

R_o = Overstrength-related force modification factor

F_i = Seismic force at storey

h_s = Inter-storey height

Table 4.12 Inter-storey Drift and Stability Factor of Four-Storey Building

Level	h_s (mm)	Δ (mm)	Δ_{mx} (mm)	Drift (%)	θ_x
4	2750	5.9	24.9	0.91	0.022
3	2750	6.2	26.5	0.96	0.028
2	2750	6.1	25.8	0.94	0.032
1	3360	7.3	31.0	0.92	0.038

The P-Δ load was calculated using the live load reduction factor (*LLRF*) given in Equation (4-17) and was only applicable if the tributary area was greater than $20m^2$ ($215ft^2$). The *LLRF* was included as it was not necessary to consider a higher load which was the case for the design of chord studs. The tributary area for P-Δ effect is the total area of the storey not including the tributary area of the shear walls. The *LLRF* did not apply to the top floor since only snow loads were applied on the roof. A summary of the P-Δ loads for the four-storey building is given in Table 4.13.

$$LLRF = 0.3 + \sqrt{\frac{9.8}{A}} \tag{4-17}$$

where,

LLRF = live load reduction factor (< 1.0)

A = cumulative tributary area of shear wall including upper storeys

Table 4.13 P-Δ Loads for Four-Storey Building

Level	P-Δ Area (m^2)	Cumulative Area (m^2)	*LLRF*	Reduced live load (kPa)	Gravity Load[1] (kPa)	P_x (kN)
4	178.2	178.2	0.53	-	1.10	196.0
3	169.3	347.4	0.47	1.14	3.44	582.4
2	154.4	501.8	0.44	1.07	3.41	526.0
1	142.5	644.4	0.42	1.03	3.39	482.7

[1] Calculated using D+0.25S+0.5Lx$LLRF$

CHAPTER 5 – DYNAMIC ANALYSIS

Dynamic analysis is an integral part of evaluating the seismic performance of structures in order to validate the design procedure outlined in Chapter 4. FEMA P695 *(FEMA, 2009)* outlines a methodology to assess the seismic performance factors through analytical processes. The methodology addresses the selection of a model building, input ground motion records and their scaling, incremental data analysis *(Vamvatsikos and Cornell, 2002)*, fragility curves based on collapse probability, validation of *R*-values and acceptable height limit. A dynamic analysis software was used to model the non-linear inelastic behaviour of steel sheathed shear walls. The analysis accounts for important characteristics of behaviour such as strength and stiffness. The software, Ruaumoko *(Carr, 2008)*, was also used to subject the model buildings to ground motion records that were recommended by FEMA P695 as well as some synthetic records.

The ground motion records were selected in accordance with the recommendations of the FEMA P695. The records were adjusted according to the uniform hazard spectrum of Vancouver where the model building was assumed to be located. Each ground motion record to which the buildings were subjected was scaled to determine the intensity that would cause failure in each building as part of an incremental dynamic analysis (IDA). Failure was determined as drifts exceeding the allowable limit of 2% as recommended in Chapter 3.

The performance of each building was based on its collapse probability, which signifies the probability of failure based on the number of ground motion records for a given scaling factor that cause the building to fail. Fragility curves are created using the collapse probability at each scaling factor. For each building to perform adequately, the collapse probability had to meet tabulated allowable criteria that account for uncertainty, as listed in FEMA P695.

5.1 Calibration of Hysteresis

The Stewart Model *(Stewart, 1987)* was chosen to simulate the hysteretic behaviour of the reversed cyclic tests of steel sheathed shear walls. Boudreault *(2005)* examined many models and found that the Stewart model matches the hysteretic behaviour of wood sheathed shear walls. Due to the similarity in behaviour of steel sheathed shear walls and wood sheathed shear walls, the Stewart Model was deemed appropriate for hysteresis matching. However, one drawback of the model is that it does not reproduce the strength degradation observed during testing. The Stewart Model was available in the HYSTERES software *(Carr, 2008)* that was used to match the hysteretic element to experimental results. The model parameters were calibrated using experimental data from reversed cyclic tests. Based on material test results, the tensile stress ratio of experimental test results and nominal values was consistent with the R_y and R_t values listed in AISI S213 *(2007)* (Section 2.9). Therefore, experimental results used for matching were not calibrated for thickness and tensile stress because it was assumed that the walls would perform in a similar manner to that of the tests and not to the nominal values.

The modeling of the hysteretic behaviour was based on stiffness and strength parameters. The initial stiffness of the shear wall, k_e, degraded as the wall was pushed past the yield point (Figure 5.1). The strength degradation began once the shear wall was pushed into inelastic displacements after reaching its ultimate resistance, however this was not captured in the hysteretic model. Degradation was visible in the stiffness and strength of subsequent cycles. The loss of stiffness and strength was due to fasteners that became loose and enlarged connection holes due to bearing of the fasteners on the steel sheathing. On the return cycle the wall was only able to resist the loads after a certain displacement was reached. Due to the slotting of the connections, the reserve strength on the return cycle is referred to as pinching where the fasteners regained bearing contact with the sheathing.

Each phase of the hysteretic behaviour was modeled with a parameter (Figure 5.1). The parameter k_o represents the initial stiffness, F_u represents the ultimate strength of the wall, F_y represents the yield strength of the wall, and F_i represents the intercept force. Other factors include the unloading stiffness factor, P_{UNL}, the tri-linear factor beyond ultimate force, F_{TRI}, the softening factor, β, and the pinch power factor, α *(Carr, 2008).*

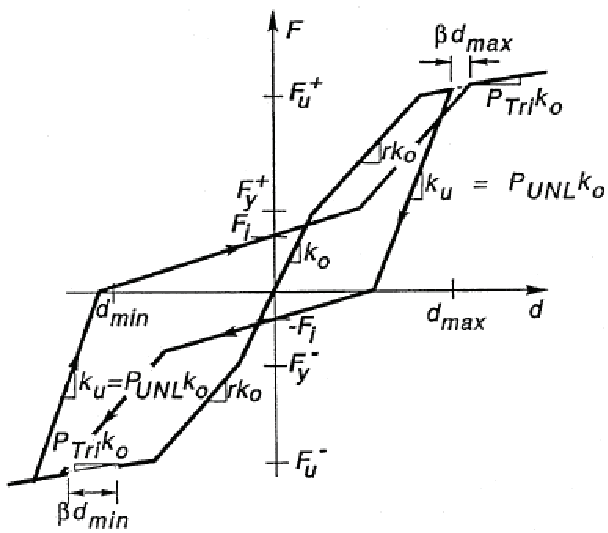


Figure 5.1 Parameters of the Stewart Element ***(Carr, 2008)***

The stiffness, ultimate and yield strengths were initially taken from the experimental results before being modified. The model was inspected visually by comparing the strength and the energy dissipation of the experimental and modeled hysteresis. The model was only compared up to the post peak displacement that corresponded to 80% of the strength as that was determined to be the failing point.

The calibration process was iterative and all specimens with the same configuration were compared to obtain a model that was applicable to all experimental tests. A comparison of an experimental hysteresis with the Stewart model is presented in Figure 5.2 for shear wall specimens constructed of 1.09mm (0.043") framing, 0.84mm (0.033") sheathing and 50mm (2") fastener spacing.

The energy dissipation of the model and the experimental hysteresis were closely matched as well (Figure 5.3). The parameters used for the calibration are listed in Table 5.1. Hysteresis matching and parameters for all the configurations used in the design of the model buildings are presented in Appendix E.

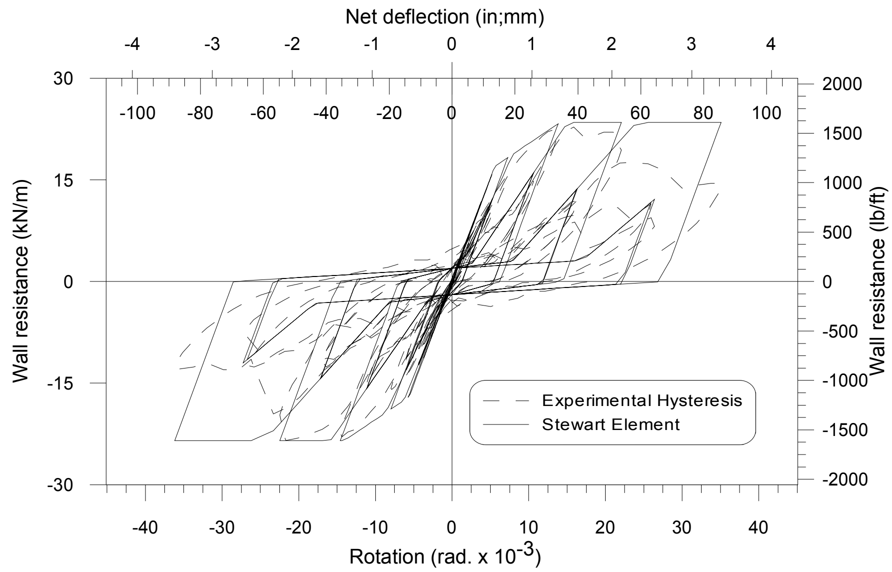


Figure 5.2 Calibration of Stewart Hysteretic Element using HYSTERES for 1.09mm Framing, 0.84mm Sheathing and 50mm Fastener Spacing

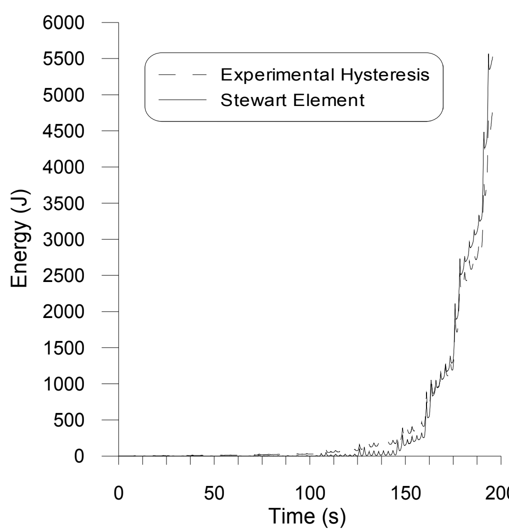


Figure 5.3 Energy Dissipation of Stewart Model and Experimental Hysteresis for 1.09mm Framing, 0.84mm Sheathing and 50mm Fastener Spacing

Table 5.1 Description of Parameters 0.84mm Sheathing, 50mm Fastener Spacing

k_o	1.25 kN/mm
R_f	0.25
F_{x+}	17.0 kN
F_{x-}	-17.0 kN
F_u	23.5 kN
F_i	1.95 kN
P_{tri}	0.0
P_{UNL}	1.0
gap^+	0.0
gap^-	0.0
β	1.09
α	0.60

5.2 Ruaumoko

Ruaumoko is a software package developed by Carr *(2008)* that was used for the inelastic dynamic modeling and analysis. The software has previously been used for the modeling and analysis of strap braced shear walls *(Comeau, (2008) and Velchev, (2008))* and of wood sheathed shear walls *(Morello (2009))*.

A single braced bay of the design building was modeled in Ruaumoko by Comeau *(2008)*, Velchev *(2008)* and Morello *(2009)* as the same number of shear walls was used on each storey. However, due to the variation of shear wall length on each storey for the design of steel sheathed shear walls (Figure 4.6), it was preferable to model the entire building as a two-dimensional model.

The building was simulated as a stick model in Ruaumoko without taking into consideration the exact location of each shear wall. A lumped mass representing the seismic weight was applied to each node at each storey level. Each floor was represented as a spring element with the parameters of the Stewart hysteretic element (Section 5.1) which is the energy dissipating element (Figure 5.4). An assumption was made that each floor behaves rigidly. A lean-on P-Δ column was represented by a stick with infinite axial stiffness and its displacement relied on the primary storey element. The gravity loads contributing to the P-Δ effect were applied at each corresponding node of the P-Δ column. The seismic weight and P-Δ loads for each storey were as calculated in Sections 4.3 and 4.7, respectively.

Each model building was subjected to 45 ground motion records in one direction to evaluate its performance with a Rayleigh damping of 5%. The input code for the four-storey building in Ruaumoko is available in Appendix F.

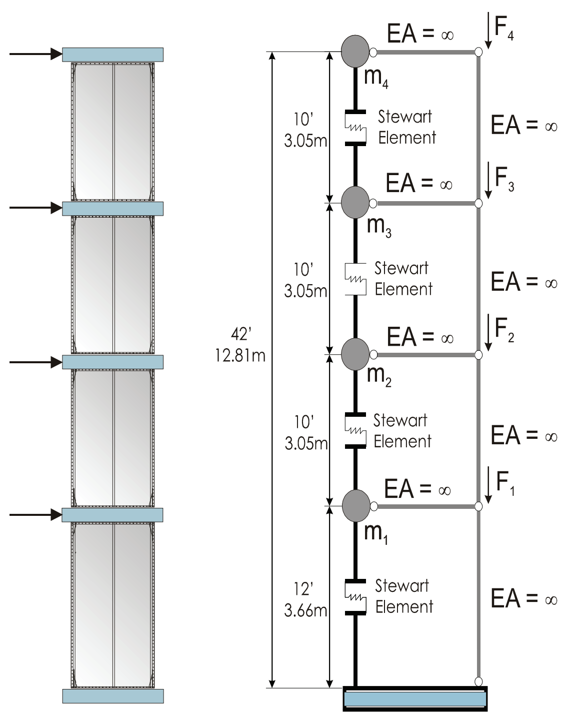


Figure 5.4 Stick Model of Building and P-Δ Column

5.2.1 Parameter Adjustments

It was decided to account for all shear walls within the storey in the Ruaumoko model, therefore, a method for modifying the spring element parameters had to be established. Morello *(2009)* found that the strength and stiffness vary directly with any change in length of the frame. However, the stiffness varied inversely with any increase in height while the strength remained the same. The relationship of variation of strength and stiffness to length and height can be compared to a cantilever beam with the height of the frame being the length of the beam and the length of the wall being the depth of the beam (Figure 5.5). An increase in depth would result in a stiffer section; however, a longer section would result in a decrease in stiffness. An increase in depth also increases resistance. Therefore, only the stiffness-related parameters were adjusted for any change in height and both the stiffness-related and strength-related parameters were adjusted for change in length.

The strength parameters (F_{x+}, F_{x-}, F_u, F_i) in Table 5.1 were multiplied by the number of shear walls on each storey and the length, which was assumed to be 1220mm (4') for all shear walls. The stiffness parameter, k_o, was also multiplied by the number of shear walls and the length. It was also adjusted based on the shear storey height not the full storey height. The bottom storey is assumed to be 3.66m (12') with a 300mm (12") floor for a shear wall height of 3.36m (11') while the upper storeys are assumed to be 3.05m (10') in height for a shear wall height of 2.75m (9'). Therefore, the stiffness parameter was divided by 1.377 (3.36/2.44) since the parameters were based on a shear wall that is 2440mm (8') in height while the upper storeys were divided by 1.127 (2.75/2.44).

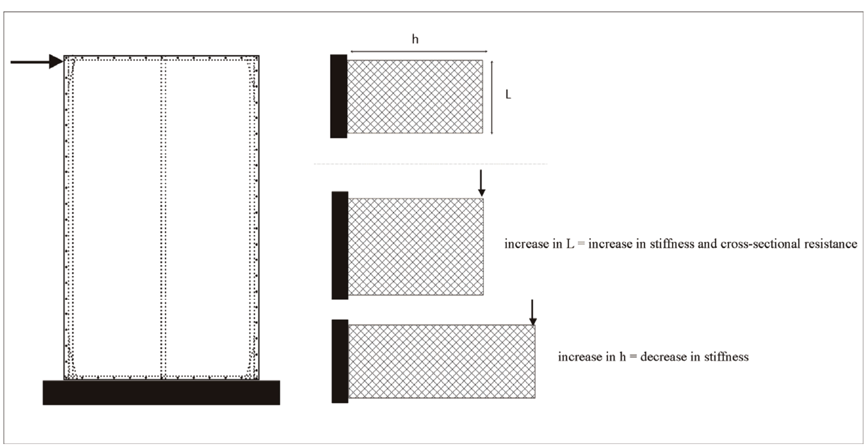


Figure 5.5 Schematic Demonstrating the Variation of Stiffness with Changes in Length and Height of a Wall *(Morello, 2009)*

5.3 Ground Motion Selection and Scaling

Each building model was subjected to a suite of ground motion records. These records were also consistent with past research by Comeau *(2008)*, Velchev *(2008)* and Morello *(2009)*. There were a total of 45 ground motion records of which 32 were synthetic records, 12 real records and one closely matched earthquake.

There are a limited number of measured ground motion records listed in the FEMA P695 document that are considered appropriate to represent the expected earthquake demand for Vancouver. For this reason a database of simulated records by Atkinson *(2009)* was utilized; these records can be scaled to match the uniform hazard spectrum (UHS) of any given city in Canada. Time histories were generated for a range of distances and magnitudes using the stochastic finite-fault method. The earthquakes selected were for Site Class C in western Canada and were categorized as earthquakes with magnitudes of M6.0 and M7.5. The 32 synthetic records were selected based on their compatibility with the UHS for Vancouver. Based on the recommendations of FEMA P695 for ground motion selection, six real earthquake records were obtained from the PEER NGA database *(PEER, 2005)* measured at Site Class C soil conditions with accelerations in the transverse and lateral directions for a total of 12 records.

The last earthquake record was closely matched with the UHS of Vancouver *(Léger et al., 1993)*. The closely matched earthquake was generated by applying the Fast Fourier Transform to a synthetic record. After several iterations, the frequency of the accelerogram was scaled according to the UHS of Vancouver. The amplitude of the spectrum was then verified with the design response spectrum (UHS for Vancouver) (Figure 5.6). All earthquake records were scaled to match the UHS for Vancouver and are summarized with their corresponding magnitude, epicentral distance and scaling factor (SF) in Table 5.2. All ground motion records were compared with the UHS for Vancouver as illustrated in Figure 5.6.

Table 5.2 Ground Motion Records for Vancouver, Site Class C[1,2]

No.	Record Number	Magnitude	Station	Deg.	PGA (g)	Epicentral Distance (km)	Scaling Factor, SF	Time Step (s)
1	7	M6.0	-	-	0.19	27.2	3.00	0.005
2	17		-	-	0.06	50.1	4.00	0.005
3	25		-	-	0.13	27.2	3.00	0.005
4	29		-	-	0.18	7.1	1.80	0.005
5	30		-	-	0.20	10.7	1.80	0.005
6	82		-	-	0.34	5.0	1.10	0.005
7	100		-	-	0.41	3.5	1.30	0.005
8	109		-	-	0.47	3.5	0.90	0.005
9	148		-	-	0.29	5.5	1.10	0.005
10	156		-	-	0.35	15.0	1.00	0.005
11	161		-	-	0.38	50.1	0.70	0.005
12	170		-	-	0.15	35.6	2.00	0.005
13	179		-	-	0.17	41.2	2.00	0.005
14	186		-	-	0.24	22.3	1.50	0.005
15	188		-	-	0.17	41.1	1.80	0.005
16	197		-	-	0.23	40.8	1.20	0.005
17	237	M7.5	-	-	0.78	1.0	0.50	0.005
18	268		-	-	0.26	28.2	1.30	0.005
19	305		-	-	0.28	50.1	1.30	0.005
20	311		-	-	0.92	1.0	0.60	0.005
21	317		-	-	1.53	7.1	0.60	0.005
22	321		-	-	0.39	21.3	1.25	0.005
23	326		-	-	2.62	7.1	0.25	0.005
24	328		-	-	0.52	14.2	0.80	0.005
25	344		-	-	1.04	9.7	0.50	0.005
26	355		-	-	1.19	13.8	0.50	0.005
27	363		-	-	1.32	1.0	0.40	0.005
28	389		-	-	0.26	7.2	1.10	0.005
29	408		-	-	0.64	8.2	0.60	0.005
30	410		-	-	0.34	13.7	0.90	0.005
31	411		-	-	0.36	16.5	0.90	0.005
32	430		-	-	0.13	21.9	2.40	0.005
33	CHICHIE	M7.6	TCU045	90.0	0.49	77.5	1.10	0.005
34	CHICHIN			0.0			1.00	0.005
35	FRULI000	M6.5	Tolmezzo	0.0	0.33	20.2	1.50	0.005
36	FRULI270			270.0			1.00	0.005
37	HECTOR000	M7.1	Hector	0.0	0.30	26.5	2.00	0.005
38	HECTOR090			90.0			1.40	0.005
39	KOBE000	M6.9	Nishi-Akashi	0.0	0.51	8.7	0.80	0.010
40	KOBE090			90.0			1.00	0.010
41	KOCAELI000	M7.5	Arcelik	0.0	0.18	53.7	3.00	0.005
42	KOCAELI090			90.0			2.80	0.005
43	MANJILL	M7.4	Abbar	-	0.51	40.4	0.90	0.020
44	MANJILT			-			0.75	0.020
45	CM	-	-	-	-	-	-	0.010

[1] Records 1 to 32 are synthetic ground motions from Atkinson *(2009)*

[2] Records 33 to 44 are ground motions from PEER NGA database *(PEER, 2005) (FEMA, 2009)*

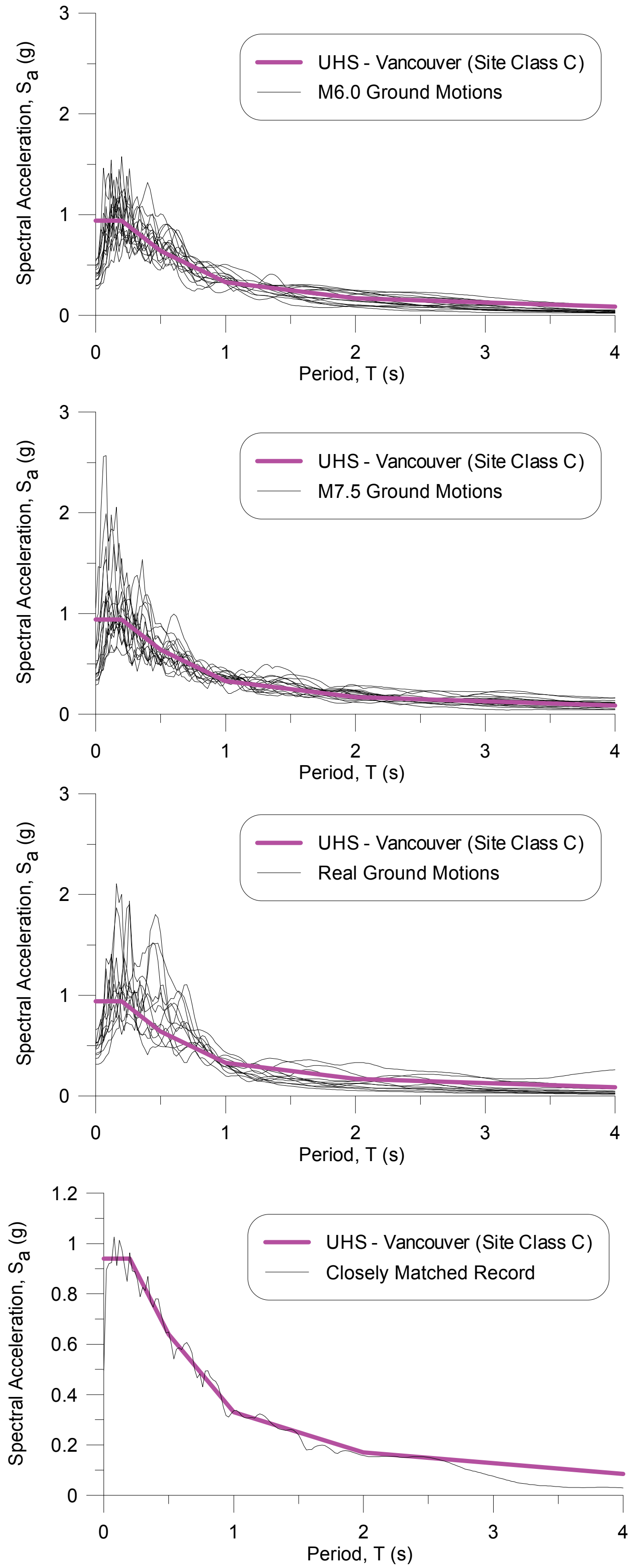


Figure 5.6 Ground Motion Records Compares with UHS for Vancouver

5.4 Response of Model Buildings to Dynamic Analysis

Initially, the design of each building did not consider irregularity (Table 4.8). Figure 5.7 presents the relationship between resistance and displacement for each storey in the four-storey building when subjected to the closely matched earthquake record. The uppermost storey in the initial design reached displacements in the inelastic region which was inadequate in terms of performance based on the results from dynamic analysis (Figure 5.7a). Therefore, the design was modified to account for irregularity which improved the performance of the model building (Section 4.4.1). The uppermost storey of the modified four-storey building remained in the elastic region as presented in Figure 5.7b. The time histories and force-displacement hysteresis at each storey for all buildings designed for irregularities and subjected to the closely matched record at the design level are presented in Appendix H.

After validating each design, based on stiffness, strength and geometrical irregularities, each model building was subjected to the 45 ground motion records. This stage of the analysis procedure provides the building response of the design level earthquakes, i.e. the records were scaled to the UHS for Vancouver. The inter-storey drifts for all buildings were less than the proposed drift limit of 2% for steel sheathed shear wall systems (Table 5.3). The highest mean drift based on the average drift of all earthquakes at the design level was 1.48% which occurred in the seven storey building, and in the majority of cases the highest mean drift occurred in the first storey. The inter-storey drifts for each storey of the four-storey building are presented in Figure 5.8. The design level earthquake inter-storey drifts of the other buildings are presented in Appendix G.

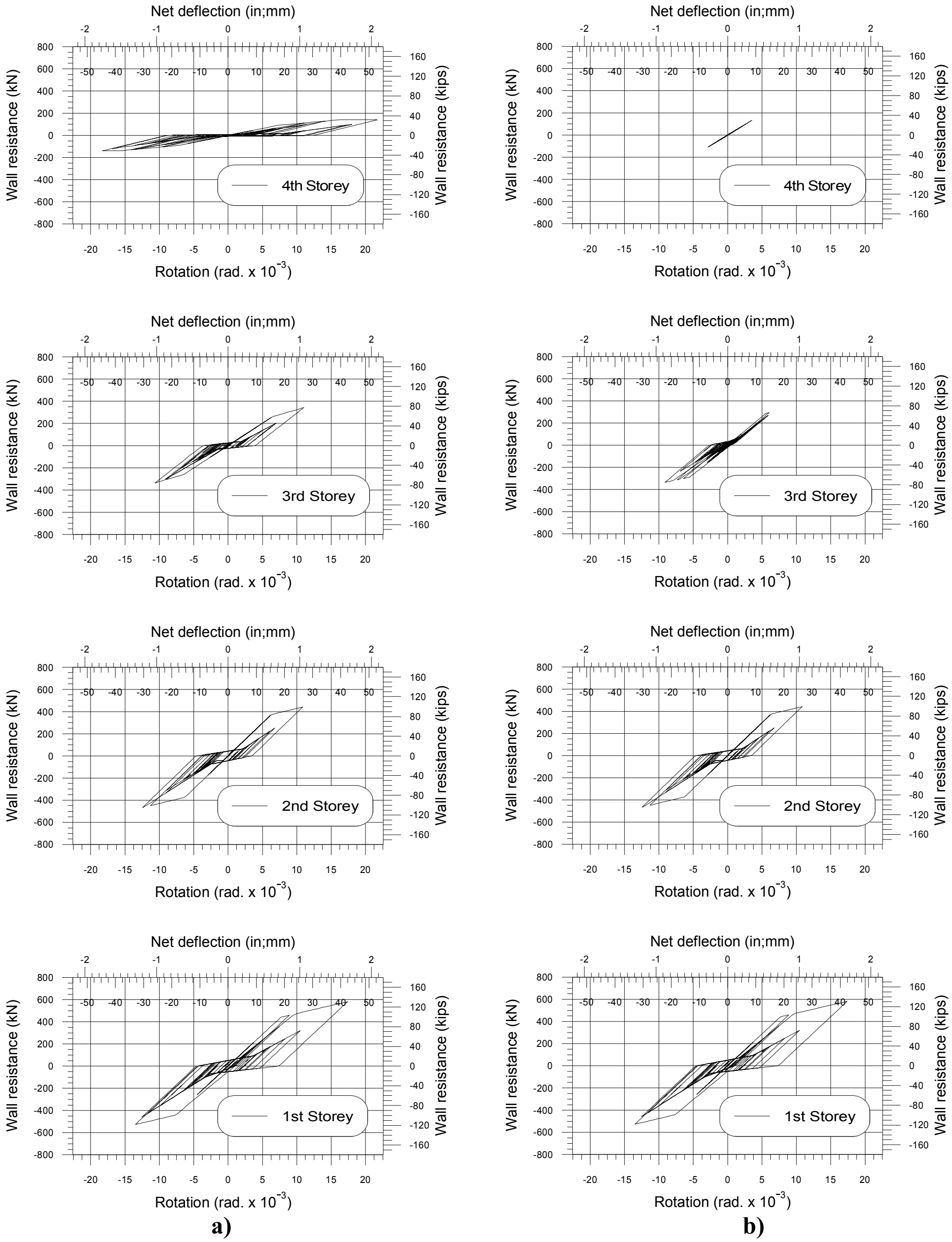


Figure 5.7 Force vs. Displacement hysteresis at each storey for Four-Storey Building a) Initial Design b) Final Design

Table 5.3 Mean Inter-storey Drifts for All Design Level Earthquakes

Storey	Inter-storey Drift (%h_s) - Ruaumoko					
	2	3	4	5	6	7
1	1.23	1.43	1.23	1.10	1.24	1.24
2	0.39	1.00	1.17	1.06	1.03	1.00
3	-	0.34	0.88	1.33	0.98	1.01
4	-	-	0.33	1.08	1.22	1.18
5	-	-	-	0.33	1.03	1.48
6	-	-	-	-	0.33	1.04
7	-	-	-	-	-	0.33
max	1.23	1.43	1.23	1.33	1.24	1.48

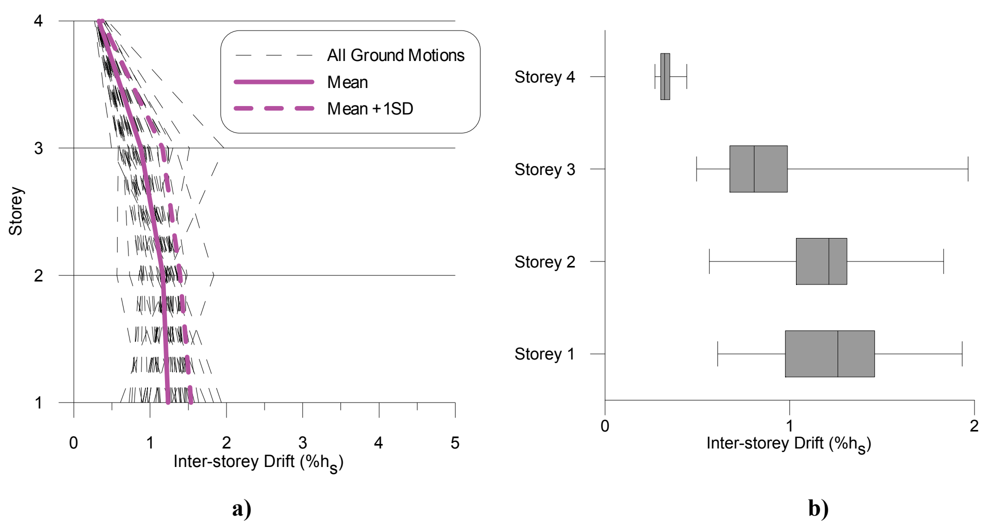


Figure 5.8 a) Inter-storey Drifts of Four-Storey Building
b) Corresponding Box and Whisker Plot

The distribution of the inter-storey drifts of the four storey building are presented in a box-and-whisker plot (Figure 5.8b), which represents various percentiles of distribution; the line in the middle of the box represents the 50th percentile while the ends of the box represent the 25th and 75th percentile drifts. The whiskers indicate the minimum and maximum values within the data. The plot assists in analyzing the data where the dispersion of data is presented. The dispersion of drift in the uppermost storey is low and the drifts are concentrated within a small range. However, for the remaining storeys, the minimum and maximum values vary greatly from the mean although the 25th and 75th percentile are contained within a 0.5% range, approximately.

5.5 Evaluation of Performance of Shear Walls based on FEMA P695

5.5.1 Incremental Dynamic Analysis

Incremental Dynamic Analysis (IDA) *(Vamvatsikos and Cornell, 2002)* using all 45 ground motion records was carried out to assess the performance of each building. The accelerogram of each record was scaled from 20% up to 800% in increments of 20% using the design level earthquake which was previously scaled to match the UHS for Vancouver. The records were scaled to determine the intensity that would cause failure of the structure. Failure was defined as the point where the inter-storey drift of any given storey surpassed the maximum allowable drift limit of 2%. The IDA curves for the four storey building are illustrated in Figure 5.9 where each point on the curve represents the maximum inter-storey drift for a scaled ground motion record.

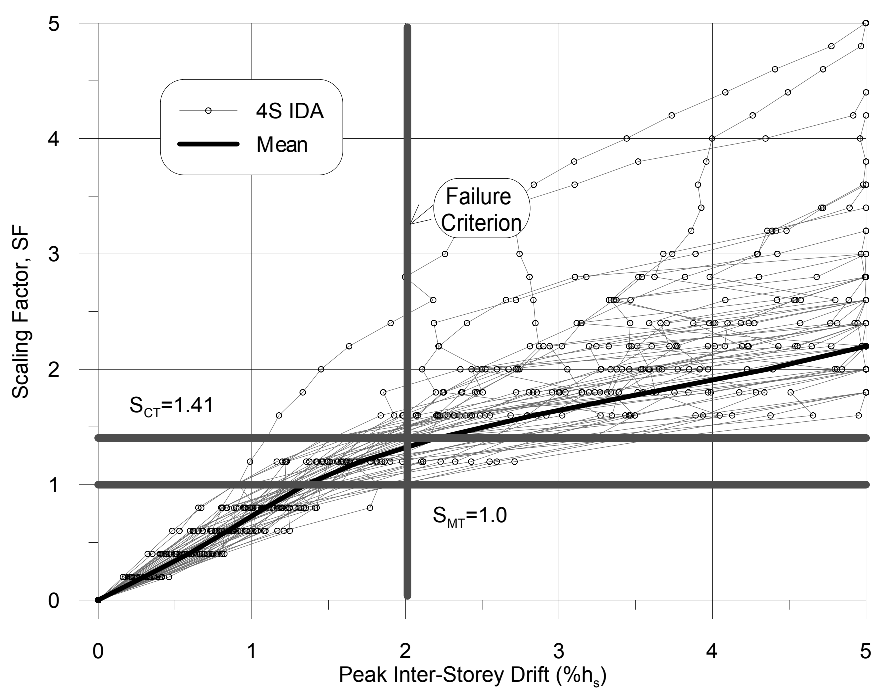


Figure 5.9 IDA for 45 Earthquake Records for the Four-Storey Building

The FEMA P695 methodology defines the median collapse, S_{CT}, as the intensity at which 50% of the earthquake records cause failure. The collapse margin ratio, *CMR*, is the ratio of the median collapse to the scaling factor of the original earthquake record, S_{MT} (Equation (5-1)). Since all the earthquakes were previously scaled to match the UHS for Vancouver, as in Table 5.2, the S_{MT} was taken as 1.0. Therefore, the *CMR* was equal to the intensity of the median collapse. For the four-storey building in Figure 5.9, the S_{CT} was 1.41 which meant that at a scaling factor of 141% of the ground motion records, 50% of the records caused damage exceeding the maximum allowable failure criterion. The IDA results for all buildings are presented in Appendix I.

$$CMR = \frac{S_{CT}}{S_{MT}} \tag{5-1}$$

where,

CMR = Collapse margin ratio

S_{CT} = Median collapse intensity

S_{MT} = Scaling factor of original earthquake record

5.5.2 Evaluation of Buildings

The collapse probability was determined from the results of the IDA response curves. It was calculated as the number of ground motion records that caused failure of the building based on the failure criterion of 2% for each scaling factor. A log-normal distribution was fit to the collapse probability data points from which a fragility curve was obtained (Figure 5.10).

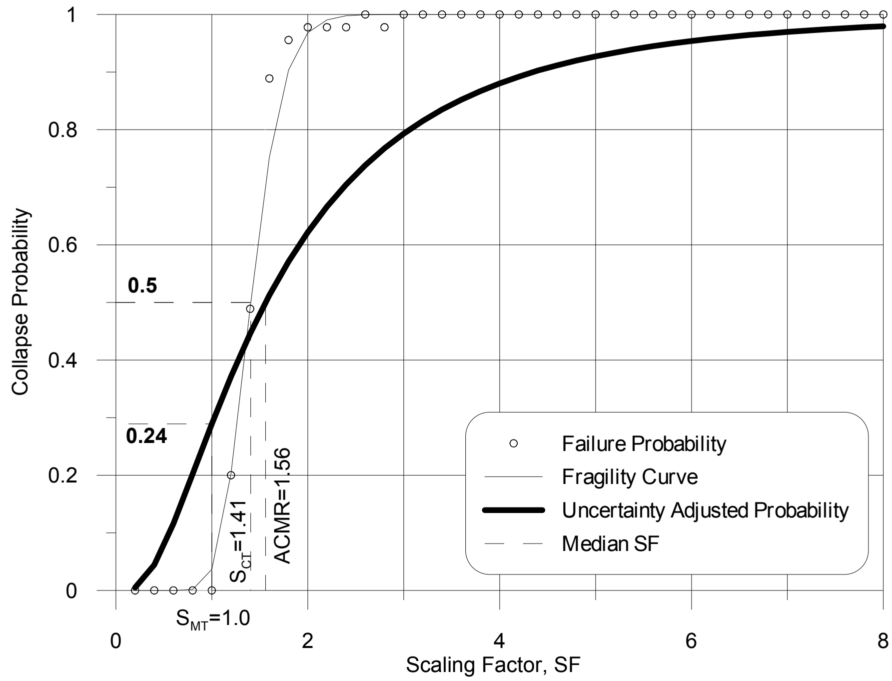


Figure 5.10 Fragility Curve for the Four-Storey Building

The *CMR* was adjusted using a spectral shape factor (*SSF*) to obtain an adjusted collapse margin ratio (*ACMR*) (Equation (5-2)). An *SSF* was used because less damage than that predicted is expected for ductile systems with long periods *(FEMA, 2009)*. The fragility curves for all buildings along with their corresponding *CMR* and *ACMR* values are presented in Appendix I. The *SSF* depended on the ductility of the system and its fundamental period which was obtained from pushover analyses.

$$ACMR_i = SSF_i \times CMR_i \tag{5-2}$$

where,

CMR_i = Collapse margin ratio of each building

$ACMR_i$= Adjusted collapse margin ratio of each building

SSF = Spectral shape factor

5.5.2.1 Pushover Analysis

A pushover analysis of each model building was carried out to determine the period based ductility and the *SSF*. The pushover analysis is a nonlinear static analysis in which a unit force was applied at each storey level with a ramp loading protocol. The proportion of the base shear force at each storey level listed in Table 5.4 was considered by including the seismic force distribution shape (Figure 5.11) from the values in Table 4.7. The pushover analysis input file using Ruaumoko for the four-storey building is provided in Appendix F.

Table 5.4 Seismic Force Distribution Shape for Four-Storey Building

Storey	F_x (kN)	Fraction
Roof	52.2	0.196
4	103.7	0.390
3	71.3	0.268
2	38.9	0.146
1	-	-
Σ	266.2	1

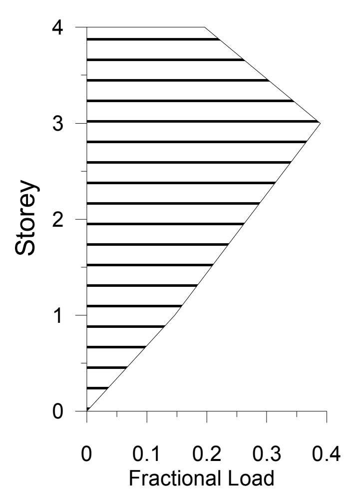


Figure 5.11 Pushover Unit Force Distribution for Four-Storey Building

The ductility (Equation (5-3)) was calculated based on the ratio of ultimate drift, δ_u, to yield drift, δ_y. Since degradation could not be modeled, 2% was assumed to be the ultimate drift. The yield drift was based on where initial elastic shear force portion of the pushover curve met the maximum shear force. The overstrength of the system was calculated by comparing the maximum shear force, V_{max}, to the design base shear force, V (Equation (5-4)). The pushover curve of the four-storey building is presented in Figure 5.12 and all pushover curves are presented in Appendix I.

$$\mu_T = \frac{\delta_u}{\delta_y} \tag{5-3}$$

$$\Omega = \frac{V_{\max}}{V} \tag{5-4}$$

where,

μ_T = Period-based ductility of structure

δ_u = ultimate drift of structure

δ_y = yield drift of structure

Ω = overstrength of structure

V_{max} = maximum shear strength

V = maximum design base shear force

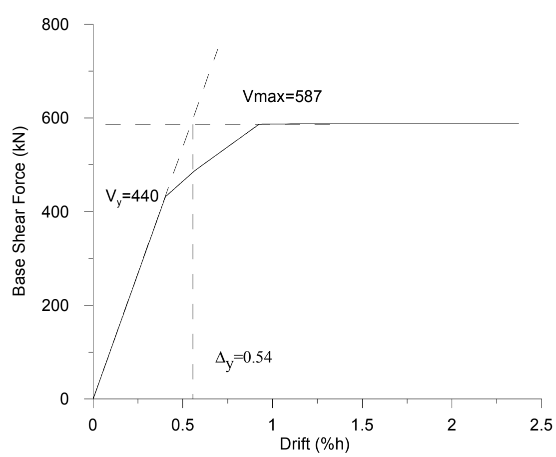


Figure 5.12 Pushover Analysis of the Four-Storey Building

5.5.2.2 Determination of Total Uncertainty

The *ACMR* has a log-normal distribution with a mean distribution calculated as the natural logarithm of the median collapse intensity, S_{CT}, and with a standard deviation of distribution given as the total uncertainty of collapse, β_{TOT}, of the system. The total uncertainty included four areas where uncertainty was expected: uncertainty due to record-to-record variation, β_{RTR}, uncertainty due to design requirements, β_{DR}, uncertainty within the test data, β_{TD}, and uncertainty related to modeling of the structure, β_{MDL} (Equation (5-5)). FEMA P695 classifies each of these uncertainties as superior (β=0.10), good (β=0.20), fair (β=0.35), or poor (β=0.50) except for β_{RTR} which is generally assigned a value of 0.40 for systems with ductility greater than 3.0.

$$\beta_{TOT} = \sqrt{\beta_{RTR}^2 + \beta_{DR}^2 + \beta_{TD}^2 + \beta_{MDL}^2} \tag{5-5}$$

where,

β_{TOT} = Total system collapse uncertainty

β_{RTR} = Record-to-record collapse uncertainty

β_{DR} = Design requirements-related collapse uncertainty

β_{TD} = Test data-related collapse uncertainty

β_{MDL} = Modeling-related collapse uncertainty

The confidence of design requirements was assumed to be of medium reliability since there are no current Canadian design guidelines for steel sheathed shear walls, however, there are design guidelines for similar systems such as wood sheathed shear walls in AISI S213. The design was carried out based on the requirements of the 2005 NBCC and AISI S213 where the design addressed properties such as stiffness and strength. The completeness and robustness of the design requirements of medium reliability was chosen because the design method was only examined by this research and quality assurance of construction in the field could not be controlled. A value of 0.35 was therefore assigned to β_{DR} for a Good rating.

The completeness and robustness of the test data was also taken as medium because many configurations were tested though the test program did not address all the test issues as defined in Section 3.5.2 of FEMA P695 *(FEMA, 2009)*.The effects of gravity loads on shear walls for example were not studied. The confidence in test results was of medium reliability because the behaviour of each test configuration was consistent and repeatable. Even though the tests were consistent with one another, they were not completely consistent with the behaviour of the tests carried out by Yu *et al. (2007)*. Therefore, an overall rating of Good with a corresponding value of 0.35 was assigned to β_{TD}.

The degradation observed in the experimental tests was not modeled in Ruaumoko which was an important aspect of the behaviour of steel sheathed shear walls. A low reliability was selected for the accuracy and robustness of the models. A reliability rating of medium was chosen for the representation of collapse characteristics because the tests assessed the inelastic behaviour of the shear walls but did not determine the mode of collapse. An overall rating of poor was assigned for $\beta_{MDL.}$

The total system collapse uncertainty was calculated to be 0.80. Each uncertainty factor is listed in Table 5.5.

Table 5.5 Determination of the Collapse Uncertainty Factor, β

<table>
<tr><th>Uncertainty Factor</th><th>Reliability</th><th>Rating</th><th>β</th></tr>
<tr><td colspan="3">Record-to-record collapse uncertainty β_{RTR}</td><td>0.40</td></tr>
<tr><td colspan="3">Design requirements-related collapse uncertainty β_{DR}</td><td></td></tr>
<tr><td>Confidence in basis of design requirements</td><td>Medium</td><td rowspan="2">Fair</td><td rowspan="2">0.35</td></tr>
<tr><td>Completeness and robustness</td><td>Medium</td></tr>
<tr><td colspan="4">Test data-related collapse uncertainty β_{TD}</td></tr>
<tr><td>Confidence in test results</td><td>Medium</td><td rowspan="2">Fair</td><td rowspan="2">0.35</td></tr>
<tr><td>Completeness and robustness</td><td>Medium</td></tr>
<tr><td colspan="4">Modeling-related collapse uncertainty β_{MDL}</td></tr>
<tr><td>Accuracy and robustness of models</td><td>Low</td><td rowspan="2">Poor</td><td rowspan="2">0.50</td></tr>
<tr><td>Representation of collapse characteristics</td><td>Medium</td></tr>
<tr><td colspan="3">Total system collapse uncertainty β_{TOT}</td><td>0.80</td></tr>
</table>

5.5.2.3 Evaluation of Structures

The evaluation of each model building was based on the acceptable values of *ACMR* listed in FEMA P695 according to the level of uncertainty of the system. The listed acceptable values were a result of established probabilities of collapse. To validate the R-values, FEMA P695 requires that each *ACMR* must be greater than the tabulated $ACMR_{20\%}$ value corresponding to the total system collapse uncertainty that was calculated (Equation (5-6)). In addition, the average of the *ACMR* for all model buildings must be greater than the listed value for $ACMR_{10\%}$ (Equation (5-7)).

$$ACMR_i \geq ACMR20\% \tag{5-6}$$

$$\overline{ACMR_i} \geq ACMR10\% \tag{5-7}$$

where,

$\overline{ACMR_i}$ = average adjusted collapse margin ratio of all buildings

$ACMR_i$ = adjusted collapse margin ratio of each building

The S_{CT}, β_{TOT}, and SSF for each building are presented in Table 5.6 from which the $ACMR$ was calculated. In all cases the $ACMR$ value was below the allowable value of $ACMR_{20\%}$ and as a result, the average value was also below the acceptable value for $ACMR_{10\%}$.

Table 5.6 Summary of FEMA P695 Values

Storeys	S_{MT}	S_{CT}	CMR	β_{TOT}	SSF	$ACMR_i$	$ACMR_{20\%}$	$ACMR$ average	$ACMR_{10\%}$	Overstrength Ω_o
2	1.00	1.43	1.43	0.800	1.12	1.60	1.96	1.50	2.79	1.30
3	1.00	1.30	1.30	0.800	1.10	1.43	1.96			1.37
4	1.00	1.41	1.41	0.800	1.11	1.56	1.96			1.33
5	1.00	1.29	1.29	0.800	1.12	1.45	1.96			1.29
6	1.00	1.29	1.29	0.800	1.14	1.47	1.96			1.30
7	1.00	1.27	1.27	0.800	1.15	1.46	1.96			1.32

An evaluation for the overstrength value was also provided in FEMA P695. The overstrength, Ω_o, value was determined by the pushover analysis for each building (Table 5.4). A maximum value of 3.0 is allowed for overstrength which was higher than the calculated values of overstrength. Furthermore, each overstrength value, Ω_o, was less than the proposed overstrength value of 1.4. Therefore, Ω_o can be conservatively increased to 1.4.

The results proved not to meet the acceptance criteria of the FEMA P695 evaluation procedure for building performance of structures designed using the test-based seismic force modification factors. A revision of the R_d and R_o values obtained directly from the test data is warranted.

5.6 Design and Analysis of Phase II

An alternate design was needed based on the findings presented in Section 5.5.2.3 where the performance of the model buildings was inadequate. The design procedure for Phase II followed that outlined in Chapter 4 except for the R-values which were modified; R_d was reduced to 2.0 and R_o was reduced to 1.3.

The design relied on shear walls tested at McGill. Therefore, only walls with 1.09mm (0.043”) framing, 0.46mm (0.018”) or 0.76mm (0.030”) sheathing were used. In the data analyzed by Velchev *(2009)*, the stiffness values for the US tests were relatively low compared with the data obtained from tests at McGill. The low stiffness is likely attributed to the 12.7mm (1/2”) hold-down anchor rods used by Yu *et al. (2007)* to fasten the shear wall to the test frame, as opposed to the 22.2mm (7/8”) threaded anchor rods used for the wall tests described herein. It is doubtful that 12.7 mm anchor rods would be sufficient for the buildings used in the dynamic analysis study, making the shear walls at McGill the choice of walls for design. A maximum number of shear walls was determined in Section 4.4 to be approximately 37 per storey. However, the shear walls with 0.76mm (0.030”) sheathing had lower shear resistance values than the walls with 0.84mm (0.033”) (Table 3.11). Therefore, the design was modified to allow the sheathing to be doubled on each shear wall for a maximum number of shear walls of 74 with the assumption that the shear wall will have double the resistance and stiffness. The Stewart hysteretic element parameters for the 0.76mm (0.030”) sheathed walls for 50mm (2”), 100mm (4”), and 150mm (6”) are listed in Appendix E with a comparison with the reversed cyclic test data and energy dissipation.

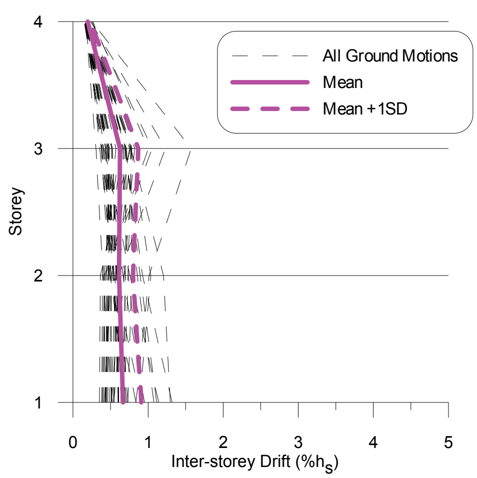
The initial verification of the design included verification of the design period using Ruaumoko. The results of the preliminary analysis showed that the period for each building was less than the maximum allowable of $2T_a$ (See Section 4.3). The design period was modified using the periods resulting from the preliminary analysis. The secondary analysis showed that the period was reduced further because the number of walls was increased. A higher number of shear walls causes the stiffness of the building to increase which reduces the natural period of the building (Table 5.7). The iteration process would not converge because the number of walls would continuously need to be increased. Therefore, a decision was made to carry out the design of the model using the period from the preliminary analysis. A summary of all design details are presented in Appendix J.

Table 5.7 Phase II Period Verification

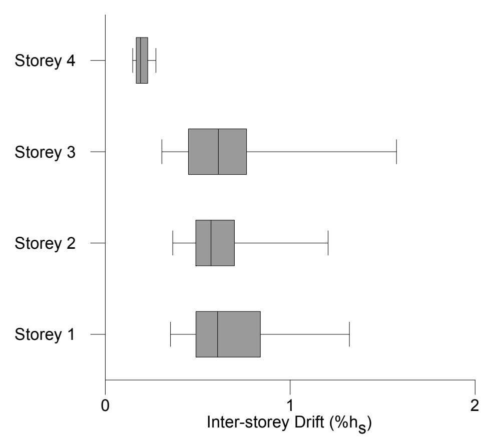
Storeys	Design Period [1], $2T_a$ (s)	Ruaumoko Preliminary Verification, T (s)	Ruaumoko Secondary Verification, T (s)
2	0.417	0.343	0.343
3	0.552	0.431	0.427
4	0.677	0.515	0.478
5	0.795	0.643	0.570
6	0.907	0.756	0.662
7	1.014	0.879	0.772

[1] Design Period from Table 4.2

Following the same analysis procedure for Phase I, the model buildings were subjected to 45 ground motion records at 100% scaling. The inter-storey drifts of each storey of the four-storey building are presented in Figure 5.13a and the distribution of the results is presented in a box and whisker plot in Figure 5.13b. The mean drift values for the Phase II design are lower than those obtained from the results of Phase I (Tables 5.3 and 5.8). The maximum mean drift for Phase II was 1.02% which is well below the allowable drift of 2%.



a)



b)

Figure 5.13 a) Inter-storey Drifts of Four-Storey Building of Phase II b) Corresponding Box and Whisker Plot

Table 5.8 Mean Inter-storey Drifts for All Design Level Earthquakes for Phase II

Storey	Inter-storey Drift (%h_s) - Ruaumoko					
	2	3	4	5	6	7
1	0.84	0.80	0.67	0.61	0.58	0.55
2	0.28	0.59	0.61	0.62	0.53	0.52
3	-	0.23	0.63	0.63	0.59	0.52
4	-	-	0.20	0.84	0.66	0.58
5	-	-	-	0.23	0.95	0.71
6	-	-	-	-	0.24	1.02
7	-	-	-	-	-	0.24
max	0.84	0.80	0.67	0.84	0.95	1.02

The performance of each model building was then evaluated through an incremental data analysis followed by an evaluation of collapse probability. The IDA and fragility curves for the revised design of the four storey building are presented in Figures 5.14 and 5.15, respectively. The IDA and fragility curves for each building are provided in Appendix K along with their corresponding pushover analysis curve. A summary of the results is provided in Table 5.9.

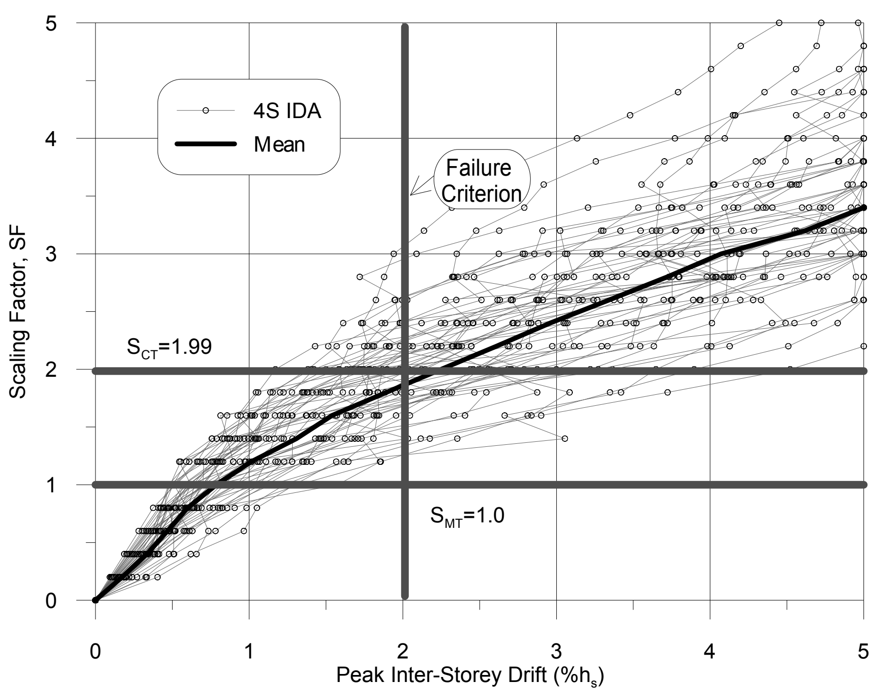


Figure 5.14 IDA for 45 Earthquake Records for the Four-Storey Building – Phase II

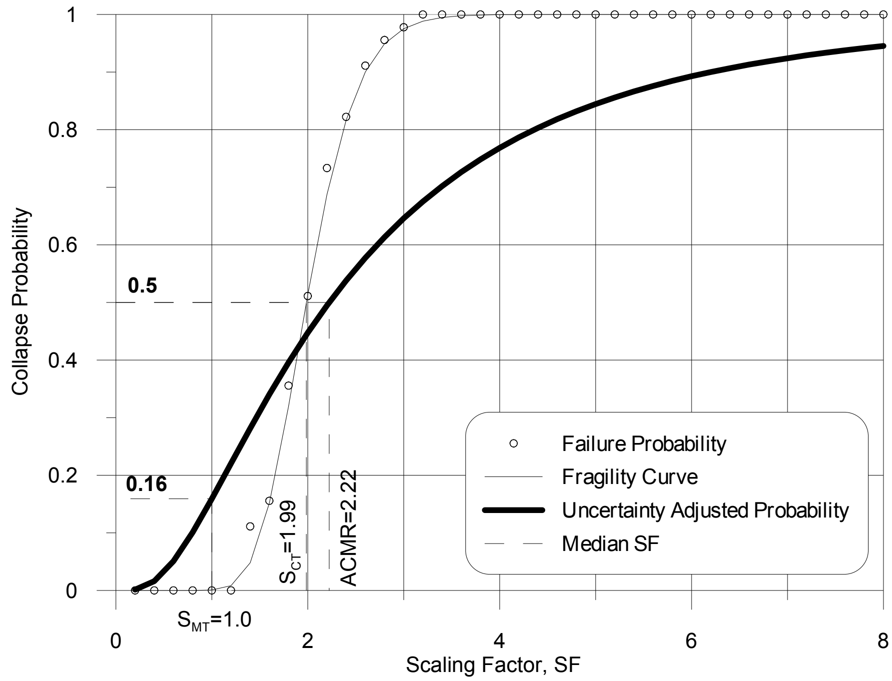


Figure 5.15 Fragility Curve for Four-Storey Building – Phase II

Table 5.9 Summary of FEMA P695 Values for Phase II

Storeys	S_{MT}	S_{CT}	CMR	β_{TOT}	SSF	$ACMR_i$	$ACMR_{20\%}$	$ACMR$ average	$ACMR_{10\%}$	Overstrength Ω_o
2	1.00	1.91	1.91	0.800	1.12	2.14	1.96	2.08	2.79	1.26
3	1.00	1.93	1.93	0.800	1.11	2.14	1.96			1.32
4	1.00	1.99	1.99	0.800	1.12	2.22	1.96			1.38
5	1.00	1.79	1.79	0.800	1.13	2.03	1.96			1.33
6	1.00	1.89	1.89	0.800	1.15	2.17	1.96			1.29
7	1.00	1.54	1.54	0.800	1.17	1.80	1.96			1.34

Based on the results listed in Table 5.9, the individual ACMR values exceed the minimum $ACMR_{20\%}$ value of 1.96 except for the seven-storey building that falls short of the minimum. The average ACMR value for all buildings within the performance group is lower than the minimum $ACMR_{10\%}$ value of 2.79. The overstrength, Ω_o, values are all lower than 1.4 which validates the recommended conservative value of 1.4.

The FEMA P695 methodology requires that the evaluation be based on multiple performance groups that vary in configuration design, seismic load intensity and structural period. As described in Chapter 4 of FEMA P695 *(FEMA, 2009)*, the performance groups should not be biased towards certain variations and should reflect the spectrum of possible behaviour. As a minimum, one structural configuration should be examined with its response to at least two seismic design levels. The evaluation of Phase II consisted of the NEESWood Project building (*Cobeen et al., 2007)* as the structural configuration and covered the range of building heights. However, the evaluation was only carried out for Vancouver which deems the evaluation insufficient; another seismic design level should be examined. It can be concluded that the performance of the buildings is adequate based on their individual performance, except for the seven-storey building. A height limit of 15m (49.2') can be recommended which corresponds approximately to a five-storey building.

CHAPTER 6 – CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

The general objective of this research project was to develop a Canadian design method for steel sheathed / cold-formed steel framed shear walls. The approach involved a test phase, followed by the analysis of the resulting data. Design provisions were then established using this information. Finally, the design method was evaluated by means of dynamic analyses following the FEMA P695 methodology modified for use in Canada.

6.1.1 Test Program

A total of 54 tests (18 wall configurations) were carried out on single-storey steel sheathed shear walls to observe their behaviour and performance. Each configuration varied with respect to screw spacing, sheathing thickness, framing thickness, detailing and aspect ratio.

The shear wall tests were carried out using two loading protocols: monotonic and CUREE reversed-cyclic. The behaviour of the specimens within each configuration was consistent. The majority of failures occurred at the sheathing-framing connection where the fasteners pulled out of the sheathing, punched through the sheathing or tore out of the edge of the sheathing after severe bearing.

The resistance of the shear walls was dependent on the sheathing thickness, framing thickness, and fastener spacing. Fastener spacings of 50mm (2”), 100mm (4”), and 150mm (6”) were tested. An increase in shear resistance was observed as the fastener spacing decreased. Similarly, an increase in resistance was observed with the use of thick sheathing of 0.76mm (0.030”) and 1.09mm (0.043”) framing thickness.

The chord studs of the shear walls were often subjected to significant damage largely due to the tension field that would develop in the sheathing. The horizontal component of this tension field resulted in the twisting and distortion of the chord studs. Bridging was used in an attempt to restrain the chord studs from twisting, which reduced damage and resulted in higher capacities but compromised to some extent the ductility of the shear wall. Further study regarding the use of full blocking between studs for these shear walls is recommended.

The test results were compared with those published by Serrette *(1997)* and Yu *et al. (2007).* Similar shear resistances were measured, however a variation in performance was observed most likely due to the materials that had different properties than the nominal values listed. It is probable that for the Serrette test walls the sheathing was thicker than the nominal value and the yield and tensile stresses were higher than the specified minimum of 230MPa (33ksi) and 310MPa (45ksi), respectively. As well, the use of smaller hold-down anchors by Yu *et al.* may explain the difference in measured stiffness of the walls.

The results of tests by the author, Ong-Tone *(2009)*, Yu *et al. (2007)* and Ellis *(2007)* were incorporated in this study. The test results were reduced using the Equivalent Energy Elastic-Plastic approach in which a bi-linear curve was obtained from the non-linear test or backbone curve. Nominal shear resistances for each shear wall configuration were then determined based on the average yield strength that was calibrated to account for variation in thickness and tensile stress of the sheathing.

Shear walls with aspect ratios from 1:1 to 4:1 were tested to determine whether short walls can be used in design. Short walls measuring 610mm (2’) in length had high rotations which did not allow the development of shear resistance at the same drift as measured for longer walls. It was required that for design of the high aspect ratio shear walls the 2w/h strength reduction formula be used, as found in AISI S213.

In addition, a material resistance factor, $\phi = 0.70$ was proposed. An overstrength factor of 1.40 represents the reserve capacity of a shear wall for seismic capacity design. A factor of safety for limit states and allowable stress design was calculated based on the ratio of ultimate to factored shear strength. As well, a maximum drift limit of 2% was proposed for steel sheathed shear walls. Finally, seismic force modification factors were calculated from the test data; a value of 2.5 was initially proposed for R_d and 1.7 for R_o.

6.1.2 Design Provisions

The test-based parameters that were determined from the steel sheathed shear wall test data were used to determine guidelines for design. Buildings representative of low-rise to medium-rise buildings across Canada were then selected for design. The proposed design approach was then applied to these multi-storey structures (two, three, four, five, six, and seven storeys) to establish a consistent design for the range of heights of each building.

The model buildings were assumed to be located in Vancouver; this choice was made because it is located in a high seismic zone. The loads applied to the buildings followed the guidelines of the 2005 NBCC in which the critical load case included dead, earthquake, snow and live loads. The design of the building was adjusted to account for irregularity in terms of strength, stiffness and geometry. The objective of the design method was to determine and minimize the appropriate number of shear walls to resist the calculated base shear force. Therefore, shear walls with 1.09mm (0.043”) framing, 0.84mm (0.033”) and 50mm (2”) fastener spacing were used for the design of all the buildings except for the two-storey building. The use of 0.46mm (0.018”) was sufficient for the design of the two-storey building.

The seismic force resisting system (SFRS) was defined as the shear walls on each storey. More specifically, the connection between the sheathing and framing of the shear wall was selected to be the energy dissipating element in seismic design. All other elements were expected to be designed to remain elastic following the capacity based design approach. The chord studs of the shear walls were designed according to the CSA-S136 for cold-formed steel compression members.

The inelastic drift of each storey of the design was estimated according to the AISI S213 and in all cases the drift was less than the test-based maximum drift. The stability factor was calculated as well and therefore P-Δ effects were not necessary to be considered in the design.

6.1.3 Dynamic Analysis

The performance of the steel sheathed shear walls as the SFRS of the representative buildings under seismic excitations was examined by means of dynamic analysis. The shear resistance vs. displacement hysteretic behaviour of the shear walls under cyclic loading was modeled using the Stewart hysteretic element. The Stewart model captured many features of the shear wall behaviour such as elastic stiffness, strength and pinching, although strength degradation was not modeled.

Each building was modeled using Ruaumoko, which is a non-linear dynamic analysis software. The entire building was modeled as a two-dimensional stick with a lean-on P-Δ column. Each building was then subjected to 45 ground motion records that were compatible with the UHS for Vancouver. The ground motion records comprised a suite of real and synthetic records and one closely matched record. The inter-storey drifts of each storey were less than the drift limit of 2% when the building was subjected to the design level earthquakes.

Each ground motion record was further scaled to different intensities as part of an Incremental Dynamic Analysis (IDA). The results of the IDA were used to evaluate the performance of the SFRSs, and validate the *R*-values used in design, according to the FEMA P695 methodology.

The *ACMR* for each building in the Phase I design did not meet the minimum requirements and, therefore, deemed the design to be inappropriate. The R-values used in design had to be revised to determine values appropriate for use with steel sheathed shear walls in regions of high seismicity. Therefore, Phase II of the design was developed to re-evaluate the performance of the buildings where the *R*-values were modified. Based on the results of Phase II, an R_d value of 2.0, and an R_o value of 1.3 are recommended. A height limit of 15m (49.2') is recommended which is approximately equivalent to a five-storey building. To complete the verification of the recommended R-values and height limit, it is necessary to carry out the analysis for another seismic region to cover the range of building performance.

6.2 Recommendations for Future Research

The design approach used for steel sheathed shear walls was conservative due to the limited number of tests carried out at McGill and the US. An expanded test program would be useful in confirming the behaviour of the shear walls under cyclic loading and would lead to a design approach that is more representative of the behaviour.

Based on the analysis of the shear walls tested in the US by Velchev, the stiffness of the shear walls was found to be relatively low compared to the elastic stiffness values of the shear walls tested at McGill with the same detailing. In addition, Yu *et al.* carried out shear wall tests constructed with thicker sheathing which was not part of the test program at McGill. Replicates of those shear walls should be examined to compare and confirm the behaviour exhibited by walls with thicker sheathing especially since their properties were relied on for the design of the model buildings. The test program should be expanded to include design values for shear walls with thicker chord studs and varied sheathing thicknesses. Additional short walls should be tested to verify the shear resistance reduction factor for higher aspect ratio walls of various detailing.

The results of the tests showed that the framing thickness had an effect on the performance of the shear walls. However, the strength of the shear wall would not be influenced by the framing thickness if the thickness of the framing and sheathing were not similar. Therefore, it is recommended to test shear wall configurations with framing thicknesses of at least 1.37mm (0.054"). In addition, shear wall tests with gravity loads should be carried out to determine the effects of gravity on the performance of the shear wall and the chord studs. Furthermore, detailing should be devised in which the twisting of the chord stud members is reduced when subjected to tension field action in the sheathing.

The design evaluation was only carried out for Vancouver, which is located in the highest seismic region of Canada. The design approach should be further investigated for other seismic regions across Canada such as Calgary, Halifax and Montreal.

The dynamic modeling of the buildings was simplified as a two-dimensional stick model. A three-dimensional model would provide a more realistic interpretation of the behaviour of shear walls. It is also recommended that an alternative software be used to model the degradation of strength of the shear walls as it was not accounted for in the Stewart model. Finally, dynamic shake table tests on multi-storey shear walls are also recommended to provide realistic simulations of shear walls subjected to seismic loading.

REFERENCES

Al-Kharat, M., Rogers, C.A. (2005). "Testing of Light Gauge Steel Strap Braced Walls", Research Report, Dept. of Civil Engineering & Applied Mechanics, McGill University, Montreal, QC, Canada.

American Iron and Steel Institute (AISI). 2007. "AISI S213-07, North American Standard for Cold-Formed Steel Framing – Lateral Design". Washington, DC, USA.

American Society for Testing and Materials, A193. 2008. "Standard Specification for Alloy-Steel and Stainless Steel Bolting Materials for High Temperature or High Pressure Service and Other Special Purpose Applications".West Conshohocken, PA, USA.

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

American Society for Testing and Materials, A370. 2006. "Standard Test Methods and Definitions for Mechanical Testing of Steel Products". West Conshohocken, PA, USA.

American Society for Testing and Materials, A653. 2008. "Standard Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process". West Conshohocken, PA, USA.

American Society for Testing and Materials, E2126. 2007. "Standard Test Methods for Cyclic (Reversed) Load Test for Shear Resistance of Vertical Elements of the Lateral Force Resisiting Systems for Buildings". ASTM International, West Conshohocken, PA, USA.

APA-The Engineered Wood Association. 1993. "Wood Structural Panel Shear Walls". Report 154. Tacoma, WA, USA.

APA-The Engineered Wood Association. 2000. "Plywood Diaphragms". Report 138. Tacoma, WA, USA.

APA-The Engineered Wood Association. 2001. "Diaphragms and Shear Walls: Design/Construction Guide". Tacoma, WA, USA.

APA-The Engineered Wood Association. 2005. "Fastener Loads for Plywood – Screws". Technical Note E380D. Tacoma, WA, USA.

Atkinson, G. M. (2009). "Earthquake Time Histories Compatible with the 2005 NBCC Uniform Hazard Spectrum", *Canadian Journal of Civil Engineering*, Vol. 36 No. 6, 991-1000.

Blais, C. (2006). "Testing and Analysis of Light Gauge Steel Frame / 9mm OSB Wood Panel Shear Walls". M.Eng Thesis, Dept. of Civil Engineering & Applied Mechanics, McGill University, Montreal, QC, Canada.

Boudreault, F.A. 2005. "Seismic Analysis of Steel Frame / Wood Panel Shear Walls". M.Eng. thesis, Department of Civil Engineering and Applied Mechanics, McGill University. Montreal, QC, Canada.

Branston, A.E. 2004. "Development of a Design Methodology for Steel Frame / Wood Panel Shear Walls". M.Eng. thesis, Department of Civil Engineering and Applied Mechanics, McGill University. Montreal, QC, Canada.

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

Branston, A.E., Boudreault, F.A., Chen, C.Y., Rogers, C.A. 2006a. "Light-Gauge Steel-Frame / Wood Structural Panel Shear Wall Design Method". *Canadian Journal of Civil Engineering*, Vol. 33 No.7, 872-889.

Branston, A.E., Chen, C.Y., Boudreault, F.A., Rogers, C.A. 2006b. "Testing of Light-Gauge Steel-Frame / Wood Structural Panel Shear Walls". *Canadian Journal of Civil Engineering*, Vol. 33 No.5, 561-572.

Canadian Institute of Steel Construction (2004). "Handbook of Steel Construction", 8th edition, Toronto, ON, Canada.

Canadian Standards Association (CSA), S136. 2007. "North American Specification for the Design of Cold-Formed Steel Structural Members". Mississauga, ON, Canada

Canam Group (2004). "Hambro D500 floor system", www.hambrosystems.com.

Carr, A.J. (2008). "RUAUMOKO – Inelastic Dynamic Analysis, Version March 15th 2000", Dept. of Civil Eng., University of Canterbury, Christchurch, New Zealand"

Chen, C.Y. 2004. "Testing and Performance of Steel Frame / Wood Panel Shear Walls". M.Eng. thesis, Department of Civil Engineering and Applied Mechanics, McGill University. Montreal, QC, Canada.

Cobeen, K., Van de Lindt, J. W., Cronin, K. (2007). "Design of a Six-Story Woodframe Building based on the 2006 IBC Methodology", NEESwood report NW-03, In press.

Comeau, G. 2008. "Inelastic Performance of Welded Cold-Formed Steel Strap Braced Walls". M.Eng. thesis, Department of Civil Engineering and Applied Mechanics, McGill University. Montreal, QC, Canada.

Consortium of Universities for Research in Earthquake Engineering (CUREE). 2004a. "Recommendations for Earthquake Resistance in the Design and Construction of Woodframe Buildings – Part I: Recommendations". CUREE Publication No. W-30a, Richmond, CA, USA.

Consortium of Universities for Research in Earthquake Engineering (CUREE). 2004b. "Recommendations for Earthquake Resistance in the Design and Construction of Woodframe Buildings – Part II: Topical Discussions". CUREE Publication No. W-30b, Richmond, CA, USA.

Ellis, J. 2007. "Shear Resistance of Cold-Formed Steel Framed Shear Wall Assemblies using CUREE Test Protocol". Simpson Strong-Tie Co., Inc.

Federal Emergency Management Agency (2009). "Quantification of Building Seismic Performance Factors, FEMA-P695 " Redwood City, CA, USA.

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

Hikita, K. E. (2006). "Impact of Gravity Loads on the Lateral Performance of Light Gauge Steel Frame / Wood Panel Shear Walls", M.Eng. Thesis, Dept. of Civil Engineering & Applied Mechanics, McGill University, Montreal, Canada.

International Code Council. 2003. "International Building Code 2003". Falls Church, VA, USA.

International Conference of Building Officials. 1997. "Uniform Building Code". Whittier, CA, USA.

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

Léger, P., Tayebi, A. K., Paultre, P. (1993). "Spectrum-compatible Accelerograms for Inelastic Seismic Analysis of Short-period Structures located in eastern Canada", *Canadian Journal of Civil Engineering*, Vol. 20, 951-968.

Miranda, E. and Bertero, V.V. 1994. "Evaluation of Strength Reduction Factors for Earthquake-Resistant Design". *Earthquake Spectra*, Vol. 10(2).

Mitchell, D., Tremblay, R., Karacabeyli, E., Paultre, P., Saatcioglu, M., Anderson, D.L. 2003. "Seismic Force Modification Factors for the Proposed 2005 Edition of the National Building Code of Canada". *Canadian Journal of Civil Engineering*, Vol. 30, No. 2, 308 – 327.

Morello,D. (2009). "Seismic Performance of Multi-Storey Structures with Cold-Formed Steel Wood Sheathed Shear Walls". M.Eng. thesis, Department of Civil Engineering and Applied Mechanics, McGill University. Montreal, QC, Canada.

National Fire Protection Association (NFPA). 2003. "NFPA 5000, Building Construction and Safety Code". Quincy, MA, USA.

National Research Council of Canada. 2005. "National Building Code of Canada 2005, 12th Edition". Ottawa, ON, Canada.

Newmark, N.M., Hall, W.J., 1982. "Earthquake Spectra and Design". *Engineering Monograph*, Earthquake Engineering Research Institute. Berkeley, CA, USA.

Ong-Tone, C. (2009). "Tests and Evaluation of Cold-Formed Steel Frame/Steel Sheathed Shear Walls", Project Report, Dept. of Civil Engineering and Applied Mechanics, McGill University, Montreal, Qc, Canada.

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

Pacific Earthquake Engineering Research Center (PEER) (2005). "PEER NGA database", http://peer.berkeley.edu/nga. Accessed March, 2008.

Rokas, D. (2005). "Testing of Steel Frame / 9.5 mm CSP Wood Panel Shear Walls", Project Report, Dept. of Civil Engineering and Applied Mechanics, McGill University, Montreal, Qc, Canada.

Shamim, I., Morello, D., Rogers, C.A. (2010), "Dynamic testing and analyses of multi-storey wood sheathed / CFS framed shear walls", 9th US National & 10th Canadian Conference on Earthquake Engineering, Toronto, ON.

Serrette, R.L. 1995. “Shear Wall Design and Testing”. Newsletter for the Light Gauge Steel Engineers Association, Light Gauge Steel Engineers Association. Nashville, TN, USA.

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

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

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

Simpson Strong-Tie Co., Inc. 2008. “S/HDS & S/HDB Holdowns Specification”. Catalog C-CFS06, Pleasanton, CA, USA. 23.

Steel Framing Alliance (2005). “Management Report to the Steel Framing Alliance Board of Directors”. Washington, D.C., USA.

Stewart, W.G. (1987). “The Seismic Design of Plywood Sheathed Shear Walls”, PhD Thesis, University of Canterbury, New-Zealand.

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

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

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

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

Vamvatsikos D., Cornell, C. A. (2002) “Incremental Dynamic Analysis”, *Earthquake Engineering and Structural Dynamics*, 31:4, 491-514.

Velchev, K. 2008. “Inelastic Performance of Screw Connected Cold-Formed Steel Strap Braced Walls”. M.Eng. thesis, Department of Civil Engineering and Applied Mechanics, McGill University. Montreal, QC, Canada.

Velchev, K. 2009. “Analysis of US Steel Sheathed Shear Wall Data Using the Equivalent Energy Elastic-Plastic Approach”. Research Report. Department of Civil Engineering and Applied Mechanics, McGill University. Montreal, QC, Canada.

Yu, C., Vora, H., Dainard, T., Tucker, J., Veetvkuri, P. 2007. “Steel Sheet Sheathing Options for Cold-Formed Steel Framed Shear Wall Assemblies Providing Shear Resistance”. *Report No. UNT-G76234, American Iron and Steel Institute*, Department of Engineering Technology, University of North Texas, Denton, Texas, USA.

Zhao, Y. 2002. “Cyclic Performance of Cold-Formed Steel Stud Shear Walls”. M.Eng. thesis, Department of Civil Engineering and Applied Mechanics, McGill University. Montreal, QC, Canada.

