CYCLIC PERFORMANCE OF COLD-FORMED STEEL STUD SHEAR WALLS

by

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Your file Votre référence
ISBN: 0-612-85904-5
Our file Notre référence
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To my beloved daughter

刘慕晴

ABSTRACT

Specific guidelines for the design of cold-formed steel stud shear walls following Canadian procedures are not available. In term of loads, design standards allow engineers to rely on an equivalent static approach in seismic design, for which force and deflection modification factors greater than one are specified if a structure can maintain load carrying capacity with significant inelastic deformation. However, the National Building Code of Canada (NBCC) does not define an R-value for these steel stud walls. Thus, an $R = 1.0$ must be used without consideration of their possible inelastic behaviour. The American Iron and Steel Institute (AISI) and Uniform Building Code (UBC), however, do provide design tables, which are limited to certain construction types, to determine the shear capacity of steel stud walls.

This thesis contains a summary of previous cold-formed steel stud shear wall test programs in North America, as well as an overview of the seismic requirements for a number of different design standards, *i.e.* the NBCC, the UBC and the NEHRP guidelines for seismic design. A theoretical method for the prediction of shear capacity based on the first possible failure mode, which follows the adapted American wood design procedure, is presented and the results from this method are compared with peak loads obtained from existing tests. In addition, a preliminary force modification factor for use in seismic design is suggested for use with the NBCC. Finally, future tests of cold-formed steel stud shear walls are proposed and a corresponding test frame is designed.

It is shown that the predicted shear capacity is generally in agreement with the peak load results measured from shear wall tests. In addition, a preliminary R-value of 2.0 for the NBCC is suitable for the seismic design of cold-formed steel stud shear walls sheathed with wood panels. Further studies, involving laboratory tests of shear walls, development of a Canadian design method, as well as time-history analyses of different design scenarios are recommended to be carried out to verify the findings described in this thesis.

RÉSUMÉ

Aucune directive spécifique pour l'analyse de mûrs de refends à montant en acier forgé à froid n'est disponible dans les codes canadiens. Pour les calculs sismiques de structures, les ingénieurs se basent sur les méthodes statiques équivalentes suggérées par les différents normes de conception. Ces méthodes comportent des facteurs d'amplification de charge et de déplacement pour les structures conçues pour des déformations inélastiques importantes si une structure peut maintenir la capacité de charge avec la déformation non élastique significative. Cependant, pour l'analyse de ces murs de refends à montant en acier, le code national du bâtiment du Canada (*CNBC*) ne définit aucune valeur du coefficient R, donc, une valeur de R=1.0 est ultilisée avec aucun considération des effets inélastiques. D'autre part, les normes établies par la ***American Iron and Steel Institute (AISI)*** ainsi que le ***Uniform Building Code (UBC)*** présentent plusieurs valeurs de conception pour déterminer la capacité de cisaillement effectué sur les mûrs de refends à montant en acier forgé à froid mais ceux-ci limitées à certains types de construction.

Ce mémoire présente un sommaire des essais effectués sur les mûrs de refends à montant en acier forgé à froid jusqu`à ce jour en Amérique du Nord. Un aperçu des critères de conception sismique des normes: ***CNBC, UBC,*** et ***NEHRP guidelines for seismic design*** est également présenté. Une approche théorique est présentée, s'inspirant de la procédure décrite dans les critères de conception américains pour les structures de bois (***American Wood Design***), qui permet de prédire la résistance en cisaillement en se basant sur le premier mode de rupture du système étudié. Les résultats obtenus à l'aide de cette dernière approche sont comparés aux charges maximales des résultats d'essai. Aussi, des facteurs de modification de charge et de déplacement préliminaires pour l'utilisation dans l'analyse sismique sont fournis. Finalement, une série d'essais futurs sont proposés et un cache rigide en acier a été concu pour ces essais.

Il a été démontré que la capacité prévue de refends est généralement en accord avec la charge maximale des résultats d'essai. De plus, pour une analyse sismique, une valeur préliminaire du coefficient R (R=2.0) est suggérée au CNBC pour des mûrs de refends à montant en acier forgé à froid recouvert avec des panneaux en bois. Les études futures à effectuer incluent des essais en laboratoire, le développement de critères de conception canadiens, ainsi que des analyses sismiques numériques utilisant différents scénarios de charge afin de vérifier les résultats obtenus dans cette recherche.

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CHAPTER 1 INTRODUCTION

1.1 GENERAL OVERVIEW

Cold-formed steel assemblies can be used economically in low-rise buildings, residential housing and other light structures. Cold-formed steel members are fabricated in a wide range of geometric shapes through a manufacturing process that usually involves roll forming at room temperature, and in some cases utilizes a press brake or bending brake forming operation. The thickness of cold-formed steel sheets and strips ranges from 0.3 mm to 25 mm, although the most common sections are made from 0.9 mm to 2 mm thick material. Traditionally, there are two major cold-formed steel structural configurations: 1) Individual framing members with common shapes such as C-section, Z-section, angles, hat section, I-section, T-section and tubular section. These members are commonly specified as the secondary structural components in a building whose main structure is composed of hot-rolled steel or reinforced concrete sections. Additionally, these individual sections can be used as chord and web members of open web steel joists, space frames, arches, storage racks, *etc.* 2) Panels and decks; for example, roof and composite floor decks, wall panels, siding material, stay-in-place forms, *etc.* Deck sections often serve a dual purpose of resisting both gravity and lateral loads, as well as acting as a surface for flooring, roofing and concrete fill *(Yu, 2000)*.

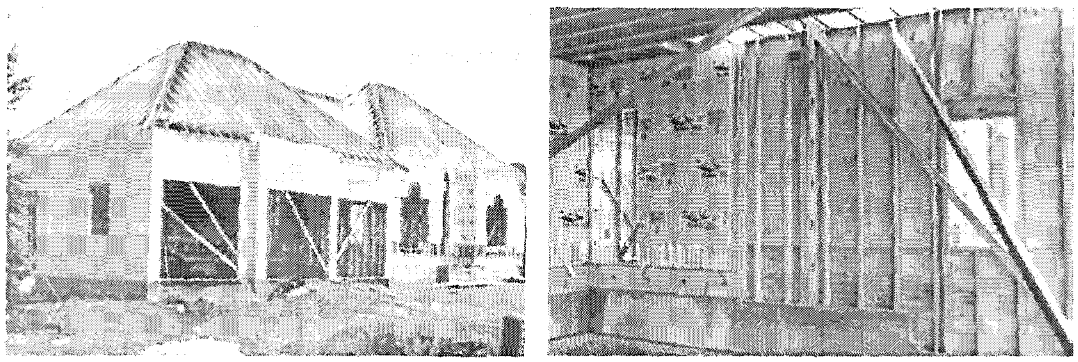


Fig. 1.1: Cold-formed Steel Stud House with Shear Walls (left-most garage wall)

More recently, cold-formed steel members have been utilised to construct the main structure of low-rise buildings, in the form of wall, roof and floor assemblies, as shown in Fig. 1.1. This includes non-loadbearing wall construction (only relied on to carry lateral loads) and loadbearing wall construction (carries gravity and in some cases lateral loads).

Shear walls have long been used as the lateral force resisting system that withstands loads caused by wind and seismic events; for example, hurricanes and earthquakes. Wood-framed buildings, which contain shear walls, historically have been the most popular form of housing in North America because wood has been readily available, cost effective, good for insulation, and construction methods have been well developed. Recently, however, the use of cold-formed steel stud framing in homes and multiple-storey buildings has increased due, in part, to the escalating construction costs of wood structures, the scarcity of adequate wood products, and in addition because of concerns such as pest resistance, product quality, *etc*. The beneficial characteristics of cold-formed steel members include: high strength and stiffness; ease of prefabrication and mass production; accurate detailing; ability to retain their size and shape at ambient temperature and humidity levels; termite-proof and rot proof; uniform quality; non-combustibility; and fabricated from recyclable material.

Similar to wood framed buildings, cold-formed steel structures are often built using the platform framing technique, where floors, roofs, and walls are composed of individual joist, rafter, and stud members. Under seismic ground motion, horizontal inertia forces develop at the roof and floor levels as a result of the horizontal accelerations experienced by the building mass. To resist these lateral loads the structure may include diagonal steel bracing, plywood sheathing, oriented strand board sheathing, gypsum wallboard or sheet steel sheathing in the walls. C-shape studs are commonly installed back-to-back at both ends of a wall segment to resist the high tension/compression forces due to overturning. Top and bottom tracks, which complete the wall system framing, are typically connected to studs with self-drilling/tapping screws. Sheathing and/or diagonal steel straps are then attached to the face of the wall to develop adequate lateral shear capacity and to maintain the structural integrity of the building. A typical cold-formed steel shear wall is illustrated in Fig. 1.2. The lateral loads from the roof and floor levels are eventually transferred to the foundations, by means of hold-downs and shear anchors.

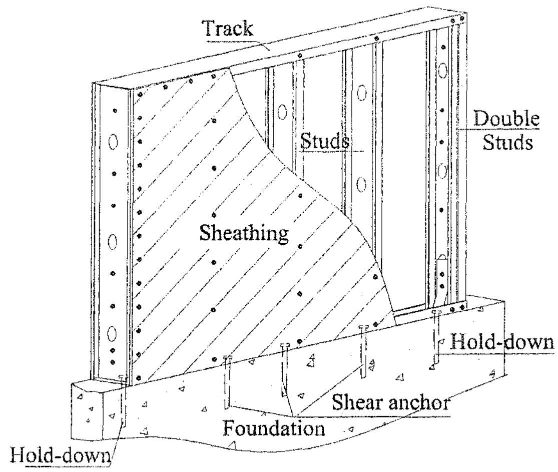


Fig. 1.2: Typical Cold-Formed Steel Shear Wall

1.2 STATEMENT OF PROBLEM

In the future, the construction of steel stud framed buildings will, in all probability, increase across North America, including earthquake prone areas such as the West Coast. Hence, it is of great importance to design engineers and contractors, as well as building owners and occupants, that the lateral performance characteristics of steel stud structures are understood and that construction and design procedures are adequate before their use in areas of known earthquake activity becomes widespread. The behaviour of steel stud wall assemblies is dependent on several variables including load type and duration, stud geometry and spacing, fasteners, bracing, sheathing, architectural details, *etc*, and hence, shear wall design procedures are generally based on the results of test programs. For the most part, researchers of cold-formed steel structures have focused their investigations on the monotonic performance of buildings without great concern for changes in behaviour caused by seismic loading. However, the American Iron and Steel Institute (AISI) has published prescriptive design tables with seismic load values based on cyclic tests for a number of construction configurations *(AISI, 1998)*. Methods for the design of lateral bracing wall systems for either static (wind) or seismic load resistance have not been extensively documented in Canada. For example, the National Building Code (NBCC) *(NRCC, 1995)* must be utilised when designing cold-formed steel structures in seismic areas, however, the code does not specifically address their intricate design in terms of shear strength and the force modification factors (*R*-value) used in the equivalent static approach for earthquake loading. Therefore, it is necessary that research concerning the cyclic performance (strength and ductility characteristics) of steel stud shear walls be carried out.

1.3 Objectives

The objectives of this thesis include an examination of the literature and a collection of existing data and information that involves the testing and design of steel stud shear walls. The second objective is to derive a numerical method with which the shear strength of steel stud walls can be estimated. Thirdly, existing test data is to be used to evaluate various lateral force design methods and to determine preliminary force modification factors for use in seismic design. The final objectives are to provide recommendations for cold-formed steel stud shear wall tests and to design a corresponding test frame.

1.4 Thesis Outline

This thesis, which consists of four main parts, is a preliminary study to evaluate the strength and ductility of cold-formed steel stud shear walls. A review of previous tests on cold-formed steel shear walls in North America is given in Chapter 2. In Chapter 3, a numerical method to predict shear strength dependent on various failure modes of steel shear walls is described and a comparison with test results is presented. The contents of Chapter 4 focus on the cyclic performance of steel stud walls. A possible procedure for determining ductility factors from quasi-static tests is explained and parameter studies are carried out. In Chapter 5, future tests of steel walls are proposed and also, a description of a shear wall test frame, along with information on its design is provided.

Chapter 2 Literature Review

In North America, the design of cold-formed steel members is carried out following the procedures provided in the CSA-S136 Standard *(1994)* or the AISI Specification *(1999)*. The design methods contained in these documents are for the most part based on research that encompassed the behaviour of monotonically loaded components of a structure. Serrette and Ogunfunmi *(1996)* noted that prior to 1996 no published standard existed that provided guidelines on the shear design of cold-formed steel wall stud assemblies. The AISI published a prescriptive shear wall design guide in 1998 *(AISI, 1998)* based on information provided by test programs carried out by Tissell *(1993)*, Serrette *(1994)*, and Serrette *et al. (1996b, 1997b)*. Subsequently, a draft design guide (*AISI, 2002)* was written to provide additional code approved strength values, and to address the issues of aspect ratio and perforations on the capacity of shear walls. However, no shear wall design method has been provided in standard or guide form in Canada.

A number of experimental research programs have been conducted to investigate the behaviour of cold-formed steel shear walls with different sheathing material, and to provide design capacities for a variety of wall types under lateral force *(Gad et al., 1997, 1998, 1999a,b,c, 2000; McCreless and Tarpy, 1978; NAHB, 1997; Salenikovich et al., 1999; Serrette, 1994, 1997; Serrette & Ogunfunmi, 1996; Serrette et al., 1996a,b, 1997a,b; Tarpy, 1980, Tarpy and Hauenstein, 1978; Tarpy and Girard, 1982; Tissell, 1993; COLA-UCI, 2001)*. Studies on the seismic behaviour of wall assemblies with diagonal strap bracing have been completed in Australia by Gad *et al.*. These tests were carried out on house structures and components built using Australian construction practice. The steel type, stud profiles, connection and blocking details, and the diagonal strapping systems that were used are considerably different from those found in North America, hence, these results are not directly applicable for use in housing design in this country. Table 2.1 provides a general listing of the existing tests of steel stud shear walls constructed following North American practice and subjected to lateral loading. A more comprehensive listing of existing test program information can be found in Appendix 'A'. Summaries of the relevant research projects are provided in the following section.

Table 2.1: Listing of North American Tests of Steel Stud Shear Walls

	Tarpy & Girard	**Salenikovich *et al.***	**Serrette *et al.***					
Type of walls	GWB, Ply.	OSB, GWB (with openings)	OSB	Ply.	GWB	X - brace	Steel sheet	FB
No. of tests	14[1]	16[1] 10[2]	47[1] 18[2]	24[1] 16[2]	13[1]	15[1] 4[2]	12[1] 6[2]	5[1]
	NAHB	**McCreless & Tarpy**	**Tissell**	**Tarpy**	**Tarpy & Hauenstein**		**COLA - UCI**	
Type of walls	OSB, GWB (with opening)	GWB	OSB, Ply.	GWB	GWB		Ply, OSB.	
No. of tests	4[1]	16[1]	8[1]	12[1] 8[2]	18[1]		12[1] 12[2]	

[1] Denotes total tests. [2] Denotes cyclic tests. GWB: gypsum wallboard; OSB: oriented strand board; Ply.: plywood; FB: FiberBond wallboard

2.1 Summary of Existing North American Test Programs of Steel Stud Shear Walls

A brief summary of the various research programs involving the testing of steel stud shear walls is contained in this section. Only those studies relevant to the monotonic and cyclic performance of walls subjected to lateral loading and constructed following North American procedures have been included.

McCreless & Tarpy *(1978)*

The objectives of the research program were to determine the effect of various aspect ratios (height/length) on the shear strength and shear resistance of steel stud wall systems, and also, to determine the degree of possible panel distortion before major wall panel damage. McCreless & Tarpy also investigated whether the addition of a single horizontal stiffener, *i.e.* blocking between stud members, located at mid-height in the plane of the wall, could increase the shear capacity of the wall system.

The experimental program consisted of the testing of sixteen full size wall panels with varying aspect ratios under monotonic load. A series of interior wall panels of 8' (2.44 m) – 12' (3.66 m) heights and 8' (2.44 m) – 24' (7.32 m) lengths were tested. Each wall panel was constructed of 3-½" (89 mm), 20 gauge Super C studs (Alabama Metal Industries Corporation) spaced at 24" (610 mm) o.c., which were attached to 3-5/8" (92 mm) web by 1-½" (38 mm) flange, 20 gauge structural track with No. 10 × ½" (12.7 mm) low profile head screws. Gypsum wallboard, ½" (12.7 mm) thick, was attached to both sides of the stud assembly using No. 6 × 1" (25.4 mm) bugle head screws spaced at 12" (305 mm) o.c. over the entire face of the panel along both studs and runner tracks. The tests were carried out under the requirements of ASTM E 564 – 76 *(1976)*.

McCreless & Tarpy noted that flexural deformation controlled the behaviour of all of the test walls, except for the longer specimens where shear deformation had more of an influence. In the case where shear deformation was more prevalent, edge screws rotated through the gypsum wallboard in the direction of loading, and finally, the stud framing sheared through the wallboard. When deflection due to bending was observed, the wall base track deformed around the clip angle at the exterior tension corner and then the screws in the tension corner rotated, followed by cracking separation of the wallboard. The shear strength of the wall panels, calculated by McCreless & Tarpy, was generally independent of the aspect ratio. The shear stiffness computed from the net deflections and total deflections increased for the shorter height and longer walls.

McCreless & Tarpy concluded that steel stud wall panels could be used as lateral load-resisting elements in building construction, provided that appropriate factors of safety and anchorage details were maintained. Using screw fasteners to attach the stud to the track provided the walls with higher capacity in comparison with using either resistance spot welds or friction connections. It was also believed that increasing gypsum wallboard attachment points around the perimeter increased the wall capacity and stiffness. Adding a horizontal stiffener at mid-height was not recommended since it increased the construction difficulties and cost without a corresponding improvement in the shear capacity of the wall.

Tarpy & Hauenstein *(1978)*

The primary objectives of the test program included determining the effect of different construction and anchorage details on the shear resistance of framed steel-stud/gypsum-wallboard partitions, and quantifying the damage threshold load level. The secondary objective was to determine how steel-stud/gypsum-wallboard shear resistance values compare to wood-stud/gypsum-wallboard values.

Eighteen full-scale walls with seven different types of wall panel construction and anchorage details were tested. One wall type consisted of 2 × 4 wood-studs, while others were constructed of 3-½" (89 mm) web by 1-½" (38 mm) flange by 0.032" (0.81 mm) thickness steel "C" studs spaced at 24" (610 mm) o.c. (lip dimension not provided). Gypsum wallboard, ½" (12.7 mm), placed in the horizontal position was attached to both sides of the stud frame. The fasteners were No. 6 × 1" (25.4 mm) bugle head screws for the steel frame and 1-3/8" (35 mm) annular head nails for the wood frame.

Tarpy & Hauenstein concluded from the test results that framing wall panels with steel C-studs and gypsum wallboards is a feasible way to construct vertical diaphragms (shear walls) that resist lateral in-plane shear loads. They also recommended that a positive attachment should be furnished between the track and floor framing system in case of uplift loading. A reduction of the fastener spacing around the wall perimeter could provide larger shear strength. In addition, the wall panel diaphragm should possess no less ductility than the gypsum-paper-wallboard material. A safety factor of 2.0 was recommended for design purposes to ensure that the design load level does not exceed the damage threshold load level.

Tarpy *(1980)*

Nine different types of wall panel construction and anchorage details were tested under monotonic and cyclic loading protocols for this research program. The parameters that were considered included: the effect of wall panel anchorage techniques, gypsum wallboard thickness, sheathing materials, monotonic versus cyclic loading conditions, gypsum wallboard fastener spacing, and the use of a diagonal corner brace. The objectives were:

(1) To determine the effect of different construction techniques and anchorage details on the shear capacity of steel-stud shear walls with different types of sheathing and plaster.
(2) To determine the threshold for damage of the walls due to lateral in-plane displacement.
(3) To determine the effect of cyclic loading versus monotonic unidirectional loading on the shear capacity.

The wall panels consisted of 3-½" (89 mm) × 1-½" (38 mm) × ½" (12.7 mm) 20 gauge (0.0359": 0.91 mm) steel studs spaced at 24" (610 mm) o.c.. The studs (with a mean yield strength of 48 ksi: 331 MPa from coupon tests) were attached to 3-5/8" (92 mm) × 1-½" (38 mm) 20 gauge tracks (0.0359": 0.91 mm) with No. 10 × ½" (12.7 mm) low profile head screws. The sheathing was attached to the frame in different scenarios were the fastener type, as well as perimeter and field spacing were varied. For example, No. 6 × 1" (25.4 mm) bugle head screws spaced at 12" (305 mm) o.c. except for the type C walls, where the perimeter spacing was reduced to 6" (152 mm). In other cases, No. 6 × 1" (25.4 mm) bugle head screws spaced at 24" (610 mm) o.c. and No. 6 × 3/8" bugle head screw spaced at 12" (305 mm) o.c. were used to connect the sheathing to the frame. Also, No. 6 × ½" (12.7 mm) pan washer head screws with a spacing of 7-3/4" (197 mm) were used in type I walls. The number of tests for each wall type was based on the requirements of ASTM E564 – 76 *(1976)*.

Tarpy concluded that the wall panels framed with C-shaped steel studs and gypsum are practical in terms of resisting lateral shear loads. The corner anchorage did influence the shear stiffness and threshold load level dramatically, which was seen by the significant decrease in the shear strength when corner angles (hold-down device) were replaced with bolt and washer anchors. Densely spaced powder actuated fasteners (connected to a supporting concrete beam) provided similar restraint to the corner angles, and thus the walls constructed with these two kinds of anchorage exhibited similar shear capacities. The author further argued that the shear resistance did not vary extensively when using different types of interior shear anchorage. The use of two layers of gypsum wallboard increased the shear capacity of the wall, while decreasing the shear stiffness, in comparison with single layer systems. The use of cement plaster over the surface of the wall resulted in an increase of the shear strength and stiffness. Cyclic load weakened the wall panels by decreasing the damage threshold load level and the ultimate shear strength. Finally, Tarpy stated that a decrease in the spacing of fasteners increased the panel shear capacity and stiffness. Adding a corner brace (a 45° stud placed at the bottom corner between the chord members and the adjacent stud) had little effect on the ultimate load capacity.

Tarpy & Girard *(1982)*

This project consisted of the testing of different of wall panel and anchorage construction techniques using monotonic lateral loading procedures. The overall objectives were:

(1) To determine the effect of different construction techniques and anchorage details on the in-plane shear resistance of steel-stud shear walls with different types of sheathing.
(2) To determine the threshold for damage in the walls due to in-plane displacement.

The wall panels were constructed with 3-½" (89 mm) × 1-½" (38 mm) × ½" (12.7 mm) 20 gauge (0.0359": 0.91 mm) C-shape studs and gypsum wallboard (½": 12.7 mm) attached to both sides of the stud frame (some wall specimens were combined with ½": 12.7 mm exterior gypsum sheathing). The parameters considered in this study were:

a) The effect of using light gauge clip angles and powder actuated fasteners in place of bolts and washers to anchor the base of the wall panel.
b) The effect of anchoring the wall panel through transverse floor joists.
c) The effect of plywood or gypsum exterior sheathing in place of gypsum wallboard as a diaphragm material.

d) The effect of using fillet welds instead of self-drilling screws to attach the studs to the runner tracks.
e) The effect of using a 400 mm (16") rather than a 600 mm (24") stud spacing.

It was observed that all of the wall panels experienced the same basic failure mode. The tracks deformed close to the anchorage device at the uplift corner of the wall and the gypsum wallboard cracked along the panel edge at the corner fastener location. Tarpy & Girard concluded that when the bolt and washer anchorage detail was used without clip angles the shear capacity of the wall decreased, while the use of closely spaced powder actuated fasteners negligibly increased the shear capacity in comparison to using corner clips. The wall, when anchored through floor joists, exhibited lower shear resistance and stiffness than when connected directly to the test frame. In addition, Tarpy & Girard indicated that the use of plywood sheathing increased the shear capacity to a great extent over that which was reached with gypsum wallboard. The use of welded stud to track connections provided the same shear strength as self-drilling screw connections. A decrease in the stud spacing resulted in a slight increase in the shear resistance.

Tissell *(1993)*

The tests completed by Tissell were carried out to provide information on the influence of fastener size and spacing, along with framing thickness on shear wall capacity. Eight specimens, sheathed with either oriented strand board (OSB) or plywood, with steel stud thickness values of 14-ga., 16-ga., and 18-ga. were included. Screw sizes were No. 10 for the 14-ga. and 16-ga. studs, and No. 8 for the 18-ga. studs, although a different fastener schedule was used for the later case.

The predicted design shears were calculated based on the assumption that No. 10 fasteners in 14-ga. studs were able to fully develop the recognised shear capacity for a wood framed wall with 10d nails, No. 10 in 16-ga. for 8d nails, and No. 8 in 18-ga. for 6d nails. Tissell reported that most of the walls failed due to buckling of the end studs or the bottom track at the anchor bolts before the full shear capacity of the panels could be developed, and thus the tests did not provide a true indicator of the capacity of the sheathing panels. Tissell also recommended that additional tests should be completed to refine the method of determining design shear values for steel stud shear walls.

Serrette *et al.* *(1996a,b)* & Serrette *(1997)*

The purpose of this experimental research program was to investigate the behaviour of light gauge steel framed shear walls sheathed with plywood, oriented strand board (OSB), and gypsum wallboard (GWB). The test program was divided into three phases. The main objective of Phase 1 was to investigate differences in the monotonic behaviour of plywood (15/32" or 11.9 mm) APA rated 4-ply plywood sheathing *(APA, 1991)* and OSB (7/16" or 11.0 mm APA rated OSB sheathing) shear walls. Phase 2 focused on the behaviour of OSB walls with dense fastener schedules, wall panels with OSB on one side and GWB panels on the other side, as well as walls with GWB panels on both sides using monotonic load protocols. The final phase included the cyclic testing of OSB and plywood walls using various fastener schedules.

The authors concluded that the overall behaviour of the plywood and OSB panel assemblies was practically identical for both the monotonic and cyclic tests, while, in general the plywood walls carried slightly higher loads. The walls with different aspect ratios were observed to have the same shear capacity if a consistent panel orientation was used. The OSB walls with a dense fastener schedule could resist higher shear loads with lower displacements compared with the walls constructed using a smaller number of screws. The authors also indicated that blocked walls with panels perpendicular to the studs provided higher shear capacity than those with panels parallel to the framing (Fig. 2.1) from the test results. The walls with OSB panels on one side and GWB on the other side exhibited similar failure behaviour, but degraded more gradually than the walls with OSB on one side alone in terms of shear capacity. Gypsum wallboard improved the strength and stiffness of the wall only for the case where a sparse fastener schedule was specified.

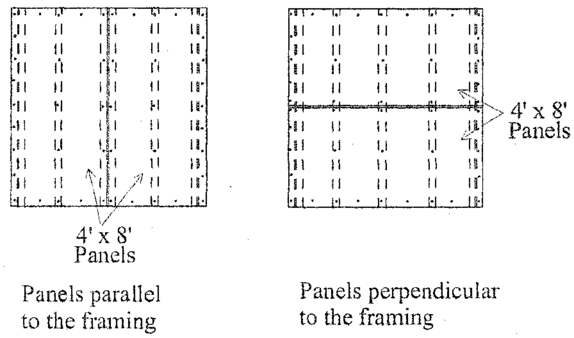


Fig. 2.1: Orientation of Panels

<u>Serrette & Ogunfunmi</u> *(1996)*

In this investigation, the main objectives were to study the contribution of flat strap tension X-bracing, gypsum sheathing board (GSB), gypsum wallboard, and the combination of X-bracing, GSB, and GWB to the in-plane shear resistance of steel stud walls. GWB is a building panel mainly consisting of a noncombustible gypsum core material sandwiched between two durable paper faces, and GSB combines a treated gypsum core with a water-repellent face paper to improve structural strength and increase the resistance to weather and fire. A total of thirteen 8' × 8' (2.44 m × 2.44 m) shear walls, with three different sheathing types, were tested under monotonic load conditions. The studs, spaced at 610 mm (24") o.c., were C-shaped 150 mm (6") 20 ga. (0.88 mm) MSMA 600IC20 sections, while the MSMA 600ST20 tracks had the same width and thickness as the studs. The different shear walls included: Type A – framed wall with 50.8 mm (2") wide, 0.88 mm (20-ga. or 0.0346") flat strap X-bracing on the face; Type B – framed wall with ½" (12.7 mm) single-ply gypsum wallboard on the back and ½" (12.7 mm) single-ply gypsum sheathing board on the face; and Type C – framed wall with ½" (12.7 mm) single-ply gypsum wallboard on the back and ½" (12.7 mm) single-ply gypsum sheathing board and 20-ga. (0.88mm or 0.0346") flat strap X-bracing on the face.

It was reported that for Type A walls, failure resulted from excessive lateral deflection which followed yielding of the tension X-bracing. For Type B and C walls, screw rotation occurred at the perimeter edges when half of the sustained maximum load was reached, and final failure was always governed by the gypsum wallboard in the form of breaking/cracking of the paper cover and underlying gypsum due to its tapered edges. Type B walls provided approximately 2.38 times the shear capacity of Type A walls, and Type C walls were 3.06 times stronger than Type A walls. The authors stated that this demonstrated the significant contribution to shear capacity from the gypsum panels. The use of steel straps in Type C walls reduced the permanent deflection of the wall and increased the load capacity without enhancing the shear stiffness. The contribution of the studs to overall wall shear resistance was relatively small and the lateral resistance of the intermediate studs was not considered.

Serrette and Ogunfunmi concluded from the results of the monotonic tests that the gypsum board provided significant shear strength to the wall. In addition, although flat strap tension braced walls have high shear strength, the use of straps plus gypsum sheathing is not practical due to two drawbacks: (1) the need to pretension the straps, and (2) the need for additional screws to connect the straps.

<u>NAHB Research Center</u> *(1997)*

The National Association of Home Builders (NAHB) Research Center tests were carried out to assess the suitability of using the perforated shear wall design method for wood structures with light-gauge steel-framed shear walls. A secondary objective was to provide a direct comparison between the performance of wood-framed and steel-framed shear walls. A total of four 40' × 8' (12.2 m × 2.44 m) shear wall specimens were tested under monotonic load. Three of the walls were constructed with typical details, although the sheathing ratio (*r*, as defined in Equation (2 – 1)) was varied as 1.0, 0.76, 0.48, respectively. An additional specimen, with a sheathing ratio of 0.76, was constructed without hold-down anchors; rather anchor bolts spaced at 6" (152 mm) o.c. were installed. All walls were constructed of 33-mil (0.033": 0.88mm) thickness stud and track sections. Exterior sheathing consisted of 7/16" (11.1 mm) OSB panels oriented vertically with No. 8 screws spaced at 6" (152 mm) o.c. along the perimeter and at 12" (305 mm) o.c. in the field of the panels. Interior sheathing was ½" (12.7 mm) GWB with No. 6 screws spaced at 7" (178 mm) o.c. along the perimeter and at 10" (254 mm) o.c. in the field.

$$r = \frac{1}{1 + \frac{A_o}{H \sum L_i}} \qquad (2-1)$$

where:

A_o = Total area of openings

H = Height of the wall

L_i = Length of the full height wall segment

Sugiyama and Matsumoto *(1994)* proposed an empirical equation for the shear capacity ratio, F (the ratio of the strength of a shear wall segment with openings to the strength of a fully sheathed shear wall segment without openings):

$$F = \frac{r}{3 - 2r} \qquad (2-2)$$

where r is the sheathing area ratio as defined in Equation (2 – 1).

It was reported in the NAHB document that all of the specimens with hold-down anchors underwent a similar mode of failure. Initially the screws began to pull through the GWB, and the OSB experienced cracking at the perimeter screw connections when the shear load reached its ultimate level. The wall without hold-down anchors also exhibited failure of the interior sheathing, although the OSB remained intact after completion of the test, except at the location of the first anchor bolt where the bottom track failed in bending due to uplift.

The NAHB also concluded that calculation of the shear capacity for perforated steel stud walls using the empirical equation by Sugiyama and Matsumoto *(1994)* multiplying the shear capacity of the fully sheathed walls without openings from test results appears to be valid, although it provides a conservative estimate. The steel stud shear walls exhibited similar lateral load resisting mechanisms to wood shear walls. Also, the use of hold-down anchors decreased uplift and improved the failure loads of the wall by allowing more sheathing fasteners to resist shear.

Serrette *et al.* *(1997b)*

A follow-up research program was initiated to provide a wider range of design options for steel stud shear walls and to clarify some of the test values recorded during the previous research programs *(Serrette et al., 1996a,b; Serrette & Ogunfunmi, 1996; Serrette, 1997)*. The following wall assemblies were included: flat strap X-braced walls, steel sheathed walls, high aspect ratio walls, and walls framed with 1.37 mm (0.054") and 1.09 mm (0.043") thick studs. The test program was divided into five phases, each with a different objective, as described below:

Phase 1 (Cyclic Tests) – To determine the wall capacity when chord stud buckling does not govern: 4' × 8' (1.22 m × 2.44 m), 15/32" (11.9 mm) plywood and 7/16" (11.1 mm) OSB wall assemblies framed with 0.033" (0.84 mm) studs (chords – 0.043" or 1.09 mm studs) and with fasteners at 3"/12" (76 mm/305 mm) and 2"/12"(51 mm/305 mm).

Phase 2 (Cyclic Tests) – To define the limits for framing member thickness for sheathing attached with No. 8 screws: 4' × 8' (1.22 m × 2.44 m), 15/32" (11.9 mm) plywood walls framed with 0.043" (1.09 mm) & 0.054" (1.37 mm) studs attached with No. 8 screws.

Phase 3 (Cyclic & Monotonic Tests) – To evaluate the performance of flat strap X-braced walls: 0.033" (0.84 mm) flat strap X-braced walls framed with 0.033" (0.84 mm) and 0.043" (1.09 mm) studs.

Phase 4 (Cyclic & Monotonic Tests) – To evaluate the performance of sheet steel shear walls: 0.027" (0.69 mm) and 0.018" (0.46 mm) sheet steel shear wall assemblies with aspect ratios of 2:1 and 4:1.

Phase 5 (Cyclic & Monotonic Tests) – To observe the behaviour of sheathed OSB and plywood walls with high aspect ratios: 2' × 8' (0.61 m × 2.44 m), 15/32" (11.9 mm) plywood and 7/16" (11.1 mm) OSB walls were constructed with various fastener schedules.

Serrette *et al.* concluded from the test results that the use of thicker and multiple chord studs for the plywood and OSB walls with dense fastener schedules permitted the assemblies to fully develop the shear capacity of the sheathing. No. 8 screws behaved well in the 1.09 mm

(0.043") studs but fractured in shear when 1.37 mm (0.054") studs were used. In the design of X-braced walls, the designer must be concerned with strap yield strengths in excess of the specified nominal values, which may result in connection or chord stud failure, and the eccentricity due to installing straps on one side of the wall only. The steel sheathed wall assemblies behaved in a ductile fashion without sudden decreases in load carrying capacity. Furthermore, the authors stated that the use of thicker steel sheathing increased the shear capacities, but the failure mode moved from rupture at the edges of the sheathing to screw pullout from the framing. High aspect ratio walls can carry comparable loads to the low aspect ratio walls at relatively large displacements, however the initial stiffness of the wall was reduced to low or near zero values after these large displacements.

Serrette *et al.* *(1997a)*

In this project, the main objective was to investigate the behaviour of the sheathing material and fasteners. Full-scale tests were designed to force failure in the sheathing, while small-scale tests were designed to evaluate failure at the connections. Two different set-ups were used in the full-scale tests: Type A and Type B. The type A frame was 2.44 m × 2.44 m (8' × 8') in size with galvanized 152 mm (6"), 20-ga. (0.88 mm or 0.0346") C-studs and tracks. The studs were spaced 610 mm (24") o.c. and the chord studs were made of two 152 mm (6"), 20-ga. (0.88 mm or 0.0346") studs placed back-to-back. The Type B frame was identical to Type A except that a 50.8 mm (2") wide 20-gauge (0.88 mm or 0.0346") steel flat strap was attached across the mid-height of the wall and that solid blocking was installed above the strap in the end bays. The type A and B frames were sheathed with plywood, OSB, gypsum wallboard, and FiberBond wallboard on either one side or both sides. The panels were oriented either parallel or perpendicular to the framing depending on the test.

Twenty small-scale walls consisting of single 610 mm × 610 mm (2' × 2') panels were constructed, with five specimens selected for the purpose of calibrating and investigating the test setup and procedure. C-shaped galvanized 152 mm (6"), 20-ga. (0.88 mm or 0.0346") studs with 41.3 mm (1-5/8") flanges were oriented horizontally with a single stud on top and double studs back-to-back on the bottom. The walls were sheathed with panels including 12.7-mm (1/2") gypsum wallboard, 11.1-mm (7/16") OSB, 12.7-mm (1/2") FiberBond wallboard, and 11.9-mm (15/32") plywood were attached with three No. 6 × 25.4-mm (1") bugle head self-drilling and self-tapping screws at the top stud (152 mm (6") o.c. spacing) and six at the bottom studs (102 mm (4") o.c. spacing in two rows).

The full-scale walls failed initially by fastener rotation (tilting) about the plane of the stud flange. The chord studs were subjected to local crushing at the bearing end in some tests. Otherwise, generally the walls failed either when the edges of the panels broke off at the screw fastener or when the panel pulled over the head of the screw (The walls with the No. 6 screws failed due to fracture of the screws). Tests on nominally identical walls, where only the fastener size was changed, revealed that the size of the fastener does affect the maximum attainable shear strength. The wall with blocked panels oriented perpendicular to the framing provided essentially the same capacity, although it was stiffer than a comparable wall with panels parallel to the studs. When blocking was omitted from the walls with perpendicular panels, the shear capacity of the wall was reduced by more than 50%. The use of GWB on the interior of the wall and plywood on the exterior resulted in a higher capacity, by approximately 18%, in comparison with the plywood wall. The nailed plywood wall exhibited about 42% less maximum strength than a similar wall with No. 8 screws. The nailed OSB walls provided essentially the same strength as the nail-fastened plywood walls. It was showed from the small-scale test results that both OSB and plywood could carry relatively high shear load while the plywood specimens provided greater stiffness and strength.

