SEISMIC DESIGN OF COLD FORMED STEEL STRUCTURES IN RESIDENTIAL APPLICATIONS

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ABSTRACT

SEISMIC DESIGN OF COLD FORMED STEEL STRUCTURES IN RESIDENTIAL APPLICATIONS

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In this study, lateral load bearing capacities of cold formed steel framed wall panels are investigated. For this purpose lateral load bearing alternatives are analyzed numerically by computer models and results are compared with already done experimental studies and approved codes.

In residential cold formed steel construction, walls are generally covered with cladding material like oriented strand board (OSB) or plywood on the exterior wall surface and these sheathed light gauge steel walls behave as shear walls with significant capacity. Oriented strand board is used in analytical models since OSB claddings are most commonly used in residential applications. The strength of shear walls depends on different parameters like screw spacing, strength of sheathing, size of fasteners used and aspect ratio. SAP2000 software is used for structural analysis of walls and joint force outputs are collected by Microsoft Excel. The yield strength of shear walls at which first screw connection reaches its shear capacity is calculated and load carrying capacity per meter length is found. The nonlinear analysis is also done by modeling the screw connections between OSB and frame as non-linear link and the nominal shear capacities of walls are calculated for different screw spacing combinations. The results are consistent with the values in shear wall design Guide and International Building Code 2003.

The other lateral load bearing method is flat strap X-bracing on wall surfaces. Various parameters like wall frame section thickness, flat strap area, aspect ratio and bracing number are investigated and results are evaluated.

The shear walls in which X-bracing and OSB sheathing used together are also analyzed and the results are compared with separate analyses.

Keywords: cold formed steel shear walls, OSB sheathing, flat strap bracing, lateral stiffness.

ÖZ

KONUT TÜRÜ YAPILARDA SOĞUKTA ŞEKİL VERİLMİŞ ÇELİK YAPILARIN DEPREME DAYANIKLI TASARIMI

Uygar, Celaletdin
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Mayıs 2006, 82 sayfa

Bu çalışmada soğukta şekil verilmiş çelik çerçeveli duvar panellerinin yatay yük taşıma kapasitesi araştırılmıştır. Bu amaçla yatay yük taşıma sistemleri alternatifleri bilgisayar modelleri kullanılarak sayısal olarak modellenmiş, analizleri yapılmış ve bulunan sonuçlar daha önceden yapılmış olan deneysel çalışmalar ve yönetmeliklerdeki değerlerle karşılaştırılmıştır.

Konut türü hafif çelik yapılarda duvarlar dış yüzeylerinden genellikle yönlendirilmiş ahşap yonga levha (OSB) ve kontraplak levha kaplama malzemeleriyle kaplanmaktadır ve bu malzemelerle kaplanmış hafif çelik duvarlar ciddi yatay yük taşıma kapasitesine sahip perde duvar olarak davranırlar. Konut türü yapılarda çok daha yaygın olarak kullanıldığından analitik modellerde OSB plakalar kullanılmıştır. Perde duvarların kapasiteleri vida aralığı, panel mukavemeti, vida boyutu ve en boy oranı gibi çeşitli parametrelere bağlı olarak değişmektedir. Duvarların statik analizinde SAP2000 yazılımı, bağlantı noktası kuvvet sonuçlarının derlenmesinde de Microsoft Excel yazılımı kullanılmıştır. Duvarlardaki ilk vida bağlantısının kesme dayanımına ulaştığı andaki kapasitesi olan duvar akma kapasiteleri hesaplanıp birim genişlikteki perde duvarın kesme kapasitesi bulunmuştur. Duvar panellerinin lineer olmayan analizleri, duvar elemanları ve OSB plakalar arasındaki bağlantılar lineer olmayan bağlantı elemanı olarak modellenerek yapılmış ve nihai kapasiteleri değişik vida aralığı durumları için hesaplanmıştır. Bulunan sonuçların perde duvar tasarım kılavuzu ve uluslararası bina yönetmeliği 2003'te verilen sonuçlarla tutarlı olduğu görülmüştür.

Diğer bir yatay yük taşıma mekanizması da duvar yüzeyinde X-biçimli düz şerit çelik çaprazlar kullanılmasıdır. Duvar kesit kalınlığı, düz çelik şerit kesit alanı, yükseklik-genişlik oranı ve çapraz sayısı gibi çeşitli parametreler incelenerek kapasite ve rijitlikler hesaplanmış ve sonuçlar değerlendirilmiştir.

X-biçimli düz şerit çelik çaprazlar ve OSB plakaların beraber kullanıldığı perde duvarların da analizleri yapılmış ve çıkan sonuçlar ayrı ayrı yapılan analizlerle karşılaştırılmıştır.

Anahtar Kelimeler: soğukta sekil verilmiş çelik perde duvarlar, OSB kaplaması, düz şerit çaprazlar, yatay rijitlik.

To my family...

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CHAPTER I

INTRODUCTION

1.1 Objective and Scope of the Study

The objective of this study is to investigate analytically the lateral load capacities of shear walls used in cold formed steel framed residential buildings. The capacity of the shear walls depend on the interaction of several parameters like screw spacing on perimeter and field, screw type and size, plate type and thickness, plate strength and aspect ratio. The main parameters are studied and some of them are assumed constant. In experimental studies, it was observed that the failure mechanism of shear walls is OSB sheathing and steel frame connection and the connection shear forces are calculated in all the computer models and shear wall capacities are calculated according to this parameter.

Shear walls with X-type flat strap diagonal bracing are also analyzed and parameters for design are investigated. Since the X-bracing shear wall members are designed against imposed design loads, in this study it is focused on the design parameter than lateral load capacity. Also combination of X-bracing and OSB sheathing together are analyzed and contributions are determined.

1.2 General

Cold formed steel sections are being increasingly used in residential construction all over the world and light gauge steel houses are becoming preferable in Turkey since last severe earthquakes. Its advantages like high strength vs. weight ratio, very short construction time, great resistance to earthquake because of its low weight, environmentally friendly, high sound and heat isolation are the main advantageous of this construction technique (Fig 1.1 and Fig 1.2). Usage of galvanized steel brings a perfect solution to corrosion problem and since the steel is purchased as galvanized, it eliminates the additional corrosion protection.

Figure 1.1 Steel frame of a light gauge steel house

Figure 1.2 Completed light gauge steel house

The cold formed steel sections are produced from steel coils by roll-formers or hydraulic press machine. Production with roll-formers is faster and more accurate because in roll-formers the whole production process is controlled by a computer. Today it is possible by the developments in roll-former technology (Fig 1.3) to produce the frames of several houses in a day and finish hundreds of houses in a few months. Especially the wall frames are assembled in factory and transported to the site as frames and at site these frames are assembled to each other. According to scale of project, the roll-formers can be moved to site and the production can be done in a workshop at site, which results in great savings in transportation costs.

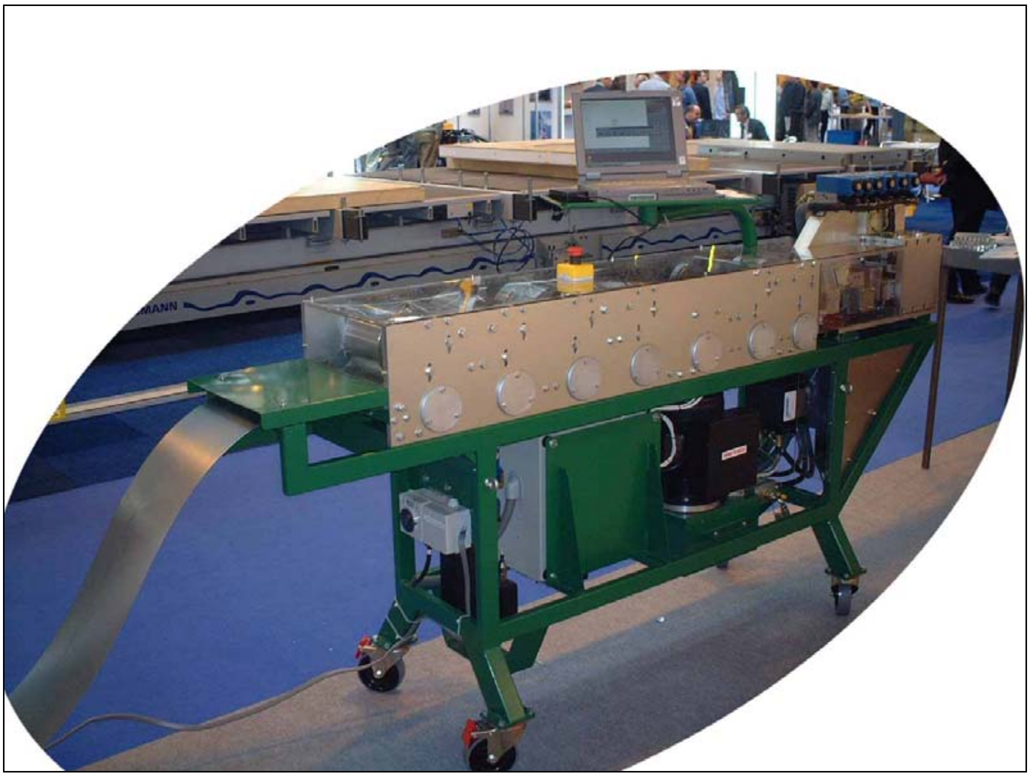

Figure 1.3 An example to Roll-Former

1.3 Components of a cold formed steel Structure

A cold formed steel panel normally consists of top and bottom track, studs, nogs(blockings), bracing and sheathing. The thicknesses of sections used in cold formed steel construction mainly vary between 0.70 mm to 2.0 mm. C-section with lips are used as studs and C-section without lips are used as tracks. Self-drilling screws or pneumatically driven steel rivets are used in frame member connections. Usage of self-drilling screws saves the assembly time in factory and at site.

Figure 1.4 OSB Sheathed light gauge steel houses

Cold formed steel houses are composed of wall, ceiling and roof panels, floor joists and roof trusses. Generally most of the walls are load bearing and the lateral loads are transferred to shear walls via roof and floor diaphragms. All the walls are covered by wooden based cladding material on the exterior. The most common materials are Oriented Strand Board (OSB) and plywood. In this research OSB cladding (Fig 1.4) is investigated as the structural member since it is the most common material and the shear wall capacity of OSB sheathed walls are conservative as compared to plywood. The most common thickness of OSB for external cladding is 11 mm (7/16 in) for residential applications. The sheathed light gauge steel wall panels provide significant shear values against lateral forces caused by earthquake and wind loads. The shear walls are anchored to foundation by hold-down anchors and the shear couple is transferred to ground. (Fig 1.5)

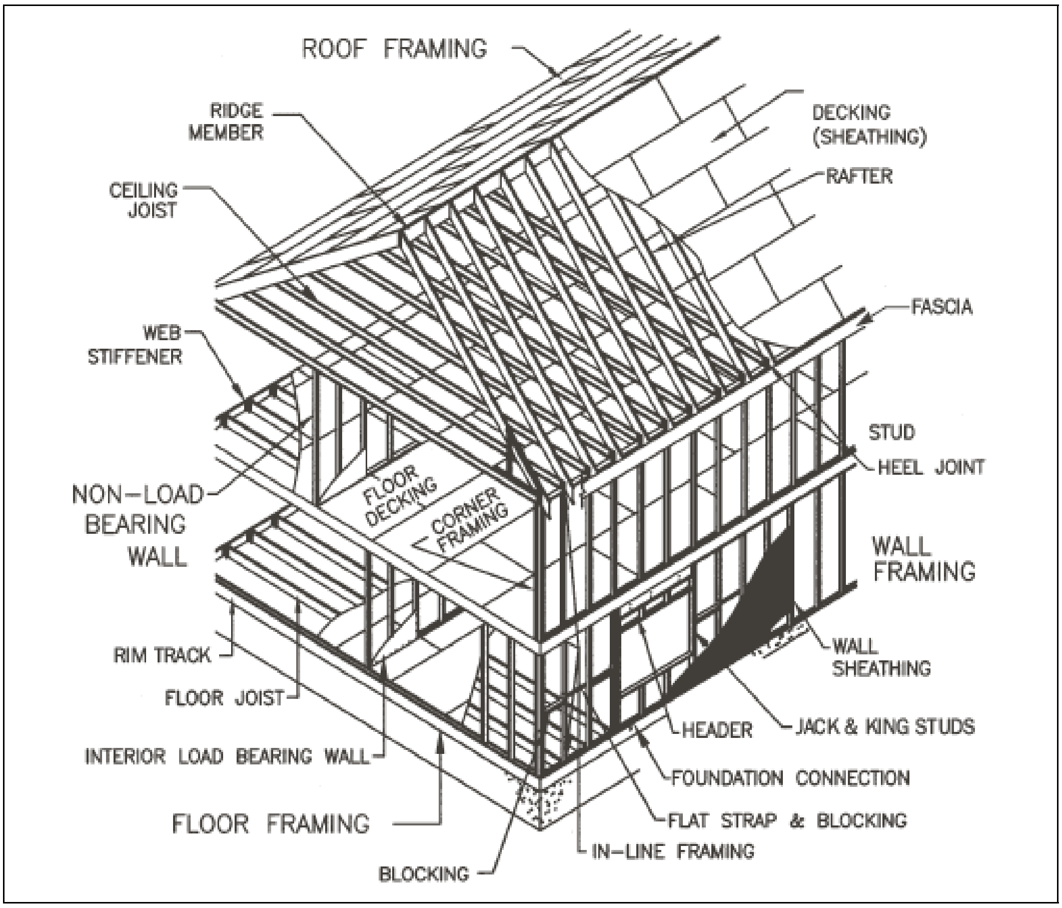

Figure 1.5 Overall view of residential steel framing and the basic components, NASFA Publication NT600

1.4 Bracing Types used in Cold Formed Steel Residential Structures

In cold formed steel residential structures, the shear walls can be designed with structural panel sheathing, flat steel sheathing, flat strap diagonal steel bracing, vertical truss wall section or combination of these methods (Fig 1.6). The frame itself has no lateral load capacity and all the capacity is achieved by bracing methods. The most common bracing methods are structural panel sheathing and flat strap diagonal steel bracing (Fig 1.6) which are evaluated in this study.