APPENDIX A

TEST CONFIGURATIONS

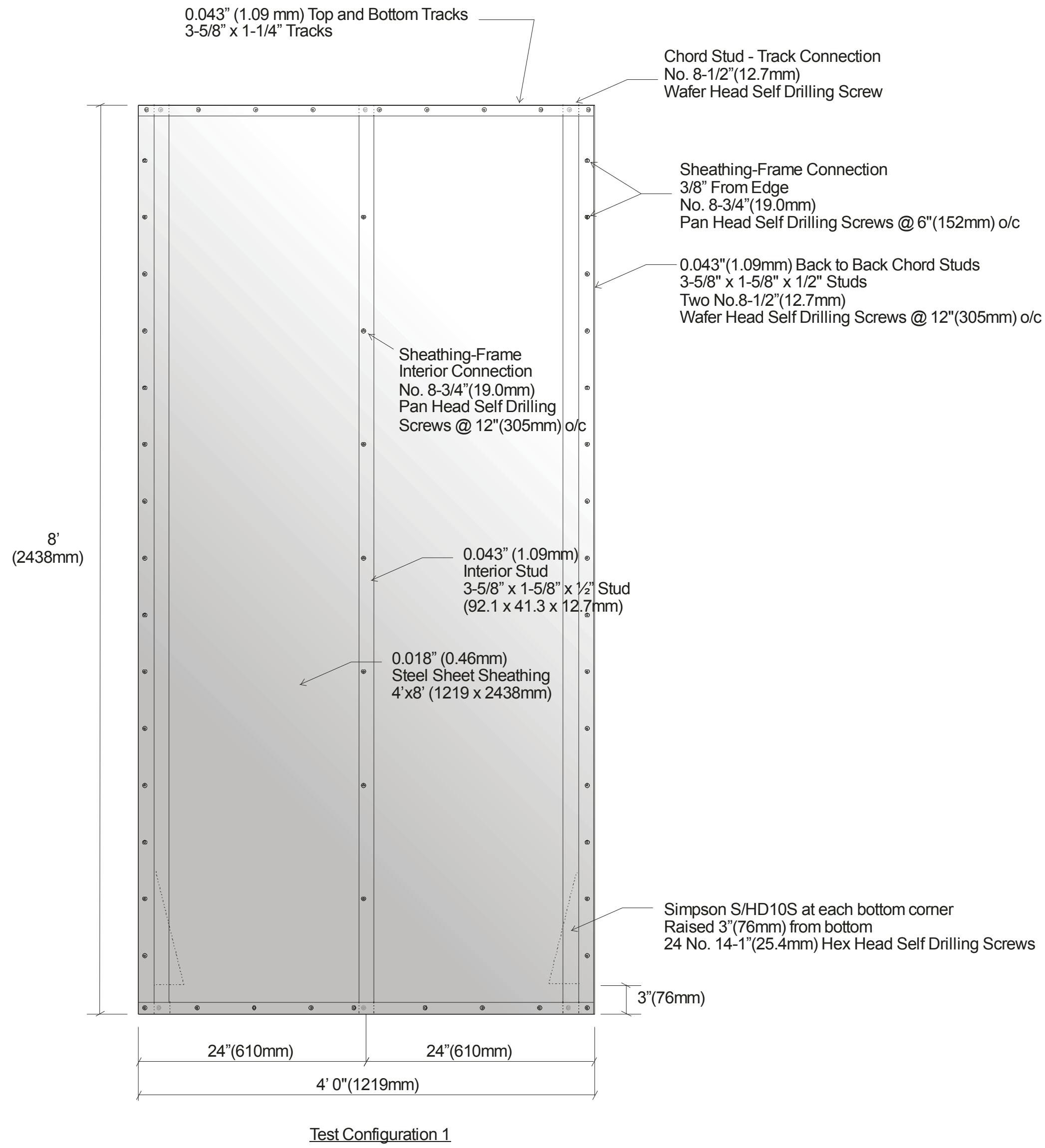


Figure A.1 Nominal Dimensions and Specifications for Test Configuration 1

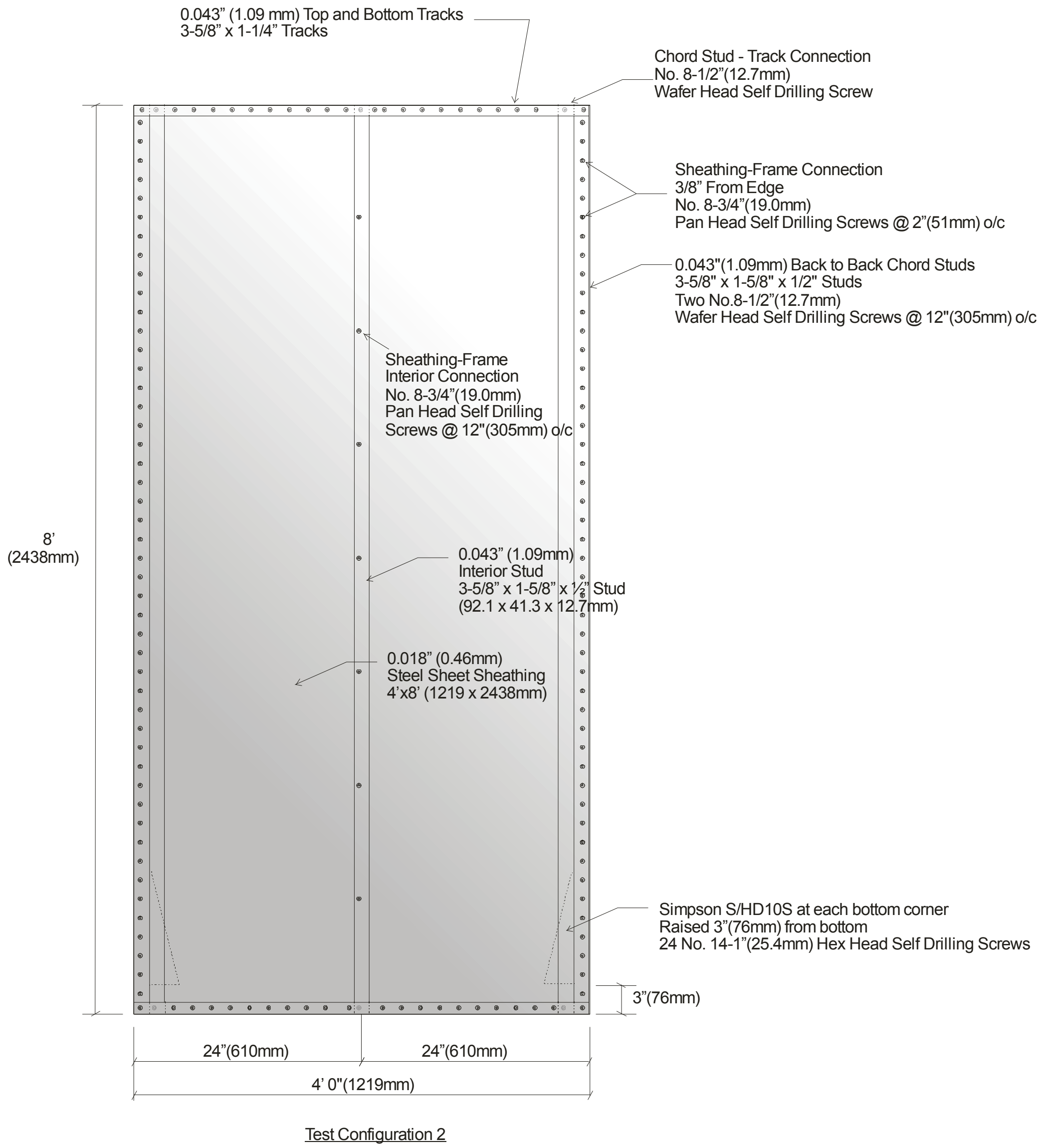


Figure A.2 Nominal Dimensions and Specifications for Test Configuration 2

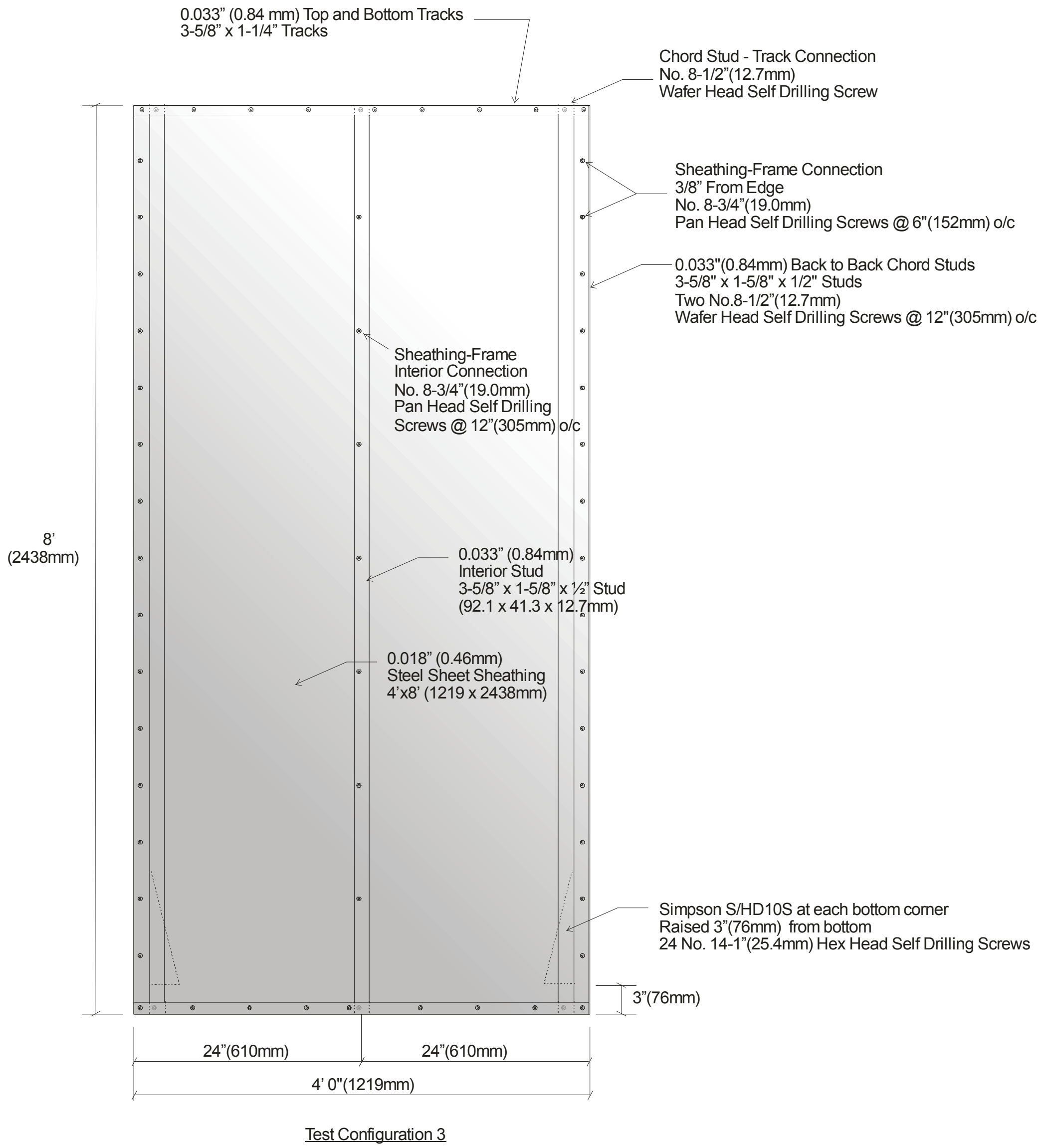


Figure A.3 Nominal Dimensions and Specifications for Test Configuration 3

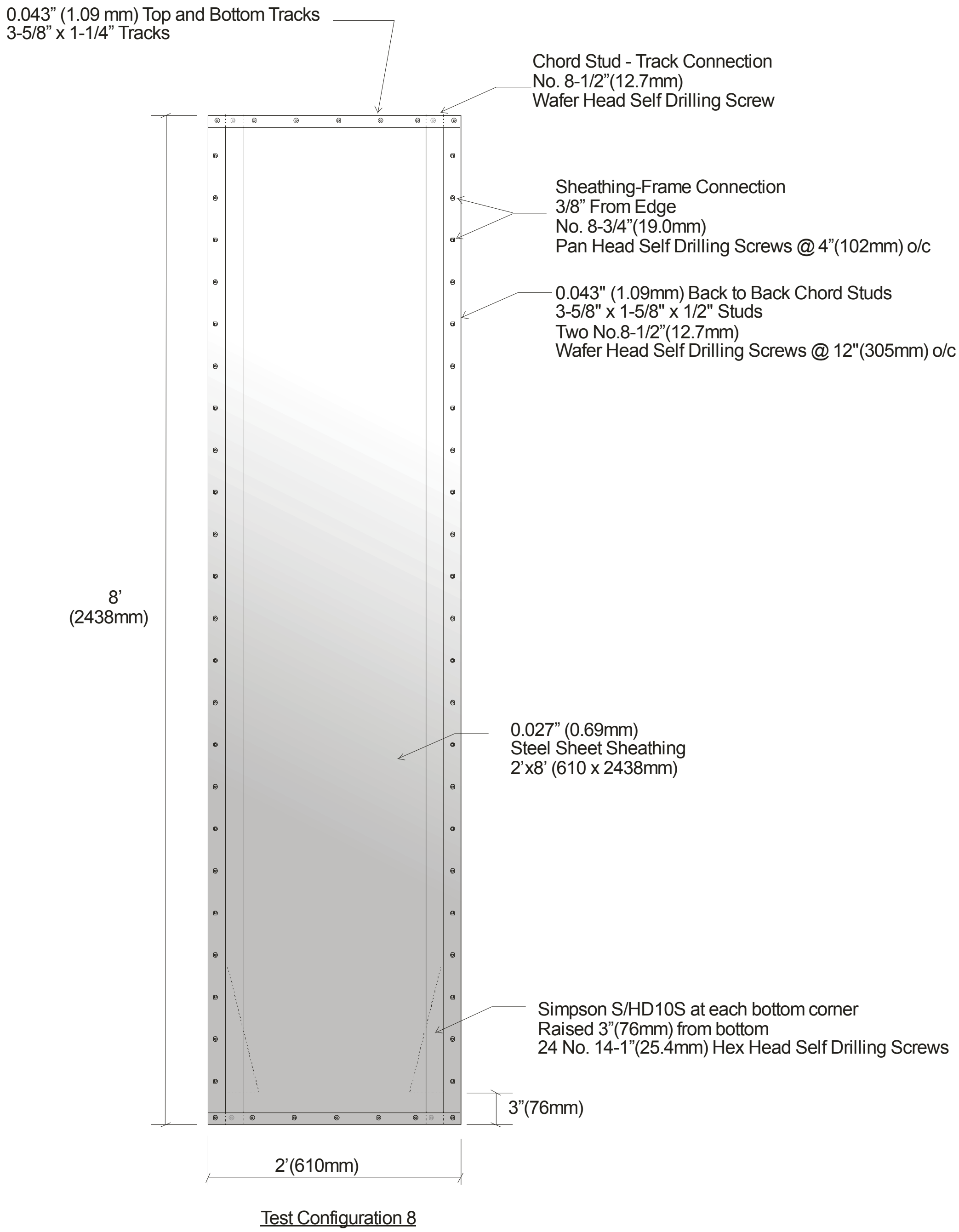


Figure A.4 Nominal Dimensions and Specifications for Test Configuration 8

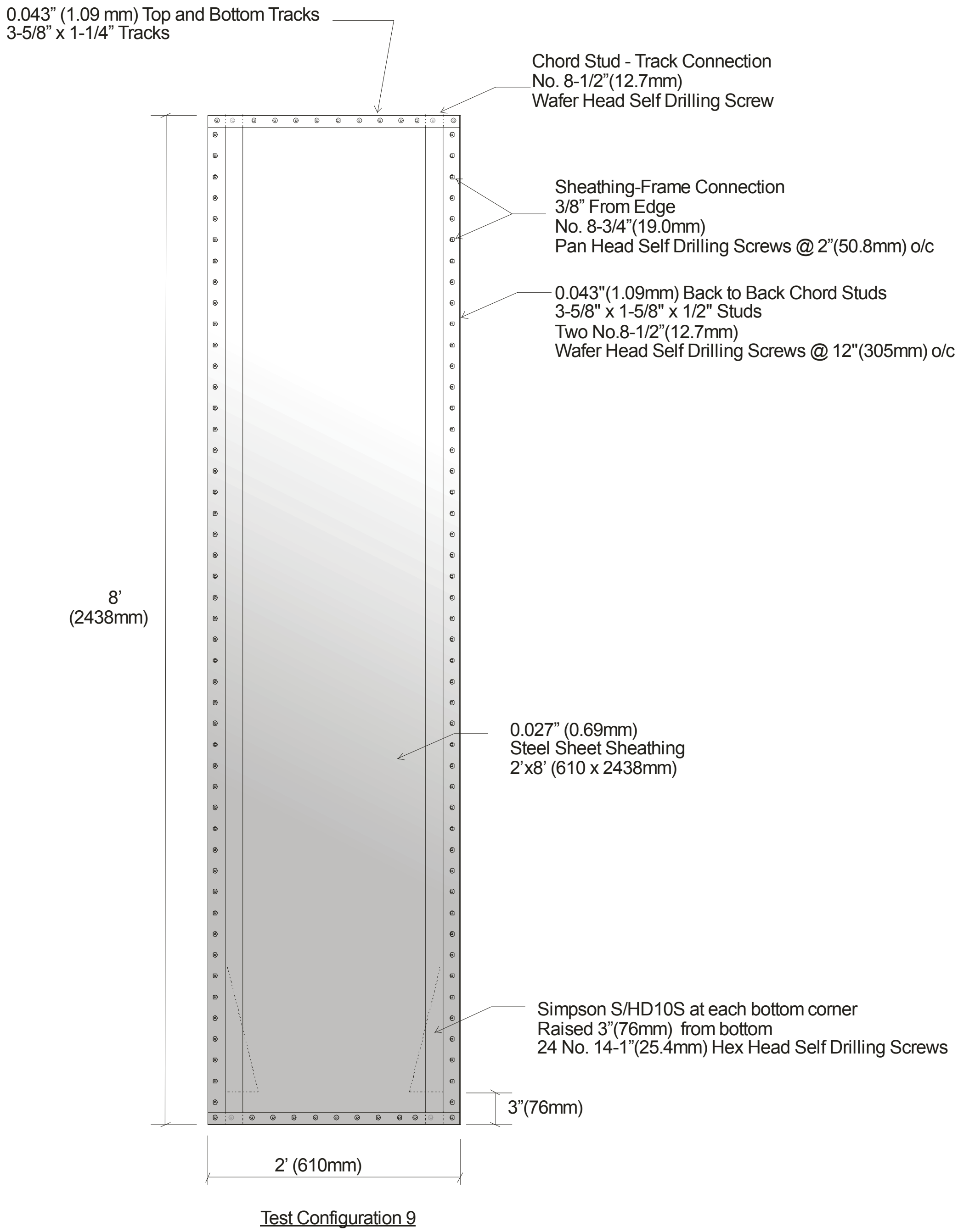


Figure A.5 Nominal Dimensions and Specifications for Test Configuration 9

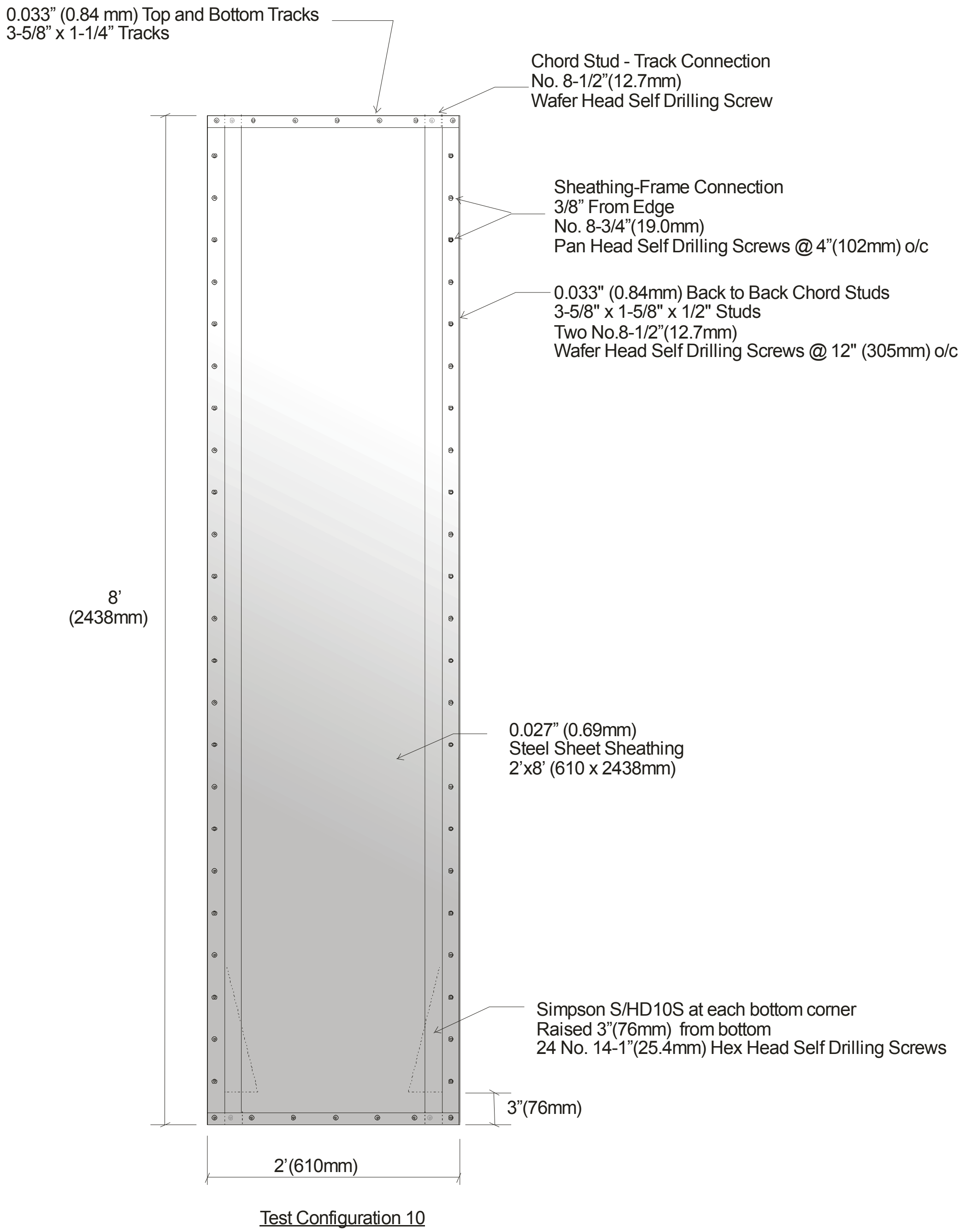
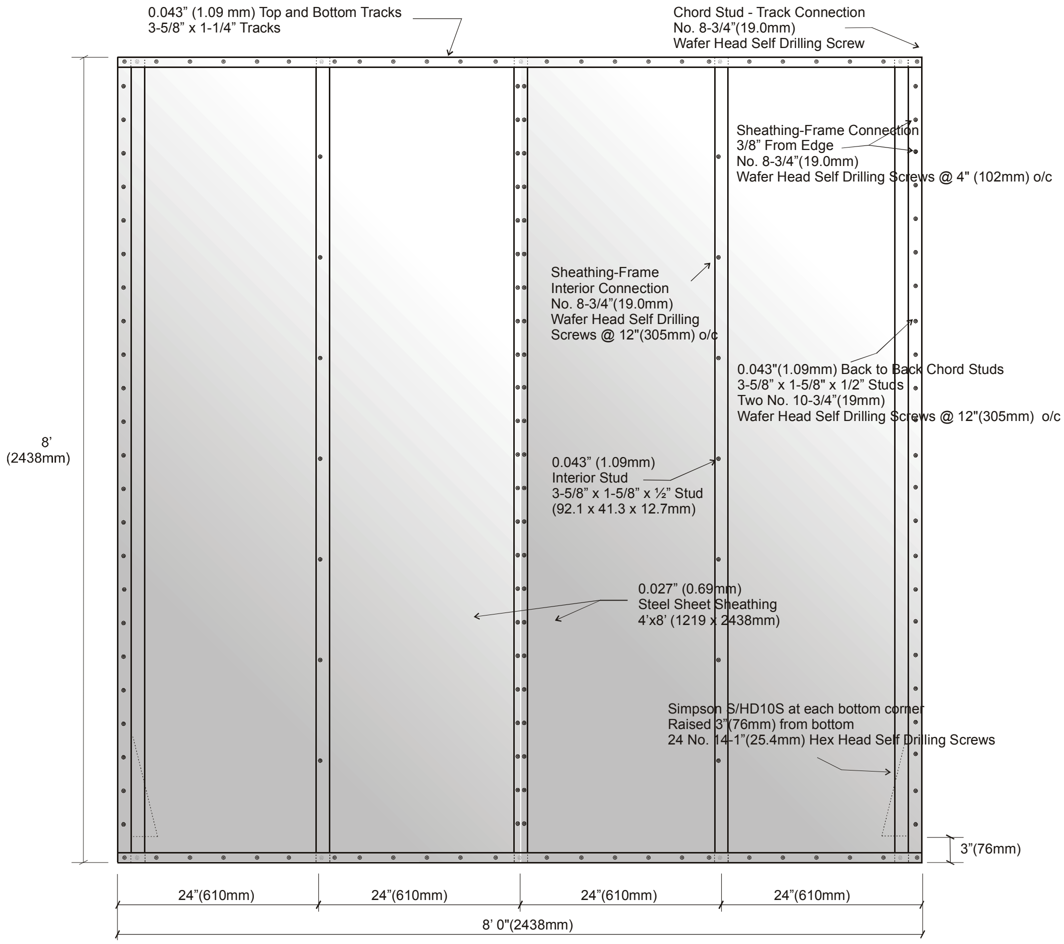


Figure A.6 Nominal Dimensions and Specifications for Test Configuration 10



Test Configuration 11

Figure A.7 Nominal Dimensions and Specifications for Test Configuration 11

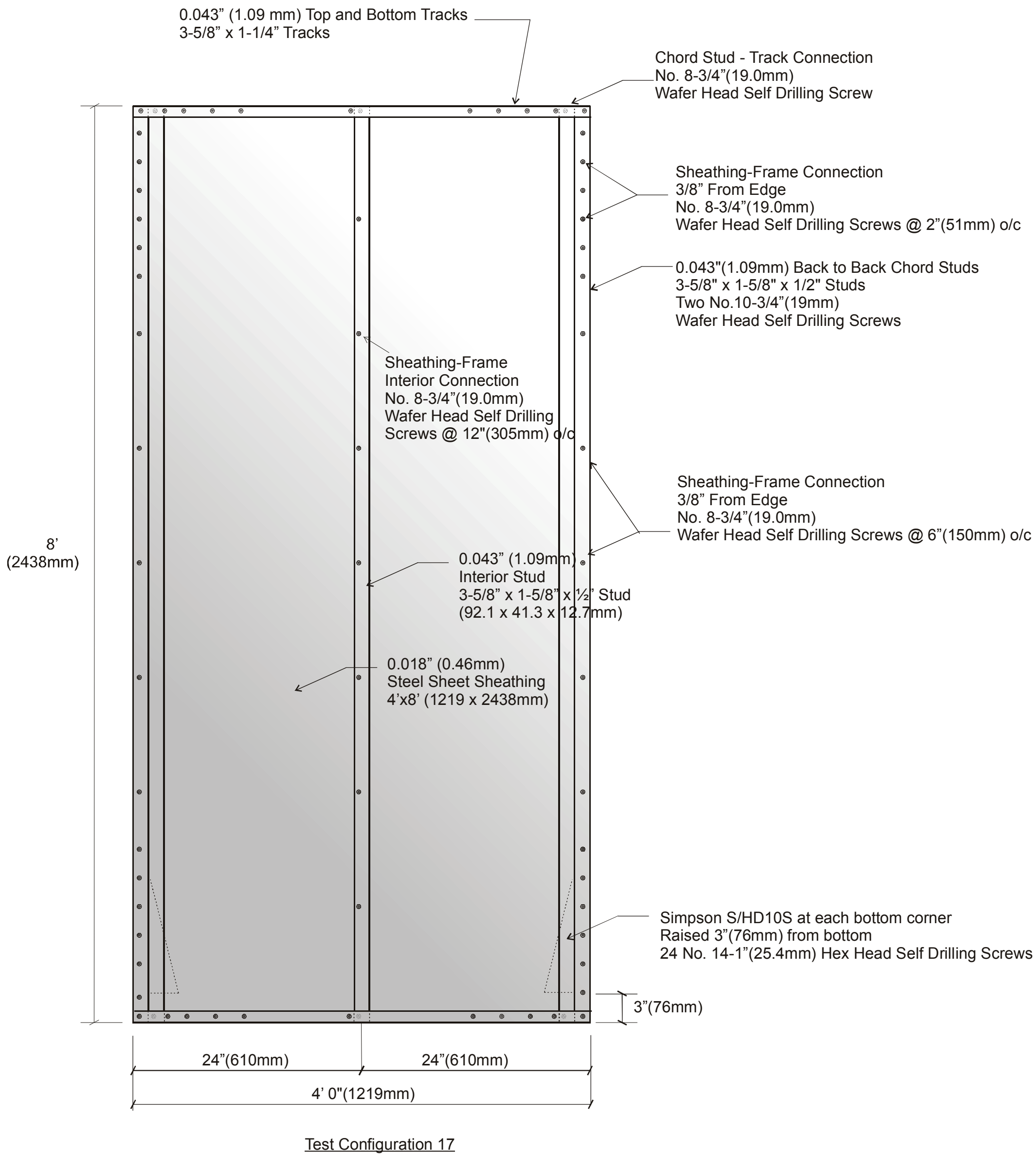


Figure A.8 Nominal Dimensions and Specifications for Test Configuration 17

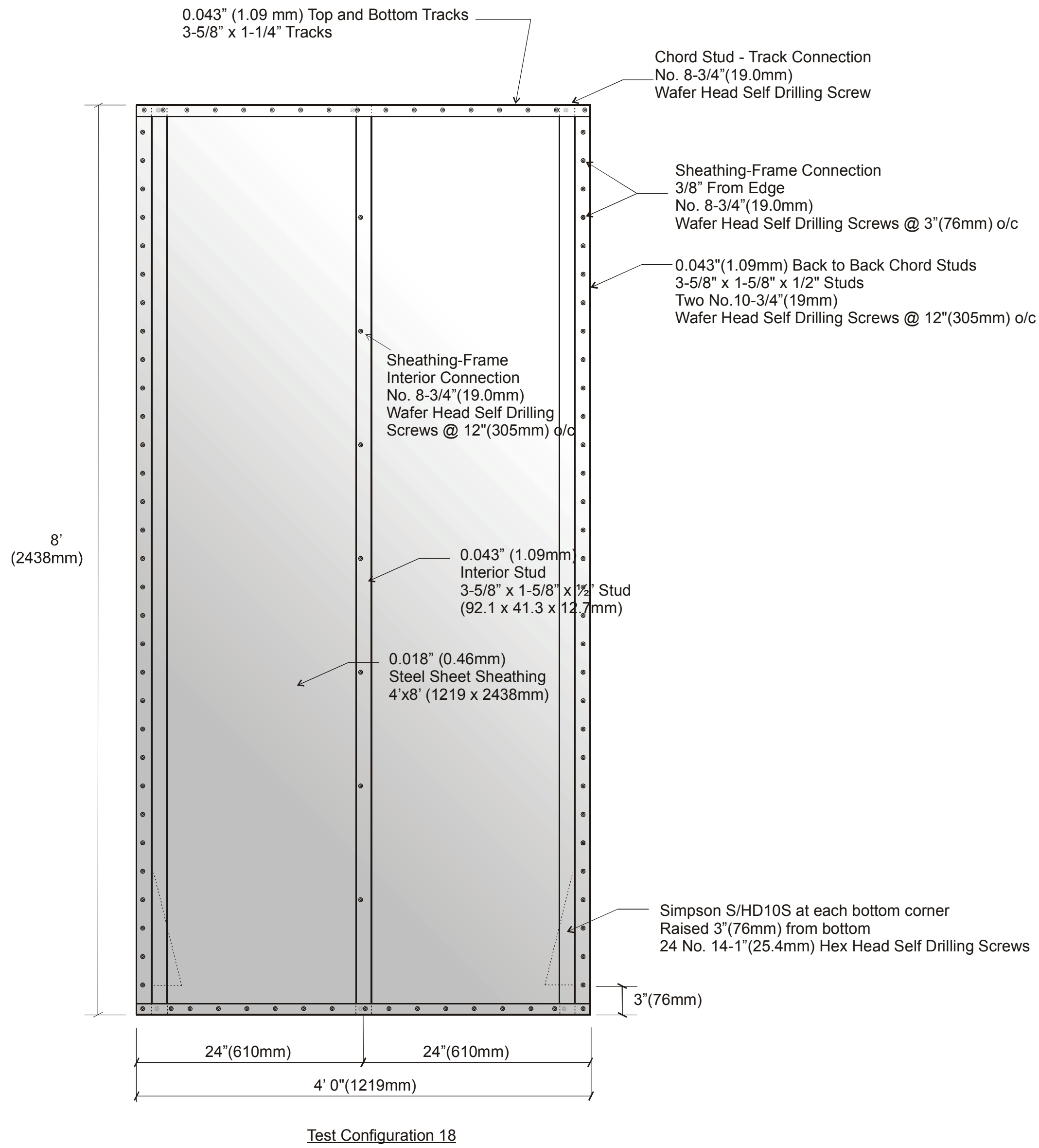


Figure A.9 Nominal Dimensions and Specifications for Test Configuration 18

APPENDIX B

TEST DATA AND OBSERVATION SHEETS

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 1M-a

RESEARCHER: Nisreen Balh **ASSISTANTS:** Cheryl Ong-Tone, Anthony Caruso, Gabriele Rotili

DATE: 14-May-08 **TIME:** 3:45 PM

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

SHEATHING:
[X] 0.018" Sheet Steel 33ksi (230 MPa)
[ ] 0.027" Sheet Steel 33ksi (230 MPa)

Connections
- Sheathing: [X] No.8 gauge 0.75" self-drilling pan head (Grabber Superdrive)
- Framing: [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive
- Hold downs: [X] No.14 gauge 0.75" self-drilling Hex washer head
- Anchor Rods [X] 7/8" rod
- Loading Beam: [X] A325 3/4" bolts [X] 4 bolts [ ] 6 bolts [ ] 12 bolts [ ] Other:
- Base [X] A325 3/4" bolts [X] 4 bolts [ ] 6 bolts [ ] 12 bolts [ ] Other:
- Back-to-Back
- Chord Studs: [X] No.8 gauge 0.75" self-drilling wafer head (2@12" O.C.)

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

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

STUDS:
[ ] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.033" thickness) 33ksi (230 MPa)
[X] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.043" thickness) 33ksi (230 MPa)
[X] Double chord studs used at each end
[ ] Other

STUD SPACING: [X] 24" O.C.

TRACK:
Web: 3-5/8" inches [ ] (0.033" thickness) 33ksi (230 MPa)
Flange: 1-1/4" inches [X] (0.043" thickness) 33ksi (230 MPa)

HOLD DOWNS:
[X] Simpson Strong-Tie S/HD10S (# of screws): 24
[ ] Other

TEST PROTOCOL AND DESCRIPTION:
[X] Monotonic rate of loading 2.5mm/min
[ ] Cyclic

LVDT MEASUREMENTS:
[X] Actuator LVDT [X] North Uplift
[X] North Slip [X] South Uplift
[X] South Slip [X] Top of Wall Lateral **TOTAL:** 6

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 10 scan/sec

COMMENTS:
-Shear anchors torqued for 10s with impact wrench
-Hold down anchors at approximately 7.5 kN (load cells used on both hold-downs)
-Ambient temperature 23 °C
-Square plate washers (2.5"x2.5") used in all top track connections

Figure B.1 Data Sheet for Test 1M-a

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 1M-b

RESEARCHER: Nisreen Balh **ASSISTANTS:** Cheryl Ong-Tone, Anthony Caruso, Gabriele Rotili

DATE: 16-May-08 **TIME:** 10:00AM

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

Sheathing one side

SHEATHING:
- [X] 0.018" Sheet Steel 33ksi (230 MPa)
- [ ] 0.027" Sheet Steel 33ksi (230 MPa)

Connections

Sheathing:	[X] No.8 gauge 0.75" self-drilling pan head (Grabber Superdrive)					
Framing:	[X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive					
Hold downs:	[X] No.14 gauge 0.75" self-drilling Hex washer head					
Anchor Rods	[X] 7/8" rod					
Loading Beam:	[X] A325 3/4" bolts	[X] 4 bolts	[ ] 6 bolts	[ ] 12 bolts	[ ] Other:	
Base	[X] A325 3/4" bolts	[X] 4 bolts	[ ] 6 bolts	[ ] 12 bolts	[ ] Other:	
Back-to-Back Chord Studs:	[X] No.8 gauge 0.75" self-drilling wafer head (2@12" O.C.)					

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

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

STUDS:
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.033" thickness) 33ksi (230 MPa)
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.043" thickness) 33ksi (230 MPa)
- [X] Double chord studs used at each end
- [ ] Other

STUD SPACING: [X] 24" O.C.

TRACK:
- Web: 3-5/8" inches — [ ] (0.033" thickness) 33ksi (230 MPa)
- Flange: 1-1/4" inches — [X] (0.043" thickness) 33ksi (230 MPa)

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10S (# of screws): 24
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [X] Monotonic — rate of loading 2.5mm/min
- [ ] Cyclic

LVDT MEASUREMENTS:
- [X] Actuator LVDT [X] North Uplift
- [X] North Slip [X] South Uplift
- [X] South Slip [X] Top of Wall Lateral **TOTAL:** 6

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 10 scan/sec

COMMENTS:
- -Shear anchors torqued for 10s with impact wrench
- -Hold down anchors at approximately 7.5 kN (load cells used on both hold-downs)
- -Ambient temperature 23 °C
- -Square plate washers (2.5"x2.5") used in all top track connections

Figure B.2 Data Sheet for Test 1M-b

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 1M-c

RESEARCHER: Nisreen Balh **ASSISTANTS:** Cheryl Ong-Tone, Anthony Caruso, Gabriele Rotili

DATE: 21-May-08 **TIME:** 9:30AM

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

SHEATHING:
- [X] 0.018" Sheet Steel 33ksi (230 MPa)
- [ ] 0.027" Sheet Steel 33ksi (230 MPa)

Connections

Sheathing:	X	No.8 gauge 0.75" self-drilling pan head (Grabber Superdrive)				
Framing:	X	No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive				
Hold downs:	X	No.14 gauge 0.75" self-drilling Hex washer head				
Anchor Rods	X	7/8" rod				
Loading Beam:	X	A325 3/4" bolts	X 4 bolts	6 bolts	12 bolts	Other:
Base	X	A325 3/4" bolts	X 4 bolts	6 bolts	12 bolts	Other:
Back-to-Back Chord Studs:	X	No.8 gauge 0.75" self-drilling wafer head (2@12" O.C.)				

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

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

STUDS:
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.033" thickness) 33ksi (230 MPa)
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.043" thickness) 33ksi (230 MPa)
- [X] Double chord studs used at each end
- [ ] Other

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

TRACK:
Web: 3-5/8" inches
Flange: 1-1/4" inches
- [ ] (0.033" thickness) 33ksi (230 MPa)
- [X] (0.043" thickness) 33ksi (230 MPa)

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10S (# of screws): 24
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [X] Monotonic rate of loading 2.5mm/min
- [ ] Cyclic

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral

TOTAL: 6

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 10 scan/sec

COMMENTS:
-Shear anchors torqued for 10s with impact wrench
-Hold down anchors at approximately 7.5 kN (load cells used on both hold-downs)
-Ambient temperature 23 °C
-Square plate washers (2.5"x2.5") used in all top track connections

Figure B.3 Data Sheet for Test 1M-c

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 1C-a

RESEARCHER: Nisreen Balh **ASSISTANTS:** Cheryl Ong-Tone, Anthony Caruso, Gabriele Rotili

DATE: 22-May-08 **TIME:** 11:00 AM

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

SHEATHING:
[X] 0.018" Sheet Steel 33ksi (230 MPa)
[ ] 0.027" Sheet Steel 33ksi (230 MPa)

Connections

Sheathing:	[X] No.8 gauge 0.75" self-drilling pan head (Grabber Superdrive)					
Framing:	[X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive					
Hold downs:	[X] No.14 gauge 0.75" self-drilling Hex washer head					
Anchor Rods	[X] 7/8" rod					
Loading Beam:	[X] A325 3/4" bolts	[X] 4 bolts	[ ] 6 bolts	[ ] 12 bolts	[ ] Other:	
Base	[X] A325 3/4" bolts	[X] 4 bolts	[ ] 6 bolts	[ ] 12 bolts	[ ] Other:	
Back-to-Back Chord Studs:	[X] No.8 gauge 0.75" self-drilling wafer head (2@12" O.C.)					

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

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

STUDS:
[ ] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.033" thickness) 33ksi (230 MPa)
[X] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.043" thickness) 33ksi (230 MPa)
[X] Double chord studs used at each end
[ ] Other

STUD SPACING: [X] 24" O.C.

TRACK:
Web: 3-5/8" inches [ ] (0.033" thickness) 33ksi (230 MPa)
Flange: 1-1/4" inches [X] (0.043" thickness) 33ksi (230 MPa)

HOLD DOWNS:
[X] Simpson Strong-Tie S/HD10S (# of screws): 24
[ ] Other

TEST PROTOCOL AND DESCRIPTION:
[ ] Monotonic
[X] Cyclic CUREE cyclic protocol

LVDT MEASUREMENTS:
[X] Actuator LVDT [X] North Uplift
[X] North Slip [X] South Uplift
[X] South Slip [X] Top of Wall Lateral **TOTAL:** 6

DATA ACQ. RECORD RATE: 100 scan/sec **MONITOR RATE:** 100 scan/sec

COMMENTS:
-Shear anchors torqued for 10s with impact wrench
-Hold down anchors at approximately 7.5 kN (load cells used on both hold-downs)
-Ambient temperature 23 °C
-Square plate washers (2.5"x2.5") used in all top track connections

Figure B.4 Data Sheet for Test 1C-a

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 1C-b

RESEARCHER: Nisreen Balh **ASSISTANTS:** Cheryl Ong-Tone, Anthony Caruso, Gabriele Rotili

DATE: 22-May-08 **TIME:** 2:00 PM

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

SHEATHING:
- [X] 0.018" Sheet Steel 33ksi (230 MPa)
- [ ] 0.027" Sheet Steel 33ksi (230 MPa)

Connections
- Sheathing: [X] No.8 gauge 0.75" self-drilling pan head (Grabber Superdrive)
- Framing: [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive
- Hold downs: [X] No.14 gauge 0.75" self-drilling Hex washer head
- Anchor Rods: [X] 7/8" rod
- Loading Beam: [X] A325 3/4" bolts [X] 4 bolts [ ] 6 bolts [ ] 12 bolts [ ] Other:
- Base: [X] A325 3/4" bolts [X] 4 bolts [ ] 6 bolts [ ] 12 bolts [ ] Other:
- Back-to-Back Chord Studs: [X] No.8 gauge 0.75" self-drilling wafer head (2@12" O.C.)

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

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

STUDS:
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.033" thickness) 33ksi (230 MPa)
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.043" thickness) 33ksi (230 MPa)
- [X] Double chord studs used at each end
- [ ] Other

STUD SPACING: [X] 24" O.C.

TRACK:
- Web: 3-5/8" inches [ ] (0.033" thickness) 33ksi (230 MPa)
- Flange: 1-1/4" inches [X] (0.043" thickness) 33ksi (230 MPa)

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10S (# of screws): 24
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [ ] Monotonic
- [X] Cyclic CUREE cyclic protocol

LVDT MEASUREMENTS:
- [X] Actuator LVDT [X] North Uplift
- [X] North Slip [X] South Uplift
- [X] South Slip [X] Top of Wall Lateral **TOTAL:** 6

DATA ACQ. RECORD RATE: 100 scan/sec **MONITOR RATE:** 100 scan/sec

COMMENTS:
- -Shear anchors torqued for 10s with impact wrench
- -Hold down anchors at approximately 7.5 kN (load cells used on both hold-downs)
- -Ambient temperature 23 °C
- -Square plate washers (2.5"x2.5") used in all top track connections

Figure B.5 Data Sheet for Test 1C-b

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 2M-a

RESEARCHER: Nisreen Balh **ASSISTANTS:** Cheryl Ong-Tone, Anthony Caruso, Gabriele Rotili

DATE: 16-May-08 **TIME:** 3:30 PM

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

SHEATHING:
[X] 0.018" Sheet Steel 33ksi (230 MPa)
[ ] 0.027" Sheet Steel 33ksi (230 MPa)

Connections

Sheathing:	[X] No.8 gauge 0.75" self-drilling pan head (Grabber Superdrive)				
Framing:	[X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive				
Hold downs:	[X] No.14 gauge 0.75" self-drilling Hex washer head				
Anchor Rods	[X] 7/8" rod				
Loading Beam:	[X] A325 3/4" bolts	[X] 4 bolts	[ ] 6 bolts	[ ] 12 bolts	[ ] Other:
Base	[X] A325 3/4" bolts	[X] 4 bolts	[ ] 6 bolts	[ ] 12 bolts	[ ] Other:
Back-to-Back					
Chord Studs:	[X] No.8 gauge 0.75" self-drilling wafer head (2@12" O.C.)				

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

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

STUDS:
[ ] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.033" thickness) 33ksi (230 MPa)
[X] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.043" thickness) 33ksi (230 MPa)
[X] Double chord studs used at each end
[ ] Other

STUD SPACING: [X] 24" O.C.

TRACK:
Web: 3-5/8" inches [ ] (0.033" thickness) 33ksi (230 MPa)
Flange: 1-1/4" inches [X] (0.043" thickness) 33ksi (230 MPa)

HOLD DOWNS:
[X] Simpson Strong-Tie S/HD10S (# of screws): 24
[ ] Other

TEST PROTOCOL AND DESCRIPTION:
[X] Monotonic rate of loading 2.5mm/min
[ ] Cyclic

LVDT MEASUREMENTS:
[X] Actuator LVDT [X] North Uplift
[X] North Slip [X] South Uplift
[X] South Slip [X] Top of Wall Lateral **TOTAL:** 6

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 10 scan/sec

COMMENTS:
-Shear anchors torqued for 10s with impact wrench
-Hold down anchors at approximately 7.5 kN (load cells used on both hold-downs)
-Ambient temperature 23 °C
-Square plate washers (2.5"x2.5") used in all top track connections

Figure B.6 Data Sheet for Test 2M-a

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 2M-b

RESEARCHER: Nisreen Balh **ASSISTANTS:** Cheryl Ong-Tone, Anthony Caruso, Gabriele Rotili

DATE: 20-May-08 **TIME:** 1:30 PM

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

SHEATHING:
- [X] 0.018" Sheet Steel 33ksi (230 MPa)
- [ ] 0.027" Sheet Steel 33ksi (230 MPa)

Connections
- Sheathing: [X] No.8 gauge 0.75" self-drilling pan head (Grabber Superdrive)
- Framing: [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive
- Hold downs: [X] No.14 gauge 0.75" self-drilling Hex washer head
- Anchor Rods: [X] 7/8" rod
- Loading Beam: [X] A325 3/4" bolts [X] 4 bolts [ ] 6 bolts [ ] 12 bolts [ ] Other:
- Base: [X] A325 3/4" bolts [X] 4 bolts [ ] 6 bolts [ ] 12 bolts [ ] Other:
- Back-to-Back Chord Studs: [X] No.8 gauge 0.75" self-drilling wafer head (2@12" O.C.)

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

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

STUDS:
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.033" thickness) 33ksi (230 MPa)
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.043" thickness) 33ksi (230 MPa)
- [X] Double chord studs used at each end
- [ ] Other

STUD SPACING: [X] 24" O.C.

TRACK:
- Web: 3-5/8" inches [ ] (0.033" thickness) 33ksi (230 MPa)
- Flange: 1-1/4" inches [X] (0.043" thickness) 33ksi (230 MPa)

HOLD DOWNS: [X] Simpson Strong-Tie S/HD10S (# of screws): 24
[ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [X] Monotonic rate of loading 2.5mm/min
- [ ] Cyclic

LVDT MEASUREMENTS:
- [X] Actuator LVDT [X] North Uplift
- [X] North Slip [X] South Uplift
- [X] South Slip [X] Top of Wall Lateral **TOTAL:** 6

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 10 scan/sec

COMMENTS:
-Shear anchors torqued for 10s with impact wrench
-Hold down anchors at approximately 7.5 kN (load cells used on both hold-downs)
-Ambient temperature 23 °C
-Square plate washers (2.5"x2.5") used in all top track connections

Figure B.7 Data Sheet for Test 2M-b

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 2C-a

RESEARCHER: Nisreen Balh **ASSISTANTS:** Cheryl Ong-Tone, Anthony Caruso, Gabriele Rotili

DATE: 22-May-08 **TIME:** 4:00 PM

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

SHEATHING:
- [X] 0.018" Sheet Steel 33ksi (230 MPa)
- [ ] 0.027" Sheet Steel 33ksi (230 MPa)

Connections

Sheathing:	X No.8 gauge 0.75" self-drilling pan head (Grabber Superdrive)					
Framing:	X No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive					
Hold downs:	X No.14 gauge 0.75" self-drilling Hex washer head					
Anchor Rods	X 7/8" rod					
Loading Beam:	X A325 3/4" bolts	X 4 bolts	6 bolts	12 bolts	Other:	
Base	X A325 3/4" bolts	X 4 bolts	6 bolts	12 bolts	Other:	
Back-to-Back Chord Studs:	X No.8 gauge 0.75" self-drilling wafer head (2@12" O.C.)					

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

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

STUDS:
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.033" thickness) 33ksi (230 MPa)
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.043" thickness) 33ksi (230 MPa)
- [X] Double chord studs used at each end
- [ ] Other

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

TRACK:
Web: 3-5/8" inches
Flange: 1-1/4" inches
- [ ] (0.033" thickness) 33ksi (230 MPa)
- [X] (0.043" thickness) 33ksi (230 MPa)

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10S (# of screws): 24
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [ ] Monotonic
- [X] Cyclic CUREE cyclic protocol

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Uplift
- [X] North Slip
- [X] South Uplift
- [X] South Slip
- [X] Top of Wall Lateral

TOTAL: 6

DATA ACQ. RECORD RATE: 100 scan/sec **MONITOR RATE:** 100 scan/sec

COMMENTS:
-Shear anchors torqued for 10s with impact wrench
-Hold down anchors at approximately 7.5 kN (load cells used on both hold-downs)
-Ambient temperature 23 °C
-Square plate washers (2.5"x2.5") used in all top track connections

Figure B.8 Data Sheet for Test 2C-a

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 2C-b

RESEARCHER: Nisreen Balh **ASSISTANTS:** Cheryl Ong-Tone, Anthony Caruso, Gabriele Rotili

DATE: 23-May-08 **TIME:** 10:00 AM

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

SHEATHING:
[X] 0.018" Sheet Steel 33ksi (230 MPa)
[ ] 0.027" Sheet Steel 33ksi (230 MPa)

Connections

Sheathing:	[X] No.8 gauge 0.75" self-drilling pan head (Grabber Superdrive)				
Framing:	[X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive				
Hold downs:	[X] No.14 gauge 0.75" self-drilling Hex washer head				
Anchor Rods	[X] 7/8" rod				
Loading Beam:	[X] A325 3/4" bolts	[X] 4 bolts	[ ] 6 bolts	[ ] 12 bolts	[ ] Other:
Base	[X] A325 3/4" bolts	[X] 4 bolts	[ ] 6 bolts	[ ] 12 bolts	[ ] Other:
Back-to-Back Chord Studs:	[X] No.8 gauge 0.75" self-drilling wafer head (2@12" O.C.)				

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

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

STUDS:
[ ] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.033" thickness) 33ksi (230 MPa)
[X] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.043" thickness) 33ksi (230 MPa)
[X] Double chord studs used at each end
[ ] Other

STUD SPACING: [X] 24" O.C.

TRACK:
Web: 3-5/8" inches [ ] (0.033" thickness) 33ksi (230 MPa)
Flange: 1-1/4" inches [X] (0.043" thickness) 33ksi (230 MPa)

HOLD DOWNS:
[X] Simpson Strong-Tie S/HD10S (# of screws): 24
[ ] Other

TEST PROTOCOL AND DESCRIPTION:
[ ] Monotonic
[X] Cyclic CUREE cyclic protocol

LVDT MEASUREMENTS:
[X] Actuator LVDT [X] North Uplift
[X] North Slip [X] South Uplift
[X] South Slip [X] Top of Wall Lateral **TOTAL:** 6

DATA ACQ. RECORD RATE: 100 scan/sec **MONITOR RATE:** 100 scan/sec

COMMENTS:
-Shear anchors torqued for 10s with impact wrench
-Hold down anchors at approximately 7.5 kN (load cells used on both hold-downs)
-Ambient temperature 23 °C
-Square plate washers (2.5"x2.5") used in all top track connections

Figure B.9 Data Sheet for Test 2C-b

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 3M-a

RESEARCHER: Nisreen Balh **ASSISTANTS:** Cheryl Ong-Tone, Anthony Caruso, Gabriele Rotili

DATE: 21-May-08 **TIME:** 1:00 PM

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

SHEATHING:
- [X] 0.018" Sheet Steel 33ksi (230 MPa)
- [ ] 0.027" Sheet Steel 33ksi (230 MPa)

Connections

Sheathing:	[X] No.8 gauge 0.75" self-drilling pan head (Grabber Superdrive)					
Framing:	[X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive					
Hold downs:	[X] No.14 gauge 0.75" self-drilling Hex washer head					
Anchor Rods	[X] 7/8" rod					
Loading Beam:	[X] A325 3/4" bolts	[X] 4 bolts	[ ] 6 bolts	[ ] 12 bolts	[ ] Other:	
Base	[X] A325 3/4" bolts	[X] 4 bolts	[ ] 6 bolts	[ ] 12 bolts	[ ] Other:	
Back-to-Back Chord Studs:	[X] No.8 gauge 0.75" self-drilling wafer head (2@12" O.C.)					

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

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

STUDS:
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.033" thickness) 33ksi (230 MPa)
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.043" thickness) 33ksi (230 MPa)
- [X] Double chord studs used
- [ ] Other

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

TRACK:
Web: 3-5/8" inches
Flange: 1-1/4" inches
- [X] (0.033" thickness) 33ksi (230 MPa)
- [ ] (0.043" thickness) 33ksi (230 MPa)

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10S (# of screws): 24
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [X] Monotonic rate of loading 2.5mm/min
- [ ] Cyclic

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral

TOTAL: 6

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 10 scan/sec

COMMENTS:
-Shear anchors torqued for 10s with impact wrench
-Hold down anchors at approximately 7.5 kN (load cells used on both hold-downs)
-Ambient temperature 23 °C
-Square plate washers (2.5"x2.5") used in all top track connections

Figure B.10 Data Sheet for Test 3M-a

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 3M-b

RESEARCHER: Nisreen Balh **ASSISTANTS:** Cheryl Ong-Tone, Anthony Caruso, Gabriele Rotili

DATE: 21-May-08 **TIME:** 3:10 PM

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

SHEATHING:
[X] 0.018" Sheet Steel 33ksi (230 MPa)
[ ] 0.027" Sheet Steel 33ksi (230 MPa)

Connections
Sheathing: [X] No.8 gauge 0.75" self-drilling pan head (Grabber Superdrive)
Framing: [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive
Hold downs: [X] No.14 gauge 0.75" self-drilling Hex washer head
Anchor Rods [X] 7/8" rod
Loading Beam: [X] A325 3/4" bolts [X] 4 bolts [ ] 6 bolts [ ] 12 bolts [ ] Other:
Base [X] A325 3/4" bolts [X] 4 bolts [ ] 6 bolts [ ] 12 bolts [ ] Other:
Back-to-Back
Chord Studs: [X] No.8 gauge 0.75" self-drilling wafer head (2@12" O.C.)

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

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

STUDS:
[X] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.033" thickness) 33ksi (230 MPa)
[ ] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.043" thickness) 33ksi (230 MPa)
[X] Double chord studs used at each end
[ ] Other

STUD SPACING: [X] 24" O.C.

TRACK:
Web: 3-5/8" inches [X] (0.033" thickness) 33ksi (230 MPa)
Flange: 1-1/4" inches [ ] (0.043" thickness) 33ksi (230 MPa)

HOLD DOWNS:
[X] Simpson Strong-Tie S/HD10S (# of screws): 24
[ ] Other

TEST PROTOCOL AND DESCRIPTION:
[X] Monotonic rate of loading 2.5mm/min
[ ] Cyclic

LVDT MEASUREMENTS:
[X] Actuator LVDT [X] North Uplift
[X] North Slip [X] South Uplift
[X] South Slip [X] Top of Wall Lateral **TOTAL:** 6

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 10 scan/sec

COMMENTS:
-Shear anchors torqued for 10s with impact wrench
-Hold down anchors at approximately 7.5 kN (load cells used on both hold-downs)
-Ambient temperature 23 °C
-Anchor Rods used at loading beam North and South ends
-Square plate washers (2.5"x2.5") used in all top track connections

Figure B.11 Data Sheet for Test 3M-b

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 3C-a

RESEARCHER: Nisreen Balh **ASSISTANTS:** Cheryl Ong-Tone, Anthony Caruso, Gabriele Rotili

DATE: 23-May-08 **TIME:** 11:45AM

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

SHEATHING:
- [X] 0.018" Sheet Steel 33ksi (230 MPa)
- [ ] 0.027" Sheet Steel 33ksi (230 MPa)

Connections
- Sheathing: [X] No.8 gauge 0.75" self-drilling pan head (Grabber Superdrive)
- Framing: [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive
- Hold downs: [X] No.14 gauge 0.75" self-drilling Hex washer head
- Anchor Rods: [X] 7/8" rod
- Loading Beam: [X] A325 3/4" bolts [X] 4 bolts [ ] 6 bolts [ ] 12 bolts [ ] Other:
- Base: [X] A325 3/4" bolts [X] 4 bolts [ ] 6 bolts [ ] 12 bolts [ ] Other:
- Back-to-Back Chord Studs: [X] No.8 gauge 0.75" self-drilling wafer head (2@12" O.C.)

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

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

STUDS:
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.033" thickness) 33ksi (230 MPa)
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.043" thickness) 33ksi (230 MPa)
- [X] Double chord studs used at each end
- [ ] Other

STUD SPACING: [X] 24" O.C.

TRACK:
- Web: 3-5/8" inches [X] (0.033" thickness) 33ksi (230 MPa)
- Flange: 1-1/4" inches [ ] (0.043" thickness) 33ksi (230 MPa)

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10S (# of screws): 24
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [ ] Monotonic
- [X] Cyclic CUREE cyclic protocol

LVDT MEASUREMENTS:
- [X] Actuator LVDT [X] North Uplift
- [X] North Slip [X] South Uplift
- [X] South Slip [X] Top of Wall Lateral **TOTAL:** 6

DATA ACQ. RECORD RATE: 100 scan/sec **MONITOR RATE:** 100 scan/sec

COMMENTS:
- -Shear anchors torqued for 10s with impact wrench
- -Hold down anchors at approximately 7.5 kN (load cells used on both hold-downs)
- -Ambient temperature 23 °C
- -Square plate washers (2.5"x2.5") used in all top track connections

Figure B.12 Data Sheet for Test 3C-a

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 3C-b

RESEARCHER: Nisreen Balh **ASSISTANTS:** Cheryl Ong-Tone, Anthony Caruso, Gabriele Rotili

DATE: 23-May-08 **TIME:** 2:30 PM

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

SHEATHING:
- [X] 0.018" Sheet Steel 33ksi (230 MPa)
- [ ] 0.027" Sheet Steel 33ksi (230 MPa)

Connections

Sheathing:	[X] No.8 gauge 0.75" self-drilling pan head (Grabber Superdrive)					
Framing:	[X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive					
Hold downs:	[X] No.14 gauge 0.75" self-drilling Hex washer head					
Anchor Rods	[X] 7/8" rod					
Loading Beam:	[X] A325 3/4" bolts	[X] 4 bolts	[ ] 6 bolts	[ ] 12 bolts	[ ] Other:	
Base	[X] A325 3/4" bolts	[X] 4 bolts	[ ] 6 bolts	[ ] 12 bolts	[ ] Other:	
Back-to-Back						
Chord Studs:	[X] No.8 gauge 0.75" self-drilling wafer head (2@12" O.C.)					

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

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

STUDS:
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.033" thickness) 33ksi (230 MPa)
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.043" thickness) 33ksi (230 MPa)
- [X] Double chord studs used at each end
- [ ] Other

STUD SPACING: [X] 24" O.C.

TRACK:
- Web: 3-5/8" inches — [X] (0.033" thickness) 33ksi (230 MPa)
- Flange: 1-1/4" inches — [ ] (0.043" thickness) 33ksi (230 MPa)

HOLD DOWNS: [X] Simpson Strong-Tie S/HD10S (# of screws): 24; [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [ ] Monotonic
- [X] Cyclic — CUREE cyclic protocol

LVDT MEASUREMENTS:
- [X] Actuator LVDT — [X] North Uplift
- [X] North Slip — [X] South Uplift
- [X] South Slip — [X] Top of Wall Lateral

TOTAL: 6

DATA ACQ. RECORD RATE: 100 scan/sec **MONITOR RATE:** 100 scan/sec

COMMENTS:
-Shear anchors torqued for 10s with impact wrench
-Hold down anchors at approximately 7.5 kN (load cells used on both hold-downs)
-Ambient temperature 23 °C
-Square plate washers (2.5"x2.5") used in all top track connections

Figure B.13 Data Sheet for Test 3C-b

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST:	3C-c		
RESEARCHER:	Nisreen Balh	**ASSISTANTS:**	Cheryl Ong-Tone, Anthony Caruso, Gabriele Rotili
DATE:	09-Oct-08	**TIME:**	4:30 PM
DIMENSIONS OF WALL:	4 FT X 8 FT	**PANEL ORIENTATION:**	Vertical Sheathing one side

SHEATHING:
- [X] 0.018" Sheet Steel 33ksi (230 MPa)
- [ ] 0.027" Sheet Steel 33ksi (230 MPa)

Connections
- Sheathing: [X] No.8 gauge 0.75" self-drilling pan head (Grabber Superdrive)
- Framing: [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive
- Hold downs: [X] No.14 gauge 0.75" self-drilling Hex washer head
- Anchor Rods: [X] 7/8" rod
- Loading Beam: [X] A325 3/4" bolts — [X] 4 bolts [ ] 6 bolts [ ] 12 bolts [ ] Other:
- Base: [X] A325 3/4" bolts — [X] 4 bolts [ ] 6 bolts [ ] 12 bolts [ ] Other:
- Back-to-Back
- Chord Studs: [X] No.8 gauge 0.75" self-drilling wafer head (2@12" O.C.)

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

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

STUDS:
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.033" thickness) 33ksi (230 MPa)
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.043" thickness) 33ksi (230 MPa)
- [X] Double chord studs used at each end
- [ ] Other

STUD SPACING: [X] 24" O.C.

TRACK:
- Web: 3-5/8" inches — [X] (0.033" thickness) 33ksi (230 MPa)
- Flange: 1-1/4" inches — [ ] (0.043" thickness) 33ksi (230 MPa)

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10S (# of screws): 24
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [ ] Monotonic
- [X] Cyclic — CUREE cyclic protocol

LVDT MEASUREMENTS:
- [X] Actuator LVDT [X] North Uplift
- [X] North Slip [X] South Uplift
- [X] South Slip [X] Top of Wall Lateral

TOTAL: 6

DATA ACQ. RECORD RATE: 100 scan/sec **MONITOR RATE:** 100 scan/sec

COMMENTS:
-Shear anchors torqued for 10s with impact wrench
-Hold down anchors at approximately 7.5 kN (load cells used on both hold-downs)
-Ambient temperature 23 °C
-Square plate washers (2.5"x2.5") used in all top track connections

Figure B.14 Data Sheet for Test 3C-c

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 8M-a

RESEARCHER: Nisreen Balh **ASSISTANTS:** Cheryl Ong-Tone, Anthony Caruso, Gabriele Rotili

DATE: 9-Jun-08 **TIME:** 9:30 AM

DIMENSIONS OF WALL: 2 FT X 8 FT **PANEL ORIENTATION:** Vertical
Sheathing one side

SHEATHING:
[ ] 0.018" Sheet Steel 33ksi (230 MPa)
[X] 0.027" Sheet Steel 33ksi (230 MPa)

Connections
Sheathing: [X] No.8 gauge 0.75" self-drilling pan head (Grabber Superdrive)
Framing: [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive
Hold downs: [X] No.14 gauge 0.75" self-drilling Hex washer head
Anchor Rods [X] 7/8" rod
Loading Beam: [X] A325 3/4" bolts [ ] 4 bolts [ ] 6 bolts [ ] 12 bolts [X] Other: 1
Base [X] A325 3/4" bolts [ ] 4 bolts [ ] 6 bolts [ ] 12 bolts [X] Other: 1
Back-to-Back
Chord Studs: [X] No.8 gauge 0.75" self-drilling wafer head (2@12" O.C.)

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

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

STUDS:
[ ] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.033" thickness) 33ksi (230 MPa)
[X] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.043" thickness) 33ksi (230 MPa)
[X] Double chord studs used at each end
[ ] Other

STUD SPACING: [X] 24" O.C.

TRACK:
Web: 3-5/8" inches [ ] (0.033" thickness) 33ksi (230 MPa)
Flange: 1-1/4" inches [X] (0.043" thickness) 33ksi (230 MPa)

HOLD DOWNS:
[X] Simpson Strong-Tie S/HD10S (# of screws): 24
[ ] Other

TEST PROTOCOL AND DESCRIPTION:
[X] Monotonic rate of loading 2.5mm/min
[ ] Cyclic

LVDT MEASUREMENTS:
[X] Actuator LVDT [X] North Uplift
[X] North Slip [X] South Uplift
[X] South Slip [X] Top of Wall Lateral **TOTAL:** 6

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 10 scan/sec

COMMENTS:
-Shear anchors torqued for 10s with impact wrench
-Hold down anchors at approximately 7.5 kN (load cells used on both hold-downs)
-Ambient temperature 23 °C
-Square plate washers (2.5"x2.5") used in all top track connections

Figure B.15 Data Sheet for Test 8M-a

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 8M-b

RESEARCHER: Nisreen Balh **ASSISTANTS:** Cheryl Ong-Tone, Anthony Caruso, Gabriele Rotili

DATE: 9-Jun-08 **TIME:** 11:30 AM

DIMENSIONS OF WALL: 2 FT X 8 FT **PANEL ORIENTATION:** Vertical

Sheathing one side

SHEATHING:
- [ ] 0.018" Sheet Steel 33ksi (230 MPa)
- [X] 0.027" Sheet Steel 33ksi (230 MPa)

Connections

Sheathing:	X	No.8 gauge 0.75" self-drilling pan head (Grabber Superdrive)
Framing:	X	No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive
Hold downs:	X	No.14 gauge 0.75" self-drilling Hex washer head
Anchor Rods	X	7/8" rod
Loading Beam:	X	A325 3/4" bolts; [ ] 4 bolts; [ ] 6 bolts; [ ] 12 bolts; [X] Other: 1
Base	X	A325 3/4" bolts; [ ] 4 bolts; [ ] 6 bolts; [ ] 12 bolts; [X] Other: 1
Back-to-Back Chord Studs:	X	No.8 gauge 0.75" self-drilling wafer head (2@12" O.C.)

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

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

STUDS:
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.033" thickness) 33ksi (230 MPa)
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.043" thickness) 33ksi (230 MPa)
- [X] Double chord studs used at each end
- [ ] Other

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

TRACK:

Web: 3-5/8" inches
Flange: 1-1/4" inches

- [ ] (0.033" thickness) 33ksi (230 MPa)
- [X] (0.043" thickness) 33ksi (230 MPa)

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10S (# of screws): 24
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [X] Monotonic rate of loading 2.5mm/min
- [ ] Cyclic

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral

TOTAL: 6

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 10 scan/sec

COMMENTS:
-Shear anchors torqued for 10s with impact wrench
-Hold down anchors at approximately 7.5 kN (load cells used on both hold-downs)
-Ambient temperature 23 °C
-Square plate washers (2.5"x2.5") used in all top track connections

Figure B.16 Data Sheet for Test 8M-b

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 8C-a

RESEARCHER: Nisreen Balh **ASSISTANTS:** Cheryl Ong-Tone, Anthony Caruso, Gabriele Rotili

DATE: 10-Jun-08 **TIME:** 2:30 PM

DIMENSIONS OF WALL: 2 FT X 8 FT **PANEL ORIENTATION:** Vertical
Sheathing one side

SHEATHING:
[ ] 0.018" Sheet Steel 33ksi (230 MPa)
[X] 0.027" Sheet Steel 33ksi (230 MPa)

Connections
- Sheathing: [X] No.8 gauge 0.75" self-drilling pan head (Grabber Superdrive)
- Framing: [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive
- Hold downs: [X] No.14 gauge 0.75" self-drilling Hex washer head
- Anchor Rods [X] 7/8" rod
- Loading Beam: [X] A325 3/4" bolts [ ] 4 bolts [ ] 6 bolts [ ] 12 bolts [X] Other: 1
- Base [X] A325 3/4" bolts [ ] 4 bolts [ ] 6 bolts [ ] 12 bolts [X] Other: 1
- Back-to-Back
- Chord Studs: [X] No.8 gauge 0.75" self-drilling wafer head (2@12" O.C.)

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

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

STUDS:
[ ] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.033" thickness) 33ksi (230 MPa)
[X] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.043" thickness) 33ksi (230 MPa)
[X] Double chord studs used at each end
[ ] Other

STUD SPACING: [X] 24" O.C.

TRACK:
Web: 3-5/8" inches [ ] (0.033" thickness) 33ksi (230 MPa)
Flange: 1-1/4" inches [X] (0.043" thickness) 33ksi (230 MPa)

HOLD DOWNS:
[X] Simpson Strong-Tie S/HD10S (# of screws): 24
[ ] Other

TEST PROTOCOL AND DESCRIPTION:
[ ] Monotonic
[X] Cyclic CUREE cyclic protocol

LVDT MEASUREMENTS:
[X] Actuator LVDT [X] North Uplift
[X] North Slip [X] South Uplift
[X] South Slip [X] Top of Wall Lateral **TOTAL:** 6

DATA ACQ. RECORD RATE: 100 scan/sec **MONITOR RATE:** 100 scan/sec

COMMENTS:
-Shear anchors torqued for 10s with impact wrench
-Hold down anchors at approximately 7.5 kN (load cells used on both hold-downs)
-Ambient temperature 23 °C
-Square plate washers (2.5"x2.5") used in all top track connections

Figure B.17 Data Sheet for Test 8C-a

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 8C-b

RESEARCHER: Nisreen Balh **ASSISTANTS:** Cheryl Ong-Tone, Anthony Caruso, Gabriele Rotili

DATE: 11-Jun-08 **TIME:** 12:00 PM

DIMENSIONS OF WALL: 2 FT X 8 FT **PANEL ORIENTATION:** Vertical
Sheathing one side

SHEATHING:
[ ] 0.018" Sheet Steel 33ksi (230 MPa)
[X] 0.027" Sheet Steel 33ksi (230 MPa)

Connections
Sheathing: [X] No.8 gauge 0.75" self-drilling pan head (Grabber Superdrive)
Framing: [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive
Hold downs: [X] No.14 gauge 0.75" self-drilling Hex washer head
Anchor Rods [X] 7/8" rod
Loading Beam: [X] A325 3/4" bolts [ ] 4 bolts [ ] 6 bolts [ ] 12 bolts [X] Other: 1
Base [X] A325 3/4" bolts [ ] 4 bolts [ ] 6 bolts [ ] 12 bolts [X] Other: 1
Back-to-Back
Chord Studs: [X] No.8 gauge 0.75" self-drilling wafer head (2@12" O.C.)

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

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

STUDS:
[ ] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.033" thickness) 33ksi (230 MPa)
[X] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.043" thickness) 33ksi (230 MPa)
[X] Double chord studs used at each end
[ ] Other

STUD SPACING: [X] 24" O.C.

TRACK:
Web: 3-5/8" inches [ ] (0.033" thickness) 33ksi (230 MPa)
Flange: 1-1/4" inches [X] (0.043" thickness) 33ksi (230 MPa)

HOLD DOWNS:
[X] Simpson Strong-Tie S/HD10S (# of screws): 24
[ ] Other

TEST PROTOCOL AND DESCRIPTION:
[ ] Monotonic
[X] Cyclic CUREE cyclic protocol

LVDT MEASUREMENTS:
[X] Actuator LVDT [X] North Uplift
[X] North Slip [X] South Uplift
[X] South Slip [X] Top of Wall Lateral **TOTAL:** 6

DATA ACQ. RECORD RATE: 100 scan/sec **MONITOR RATE:** 100 scan/sec

COMMENTS:
-Shear anchors torqued for 10s with impact wrench
-Hold down anchors at approximately 7.5 kN (load cells used on both hold-downs)
-Ambient temperature 23 °C
-Square plate washers (2.5"x2.5") used in all top track connections

Figure B.18 Data Sheet for Test 8C-b

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 9M-a

RESEARCHER: Nisreen Balh **ASSISTANTS:** Cheryl Ong-Tone, Anthony Caruso, Gabriele Rotili

DATE: 5-Jun-08 **TIME:** 2:30 PM

DIMENSIONS OF WALL: 2 FT X 8 FT **PANEL ORIENTATION:** Vertical
Sheathing one side

SHEATHING:
[ ] 0.018" Sheet Steel 33ksi (230 MPa)
[X] 0.027" Sheet Steel 33ksi (230 MPa)

Connections
Sheathing: [X] No.8 gauge 0.75" self-drilling pan head (Grabber Superdrive)
Framing: [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive
Hold downs: [X] No.14 gauge 0.75" self-drilling Hex washer head
Anchor Rods [X] 7/8" rod
Loading Beam: [X] A325 3/4" bolts [ ] 4 bolts [ ] 6 bolts [ ] 12 bolts [X] Other: 1
Base [X] A325 3/4" bolts [ ] 4 bolts [ ] 6 bolts [ ] 12 bolts [X] Other: 1
Back-to-Back
Chord Studs: [X] No.8 gauge 0.75" self-drilling wafer head (2@12" O.C.)

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

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

STUDS:
[ ] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.033" thickness) 33ksi (230 MPa)
[X] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.043" thickness) 33ksi (230 MPa)
[X] Double chord studs used at each end
[ ] Other

STUD SPACING: [X] 24" O.C.

TRACK:
Web: 3-5/8" inches [ ] (0.033" thickness) 33ksi (230 MPa)
Flange: 1-1/4" inches [X] (0.043" thickness) 33ksi (230 MPa)

HOLD DOWNS:
[X] Simpson Strong-Tie S/HD10S (# of screws): 24
[ ] Other

TEST PROTOCOL AND DESCRIPTION:
[X] Monotonic rate of loading 2.5mm/min
[ ] Cyclic

LVDT MEASUREMENTS:
[X] Actuator LVDT [X] North Uplift
[X] North Slip [X] South Uplift
[X] South Slip [X] Top of Wall Lateral **TOTAL:** 6

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 10 scan/sec

COMMENTS:
-Shear anchors torqued for 10s with impact wrench
-Hold down anchors at approximately 7.5 kN (load cells used on both hold-downs)
-Ambient temperature 23 °C
-Square plate washers (2.5"x2.5") used in all top track connections

Figure B.19 Data Sheet for Test 9M-a

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 9M-b

RESEARCHER: Nisreen Balh **ASSISTANTS:** Cheryl Ong-Tone, Anthony Caruso, Gabriele Rotili

DATE: 6-Jun-08 **TIME:** 10:15 AM

DIMENSIONS OF WALL: 2 FT X 8 FT **PANEL ORIENTATION:** Vertical
Sheathing one side

SHEATHING:
[ ] 0.018" Sheet Steel 33ksi (230 MPa)
[X] 0.027" Sheet Steel 33ksi (230 MPa)

Connections

Sheathing:	[X] No.8 gauge 0.75" self-drilling pan head (Grabber Superdrive)					
Framing:	[X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive					
Hold downs:	[X] No.14 gauge 0.75" self-drilling Hex washer head					
Anchor Rods	[X] 7/8" rod					
Loading Beam:	[X] A325 3/4" bolts	[ ] 4 bolts	[ ] 6 bolts	[ ] 12 bolts	[X] Other:	1
Base	[X] A325 3/4" bolts	[ ] 4 bolts	[ ] 6 bolts	[ ] 12 bolts	[X] Other:	1
Back-to-Back						
Chord Studs:	[X] No.8 gauge 0.75" self-drilling wafer head (2@12" O.C.)					

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

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

STUDS:
[ ] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.033" thickness) 33ksi (230 MPa)
[X] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.043" thickness) 33ksi (230 MPa)
[X] Double chord studs used at each end
[ ] Other

STUD SPACING: [X] 24" O.C.

TRACK:
Web: 3-5/8" inches [ ] (0.033" thickness) 33ksi (230 MPa)
Flange: 1-1/4" inches [X] (0.043" thickness) 33ksi (230 MPa)

HOLD DOWNS: [X] Simpson Strong-Tie S/HD10S (# of screws): 24
[ ] Other

TEST PROTOCOL AND DESCRIPTION:
[X] Monotonic rate of loading 2.5mm/min
[ ] Cyclic

LVDT MEASUREMENTS:
[X] Actuator LVDT [X] North Uplift
[X] North Slip [X] South Uplift
[X] South Slip [X] Top of Wall Lateral **TOTAL:** 6

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 10 scan/sec

COMMENTS:
-Shear anchors torqued for 10s with impact wrench
-Hold down anchors at approximately 7.5 kN (load cells used on both hold-downs)
-Ambient temperature 23 °C
-Square plate washers (2.5"x2.5") used in all top track connections

Figure B.20 Data Sheet for Test 9M-b

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 9M-c

RESEARCHER: Nisreen Balh **ASSISTANTS:** Cheryl Ong-Tone, Anthony Caruso, Gabriele Rotili

DATE: 6-Jun-08 **TIME:** 3:30 PM

DIMENSIONS OF WALL: 2 FT X 8 FT **PANEL ORIENTATION:** Vertical
Sheathing one side

SHEATHING:
[ ] 0.018" Sheet Steel 33ksi (230 MPa)
[X] 0.027" Sheet Steel 33ksi (230 MPa)

Connections
Sheathing: [X] No.8 gauge 0.75" self-drilling pan head (Grabber Superdrive)
Framing: [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive
Hold downs: [X] No.14 gauge 0.75" self-drilling Hex washer head
Anchor Rods [X] 7/8" rod
Loading Beam: [X] A325 3/4" bolts [ ] 4 bolts [ ] 6 bolts [ ] 12 bolts [X] Other: 1
Base [X] A325 3/4" bolts [ ] 4 bolts [ ] 6 bolts [ ] 12 bolts [X] Other: 1
Back-to-Back
Chord Studs: [X] No.8 gauge 0.75" self-drilling wafer head (2@12" O.C.)

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

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

STUDS:
[ ] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.033" thickness) 33ksi (230 MPa)
[X] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.043" thickness) 33ksi (230 MPa)
[X] Double chord studs used at each end
[ ] Other

STUD SPACING: [X] 24" O.C.

TRACK:
Web: 3-5/8" inches [ ] (0.033" thickness) 33ksi (230 MPa)
Flange: 1-1/4" inches [X] (0.043" thickness) 33ksi (230 MPa)

HOLD DOWNS:
[X] Simpson Strong-Tie S/HD10S (# of screws): 24
[ ] Other

TEST PROTOCOL AND DESCRIPTION:
[X] Monotonic rate of loading 2.5mm/min
[ ] Cyclic

LVDT MEASUREMENTS:
[X] Actuator LVDT [X] North Uplift
[X] North Slip [X] South Uplift
[X] South Slip [X] Top of Wall Lateral **TOTAL:** 6

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 10 scan/sec

COMMENTS:
-Shear anchors torqued for 10s with impact wrench
-Hold down anchors at approximately 7.5 kN (load cells used on both hold-downs)
-Ambient temperature 23 °C
-Square plate washers (2.5"x2.5") used in all top track connections
-Bridging installed through all stud cutholes

Figure B.21 Data Sheet for Test 9M-c

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST:	9C-a					
RESEARCHER:	Nisreen Balh		**ASSISTANTS:**	Cheryl Ong-Tone, Anthony Caruso, Gabriele Rotili		
DATE:	10-Jun-08		**TIME:**	9:45 AM		
DIMENSIONS OF WALL:	2 FT X 8 FT		**PANEL ORIENTATION:**	Vertical		
				Sheathing one side		
SHEATHING:	[ ] 0.018" Sheet Steel 33ksi (230 MPa)					
	[X] 0.027" Sheet Steel 33ksi (230 MPa)					
Connections Sheathing:	[X] No.8 gauge 0.75" self-drilling pan head (Grabber Superdrive)					
Framing:	[X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive					
Hold downs:	[X] No.14 gauge 0.75" self-drilling Hex washer head					
Anchor Rods	[X] 7/8" rod					
Loading Beam:	[X] A325 3/4" bolts	[ ] 4 bolts	[ ] 6 bolts	[ ] 12 bolts	[X] Other:	1
Base	[X] A325 3/4" bolts	[ ] 4 bolts	[ ] 6 bolts	[ ] 12 bolts	[X] Other:	1
Back-to-Back Chord Studs:	[X] No.8 gauge 0.75" self-drilling wafer head (2@12" O.C.)					
SHEATHING FASTENER SCHEDULE:	[X] 2"/12"	[ ] 3"/12"	[ ] Other:			
	[ ] 4"/12"	[ ] 6"/12"				
EDGE PANEL DISTANCE:	[X] 3/8"	[ ] 1/2"	[ ] Other:			
STUDS:	[ ] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.033" thickness) 33ksi (230 MPa)					
	[X] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.043" thickness) 33ksi (230 MPa)					
	[X] Double chord studs used at each end					
	[ ] Other					
STUD SPACING:	[X] 24" O.C.					
TRACK:	Web: 3-5/8" inches		[ ] (0.033" thickness) 33ksi (230 MPa)			
	Flange: 1-1/4" inches		[X] (0.043" thickness) 33ksi (230 MPa)			
HOLD DOWNS:	[X] Simpson Strong-Tie S/HD10S		(# of screws):	24		
	[ ] Other					
TEST PROTOCOL AND DESCRIPTION:	[ ] Monotonic					
	[X] Cyclic	CUREE cyclic protocol				
LVDT MEASUREMENTS:	[X] Actuator LVDT	[X] North Uplift				
	[X] North Slip	[X] South Uplift				
	[X] South Slip	[X] Top of Wall Lateral	**TOTAL:** 6			
DATA ACQ. RECORD RATE:	100 scan/sec	**MONITOR RATE:**	100 scan/sec			
COMMENTS:	-Shear anchors torqued for 10s with impact wrench					
	-Hold down anchors at approximately 7.5 kN (load cells used on both hold-downs)					
	-Ambient temperature 23 °C					
	-Square plate washers (2.5"x2.5") used in all top track connections					

Figure B.22 Data Sheet for Test 9C-a

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 9C-b

RESEARCHER: Nisreen Balh **ASSISTANTS:** Cheryl Ong-Tone, Anthony Caruso, Gabriele Rotili

DATE: 10-Jun-08 **TIME:** 11:00 AM

DIMENSIONS OF WALL: 2 FT X 8 FT **PANEL ORIENTATION:** Vertical
Sheathing one side

SHEATHING:
- [ ] 0.018" Sheet Steel 33ksi (230 MPa)
- [X] 0.027" Sheet Steel 33ksi (230 MPa)

Connections

Sheathing:	X No.8 gauge 0.75" self-drilling pan head (Grabber Superdrive)					
Framing:	X No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive					
Hold downs:	X No.14 gauge 0.75" self-drilling Hex washer head					
Anchor Rods	X 7/8" rod					
Loading Beam:	X A325 3/4" bolts	4 bolts	6 bolts	12 bolts	X Other:	1
Base	X A325 3/4" bolts	4 bolts	6 bolts	12 bolts	X Other:	1
Back-to-Back						
Chord Studs:	X No.8 gauge 0.75" self-drilling wafer head (2@12" O.C.)					

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

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

STUDS:
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.033" thickness) 33ksi (230 MPa)
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.043" thickness) 33ksi (230 MPa)
- [X] Double chord studs used at each end
- [ ] Other

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

TRACK:
Web: 3-5/8" inches
Flange: 1-1/4" inches
- [ ] (0.033" thickness) 33ksi (230 MPa)
- [X] (0.043" thickness) 33ksi (230 MPa)

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10S (# of screws): 24
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [ ] Monotonic
- [X] Cyclic CUREE cyclic protocol

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral

TOTAL: 6

DATA ACQ. RECORD RATE: 100 scan/sec **MONITOR RATE:** 100 scan/sec

COMMENTS:
-Shear anchors torqued for 10s with impact wrench
-Hold down anchors at approximately 7.5 kN (load cells used on both hold-downs)
-Ambient temperature 23 °C
-Square plate washers (2.5"x2.5") used in all top track connections

Figure B.23 Data Sheet for Test 9C-b

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 10M-a

RESEARCHER: Nisreen Balh **ASSISTANTS:** Cheryl Ong-Tone, Anthony Caruso, Gabriele Rotili

DATE: 9-Jun-08 **TIME:** 2:45 PM

DIMENSIONS OF WALL: 2 FT X 8 FT **PANEL ORIENTATION:** Vertical

Sheathing one side

SHEATHING:
- [ ] 0.018" Sheet Steel 33ksi (230 MPa)
- [X] 0.027" Sheet Steel 33ksi (230 MPa)

Connections
- Sheathing: [X] No.8 gauge 0.75" self-drilling pan head (Grabber Superdrive)
- Framing: [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive
- Hold downs: [X] No.14 gauge 0.75" self-drilling Hex washer head
- Anchor Rods [X] 7/8" rod
- Loading Beam: [X] A325 3/4" bolts [ ] 4 bolts [ ] 6 bolts [ ] 12 bolts [X] Other: 1
- Base [X] A325 3/4" bolts [ ] 4 bolts [ ] 6 bolts [ ] 12 bolts [X] Other: 1
- Back-to-Back
- Chord Studs: [X] No.8 gauge 0.75" self-drilling wafer head (2@12" O.C.)

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

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

STUDS:
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.033" thickness) 33ksi (230 MPa)
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.043" thickness) 33ksi (230 MPa)
- [X] Double chord studs used
- [ ] Other

STUD SPACING: [X] 24" O.C.

TRACK:
Web: 3-5/8" inches [X] (0.033" thickness) 33ksi (230 MPa)
Flange: 1-1/4" inches [ ] (0.043" thickness) 33ksi (230 MPa)

HOLD DOWNS: [X] Simpson Strong-Tie S/HD10S (# of screws): 24
[ ] Other

TEST PROTOCOL AND DESCRIPTION:
[X] Monotonic rate of loading 2.5mm/min
[ ] Cyclic

LVDT MEASUREMENTS:
[X] Actuator LVDT [X] North Uplift
[X] North Slip [X] South Uplift
[X] South Slip [X] Top of Wall Lateral **TOTAL:** 6

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 10 scan/sec

COMMENTS:
-Shear anchors torqued for 10s with impact wrench
-Hold down anchors at approximately 7.5 kN (load cells used on both hold-downs)
-Ambient temperature 23 °C
-Double chord studs used
-Square plate washers (2.5"x2.5") used in all top track connections

Figure B.24 Data Sheet for Test 10M-a

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 11M-a

RESEARCHER: Nisreen Balh **ASSISTANTS:** Cheryl Ong-Tone, Anthony Caruso, Gabriele Rotili

DATE: 2-Jul-08 **TIME:** 10:15 AM

DIMENSIONS OF WALL: 8 FT X 8 FT **PANEL ORIENTATION:** Vertical, 2 - 4'x8' sheets; Sheathing one side

SHEATHING:
- [ ] 0.018" Sheet Steel 33ksi (230 MPa)
- [X] 0.027" Sheet Steel 33ksi (230 MPa)

Connections
- Sheathing: [X] No.8 gauge 0.75" self-drilling pan head (Grabber Superdrive)
- Framing: [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive
- Hold downs: [X] No.14 gauge 0.75" self-drilling Hex washer head
- Anchor Rods: [X] 7/8" rod
- Loading Beam: [X] A325 3/4" bolts [ ] 4 bolts [ ] 6 bolts [X] 12 bolts [ ] Other:
- Base: [X] A325 3/4" bolts [ ] 4 bolts [X] 6 bolts [ ] 12 bolts [ ] Other:
- Back-to-Back Chord Studs: [X] No.8 gauge 0.75" self-drilling wafer head (2@12" O.C.)

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

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

STUDS:
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.033" thickness) 33ksi (230 MPa)
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.043" thickness) 33ksi (230 MPa)
- [X] Double chord studs used at each end
- [ ] Other

STUD SPACING: [X] 24" O.C.

TRACK:
- Web: 3-5/8" inches [ ] (0.033" thickness) 33ksi (230 MPa)
- Flange: 1-1/4" inches [X] (0.043" thickness) 33ksi (230 MPa)

HOLD DOWNS: [X] Simpson Strong-Tie S/HD10S (# of screws): 24
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [X] Monotonic rate of loading 2.5mm/min
- [ ] Cyclic

LVDT MEASUREMENTS:
- [X] Actuator LVDT [X] North Uplift
- [X] North Slip [X] South Uplift
- [X] South Slip [X] Top of Wall Lateral **TOTAL:** 6

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 10 scan/sec

COMMENTS:
-Shear anchors torqued for 10s with impact wrench
-Hold down anchors at approximately 7.5 kN (load cells used on both hold-downs)
-Ambient temperature 23 °C
-Square plate washers (2.5"x2.5") used in all top track connections

Figure B.25 Data Sheet for Test 11M-a

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 11M-b

RESEARCHER: Nisreen Balh **ASSISTANTS:** Cheryl Ong-Tone, Anthony Caruso, Gabriele Rotili

DATE: 2-Jul-08 **TIME:** 3:00 PM

DIMENSIONS OF WALL: 8 FT X 8 FT **PANEL ORIENTATION:** Vertical, 2 - 4'x8' sheets; Sheathing one side

SHEATHING:
- [ ] 0.018" Sheet Steel 33ksi (230 MPa)
- [X] 0.027" Sheet Steel 33ksi (230 MPa)

Connections

Sheathing:	X No.8 gauge 0.75" self-drilling pan head (Grabber Superdrive)				
Framing:	X No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive				
Hold downs:	X No.14 gauge 0.75" self-drilling Hex washer head				
Anchor Rods	X 7/8" rod				
Loading Beam:	X A325 3/4" bolts	4 bolts	6 bolts	X 12 bolts	Other:
Base	X A325 3/4" bolts	4 bolts	X 6 bolts	12 bolts	Other:
Back-to-Back					
Chord Studs:	X No.8 gauge 0.75" self-drilling wafer head (2@12" O.C.)				

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

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

STUDS:
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.033" thickness) 33ksi (230 MPa)
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.043" thickness) 33ksi (230 MPa)
- [X] Double chord studs used at each end
- [ ] Other

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

TRACK:
Web: 3-5/8" inches
Flange: 1-1/4" inches
- [ ] (0.033" thickness) 33ksi (230 MPa)
- [X] (0.043" thickness) 33ksi (230 MPa)

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10S (# of screws): 24
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [X] Monotonic rate of loading 2.5mm/min
- [ ] Cyclic

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral

TOTAL: 6

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 10 scan/sec

COMMENTS:
-Shear anchors torqued for 10s with impact wrench
-Hold down anchors at approximately 7.5 kN (load cells used on both hold-downs)
-Ambient temperature 23 °C
-Square plate washers (2.5"x2.5") used in all top track connections

Figure B.26 Data Sheet for Test 11M-b

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 11C-a

RESEARCHER: Nisreen Balh **ASSISTANTS:** Cheryl Ong-Tone, Anthony Caruso, Gabriele Rotili

DATE: 03-Jul-08 **TIME:** 11:30 AM

DIMENSIONS OF WALL: 8 FT X 8 FT **PANEL ORIENTATION:** Vertical, 2 - 4'x8' sheets; Sheathing one side

SHEATHING:
- [ ] 0.018" Sheet Steel 33ksi (230 MPa)
- [X] 0.027" Sheet Steel 33ksi (230 MPa)

Connections
- Sheathing: [X] No.8 gauge 0.75" self-drilling pan head (Grabber Superdrive)
- Framing: [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive
- Hold downs: [X] No.14 gauge 0.75" self-drilling Hex washer head
- Anchor Rods: [X] 7/8" rod
- Loading Beam: [X] A325 3/4" bolts [ ] 4 bolts [ ] 6 bolts [x] 12 bolts [ ] Other:
- Base: [X] A325 3/4" bolts [ ] 4 bolts [X] 6 bolts [ ] 12 bolts [ ] Other:
- Back-to-Back Chord Studs: [X] No.8 gauge 0.75" self-drilling wafer head (2@12" O.C.)

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

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

STUDS:
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.033" thickness) 33ksi (230 MPa)
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.043" thickness) 33ksi (230 MPa)
- [X] Double chord studs used at each end
- [ ] Other

STUD SPACING: [X] 24" O.C.

TRACK:
- Web: 3-5/8" inches [ ] (0.033" thickness) 33ksi (230 MPa)
- Flange: 1-1/4" inches [X] (0.043" thickness) 33ksi (230 MPa)

HOLD DOWNS: [X] Simpson Strong-Tie S/HD10S (# of screws): 24
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [ ] Monotonic
- [X] Cyclic CUREE cyclic protocol

LVDT MEASUREMENTS:
- [X] Actuator LVDT [X] North Uplift
- [X] North Slip [X] South Uplift
- [X] South Slip [X] Top of Wall Lateral **TOTAL:** 6

DATA ACQ. RECORD RATE: 100 scan/sec **MONITOR RATE:** 100 scan/sec

COMMENTS:
- -Shear anchors torqued for 10s with impact wrench
- -Hold down anchors at approximately 7.5 kN (load cells used on both hold-downs)
- -Ambient temperature 23 °C
- -Square plate washers (2.5"x2.5") used in all top track connections

Figure B.27 Data Sheet for Test 11C-a

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 11C-b

RESEARCHER: Nisreen Balh **ASSISTANTS:** Cheryl Ong-Tone, Anthony Caruso, Gabriele Rotili

DATE: 04-Jul-08 **TIME:** 9:15 AM

DIMENSIONS OF WALL: 8 FT X 8 FT **PANEL ORIENTATION:** Vertical, 2 - 4'x8' sheets; Sheathing one side

SHEATHING:
- [ ] 0.018" Sheet Steel 33ksi (230 MPa)
- [X] 0.027" Sheet Steel 33ksi (230 MPa)

Connections

Sheathing:	X No.8 gauge 0.75" self-drilling pan head (Grabber Superdrive)				
Framing:	X No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive				
Hold downs:	X No.14 gauge 0.75" self-drilling Hex washer head				
Anchor Rods	X 7/8" rod				
Loading Beam:	X A325 3/4" bolts	4 bolts	6 bolts	x 12 bolts	Other:
Base	X A325 3/4" bolts	4 bolts	X 6 bolts	12 bolts	Other:
Back-to-Back					
Chord Studs:	X No.8 gauge 0.75" self-drilling wafer head (2@12" O.C.)				

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

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

STUDS:
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.033" thickness) 33ksi (230 MPa)
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.043" thickness) 33ksi (230 MPa)
- [X] Double chord studs used at each end
- [ ] Other

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

TRACK:
Web: 3-5/8" inches
Flange: 1-1/4" inches
- [ ] (0.033" thickness) 33ksi (230 MPa)
- [X] (0.043" thickness) 33ksi (230 MPa)

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10S (# of screws): 24
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [ ] Monotonic
- [X] Cyclic CUREE cyclic protocol

LVDT MEASUREMENTS:
- [X] Actuator LVDT
- [X] North Slip
- [X] South Slip
- [X] North Uplift
- [X] South Uplift
- [X] Top of Wall Lateral

TOTAL: 6

DATA ACQ. RECORD RATE: 100 scan/sec **MONITOR RATE:** 100 scan/sec

COMMENTS:
-Shear anchors torqued for 10s with impact wrench
-Hold down anchors at approximately 7.5 kN (load cells used on both hold-downs)
-Ambient temperature 23 °C
-Square plate washers (2.5"x2.5") used in all top track connections

Figure B.28 Data Sheet for Test 11C-b

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 17M-a

RESEARCHER: Nisreen Balh **ASSISTANTS:** Cheryl Ong-Tone, Anthony Caruso, Gabriele Rotili

DATE: 26-May-08 **TIME:** 10:30 AM

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

SHEATHING:
- [X] 0.018" Sheet Steel 33ksi (230 MPa)
- [ ] 0.027" Sheet Steel 33ksi (230 MPa)

Connections
- Sheathing: [X] No.8 gauge 0.75" self-drilling pan head (Grabber Superdrive)
- Framing: [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive
- Hold downs: [X] No.14 gauge 0.75" self-drilling Hex washer head
- Anchor Rods [X] 7/8" rod
- Loading Beam: [X] A325 3/4" bolts [X] 4 bolts [ ] 6 bolts [ ] 12 bolts [ ] Other:
- Base [X] A325 3/4" bolts [X] 4 bolts [ ] 6 bolts [ ] 12 bolts [ ] Other:
- Back-to-Back
- Chord Studs: [X] No.8 gauge 0.75" self-drilling wafer head (2@12" O.C.)

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

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

STUDS:
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.033" thickness) 33ksi (230 MPa)
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.043" thickness) 33ksi (230 MPa)
- [X] Double chord studs used at each end
- [ ] Other

STUD SPACING: [X] 24" O.C.

TRACK:
- Web: 3-5/8" inches [ ] (0.033" thickness) 33ksi (230 MPa)
- Flange: 1-1/4" inches [X] (0.043" thickness) 33ksi (230 MPa)

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10S (# of screws): 24
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [X] Monotonic rate of loading 2.5mm/min
- [ ] Cyclic

LVDT MEASUREMENTS:
- [X] Actuator LVDT [X] North Uplift
- [X] North Slip [X] South Uplift
- [X] South Slip [X] Top of Wall Lateral **TOTAL:** 6

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 10 scan/sec

COMMENTS:
- -Shear anchors torqued for 10s with impact wrench
- -Hold down anchors at approximately 7.5 kN (load cells used on both hold-downs)
- -Ambient temperature 23 °C
- -Square plate washers (2.5"x2.5") used in all top track connections

Figure B.29 Data Sheet for Test 17M-a

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 17M-b

RESEARCHER: Nisreen Balh **ASSISTANTS:** Cheryl Ong-Tone, Anthony Caruso, Gabriele Rotili

DATE: 26-May-08 **TIME:** 11:10 AM

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

SHEATHING:
- [X] 0.018" Sheet Steel 33ksi (230 MPa)
- [ ] 0.027" Sheet Steel 33ksi (230 MPa)

Connections
- Sheathing: [X] No.8 gauge 0.75" self-drilling pan head (Grabber Superdrive)
- Framing: [X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive
- Hold downs: [X] No.14 gauge 0.75" self-drilling Hex washer head
- Anchor Rods: [X] 7/8" rod
- Loading Beam: [X] A325 3/4" bolts [X] 4 bolts [ ] 6 bolts [ ] 12 bolts [ ] Other:
- Base: [X] A325 3/4" bolts [X] 4 bolts [ ] 6 bolts [ ] 12 bolts [ ] Other:
- Back-to-Back
- Chord Studs: [X] No.8 gauge 0.75" self-drilling wafer head (2@12" O.C.)

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

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

STUDS:
- [ ] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.033" thickness) 33ksi (230 MPa)
- [X] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.043" thickness) 33ksi (230 MPa)
- [X] Double chord studs used at each end
- [ ] Other

STUD SPACING: [X] 24" O.C.

TRACK:
- Web: 3-5/8" inches [ ] (0.033" thickness) 33ksi (230 MPa)
- Flange: 1-1/4" inches [X] (0.043" thickness) 33ksi (230 MPa)

HOLD DOWNS:
- [X] Simpson Strong-Tie S/HD10S (# of screws): 24
- [ ] Other

TEST PROTOCOL AND DESCRIPTION:
- [X] Monotonic rate of loading 2.5mm/min
- [ ] Cyclic

LVDT MEASUREMENTS:
- [X] Actuator LVDT [X] North Uplift
- [X] North Slip [X] South Uplift
- [X] South Slip [X] Top of Wall Lateral **TOTAL:** 6

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 10 scan/sec

COMMENTS:
-Shear anchors torqued for 10s with impact wrench
-Hold down anchors at approximately 7.5 kN (load cells used on both hold-downs)
-Ambient temperature 23 °C
-Square plate washers (2.5"x2.5") used in all top track connections

Figure B.30 Data Sheet for Test 17M-b

Cold Formed Steel Framed Shear Walls
McGill University, Montreal

TEST: 18M-a

RESEARCHER: Nisreen Balh **ASSISTANTS:** Cheryl Ong-Tone, Anthony Caruso, Gabriele Rotili

DATE: 26-May-08 **TIME:** 4:30 PM

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

SHEATHING:
[X] 0.018" Sheet Steel 33ksi (230 MPa)
[ ] 0.027" Sheet Steel 33ksi (230 MPa)

Connections

Sheathing:	[X] No.8 gauge 0.75" self-drilling pan head (Grabber Superdrive)					
Framing:	[X] No.8 gauge 0.5" self-drilling wafer head (mod. Truss) Phillips drive					
Hold downs:	[X] No.14 gauge 0.75" self-drilling Hex washer head					
Anchor Rods	[X] 7/8" rod					
Loading Beam:	[X] A325 3/4" bolts	[X] 4 bolts	[ ] 6 bolts	[ ] 12 bolts	[ ] Other:	
Base	[X] A325 3/4" bolts	[X] 4 bolts	[ ] 6 bolts	[ ] 12 bolts	[ ] Other:	
Back-to-Back						
Chord Studs:	[X] No.8 gauge 0.75" self-drilling wafer head (2@12" O.C.)					

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

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

STUDS:
[ ] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.033" thickness) 33ksi (230 MPa)
[X] 3-5/8"Wx1-5/8"Fx1/2"Lip (0.043" thickness) 33ksi (230 MPa)
[X] Double chord studs used at each end
[ ] Other

STUD SPACING: [X] 24" O.C.