The authors concluded, from the results of the full-scale tests, that the shear values for plywood and OSB panels on steel studs are comparable, and that the strength of gypsum board sheathed walls is relatively low. From the results of the small-scale tests, it was shown that the plywood wall connections provided 23% higher capacity in comparison to the OSB specimens, while the capacities of the gypsum and FiberBond walls were much lower than those of the OSB and plywood specimens. Also, the papered edge of the GWB panel contributed significantly to the overall strength. The normalized shear values for small-scale tests are similar as those for full-scale tests and therefore, the small-scale tests were considered to be useful in a simple evaluation of the relative resistance of different full-scale wall assemblies.

Salenikovich *et al.* *(1999)*

The objective of the research program was to determine the effects of: (a) size of openings, (b) cyclic loading, and (c) gypsum drywall sheathing and steel framing on shear wall performance, and to compare the strength of the walls with predicted capacities.

All specimens were 12.2 m (40') long and 2.44 m (8') in height with consistent framing, sheathing, fasteners, and fastener schedules. The test program consisted of five different scenarios, where walls with different sheathing area ratios were subjected to both monotonic and cyclic loads. The predicted shear capacities for perforated steel stud walls were calculated as shear capacity ratios, F, from Equation (2 – 2) multiplying the shear capacities of the fully sheathed walls without openings obtained from test results. The Equation (2 – 2) was derived based on the results of monotonic racking tests on 1/3-scale walls and is applicable for apparent shear deformation angles of 1/100 radians or less and for ultimate capacity.

Salenikovich *et al.* concluded the following:

a) A comparison of test results with the design equation recommended by Sugiyama and Matsumoto revealed that a conservative prediction of steel stud wall shear resistance at all levels of monotonic and cyclic loading can be obtained.
b) Long, fully sheathed walls had significantly higher stiffness and greater shear capacity, but were less ductile than walls with openings.
c) Cyclic loading did not influence the elastic performance of the walls but did reduce their deformation capacity.
d) The strength of fully sheathed walls was affected more significantly by cyclic loading than were walls with openings.
e) Gypsum wallboard increased the elastic stiffness and strength of the fully sheathed walls under monotonic load.

COLA – UCI *(2001)*

This experimental program included four groups of shear walls sheathed with plywood and OSB panels that were attached to either light gauge steel stud framing or wood stud framing with different fastener schedules. The test results were used to develop experimental shear strength values for light-gauge steel-framed walls and to compare the cyclic response of steel-framed walls and wood-framed walls.

A total of twelve 8' × 8' (2.44 m × 2.44 m) shear wall specimens were tested under cyclic load. Panels included: 7/16" (11.1 mm) OSB or 15/32" (11.9 mm) plywood (APA rated sheathing - Structure I), which were attached to the steel stud framing with No. 8 × 1" (25.4 mm) bugle head screws spaced 12" (305 mm) o.c. in the field and 6", 4", or 2" o.c. along the edge. The 20-gauge (0.033": 0.84 mm) C-shaped steel studs, with a 3.5" (89 mm) web, a 1.625" (41.3 mm) flange and a 0.375" (9.5 mm) lip were connected to the 20-gauge, 3.5" × 1.50" (89 mm × 38 mm) steel tracks using No. 8 × ½" (12.7 mm) modified truss screws. Double steel studs were attached back-to-back at the end of the wall with No. 10 × ¾" (19 mm) hex washer head self-drilling and self-tapping screws to prevent local and flexural buckling in the chords.

The test results showed that a more dense fastener schedule would nonlinearly increase the shear capacity and stiffness for both the light-gauge steel and wood-framed stud walls. Generally, with the same sheathing and fastener spacing, steel-framed shear walls exhibited somewhat higher shear capacity than the wood-framed walls. The results also revealed that in comparison to wood shear walls, steel-stud-framed walls had higher overstrength and ductility factors, but less hysteretic damping.

2.2 Summary of Existing Test Programs of Steel Stud Shear Walls Outside of North America

In this section, the summaries of some existing test programs of steel stud shear walls completed outside of North America are presented.

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

These papers present the findings of investigations into the behaviour of Australian domestic structures constructed with cold-formed steel stud walls. The contribution of plasterboard to the lateral resistance and seismic design of the shear walls was identified, and a comparison of the laboratory-based tests with field tests using modal analysis was provided.

The majority of test specimens consisted of one-room-houses measuring 2.3 m × 2.4 m × 2.4 m high, which were constructed from full-scale components. The test houses were subjected to specified simulated earthquakes in two directions using a shake table. A concrete slab with a mass of 2350 kg was added to the top of the house to simulate the mass of the roof tiles, battens, insulation, ceiling lining and trusses. The framing members were standard Grade 550 C-sections (F_y = 550 Mpa) (75 mm × 35 mm × 1 mm: 3.0" × 1.375" × 0.04"). A 10 mm thick plasterboard lining was connected to the ceiling and walls with No. 6 × 25mm (1") bugle head screws. Metal clip-on brick ties were used to connect the brick veneer walls to the studs.

Gad *et al.* concluded that the overall lateral performance of the steel stud structures was influenced by many components, such as steel strap bracing, plasterboard and boundary conditions. The plasterboard (non-structural component) enhanced the shear stiffness, shear capacity, damping and energy absorption capacity of the structural system. The in-plane brick veneer walls did not improve the stiffness of the system. The boundary conditions, in particular the return walls, dramatically increased the load carrying capacity of the plasterboard-lined structure and provided a larger overstrength factor for the system.

2.3 AISI Design Guide for Steel Stud Shear Walls

AISI *(1998)*

This steel stud wall design guide for lateral loads incorporates the results of tests carried out by Serrette *(1994)*, Serrette *et al.* *(1996b, 1997b)* and Tissell *(1993)*. Nominal prescriptive shear strength values for walls of different construction, most of which consist of 3.0" × 1-5/8" studs attached to the tracks with self drilling screws, were included in this document. Examples include: 1) plywood and oriented strand board (OSB) on the exterior wall surface, with or without gypsum wall board (GWB) on the interior wall surface, 2) GWB on both surfaces, 3) steel sheathing on one side, and 4) steel X-bracing on one side. Some of the values in the tables of the guide have been approved by different American model codes, *e.g.* UBC *(ICBO, 1997)* and IBC *(ICC, 2000)*.

AISI *(2002)*

This AISI draft design guide provides additional code-approved values for the nominal prescriptive shear capacity of cold-formed steel stud shear walls when designing for wind and earthquake loads. Along with the wall configurations listed in the previous AISI guide *(1998)*, walls constructed using sheet steel panels (0.018" and 0.027"), thicker studs (0.043") and thicker end studs (0.043") for dense fastener spacing, and panels oriented perpendicular to framing have been included. These additional shear wall configurations are also found in the IBC *(ICC, 2000)*. The AISI draft document consists of three main sections: 1) information on safety and resistance factors: a factor of safety of 2.5 and a resistance factor of 0.55 are proposed for both wind and seismic load; 2) height to width aspect ratio (h/w): when this aspect ratio exceeds 2:1 and is less than 4:1, the available shear strength shall be adjusted by multiplying by 2w/h; 3) perforations: perforated shear walls sheathed with wood structural panels or sheet steel are allowed to carry the lateral load if properly designed.

The scope of the two versions of the AISI shear wall design guide in terms of allowable wall configurations can be found in Appendix D5.

CHAPTER 3 DESIGN METHODS FOR SHEAR CAPACITY

In North America, the CSA-S136 Standard *(1994)* and the AISI Specification *(1999)* provide design procedures for cold-formed steel structures in general, however, not all possible assembly situations are covered. The AISI has also published a Shear Wall Design Guide *(AISI, 1998)* that lists the nominal strength for different steel stud wall assemblies. Likewise, the UBC *(ICBO, 1997)*, IBC *(ICC, 2000)* and NEHRP *(FEMA, 1997a)* provide nominal shear value tables (seismic and wind forces) for specific wall configurations. The recommendations contained in these design guides are based on the results of shear wall tests, and hence are limited to the scope of the different research programs. In terms of guidance for designers using construction products that are available in Canada, no design standard or guide exists.

Design methods and shear resistance values for numerous wood-framed shear wall configurations can be obtained from different codes and standards *(UBC, 1997; IBC, 2000; CWC, 1995, 2001; CSA O86, 2001; APA, 1997; AWC, 1996)*. In this chapter, the prescriptive design tables that are available for cold-formed steel-stud shear walls are summarised, and a review of various design methods for wood-framed walls in both Canada and the USA is presented. Furthermore, a numerical method for calculating the shear strength of a steel-stud wall, which follows a similar procedure to that used for wood-framed walls is described. A comparison of the strength values determined using this numerical method with those obtained from test results is then presented.

3.1 DESIGN SHEAR CAPACITY

i. AISI Design Tables

The AISI Shear Wall Design Guide *(1998)* provides nominal shear strengths for cold-formed steel walls with plywood, OSB and gypsum wallboard sheathing, with fastener schedules from 6"/12" to 2"/12" (150 mm / 300 mm to 50 mm / 300 mm), and with studs having a minimum thickness of 0.84 mm. The allowable shear value can be obtained by dividing the listed nominal strength, based on the results of test programs carried out by Tissell *(1993)*, Serrette *(1994)*, and Serrette *et al. (1996b, 1997b)*, by a safety factor. It is suggested in the AISI Design Guide that a safety factor of 2.5 and 2.0 should be implemented for seismic and wind loads, respectively, in the absence of other requirements. If the Design Guide is used with the Load and Resistance Factor Design (LRFD) method or Limit States Design (LSD) method, then the recommended resistance factors are 0.6 for seismic loads and 0.65 for wind loads.

The AISI Design Guide is in the process of being updated, however a draft version is available *(AISI, 2002)* which contains additional nominal shear values for walls with sheathing oriented perpendicular to the framing, with steel sheet sheathing, and with stud thickness of 1.09 mm. The draft guide also contains considerations for the effect of aspect ratio and perforations, and proposes the use of a safety factor of 2.5 and a resistance factor of 0.55 for both seismic and wind loads.

ii. UBC *(1997)*

The Uniform Building Code *(ICBO, 1997)* specifies that the allowable shear capacity of a wall is determined by dividing the nominal shear value (shown in Table 22-VIII-A to C of UBC 97) by a safety factor, which can be taken as 3.0 for wind forces or 2.5 for seismic forces. When using the Load and Resistance Factor Design (or Limit States Design) procedure, the resistance factor (ϕ) shall be taken as 0.45 for wind load and 0.55 for seismic load.

3.2 Adapted Wood Design Method to Determine Shear Strength

In this section, the design methods for wood walls in both Canada and the USA are examined and a numerical method is derived to evaluate the shear strength of a steel-stud wall following a procedure similar to that used for wood-framed walls in the USA.

3.2.1 Adapted American Wood Design Method

In the design of a wood-framed shear wall the shear capacity is affected by many factors: such as the sheathing type and thickness, the fastener schedule, the orientation of sheathing panels, blocking, *etc.* In most cases, a shear wall fails due to the nails pulling through the panels and/or the nails bending and withdrawing from the framing. Thus, the capacity of a wood shear wall is usually governed by the connection capacity, rather than by the strength of the panels.

The most common yield limit modes for a nailed connection include dowel bearing failure under uniform bearing (mode I), dowel bearing failure under nonuniform bearing (mode II), plastic hinge located near each shear plane (mode III), as well as two plastic hinges near each shear plane (mode IV) *(Faherty & Williamson, 1999; Breyer et al., 1998)*. Since a wood connection will reach the design capacity when any one of these yield mechanisms is formed, the nominal strength of a single connection can be evaluated as the smallest load capacity of the four modes. This nominal strength is obtained from the equations for the respective connection failure modes, in which a safety factor of approximately 3.5 has been incorporated for softwood. The allowable connection shear value used for design can be obtained by using adjustment factors, such as that specified for the loading duration (C_D), which is equal to 1.6 for wind and earthquake forces, moisture content conditions (C_M), temperature (C_t), length of penetration (C_d), *etc.* The allowable shear strength of the wall is then obtained by multiplying the allowable design value per fastener by the total number of edge fasteners. Tissell *(1993)* stated that this design shear capacity could still be adjusted by factors that account for the influence of framing lumber width and panel thickness versus fastener size, *etc.* It should be noted that the design shear capacity obtained from this method will overestimate the true shear strength if the wall fails in another limit state prior to yielding of the nailed connections, such as buckling of the studs, splitting of the sheathing, splitting of the bottom plate, anchorage failure, *etc.*

The ASD (allowable stress design) method stipulates that the stresses and deflections under the specified design loads must not exceed the prescribed allowable stress and deflection limits. The general design philosophy is based on the following equation:

Applied stress ≤ Allowable stress

Since cold-formed steel studs are relatively thin (0.2 mm ~ 2 mm) compared with the fastener diameter (4 mm ~ 6 mm), it is impractical to assume that a plastic hinge forms in the fasteners as per the wood assembly design method. Thus, the equations used for calculating the shear capacity of a wood-framed wall cannot be directly applied to a steel-stud wall. Instead, the formula for shear resistance should be modified to incorporate all possible failure modes of the steel-stud wall. Common failure modes for a steel-stud shear wall include connection shear failure (such as bearing failure in the studs or panels, and fastener tilting failure), fastener shear failure, pull-over failure (for wood panels or steel sheet), steel sheet failure, and yielding or fracture of braces. A wall could also fail due to stud buckling, flexural failure of the bottom track if hold-down anchors are not used, and gypsum sheet fracture when the edge distance is inadequate. As per limit states design philosophy, the smallest value obtained from all of these possible failure modes will control the shear capacity of the wall. Strength calculations for the different failure modes are detailed below:

- **Connection Bearing/Tilting Strength of Screws in Shear P_{ns} *(AISI, 1999)*:**

For $t_2/t_1 \leq 1.0$, P_{ns} shall be taken as the smallest of

$$P_{ns} = 4.2(t_2^3 d)^{1/2} F_{u2} \qquad \text{(tilting strength)} \qquad (3-1.1)$$

$$P_{ns} = 2.7 t_1 d F_{u1} \qquad \text{(bearing strength for steel sheet)} \qquad (3-1.2)$$

$$P_{ns} = 2.7 t_2 d F_{u2} \qquad \text{(bearing strength for steel sheet)} \qquad (3-1.3)$$

For $t_2/t_1 \geq 2.5$, P_{ns} shall be taken as the smaller of

$$P_{ns} = 2.7 t_1 d F_{u1}$$

$$P_{ns} = 2.7 t_2 d F_{u2}$$

For $1.0 < t_2/t_1 < 2.5$, P_{ns} shall be determined by linear interpolation between the above two cases.

where:

t_1 = Thickness of the connected component in contact with the screw head

t_2 = Thickness of the connected component not in contact with the screw head

d = Diameter of the fastener

F_{u1} = Tensile strength of the member in contact with the screw head

F_{u2} = Tensile strength of the member not in contact with the screw head

If the sheathing panels are made of wood plate, then the bearing strength of the panels should follow the standard wood design procedures *(Faherty & Williamson, 1999, Breyer et al., 1998)*.

- **Bearing Strength for wood panel (*Faherty & Williamson, 1999*):**

$$Z = \frac{Dt_s F_{es}}{K_D} \tag{3 – 2.1}$$

where:

t_s = Thickness of panel, inches

F_{es} = Dowel bearing strength of panel, psi

D = Unthreaded shank diameter of the screw, inches

K_D= 2.2 for $D \leq 0.17"$, $10D+0.5$ for $0.17" < D < 0.25"$, and 3.0 for $D \geq 0.25"$.

It is noted by Faherty & Williamson *(1999)* that Equation (3 – 2.1) has a built-in safety factor of about 3.5 for softwood. Furthermore, considering that the dowel bearing strength of a panel is based on the standard duration load, while the test walls are subjected to short-term loading, the nominal bearing strength (P_{nw}) can be approximately evaluated as the product of the Z-value from Equation (3 – 2.1), a safety factor of 3.5, and a duration factor C_D (1.6), *i.e.* $P_{nw} = Z \times 3.5 \times 1.6$

- **Pull-over Strength P_{nov} for cold-formed steel sheet *(AISI, 1999)*:**

$$P_{nov} = 1.5 t_1 d_w F_{u1} \qquad (3-3.1)$$

where d_w is the larger of the screw head diameter or the washer diameter, and shall be taken not larger than ½" (12.7 mm), and F_{u1} and t_1 are as defined previously.

- **Shear rupture strength V_n:**

In general, the shear capacity for screws is provided by the screw manufacturer and is reduced by 20% to obtain the corresponding nominal code design value.

The minimum value obtained from the above equations governs the capacity of one connection or one screw. For simplification, the shear capacity for the whole wall can be taken as the product of this value and the number of edge screws per foot. This product should be compared with the shear load that causes overall failure in other modes, such as stud buckling, tension yielding or fracture of the straps for diagonally braced walls, as well as shear buckling and post tension yielding of walls sheathed with steel sheet panels.

- **Stud Buckling *(AISI, 1999)*:**

Considering that the chord members are subjected to high compression force due to overturning, it is possible that these members will fail due to stud buckling before reaching the load that causes failure of the connections. The nominal axial strength, P_n, for concentrically loaded compression members shall be calculated as defined in Equation (3 – 4.1). This nominal axial strength should not be greater than the yielding load.

$$P_n = A_e F_n \qquad (3-4.1)$$

and

$$P_n \le A_{ey} F_y$$

where:

A_e, A_{ey} = Effective area at the stress F_n and F_y, respectively

F_n is determined as follows:

For $\lambda_c \le 1.5$ $\quad F_n = \left(0.658^{\lambda_c^2}\right) F_y$ (3 – 4.2)

For $\lambda_c > 1.5$ $\quad F_n = \left[\dfrac{0.877}{\lambda_c^2}\right] F_y$ (3 – 4.3)

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} \qquad (3-4.4)$$

F_e = The least of the elastic flexural, torsional and torsional-flexural buckling stresses.

For doubly symmetric sections, closed cross-sections and any other section that can be shown not to be susceptible to torsional or torsional-flexural buckling, the buckling stress will be controlled by flexural buckling. For example, it is possible for the wood panels to act as lateral and torsional braces for the studs. Hence, the buckling stress, F_e, can be taken as the smaller of the elastic flexural buckling stresses, F_{ex} and F_{ey}, which can be determined as follows:

$$F_{ex} = \frac{\pi^2 E}{(K_x L_x / r_x)^2} \qquad (3-5.1)$$

$$F_{ey} = \frac{\pi^2 E}{(K_y L_y / r_y)^2} \qquad (3-5.2)$$

where:

E = Elastic modulus of the member

K_x, K_y = Effective length factors for bending about x- and y-axes

L_x, L_y = Unbraced length of member for bending about x- and y-axes

r_x, r_y = Radius of gyration of the full cross-section about the centroidal principal axes

If no sheathing is attached to the framing, or if the sheathing, such as steel sheet, cannot provide enough restraint to prevent torsional buckling of the end studs, the buckling stress, F_e, should also include torsional buckling and the interaction of torsional and flexural buckling. Since double studs are usually used back-to-back at the wall end to increase the buckling resistance, there is no interaction of torsional buckling and flexural buckling for this doubly symmetric section. The governing buckling stress shall be the minimum value of the flexural buckling stresses, F_{ex} and F_{ey}, and the torsional buckling stress F_t *(CSA S136, 1994; AISI, 1999)*.

$$F_t = \frac{1}{AR_0^2}\left[GJ + \frac{\pi^2 EC_w}{(K_t L_t)^2}\right] \qquad (3-5.3)$$

where:

$R_0 = \sqrt{r_x^2 + r_y^2 + x_0^2}$

A = Area of the full, unreduced cross-section

x_0 = Distance from shear centre to centroid along the principal x-axis.

C_w = Warping constant of the cross-section

J = St. Venant torsion constant of the cross-section

K_t = Effective length factors for twisting

L_t = Unbraced length of member for twisting

G = Shear modulus

E, r_x, r_y, K_x, K_y, L_x, L_y are as defined in Equation (3 – 5.2).

3.2.1.1 Possible Limit States for Other Wall Configurations

- **Elastic Plate Buckling** *(CSA S16.1, 1994; AISI, 1999)*:

Shear walls which are sheathed with steel sheet panels, may fail by elastic shear buckling of the sheathing due to the low shear stiffness of the sheet.

$$F_{CR} = K\frac{\pi^2 E}{12(1-\nu^2)(b/t)^2} \qquad (3-6.1)$$

$$K = 4 + \frac{5.34}{(a/h)^2} \qquad a/h < 1$$

$$K = 5.34 + \frac{4}{(a/h)^2} \qquad a/h \geq 1$$

where:

ν = Poisson ratio

t = Plate thickness

b = Length of loaded edge of plate (the shorter side dimension for shear loading)

K = Plate buckling constant

a/h = Aspect ratio, the ratio of the distance between stiffeners (stud spacing, a) to web depth (height of steel plate, h)

In plate girder design, the post-buckling strength of the web allows for an increase in the shear load if transverse stiffeners are added. Basler *(1962)* stated that part of the girder web (within a distance of s as shown in Fig. 3.1) develops a tension field due to the connection of web and stiffeners, which are capable of transmitting the vertical stresses to the adjacent panel. Similarly, for walls sheathed with steel sheets, the shear capacity can also be enhanced by relying on this post elastic buckling tension field action of the steel sheets. However, since the steel sheets are attached to the steel framing with screws spaced at a discrete distance, the continuous tension-bar model used for plate girders must be substituted by a discrete tension-bar model. A conservative approach to determine the number of these tension bars is to assume that within the width (s) of the tension field, only the area of steel directly restrained by the screws (screw diameter) contributes to the tension resistance. Therefore, taking the area of one bar as the product of the screw diameter and the sheet thickness, the number of the bars can be determined by dividing this s distance by the screw spacing in the desired direction. The method is shown below and the specific calculations of tension field shear capacity for walls sheathed with steel sheet are presented in Appendix B4.

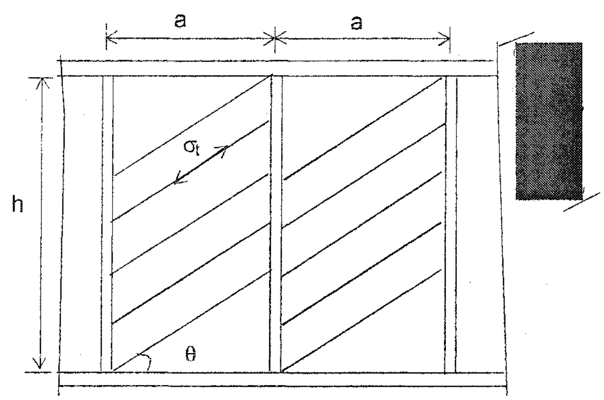


Fig. 3.1: Tension Field Action

- **Steel Sheet Tension Yielding *(CISC, 2000)*:**

From Fig. 3.1, the tension field width can be evaluated as follows:

$$s = h\cos\theta - a\sin\theta \qquad (3-7.1)$$

where:

θ = The inclination of tension stresses

a, h are as defined in Equation (3 – 6.1).

The number of the screws included in this tension field width is:

$$N = \frac{s}{s_1 \cos\theta} \tag{3-7.2}$$

where:

s_l = The spacing of screws at the perimeter

For a conservative capacity prediction, one can ignore the tension strength of the area between the screws, and hence, the vertical component of the force developed by the tension field is:

$$V_T = \sigma_t AN \sin\theta = \frac{\sigma_t A(h\cos\theta - a\sin\theta)}{s_1 \cos\theta} \tag{3-7.3}$$

where:

A = Area of each tension bar, which is equal to the product of the diameter of the screw (d) and the thickness of the sheet steel (t).

σ_t =Tension stress of the bars

To find the value of θ, differentiate V_T and set $dV_T/d\theta$=0, then solve for θ.

$$\tan^3\theta + 2\tan\theta - h/a = 0$$

$$\tan\theta = \frac{\left(27a^2h + \sqrt{864a^6 + 729a^4h^2}\right)^{1/3}}{32^{1/3}a} - \frac{22^{1/3}a}{\left(27a^2h + \sqrt{864a^6 + 729a^4h^2}\right)^{1/3}} \tag{3-7.4}$$

Then V_T can be found using the equilibrium equations from a free body diagram as shown in Fig. 3.2. The number of the screws included in the horizontal distance a is:

$$N_1 = \frac{a \tan\theta}{s_1} \qquad (3-7.5)$$

$$\sum F_{hor} = 0 \qquad \Delta T = (\sigma_t dt N_1)\cos\theta = \frac{\sigma_t dta \sin\theta}{s_1} \qquad (3-7.6)$$

$$\sum M_o = 0 \qquad V_T = \frac{\Delta T \times h}{a} = \frac{\sigma_t dth \sin\theta}{s_1} \qquad (3-7.7)$$

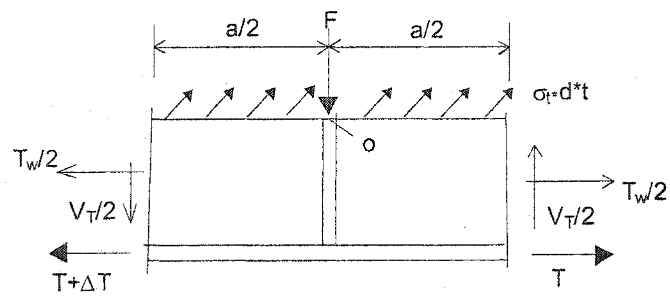


Fig. 3.2: Equilibrium Condition Applied to Free Body

- **Strap Tension Strength** ***(CSA S136, 1994; AISI, 1999)***

In the case where X-braces are to be utilized as the lateral resisting system, it is necessary to consider strap yielding and fracture of the net section, as well as connection failure. Furthermore, a failure mode due to a combination of torsion, bending and axial force in the end studs should be also taken into account when braces only exist on one side of the shear wall. It is noted that the braces are assumed to carry load in tension only.

The wall would fail if the straps yield or fracture under the applied load. From Fig. 3.3,

$$F = P_y \cos\theta \qquad (3-8.1)$$

P_y should be taken as the smaller of:

$$P_y = AF_y \qquad \text{(yielding)}$$

$$P_y = A_n F_u \qquad \text{(fracture)}$$

where:

F_y = Yield stress of the strap

A = Gross cross-sectional area of one strap

F_u = Tensile strength of the strap

A_n = Net cross-sectional area of one strap

θ = The inclination of the strap

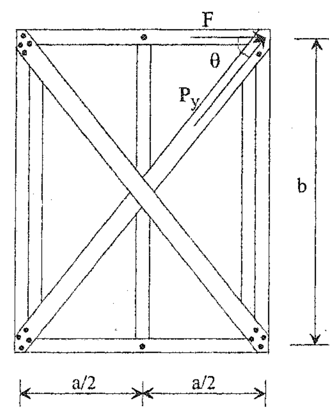


Fig. 3.3: Steel Walls Sheathed with X-braces

- **Combination of torsion, bending warping and compression** ***(Seaburg and Carter, 1997;Salmon and Johnson, 1996)***

For open cross-sections, the total stress of a member can be taken as a combination of stresses due to torsion and all other stresses.

The total normal stress f_n is:

$$f_n = \sigma_a \pm \sigma_{bx} \pm \sigma_{by} \pm \sigma_w \qquad (3-9.1)$$

The total shear stress f_v is:

$$f_v = \tau_{bx} \pm \tau_{by} \pm \tau_w \pm \tau_t \qquad (3-9.2)$$

a) Normal stress due to axial load

$$\sigma_a = \frac{N}{A} \qquad (3-9.3)$$

where:

σ_a = Normal stress due to axial loading

A = Cross-sectional area of the member

b) Stress due to bending moment

$$\sigma_b = \frac{M}{S} \qquad (3-9.4)$$

$$\tau_b = \frac{VQ}{It} \qquad (3-9.5)$$

where:

σ_b = Normal stress due to bending about either the x or y axis

M = Bending moment about either the x or y axis

S = Elastic section modulus

τ_b = Shear stress due to applied shear in either x or y direction

V = Shear in either x or y direction

Q = Static moment of area about either the x or y axis

I = Moment of inertia I_x or I_y

t = Thickness of the member

c) Torsional Shear Stress:

$$\tau_t = Gt\theta' \qquad (3-9.6)$$

where:

τ_t = Pure torsional (Saint-Venant torsion) shear stress at member edge

G = Elastic Shear Modulus of the member

t = Thickness of the member

θ' = Rate of change of angle of rotation θ (as shown in Fig. 3.4), first derivative of θ with respect to z (measured along longitudinal axis of member)

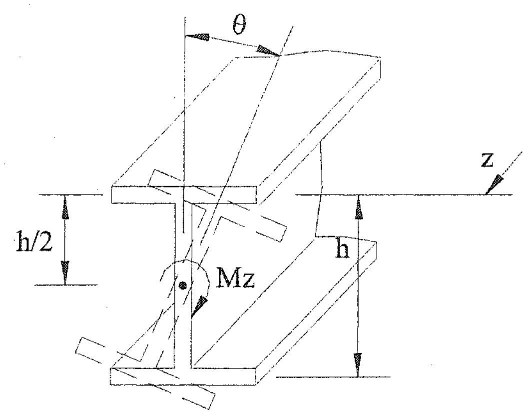


Fig. 3.4: Torsion of an I-Shaped Section

d) Stress due to warping torsional moment

For I-shaped sections, the maximum normal stress (σ_w) and shear stress (τ_w) due to warping can be approximated as follows, respectively:

$$\sigma_w = \frac{Ebh}{4}\theta'' \qquad (3-9.7)$$

$$\tau_w = \frac{Eb^2h}{16}\theta''' \qquad (3-9.8)$$

where:

E = Modulus of elasticity of the member

θ'', θ''' = The second and third derivatives of the rotation angle, θ, with respect to z (measured along the longitudinal axis of member), respectively

b, h = the width and height of the member, respectively

The expression of the rotation angle, θ, with respect to z varies depending on the different support and loading conditions. Considering the complexity of the actual support restraint of test specimens along the top track *(Serrette et al. 1996b, 1997b; COLA-UCI, 2001)*, which depends on the connection between the studs & tracks, along with the top track & load beam, no torsional calculation is presented in this thesis for the prediction of the shear capacity of walls braced with steel straps on one side. It is important to note that the torsional effect must be taken into account when using diagonal strap braces on one side of the wall. The approach highlighted above did not at this stage provide any conclusive results, hence, further study is required to more fully understand the torsional behaviour of the shear wall stud, track and brace connection and to develop design procedures.

3.2.2 Adapted Canadian Wood Design Method

For the shear wall with panel sheathing arranged vertically or horizontally, the factored shear resistance (force per length) of nailed shear panels sheathed with plywood, waferboard or OSB is determined as follows *(CWC, 1995)*:

$$v_r = \phi V_d J_{sp} \qquad (3-10.1)$$

where

$\phi = 0.7$

$V_d = v_d\,(K_D\,K_{SF})$

v_d = Specified shear strength for walls sheathed with plywood, waferboard or OSB (kN/m)

K_D = Load duration factor (1.15 for short-term loading)

K_{SF} = Service condition factor for fastenings

J_{sp} = Species factor for the framing material

The CSA O86 Design Standard *(2001)* and Canadian Wood Council (CWC) Wood Design Manual *(2001)* incorporate additional modification factors in the procedure to determine the factored shear resistance of a wood stud wall. The effect of construction details such as blocking at panel edges, different types of nails, and hold-down connections must be considered. The contribution of gypsum wallboard may also be included in the design strength of the entire shear wall.

- **For shear walls constructed with wood-based panels (*CWC, 2001*):**

$$v_r = \phi V_d J_{sp} J_n J_{ub} J_{hd} \tag{3 – 10.2}$$

where:

ϕ, V_d and J_{sp} are as defined in Equation (3 – 10.1).

J_n = Nail diameter factor which is equal to $(d_p/d_c)^2$.

d_p = Diameter of the alternate nail being considered

d_c = Diameter of the common wire nail given in the tables

J_{ub} = Factor for horizontally sheathed unblocked walls

J_{hd} = Hold-down factor

- **For shear walls constructed with gypsum wallboard panels (*CWC, 2001*):**

$$v_{rg} = \phi V_{dg} J_{hd} \tag{3 – 10.3}$$

where:

ϕ and J_{hd} are as defined in Equation (3 – 10.2).

V_{dg} = Specified shear strength for walls sheathed with gypsum sheathing (kN/m)

The limit states design method specifies that the factored load effect (the product of the specified loads and load factors) must not exceed the factored resistance of the structure (the product of a resistance factor ϕ and the nominal resistance). The general design format is as follows:

Factored resistance of structure $\geq \Sigma$Factored load effects

Since in Canada, design tables that provide specified shear strengths for steel-stud walls that are equivalent to what is contained in the CSA O86 Wood Design Standard *(2001)* do not exist, the CWC procedure cannot be directly applied to predict the shear capacity of steel walls. If a similar procedure to that prescribed by the CWC is to be used for steel stud walls, then an equivalent table of design shear values would need to be prepared.

3.3 Calculation of the Shear Capacity Following the Adapted American Wood Design Method

An example calculation using shear walls AISI-OSB1 and AISI-OSB2, which were cyclically tested by Serrette *et al. (1996b)*, is contained in this section. The 4' × 8' walls were constructed of 3.5"×1.625"×0.375" 20 gauge (0.0346": 0.88 mm) studs fabricated from ASTM A446 Grade A (33 ksi) sheet steel and spaced at 24" o.c.. A 7/16" OSB APA rated sheathing wall panel was attached in the vertical position to the framing on one side of the walls with No. 8×0.5" wafer head (modified truss) self-drilling screws. The screws were spaced at 6" centres along the panel perimeter and at 12" centres in the field. The maximum shear loads reached during testing were 945.5 lb/ft and 917.8 lb/ft with an average value of 931.6 lb/ft. The nominal shear resistance for both walls was determined to be 700 lb/ft, as defined by the last stable loop in the load *vs.* deflection hysteresis. This nominal value has also been approved for use in the Shear Wall Design Guide *(AISI, 1998)*.

A number of design assumptions were necessary, including that the OSB panels were made from Southern Pine, F_{es} = 5550 psi (Table 5.35 of *Faherty & Williamson, 1999)*. For the stud members, the minimum specified nominal yield strength is F_y = 33 ksi and the corresponding ultimate tensile strength is F_u = 45 ksi *(ASTM A653, 1994)*. For the No. 8 screws, D = d = 4.2 mm (0.165"). t_2 = 0.0346" (stud thickness), $t_s = t_l$ = 7/16" (panel thickness) then $t_2/t_l < 1.0$, K_D = 2.2 (from Equation (3 – 2.1)). Considering that for all tests by Serrette *et al. (1996b, 1997b)*, the walls were only subjected to lateral loads, no direct tension force was applied to the screws. Hence, for pull-over failure to occur extensive tilting of the screws would have to take place. It was therefore assumed that the tilting capacity would be reached prior to the pull-over resistance. All of the calculations are shown in detail in Appendix 'B' with a summary of nominal results listed below.

Bearing strength of OSB panel:	$P_{nw} = 1020$ lb
Bearing strength of steel studs:	$P_{ns} = 694$ lb
Tilting strength of screws:	$P_{ns} = 494$ lb
Screw shear *(Buildex, 2002)*	$V_n = 740$ lb

In terms of connection capacity the tilting strength of the screws controls and the nominal shear resistance of one connection is 494 lb. There are 2 edge screws per foot at the panel perimeter; thus, the estimated nominal lateral strength per foot for this specimen is calculated as follows:

$$494 \times 2 = 988 \text{ lb/ft}$$

As stated previously, this estimated strength should not exceed the stud buckling capacity: $P_{nb} = 1199$ lb/ft (> 988 lb/ft)

However, for this simplified method to calculate the lateral strength of the wall it is assumed that only the end screws resist horizontal load, and hence, the contribution of side fasteners and interior fasteners is ignored (Fig. 3.5). In reality, the edge screws resist both horizontal load and vertical load due to the overturning moment (chord stud forces), and furthermore, both the side fasteners and the interior fasteners contribute to the shear resistance. Considering this contribution, Easley *et al. (1982)* developed a design procedure to relate the applied lateral force with the various fastener forces based on equilibrium (Fig. 3.5). It is assumed that the fastener forces in the panel ends have two direction components. The x-component of each fastener remains constant, whereas the y-component is proportional to the distance x_{ei} of the fastener from the panel centreline. The fastener forces in the panel sides are presumed to be uniform and to act along the long axis of the studs. At the interior stud locations the fastener forces are assumed to act in the y-direction (along the length of the stud) with values proportional to the distance x_{si} of the fastener from the panel centreline.