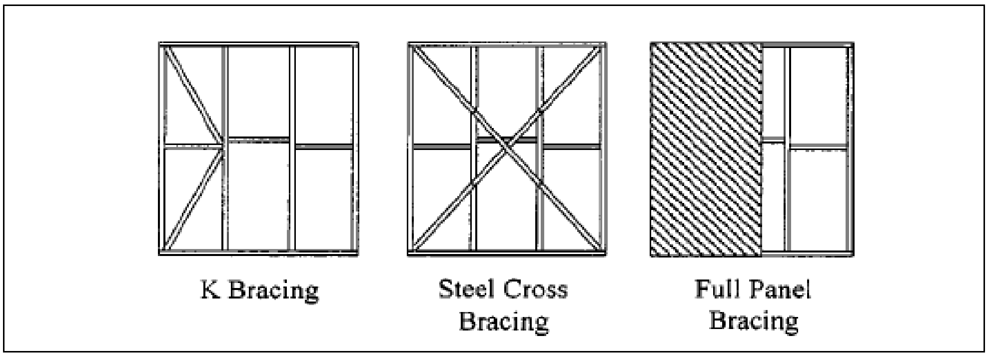

Figure 1.6 Bracing Types Used in Light Framed residential structures, Serrette (1997)

Figure 1.7 Shear Walls with X-bracing on a light gauge steel house

1.5 Load Bearing Mechanism of Cold Formed Steel Residential Structures

The main components to provide earthquake performance of a light-gauge steel house are the shear walls. A light gauge cold formed steel framed residential structure resists lateral loads caused by earthquake or wind through the use of horizontal roof and floor diaphragms and vertical shear walls. The roof and floor diaphragms take the story shear and transfer the load into the shear walls. The shear walls parallel to the force take the lateral force and transfer the load into the foundation (Fig 1.8).

The connections between the members in a cold formed steel frame are pin connections. This results in a frame with no lateral stiffness so all the lateral stiffness is provided by bracing elements. When the frame is subjected to lateral force it tends to become parallelogram but sheathed panel resist against this. This resistance is provided only if the structural panel is sufficiently connected to frame(Fig 1.9).

The walls are first designed to carry vertical loads and these walls are transformed into shear walls by addition of bracing members. These members can be flat strap diagonal bracing or structural panel sheathing. In structural sheathing, the story shear is applied to the top track of the shear wall and transferred into the sheathing panels through the screws connecting plate to the frame. Then the plate transfers the shear into the bottom plate by screws and bottom plate into the foundation by anchor bolts. An overturning moment is developed within the wall from the story shear and this moment is supported by a force couple acting to the end studs. The end studs are usually constituted from back to back studs to resist against the overturning moment and to provide enough space for the screws at the ends of the diagonal. The lateral shear force is transferred through top and bottom track and vertical shear force is transferred through end studs; it is obvious that all the shear force is transferred from frame into panel mainly through perimeter screws. The screws connecting the panel to frame along internal studs do not transfer high level of shear force and mainly inhibit out of plane buckling of panel.

In addition, the anchorage at the end studs must be designed strong enough to transfer the tension force occurred in the studs and to prevent any uplift in the frame and any bending moment in the bottom track. Mechanical or chemical anchor bolts can be used according to the forces in studs. Hold-down anchors are used at the ends of shear walls to transfer the tension force into the foundation (Fig 1.10). If the hold-down members are not used, the stud separates from the bottom track as tension load exceeds the stud-bottom track connection capacity and this disturbs the shear wall behavior and results in premature failure of the wall. Also the absence of mechanical hold-downs causes bending between the end studs and first anchor bolt. For two storey buildings, the shear walls on the upper story must be anchored to the first story walls with two hold-down members one at the bottom of upper story shear wall and one at the top of first story wall.

Several parameters affect the performance of a shear wall when subjected to the lateral force. The connection between the sheathing and framing is the most important parameter for a shear wall because all the shear force is transferred via the screws from the frame into the panel. Increasing the number of screws will increase the amount of shear force transferred from the frame into the plate, which results in an increase in the lateral load capacity of a shear wall.

In residential applications especially all the external walls are covered by single or double layers of gypsum board from internal side. This gypsum board sheathing provides additional stiffness to shear walls.

The structural panel sheathing makes contribution to the axial compression capacity of wall frames but this effect will not be taken into account in this study.

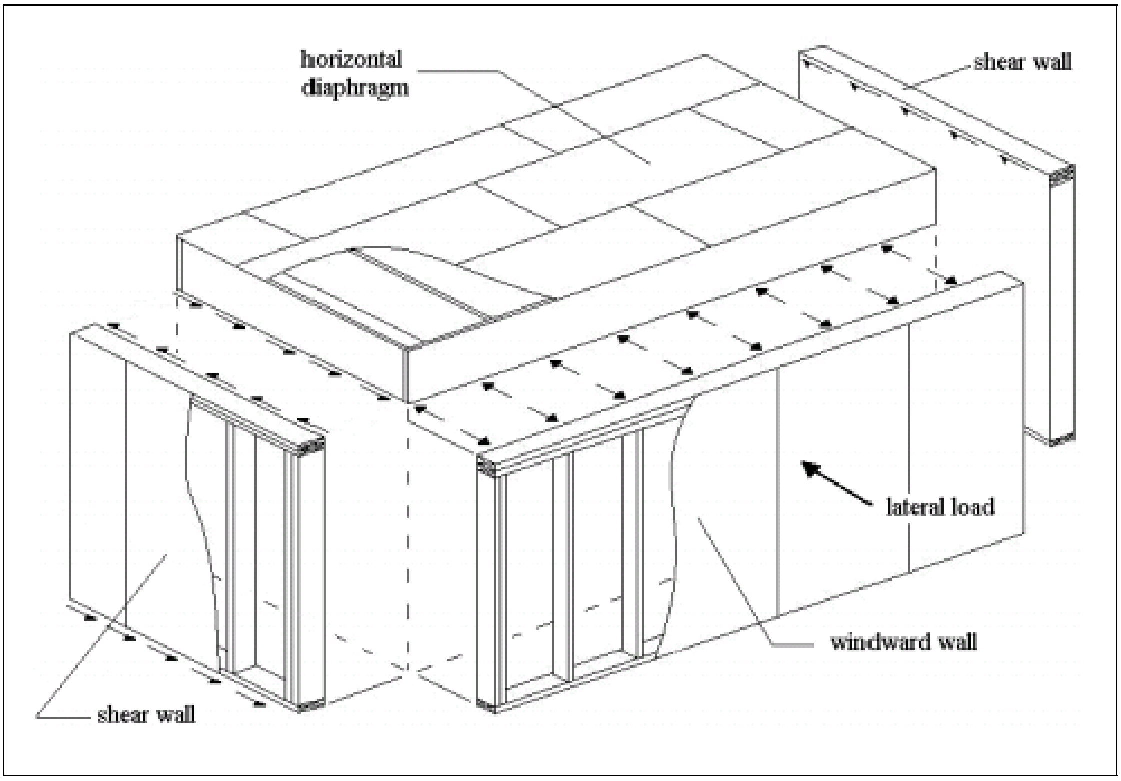

Figure 1.8 Lateral Load Carrying Mechanism of Light framed Building, Bredel (2003)

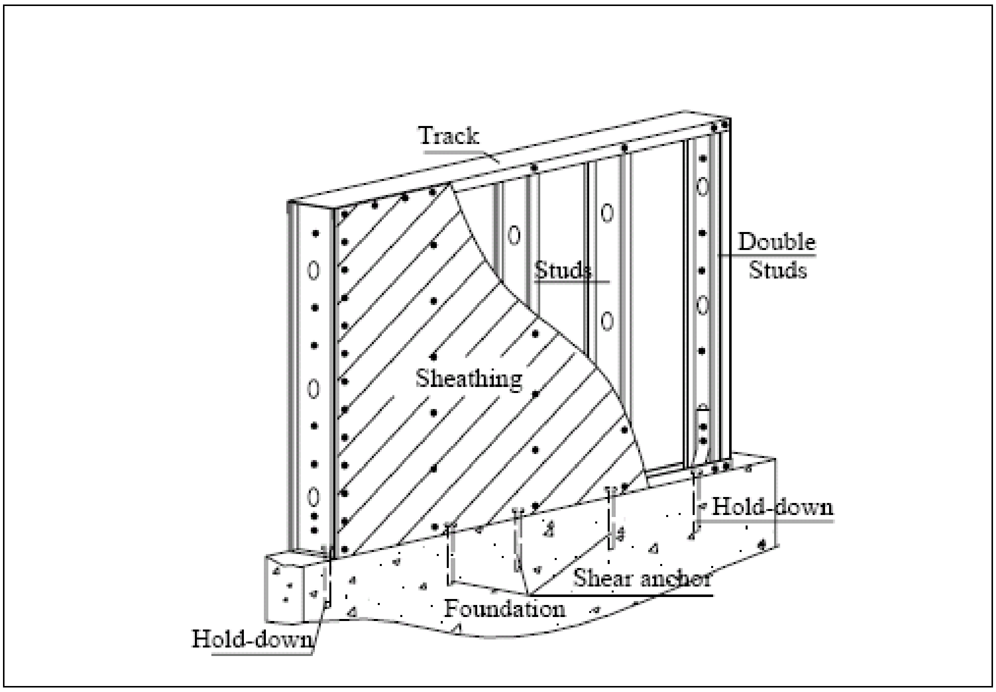

Figure 1.9 Typical Cold Formed Steel Shear Wall, Zhao and Rogers (2002)

Figure 1.10 Typical Hold-down Members

CHAPTER II

LITERATURE SURVEY

2.1 Previous Studies on Cold Formed Steel Framed Shear Walls

Experiments and analysis on cold formed steel framed shear walls is a subject that has been studied for a long time but the researches have been done mostly since 1990's.

The following results were achieved by Klippstein and Tarpy (1992) after a series of experiments.

- The results obtained from the investigation indicate that tested wall panels framed with cold-formed steel studs can substantially resist later in-plane shear loads when used as vertical shear wall diaphragms in buildings. However, certain design and construction precautions must be followed in order to take advantage of the resistance to in-plane wind and earthquake forces.
- A cold formed steel wall system with gypsum board, stucco or plywood cladding may be used with rigid or semi-rigid wall to floor attachments at both ends and/or at or between intermittent studs to act as a wind or earthquake resistant shear wall.
- A proper transfer of gravity, uplift and transverse or in-plane forces must be provided to transmit these loads to lower floor levels through floor joist as necessary to prevent local joist failure. This could be accomplished with transverse spacers between joist or other equivalent means.
- Additional or heavier end studs may be required to transmit vertical components of shear walls.
- Welding to connect the studs to the track or using self-drilling screws to connect the stud to the track are acceptable provided that welds or fasteners are designed in accordance with the current specification.

- The use of plywood sheathing, stucco or plaster increases the shear resistance of the wall panel over that with gypsum wallboard.
- Decreasing the stud spacing alone slightly increases the shear strength.
- For design purposes, a factor of safety in compliance with design philosophy of the current AISI specification is recommended.

After Klippstein and Tarpy, (1992) the failure mechanism of cold formed steel shear wall panels are described by Serrette (1997) with the results of the full scale and small-scale tests. The mode of failure of OSB sheathed panels is bend-breaking of material around screw followed by screws pulling out at the end of material.

Serrette (1997) also describes the effect of fastener spacing according to test done. In one series of tests, the screw spacing was held at 12 in. along intermediate members and decreased from 6 in. to 2 in. along the panel edges. The results showed that the wall shear strength can be significantly increased by decreasing the edge fastener spacing as shown by the comparison below (Table 2.1 and 2.2). Even though double studs were used at the ends of the wall (back-to-back with the sheathing attached only to the outer studs), for the 2 in. and 3 in. spacing Nominal Shear failure was triggered by crippling of the end studs.

Table 2.1 Nominal Static shear values of Tests done by Serrette (1997)

Test Ref. No.	Fastener Spacing (in.)	Nominal Shear (lb/ft)	Static Shear Strength Ratio
1A2/3	6/12	911	1.00
1D3/4	4/12	1412	1.55
1D5/6	3/12	1736	1.91
1D7/8	2/12	1912	2.10

Table 2.2 Nominal Cyclic shear values of Tests done by Serrette (1997)

Fastener Spacing (in)	OSB Shear Strength Ratio	Plywood Shear Strength Ratio	Average Cyclic Shear Strength Ratio
6/12	1.00	1.00	1.00
4/12	1.30	1.27	1.28
3/12	1.82	1.87	1.84
2/12	2.42	2.08	2.25

Later on Kawai, Kanno, Uno, Sakumoto (1999) suggested the story drift angle limits of light gauged steel framed houses to have safety against earthquakes encountered in Japan. The design methods of steel-framed houses were proposed based on the direct evaluation of seismic resistance by seismic response analyses. Holding the maximum story drift angle to 1/50 rad in a severe earthquake was proposed as a criterion for steel-framed houses. The Nominal Shear displacement of earthquake resistant elements in low-rise buildings is typically specified as a maximum story drift angle of 1/30 to 1/50 rad. In addition, a story angle of 1/60 rad is applied as a repairable displacement limit for houses

After these studies on displacement limit, Zhao and Rogers (2002) describes the lateral force resisting mechanism of a cold formed steel residential structure. Under seismic ground motion, horizontal inertia forces develop at the roof and floor levels as a result of the 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. These structural shear wall systems maintain the structural integrity of the building by transferring the seismic loads from the diaphragms at the roof and floor levels to the foundations.