TRACK:
Web: 3-5/8" inches [ ] (0.033" thickness) 33ksi (230 MPa)
Flange: 1-1/4" inches [X] (0.043" thickness) 33ksi (230 MPa)

HOLD DOWNS:
[X] Simpson Strong-Tie S/HD10S (# of screws): 24
[ ] Other

TEST PROTOCOL AND DESCRIPTION:
[X] Monotonic rate of loading 2.5mm/min
[ ] Cyclic

LVDT MEASUREMENTS:
[X] Actuator LVDT [X] North Uplift
[X] North Slip [X] South Uplift
[X] South Slip [X] Top of Wall Lateral **TOTAL:** 6

DATA ACQ. RECORD RATE: 2 scan/sec **MONITOR RATE:** 10 scan/sec

COMMENTS:
-Shear anchors torqued for 10s with impact wrench
-Hold down anchors at approximately 7.5 kN (load cells used on both hold-downs)
-Ambient temperature 23 °C
-Square plate washers (2.5"x2.5") used in all top track connections

Figure B.31 Data Sheet for Test 18M-a

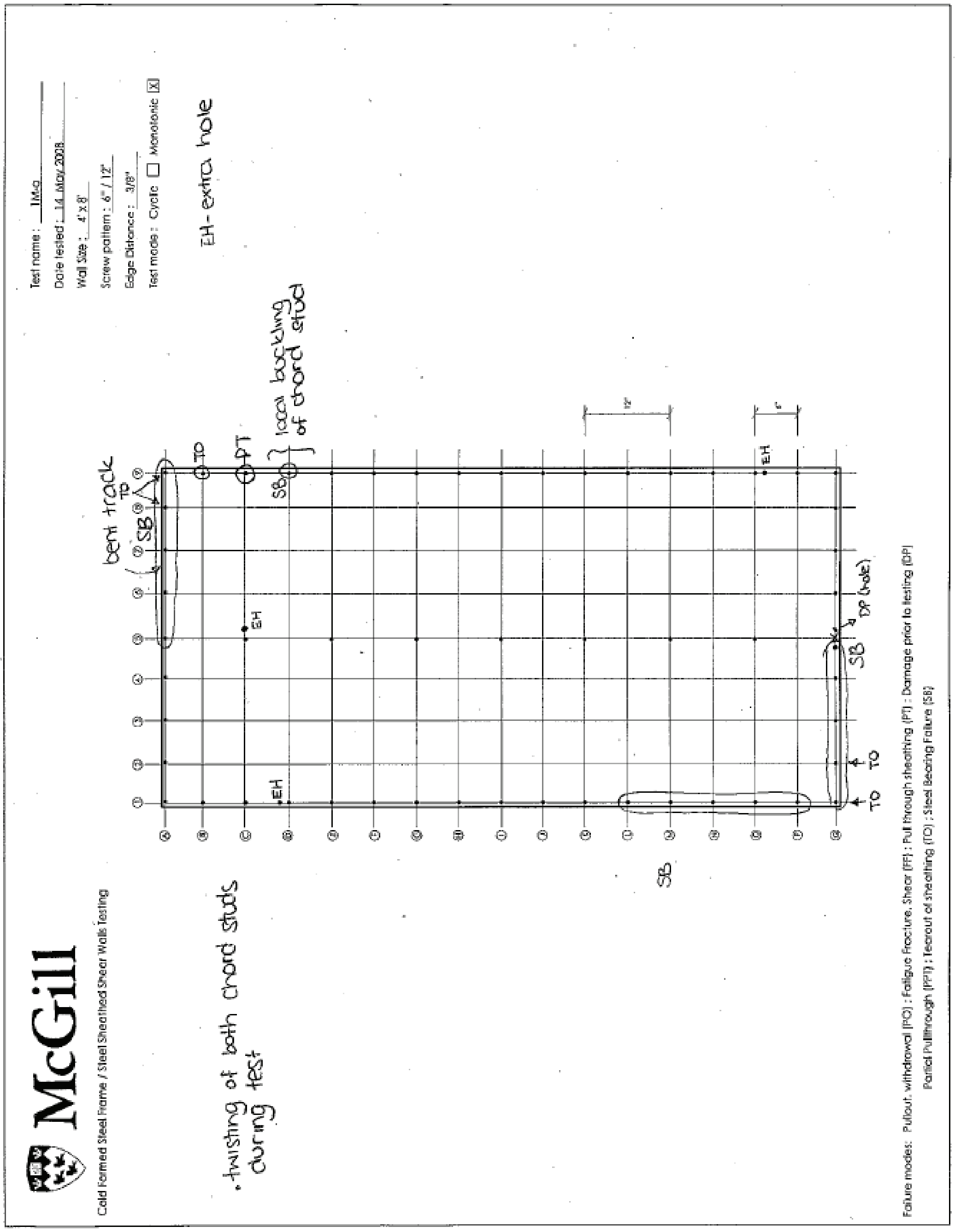


Figure B.32 Observations for Test 1M-a

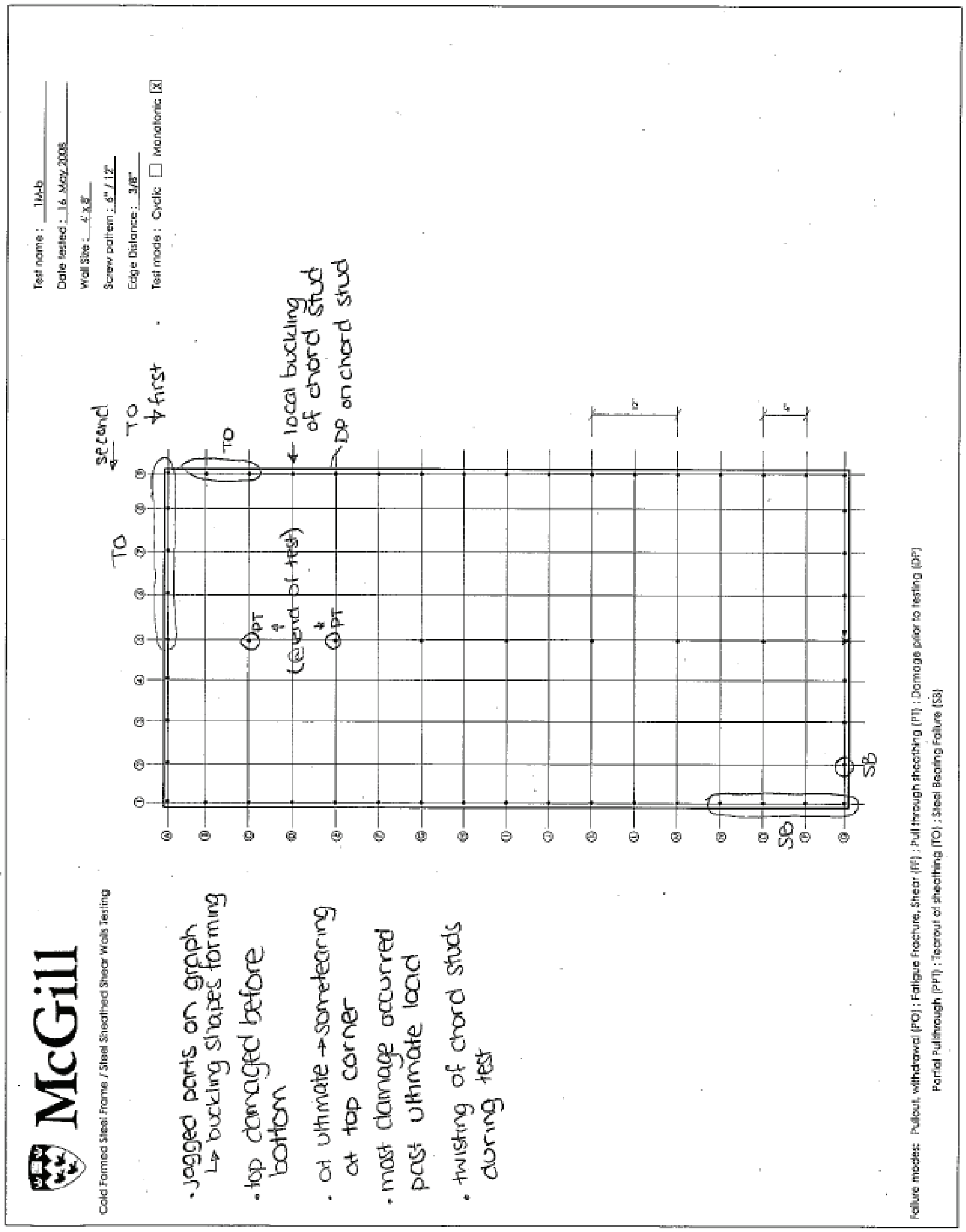


Figure B.33 Observations for Test 1M-b

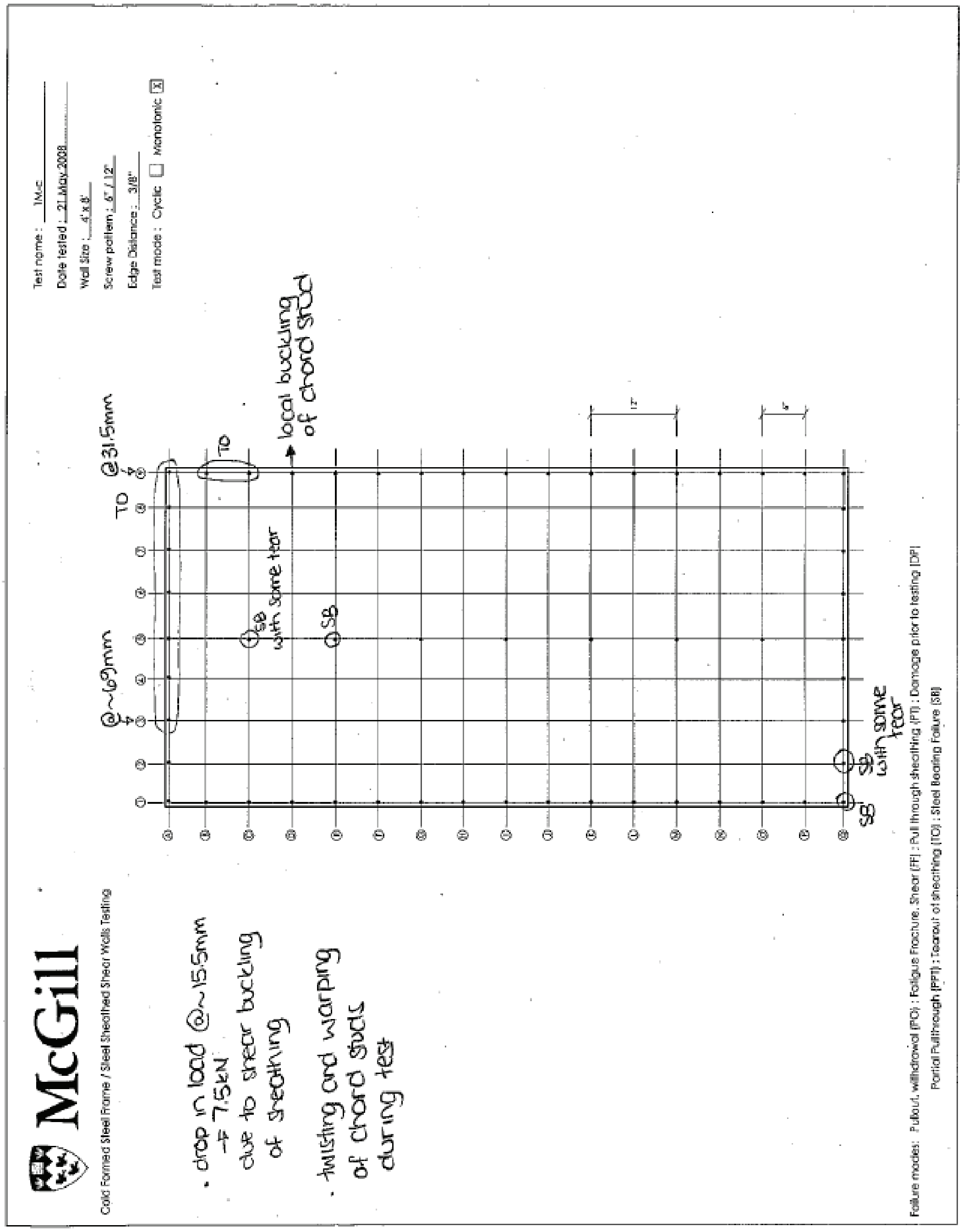


Figure B.34 Observations for Test 1M-c

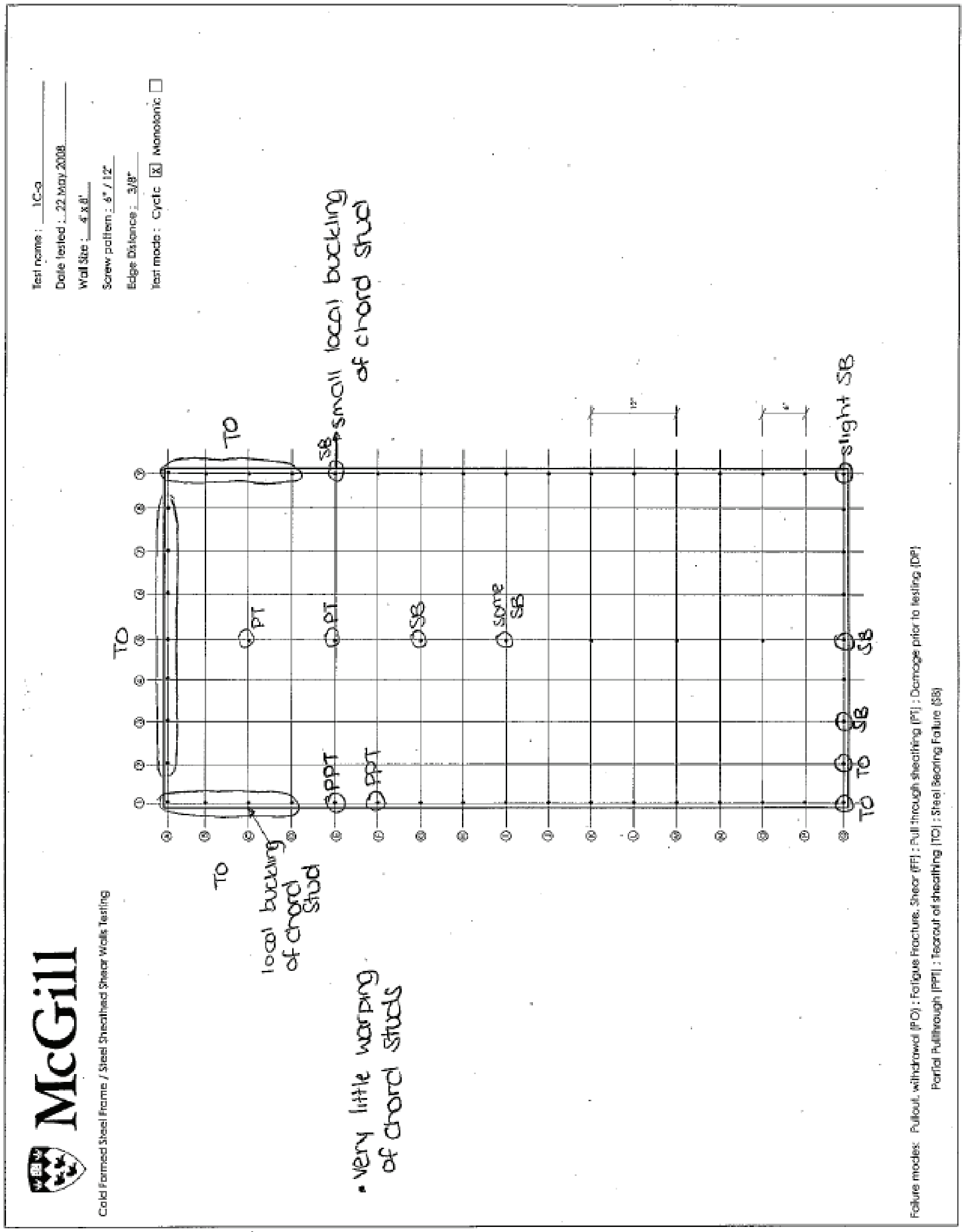


Figure B.35 Observations for Test 1C-a

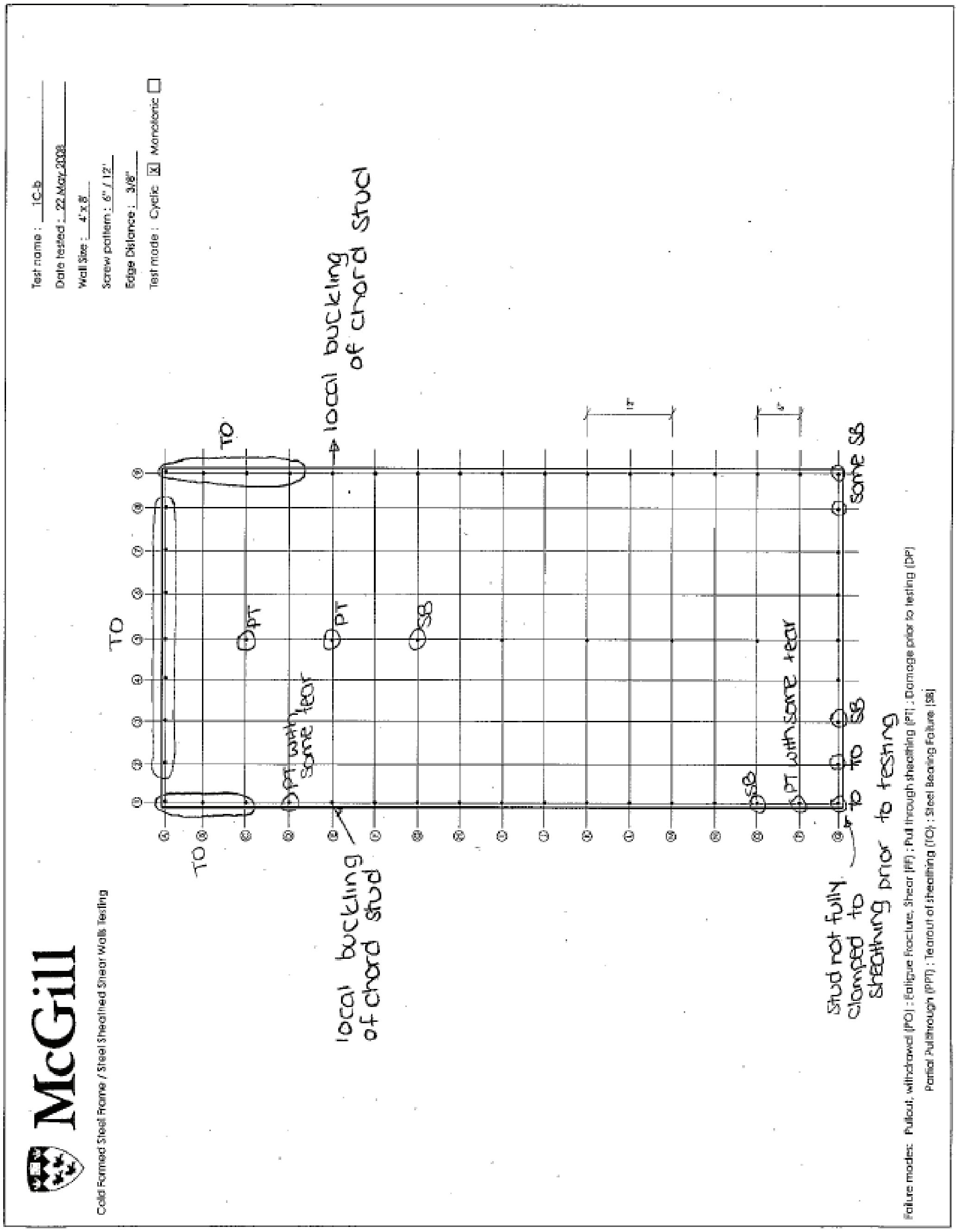


Figure B.36 Observations for Test 1C-b

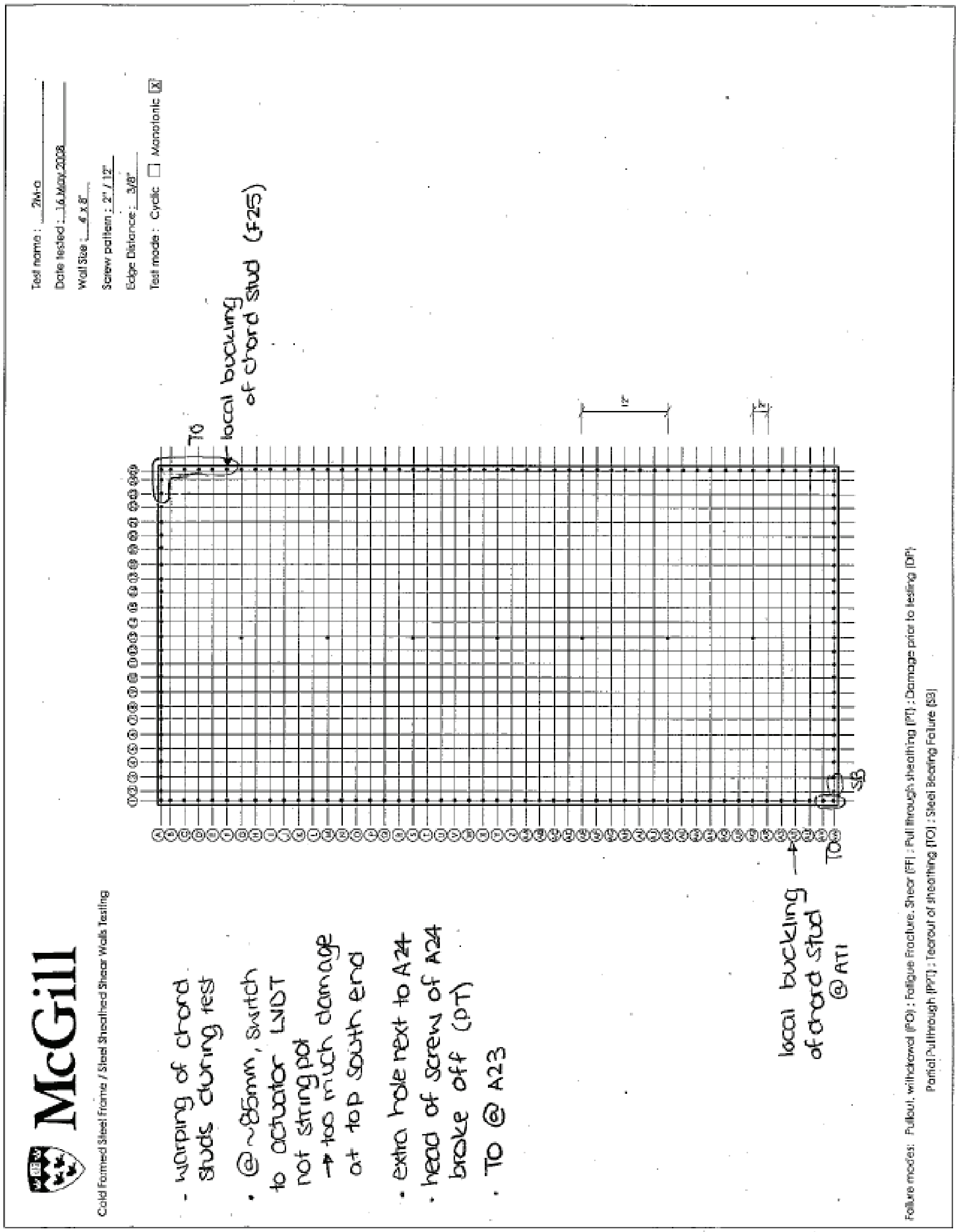


Figure B.37 Observations for Test 2M-a

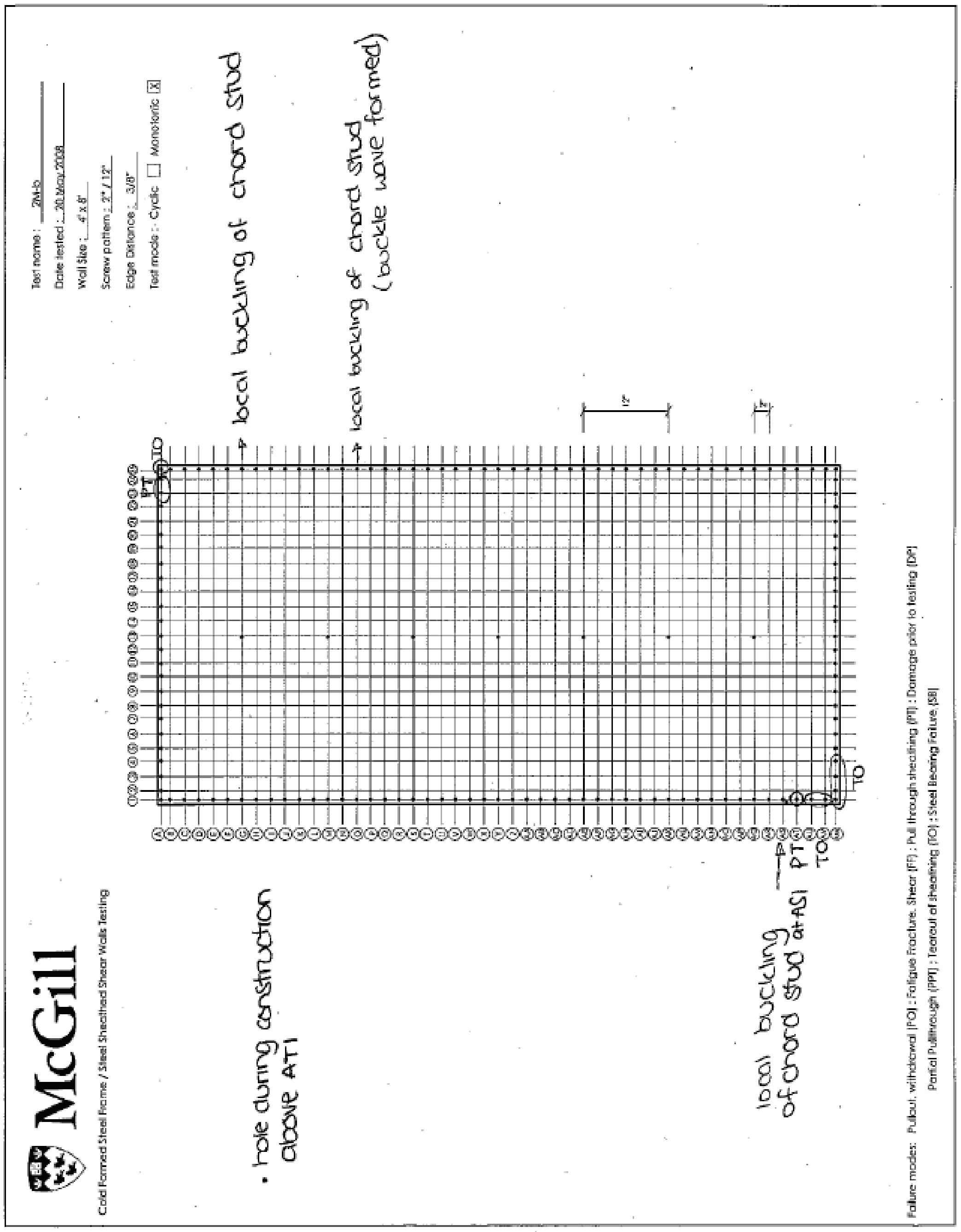


Figure B.38 Observations for Test 2M-b

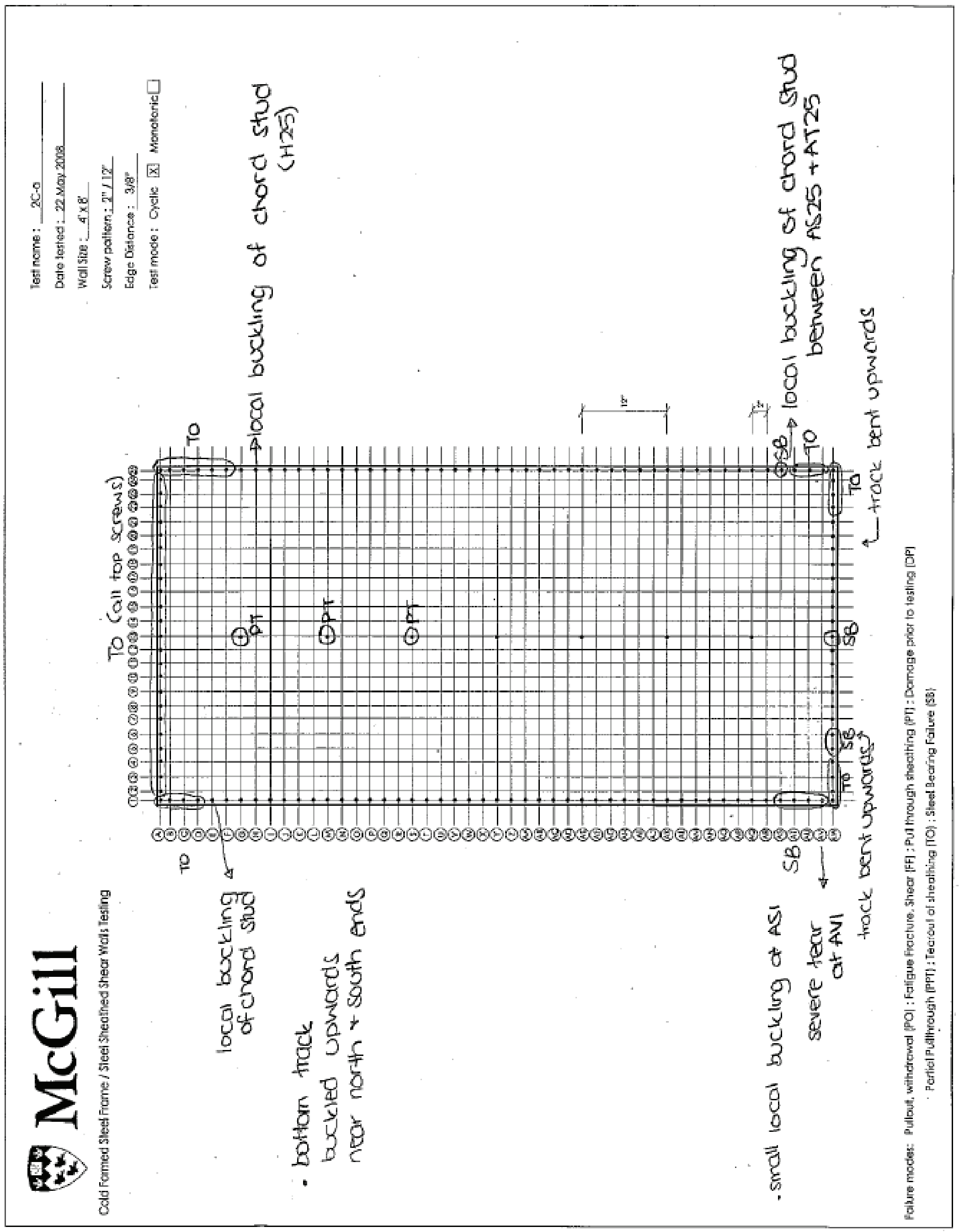


Figure B.39 Observations for Test 2C-a

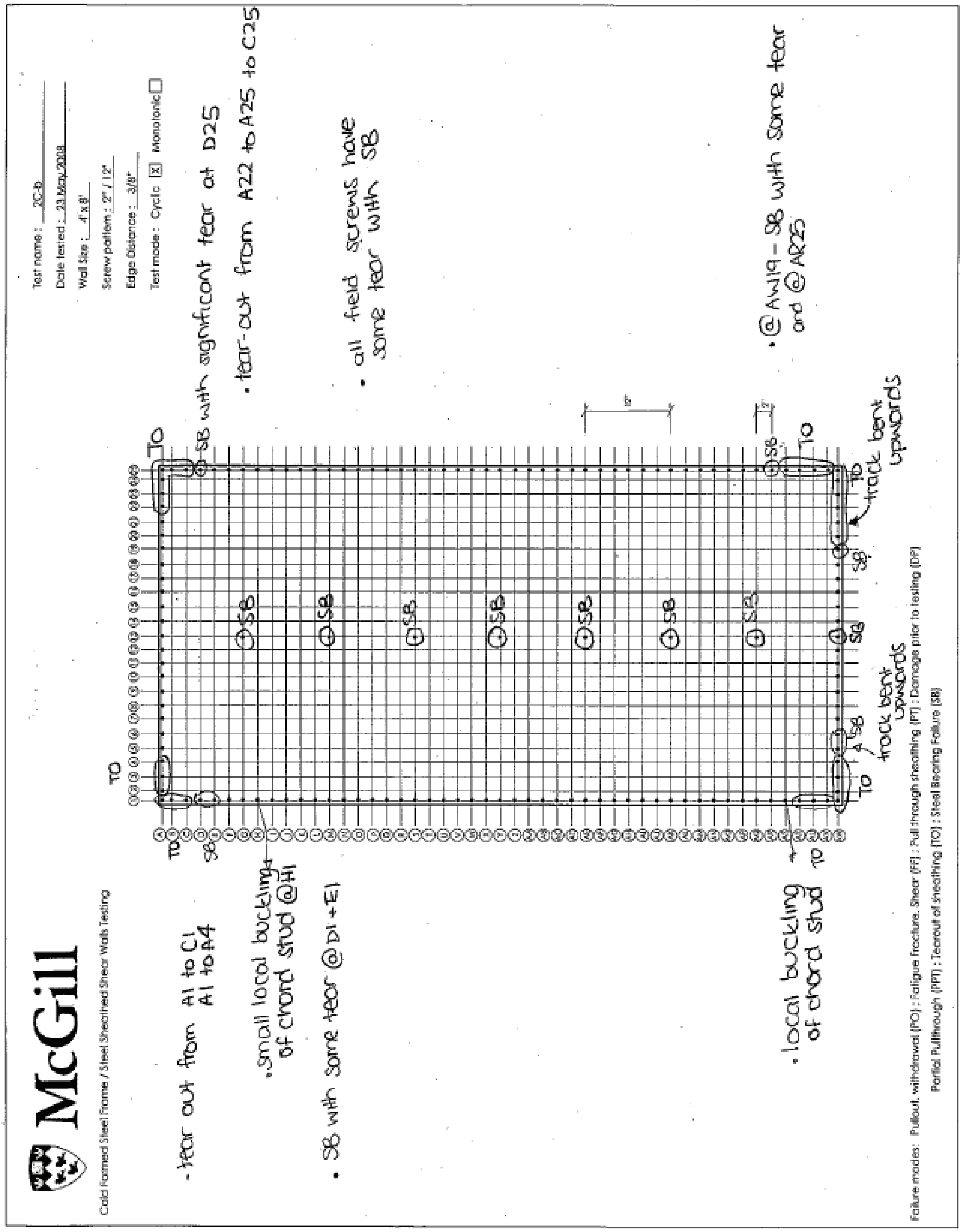


Figure B.40 Observations for Test 2C-b

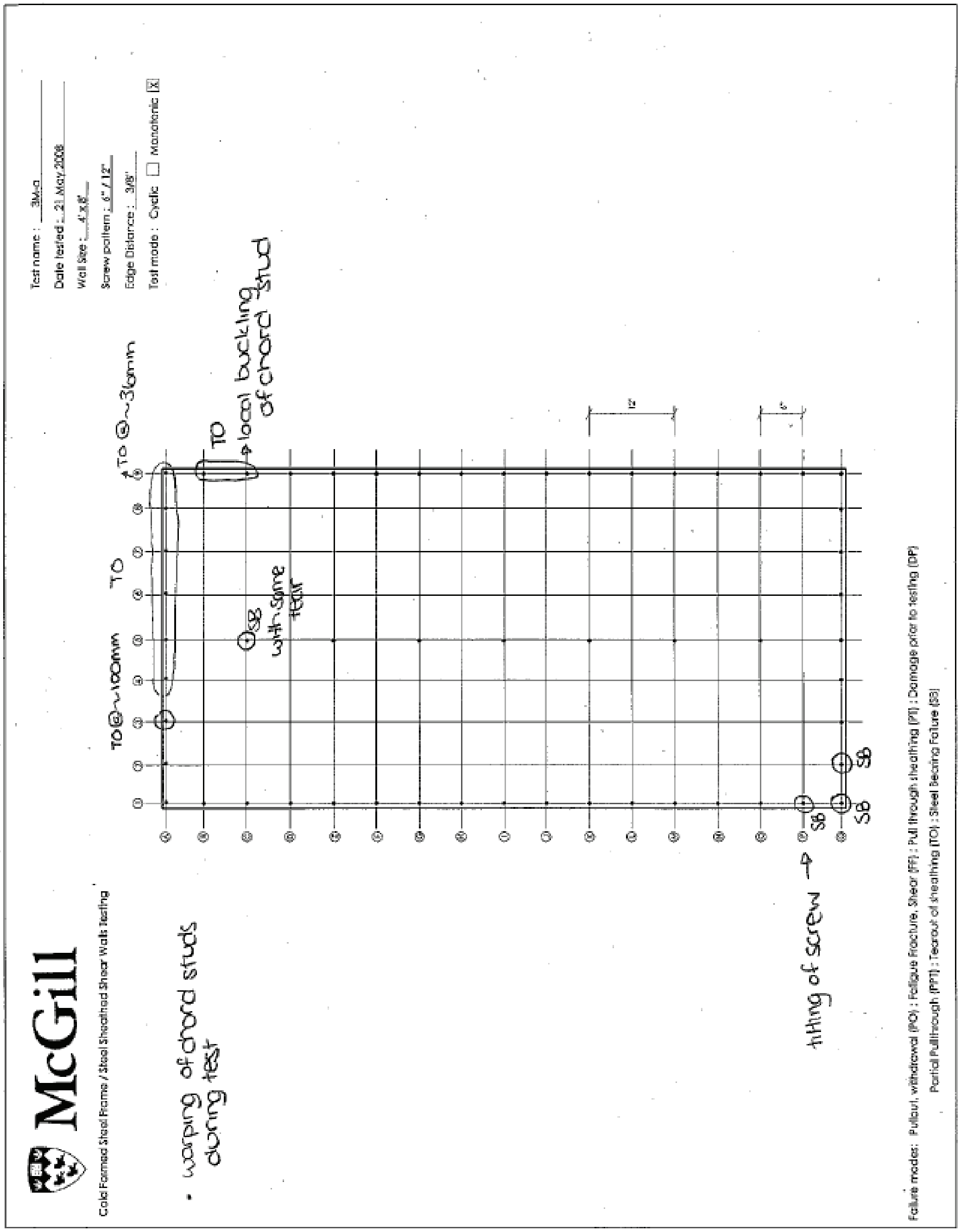


Figure B.41 Observations for Test 3M-a

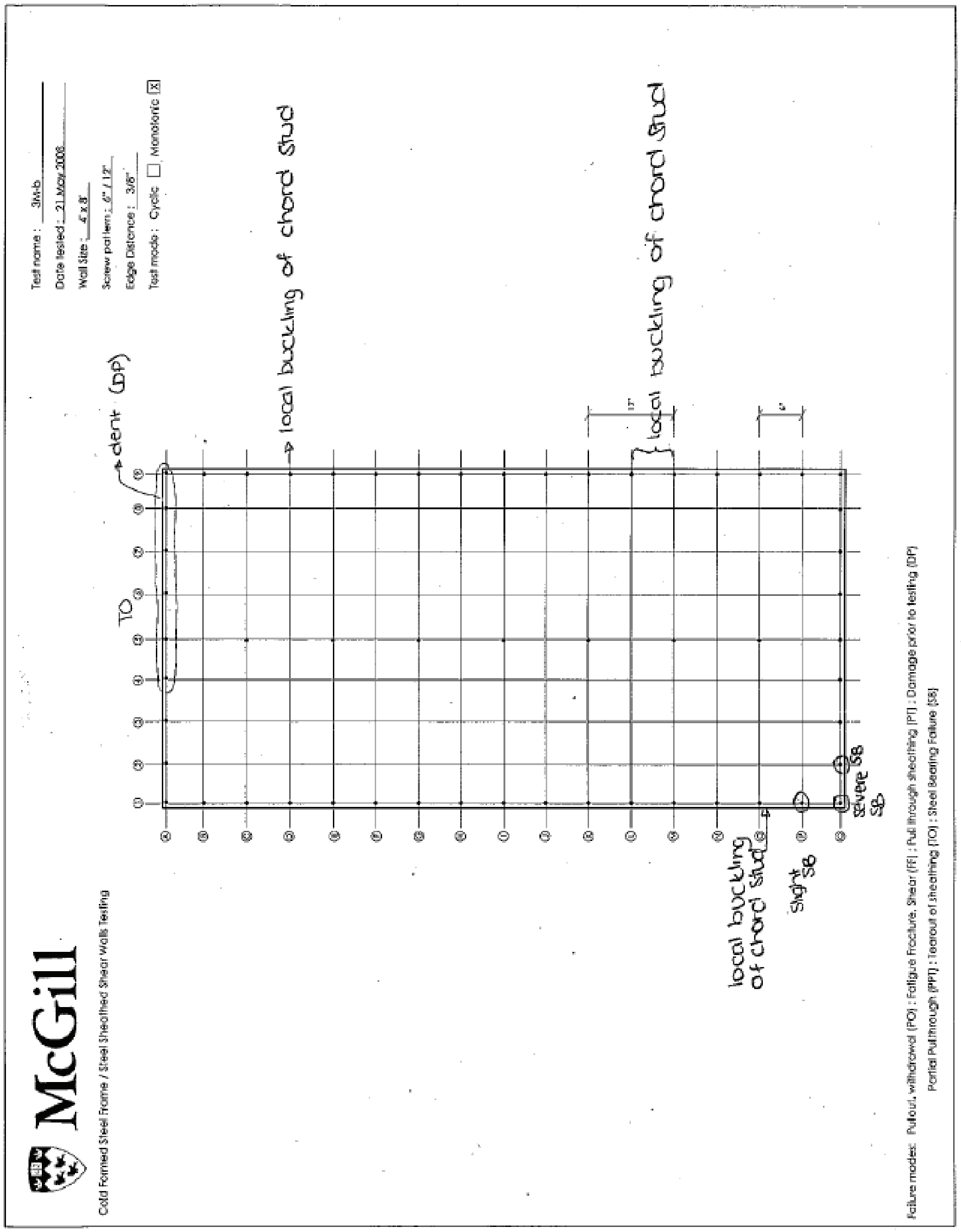


Figure B.42 Observations for Test 3M-b

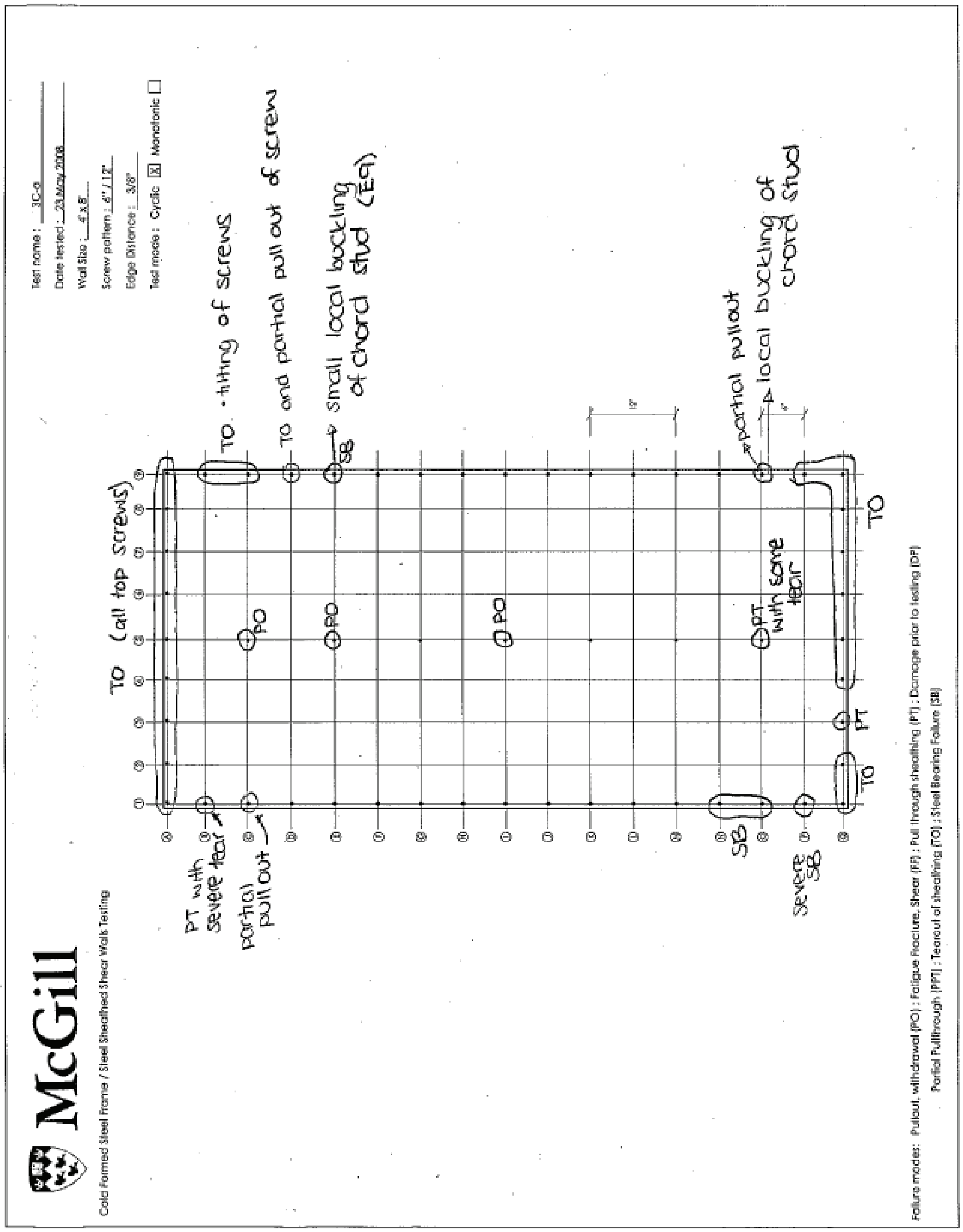


Figure B.43 Observations for Test 3C-a

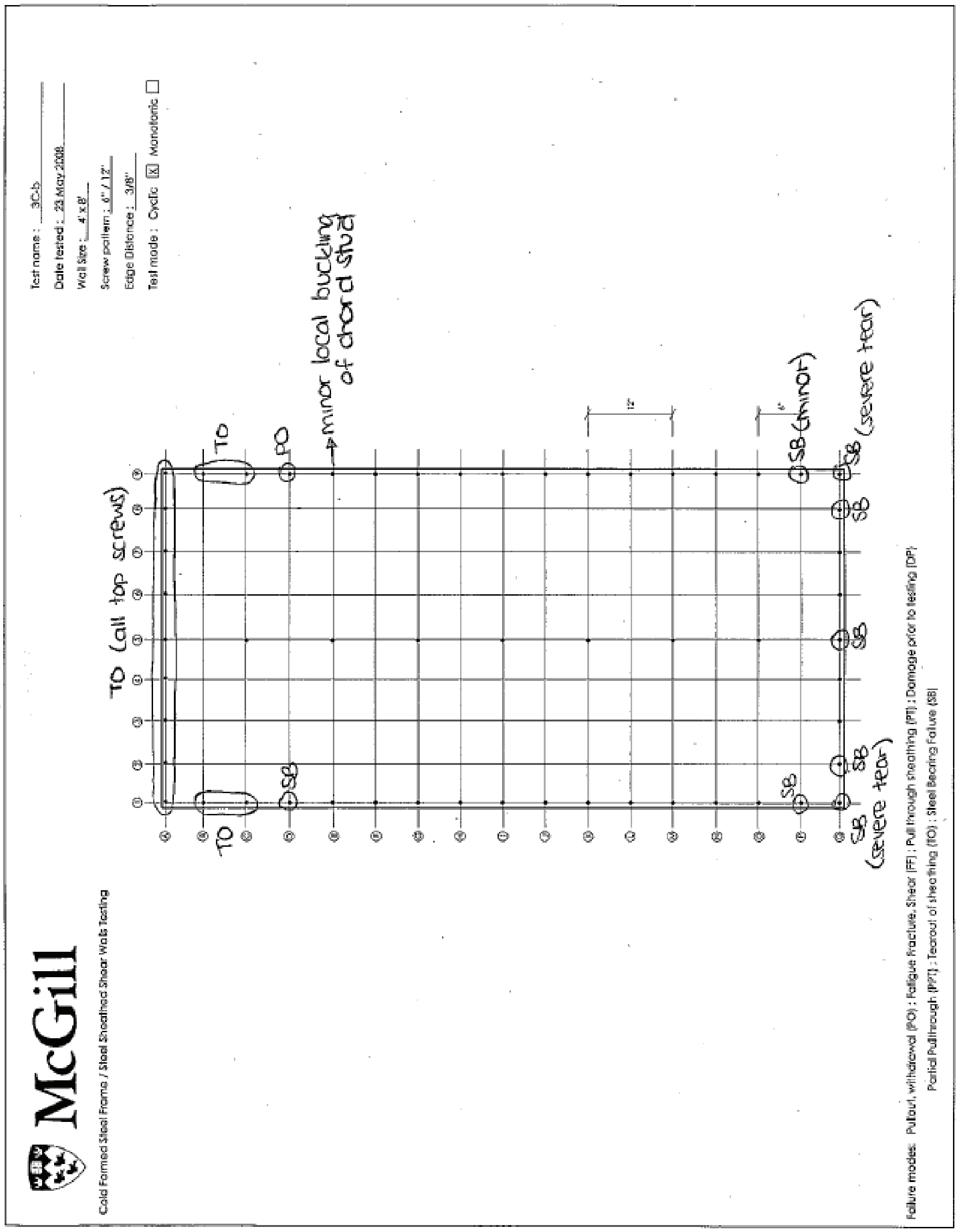


Figure B.44 Observations for Test 3C-b

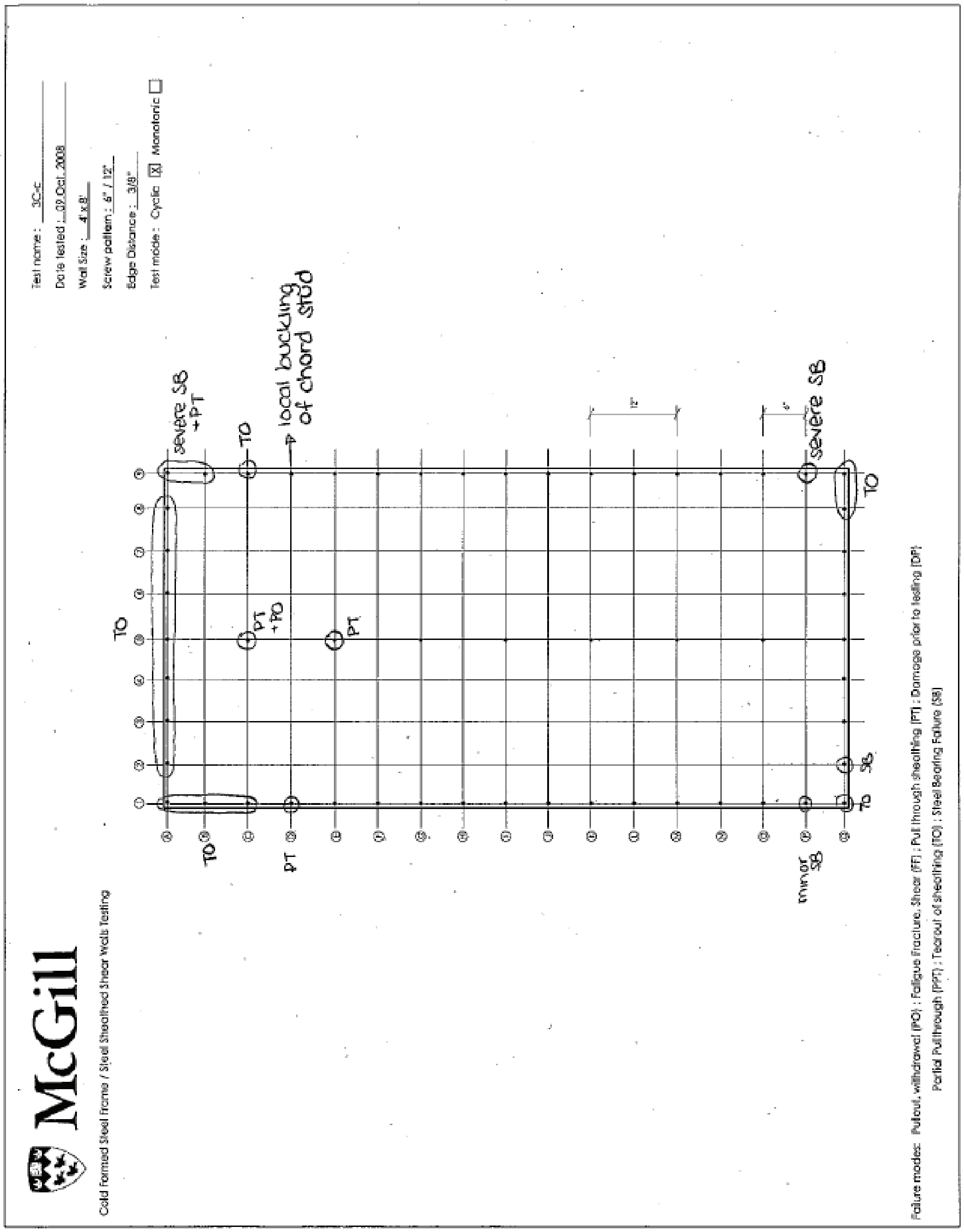


Figure B.45 Observations for Test 3C-c

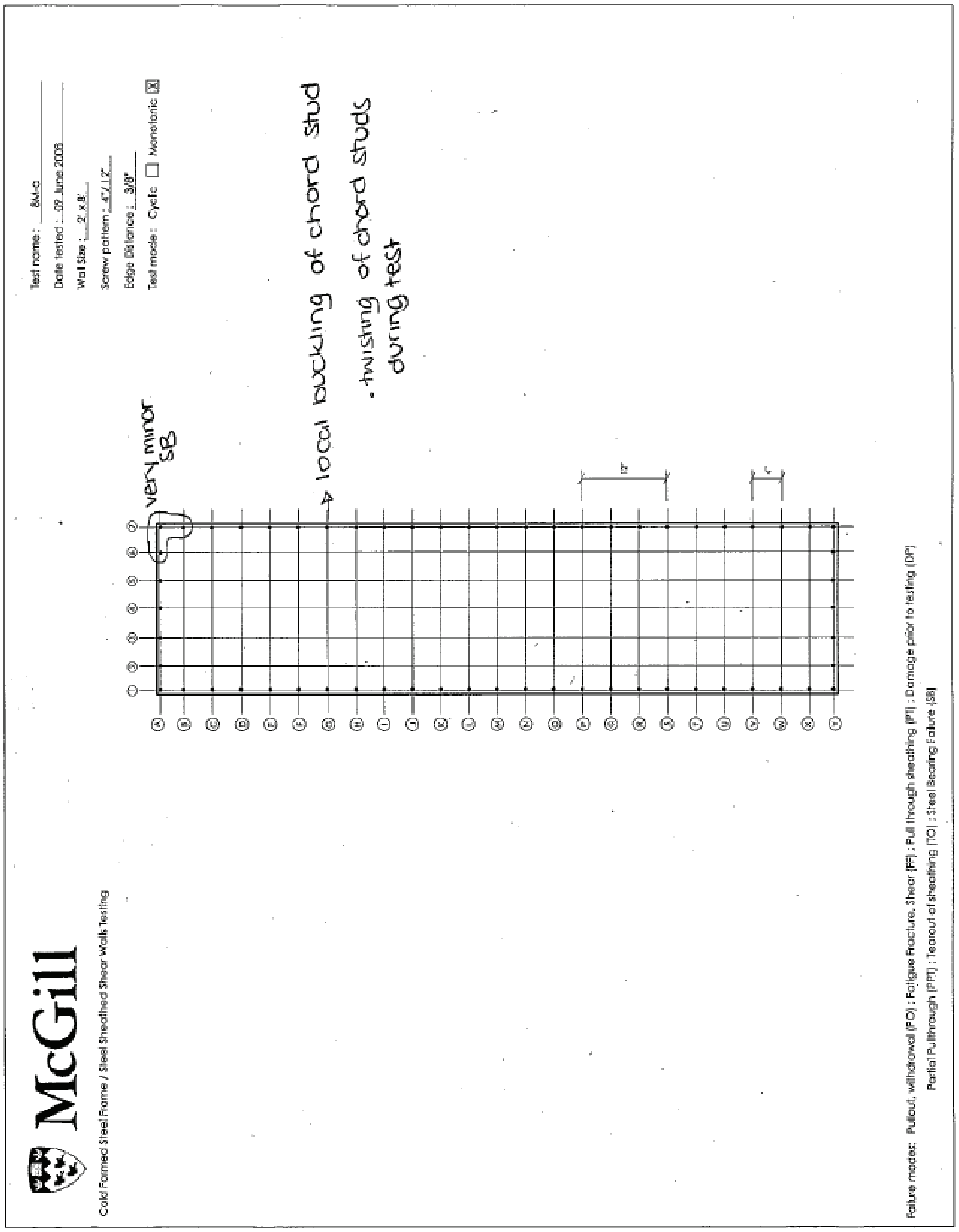


Figure B.46 Observations for Test 8M-a

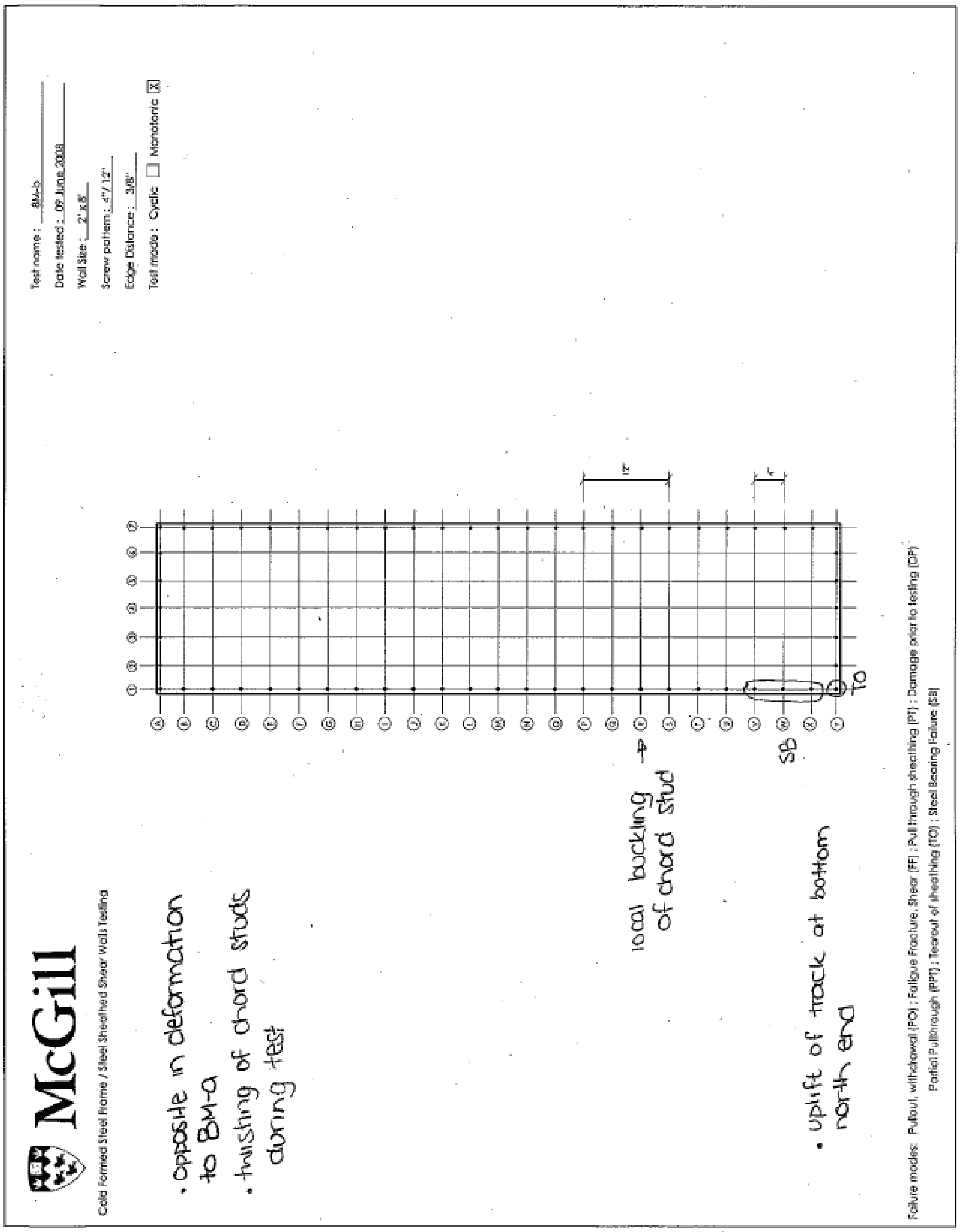


Figure B.47 Observations for Test 8M-b

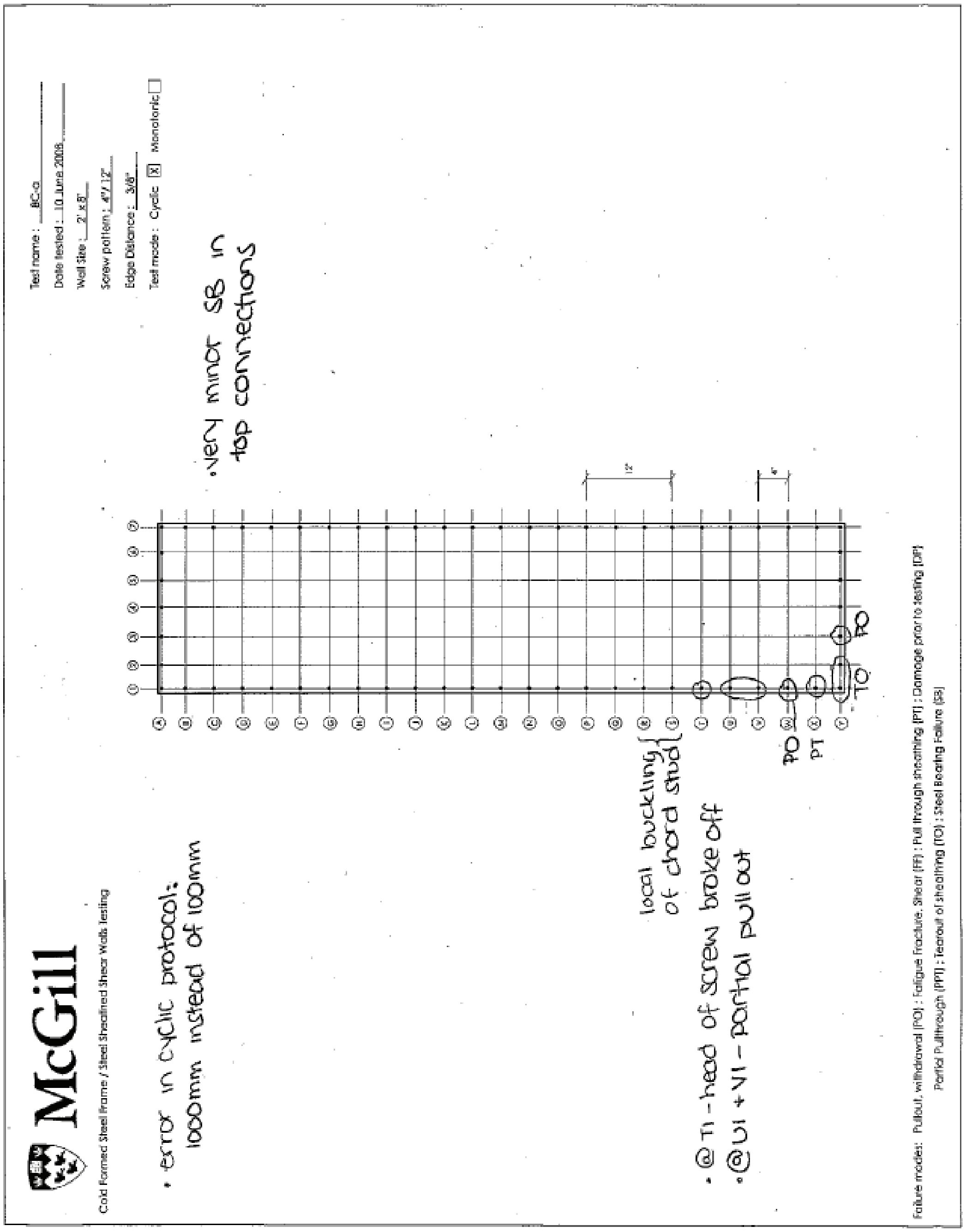


Figure B.48 Observations for Test 8C-a

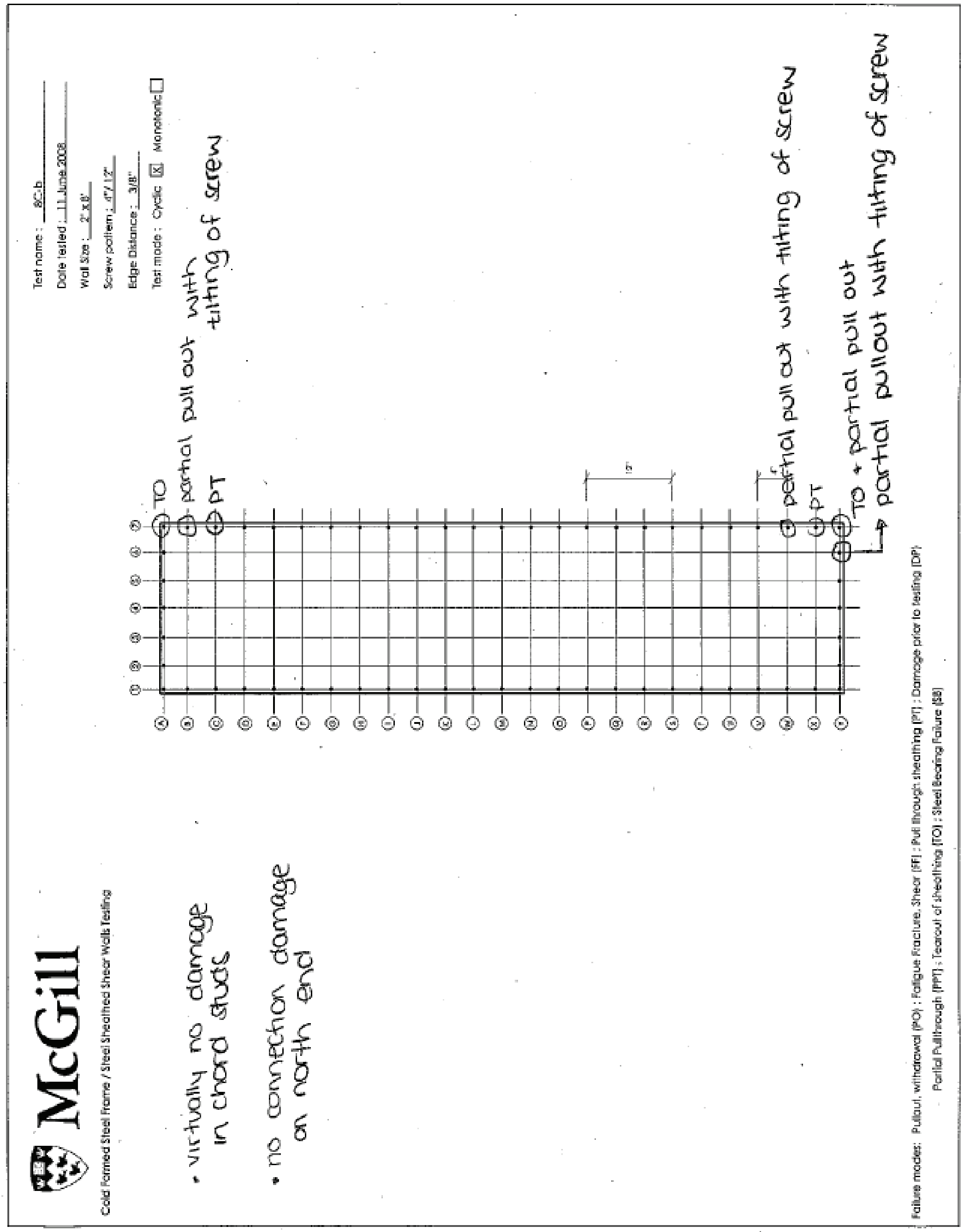


Figure B.49 Observations for Test 8C-b

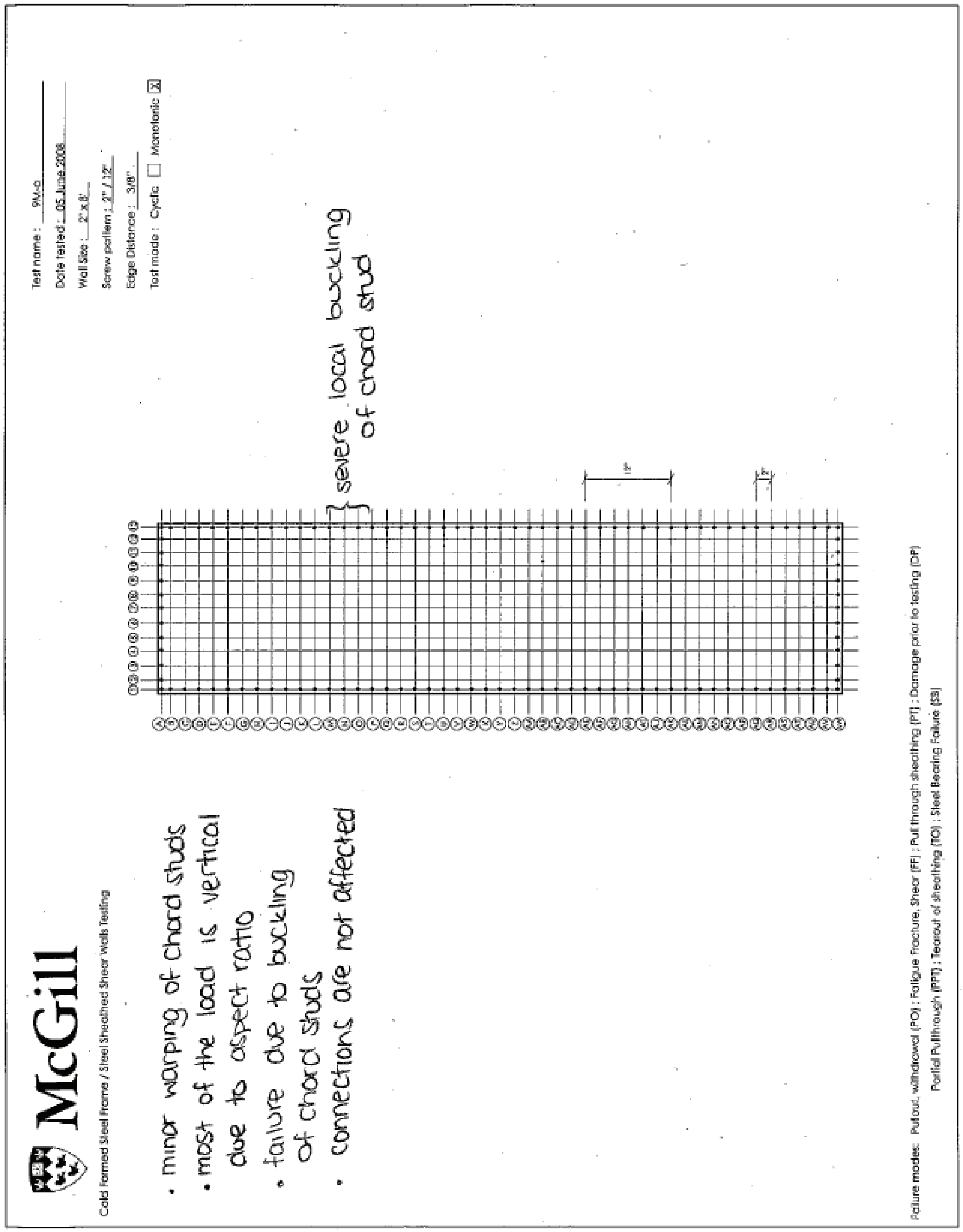


Figure B.50 Observations for Test 9M-a

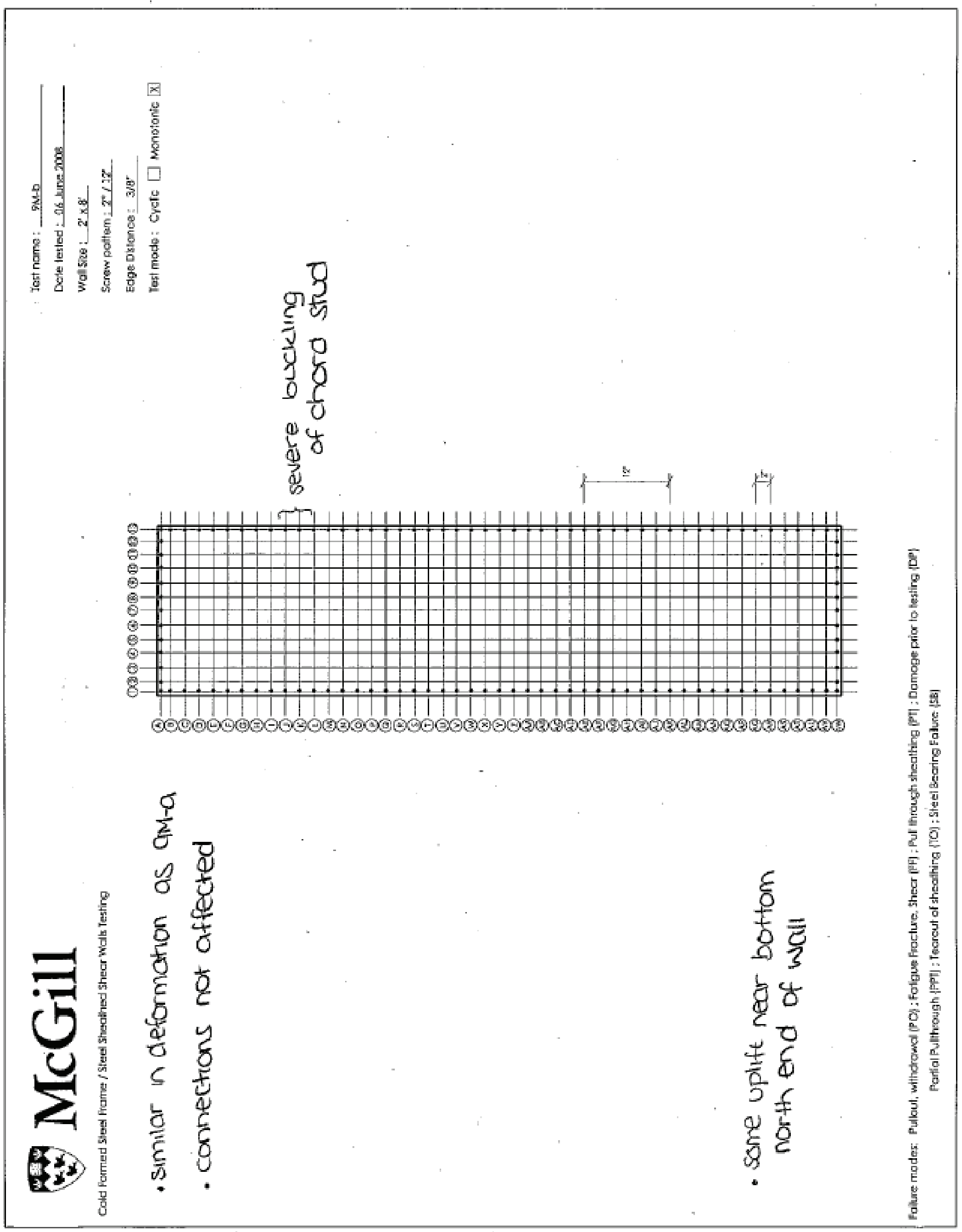


Figure B.51 Observations for Test 9M-b

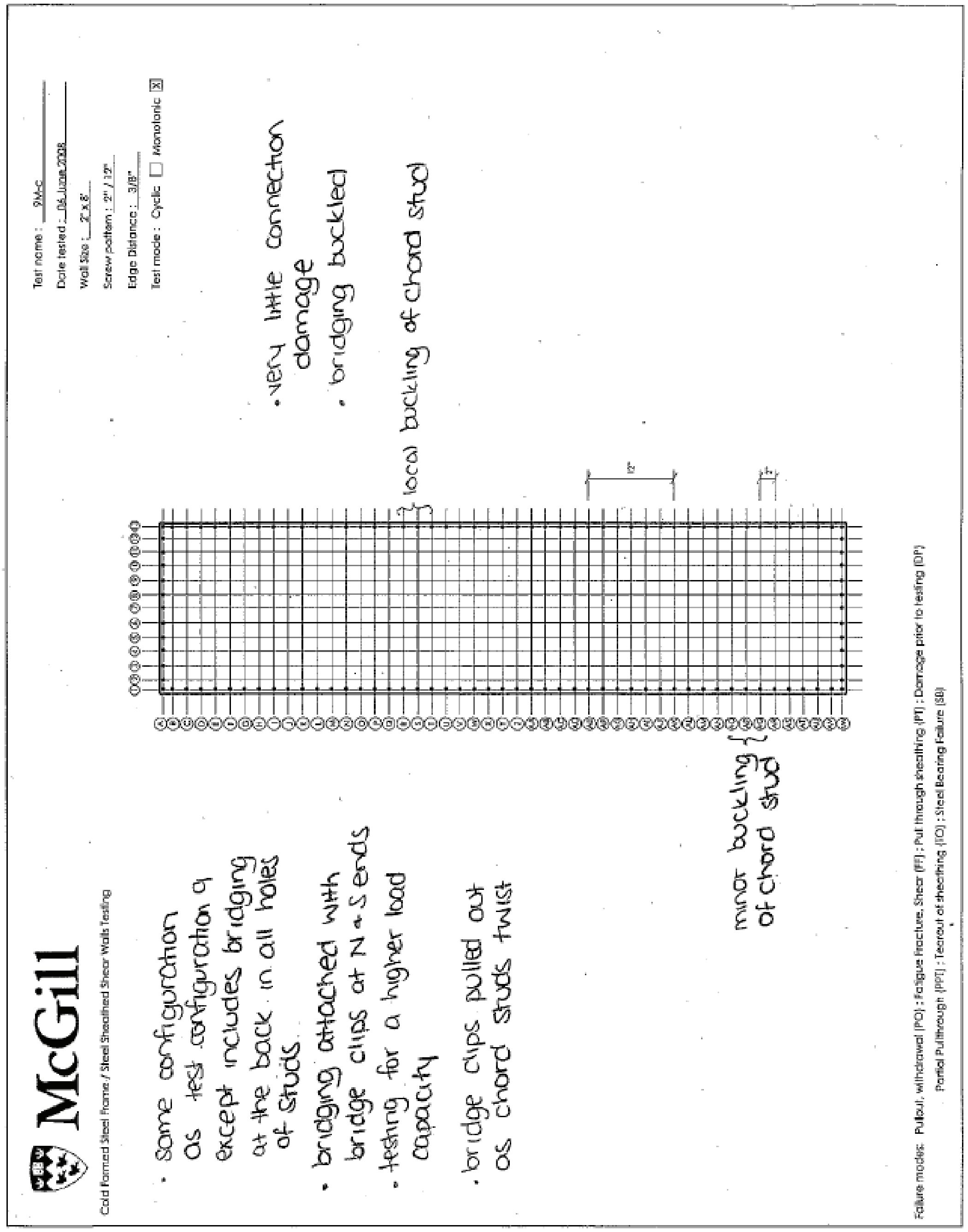


Figure B.52 Observations for Test 9M-c

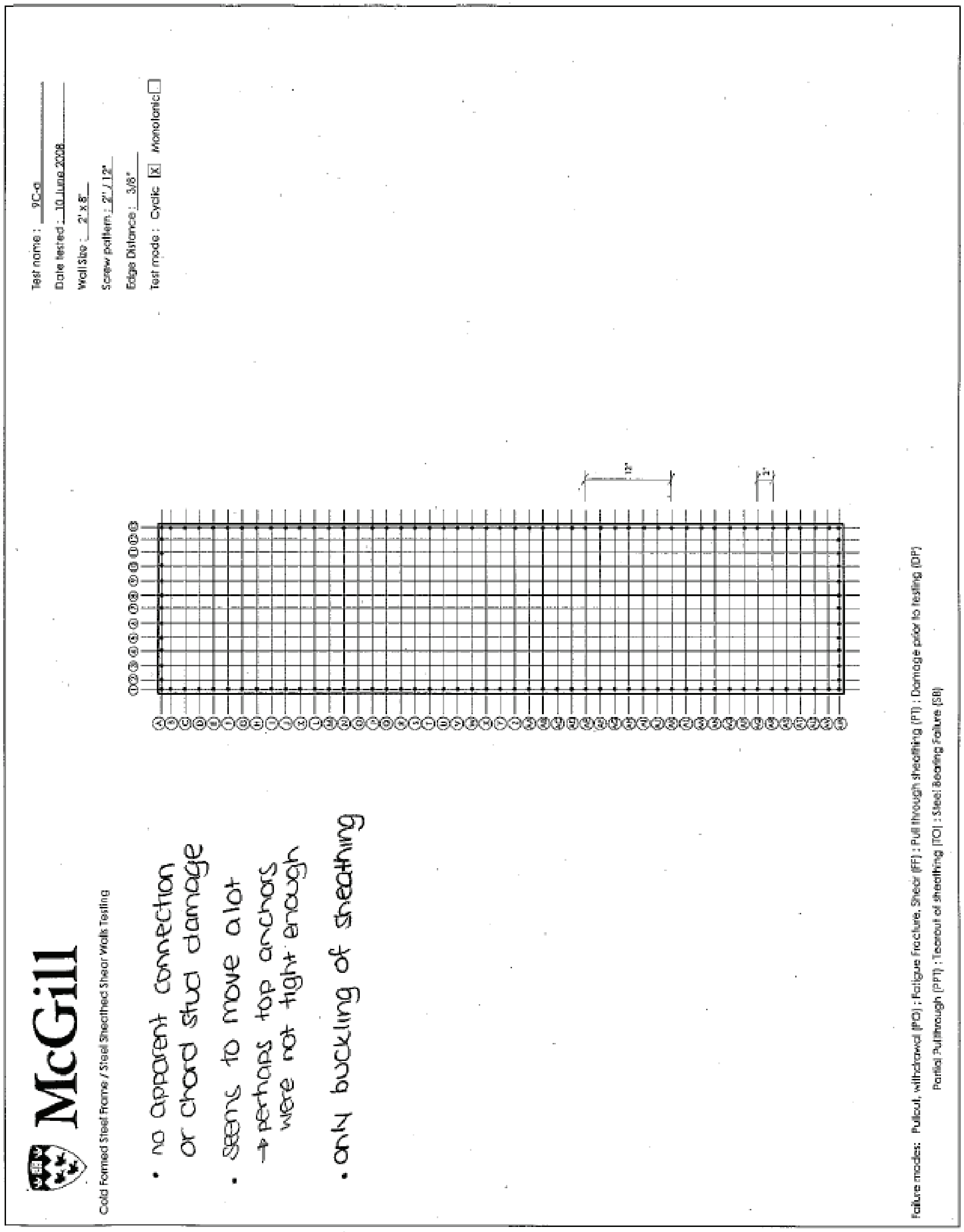


Figure B.53 Observations for Test 9C-a

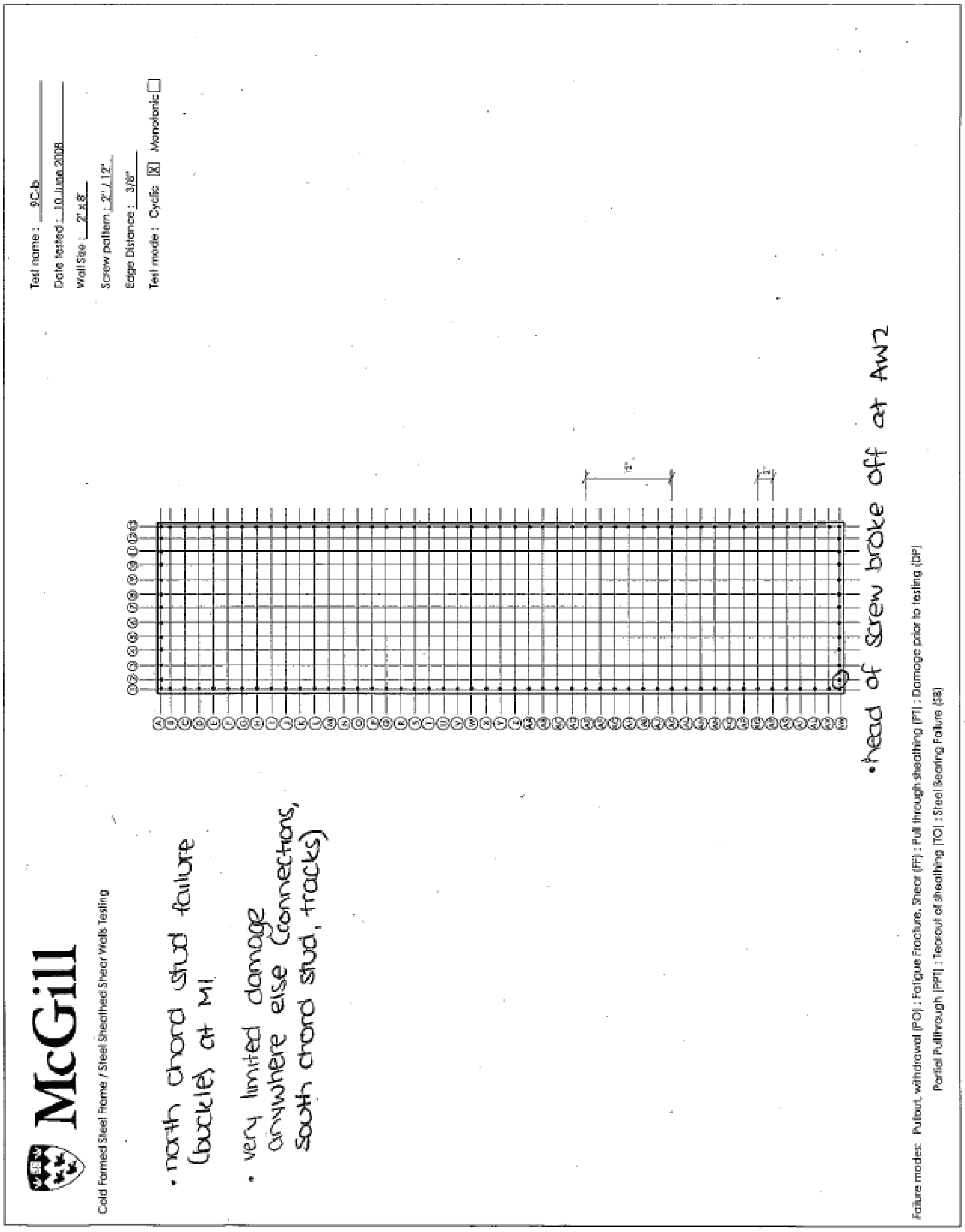


Figure B.54 Observations for Test 9C-b

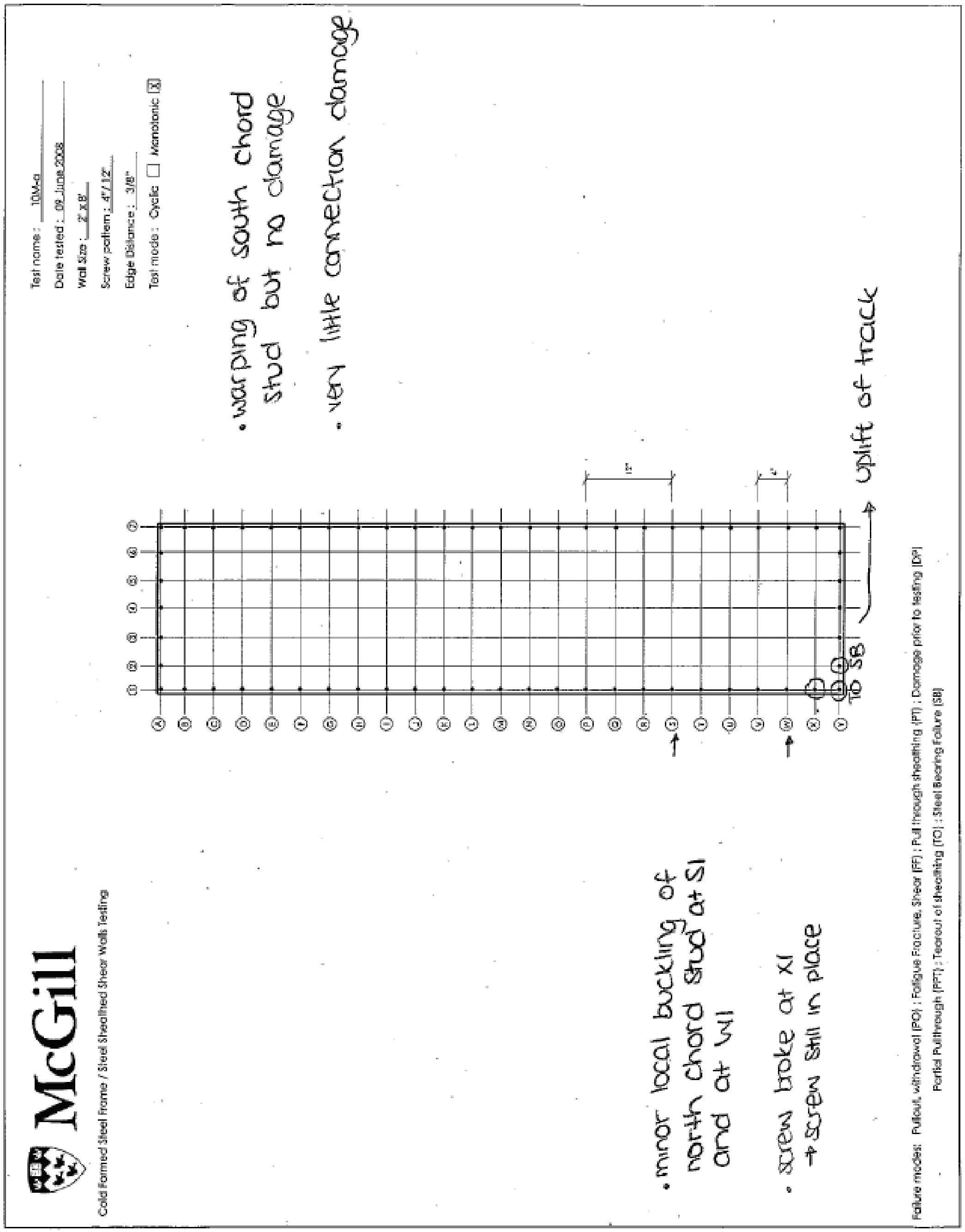


Figure B.55 Observations for Test 10M-a

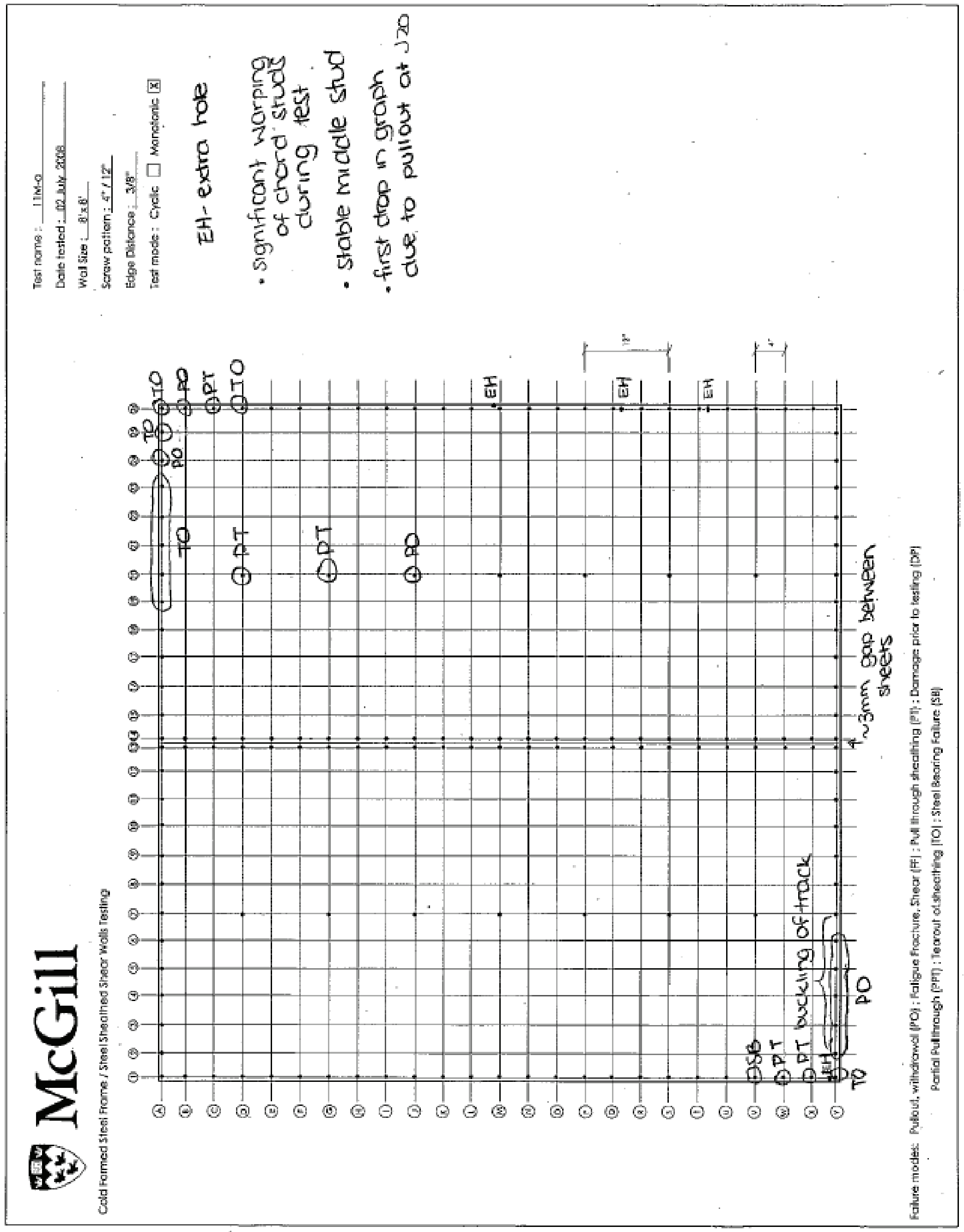


Figure B.56 Observations for Test 11M-a

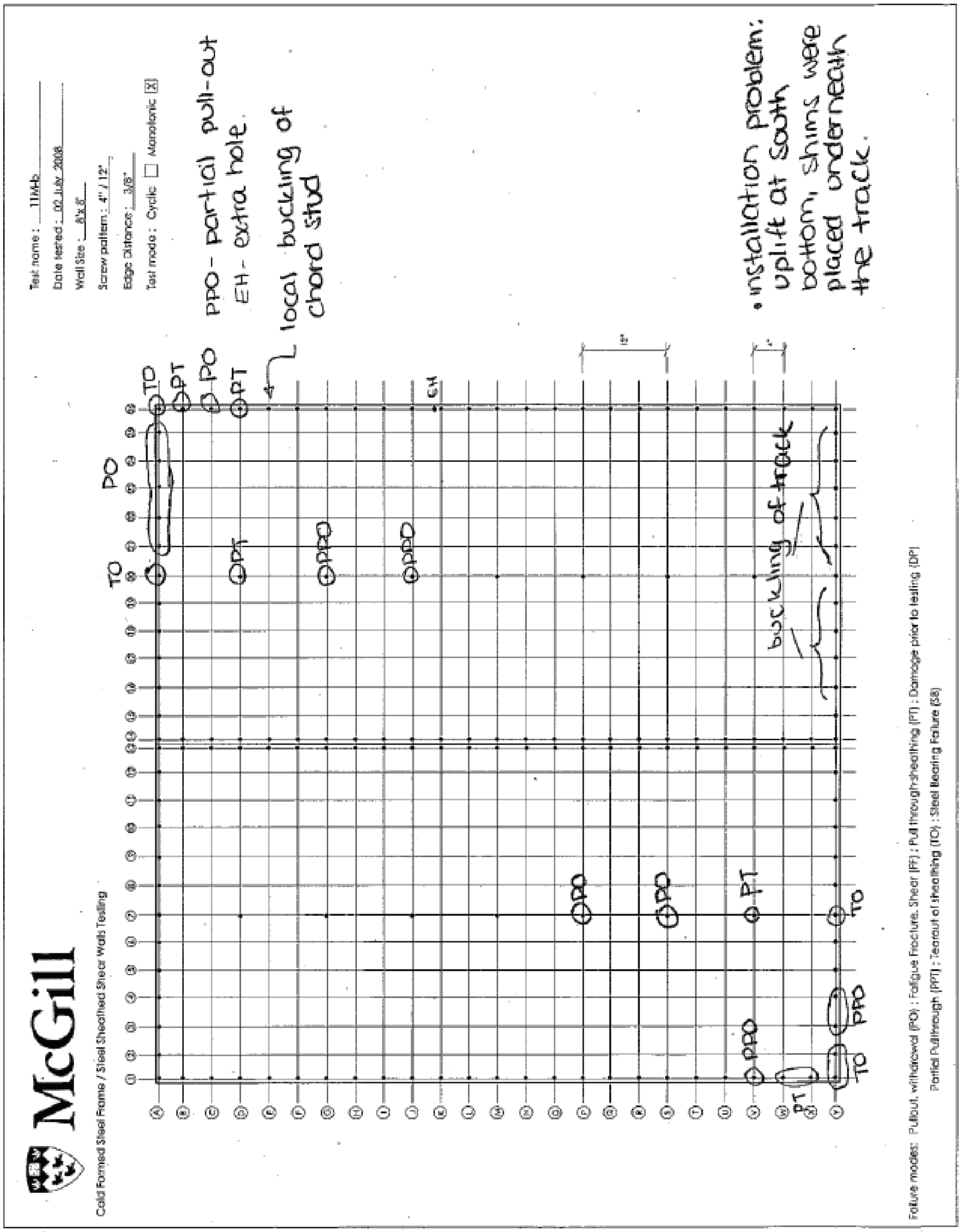


Figure B.57 Observations for Test 11M-b

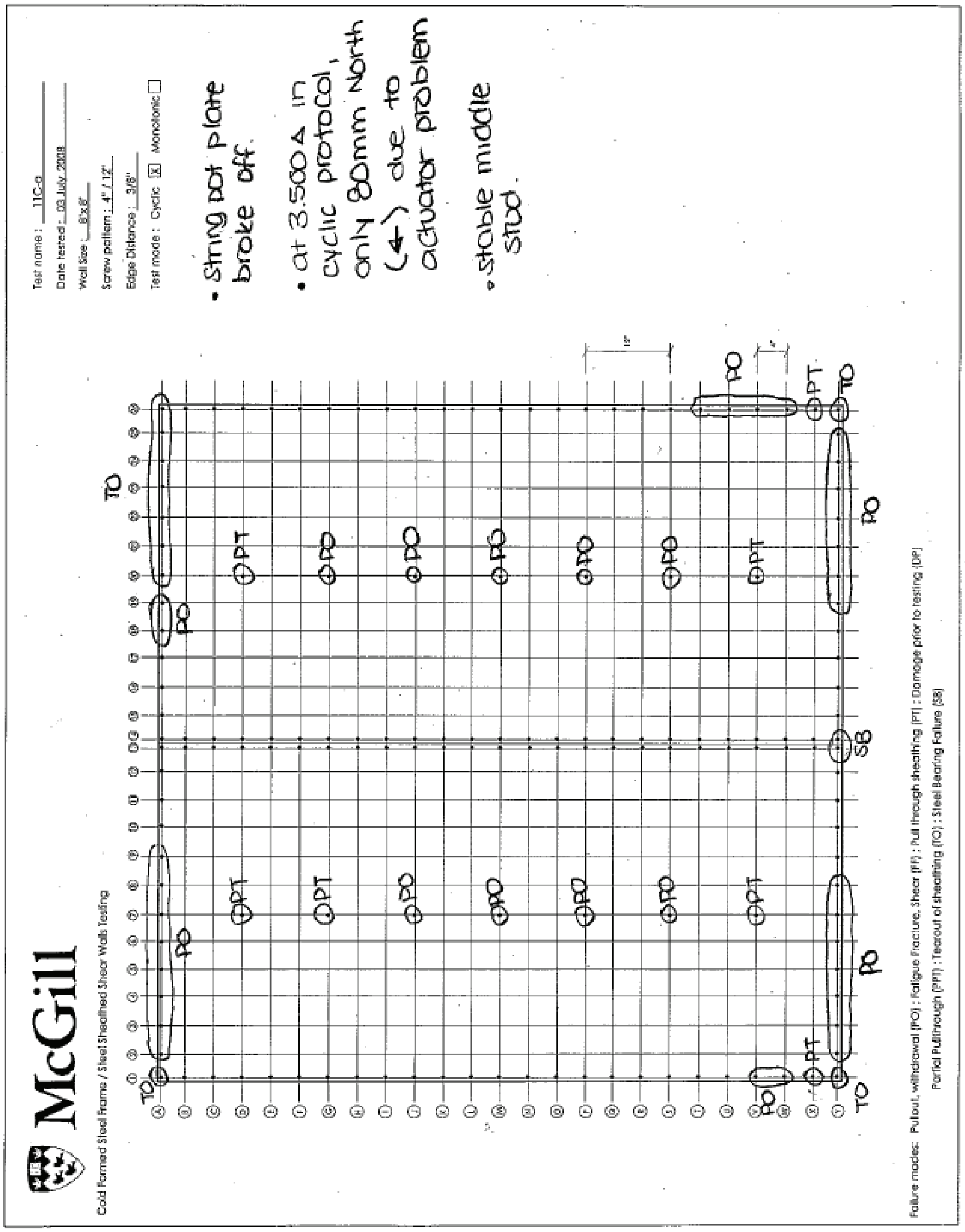


Figure B.58 Observations for Test 11C-a

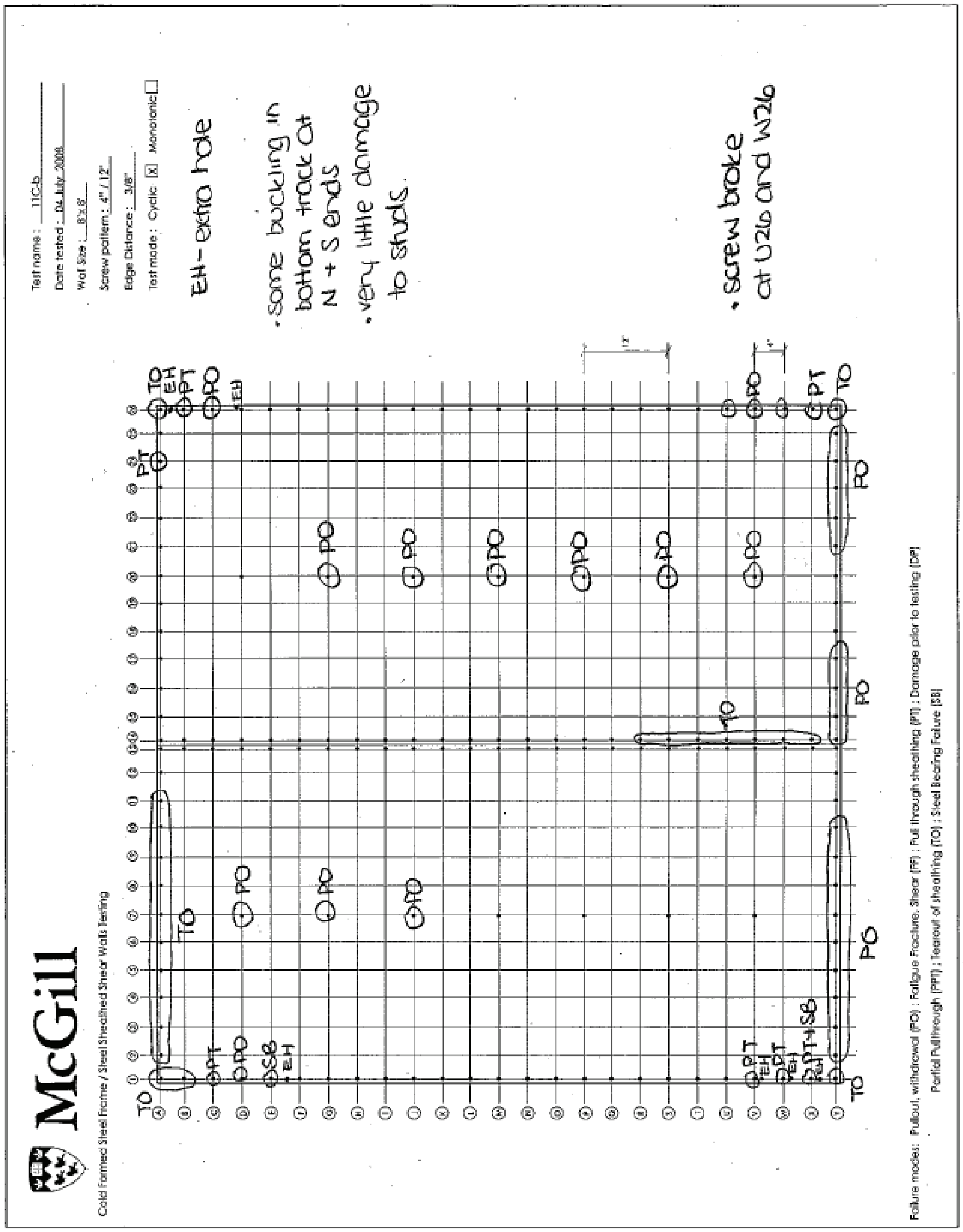


Figure B.59 Observations for Test 11C-b

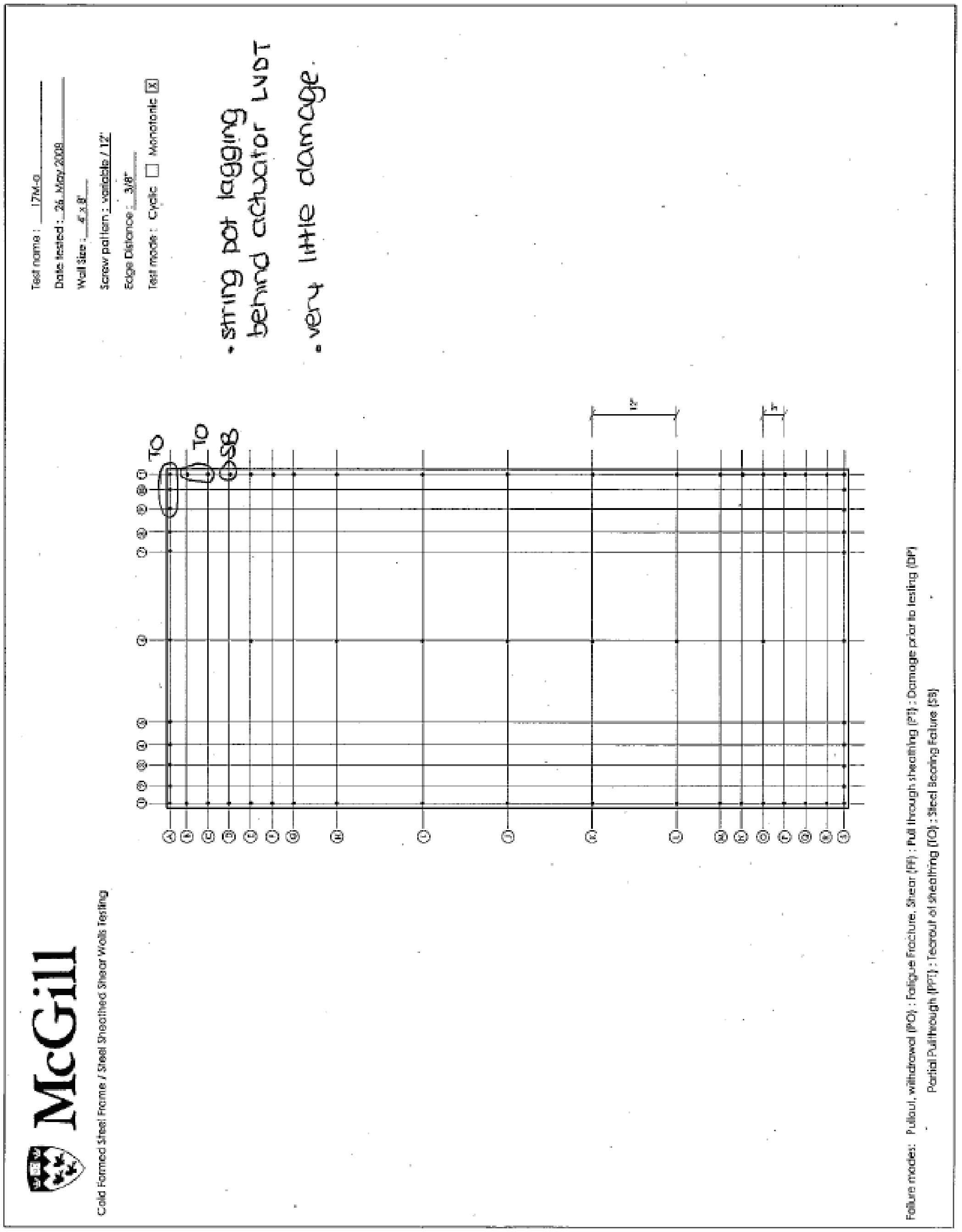


Figure B.60 Observations for Test 17M-a

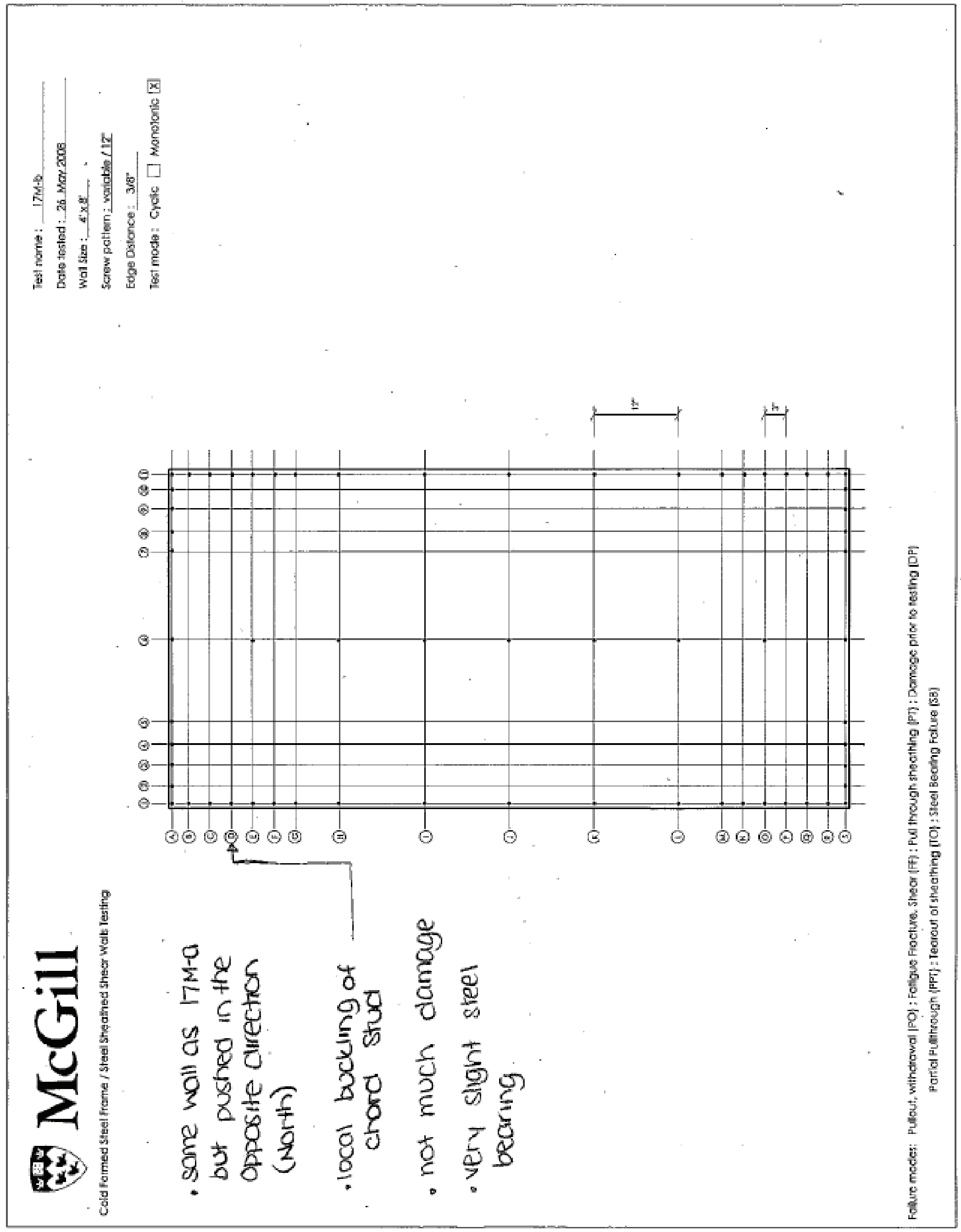


Figure B.61 Observations for Test 17M-b

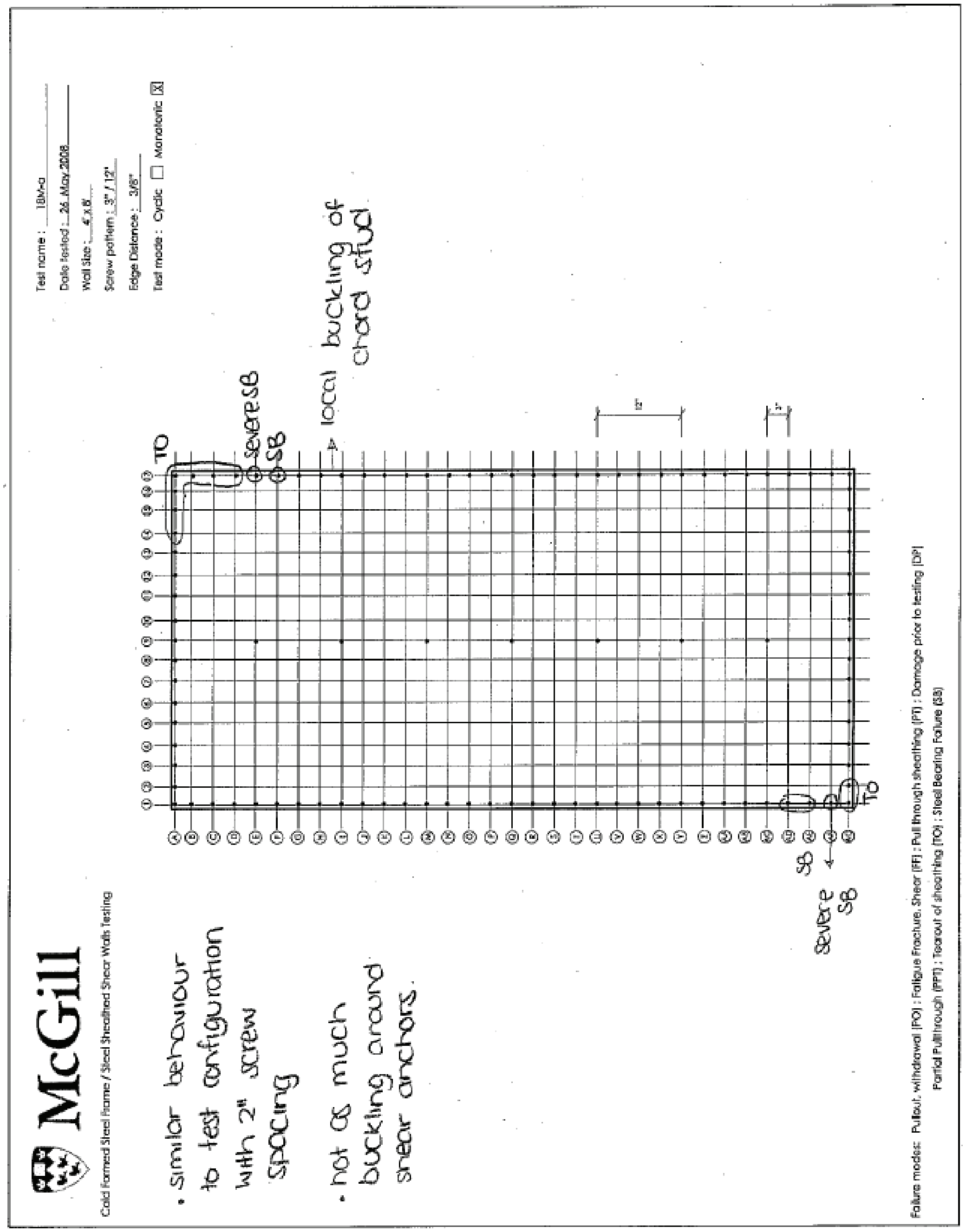


Figure B.62 Observations for Test 18M-a

APPENDIX C

TEST ANALYSIS USING THE EEEP APPROACH

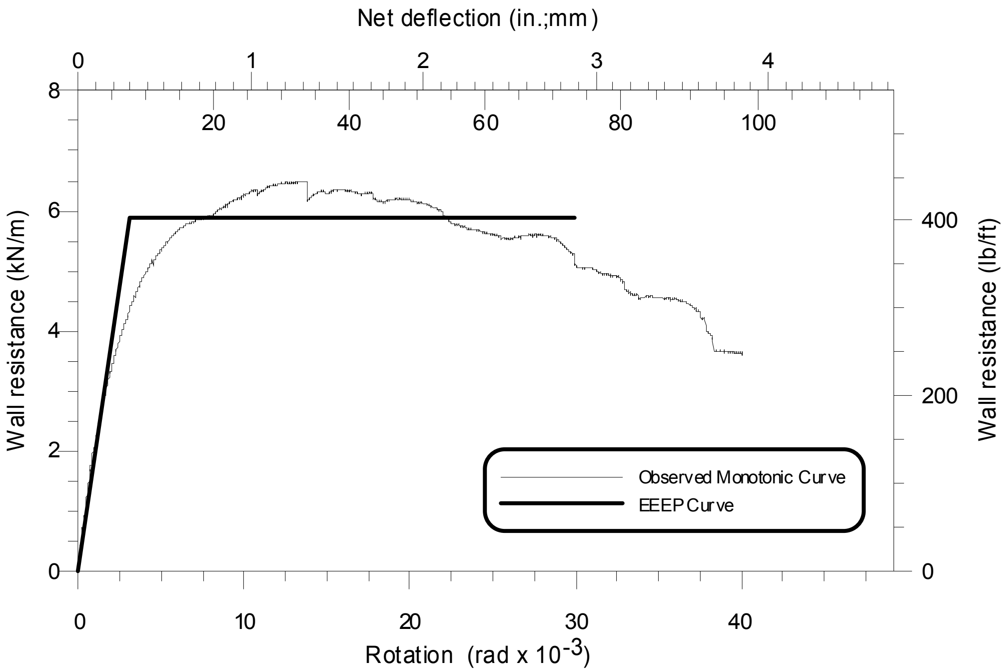


Figure C.1 Observation and EEEP Curves for Test 1M-a

Table C.1 Results for Test 1M-a

Parameters		**Units**
$\mathbf{F_u}$	7.92	kN
$\mathbf{F_{0.8u}}$	6.34	kN
$\mathbf{F_{0.4u}}$	3.17	kN
$\mathbf{F_y}$	7.15	kN
$\mathbf{K_e}$	0.96	kN/mm
Ductility (μ)	9.79	-
$\mathbf{\Delta_{net,y}}$	7.45	mm
$\mathbf{\Delta_{net,u}}$	33.13	mm
$\mathbf{\Delta_{net,0.8u}}$	72.99	mm
$\mathbf{\Delta_{net,0.4u}}$	3.30	mm
Energy	495.52	J
$\mathbf{R_d}$	4.31	-
$\mathbf{S_y}$	5.87	kN/m

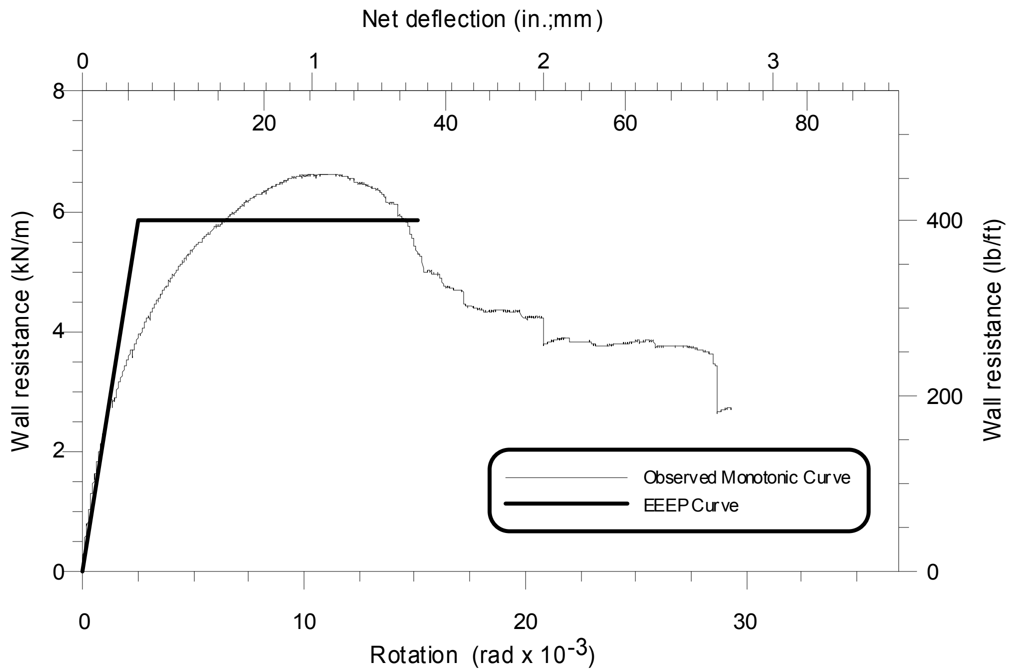


Figure C.2 Observation and EEEP Curves for Test 1M-b

Table C.2 Results for Test 1M-b

Parameters		**Units**
$\mathbf{F_u}$	8.08	kN
$\mathbf{F_{0.8u}}$	6.46	kN
$\mathbf{F_{0.4u}}$	3.23	kN
$\mathbf{F_y}$	7.13	kN
$\mathbf{K_e}$	1.15	kN/mm
Ductility (μ)	5.97	-
$\mathbf{\Delta_{net,y}}$	6.20	mm
$\mathbf{\Delta_{net,u}}$	26.34	mm
$\mathbf{\Delta_{net,0.8u}}$	37.02	mm
$\mathbf{\Delta_{net,0.4u}}$	2.81	mm
Energy	241.76	J
$\mathbf{R_d}$	3.31	-
$\mathbf{S_y}$	5.85	kN/m

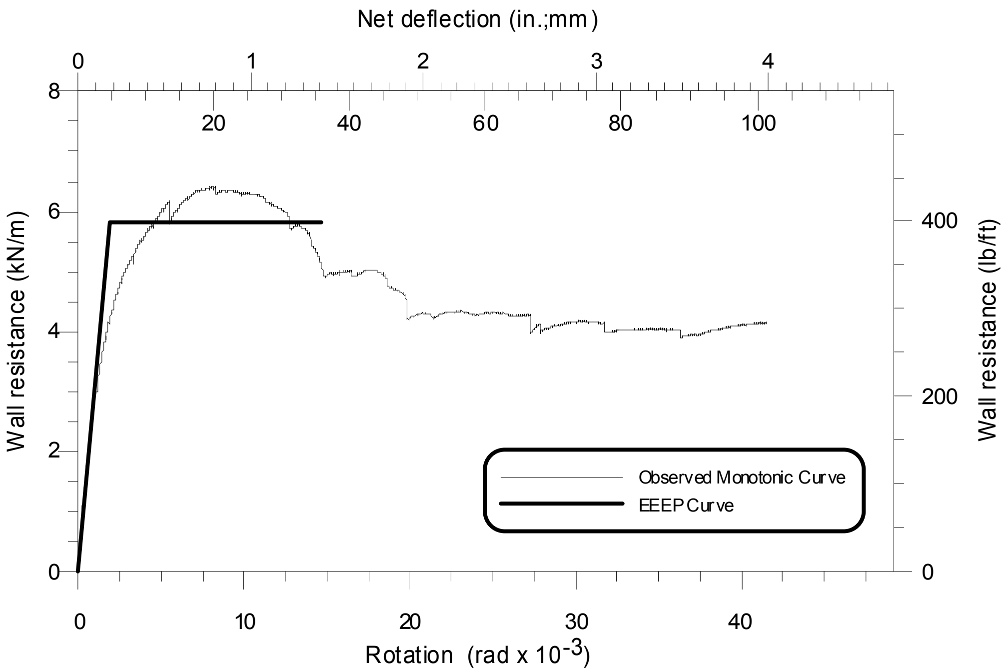


Figure C.3 Observation and EEEP Curves for Test 1M-c

Table C.3 Results for Test 1M-c

Parameters		**Units**
$\mathbf{F_u}$	7.81	kN
$\mathbf{F_{0.8u}}$	6.25	kN
$\mathbf{F_{0.4u}}$	3.13	kN
$\mathbf{F_y}$	7.11	kN
$\mathbf{K_e}$	1.53	kN/mm
Ductility (μ)	7.70	-
$\mathbf{\Delta_{net,y}}$	4.64	mm
$\mathbf{\Delta_{net,u}}$	19.69	mm
$\mathbf{\Delta_{net,0.8u}}$	35.73	mm
$\mathbf{\Delta_{net,0.4u}}$	2.04	mm
Energy	237.60	J
$\mathbf{R_d}$	3.79	-
$\mathbf{S_y}$	5.83	kN/m

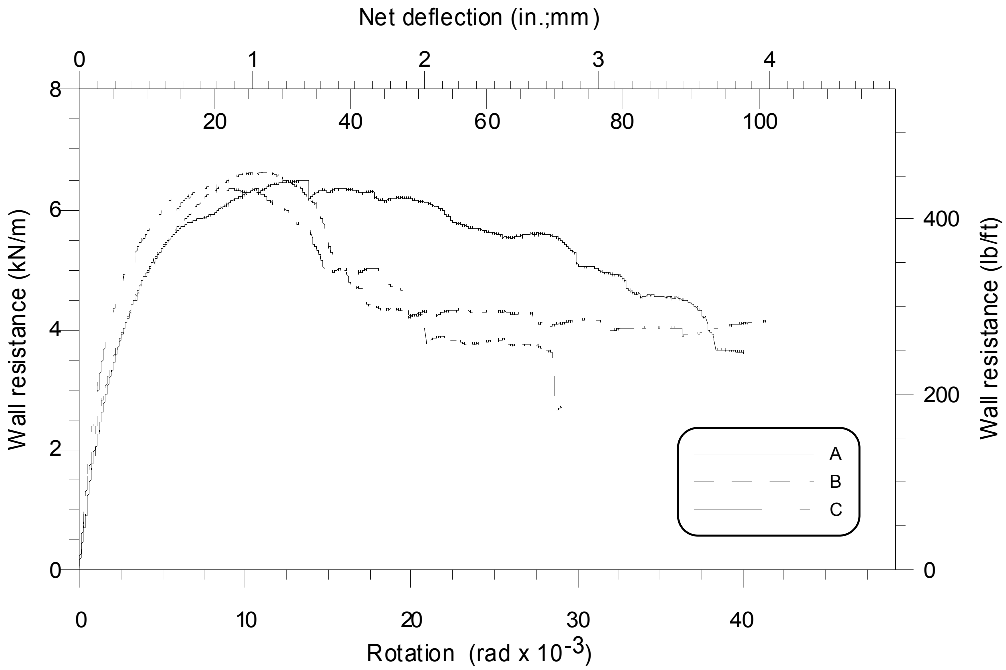


Figure C.4 Comparison of Test Results for Tests 1M-a,b,c

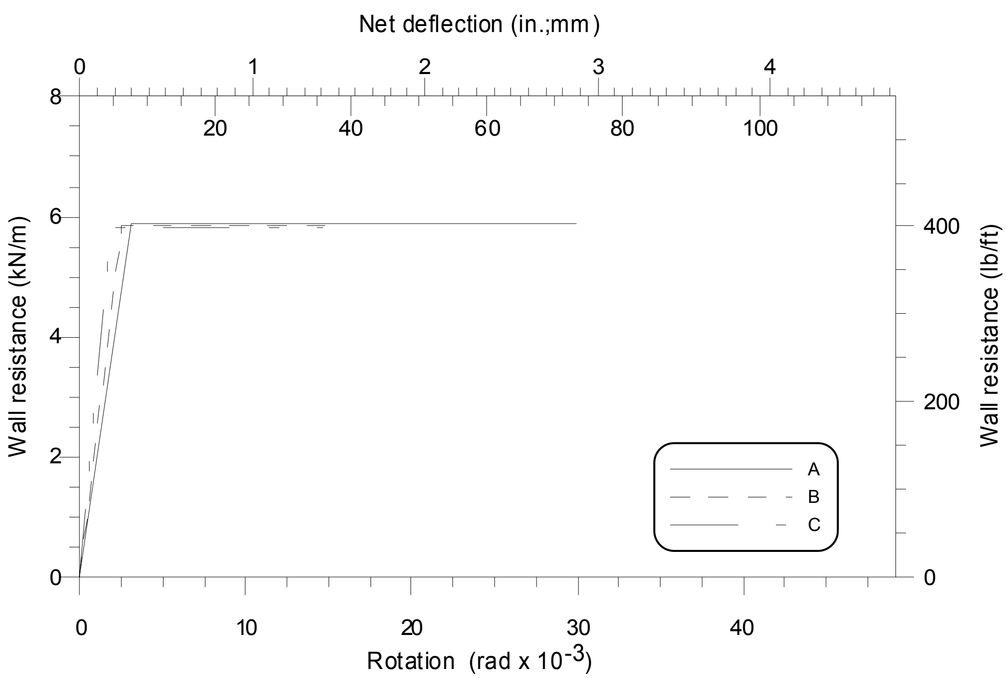


Figure C.5 Comparison of EEEP Results for Tests 1M-a,b,c

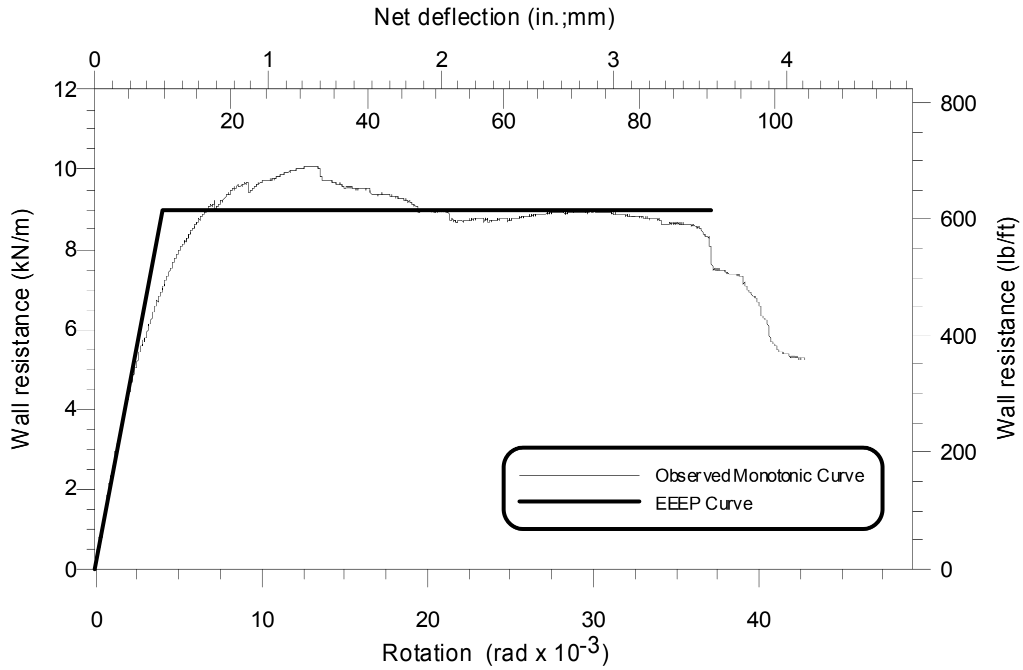


Figure C.6 Observation and EEEP Curves for Test for 2M-a

Table C.4 Results for Test 2M-a

Parameters		**Units**
$\mathbf{F_u}$	12.31	kN
$\mathbf{F_{0.8u}}$	9.85	kN
$\mathbf{F_{0.4u}}$	4.92	kN
$\mathbf{F_y}$	10.97	kN
$\mathbf{K_e}$	1.10	kN/mm
Ductility (μ)	9.10	-
$\mathbf{\Delta_{net,y}}$	9.94	mm
$\mathbf{\Delta_{net,u}}$	31.54	mm
$\mathbf{\Delta_{net,0.8u}}$	90.42	mm
$\mathbf{\Delta_{net,0.4u}}$	4.46	mm
Energy	937.19	J
$\mathbf{R_d}$	4.15	-
$\mathbf{S_y}$	9.00	kN/m

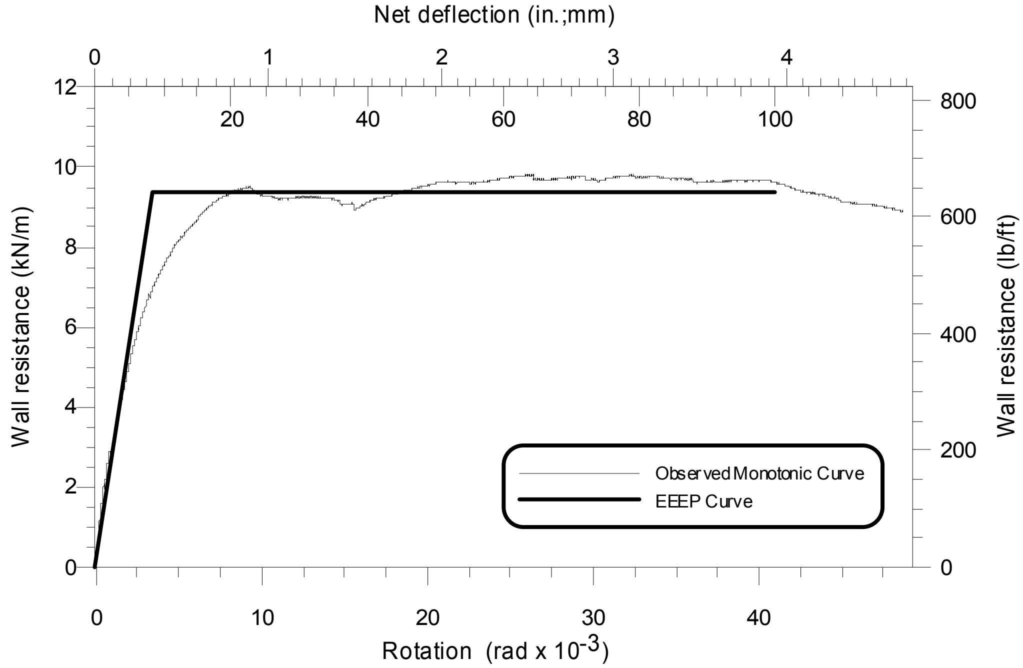


Figure C.7 Observation and EEEP Curves for Test 2M-b

Table C.5 Results for Test 2M-b

Parameters		**Units**
$\mathbf{F_u}$	11.96	kN
$\mathbf{F_{0.8u}}$	9.57	kN
$\mathbf{F_{0.4u}}$	4.79	kN
$\mathbf{F_y}$	11.41	kN
$\mathbf{K_e}$	1.36	kN/mm
Ductility (μ)	11.91	-
$\mathbf{\Delta_{net,y}}$	8.40	mm
$\mathbf{\Delta_{net,u}}$	64.24	mm
$\mathbf{\Delta_{net,0.8u}}$	100.00	mm