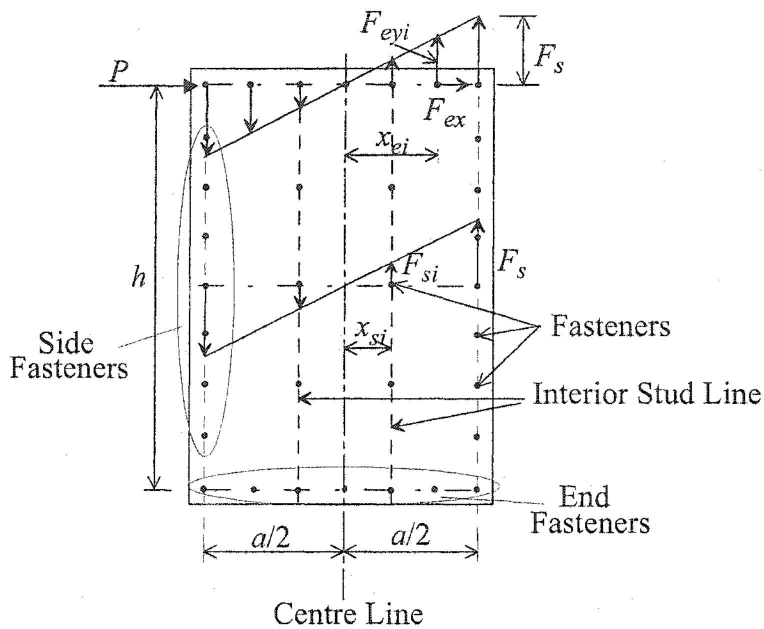


Fig. 3.5: Assumed Directions and Distributions of Sheathing Fastener Forces (*Easley et al., 1982*)

For side fastener forces (Fig. 3.5):

$$F_s = \frac{Pb}{\beta} \tag{3-11.1}$$

For end fastener forces (Fig. 3.5):

$$F_{ei} = P\left[\left(\frac{a}{n_e}\right)^2 + \left(2x_{ei}\frac{h}{a\beta}\right)^2\right]^{1/2} \tag{3-11.2}$$

where

$$\beta = n_s + \frac{4I_e + 2n_{si}I_s}{a^2}$$

$$I_e = \sum_{i=1}^{n_e} x_{ei}^{\ 2} \text{ and } I_s = \sum_{i=1}^{m} x_{si}^{\ 2}$$

n_s = The number of side fasteners, excluding those at the end

n_e = The number of end fasteners

n_{si} = The number of fasteners in each interior stud, excluding those at the end

m = The number of interior studs

a = Wall length (Fig. 3.5)

h = Wall height (Fig. 3.5)

P = The shear force per unit length on the shear wall

In this test, the wall panel was 4'×8' and the studs were spaced 24" o.c., thus m = 1. The fastener schedule required screws to be placed at every 6" along the perimeter and at every 12" in the field, hence, $n_s = 15$, $n_e = 9$, and $n_{si} = 7$. x_{ei} = 24" for all side fasteners, 0" for interior fasteners and 0", 6", 12", 18", and 24" for end fasteners. Calculations then show that $I_e = 2160$ in^2 and $I_s = 0$.

$$\beta = 15 + \frac{4 \times 2160 + 2 \times 7 \times 0}{(4 \times 12)^2} = 18.75$$

The force in the side fasteners is:

$$F_s = \frac{P \times 8 \times 12}{18.75} = 5.12P$$

The maximum force in the end fasteners is:

$$F_{ei} = P\left[\left(\frac{4 \times 12}{9}\right)^2 + \left(2 \times 24 \times \frac{8 \times 12}{4 \times 12 \times 18.75}\right)^2\right]^{1/2} = 7.4P$$

Thus the maximum force in the fasteners is F_{ei} = 7.4P, which must not exceed the connection shear capacity as evaluated previously. The lateral strength of the wall can then be determined by rearranging the above equation, so that $P = 0.135\ F_{nominal}$ (lb/in) = 1.623 $F_{nominal}$ (lb/ft). Again, this shear load should be no greater than that reached when the chord studs buckle. Given that the nominal strength for the fasteners is 494 lb/ft as shown above, the nominal shear strength for this specific wall is as follows:

$$494 \times 1.623 = 802 \text{ lb/ft} < P_{nb} = 1199 \text{ lb/ft (stud-buckling load)}$$

The test-to-predicted ratio for this nominal value is 932 / 802 = 1.16 (>1.0), while the ratio for the nominal value obtained from the simplified method (where the edge screws only resist horizontal load) is 932 / 988 = 0.943, which indicates that the predicted nominal strength based on the design procedure by Easley *et al. (1962)* provides a more conservative result. Therefore, the predicted nominal shear strength in this thesis is based on the Easley *et al.* procedure. However, further studies which incorporate the behaviour of the steel to wood fasteners when subjected to lateral loading are required to verify the assumptions made in the Easley *et al.* procedure.

3.4 COMPARISONS AND ANALYSIS

The measured shear capacity (peak load) for the walls tested at Santa Clara University (*Serrette et al. 1996b, 1997b*) and at the University of California at Irvine *(COLA-UCI, 2001)* were utilised in a comparison with the results obtained from the previously described numerical procedure. A summary of the nominal test-to-predicted ratios is provided in Table 3.1 with statistical parameters shown in Table 3.2. More detailed calculations and results are presented in Appendix 'B'. A comparison of the predicted shear strengths and the actual shear strength obtained from the test results is also shown in Fig.3.6.

Table 3.1: Comparison of the Predicted Shear Capacity and the Peak Load from Tests *(Serrette et al. 1996b, 1997b; COLA-UCI, 2001)*

Specimens	Predicted Shear (lb/ft)	Peak Load (lb/ft)	Test to Predicted Ratio	Specimens	Predicted Shear (lb/ft)	Peak Load (lb/ft)	Test to Predicted Ratio
AISI – A1, 2[w]	1553	1933	1.24	AISI – OSB3, 4[w]	1178	1269	1.08
AISI – A3, 4[w]	1781	2397	1.35	AISI – OSB5, 6[w]	1199	1725	1.44
AISI – A5, 6[w]	1553	1652	1.06	AISI – OSB7, 8[w]	1199	1985	1.66
AISI – A7, 8[w]	1781	2263	1.27	AISI – PLY1, 2[w]	802	990	1.23
AISI – B1, 2[w]	1111	1015	0.91	AISI – PLY3, 4[w]	1178	1312	1.11
AISI – B3, 4[w]	1201	1063	0.88	AISI – PLY5, 6[w]	1199	1753	1.46
AISI – D1, 2[s]	191	438	2.29	AISI – PLY7, 8[w]	1199	1928	1.61
AISI – E1, 2[w]	813	804	0.99	Group 14 A, B, C[w]	669	891	1.33
AISI – E3, 4[w]	1116	1288	1.15	Group 15A, B, C[w]	1002	1222	1.22
AISI – E5, 6[w]	1116	1808	1.62	Group 16A, B, C[w]	1116	2067	1.85
AISI – F1, 2[s]	447	538	1.20	Group 17A, B, C[w]	669	726	1.09
AISI – F3, 4[s]	828	1249	1.51	Group 18A, B, C[w]	1002	1065	1.06
AISI – OSB1, 2[w]	802	932	1.16	Group 19A, B, C[w]	1116	2006	1.80

[w] denotes steel walls sheathed with wood panels [s] denotes steel walls sheathed with sheet steel

Table 3.2: Statistical Parameters of Test-to-Predicted Ratios

	Number of Tests	Mean Value	Standard Deviation	Coefficient of Variation
All Tests	58	1.33	0.326	24.5%
Serrette Tests	40	1.31	0.326	24.8%
UCI Tests	18	1.39	0.350	25.1%
Wood Panel Tests	52	1.29	0.273	21.2%
OSB Tests	27	1.28	0.276	21.5%
Plywood Tests	25	1.12	0.333	29.7%
Steel Sheet Tests	6	1.67	0.563	33.7%

As can be seen From Table 3.1 and Figure 3.6, the calculated shear capacities using the numerical method generally provide lower values than the measured peak load from tests, except for specimens AISI – B1,2, AISI – B3,4, and AISI – E1,2. The results also show that this numerical method provides a better prediction of the shear strength for the walls sheathed with wood panels. For example, the wood panel walls have a mean test-to-predicted ratio of 1.29 and the coefficient of variation of 21.2%, in comparison with the walls sheathed with sheet steel, which have corresponding values of 1.67 and 33.7%, respectively.

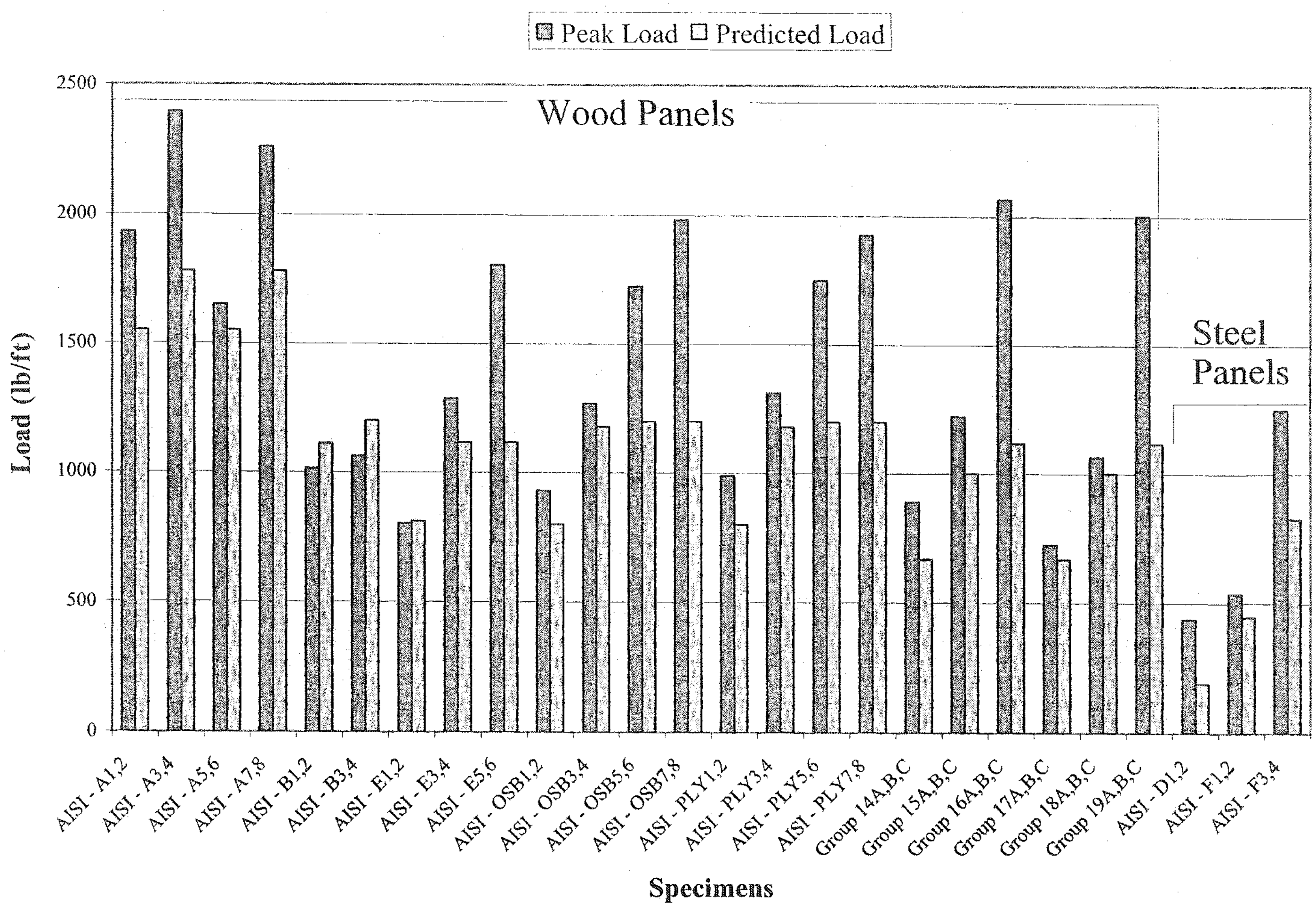


Fig. 3.6: Comparison of the Predicted Shear Capacity and the Peak Load from Tests (*Serrette et al. 1996b, 1997b; COLA-UCI, 2001*)

For wood panel walls AISI - A1,2,5,6, AISI – OSB1,2,3,4, AISI – PLY1,2,3,4, as well as Group 14A,B,C, 15A,B,C, 17A,B,C, 18A,B,C, the calculated shear capacities are somewhat smaller than those measured during testing (the test-to-predicted ratios range from 1.06 to 1.33). This conservative nature of the prediction method may be attributed to the following: 1) The use of minimum specified ASTM A653 material properties, including F_y and F_u, in the calculation process, instead of the actual material properties as determined by means of coupon tests. It is common for the actual material properties to exceed those specified in the ASTM material standards. 2) The predicted method is based on the first failure modes, for which the capacity is controlled by the tilting strength of the corner screws. At this stage the majority of the remaining screws would not have been subjected to a shear load that matches their ultimate capacity. Therefore, it is probable that after tilting of the corner screws, the wall would deform inelastically and the loads would redistribute to other less highly stressed fasteners. In this fashion it is possible that the walls were able to carry increased shear loads at ultimate failure.

In the calculation of the shear capacity for the walls sheathed with sheet steel, specimens AISI - D1,2, AISI - F1,2, and AISI – F3,4, only the post-buckling strength of the bars within the area of the screw diameter and the sheet thickness was considered to contribute to the increased strength of the tension field. It is most likely that the regions of sheet steel near the screws were restrained, and hence, provided additional shear capacity to the wall system. The closer the screws are spaced, the more pronounced the difference in test-to-predicted results due to the greater amount of sheet steel that is able to resist load. This can be seen in the results obtained for specimens AISI – F3,4 (screws spaced at 2"/12") where the average test-to-predicted ratio is 1.51, in comparison to walls AISI – F1,2 (screws spaced at 4"/12"), which have an average test-to-predicted ratio of 1.20. Furthermore, the capacity may also increase due to the actual material strength being higher than the nominal as discussed previously.

In the twenty four design cases where the predicted strength is controlled by stud buckling, such as for walls AISI – A3,4, AISI – A7,8, AISI – E3,4, AISI – E5,6, AISI – F3,4, AISI – OSB5,6, AISI – OSB7,8, AISI – PLY5,6, AISI – PLY7,8, and Group

16A,B,C, 19A,B,C, the calculated shear capacities are significantly lower than those obtained from the test results. This may be due to the following:

a) The yield stress, F_y, used in the numerical method to evaluate the shear strength is taken as the minimum specified ASTM A653 strength of 33 ksi ($\approx$ 230 MPa), which in all probability is lower than the actual strength if measured using coupon tests ($\approx$ 300 MPa for common mild grade sheet steels). If the assumed increased yield stress value were to be used in the calculations (coupon test results were not available), then an increase in the shear capacity of approximately 15% would be realized.

b) The test information showed that 20 ga. studs were used for all specimens. It was assumed that this corresponded to a 0.84 mm thickness for the sheet steel. However, the thickness range of 20 ga. material can reach as high as 0.91 mm. If the thicker stud measurement were used in the calculations, then a 13% increase in the buckling capacity of the chord members would occur.

c) In the model used to calculate the stud buckling capacity it was assumed that the chord studs were concentrically loaded with a constant axial stress along the entire member ((1) of Fig. 3.7). Furthermore, the unbraced length for the weak axis was defined as the screw spacing and for the strong axis was taken as the height of the wall. This model provides a conservative stud resistance for the following reasons. When the wall is subjected to lateral load only, which is the actual loading condition of tests by Serrette *et al.* and UCI, the stress in the studs varies along their length, with minimum and maximum forces at either end, (Fig. 3.7 (2)), because of the shear flow from the panel sections. Therefore, the stress pattern can be taken as a triangular shape if the spacing of screws is rather small compared to the length of the studs. For simplification, the average stress can be used for design purposes. Furthermore, the wood panel is attached to the flange of the studs through screws, and hence, will restrain the strong axis buckling of studs to some extent. If this restraint were sufficient, then the studs would be forced to buckle between the screws in both the strong and weak axes, which may result in failure due to squashing of the cross section instead of buckling. If all of these variations were taken into account, the predicted strength, governed by yielding of the cross section, would be approximately 15% higher than that obtained for the procedure detailed previously.

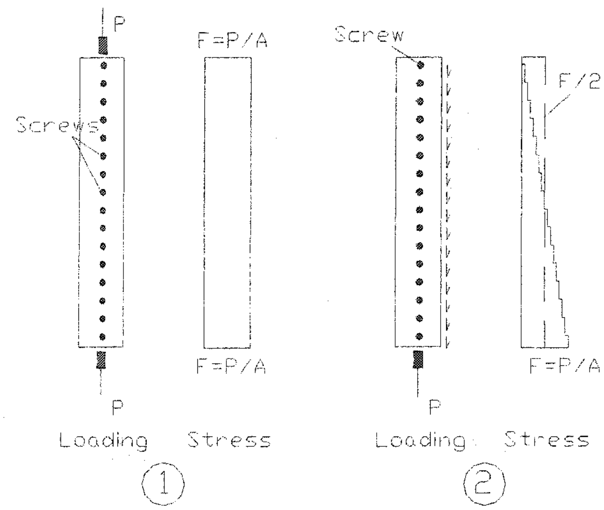


Fig. 3.7: Two Models for Stud Buckling

Shear capacity calculations were carried out for the walls with wood panels accounting for the aspects discussed above (a, b, c). This directly affected the stud buckling resistance, which increased by approximately 38%. The resulting test-to-predicted ratios (shear resistance) for walls sheathed with wood panels are listed in Table 3.3. The walls where the design calculations show that the shear capacity is controlled by stud buckling include AISI – E5,6, AISI – OSB7,8, AISI – PLY7,8 and Group 16A,B,C, 19A,B,C. These twelve test specimens were also the only walls that were recorded to have failed by stud buckling during testing. In contrast, when nominal material values were used and when the strong axis braced length was assumed to be the wall height, the predicted failure mode of stud buckling occurred for additional tests, including AISI – A3,4, AISI – A7,8, AISI – E3,4, AISI – OSB5,6, AISI – PLY5,6, which actually failed by screws pulling through wood panels (shown in Appendix 'B', Table B5 – B7). This shows that the increased stud buckling resistance provides results, with respect to failure mode prediction, that are more in line with the laboratory observations. The mean value for the test-to-predicted ratios is 1.11 and the coefficient of variation is 10.0%, which indicates that the predicted capacity for the steel stud walls sheathed with wood panels obtained from the numerical method is generally in agreement with the test results if elevated material properties are used in place of nominal values, and if the bracing of studs is considered as the screw spacing for both the strong and weak bending axes. It is important to reiterate that axial gravity loads were not taken into consideration in the design method because these loads were not applied during testing.

Table 3.3: Test-to-Predicted Ratios of Wood Panel Walls based on the Increased Stud Buckling Resistance *(Serrette et al. 1996b, 1997b; COLA-UCI, 2001)*

Specimens	Predicted Shear (lb/ft)	Peak Load (lb/ft)	Test to Predicted Ratio	Specimens	Predicted Shear (lb/ft)	Peak Load (lb/ft)	Test to Predicted Ratio
AISI – A1, 2^{W}	1553	1933	1.24	AISI – OSB7, 8^{W}	1658	1985	1.20
AISI – A3, 4^{W}	2304	2397	1.04	AISI – PLY1, 2^{W}	802	990	1.23
AISI – A5, 6^{W}	1553	1652	1.06	AISI – PLY3, 4^{W}	1178	1312	1.11
AISI – A7, 8^{W}	2304	2263	0.98	AISI – PLY5, 6^{W}	1553	1753	1.12
AISI – B1, 2^{W}	1111	1015	0.91	AISI – PLY7, 8^{W}	1658	1928	1.16
AISI – B3, 4^{W}	1201	1063	0.88	Group 14A, B, C^{W}	669	891	1.33
AISI – E1, 2^{W}	813	804	0.99	Group 15A, B, C^{W}	1002	1222	1.22
AISI – E3, 4^{W}	1179	1288	1.09	Group 16A, B, C^{W}	1658	2067	1.25
AISI – E5, 6^{W}	1658	1808	1.09	Group 17A, B, C^{W}	669	726	1.09
AISI – OSB1, 2^{W}	802	932	1.16	Group 18A, B, C^{W}	1002	1065	1.06
AISI – OSB3, 4^{W}	1178	1269	1.08	Group 19A, B, C^{W}	1658	2006	1.21
AISI – OSB5, 6^{W}	1553	1725	1.11				

^{W} denotes steel walls sheathed with wood panels

3.5 CONCLUSIONS

The method described in the previous section is a preliminary attempt to numerically estimate the shear capacity of cold-formed steel shear walls subjected to in-plane lateral loading. The comparisons and analyses indicate that this numerical method is feasible, in that it provides an approximate prediction of the shear strength. However, since the method relies on the assumption that the wall exhibits elastic behaviour prior to the first connection failure, it is necessary to revise the approach to include the inelastic effects that occur before failure of the entire wall. In addition, this numerical method is derived from static analyses without considering the effect of cyclic performance, and thus further studies are necessary to incorporate the degradation effects of cyclic wind and earthquake of loading.

CHAPTER 4 FORCE MODIFICATION FACTOR FOR SEISMIC DESIGN

Seismic design codes recognise the ability of some structures to undergo significant inelastic deformation while maintaining their load carrying capacity. This behaviour allows the engineer to rely on an elastic design approach with reduced seismic forces if the structure is detailed such that it will carry load and dissipate energy when displaced into the inelastic range, *i.e.* the 'equivalent static approach to seismic design'. The true lateral deflections of a structure that behaves inelastically can be several times that of the same structure when designed using an elastic analysis with reduced loads (Fig. 4.1). Most building codes provide force and deflection modification factors to account for the reduced loads and increased deflections that occur under seismic loading in a building that has been designed to provide a ductile load *vs.* displacement response. An overview of a number of different design standards has been included in this chapter in order to compare the different methods used in the equivalent static approach to seismic design. This includes the National Building Code of Canada *(NRCC, 1995, 2004)*, and two of the US model codes NEHRP *(FEMA, 1997a,b)* and UBC *(ICBO, 1994, 1997)*.

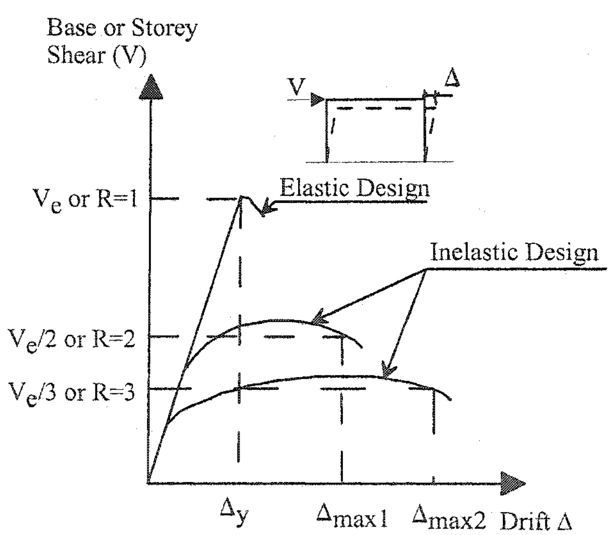


Fig. 4.1: Design Shear *vs.* Required Drift

4.1 Comparison of Different Design Standards

i. **NBCC** (National Building Code of Canada)

For regular structures, the NBCC *(NRCC, 1995)* allows engineers to use an equivalent static design method to determine the seismic load. The NBCC states that regular structures must be designed to sustain a base shear, V, in each of their principal directions, given by the following formula:

$$V = \frac{V_e}{R} U \qquad (4-1)$$

where $U = 0.6$, V_e is the base shear corresponding to an elastic response of the structure, and R is the force modification factor. The maximum lateral deflection that the structure must resist without collapse is:

$$\Delta_{\max} = \Delta_y \times R \qquad (4-2)$$

where Δ_y is the lateral deformation of the structure based on an elastic analysis. The NBCC specifies a drift limit of 2% of the storey height. The factor, R, depends on the ability of the structure to maintain its load carrying capacity over extended lateral displacement; hence, it will vary depending on the type of structural system that is specified. Force modification factors are determined based on experience acquired in terms of design and construction and also from the study of building behaviour during earthquakes. The values of R vary from 1.0 to 4.0 (R/0.6 vary from 1.7 to 6.7), where the lower is recommended for an unreinforced masonry wall and the higher for a ductile moment frame.

- Draft NBCC 2004

The base design shear under earthquake loading in the Draft NBCC 2004 *(NRCC, 2004)* is given by:

$$V = \frac{V_e}{R_d R_0} \tag{4-3}$$

where R_d is a ductility related force modification factor that reflects the capability of a structure to dissipate energy through inelastic behaviour, and R_0 is an overstrength related force modification factor that accounts for the dependable portion of reserve strength in a structure designed according to the NBCC provisions. The R_0 factor is evaluated by:

$$R_0 = R_\phi \times R_{yield} \times R_{sh} \times R_{size} \times R_{mech} \tag{4-4}$$

where:

$R_\phi = 1/\phi$;

R_{yield} = Ratio of probable yield strength to minimum specified yield strength

R_{sh} = Overstrength due to strain hardening

R_{size} = Overstrength due to discrete member sizes

R_{mech} = Overstrength developed when a full collapse mechanism is formed

The maximum lateral deflection that a structure is expected to resist without collapse is:

$$\Delta_{\max} = \Delta_y \times R_0 \times R_d \tag{4-5}$$

where Δ_y is the lateral deformation of the structure based on an elastic analysis using the reduced seismic loads. The drift limit is 2.5% of the storey height. The values of $R_d R_0$ vary from 1.0 to 7.5, where the lower is recommended for an unreinforced masonry wall and the higher for a ductile moment frame

ii. **NEHRP** (National Earthquake Hazards Reduction Program)

The National Earthquake Hazards Reduction Program *(FEMA, 1997a,b)* relies on a different method to determine the equivalent static loads for seismic design. The force reduction factor (FRF), expressed as a response modification factor, *R*, is used to reduce the linear elastic design response spectra. The design seismic base shear, V, in a given direction is determined in accordance with the following equation:

$$V = \frac{V_e}{R} \tag{4-6}$$

where V_e is the base shear corresponding to an elastic response of the structure. A displacement amplification factor (DAF), C_d, is used to compute the expected maximum inelastic displacement from the elastic displacement induced by the design seismic forces.

$$\Delta_{\max} = \Delta_s \times C_d \tag{4-7}$$

Uang *(1991)* showed that if the actual response envelope of the structure, considering drift *vs.* base or storey shear (Fig. 4.2), can be idealised as an elasto-perfectly plastic response curve, the following factors can be defined:

System ductility reduction factor

$$R_\mu = \frac{V_e}{V_y} \tag{4-8}$$

Structural overstrength factor

$$\Omega = \frac{V_y}{V_s} \tag{4-9}$$

where V_y is the actual yield strength level and V_s is the first significant yield strength level (prescribed design force). Therefore:

$$R = \frac{V_e}{V_s} = R_\mu \Omega \tag{4-10}$$

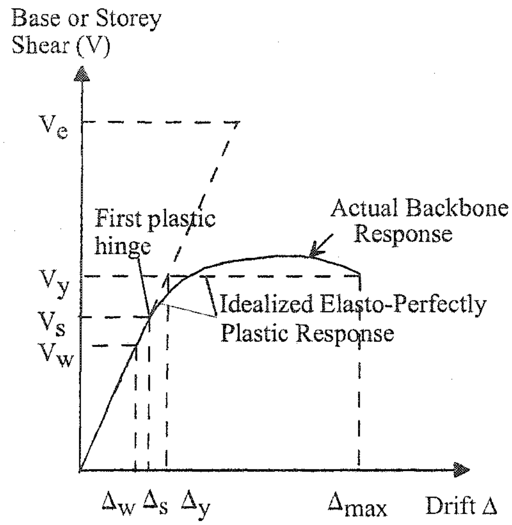


Fig 4.2: Base or Storey Shear *vs.* Drift

The values of R vary from 1.25 to 8.0, where the lower is recommended for an ordinary masonry wall and the higher for a ductile moment frame (*FEMA, 1997a*). Uang also showed that the displacement amplification factor C_d could be derived from Fig. 4.2 as follows:

$$C_d = \frac{\Delta_{\max}}{\Delta_s} = \frac{\Delta_{\max}}{\Delta_y}\frac{\Delta_y}{\Delta_s} \qquad (4-11)$$

where Δ_{max} / Δ_y = the structural ductility factor μ; and from Fig. 4.2:

$$\frac{\Delta_y}{\Delta_s} = \frac{V_y}{V_s} = \Omega \qquad (4-12)$$

Therefore $C_d = \mu\Omega$

The overstrength factor, Ω, is the ratio of the full yield strength of the structure (nominal strength) to the first significant yield strength. However, there are several other sources of strength that may further increase structural overstrength. For example: the material overstrength (due to the true material properties being higher than the nominally specified values), the resistance factor used in design, the designer intentionally introducing additional overstrength, *etc*. Another overstrength factor that may need to be considered in the design of steel stud shear walls is the system overstrength factor, which is dependent on the amount of redundancy in the structure. A structure that has a high degree of redundancy will typically be able to carry a higher load after first significant yield due to force redistribution; and thus, the greater the system overstrength factor. Furthermore, a structure whose design is controlled by the drift limit could have a rather high overstrength factor since the structure's actual strength could be significantly higher than that required by ultimate strength design provisions.

iii. **UBC** (Uniform Building Code)

The Uniform Building Code *(ICBO, 1994)* specifies a design seismic force level for working stress design. The required elastic seismic force level can be reduced by a force reduction factor (FRF) R_w *(SEAOC, 1996)*.

$$V = \frac{V_e}{R_w} \tag{4 – 13}$$

where V_e is base shear corresponding to an elastic response of the structure. The total reduction factor corresponding to the UBC allowable-stress-design format is:

$$R_w = \frac{V_e}{V_w} = \frac{V_e}{V_y}\frac{V_y}{V_s}\frac{V_s}{V_w} \tag{4 – 14}$$

Uang *(1991)* showed that an additional factor, the allowable stress factor Y, is required for allowable stress design.

$$Y = \frac{V_s}{V_w} \tag{4 – 15}$$

where V_w is the corresponding design force level for allowable stress design. Along with the system ductility reduction factor, R_μ, and structural over-strength factor, Ω, defined previously, the force reduction factor R_w is expressed as follows:

$$R_w = R_\mu \Omega Y \tag{4 – 16}$$

The R and R_w factors differ by a constant load factor ($Y \approx 1.4$) that is dependent on the structural system. The values of R_w for use in seismic design vary up to 12.0, which is the factor specified for ductile moment frames.

To estimate the maximum inelastic deflections, Δ_{max} that may develop in a major earthquake, the design lateral deflections computed using an elastic structural analysis, Δ_w, are amplified as follows:

$$\Delta_{\max} = \Delta_w \times \frac{3}{8} R_w \tag{4 – 17}$$

Based on observations of building performance during actual seismic events the $3R_w/8$ factor was included in the deflection calculation for the UBC. NEHRP *(1997b)* reports that "this is a somewhat arbitrary factor that attempts to quantify the maximum force that can be delivered to sensitive elements based on historic observations that the real force that could develop in a structure may be 3 to 4 times the design levels".

The base shear equation in UBC *(ICBO, 1997)* had been revised from the allowable stress design approach to that appropriate for the load and resistance factor design philosophy, and hence is consistent with the 1994 NEHRP equation. The original R_w factor was replaced by an R factor that has a value of approximate $R_w/1.4$. The maximum inelastic displacement Δ_{max} can be calculated as follows:

$$\Delta_{\max} = \Delta_s \times 0.7R \tag{4 – 18}$$

where Δ_s is elastic drift under the design seismic force.

4.2 PROCEDURE TO DETERMINE FORCE MODIFICATION FACTORS FROM TESTS

In this section a description of a possible method that can be used to determine force modification factors, R, from quasi-static cyclic tests for use with the various model building codes is provided. The steps are as listed:

i. Depict the unidirectional "actual response" (backbone curve as shown in Fig. 4.3)

The backbone curve is taken as the envelope of cyclic curves based on the highest strength hysteretic response from a plot of storey shear *vs.* storey deflection (recommended by SEAOSC *(1997)*).

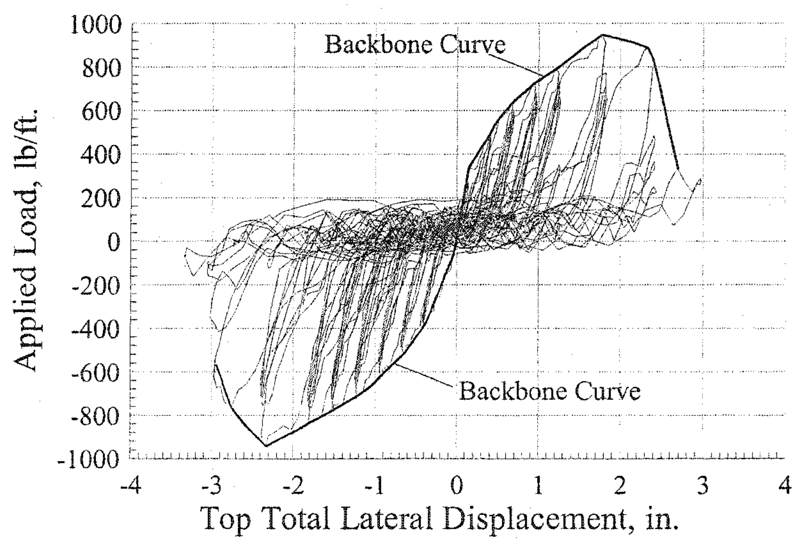


Fig. 4.3: Typical Steel Stud Shear Wall Load *vs.* Displacement Hysteresis *(Serrette et al., 1996b)*

ii. Evaluate the ideal bilinear curve and the ductility factor, μ

An ideal bilinear curve is comprised of two segments, where the first segment represents the shear stiffness of the wall, which is dependent on the definition of the yield displacement. Park (*1989*) provided various definitions for the yield displacement (Fig. 4.4), and recommended that the most realistic definition is as shown in Fig. 4.4d. However, for cold-formed steel shear walls sheathed with wood panels, sheet steel or steel straps using screws, it is difficult to define the first yield point (Fig. 4.4a). Park's recommendation is more appropriate for reinforced concrete or hot-rolled steel structures, where the first instance of steel yielding can be more easily identified. Furthermore, use of the yield displacement based on equivalent elasto-plastic energy absorption (Fig. 4.4c) is feasible for computer models, but it is not practical when attempted manually because of the difficulty in obtaining two equivalent areas. Thus in the method utilised for this research project, the yield displacement is evaluated based on the equivalent elasto-plastic yield, which is defined to have the same elastic stiffness as the test wall, as shown in Fig. 4.4b.

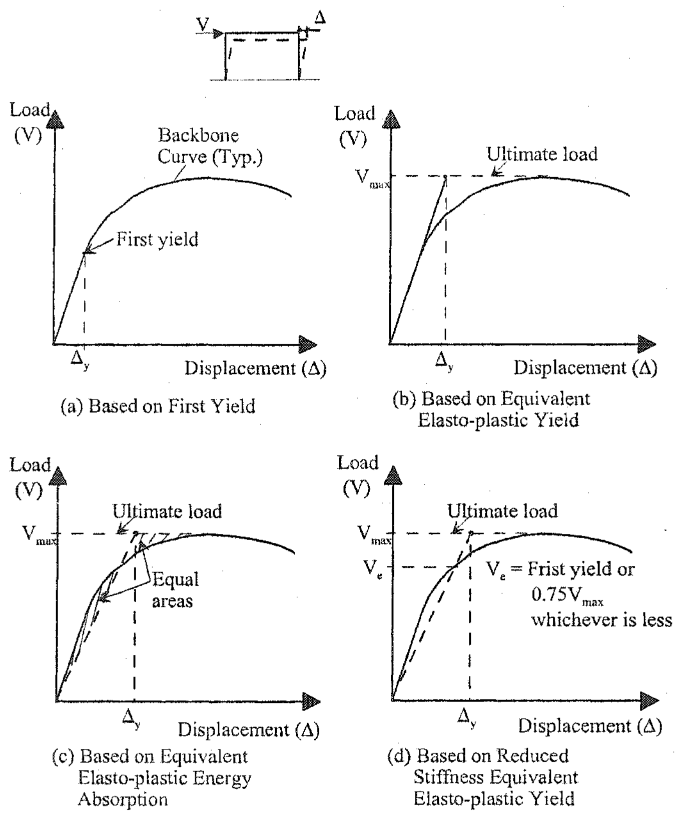


Fig. 4.4: Alternative Definitions for Yield Displacement *(Park, 1989)* for Cyclic Loading

The second segment of the ideal bilinear curve consists of a plateau over which the shear displacement of the wall increases while the applied load remains constant. A lower and upper bound value for the plateau portion can be established from the storey shear *vs.*

storey deflection envelope curves (Fig. 4.5). The lower and upper bounds occur when the plateau portion of the curve (plastic behaviour) intersects the maximum load in the envelope and the failure load (typically, 80%V_{max} for wood walls), respectively. An idealised bilinear elasto-plastic shear *vs.* deflection plateau can be selected within these bounds. The properties that need to be considered in this evaluation include: ductility, hysteretic energy dissipation, resistance to degradation, inherent redundancy, number of cycles resisted, and failure mode, *etc (Driver et al., 2000)*. The ductility demand factor, μ, is defined by using the following equation:

$$\mu = \frac{\Delta_{max}}{\Delta_y} \tag{4-19}$$

where Δ_{max} is the measured deflection that occurs at the intersection of the actual and idealised bilinear response curves, and Δ_y is the pseudo yield deflection, which occurs at the intersection of the two segments of the idealised bilinear curve.

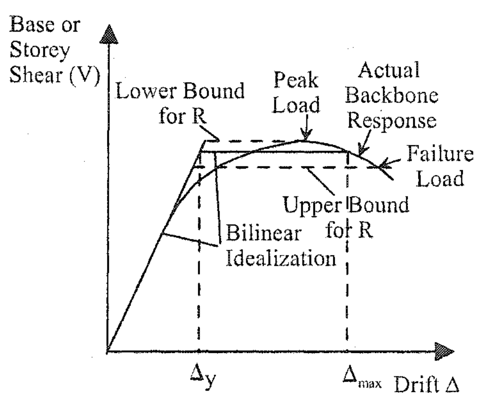


Fig. 4.5: Backbone Curve for *R*-Value Calculation

iii. Estimate the ductility modification factor R_μ

The relationship between the ductility modification factor, R_μ, and the ductility demand factor, μ, must be determined. Newmark and Hall *(1982)* demonstrated that for a single degree of freedom system with a period greater than 0.5 seconds, the maximum lateral displacement of a non-linear system is almost equal to the maximum displacement of the corresponding linear system. Therefore:

$$R_\mu = \mu \quad (\mathrm{T} \geq 0.5) \tag{4-20}$$

In addition, Newmark and Hall have shown that for a system with a period from approximately 0.125 to 0.5 seconds, the strain energies for the elastic and the elasto-plastic cases are approximately the same. Equating the areas under the two curves, *i.e.* elastic and elasto-plastic curves, leads to:

$$R_\mu = \sqrt{2\mu - 1} \quad (\mathrm{T} < 0.5) \tag{4-21}$$

iv. Establish the overstrength factor Ω

It is possible to determine the yield strength, V_y, of a shear wall from the load reached during the last stable loop. A stable loop is obtained when the shear force in the first and the fourth cycle of a given displacement amplitude is within 5% *(SEAOSC, 1997)*. The first significant yield strength, V_s, occurs at the point in the elastic segment of the ideal bilinear curve beyond which the backbone curve deviates significantly from the idealised curve. The value of V_s can also be taken as the prescribed design strength given by the respective design standards, which is adopted in this thesis as well. Thus, as shown for the definition of the ductility factor, one can define the over-strength factor, Ω_0, as a function of the nominal and first significant yield strengths. As noted previously in Section 4.1, this overstrength factor can also be further increased by other sources.

$$\Omega_0 = \frac{V_y}{V_s} \tag{4-22}$$

The Applied Technology Council *(1995)* recommends another method with which overstrength factors can be evaluated. Once the backbone curve is plotted one can calculate the base shear force, V_0, at the drift corresponding to the limiting state of response. The typical limiting responses include maximum inter-storey drift and maximum plastic hinge rotation. The design base shear force at the working stress level is given by the following equation in the 1985 UBC *(ICBO, 1985)*:

$$V_D = (ZIKCS) \times W \tag{4 – 23}$$

The parameters Z and I are used to quantify the seismic zone and the importance of the building, respectively. The parameter S is used to account for site characteristics, and C is a numerical period of vibration of the building and the defined spectral shape. K is a numerical coefficient referred to as the horizontal force factor, and W is the total dead load of the building.