Zhao and Rogers (2002) also explain the failure mechanism of the shear walls recorded during testing, the steel stud shear walls failed when one of the following took place: screws pulled through the wood sheathing, 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).

Fülop and Dubina (2004) suggested the drift angle for wall displacement as a maximum 1/50 rad storey drift angle limit is also suggested as acceptable during severe earthquakes. Authors also describe the failure mechanism of OSB sheathed walls observed from the experiments they did. Due to increased load bearing capacity uplift effect induced in the corner was more important. The three OSB panels placed vertically produced rigid body rotations during deformation and difference of deformation between panel and skeleton had to be accommodated by the screws. This led to important deformation of the fixing screws and relative vertical slip of one OSB panel to the other. Failure of the specimen was sudden when one vertical row of screws unzipped from the stud and both pull over the screw head, and failure of OSB margins was observed.

Fülop and Dubina (2004) also stated the differences between cyclic and monotonic performance of shear walls. Qualitatively observing comparative monotonic to cyclic curves, a reduction of strength of about 10% can be identified in case of cyclic loading. Hence, if only monotonic response is considered for an analysis (e.g. push-over analysis), the performance of the panel will be overestimated. The allowable strength is referred as the minimum of the force at storey drift angle 1/300. Differences between monotonic and cyclic values can be observed as follows. Initial rigidity is not affected, values of cyclic and monotonic tests range within a difference of less than 20%. The same can be noted for ductility, exception being in case of OSB specimens where ductility is reduced by 10–25% for cyclic results. One important observation concerns Nominal Shear load (Fu), where cyclic results are lower than monotonic ones by 5–10 % even if we consider unstabilised envelope curve.

The performance criteria were defined by authors. An important aspect of the experiments is to define acceptable damage levels and relate it to the performance objectives for the panels. Recent performance objective proposals are based on three or four generally stated goals: (1) serviceability under ordinary occupancy conditions; (2) immediate occupancy following moderate earthquakes; (3) life safety under design-basis events; (4) collapse prevention under maximum considered event.

After experimental and analytical studies, the authors concluded that the shear-resistance of wall panels is significant both in terms of rigidity and load bearing capacity, and can effectively resist lateral loads.

Failure starts at the bottom track in the anchor bolt region, therefore strengthening of the corner detail is crucial. The ideal shape of corner detail is such that uplift force is directly transmitted from the brace (or corner stud) to the anchoring bolt, without inducing bending in the bottom track. Failing to strengthen wall panel corners has important effects on the initial rigidity of the system and can be the cause of large sway and premature failure for the panel

Fülop and Dubina (2004) also describe the failure mechanisms of OSB sheathed walls. During experiments, two distinct failure mechanisms were identified for wall panels sheathed with corrugated sheeting and OSB. The lateral deformation of a panel is dependent on: (1) shear deformation of the sheeting material, (2) deformation due to corner uplift and most significantly on (3) nonlinear deformation of the connections between shear panel and skeleton. In case of wall panels sheeted with OSB, as the skeleton deforms into a parallelogram, the OSB panels have ‘rigid body’ rotation. As a consequence connections at the corners of OSB panels will be the ones which have to accommodate the largest slip and will be damaged.

Authors also derive a calculation procedure for calculating the load bearing capacity of walls. In the case of OSB panels procedure has been adopted based on the observation that such panels behave like a series of 'cells'. Therefore, in a long wall sheathed with several similar OSB panels the effect of these 'cells' is cumulative and load bearing capacity per unit length can be defined. The total capacity of a wall is then the capacity per unit length multiplied by the sheathed length of the wall. In the case of walls with openings, this value is then reduced by a factor taking into account the ratio of openings from the total wall area. A more general approach is to relate sheathed-to-frame connector slip to the lateral displacement of the wall panel. In this way a panel can be analyzed under increasing lateral displacement and based on the individual connector properties, the load bearing capacity can be calculated. As the number of connectors is usually large, it is convenient to perform the analysis by computer. As experimental curves are non-linear from the beginning, the 'elastic' design capacity of the panels can be defined only in a conventional way. Obviously, any assumption of 'elastic' design limit, like in case of the ECCS Recommendation, is to be related to a tolerable deformation of the relevant group of connectors subjected to the highest forces. Therefore the 'design' capacity of the panel is mainly based on serviceability than strength criteria. Consequently, there will be an important strength reserve beyond any design limit considered, due to load bearing capacity of the remaining connectors when the few most damaged ones have excessive deformations or fail.

Tian, Wang and Lu (2003) describes the parts and types of the cold formed steel wall panels. Wall panel normally consists of top/bottom tracks, stud, bracing and connections, which can be assembled together on site or manufactured in the factory. With the latter method, intensive labour can be saved on site, and hence the construction time may be shortened. However, transportation of the assembled panels may pose a potential problem due to weak racking stiffness/strength.

From the loading point of view, there are typically two types of wall: load-bearing wall and partition wall. For a partition wall, no special requirement on the load carrying capacity of the wall frame is needed. A load bearing wall frame will not only support vertical load but also resist racking load caused by wind, earthquake or even transportation. The gauge of a cold-formed steel section is in general very thin and hence the racking resistance of a wall frame will be unacceptably low if no bracing is used. If the racking load carrying capacity is needed for a wall frame, bracing with boards, cold-formed steel sections or flat straps is desirable

Authors also emphasize the effect of bracing on vertical load capacity of walls. Bracing of a frame can significantly increase its capability to carry the vertical as well as lateral load. Miller and Pekoz [18, 19] studied the effect of sheathing on the vertical load capacity of cold-formed steel studs, whilst Telue and Mahendran [28] examined the behaviour of cold-formed steel wall frames braced with plasterboards. It was found that the vertical load carrying capacity of wall studs increases significantly after bracing with boards.

It was observed from the experiments of frames braced with one OSB board done by authors that the net lateral deflection from the damage load (10 kN) to the maximum load was large, about 40 mm. This means that the frame was damaged gradually, accompanied with considerable deformation. In the final stage, as the deflection increased, the board was almost completely disconnected from the left and right track (Figure 2.1), and the load dropped sharply. When the lateral deflection reached about 80 mm, the test was stopped. It was then observed that on the middle stud, the screw connections were still intact, suggesting that the shear force was mainly passed by the sidetrack connections to the board. Upon complete unloading, about 50 mm residual deflection was measured. When the board was removed and the frame carefully examined, it was found that all local buckling had been recovered and there was no significant damage on the track and the stud, except local areas around screw connections.

Figure 2.1 Deformation on OSB and screw pulling-out from OSB at tests done by Tian, Wang and Lu (2003)

The authors statements about the flat strap X-bracing are that Strap width has a relatively small effect on racking failure load and strap force, but great influence on the deflection. For a 10 times increase in strap width, the racking failure load only increases by about 10%, whereas the deflection is reduced by about 5 times. This indicates that when the strap width is sufficiently large, the frame is stiff with small deflection, and its members fail mainly by compression. For such cases, the section design of a frame member should be based on compression, and the section design of bracing straps should be based on frame deflection and strength of the strap material. For frames braced with boards, the racking resistance is governed by board properties. By increasing board thickness, complemented with optimized screw spacing, the racking resistance can be enhanced. For frames braced with steel straps, according to the observed failure modes, the racking resistance enhancement can be achieved by considering different bracing methods, individual member sections, strap–frame connections, amongst others. For bracing methods, generally speaking, the larger the ratio of bracing unit width to frame length W/L is, the better. For individual member section, frames consisting of sidetracks made with lipped channel sections can carry larger racking load in comparison with frames using plain channel sections as sidetracks. For strap–frame connection, by increasing the number of rivets or modifying the connection type, the frame racking resistance can be improved and immature connection failures prevented. Furthermore, by increasing the strap width or cross-sectional area, the stiffness and racking behavior of the frame can be much improved.

The cross-sectional area of a strap significantly affects the deflection (stiffness) of a frame, but has little influence on its racking load capacity. Amongst the bracing methods studied, frames with 2 side X-straps have the best racking performance. By careful selection of the bracing method, individual member section, strap geometry and connection method, the frame performance can be optimized (Table 2.3).

Table 2.3 Racking Test Results and Failure Modes by Tian et al (2002).

Bracing type	Loading steps	Failure mode
No bracing	1	Plastic deformation around corners
OSB-1 sides	1	Screws
OSB-1 side	3	Screws
CPB-1 side	1	Screws
CPB-1 side	3	Screws
1X-2 sides	1	Top-left rivets
1X-2 sides	3	Left track
1X-1 side	1	Bottom-right rivets
2X-2 sides	1	Left track
2X-2 sides	3	Left track

The authors reach the below results from their experimental and analytical results.

- A frame without any bracing has a racking strength less than 5% of that of the same frame with bracing.

- Strap width has relatively small influence on racking resistance, but affects frame stiffness significantly. The lateral deflection of the frame decreases dramatically with increasing strap width.
- The performance of a frame under racking depends on several key factors, including individual member section design, bracing method, connection method, and strap size. All these aspects need to be carefully examined if the racking performance of the frame is to be optimized.
- For frames braced with boards, failure occurs on the board near screw connections. If the board thickness increases or screw spacing decreases, it is possible to increase the racking strength.
- Bracing with a large ratio of bracing unit width to frame length is preferable.

Fülop and Dubina (2006) performed experimental studies on Oriented Strand Board to Steel connection Strength. A connection typology used in the wall panel test was the one connecting OSB to the steel skeleton. In order to test this typology of connections, specimens as presented in Figure 2.2 were prepared. The testing of these specimens yielded very inhomogeneous results (Figure 2.3), depending on the direction and density of fibers in the vicinity of the screw and between the screw and the margin of the OSB panel. No generalizing conclusion can be drawn from these experiments; only that OSB connections possess less ductility, this being the most likely reason for the less ductile failure of the wall panels sheeted with OSB.
No generalizing conclusion can be drawn regarding OSB sheeted wall panels, due to the low homogeneity of the OSB-to steel connection tests, only that the lower ductility of the connection is responsible for the nonductile failure of these wall panels.

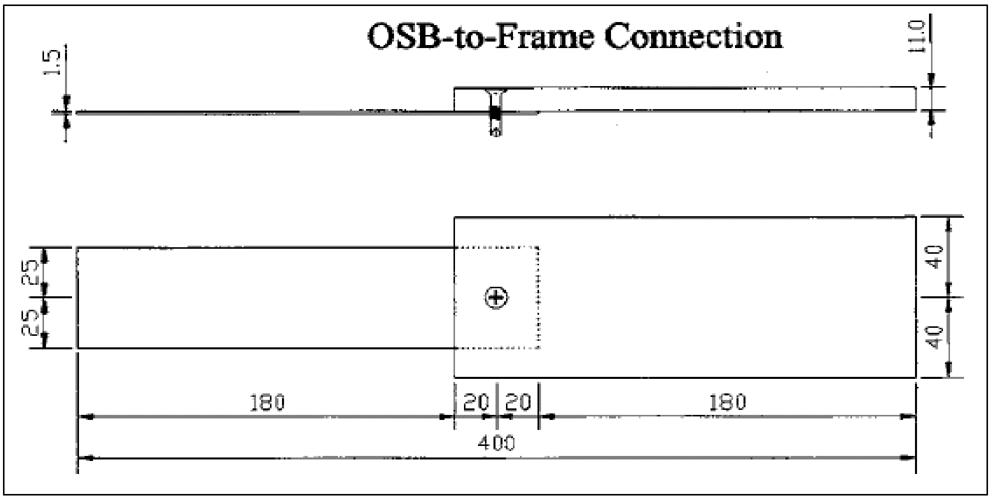

Figure 2.2. Drawing of tested OSB to Steel skeleton connections by Fülop and Dubina (2006)

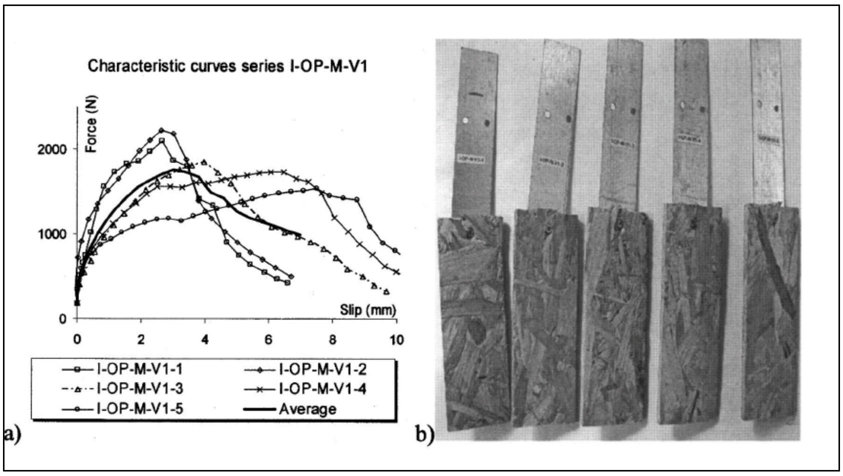

Figure 2.3 Experimental curves and failure modes of OSB to steel connections by Fülop and Dubina (2006)

CHAPTER III

DESCRIPTION OF OSB SHEATHED SHEAR WALLS

3.1. Materials Used in a Cold Formed Steel Shear Wall

3.1.1. Oriented Strand board (OSB)

Oriented strand board (OSB) is a performance-rated structural wood-based panel engineered, uniformity, strength, versatility and workability. It is utilized internationally in a wide array of applications including residential and commercial construction (Figure 3.1) and renovation, packaging/crating, furniture and shelving, and do-it-yourself projects. Because it is engineered, OSB can be custom manufactured to meet specific requirements in thickness, density, panel size, surface texture, strength and rigidity. This engineering process makes OSB the most widely accepted and preferred structural panel among architects, specifiers and contractors. (Structural Board Association, 2002)

Mechanical Properties of OSB:
Minimum Modulus of Elasticity (Parallel):3500 MPa
Minimum Modulus of Elasticity (Perpendicular):1500 MPa
Dowel Bearing Strength: 41.4 MPA (APA, The Engineered Wood Association Form no: TT-020, 2002)

Figure 3.1 OSB Sheathed Light Gauge Steel House.