$\mathbf{\Delta_{net,0.4u}}$	3.52	mm
Energy	1093.50	J
$\mathbf{R_d}$	4.78	-
$\mathbf{S_y}$	9.36	kN/m

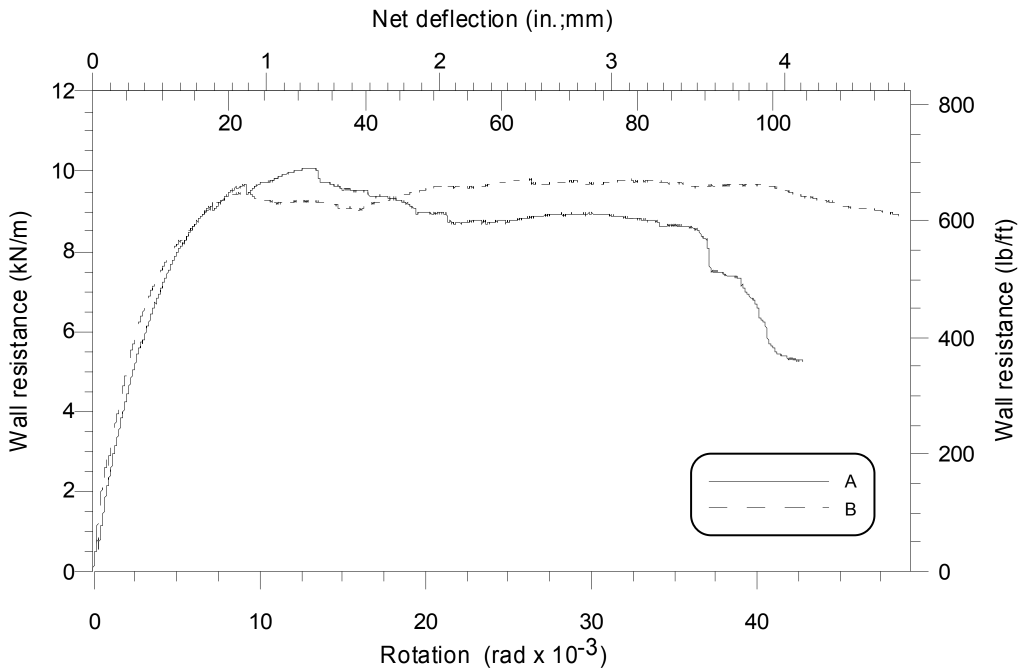


Figure C.8 Comparison of Test Results for Tests 2M-a,b

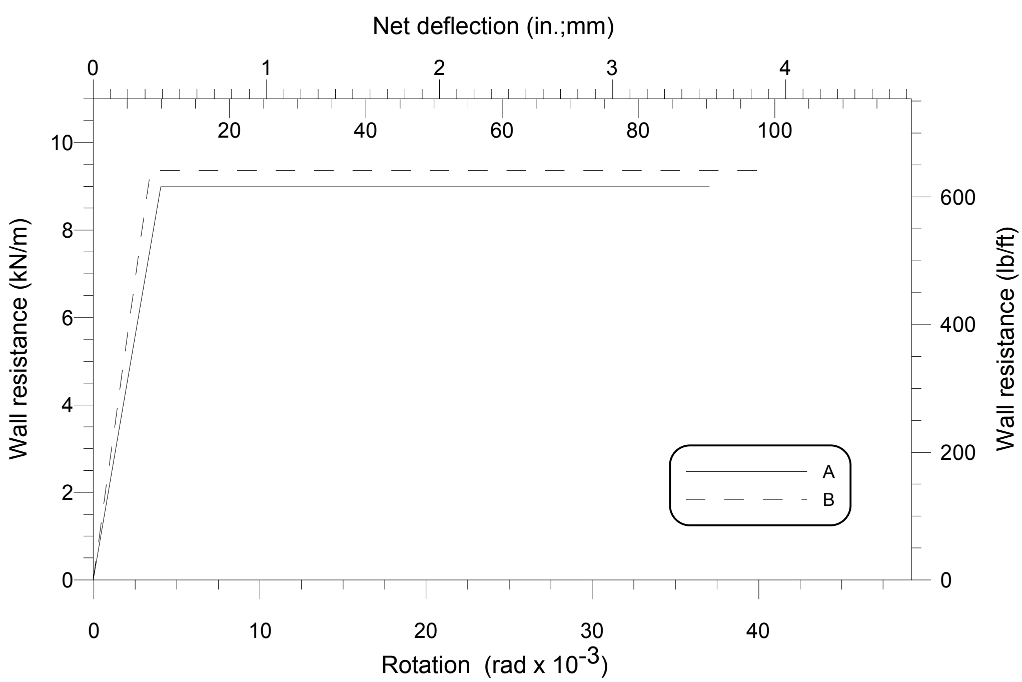


Figure C.9 Comparison of EEEP Results for Tests 2M-a,b

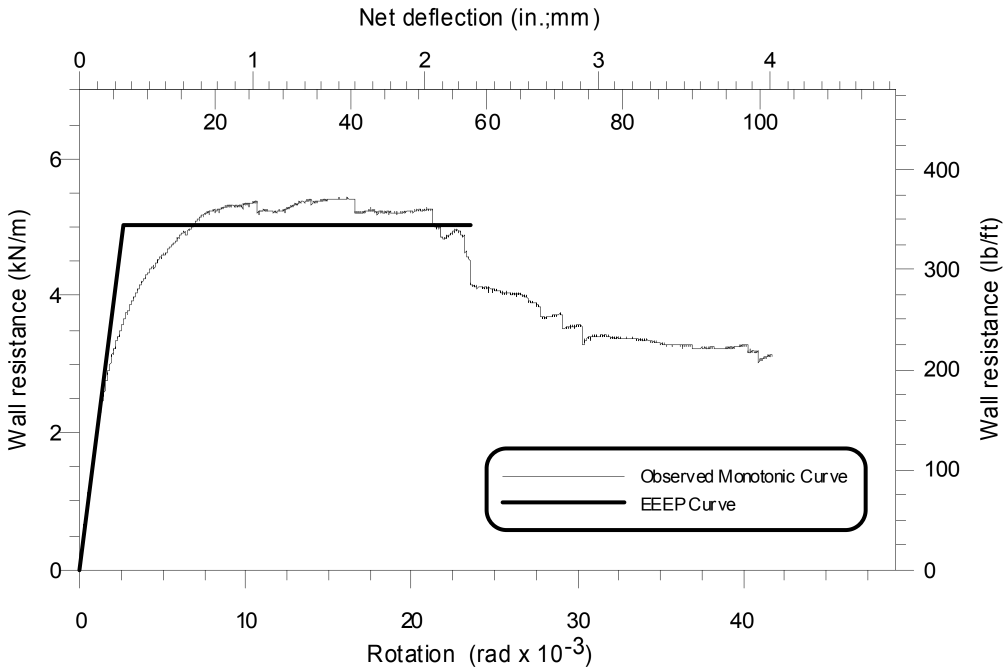


Figure C.10 Observation and EEEP Curves for Test 3M-a

Table C.6 Results for Test 3M-a

Parameters		**Units**
$\mathbf{F_u}$	6.63	kN
$\mathbf{F_{0.8u}}$	5.30	kN
$\mathbf{F_{0.4u}}$	2.65	kN
$\mathbf{F_y}$	6.14	kN
$\mathbf{K_e}$	0.93	kN/mm
Ductility (μ)	8.75	-
$\mathbf{\Delta_{net,y}}$	6.58	mm
$\mathbf{\Delta_{net,u}}$	39.48	mm
$\mathbf{\Delta_{net,0.8u}}$	57.56	mm
$\mathbf{\Delta_{net,0.4u}}$	2.84	mm
Energy	333.39	J
$\mathbf{R_d}$	4.06	-
$\mathbf{S_y}$	5.04	kN/m

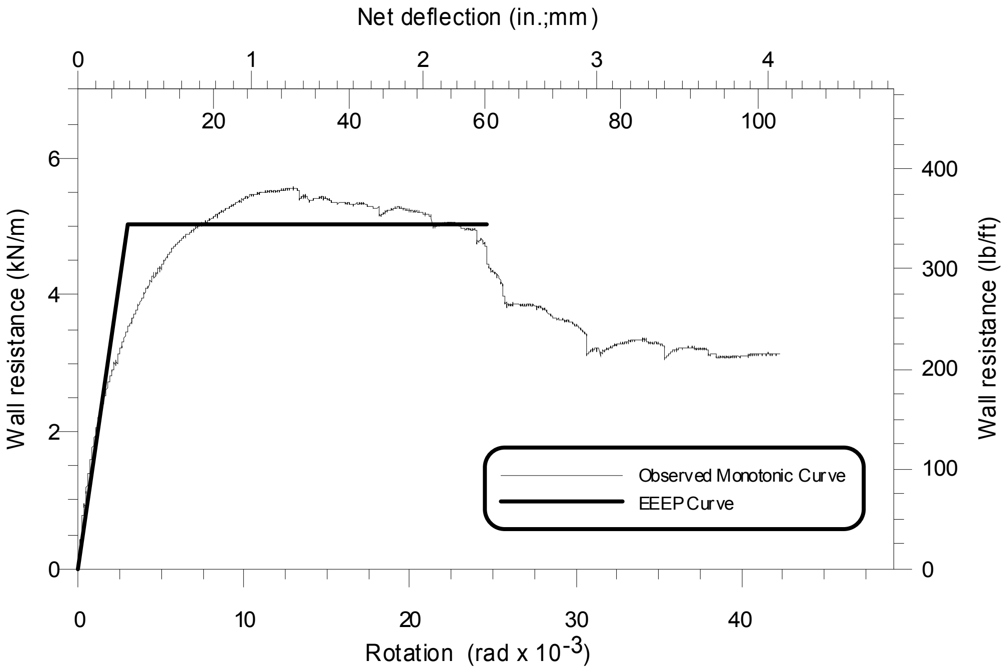


Figure C.11 Observation and EEEP Curves for Test 3M-b

Table C.7 Results for Test 3M-b

Parameters		**Units**
$\mathbf{F_u}$	6.80	kN
$\mathbf{F_{0.8u}}$	5.44	kN
$\mathbf{F_{0.4u}}$	2.72	kN
$\mathbf{F_y}$	6.15	kN
$\mathbf{K_e}$	0.86	kN/mm
Ductility (μ)	8.43	-
$\mathbf{\Delta_{net,y}}$	7.15	mm
$\mathbf{\Delta_{net,u}}$	31.72	mm
$\mathbf{\Delta_{net,0.8u}}$	60.23	mm
$\mathbf{\Delta_{net,0.4u}}$	3.16	mm
Energy	348.41	J
$\mathbf{R_d}$	3.98	-
$\mathbf{S_y}$	5.04	kN/m

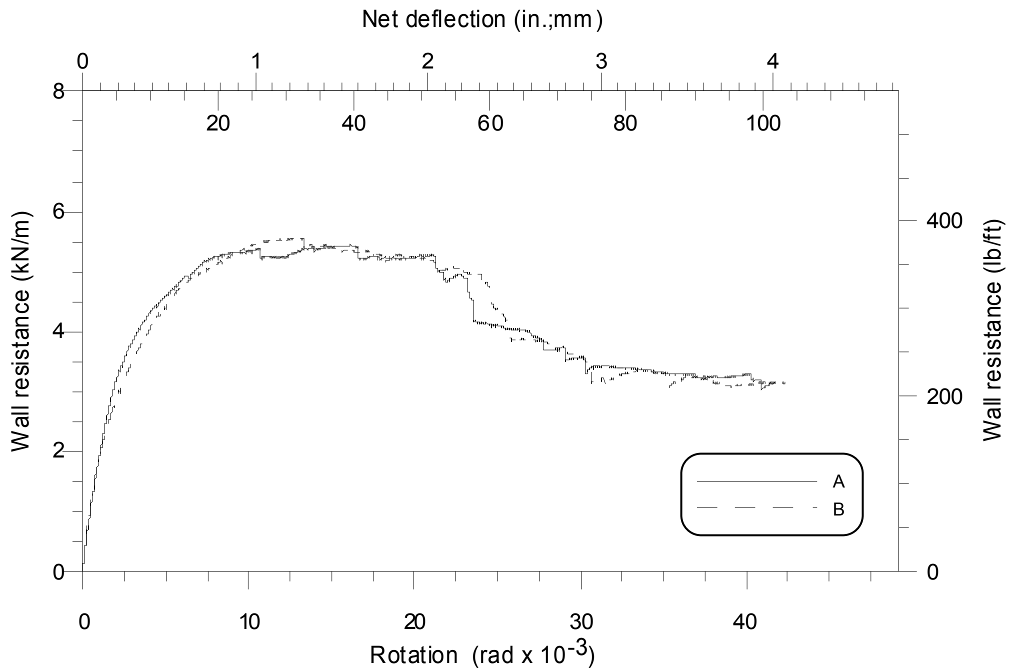


Figure C.12 Comparison of Test Results for Tests 3Ma,b

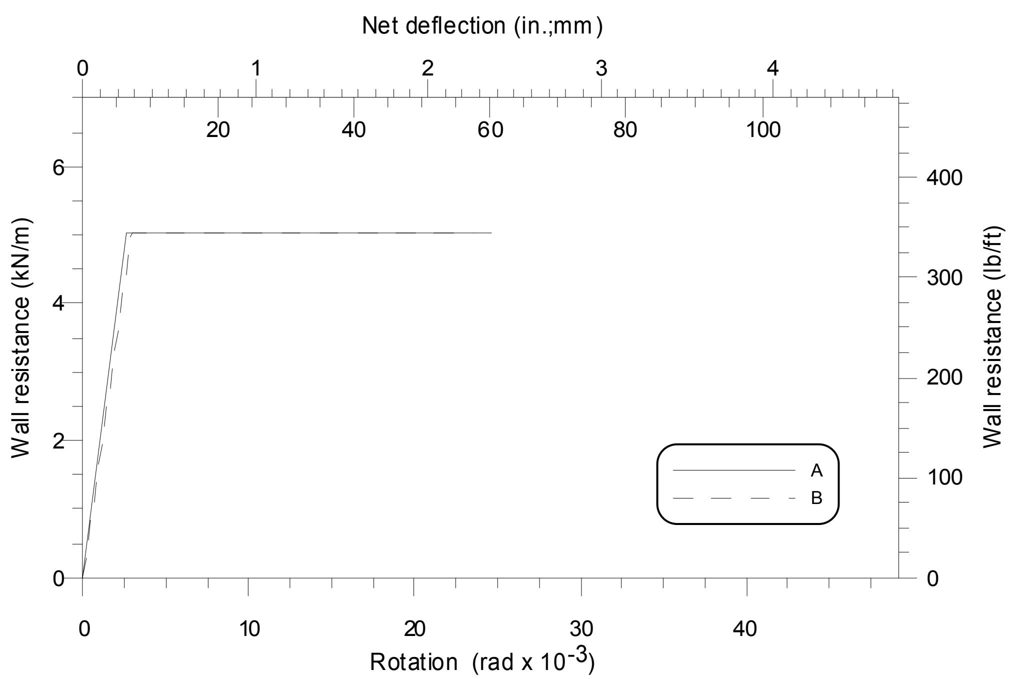


Figure C.13 Comparison of EEEP Results for Tests 3Ma,b

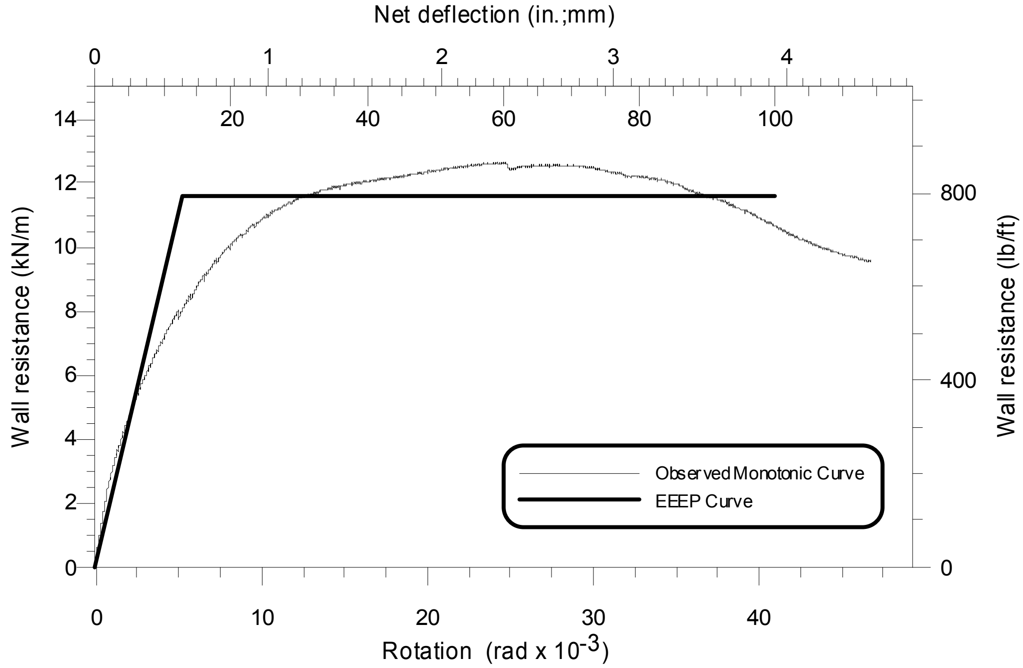


Figure C.14 Observation and EEEP Curves for Test 8M-a

Table C.8 Results for Test 8M-a

Parameters		**Units**
$\mathbf{F_u}$	7.72	kN
$\mathbf{F_{0.8u}}$	6.18	kN
$\mathbf{F_{0.4u}}$	3.09	kN
$\mathbf{F_y}$	7.07	kN
$\mathbf{K_e}$	0.56	kN/mm
Ductility (μ)	7.86	-
$\mathbf{\Delta_{net,y}}$	12.73	mm
$\mathbf{\Delta_{net,u}}$	59.01	mm
$\mathbf{\Delta_{net,0.8u}}$	100.00	mm
$\mathbf{\Delta_{net,0.4u}}$	5.56	mm
Energy	662.18	J
$\mathbf{R_d}$	3.84	-
$\mathbf{S_y}$	11.60	kN/m

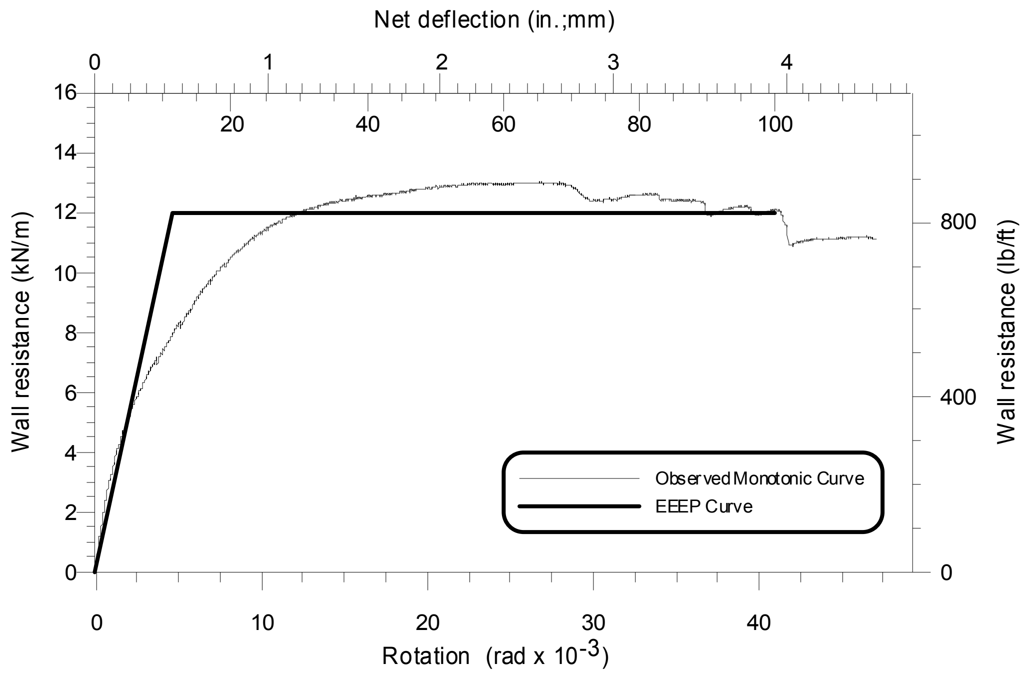


Figure C.15 Observation and EEEP Curves for Test 8M-b

Table C.9 Results for Test 8M-b

Parameters		**Units**
$\mathbf{F_u}$	7.94	kN
$\mathbf{F_{0.8u}}$	6.35	kN
$\mathbf{F_{0.4u}}$	3.17	kN
$\mathbf{F_y}$	7.32	kN
$\mathbf{K_e}$	0.64	kN/mm
Ductility (μ)	8.73	-
$\mathbf{\Delta_{net,y}}$	11.45	mm
$\mathbf{\Delta_{net,u}}$	65.32	mm
$\mathbf{\Delta_{net,0.8u}}$	100.00	mm
$\mathbf{\Delta_{net,0.4u}}$	4.97	mm
Energy	689.67	J
$\mathbf{R_d}$	4.06	-
$\mathbf{S_y}$	12.00	kN/m

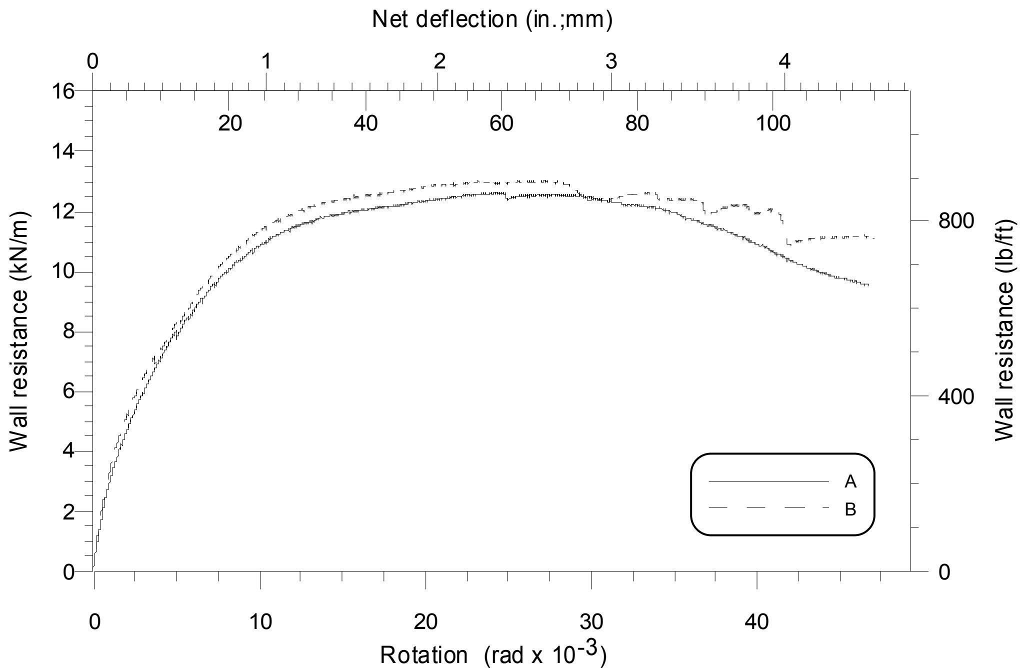


Figure C.16 Comparison of Test Results for Tests 8M-a,b

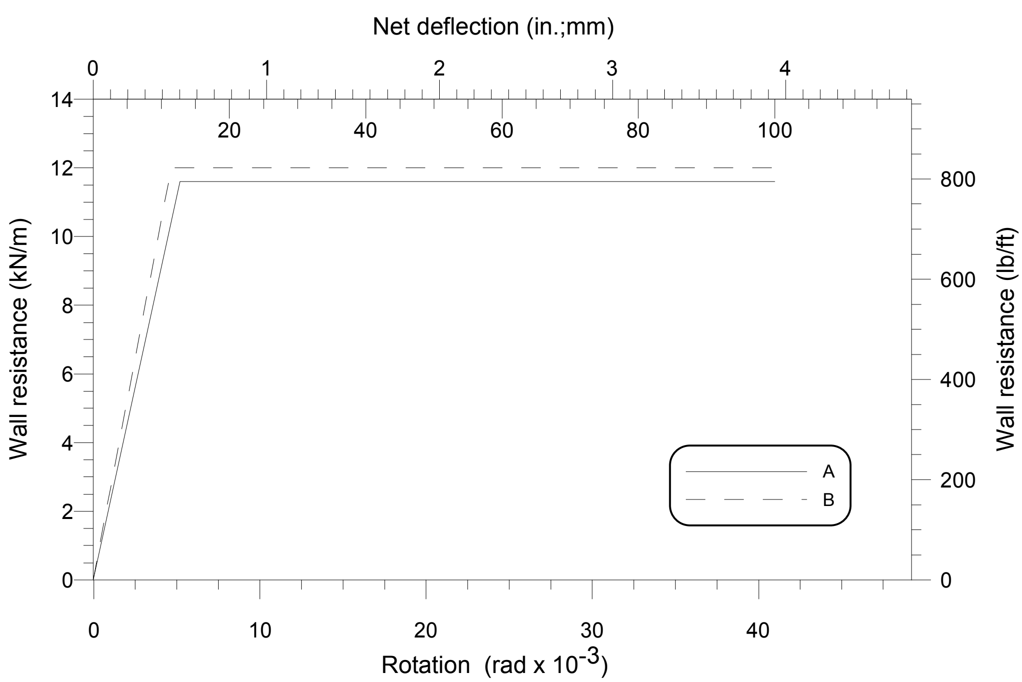


Figure C.17 Comparison of EEEP Results for Tests 8M-a,b

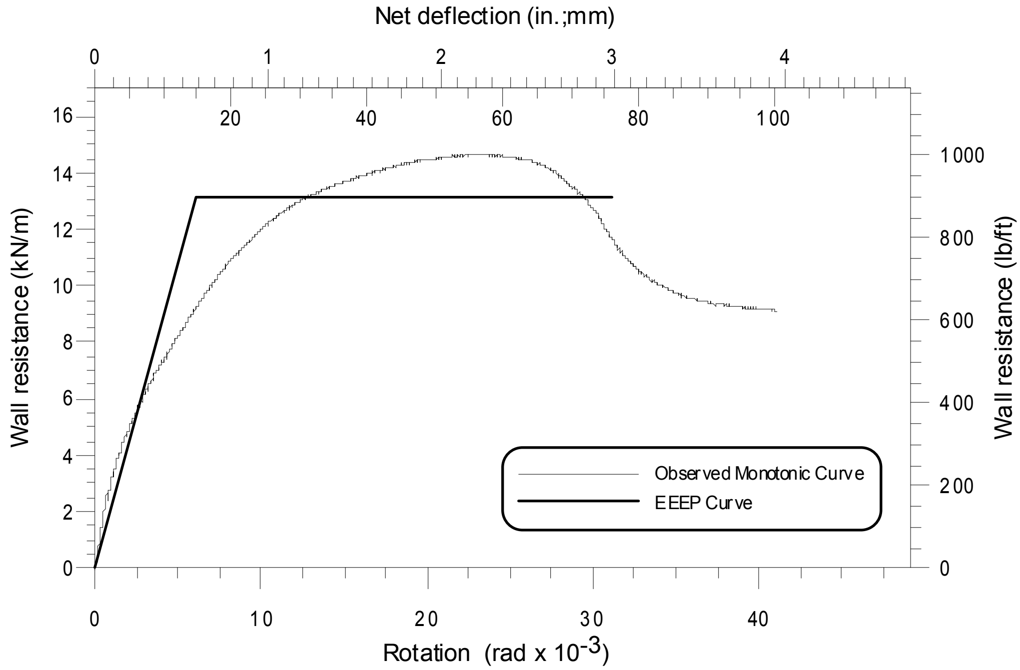


Figure C.18 Observation and EEEP Curves for Test 9M-a

Table C.10 Results for Test 9M-a

Parameters		**Units**
$\mathbf{F_u}$	8.94	kN
$\mathbf{F_{0.8u}}$	7.15	kN
$\mathbf{F_{0.4u}}$	3.57	kN
$\mathbf{F_y}$	8.02	kN
$\mathbf{K_e}$	0.54	kN/mm
Ductility (μ)	5.06	-
$\mathbf{\Delta_{net,y}}$	14.98	mm
$\mathbf{\Delta_{net,u}}$	53.17	mm
$\mathbf{\Delta_{net,0.8u}}$	75.85	mm
$\mathbf{\Delta_{net,0.4u}}$	6.68	mm
Energy	548.05	J
$\mathbf{R_d}$	3.02	-
$\mathbf{S_y}$	13.15	kN/m

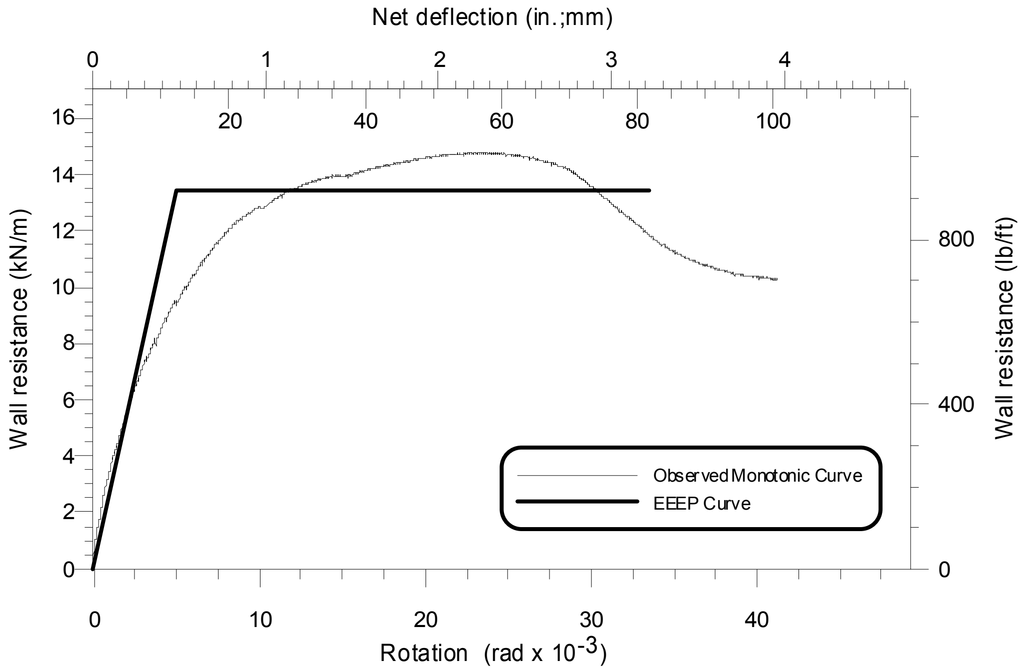


Figure C.19 Observation and EEEP Curves for Test 9M-b

Table C.11 Results for Test 9M-b

Parameters		**Units**
$\mathbf{F_u}$	9.01	kN
$\mathbf{F_{0.8u}}$	7.21	kN
$\mathbf{F_{0.4u}}$	3.60	kN
$\mathbf{F_y}$	8.17	kN
$\mathbf{K_e}$	0.67	kN/mm
Ductility (μ)	6.67	-
$\mathbf{\Delta_{net,y}}$	12.26	mm
$\mathbf{\Delta_{net,u}}$	55.88	mm
$\mathbf{\Delta_{net,0.8u}}$	81.84	mm
$\mathbf{\Delta_{net,0.4u}}$	5.41	mm
Energy	618.56	J
$\mathbf{R_d}$	3.51	-
$\mathbf{S_y}$	13.40	kN/m

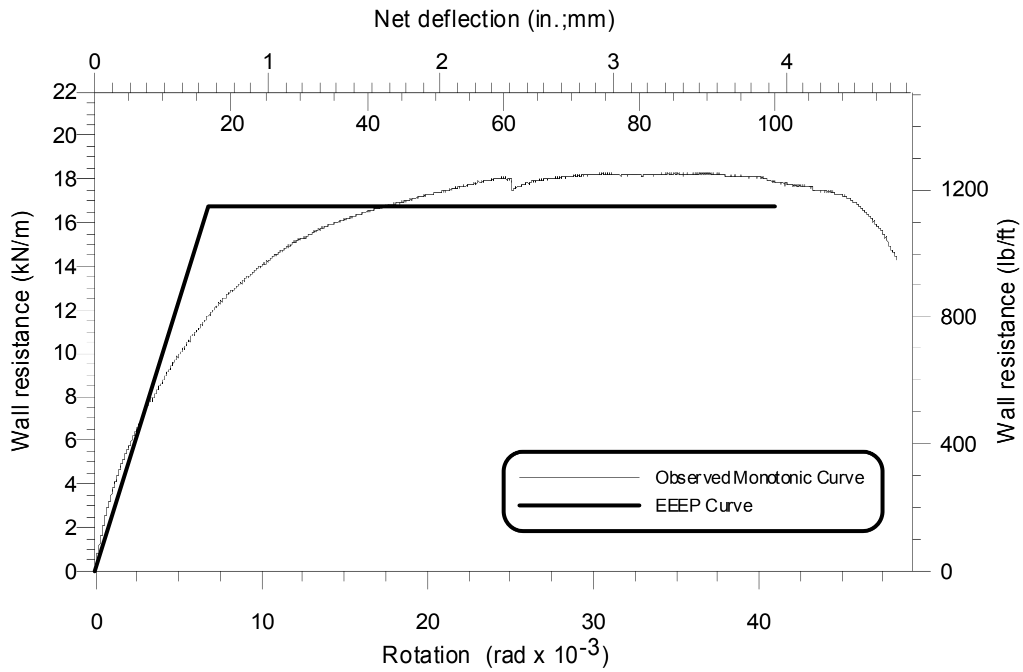


Figure C.20 Observation and EEEP Curves for Test 9M-c

Table C.12 Results for Test 9M-c

Parameters		**Units**
$\mathbf{F_u}$	11.16	kN
$\mathbf{F_{0.8u}}$	8.92	kN
$\mathbf{F_{0.4u}}$	4.46	kN
$\mathbf{F_y}$	10.22	kN
$\mathbf{K_e}$	0.61	kN/mm
Ductility (μ)	6.00	-
$\mathbf{\Delta_{net,y}}$	16.67	mm
$\mathbf{\Delta_{net,u}}$	88.53	mm
$\mathbf{\Delta_{net,0.8u}}$	100.00	mm
$\mathbf{\Delta_{net,0.4u}}$	7.28	mm
Energy	936.53	J
$\mathbf{R_d}$	3.32	-
$\mathbf{S_y}$	16.76	kN/m

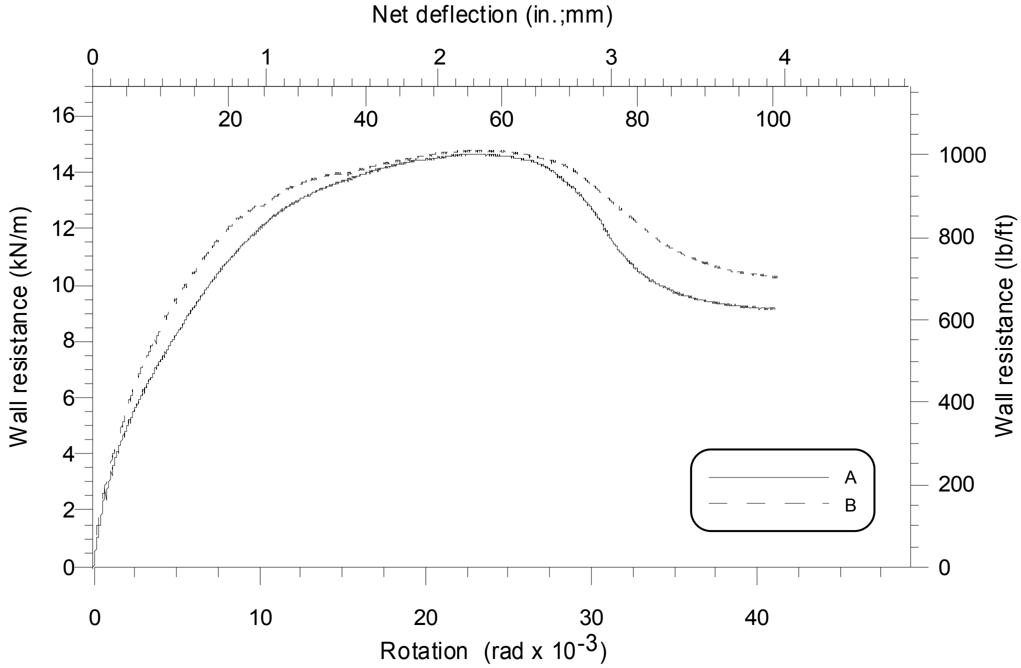


Figure C.21 Comparison of Test Results for Tests 9M-a,b

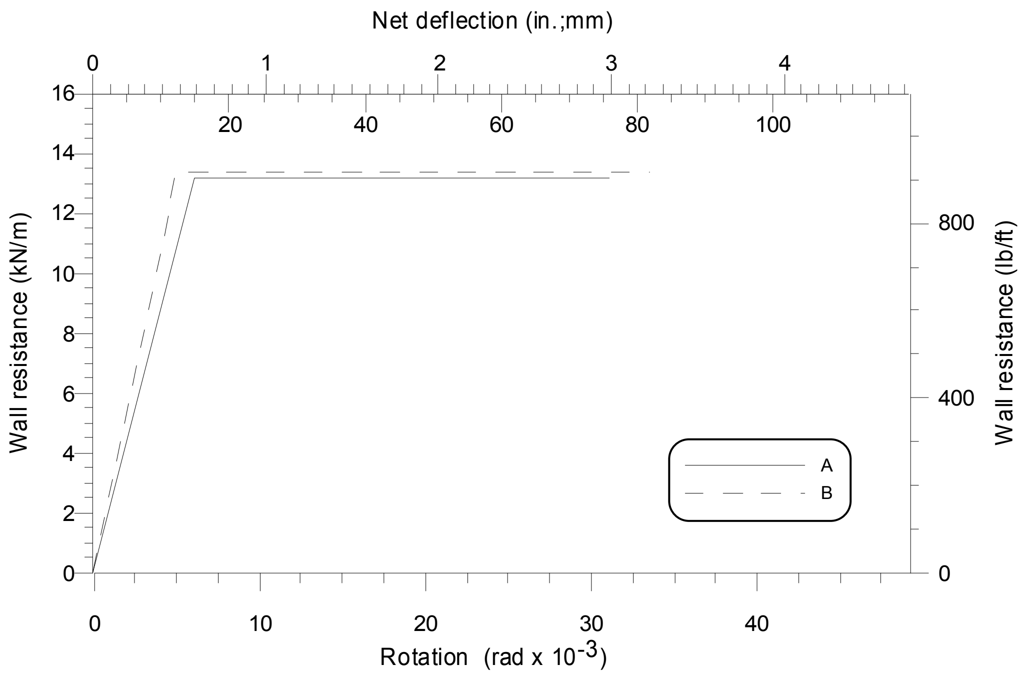


Figure C.22 Comparison of EEEP Results for Tests 9M-a,b

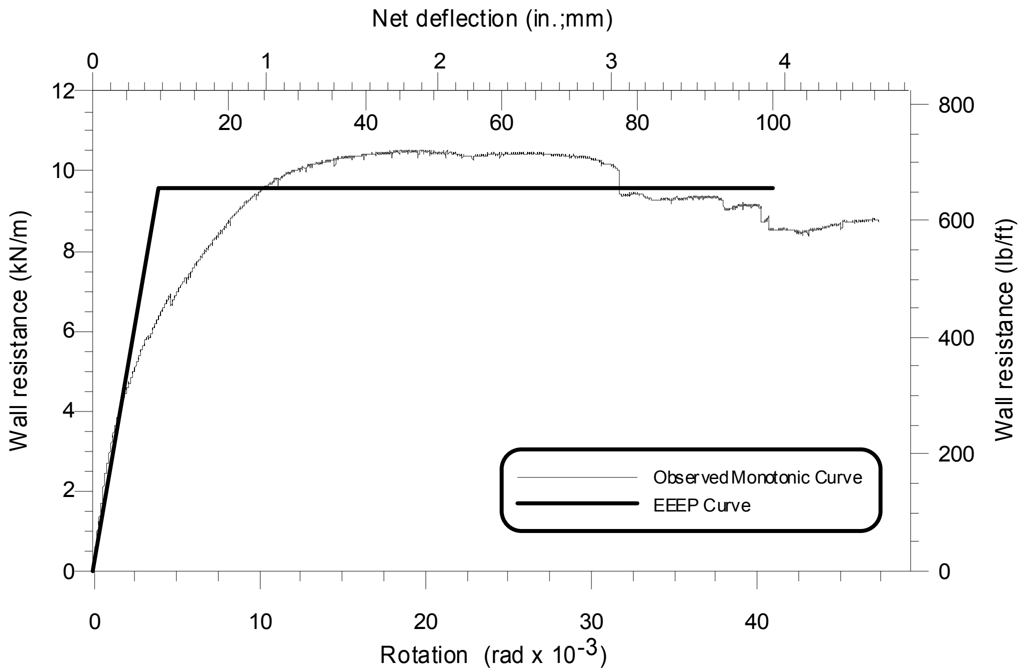


Figure C.23 Observation and EEEP Curves for Test 10M-a

Table C.13 Results for Test 10M-a

Parameters		**Units**
$\mathbf{F_u}$	6.42	kN
$\mathbf{F_{0.8u}}$	5.14	kN
$\mathbf{F_{0.4u}}$	2.57	kN
$\mathbf{F_y}$	5.85	kN
$\mathbf{K_e}$	0.61	kN/mm
Ductility (μ)	10.46	-
$\mathbf{\Delta_{net,y}}$	9.56	mm
$\mathbf{\Delta_{net,u}}$	44.18	mm
$\mathbf{\Delta_{net,0.8u}}$	100.00	mm
$\mathbf{\Delta_{net,0.4u}}$	4.20	mm
Energy	557.07	J
$\mathbf{R_d}$	4.46	-
$\mathbf{S_y}$	9.60	kN/m

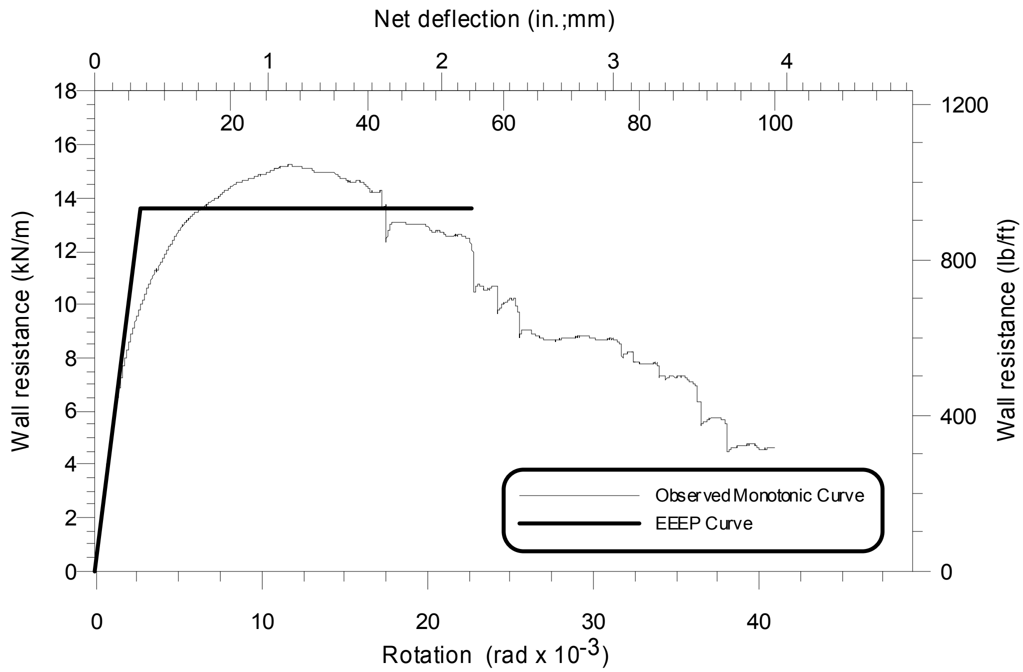


Figure C.24 Observation and EEEP Curves for Test 11M-a

Table C.14 Results for Test 11M-a

Parameters		**Units**
$\mathbf{F_u}$	37.19	kN
$\mathbf{F_{0.8u}}$	29.75	kN
$\mathbf{F_{0.4u}}$	14.88	kN
$\mathbf{F_y}$	33.18	kN
$\mathbf{K_e}$	5.01	kN/mm
Ductility (μ)	8.34	-
$\mathbf{\Delta_{net,y}}$	6.63	mm
$\mathbf{\Delta_{net,u}}$	28.66	mm
$\mathbf{\Delta_{net,0.8u}}$	55.26	mm
$\mathbf{\Delta_{net,0.4u}}$	2.97	mm
Energy	1723.80	J
$\mathbf{R_d}$	3.96	-
$\mathbf{S_y}$	13.61	kN/m

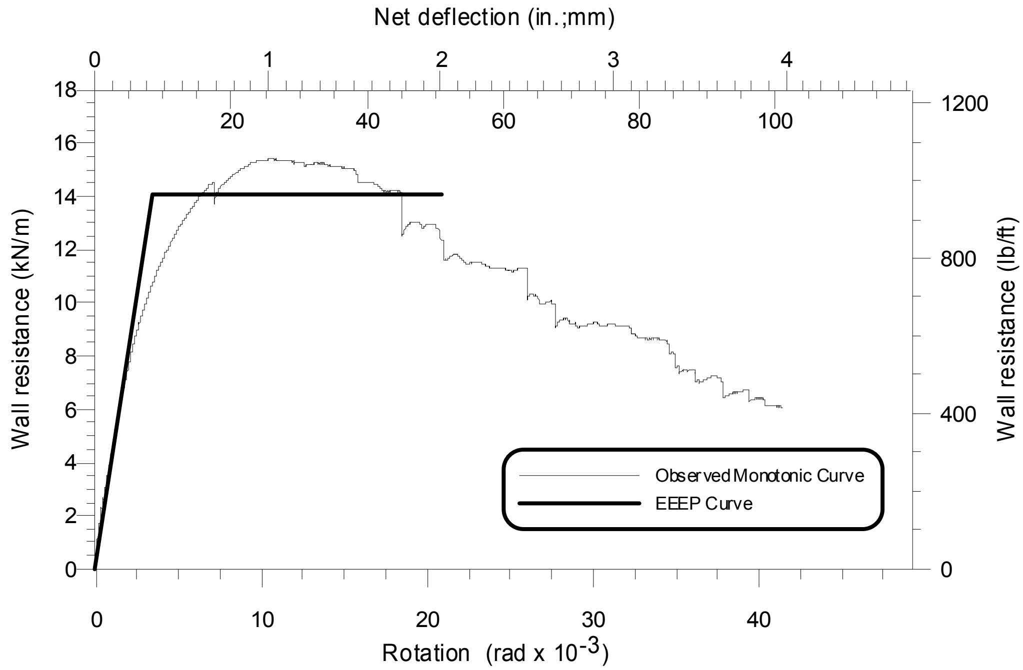


Figure C.25 Observation and EEEP Curves for Test 11M-b

Table C.15 Results for Test 11M-b

Parameters		**Units**
$\mathbf{F_u}$	37.57	kN
$\mathbf{F_{0.8u}}$	30.06	kN
$\mathbf{F_{0.4u}}$	15.03	kN
$\mathbf{F_y}$	34.38	kN
$\mathbf{K_e}$	4.08	kN/mm
Ductility (μ)	6.05	-
$\mathbf{\Delta_{net,y}}$	8.42	mm
$\mathbf{\Delta_{net,u}}$	25.84	mm
$\mathbf{\Delta_{net,0.8u}}$	50.96	mm
$\mathbf{\Delta_{net,0.4u}}$	3.68	mm
Energy	1607.28	J
$\mathbf{R_d}$	3.33	-
$\mathbf{S_y}$	14.10	kN/m