The design base shear at the strength level (V_d) can be calculated by multiplying the working-stress design base shear (V_D) by a seismic load factor ($V_d \approx V_D \times 1.40$). It follows that the over-strength factor can be calculated using the following expression:

$$\Omega = \frac{V_0}{V_d} \tag{4 – 24}$$

v. Evaluate the force modification factor

The definition of the force modification factor depends on the building code that is to be implemented in the design of the shear wall. For the codes that are under consideration R is defined as:

For NBCC, $$R = R_\mu \tag{4 – 25}$$

For UBC 94, $$R_w = R_\mu \Omega Y \tag{4 – 26}$$

For NEHRP and UBC 97, $$R = R_\mu \Omega \tag{4 – 27}$$

4.3 Example of Force Modification Factor Calculation

The load *vs.* deflection hysteresis of shear wall specimen No. AISI-OSB1 tested by Serrette *et al. (1996b)* (Fig. 4.6) was used to provide an example of the procedure that was followed to determine an appropriate force modification factor.

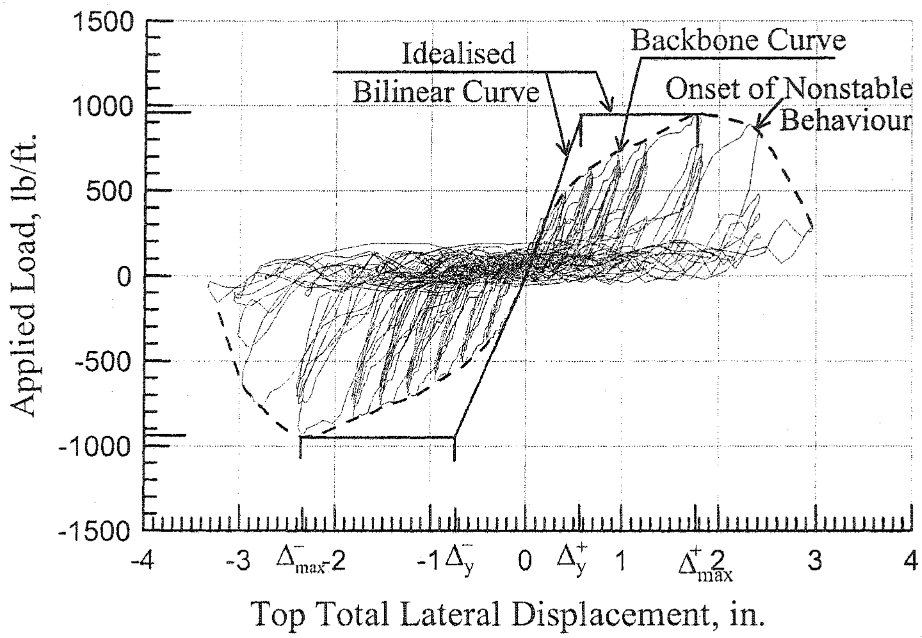


Fig. 4.6: Load *vs.* Deflection Hysteresis for Specimen No. AISI-OSB1 *(Serrette et al., 1996b)*

i. Backbone curve.

A smooth curve is drawn to connect the peak load points of the successive cycles in order to obtain the unidirectional backbone curve, shown as a dashed line in Fig. 4.6.

ii. Idealised bilinear curve.

The area enclosed by the shear load *vs.* deflection hysteresis is considered as a measure of the dissipated energy. Typically, after the peak load is reached in the first cycle of a given displacement amplitude, the maximum load that can be carried in subsequent cycles degrades significantly, *i.e.* the behaviour is not stable. This indicates that there will be sudden losses in shear stiffness and capacity of the wall under severe cyclic loading. From the discussion of test results provided in Serrette *et al. (1996b)*, it is known that wall No. AISI-OSB1 failed when the panel pulled over the screw heads and became unzipped. Fastener unzipping is considered as a brittle failure mode because of the sudden drop in load carrying capacity of the shear wall. Due to this brittle failure a conservative approach was taken where the plastic plateau was defined as the maximum load reached, which provides a lower bound solution for the calculated R-value. The first segment of the idealised bilinear elasto-plastic curve begins at the origin and has the same slope as the elastic stiffness of the test wall during the initial quasi-static displacement cycles (Fig. 4.6).

iii. Ductility factor μ

The ductility factor is also obtained from information contained in Fig. 4.6. The yield (Δ_y) and maximum displacement (Δ_{max}) are taken as the average of the positive value and the negative value for yield (Δ_y^+ and Δ_y^-) and maximum displacement (Δ_{max}^+ and Δ_{max}^-), respectively. Therefore, Δ_y = 0.65" and Δ_{max} = 2.05" are identified and $\mu = \Delta_{max} / \Delta_y$ = 3.15.

iv. Ductility modification factor R_μ

Additional information with regards to the shear wall test is required to calculate the ductility modification factor. Serrette *et al. (1996b)* have recorded the height of the wall as 8' (2.44 m) and the length as 4' (1.22 m). The natural period of the wall system, T, can be approximated by using the empirical formula given by the NBCC *(NRCC, 1995)*:

$$T = \frac{0.09 h_s}{(D_s)^{1/2}} \qquad (4-28)$$

where h_s and D_s are the height and length of the structure, respectively. In this case, h_s = 2.44 m and D_s = 1.22 m; hence T = 0.2s (< 0.5s), therefore:

$$R_{\mu} = \sqrt{2\mu - 1} = 2.3$$

v. Overstrength factor Ω

As noted previously, the overstrength factor is defined as the ratio of the nominal strength to the code defined design capacity. Considering that the Uniform Building Code *(ICBO, 1997)* specifies that the shear capacity for load and resistance factor design is the nominal strength multiplied by a resistance factor of 0.55, the overstrength, Ω, can be taken as 1/0.55 = 1.82. On the other hand, for this test, the wall assembly was only subjected to an in-plane lateral load without additional gravity forces that simulate the self-weight of the building. Hence the ATC-19 *(1995)* method to calculate the design base shear force cannot be followed. Furthermore, considering that the existing shear wall test programs in North America consisted of single wall panel specimens (only one element) instead of a complete structure, as detailed in Chapter 2, the overstrength considered in this thesis refers to the element overstrength. A conservative approach will be taken where the system over-strength is assumed equal to 1.0.

vi. Force modification factor R

The force modification factors for seismic design are given as follows:

NBCC	$R = R_{\mu} = 2.3$
UBC 94	$R_w = R_{\mu}\,\Omega\,Y = 2.3 \times 1.82 \times 1.4 = 5.9$
NEHRP and UBC 97	$R = R_{\mu}\,\Omega = 2.3 \times 1.82 = 4.2$

Additional force modification factors for other tests completed by Serrette *et al. (1996b, 1997b)* and COLA-UCI *(2001)* can be found in Appendix 'C' from Table C1 to Table C4, and a summary of the results is provided in Table 4.1. All those R-values are only applied to single-storey walls. The results are all based on peak load, which can be considered as a lower bound for the R-values, and the overstrength is taken as 1.82 as discussed previously. It must also be noted that values listed in Table 4.1 were evaluated from the results of single shear wall tests, which may not necessarily correspond to the overall behaviour of a real building. The contributions of various boundary conditions, connections and non-structural components, gravity loads, *etc,* may impact on the distribution of seismic forces to structural shear walls.

Table 4.1: Calculated *R*-Values for Steel Stud Shear Walls

Specimen	R_w (UBC, 94)	R (NEHRP, UBC, 97)	R (NBCC)	*Specimen*	R_w (UBC, 94)	R (NEHRP, UBC, 97)	R (NBCC)
AISI-OSB1[1w]	5.9	4.2	2.3	AISI-D1[1s]	6.9	4.9	2.7
AISI-OSB2[1w]	6.3	4.5	2.5	AISI-D2[1s]	6.5	4.7	2.6
AISI-OSB3[1w]	4.8	3.4	1.9	AISI-E1[1w]	8.3	5.9	3.2
AISI-OSB4[1w]	6.1	4.4	2.4	AISI-E2[1w]	7.2	5.1	2.8
AISI-OSB5[1w]	5.0	3.6	2.0	AISI-E3[1w]	6.8	4.9	2.7
AISI-OSB6[1w]	5.1	3.6	2.0	AISI-E4[1w]	6.8	4.9	2.7
AISI-OSB7[1w]	3.9	2.8	1.6	AISI-E5[1w]	4.6	3.3	1.8
AISI-OSB8[1w]	4.1	2.9	1.6	AISI-E6[1w]	5.0	3.6	2.0
AISI-PLY1[1w]	4.7	3.3	1.8	AISI-F1[1s]	6.6	4.7	2.6
AISI-PLY2[1w]	5.7	4.1	2.2	AISI-F2[1s]	6.9	4.9	2.7
AISI-PLY3[1w]	5.9	4.2	2.3	AISI-F3[1s]	5.9	4.2	2.3
AISI-PLY4[1w]	5.0	3.6	2.0	AISI-F4[1s]	5.6	4.0	2.2
AISI-PLY5[1w]	5.0	3.6	2.0	Group14A[2w]	6.6	4.7	2.6
AISI-PLY6[1w]	5.2	3.7	2.0	Group14B[2w]	6.5	4.6	2.5
AISI-PLY7[1w]	4.1	2.9	1.6	Group14C[2w]	6.0	4.3	2.4
AISI-PLY8[1w]	4.1	3.0	1.6	Group15A[2w]	6.3	4.5	2.5
AISI-A1[1w]	6.1	4.4	2.4	Group15B[2w]	6.0	4.3	2.4
AISI-A2[1w]	5.1	3.7	2.0	Group15C[2w]	6.1	4.4	2.4
AISI-A3[1w]	5.0	3.6	2.0	Group16A[2w]	6.0	4.3	2.4
AISI-A4[1w]	4.8	3.5	1.9	Group16B[2w]	5.2	3.7	2.0
AISI-A5[1w]	5.0	3.6	2.0	Group16C[2w]	5.1	3.6	2.0
AISI-A6[1w]	4.8	3.4	1.9	Group17A[2w]	5.7	4.0	2.2
AISI-A7[1w]	5.1	3.6	2.0	Group17B[2w]	6.6	4.7	2.6
AISI-A8[1w]	5.1	3.7	2.0	Group17C[2w]	6.2	4.4	2.4
AISI-B1[1w]	5.4	3.8	2.3	Group18A[2w]	6.9	4.9	2.7
AISI-B2[1w]	6.2	4.4	2.4	Group18B[2w]	6.6	4.7	2.6
AISI-B3[1w]	5.3	3.8	2.1	Group18C[2w]	6.9	5.0	2.7
AISI-B4[1w]	5.8	4.1	2.3	Group19A[2w]	7.0	5.0	2.8
AISI-C1[1x]	4.0	2.9	1.6	Group19B[2w]	6.5	4.7	2.7
AISI-C2[1x]	4.6	3.3	1.8	Group19C[2w]	5.8	4.2	2.3

[1]Serrette *et al. (1996b, 1997b)* [2] COLA-UCI *(2001)* [w] denotes steel walls sheathed with wood panels [s] denotes steel walls sheathed with sheet steel [x] denotes steel walls with X-braces

4.4 *Comparison with Existing R-Values and Hysteretic Curves for Wood and Masonry Shear Walls*

i. Wood Shear Walls

Wood shear walls have long been utilised to withstand high wind and earthquake loading. Timber structures in general, provide good insulation value, are versatile and easily constructed, are readily available, and posses a high strength-to-weight ratio, which has made them popular throughout North America. Furthermore, when adequately detailed for seismic loading the lightweight nature and flexibility of wood structures result in a small inertia force, and hence wood shear walls act as an efficient energy absorber during earthquakes. In general, building codes and standards, *e.g. UBC (ICBO, 1997)* and *CSA O86-01 (2001)* provide design shear capacity values for wood shear walls based on test data. To evaluate the cyclic response of wood shear walls, most experimental studies focused on the effect of shear capacity and ductility under cyclic loading, where specimens were constructed with different details; such as oversize panels, aspect ratio, openings, blocking, double-sheathing, connection details, *etc* (*Dolan and Johnson, 1997; Durham et al., 1999; Filiatrault, 1990; Folz and Filiatrault, 2001; Gutkowski and Castillo, 1988; He et al., 1999; Popovski et al., 1998; Rose, 1998; Tissell, 1993).*

The Canadian Wood Council Wood Design Manual *(CWC, 1995)* prescribes the use of different R-values depending on the type of lateral load resisting system and its ability to absorb earthquake induced energy. An R-value of 3.0 is assigned to all nailed plywood, waferboard and oriented strand board (OSB) shear walls which satisfy certain requirements, including; panel orientation and configuration, panel thickness, width of framing members, fastener schedule, *etc*. A more recent design manual from the CWC *(2001)* recommends that a lower force modification factor ($R = 2$) be used for the design of shear walls sheathed with a combination of wood-based panel and gypsum wallboard when considering the contribution of the gypsum wallboard to the shear resistance of the walls. Walls that are sheathed with wood-based panels alone, or when the contribution of the gypsum wallboard is neglected may still be designed with $R = 3$. The CSA O86-01 *(2001)* design standard prescribes the main requirements for the design of wood shear walls as follows:

1) The maximum aspect ratio (height-to-length ratio) of a shear wall segment shall be 3.5:1.

2) When the factored dead loads are not sufficient to prevent overturning, hold-down connections shall be provided to resist the factored uplift forces, or an anchorage shall transfer the uplift force, located on the bottom plate within 300 mm from both ends of the shear wall segment.

3) Framing members shall be at least 38 mm wide and be spaced no greater than 600 mm apart. The panels can be installed horizontally or vertically, with nails spaced at 300 mm on centre along intermediate framing members.

4) The nominal thickness of panels shall be no less than 7.5 mm.

5) Common wire nails, with a minimum diameter of 2.84 mm, shall be spaced between 50 mm and 150 mm at panel edges. Nails are to meet the minimum penetration distance and be firmly driven into the framing but not over-driven.

Wood shear wall specimens that were tested at Virginia Polytechnic Institute and State University *(Salenikovich et al., 2000)* and at the University of California at Irvine *(COLA-UCI, 2001)* were used in the comparison of steel stud shear walls. To represent the typical behaviour of wood shear walls, the specimens (listed in detail in Appendix 'C' Table C5) were chosen according to the following criteria:

- Satisfy all of the requirements in the CSA 086-01 Wood Design Standard;
- Include panels with different aspect ratio, fastener spacing, type of sheathing, and orientation;
- Panels attached on one side or on two-sides.
- Sheathed with panels of typical thickness and size.

The R-values of the selected specimens were calculated based on the peak load following the procedure described in Section 4.2. The design shear capacities for wood walls were obtained from the Shearwall Selection Tables in the Wood Design Manual (*CWC, 2001*). The equivalent wood panel thickness listed in the tables for 15/32" (11.9 mm) plywood was taken as 12.5 mm. For those wood walls sheathed with two panel layers, one on each side of the studs, the shear resistance is additive, while for those sheathed with more than one layer on one side, the shear resistance is taken as the resistance of the innermost panel. The overstrength, listed in Appendix 'C', Table C5, was obtained by dividing the nominal strength of the walls obtained from the test data by the corresponding design strength (Appendix 'D', Table D1). Therefore, the overstrength for those walls with more than one layer on one side such as walls Group 35A,B,C, is very high since the outmost panel would also contribute to the nominal strength, while this outmost panel is ignored when calculating the design strength using method in Wood Design Manual. The summarised results are shown in Table 4.2, with more detailed information listed in Appendix 'C', Table C5.

Table 4.2: Calculated R-Values for Wood Walls

Specimen	R_w (UBC, 94)	R (NEHRP, UBC, 97)	R (NBCC)	*Specimen*	R_w (UBC, 94)	R (NEHRP, UBC, 97)	R (NBCC)
Group 03A[1]	7.7	5.5	2.9	Group 23A[1]	7.2	5.1	3.0
Group 03B[1]	6.6	4.7	2.5	Group 23B[1]	7.0	5.0	2.8
Group 03C[1]	8.3	5.9	3.0	Group 23C[1]	6.4	4.6	2.8
Group 04A[1]	7.1	5.1	2.8	Group 34A[1]	5.3	3.8	2.5
Group 04B[1]	6.6	4.7	2.6	Group 34B[1]	6.2	4.4	3.0
Group 04C[1]	5.9	4.2	2.7	Group 34C[1]	5.5	3.9	2.7
Group 06A[1]	6.0	4.3	2.4	Group 35A[1]	12.4	8.9	2.6
Group 06B[1]	6.1	4.3	2.3	Group 35B[1]	13.3	9.5	2.7
Group 06C[1]	6.4	4.6	2.4	Group 35C[1]	12.1	8.7	2.4
Group 09A[1]	6.4	4.6	2.5	Group 36A[1]	7.0	5.0	2.9
Group 09B[1]	6.2	4.4	2.5	Group 36B[1]	4.0	2.9	2.1
Group 09C[1]	6.3	4.5	2.4	Group 36C[1]	5.7	4.1	2.5
Group 12A[1]	8.6	6.1	3.7	04FAc1[2]	7.8	5.6	3.0
Group 12B[1]	6.9	4.9	3.0	04FAc2[2]	9.4	6.7	2.9
Group 12C[1]	7.8	5.5	3.3	08FAc1[2]	9.3	6.6	3.2
Group 13A[1]	6.0	4.3	2.6	08FAc2[2]	8.2	5.8	2.9
Group 13B[1]	6.7	4.8	2.6	12FAc1[2]	9.9	7.1	3.3
Group 13C[1]	6.7	4.8	2.7	12FAc2[2]	10.7	7.7	3.3

[1]COLA-UCI *(2001)* [2]Salenikovich *et al. (2000)*

ii. Masonry Shear Walls

Masonry is a common material that has been used in building construction for a number of centuries. Although brittle as a material, the addition of reinforcement helps to improve the seismic behaviour of masonry shear walls. According to Tomaževič *(1999)*, when subjected to seismic load, structural masonry walls develop three types of failure mechanism, including; sliding shear, shear, and flexural failure, which depend on the aspect ratio, load condition, quality of materials, and amount of vertical and horizontal reinforcement. Experimental studies by Shing *et al. (1989, 1990a, 1990b, 1991)* and Tomaževič *(1999)* indicate that the walls dominated by flexural yielding at failure exhibit a more ductile behaviour in comparison to those dominated by a shear failure mechanism.

The NBCC *(NRCC, 1995)* has three classifications for masonry structures, each with a different value for the force modification factor to account for the inherent capability of reinforcement to exhibit inelastic performance. Values are defined as follows: i) R = 1.0 for unreinforced masonry, ii) R = 1.5 for reinforced masonry, and iii) R = 2.0 for reinforced masonry walls with nominal ductility.

The CSA S304.1 *(1994)* Masonry Design Standard requires minimum reinforcement for walls designed to resist seismic forces. This includes vertical and horizontal steel having a minimum total area of $0.002A_g$, distributed as $A_{sv} = 0.002A_g\alpha$, $A_{sh} = 0.002A_g(1-\alpha)$ (where $0.33 \leq \alpha \leq 0.67$), where A_g is the gross cross-sectional area of the wall (wall thickness $\times 1$ m), and A_{sv} and A_{sh} are the areas of vertical and horizontal reinforcement, respectively. In addition, reinforcement shall be spaced at centre-to-centre intervals of no more than 6 times the wall thickness or 1.2 m. Shear walls that are considered to have nominal ductility (R = 2.0), should be designed in accordance with Appendix A of the S304.1 Standard, where requirements for plastic hinge length, factored shear resistance, ductility and minimum reinforcement are addressed. The main features of Appendix A of the S304.1 Standard include:

1) Structures are required to be of reinforced masonry and the redistribution of moments obtained from an elastic analysis is not permitted.

2) Vertical reinforcement shall be spaced not more than one-quarter of the wall effective depth, six times the wall thickness or 1200 mm, whichever is less. Horizontal reinforcement shall also be continuous to the ends of the wall with 180° hooks around vertical reinforcing bars and they shall not be lapped within 600 mm or the neutral axis depth, c, (the distance from extreme compression fibre to the neutral axis in a flexural member), whichever is greater, from the end of the wall.

3) Other requirements relating to the plastic hinge region: limiting the extent of the plastic hinge region, limiting the maximum compressive strain to 0.0025 (which can be considered satisfied if $c < 0.2\ l_w$, where l_w is the length of the wall), and reducing the shear resistance contributed by masonry and axial compressive load by one-half.

Masonry shear wall specimens tested at the University of Colorado *(Shing et al., 1991)* were used in a comparison of the behaviour of wood stud, masonry and steel stud walls. It was necessary to assign NBCC defined R-values to these walls following the CSA S304.1 requirements. All of the masonry shear walls that were considered met the minimum requirement for total reinforcement ratio and distribution. For those walls with $R = 2.0$, the extent of the plastic hinge region above the base of the wall is defined in Appendix A of the S304.1 as l_p = greater of l_w or $h_w/6$, where l_w and h_w are the length and height of the wall, respectively. For the Shing *et al.* test specimens, $l_w = h_w$, hence the plastic hinge length was considered as the entire height of the wall.

For a wall with $R = 2.0$, the factored shear capacity of the portion of the wall that lies outside of the plastic hinge region shall be calculated according to CSA S304.1 Clause 11.5.3. This is the same approach used to calculate the factored shear resistance of a wall with $R = 1.5$, considering the contribution of the masonry wall, the axial compressive load and the shear reinforcement, as shown in Equations (4 – 29) and (4 – 30). However, for the section of wall that is within the plastic hinge region, Clause A6.1 of CSA S304.1

specifies that the factored shear resistance, as determined using Clause 11.5.3, which depends on the masonry and the axial compressive load, shall be reduced by one-half.

For the Shing *et al.* shear wall tests the compressive strength of the masonry materials were determined by prism tests and the tensile strength of the reinforcing steels were obtained from coupon tests. Hence, the resistance factor values, ϕ_m and ϕ_s, which account for some uncertainties with respect to material strength, were both taken as 1.0 instead of 0.55 and 0.85, respectively. An example calculation is provided in Appendix D4 and the results for all masonry shear wall specimens are listed in Table 4.3. The calculated ultimate shear values in Table 4.3 correspond to the flexural ultimate states following Clause 10.2.3.1.3 of CSA S304.1 (ε_u =0.003).

- Analysis results

R-values were assigned to the walls according to the CSA S304.1 Masonry Design Standard. CSA S304.1 requires that besides meeting the minimum reinforcement requirement, reinforced masonry walls should have enough flexural and shear resistance to carry the expected applied loads. Thus, the calculated shear capacity (V_R) from the CSA standard was compared with the maximum applied shear load (V_F) of the wall before the wall failed in flexure. The maximum applied shear load was considered as the smaller of the measured test shear at diagonal cracking, and the calculated shear at the flexural ultimate limit state. For the walls with $V_R < V_F$ (within 5% difference it can be assumed $V_R = V_F$), such as wall 9, 13, and 14, an *R*-value of 1.0 was assigned since the shear performance did not improve significantly in comparison with an unreinforced masonry wall, due to lack of adequate shear resistance. According to Appendix A of CSA S304.1, for an reinforced wall with nominal ductility, the factored shear resistance within the plastic hinge region (V_{RP}) should be recalculated to reduce the contribution of the masonry and axial force. Therefore, for the walls with $V_{RP} > V_F$, that met both the ductility requirement ($c < 0.2l_w$, then the limiting strain requirement is satisfied according to Appendix A in CSA S304.1) and the minimum reinforcement requirement, an *R*-value of 2.0 was defined, *e.g.* wall 8. For other reinforced masonry walls that did not fall into these two categories, $R = 1.5$ was utilised. The results are listed in Table 4.3.

Table 4.3: Assignment of *R*-Values in NBCC for Masonry Walls *(Shing et al., 1991)*

Wall No.	f_m' (psi)	Ver. steel	Hor. steel	*Shear capacity* (Clause 11.5.3) (V_R kN)	*Shear capacity* (App. A) (V_{RP} kN)	*Axial Force* (kips)	Ultimate Shear (kN) Final	Ultimate Shear (kN) Crack	Cal.[1]	ε @ ultimate	$c < 0.2l_w$	Design *R* in NBCC
1	2900	5 x #5	5 x #4	357	240	80	367	352	342	0.003	Y	1.5
2	2900	5 x #5	9 x #3	374	246	108	403	370	383	0.003	Y	1.5
3	3000	5 x #7	5 x #3	328	192	108	456	356	565	< 0.003	N	1.0
4	2600	5 x #7	5 x #3	197	127	0	354	236	407	< 0.003	Y	1.0
5	2600	5 x #7	5 x #3	242	149	40	385	267	462	< 0.003	N	1.0
6	2600	5 x #5	5 x #3	197	127	0	220	216	200	0.003	Y	1.5
7	3000	5 x #7	5 x #3	252	154	40	432	278	473	< 0.003	Y	1.0
8	3000	5 x #5	5 x #4	273	197	0	216	---	202	0.003	Y	2.0
9	3000	5 x #5	5 x #3	328	192	108	427	409	385	0.003	Y	1.0
10	3200	5 x #5	5 x #3	257	157	40	303	263	278	0.003	Y	1.5
11	3200	5 x #7	5 x #4	277	199	0	409	249	420	< 0.003	Y	1.5
12	3200	5 x #5	5 x #4	322	221	40	316	310	278	0.003	Y	1.5
13	3300	5 x #6	5 x #4	399	260	108	501	498	463	0.003	Y	1.0
14	3300	5 x #6	5 x #3	335	196	108	467	452	463	0.003	Y	1.0
15	3300	5 x #6	5 x #4	324	222	40	392	327	361	0.003	Y	1.5
16	2500	5 x #7	5 x #4	347	250	108	536	383	603	< 0.003	N	1.0

[1] the shear is the calculated maximum horizontal force based on strain limits.

The masonry wall *R*-values for the different codes were calculated using the method described in Section 4.2. As discussed previously, the overstrength factor can be obtained by dividing the nominal strength (from tests) by the design shear strength. The factored shear resistance was calculated following Clause 11.5.3.1 in the CSA S304.1 Masonry Design Standard:

$$V_r = \phi_m(v_m b_w d + 0.25P)\gamma_g + \phi_s(0.60 A_v f_y \frac{d}{s}) \tag{4-29}$$

and

$$V_r \le \phi_m(0.4)\sqrt{f_m^{'}}\, b_w d \gamma_g \tag{4-30}$$

where:

ϕ_m = Resistance factor for masonry, $\phi_m = 0.55$

v_m = Shear strength attributed to the masonry

$$v_m = 0.16\left(2 - \frac{M_f}{V_f d}\right)\sqrt{f_m^{'}} \tag{4-31}$$

b_w = Width of the wall

d = Effective depth of the wall, distance from extreme compression fibre to centroid of tension reinforcement, which need not be taken less than $0.8l_w$ for walls with flexure reinforcement distributed along the length

P = Axial compressive load of the wall, based on 0.85 times dead load of the wall

ϕ_s = Resistance factor for reinforcement, $\phi_m = 0.85$

A_v = Cross-sectional area of shear reinforcement

f_y = Yield strength of reinforcement

s = Spacing of shear reinforcement measured parallel to the longitudinal axis of the member

$f_m^{'}$ = Compressive strength of masonry at 28 days

γ_g = Factor to account for partially grouted walls, 1.0 for fully grouted walls

M_f and V_f are the concurrent factored moment and factored shear at the section under consideration and $\frac{M_f}{V_f d}$ need not be taken more than 1 nor less than 0.25

The results of the calculated factored shear strength and the corresponding overstrength factors are listed in Appendix 'D', Table D1. A summary of results is shown in Table 4.4 and more detailed information is available in Appendix 'C', Table C6.

Table 4.4: Calculated *R*-Values for Masonry Walls *(Shing et al., 1991)*

Specimen	R_w (UBC, 94)	R (NEHRP, UBC, 97)	R (NBCC)	*Specimen*	R_w (UBC, 94)	R (NEHRP, UBC, 97)	R (NBCC)
Specimen 1	6.8	4.8	2.8	Specimen 9	8.4	6.0	2.9
Specimen 2	10.2	7.3	4.0	Specimen 10	7.8	5.6	3.4
Specimen 3	9.7	6.9	3.8	Specimen 11	5.7	4.1	3.1
Specimen 4	6.7	4.8	2.6	Specimen 12	7.6	5.4	3.7
Specimen 5	5.1	3.6	2.0	Specimen 13	9.7	6.9	3.0
Specimen 6	6.3	4.5	2.6	Specimen 14	8.1	5.8	2.6
Specimen 7	7.0	5.0	2.8	Specimen 15	9.6	6.9	4.5
Specimen 8	6.4	4.6	3.1	Specimen 16	8.3	5.9	2.9

iii. Comparison of Steel, Wood and Masonry Force Modification Factors

Force modification factors were determined for the wood and masonry shear walls following the procedure detailed previously. These test-based values were then compared with those R-values specified in the NBCC (Fig. 4.7), as well as NEHRP & UBC 97 (Fig. 4.8). As indicated previously, the values in UBC 94 differ from those in NEHRP & UBC 97 by a constant, therefore, the comparison between the test-based values and UBC 94 is similar to that found for NEHRP & UBC 97.

- NBCC

The current NBCC *(NRCC, 1995)* does not list an *R*-value for steel-stud shear walls, hence $R = 1$ must be used for design. In general, the use of the procedure described in this paper to determine force modification factors for masonry walls yields a high ductility ratio because these walls are significantly stiffer, and hence have small yield displacement values, in comparison with steel and wood walls. Thus, a direct comparison of the lateral ductility cannot be made because of the substantial variation in behaviour between the wall types. In contrast, the construction of wood-stud walls and steel-stud walls is similar; hence, a direct comparison of the force modification values calculated using test results may be carried out. As shown in Fig. 4.7, the calculated test-based R-values for wood walls are relatively similar to those defined in the NBCC. The calculated R-values for steel walls fluctuate around 2.0, ranging from 1.6 to 2.8 (except AISI-E1) while those of wood walls range from 2.1 to 3.3 (except 12A). It can be seen from Table 4.5 that the mean R-value for steel wall is 2.2 and the mean calculated values for wood walls, separated according to the code defined force modification factors, with R = 3.0 and R = 2.0 are 2.8 and 2.6, respectively. Wood walls defined at R = 2 provide more consistent calculated R-values, which may be due to the smaller number of construction configurations that have been tested, in contrast to the wood walls with R = 3.0 and steel walls (Table 4.5). In general the steel stud walls exhibit a slightly lower ability to maintain load-carrying ability in the inelastic lateral deformation range.

Table 4.5: Statistic Parameters of R-Values (*NBCC*) for Steel and Wood Walls

	Number of Specimens	Mean Value	Standard Deviation	Coefficient of Variation
Steel Walls	60	2.2	0.367	16.4%
Wood Walls (R = 3)	30	2.8	0.339	12.2%
Wood Walls (R = 2)	6	2.6	0.242	9.5%

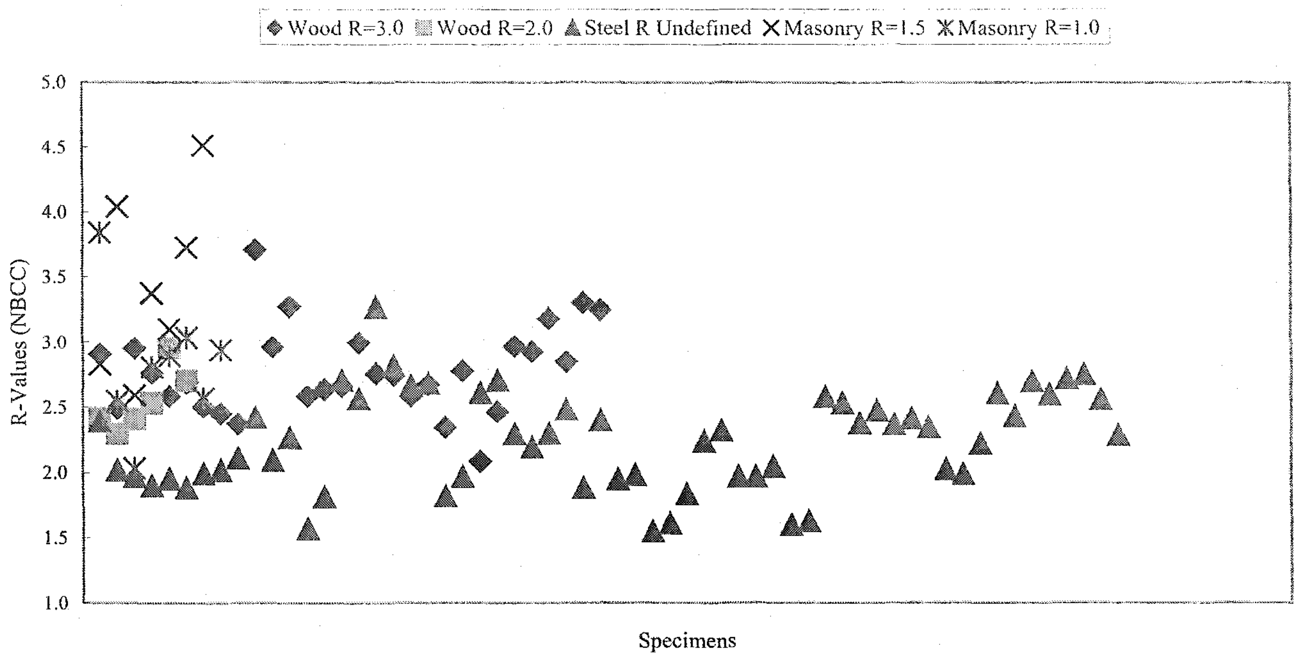


Fig. 4.7: Comparison of Calculated *R*-Values *(NRCC, 1995)* for Wood, Masonry and Steel Shear Wall

- **NEHRP & UBC 97**

In NEHRP and UBC, the assignment of R-values is based on the materials used in the structure, as well as the basic structural system, which should fall into one of the following categories; bearing wall, building frame, moment-resisting frame, and dual system. The 1997 UBC assigns an R-value of 6.5 for light-framed walls sheathed with wood structural panels in a building frame system, when the wall is relied on to resist lateral load only, while $R = 5.0$ is prescribed for all other light-framed walls in the same category.

It can be seen from Fig. 4.8 that the calculated test-based R-values for wood walls (without gypsum) fluctuate around 5.5 (with a range from 4.1 to 9.5), while those with gypsum wallboard fluctuate around 4.5 (range from 3.8 to 4.6). These values are generally lower than the code prescribed values of 6.5 and 5.0, respectively. This result may have occurred because these R-values are based on peak load and therefore provide the lower bound solution as stated previously. Steel walls fall in a similar range (from 2.8 to 5.0, except AISI-E1) to that of wood walls with gypsum. Steel walls sheathed with sheet steel tend to exhibit higher R-values (4.0 to 4.9).

Table 4.6 shows that steel walls provide a mean R-value of 4.1, which is lower than that associated with wood walls without gypsum, although similar to the mean value of wood walls with gypsum wallboard (with a value of 4.2). It can also be seen that the calculated R-values of wood walls vary to a greater extent than steel walls due to the use of a constant overstrength factor for steel walls, while for wood walls, the overstrength factors are determined by taking the nominal strengths from test results and divided by the design shear capacities obtained from the Shearwall Selection Tables in the Wood Design Manual *(CWC, 2001)*. Wood walls with gypsum wallboard exhibit rather consistent R-values, which, as stated previously, may be due to the smaller number of specimens under consideration.

Table 4.6: The Statistic Parameters of *R*-Values (*NEHRP & UBC 97*) for Steel and Wood Walls

	Number of Specimens	Mean Value	Standard Deviation	Coefficient of Variation
Steel Walls	60	4.1	0.67	16.4%
Wood Walls without gypsum	30	5.6	1.52	27.4%
Wood Walls with gypsum	6	4.2	0.31	7.2%

It is relevant to note that in general, the calculated overstrength factor for wood walls with structural panels ranges from 1.7 to 2.4, except those walls sheathed with two panels on the same side where only one panel layer is considered to contribute to the shear resistance. This may lead to an underestimate of the shear resistance and a corresponding increase in the overstrength factor. However, for wood walls with gypsum wallboard, the overstrength factor values are approximately 1.5, while for masonry wall, the values range from 1.5 to 2.3. The overstrength factor for steel walls is taken as 1.82 as analysed previously, which falls into the overstrength range for both wood and masonry walls.