3.1.2. Steel

Steel for cold forming is mostly slitted as coils in required width from big galvanized steel coils and it is ready to be used by roll- formers. (Figure 3.2)
Minimum Yield Strength: Fy =228 MPa (33 ksi)
Minimum Tensile Strength: Fu=310 MPa (45 ksi)
All Cold Formed steel section Capacities are calculated according to the AISI 2001 specification.

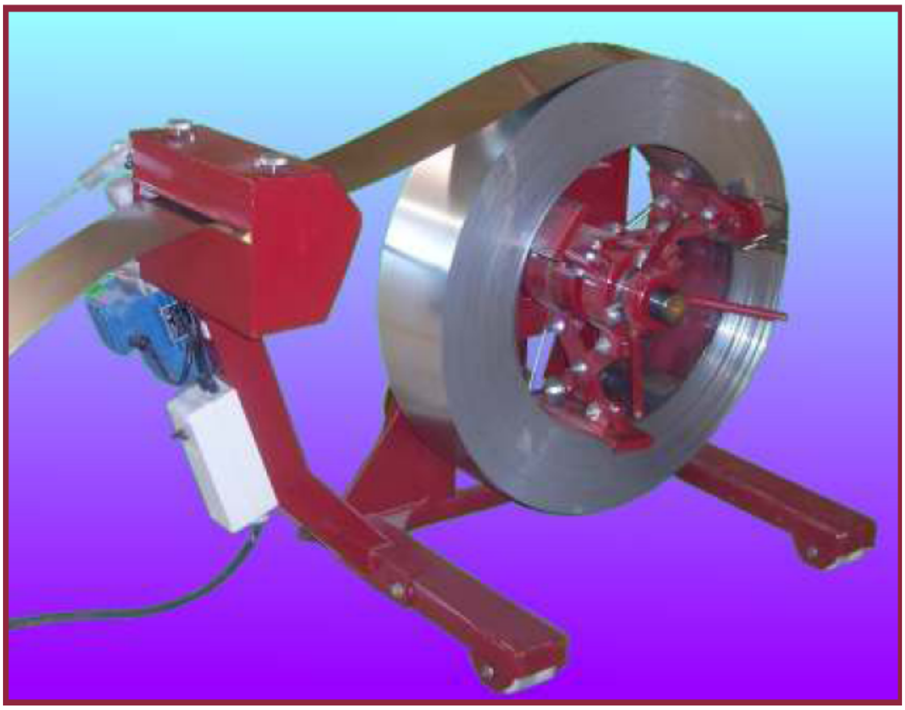

Figure 3.2 Galvanized Steel coil on decoiler of

3.1.3. Screws

Screws with different types of heads can be used in construction according to the requirements of design and material (Figure 3.3). In analyses the values for the screws below are used because these are the ones used in experiments and specified in codes.

For framing No. 8 x 5/8 in. wafer head, self drilling. (D=4.17 mm)

For OSB sheathing No. 8 x 1 in. flat head, sharp point, self drilling. (D=4.17 mm)

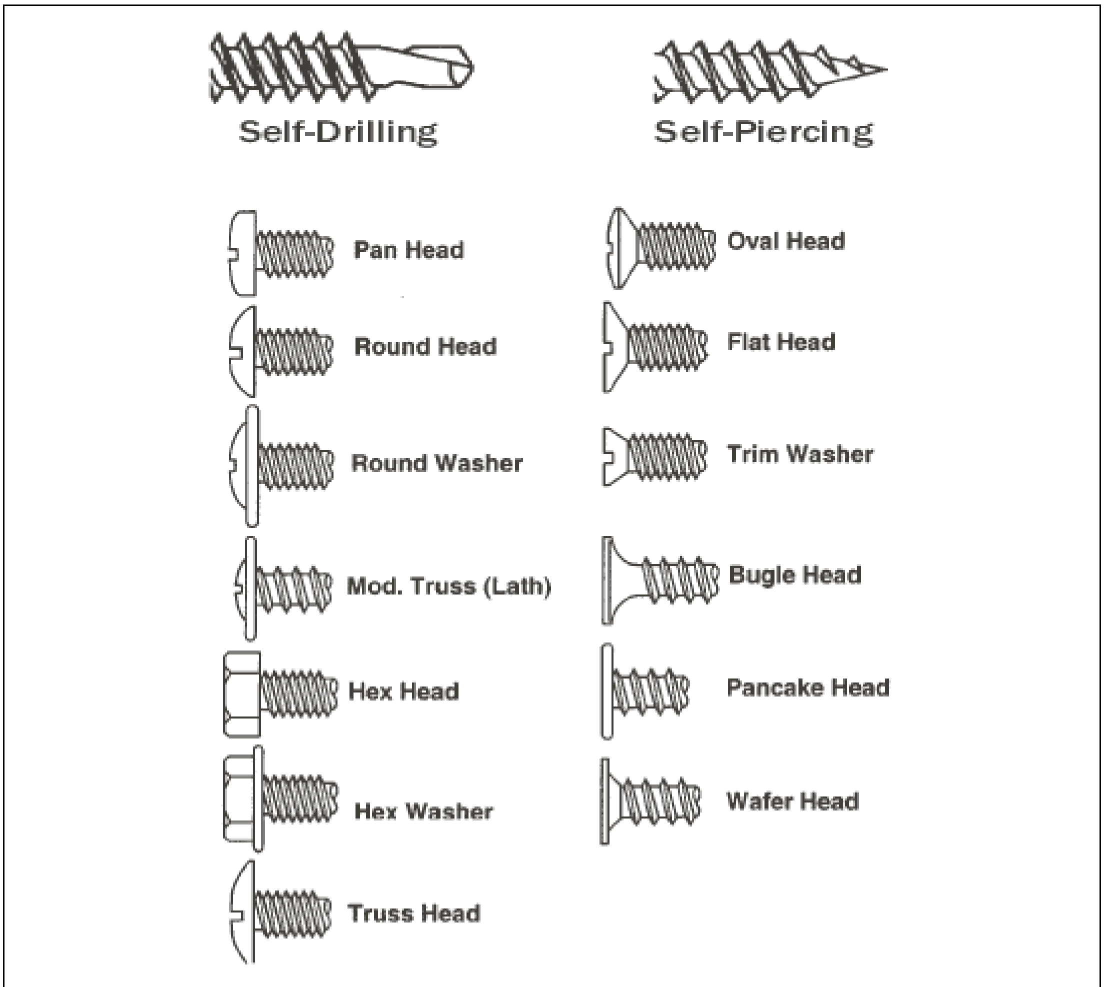

Figure 3.3 Head and Types of screws, Prescriptive Method (2000)

3.2. Analytical Computer Model

A frame of 244cm x 244cm (8ft x 8ft) is modelled in SAP2000 software. All frame member connections are modelled as pin connection and supports as simple support. The frame members are divided into pieces according to screw spacing and the OSB plates are meshed according to screw spacing too (Figure 3.4). Then for the analysis of constraint-defined model, constraints are defined to corresponding joints of frame and shell in 3-translation degree of freedom and moments are released since screw connections do not carry any moment. Different constraints are defined to each joint.

The load is applied to upper track at the left corner (Figure 3.5). The model is analyzed and area elements joint forces are exported to excel and the forces at each joint are summed by an ms-excel macro. The maximum joint forces are determined at two directions (F1 and F3) and the value is compared with the OSB-frame screw connection strength. If the joint forces are less than the strength, the applied force is increased until the first joint reaches the capacity. Then the corresponding applied load to frame is accepted as the yield capacity of the constraint defined model and this load is divided to frame length to find the capacity per length. (KN/m). The results of this analysis is given below and named as constraint defined model yield capacity.

The results of this model give the yield shear capacity of the frame, but not the Nominal Shear capacity. To reach the Nominal Shear capacity screw connections should be modelled in such a way that they should carry load until their yield capacity and then carry the same load. To have such behaviour screw connections are modelled as nonlinear-link element in SAP2000 software. The nonlinear link properties are defined according to the material properties of OSB and the links are drawn between panel nodes and frame nodes. Then the model is analyzed nonlinearly and the yield capacity of the model is determined when the first link reaches its yield capacity, the results of these analyses are given below and named as Yield Capacity of Link defined model. These results are compared with the result obtained from constraint defined model. The yield capacities are nearly the same for 5 cm, 7.5 cm 10 cm and 15 cm screw spacing model.

To reach the Nominal Shear capacity the load applied is increased and the yielded joints are observed. As expected first the joints on perimeter yielded. The yielded joints and deflection limit are accepted as Nominal Shear capacity criteria of frames for aspect ratio equal to one. Deflection limit is taken as L/240 from International Building Code Table 1603.4 (Table 3.1). When the frame reaches its deflection limit the corresponding applied load is assumed as nominal wall shear capacity.

Table 3.1 Deflection Limits for Structural Members in International Building Code 2003.

CONSTRUCTION	*L*	*S* or *W* [f]	*D* + *L* [d,g]
Roof members: [e]			
Supporting plaster ceiling	*l*/360	*l*/360	*l*/240
Supporting nonplaster ceiling	*l*/240	*l*/240	*l*/180
Not supporting ceiling	*l*/180	*l*/180	*l*/120
Floor members	*l*/360	—	*l*/240
Exterior walls and interior partitions:			
With brittle finishes	—	*l*/240	—
With flexible finishes	—	*l*/120	—
Farm buildings	—	—	*l*/180
Greenhouses	—	—	*l*/120

D=Dead load

S=Snow load

L=Live load, except roof live load, including any permitted live load reduction

W=Load due to wind pressure

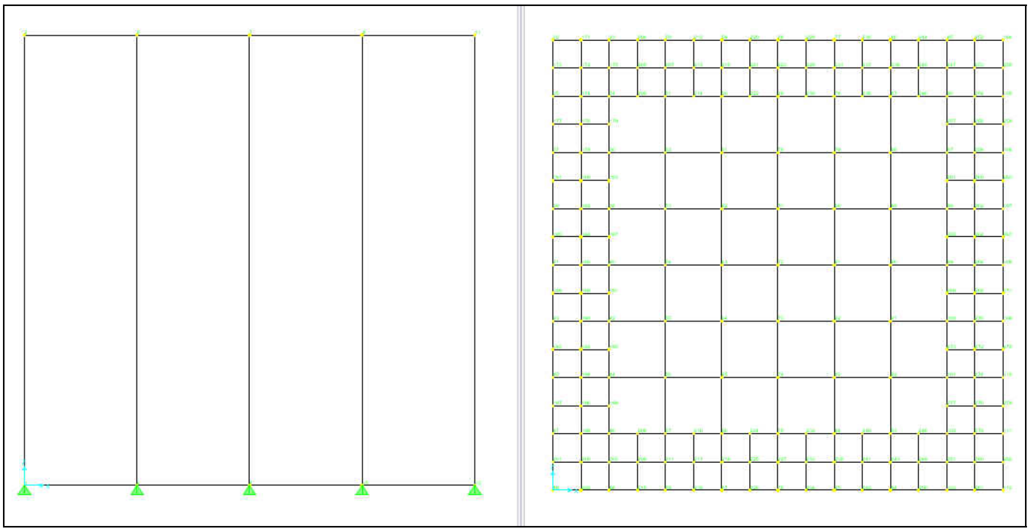

Figure 3.4 General Description of Steel Frame and Meshed shell used in numerical model.