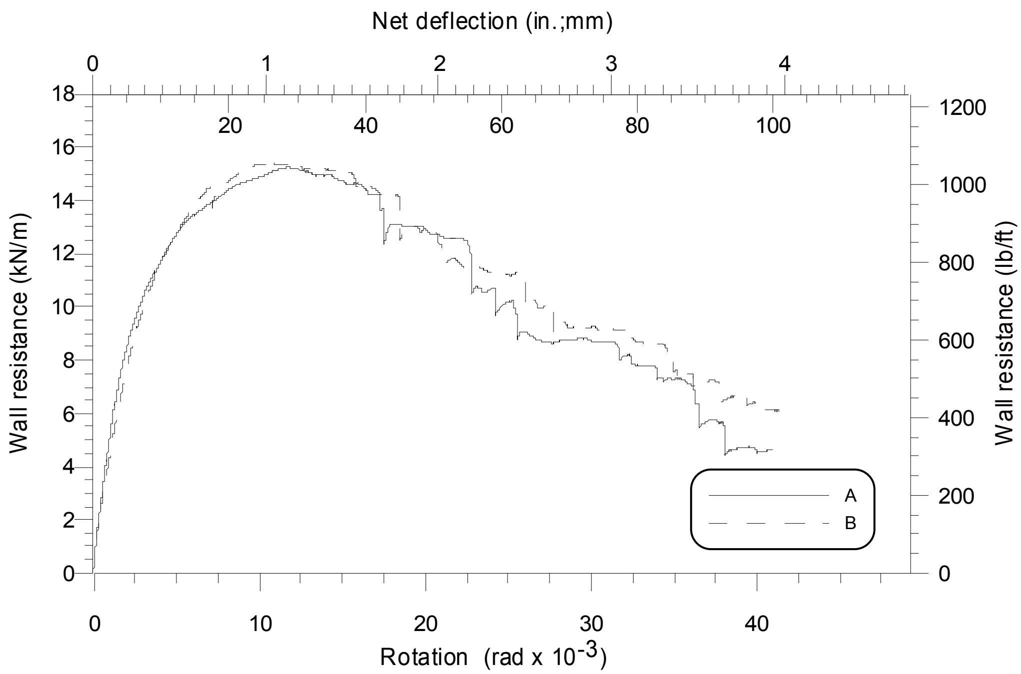


Figure C.26 Comparison of Test Results for Tests 11M-a,b

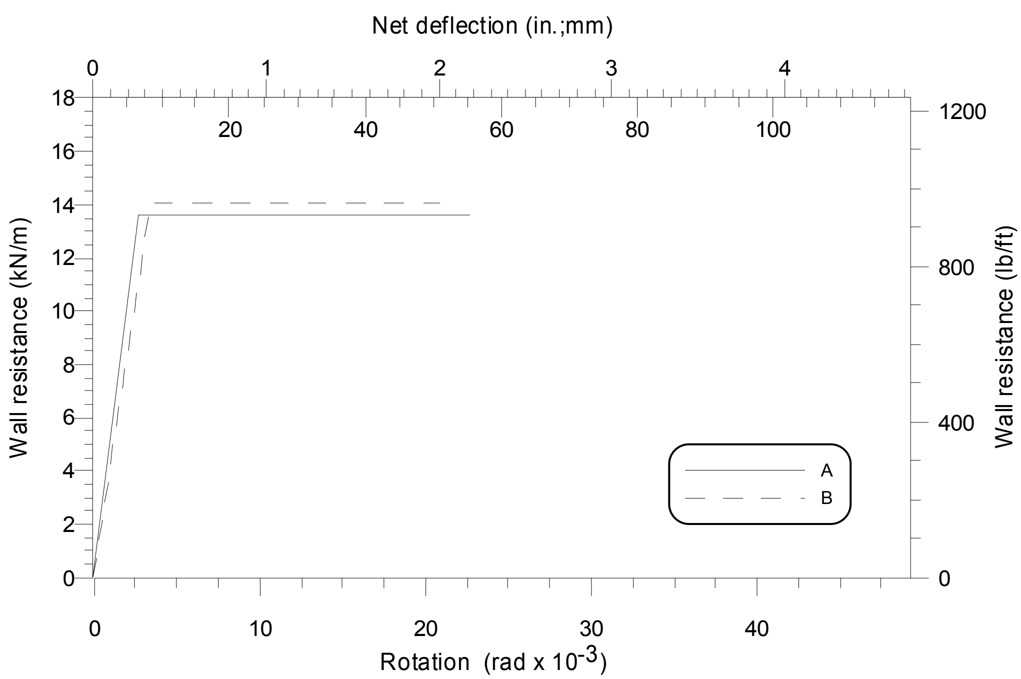


Figure C.27 Comparison of EEEP Results for Tests 11M-a,b

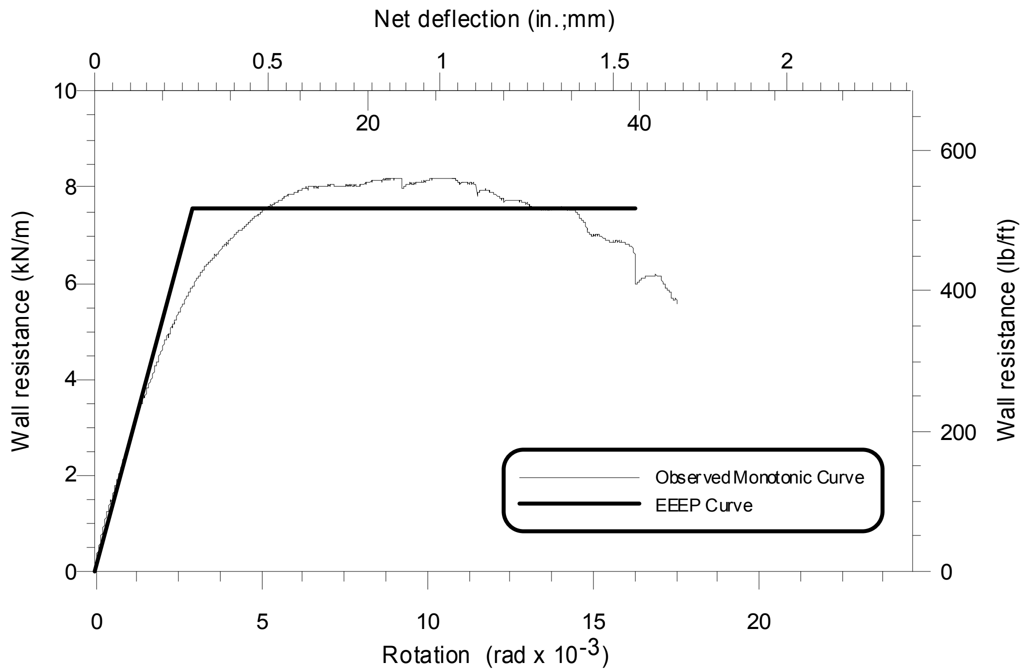


Figure C.28 Observation and EEEP Curves for Test 17M-a

Table C.16 Results for Test 17M-a

Parameters		**Units**
$\mathbf{F_u}$	10.00	kN
$\mathbf{F_{0.8u}}$	8.00	kN
$\mathbf{F_{0.4u}}$	4.00	kN
$\mathbf{F_y}$	9.20	kN
$\mathbf{K_e}$	1.28	kN/mm
Ductility (μ)	5.51	-
$\mathbf{\Delta_{net,y}}$	7.20	mm
$\mathbf{\Delta_{net,u}}$	25.34	mm
$\mathbf{\Delta_{net,0.8u}}$	39.69	mm
$\mathbf{\Delta_{net,0.4u}}$	3.13	mm
Energy	332.02	J
$\mathbf{R_d}$	3.17	-
$\mathbf{S_y}$	7.55	kN/m

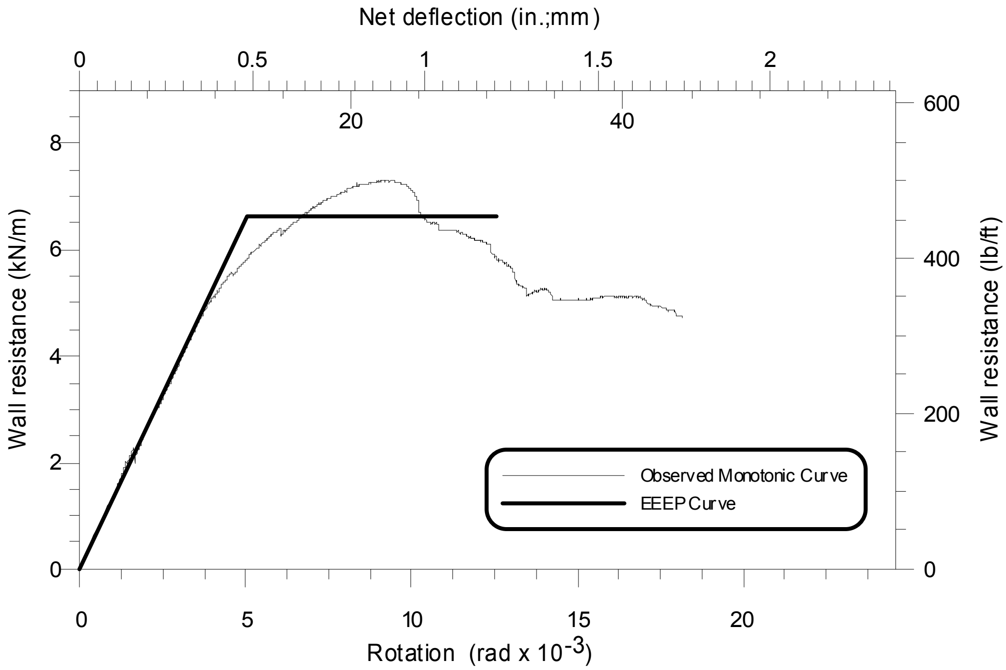


Figure C.29 Observation and EEEP Curves for Test 17M-b

Table C.17 Results for Test 17M-b

Parameters		**Units**
F_u	8.90	kN
$F_{0.8u}$	7.12	kN
$F_{0.4u}$	3.56	kN
F_y	8.06	kN
K_e	0.65	kN/mm
Ductility (μ)	2.48	-
$\Delta_{net,y}$	12.38	mm
$\Delta_{net,u}$	22.49	mm
$\Delta_{net,0.8u}$	30.65	mm
$\Delta_{net,0.4u}$	5.47	mm
Energy	197.13	J
R_d	1.99	-
S_y	6.61	kN/m

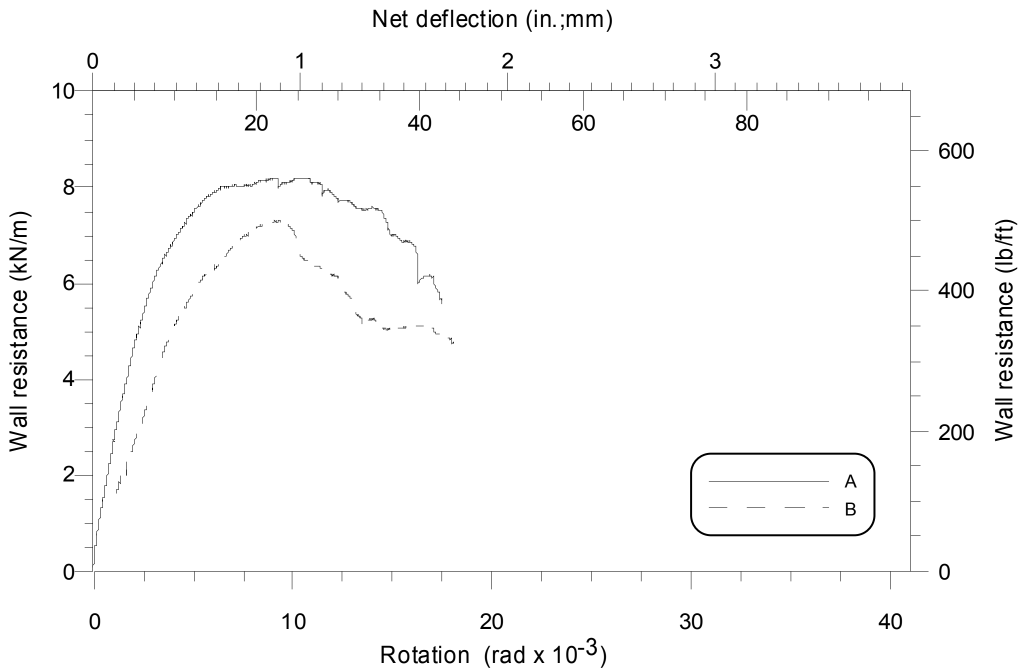


Figure C.30 Comparison of Test Results for Tests 17M-a,b

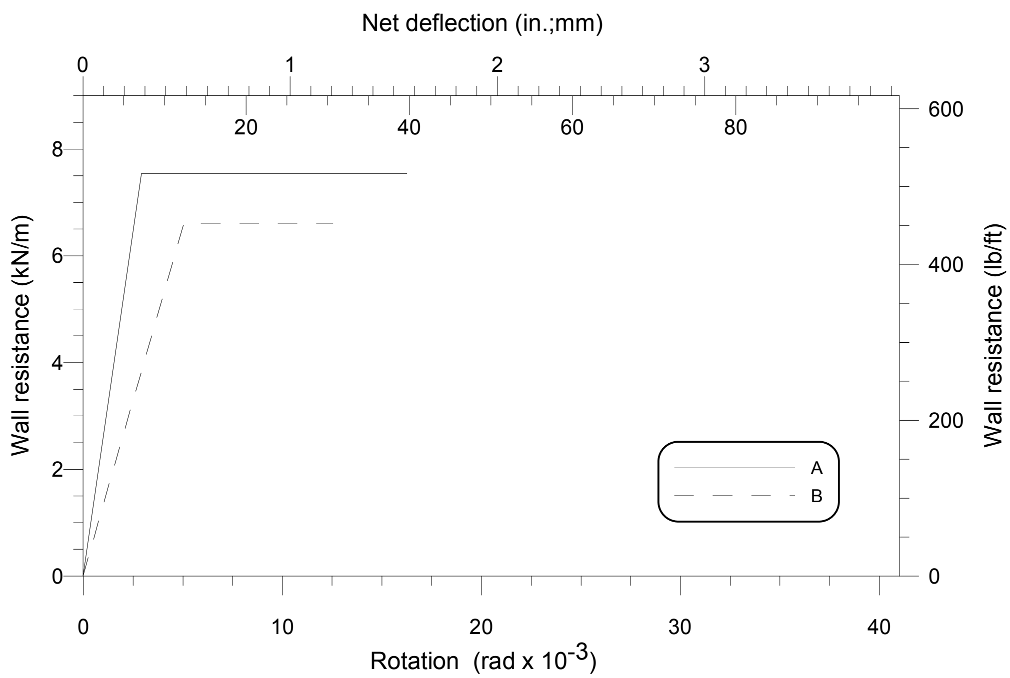


Figure C.31 Comparison of EEEP Results for Tests 17M-a,b

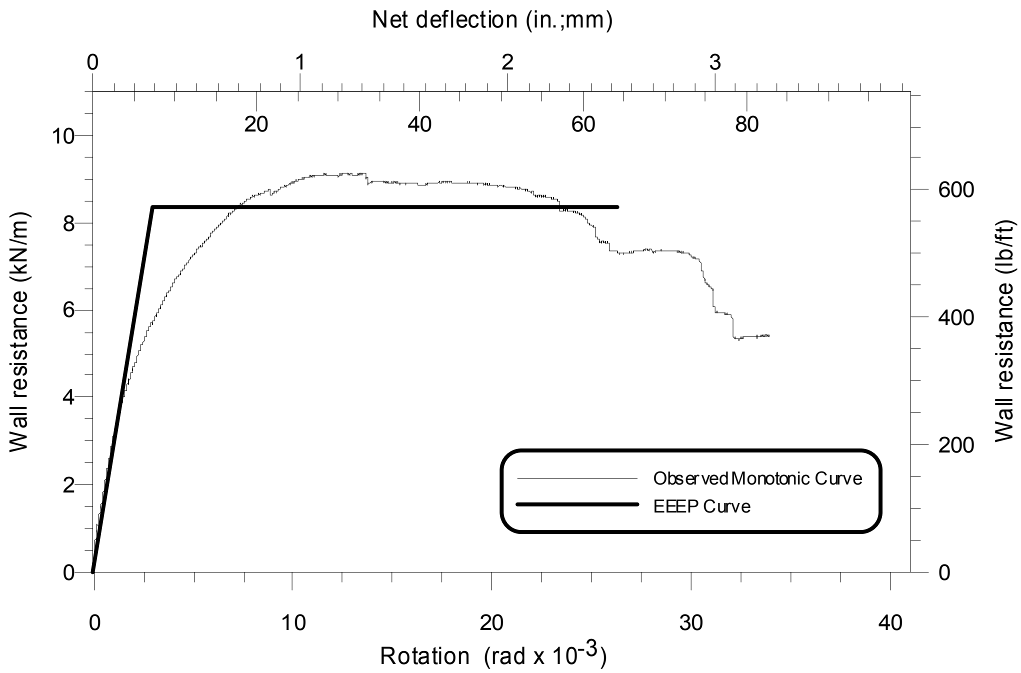


Figure C.32 Observation and EEEP Curves for Test 18M-a

Table C.0.18 Results for Test 18M-a

Parameters		**Units**
$\mathbf{F_u}$	11.16	kN
$\mathbf{F_{0.8u}}$	8.92	kN
$\mathbf{F_{0.4u}}$	4.46	kN
$\mathbf{F_y}$	10.22	kN
$\mathbf{K_e}$	1.40	kN/mm
Ductility (μ)	8.82	-
$\mathbf{\Delta_{net,y}}$	7.29	mm
$\mathbf{\Delta_{net,u}}$	33.21	mm
$\mathbf{\Delta_{net,0.8u}}$	64.27	mm
$\mathbf{\Delta_{net,0.4u}}$	3.18	mm
Energy	619.82	J
$\mathbf{R_d}$	4.08	-
$\mathbf{S_y}$	8.39	kN/m

Table C.19 Reversed-Cyclic Loading Protocol for Configuration 1

$\Delta=0.6*\Delta_m$	29.14	Screw Pattern: 6"/12"
		Sheathing: 0.018"

Displ.	Actuator Input (mm)	No. Of cycles	
0.050 Δ	1.457	6	Initiation
0.075 Δ	2.185	1	Primary
0.056 Δ	1.639	6	Trailing
0.100 Δ	2.914	1	Primary
0.075 Δ	2.185	6	Trailing
0.200 Δ	5.827	1	Primary
0.150 Δ	4.370	3	Trailing
0.300 Δ	8.741	1	Primary
0.225 Δ	6.556	3	Trailing
0.400 Δ	11.654	1	Primary
0.300 Δ	8.741	2	Trailing
0.700 Δ	20.395	1	Primary
0.525 Δ	15.296	2	Trailing
1.000 Δ	29.136	1	Primary
0.750 Δ	21.852	2	Trailing
1.500 Δ	43.704	1	Primary
1.125 Δ	32.778	2	Trailing
2.000 Δ	58.272	1	Primary
1.500 Δ	43.704	2	Trailing
2.500 Δ	72.840	1	Primary
1.875 Δ	54.630	2	Trailing
3.000 Δ	87.408	1	Primary
2.250 Δ	65.556	2	Trailing
3.500 Δ	100.000	1	Primary
2.625 Δ	75.000	2	Trailing

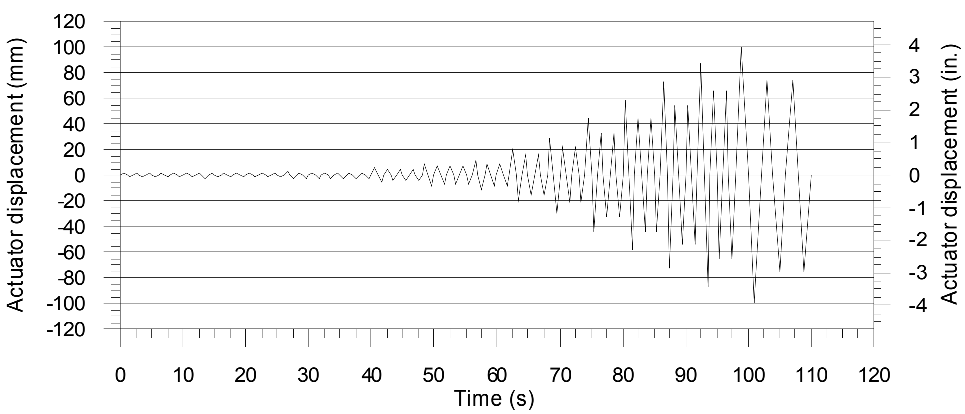


Figure C.33 Displacement Time History for Configuration 1

Table C.20 Reversed-Cyclic Loading Protocol for Configuration 2

$\Delta=0.6*\Delta_m$	57.12	Screw Pattern:	2"/12"
		Sheathing:	0.018"

Displ.	Actuator Input (mm)	No. Of cycles	
0.050 Δ	2.856	6	Initiation
0.075 Δ	4.284	1	Primary
0.056 Δ	3.213	6	Trailing
0.100 Δ	5.712	1	Primary
0.075 Δ	4.284	6	Trailing
0.200 Δ	11.424	1	Primary
0.150 Δ	8.568	3	Trailing
0.300 Δ	17.136	1	Primary
0.225 Δ	12.852	3	Trailing
0.400 Δ	22.848	1	Primary
0.300 Δ	17.136	2	Trailing
0.700 Δ	39.984	1	Primary
0.525 Δ	29.988	2	Trailing
1.000 Δ	57.120	1	Primary
0.750 Δ	42.840	2	Trailing
1.500 Δ	85.680	1	Primary
1.125 Δ	64.260	2	Trailing
2.000 Δ	100.000	1	Primary
1.500 Δ	75.000	2	Trailing

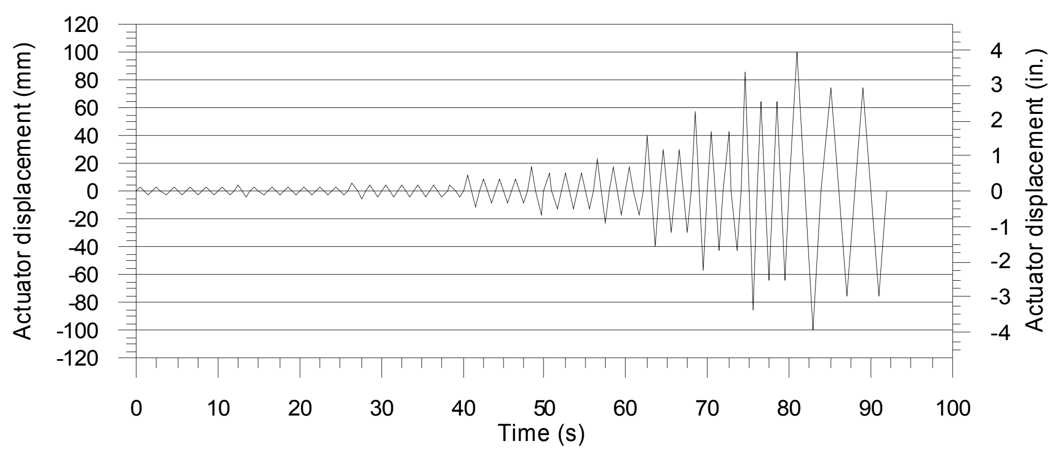


Figure C.34 Displacement Time History for Configuration 2

Table C.21 Reversed-Cyclic Loading Protocol for Configuration 3

Δ=0.6*Δ_m	35.33	Screw Pattern:	6"/12"
		Sheathing:	0.018"

Displ.	Actuator Input (mm)	No. Of cycles	
0.050 Δ	1.766	6	Initiation
0.075 Δ	2.649	1	Primary
0.056 Δ	1.987	6	Trailing
0.100 Δ	3.533	1	Primary
0.075 Δ	2.649	6	Trailing
0.200 Δ	7.065	1	Primary
0.150 Δ	5.299	3	Trailing
0.300 Δ	10.598	1	Primary
0.225 Δ	7.948	3	Trailing
0.400 Δ	14.130	1	Primary
0.300 Δ	10.598	2	Trailing
0.700 Δ	24.728	1	Primary
0.525 Δ	18.546	2	Trailing
1.000 Δ	35.325	1	Primary
0.750 Δ	26.494	2	Trailing
1.500 Δ	52.988	1	Primary
1.125 Δ	39.741	2	Trailing
2.000 Δ	70.650	1	Primary
1.500 Δ	52.988	2	Trailing
2.500 Δ	88.313	1	Primary
1.875 Δ	66.234	2	Trailing
3.000 Δ	100.000	1	Primary
2.250 Δ	75.000	2	Trailing

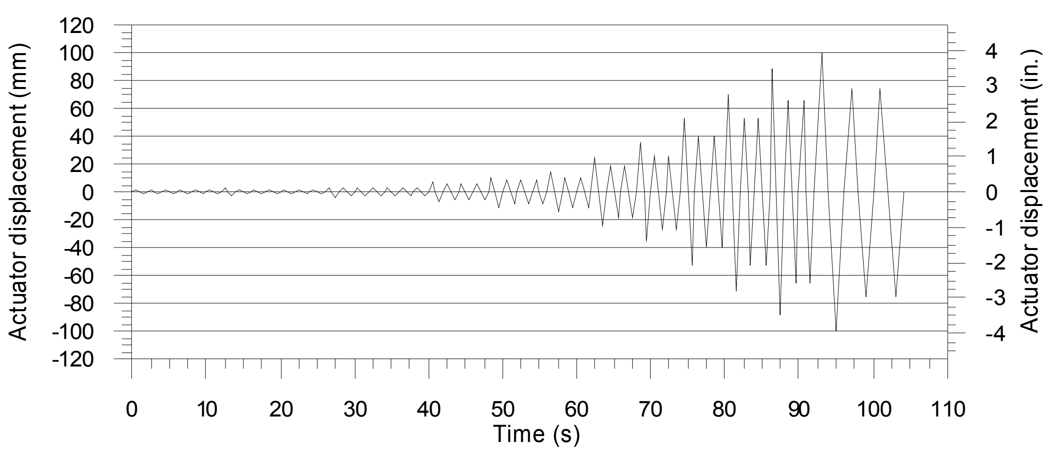


Figure C.35 Displacement Time History for Configuration 3

Table C.22 Reversed-Cyclic Loading Protocol for Configuration 8

$\Delta=0.6*\Delta_m$	60.00	Screw Pattern:	4"/12"
		Sheathing:	0.027"

Displ.	Actuator Input (mm)	No. Of cycles	
0.050 Δ	3.000	6	Initiation
0.075 Δ	4.500	1	Primary
0.056 Δ	3.375	6	Trailing
0.100 Δ	6.000	1	Primary
0.075 Δ	4.500	6	Trailing
0.200 Δ	12.000	1	Primary
0.150 Δ	9.000	3	Trailing
0.300 Δ	18.000	1	Primary
0.225 Δ	13.500	3	Trailing
0.400 Δ	24.000	1	Primary
0.300 Δ	18.000	2	Trailing
0.700 Δ	42.000	1	Primary
0.525 Δ	31.500	2	Trailing
1.000 Δ	60.000	1	Primary
0.750 Δ	45.000	2	Trailing
1.500 Δ	90.000	1	Primary
1.125 Δ	67.500	2	Trailing
2.000 Δ	100.000	1	Primary
1.500 Δ	75.000	2	Trailing

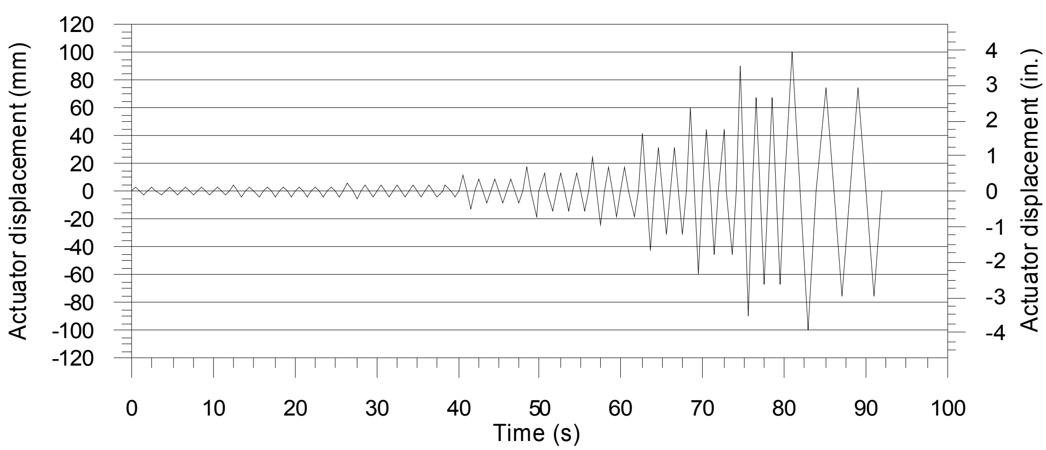


Figure C.36 Displacement Time History for Configuration 8

Table C.23 Reversed-Cyclic Loading Protocol for Configuration 9

Δ=0.6*Δ_m	47.29	Screw Pattern:	2"/12"
		Sheathing:	0.027"

Displ.	Actuator Input (mm)	No. Of cycles	
0.050 Δ	2.364	6	Initiation
0.075 Δ	3.546	1	Primary
0.056 Δ	2.660	6	Trailing
0.100 Δ	4.729	1	Primary
0.075 Δ	3.546	6	Trailing
0.200 Δ	9.457	1	Primary
0.150 Δ	7.093	3	Trailing
0.300 Δ	14.186	1	Primary
0.225 Δ	10.639	3	Trailing
0.400 Δ	18.914	1	Primary
0.300 Δ	14.186	2	Trailing
0.700 Δ	33.100	1	Primary
0.525 Δ	24.825	2	Trailing
1.000 Δ	47.286	1	Primary
0.750 Δ	35.465	2	Trailing
1.500 Δ	70.929	1	Primary
1.125 Δ	53.197	2	Trailing
2.000 Δ	94.572	1	Primary
1.500 Δ	70.929	2	Trailing
2.500 Δ	100.000	1	Trailing
1.875 Δ	75.000	2	Primary

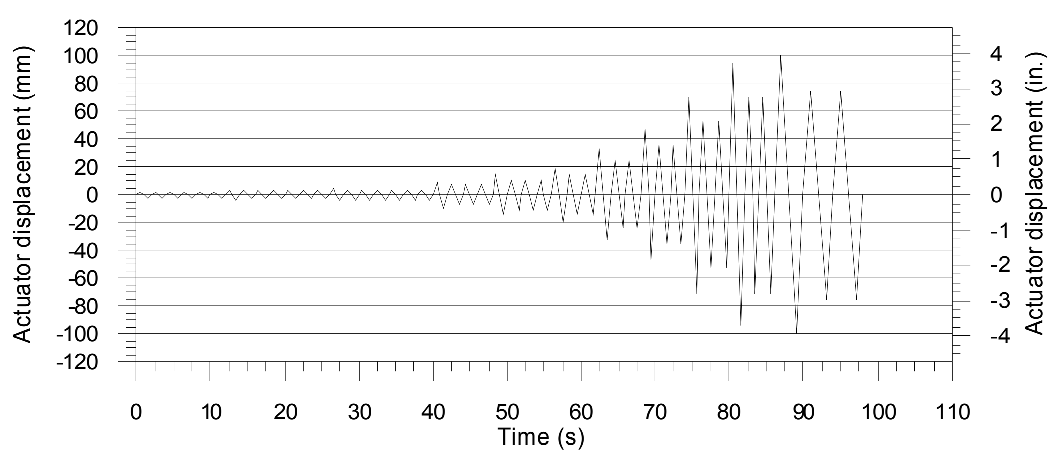


Figure C.37 Displacement Time History for Configuration 9

Table C.24 Reversed-Cyclic Loading Protocol for Configuration 11

$\Delta=0.6*\Delta_m$	31.94	Screw Pattern: 4"/12"
		Sheathing: 0.027"

Displ.	Actuator Input (mm)	No. Of cycles	
0.050 Δ	1.597	6	Initiation
0.075 Δ	2.396	1	Primary
0.056 Δ	1.797	6	Trailing
0.100 Δ	3.194	1	Primary
0.075 Δ	2.396	6	Trailing
0.200 Δ	6.388	1	Primary
0.150 Δ	4.791	3	Trailing
0.300 Δ	9.582	1	Primary
0.225 Δ	7.187	3	Trailing
0.400 Δ	12.776	1	Primary
0.300 Δ	9.582	2	Trailing
0.700 Δ	22.359	1	Primary
0.525 Δ	16.769	2	Trailing
1.000 Δ	31.941	1	Primary
0.750 Δ	23.956	2	Trailing
1.500 Δ	47.912	1	Primary
1.125 Δ	35.934	2	Trailing
2.000 Δ	63.882	1	Primary
1.500 Δ	47.912	2	Trailing
2.500 Δ	79.853	1	Primary
1.875 Δ	59.889	2	Trailing
3.000 Δ	95.823	1	Primary
2.250 Δ	71.867	2	Trailing
3.500 Δ	100.000	1	Primary
2.625 Δ	75.000	2	Trailing

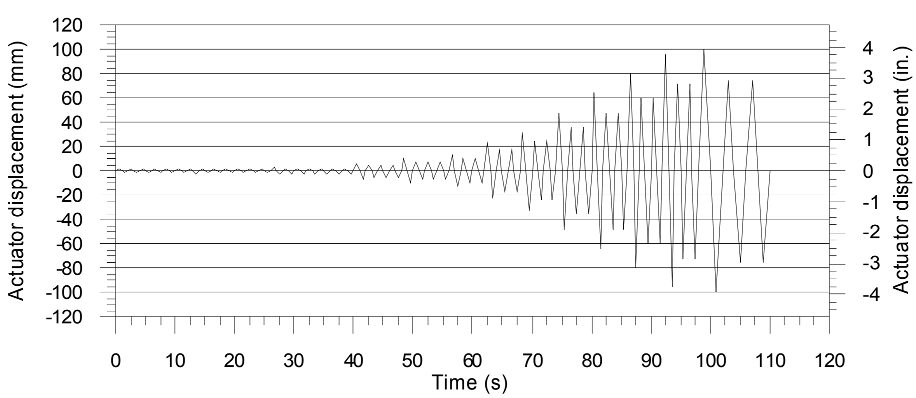


Figure C.38 Displacement Time History for Configuration 11

Table C.25 Results of Test 1C-a

Parameters			Units
	Positive	**Negative**	
F_u	7.43	-7.98	kN
$F_{0.8u}$	5.94	-6.38	kN
$F_{0.4u}$	2.97	-3.19	kN
F_y	6.92	-7.19	kN
K_e	1.10	1.03	kN/mm
Ductility (μ)	8.18	5.75	-
$\Delta_{net,y}$	6.29	-6.99	mm
$\Delta_{net,u}$	34.55	-22.59	mm
$\Delta_{net,0.8u}$	51.40	-40.20	mm
$\Delta_{net,0.4u}$	2.70	-3.10	mm
Energy	333.89	264.01	J
R_d	3.92	3.24	-
S_y	5.68	-5.90	kN/m

Table C.26 Results of Test 1C-b

Parameters			Units
	Positive	**Negative**	
F_u	7.77	-7.45	kN
$F_{0.8u}$	6.22	-5.96	kN
$F_{0.4u}$	3.11	-2.98	kN
F_y	7.02	-6.73	kN
K_e	1.00	1.03	kN/mm
Ductility (μ)	5.74	5.28	-
$\Delta_{net,y}$	7.00	-6.56	mm
$\Delta_{net,u}$	19.34	-19.70	mm
$\Delta_{net,0.8u}$	40.20	-34.60	mm
$\Delta_{net,0.4u}$	3.10	-2.90	mm
Energy	257.56	210.95	J
R_d	3.24	3.09	-
S_y	5.76	-5.52	kN/m

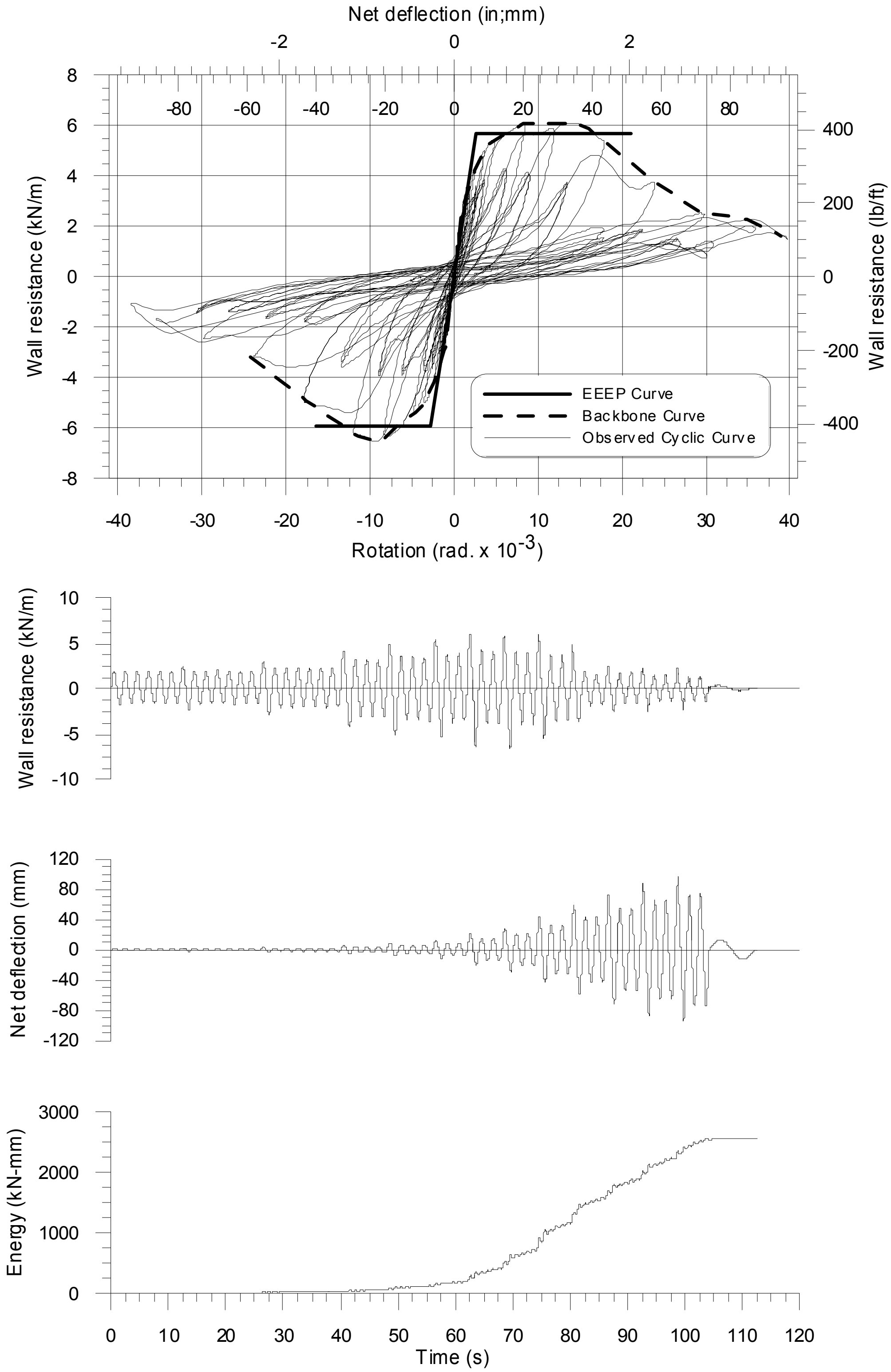


Figure C.39 Observation and EEEP Curves and Time History for Test 1C-a

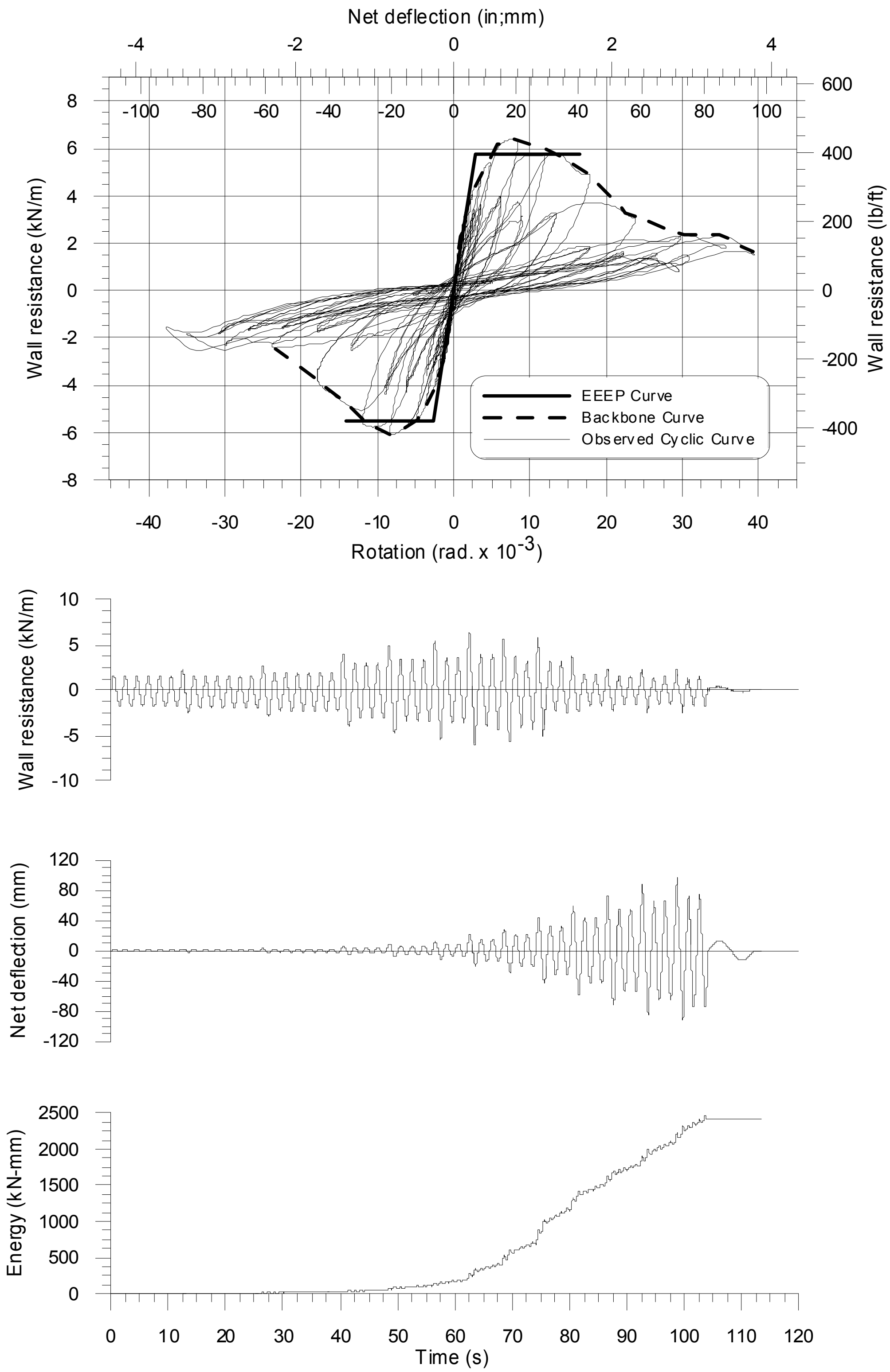


Figure C.40 Observation and EEEP Curves and Time History for Test 1C-b

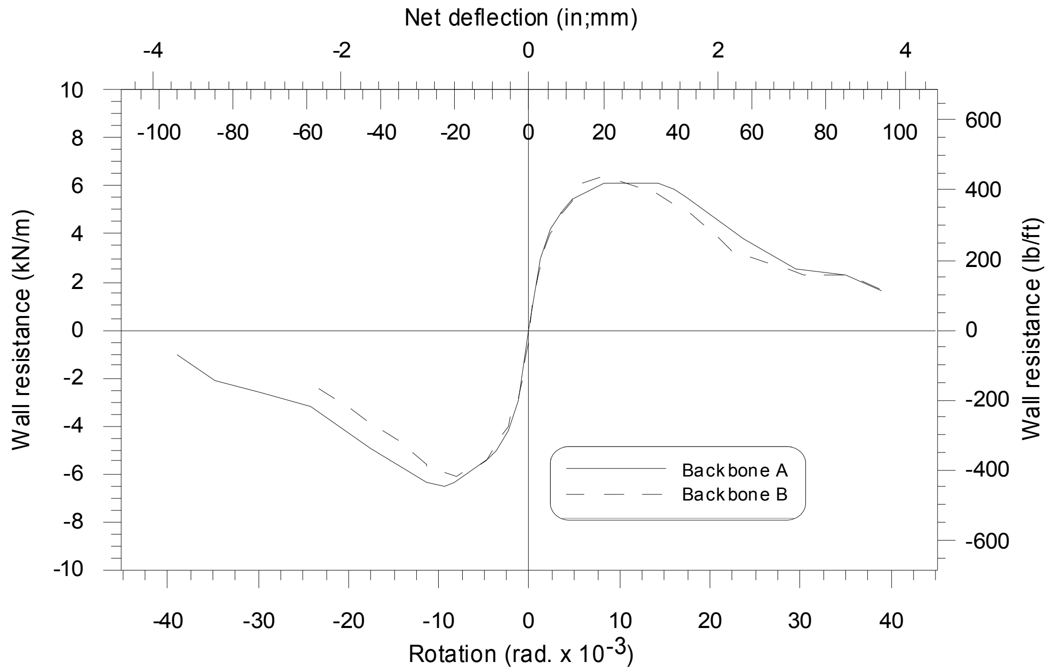


Figure C.41 Comparison of Test Results for Tests 1C-a,b

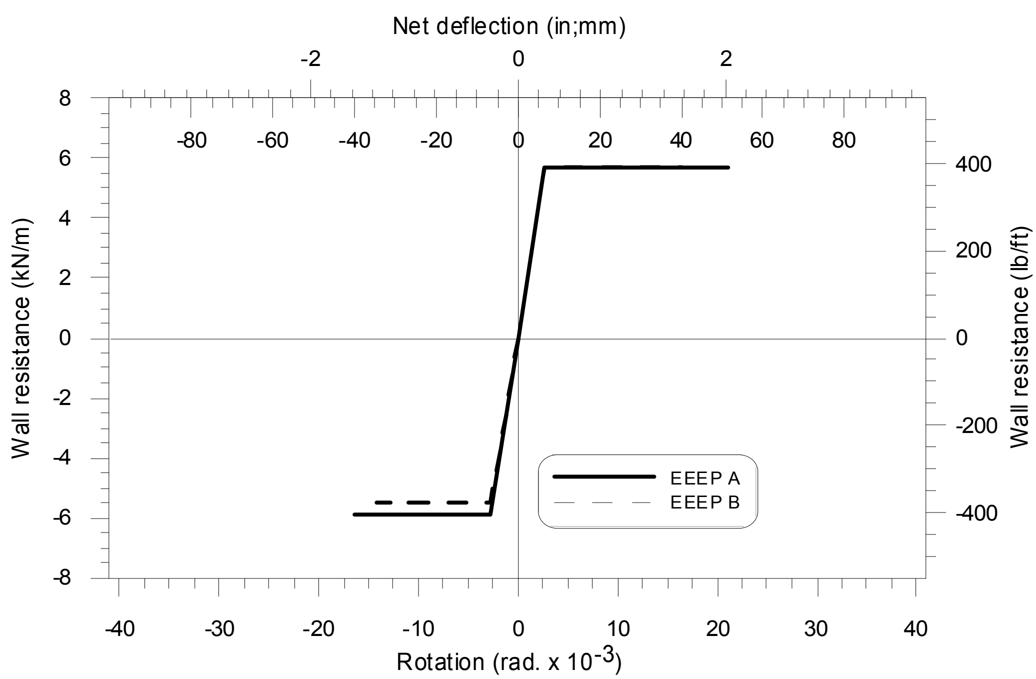


Figure C.42 Comparison of EEEP Results for Tests 1C-a,b

Table C.27 Results of Test 2C-a

Parameters			Units
	Positive	Negative	
F_u	13.54	-13.12	kN
$F_{0.8u}$	10.83	-10.50	kN
$F_{0.4u}$	5.42	-5.25	kN
F_y	12.17	-12.37	kN
K_e	1.23	1.31	kN/mm
Ductility (μ)	8.22	8.99	-
$\Delta_{net,y}$	9.88	-9.43	mm
$\Delta_{net,u}$	29.00	-28.21	mm
$\Delta_{net,0.8u}$	81.20	-84.80	mm
$\Delta_{net,0.4u}$	4.40	-4.00	mm
Energy	927.76	990.89	J
R_d	3.93	4.12	-
S_y	9.98	-10.15	kN/m

Table C.28 Results of Test 2C-b

Parameters			Units
	Positive	Negative	
F_u	13.12	-12.98	kN
$F_{0.8u}$	10.49	-10.39	kN
$F_{0.4u}$	5.25	-5.19	kN
F_y	12.19	-12.22	kN
K_e	1.25	1.37	kN/mm
Ductility (μ)	9.83	9.83	-
$\Delta_{net,y}$	9.76	-8.94	mm
$\Delta_{net,u}$	29.52	-38.57	mm
$\Delta_{net,0.8u}$	95.90	-87.90	mm
$\Delta_{net,0.4u}$	4.20	-3.80	mm
Energy	1109.27	1019.41	J
R_d	4.32	4.32	-
S_y	10.00	-10.02	kN/m

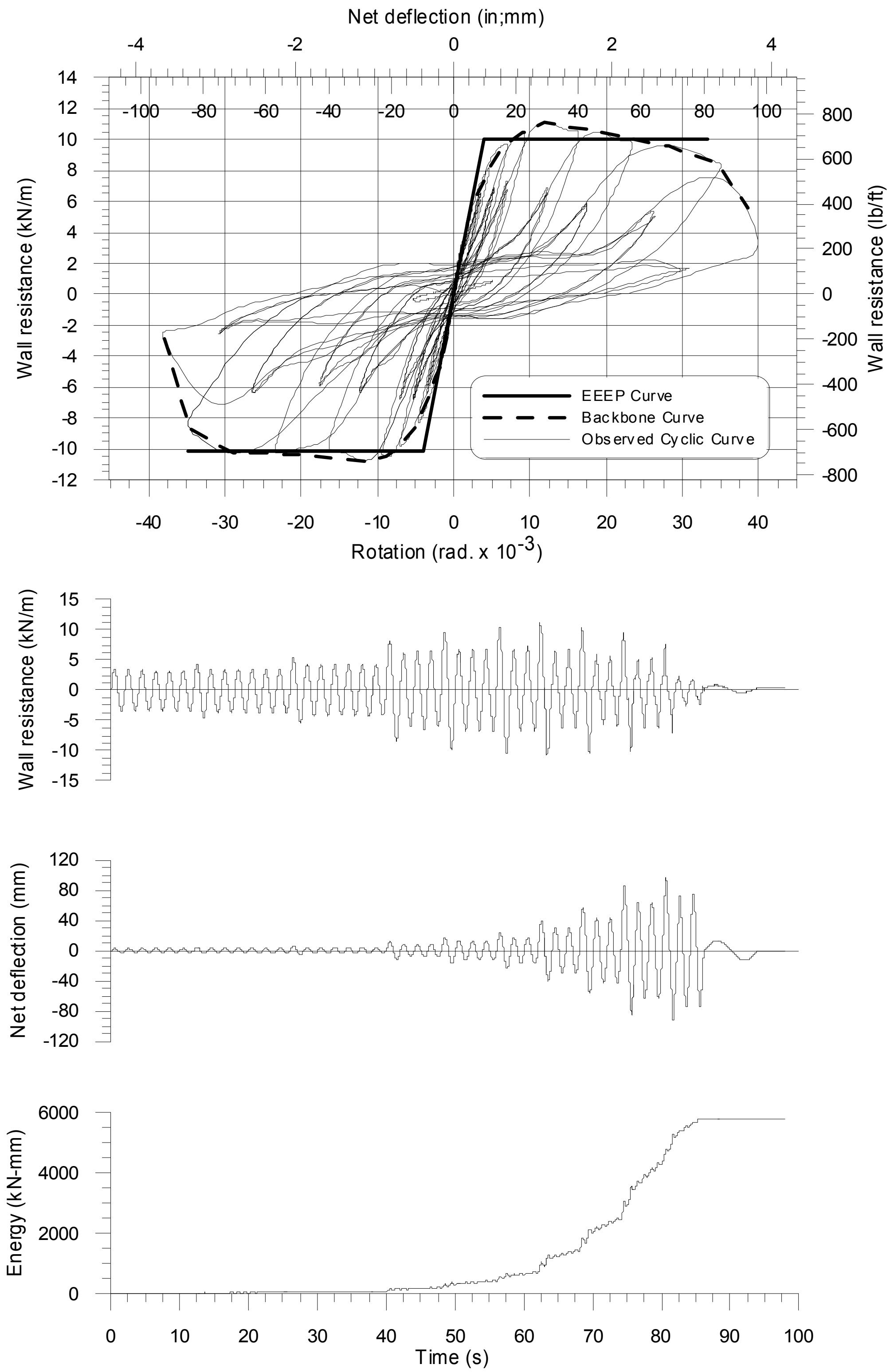


Figure C.0.43 Observation and EEEP Curves and Time History for Test 2C-a

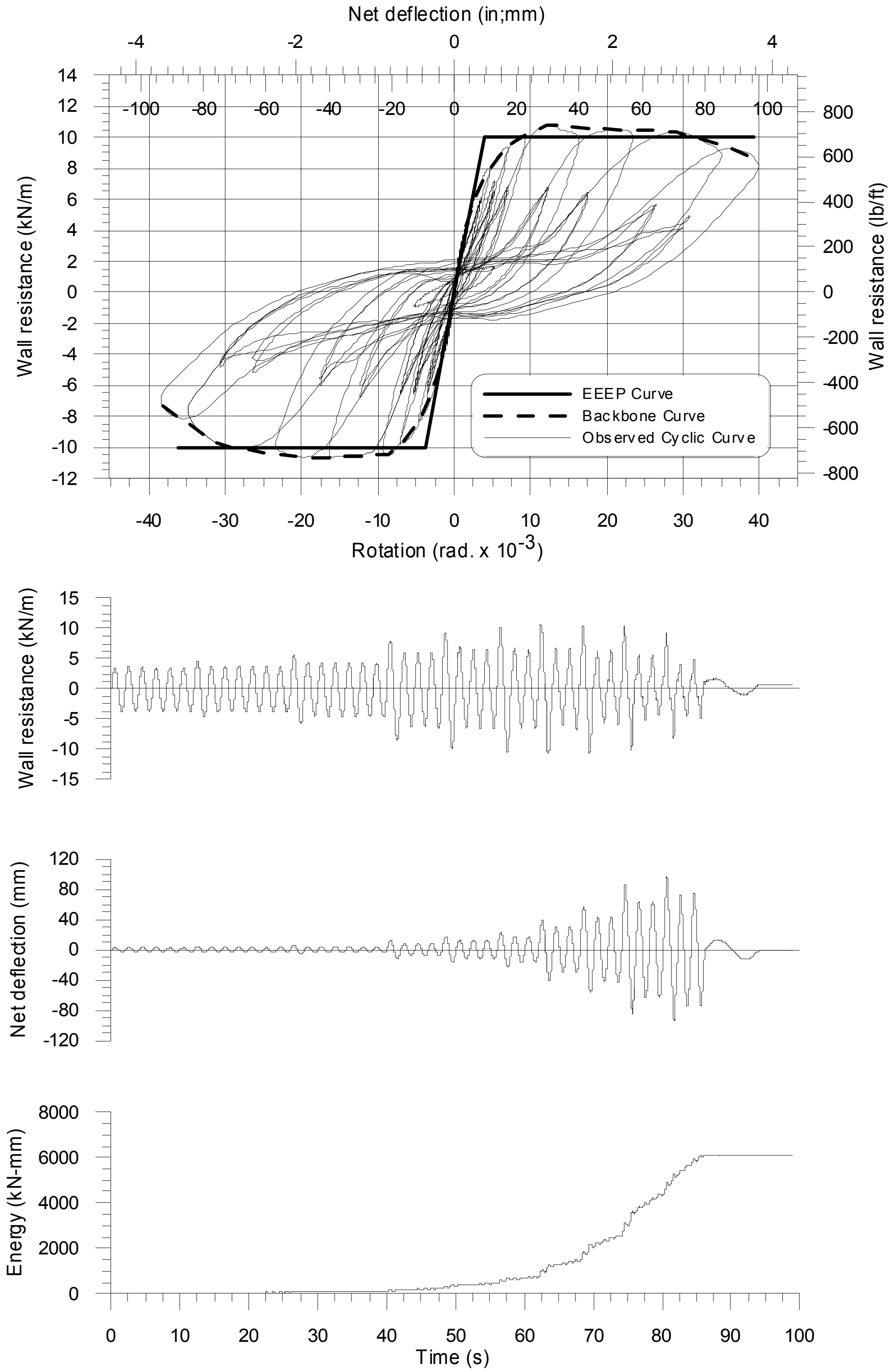


Figure C.44 Observation and EEEP Curves and Time History for Test 2c-b

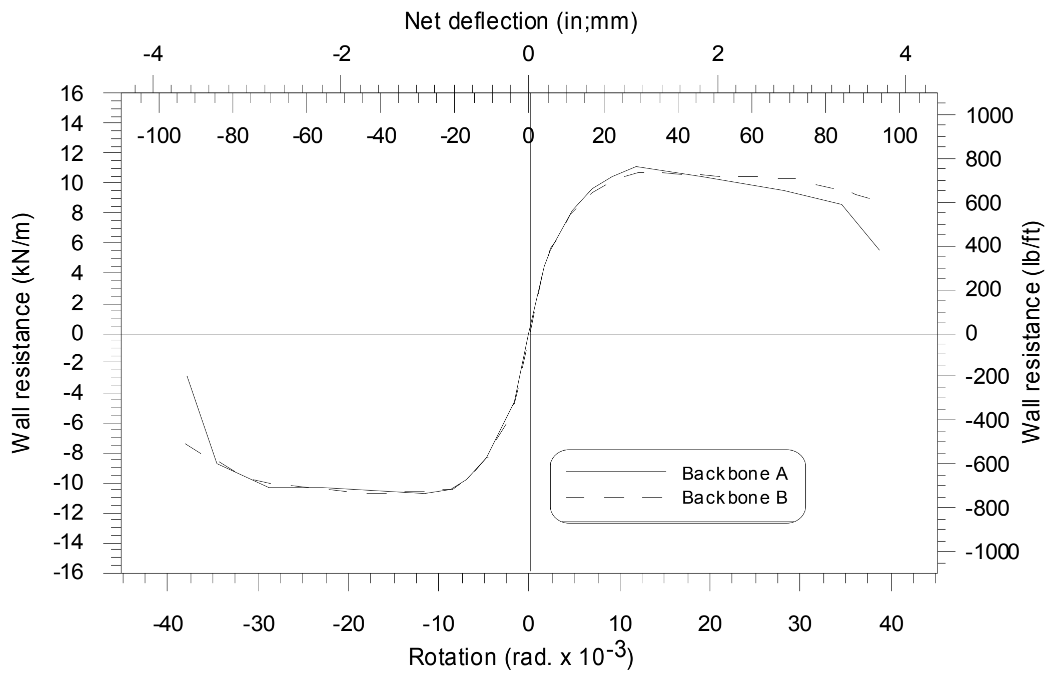


Figure C.45 Comparison of Test Results for Tests 2C-a,b

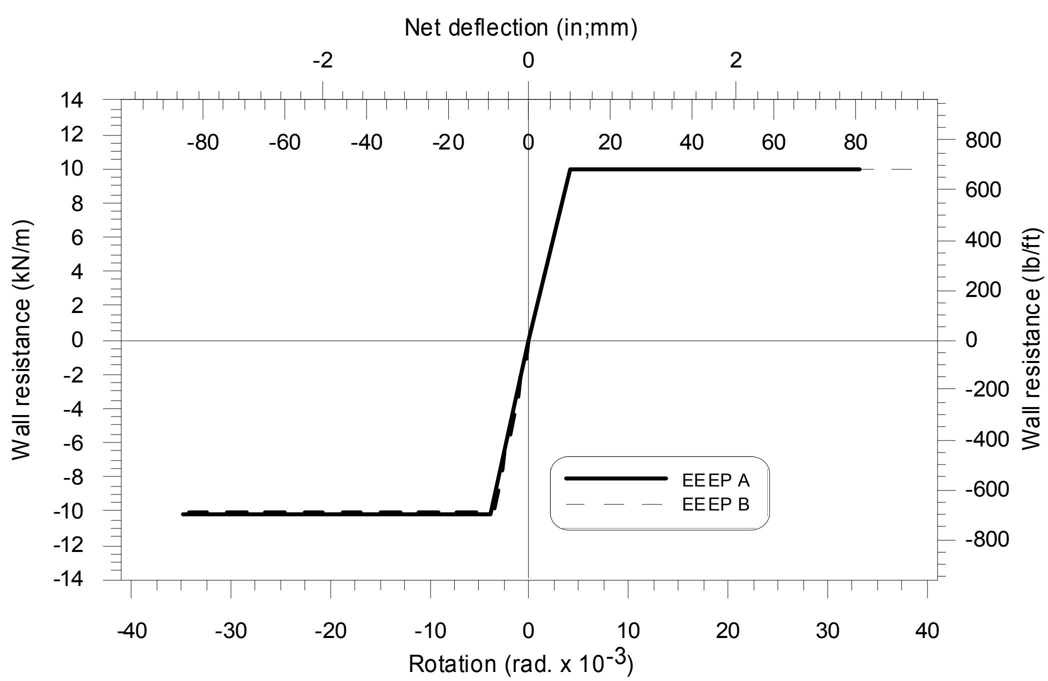


Figure C.46 Comparison of EEEP Results for Tests 2C-a,b

Table C.29 Results of Test 3C-a

Parameters			Units
	Positive	**Negative**	
F_u	7.36	-6.69	kN
$F_{0.8u}$	5.89	-5.35	kN
$F_{0.4u}$	2.94	-2.68	kN
F_y	6.87	-6.24	kN
K_e	0.89	0.86	kN/mm
Ductility (μ)	8.91	7.88	-
$\Delta_{net,y}$	7.70	-7.22	mm
$\Delta_{net,u}$	50.34	-43.71	mm
$\Delta_{net,0.8u}$	68.60	-56.90	mm
$\Delta_{net,0.4u}$	3.30	-3.10	mm
Energy	444.70	332.27	J
R_d	4.10	3.84	-
S_y	5.63	-5.11	kN/m

Table C.30 Results of Test 3C-c

Parameters			Units
	Positive	**Negative**	
F_u	7.21	-7.64	kN
$F_{0.8u}$	5.77	-6.11	kN
$F_{0.4u}$	2.88	-3.06	kN
F_y	6.80	-6.95	kN
K_e	1.11	0.85	kN/mm
Ductility (μ)	9.02	5.41	-
$\Delta_{net,y}$	6.13	-8.19	mm
$\Delta_{net,u}$	28.98	-19.35	mm
$\Delta_{net,0.8u}$	55.30	-44.30	mm
$\Delta_{net,0.4u}$	2.60	-3.60	mm
Energy	355.36	279.38	J
R_d	4.13	3.13	-
S_y	5.58	-5.70	kN/m

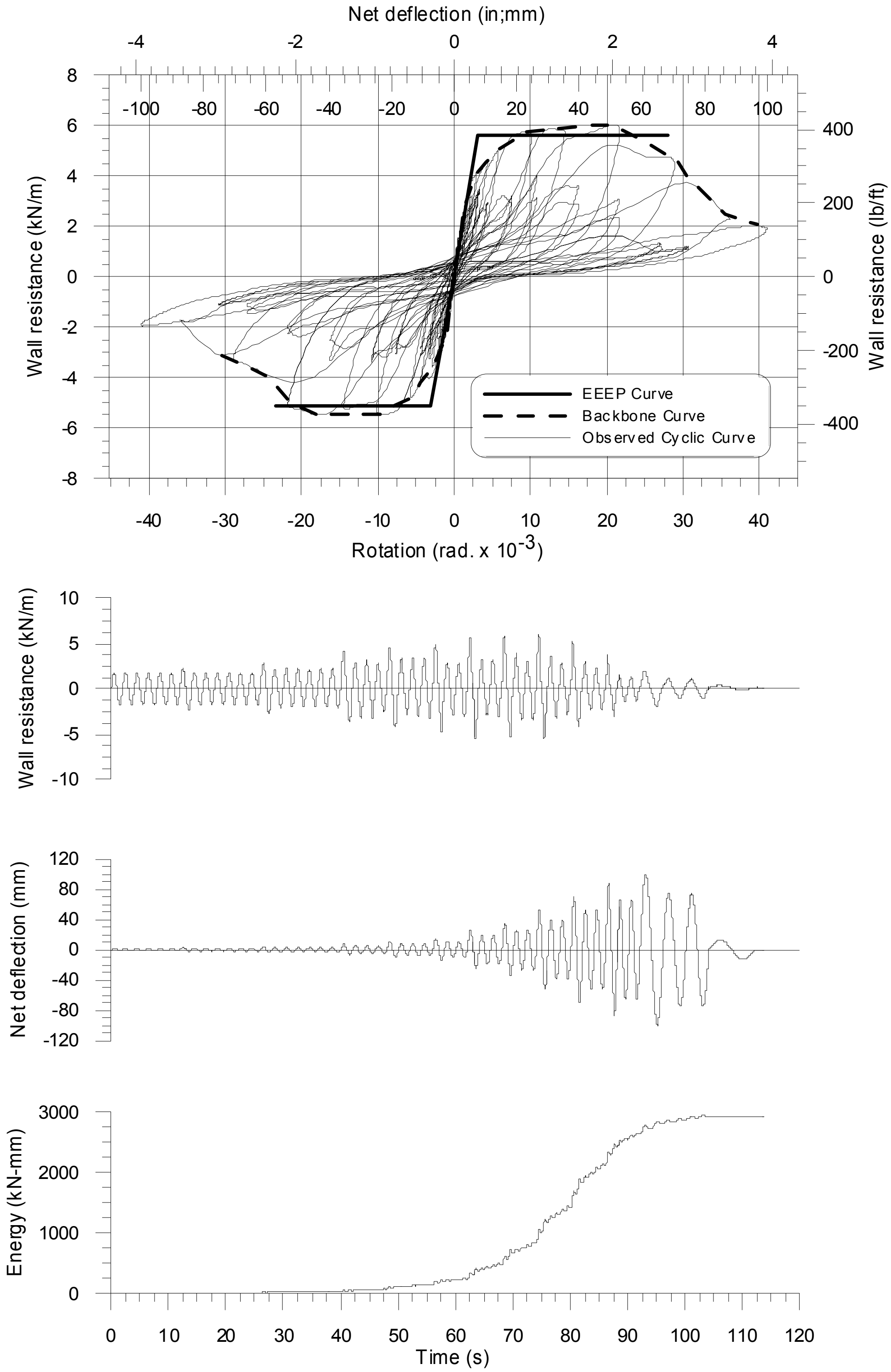


Figure C.47 Observation and EEEP Curves and Time History for Test 3C-a

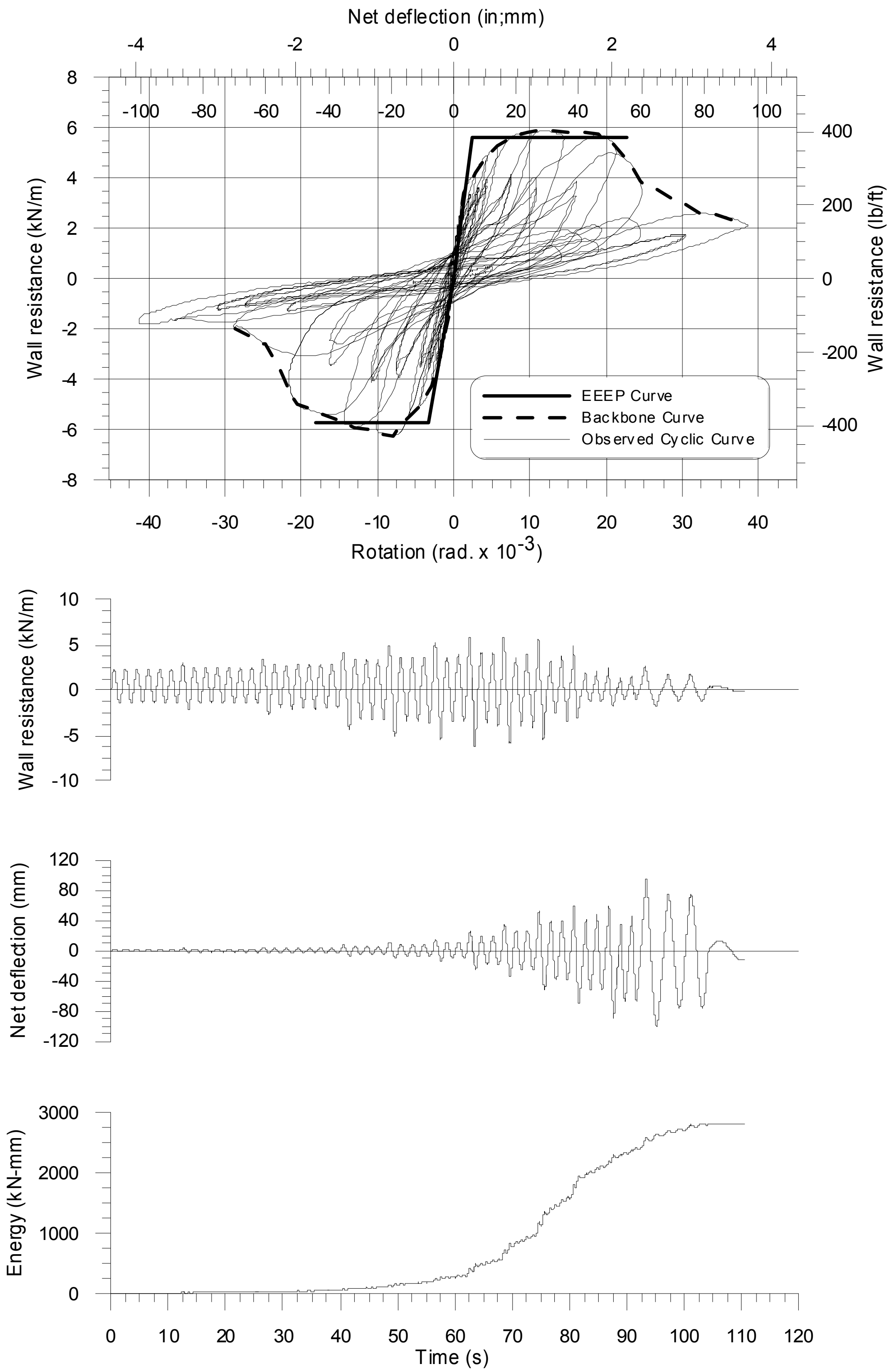


Figure C.48 Observation and EEEP Curves and Time History for Test 3C-c

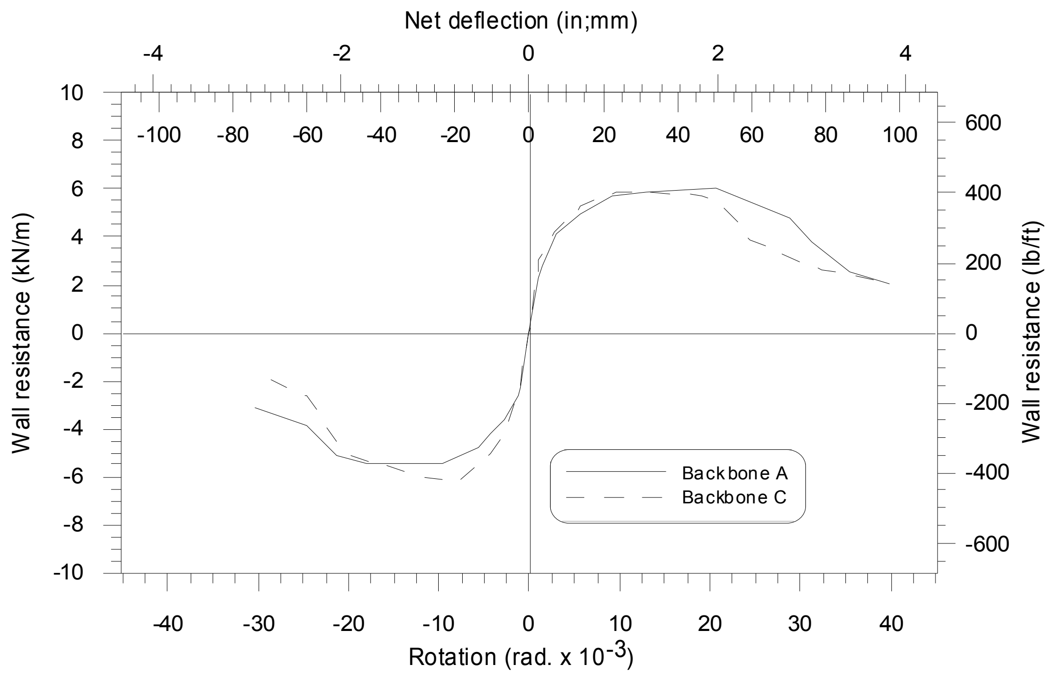


Figure C.49 Comparison of Test Results for Tests 3C-a,c

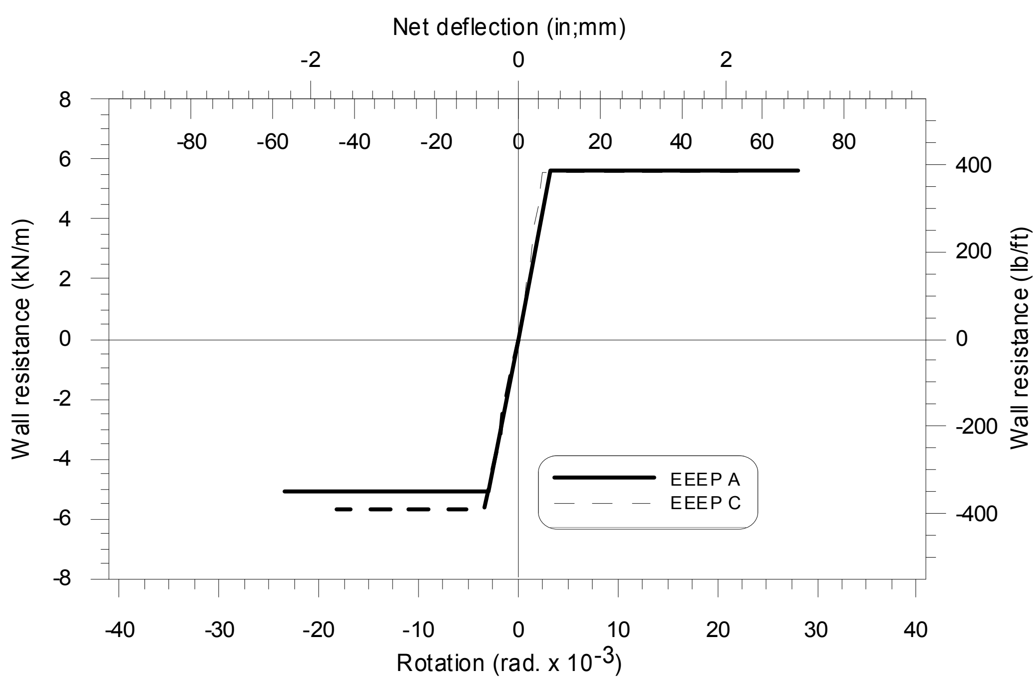


Figure C.50 Comparison of EEEP Results for Tests 3C-a,c

Table C.31 Results of Test 8C-a

Parameters			Units
	Positive	Negative	
F_u	8.40	-8.50	kN
$F_{0.8u}$	6.72	-6.80	kN
$F_{0.4u}$	3.36	-3.40	kN
F_y	7.56	-7.55	kN
K_e	0.56	0.63	kN/mm
Ductility (μ)	6.72	7.33	-
$\Delta_{net,y}$	13.50	-11.99	mm
$\Delta_{net,u}$	76.27	-76.25	mm
$\Delta_{net,0.8u}$	90.70	-87.90	mm
$\Delta_{net,0.4u}$	6.00	-5.40	mm
Energy	634.44	618.59	J
R_d	3.53	3.70	-
S_y	12.40	-12.39	kN/m

Table C.32 Results of Test 8C-b

Parameters			Units
	Positive	Negative	
F_u	8.34	-7.91	kN
$F_{0.8u}$	6.67	-6.33	kN
$F_{0.4u}$	3.34	-3.16	kN
F_y	7.64	-7.33	kN
K_e	0.63	0.52	kN/mm
Ductility (μ)	7.40	7.08	-
$\Delta_{net,y}$	12.15	-14.12	mm
$\Delta_{net,u}$	71.92	-53.63	mm
$\Delta_{net,0.8u}$	89.90	-100.00	mm
$\Delta_{net,0.4u}$	5.30	-6.10	mm
Energy	640.85	680.90	J
R_d	3.72	3.63	-
S_y	12.54	-12.02	kN/m

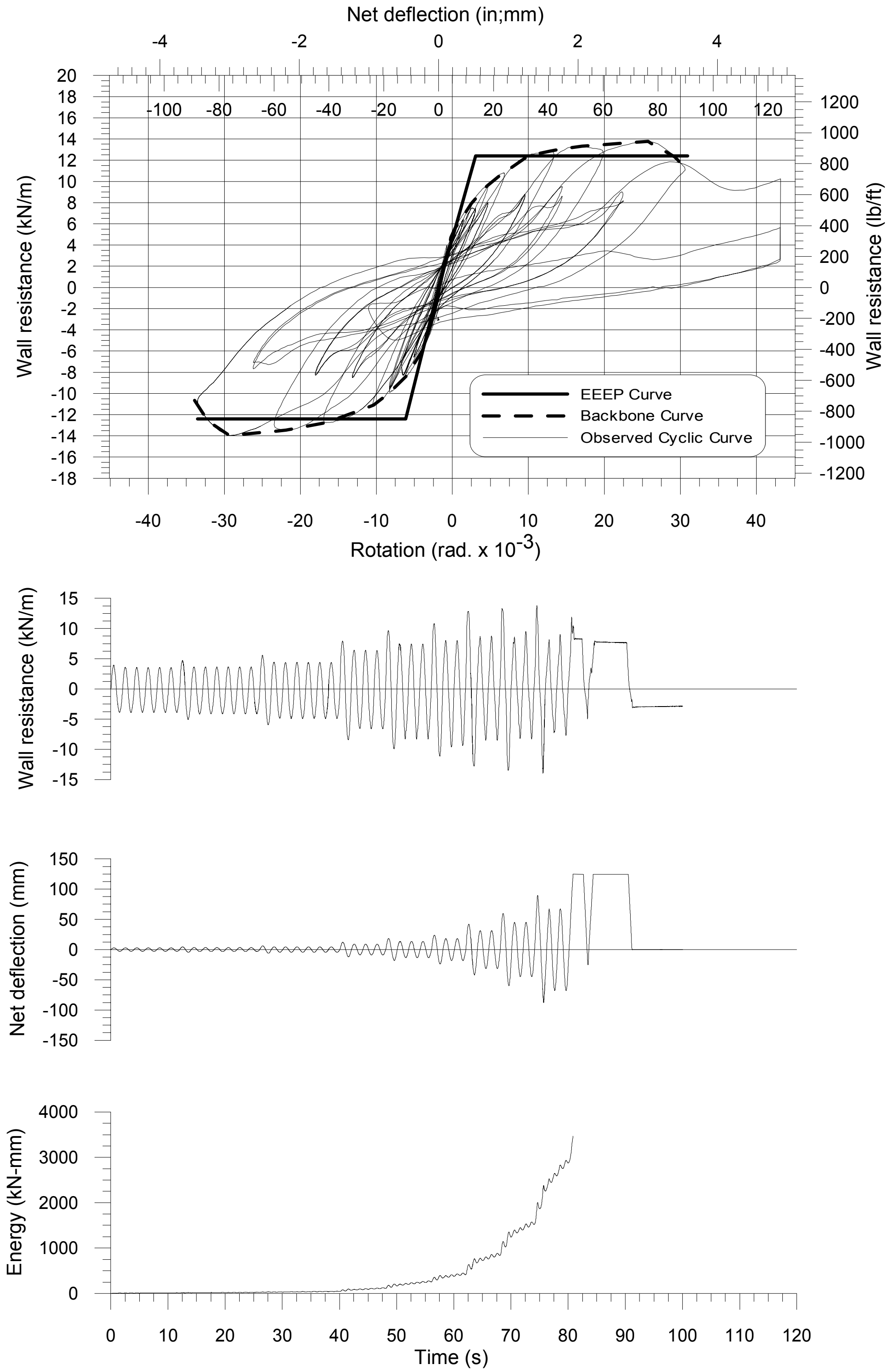


Figure C.51 Observation and EEEP Curves and Time History for Test 8C-a

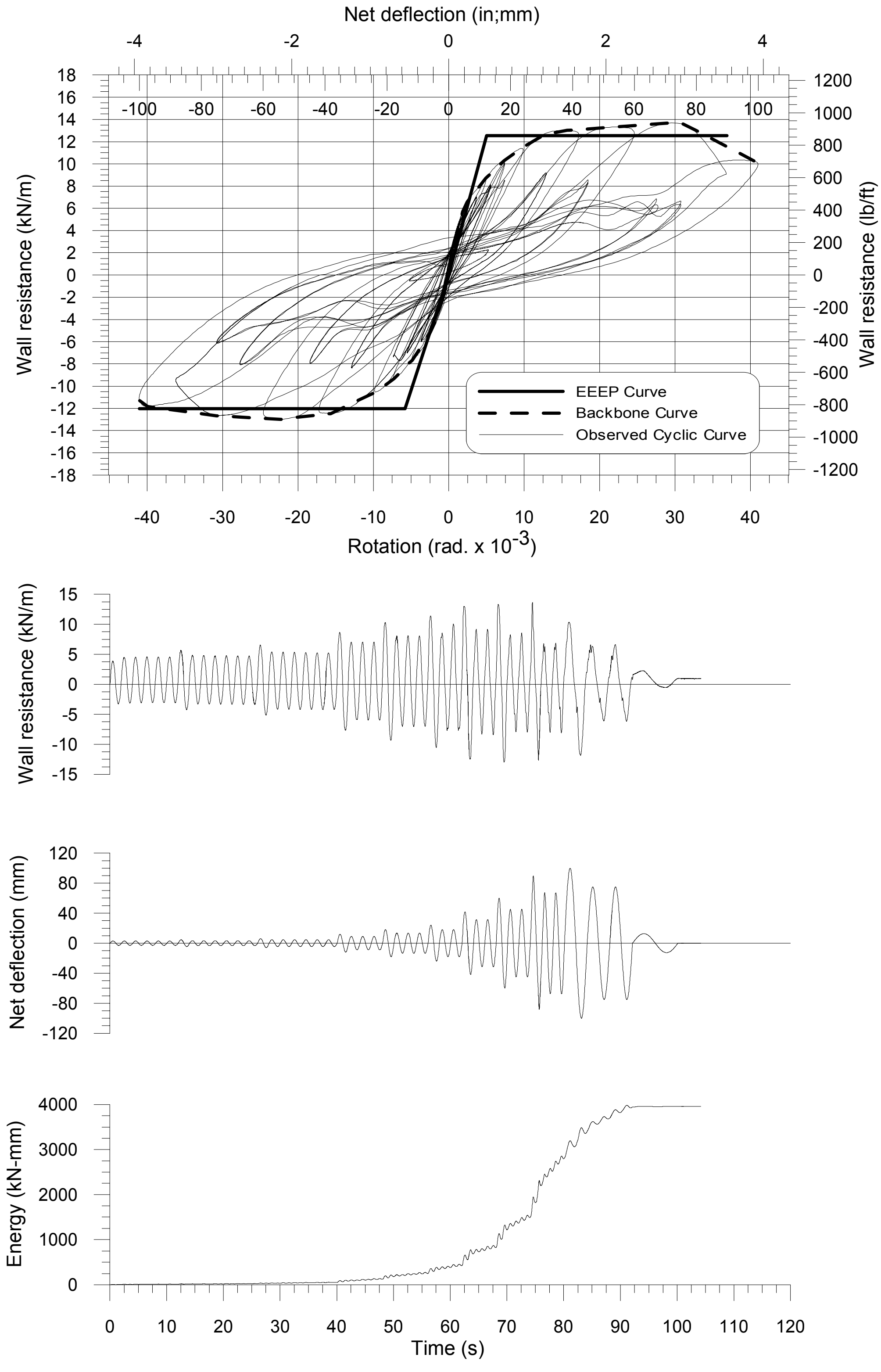


Figure C.52 Observation and EEEP Curves and Time History for Test 8C-b

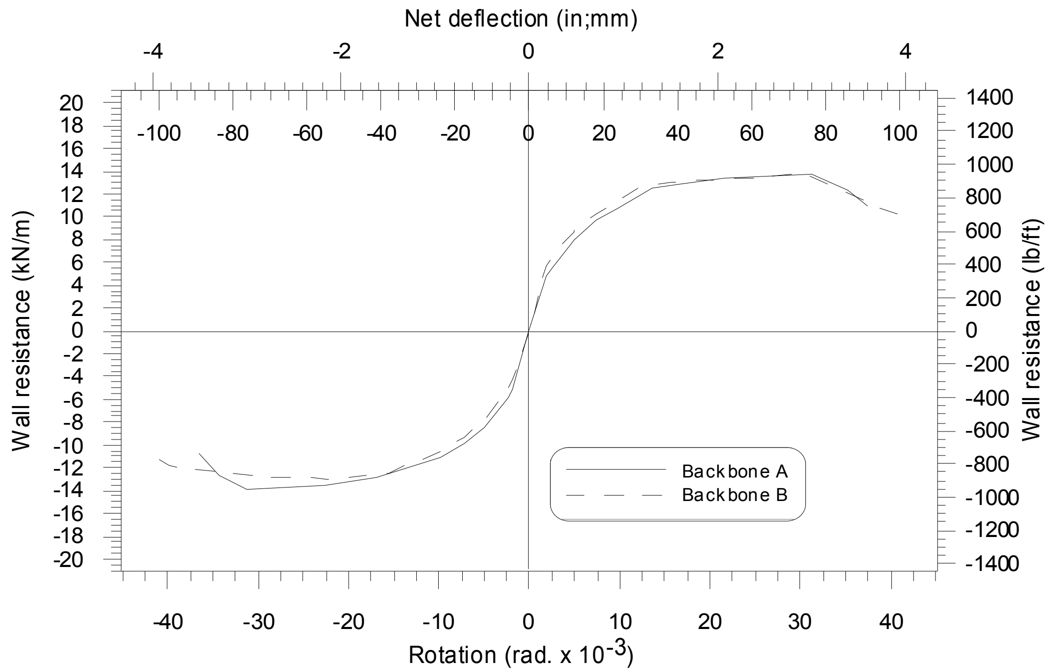


Figure C.53 Comparison of Test Results for Tests 8C-a,b

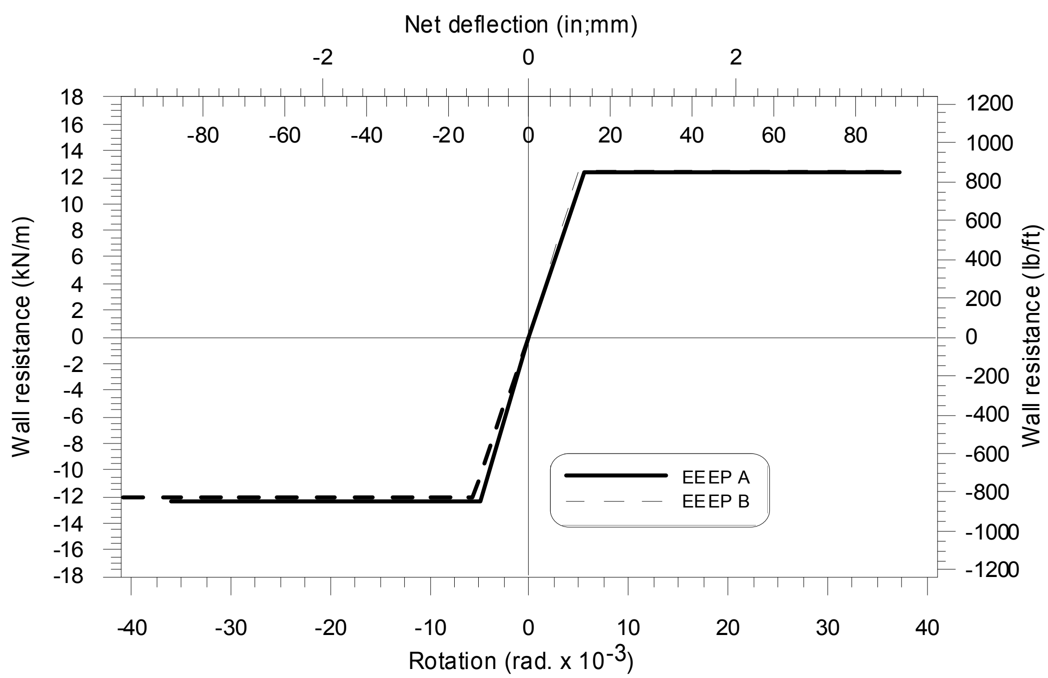


Figure C.54 Comparison of EEEP Results for Tests 8C-a,b

Table C.33 Results of Test 9C-a

Parameters			Units
	Positive	Negative	
F_u	9.86	-9.55	kN
$F_{0.8u}$	7.89	-7.64	kN
$F_{0.4u}$	3.95	-3.82	kN
F_y	9.24	-8.89	kN
K_e	0.49	0.42	kN/mm
Ductility (μ)	5.24	4.67	-
$\Delta_{net,y}$	18.96	-21.41	mm
$\Delta_{net,u}$	55.20	-77.89	mm
$\Delta_{net,0.8u}$	99.40	-100.00	mm
$\Delta_{net,0.4u}$	8.10	-9.20	mm
Energy	830.62	794.04	J
R_d	3.08	2.89	-
S_y	15.15	-14.59	kN/m