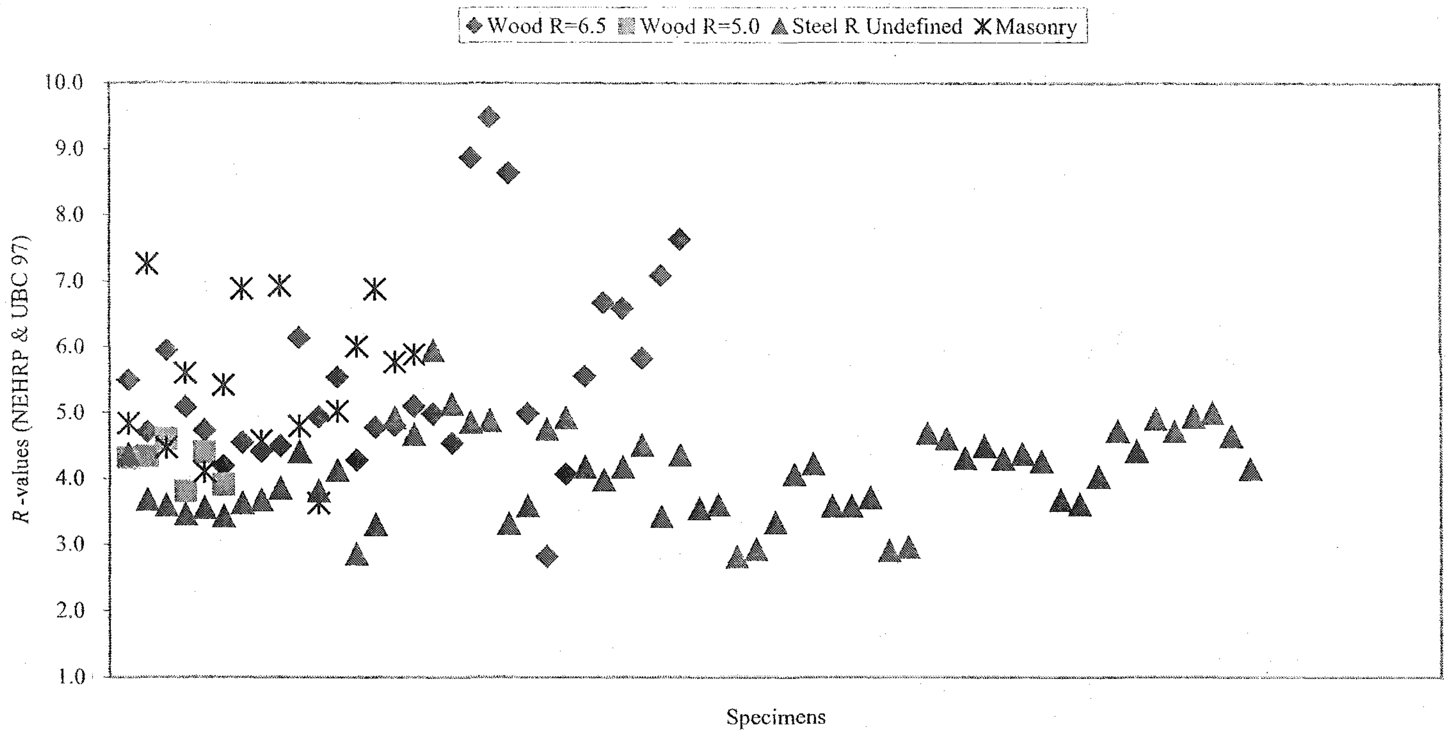


Fig. 4.8: Comparison of Calculated *R*-Values (*NEHRP&UBC 97*) for Wood, Masonry and Steel Shear Walls

4.5 Parameter Study

While it is of importance that a structure exhibits high ductility to resist possible earthquake forces; other parameters, including hysteretic energy dissipation, resistance to degradation, inherent redundancy, load level, failure mode, *etc*, also play an important role and should be considered in the determination of force modification values for design. Furthermore, the calculated R-values listed in this thesis were based on the measured peak load during a test without consideration of the subsequent degradation in load. Hence, post peak load behaviour of the shear walls has been overlooked. In an attempt to better understand the behaviour of laterally loaded shear walls, and to assess the appropriateness of the calculated R-values for cold-formed steel stud shear walls, a parameter study was completed. Comments on the various parameters that were taken into consideration are provided in the following sections and a listing of the characteristics that were compared is located in Appendix 'D', Tables D2 and D3.

i. The ratio of displacements at failure to those at peak load ($\delta = \Delta_{fail}/\Delta_{peak}$)

The ratio $\delta (= \Delta_{fail}/\Delta_{peak})$ is an indication of the ability of a shear wall to limit the amount of load degradation with increasing lateral deflection. In a best-case scenario the capacity of a structure should be roughly maintained, with no sudden decrease, if earthquake energy is to be more efficiently absorbed. The failure load for wood structures and masonry structures is $0.8F_{peak}$ *(ISO, 1998)* and $0.9F_{peak}$ *(CSA S304, 1994)*, respectively. For this comparison, the failure load for cold-formed steel shear walls was assumed to be $0.9F_{peak}$. In terms of performance, the higher the δ-value, the better the resistance to load degradation. A comparison of δ-values is shown in Fig. 4.9.

ii. Hysteretic energy dissipation (W_D)

The dissipation of hysteretic energy during cyclic loading is an important attribute for a structure to possess if it is to survive an earthquake. Favourable energy dissipation characteristics enable a better seismic response and thus, support the assignment of higher R-values. The energy dissipated in one complete cycle (W_D) is measured as the area enclosed by the storey shear *vs.* deflection curve, which can be obtained by carrying out a numerical integration of the recorded test results. The cumulative dissipated energy of all cycles up to the peak load cycle was calculated and normalised by the peak load in order that a comparison of the different tests could be made (Fig. 4.10).

iii. Damping ratio (ζ_{eq})

Damping is another important characteristic in seismic design, as it reflects the ability of a structure to dissipate the energy due to internal or external friction. Normally, the damping ratio used for design is 5% of critical damping. The higher the damping ratio of a structure, the better it can dissipate energy during earthquakes. The equivalent viscous damping ratio for each cycle, ζ_{eq}, can be approximated as follows *(Salenikovich et al., 1999)*:

$$\zeta_{eq} = \frac{1}{4\pi}\frac{W_D}{U_0} = \frac{W_D}{2\pi PA} \qquad (4-32)$$

where W_D and U_0 are dissipated energy and the strain energy of the cycle under consideration, respectively. P and A are the average peak load and the average amplitude in that cycle, respectively. The comparison of damping ratio for wood and steel walls is shown in Fig 4.11. Due to the unavailability of the original test data of masonry walls, the damping ratio for masonry walls at peak load could not be determined.

iv. Material Overstrength Factor *(Ω_M)*

Generally, design codes and standards allow for the determination of the capacity of a shear wall with respect to the nominal strength (F_n), however, the maximum load that a structure can carry may be much higher. Uang *(1994)* stated that the actual strength of the structure greatly contributes to its ability to survive severe earthquakes. Uang also recommended that a balance between strength and ductility requirements should be made to take advantage of the reserve strength when considering the assignment of an R-factor. The material over-strength factor, Ω_M, is defined as the ratio of the maximum strength the system can attain, F_{max}, to the nominal strength (F_n) that is used in design, which can be determined from the last stable hysteresis hoop as stated in Section 4.2:

$$\Omega_M = \frac{F_{\max}}{F_n} \tag{4-33}$$

v. **Failure Modes**

Acceptable seismic performance requires a ductile failure mode without a rapid or complete loss of load carrying capacity. Driver *et al. (2000)* state that a conservative value of R is suitable for a structure that fails in a nonductile fashion, whereas for a structure that exhibits a gradual degradation of load before final failure, a more elevated R-value will still result in adequate performance during an earthquake. With respect to masonry walls, where R = 1.5 in the NBCC, flexural failure of the wall is expected with yielding of the tension reinforcement. In contrast, the unreinforced masonry walls that must be designed with R = 1.0, fail in a brittle shear mode. As recorded during testing, the steel stud shear walls failed when one of the following took place: screws pulled through the wood sheathing, wood panels pulled over screws, studs buckled, screws pulled out of the studs and/or tracks, screws sheared, tracks pulled out of the plane, *etc (Serrette et al., 1996b, 1997b; COLA-UCI, 2001)*. For wood walls, the failure modes that were most frequently observed were: nails failed in fatigue, nails pulled out of wood studs and/or through the panels, nails tore through the sheathing edge, and combinations of these modes *(Salenikovich et al., 2000; COLA-UCI, 2001)*. In general, the first instance at which load-carrying capacity decreases in steel stud walls is typically at a lower lateral displacement from that measured for wood stud walls.

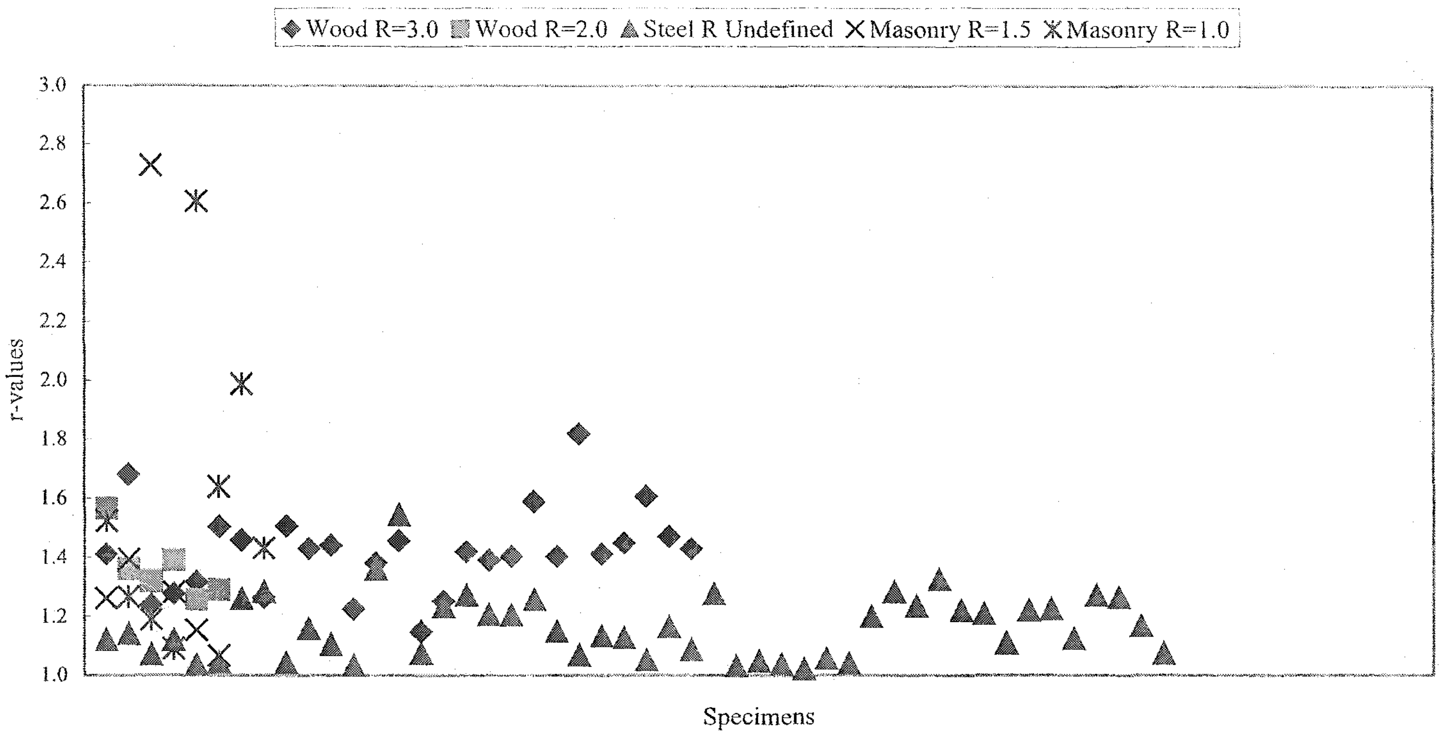


Fig. 4.9: Comparison of δ-values for Different Walls (*NBCC* defined *R*-Values)

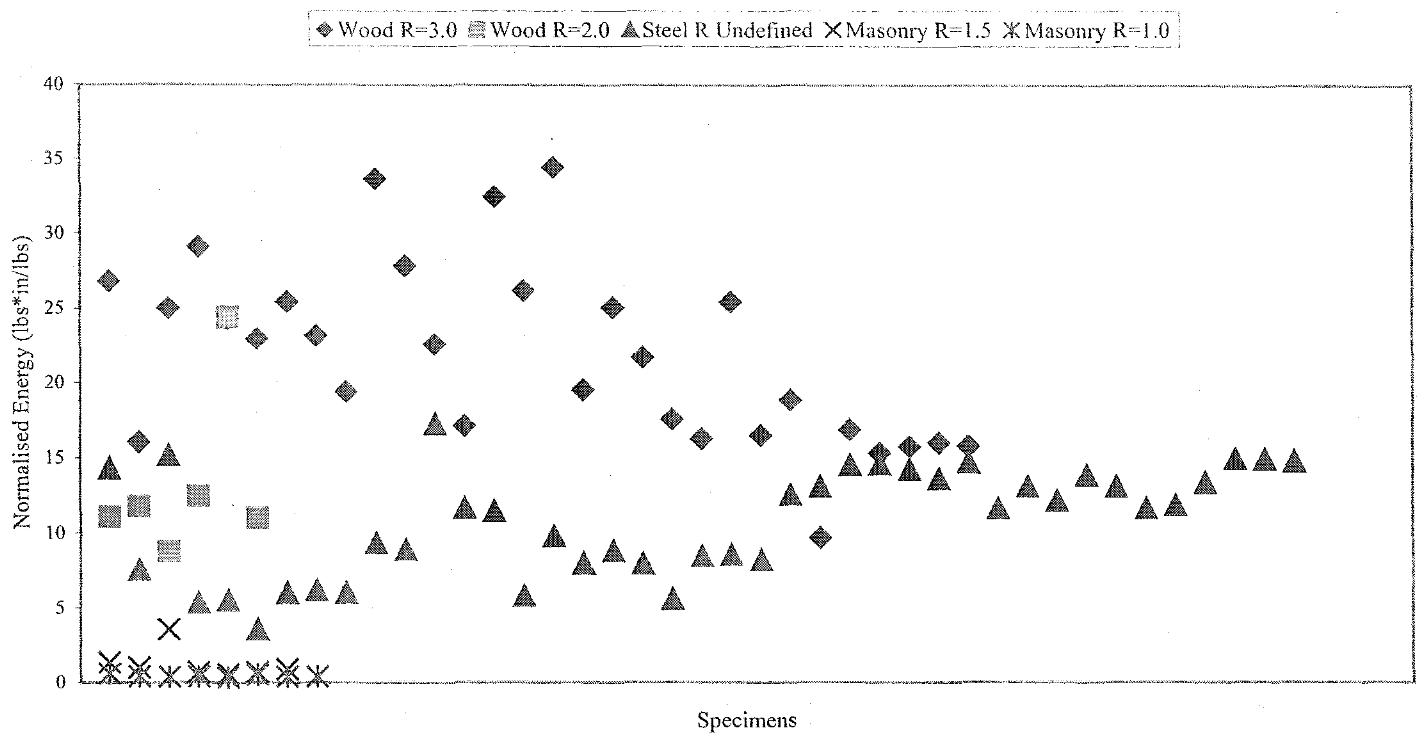


Fig. 4.10: Comparison of Normalised Dissipated Energy for Different Walls (*NBCC* defined *R*-Values)

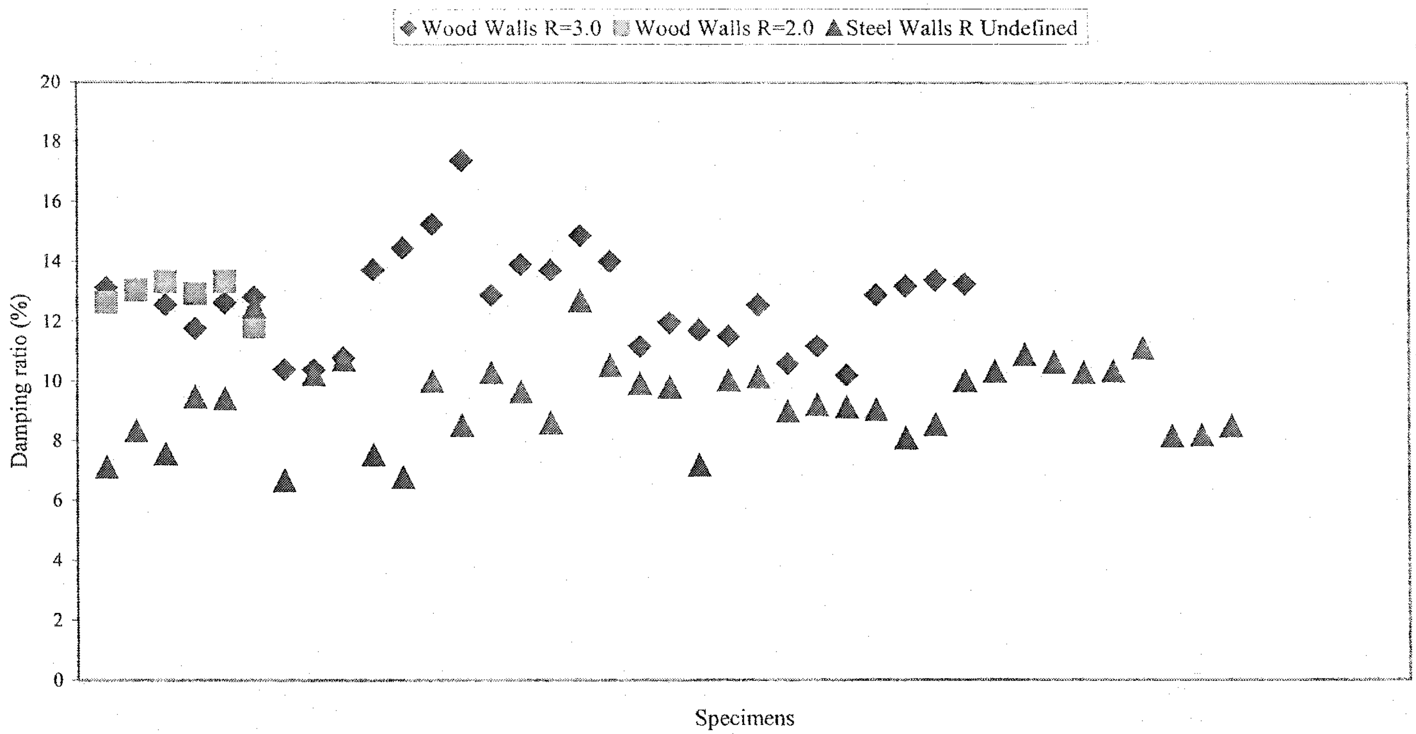


Fig. 4.11: Comparison of Damping Ratio for Different Walls (*NBCC* defined *R*-Values)

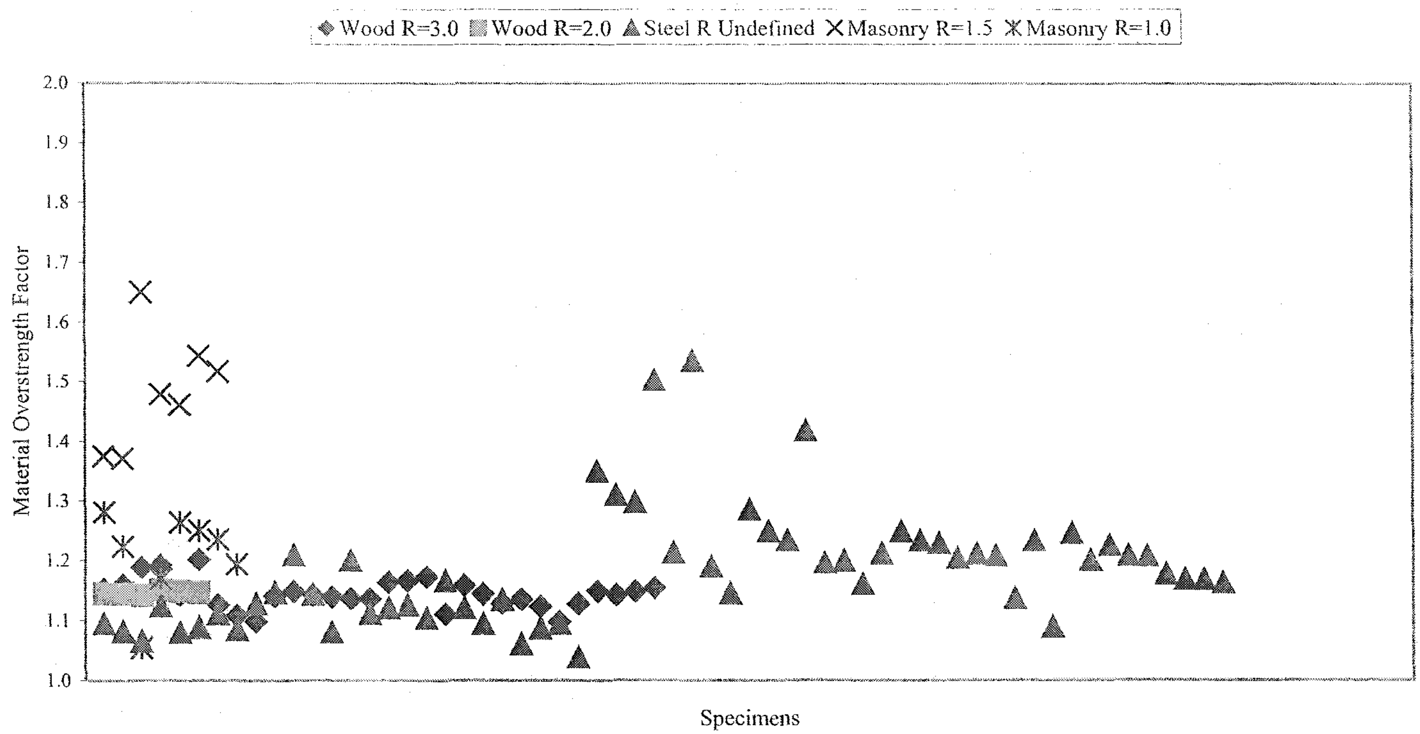


Fig. 4.12: Comparison of Material Overstrength for Different Walls (*NBCC* defined *R*-Values)

- **Results of Parameter Study**

The parameters discussed above were determined based on the results of shear wall tests completed by Serrette *et al. (1996b, 1997b)*, COLA-UCI *(2001)*, Salenikovich *et al. (2000)*, and Shing *et al. (1989, 1990a,b, 1991)*. As shown in Fig. 4.9, wood walls with $R = 3.0$ have similar δ-values (from 1.3 to 1.6 except specimens Group 23C and 04Fac-1) to wood walls with $R = 2.0$ (*i.e.* gypsum sheathed walls). This may due to the construction configuration used, where walls with an NBCC defined $R = 2.0$ were constructed with a combination of wood and gypsum sheathing, which increased the lateral stiffness and also, decreased the displacement at peak load. Thus a higher δ-value resulted even though the load that these walls can carry dropped quickly after the peak load. Masonry walls are significantly stiffer than wood and steel walls; thus, the measured δ-values are high, ranging from 1.1 to 2.7, even though the shear resistance diminished rapidly after the peak load was reached. Steel stud walls generally have lower δ-values, ranging from 1.0 to 1.3 except for AISI-D1 and D2 (sheathed with sheet steel), than wood walls, which is an indication that the steel walls do not have the same capacity to resist shear loads in the post peak range.

Wood walls with higher measured R-values tend to have elevated normalised energy values (E_{nor}), as illustrated in Fig. 4.10. For example, the normalised energy values of the wood shear walls with $R = 3.0$ are in the range of 20 lbs in./lbs (except 04Fac-1), and those of the wood shear walls with $R = 2.0$ are noticeably lower ($\approx$ 11 lbs in./lbs) (except Group 34B). The E_{nor} values for the masonry shear walls were determined for the 50% degradation post peak load position, rather than at the peak load, which would provide a slight advantage in terms of energy dissipation. However, in comparison with the steel stud and wood shear walls, the normalised energy values are dramatically lower, especially for the unreinforced walls with $R = 1.0$ ($<$ 1.0 lbs in./lbs). The steel wall normalised energy values are in the range of 8 lbs in./lbs for the specimens tested by Serrette *et al. (1996b, 1997b)*, and approximately 14 lbs in./lbs for those tested by COLA-UCI *(2001)*. The discrepancy between these two test programs may have resulted from the use of different aspect ratios (height *vs.* length) for the test specimens in the two test programs or slight differences in the displacement protocol, test set-up, humidity levels and the corresponding wood moisture content, or specimen components. The steel stud shear wall tests with lower aspect ratios tended to display a better ability to dissipate energy. In general, the wood stud walls were able to dissipate the greatest amount of energy, followed by the steel stud walls, with the masonry walls showing only minimal energy absorption ability.

As shown in Fig 4.11, wood walls with and without gypsum wallboard have a critical damping ratio around 13% at peak load, whereas the steel stud shear walls provide a critical damping ratio near 9%. It can also be seen that increasing the number of panel layers does not significantly affect the damping ratio, as shown by comparing specimens Group 35, 36 (two panels on one/both sides) with wood walls constructed with gypsum wallboard and other wood walls with only one panel on one side. Decreasing the aspect ratio, which may improve the dissipated energy as discussed previously, does not enhance the damping ratio in the comparison of steel wall specimens *(Serrette et al., 1996b, 1997b and COLA-UCI, 2001.)* with different aspect ratio, as well as wood wall specimens *(COLA-UCI, 2001* and *Salenikovich et al., 2000).*

In terms of material over-strength, the masonry shear walls, with values from 1.2 to 1.7 except Specimen 5, have higher factors (Ω_m) than both steel and wood walls, which range from 1.1 to 1.2 and from 1.0 to 1.5, respectively (Fig. 4.12). This characteristic may aid in their ability to survive severe earthquakes. The material-overstrength-values for wood walls are stable (1.1 ~ 1.2), while those for steel walls are in the same overall range, although the results fluctuate to a larger degree.

4.6 Other Methods to Determine an R-Value

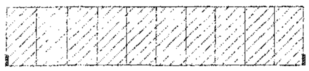
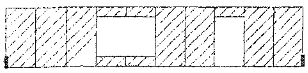
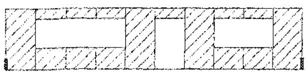
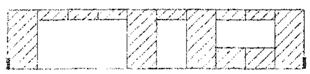
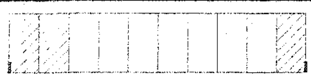
It must also be noted that the *R*-values presented in this thesis were evaluated from the results of quasi-static cyclic shear wall tests, which may not necessarily correspond to the behaviour of a real building including inertia effects. The contributions of various boundary conditions, connections and non-structural components, gravity loads, *etc*, may impact on the distribution of seismic forces to the structural shear walls. Ceccotti and

Karacabeyli *(2000)* presented a methodology to assess *R*-values, for which full-size shear wall specimens are tested under both static and cyclic load to determine the near-collapse criterion. A hysteretic model is then fit to the cyclic test data; the walls are designed for use in a selected building according to the code peak ground acceleration following various design scenarios; then a time-history dynamic analysis is carried out to obtain the ultimate peak ground acceleration for the different design scenarios. With this type of study the performance of a shear wall when subjected to seismic inertia loading can be predicted and more appropriate *R*-values can be selected. Shake-table tests of the shear walls would also need to be performed to verify the analytical conclusions. Ceccotti and Karacabeyli confirmed that wood-stud shear walls sheathed with gypsum wallboard should be designed using an *R*-value of 2.0 with this procedure. Future studies of the seismic performance of steel stud shear walls along the lines of the procedure followed by Ceccotti and Karacabeyli are necessary.

4.7 ADDITIONAL INFORMATION FOR DETERMINING R-VALUES

Experimental studies at Virginia Polytechnic Institute and State University have been carried out to evaluate the strength and cyclic behaviour of long shear walls with openings, both wood-frame *(Dolan and Johnson, 1997)* and steel-frame *(Salenikovich et al., 1999)*. The wall configuration and opening size, as well as the wall materials and construction data are shown in Table 4.7 and 4.8, respectively. As listed, the walls tested in these two studies are of similar configuration, with identical opening size, although different types of wood panel (15/32" plywood and 7/16" OSB) and different stud spacing (16" and 24") were used. Considering that Serrette *(1996b)* observed that steel stud walls sheathed with 15/32" (11.9 mm) plywood generally exhibited similar behaviour to those sheathed with 7/16" (11.1 mm) OSB, and that Tarpy and Girard *(1982)* concluded that the shear wall capacity was only slightly enhanced when the stud spacing is decreased, the results of the wood and steel stud long shear wall tests were considered to be directly comparable. Therefore, a comparison of the findings of these two studies was completed to better understand the relative behaviour of wood and steel stud shear walls.

Table 4.7: Wall Configurations and Opening Sizes for Wood and Steel Walls *(Dolan and Johnson, 1997)* and *(Salenikovich et al., 1999)*

Wall Configuration	Wall Type	Sheathing Area Ratio, (r)	Opening Size	
			Door	Window
	A	1.0	–	–
	B	0.76	6'-8" × 4'-0"	5'-8" × 7'-10$^{1/2}$"
	C	0.55	6'-8" × 4'-0"	4'-0" × 11'-10$^{1/2}$" 4'-0" × 7'-10$^{1/2}$"
	D	0.48	6'-8" × 4'-0" 6'-8" × 12'-0"	4'-0" × 7'-10$^{1/2}$"
	E	0.30	(Sheathed at ends) 8'-0" × 28'-0"	–

Note: r is as defined in equation (2 – 1) in Chapter 2. Shaded areas represent sheathing.

Table 4.8: Wall Materials and Construction Data for Wood and Steel Walls *(Dolan and Johnson, 1997)* and *(Salenikovich et al., 1999)*

	Wall Aspect	Studs	Sheathing			Sheathing Fasteners	
			Exterior	Interior	Orientation	Size	Spacing
Wood Walls	40' × 8'	No. 2, SPF 2 × 4 Spaced 16"	15/32" Plywood	1/2" Gypsum Wallboard	Vertically	8d[1] common nail (Plywood) 13 ga. × 1-1/2" (Gypsum)	6"/12" (Plywood) 7"/10" (Gypsum)
Steel Walls	40' × 8'	350S150-33 2 × 4 C-section cold-formed steel stud, 0.033" Spaced 24"	7/16" OSB	1/2" Gypsum Wallboard	Vertically	#8 self-drilling, bugle-head screws	6"/12" (OSB) 7"/10" (Gypsum)

Note: 1' = 304.8 mm, 1" = 25.4 mm [1] 0.131" diameter and 2.5" length

The definition of ductility ratio under cyclic load is consistent for both of these experimental studies *(Dolan and Johnson, 1997, Salenikovich et al., 1999)*. As shown in Fig 4.13, after the load-displacement curve (backbone curve) was obtained, an equivalent elastic-plastic curve (EEPC) was drawn to determine the yield (Δ_{yield}) and failure displacement ($\Delta_{failure}$). $\Delta_{failure}$ is defined as the post-peak deflection corresponding to the first significant drop in resistance for the load-displacement curve or 80% of the peak load, whichever is greater. The elastic portion of this EEPC passed through the origin and the point on the load-displacement curve where the applied load is $0.4F_{max}$. The plastic portion of the EEPC was based on an equivalent area approach (A1 = A2) as shown in Fig. 4.13. Δ_{yield} is identified as the intersection of the two portions of the EEPC. The ductility ratio, D, is then defined as:

$$D = \frac{\Delta_{failure}}{\Delta_{yield}} \tag{4-34}$$

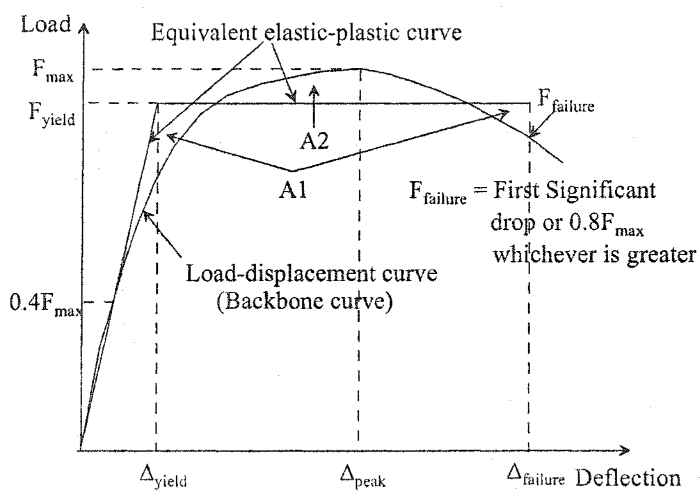


Fig. 4.13: Performance Parameter of Shear Walls *(Dolan and Johnson, 1997, Salenikovich et al., 1999)*

The ductility ratios of the initial and stabilised load curves were calculated and listed by Dolan and Johnson *(1997)* and Salenikovich *et al. (1999)*. The initial load curves were defined as the load-displacement curve for the initial input cycle, and the stabilised curves represented the load-displacement behaviour when the shear resistance of the wall decreased by less than 5% in two successive same amplitude cycles. A comparison of these two ratios for wood walls and steel walls is provided in Figs. 4.14 and 4.15. It can be seen that the steel walls generally exhibit a similar initial and stabilised ductility ratio (at failure loads) except for Walls E, where steel-framed one provide higher ratio. Figures 4.16 and 4.17 illustrate the comparison of the initial and stabilised ratio of the displacements at failure load to those at peak load for wood walls and steel walls, which is the δ-value as described in Section 4.5. As shown, the steel walls provided close initial and stabilised δ-values in comparison with the wood walls, which indicates that both wall types exhibit similar degradation after peak load. This corroborates the finding that the seismic behaviour of cold-formed steel-stud shear walls is comparable to that of wood-stud shear walls, and hence the assignment of an R-value greater than 1.0 is appropriate.

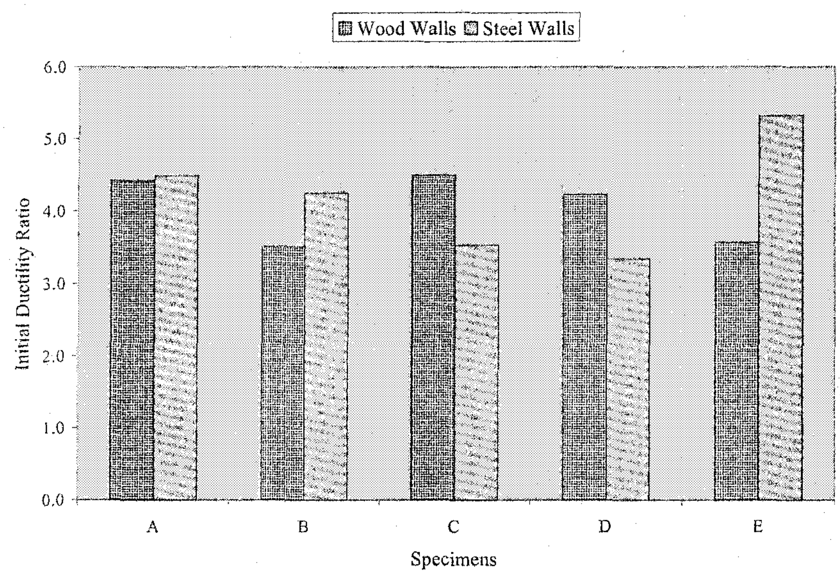


Fig. 4.14: Comparison of Initial Ductility Ratio – Perforated Shear Walls: Wood Tests *(Dolan and Johnson, 1997) vs.* Steel Tests *(Salenikovich et al., 1999)*

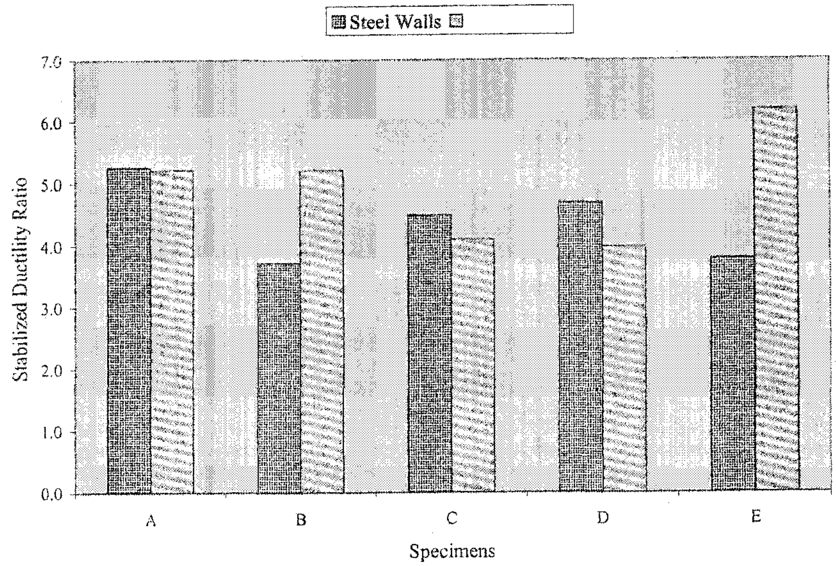


Fig. 4.15: Comparison of Stabilised Ductility Ratio – Perforated Shear Walls: Wood Tests *(Dolan and Johnson, 1997) vs.* Steel Tests *(Salenikovich et al., 1999)*

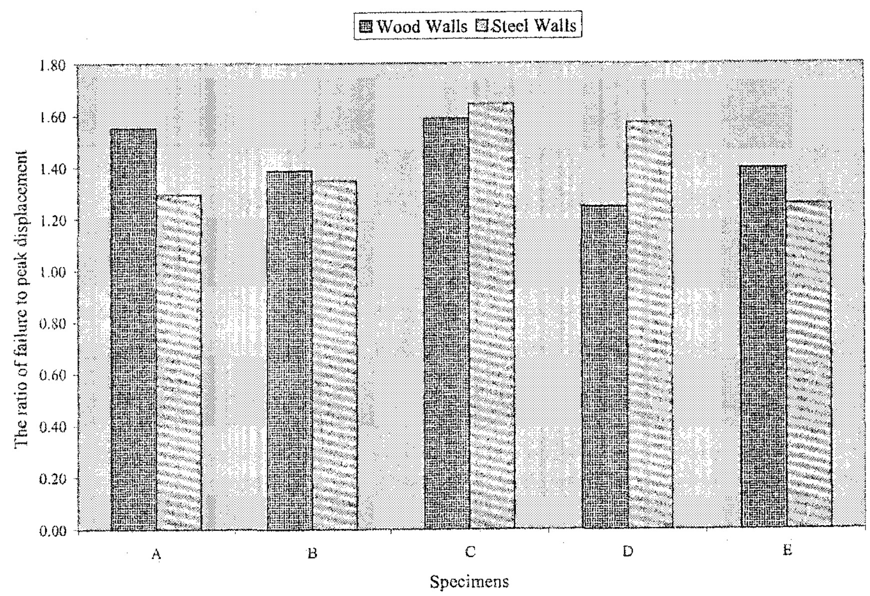


Fig. 4.16: Comparison of Initial Displacement Ratio at Failure to Peak: Wood Tests *(Dolan and Johnson, 1997) vs.* Steel Tests *(Salenikovich et al., 1999)*

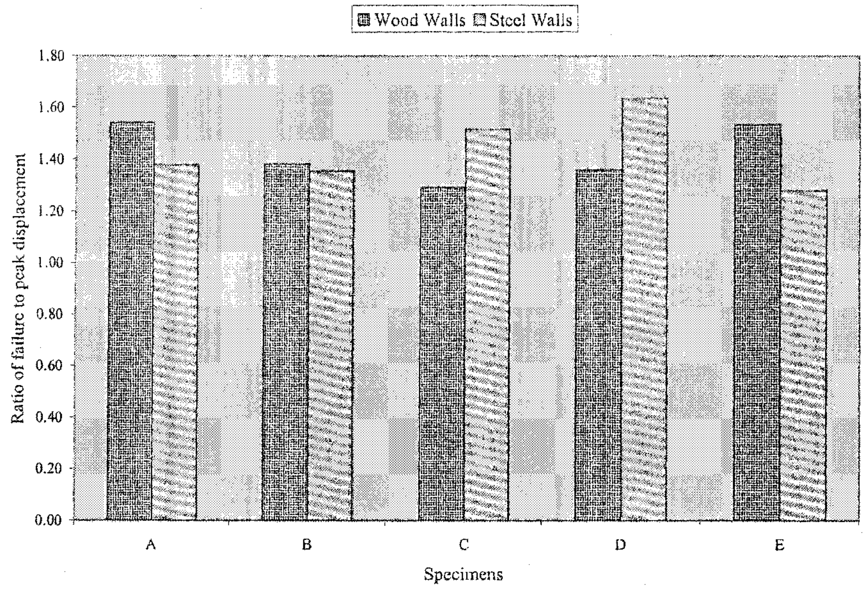


Fig. 4.17: Comparison of Stabilised Displacement Ratio at Failure to Peak: Wood Tests *(Dolan and Johnson, 1997) vs.* Steel Tests *(Salenikovich et al., 1999)*

4.8 POSSIBLE R VALUES FOR USE IN DESIGN

A procedure to determine R-values for use in the design of lateral load-resisting systems based on the results of quasi-static cyclic shear wall tests was presented. According to the comparison of calculated R-values, and other parameters of steel, wood, and masonry shear walls, and considering the variation of test results and the limitation of test data, at this time a preliminary, and possibly conservative, R-value of 2.0 is suggested for use in the design of cold-formed steel stud single-storey shear walls sheathed with wood panels following the NBCC equivalent static load procedure. Further studies are necessary to evaluate the effects of aspect ratio, gravity loads and construction configuration, as well as the influence of dynamic forces. A more advanced study that includes time-history analyses of different design scenarios and a comparison with additional test data must be carried out to confirm this suggested force modification factor. The expected displacement of the structure must also be adjusted accordingly if an R-value greater than 1.0 is used in design.