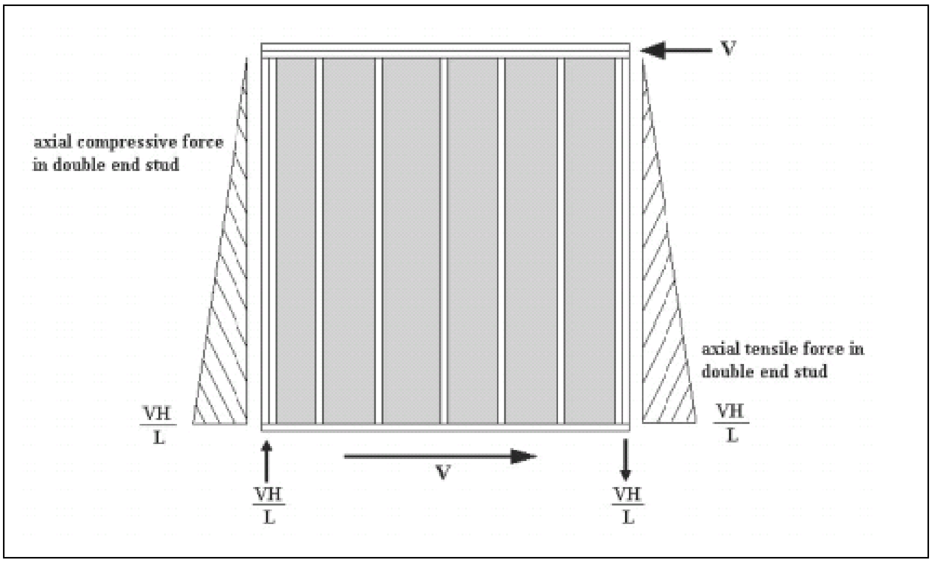

Figure 3.5 Free-Body Diagram of the Shear Walls, Bredel (2003)

The results of computer analyses are compared with the values of shear wall design guide and international building code tables which are same.

3.3 OSB-Steel Frame Connection Shear Strength

The connection shear capacity between the OSB and steel member is calculated by different methods and formulas and the average of these results is taken as the OSB-Steel frame connection shear strength.

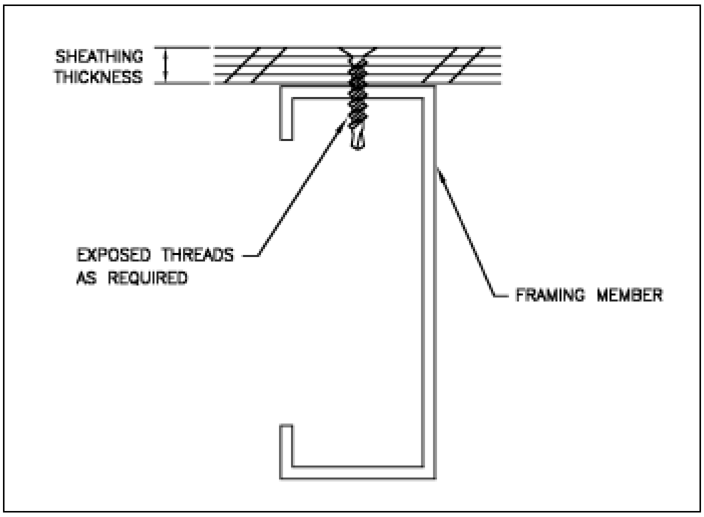

Figure 3.6 Oriented Strand Board and steel stud screw connection, NASFA Publication NT600.

3.3.1. Calculation 1

Dowel bearing strength of OSB has been measured in a limited number of tests conducted at APA (2002). Average results exceeded 6.000 psi.

6000 psi=41.37 MPa

Bearing strength of OSB:

$V = t \times d \times \sigma$ (3.1)

V=11.1x4.2x41.37=1929 N=1.93 kN

V: Connection shear Capacity

t: Thickness of OSB

d: Screw Diameter

σ: Dowel Bearing Strength of OSB

3.3.2. Calculation 2

The ultimate Shear capacity for OSB to 0.84 mm thick steel by #8 self-drilling tapping screws with screw bugle heads is 1.73 kN. (U.S. Department of Housing and Urban Development, 2003)

The ultimate Shear capacity for OSB to 1.38 mm thick steel by #8 self-drilling tapping screws with screw bugle heads is 2.07 kN. (U.S. Department of Housing and Urban Development, 2003)

3.3.3. Calculation 3

The ultimate Shear capacities for 18mm OSB to 1.38 mm thick steel by #8 self-drilling tapping screws with screw bugle heads with 12.5 mm screw edge distance are 2.67 kN and 2.75 kN parallel to grain and perpendicular to grain respectively. Since shear strength of OSB governs the test, interpolation to 11.1mm can be done easily. (U.S. Department of Housing and Urban Development, 1999)

$$V = \sigma \times t1 / t2 \qquad (3.2)$$

V: Connection shear Capacity

t1: Thickness of OSB that capacity is to be found

t2: Thickness of OSB that capacity is known

V=2.68*11.1/18=1.65 kN parallel to grain

V=2.75*11.1/18=1.70 kN perpendicular to grain

3.3.4. Calculation 4

The average value for OSB to steel connection is found as 1.8 kN in tests done by Fülop and Dubina (2006)

OSB-Steel Connection Shear Strength is assumed as the average of the four references, 1.80 kN for analytical calculations.

3.4 Comparison of Cyclic vs. Static Tests in Shear Wall Design Guide

The difference between cyclic tests and static tests are defined below. All the graphs and explanations are taken from Nasfa Publication RG-8904, "Shear Wall Design Guide". The test results show the difference between cyclic tests and static tests.

Table 3.3 shows the static test results which are used as "nominal shear values for wind forces in pounds per foot for shear walls framed with cold-formed steel studs" in IBC 2003.

Table 3.4 shows the cyclic test results which are used as "nominal shear values for seismic forces in pounds per foot for shear walls framed with cold-formed steel studs" in IBC 2003.

As expected, the cyclic test results were somewhat lower than static test results for walls of similar construction. For walls with OSB sheathing on one side, the ratio of cyclic strength to static strength varied somewhat with the fastener spacing (Table 3.2).

Table 3.2 Ratio of Cyclic Strength to Static Strength according to screw spacing

Screw Spacing (in)	Ratio of Cyclic Strength to Static Strength
6/12	0.77
4/12	0.65
3/12	0.89
2/12	0.73

The overall average was 0.76.

Table 3.3 Nominal shear Strength of Walls on Static Tests by Serrette (1996)

Ref. No.	Sheathing Thickness and Type	Sheathing Orientation	Screw Spacing (in)	Nominal Shear (lb/ft)
1A6, 1A7	15/32" 4-ply plywood	V	6/12	1062
1A2, 1A3	7/16" OSB	V	6/12	911
1A5, 1A6	7/16" OSB	H	6/12	1022
1E1, 1E2	7/16" OSB	H	6/12	1025
1D3, 1D4	7/16" OSB	V	4/12	1412
1D5, 1D6	7/16" OSB	V	3/12	1736
1D7, 1D8	7/16" OSB	V	2/12	1912
1F1, 1F2	7/16" OSB 1/2" GWB	V	6/12 7/7	1216
1F3, 1F4	7/16" OSB 1/2" GWB	V	4/12 7/7	1560
1F5, 1F6	7/16" OSB 1/2" GWB	V	2/12 7/7	1884
2A1, 1A3	7/16" OSB 1/2" GWB	H	7/7 7/7	583
2A2, 2A4	7/16" OSB 1/2" GWB	H	4/4 4/4	849

Notes:
1. See Serrette (1996) for further details.
2. Nominal (ultimate) shears listed are average of two tests.
3. Sheathing on one side only except for tests with GWB.
Horizontal strap, 0.033 x 1.5 in., at midheight of studs.
V indicates sheathing parallel to framing, H indicates sheathing perpendicular.
4. Screw spacing 6/12 indicates 6 in. on panel edges, 12 in. on intermediate members.
Screws for plywood and OSB were No. 8 x 1in. self drilling, flat head with counter-sinking nibs under the head, type 17 point, coarse high thread, zinc plated.
Screws for GWB were No. 6 x 1-1/4 in. self drilling, bugle head, type S point.
5. Studs were 3-1/2 x 1-5/8 x 0.033 in. spaced at 24 in., A653 Grade 33 steel.
Double studs (back-to-back) were used at the ends of the wall.
Track was 3-1/2 x 1-1/4 x 0.033 in., top and bottom, A653 Grade 33 steel.
Thicknesses refer to minimum metal base thickness.
6. For design, divide by a safety factor (ASD) or multiply by a reduction factor (LRFD).

Table 3.4 Nominal shear Strength of Walls on Cyclic Tests by Serrette (1996)

Ref. No.	Sheathing Thickness and Type	Sheathing Orientation n	Screw Spacing (in)	Nominal Shear (lb/ft)
OSB1, OSB2	7/16" OSB	V	6/12	700
OSB3,OSB4	7/16" OSB	V	4/12	912
OSB5,OSB6	7/16" OSB	V	3/12	1275
OSB7,OSB8	7/16" OSB	V	2/12	1700
PLY1,PLY2	15/32" 4-ply plywood	V	6/12	780
PLY3, PLY4	15/32" 4-ply plywood	V	4/12	988
PLY5,PLY 6	15/32" 4-ply plywood	V	3/12	1462
PLY7,PLY 8	15/32" 4-ply plywood	V	2/12	1625

Notes:
1. See Serrette (1996) for further details.
2. Nominal (ultimate) shears listed are average of two tests. Each is based on average values for last stable hysterectic loop.
3. Sheathing on one side only.
Horizontal strap, 0.033 x 1.5 in., at midheight of studs.
V indicates sheathing parallel to framing.
4. Screw spacing 6/12 indicates 6 in. on panel edges, 12 in. on intermediate members.
Screws for plywood and OSB were No. 8 x 1 in. self drilling, flat head with counter-sinking nibs under the head, type 17 point, coarse high thread, zinc plated.
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5. Studs were 3-1/2 x 1-5/8 x 0.033 in. spaced at 24 in., A653 Grade 33 steel.
Double studs (back-to-back) were used at the ends of the wall.
Track was 3-1/2 x 1-1/4 x 0.033 in., top and bottom, A653 Grade 33 steel.
Thicknesses refer to minimum metal base thickness.
6. For design, divide by a safety factor (ASD) or multiply by a reduction factor (LRFD).

CHAPTER IV

NUMERICAL ANALYSIS OF OSB SHEATHED SHEAR WALLS

4.1 Analytical Models of Shear Walls and Analysis Results

Four different computer models have been analyzed and the properties of the models and analysis results are described below. To check the analysis results the models are determined according to Shear wall design guide, 1998 by American Iron and Steel Institute and IBC 2003.

4.1.1. MODEL 1-Fastener Spacing at Panel Edges: 152 mm (6 in)

Dimensions: 2.44m x 2.44m

Screw Spacing: 152 mm on centre at perimeter and 305 mm on centre in field.

Number of Constraints or nonlinear-links defined: 85 (Figure 4.1).

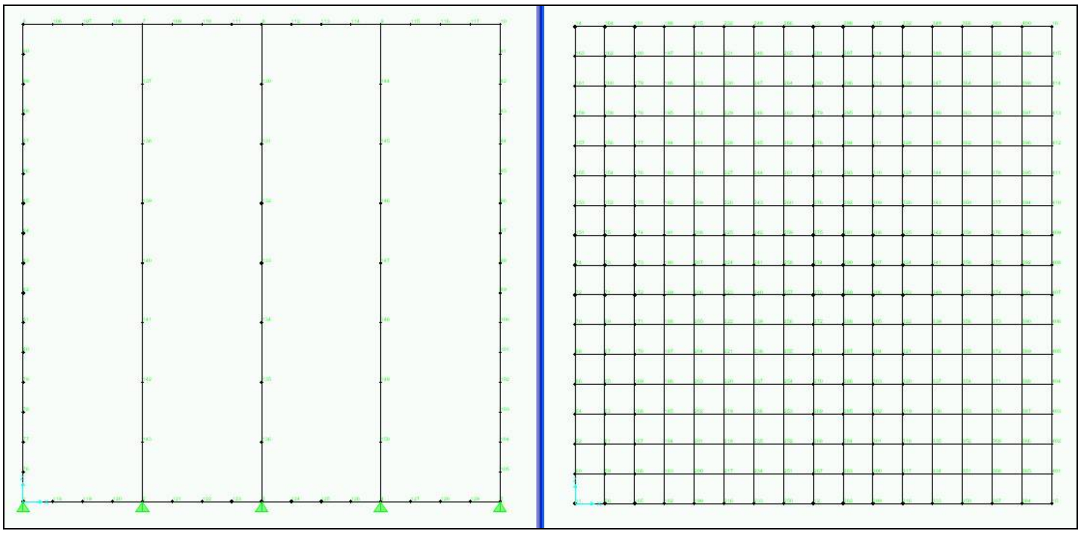

Figure 4.1 Steel Frame and Meshed Shell of Sap2000 model

Geometric and material properties of sections used in models are given in (Table 4.1)

Table 4.1 Geometric and Material properties of Sections used in Analysis of shear wall with screw spacing 152 mm on centre at perimeter

	Web (mm)	Flange (mm)	Lip (mm)	Thickness (mm)	Fy / Fu (MPa)
Studs	89	41.2	12.5	0.84	228 /310
Tracks	89	32	0	0.84	228 /310

- End Studs are Back to Back

Unit wall shear capacities are obtained from analyses and values are divided by IBC values for wind forces to compare the results. The results are defined as Normalized values according to IBC values for wind forces (Table 4.2).