Table C.34 Results of Test 9C-b

Parameters			Units
	Positive	Negative	
F_u	9.78	-9.40	kN
$F_{0.8u}$	7.83	-7.52	kN
$F_{0.4u}$	3.91	-3.76	kN
F_y	9.07	-7.82	kN
K_e	0.50	0.64	kN/mm
Ductility (μ)	5.52	8.14	-
$\Delta_{net,y}$	18.08	-12.28	mm
$\Delta_{net,u}$	57.03	-55.31	mm
$\Delta_{net,0.8u}$	99.90	-100.00	mm
$\Delta_{net,0.4u}$	7.80	-5.90	mm
Energy	824.40	734.44	J
R_d	3.17	3.91	-
S_y	14.88	-12.84	kN/m

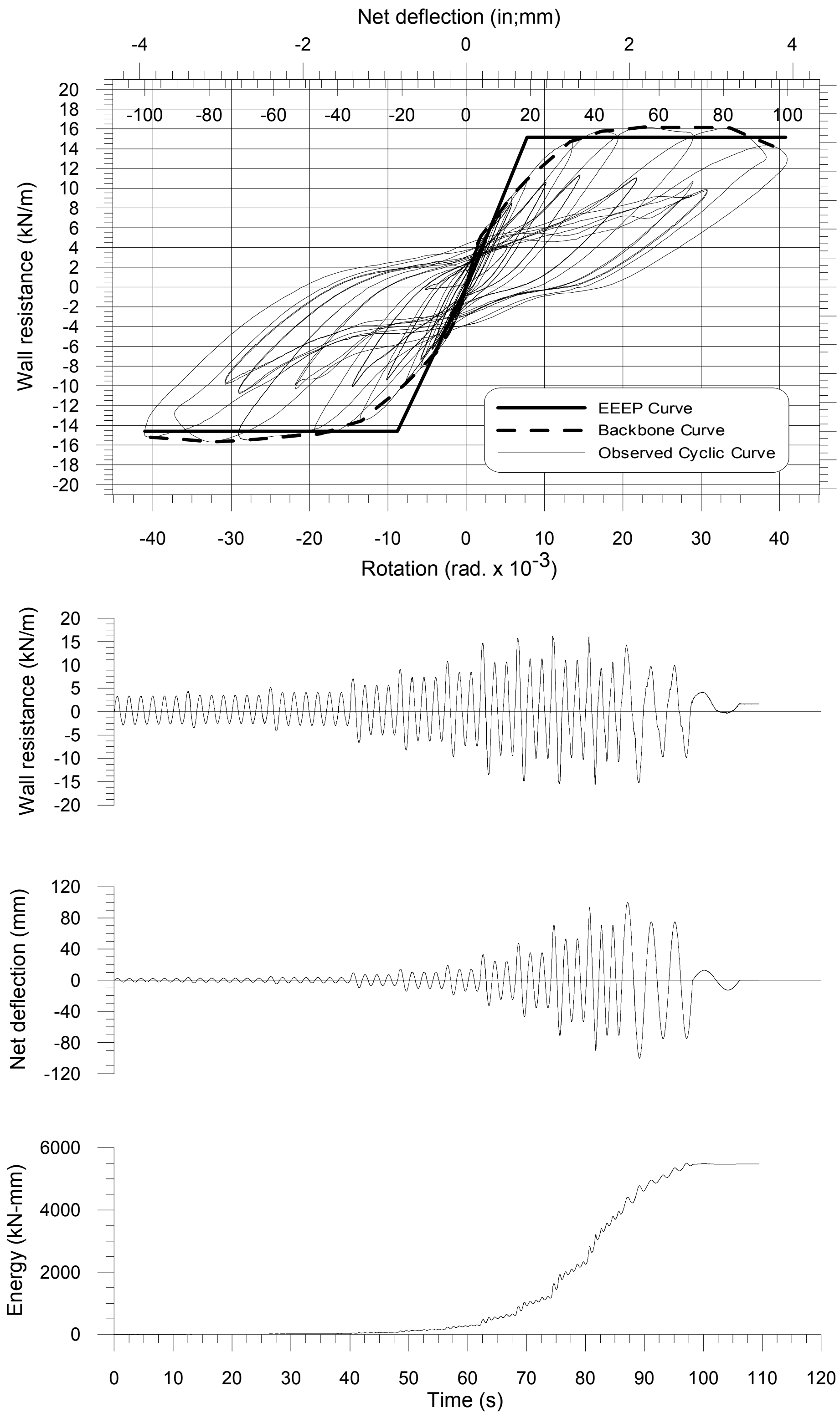


Figure C.55 Observation and EEEP Curves and Time History for Test 9C-a

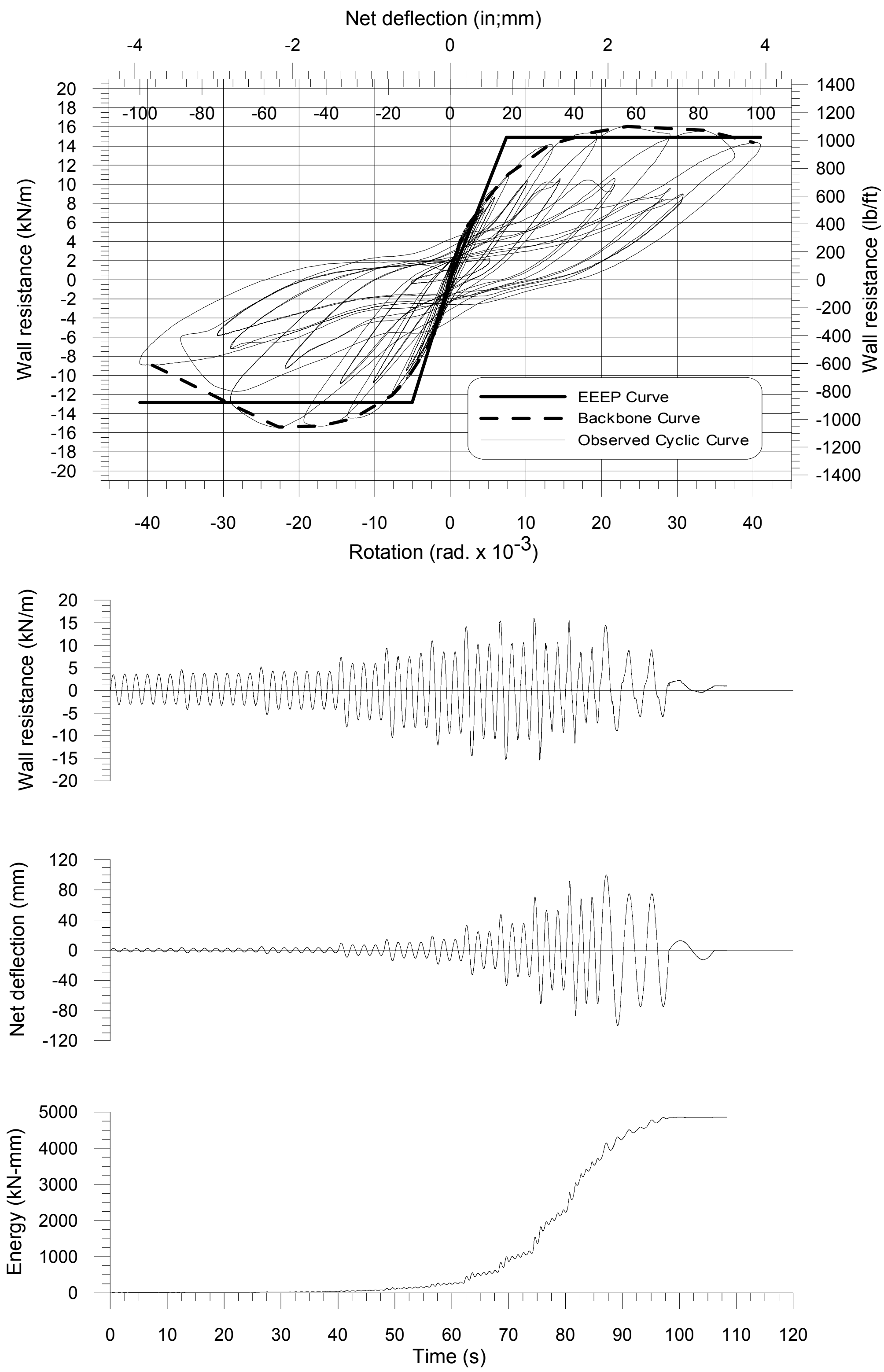


Figure C.56 Observation and EEEP Curves and Time History for Test 9C-b

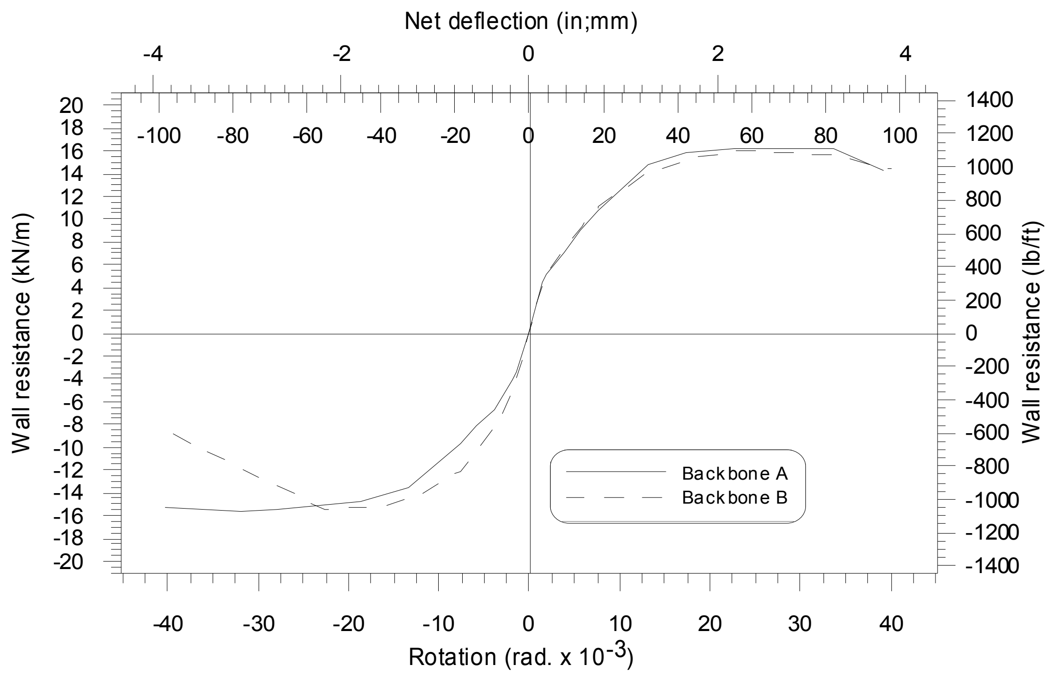


Figure C.57 Comparison of Test Results for Tests 9C-a,b

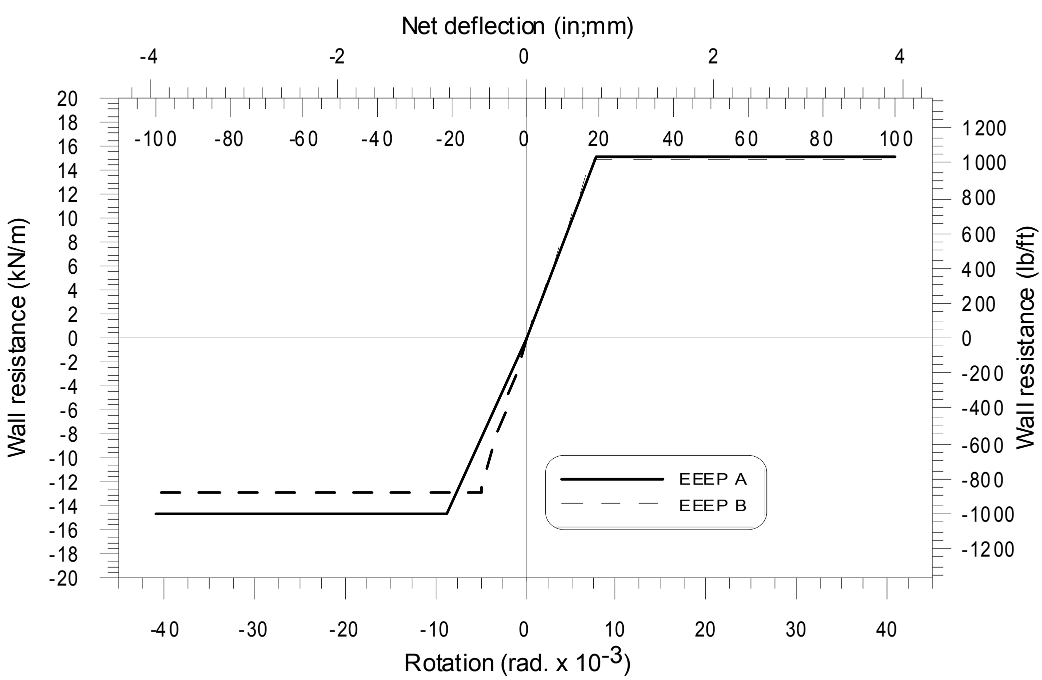


Figure C.58 Comparison of EEEP Results for Tests 9C-a,b

Table C.35 Results of Test 11C-a

Parameters			Units
	Positive	Negative	
F_u	39.31	-39.44	kN
$F_{0.8u}$	31.45	-31.55	kN
$F_{0.4u}$	15.72	-15.78	kN
F_y	36.11	-35.92	kN
K_e	4.91	4.26	kN/mm
Ductility (μ)	7.08	7.13	-
$\Delta_{net,y}$	7.35	-8.43	mm
$\Delta_{net,u}$	26.04	-29.35	mm
$\Delta_{net,0.8u}$	52.00	-60.10	mm
$\Delta_{net,0.4u}$	3.20	-3.70	mm
Energy	1744.98	2007.64	J
R_d	3.63	3.64	-
S_y	14.81	-14.73	kN/m

Table C.36 Results of Test 11C-b

Parameters			Units
	Positive	Negative	
F_u	39.47	-38.53	kN
$F_{0.8u}$	31.57	-30.83	kN
$F_{0.4u}$	15.79	-15.41	kN
F_y	36.47	-35.25	kN
K_e	5.85	5.31	kN/mm
Ductility (μ)	7.84	7.43	-
$\Delta_{net,y}$	6.24	-6.63	mm
$\Delta_{net,u}$	27.81	-26.97	mm
$\Delta_{net,0.8u}$	48.90	-49.30	mm
$\Delta_{net,0.4u}$	2.70	-2.90	mm
Energy	1669.55	1621.13	J
R_d	3.83	3.72	-
S_y	14.96	-14.46	kN/m

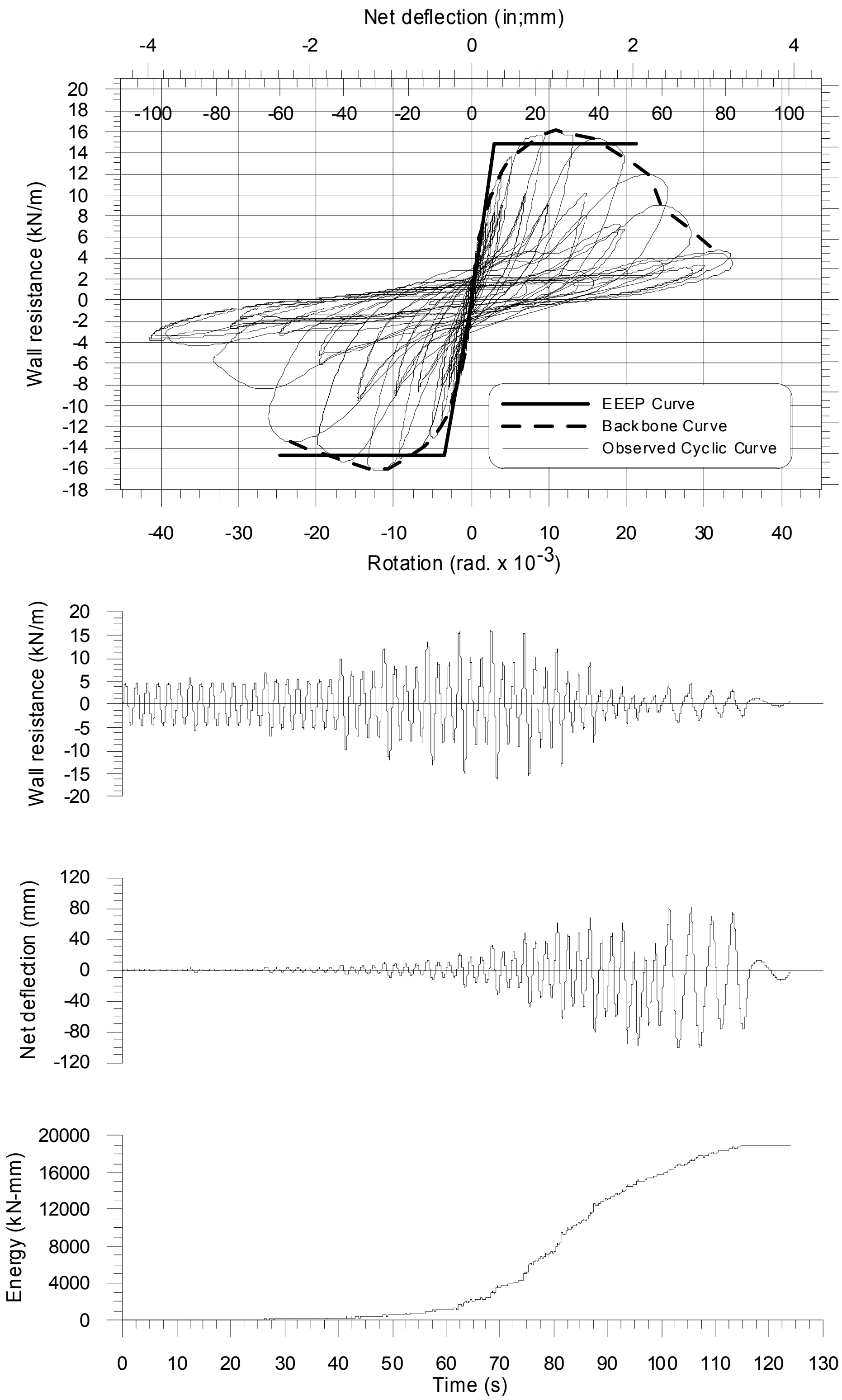


Figure C.59 Observation and EEEP Curves and Time History for Test 11C-a

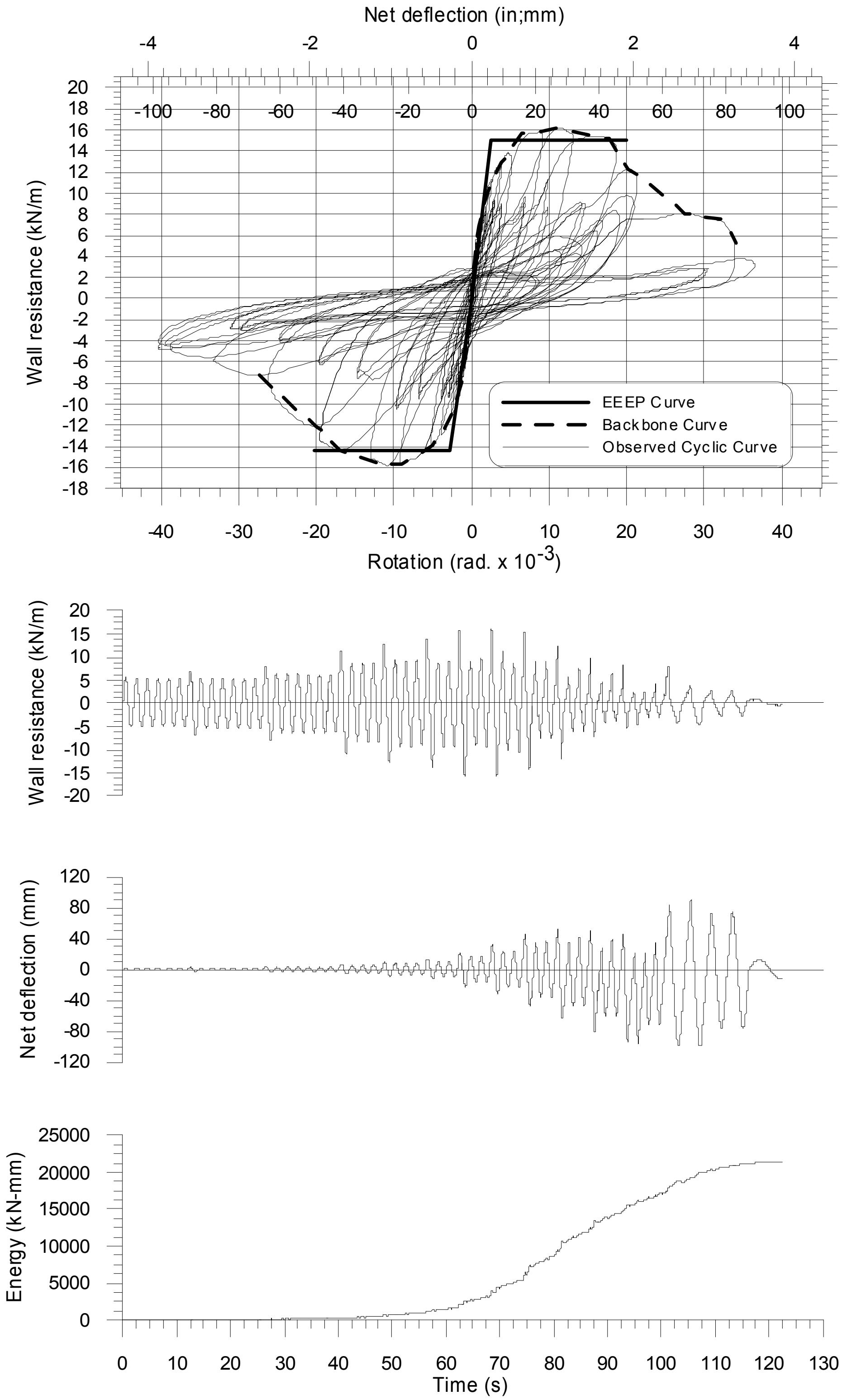


Figure C.60 Observation and EEEP Curves and Time History for Test 11C-b

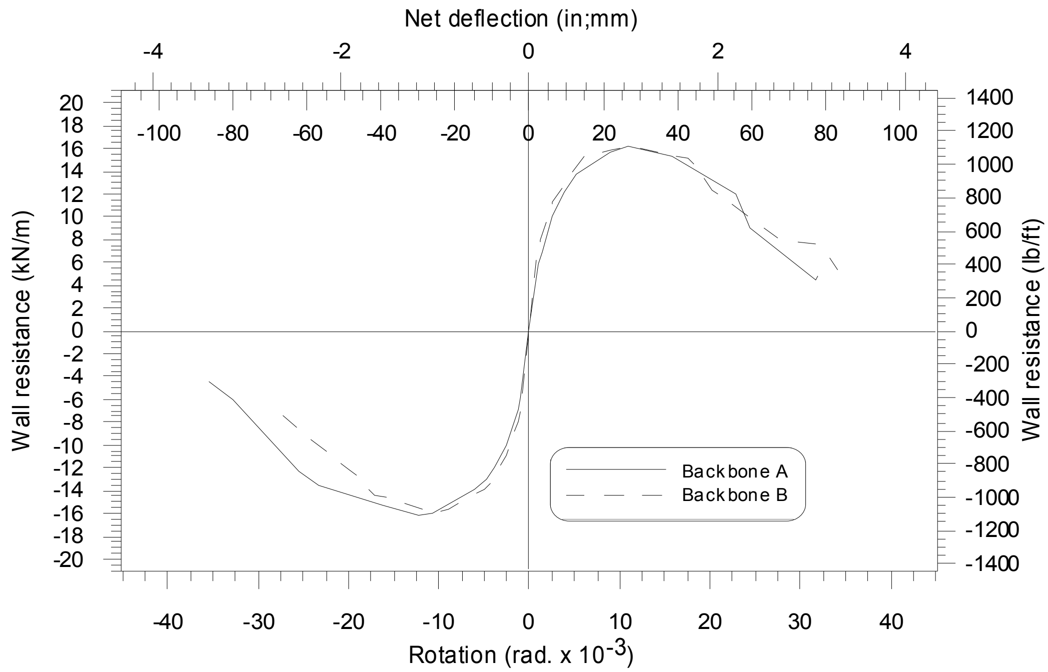


Figure C.61 Comparison of Test Results for Tests 11C-a,b

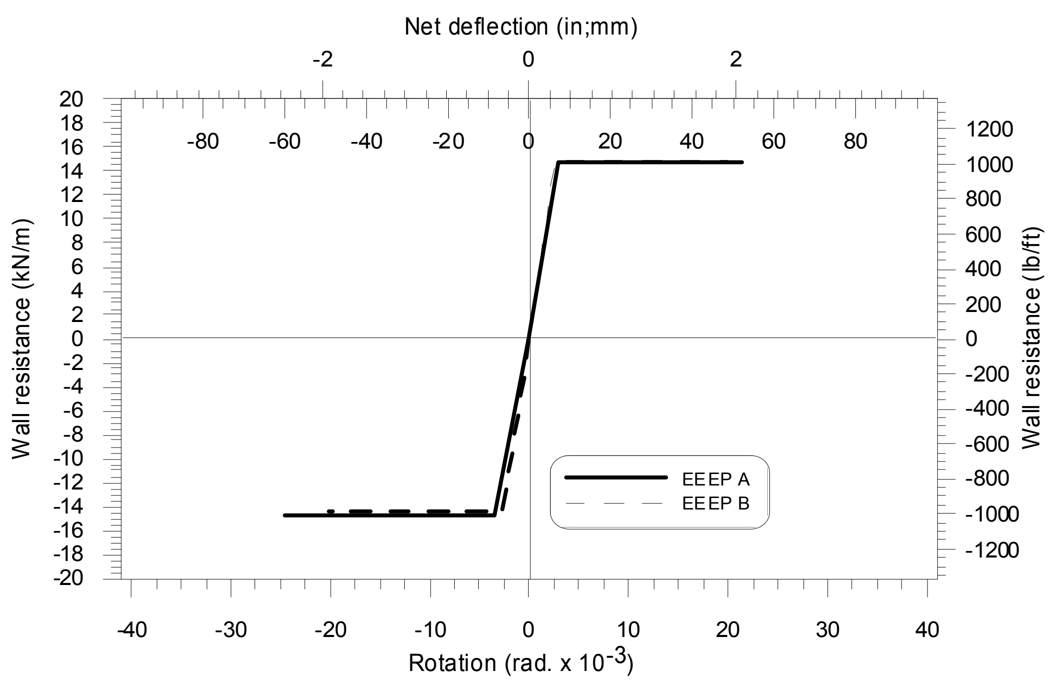


Figure C.62 Comparison of EEEP Results for Tests 11C-a,b

APPENDIX D

SHEAR WALL RESISTANCE VALUE MODIFICATION

Table D.1 Material Properties

Component	Nominal Thickness	Measured Base Thickness		Yield Stress	Tensile Stress	Reference
	mils	mm	in	MPa	MPa	
Sheathing	18	0.46	0.018	300	395	McGill
	27	0.61	0.024	347	399	Yu *et al.*
	30	0.73	0.029	337	383	Yu *et al.*
		0.76	0.030	307	385	Ellis
		0.76	0.030	284	373	McGill
	33	0.91	0.036	299	371	Yu *et al.*

Table D.2 Thickness Resistance Modification Factor

Thickness (mm)		Thickness Ratio	Thickness Modification Factor	Reference
Nominal	Measured			
0.46	0.46	1.00	1.00	McGill
0.68	0.61	1.11	1.00	Yu *et al.*
0.76	0.73	1.04	1.00	Yu *et al.*
0.76	0.76	1.00	1.00	Ellis
0.76	0.76	1.00	1.00	McGill
0.84	0.91	0.92	0.92	Yu *et al.*

Table D.3 Tensile Stress Resistance Modification Factor

Nominal Thickness	Tensile Stress (mm)		Tensile Stress Ratio	Tensile Stress Modification Factor	Reference
mils	Nominal	Measured			
18	310	395	0.78	0.78	McGill
27	310	399	0.78	0.78	Yu *et al.*
30	310	383	0.81	0.81	Yu *et al.*
	310	385	0.80	0.80	Ellis
	310	373	0.83	0.83	McGill
33	310	371	0.84	0.84	Yu *et al.*

Table D.4 Modified Resistance Values

Group	Nominal Framing (mils)	Nominal Sheathing (mils)	Fastener Spacing (mm/mm)	Test Name	Test S_y kN/m	Thickness ratio	Tensile Stress Ratio	Test S_y reduced (kN/m)	Test S_y reduced average (kN/m)		Nominal S_y (kN/m)
1	33	18	150/300	3M-a	5.04	1.00	0.78	3.95	3.95	3.95	4.13
				3M-b	5.04	1.00	0.78	3.95	3.95		
				3C-a	5.63	1.00	0.78	4.41	4.21	4.32	
					-5.11	1.00	0.78	-4.01			
				3C-c	5.58	1.00	0.78	4.37	4.42		
					-5.70	1.00	0.78	-4.47			
2		27	50/300	Y7M1	11.36	1.00	0.78	8.84	8.84	8.65	8.69
				Y7M2	10.88	1.00	0.78	8.46	8.46		
				Y7C1	10.63	1.00	0.78	8.27	8.12	8.72	
					-10.25	1.00	0.78	-7.98			
				Y7C2	12.60	1.00	0.78	9.80	9.32		
					-11.37	1.00	0.78	-8.84			
3			100/300	Y8M1	9.03	1.00	0.78	7.02	7.02	7.10	7.17
				Y8M2	9.21	1.00	0.78	7.17	7.17		
				Y8C1	9.43	1.00	0.78	7.34	7.38	7.25	
					-9.54	1.00	0.78	-7.42			
				Y8C2	9.13	1.00	0.78	7.10	7.13		
					-9.20	1.00	0.78	-7.16			

Table D.5 Modified Resistance Values *(Continued)*

Group	Nominal Framing (mils)	Nominal Sheathing (mils)	Fastener Spacing (mm/mm)	Test Name	Test S_y kN/m	Thickness ratio	Tensile Stress Ratio	Test S_y reduced (kN/m)	Test S_y reduced average (kN/m)		Nominal S_y (kN/m)
4	33	27	150/300	Y9M1	8.52	1.00	0.78	6.63	6.63	6.48	6.48
				Y9M2	8.14	1.00	0.78	6.33	6.33		
				Y9C1	8.66	1.00	0.78	6.74	6.55	6.48	
					-8.19	1.00	0.78	-6.37			
				Y9C2	8.09	1.00	0.78	6.29	6.40		
					-8.38	1.00	0.78	-6.52			
5	43	18	50/300	2M-a	9.00	1.00	0.78	7.06	7.06	7.20	7.53
				2M-b	9.36	1.00	0.78	7.34	7.34		
				2C-a	9.98	1.00	0.78	7.82	7.89	7.87	
					-10.15	1.00	0.78	-7.96			
				2C-b	10.00	1.00	0.78	7.84	7.85		
					-10.02	1.00	0.78	-7.86			
6			150/300	1M-a	5.87	1.00	0.78	4.60	4.60	4.59	4.53
				1M-b	5.85	1.00	0.78	4.59	4.59		
				1M-c	5.83	1.00	0.78	4.57	4.57		
				1C-a	5.68	1.00	0.78	4.45	4.54	4.48	
					-5.90	1.00	0.78	-4.63			
				1C-b	5.76	1.00	0.78	4.52	4.42		
					-5.52	1.00	0.78	-4.33			

Table D.6 Modified Resistance Values *(Continued)*

Group	Nominal Framing (mils)	Nominal Sheathing (mils)	Fastener Spacing (mm/mm)	Test Name	Test S_y kN/m	Thickness ratio	Tensile Stress Ratio	Test S_y reduced (kN/m)	Test S_y reduced average (kN/m)		Nominal S_y (kN/m)
7	43	30	50/300	Y4M1	14.60	1.00	0.81	11.81	11.81	12.55	12.54
				Y4M2	14.29	1.00	0.81	11.56	11.56		
				6M-a	15.47	1.00	0.83	12.86	12.86		
				6M-b	15.05	1.00	0.83	12.51	12.51		
				13M-a	16.89	1.00	0.83	14.04	14.04		
				Y4C1	13.65	1.00	0.81	11.04	11.42	12.52	
					-14.61	1.00	0.81	-11.81			
				6C-a	16.43	1.00	0.83	13.65	12.92		
					-14.67	1.00	0.83	-12.19			
				6C-b	15.72	1.00	0.83	13.06	13.21		
					-16.07	1.00	0.83	-13.36			
8	43		100/300	Y5M1	12.42	1.00	0.81	10.04	10.04	10.79	10.58
				Y5M2	13.45	1.00	0.81	10.88	10.88		
				5M-a	12.90	1.00	0.83	10.72	10.72		
				5M-b	12.41	1.00	0.83	10.31	10.31		
				11M-a	13.61	1.00	0.83	11.31	11.31		
				11M-b	14.10	1.00	0.83	11.72	11.72		
				12M-a	13.16	1.00	0.83	10.93	10.93		
				15M-a	12.56	1.00	0.83	10.44	10.44		

Table D.7 Modified Resistance Values *(Continued)*

Group	Nominal Framing (mils)	Nominal Sheathing (mils)	Fastener Spacing (mm/mm)	Test Name	Test S_y kN/m	Thickness ratio	Tensile Stress Ratio	Test S_y reduced (kN/m)	Test S_y reduced average (kN/m)		Nominal S_y (kN/m)
8	43	30	100/300	Y5C1	12.98	1.00	0.81	10.49	10.82	10.37	10.58 *(cont'd)*
					-13.78	1.00	0.81	-11.14			
				5C-a	13.28	1.00	0.83	11.04	10.70		
					-12.47	1.00	0.83	-10.36			
				5C-b	13.38	1.00	0.83	11.12	10.62		
					-12.18	1.00	0.83	-10.12			
				11C-a	14.81	1.00	0.83	12.31	12.28		
					-14.73	1.00	0.83	-12.24			
				11C-b	14.96	1.00	0.83	12.43	12.23		
					-14.46	1.00	0.83	-12.02			
				E114	13.98	1.00	0.80	11.25	10.29		
					-11.59	1.00	0.80	-9.32			
				E115	12.95	1.00	0.80	10.42	10.30		
					-12.66	1.00	0.80	-10.19			
				E116	11.80	1.00	0.80	9.49	9.28		
					-11.27	1.00	0.80	-9.06			
				E117	12.32	1.00	0.80	9.91	9.95		
					-12.41	1.00	0.80	-9.99			
				E118	10.87	1.00	0.80	8.74	9.18		
					-11.94	1.00	0.80	-9.61			
				E119	11.13	1.00	0.80	8.95	9.37		
					-12.16	1.00	0.80	-9.78			
				E120	11.12	1.00	0.80	8.95	9.43		
					-12.32	1.00	0.80	-9.91			

Table D.8 Modified Resistance Values *(Continued)*

Group	Nominal Framing (mils)	Nominal Sheathing (mils)	Fastener Spacing (mm/mm)	Test Name	Test S_y kN/m	Thickness ratio	Tensile Stress Ratio	Test S_y reduced (kN/m)	Test S_y reduced average (kN/m)		Nominal S_y (kN/m)
9	43	30	150/300	Y6M1	10.66	1.00	0.81	8.62	8.62	8.47	8.89
				Y6M2	10.58	1.00	0.81	8.56	8.56		
				4M-a	10.08	1.00	0.83	8.37	8.37		
				4M-b	10.03	1.00	0.83	8.33	8.33		
				Y6C1	11.03	1.00	0.84	9.21	9.56	9.32	
					-12.25	1.00	0.81	-9.91			
				Y6C2	12.12	1.00	0.81	9.80	9.68		
					-11.83	1.00	0.81	-9.56			
				4C-a	11.60	1.00	0.83	9.64	9.13		
					-10.37	1.00	0.83	-8.62			
				4C-b	10.81	1.00	0.83	8.98	8.90		
					-10.60	1.00	0.83	-8.81			
10		33	50/300	Y1M1	18.02	0.92	0.84	13.88	13.88	14.08	13.93
				Y1M2	18.52	0.92	0.84	14.27	14.27		
				Y1C1	18.41	0.92	0.84	14.18	14.21	13.79	
					-18.47	0.92	0.84	-14.23			
				Y1C2	17.16	0.92	0.84	13.22	13.38		
					-17.59	0.92	0.84	-13.55			
11			100/300	Y2M1	14.89	0.92	0.84	11.47	11.47	11.82	12.01
				Y2M2	15.80	0.92	0.84	12.17	12.17		
				Y2C1	16.14	0.92	0.84	12.43	11.92	12.20	
					-14.81	0.92	0.84	-11.41			
				Y2C2	15.86	0.92	0.84	12.22	12.49		
					-16.56	0.92	0.84	-12.76			

Table D.9 Modified Resistance Values *(Continued)*

Group	Nominal Framing (mils)	Nominal Sheathing (mils)	Fastener Spacing (mm/mm)	Test Name	Test S_y kN/m	Thickness ratio	Tensile Stress Ratio	Test S_y reduced (kN/m)	Test S_y reduced average (kN/m)		Nominal S_y (kN/m)
12	43	33	150/300	Y3M1	13.37	0.92	0.84	10.30	10.30	10.47	10.69
				Y3M2	13.80	0.92	0.84	10.63	10.63		
				Y3C1	14.88	0.92	0.84	11.46	11.04	10.91	
					-13.79	0.92	0.84	-10.62			
				Y3C2	14.79	0.92	0.84	11.39	10.78		
					-13.19	0.92	0.84	-10.16			

APPENDIX E

HYSTERESIS MATCHING

Hysteresis Matching

Framing: 1.09mm (0.043") framing,
Sheathing: 0.46mm (0.018")
Fastener Spacing: 50mm (2")

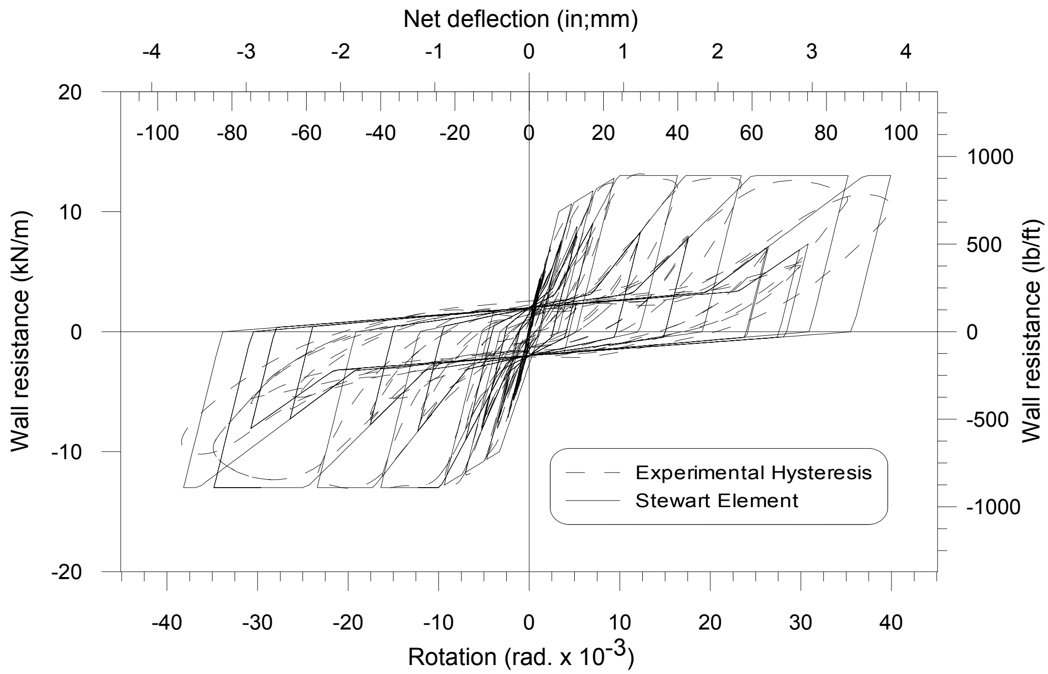


Figure E.1 Hysteresis Matching of 0.46mm Sheathing and 50mm Fastener Spacing

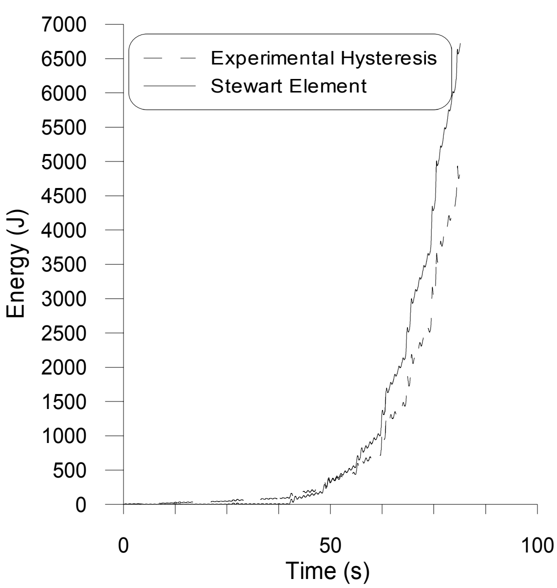


Figure E.2 Comparison of Dissipated Energy

Table E.1 Description of Parameters 0.46mm Sheathing, 50mm Fastener Spacing

K_o	1.25 kN/mm
R_f	0.15
F_{x+}	10.0 kN
F_{x-}	-10.0 kN
F_u	13.0 kN
F_i	2.0 kN
P_{tri}	0.0
P_{UNL}	1.0
gap^+	0.0
gap^-	0.0
beta	1.05
alpha	0.70

Hysteresis Matching

Framing: 1.09mm (0.043”) framing,
Sheathing: 0.46mm (0.018”)
Fastener Spacing: 150mm (6”)

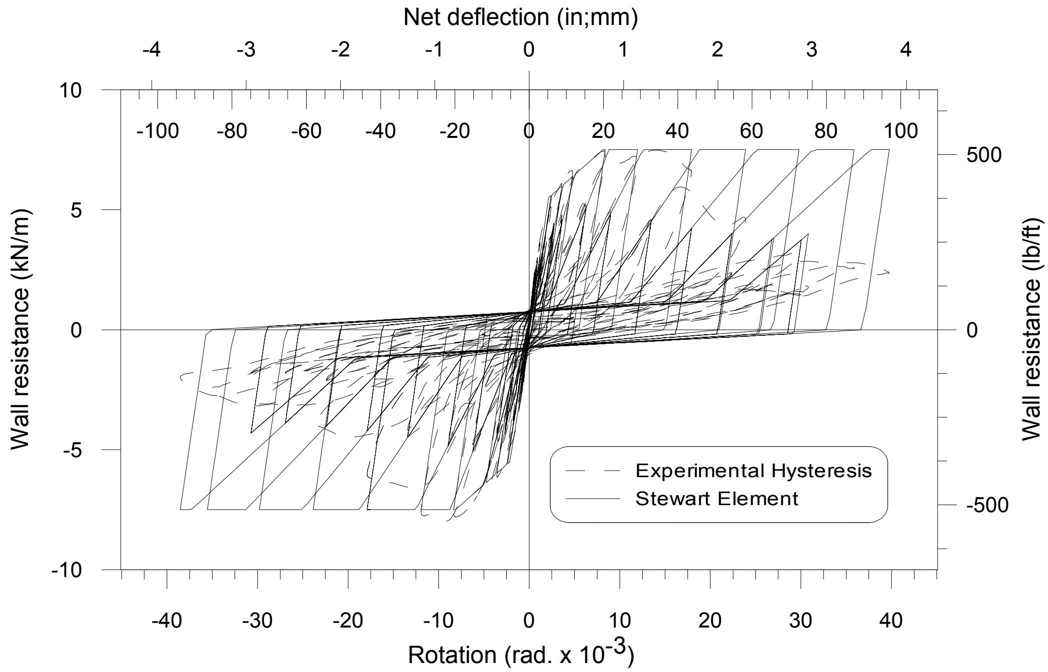


Figure E.3 Hysteresis Matching of 0.46mm Sheathing and 150mm Fastener Spacing

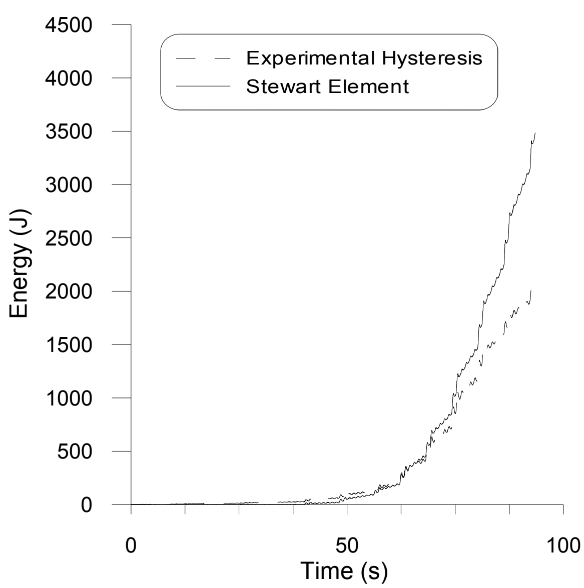


Table E.2 Description of Parameters 0.46mm Sheathing, 150mm Fastener Spacing

K_o	1.05 kN/mm
R_f	0.13
F_{x+}	5.5 kN
F_{x-}	-5.5 kN
F_u	7.5 kN
F_i	0.75 kN
P_{tri}	0.0
P_{UNL}	1.0
gap^+	0.0
gap^-	0.0
beta	1.05
alpha	0.70

Figure E.4 Comparison of Dissipated Energy

Hysteresis Matching

Framing: 1.09mm (0.043”) framing,
Sheathing: 0.76mm (0.030”)
Fastener Spacing: 50mm (2”)

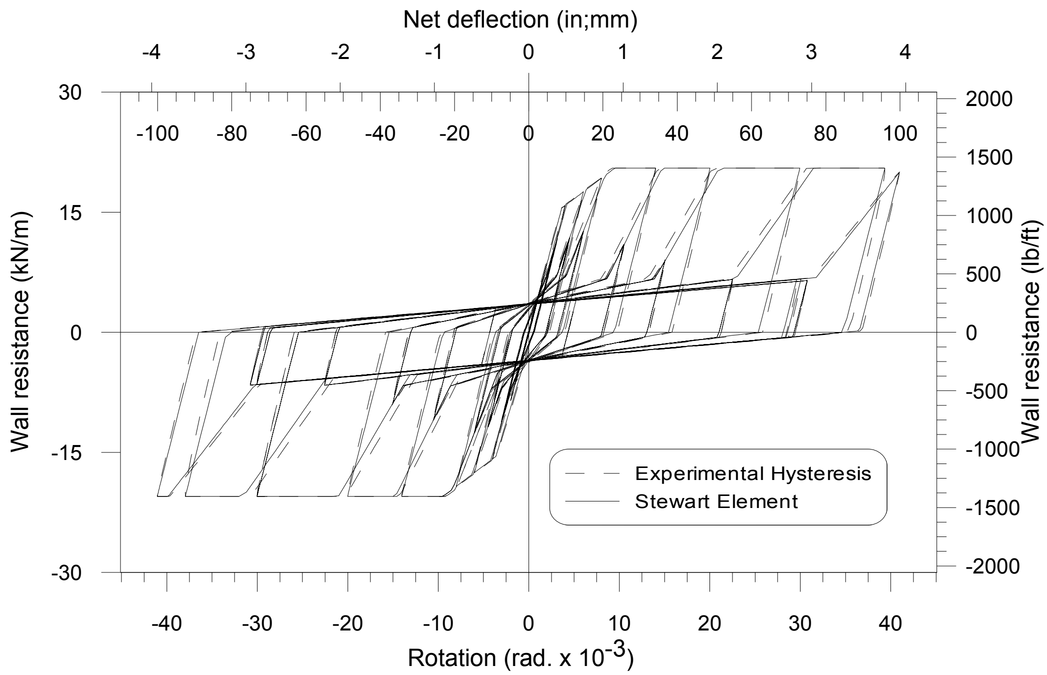


Figure E.5 Hysteresis Matching of 0.76mm Sheathing and 50mm Fastener Spacing

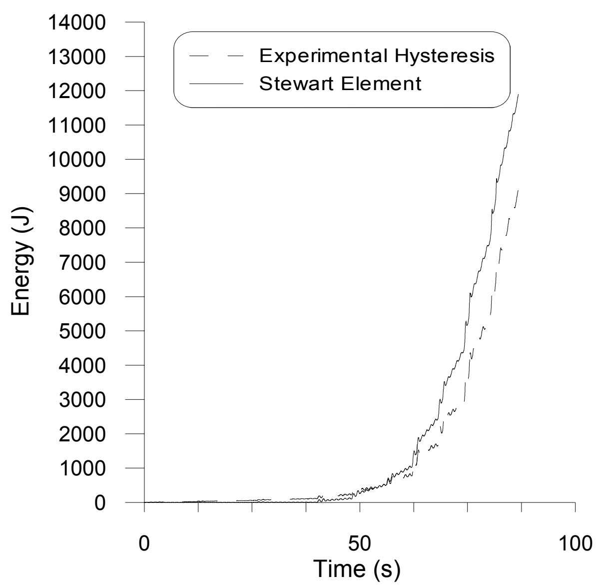


Figure E.6 Comparison of Dissipated Energy

Table E.3 Description of Parameters 0.76mm Sheathing, 50mm Fastener Spacing

K_o	1.75 kN/mm
R_f	0.20
F_{x+}	15.5 kN
F_{x-}	-15.5 kN
F_u	20.5 kN
F_i	3.5 kN
P_{tri}	0.0
P_{UNL}	1.05
gap^+	0.0
gap^-	0.0
beta	1.05
alpha	0.45

Hysteresis Matching

Framing: 1.09mm (0.043") framing,
Sheathing: 0.76mm (0.030")
Fastener Spacing: 100mm (4")

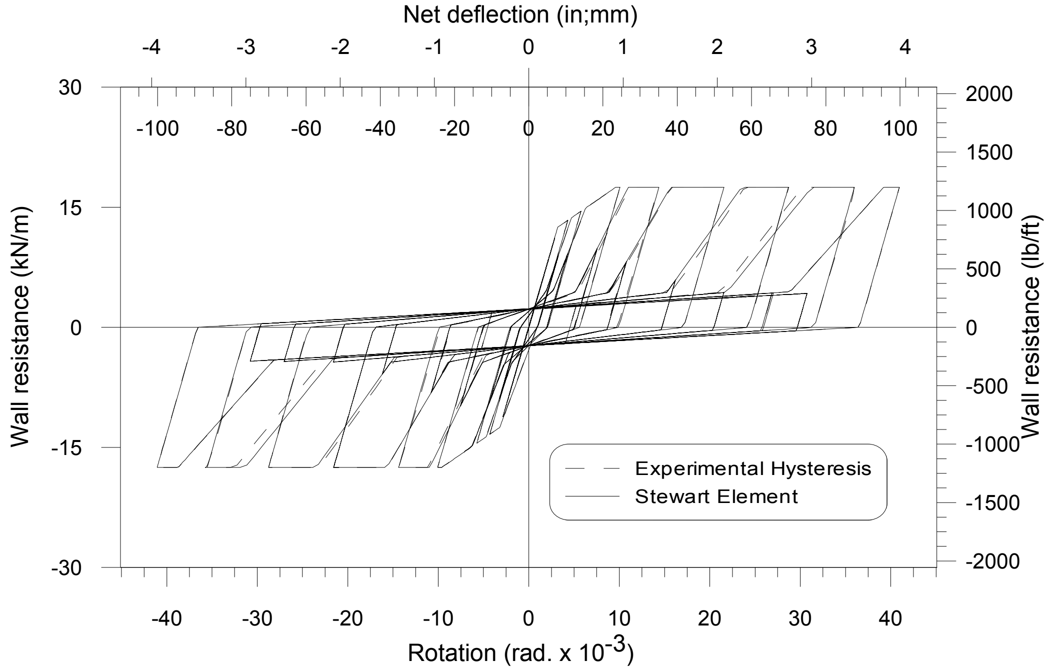


Figure E.7 Hysteresis Matching of 0.76mm Sheathing and 100mm Fastener Spacing

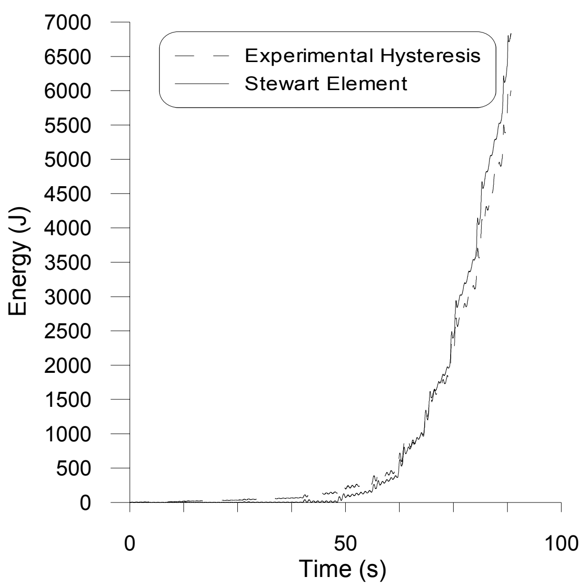


Figure E.8 Comparison of Dissipated Energy

Table E.4 Description of Parameters 0.76mm Sheathing, 100mm Fastener Spacing

K_o	1.60 kN/mm
R_f	0.20
F_{x+}	12.5 kN
F_{x-}	-12.5 kN
F_u	17.5 kN
F_i	2.3 kN
P_{tri}	0.0
P_{UNL}	1.0
gap^+	0.0
gap^-	0.0
beta	1.09
alpha	0.45

Hysteresis Matching

Framing: 1.09mm (0.043”) framing,
Sheathing: 0.76mm (0.030”)
Fastener Spacing: 150mm (6”)

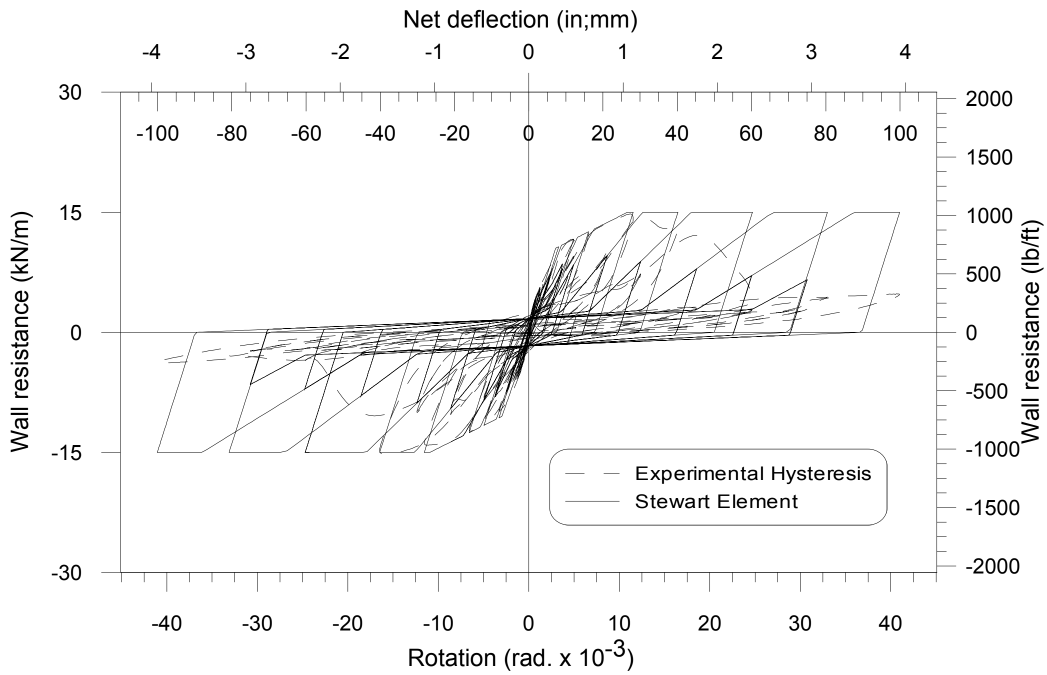


Figure E.9 Hysteresis Matching of 0.76mm Sheathing and 150mm Fastener Spacing

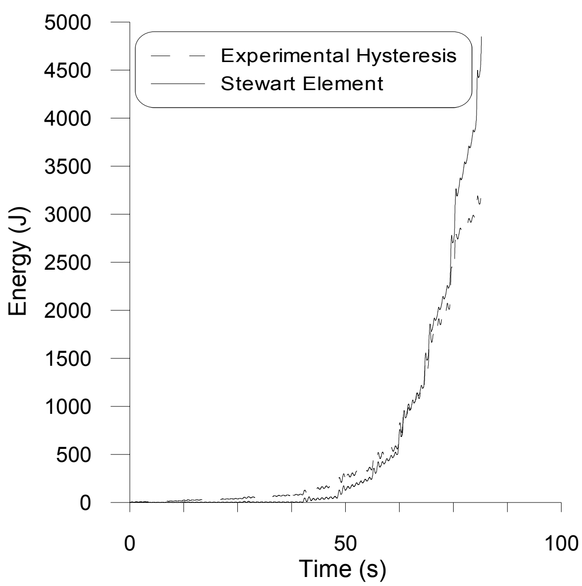


Figure E.10 Comparison of Dissipated Energy

Table E.5 Description of Parameters 0.76mm Sheathing, 150mm Fastener Spacing

K_o	1.45 kN/mm
R_f	0.16
F_{x+}	10.5 kN
F_{x-}	-10.5 kN
F_u	15.0 kN
F_i	1.70 kN
P_{tri}	0.0
P_{UNL}	1.0
gap^+	0.0
gap^-	0.0
beta	1.09
alpha	0.65

Hysteresis Matching

Framing: 1.09mm (0.043”) framing,
Sheathing: 0.84mm (0.033”)
Fastener Spacing: 50mm (2”)

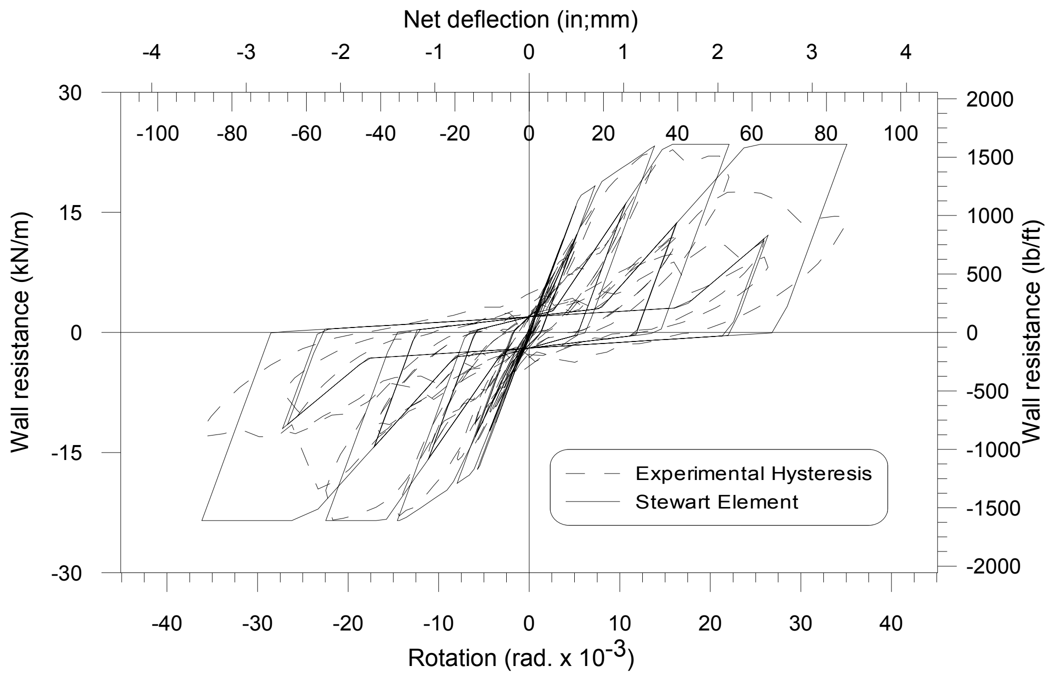


Figure E.11 Hysteresis Matching of 0.84mm Sheathing and 50mm Fastener Spacing

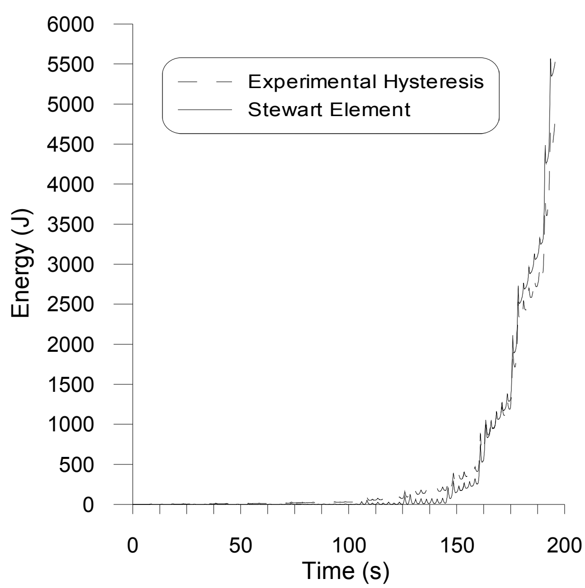


Figure E.12 Comparison of Dissipated Energy

Table E.6 Description of Parameters 0.84mm Sheathing, 50mm Fastener Spacing

K_o	1.25 kN/mm
R_f	0.25
F_{x+}	17.0 kN
F_{x-}	-17.0 kN
F_u	23.5 kN
F_i	1.95 kN
P_{tri}	0.0
P_{UNL}	1.0
gap^+	0.0
gap^-	0.0
beta	1.09
alpha	0.60

Hysteresis Matching

Framing: 1.09mm (0.043”) framing,
Sheathing: 0.84mm (0.033”)
Fastener Spacing: 100mm (4”)

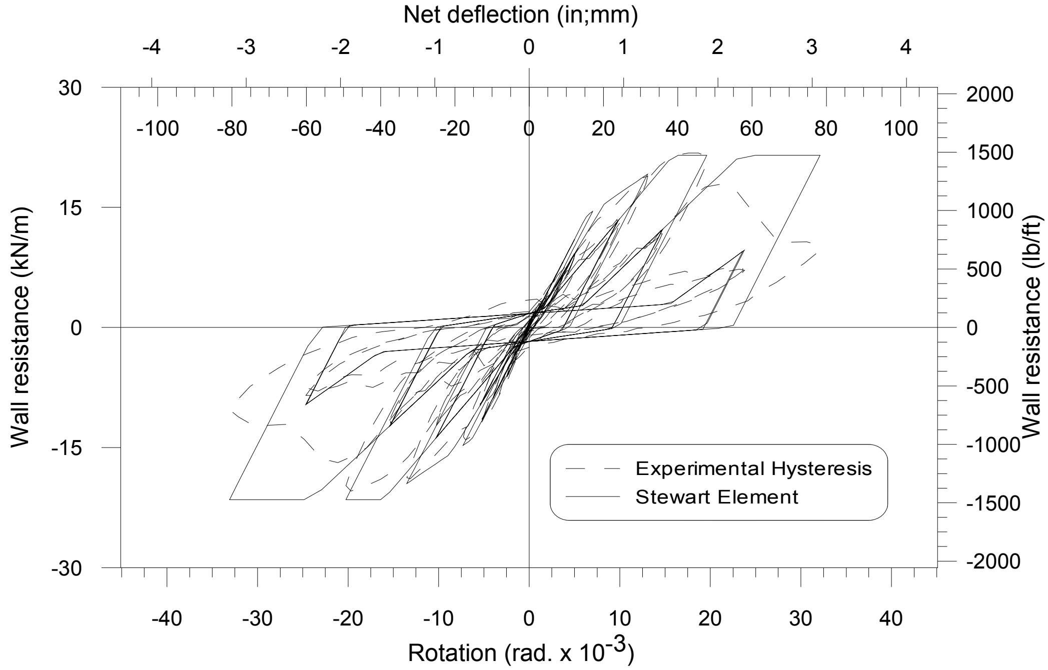


Figure E.13 Hysteresis Matching of 0.84mm Sheathing and 100mm Fastener Spacing

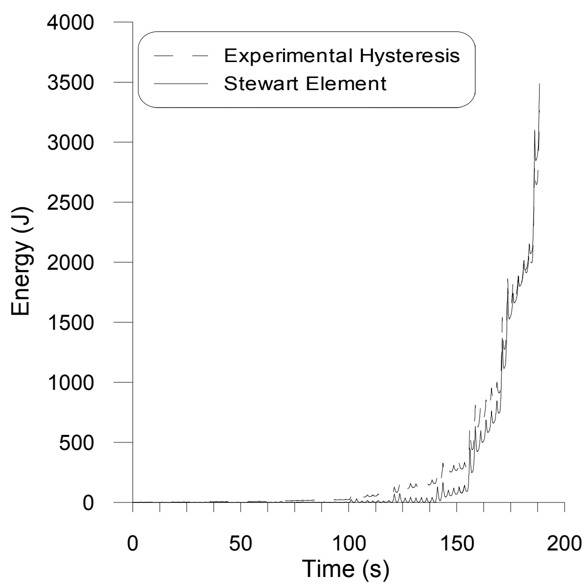


Figure E.14 Comparison of Dissipated Energy

Table E.7 Description of Parameters 0.84mm Sheathing, 100mm Fastener Spacing

K_o	0.9 kN/mm
R_f	0.35
F_{x+}	14.0 kN
F_{x-}	-14.0 kN
F_u	21.5 kN
F_i	1.75 kN
P_{tri}	0.0
P_{UNL}	1.0
gap^+	0.0
gap^-	0.0
beta	1.19
alpha	0.55

Hysteresis Matching

Framing: 1.09mm (0.043”) framing,
Sheathing: 0.84mm (0.033”)
Fastener Spacing: 150mm (6”)

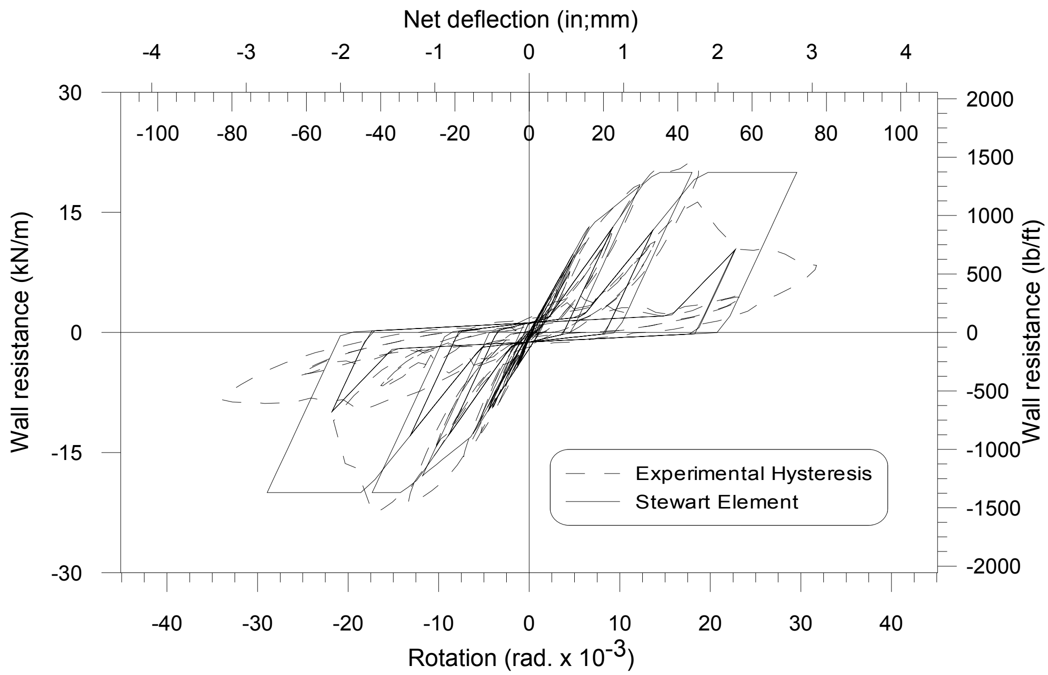


Figure E.15 Hysteresis Matching of 0.84mm Sheathing and 150mm Fastener Spacing

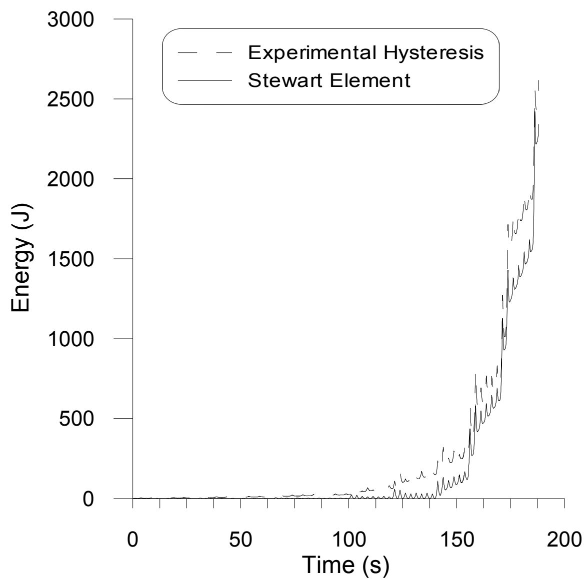


Figure E.16 Comparison of Dissipated Energy

Table E.8 Description of Parameters 0.84mm Sheathing, 150mm Fastener Spacing

K_o	0.83 kN/mm
R_f	0.45
F_{x+}	13.0 kN
F_{x-}	-13.0 kN
F_u	20.0 kN
F_i	1.2 kN
P_{tri}	0.0
P_{UNL}	1.2
gap^+	0.0
gap^-	0.0
beta	1.05
alpha	0.35

APPENDIX F

SAMPLE INPUT CODES FOR RUAUMOKO

```
1.25 0.25 17 -17                    !KX  RF  FX+  FX-
9                                   !9 = Wayne Stewart Hysteresis Model
0                                   !0 = No Strength Degradation (Not available for Stewart)
23.5 1.95 0.0 1.0 0 0 1.09 0.6 0    !FU  FI  PTRI  PUNL  GAP+  GAP-  BETA  ALPHA
1
0
3 1
0
0.0815594
0.0815594
0.0543814
0.1087374
0.067945
0.0815594
0.1087374
0.0951484
0.1359408
0.0951484
0.0543814
0.0951484
0.1223264
0.0679704
 (continued – values not shown)
0
STOP
1
```

Figure F.1 Sample Code for Hysteres (0.84mm Sheathing, 50mm Fastener Spacing)

```
4 storey shear wall Rd = 2.5       Ro = 1.7          ! Units kN, m and s
2 0 1 0 0 0 1 0 0                                    ! Principal Analysis Options
10 8 5 6 1 2 9.81 5 5 0.005 40.95 1                  ! Frame Control Parameters
0 0 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      3.66   0      0      1      0      0      0      0
3      0      6.71   0      0      1      0      0      0      0
4      0      9.76   0      0      1      0      0      0      0
5      0      12.81  0      0      1      0      0      0      0
6      3      0      1      1      1      0      0      0      3
7      3      3.66   0      0      1      2      0      0      0
8      3      6.71   0      0      1      3      0      0      0
9      3      9.76   0      0      1      4      0      0      0
10     3      12.81  0      0      1      5      0      0      0

ELEMENTS
1      1      1      2      0      0      0
2      2      2      3      0      0      0
3      3      3      4      0      0      0
4      4      4      5      0      0      0
5      5      6      7      0      0      0
6      5      7      8      0      0      0
7      5      8      9      0      0      0
8      5      9      10     0      0      0
```

Figure F.2 Sample Code for Ruaumoko – Four-Storey Building

```
PROPS

1        SPRING                                     ! BRACE:50/300 0.033"
1 9 0 0 1000000 23602.03 0 0    0.25                ! Itype 1, Ihyst = Wayne Stewart,Ilos = No Strength Degradation,IDAMG,Kx,Ky,GJ,WGT,RF
442.00  -442.00 442.00  -442.00                     ! FY+ FY- FX+  FX-
611.00  50.70   0.00    1.00    0 0 1.09 0.6        ! FU  FI  PTRI  PUNL  GAP+  GAP-  BETA  ALPHA

2        SPRING                                     ! BRACE:        50/300 0.033"
1 9 0 0 1000000 24401.06 0 0    0.25                ! Itype 1, Ihyst = Wayne Stewart,Ilos = No Strength Degradation,IDAMG,Kx,Ky,GJ,WGT,RF
374.00  -374.00 374.00  -374.00                     ! FY+ FY- FX+  FX-
517.00  42.90   0.00    1.00    0 0 1.09 0.6        ! FU  FI  PTRI  PUNL  GAP+  GAP-  BETA  ALPHA

3        SPRING                                     ! BRACE:        50/300 0.033"
1 9 0 0 1000000 21250.00 0 0    0.25                ! Itype 1, Ihyst = Wayne Stewart,Ilos = No Strength Degradation,IDAMG,Kx,Ky,GJ,WGT,RF
289.00  -289.00 289.00  -289.00                     ! FY+ FY- FX+  FX-
399.50  33.15   0.00    1.00    0 0 1.09 0.6        ! FU  FI  PTRI  PUNL  GAP+  GAP-  BETA  ALPHA

4        SPRING                                     ! BRACE:        50/300 0.033"
1 9 0 0 1000000 15527.95 0 0    0.25                ! Itype 1, Ihyst = Wayne Stewart,Ilos = No Strength Degradation,IDAMG,Kx,Ky,GJ,WGT,RF
238.00  -238.00 238.00  -238.00                     ! FY+ FY- FX+  FX-
329.00  27.30   0.00    1.00    0 0 1.09 0.6        ! FU  FI  PTRI  PUNL  GAP+  GAP-  BETA  ALPHA

5        SPRING
1 0 0 0 1000000
```

Figure F.3 Sample Code for Ruaumoko – Four-Storey Building *(Continued)*

```
WEIGHT
1       0
2       630.6
3       630.6
4       630.6
5       241.7
6       0
7       0
8       0
9       0
10      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    -486.11      0
8       0    -528.90      0
9       0    -584.64      0
10      0    -196.59      0

EQUAKE
3  1    0.01    1      40.95   0 0 1.0

START
```

Figure F.4 Sample Code for Ruaumoko – Four-Storey Building *(Continued)*

```
4 storey shear wall Rd = 2.5     Ro = 1.7          ! Units kN, m and s
2 0 1 0 0 -1 1 0 0                                 ! Principal Analysis Options
10 8 5 6 1 2 9.81 5 5 0.005 60 1                   ! Frame Control Parameters
0 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     3.66    0     0     1     0     0     0     0
3     0     6.71    0     0     1     0     0     0     0
4     0     9.76    0     0     1     0     0     0     0
5     0     12.81   0     0     1     0     0     0     0
6     3     0       1     1     1     0     0     0     3
7     3     3.66    0     0     1     2     0     0     0
8     3     6.71    0     0     1     3     0     0     0
9     3     9.76    0     0     1     4     0     0     0
10    3     12.81   0     0     1     5     0     0     0

ELEMENTS
1     1     1     2     0     0     0
2     2     2     3     0     0     0
3     3     3     4     0     0     0
4     4     4     5     0     0     0
5     5     6     7     0     0     0
6     5     7     8     0     0     0
7     5     8     9     0     0     0
8     5     9     10    0     0     0
```

Figure F.5 Sample Code for Pushover Analysis – Four-Storey Building

```
PROPS

1         SPRING                                    ! BRACE:        50/300 0.033"
1 9 0 0 1000000  23602.03   0 0  0.25               ! Itype 1, Ihyst = Wayne Stewart,Ilos = No Strength Degradation,IDAMG,Kx,Ky,GJ,WGT,RF
442.00  -442.00 442.00  -442.00                     ! FY+ FY- FX+  FX-
611.00  50.70    0.00      1.00      0 0 1.09 0.6   ! FU  FI  PTRI  PUNL  GAP+  GAP-  BETA  ALPHA

2         SPRING                                    ! BRACE:        50/300 0.033"
1 9 0 0 1000000  24401.06  0 0   0.25               ! Itype 1, Ihyst = Wayne Stewart,Ilos = No Strength Degradation,IDAMG,Kx,Ky,GJ,WGT,RF
374.00  -374.00 374.00  -374.00                     ! FY+ FY- FX+  FX-
517.00  42.90    0.00      1.00      0 0 1.09 0.6   ! FU  FI  PTRI  PUNL  GAP+  GAP-  BETA  ALPHA

3         SPRING                                    ! BRACE:        50/300 0.033"
1 9 0 0 1000000 21250.00  0 0   0.25                ! Itype 1, Ihyst = Wayne Stewart,Ilos = No Strength Degradation,IDAMG,Kx,Ky,GJ,WGT,RF
289.00  -289.00 289.00  -289.00                     ! FY+ FY- FX+  FX-
399.50  33.15    0.00      1.00      0 0 1.09 0.6   ! FU  FI  PTRI  PUNL  GAP+  GAP-  BETA  ALPHA

4         SPRING                                    ! BRACE:        50/300 0.033"
1 9 0 0 1000000  15527.95  0 0   0.25               ! Itype 1, Ihyst = Wayne Stewart,Ilos = No Strength Degradation,IDAMG,Kx,Ky,GJ,WGT,RF
238.00  -238.00 238.00  -238.00                     ! FY+ FY- FX+  FX-
329.00  27.30    0.00      1.00      0 0 1.09 0.6   ! FU  FI  PTRI  PUNL  GAP+  GAP-  BETA  ALPHA

5         SPRING
1 0 0 0 1000000
```

Figure F.6 Sample Code for Pushover Analysis – Four-Storey Building (Continuted)

```
WEIGHT
1	0
2	1
3	1
4	1
5	1
6	0
7	0
8	0
9	0
10	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	-482.67	0
8	0	-525.99	0
9	0	-582.39	0
10	0	-195.98	0
```

Figure F.7 Sample Code for Pushover Analysis – Four-Storey Building (Continuted)

```
SHAPE
2 0.146
3 0.268
4 0.390
5 0.196

EQUAKE
3

START
1 0 0
2 60 600
```

Figure F.8 Sample Code for Pushover Analysis – Four-Storey Building (Continuted)

APPENDIX G

DESIGN PROCEDURE – PHASE I

Two-Storey Building –Vancouver, BC

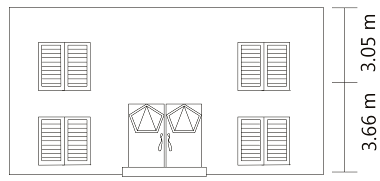


Figure G.1 Elevation View of Two-Storey Model Building

Table G.1 Seismic Weight Distribution for Two-Storey Building

Level	Storey Height (m)	Area (m^2)	Dead (kPa)	Snow (kPa)	Live (kPa)	Seismic Weight (kN)	Cumulative (kN)
Roof		220	0.69	1.64		241.71	241.71
2	3.05	220	2.87		2.44	630.64	872.34
1	3.66	220	2.87		2.44		872.34

Table G.2 Design Base Shear Distribution for Two-Storey Building

Storey	W_i (kN)	h_i (m)	W_i x h_i	F_x (kN)	T_x (kN)	N_x(kN)	Vf_x (kN)
Roof	241.7	6.71	1622	53.08	5.31	1.21	59.60
2	630.6	3.66	2308	75.54	7.55	4.49	87.59
1	-	-	-	-	-	-	-
Σ			3930	129			147

Table G.3 Design of Two-Storey Building Adjusted for Irregularity

Level	Shear (kN)	Framing (mm)	Sheathing (mm)	Fastener Spacing (mm)	S_v (kN/m)	S_r (kN/m)	Min L (m)	# walls[1]	Rounded # walls[1]
2	59.60	1.09	0.46	150	4.53	3.17	18.78	15.40	18
1	147.19	1.09	0.46	50	7.53	5.27	27.91	22.88	23

[1] 1220mm (4’) wall segments

Table G.4 Design of Double Chord Studs of Two-Storey Building

Storey	Compression – shear (kN)	Compression – gravity (kN)	Compression – total (kN)	DCS Thickness (mm)	# DCS	Area DCS (mm^2)	DCS Pn (kN)
2	19.36	1.61	20.97	1.09	1	417.32	56.6
1	38.60	5.99	65.56	1.37	1	541.19	100

Table G.5 Inter-storey Drift and Stability Factor of Two-Storey Building

Level	h_s (mm)	Δ (mm)	Δ_{mx} (mm)	Drift (%)	θ_x
2	2750	5.9	25.3	0.92	0.022
1	3360	7.8	33.0	0.98	0.034

Table G.6 P-Δ Loads for Two-Storey Building

Level	P-Δ Area (m^2)	Cumulative Area (m^2)	LLRF	Reduced live load (kPa)	Gravity Load (kPa)	P_x (kN)
2	167.0	167.0	0.54	0	1.10	183.7
1	152.3	319.3	0.48	1.16	3.45	525.6

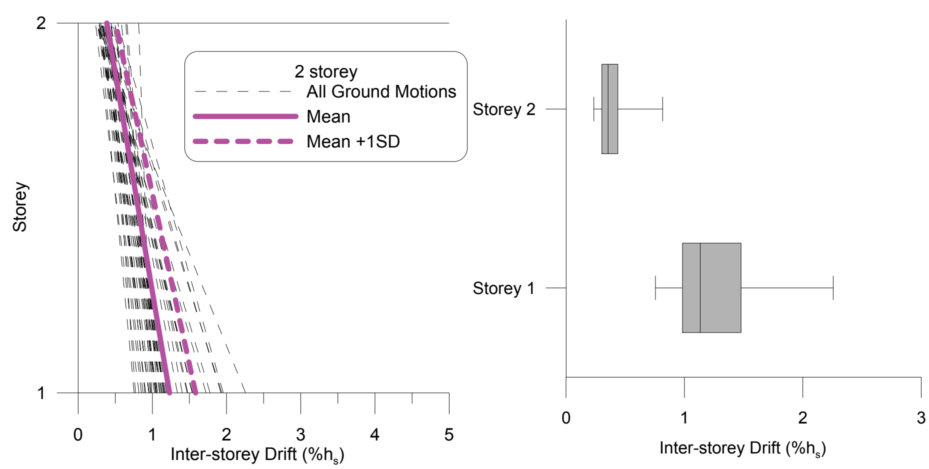


Figure G.2 Inter-Storey Drifts of Two-Storey Building for All 45 Records at Design Level

Three-Storey Building –Vancouver, BC

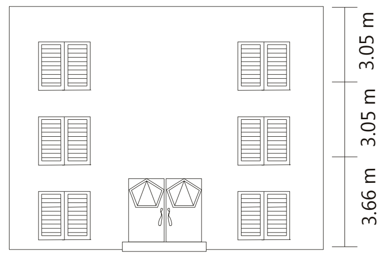


Figure G.3 Elevation View of Three-Storey Building

Table G.7 Seismic Weight Distribution for Three-Storey Building

Level	Storey Height (m)	Area (m^2)	Dead (kPa)	Snow (kPa)	Live (kPa)	Seismic Weight (kN)	Cumulative (kN)
Roof		220	0.69	1.64		241.71	241.71
3	3.05	220	2.87		2.44	630.64	872.34
2	3.05	220	2.87		2.44	630.64	1502.98
1	3.66	220	2.87		2.44		1502.98

Table G.8 Design Base Shear Distribution for Three-Storey Building

Storey	W_i (kN)	h_i (m)	W_i x h_i	F_x (kN)	T_x (kN)	N_x(kN)	Vf_x (kN)
Roof	241.7	9.76	2359	56.97	5.70	1.21	63.87
3	630.6	6.71	4232	102.18	10.22	4.49	116.90
2	630.6	3.66	2308	55.74	5.57	4.49	65.80
1	-	-	-	-	-	-	-
Σ			8899	215			247

G.9 Design of Three-Storey Building Adjusted for Irregularity

Level	Shear (kN)	Framing (mm)	Sheathing (mm)	Fastener Spacing (mm)	S_v (kN/m)	S_r (kN/m)	Min L (m)	# walls[1]	Rounded # walls[1]
3	63.87	1.09	0.84	50	13.94	9.75	6.55	5.37	14
2	180.77	1.09	0.84	50	13.94	9.75	18.53	15.19	17
1	246.57	1.09	0.84	50	13.94	9.75	25.28	20.72	21

[1] 1220mm (4') wall segments

Table G.10 Design of Double Chord Studs of Three-Storey Building

Storey	Compression – shear (kN)	Compression – gravity (kN)	Compression – total (kN)	DCS Thickness (mm)	# DCS	Area DCS (mm^2)	DCS Pn (kN)
3	59.48	1.63	61.11	1.37	1	541.2	100.0
2	59.48	6.07	126.67	1.73	1	670.4	128.8
1	71.38	6.07	204.12	2.46	1.5	1384.6	264.6

Table G.11 Inter-storey Drift and Stability Factor of Three-Storey Building

Level	h_s (mm)	Δ (mm)	Δ_{mx} (mm)	Drift (%)	θ_x
3	2750	5.9	25.1	0.91	0.020
2	2750	6.3	26.6	0.97	0.027
1	3360	7.5	31.7	0.94	0.034

Table G.12 P-Δ Loads for Three-Storey Building

Level	P-Δ Area (m^2)	Cumulative Area (m^2)	LLRF	Reduced live load (kPa)	Gravity Load (kPa)	P_x (kN)
3	178.2	178.2	0.53	0.00	1.10	196.0
2	169.3	347.4	0.47	1.14	3.44	582.4
1	157.4	504.8	0.44	1.07	3.41	536.0

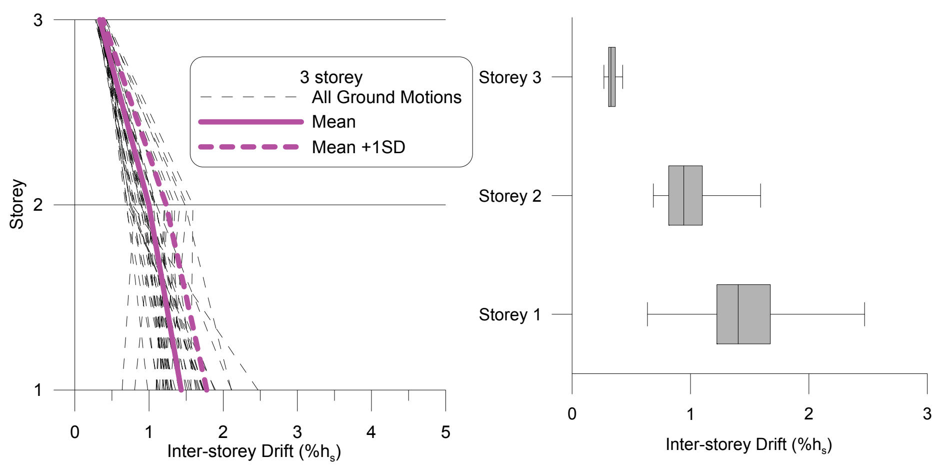


Figure G.4 Inter-Storey Drifts of Three-Storey Building for All 45 Records at Design Level

Four-Storey Building –Vancouver, BC

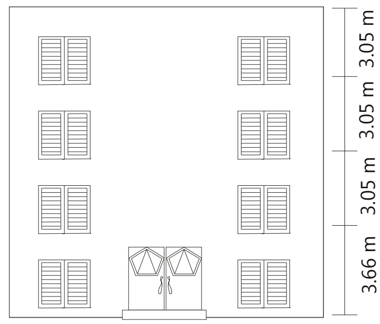


Figure G.5 Elevation View of Four-Storey Building

Table G.13 Seismic Weight Distribution for Four-Storey Building

Level	Storey Height (m)	Area (m^2)	Dead (kPa)	Snow (kPa)	Live (kPa)	Seismic Weight (kN)	Cumulative Seismic Weight (kN)
Roof	-	220	0.69	1.64	-	241.71	241.71
4	3.05	220	2.87	-	2.44	630.64	872.34
3	3.05	220	2.87	-	2.44	630.64	1502.98
2	3.05	220	2.87	-	2.44	630.64	2133.62
1	3.66	220	2.87	-	2.44	-	2133.62

Table G.14 Design Base Shear Distribution for Four-Storey Building

Storey	W_i (kN)	h_i (m)	W_i x h_i	F_x (kN)	T_x (kN)	N_x (kN)	Vf_x (kN)
Roof	241.7	12.81	3096	52.19	5.22	1.21	58.62
4	630.6	9.76	6155	103.75	10.37	4.49	118.62
3	630.6	6.71	4232	71.33	7.13	4.49	82.95
2	630.6	3.66	2308	38.91	3.89	4.49	47.29
1	-	-	-	-	-	-	-
Σ			15791	266			307

Table G.15 Design of Four-Storey Building Adjusted for Irregularity

Level	Shear (kN)	Framing (mm)	Sheathing (mm)	Fastener Spacing (mm)	S_v (kN/m)	S_r (kN/m)	Min L (m)	# walls[1]	Rounded # walls[1]
4	58.62	1.09	0.84	50	13.94	9.75	6.01	4.93	14
3	177.23	1.09	0.84	50	13.94	9.75	18.17	14.89	17
2	260.19	1.09	0.84	50	13.94	9.75	26.67	21.86	22
1	307.48	1.09	0.84	50	13.94	9.75	31.52	25.84	26

[1] 1220mm (4') wall segments

Table G.16 Design of Double Chord Studs of Four-Storey Building

Storey	Compression – shear (kN)	Compression – gravity (kN)	Compression – total (kN)	DCS Thickness (mm)	# DCS	Area DCS (mm^2)	DCS Pn (kN)
4	59.50	1.63	61.14	1.37	1	541.2	100.0
3	59.50	6.07	126.71	1.73	1	670.4	128.8
2	59.50	6.07	192.29	1.73	1.5	1005.6	193.2
1	71.40	6.07	269.76	2.46	2	1846.1	352.8

Table G.17 Inter-storey Drift and Stability Factor of Four-Storey Building

Level	h_s (mm)	Δ (mm)	Δ_{mx} (mm)	Drift (%)	θ_x
4	2750	5.9	24.9	0.91	0.022
3	2750	6.2	26.5	0.96	0.028
2	2750	6.1	25.8	0.94	0.032
1	3360	7.3	31.0	0.92	0.038

Table G.18 P-Δ Loads for Four-Storey Building

Level	P-Δ Area (m^2)	Cumulative Area (m^2)	LLRF	Reduced live load (kPa)	Gravity Load (kPa)	P_x (kN)
4	178.2	178.2	0.53	-	1.10	196.0
3	169.3	347.4	0.47	1.14	3.44	582.4
2	154.4	501.8	0.44	1.07	3.41	526.0
1	142.5	644.4	0.42	1.03	3.39	482.7