CHAPTER 5 PROPOSED SHEAR WALL TESTS AND TEST FRAME

In Canada, no specific design method for steel stud shear walls is contained in the National Building Code *(NRCC, 1995)* or in the S136 Design Standard *(CSA, 1994)*. In general, prescriptive shear capacity tables provided in the AISI *(1998)* and UBC *(ICBO, 1997)* design documents are based on the results of tests carried out by Tissell *(1993)*, Serrette *(1994)*, and Serrette *et al. (1996b, 1997b)*. An equivalent Canadian design guide does not exist, and hence, due to the different wood products available in the USA, it is necessary to complete tests with local products before adopting the shear capacity values specified by the AISI or UBC. Recent shear wall tests, performed at the University of California at Irvine *(COLA-UCI, 2001)* provide consistent shear capacity and energy-dissipation results, which are somewhat different from those reported by Serrette *et al.*, as discussed in Chapter 4. It is possible that this slight inconsistency is due to different test setups used by the respective researchers, or perhaps due to the influence of aspect ratio on shear capacity. Another possibility is a change in materials: the wood panels used by Serrette *et al.* were APA Rated Sheathing Exposure 1 grade while those used in UCI tests were APA Rated Sheathing Structural I grade. Structural I grade panels provide enhanced shear strength (*APA, 1999*) and may give better ductility and consistency and hence better test results. Supplementary tests should be completed to ascertain the nature of this discrepancy. Furthermore, additional tests should be completed to verify the analytical results described in Chapters 3 and 4.

It is anticipated that the findings contained in this thesis will be used as a starting point for further research into the design and behaviour of steel stud shear walls. Included in the scope of these future studies will be shear wall testing, hence the construction of a test frame is necessary and the proposal of a series of shear wall tests that can be used to establish a link between the existing data *(Serrette et al., 1996b, 1997b; COLA-UCI, 2001)* and any future tests is required. This Chapter contains a discussion of the possible tests that will allow for a link to be drawn between the existing and any future studies, as well as recommendations concerning tests that should be carried out to complete the full range of construction configurations that are currently in use. Additional details on the design and use of the shear wall test frame are also provided.

5.1 Proposed Tests

A preliminary listing of tests that will extend and complement the existing steel stud shear wall data is provided in Tables 5.1 and 5.2. Table 5.1 includes information on shear walls sheathed with plywood or OSB that match specimens which have already been tested by Serrette *et al.* and UCI, and which will be used to establish a link with the existing data base. Steel studs and tracks for 4'×8' and 8'×8' walls will be fabricated in the United States from material which is the current equivalent of ASTM A446 Grade A steel *(1993)* (F_y = 33 ksi: 230 MPa) and of ASTM A 653 SQ 33 steel *(1994)*, respectively. APA rated plywood and OSB sheathing will also be sourced from the United States. Other construction and testing requirements, as well as the loading protocols, will be as specified by Serrette *et al.* and UCI.

Table 5.1: Preliminary Proposed Repeated Laboratory Shear Wall Tests

Type of wall (Test walls by Serrette *et al.* and UCI)		OSB (OSB – 1D3,4[1] &AISI – OSB 3,4[1])	Plywood (PLY – 1A6,7[1] & Group 14[2])
Type of loading		Monotonic + Cyclic test	Monotonic + Cyclic test
Type of sheathing or brace	*Thickness*	7/16"	15/32"
	Orientation	Vertical	Vertical
Screw	*Type*	No. 8 × 1" flat head (sheathing)	No. 8 × 1" bugle head (sheathing)
	Spacing	4"/12"	6"/12"
Stud size & thickness		C-shaped, 3-1/2" × 1-5/8"× 3/8" 0.0346" (double studs back to back at the end)	C-shaped, 3-1/2" × 1-5/8"× 3/8" 0.0346" (double studs back to back at the end)
Stud spacing		24"	24"
Size of test (aspect ratio)		4'×8'	8'×8'

Note: 1" = 25.4 mm, 1' = 304.8 mm [1] denotes tests by Serrette *et al.* [2] denotes tests by UCI

In Table 5.2, a listing of walls with 8'-height and 8'-width or 12'-width is provided with which a better understanding of the strength, ductility and energy-absorption abilities of shear walls constructed using Canadian Standards Association wood sheathing products, as well as steel studs and tracks that are fabricated in Canada, can be established. The tests also include some walls with wood panels oriented horizontally, with or without blocking, in order to investigate the effect of sheathing orientation and blocking on lateral load carrying performance. Steel studs, tracks, steel sheets and steel straps all meet ASTM A 653 SQ 33 *(2001)* material requirements with a minimum specified yield strength (F_y) of 33 ksi (230 MPa). As typically found in construction, studs are to be spaced at 24" (610 mm) centres and it is anticipated that No. 8 and 10 screws are to be used. In addition, double chord studs will be installed at both ends of the test walls to enhance the buckling and torsional resistance of these shear wall edge members. It is suggested that for the X-braced walls a sufficient number of screws must be utilised in order to develop 1.5 times the yield strength of the strap in the gusset plate to strap connection. Cyclic tests are to be carried out to extend the existing range of data, while monotonic tests are required to develop a more precise definition of the first major event and to help define the test protocol for the corresponding cyclic tests. Cyclic tests are to be quasi-static in nature with a frequency of 1.0 Hz. An example test protocol is illustrated in Fig 5.1, which is as utilised in previous tests by Serrette *et al. (1997b)*.

Table 5.2: Preliminary Proposed Extended Laboratory Shear Wall Tests

Type of wall		Steel X-bracing	GWB (both)	Steel sheet sheathing	Plywood, OSB
Type of loading		Monotonic + Cyclic test	Cyclic test	Monotonic + Cyclic test	Cyclic test
Type of sheathing or brace	Thickness	4-1/2" x 0.033"	1/2"	0.027"	15/32", 7/16"
	Orientation	---	Vertical & Horizontal	---	Vertical & Horizontal
Screw	*Spacing*	20 No.8 screws	7"/7"; 4"/4"	6"/12", 4"/12"	6"/12", 4"/12", 2"/12"
Blocking		---	1-1/2" × 0.0346" strap & No blocking	---	1-1/2" × 0.0346" strap & No blocking
Stud size & thickness		C-shaped, 3-1/2" × 1-5/8"× 0.043"	C-shaped, 3-1/2" × 1-5/8"× 0.033"	C-shaped, 3-1/2" × 1-5/8"× 0.033" & C-shaped, 3-1/2" × 1-5/8"× 0.043"	C-shaped, 3-1/2" × 1-5/8"× 0.0346"
Stud spacing		24"	24"	24"	24"
Size of test (aspect ratio)		8'×8' 12'×8'	8'×8' 12'×8'	8'×8' 12'×8'	8'×8' 12'×8'

Note: 1" = 25.4 mm, 1' = 304.8 mm

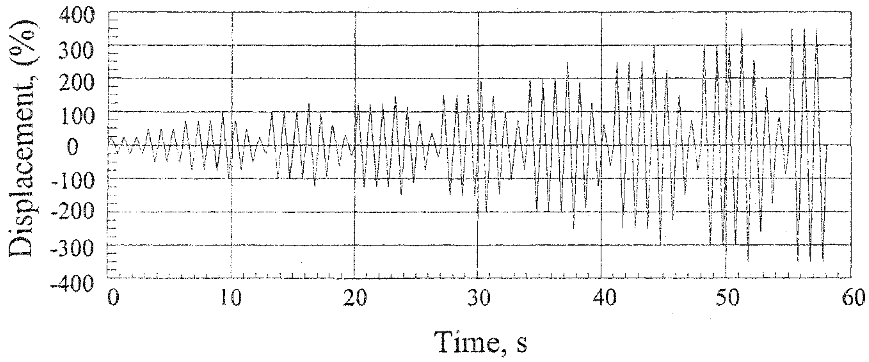


Fig. 5.1: Cyclic Test Protocol as per Serrette *(1997b)*.

It is recommended that shear wall specimens be chosen from the listing in Table 5.2 for the extended tests planned for Phase II of this research project. Furthermore, some of the previous duplicate tests exhibited results that varied by over 5% (AISI – OSB3, 4, 5, 6, 7, 8, AISI – PLY7,8, AISI – A3, 4, AISI – B1, 2, 3, 4, AISI – C3, 4, AISI – D1, 2, AISI – E5,6, and AISI – F1, 2, 3, 4); these tests should also be repeated to determine if more consistent results can be obtained. In addition, other shear wall configurations may be included.

5.2 Test Frame

The most common method of testing a shear wall is to mount an actuator on a strong wall, which resists the reaction force when applying a horizontal load to a test specimen. Considering that these facilities do not exist in the structural laboratory at McGill University, it was decided to construct a test frame in order to carry out the shear wall tests (Figs. 5.2 & 5.3). The frame, 11 m in length (centre to centre of end channels) with a clear height of 4 m, is built to accommodate walls with a length and a height of up to 12' (3.66 m). The maximum input displacement for the available actuator is ±5" (±127 mm). Two lower beams have been specified such that the test wall anchors can be installed, and to provide support for lateral braces. In the design of the frame it was assumed that the strong floor of the lab could only be relied on to resist uplift force through the anchor rods. For this reason the test frame was designed to transfer the lateral applied shear force internally through the frame without considering the contribution of the floor. In order to satisfy these requirements and to facilitate transportation and erection of the frame, as well as the eventual testing needs, the structure was divided into seven components as shown in Fig. 5.2. These components are listed as follows:

a) Part A – Triangular section that provides support for the actuator (North end) and carries the reaction force, decomposing it as an uplift force and a shear force, to the base and lower beams of the assembly.

b) Part B – Lower beams that serve two roles: resisting the shear force of the test wall and transferring the shear force between two triangular sections (Part A); meanwhile, conveying the uplift force from the test walls to the channel anchors (Parts F and G).

c) Part C – Upper beam acts as one end support for two triangular sections (Part A), where it helps to distribute the internal shear forces, and also to restrain the horizontal displacement of the two Part A sections through axial tension and compression. It also acts as a lateral support, along with the attached HSS frames, for the test walls.

d) Part D – Second column (pinned base) is necessary to support the weight of the actuator and ensure that only horizontal loading will take place even when the height of the test walls decreases with lateral deformations. It can also be used to increase the actuator displacement by lowering the position of the actuator while maintaining the attachment point to the test walls.

e) Parts E and H – Load beam and lateral wall brace, allow for only in-plane shear displacement of the test walls by preventing the top of the wall from twisting outside of the load application plane.

f) Parts F and G – Channels and their anchor rods transfer the uplift forces from the test frame to the strong floor.

g) Part L – Plate enables the test walls to be anchored to the lower beams in the same fashion as found in a building. The plate transfers both the shear force and the uplift force from the test walls to the lower beams.

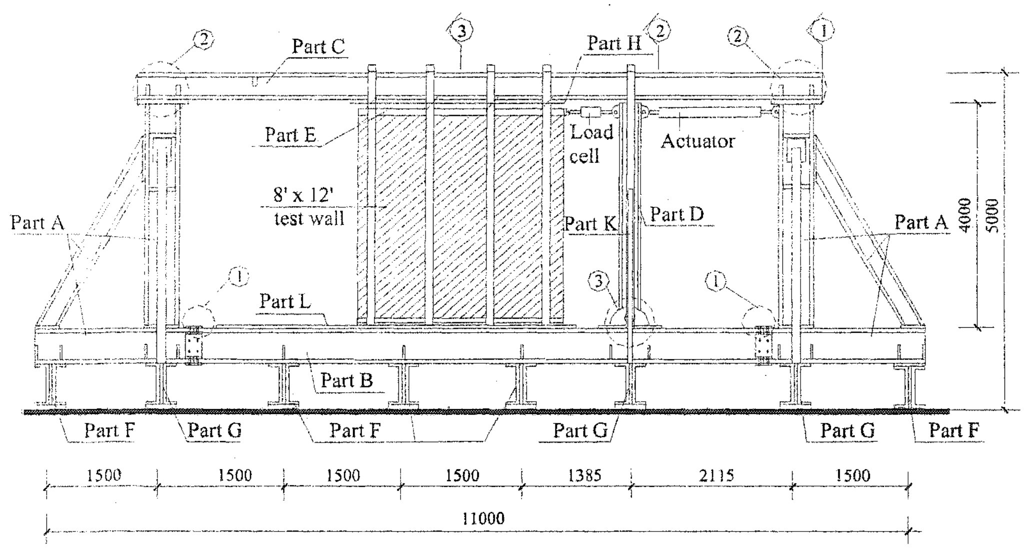


Fig. 5.2: Elevation of the Test Frame

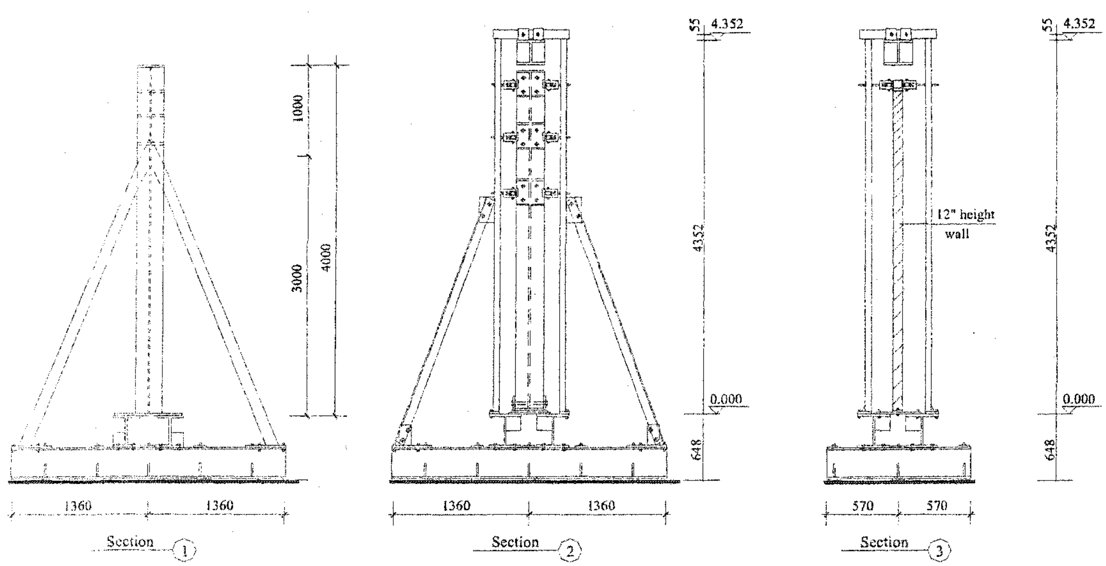


Fig. 5.3: Sections of the Test Frame

5.3 *DESIGN OF THE TEST FRAME*

5.3.1 Overall Design

In this section, details of the test frame including the design criteria, member dimensions, anchors and bracing systems are described.

- **Design Data**

The test frame was designed using the VisualDesign Program *(VisualDesign, 2001)* incorporating different actuator capacities, the expected test wall heights (8' – 12': 2.44 m – 3.66 m) and considering the cyclic load reversal needed for testing. Four design scenarios were applied to obtain the most adverse loading situation. Fig. 5.4 represents one scenario of an 8' (2.44 m) height test wall with the actuator in compression, while Fig. 5.5 denotes a 12' (3.66 m) height wall with the actuator in tension. The remaining two design scenarios were as shown in Figs. 5.4 and 5.5 except that reverse loads were used.

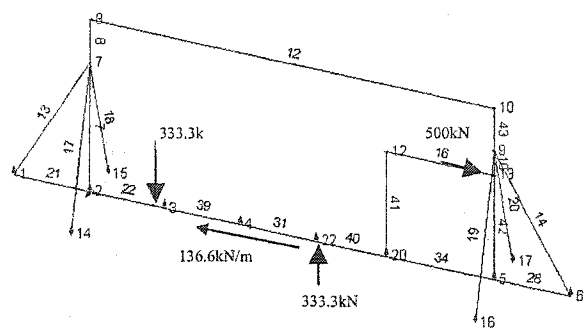


Fig. 5.4: Computer Model and Input Loading (+) for 8'-Walls

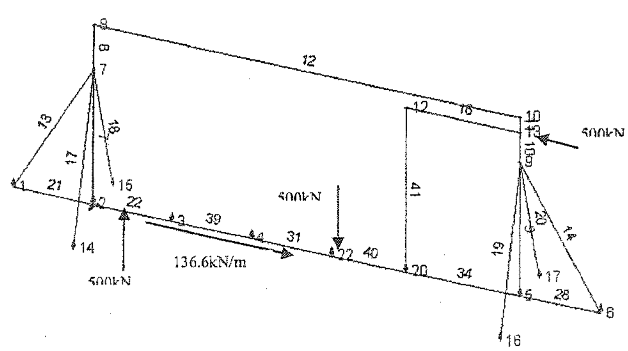


Fig. 5.5: Computer Model and Input Loading (-) for 12'-Walls

Even though the most common walls are 8' (2.44 m) in height, considering the possibility of increased storey heights up to 12' (3.66 m), the clear height of the test frame was chosen as 4 m to accommodate these taller test walls. In the analytical model the actuator was replaced with a stiff beam of the same self-weight. It was anticipated that the design of the frame would be controlled by maximum displacement allowances instead of member capacities. This was due to the use of a displacement limit of $L/800$ in order that only minimal frame distortion occurs under loading. Another criterion included in the design was to control the maximum stress in the members so that they remained in the elastic range under 0.4 F_y at all times. As a final design step, the frame was checked for ultimate strength following the CSA S16.1 Design Standard *(1994)*. The maximum input force was considered to be 500 kN, twice the capacity of the available actuator, which will enable the frame to provide enough stiffness if in the future a higher capacity actuator is installed. To ensure that the shear forces in the frame are not transferred to the supporting concrete slab, all of the model supports, except for one end support, were set as rollers to avoid the formation of a failure mechanism during analysis runs. A summary of analysis results including internal forces and deflections for the different design scenarios is provided in Table 5.3. From the analysis output, it can be seen that the design scenarios with the input load applied at a height of 8' (2.44 m) result in larger displacement and uplift forces, and higher stress levels. This may be attributed to the brace positions for the vertical column of Part A, which are located at the 3 m and 4 m positions, as well as the column base. Thus the bending span that corresponds to the 8' height input load is 3 m, and for the 12' height input load is 1 m. In the later case, a greater proportion of the reaction forces is transferred to the lower beams through the inclined member in Part A.

As shown in Table 5.3, the maximum displacement is $L/748$, which slightly exceeds the $L/800$ limit. However, considering that the input load used in the design of the frame is twice the capacity of the available actuator (250 kN), and is significantly greater than the anticipated resistance of the test walls (based on tests by Serrette and UCI, the maximum shear capacity is 2423 lb/ft which for a 12' wall ≈ 130 kN), the resulting displacements were considered to be acceptable.

Table 5.3: Internal Forces and Deflections of the Test Frame

	8'-wall (+)	8'-wall (-)	12'-wall (+)	12'-wall (-)
Maximum displacement	$\frac{L}{748}$	$\frac{L}{763}$	$\frac{L}{1040}$	$\frac{L}{1049}$
Maximum uplift (kN)	614	631	577	623
Maximum stress	$0.396F_y$	$0.397F_y$	$0.305F_y$	$0.285F_y$

Note: 8'-wall and 12'-wall denote the input force at the height of 8' (2.44 m) and 12' (3.66 m), respectively. (+) denotes compression in the actuator and (-) denotes tension in the actuator.

- **Anchors**

The support reactions are shown in Table 5.4, where the support numbers are as illustrated in Figs. 5.4 and 5.5. The negative values signify uplift forces and the positive values denote compression forces.

Table 5.4: Reactions at Supports of the Test Frame

Number	8'-wall (+) (kN)	8'-wall (-) (kN)	12'-wall (+) (kN)	12'-wall (-) (kN)
1	-125	177	-457	511
2	283	-234	644	-598
20	-34	70	-48	82
22	-313	324	-464	471
3	263	-247	395	-379
4	-7	26	-18	37
5	-614	661	-577	649
6	688	-631	676	-623

The values listed in Table 5.4 are design anchor forces for the frame when it is attached to the strong floor. These reaction forces occur because the strong floor is relatively stiff in comparison to the steel frame. However currently, a significant portion of the test frame will be located on the weak floor (≈200 mm thick lightly reinforced concrete slab), and hence these reaction forces will not develop because the concrete slab is flexible in comparison to the test frame. Considering that the typical capacity of a 12' (3.66 m) long shear wall is less than 150 kN, as indicated in the results of previous tests *(Serrette et al., 1996b, 1997b, COLA – UCI, 2001)*, the design anchor forces for the temporary weak floor installation can be based on an input load of 150 kN instead of 500 kN, which is still conservative. Therefore, the uplift force for designing the anchors for use in the weak floor can be scaled down by 500/150 from those values listed in Table 5.4. The maximum factored bond capacity that an Hilti HIT HY 150 Injection Adhesive anchor with a 1" (25.4 mm) diameter threaded rod (6" or 150 mm embedment depth) can provide is 29,615 lb (131.7 kN) *(Hilti, 2001)*. Generally, three anchors are arranged to resist the uplift force from each reaction except in the lateral brace positions where five anchors are specified. Considering that the anchor rods will deflect axially under loading, springs were used to simulate the actual anchor support, where the stiffness was obtained as follows:

$$K_1 = \frac{EA}{L} \qquad (5-1)$$

where

E – Elastic modulus of anchor rods = 200000 MPa

A – Area of anchor rods = 507 mm^2 for 1" anchor rods

L – Calculated length of anchor rods, which is equal to the height of channels plus thickness of two bearing plates. Therefore, L = 337mm.

$$K_1 = \frac{200x507}{337} = 301 \text{ kN/mm}$$

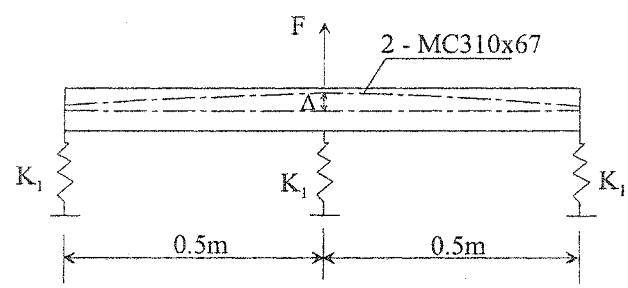


Fig 5.6: Deflection of Channels under Uplift Force

The channel and its anchor rods then work in combination as a spring, as shown in Fig. 5.6, with a stiffness K' (=F/Δ) of 694.4 kN/mm obtained from the VisualDesign program. The analysis model shown in Fig. 5.6 represents the three-anchor-rod channel section viewed in Fig. 5.3.

The results from the VisualDesign program also show that due to the bending flexibility of the channel, the contribution with respect to the uplift force (F) of the middle spring (in Fig 5.6) is 43.7%, while each of the end springs contribute only 28.2%. The computer models were then revised to account for the flexibility of the supports. A comparison of results for the different test frame models in which the support reactions have been adjusted accordingly are shown in Table 5.5. As revealed, the maximum displacements and uplift forces are significantly affected while the maximum stresses of members remain near constant. In these design scenarios the predicted maximum displacement increases by up to 31.9% for the 12' wall and the maximum uplift force decreases by 30.8% for the 8' wall. However, it is important to note that the values contained in Table 5.5 are based on a 500 kN load, which exceeds the expected shear wall capacity by over 300%, hence the listed deflections, loads and stresses will not be reached during testing.

Table 5.5: Internal Forces and Deflections of the Test Frame with Modified Supports

	8'-wall (+)	8'-wall (-)	12'-wall (+)	12'-wall (-)
Maximum displacement	$\frac{L}{594}$ (20.6% ↑)	$\frac{L}{570}$ (25.3% ↑)	$\frac{L}{761}$ (26.8% ↑)	$\frac{L}{715}$ (31.9% ↑)
Maximum uplift (kN)	425 (30.8% ↓)	525 (16.7% ↓)	421 (27.0% ↓)	545 (12.4% ↓)
Maximum stress	$0.386F_y$ (2.5% ↓)	$0.388F_y$ (2.3% ↓)	$0.303F_y$ (0.7% ↓)	$0.279F_y$ (2.1% ↓)

Note: 8'-wall and 12'-wall denote the input force at the height of 8' (2.44 m) and 12' (3.66 m), respectively. (+) denotes compression in the actuator and (-) denotes tension in the actuator.

Finally, the anchor forces for each rod were recalculated by incorporating the spring support conditions and the more realistic 150 kN load situation, and then checked to ensure that all forces will not exceed the factored anchor bond capacity (Table 5.6). For those supports that contain five rods, the anchor rod force is generally less than that obtained for the corresponding three-rod model.

Table 5.6: Maximum Force for Anchor Rods with Input Load of 150 kN

	1	2	20	22	3	4	5	6
Middle Rod (kN)	53	64	28	41	48	48	56	72
End Rods (kN)	34	41	18	27	31	31	36	46

- **Member sections**

The member sizes, detailed below, were chosen to meet the deflection and stress limits. All members are made of G40.21 – 350W grade steel *(CSA-G40.21, 1992)*.

a) W310×158 sections are specified for Parts A and C, whereas 2 parallel W310×158 members are utilised for Part B.

b) Part D: In the best case scenario this column is subjected to two lateral forces equal in value but opposite in direction, and hence would not develop an axial force. However, the section must provide enough width to connect the actuator with the bolts spaced at 7.25" (184 mm) and must have adequate web crippling resistance. Based on the attachment requirements the minimum width of the section is 184 mm + 2×50 (edge distance) = 284 mm. A W310×97 with a width of 305 mm was chosen for this member and web stiffeners were added at the actuator connection locations.

c) Part E: The load beam is comprised of two components, shown in Fig. 5.7, including an HSS and a cold-formed steel C-section (1.5-mm thick) to facilitate the changing of test specimens. An HSS 89×89×6.4 member is to be used to match the common width of track section found for a typical steel stud wall. 1/2" bolts are spaced at 8" along this HSS in order that the maximum 250 kN actuator load can be transferred to the test wall.

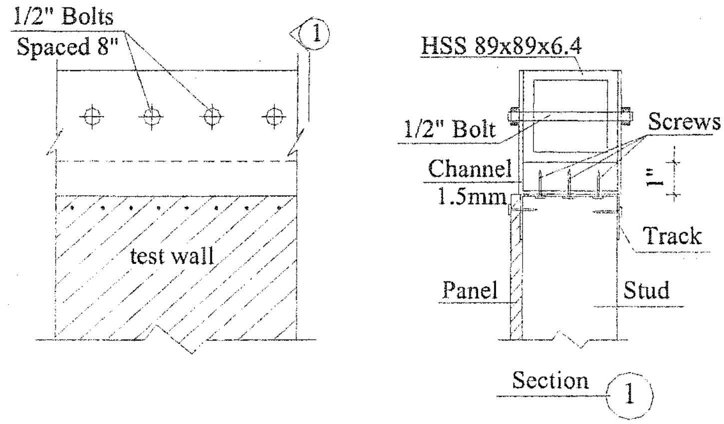


Fig. 5.7: Configuration of Part E – Load Beam

d) Parts G and F: Channels were specified to act as supports for the lower beams and also, to provide enough width to connect the lateral braces as shown in Figs. 5.2 and 5.3. Two MC 310×67 members are to be attached back-to-back (40 mm gap) with 16 mm plates to accommodate for positioning of the anchor rods.

e) Part L: Plate L has three rows of holes, where the outer holes are be used to connect the plate to the lower beams and the central hole to connect the plate to the test specimen. The connection requirements were based on slip critical resistance of the bolts under both shear and tension loading. The plate itself was sized to possess enough moment and shear capacity to carry the applied shear and uplift forces. At the uplift locations an HSS 203×203×9.5 section was welded to the bottom of the plate to enhance the moment capacity.

5.3.2 Connections

a) Connection 1 (Fig. 5.2): Part A – Triangular section with Part B – Lower beams

The connection capacity relies on the slip critical shear resistance of the bolts and the design load can be taken as the axial force in the lower beam, obtained from the VisualDesign program analysis results for the overall frame. Twelve 1" A325 bolts were utilised on each side of the connection.

b) Connection 2 (Fig. 5.2): Part A – Triangular section with Part C – Upper beam

Considering that the restraint of this connection significantly influences the overall drift, which is critical to the design of the frame, the connection was chosen based on the slip critical shear resistance of the bolts, rather than the tension resistance of the bolts. Since these bolts are subjected to a shear force and a moment from the end reaction of the triangular Part A, they are placed outside of column to increase the moment capacity of connection.

c) Connection 3 (Fig. 5.2): Pin connection of Part D – Second column

The hinge at the base of Part D allows for the transfer of the racking shear loads to the test wall without a resulting moment at the base of the column. This column is sandwiched by two plates and connected by a pin and bearing assembly with pairs of Teflon pads fastened between the column and plates to reduce friction. The connection restrains displacement normal to the plane of the wall, while allowing free rotation in the plane of the wall.

5.3.3 Braces

Lateral braces are necessary to ensure that all loads and displacements are in the plane of the test wall. Under ideal conditions, where all loads and resistances are in line with the frame no force will be generated in the braces. However, as behaviour becomes non-linear, or if the alignment is not perfect, then the braces will carry load. Hence, member sizes were selected to provide enough stiffness to minimise the possible out-of-plane deflection of the braced members.

a) Brace part H (Section 3 in Fig. 5.3)

The purpose of these members is to laterally brace the test specimen. Attachments have been provided at 0.6 m spacing so that even a 2' (0.61 m) long wall can be tested. In the initial set-up the loading beam (Part E) may be subjected to a maximum 250 kN (the capacity of the available actuator) compression force at one end. This force is assumed to be distributed along the test wall, and essentially, is reduced to zero at the far end. For simplicity Part E was regarded as a braced column subjected to an average concentric loading of 125 kN. The required stiffness for this loading beam is stipulated in CSA S16.1 *(1994)* as follows:

$$k_b = \frac{\beta C_f}{L}(1 + \frac{\Delta_0}{\Delta_b}) \qquad (5-2)$$

where

k_b = Required stiffness of the bracing assembly

Δ_0 = Initial misalignment of the braced member (loading beam Part E) at the point of support as shown in Fig 5.8

Δ_b = Displacement of braced member (loading beam Part E) at the point of support under force C_f. May be taken equal to Δ_0

β = 2, 3, 3.41, or 3.63 for 1, 2, 3, or 4 equally spaced braces, respectively

C_f = Force in a column, the compressed portion of a flexural member (125 kN)

L = Length of the braced member between brace points (0.6 m)

The permissible variation in straightness (Δ_0) is $L/1000$ for a W-shape beam with flange width 150mm, $L/500$ for those with a flange width < 150mm, and $L/500$ for HSS members.

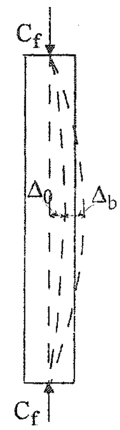


Fig. 5.8: Assumed Initial Deflection Δ_0 and Deflection Δ_b of the Loading Beam (Part E)

Due to the possible out-of-plane displacement ($\Delta_0 + \Delta_b$) of the braced member, the brace part H tends to bend and thus, it must possess enough flexural stiffness to restrict this displacement. Assuming fixed-end conditions (using appropriate connection details), an HSS 102×51×8.0 was chosen as a brace to achieve the required bending stiffness as follows:

$$I_x = \frac{k_b a^3 b^3}{3EL^3} \qquad (5-3)$$

where

k_b = Required stiffness of the bracing assembly from Equation (5 – 2)

a, b = Distances from braced point to both ends

E = Elastic modulus of the brace

L = Length of the brace

b) Brace Part A (Section 1 in Fig. 5.3)

Considering that the vertical section of Part A is mainly subjected to shear and bending, the brace requirement for beams was utilized. The required stiffness k_b is similar to that expressed in Equation (5 – 2) except the definition of C_f is modified, as shown in Equation (5 – 4):

$$C_f = Max(C_b P_E, \frac{M_F}{h}) \tag{5 – 4}$$

where

$$P_E = \frac{\pi^2 E I_{yc}}{L_b^2}$$

I_{yc} = Out-of-plane moment of inertia of the compression flanges

C_b = Factor for non-uniform moment

L_b = Braced length of the braced member

M_F = Applied moment

h = Depth of the braced member

In this case the brace acts both in compression and in tension, and hence an HSS 102×51×8.0 member was selected to meet the cross-section area requirement.

$$A_s = \frac{k_b L}{2E \sin\theta} \tag{5 – 5}$$

where

θ = Angle between the vertical column and the brace

Other definitions are as found in Equation (5 – 3)

c) Brace part K (Section 2 in Fig. 5.3):

Ideally, the second column is subjected to balanced loads at its top, which would not result in an out-of-plane deflection. However, due to misalignment and possible lateral movement of the shear wall, the required stiffness for the braces was determined using the same procedure as described for Part H, except that C_f was conservatively replaced by the expected compression resistance of the column. To significantly increase the stiffness of the braces, a combination of an HSS 102×51×8.0 in bending and 2-L89×89×9.5 members in axial compression was utilised.

Chapter 6 Conclusions and Recommendations

6.1 Conclusions

This thesis includes an extensive literature review that contains information on existing cold-formed steel stud shear wall test programs. The results of these studies were relied on to develop a numerical method with which the shear strength of steel stud walls with wood or steel sheathing can be estimated. In addition, the results of some of the existing test programs were used to evaluate various lateral force design methods and to determine a preliminary force modification factor for use in the seismic design of steel stud shear walls following the equivalent static approach prescribed by the National Building Code of Canada. Furthermore, recommendations for future testing of cold-formed steel stud shear walls were made and a review of the design of a corresponding test frame was provided.

The following conclusions are drawn from the theoretical investigation of shear capacity and ductility of cold-formed steel stud shear walls using existing test data:

1) The described numerical method is feasible to approximately evaluate the shear capacity of steel stud walls sheathed with wood panels and steel sheet, where the method provided a more accurate estimate for walls sheathed with wood panels. Refinement of the design method is warranted in order that inelastic effects are accounted for and to incorporate Canadian limit states design philosophy.

2) An *R*-value of 2.0 is shown to be suitable for use in the NBCC design of steel stud single-storey walls sheathed with wood panels. The value of this force modification factor is preliminary and further studies with respect to the seismic performance of steel stud shear walls need to be carried out.

6.2 RECOMMENDATIONS

To obtain a better evaluation of the shear capacity and ductility of steel stud shear walls the following recommendations are made:

1) To better predict the seismic shear capacity, the proposed numerical approach should be revised to include the inelastic effects before first failure, as well as the degradation effects of cyclic loading. It will also be necessary to develop resistance factors that are consistent with the National Building Code of Canada.

2) Further studies should be conducted to evaluate the effect of aspect ratio, Canadian construction configuration and dynamic forces on the ductility of steel walls.

3) Additional cyclic tests for wall segments and complete structures, as well as more advanced studies including time-history analyses of different design scenarios should be carried out to confirm the suggested force modification factor. In addition, studies which include an evaluation of the loading protocol that was used for the Serrette and UCI tests, the assumptions made in the bi-linear behaviour model used to evaluate R-values from existing tests, and the natural period of steel-stud shear walls should be completed.

4) A design method is required to estimate the shear capacity of walls that contain diagonal strap bracing. This is especially important for walls with strap braces on one side only, due to the possible torsion mode of failure at the chord, track and brace connection.