Table 4.2 Analysis Results for Shear Wall with screw spacing 152 mm on centre at perimeter

	Wall Shear Capacity (kN/m)	Normalized Values According to IBC values for wind forces
IBC values for wind forces Capacity	13.27	1.0
IBC values for seismic forces Capacity	10.21	0.77
Constraint Defined Model Yield Capacity	9.68	0.73
Link Defined Model Yield Capacity	10.08	0.76
Link Defined Model Nominal Shear Capacity	15.20	1.15

Lateral force is increased up to first nonlinear link yielded and the corresponding displacement is obtained as yield displacement. After yield point lateral force is increased incrementally as other links are yielded up to the frame become unstable and the corresponding displacements are read. By this way, lateral capacity vs. displacement curve is obtained. (Figure 4.2)

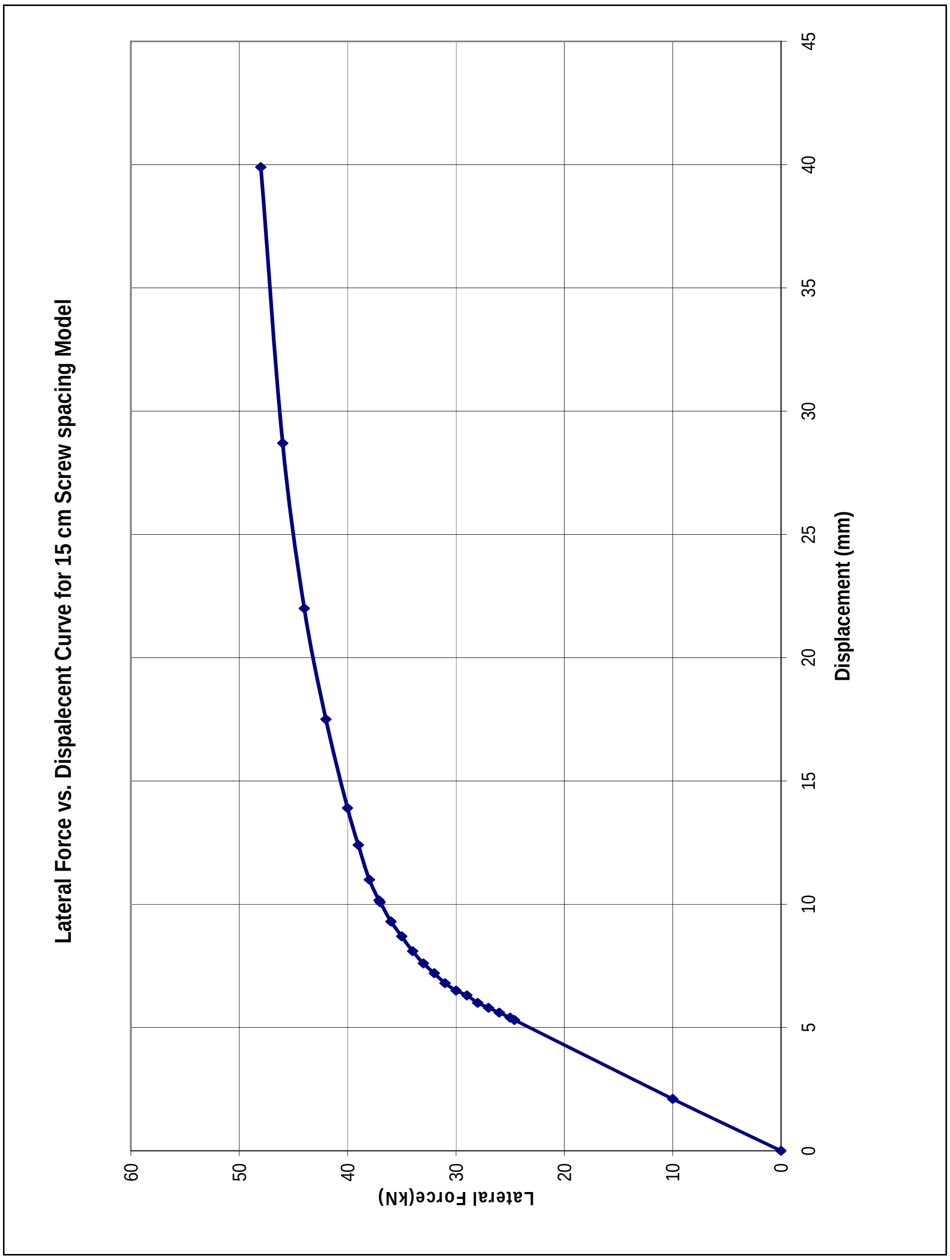

Figure 4.2 Lateral Force vs. Displacement Curve for 15 cm Screw Spacing Model

4.1.2. MODEL 2-Fastener Spacing at Panel Edges: 102 mm (4 in)

Dimensions: 2.44m x 2.44m

Screw Spacing: 102 mm on centre at perimeter and 305 mm on centre in field.

Number of Constraints or nonlinear-links defined**:** 117 (Figure 4.3)

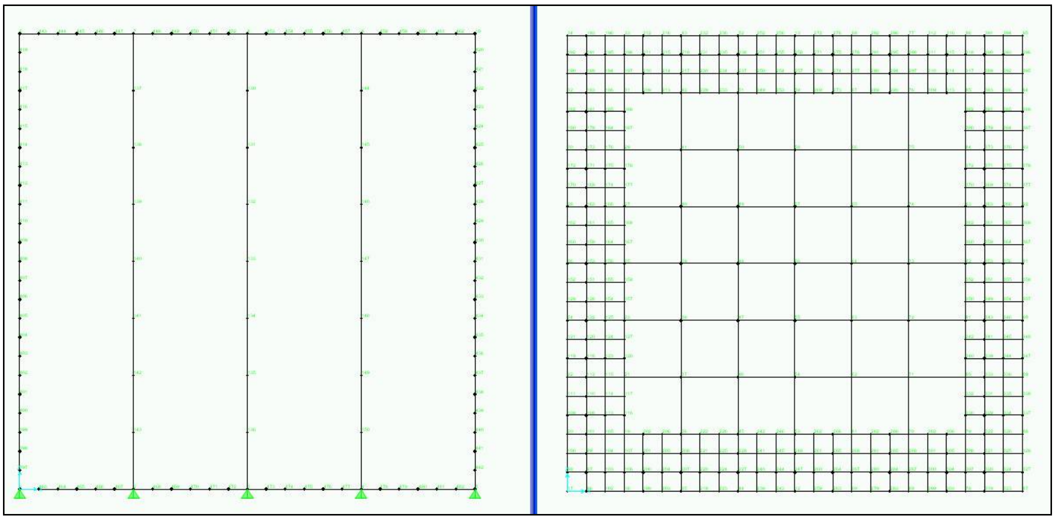

Figure 4.3 Steel Frame and Meshed Shell of Sap2000 model

Geometric and material properties of sections used in models are given in (Table 4.3)

Table 4.3 Geometric and Material properties of Sections used in Analysis of shear wall with screw spacing 102 mm on centre at perimeter

	Web (mm)	Flange(mm)	Lip(mm)	Thickness(mm)	Fy / Fu (MPa)
Studs	89	41.2	12.5	0.84	228 /310
Tracks	89	32	0	0.84	228 /310

- End Studs are Back to Back

Unit wall shear capacities are obtained from analyses and values are divided by IBC values for wind forces to compare the results. The results are defined as Normalized values according to IBC values for wind forces (Table 4.4).

Table 4.4 Analysis Results for Shear Wall with screw spacing 102 mm on centre at perimeter

	Wall Shear Capacity (kN/m)	Normalized Values According to IBC values for wind forces Capacities
IBC values for wind forces Capacity	20.57	1.0
IBC values for seismic forces Capacity	13.35	0.65
Constraint Defined Model Yield Capacity	13.30	0.65
Link Defined Model Yield Capacity	13.73	0.67
Link Defined Model Nominal Shear Capacity	20.98	1.02

Lateral force is increased up to first nonlinear link yielded and the corresponding displacement is obtained as yield displacement. After yield point lateral force is increased incrementally as other links are yielded up to the frame become unstable and the corresponding displacements are read. By this way, lateral capacity vs. displacement curve is obtained. (Figure 4.4)

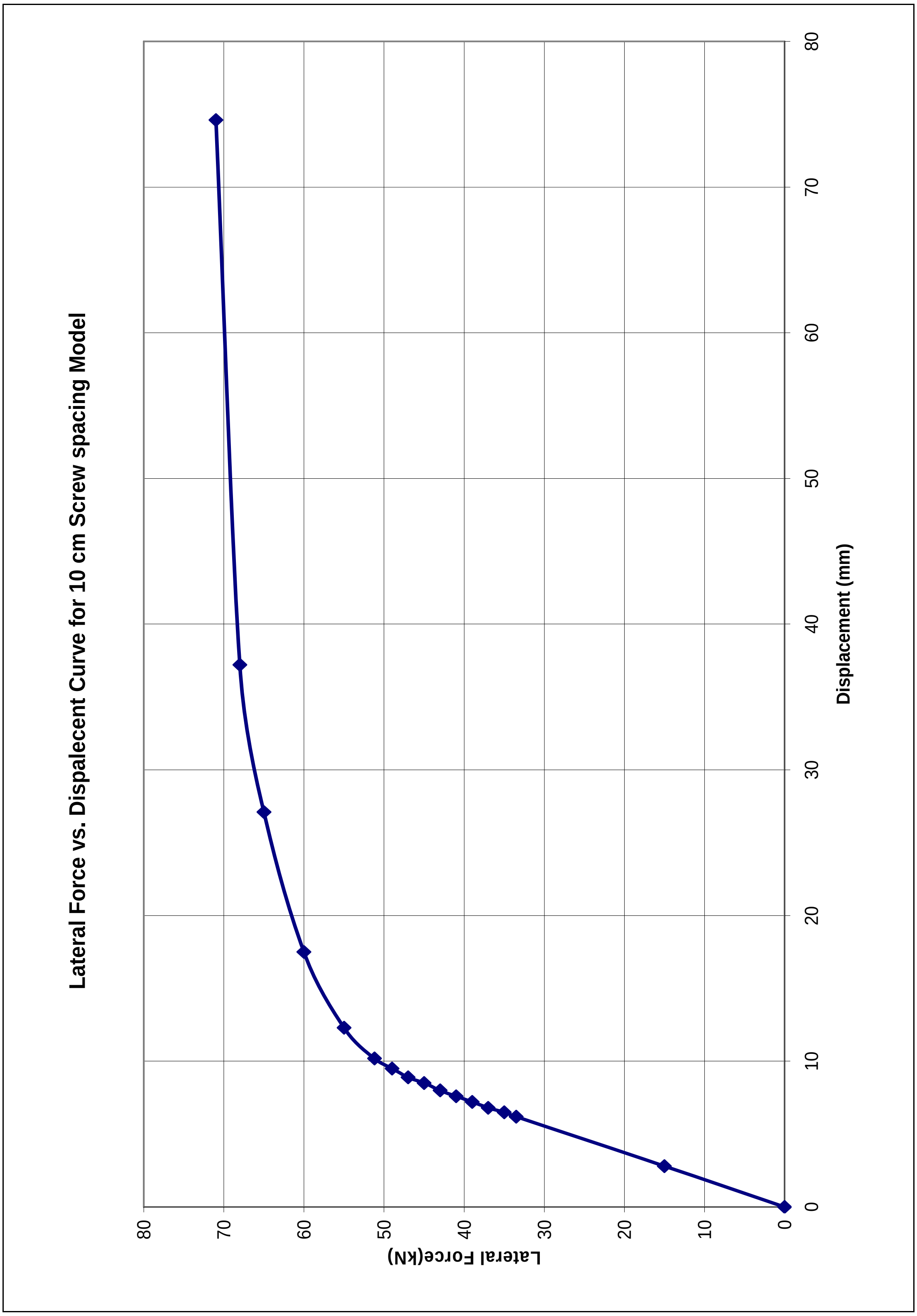

Figure 4.4 Lateral Force vs. Displacement Curve for 10 cm Screw Spacing Model

4.1.3. MODEL 3-Fastener Spacing at Panel Edges: 76 mm (3 in)

Dimensions: 2.44m x 2.44m

Screw Spacing: 76 mm on centre at perimeter and 305 mm on centre in field.

Number of Constraints or nonlinear-links defined: 149 (Figure 4.5)

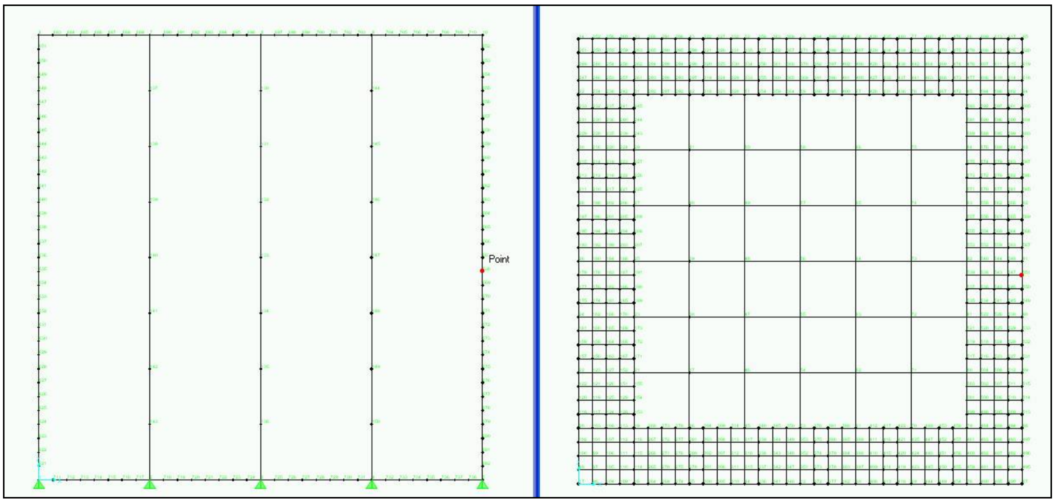

Figure 4.5 Steel Frame and Meshed Shell of Sap2000 model

Geometric and material properties of sections used in models are given in (Table 4.5)

Table 4.5 Geometric and Material properties of Sections used in Analysis of shear wall with screw spacing 76 mm on centre at perimeter

	Web (mm)	Flange(mm)	Lip(mm)	Thickness(mm)	Fy / Fu (MPa)
Studs	89	41.2	12.5	0.84	228 /310
Tracks	89	32	0	0.84	228 /310

- End Studs are Back to Back

Unit wall shear capacities are obtained from analyses and values are divided by IBC values for wind forces to compare the results. The results are defined as Normalized values according to IBC values for wind forces (Table 4.6).