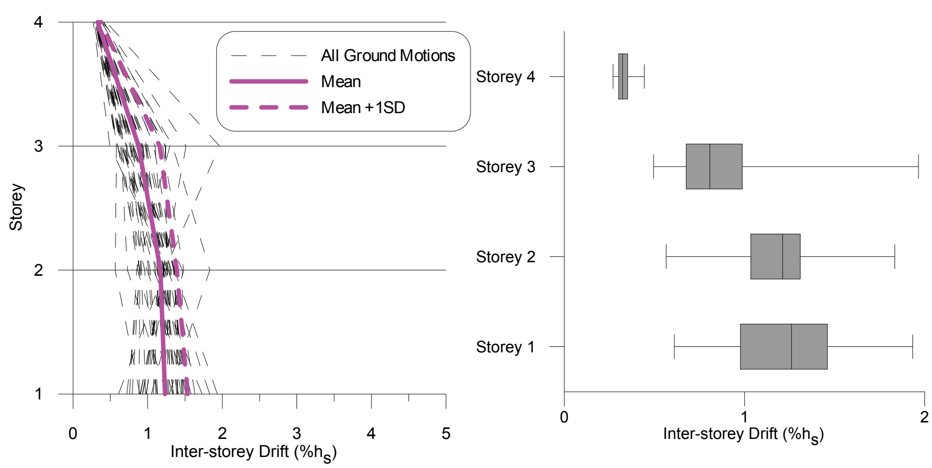


Figure G.6 Inter-Storey Drifts of Four-Storey Building for All 45 Records at Design Level

Five-Storey Building –Vancouver, BC

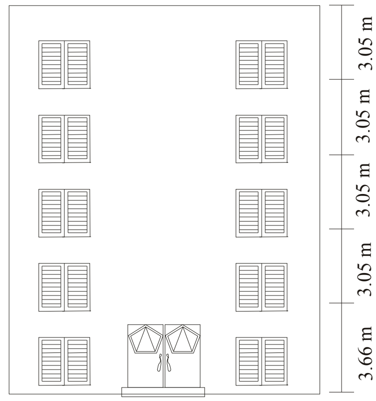


Figure G.7 Elevation View of Five-Storey Building

Table G.19 Seismic Weight Distribution for Five-Storey Building

Level	Storey Height (m)	Area (m^2)	Dead (kPa)	Snow (kPa)	Live (kPa)	Seismic Weight (kN)	Cumulative (kN)
Roof		220	0.69	1.64		241.71	241.71
5	3.05	220	2.87		2.44	630.64	872.34
4	3.05	220	2.87		2.44	630.64	1502.98
3	3.05	220	2.87		2.44	630.64	2133.62
2	3.05	220	2.87		2.44	630.64	2764.25
1	3.66	220	2.87		2.44		2764.25

Table G.20 Design Base Shear Distribution for Five-Storey Building

Storey	W_i (kN)	h_i (m)	W_i x h_i	F_x (kN)	T_x (kN)	N_x(kN)	Vf_x (kN)
Roof	241.7	15.86	3833	43.76	4.38	1.21	49.34
5	630.6	12.81	8078	92.21	9.22	4.49	105.92
4	630.6	9.76	6155	70.25	7.03	4.49	81.77
3	630.6	6.71	4232	48.30	4.83	4.49	57.62
2	630.6	3.66	2308	26.35	2.63	4.49	33.47
1	-	-	-	-	-	-	-
Σ			24607	281			328

Table G.21 Design of Five-Storey Building Adjusted for Irregularity

Level	Shear (kN)	Framing (mm)	Sheathing (mm)	Fastener Spacing (mm)	S_v (kN/m)	S_r (kN/m)	Min L (m)	# walls[1]	Rounded # walls[1]
5	49.34	1.09	0.84	50	13.94	9.75	5.06	4.15	13
4	155.26	1.09	0.84	50	13.94	9.75	15.92	13.05	16
3	237.03	1.09	0.84	50	13.94	9.75	24.30	19.92	20
2	294.66	1.09	0.84	50	13.94	9.75	30.21	24.76	25
1	328.13	1.09	0.84	50	13.94	9.75	33.64	27.57	28

[1] 1220mm (4') wall segments

Table G.22 Design of Double Chord Studs of Five-Storey Building

Storey	Compression – shear (kN)	Compression – gravity (kN)	Compression – total (kN)	DCS Thickness (mm)	# DCS	Area DCS (mm^2)	DCS Pn (kN)
5	59.50	1.61	61.11	1.37	1	541.2	100.0
4	59.50	5.99	126.61	1.73	1	670.4	128.8
3	59.50	5.99	192.10	1.73	1.5	1005.6	193.2
2	59.50	5.99	257.60	2.46	1.5	1384.6	264.6
1	71.40	5.99	334.99	2.46	2	1846.1	352.8

Table G.23 Inter-storey Drift and Stability Factor of Five-Storey Building

Level	h_s (mm)	Δ (mm)	Δ_{mx} (mm)	Drift (%)	θ_x
5	2750	5.8	24.8	0.90	0.026
4	2750	6.2	26.3	0.96	0.032
3	2750	6.1	25.8	0.94	0.035
2	2750	5.9	25.1	0.91	0.039
1	3360	7.3	31.0	0.92	0.046

Table G.24 P-Δ Loads for Five-Storey Building

Level	P-Δ Area (m^2)	Cumulative Area (m^2)	LLRF	Reduced live load (kPa)	Gravity Load (kPa)	P_x (kN)
5	181.1	181.1	0.53	0.00	1.10	199.2
4	172.2	353.4	0.47	1.14	3.44	592.3
3	160.3	513.7	0.44	1.07	3.40	545.9
2	145.5	659.2	0.42	1.03	3.38	492.5
1	136.6	795.8	0.41	1.00	3.37	460.5

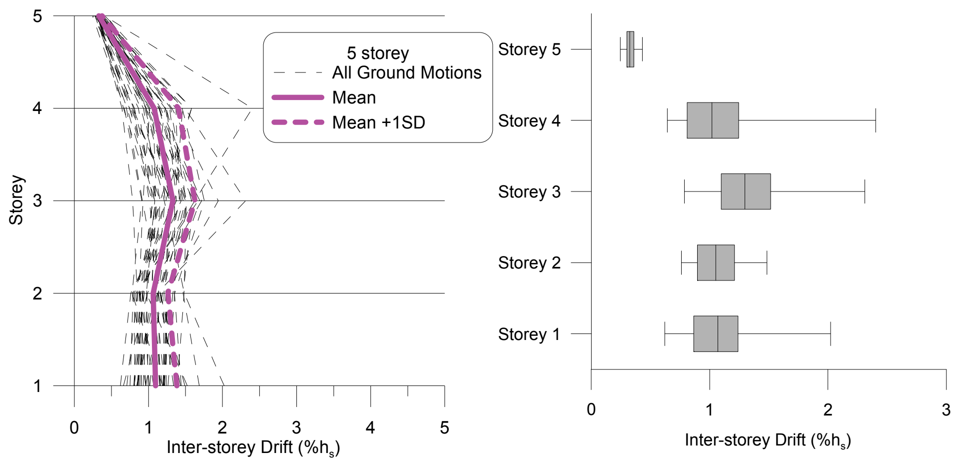


Figure G.8 Inter-Storey Drifts of Five-Storey Building for All 45 Records at Design Level

Six-Storey Building –Vancouver, BC

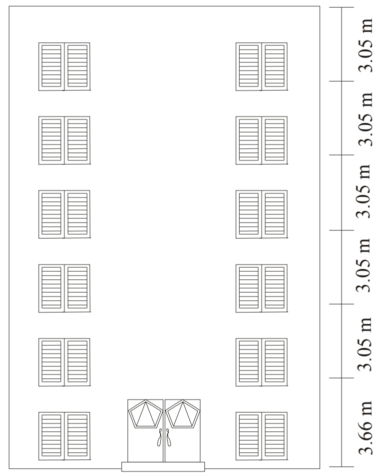


Figure G.9 Elevation View of Six-Storey Building

Table G.25 Seismic Weight Distribution for Six-Storey Building

Level	Storey Height (m)	Area (m^2)	Dead (kPa)	Snow (kPa)	Live (kPa)	Seismic Weight (kN)	Cumulative (kN)
Roof		220	0.69	1.64		241.71	241.71
6	3.05	220	2.87		2.44	630.64	872.34
5	3.05	220	2.87		2.44	630.64	1502.98
4	3.05	220	2.87		2.44	630.64	2133.62
3	3.05	220	2.87		2.44	630.64	2764.25
2	3.05	220	2.87		2.44	630.64	3394.89
1	3.66	220	2.87		2.44		3394.89

Table G.26 Design Base Shear Distribution for Six-Storey Building

Storey	W_i (kN)	h_i (m)	W_i x h_i	F_x (kN)	T_x (kN)	N_x(kN)	Vf_x (kN)
Roof	241.7	18.91	4571	37.51	3.75	1.21	42.47
6	630.6	15.86	10002	82.09	8.21	4.49	94.79
5	630.6	12.81	8078	66.30	6.63	4.49	77.43
4	630.6	9.76	6155	50.52	5.05	4.49	60.06
3	630.6	6.71	4232	34.73	3.47	4.49	42.70
2	630.6	3.66	2308	18.94	1.89	4.49	25.33
1	-	-	-	-	-	-	-
Σ			35346	290			343

Table G.27 Design of Six-Storey Building Adjusted for Irregularity

Level	Shear (kN)	Framing (mm)	Sheathing (mm)	Fastener Spacing (mm)	S_v (kN/m)	S_r (kN/m)	Min L (m)	# walls[1]	Rounded # walls[1]
6	42.47	1.09	0.84	50	13.94	9.75	4.35	3.57	12
5	137.26	1.09	0.84	50	13.94	9.75	14.07	11.53	15
4	214.69	1.09	0.84	50	13.94	9.75	22.01	18.04	19
3	274.75	1.09	0.84	50	13.94	9.75	28.17	23.09	24
2	317.45	1.09	0.84	50	13.94	9.75	32.54	26.67	27
1	342.78	1.09	0.84	50	13.94	9.75	35.14	28.80	29

[1] 1220mm (4') wall segments

Table G.28 Design of Double Chord Studs of Six-Storey Building

Storey	Compression – shear (kN)	Compression – gravity (kN)	Compression – total (kN)	DCS Thickness (mm)	# DCS	Area DCS (mm^2)	DCS Pn (kN)
6	59.50	1.61	61.11	1.37	1	541.2	100.0
5	59.50	5.99	126.61	1.73	1	670.4	128.8
4	59.50	5.99	192.10	1.73	1.5	1005.6	193.2
3	59.50	5.99	257.60	2.46	1.5	1384.6	264.6
2	59.50	5.99	323.09	2.46	2	1846.1	352.8
1	71.40	5.99	400.48	2.46	2.5	2307.7	441.0

Table G.29 Inter-storey Drift and Stability Factor of Six-Storey Building

Level	h_s (mm)	Δ (mm)	Δ_{mx} (mm)	Drift (%)	θ_x
6	2750	5.8	24.7	0.90	0.030
5	2750	6.1	26.1	0.95	0.036
4	2750	6.0	25.7	0.93	0.038
3	2750	5.9	25.1	0.91	0.042
2	2750	5.8	24.7	0.90	0.046
1	3360	7.2	30.5	0.91	0.053

Table G.30 P-Δ Loads for Six--Storey Building

Level	P-Δ Area (m^2)	Cumulative Area (m^2)	LLRF	Reduced live load (kPa)	Gravity Load (kPa)	P_x (kN)
6	184.1	184.1	0.53	0.00	1.10	202.5
5	175.2	359.3	0.47	1.13	3.44	602.2
4	163.3	522.6	0.44	1.07	3.40	555.8
3	148.5	671.1	0.42	1.03	3.38	502.3
2	139.6	810.6	0.41	1.00	3.37	470.3
1	133.6	944.3	0.40	0.98	3.36	449.0

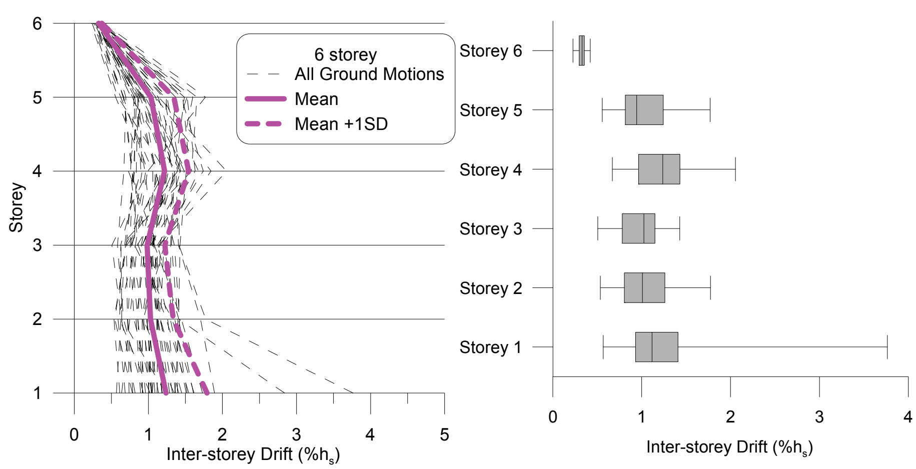


Figure G.10 Inter-Storey Drifts of Six-Storey Building for All 45 Records at Design Level

Seven-Storey Building –Vancouver, BC

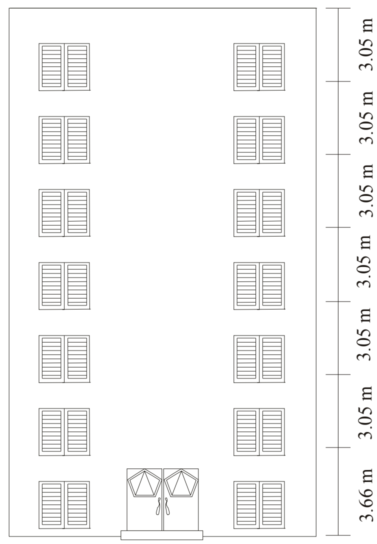


Figure G.11 Elevation View of Seven-Storey Building

Table G.31 Seismic Weight Distribution for Seven-Storey Building

Level	Storey Height (m)	Area (m^2)	Dead (kPa)	Snow (kPa)	Live (kPa)	Seismic Weight (kN)	Cumulative (kN)
Roof		220	0.69	1.64	0	241.71	241.71
7	3.05	220	2.87		2.44	630.64	872.34
6	3.05	220	2.87		2.44	630.64	1502.98
5	3.05	220	2.87		2.44	630.64	2133.62
4	3.05	220	2.87		2.44	630.64	2764.25
3	3.05	220	2.87		2.44	630.64	3394.89
2	3.05	220	2.87		2.44	630.64	4025.53
1	3.66	220	2.87		2.44		4025.53

Table G.32 Design Base Shear Distribution for Seven-Storey Building

Storey	W_i (kN)	h_i (m)	W_i x h_i	F_x (kN)	T_x (kN)	N_x(kN)	Vf_x (kN)
Roof	241.7	21.96	5308	32.0	3.2	1.21	36.4
7	630.6	18.91	11925	71.8	7.2	4.49	83.5
6	630.6	15.86	10002	60.2	6.0	4.49	70.8
5	630.6	12.81	8078	48.7	4.9	4.49	58.0
4	630.6	9.76	6155	37.1	3.7	4.49	45.3
3	630.6	6.71	4232	25.5	2.5	4.49	32.5
2	630.6	3.66	2308	13.9	1.4	4.49	19.8
1	-	-	-	-	-	-	-
Σ			48008	289.2			346.3

Table G.33 Design of Seven-Storey Building Adjusted for Irregularity

Level	Shear (kN)	Framing (mm)	Sheathing (mm)	Fastener Spacing (mm)	S_v (kN/m)	S_r (kN/m)	Min L (m)	# walls[1]	Rounded # walls[1]
7	36.38	1.09	0.84	50	13.94	9.75	3.73	3.06	11
6	119.89	1.09	0.84	50	13.94	9.75	12.29	10.07	14
5	190.65	1.09	0.84	50	13.94	9.75	19.54	16.02	17
4	248.67	1.09	0.84	50	13.94	9.75	25.49	20.90	21
3	293.94	1.09	0.84	50	13.94	9.75	30.13	24.70	25
2	326.48	1.09	0.84	50	13.94	9.75	33.47	27.43	28
1	346.26	1.09	0.84	50	13.94	9.75	35.50	29.10	30

[1] 1220mm (4’) wall segments

Table G.34 Design of Double Chord Studs of Seven-Storey Building

Storey	Compression – shear (kN)	Compression – gravity (kN)	Compression – total (kN)	DCS Thickness (mm)	# DCS	Area DCS (mm^2)	DCS Pn (kN)
7	59.5	1.6	61.1	1.37	1	541.2	100.0
6	59.5	6.1	126.7	1.73	1	670.4	128.8
5	59.5	6.1	192.2	1.73	1.5	1005.6	193.2
4	59.5	6.1	257.8	2.46	1.5	1384.6	264.6
3	59.5	6.1	323.3	2.46	2	1846.1	352.8
2	59.5	6.1	388.9	2.46	2.5	2307.7	441.0
1	71.4	6.1	466.3	2.46	3	2769.2	529.2

Table G.35 Inter-storey Drift and Stability Factor of Seven-Storey Building

Level	h_s (mm)	Δ (mm)	Δ_{mx} (mm)	Drift (%)	θ_x
7	2750	5.8	24.6	0.89	0.035
6	2750	6.1	25.9	0.94	0.040
5	2750	6.0	25.7	0.93	0.043
4	2750	5.9	25.2	0.91	0.046
3	2750	5.8	24.7	0.90	0.050
2	2750	5.8	24.5	0.89	0.054
1	3360	7.1	30.2	0.90	0.061

Table G.36 P-Δ Loads for Seven-Storey Building

Level	P-Δ Area (m^2)	Cumulative Area (m^2)	LLRF	Reduced live load (kPa)	Gravity Load (kPa)	P_x (kN)
7	187.1	187.1	0.53	0	1.10	205.8
6	178.2	365.2	0.46	1.13	3.44	612.1
5	169.3	534.5	0.44	1.06	3.40	575.7
4	157.4	691.9	0.42	1.02	3.38	532.1
3	145.5	837.4	0.41	1.00	3.37	490.0
2	136.6	974.0	0.40	0.98	3.36	458.7
1	130.7	1104.6	0.39	0.96	3.35	437.8

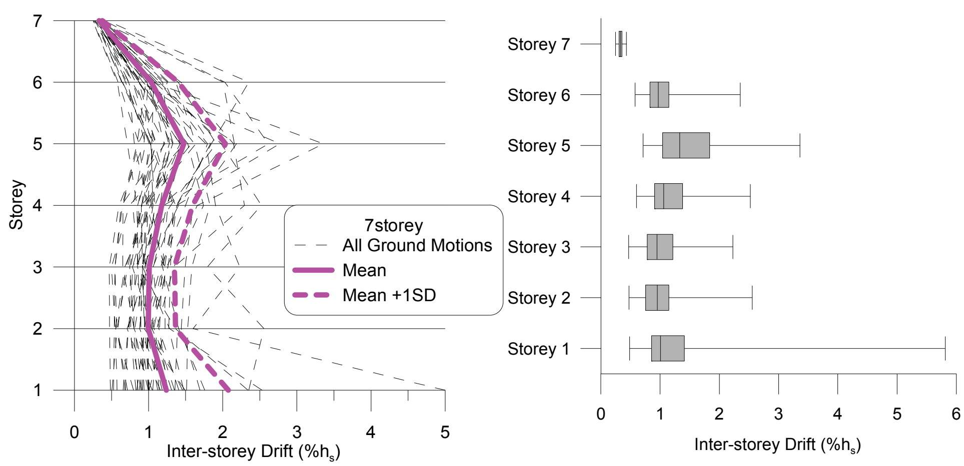


Figure G.12 Inter-Storey Drifts of Seven-Storey Building for All 45 Records at Design Level

APPENDIX H

HYSTERESIS AND TIME HISTORY FOR BUILDINGS SUBJECTED TO CM GROUND MOTIONS – PHASE I

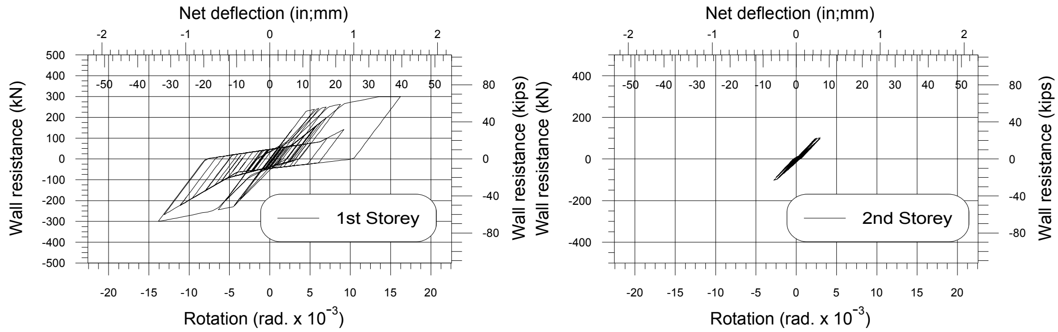


Figure H.1 Hysteresis for Each Storey, CM Earthquake Record, Two-Storey Building

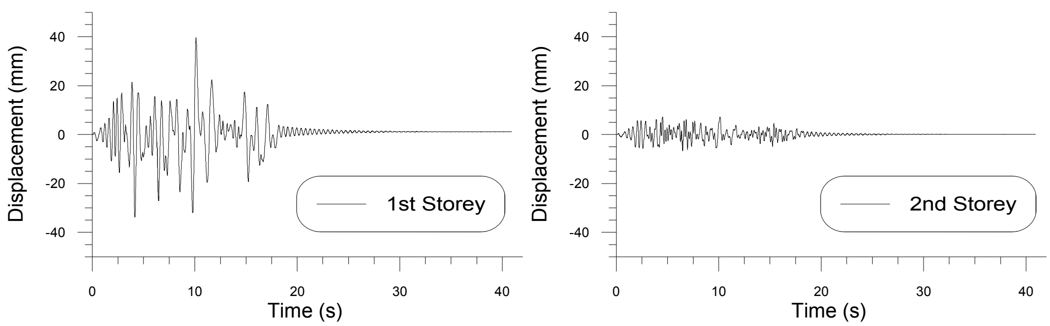


Figure H.2 Time History Showing Displacement Vs. Time for Each Storey, CM Earthquake Record, Two-Storey Building

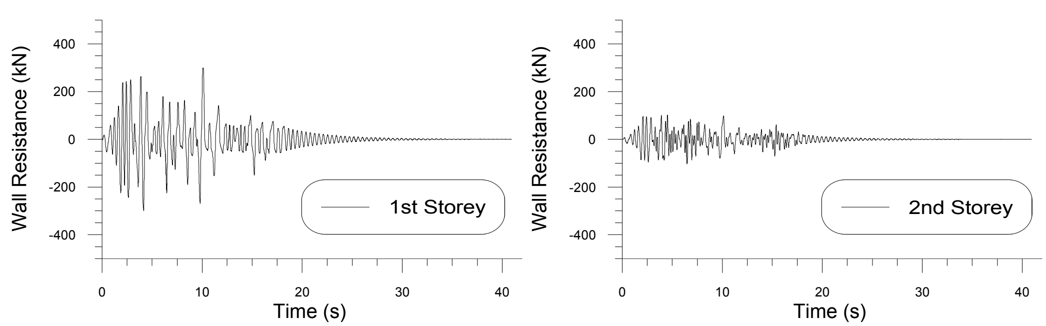


Figure H.3 Time History Showing Resistance Vs. Time for Each Storey, CM Earthquake Record,

Two-Storey Building

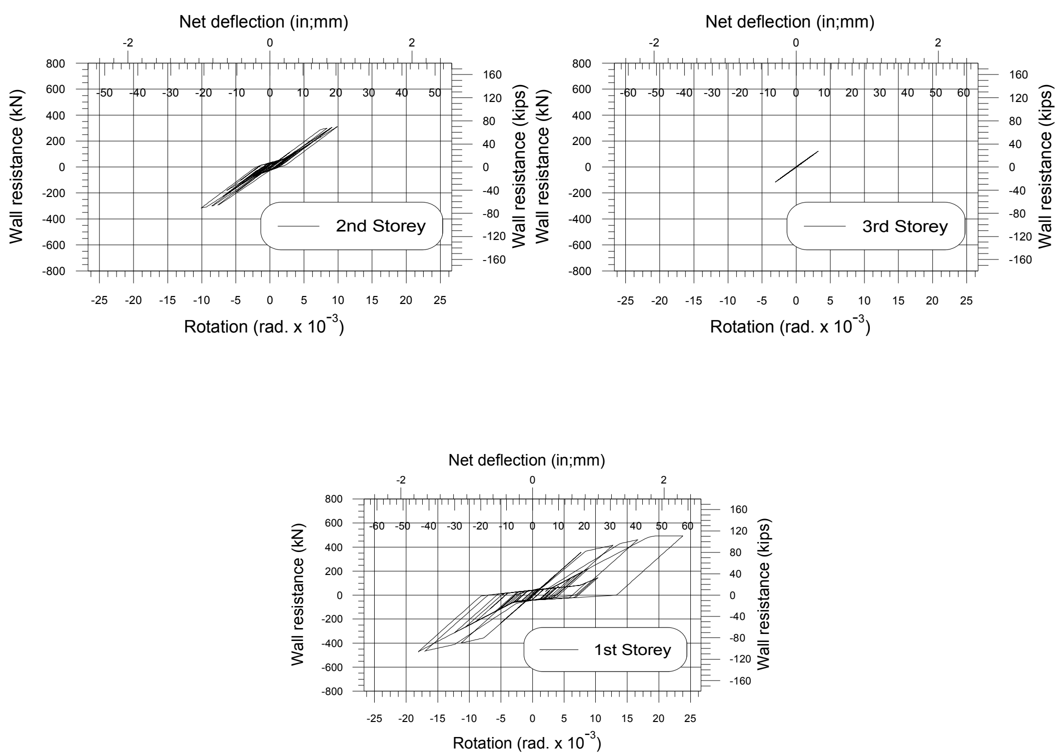


Figure H.4 Hysteresis for Each Storey, CM Earthquake Record, Three-Storey Building

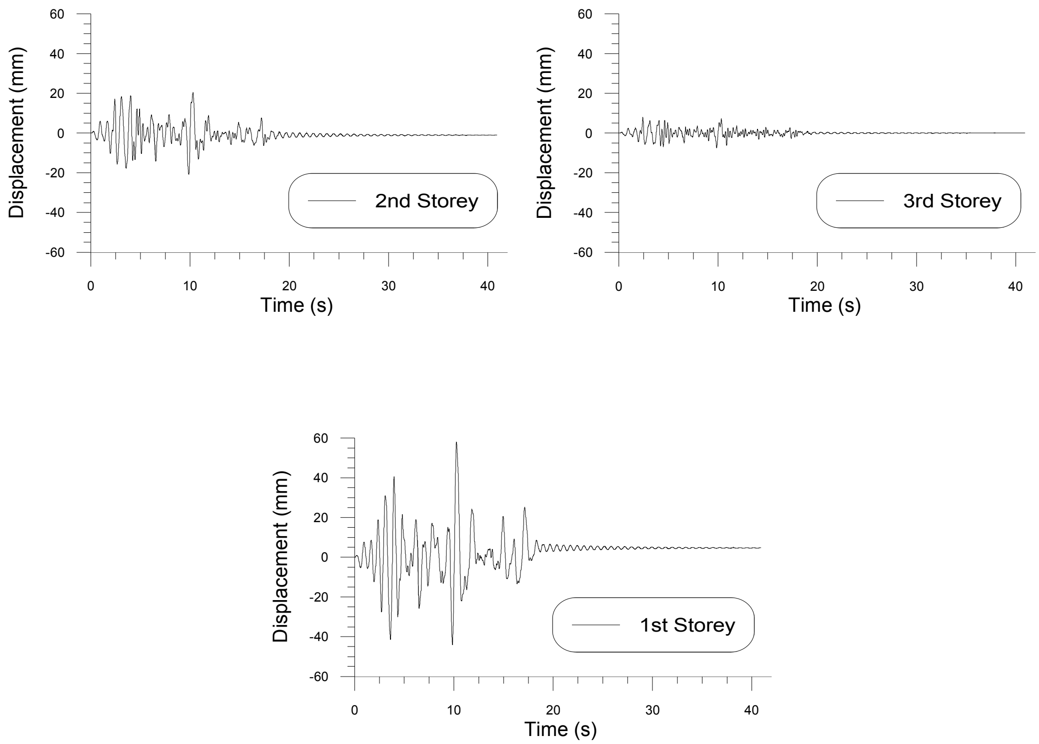


Figure H.5 Time History Showing Displacement Vs. Time for Each Storey, CM Earthquake Record, Three-Storey Building

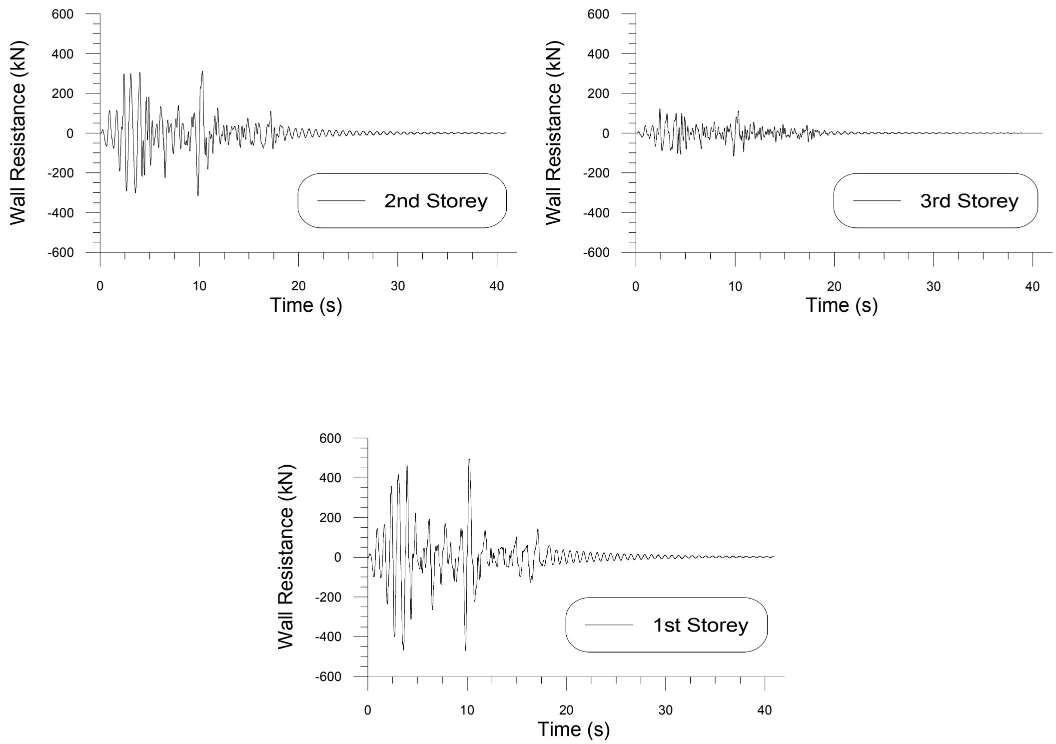


Figure H.6 Time History Showing Resistance Vs. Time for Each Storey, CM Earthquake Record, Three-Storey Building

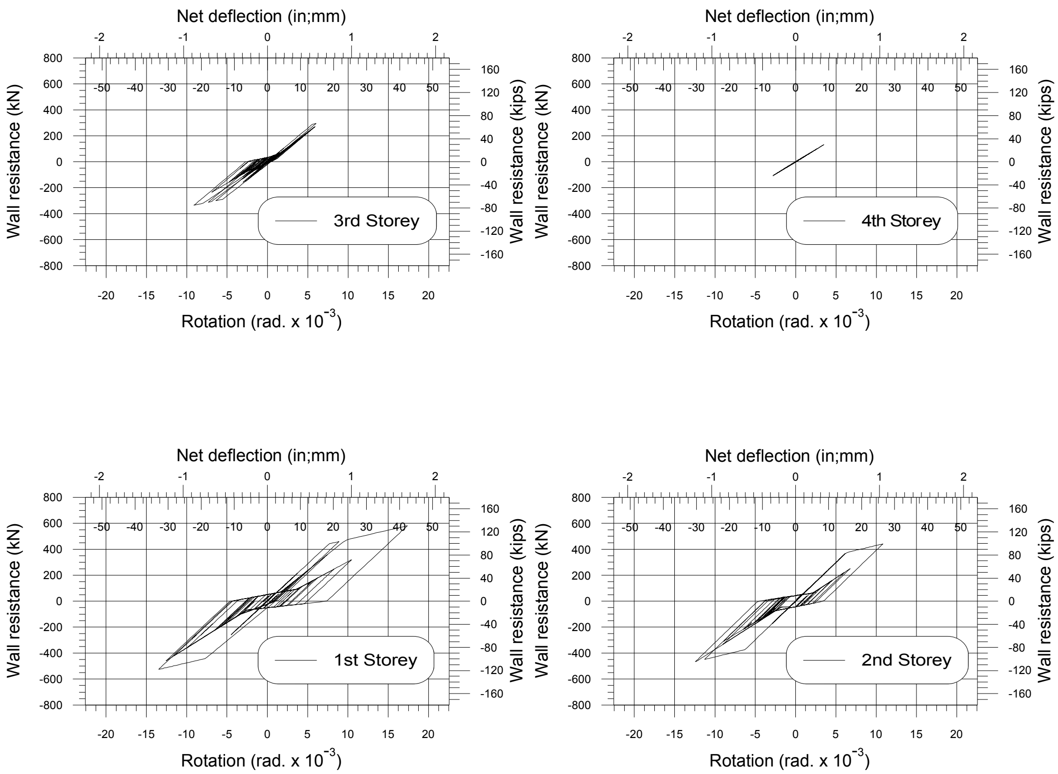


Figure H.7 Hysteresis for Each Storey, CM Earthquake Record, Four-Storey Building

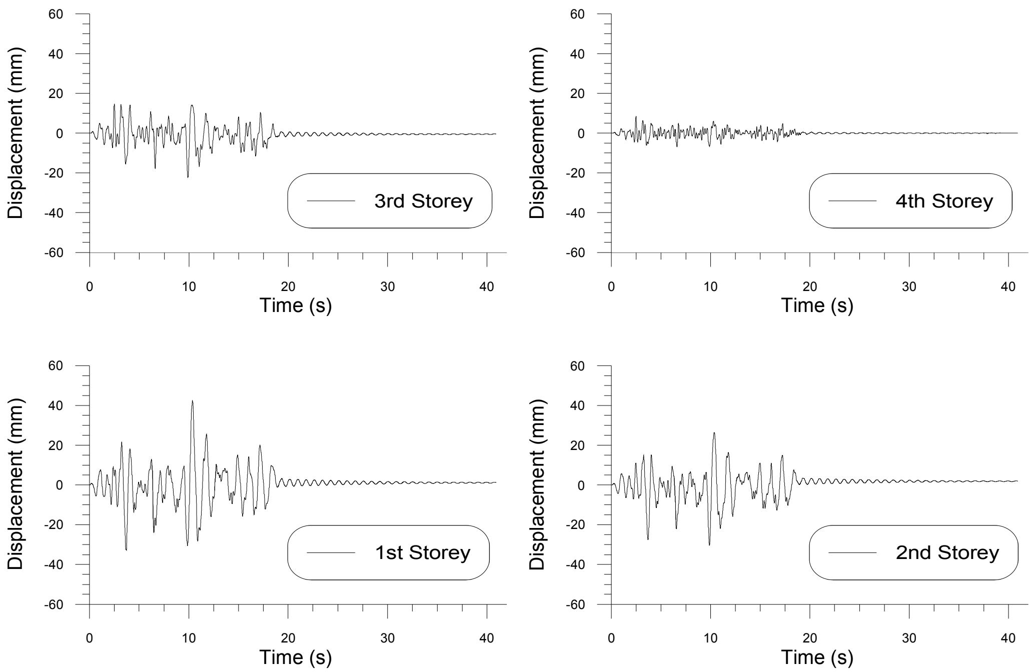


Figure H.8 Time History Showing Displacement Vs. Time for Each Storey, CM Earthquake Record, Four-Storey Building

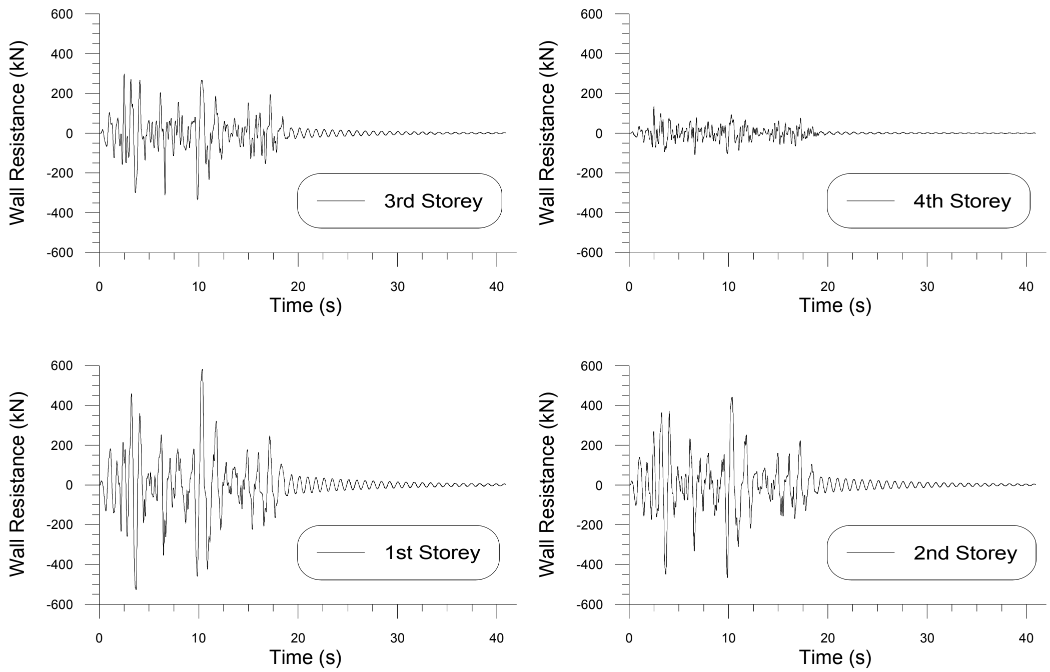


Figure H.9 Time History Showing Resistance Vs. Time for Each Storey, CM Earthquake Record, Four-Storey Building

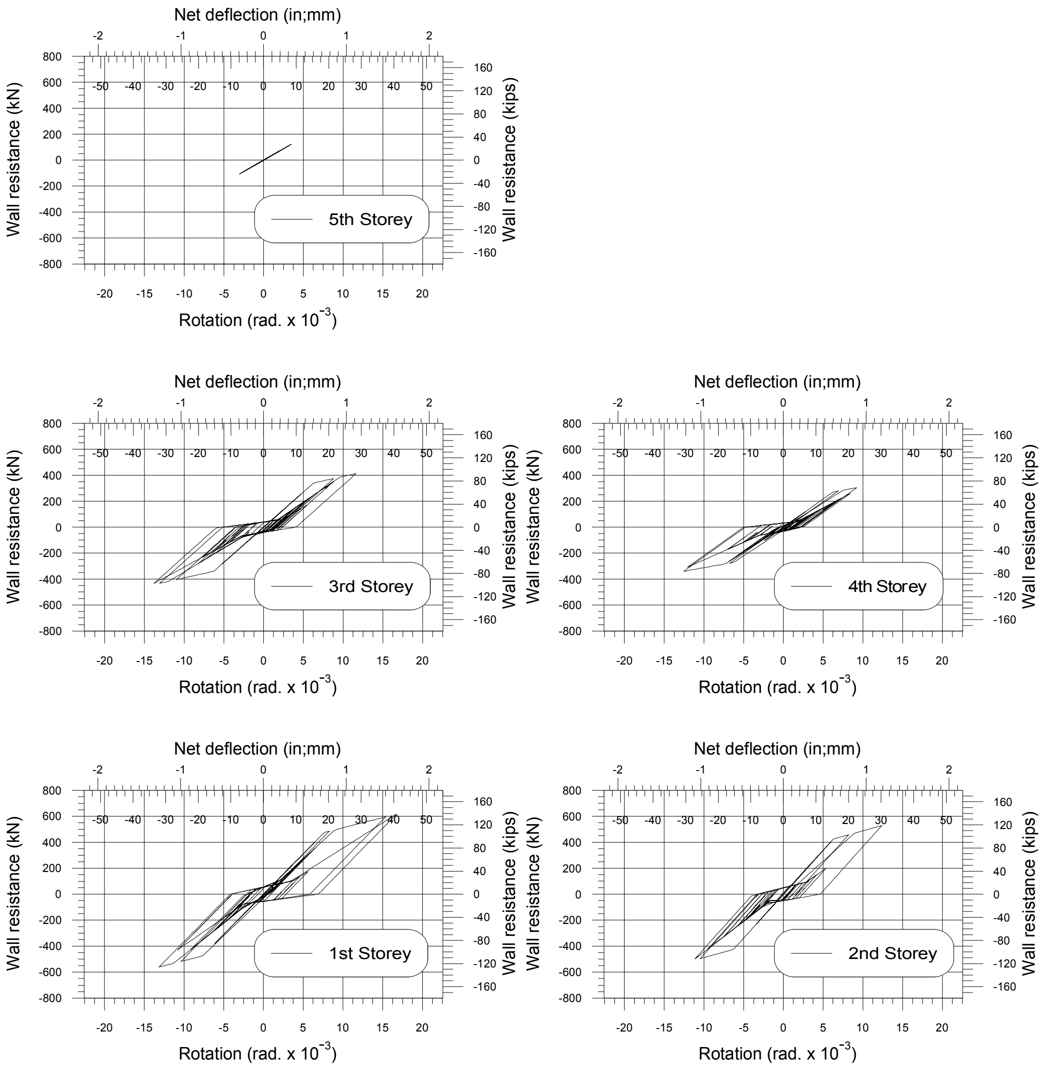


Figure H.10 Hysteresis for Each Storey, CM Earthquake Record, Five-Storey Building

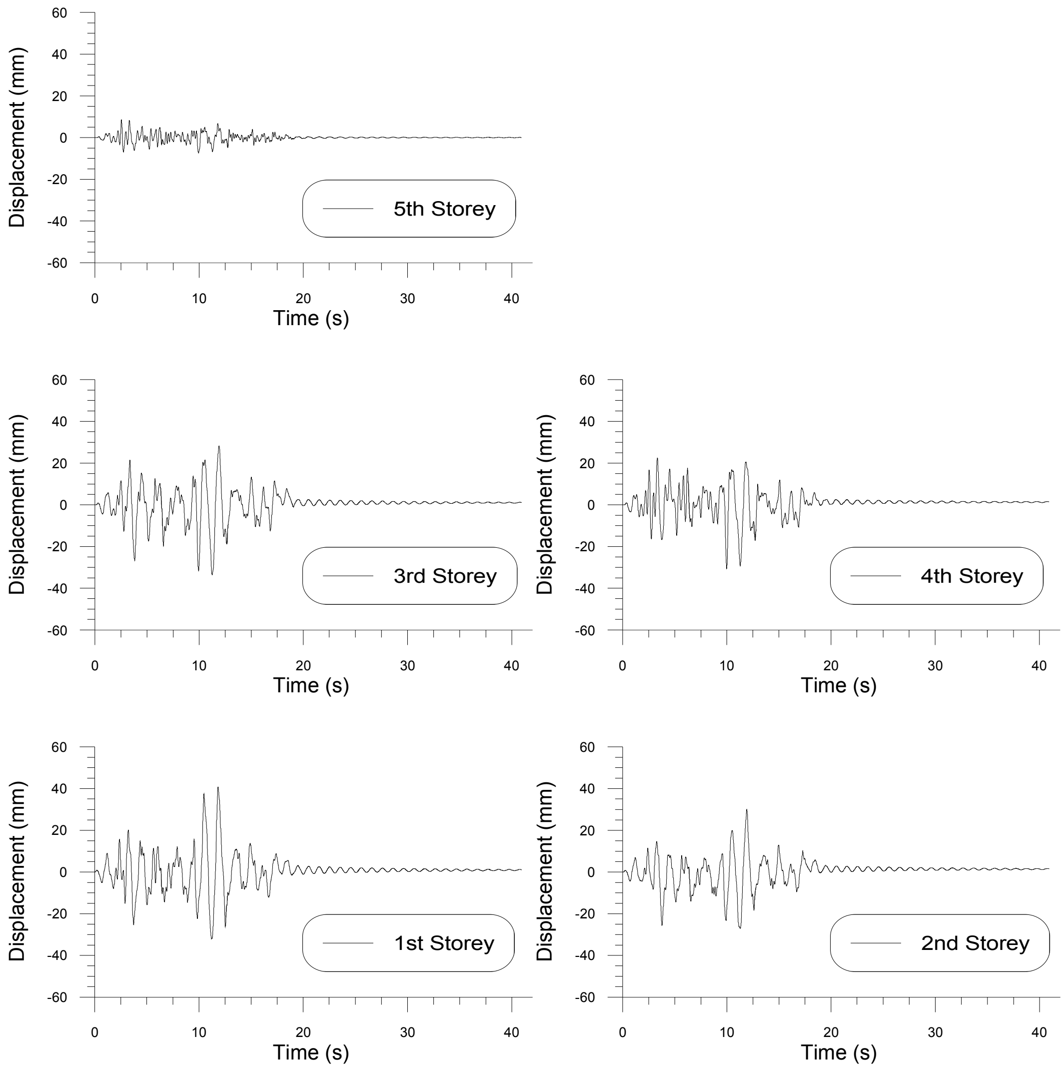


Figure H.11 Time History Showing Displacement Vs. Time for Each Storey, CM Earthquake Record, Five-Storey Building

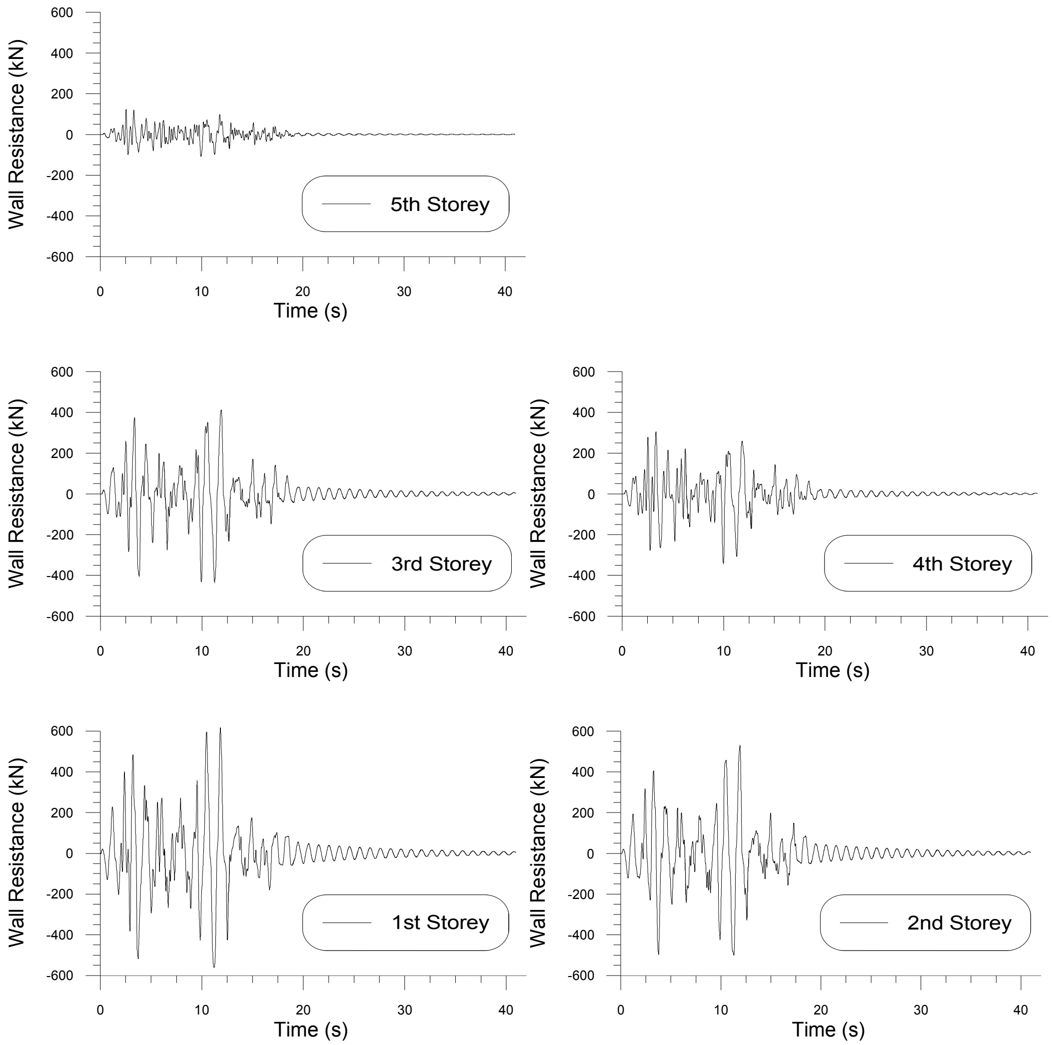


Figure H.12 Time History Showing Resistance Vs. Time for Each Storey, CM Earthquake Record, Five-Storey Building

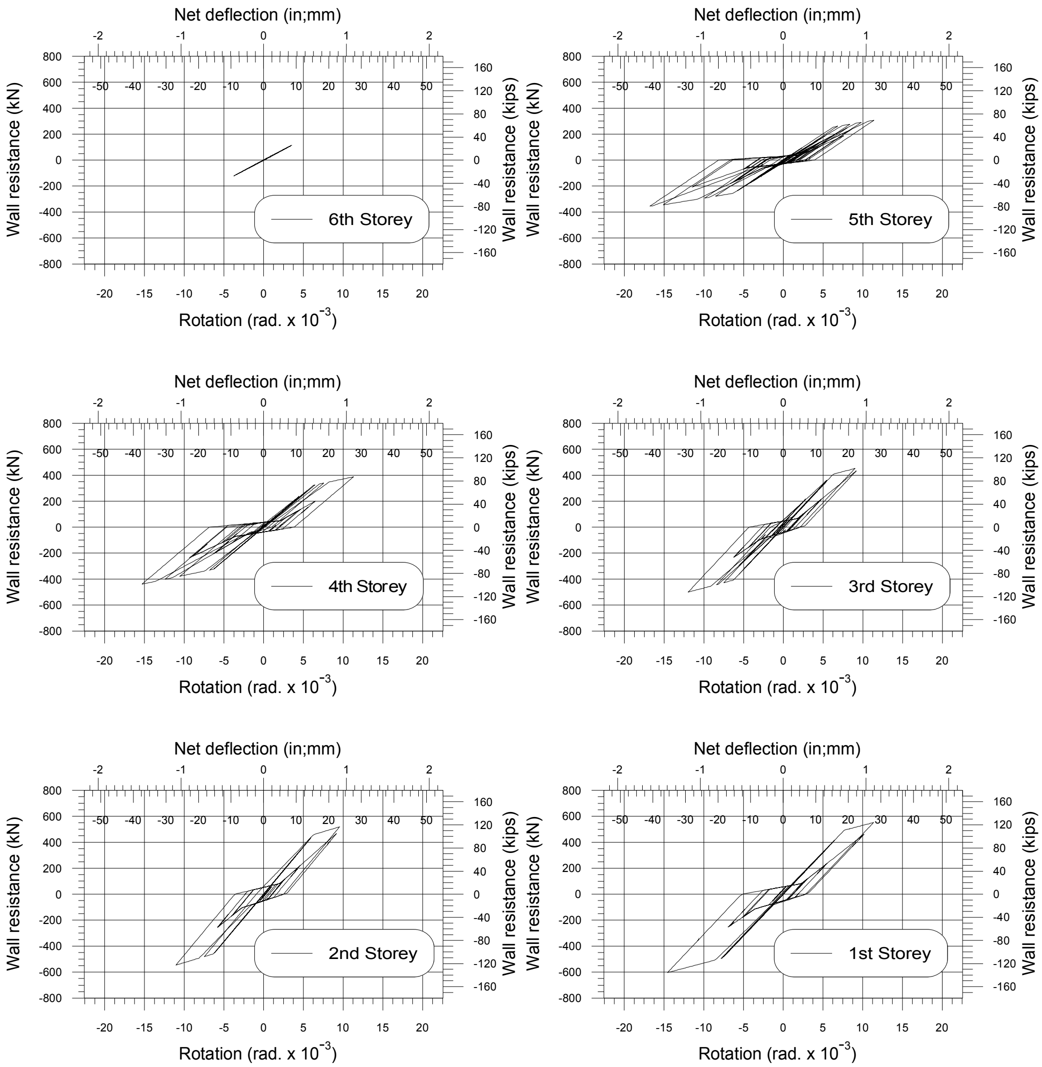


Figure H.13 Hysteresis for Each Storey, CM Earthquake Record, Six-Storey Building

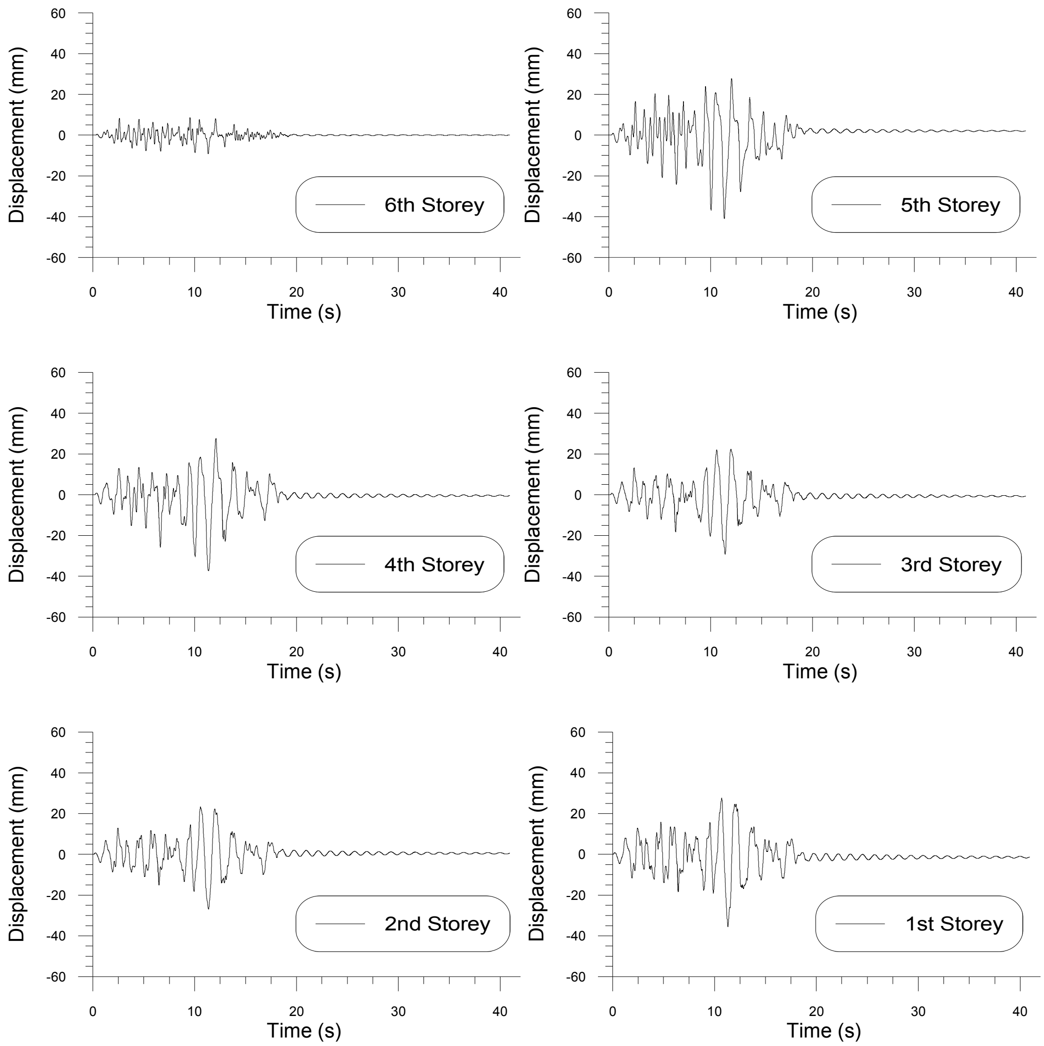


Figure H.14 Time History Showing Displacement Vs. Time for Each Storey, CM Earthquake Record, Six-Storey Building

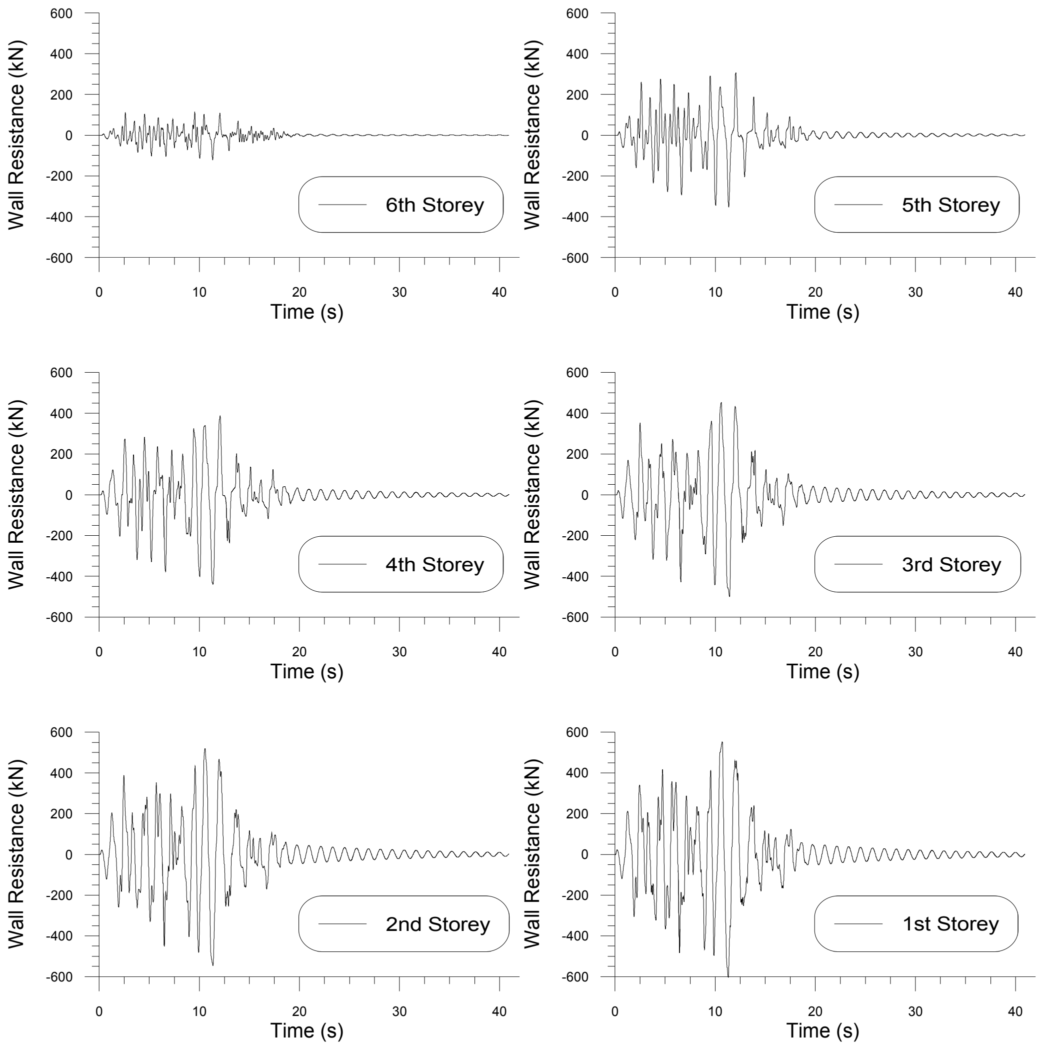


Figure H.15 Time History Showing Resistance Vs. Time for Each Storey, CM Earthquake Record, Six-Storey Building

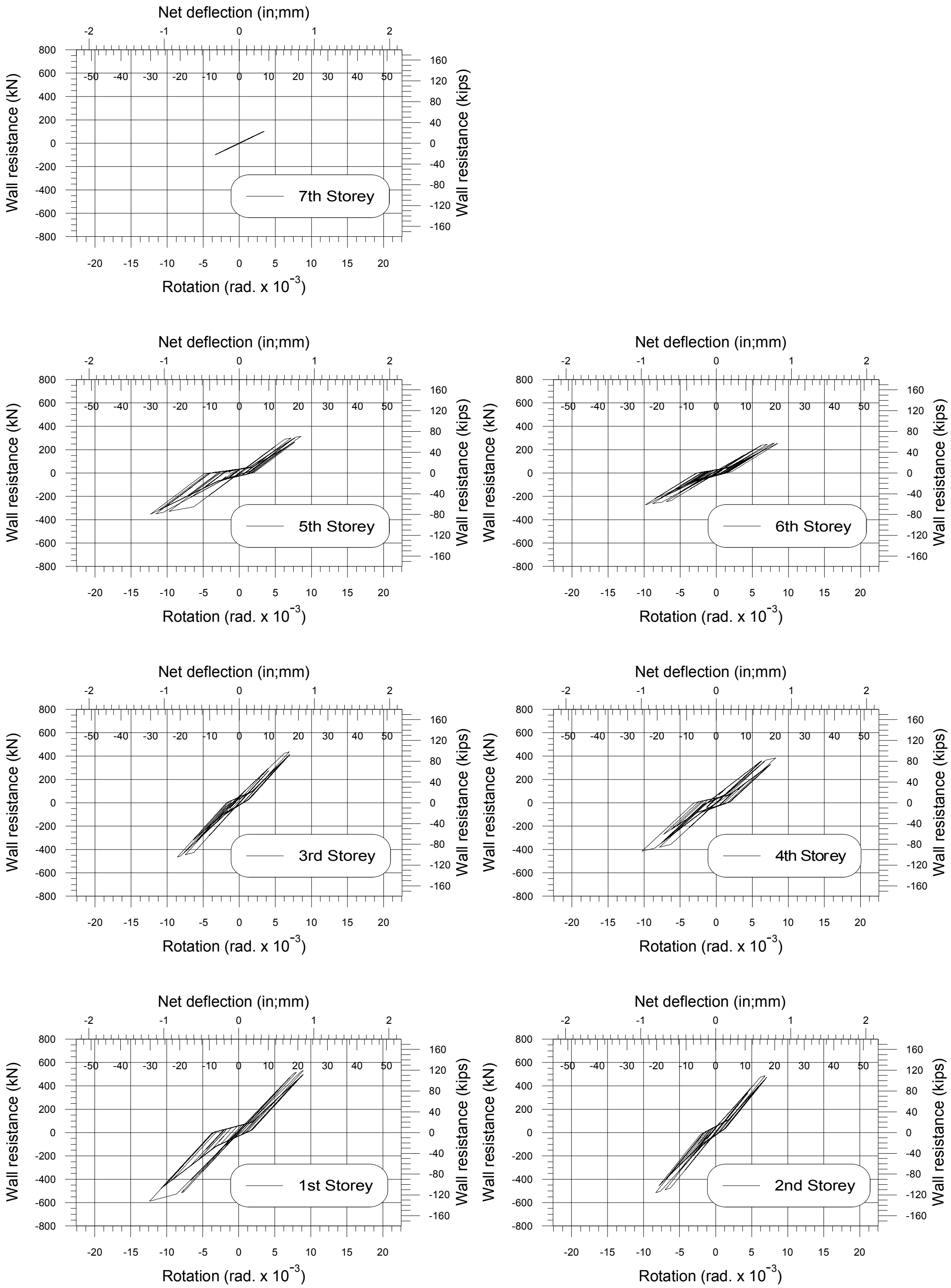


Figure H.16 Hysteresis for Each Storey, CM Earthquake Record, Seven-Storey Building

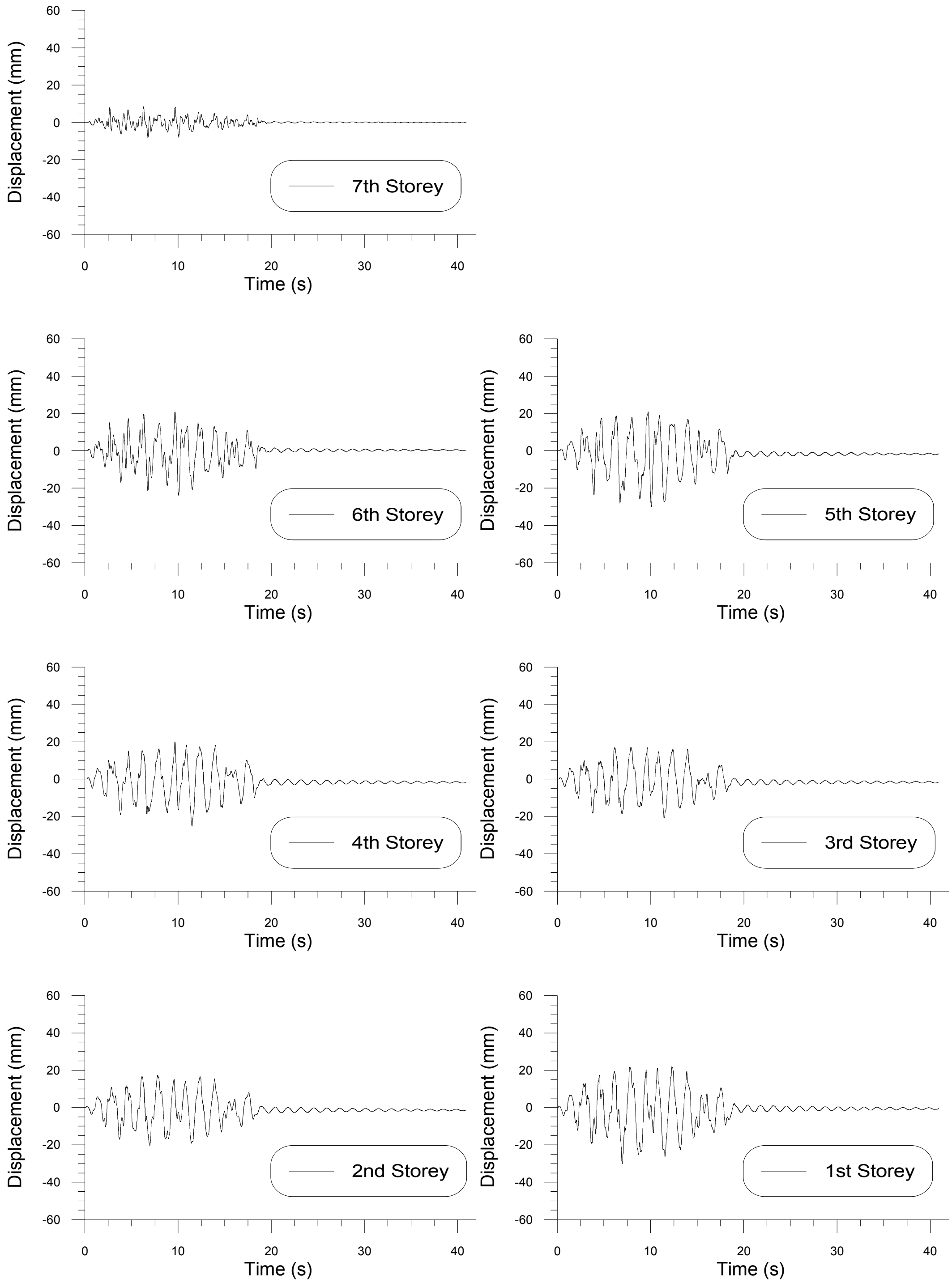


Figure H.17 Time History Showing Displacement Vs. Time for Each Storey, CM Earthquake Record, Seven-Storey Building

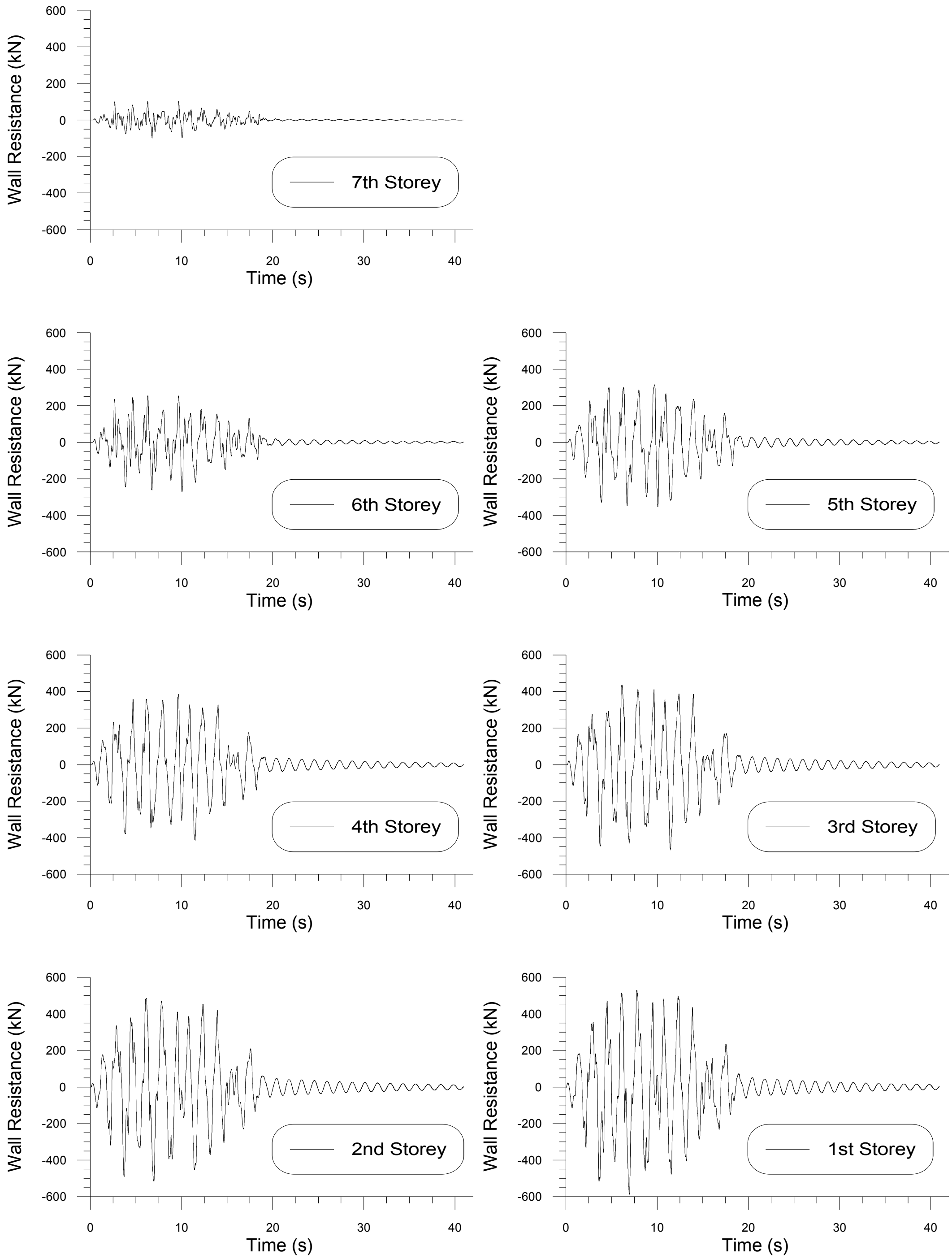


Figure H.18 Time History Showing Resistance Vs. Time for Each Storey, CM Earthquake Record, Seven-Storey Building

Appendix I

Phase I –

FEMA P695 Summary:

Pushover and IDA Analyses

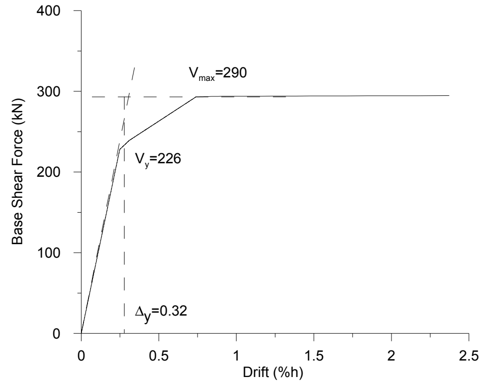


Figure I.1 Pushover Curve for Two-Storey Building

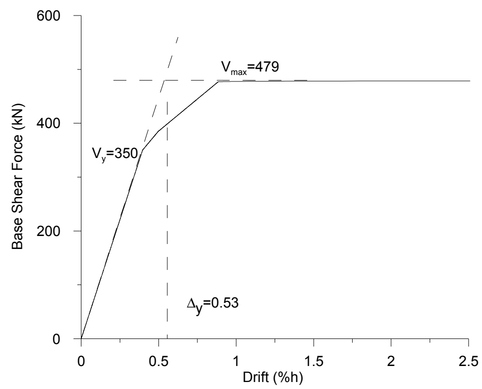


Figure I.2 Pushover Curve for Three-Storey Building

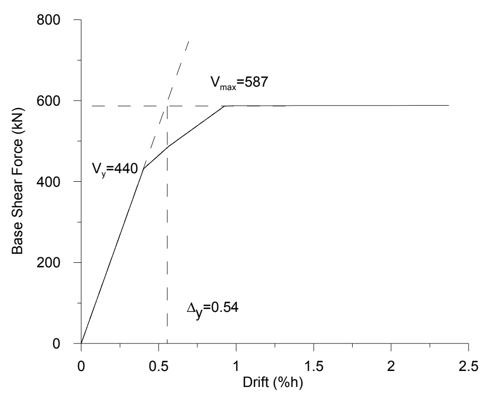


Figure I.3 Pushover Curve for Four-Storey Building

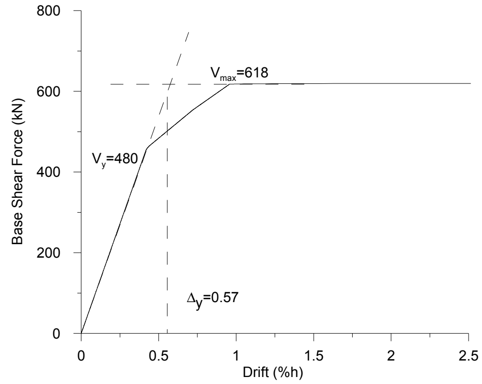


Figure I.4 Pushover Curve for Five-Storey Building

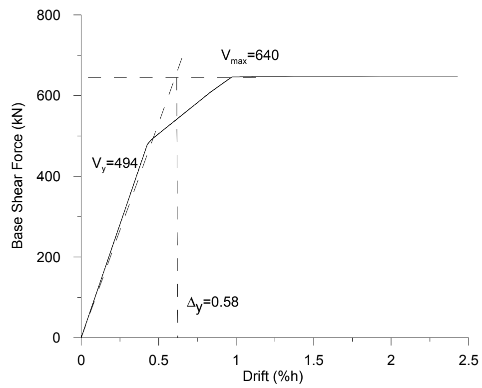


Figure I.5 Pushover Curve for Six-Storey Building

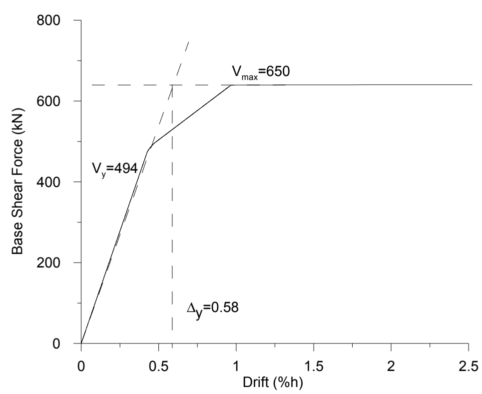


Figure I.6 Pushover Curve for Seven-Storey Building

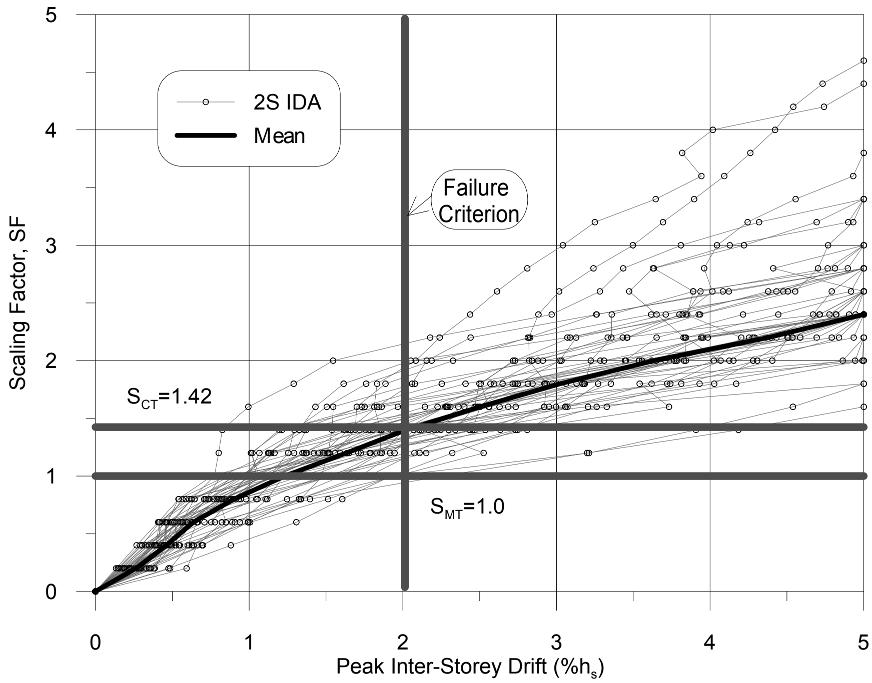


Figure I.7 IDA Curves for 45 Ground Motions (Two-Storey Building)

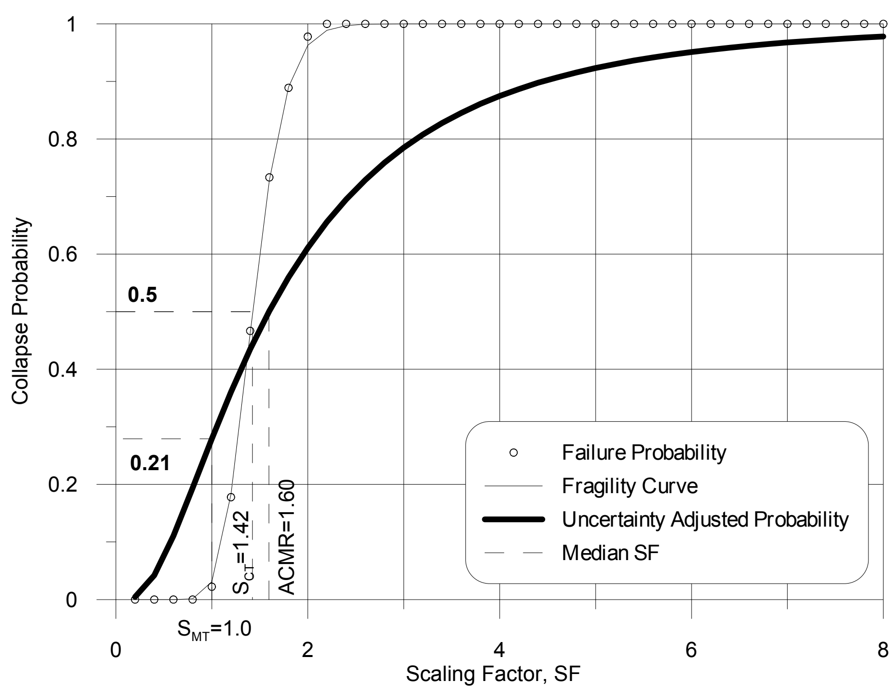


Figure I.8 Fragility Curve for Two-Storey Building

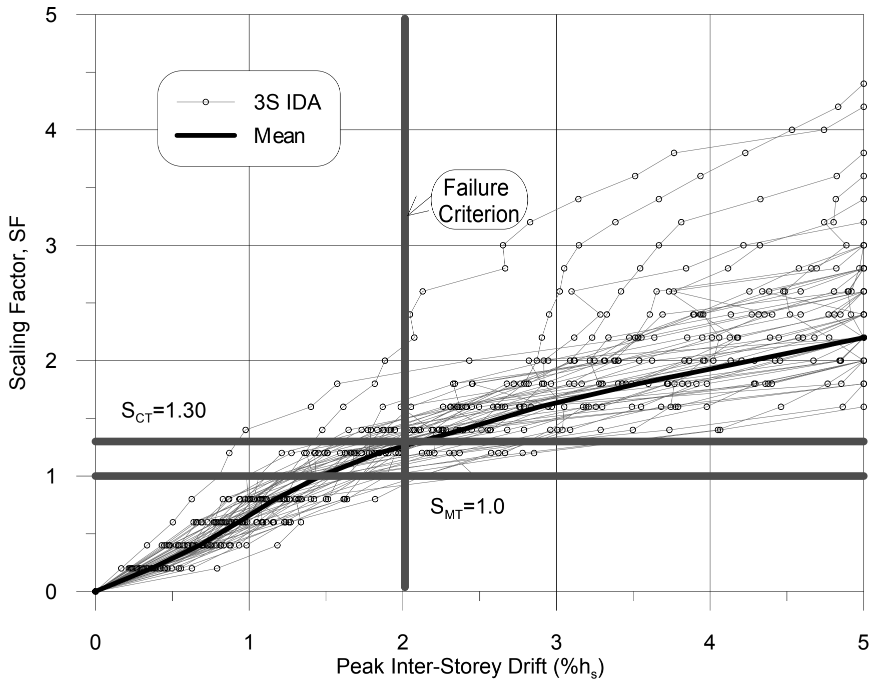


Figure I.9 IDA Curves for 45 Ground Motions (Three-Storey Building)

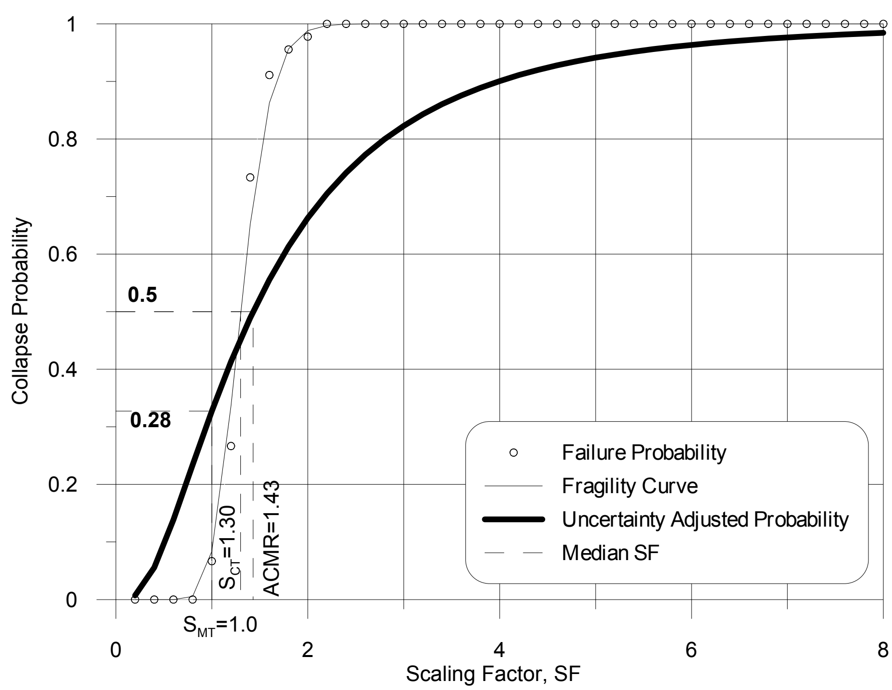


Figure I.10 Fragility Curve for Three-Storey Building

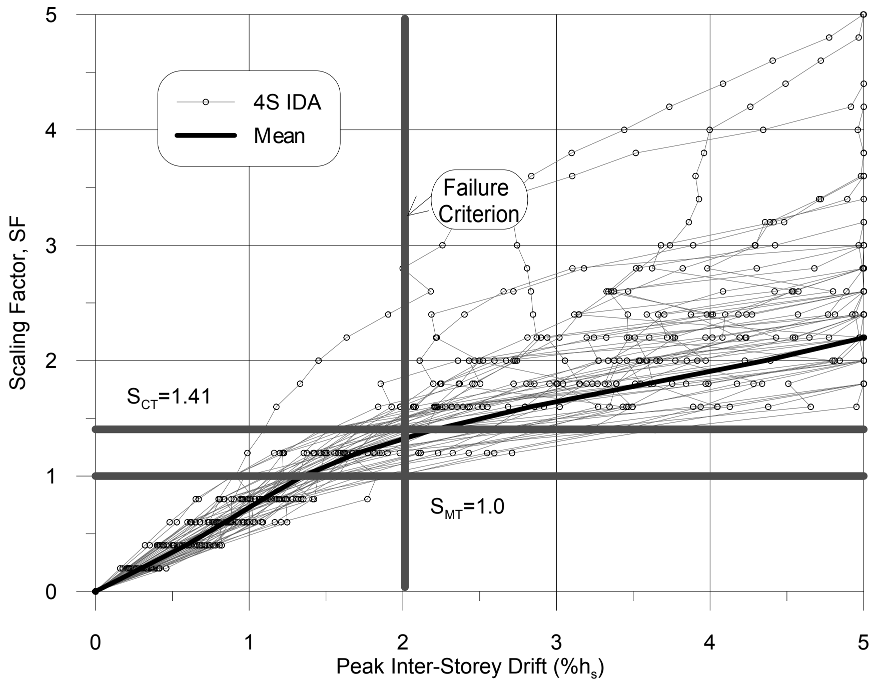


Figure I.11 IDA Curves for 45 Ground Motions (Four-Storey Building)

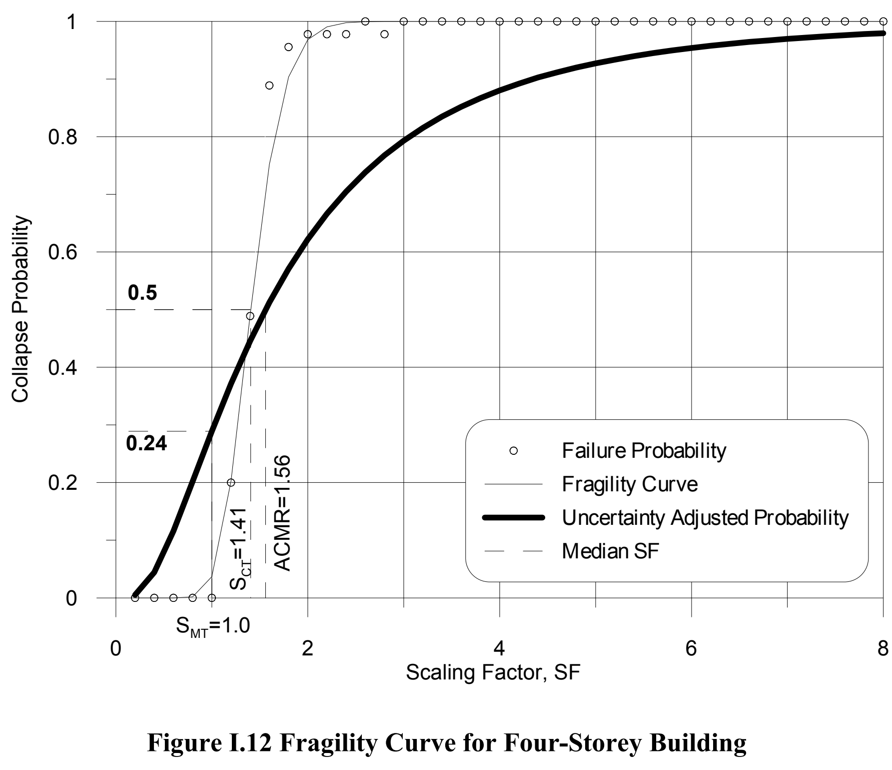


Figure I.12 Fragility Curve for Four-Storey Building

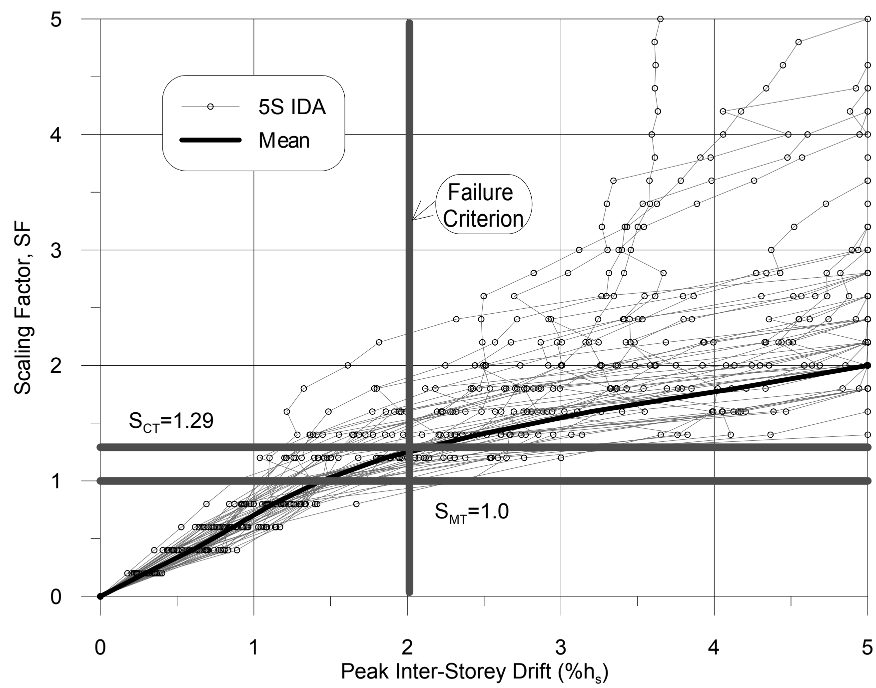


Figure I.13 IDA Curves for 45 Ground Motions (Five-Storey Building)