5) The performance of multi storey buildings with cold-formed steel stud shear walls that extend over the full height of the structure will also require extensive study.

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Appendix 'A' Summary of Existing Steel Stud Shear Wall Test Data

Authors		**Tarpy & Girard Tarpy & Hauenstein**	**Salenikovich *et al.***	**Serrette & Ogunfunmi**
Type of wall		Gypsum wallboard; plywood	OSB; Gypsum wallboard	X-bracing-A; GWB (in) & GSB (out) -B; both -C
Opening (Sheathing area ratio)		-----	1.0, 0.76, 0.56, 0.48, 0.3	-----
Type of loading		Monotonic	***Monotonic & cyclic***	Monotonic
Type of sheathing or brace	Thickness	1/2".	7/16"; 1/2"	12.5mm; 20 gauge-brace
	Orientation	Vertical Horizontal	Vertical	Vertical
	Grade	-----	-----	N/A
Screw	Type	Low profile head (framing) Hex head (sheathing)	Low profile head (framing) Bugle head (sheathing)	Wafer Head- A; Bugle-head- B
	Size	No.10 x 1/2" (framing) No. 6 x 1" (sheathing)	No. 8	No.8 x 0.5in (wood); No.6 x 1in (brace)
	Spacing	12"	6"/12" (exterior) 7"/10" (interior)	6"/12" – B, C
Blocking		@ mid-height	-----	-----
Stud	Grade	A 36 Grade	350S150-33	ASTM A446 Grade A
	Size & thickness	C-shape, 3-1/2" × 1-1/2" 0.036"	C-shape, 3-1/2" × 1-1/2", 0.033"	C-shape, 6" 20 gauge (0.88mm)
	Spacing	24", 16"	24"	24"
Size of test		8' × 8'; 12' × 8'	40' × 8'	8' × 8'

Authors		**Serrette *et al.***	**Serrette *et al.***	**Serrette *et al.***
Type of wall		Plywood, OSB	OSB	OSB
Type of loading		Monotonic	Monotonic	Monotonic
Type of sheathing or brace	Thickness	15/32"; 7/16"	7/16"	7/16"
	Orientation	Vertical	Horizontal	Vertical
Screw	Type	No. 8 × 1/2" wafer head (frame) No. 8 × 1" flat head (sheathing)	No. 8 × 1/2" wafer head (frame) No. 8 × 1" flat head (sheathing)	No. 8 × 1/2" wafer head (frame) No. 8 × 1" flat head (sheathing)
	Spacing	6"/12"	6"/12"	6"/12"
Blocking		-----	1-1/2" × 0.0346" strap @ mid-height	-----
Stud	Steel grade	ASTM A446 Grade A	ASTM A446 Grade A	ASTM A446 Grade A
	Size	C-shape, 3-1/2" × 1-5/8" × 3/8" 0.0346"	C-shape, 3-1/2" × 1-5/8" × 3/8" 0.0346"	C-shape, 3-1/2" × 1-5/8" × 3/8" 0.0346"
	Spacing	24"	24"	16"
Size of test		8' × 8'	4' × 8'; 8' × 8'	4' × 8'

Authors		**McCreless & Tarpy**	**NAHB**	**Tissell**
Type of wall		Gypsum wallboard	OSB; Gypsum wallboard	OSB, Plywood
Opening (Sheathing area ratio)		-----	1.0, 0.76, 0.48	-----
Type of loading		Monotonic	Monotonic	Monotonic
sheathing	Thickness	1/2"	7/16"; 1/2"	7/16" (19/32"), 3/8" (5/8")
	Orientation	Vertical Horizontal	*Vertical*	*Vertical*
Screw	*Type*	Low profile head (frame) Bugle head (sheathing)	N/A	-----
	Size	No. 10 × 1/2" (frame) No. 6 × 1" (sheathing)	No. 8 (OSB & framing) No.6 (GWB)	No.10-24 (14ga.,16ga.) No.8-18 (18ga.) 0.144"dia. × 1-1/4" pin
	Spacing	12"	6"/12" (exterior) 7"/10" (interior)	6"/12"; 4"/12"; 3"/12"
Blocking		@ mid-height	-----	-----
Stud	*Grade*	N/A	N/A	
	Size & thickness	C-shape, 3-1/2" 20 gauge (0.036")	C-shape, 3-1/2" × 1-1/2" 0.033"	14ga.(0.068"), 16ga.(0.054") 18ga.(0.043")
	Spacing	24"	24"	24"
Size of test		12' (16', 24') × 12'; 12' (16', 24') × 10'; 8' (12', 16', 24') × 8'	40' × 8'	4' × 8'

Authors		**Serrette *et al.***	**Serrette *et al.***	**Serrette *et al.***
Type of wall		OSB	OSB; GWB (other side)	GWB (both side)
Type of loading		Monotonic	Monotonic	Monotonic
Type of sheathing or brace	Thickness	7/16"	7/16"; 1/2"	7/16"; 1/2"
	Orientation	Vertical	Vertical	Horizontal
Screw	Type	No. 8 × 1/2" wafer head (frame) No. 8 × 1" flat head (sheathing)	No. 8 × 1/2" wafer head (frame); No. 8 × 1" flat head (OSB); No. 6 × 1-1/4" bugle head (gypsum)	No. 8 × 1/2" wafer head (frame) No. 6 × 1-1/4" bugle head (sheathing)
	Spacing	4"/12"; 3"/12"; 2"/12"	6"/12"; 4"/12"; 2"/12"; 7"/7" (gypsum)	7"/7"; 4"/4"
Blocking		-----	-----	1-1/2" × 0.0346" strap @ mid-height
Stud	*Steel grade*	ASTM A446 Grade A	ASTM A446 Grade A	ASTM A446 Grade A
	Size	C-shape, 3-1/2" × 1-5/8" × 3/8" 0.0346"	C-shape, 3-1/2" × 1-5/8" × 3/8" 0.0346"	C-shape, 3-1/2" × 1-5/8" × 3/8" 0.0346"
	Spacing	24"	24"	24"
Size of test		4' × 8'	4' × 8'; 8' × 8'	8' × 8'

Authors		**Serrette *et al.***	**Serrette *et al.***	**Serrette *et al.***
Type of wall		OSB; Plywood	OSB	OSB; Plywood
Type of loading		Cyclic	Monotonic	Cyclic
Type of sheathing or brace	*Thickness*	7/16"; 15/32"	7/16"	7/16"; 15/32"
	Orientation	Vertical	Vertical	Vertical
Screw	Type	No. 8 × 1/2" wafer head (frame) No. 8 × 1" flat head (sheathing)	No. 8-18 × 1/2" truss head (frame); No. 8-18 × 1" flat head (sheathing)	No. 8-18 × 1/2" truss head (frame); No. 8-18 × 1" flat head (sheathing)
	Spacing	6"/12"; 4"/12"; 3"/12"; 2"/12"	6"/12"; 4"/12"; 2"/12"	3"/12"; 2"/12"
Blocking		-----	-----	-----
Stud	Steel grade	ASTM A446 Grade A	ASTM A 653 SQ33	ASTM A446 Grade A
	Size	C-shape, 3-1/2" × 1-5/8" × 3/8" 0.0346"	C-shape, 3-1/2" × 1-1/2"× 1/2" 0.033"	C-shape, 3-1/2" × 1-1/2"× 1/2" 0.033" 0.043" (chord)
	Spacing	24"	24"	24"
Size of test		4' × 8'	2' × 8'	4' × 8'

Authors		**Serrette *et al.***	**Serrette *et al.***	**Serrette *et al.***
Type of wall		Steel X-bracing	Steel X-bracing;	Steel sheet sheathing
Type of loading		***Monotonic & cyclic***	***Monotonic & cyclic***	***Monotonic & cyclic***
Type of sheathing or brace	Thickness	4-1/2" × 0.033" (Width × Thickness)	7-1/2" × 0.033" in. (Width × Thickness)	0.018"
	Orientation	-----	-----	-----
Screw	Type	20 No. 8-18 × 1/2" modified truss head	30 No. 8-18 × 1/2" modified truss head	No. 8-18 × 1/2" modified truss head
	Spacing	-----	-----	6"/12"
Blocking		-----	-----	-----
Stud	*Steel grade*	ASTM A 653 SQ33	ASTM A 653 SQ33	ASTM A 653 SQ33
	Size	C-shape, 3-1/2" × 1-1/2"× 1/2" 0.033"	C-shape, 3-1/2" × 1-1/2"× 1/2" 0.033"	C-shape, 3-1/2" × 1-1/2"× 1/2" 0.033"
	Spacing	24"	24"	24"
Size of test		4' × 8'	4' × 8'	4' × 8'; 2' × 8' (monotonic only)

Authors		**Serrette *et al.***	**Serrette *et al.***	**Serrette *et al.***
Type of wall		Plywood	OSB	Steel sheet sheathing
Type of loading		***Cyclic***	***Monotonic & cyclic***	***Monotonic & cyclic***
Type of sheathing or brace	Thickness	15/32"	7/16"	0.027"
	Orientation	Vertical	Vertical	-----
Screw	Type	No. 8-18 × 1/2" truss head (frame); No. 8-18 × 1" flat head (sheathing)	No. 8-18 × 1/2" truss head (frame); No. 8-18 × 1" flat head (sheathing)	No. 8-18 × 1/2" modified truss head
	Spacing	6"/12"	6"/12"; 4"/12"; 2"/12"	4"/12"; 2"/12"(cyclic only)
Blocking		-----		-----
Stud	*Steel grade*	ASTM A 653 SQ33	ASTM A 653 SQ33	ASTM A 653 SQ33
	Size	C-shape, 3-1/2" × 1-1/2"× 1/2" 0.043" & 0.054"	C-shape, 3-1/2" × 1-1/2"× 1/2" 0.033"	C-shape, 3-1/2" × 1-1/2"× 1/2" 0.033"
	Spacing	24"	24"	24"
Size of test		4' × 8'	2' × 8'	2' × 8'

Authors		**Serrette *et al.***	TARPY	**COLA - UCI**
Type of wall		Plywood; OSB; GWB; Fiberbond	Gypsum Wallboard; Cement plaster	Plywood; OSB,
Type of loading		Monotonic	***Monotonic & cyclic***	***Cyclic***
Type of sheathing or brace	Thickness	11.9mm (Ply.), 11.1mm (OSB), 12.7mm (FB), 12.7mm (GWB)	1/2" (GWB); 7/8" (Cement Plaster)	15/32"; 7/16"
	Orientation	Vertical	Vertical	Vertical
Screw	Type	No. 8 × 12.7-mm wafer head (frame); No. 6 × 25.4-mm bugle head or No. 8 × 31.7-mm flat head (sheathing)	No.10 × 1/2" low profile head (frame); No. 6 × 1" bugle head or No. 8 × 1/2" pan head (sheathing)	No. 18 × 1/2" button head (framing) No. 8 bugle head (sheathing)
	Spacing	12"	12", 6"/12", 24"; 7-3/4"	6"/12"; 4"/12"; 2"/12"
Blocking		-----	-----	-----
Stud	*Steel grade*	N/A	N/A	N/A
	Size	C-shape152-mm, 20 gauge (0.88 mm)	C-shape, 3-1/2" × 1-1/2"× 1/2" 0.0359"	C-shape, 3-1/2" × 1-1/2"× 1/2" 0.033"
	Spacing	24"	24"	24"
Size of test		8' × 8'	8' × 8'; 12' × 8'	8' × 8'

APPENDIX 'B'
CALCULATIONS AND RESULTS OF DESIGN SHEAR STRENGTH

Appendix B1 Calculation of the shear strength for the steel stud wall (4ft.x8ft.) with stud thickness as 20 ga. sheathed with 7/16" OSB

Thickness of the panel t_s = 7/16 in. = 0.44 in. (OSB)
Dowel bearing strength of panel: F_{es} = 5550 psi
Diameter of fastener: D = 0.165 in. for 8d screw K_D = 2.2
Thickness of studs t_2 = 0.0346 in.
Ultimate tensile strength for studs: F_u = 45000 psi
Length of the wall: a = 4 ft. Height of the wall: h = 8 ft.

Bearing strength of wood panel (nominal and considering the load duration):
$Z = Dt_sF_{es}/K_D*3.5*C_D=$ 1020 lb
Bearing strength of steel studs:
$P_{ns} = 2.7t_2dF_{u2} =$ 694 lb
Tilting strength:
$P_{ns} = 4.2(t_2^3d)^{1/2}F_{u2} =$ 494 lb

For OSB with screws 6in./12in.
Studs spacing is : 24 in. m = 1 (interior stud)
Screws spacing: 6 in. (edges) 12 in. (fields)
n_s = 15 (one side screws) n_e = 9 (end screws)
n_{si} = 7 (interior screws per stud)
x_{ei} = 0,6,12,18,24 in. x_{si} = 0 in.
I_e = 2160 I_s = 0
$\beta = n_s + (4I_e + 2n_{si}I_s)/w^2 =$ 18.75
Force in the side fasteners: Maximum force in the end fasteners:
$F_s/P = b/\beta =$ 5.12 $F_s/P =$ 7.39

Maximum load P = 1.623 F_s lb/ft.
= 801.99 lb/ft. < P_{buckle} = 1199 lb/ft.

For OSB with screws 4in./12in.
Studs spacing is : 24 in. m = 1 (interior stud)
Screws spacing: 4 in. (edges) 12 in. (fields)
n_s = 23 (one side screws) n_e = 13 (end screws)
n_{si} = 7 (interior screws per stud)
x_{ei} = 0,4,8,12,16,20,24 in. x_{si} = 0 in.
I_e = 2912 I_s = 0
$\beta = n_s + (4I_e + 2n_{si}I_s)/w^2 =$ 28.06
Force in the side fasteners: Maximum force in the end fasteners:
$F_s/P = b/\beta =$ 3.42 $F_s/P =$ 5.03
Maximum load P = 2.384 F_{allow} lb/ft.
= 1177.83 lb/ft. < P_{buckle} = 1199 lb/ft.

For OSB with screws 3in./12in.

Studs spacing is :		24	in.	$m =$	1	(interior stud)
Screws spacing:		3	in. (edges)		12	in. (fields)
$n_s =$	31	(one side screws)		$n_e =$	17	(end screws)
$n_{si} =$	7	(interior screws per stud)				
$x_{ei} =$	0,3,6,9,12,15,18,21,24 in.			$x_{si} =$	0	in.
$I_e =$	3672			$I_s =$	0	

$\beta = n_s + (4I_e + 2n_{si}I_s)/w^2 =$ 37.38

Force in the side fasteners: $F_s/P = b/\beta =$ 2.57

Maximum force in the end fasteners: $F_s/P =$ 3.82

Maximum load	$P =$	3.144	F_{allow}	lb/ft.	
	=	1553.36	lb/ft.	$> P_{buckle} =$	1199 lb/ft.

For OSB with screws 2in./12in.

Studs spacing is :		24	in.	$m =$	1	(interior stud)
Screws spacing:		2	in. (edges)		12	in. (fields)
$n_s =$	47	(one side screws)		$n_e =$	25	(end screws)
$n_{si} =$	7	(interior screws per stud)				
$x_{ei} =$	0,2,4,6,8,10,12,14,16,18,20,22,24 in.			$x_{si} =$	0	in.
$I_e =$	5200			$I_s =$	0	

$\beta = n_s + (4I_e + 2n_{si}I_s)/w^2 =$ 56.03

Force in the side fasteners: $F_s/P = b/\beta =$ 1.71

Maximum force in the end fasteners: $F_s/P =$ 2.57

Maximum load	$P =$	4.663	F_{allow}	lb/ft.	
	=	2304.07	lb/ft.	$> P_{buckle} =$	1199 lb/ft.

Appendix B2 Calculation of the stud buckling for the steel stud wall with end stud thickness as 20 ga. sheathed with wood panel

Stud: 3.5in. x 1.625in. x 0.375in. (20 gauge = 0.0346in.)

w_w =	88.9	mm	w_f =	41.275	mm			
d =	9.525	mm	t =	0.879	mm			
E =	203000	Mpa	F_y =	33	ksi =		227.37	Mpa

Gross Properties: (neglecting round corner)

A =	328.659162	mm^2		I_x =	424978.7	mm^4
I_y =	129355.971	mm^4	r_x =	35.96	mm	
r_y =	19.84	mm				

Heigth of the wall:h = 8 ft.

When the studs are sheathed, no need to consider the torsional buckling

X-direction:			Y-direction:				
L_x = 8ft =	2438.4	mm	L_y = 6in. =	152.4	mm	(screw spacing)	
F_{ex} =	435.7	Mpa	F_{ey} =	33952.10	Mpa		
λ_c =	0.72	=< 1.5	F_n =	182.76	Mpa	<	F_y

Check under F_n the effective area

Web:	$W = w_w/t$ =	101.16	< 500 OK	W_{Lim} =	42.93	<	W	
Effective width must be used								
B =	54.644		b =	48.023	mm			
Flange:	$W = w_f/t$ =	46.97	< 60 OK					
W_{Lim1} =	14.07	W_{Lim2} =	42.93		Case III Flange			
I_s =	63.288	mm^4	I_A =	418.067	mm^4			
$I_R = I_s/I_A$ =	0.15	<1	d/w =	0.23	< 0.25	k =	2.332658	
B =	37.46		b =	32.917	mm			
Lip:	W = d/t =	10.84	< 14 OK!	W_{Lim} =	13.99	Lip is fully effective		
A_e =	233.61	mm^2						

$P_n = A_eF_n$ = 42694.3473 N = 9594.2 lb

The applied shear force caused stud buckling:

$P = P_n/h$ = 1199 lb/ft.

Appendix B3 Calculation of the shear strength for the steel stud wall
(4ft.x8ft.) with stud thickness as 20 ga. sheathed with 25ga. Sheet steel

Thickness of the steel sheet t_1 is		25 ga. =	0.018 in.			
Diameter of fastener: D =		0.164	in. for 8d screw			
Thickness of the stud t_2 =		0.033	in.	F_y =	230	Mpa
Ultimate tensile strength for stud:		F_u =	45000 psi			
Length of the wall: a =	4	ft.	Height of the wall: h =		8	ft.

t_2/t_1 = 1.83

Connection shear P_{ns} :

Bearing strength of steel sheet:			Bearing strength of steel studs:		
$P_{ns1} = 2.7t_1dF_{u1}$ =	359	lb	$P_{ns2} = 2.7t_2dF_{u2}$ =	658	lb
Tilting strength:					
$P_{ns} = 4.2(t_2^3d)^{1/2}F_{u2}$ =	459	lb			
for t_2/t_1 =< 1.0, P_{ns} =	359	lb			
for t_2/t_1 >= 2.5, P_{ns} =	359	lb			
Thus for t_2/t_1 =1.833, P_{ns} =	359	lb			

For sheet with screws 6in./12in.

Studs spacing is :	24	in.	m =	1	(interior stud)
Screws spacing:	6	in. (edges)		12	in. (fields)
n_s =	15	(one side screws)	n_e =	9	(end screws)
n_{si} =	7	(interior screws per stud)			
x_{ei} =	0,6,12,18,24 in.		x_{si} =	0	in.
I_e =	2160		I_s =	0	

$\beta = n_s + (4I_e + 2n_{si}I_s)/w^2$ = 18.75

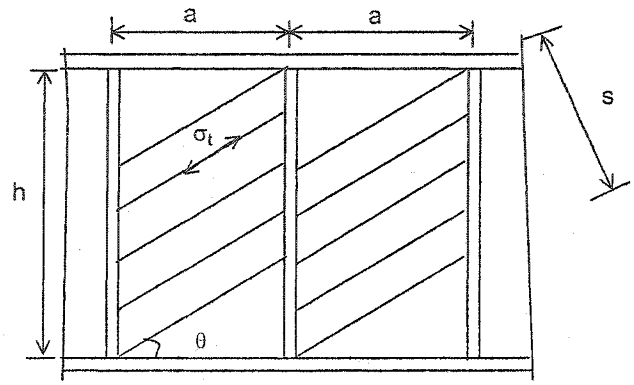
Force in the side fasteners: $F_s/P = b/\beta$ = 5.12

Maximum force in the end fasteners: F_s/P = 7.39

Maximum load P = 1.623 F_s lb/ft.

= 582 lb/ft.

Appendix B4 Calculation of the tension field force for the steel stud wall (4ft.x8ft.) with stud thickness as 20 ga. sheathed with 25ga. Sheet steel



$s = h\cos\theta - a\sin\theta$ Screws were spaced 6in. $s_1 =$ 6 in.

The number of the screws included in distance s is: $N = s/(s_1\cos\theta)$

Ignoring the tension force of the area between the screws,

The vertical component of force developed by tension field is:

$V_T = \sigma_t{*}A{*}N{*}\sin\theta = \sigma_t{*}A{*}s{*}\tan\theta/s_1 = \sigma_t sA(h\sin\theta - a\sin^2\theta/\cos\theta)/s_1$ where $A = d{*}t$

d is the diameter of the screw and t is the thickness of the sheet steel

$d =$ 0.165 in. $t =$ 0.018 in.

To find the value of θ, set $dV_T/d\theta = 0$ $tg^3\theta + 2tg\theta - h/a = 0$

Height of the wall is: $h =$ 8 ft.

Length of the wall is: $a =$ 4 ft.

Since the interior stud works as a stiffener, a can be taken as the space of the studs

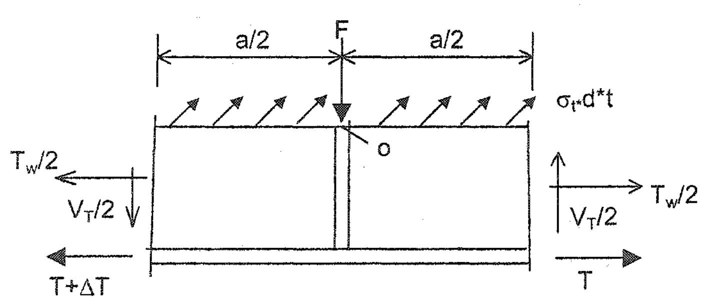
$a =$ 2 ft. $E =$ 203000 Mpa

$tg\theta =$ 1.7971 $\theta =$ 60.91

$F_y =$ 230 Mpa $K_v = 5.34 + 4.0(a/h)^2 =$ 5.59

$F_{CR} =$ 0.58 Mpa

$\tau_y = F_y/(3)^{1/2} =$ 132.8 Mpa $\sigma_t = F_y(1 - F_{CR}/\tau_y) =$ 229.0 Mpa



The number of the screws included in horizontal distance a is: $N_1 = a{*}\tan\theta/s_1$

$\Sigma F_{horz} = 0$ $\Delta_T = (\sigma_t d t N_1)\cos\theta =$ $\sigma_t{*}d{*}t{*}a{*}\sin\theta/s_1$

Σmoment about O =0

$V_T = \Delta_T{*}h/a = \sigma_t{*}d{*}t{*}h{*}\sin\theta/s_1 =$ 6.14 KN = 1379.2 lb

The applied shear load is: $V_T / h =$ 190.5 lb/ft.

APPENDIX 'B' CALCULATIONS AND RESULTS OF DESIGN SHEAR STRENGTH

Table B5: Steel Stud Wall Nominal Shear Capacity (Data from *Serrette et al. 1996b* and *COLA-UCI, 2001*)

Sample	Description	Predicted Strength (lb/ft.)			Peak Load from Tests (lb/ft.)	First controlled failure mode
		Connection Strength	Stud Buckling	Yielding in Tension Field		
AISI – OSB1,2	7/16" APA rated OSB sheathing w/panels on one side – parallel to framing (framing at 24" o.c. – fasteners at 6"/12" – 4'x 8' wall	802	1199 (1658)	—	932	Tilting of screws
AISI – OSB3,4	Same as AISI – OSB1 except fasteners at 4"/12"	1178	1199 (1658)	—	1269	Tilting of screws
AISI – OSB5,6	Same as AISI – OSB1 except fasteners at 3"/12"	1553	1199 (1658)	—	1725	Stud buckling Tilting of screw*
AISI – OSB7,8	Same as AISI – OSB1 except fasteners at 2"/12"	2304	1199 (1658)	—	1985	Stud buckling
AISI – PLY1,2	15/32" APA rated Plywood sheathing (4-ply) w/panels on one side – parallel to framing (framing at 24" o.c. – fasteners at 6"/12" – 4'x 8' wall	802	1199 (1658)	—	990	Tilting of screws
AISI – PLY3,4	Same as AISI – PLY1 except fasteners at 4"/12"	1178	1199 (1658)	—	1312	Tilting of screws
AISI – PLY5,6	Same as AISI – PLY1 except fasteners at 3"/12"	1553	1199 (1658)	—	1753	Stud buckling Tilting of screw*
AISI – PLY7,8	Same as AISI – PLY1 except fasteners at 2"/12"	2304	1199 (1658)	—	1928	Stud buckling
Group 14A,B,C	15/32" STR I APA rated Plywood sheathing (4-ply) w/panels on one side – parallel to framing (20ga. LGS framing at 24" o.c. – fasteners at 6"/12" – 8'x 8' wall	669	1116 (1658)	—	891	Tilting of screws
Group 15A,B,C	Same as Group 14 except fasteners at 4"/12"	1002	1116 (1658)	—	1222	Tilting of screws
Group 16A,B,C	Same as Group 14 except fasteners at 2"/12"	1998	1116 (1658)	—	2067	Stud buckling

Note: the underlined values govern the predicted strength; * denotes the first controlled failure mode when consider the increased stud buckling resistance as discussed in Section 3.4. The values in the brackets are the increased stud buckling resistance.

Table B6: Steel Stud Wall Nominal Shear Capacity (Data from *Serrette et al. 1996b* and *COLA-UCI, 2001*)

Sample	Description	Predicted Strength (lb/ft.)			Peak Load from Tests (lb/ft.)	First controlled failure mode
		Connection Strength	Stud Buckling	Yielding in Tension Field		
Group 17A,B,C	7/16" APA rated OSB sheathing w/panels on one side – parallel to framing (20ga. LGS framing at 24" o.c. – fasteners at 6"/12" – 8'x 8' wall	<u>669</u>	1116 (1658)	—	726	Tilting of screws
Group 18A,B,C	Same as Group 17 except fasteners at 4"/12"	<u>1002</u>	1116 (1658)	—	1065	Tilting of screws
Group 19A,B,C	Same as Group 17 except fasteners at 2"/12"	1998	<u>1116</u> (1658)	—	2006	Stud buckling
AISI – A1,2	15/32" APA rated sheathing Plywood one side 18 ga. 3-1/2 in. back to back chord studs; 20 ga. 3-1/2" interior studs and tracks; studs at 24" o.c. – fasteners at 3"/12" – 4'x 8' wall	<u>1553</u>	1781 (2388)	—	1933	Tilting of screws
AISI – A3,4	Same as AISI – A1 except fasteners at 2"/12"	2304	<u>1781</u> (2388)	—	2397	Stud buckling Tilting of screw*
AISI – A5,6	7/16" OSB APA rated sheathing one side 18 ga. 3-1/2 in. back to back chord studs; 20 ga. 3-1/2" interior studs and tracks; studs at 24" o.c. – fasteners at 3"/12" – 4'x 8' wall	<u>1553</u>	1781 (2388)	—	1652	Tilting of screws
AISI – A7,8	Same as AISI – A5 except fasteners at 2"/12"	2304	<u>1781</u> (2388)	—	2263	Stud buckling Tilting of screw*
AISI – B1,2	15/32" APA rated sheathing Plywood one side 18 ga. 3-1/2 in. studs; 20 ga. 3-1/2" tracks; studs at 24" o.c. – fasteners at 6"/12" – 4'x 8' wall	<u>1111</u>	1781 (2388)	—	1015	Tilting of screws
AISI – B3,4	Same as AISI – A1 except using 16 ga. 3-1/2" studs	<u>1201</u>	1781 (2388)	—	1063	Screw Fracture

Note: the underlined values govern the predicted strength; * denotes the first controlled failure mode when consider the increased stud buckling resistance as discussed in Section 3.4. The values in the brackets are the increased stud buckling resistance.

Table B7: Steel Stud Wall Nominal Shear Capacity (Data from *Serrette et al. 1996b* and *COLA-UCI, 2001*)

Sample	Description	Predicted Strength (lb/ft.)			Peak Load from Tests (lb/ft.)	First controlled failure mode
		Connection Strength	Stud Buckling	Yielding in Tension Field		
AISI – D1,2	25 ga. Steel sheathing one side; 20 ga. 3-1/2" studs and tracks; studs at 24" o.c. screws at 6"/12" – 4'x 8' wall	586	828	<u>191</u>	438	Yielding in the tension field
AISI – E1,2	7/16" APA rated OSB sheathing w/panels on one side; 20 ga. 3-1/2" tracks; studs at 24" o.c. – fasteners at 6"/12" – 2'x 8' wall	<u>813</u>	1116 (1658)	—	1065	Tilting of screws
AISI – E3,4	Same as AISI – E1 except fasteners at 4"/12"	1179	<u>1116</u> (1658)	—	2006	Stud buckling Tilting of screw*
AISI – E5,6	Same as AISI – E1 except fasteners at 2"/12"	2272	<u>1166</u> (<u>1658</u>)	—	1933	Stud buckling
AISI – F1,2	25 ga. Steel sheathing one side; 20 ga. 3-1/2" studs and tracks; studs at 24" o.c. screws at 4"/12" – 2'x 8' wall	1123	828	<u>447</u>	538	Yielding in the tension field
AISI – F3,4	Same as AISI – F1 except fasteners at 2"/12"	2164	<u>828</u>	<u>833</u>	1249	Both yielding in the tension field and stud buckling

Note: the underlined values govern the predicted strength; * denotes the first controlled failure mode when consider the increased stud buckling resistance as discussed in Section 3.4. The values in the brackets are the increased stud buckling resistance.

APPENDIX 'C' FORCE MODIFICATION FACTORS BASED ON EXISTING STEEL STUD SHEAR WALL TEST DATA (ALL VALUES ARE BASED ON PEAK LOAD)

Table C1: Steel Stud Shear Wall *R*-Values (Test Data from *Serrette et al. 1996b, 1997b*)

Sample	Description	$R_\mu = \sqrt{2\mu - 1}$	R_w (UBC, 94)	R (NEHRP) (UBC, 97)	R (NBCC)
AISI – OSB1	7/16" APA rated OSB sheathing w/panels on one side – parallel to framing (framing at 24" o.c. – fasteners at 6"/12" – 4'x 8' wall	2.3	8.1	5.8	2.3
AISI– OSB2	Same as AISI – OSB1	2.5	8.7	6.2	2.5
AISI – OSB3	Same as AISI – OSB1 except fasteners at 4"/12"	1.9	6.6	4.7	1.9
AISI – OSB4	Same as AISI – OSB3	2.4	8.4	6.0	2.4
AISI – OSB5	Same as AISI – OSB1 except fasteners at 3"/12"	2.0	6.9	4.9	2.0
AISI – OSB6	Same as AISI – OSB5	2.0	7.0	5.0	2.0
AISI – OSB7	Same as AISI – OSB1 except fasteners at 2"/12"	1.6	5.4	3.9	1.6
AISI – OSB8	Same as AISI – OSB7	1.6	5.4	3.9	1.6
AISI – PLY1	15/32" APA rated Plywood sheathing (4-ply) w/panels on one side – parallel to framing (framing at 24" o.c. – fasteners at 6"/12" – 4'x 8' wall	1.8	6.4	4.6	1.8
AISI – PLY2	Same as AISI – PLY1	2.2	7.9	5.6	2.2
AISI – PLY3	Same as AISI – PLY1 except fasteners at 4"/12"	2.3	8.1	5.8	2.3
AISI – PLY4	Same as AISI – PLY3	2.0	6.9	4.9	2.0
AISI – PLY5	Same as AISI – PLY1 except fasteners at 3"/12"	2.0	6.9	4.9	2.0
AISI – PLY6	Same as AISI – PLY5	2.0	7.1	5.1	2.0
AISI – PLY7	Same as AISI – PLY1 except fasteners at 2"/12"	1.6	5.4	3.9	1.6
AISI – PLY8	Same as AISI – PLY7	1.6	5.4	3.9	1.6

Table C2: Steel Stud Shear Wall *R*-Values (Test Data from *Serrette et al. 1996b, 1997b*)

Sample	Description	$R_\mu = \sqrt{2\mu - 1}$	R_w (UBC, 94)	R (NEHRP) (UBC, 97)	R (NBCC)
AISI – A1	15/32" APA rated sheathing Plywood one side 18 ga. 3-1/2 in. back to back chord studs; 20 ga. 3-1/2" interior studs and tracks; studs at 24" o.c. – fasteners at 3"/12" – 4'x 8' wall	2.4	8.4	6.0	2.4
AISI – A2	Same as AISI – A1	2.0	7.1	5.1	2.0
AISI – A3	Same as AISI – A1 except fasteners at 2"/12"	2.0	6.9	4.9	2.0
AISI – A4	Same as AISI – A3	1.9	6.6	4.7	1.9
AISI – A5	7/16" OSB APA rated sheathing one side 18 ga. 3-1/2 in. back to back chord studs; 20 ga. 3-1/2" interior studs and tracks; studs at 24" o.c. – fasteners at 3"/12" – 4'x 8' wall	2.0	6.8	4.9	2.0
AISI – A6	Same as AISI – A5	1.9	6.6	4.7	1.9
AISI – A7	Same as AISI – A5 except fasteners at 2"/12"	2.0	7.0	5.0	2.0
AISI – A8	Same as AISI – A7	2.0	7.0	5.0	2.0
AISI – B1	15/32" APA rated sheathing Plywood one side 18 ga. 3-1/2 in. studs; 20 ga. 3-1/2" tracks; studs at 24" o.c. – fasteners at 6"/12" – 4'x 8' wall	2.3	8.1	5.8	2.3
AISI – B2	Same as AISI – B1	2.4	8.4	6.0	2.4
AISI – B3	Same as AISI – A1 except using 16 ga. 3-1/2"studs	2.1	7.4	5.3	2.1
AISI – B4	Same as AISI – B3	2.3	8.1	5.8	2.3

Table C3: Steel Stud Shear Wall *R*-Values (Test Data from *Serrette et al. 1996b, 1997b*)

Sample	**Description**	$R_\mu = \sqrt{2\mu - 1}$	R_w **(UBC, 94)**	**R (NEHRP) (UBC, 97)**	**R (NBCC)**
AISI – C1	4-1/2" 20 ga. flat strap X-bracing one side; 20 ga. 3-1/2" studs and tracks; studs at 24" on center; No.8-18 x 1/2" self-drilling modified truss head screws – 4'x 8' wall	1.6	5.4	3.9	1.6
AISI – C2	Same as AISI – C2	1.8	6.4	4.6	1.8
AISI – D1	25 ga. Steel sheathing one side; 20 ga. 3-1/2" studs and tracks; studs at 24" o.c. screws at 6"/12" – 4'x 8' wall	2.7	9.5	6.8	2.7
AISI – D2	Same as AISI – D1	2.6	9.0	6.4	2.6
AISI – E1	7/16" APA rated OSB sheathing w/panels on one side; 20 ga. 3-1/2" tracks; studs at 24" o.c. – fasteners at 6"/12" – 2'x 8' wall	3.2	11.5	8.2	3.2
AISI – E2	Same as AISI – E1	2.8	9.8	7.0	2.8
AISI – E3	Same as AISI – E1 except fasteners at 4"/12"	2.7	9.5	6.8	2.7
AISI – E4	Same as AISI – E3	2.7	9.5	6.8	2.7
AISI – E5	Same as AISI – E1 except fasteners at 2"/12"	1.8	6.3	4.5	1.8
AISI – E6	Same as AISI – E5	2.0	6.9	4.9	2.0
AISI – F1	25 ga. Steel sheathing one side; 20 ga. 3-1/2" studs and tracks; studs at 24" o.c. screws at 4"/12" – 2'x 8' wall	2.6	9.1	6.5	2.6
AISI – F2	Same as AISI – F1	2.7	9.5	6.8	2.7
AISI – F3	Same as AISI – F1 except fasteners at 2"/12"	2.3	8.1	5.8	2.3
AISI – F4	Same as AISI – F3	2.2	7.8	5.6	2.2

Table C4: Steel Stud Shear Wall *R*-Values (Test Data from *COLA-UCI, 2001*)

Sample	Description	$R_\mu = \sqrt{2\mu - 1}$	R_w (UBC, 94)	R (NEHRP) (UBC, 97)	R (NBCC)
Group 14A	15/32" STR I APA rated Plywood sheathing (4-ply) w/panels on one side – parallel to framing (20ga. LGS framing at 24" o.c. – fasteners at 6"/12" – 8'x 8' wall	2.6	9.1	6.4	2.6
Group 14B	Same as Group 14A	2.5	8.9	6.3	2.5
Group 14C	Same as Group 14A	2.4	8.3	5.9	2.4
Group 15A	Same as Group 14A except fasteners at 4"/12"	2.5	8.7	6.2	2.5
Group 15B	Same as Group 15A	2.4	8.3	5.9	2.4
Group 15C	Same as Group 15A	2.4	8.5	6.0	2.4
Group 16A	Same as Group 14A except fasteners at 2"/12"	2.4	2.4	5.9	2.4
Group 16B	Same as Group 16A	2.0	7.1	5.1	2.0
Group 16C	Same as Group 16A	2.0	7.0	5.0	2.0
Group 17A	7/16" APA rated OSB sheathing w/panels on one side – parallel to framing (20ga. LGS framing at 24" o.c. – fasteners at 6"/12" – 8'x 8' wall	2.2	7.8	5.6	2.2
Group 17B	Same as Group 17A	2.6	9.1	6.5	2.6
Group 17C	Same as Group 17A	2.4	8.5	6.1	2.4
Group 18A	Same as Group 17A except fasteners at 4"/12"	2.7	9.5	6.8	2.7
Group 18B	Same as Group 18A	2.6	9.1	6.5	2.6
Group 18C	Same as Group 18A	2.7	9.5	6.8	2.7
Group 19A	Same as Group 17A except fasteners at 2"/12"	2.8	9.7	6.9	2.8
Group 19B	Same as Group 19A	2.7	9.0	6.4	2.7
Group 19C	Same as Group 19A	2.3	8.0	5.7	2.3