Table 4.6 Analysis Results for Shear Wall with screw spacing 76 mm on centre at perimeter

	Wall Shear Capacity (kN/m)	Normalized Values According to IBC values for wind forces Capacities
IBC values for wind forces Capacity	25.31	1.0
IBC values for seismic forces Capacity	18.60	0.73
Constraint Defined Model Yield Capacity	17.85	0.71
Link Defined Model Yield Capacity	17.42	0.69
Link Defined Model Nominal Shear Capacity	26.23	1.04

Lateral force is increased up to first nonlinear link yielded and the corresponding displacement is obtained as yield displacement. After yield point lateral force is increased incrementally as other links are yielded up to the frame become unstable and the corresponding displacements are read. By this way, lateral capacity vs. displacement curve is obtained. (Figure 4.6)

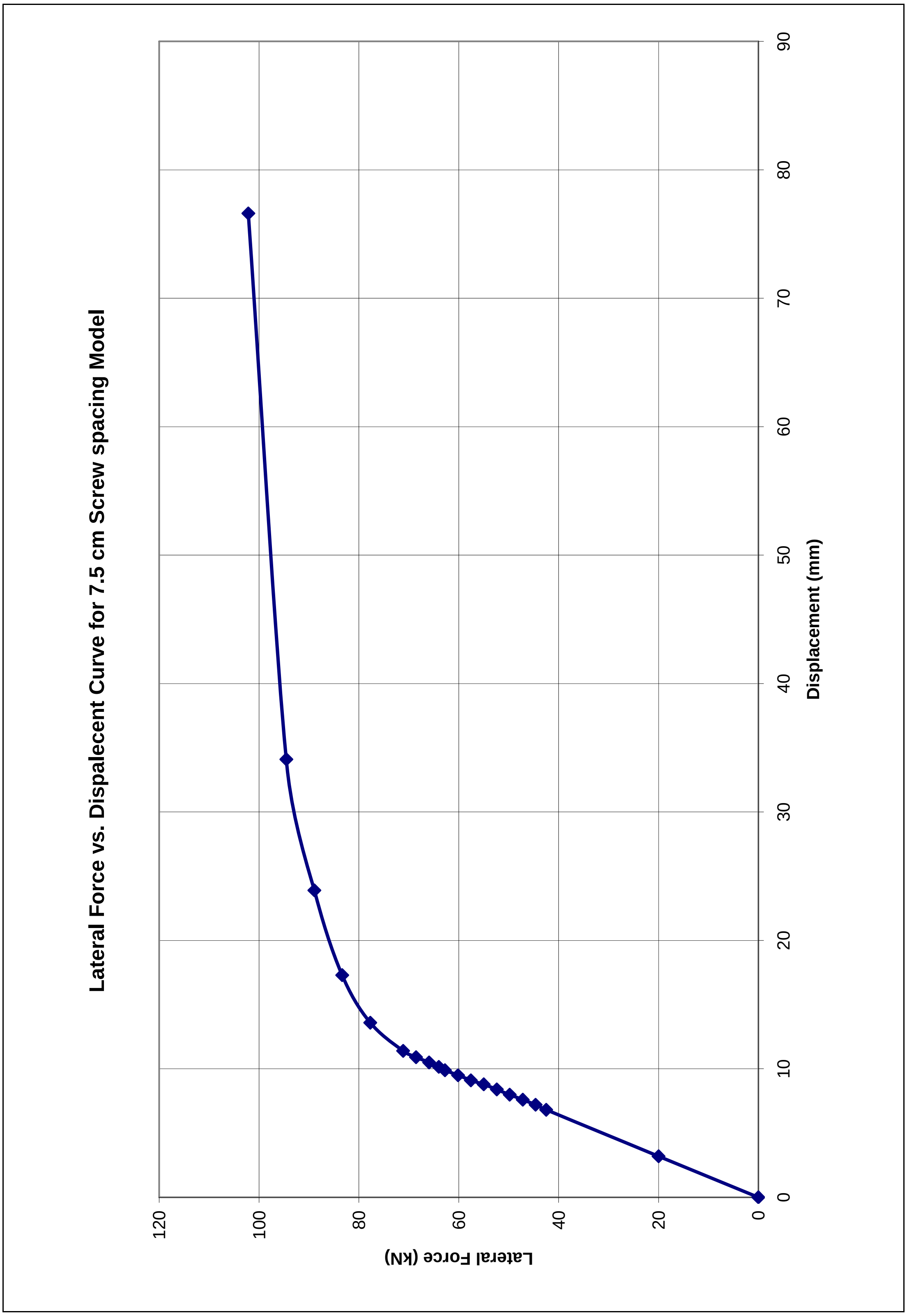

Figure 4.6 Lateral Force vs. Displacement Curve for 7.5 cm Screw Spacing Model

4.1.4. MODEL 4-Fastener Spacing at Panel Edges: 51 mm (2 in)

Dimensions: 2.44m x 2.44m

Screw Spacing: 51 mm on centre at perimeter 150 mm on centre in field.

Number of Constraints or nonlinear-links defined: 237 (Figure 4.7)

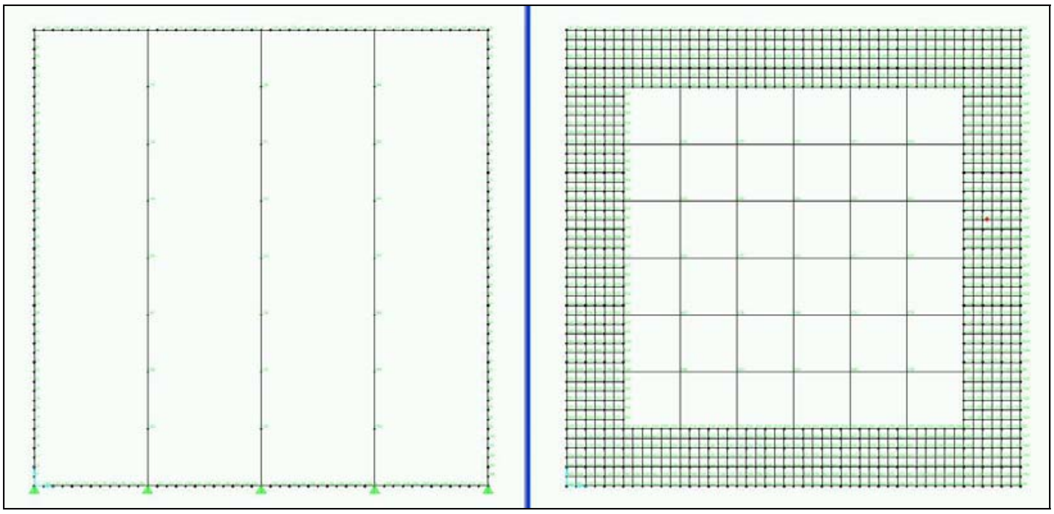

Figure 4.7 Steel Frame and Meshed Shell of Sap2000 model

Geometric and material properties of sections used in models are given in (Table 4.7)

Table 4.7 Geometric and Material properties of Sections used in Analysis of shear wall with screw spacing 51 mm on centre at perimeter

	Web (mm)	Flange(mm)	Lip(mm)	Thickness(mm)	Fy / Fu (MPa)
Studs	89	41.2	12.5	0.84	228 /310
Tracks	89	32	0	0.84	228 /310

- End Studs are Back to Back

Unit wall shear capacities are obtained from analyses and values are divided by IBC values for wind forces to compare the results. The results are defined as Normalized values according to IBC values for wind forces (Table 4.8).

Table 4.8 Analysis Results for Shear Wall with screw spacing 51 mm on centre at perimeter

	Wall Shear Capacity (kN/m)	Normalized Values According to IBC values for wind forces Capacities
IBC values for wind forces Capacity	27.87	1.0
IBC values for seismic forces Capacity	23.71	0.85
Constraint Defined Model Yield Capacity	24.44	0.88
Link Defined Model Yield Capacity	24.14	0.87
Link Defined Model Nominal Shear Capacity	29.05	1.05

* In this model with 305 mm screw spacing on centre in field the maximum screw shear force doesn't occur at the perimeter but occur in the field connections, then the field screw spacing is decreased to 150 mm on centre in field, the maximum screw shear force occur at the perimeter as expected and the capacity is increased to 24.14 kN/m.

Lateral force is increased up to first nonlinear link yielded and the corresponding displacement is obtained as yield displacement. After yield point lateral force is increased incrementally as other links are yielded up to the frame become unstable and the corresponding displacements are read. By this way, lateral capacity vs. displacement curve is obtained. (Figure 4.8)

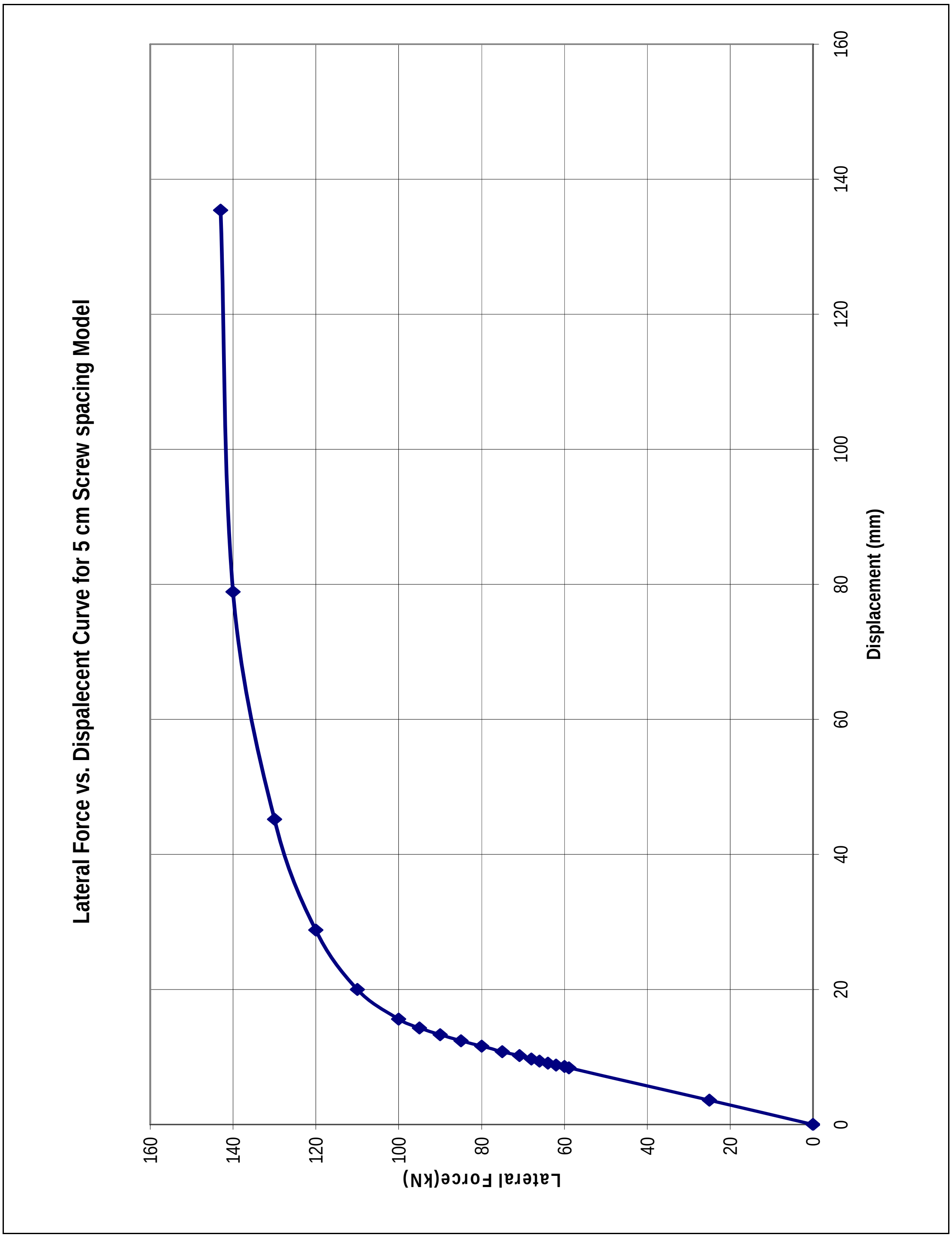

Figure 4.8 Lateral Force vs. Displacement Curve for 5 cm Screw Spacing Model

4.2 Graphical Summary and Comparison of Analyses for Different Screw Spacing

The analyses are performed for the models with four different screw spacing and the wall shear capacities for yield and nominal stages are obtained. The results are summarized and compared with IBC 2003 values (Table 4.9).