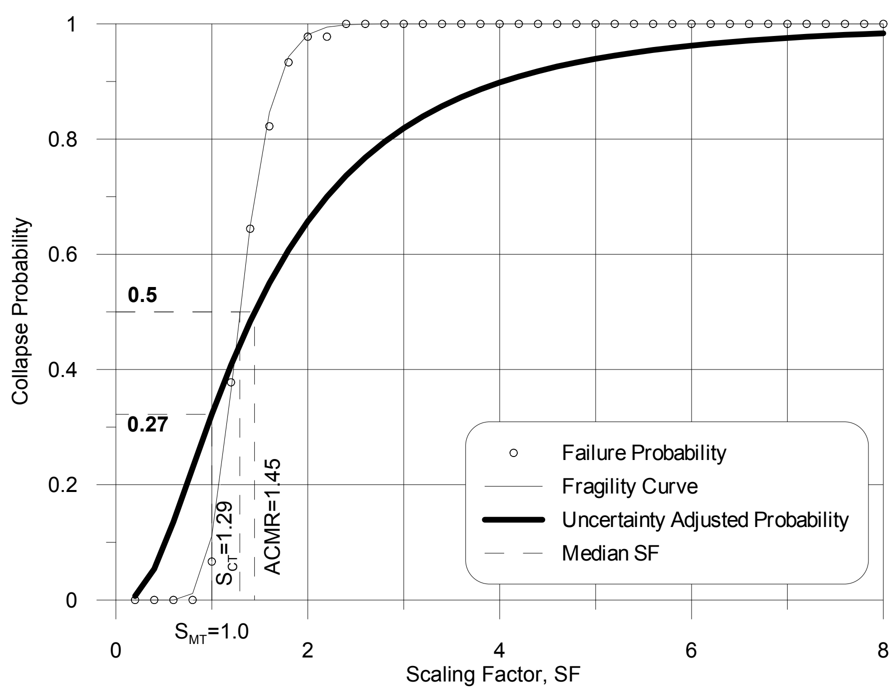


Figure I.14 Fragility Curve for Five-Storey Building

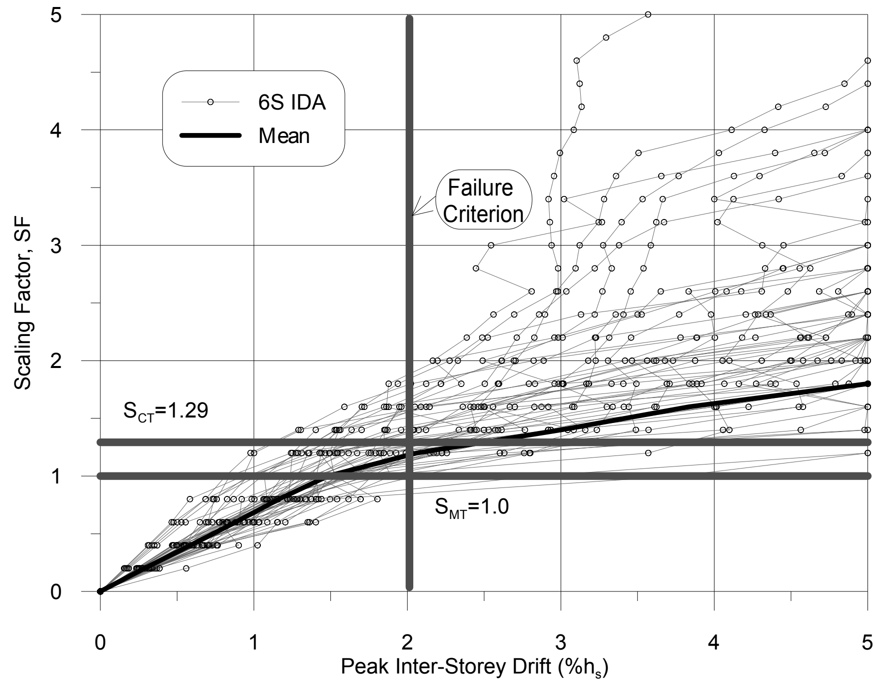


Figure I.15 IDA Curves for 45 Ground Motions (Six-Storey Building)

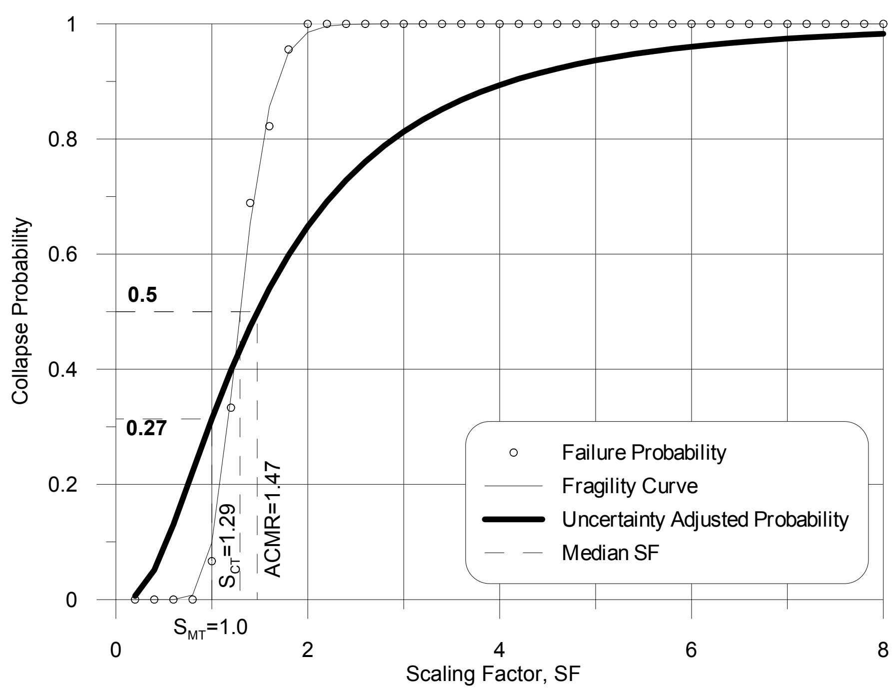


Figure I.16 Fragility Curve for Six-Storey Building

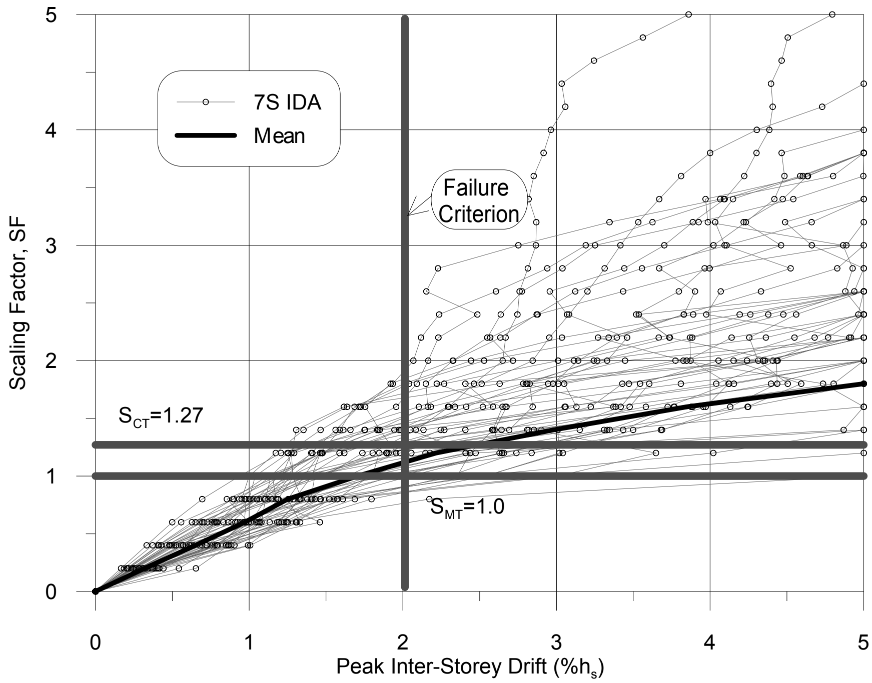


Figure I.17 IDA Curves for 45 Ground Motions (Seven-Storey Building)

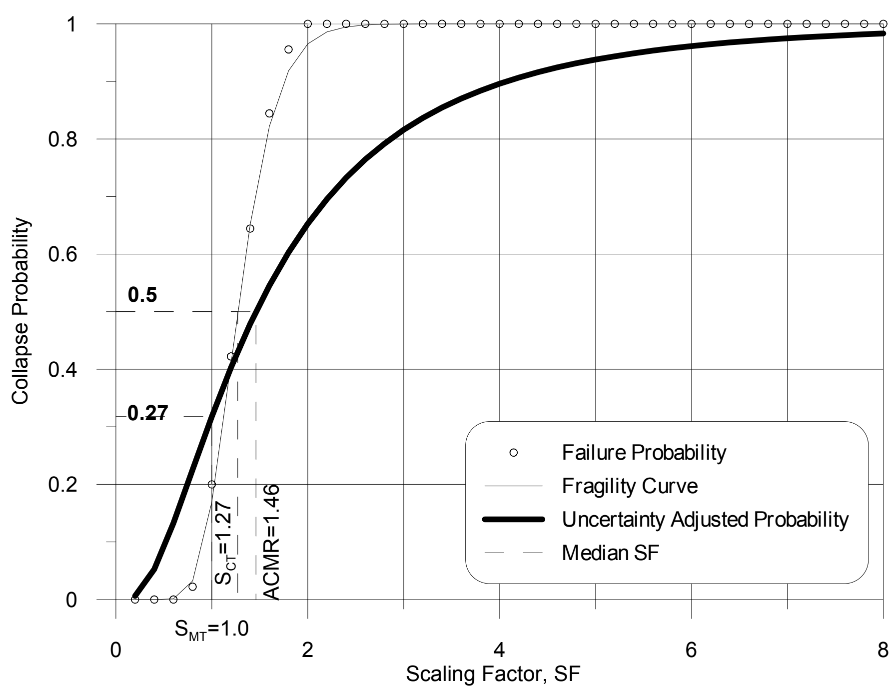


Figure I.18 Fragility Curve for Seven-Storey Building

APPENDIX J

DESIGN PROCEDURE – PHASE II

Two-Storey Building –Vancouver, BC

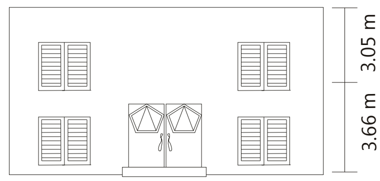


Figure J.1 Elevation View of Two-Storey Model Building

Table J.1 Seismic Weight Distribution for Two-Storey Building

Level	Storey Height (m)	Area (m^2)	Dead (kPa)	Snow (kPa)	Live (kPa)	Seismic Weight (kN)	Cumulative (kN)
Roof		220	0.69	1.64		241.71	241.71
2	3.05	220	2.87		2.44	630.64	872.34
1	3.66	220	2.87		2.44		872.34

Table J.2 Design Base Shear Distribution for Two-Storey Building

Storey	W_i (kN)	h_i (m)	W_i x h_i	F_x (kN)	T_x (kN)	N_x(kN)	Vf_x (kN)
Roof	241.7	6.71	1622	86.77	8.68	1.21	96.66
2	630.6	3.66	2308	123.49	12.35	4.49	140.33
1		-					
Σ			3930	210			237

Table J.3 Design of Two-Storey Building Adjusted for Irregularity

Level	Shear (kN)	Framing (mm)	Sheathing (mm)	Fastener Spacing (mm)	S_v (kN/m)	S_r (kN/m)	Min L (m)	# walls[1]	Rounded # walls[1]
2	96.66	1.09	0.46	150	4.53	3.17	30.46	24.97	29
1	236.99	1.09	0.46	50	7.53	5.27	44.94	36.84	37

[1] 1220mm (4’) wall segments

Table J.4 Design of Double Chord Studs of Two-Storey Building

Storey	Compression – shear (kN)	Compression – gravity (kN)	Compression – total (kN)	DCS Thickness (mm)	# DCS	Area DCS (mm^2)	DCS Pn (kN)
2	19.36	1.63	20.99	1.09	1	417.32	56.6
1	38.60	6.07	65.66	1.37	1	541.19	100

Table J.5 Inter-storey Drift and Stability Factor of Two-Storey Building

Level	h_s (mm)	Δ (mm)	Δ_{mx} (mm)	Drift (%)	θ_x
2	2750	5.9	15.5	0.56	0.011
1	3360	7.8	20.2	0.60	0.017

Table J.6 P-Δ Loads for Two-Storey Building

Level	P-Δ Area (m^2)	Cumulative Area (m^2)	LLRF	Reduced live load (kPa)	Gravity Load (kPa)	P_x (kN)
2	133.6	133.6	0.57	0	1.10	147.0
1	109.9	243.5	0.50	1.22	3.48	382.4

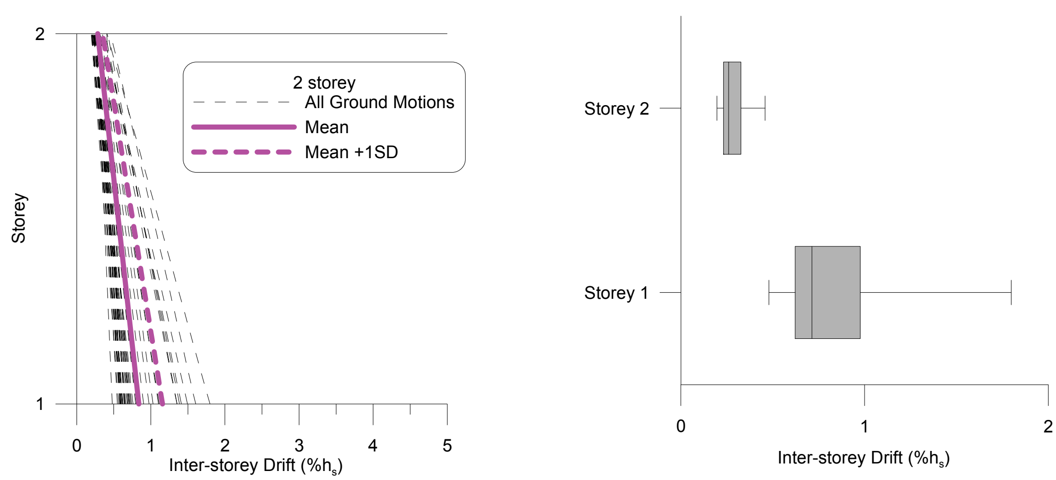


Figure J.2 Inter-Storey Drifts of Two-Storey Building for All 45 Records at Design Level

Three-Storey Building –Vancouver, BC

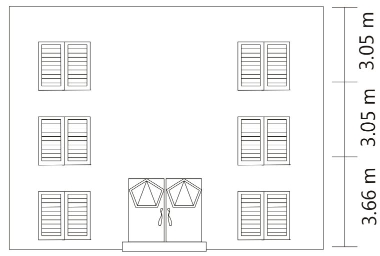


Figure J.3 Elevation View of Three-Storey Building

Table J.7 Seismic Weight Distribution for Three-Storey Building

Level	Storey Height (m)	Area (m^2)	Dead (kPa)	Snow (kPa)	Live (kPa)	Seismic Weight (kN)	Cumulative (kN)
Roof		220	0.69	1.64		241.71	241.71
3	3.05	220	2.87		2.44	630.64	872.34
2	3.05	220	2.87		2.44	630.64	1502.98
1	3.66	220	2.87		2.44		1502.98

Table J.8 Design Base Shear Distribution for Three-Storey Building

Storey	W_i (kN)	h_i (m)	W_i x h_i	F_x (kN)	T_x (kN)	N_x(kN)	Vf_x (kN)
Roof	241.7	9.76	2359	96.03	9.60	1.21	106.85
3	630.6	6.71	4232	172.26	17.23	4.49	193.98
2	630.6	3.66	2308	93.96	9.40	4.49	107.85
1	-	-	-				
Σ			8899	362			409

J.9 Design of Three-Storey Building Adjusted for Irregularity

Level	Shear (kN)	Framing (mm)	Sheathing (mm)	Fastener Spacing (mm)	S_v (kN/m)	S_r (kN/m)	Min L (m)	# walls[1]	Rounded # walls[1]
3	106.85	1.09	0.76	100	10.58	7.41	14.42	11.82	24
2	300.83	1.09	0.76	50	12.54	8.78	34.28	28.10	30
1	408.68	1.09	0.76	50	12.54	8.78	46.57	38.17	39

[1] 1220mm (4') wall segments

Table J.10 Design of Double Chord Studs of Three-Storey Building

Storey	Compression – shear (kN)	Compression – gravity (kN)	Compression – total (kN)	DCS Thickness (mm)	# DCS	Area DCS (mm^2)	DCS Pn (kN)
3	45.19	1.63	46.82	1.09	1	417.32	56.6
2	53.53	6.07	106.42	1.73	1	670.37	128.8
1	64.23	6.07	176.73	1.73	1.5	1005.56	193.2

Table J.11 Inter-storey Drift and Stability Factor of Three-Storey Building

Level	h_s (mm)	Δ (mm)	Δ_{mx} (mm)	Drift (%)	θ_x
3	2750	6.0	15.6	0.57	0.010
2	2750	6.2	16.2	0.59	0.013
1	3360	7.6	19.8	0.59	0.017

Table J.12 P-Δ Loads for Three-Storey Building

Level	P-Δ Area (m^2)	Cumulative Area (m^2)	LLRF	Reduced live load (kPa)	Gravity Load (kPa)	P_x (kN)
3	148.5	148.5	0.56	0	1.10	163.3
2	130.7	279.1	0.49	1.19	3.46	452.7
1	103.9	383.0	0.46	1.12	3.43	356.6

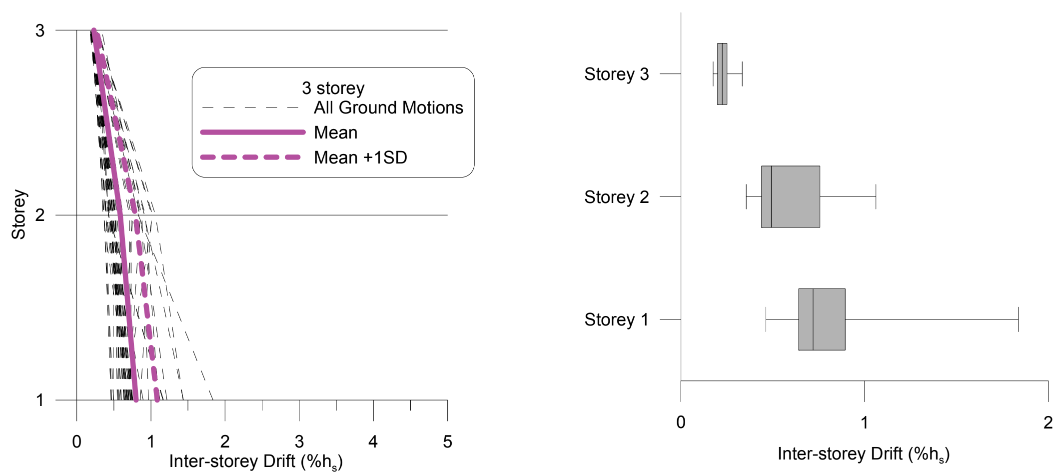


Figure J.4 Inter-Storey Drifts of Three-Storey Building for All 45 Records at Design Level

Four-Storey Building –Vancouver, BC

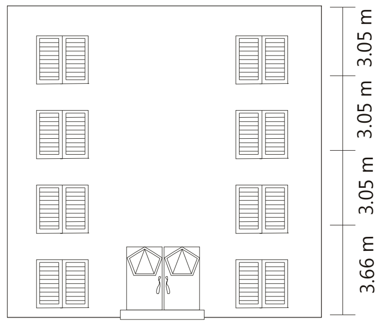


Figure J.5 Elevation View of Four-Storey Building

Table J.13 Seismic Weight Distribution for Four-Storey Building

Level	Storey Height (m)	Area (m^2)	Dead (kPa)	Snow (kPa)	Live (kPa)	Seismic Weight (kN)	Cumulative Seismic Weight (kN)
Roof	-	220	0.69	1.64	-	241.71	241.71
4	3.05	220	2.87	-	2.44	630.64	872.34
3	3.05	220	2.87	-	2.44	630.64	1502.98
2	3.05	220	2.87	-	2.44	630.64	2133.62
1	3.66	220	2.87	-	2.44	-	2133.62

Table J.14 Design Base Shear Distribution for Four-Storey Building

Storey	W_i (kN)	h_i (m)	W_i x h_i	F_x (kN)	T_x (kN)	N_x (kN)	Vf_x (kN)
Roof	241.7	12.81	3096	100.8	10.1	1.21	112.1
4	630.6	9.76	6155	200.4	20.0	4.49	225.0
3	630.6	6.71	4232	137.8	13.8	4.49	156.1
2	630.6	3.66	2308	75.2	7.5	4.49	87.2
1	-	-	-				
Σ			15791	514.3			580.4

Table J.15 Design of Four-Storey Building Adjusted for Irregularity

Level	Shear (kN)	Framing (mm)	Sheathing (mm)	Fastener Spacing (mm)	S_v (kN/m)	S_r (kN/m)	Min L (m)	# walls[1]	Rounded # walls[1]
4	112.13	1.09	0.76	100	10.58	7.41	15.14	12.41	30
3	337.11	1.09	0.76	100	10.58	7.41	45.51	37.30	38
2	493.19	1.09	0.76	50	12.54	8.78	56.20	46.07	47
1	580.37	1.09	0.76	50	12.54	8.78	66.14	54.21	55

[1] 1220mm (4') wall segments

Table J.16 Design of Double Chord Studs of Four-Storey Building

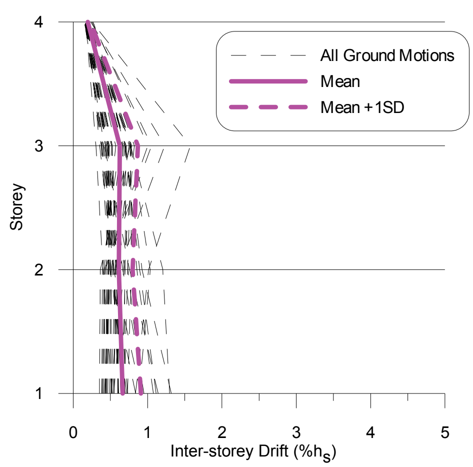
Storey	Compression – shear (kN)	Compression – gravity (kN)	Compression – total (kN)	DCS Thickness (mm)	# DCS	Area DCS (mm^2)	DCS Pn (kN)
4	45.19	1.63	46.82	1.09	1	417.3	56.6
3	45.19	6.07	98.08	1.37	1	541.2	100.0
2	53.53	6.07	157.68	2.46	1	923.1	176.4
1	64.23	6.07	227.98	2.46	1.5	1384.6	264.6

Table J.17 Inter-storey Drift and Stability Factor of Four-Storey Building

Level	h_s (mm)	Δ (mm)	Δ_{mx} (mm)	Drift (%)	θ_x
4	2750	5.9	15.4	0.56	0.009
3	2750	6.3	16.4	0.60	0.012
2	2750	6.0	15.7	0.57	0.013
1	3360	7.4	19.2	0.57	0.016

Table J.18 P-Δ Loads for Four-Storey Building

Level	P-Δ Area (m^2)	Cumulative Area (m^2)	LLRF	Reduced live load (kPa)	Gravity Load (kPa)	P_x (kN)
4	130.7	130.7	0.57	0	1.10	143.7
3	106.9	237.6	0.50	1.23	3.48	372.4
2	80.2	317.7	0.48	1.16	3.45	276.6
1	56.4	374.1	0.46	1.13	3.43	193.7



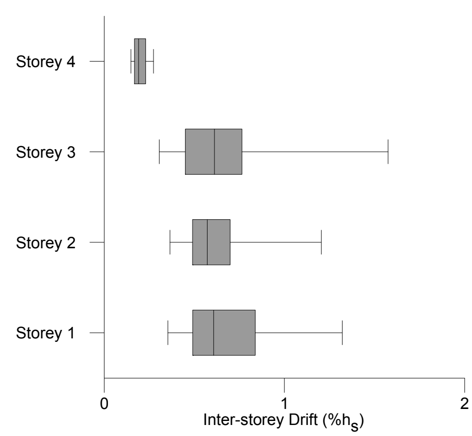


Figure J.6 Inter-Storey Drifts of Four-Storey Building for All 45 Records at Design Level

Five-Storey Building –Vancouver, BC

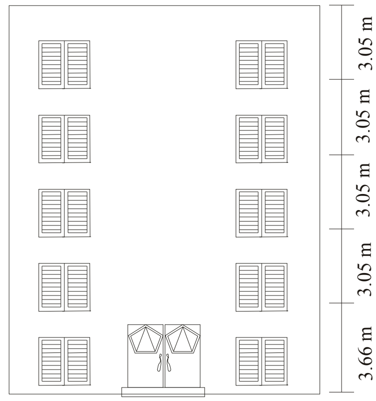


Figure J.7 Elevation View of Five-Storey Building

Table J.19 Seismic Weight Distribution for Five-Storey Building

Level	Storey Height (m)	Area (m^2)	Dead (kPa)	Snow (kPa)	Live (kPa)	Seismic Weight (kN)	Cumulative (kN)
Roof		220	0.69	1.64		241.71	241.71
5	3.05	220	2.87		2.44	630.64	872.34
4	3.05	220	2.87		2.44	630.64	1502.98
3	3.05	220	2.87		2.44	630.64	2133.62
2	3.05	220	2.87		2.44	630.64	2764.25
1	3.66	220	2.87		2.44		2764.25

Table J.20 Design Base Shear Distribution for Five-Storey Building

Storey	W_i (kN)	h_i (m)	W_i x h_i	F_x (kN)	T_x (kN)	N_x(kN)	Vf_x (kN)
Roof	241.7	15.86	3833	91.3	9.1	1.21	101.6
5	630.6	12.81	8078	192.4	19.2	4.49	216.1
4	630.6	9.76	6155	146.6	14.7	4.49	165.7
3	630.6	6.71	4232	100.8	10.1	4.49	115.4
2	630.6	3.66	2308	55.0	5.5	4.49	65.0
1	-	-	-				
Σ			24607	586.0			663.8

Table J.21 Design of Five-Storey Building Adjusted for Irregularity

Level	Shear (kN)	Framing (mm)	Sheathing (mm)	Fastener Spacing (mm)	S_v (kN/m)	S_r (kN/m)	Min L (m)	# walls[1]	Rounded # walls[1]
5	101.64	1.09	0.76	150	8.88	6.21	16.36	13.41	28
4	317.77	1.09	0.76	100	10.58	7.41	42.90	35.16	36
3	483.51	1.09	0.76	50	12.54	8.78	55.10	45.16	46
2	598.86	1.09	0.76	50	12.54	8.78	68.25	55.94	56
1	663.83	1.09	0.76	50	12.54	8.78	75.65	62.01	63

[1] 1220mm (4') wall segments

Table J.22 Design of Double Chord Studs of Five-Storey Building

Storey	Compression – shear (kN)	Compression – gravity (kN)	Compression – total (kN)	DCS Thickness (mm)	# DCS	Area DCS (mm^2)	DCS Pn (kN)
5	37.90	1.63	39.53	1.09	1	417.32	56.6
4	45.19	6.07	90.79	1.37	1	541.19	100
3	53.53	6.07	150.39	2.46	1	923.07	176.4
2	53.53	6.07	209.99	2.46	1.5	1384.61	264.6
1	64.23	6.07	280.30	2.46	2	1846.14	352.8

Table J.23 Inter-storey Drift and Stability Factor of Five-Storey Building

Level	h_s (mm)	Δ (mm)	Δ_{mx} (mm)	Drift (%)	θ_x
5	2750	5.9	15.4	0.56	0.010
4	2750	6.3	16.4	0.60	0.013
3	2750	6.1	15.7	0.57	0.014
2	2750	5.9	15.3	0.56	0.015
1	3360	7.2	18.8	0.56	0.018

Table J.24 P-Δ Loads for Five-Storey Building

Level	P-Δ Area (m^2)	Cumulative Area (m^2)	LLRF	Reduced live load (kPa)	Gravity Load (kPa)	P_x (kN)
5	136.6	136.6	0.57	0	1.10	150.3
4	112.8	249.4	0.50	1.22	3.48	392.4
3	83.1	332.6	0.47	1.15	3.45	286.5
2	53.4	386.0	0.46	1.12	3.43	183.4
1	32.7	418.7	0.45	1.11	3.42	111.8

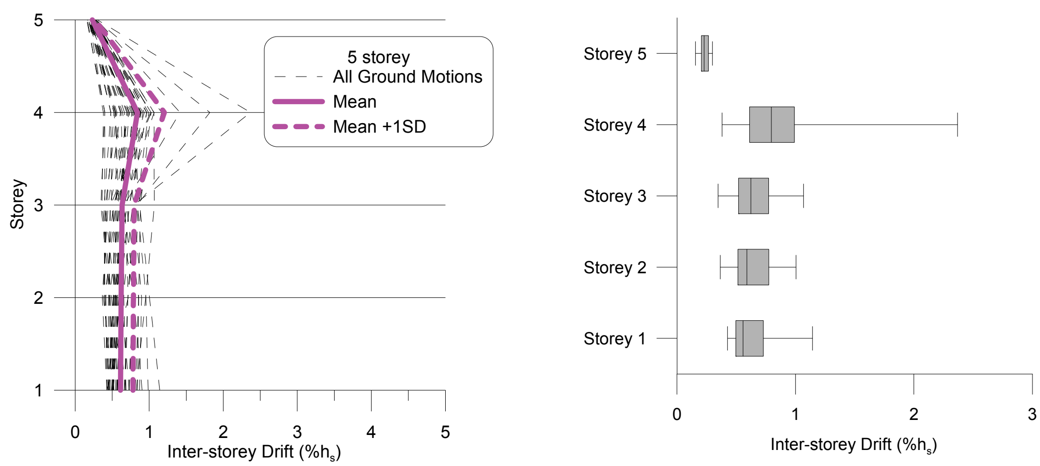


Figure J.8 Inter-Storey Drifts of Five-Storey Building for All 45 Records at Design Level

Six-Storey Building –Vancouver, BC

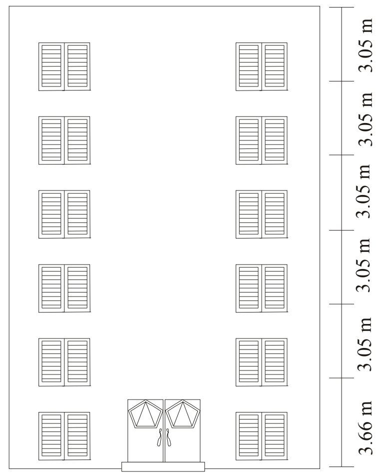


Figure J.9 Elevation View of Six-Storey Building

Table J.25 Seismic Weight Distribution for Six-Storey Building

Level	Storey Height (m)	Area (m^2)	Dead (kPa)	Snow (kPa)	Live (kPa)	Seismic Weight (kN)	Cumulative (kN)
Roof		220	0.69	1.64		241.71	241.71
6	3.05	220	2.87		2.44	630.64	872.34
5	3.05	220	2.87		2.44	630.64	1502.98
4	3.05	220	2.87		2.44	630.64	2133.62
3	3.05	220	2.87		2.44	630.64	2764.25
2	3.05	220	2.87		2.44	630.64	3394.89
1	3.66	220	2.87		2.44		3394.89

Table J.26 Design Base Shear Distribution for Six-Storey Building

Storey	W_i (kN)	h_i (m)	W_i x h_i	F_x (kN)	T_x (kN)	N_x(kN)	Vf_x (kN)
Roof	241.7	18.91	4571	81.2	8.1	1.21	90.6
6	630.6	15.86	10002	177.8	17.8	4.49	200.1
5	630.6	12.81	8078	143.6	14.4	4.49	162.4
4	630.6	9.76	6155	109.4	10.9	4.49	124.8
3	630.6	6.71	4232	75.2	7.5	4.49	87.2
2	630.6	3.66	2308	41.0	4.1	4.49	49.6
1	-	-	-				
Σ			35346	628.3			714.8

Table J.27 Design of Six-Storey Building Adjusted for Irregularity

Level	Shear (kN)	Framing (mm)	Sheathing (mm)	Fastener Spacing (mm)	S_v (kN/m)	S_r (kN/m)	Min L (m)	# walls[1]	Rounded # walls[1]
6	90.58	1.09	0.76	150	8.88	6.21	14.58	11.95	27
5	290.63	1.09	0.76	100	10.58	7.41	39.23	32.16	34
4	453.07	1.09	0.76	50	12.54	8.78	51.63	42.32	43
3	577.91	1.09	0.76	50	12.54	8.78	65.86	53.98	54
2	665.14	1.09	0.76	50	12.54	8.78	75.80	62.13	63
1	714.76	1.09	0.76	50	12.54	8.78	81.45	66.76	67

[1] 1220mm (4’) wall segments

Table J.28 Design of Double Chord Studs of Six-Storey Building

Storey	Compression – shear (kN)	Compression – gravity (kN)	Compression – total (kN)	DCS Thickness (mm)	# DCS	Area DCS (mm^2)	DCS Pn (kN)
6	37.90	1.63	39.53	1.09	1	417.3	56.6
5	45.19	6.07	90.79	1.37	1	541.2	100
4	53.53	6.07	150.39	2.46	1	923.1	176.4
3	53.53	6.07	209.99	2.46	1.5	1384.6	264.6
2	53.53	6.07	269.59	2.46	2	1846.1	352.8
1	64.23	6.07	339.90	2.46	2	1846.1	352.8

Table J.29 Inter-storey Drift and Stability Factor of Six-Storey Building

Level	h_s (mm)	Δ (mm)	Δ_{mx} (mm)	Drift (%)	θ_x
6	2750	5.9	15.3	0.56	0.011
5	2750	6.3	16.3	0.59	0.014
4	2750	6.1	15.7	0.57	0.015
3	2750	5.9	15.3	0.56	0.016
2	2750	5.8	15.1	0.55	0.018
1	3360	7.2	18.8	0.56	0.020

Table J.30 P-Δ Loads for Six--Storey Building

Level	P-Δ Area (m^2)	Cumulative Area (m^2)	LLRF	Reduced live load (kPa)	Gravity Load (kPa)	P_x (kN)
6	139.6	139.6	0.56	0	1.10	153.5
5	118.8	258.3	0.49	1.21	3.47	412.6
4	92.1	350.4	0.47	1.14	3.44	316.7
3	59.4	409.8	0.45	1.11	3.42	203.4
2	32.7	442.4	0.45	1.10	3.42	111.6
1	20.8	463.2	0.45	1.09	3.41	71.0

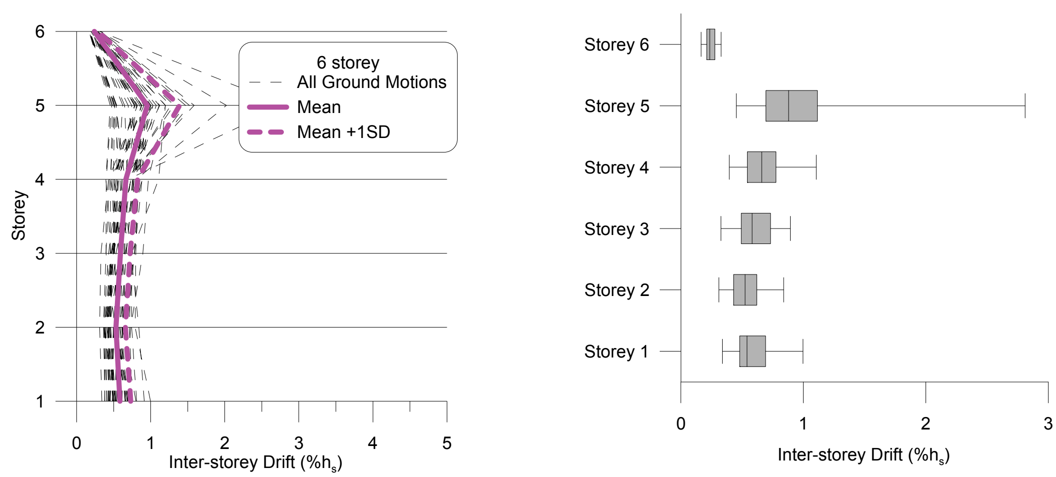


Figure J.10 Inter-Storey Drifts of Six-Storey Building for All 45 Records at Design Level

Seven-Storey Building –Vancouver, BC

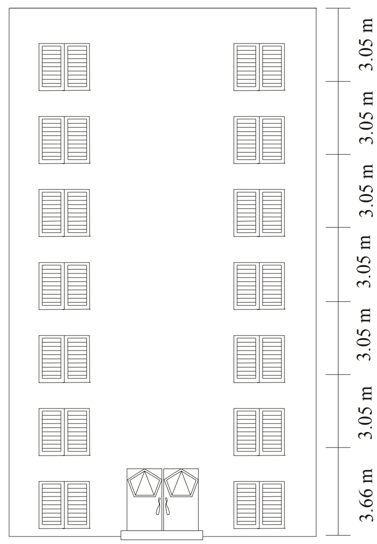


Figure J.11 Elevation View of Seven-Storey Building

Table J.31 Seismic Weight Distribution for Seven-Storey Building

Level	Storey Height (m)	Area (m^2)	Dead (kPa)	Snow (kPa)	Live (kPa)	Seismic Weight (kN)	Cumulative (kN)
Roof		220	0.69	1.64	0	241.71	241.71
7	3.05	220	2.87		2.44	630.64	872.34
6	3.05	220	2.87		2.44	630.64	1502.98
5	3.05	220	2.87		2.44	630.64	2133.62
4	3.05	220	2.87		2.44	630.64	2764.25
3	3.05	220	2.87		2.44	630.64	3394.89
2	3.05	220	2.87		2.44	630.64	4025.53
1	3.66	220	2.87		2.44		4025.53

Table J.32 Design Base Shear Distribution for Seven-Storey Building

Storey	W_i (kN)	h_i (m)	W_i x h_i	F_x (kN)	T_x (kN)	N_x(kN)	Vf_x (kN)
Roof	241.7	21.96	5308	69.3	6.9	1.21	77.5
7	630.6	18.91	11925	155.7	15.6	4.49	175.8
6	630.6	15.86	10002	130.6	13.1	4.49	148.2
5	630.6	12.81	8078	105.5	10.5	4.49	120.5
4	630.6	9.76	6155	80.4	8.0	4.49	92.9
3	630.6	6.71	4232	55.3	5.5	4.49	65.3
2	630.6	3.66	2308	30.1	3.0	4.49	37.6
1	-	-	-				
Σ			48008	626.9			717.8

Table J.33 Design of Seven-Storey Building Adjusted for Irregularity

Level	Shear (kN)	Framing (mm)	Sheathing (mm)	Fastener Spacing (mm)	S_v (kN/m)	S_r (kN/m)	Min L (m)	# walls[1]	Rounded # walls[1]
7	77.45	1.09	0.76	150	8.88	6.21	12.47	10.22	24
6	253.24	1.09	0.76	100	10.58	7.41	34.19	28.02	30
5	401.40	1.09	0.76	50	12.54	8.78	45.74	37.49	38
4	521.93	1.09	0.76	50	12.54	8.78	59.48	48.75	49
3	614.83	1.09	0.76	50	12.54	8.78	70.06	57.43	58
2	680.10	1.09	0.76	50	12.54	8.78	77.50	63.53	64
1	717.75	1.09	0.76	50	12.54	8.78	81.79	67.04	68

[1] 1220mm (4’) wall segments

Table J.34 Design of Double Chord Studs of Seven-Storey Building

Storey	Compression – shear (kN)	Compression – gravity (kN)	Compression – total (kN)	DCS Thickness (mm)	# DCS	Area DCS (mm^2)	DCS Pn (kN)
7	37.9	1.6	39.5	1.09	1	417.32	56.6
6	45.2	6.1	90.8	1.37	1	541.19	100
5	53.5	6.1	150.4	2.46	1	923.07	176.4
4	53.5	6.1	210.0	2.46	1.5	1384.61	264.6
3	53.5	6.1	269.6	2.46	2	1846.14	352.8
2	53.5	6.1	329.2	2.46	2	1846.14	352.8
1	64.2	6.1	399.5	2.46	2.5	2307.68	441

Table J.35 Inter-storey Drift and Stability Factor of Seven-Storey Building

Level	h_s (mm)	Δ (mm)	Δ_{mx} (mm)	Drift (%)	θ_x
7	2750	5.9	15.3	0.56	0.013
6	2750	6.3	16.3	0.59	0.016
5	2750	6.1	15.7	0.57	0.016
4	2750	5.9	15.3	0.56	0.017
3	2750	5.8	15.1	0.55	0.019
2	2750	5.8	15.1	0.55	0.021
1	3360	7.1	18.6	0.55	0.024

Table J.36 P-Δ Loads for Seven-Storey Building

Level	P-Δ Area (m^2)	Cumulative Area (m^2)	LLRF	Reduced live load (kPa)	Gravity Load (kPa)	P_x (kN)
7	148.5	148.5	0.56	0	1.10	163.3
6	130.7	279.1	0.49	1.19	3.46	452.7
5	106.9	386.0	0.46	1.12	3.43	366.7
4	74.2	460.3	0.45	1.09	3.41	253.4
3	47.5	507.8	0.44	1.07	3.41	161.8
2	29.7	537.5	0.44	1.06	3.40	101.0
1	17.8	555.3	0.43	1.06	3.40	60.5

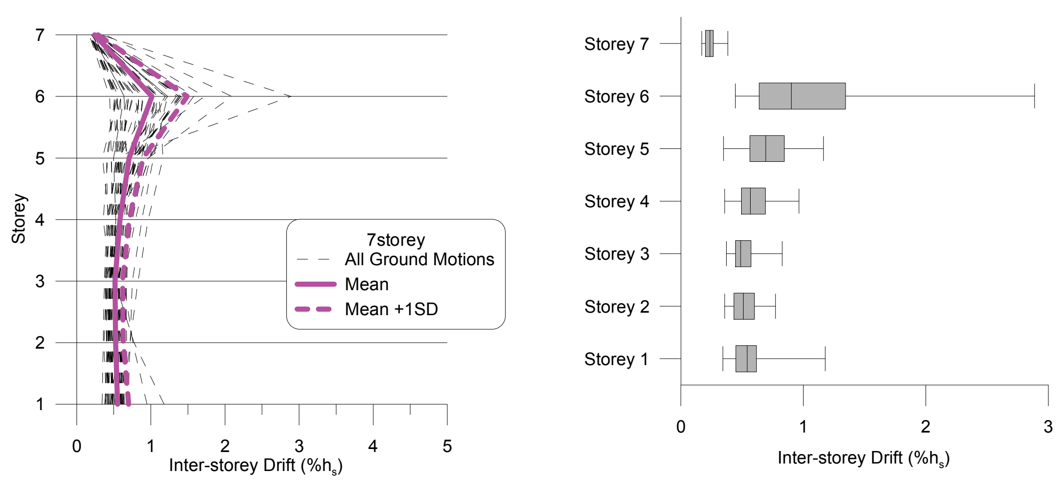


Figure J.12 Inter-Storey Drifts of Seven-Storey Building for All 45 Records at Design Level

APPENDIX K

PHASE II –

FEMA P695 SUMMARY:

PUSHOVER AND IDA ANALYSES

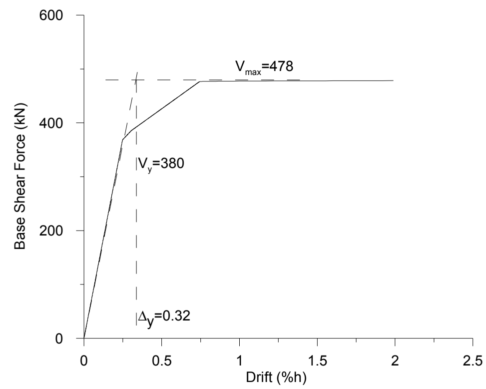


Figure K.1 Pushover Curve for Two-Storey Building

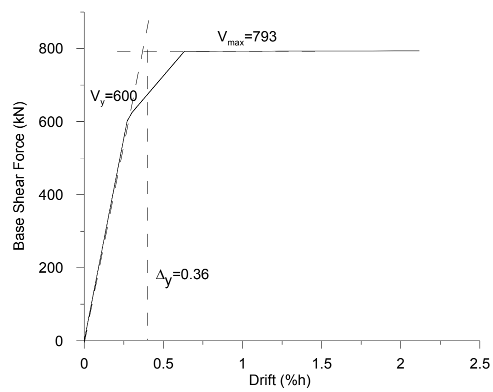


Figure K.2 Pushover Curve for Three-Storey Building

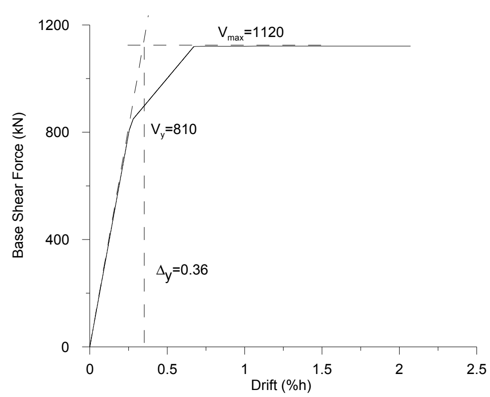


Figure K.3 Pushover Curve for Four-Storey Building

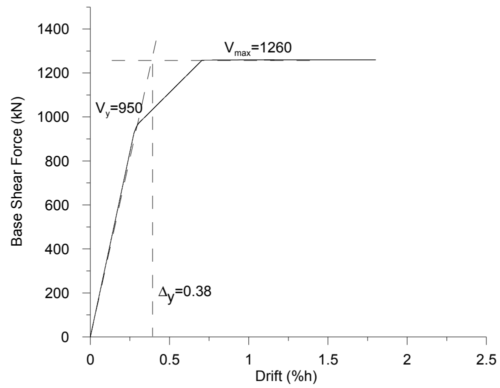


Figure K.4 Pushover Curve for Five-Storey Building

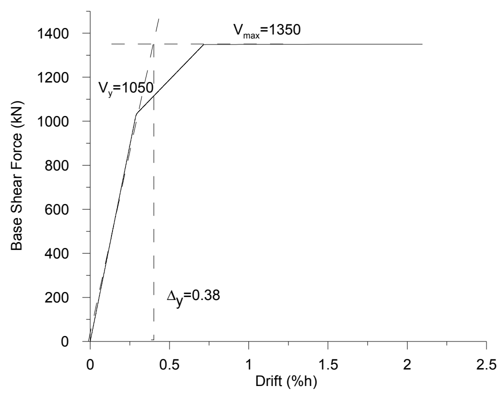


Figure K.5 Pushover Curve for Six-Storey Building

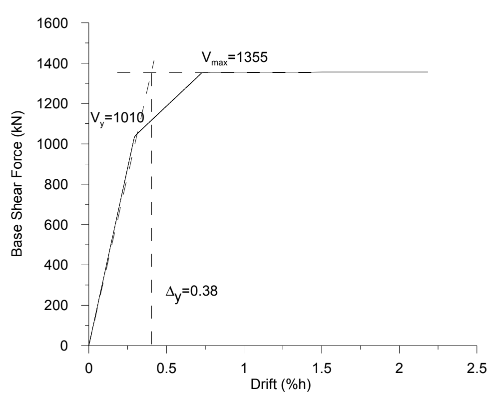


Figure K.6 Pushover Curve for Seven-Storey Building

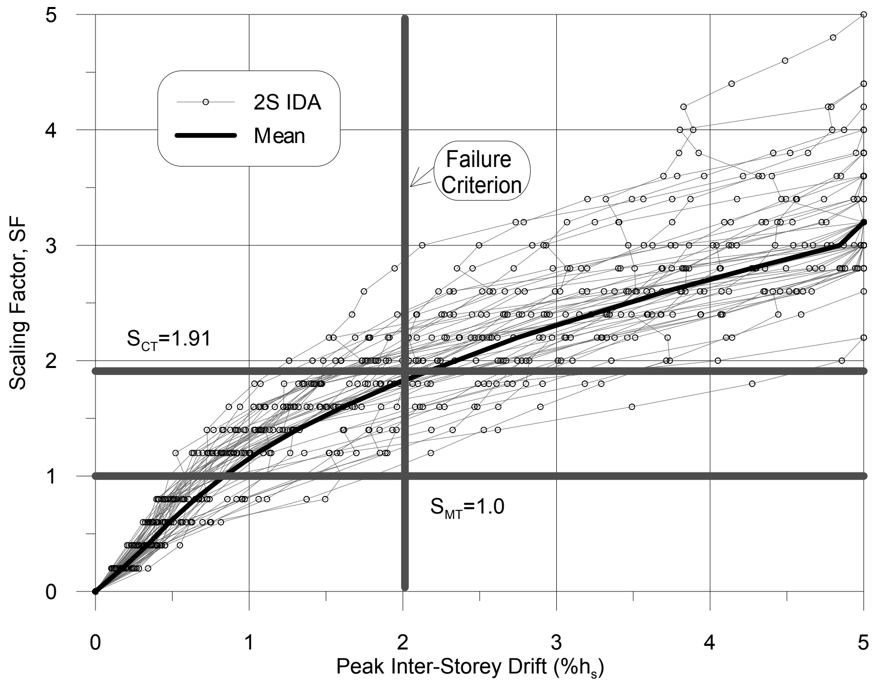


Figure K.7 IDA Curves for 45 Ground Motions (Two-Storey Building)

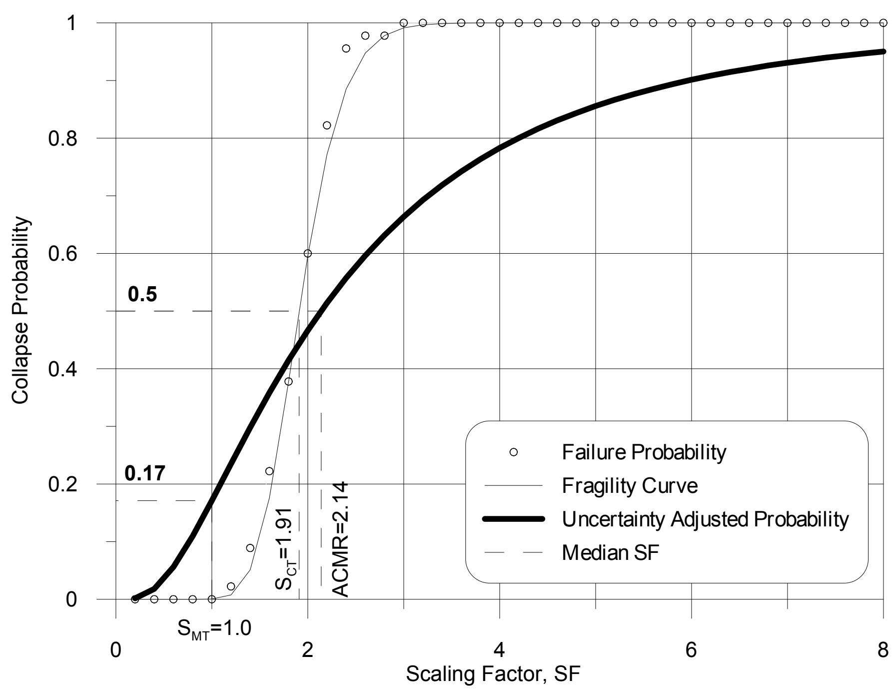


Figure K.8 Fragility Curve for Two-Storey Building

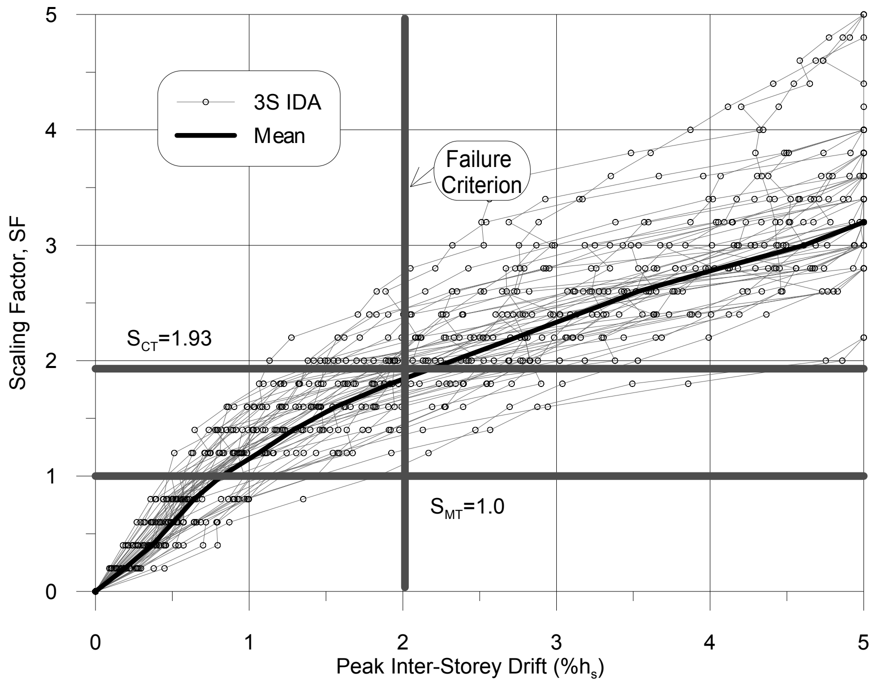


Figure K.9 IDA Curves for 45 Ground Motions (Three-Storey Building)

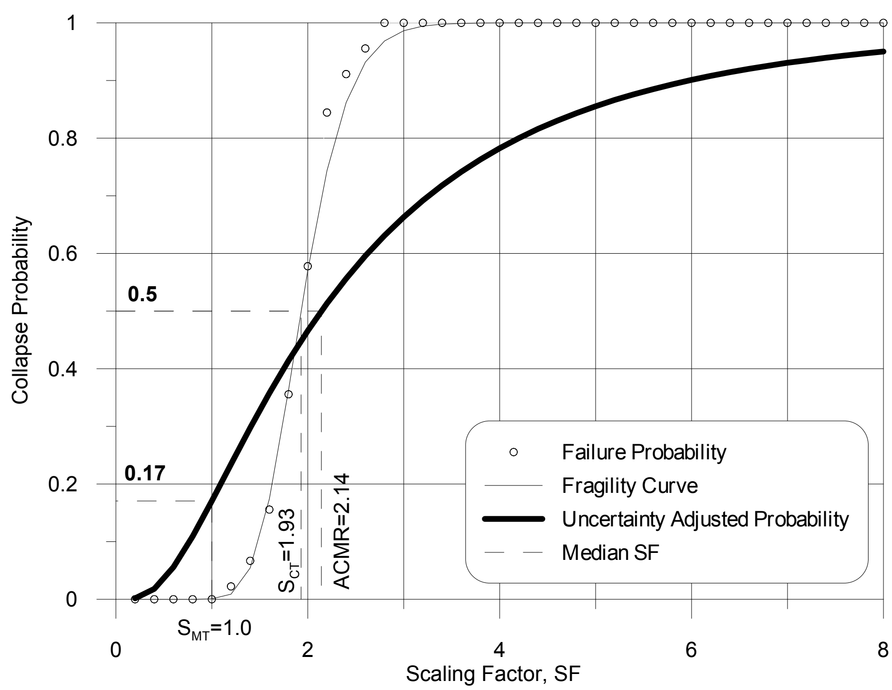


Figure K.10 Fragility Curve for Three-Storey Building

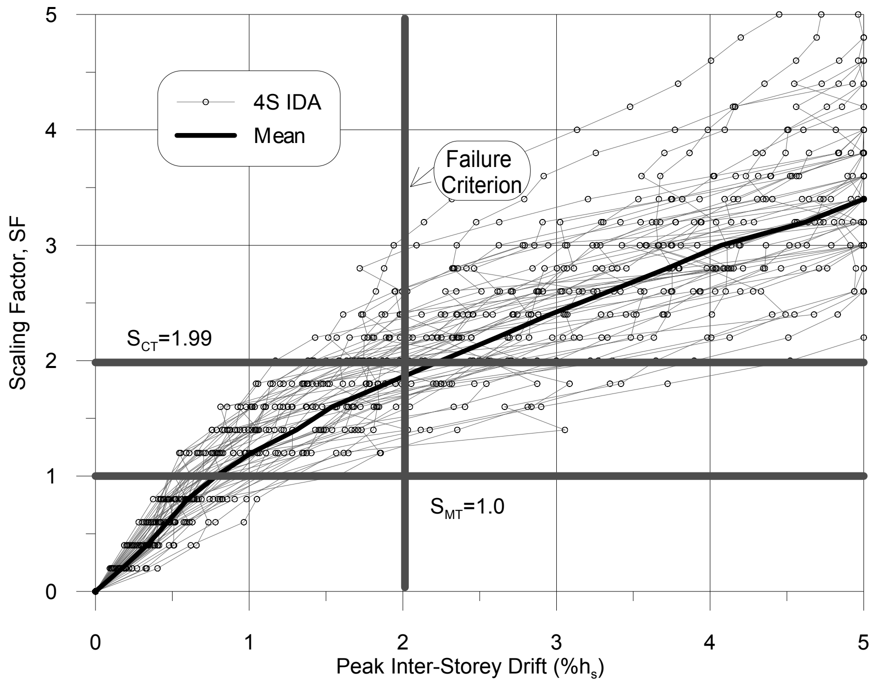


Figure K.11 IDA Curves for 45 Ground Motions (Four-Storey Building)

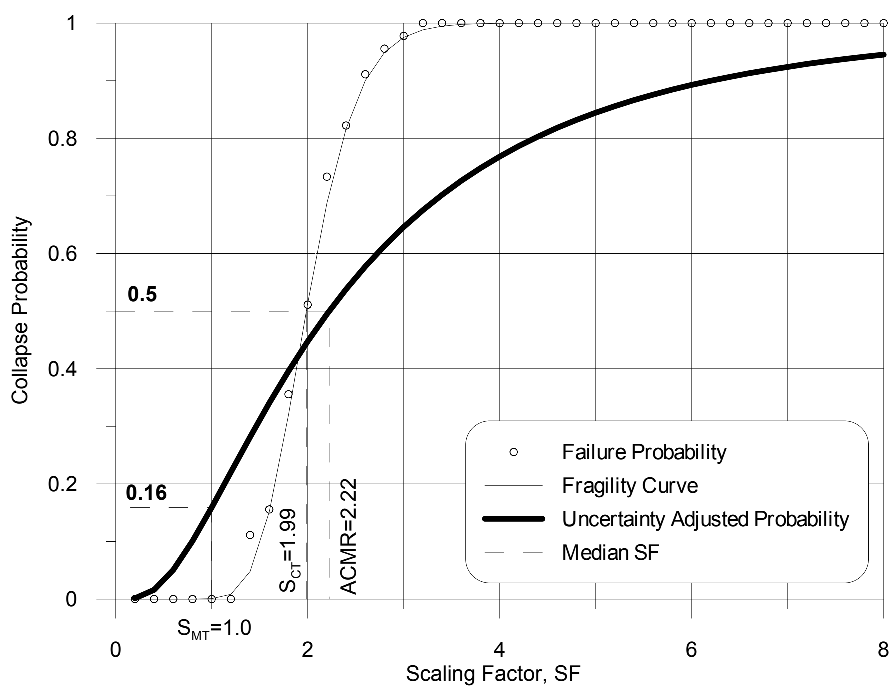


Figure K.12 Fragility Curve for Four-Storey Building

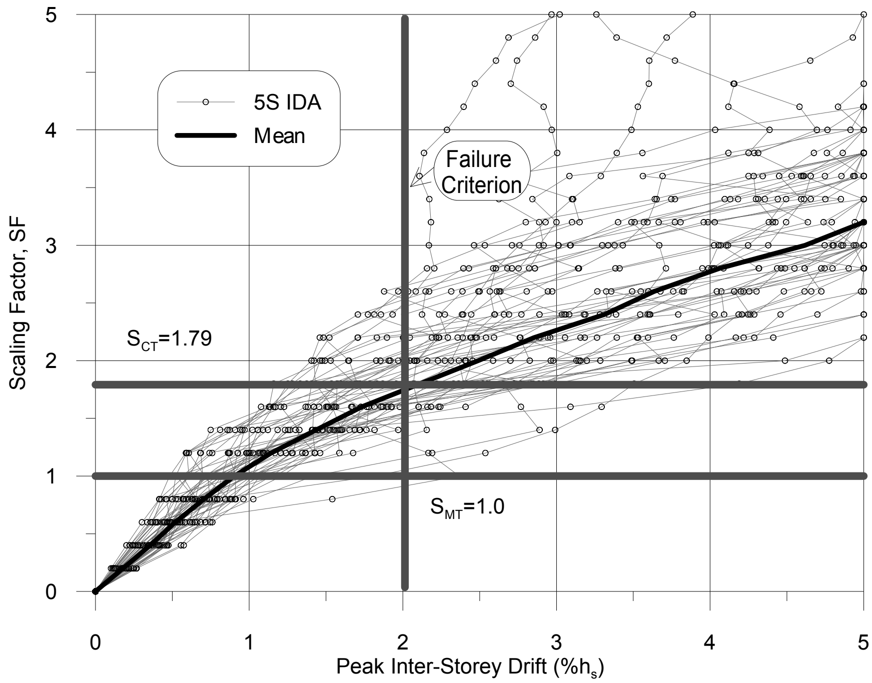


Figure K.13 IDA Curves for 45 Ground Motions (Five-Storey Building)

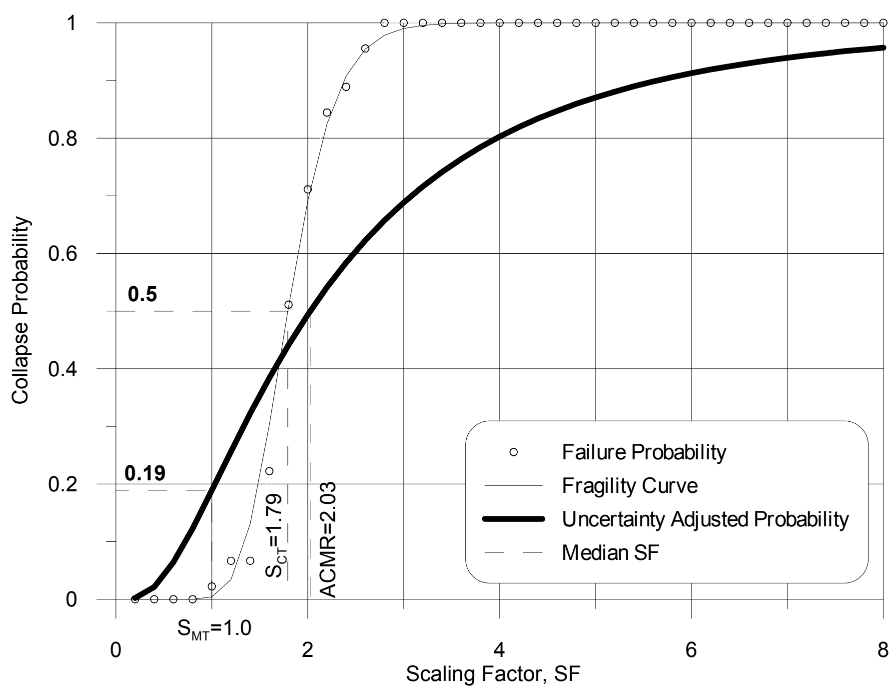


Figure K.14 Fragility Curve for Five-Storey Building

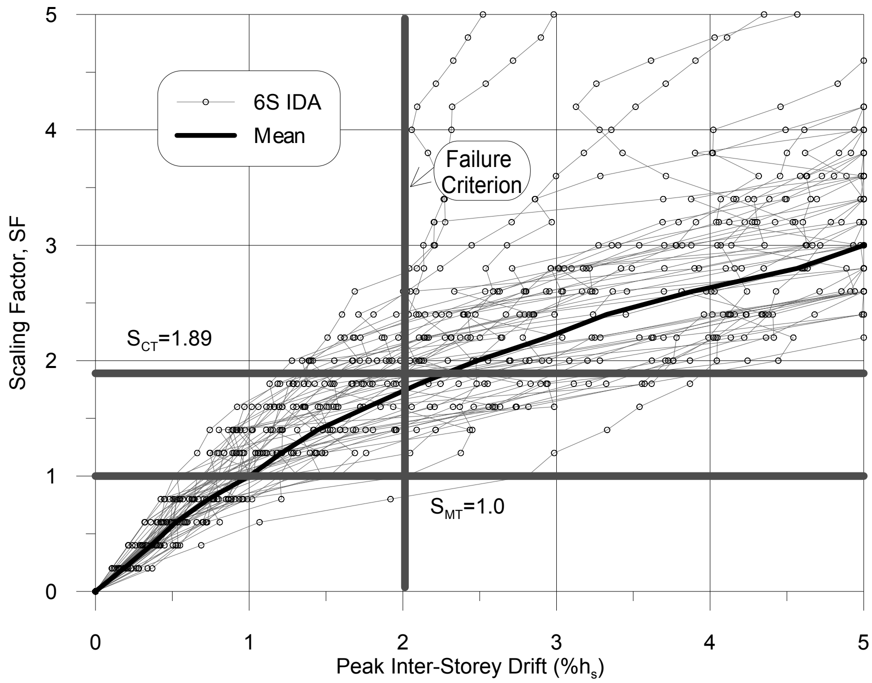


Figure K.15 IDA Curves for 45 Ground Motions (Six-Storey Building)

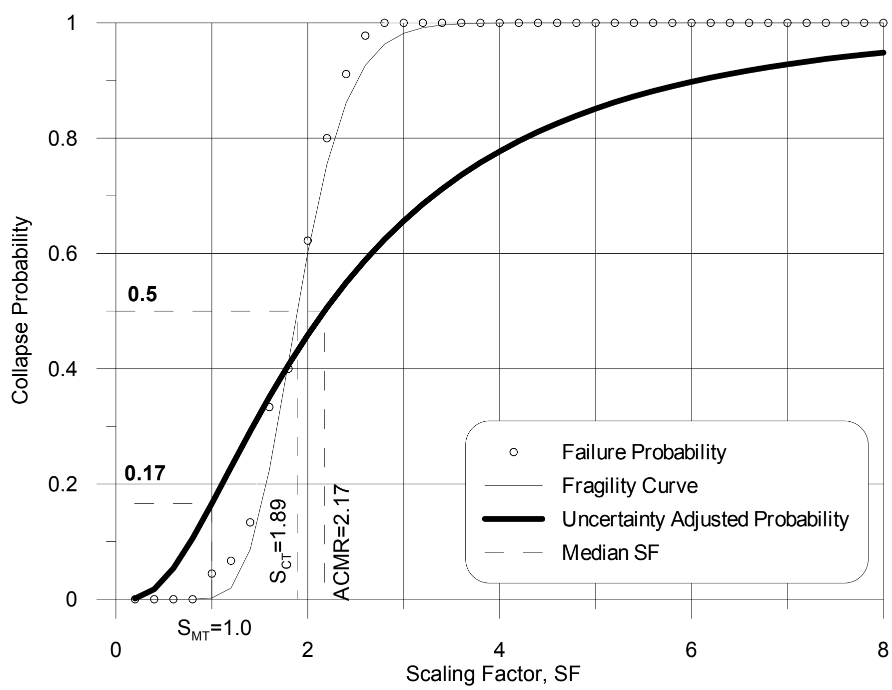


Figure K.16 Fragility Curve for Six-Storey Building

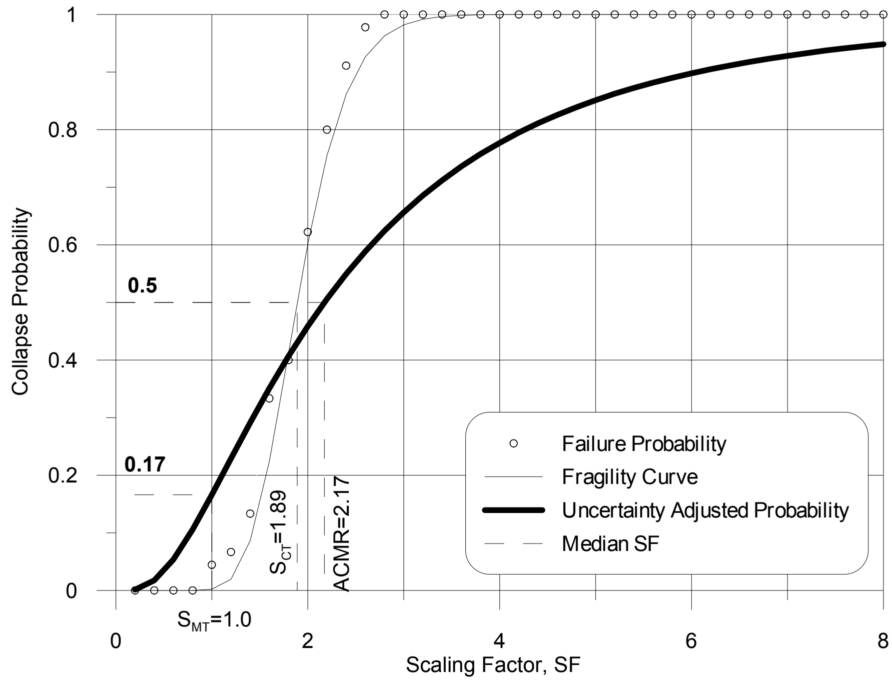


Figure K.17 IDA Curves for 45 Ground Motions (Seven-Storey Building)

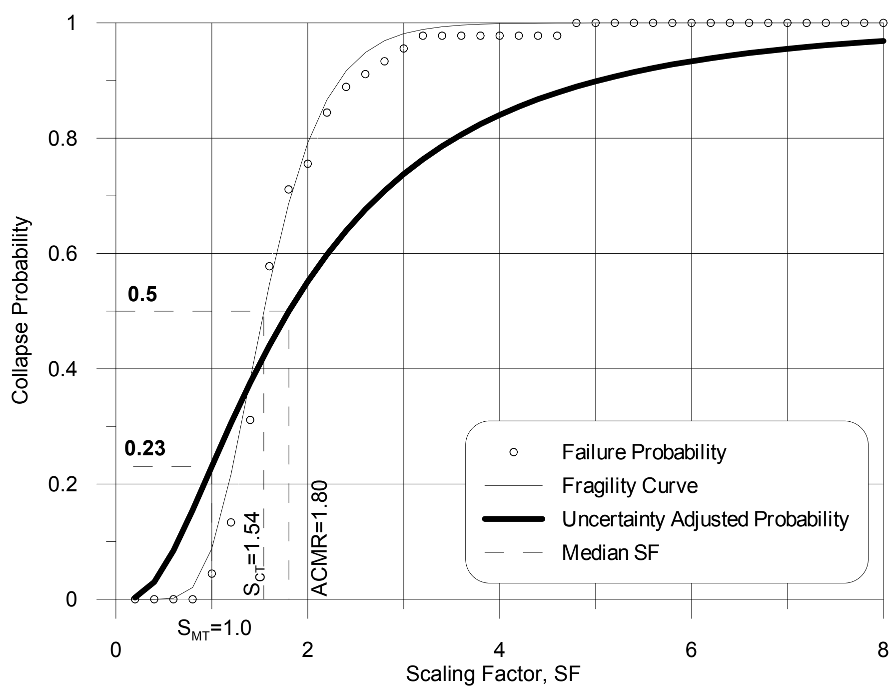


Figure K.18 Fragility Curve for Seven-Storey Building

APPENDIX L

USING EXCEL^{TM} FOR DATA ANALYSIS

Introduction

Microsoft Excel™ spreadsheets were created to speed up the analysis of data due to the number of specimens tested during the summer of 2008 at McGill University. The spreadsheets required minimal user input and they also minimized errors in computation by being applicable to all files, ensuring consistency and compatibility with all data. The spreadsheets were created using Visual Basic™ Macros; one for monotonic analysis and another for reversed cyclic analysis.

The test results were recorded in columns with many rows of unnecessary data which had to be taken out to achieve reasonable results. These unnecessary rows of data were due to a lag in data collection between the data acquisition and the actuator controller.

Monotonic

It was decided to not account for slip and uplift in net lateral deflection of the walls. The columns containing such information were therefore discarded. The only columns required were: ID, Time, MTS Load in Newtons, and MTS LVDT in millimeters. For each test the wall width in feet and the maximum drift limit as a percentage must be included. The results are copied from the test into the sheet labeled "Test Data" in the monotonic workbook (Figure L.1).

	A	B	C	D	E	F	G	H	I	J	K
1											
2	Scan Session: "Scan Session #7"										
3	Start Time: 14/05/2008 3:37:08 PM										
4											
5	ID	SecondsElapsed	MTS LC on C1	MTS LVDT on C2	Wall Top on C3	NSlip on C4	NUplift on C5	SSlip on C6	SUplift on C7	N LC on C11	S LC on C12
6	1	16.2	30.52	0.02	0.01	0.01	0.01	0	0	9249.93	7997.38

Figure L.1 Sample Test Data

	A	B	C	D	E	F	G
1	Monotonic EEEP			ID	Seconds	MTS Load	Actuator LVDT
2							
3	Test Information						
4	Name:	1M-a					
5	Date:	DD/MM/YYYY					
6	Protocol:	Monotonic					
7							
8	Wall Size(ft)	4					
9	Drift Limit (%)	2.5					
10	Drift Limit(mm)	60.96					

Figure L.2 Spreadsheet for Monotonic Test Analysis

Once the necessary data was placed in the sheet called "Monotonic Data" (Figure L.2) in the same workbook, a button on the left hand side of the sheet labeled "Calculate Shear Forces, Rotation & Energy" [Calculate Shear Forces, Rotation & Energy] was clicked to evaluate Shear Force, Rotation and incremental Energy. This was just a preliminary step so as not to overload the Excel™ sheet in waiting time. Once this step was completed, the user proceeded to click on the button labeled "Find Forces & Backbone Area" [Find Forces & Backbone Area]. This was a crucial step as it determined the yield resistance, F_y, ultimate resistance, F_u, deflections at yield point, ultimate, 40% and 80% of ultimate, and determined the Equivalent Energy Elastic-Plastic Curve for the given monotonic results.

	H	I	J	K	L
1		Shear Force (kN/m)	Shear Force (kN)	Rotation x 10^-3 (rad)	Energy (J)

Figure L.3 Example Spreadsheet for Monotonic Test Analysis

The parameters were calculated based on Equations (L-1)-(L-9) and were then tabulated as presented in Table L.1

Table L.1 Sample Monotonic Test Results Using the EEEP Analysis Approach

Parameters		Units
F_u	7.92	kN
$F_{0.8u}$	6.34	kN
$F_{0.4u}$	3.17	kN
F_y	7.15	kN
K_e	0.96	kN/mm
Ductility (μ)	9.79	-
$\Delta_{net,y}$	7.45	mm
$\Delta_{net,u}$	33.13	mm
$\Delta_{net,0.8u}$	72.99	mm
$\Delta_{net,0.4u}$	3.30	mm
$Area_{Backbone}$	495.52	J
$Area_{EEEP}$	495.52	J
Check	OK	
R_d	4.31	-
S_y	5.87	kN/m

$$F_{0.8U} = 0.8F_u \tag{L-1}$$

$$F_{0.4U} = 0.4F_u \tag{L-2}$$

$$k_e = \frac{F_{0.4U}}{\Delta_{net,0.4U}} \tag{L-3}$$

$$F_y = \frac{-\Delta_{net,0.8U} \pm \sqrt{\Delta_{net,o.8}{}^2 - \frac{2A}{k_e}}}{-1/k_e} \tag{L-4}$$

$$\Delta_{net,y} = \frac{F_y}{k_e} \tag{L-5}$$

$$S_y = \frac{F_y}{L} \tag{L-6}$$

$$\mu = \frac{\Delta_{net,0.8U}}{\Delta_{net,y}} \tag{L-7}$$

$$R_d = \sqrt{2\mu - 1} \tag{L-8}$$

$$A_{EEEP} = \frac{1}{2} F_y \cdot \Delta_{net,y} + F_y \cdot (\Delta_{net,0.8U} - \Delta_{net,y}) \tag{L-9}$$

where,

F_u = ultimate shear resistance

$F_{0.8U}$ = 80% of ultimate resistance

$F_{0.4U}$ = 40% of ultimate resistance

k_e = elastic stiffness

F_y = yield resistance

$\Delta_{net,y}$ = displacement at yield resistance

$\Delta_{net,0.8U}$ = displacement at 80% of ultimate resistance (post-peak)

S_y = yield resistance per unit length

μ = ductility

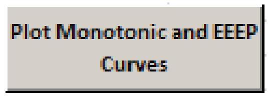
R_d = ductility-related seismic force modification factor

A_{EEEP} = area below the EEEP curve

Ultimate Shear Force, F_u, was determined as the maximum force that was reached during testing, or the peak of the curve. The forces at 40% and 80% of the peak load were determined by multiplying 0.4 and 0.8 by F_u, respectively. The corresponding displacements were based on searching through the data for the closest match to the calculated forces. For displacement corresponding to 40% of the peak load, the Macro searched for the closest matching value before the peak was reached. Similarly, the displacement corresponding to 80% of the ultimate load was found but was searched for within the post-peak section of the test. There were three scenarios that were accounted for in the Macro:

d) If the calculated 80% post-peak load was reached at a displacement greater than 100mm, then the corresponding 80% displacement was set to 100mm
e) If the calculated 80% post-peak load was lower than the last reached load, then displacement at 80% was determined as the last reached displacement or 100mm if the last displacement was greater than 100mm

f) If none of the above scenarios occur, then the displacement at 80% of the post-peak load was searched for and recorded.

Finally, after the required values were computed, the “Plot Monotonic and EEEP Curves” 

button was clicked to view a plot of the observed monotonic curve and the EEEP bilinear representation (Figure L.4).

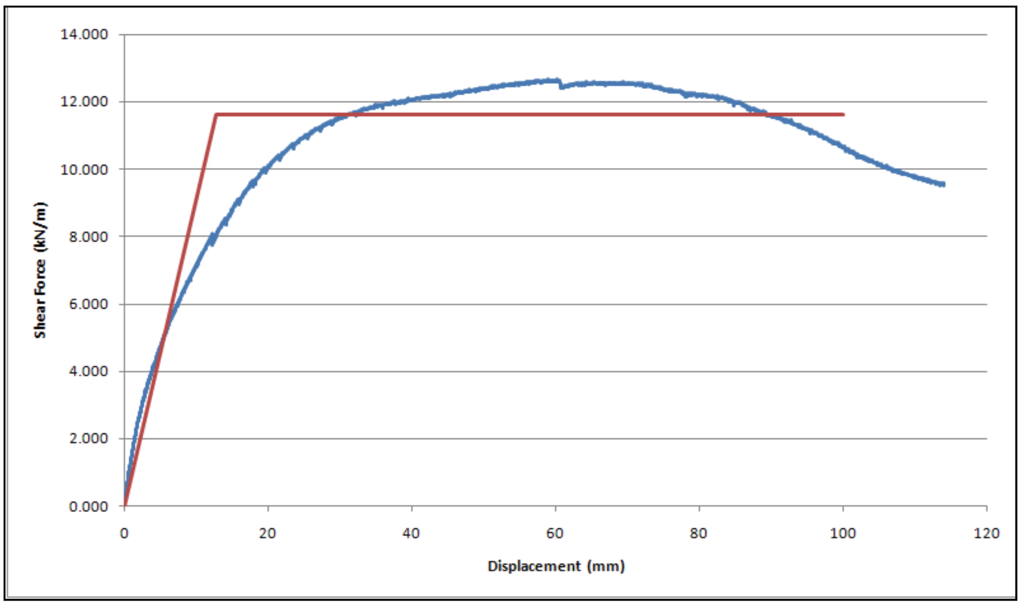


Figure L.4 Sample EEEP Curve for Monotonic Test Data

The button labeled “Reset”

erases all the data placed in the file. Before clicking this button the file should have been saved if the results were to be maintained. However, there is a warning before the workbook is reset that allows the user to confirm the command.

CUREE Cyclic

In addition to the information required from Monotonic tests, the accelerometer data was needed for input. The type of CUREE Cyclic Protocol must be selected as well. The appropriate protocol was selected from a drop down list (Figire L.6). The choices available were specific to the loading scenarios in the tests of summer 2008 at McGill University. There were four possibilities and they included:

a) Maximum amplitude at 2.0Δ with a frequency change from 0.5Hz to 0.25Hz at 2.0Δ – 92s

b) Maximum amplitude at 2.5Δ with a frequency change from 0.5Hz to 0.25Hz at 2.5Δ – 98s

c) Maximum amplitude at 3.0Δ with a frequency change from 0.5Hz to 0.25Hz at 3.0Δ – 104s

d) Maximum amplitude at 3.5Δ with a frequency change from 0.5Hz to 0.25Hz at 3.5Δ – 110s

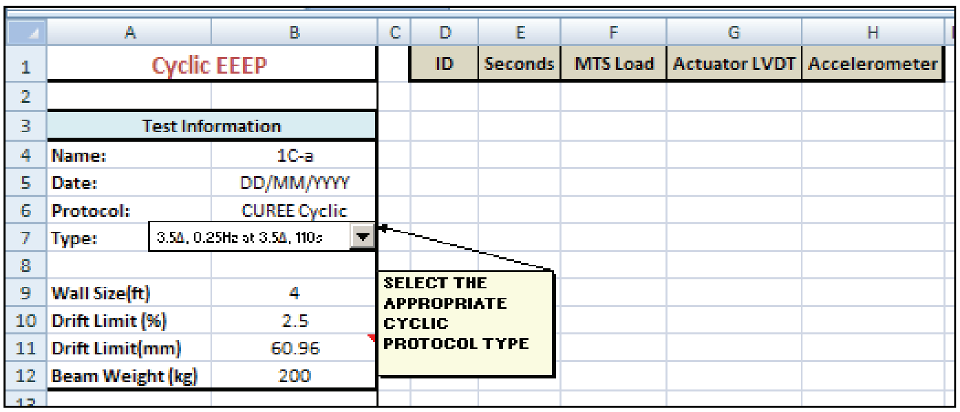


Figure L.5 Spreadsheet for CUREE Cyclic Test Analysis

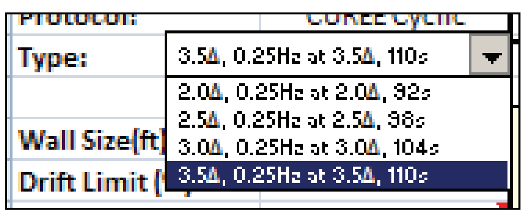
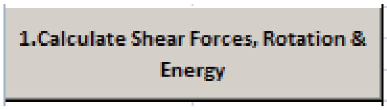


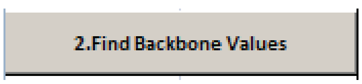
Figure L.6 Selection of CUREE Cyclic Test Frequency

After the data was placed in the appropriate columns and the corresponding CUREE Cyclic protocol type was selected for the wall specimen along with other relevant information, the command buttons were followed. To avoid confusion, the buttons were labeled to identify a sequence.

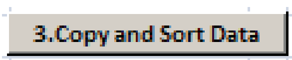
1. 

This step was the same as in the Monotonic procedure. However, the accelerometer readings were taken into account as well as the weight of the top loading beam. The beam weight was automatically adjusted for the wall size. When the wall width was 610mm or 1220mm, the beam weight was 200kg where as for a wall width of 1830mm or 2440mm, the beam weight was 250kg.

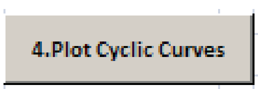
$$Force\ per\ M = \frac{MTS\ Load \pm (|Acceleration| \cdot g \cdot BeamWeight)}{L} \qquad \text{(L-10)}$$

2. 

This step determined the peak load for each primary cycle on the positive and negative side and its corresponding displacement.

3. 

The values found from step 2 are sorted and placed in order for the positive and negative sides in a separate table.

4. 

The curves of the observed cyclic curve and the backbone obtained from determining the peak point for each primary cycle from steps 2 and 3 were plotted.

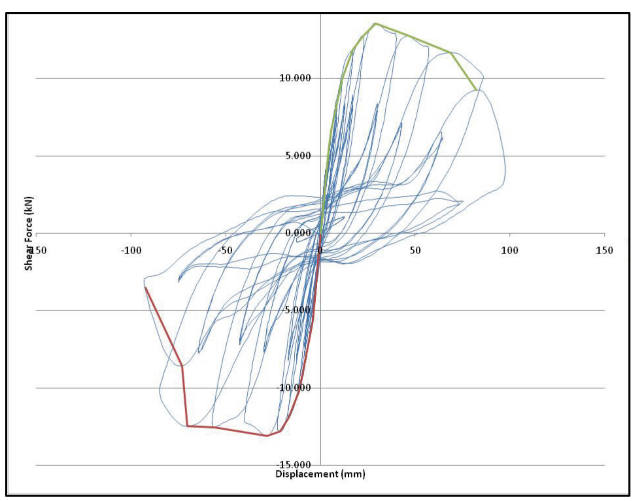


Figure L.7 Backbone Curve for CUREE Reversed-Cyclic Test Data

5. User input and manual manipulation was required in Step 5. In some cases, the backbone curve was not smooth and manipulation of the data points was necessary (Figure L.8 and L.9). A polynomial trend line was applied to the backbone curve which was defined by the user (Figure L.1).

6. 

Due to the limitations of Excel™, the maximum available trend line was a sixth order polynomial. The evaluation process involved the use of the trend line curve to obtain parameters relevant to the cyclic tests.

Trendline Data	
Force	Displacement
-3.50	-92.50
-8.61	-73.24
-12.49	-70.38
-12.56	-55.91
-13.12	-28.21
-12.82	-21.26
-11.95	-16.92
-10.19	-11.38
-6.54	-5.67
-5.64	-4.08
0.00	0.00
0.00	0.00
5.42	4.26
6.68	5.65
9.98	11.32
11.84	16.84
12.70	22.30
13.54	29.00
12.75	46.71
11.66	68.75
9.23	82.32

	Trendline Data	
	Force	Displacement
	-3.50	-92.50
new	-10.6	-84.5
	-12.49	-70.38
	-12.56	-55.91
	-13.12	-28.21
	-12.82	-21.26
	-11.95	-16.92
	-10.19	-11.38
	-6.54	-5.67
	-5.64	-4.08
	0.00	0.00
	0.00	0.00
	5.42	4.26
	6.68	5.65
	9.98	11.32
	11.84	16.84
	12.70	22.30
	13.54	29.00
	12.75	46.71
	11.66	68.75
new	10.41	84.10
new	6.80	94.20

Figure L.8 Reversed-Cyclic Backbone Curve Modification

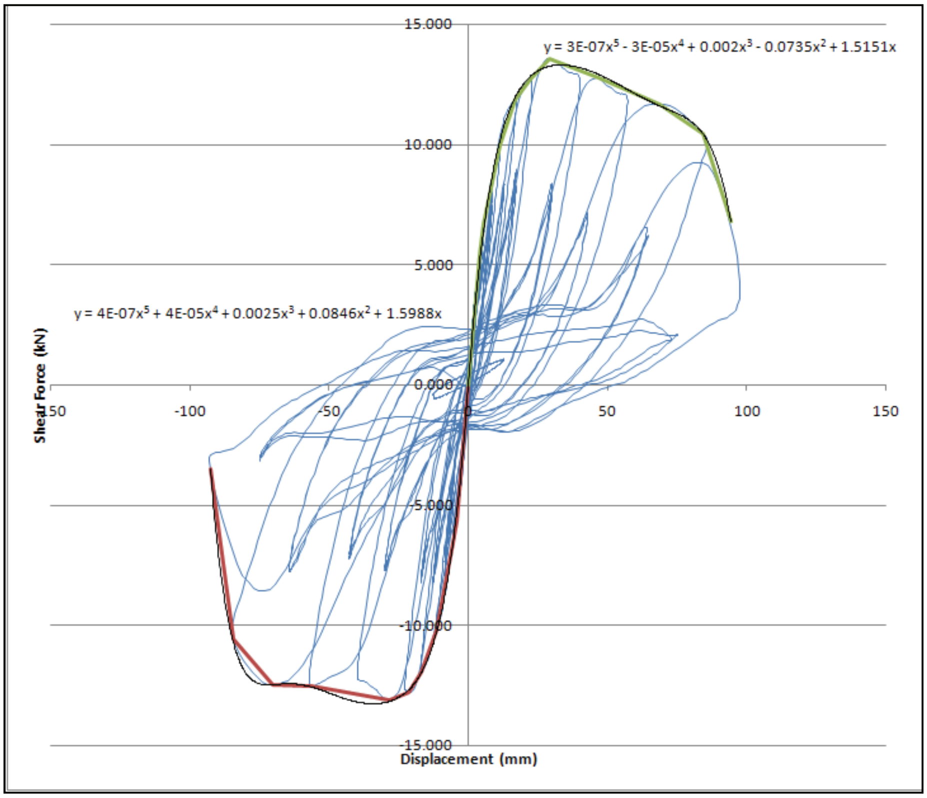


Figure L.9 Modified Backbone Curve

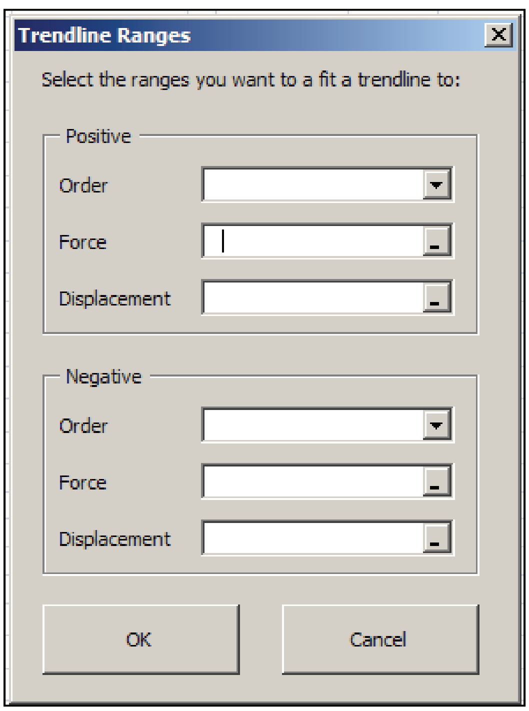


Figure L.10 Trend line Fitting User Input Window

Table L.2 Sample CUREE Reversed Cyclic Test Results Using the EEEP Analysis Approach

Parameters			Units
	Positive	Negative	
F [illegible]	7.43	-7.98	kN
F [illegible]	5.94	-6.38	kN
F [illegible]	2.97	-3.19	kN
F [illegible]	6.92	-7.19	kN
K [illegible]	1.10	1.03	kN/mm
Ductility (μ)	8.18	5.75	-
Δ [illegible]	6.29	-6.99	mm
Δ [illegible]	34.55	-22.59	mm
Δ [illegible]	51.40	-40.20	mm
Δ [illegible]	2.70	-3.10	mm
Area [illegible]	333.89	264.01	J
Area [illegible]	333.89	264.01	J
Check	OK	OK	-
R [illegible]	3.92	3.24	-
S [illegible]	5.68	-5.90	kN/m

Averages		
R_d	3.60	-
v_{yield}	5.79	(kN)/m

EEEP Bilinear Points			
Positive		Negative	
Displacement	Force	Displacement	Force
0.00	0.00	0.00	0.00
6.29	5.68	-6.99	-5.90
51.40	5.68	-40.20	-5.90

7. 6.Plot EEEP Curves The last step was used to confirm the results by viewing a plot of the curves. The "Plot EEEP Curves" allowed the user to view the created backbone curves and the EEEP bilinear curves for the positive and negative side of the hysteresis in a separate chart sheet.