Table C5: Wood Stud Shear Wall *R*-Values
(Test Data from [1]*COLA-UCI, 2001* and [2]*Salenikovich et al., 2000*)

Sample	Description	$R_\mu = \sqrt{2\mu - 1}$	Ω	R_w in UBC, 94	R in NEHRP & UBC, 97	R in NBCC	Design R NBCC[1]
Group 03A[1]	15/32" STR I APA rated Plywood sheathing (4-ply) w/panels on one side – parallel to framing, D.F. stud at 16"o.c. – 10d nails at 6"/12" – 8'x 8' wall	2.9	1.9	7.7	5.5	2.9	3.0
Group 03B[1]	Same as Group 03A	2.5	1.9	6.6	4.7	2.5	3.0
Group 03C[1]	Same as Group 03A	3.0	2.0	8.3	5.9	3.0	3.0
Group 04A[1]	Same as Group 03A except nails at 4"/12"	2.8	1.8	7.1	5.1	2.8	3.0
Group 04B[1]	Same as Group 04A	2.6	1.8	6.6	4.7	2.6	3.0
Group 04C[1]	Same as Group 04A	2.7	1.6	5.9	4.2	2.7	3.0
Group 09A[1]	Same as Group 03A except nails at 2"/12"	2.5	1.8	6.4	4.6	2.5	3.0
Group 09B[1]	Same as Group 09A	2.5	1.8	6.2	4.4	2.5	3.0
Group 09C[1]	Same as Group 09A	2.4	1.9	6.3	4.5	2.4	3.0
Group 12A[1]	15/32" STR I APA rated OSB sheathing w/panels on one side – parallel to framing, D.F. stud at 16" o.c. – 10d nails at 4"/12"– 8'x 8' wall	3.7	1.7	8.6	6.1	3.7	3.0
Group 12B[1]	Same as Group 12A	3.0	1.7	6.9	4.9	3.0	3.0
Group 12C[1]	Same as Group 12A	3.3	1.7	7.8	5.5	3.3	3.0
Group 13A[1]	Same as Group 12A except nails at 2"/12"	2.6	1.7	6.0	4.3	2.6	3.0
Group 13B[1]	Same as Group 13A	2.6	1.8	6.7	4.8	2.6	3.0
Group 13C[1]	Same as Group 13A	2.7	1.8	6.7	4.8	2.7	3.0
Group 23A[1]	15/32" STR I APA rated Plywood sheathing (4-ply) w/panels on one side – horizontal to framing, D.F. stud at 16" o.c. – 10d nails at 4"/12"– 8'x 8' wall	3.0	1.7	7.2	5.1	3.0	3.0
Group 23B[1]	Same as Group 23A	2.8	1.8	7.0	5.0	2.8	3.0
Group 23C[1]	Same as Group 23A	2.8	1.7	6.4	4.6	2.8	3.0

[1]based on CWC 2001as described in Chapter 4 Section 4.4

Table C5 Continued

Sample	Description	$R_\mu = \sqrt{2\mu - 1}$	Ω	R_w in UBC, 94	R in NEHRP & UBC, 97	R in NBCC	Design R NBCC[1]
Group 35A[1]	2 15/32" STR I APA rated Plywood sheathing (4-ply) w/panels on same side – parallel to framing, D.F. stud at 16" o.c. – 10d nails at 6"/12" (6"/6") – 8'x 8' wall	2.6	3.4	12.4	8.9	2.6	3.0
Group 35B[1]	Same as Group 35A	2.7	3.6	13.3	9.5	2.7	3.0
Group 35C[1]	Same as Group 35A	2.4	3.7	12.1	8.7	2.4	3.0
Group 36A[1]	2 15/32" STR I APA rated OSB sheathing w/panels on both side – parallel to framing, D.F. stud at 16" o.c. – 10d nails at 4"/12" – 8'x 8' wall	2.9	1.8	7.0	5.0	2.9	3.0
Group 36B[1]	Same as Group 36A	2.1	1.4	4.0	2.9	2.1	3.0
Group 36C[1]	Same as Group 36A	2.5	1.7	5.7	4.1	2.5	3.0
04Fac-1[2]	15/32" STR I APA rated OSB sheathing w/panels on one side – parallel to framing, SPF stud at 16" o.c. – 10d nails at 6"/12" – 4'x 8' wall	3.0	1.9	7.8	5.6	3.0	3.0
04Fac-2[2]	Same as 04Fac-1	2.9	2.3	9.4	6.7	2.9	3.0
08Fac-1[2]	Same as 04Fac-1 except wall length as 8ft.	3.2	2.1	9.3	6.6	3.2	3.0
08Fac-2[2]	Same as 08Fac-1	2.9	2.1	8.2	5.8	2.9	3.0
12Fac-1[2]	Same as 04Fac-1 except wall length as 12ft.	3.3	2.2	9.9	7.1	3.3	3.0
12Fac-2[2]	Same as 12Fac-1	3.3	2.4	10.7	7.7	3.3	3.0
Group 06A[1]	3/8" Plywood and two 1/2" GWB sheathing w/panels on same/other side –parallel to framing, D.F. stud at 16 in. o.c. – 10d nails at 4"/12" (7"/7") – 8'x 8' wall	2.4	1.8	6.0	4.3	2.4	2.0
Group 06B[1]	Same as Group 06A	2.3	1.9	6.1	4.3	2.3	2.0
Group 06C[1]	Same as Group 06A	2.4	1.9	6.4	4.6	2.4	2.0
Group 34A[1]	Same as Group 06A except only one GWB on other side	2.5	1.5	5.3	3.8	2.5	2.0
Group 34B[1]	Same as Group 34A	3.0	1.5	6.2	4.4	3.0	2.0
Group 34C[1]	Same as Group 34A	2.7	1.4	5.5	3.9	2.7	2.0

[1]based on CWC 2001as described in Chapter 4 Section 4.4

Table C6: Masonry Shear Wall *R*-Values (Test Data from *Shing et al., 1991*)

Sample	f_m' (psi)	Ver. steel / Hor. steel	$R_\mu = \sqrt{2\mu - 1}$	Ω	R_w in UBC, 94	R in NEHRP & UBC, 97	R in NBCC	Design R NBCC[1]
Specimen 1	2900	5 x #5 / 5 x #4	2.8	1.7	6.8	4.8	2.8	1.5
Specimen 2	2900	5 x #5 / 9 x #3	4.0	1.8	10.2	7.3	4.0	1.5
Specimen 3	3000	5 x #7 / 5 x #3	3.8	1.8	9.7	6.9	3.8	1.0
Specimen 4	2600	5 x #7 / 5 x #3	2.6	1.9	6.7	4.8	2.6	1.0
Specimen 5	2600	5 x #7 / 5 x #3	2.0	1.8	5.1	3.6	2.0	1.0
Specimen 6	2600	5 x #5 / 5 x #3	2.6	1.7	6.3	4.5	2.6	1.5
Specimen 7	3000	5 x #7 / 5 x #3	2.8	1.8	7.0	5.0	2.8	1.0
Specimen 8	3000	5 x #5 / 5 x #4	3.1	1.5	6.4	4.6	3.1	2.0
Specimen 9	3000	5 x #5 / 5 x #3	2.9	2.1	8.4	6.0	2.9	1.0
Specimen 10	3200	5 x #5 / 5 x #3	3.4	1.7	7.8	5.6	3.4	1.5
Specimen 11	3200	5 x #7 / 5 x #4	3.1	1.3	5.7	4.1	3.1	1.5
Specimen 12	3200	5 x #5 / 5 x #4	3.7	1.5	7.6	5.4	3.7	1.5
Specimen 13	3300	5 x #6 / 5 x #4	3.0	2.3	9.7	6.9	3.0	1.0
Specimen 14	3300	5 x #6 / 5 x #3	2.6	2.2	8.1	5.8	2.6	1.0
Specimen 15	3300	5 x #6 / 5 x #4	4.5	1.5	9.6	6.9	4.5	1.5
Specimen 16	2500	5 x #7 / 5 x #4	2.9	2.0	8.3	5.9	2.9	1.0

[1]based on CSA S304.1 – 94 as described in Chapter 4 Section 4.4

APPENDIX 'D' Table D1: Overstrength Factor for Wood and Masonry Wall

(Test Data from [1]*COLA-UCI, 2001*, [2]*Salenikovich et al., 2000* and [3]*Shing et al., 1991*)

Sample	Nominal Strength	Design Strength	Ω	Sample	Nominal Strength	Design Strength	Ω
Group 03A[w1]	372.7	703.7	1.9	08Fac-1[w2]	231.0	479.1	2.1
Group 03B[w1]	372.7	705.6	1.9	08Fac-2[w2]	231.0	473.0	2.1
Group 03C[w1]	372.7	750.6	2.0	12Fac-1[w2]	231.0	743.9	2.2
Group 04A[w1]	556.9	1024.1	1.8	12Fac-2[w2]	231.0	815.1	2.4
Group 04B[w1]	556.9	1018.8	1.8	Group 06A[w1]	609.2	1085.2	1.8
Group 04C[w1]	556.9	867.3	1.6	Group 06B[w1]	609.2	1151.4	1.9
Group 09A[w1]	926.32	1680.3	1.8	Group 06C[w1]	609.2	1161.6	1.9
Group 09B[w1]	926.32	1666.9	1.8	Group 34A[w1]	609.2	914.4	1.5
Group 09C[w1]	926.32	1756.2	1.9	Group 34B[w1]	609.2	905.8	1.5
Group 12A[w1]	556.9	921.3	1.7	Group 34C[w1]	609.2	879.2	1.4
Group 12B[w1]	556.9	927.2	1.7	Specimen 1[m3]	205.7	351.6	1.7
Group 12C[w1]	556.9	942.1	1.7	Specimen 2[m3]	205.7	369.4	1.8
Group 13A[w1]	926.32	1534.5	1.7	Specimen 3[m3]	197.1	356.0	1.8
Group 13B[w1]	926.32	1679.9	1.8	Specimen 4[m3]	125.3	235.9	1.9
Group 13C[w1]	926.32	1674.8	1.8	Specimen 5[m3]	149.7	267.0	1.8
Group 23A[w1]	556.9	947.7	1.7	Specimen 6[m3]	125.3	215.8	1.7
Group 23B[w1]	556.9	1006.4	1.8	Specimen 7[m3]	155.5	278.1	1.8
Group 23C[w1]	556.9	921.1	1.7	Specimen 8[m3]	144.8	215.8	1.5
Group 35A[w1]	372.7	1277.7	3.4	Specimen 9[m3]	197.1	409.4	2.1
Group 35B[w1]	372.7	1321.9	3.6	Specimen 10[m3]	158.3	262.6	1.7
Group 35C[w1]	372.7	1372.7	3.7	Specimen 11[m3]	188.6	249.2	1.3
Group 36A[w1]	1113.8	1998.3	1.8	Specimen 12[m3]	213.0	309.3	1.5
Group 36B[w1]	1113.8	1507.1	1.4	Specimen 13[m3]	219.5	498.4	2.3
Group 36C[w1]	1113.8	1837.9	1.7	Specimen 14[m3]	201.2	451.7	2.2
04Fac-1[w2]	231.0	216.2	1.9	Specimen 15[m3]	214.4	327.1	1.5
04Fac-2[w2]	231.0	263.7	2.3	Specimen 16[m3]	191.0	382.7	2.0

Note: [w] denotes wood walls and [m] denotes masonry walls

The units for wood and masonry walls are lbs/ft. and kN, respectively.

Design strength for wood walls are obtained from Shearwall Selection Tables in CWC (*2001*) and those for masonry walls are evaluated from equation (4 – 29) as discussed in Section 4.4 in Chapter 4.

Table D2: Parameters for Steel Walls
(Test Data from *Serrette et al. 1996b, 1997b* and *COLA-UCI, 2001*)

Specimen	r	$E_{nor.}$	ζ_{eq} (%)	Ω_m	*Specimen*	r	E_{nor}	ζ_{eq} (%)	Ω_m
R Undefined					AISI-C1[s,1]	1.10	3.6	12.5	1.08
AISI-OSB1[s,1]	1.26	5.8	8.5	1.35	AISI-C2[s,1]	1.03	6.0	6.7	1.20
AISI-OSB2[s,1]	1.15	9.8	10.3	1.31	AISI-D1[s,1]	1.36	9.3	10.2	1.11
AISI-OSB3[s,1]	1.07	8.0	9.7	1.30	AISI-D2[s,1]	1.55	8.9	10.7	1.12
AISI-OSB4[s,1]	1.13	8.8	8.6	1.50	AISI-E1[s,1]	---	17.3	7.5	1.13
AISI-OSB5[s,1]	1.13	---	---	1.21	AISI-E2[s,1]	---	---	---	1.10
AISI-OSB6[s,1]	1.05	---	---	1.54	AISI-E3[s,1]	---	---	---	1.17
AISI-OSB7[s,1]	1.16	8.0	12.7	1.19	AISI-E4[s,1]	---	---	---	1.12
AISI-OSB8[s,1]	1.03	---	---	1.15	AISI-E5[s,1]	---	11.7	6.8	1.09
AISI-PLY1[s,1]	1.28	5.6	10.5	1.29	AISI-E6[s,1]	1.08	11.5	10.0	1.13
AISI-PLY2[s,1]	1.06	8.5	9.9	1.25	Grp. 14A[s,3]	1.20	12.6	10.1	1.25
AISI-PLY3[s,1]	1.05	8.5	9.8	1.24	Grp. 14B[s,3]	1.28	13.1	10.2	1.23
AISI-PLY4[s,1]	1.04	8.2	7.2	1.42	Grp. 14C[s,3]	1.23	14.6	9.0	1.23
AISI-PLY5[s,1]	1.02	---	---	1.20	Grp. 15A[s,3]	1.33	14.6	9.2	1.20
AISI-PLY6[s,1]	---	---	---	1.20	Grp. 15B[s,3]	1.22	14.3	9.2	1.21
AISI-PLY7[s,1]	1.06	---	---	1.16	Grp. 15C[s,3]	1.21	13.7	9.1	1.21
AISI-PLY8[s,1]	1.04	---	---	1.21	Grp. 16A[s,3]	1.11	14.7	8.1	1.14
AISI-A1[s,1]	---	14.3	7.1	1.10	Grp. 16B[s,3]	---	11.7	8.6	1.23
AISI-A2[s,1]	1.12	7.5	8.3	1.08	Grp. 16C[s,3]	---	13.2	10.0	1.09
AISI-A3[s,1]	---	15.2	7.6	1.07	Grp. 17A[s,3]	1.22	12.2	10.4	1.25
AISI-A4[s,1]	1.14	---	---	1.12	Grp. 17B[s,3]	1.23	13.9	10.9	1.20
AISI-A5[s,1]	1.07	5.4	9.5	1.08	Grp. 17C[s,3]	1.12	13.1	10.7	1.23
AISI-A6[s,1]	1.12	5.5	9.4	1.09	Grp. 18A[s,3]	1.27	11.7	10.3	1.21
AISI-A7[s,1]	1.04	---	---	1.11	Grp. 18B[s,3]	1.26	11.9	10.4	1.21
AISI-A8[s,1]	1.04	---	---	1.09	Grp. 18C[s,3]	---	13.4	11.1	1.18
AISI-B1[s,1]	1.26	---	---	1.13	Grp.19A[s,3]	1.17	15.0	8.2	1.17
AISI-B2[s,1]	1.28	---	---	1.15	Grp. 19B[s,3]	1.08	15.0	8.2	1.17
AISI-B3[s,1]	1.04	---	---	1.21	Grp. 19C[s,3]	---	14.9	8.5	1.16
AISI-B4[s,1]	1.16	---	---	1.14					

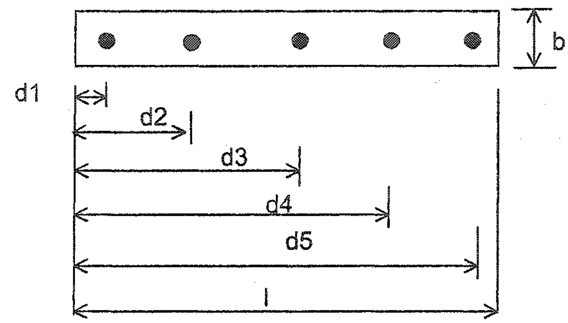
[s]cold-formed steel walls
[1]Serrette et al. *(1996b, 1997b)* [3]COLA-UCI *(2001)*
Note: E_{nor} of wood and steel walls are based on P_{peak}

Table D3: Parameters for Wood and Masonry Walls
(Test Data from *COLA-UCI, 2001*, *Salenikovich et al., 2000*, and *Shing et al., 1991*)

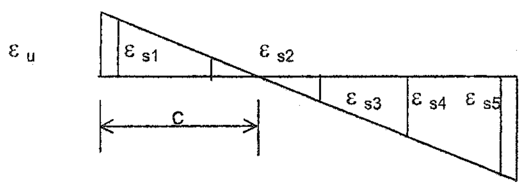
Specimen	r	$E_{nor.}$	ζ_{eq} (%)	Ω_m	*Specimen*	r	E_{nor}	ζ_{eq} (%)	Ω_m
R = 3.0					08Fac-2 w,2	1.61	15.8	13.2	1.14
Grp. 03A w,3	1.41	26.8	13.2	1.15	12Fac-1 w,2	1.47	16.1	13.4	1.15
Grp. 03B w,3	1.68	16.1	13.1	1.16	12Fac-2 w,2	1.43	15.8	13.3	1.15
Grp. 03C w,3	1.24	25.0	12.6	1.19	**R = 2.0**				
Grp. 04A w,3	---	29.1	11.8	1.19	Grp. 06A w,3	1.57	11.1	12.6	1.14
Grp. 04B w,3	1.28	24.3	12.6	1.14	Grp. 06B w,3	1.36	11.8	13.1	1.14
Grp. 04C w,3	1.32	23.0	12.8	1.20	Grp. 06C w,3	1.32	8.8	13.3	1.14
Grp. 09A w,3	1.51	25.5	10.4	1.13	Grp. 34A w,3	1.39	12.5	12.9	1.15
Grp. 09B w,3	---	23.2	10.4	1.11	Grp. 34B w,3	1.26	24.4	13.3	1.15
Grp. 09C w,3	1.46	19.4	10.8	1.10	Grp. 34C w,3	1.29	11.0	11.8	1.15
Grp. 12A w,3	1.27	33.7	13.7	1.14	Spec. 8 m,4	2.01	2.4	---	1.39
Grp. 12B w,3	1.51	27.9	14.5	1.15	**R = 1.5**				
Grp. 12C w,3	1.43	22.7	15.3	1.14	Spec. 1 m,4	1.26	1.36	---	1.38
Grp. 13A w,3	---	17.2	17.4	1.14	Spec. 2 m,4	1.39	1.02	---	1.37
Grp. 13B w,3	1.44	32.5	12.9	1.14	Spec. 6 m,4	2.73	3.58	---	1.65
Grp. 13C w,3	1.23	26.2	13.9	1.14	Spec. 10 m,4	1.65	0.72	---	1.48
Grp. 23A w,3	1.38	34.5	13.7	1.16	Spec. 11 m,4	1.44	0.58	---	1.46
Grp. 23B w,3	1.46	19.6	14.9	1.17	Spec. 12 m,4	1.48	0.72	---	1.54
Grp. 23C w,3	1.15	25.1	14.0	1.17	Spec. 15 m,4	1.43	0.91	---	1.52
Grp. 35A w,3	1.25	21.8	11.2	1.11	**R = 1.0**				
Grp. 35B w,3	1.42	17.6	12.0	1.16	Spec. 3 m,4	1.52	0.61	---	1.28
Grp. 35C w,3	1.39	16.3	11.7	1.14	Spec. 4 m,4	1.27	0.43	---	1.22
Grp. 36A w,3	1.40	25.5	11.5	1.13	Spec. 5 m,4	1.19	0.38	---	1.05
Grp. 36B w,3	1.59	16.5	12.6	1.14	Spec. 7 m,4	1.09	0.39	---	1.17
Grp. 36C w,3	1.41	18.9	10.6	1.12	Spec. 9 m,4	1.18	0.32	---	1.26
04Fac-1 w,2	1.82	9.7	11.2	1.10	Spec. 13 m,4	1.63	0.55	---	1.25
04Fac-2 w,2	1.41	16.9	10.2	1.13	Spec. 14 m,4	1.23	0.39	---	1.24
08Fac-1 w,2	1.45	15.4	12.9	1.15	Spec. 16 m,4	1.05	0.39	---	1.19

[w] wood walls [m] masonry walls
[2] Salenikovich et al. *(2000)* [3] COLA-UCI *(2001)* [4] Shing et al. *(1989, 1990a,b, 1991)*
Note: E_{nor} of wood and steel walls are based on P_{peak}, while those of masonry based on 0.5 post P_{peak}.

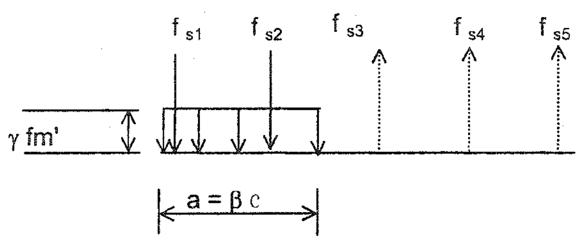
Appendix D4: Calculation of Shear and Flexural Capacity for Masonry Wall
(Test Data from *Shing et al., 1991*)



WALL CROSS SECTION



STRAIN



STRESS

d1 = 4 in. = 101.6 mm
d2 = 20 in.= 508 mm
d3 = 36 in.= 914.4 mm
d4 = 52 in.= 1320.8 mm
d5 = 68 in.= 1727.2 mm

Wall specimen 1

l = 6 ft = 1828.8 mm h = 6ft = 1.8288 m

Vertical steel 5 x # 5 A_{sv} = 199 mm^2

Horizontal steel 5 x # 4 A_{sh} = 129 mm^2 Es = 200000 Mpa

Axis Load	P =	80	kips =	356	KN
Shear Load	Hmax-ave =	82.5	kips =	367.13	KN
Masonry:	f_m' =	2900	psi =	19.981	Mpa
Steel (Ver.):	f_{yv} =	64	ksi =	440.96	Mpa
Horizontal	f_{yh} =	67	ksi =	461.63	Mpa

s = 431.8 mm γ = 0.85 b = 5-5/8 ir 143 mm

d = 0.8*l = 1463.04 mm (CSA S304.1Clause 11.5.3.1)

β = 0.8 for f_m' <20Mpa, and 0.8-0.1x(f_m'-20)/10 for f_m'>= 20Mpa (Clause 10.2.3.1.7)

β = 0.8

Shear resistance calculation: (CSA S304.1 Clause 11.5.3)

v_m = $0.16\,(2 - M_f/(V_f{*}d))f_m'^{1/2}$

$M_f/(V_f{*}d)$ = h/d = 1.250 (should >=0.25 and =<1.0)

v_m = 0.71520179

Factored shear resistance V_r:

ϕ_m = 1 ϕ_s = 1 γ_g =1.0 for fully grouted

V_r = $\phi_m(v_m b_w d + 0.25P)\gamma_g + \phi_s(0.60A_h f_y d/s)$ = 359.69 KN

$V_{rmax} = \phi_m{*}0.4(f_m')^{1/2} b_w d\gamma_g$ = 374.08 KN V_r = 359.69 KN

Considering half contribution of the masonry wall and axial load in the plastic hinge region

ϕ_m = 0.5 Vr = 240.38 KN for wall with R=: (CSA S304.1 A6.1)

Calcuate the compression strain at ultimate state:

$P = \gamma\beta f_m' cb - \Sigma A_{si}f_{si}$ $f_{si} = \varepsilon_u E_s(d_i-c)/c =< f_y$

$M = H{*}h = \gamma\beta f_m' cb(l/2-\beta c/2) + \Sigma A_{si}f_{si}(d_i - l/2)$

Five different cases were assumed for different walls.
The steel stress were checked after getting the forces.
Those met the assumption were considered as the acceptable case (expressed as good).

1) Assume $f_{s2} = f_{s3} = f_{s4} = f_{s5} = f_y$ $f_{s1} = -f_y$

Assume ε_u = 0.003

c = $(P{*}1000+f_y{*}(A_{s2}+A_{s3}+A_{s4}+A_{s5}-A_{s1}))/(\gamma{*}\beta{*}f_m'{*}b)$

= 318.72 mm < $0.2l_w$ = 365.76 mm

M = $\gamma{*}\beta{*}f_m'{*}c{*}b{*}(l/2-\beta{*}c/2)+f_y{*}(A_{s2}{*}(d_2-l/2)+A_{s3}{*}(d_3-l/2)+A_{s4}{*}(d_4-l/2)+A_{s5}{*}(d_5-l/2)-A_{s1}{*}(d_1-l/2))$

= 629946398.6 N*mm

H = M/h = 344.46 KN

check steel stress:

ε_{s1} =	$\varepsilon_u*(d_1-c)/c$=	-0.002043667	$f_{s1} = E_s*\varepsilon_{s1}$	-408.73	Mpa	<fy
ε_{s2} =	$\varepsilon_u*(d_2-c)/c$=	0.001781663	$f_{s2} = E_s*\varepsilon_{s2}$	356.33	Mpa	<fy
ε_{s3} =	$\varepsilon_u*(d_3-c)/c$=	0.005606993	$f_{s3} = E_s*\varepsilon_{s3}$	1121.40	Mpa	> fy
ε_{s4} =	$\varepsilon_u*(d_4-c)/c$=	0.009432323	$f_{s4} = E_s*\varepsilon_{s4}$	1886.46	Mpa	> fy
ε_{s5} =	$\varepsilon_u*(d_5-c)/c$=	0.013257653	$f_{s5} = E_s*\varepsilon_{s5}$	2651.531	Mpa	> fy

not good

2) Assume $f_{s3} = f_{s4} = f_{s5} = f_y$ $f_{s1} = \varepsilon_u*E_s*(d_1-c)/c$ $f_{s2} = \varepsilon_u*E_s*(d_2-c)/c$

$P = \gamma*\beta*f_m'*c*b - f_y*(A_{s3}+A_{s4}+A_{s5})-A_{s2}*f_{s2}-A_{s1}*f_{s1}$

$\gamma*\beta*f_m'*b*c^2 -(P+f_y*(A_{s3}+A_{s4}+A_{s5})-E_s*\varepsilon_u*A_{s2}-E_s*\varepsilon_u*A_{s1})*c-(A_{s2}*\varepsilon_u*E_s*d_2+A_{s1}*\varepsilon_u*E_s*d_1)=0$

Assume ε_u = 0.003

$\Delta^2 = (P+f_y*(A_{s3}+A_{s4}+A_{s5})-E_s*\varepsilon_u*A_{s2}-E_s*\varepsilon_u*A_{s1})^2 +4*\gamma*\beta*f_m'*b*(A_{s2}*\varepsilon_u*E_s*d_2+A_{s1}*\varepsilon_u*E_s*d_1)$

= 7.10425E+11

c = 314.81 mm < $0.2l_w$ = 365.76 mm

M = $\gamma*\beta*f_m'*c*b*(l/2-\beta*c/2)+f_y*(A_{s3}*(d_3-l/2)+A_{s4}*(d_4-l/2)+A_{s5}*(d_5-l/2))$
$+A_{s2}*E_s*\varepsilon_u*(d_2-c)*(d_2-l/2)/c+A_{s1}*E_s*\varepsilon_u*(d_1-c)*(d_1-l/2)/c$

= 625215042.7 N*mm

H = M/h = 341.87 KN

check steel stress:

ε_{s1} =	$\varepsilon_u*(d_1-c)/c$=	-0.002031796	$f_{s1} = E_s*\varepsilon_{s1}$	-406.36	Mpa	<fy
ε_{s2} =	$\varepsilon_u*(d_2-c)/c$=	0.00184102	$f_{s2} = E_s*\varepsilon_{s2}$	368.20	Mpa	<fy
ε_{s3} =	$\varepsilon_u*(d_3-c)/c$=	0.005713836	$f_{s3} = E_s*\varepsilon_{s3}$	1142.77	Mpa	> fy
ε_{s4} =	$\varepsilon_u*(d_4-c)/c$=	0.009586652	$f_{s4} = E_s*\varepsilon_{s4}$	1917.33	Mpa	> fy
ε_{s5} =	$\varepsilon_u*(d_5-c)/c$=	0.013459468	$f_{s5} = E_s*\varepsilon_{s5}$	2691.89	Mpa	> fy

good

3) Assume $f_{s3} = f_{s4} = f_{s5} = f_y$, $f_{s1} = -f_y$ $f_{s2} = \varepsilon_u*E_s*(d_2-c)/c$

$P = \gamma*\beta*f_m'*c*b - f_y*(A_{s3}+A_{s4}+A_{s5}-A_{s1})-A_{s2}*f_{s2}$

$\gamma*\beta*f_m'*b*c^2 -(P+f_y*(A_{s3}+A_{s4}+A_{s5}-A_{s1})-E_s*\varepsilon_u*A_{s2})*c-A_{s2}*\varepsilon_u*E_s*d_2=0$

Assume ε_u = 0.003

$\Delta^2 = (P+f_y*(A_{s3}+A_{s4}+A_{s5}-A_{s1})-E_s*\varepsilon_u*A_{s2})^2 +4*\gamma*\beta*f_m'*b*A_{s2}*\varepsilon_u*E_s*d_2$

= 6.41229E+11

c = 312.12 mm < $0.2l_w$ = 365.76 mm

M = $\gamma*\beta*f_m'*c*b*(l/2-\beta*c/2)+f_y*(A_{s3}*(d_3-l/2)+A_{s4}*(d_4-l/2)+A_{s5}*(d_5-l/2)-A_{s1}*(d_1-l/2))$
$+A_{s2}*E_s*\varepsilon_u*(d_2-c)*(d_2-l/2)/c$

= 626669219 N*mm

H = M/h = 342.67 KN

check steel stress:

ε_{s1} =	$\varepsilon_u*(d_1-c)/c$=	-0.002023453	$f_{s1} = E_s*\varepsilon_{s1}$	-404.69	Mpa	<fy
ε_{s2} =	$\varepsilon_u*(d_2-c)/c$=	0.001882733	$f_{s2} = E_s*\varepsilon_{s2}$	376.55	Mpa	<fy
ε_{s3} =	$\varepsilon_u*(d_3-c)/c$=	0.00578892	$f_{s3} = E_s*\varepsilon_{s3}$	1157.78	Mpa	> fy

ε_{s4} =	ε_u*(d_4-c)/c=	0.009695106	$f_{s4} = E_s*\varepsilon_{s4}$:	1939.02	Mpa	> fy
ε_{s5} =	ε_u*(d_5-c)/c=	0.013601292	$f_{s5} = E_s*\varepsilon_{s5}$:	2720.26	Mpa	> fy
						not good

4) Assume $f_{s2} = f_{s3} = f_{s4} = f_{s5} = f_y$ $\quad\quad f_{s1} = \varepsilon_u*E_s*(d_1-c)/c$

$P = \gamma*\beta*f_m'*c*b - f_y*(A_{s2}+A_{s3}+A_{s4}+A_{s5})-A_{s1}*f_{s1}$

$\gamma*\beta*f_m'*b*c^2 -(P+f_y*(A_{s2}+A_{s3}+A_{s4}+A_{s5})-E_s*\varepsilon_u*A_{s1})*c-A_{s1}*\varepsilon_u*E_s*d_1=0$

Assume ε_u = 0.003

$\Delta^2 = (P+f_y*(A_{s2}+A_{s3}+A_{s4}+A_{s2})-E_s*\varepsilon_u*A_{s1})^2 +4*\gamma*\beta*f_m'*b*A_{s1}*\varepsilon_u*E_s*d_1$

= 4.39559E+11

c = 321.83 mm < $0.2l_w$ = 365.76 mm

M = $\gamma*\beta*f_m'*c*b*(l/2-\beta*c/2)+f_y*(A_{s3}*(d_2-l/2)+A_{s4}*(d_3-l/2)+A_{s5}*(d_4-l/2)+A_{s1}*(d_5-l/2))$
$+A_{s1}*E_s*\varepsilon_u*(d_1-c)*(d_1-l/2)/c$

= 629011712.6 N*mm

H = M/h = 343.9477868 KN

check steel stress:

ε_{s1} =	ε_u*(d_1-c)/c=	-0.002052913	$f_{s1} = E_s*\varepsilon_{s1}$:	-410.58	Mpa	<fy
ε_{s2} =	ε_u*(d_2-c)/c=	0.001735436	$f_{s2} = E_s*\varepsilon_{s2}$:	347.09	Mpa	<fy
ε_{s3} =	ε_u*(d_3-c)/c=	0.005523784	$f_{s3} = E_s*\varepsilon_{s3}$:	1104.76	Mpa	> fy
ε_{s4} =	ε_u*(d_4-c)/c=	0.009312133	$f_{s4} = E_s*\varepsilon_{s4}$:	1862.43	Mpa	> fy
ε_{s5} =	ε_u*(d_5-c)/c=	0.013100481	$f_{s5} = E_s*\varepsilon_{s5}$:	2620.10	Mpa	> fy
						not good

5) Assume $f_{s4} = f_{s5} = f_y$ $\quad\quad f_{si} = \varepsilon_u*E_s*(d_i-c)/c \quad i = 1,2,3$

$P = \gamma*\beta*f_m'*c*b - f_y*(A_{s4}+A_{s5})-A_{s3}*f_{s3}-A_{s2}*f_{s2}-A_{s1}*f_{s1}$

$\gamma*\beta*f_m'*b*c^2 -(P+f_y*(A_{s4}+A_{s5})-E_s*\varepsilon_u*A_{s3}-E_s*\varepsilon_u*A_{s2}-E_s*\varepsilon_u*A_{s1})*c-(A_{s3}*\varepsilon_u*E_s*d_3+A_{s2}*\varepsilon_u*E_s*d_2+A_{s1}*\varepsilon_u*E_s*d_1)=0$

Assume ε_u = 0.003

$\Delta^2 = (P+f_y*(A_{s4}+A_{s5})-E_s*\varepsilon_u*(A_{s3}+A_{s2}+A_{s1})^2 +4*\gamma*\beta*f_m'*b*\varepsilon_u*E_s(A_{s3}*d_3+A_{s2}*d_2+A_{s1}*d_1)$

= 1.44424E+12

c = 407.17 mm > $0.2l_w$ = 365.76 mm

M = $\gamma*\beta*f_m'*c*b*(l/2-\beta*c/2)+f_y*(A_{s4}*(d_4-l/2)+A_{s5}*(d_5-l/2))$
$+E_s*\varepsilon_u*(A_{s3}*(d_3-c)*(d_3-l/2)+A_{s2}*(d_2-c)*(d_2-l/2)+A_{s1}*(d_1-c)*(d_1-l/2))/c$

= 762345286.6 N*mm

H = M/h = 416.86 KN

check steel stress:

ε_{s1} =	ε_u*(d_1-c)/c=	-0.002251415	$f_{s1} = E_s*\varepsilon_{s1}$:	-450.28	Mpa	> fy
ε_{s2} =	ε_u*(d_2-c)/c=	0.000742925	$f_{s2} = E_s*\varepsilon_{s2}$:	148.59	Mpa	<fy
ε_{s3} =	ε_u*(d_3-c)/c=	0.003737266	$f_{s3} = E_s*\varepsilon_{s3}$:	747.45	Mpa	> fy
ε_{s4} =	ε_u*(d_4-c)/c=	0.006731606	$f_{s4} = E_s*\varepsilon_{s4}$:	1346.32	Mpa	> fy
ε_{s5} =	ε_u*(d_5-c)/c=	0.009725946	$f_{s5} = E_s*\varepsilon_{s5}$:	1945.19	Mpa	> fy
						not good

Table D5: Scope of AISI Shear Wall Design Guides (*AISI, 1998; AISI, 2002*)

Assembly Description	Wind or Earthquake	AISI 1998	AISI 2002 Draft
15/32" Structural 1 Sheathing (4-ply) one side, fasteners spaced at 6"; Studs maximum 24" o/c Max aspect ratio 2:1	W	Approved	Approved
7/16" Rated Sheathing (OSB) one side spaced at 6", 4", 3", 2"; Studs maximum 24" o/c Max aspect ratio 2:1	W	Approved	Approved
7/16" Rated Sheathing (OSB) one side oriented perpendicular to framing spaced at 6"; Studs maximum 24" o/c Max aspect ratio 2:1	W	Proposed	Approved
7/16" Rated Sheathing (OSB) one side spaced at 4", 3", 2" Max aspect ratio 4:1	W	Proposed	Approved
0.018 inch steel sheet, one side, screws spaced at 6"; Studs maximum 24" o/c Max aspect ratio 2:1	W	Proposed	Approved
0.027 inch steel sheet, one side, spaced at 4", 3", 2"; Studs maximum 24" o/c Max aspect ratio 4:1	W	Proposed for 4"	Approved
½ gypsum board on one side of wall with screws spaced at 7", 4"; Studs maximum 24" o/c Max aspect ratio 2:1	W	Approved	Approved
15/32 Structural 1 Sheathing (4-ply) plywood one side spaced at 6", 4", 3", 2"; Studs maximum 24" o/c Max aspect ratio 4:1	E	Approved	Approved
15/32 Structural 1 Sheathing (4-ply) plywood one side spaced at 3", 2"; end studs 0.043 inch min thickness; Studs maximum 24" o/c Max aspect ratio 2:1	E	Proposed	Approved
15/32 Structural 1 Sheathing (4-ply) plywood one side spaced at 6", 4", 3", 2"; all studs 0.043 inch min thickness; Studs maximum 24" o/c Max aspect ratio 2:1	E	Proposed	Approved
7/16" OSB one side spaced at 6", 4", 3", 2"; Studs maximum 24" o/c	E	Approved for Max aspect ratio 2:1 Proposed for Max aspect ratio 4:1	Approved
7/16" OSB one side spaced at 3", 2"; end studs 0.043 inch min thickness; Studs maximum 24" o/c Max aspect ratio 2:1	E	Proposed	Approved
0.018 inch steel sheet, one side, screws spaced at 6"; Studs maximum 24" o/c Max aspect ratio 2:1	E	Proposed	Approved
0.027 inch steel sheet, one side, spaced at 4", 3", 2"; Studs maximum 24" o/c Max aspect ratio 4:1	E	Proposed	Approved
Perforated Walls	W &E	Not Included	Approved