Table 4.9 Summary of Wall shear Capacities for different screw spacings and different cases

	Unit Wall Shear Capacity (kN/m)			
	15 cm Spacing Model	10 cm Spacing Model	7.5 cm Spacing Model	5 cm Spacing Model
IBC values for wind forces	13.27	20.57	25.31	27.87
IBC values for seismic forces	10.21	13.35	18.60	23.71
Constraint Defined Model Yield Capacity	9.68	13.30	17.85	24.44
Link Defined Model Yield Capacity	10.08	13.73	17.85	24.44
Link Defined Model Nominal Shear Capacity	14.34	21.31	26.23	32.79

The explanations in the legends of the graphs refer to the following analyses results

- "IBC values for wind forces" refer to Table 2211.2(1) of IBC 2003 "nominal shear values for wind forces in pounds per foot for shear walls framed with cold-formed steel studs"

- "IBC values for seismic forces" refer to Table 2211.2(3) of IBC 2003 "nominal shear values for seismic forces in pounds per foot for shear walls framed with cold-formed steel studs"

- "Constrained Defined Model Yield Capacity" refers to the yield capacity of the model that OSB-steel connections are defined as constraints.

- "Link Defined Model Yield Capacity" refers to the yield capacity of the model that OSB-steel connections are defined as non-linear links.

- "Link Defined Model Nominal Shear Values" refers to the nominal shear capacity of the model that OSB-steel connections are defined as non-linear links

- "Nominal Capacity of OSB and Bracing" refers to the nominal shear capacity of the model that X-bracing is added and OSB-steel connections are defined as non-linear links.

- "Stiffness of Link Defined Model" refers to the stiffness of the model that OSB-steel connections are defined as non-linear links

- "Stiffness of OSB and Bracing" refers to the stiffness of the model that X-bracing is added and OSB-steel connections are defined as non-linear links.

• Constraint defined model (Figure 4.9) and nonlinear link defined model (Figure 4.10) yield capacities are so close to the results of IBC nominal shear values for seismic forces. On the other side, the yield capacities are less than the IBC nominal shear values for wind forces. The ratio of yield capacities of models to corresponding IBC values are between 0.65 and 0.88.

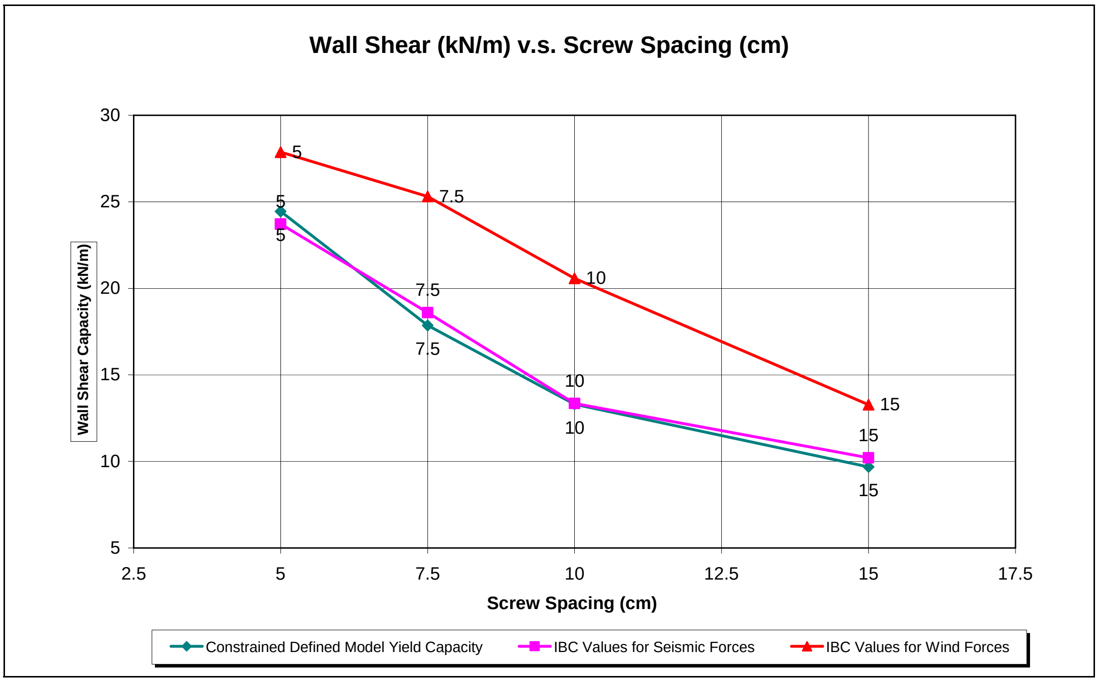

Figure 4.9 Comparison of Constraint defined Model yield Capacity with IBC nominal shear values for wind and seismic forces

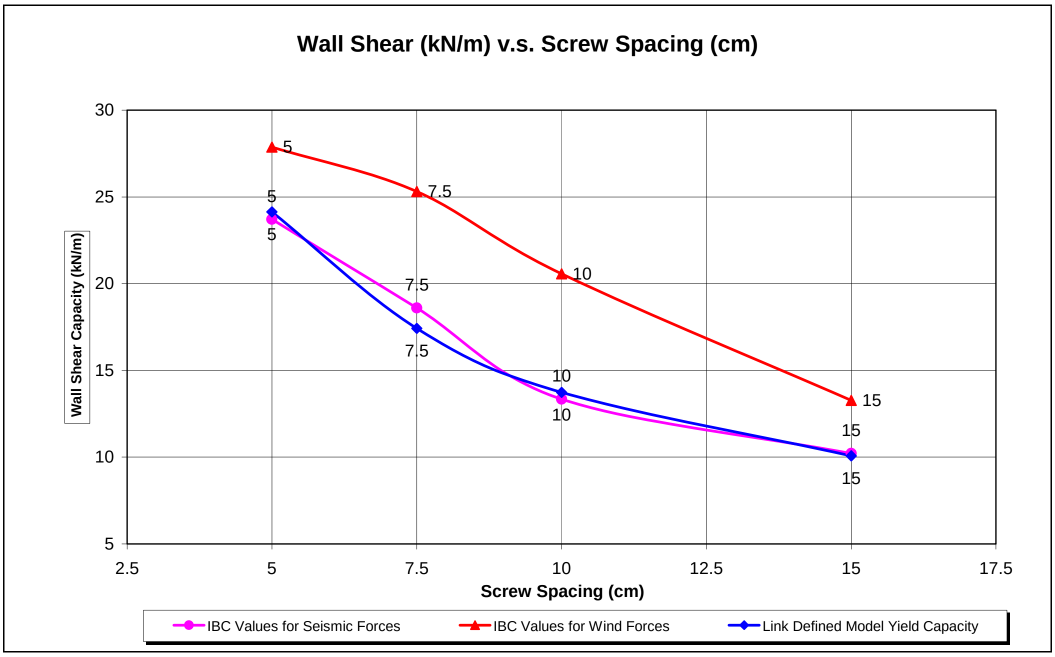

Figure 4.10 Comparison of Nonlinear Link defined Model yield Capacity with IBC nominal shear values for wind and seismic forces

- The differences between the constraint defined Model yield capacity and Link Defined Model Yield Capacity are differences around %3 which is reasonable (Figure 4.11). The nonlinear-link defined model will be used to find the nominal shear values of the walls by performing static nonlinear analysis.

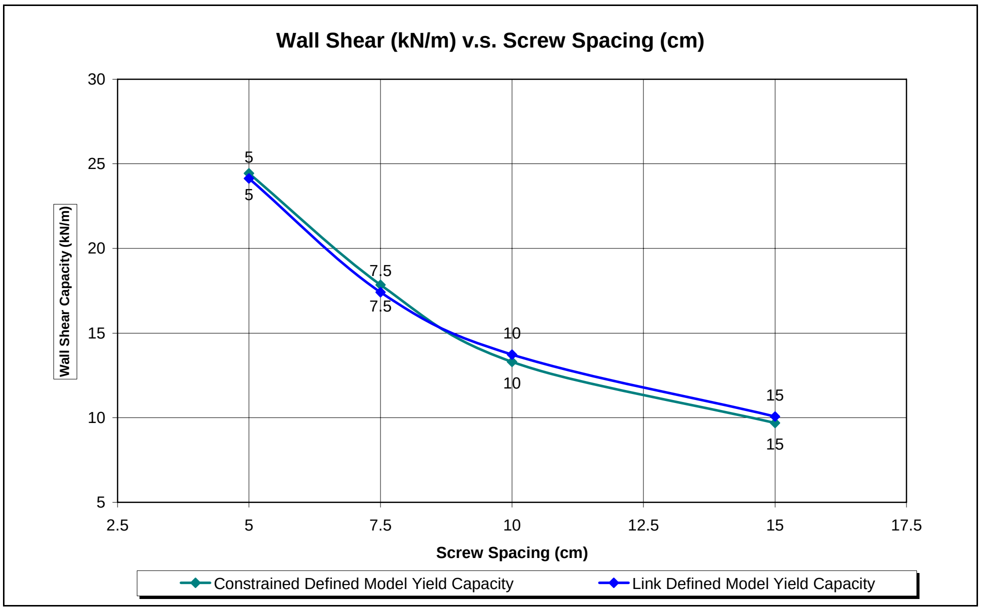

Figure 4.11 Comparison of Nonlinear Link defined Model yield Capacity with Constraint defined Model yield Capacity

- Link defined Model Nominal Shear Capacities are %3 - %7 greater than the IBC values for wind forces which is sensible and as expected (Figure 4.12).

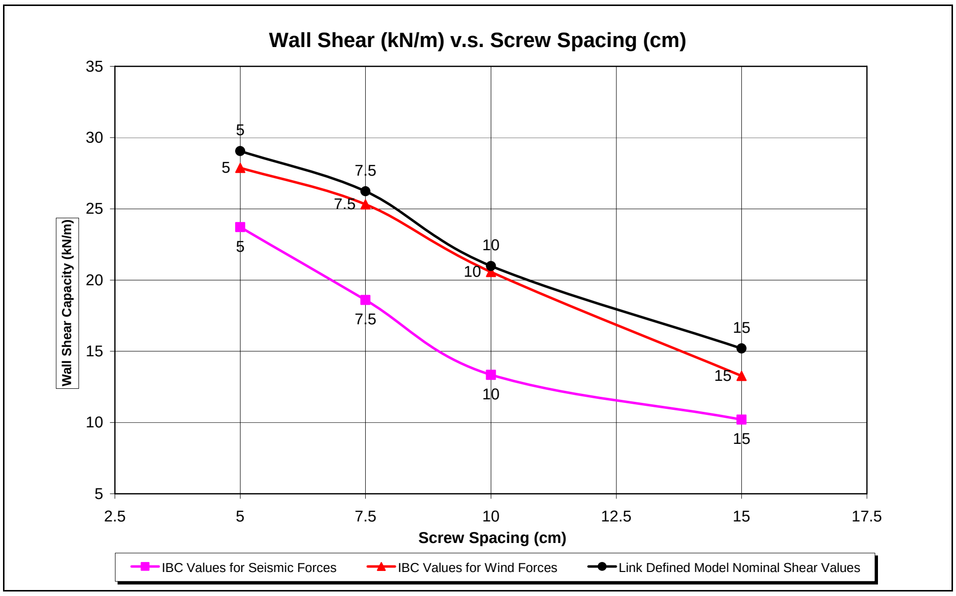

Figure 4.12 Comparison of Nonlinear Link defined Model Nominal Shear Capacity with IBC nominal shear values for wind and seismic forces

• The Nominal Shear capacities are % 42, %53, %47 and %19 more than the yield capacities for 15 cm, 10 cm, 7.5 cm and 5 cm screw spacing relatively (Figure 4.13).

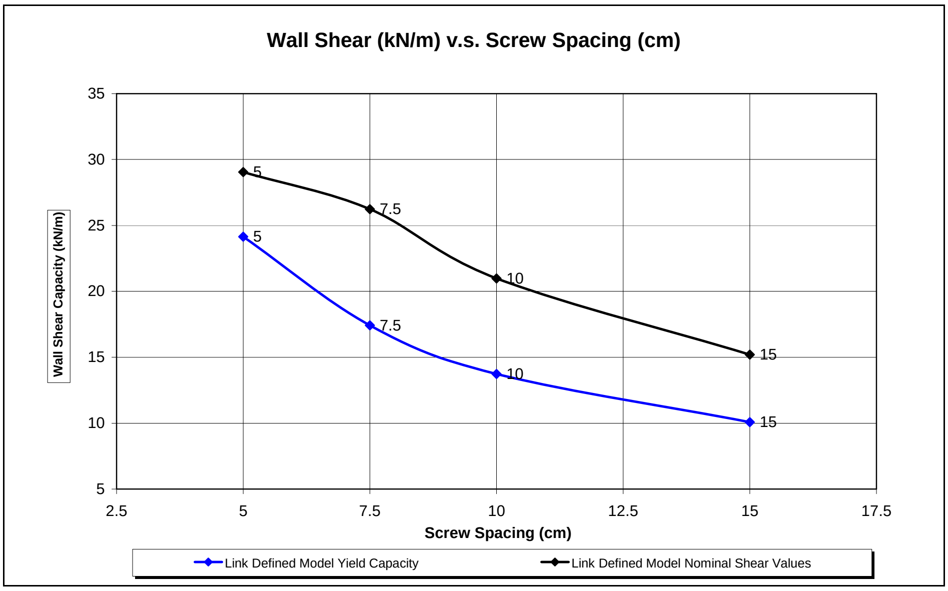

Figure 4.13 Yield and nominal shear values for nonlinear link defined models

• The values of nonlinear link defined model yield and nominal capacities and IBC 2003 values are compared and it is observed that the yield capacities are very close to IBC values for seismic forces and nominal shear values of link defined models are very close to IBC values for wind forces(Figure 4.14).

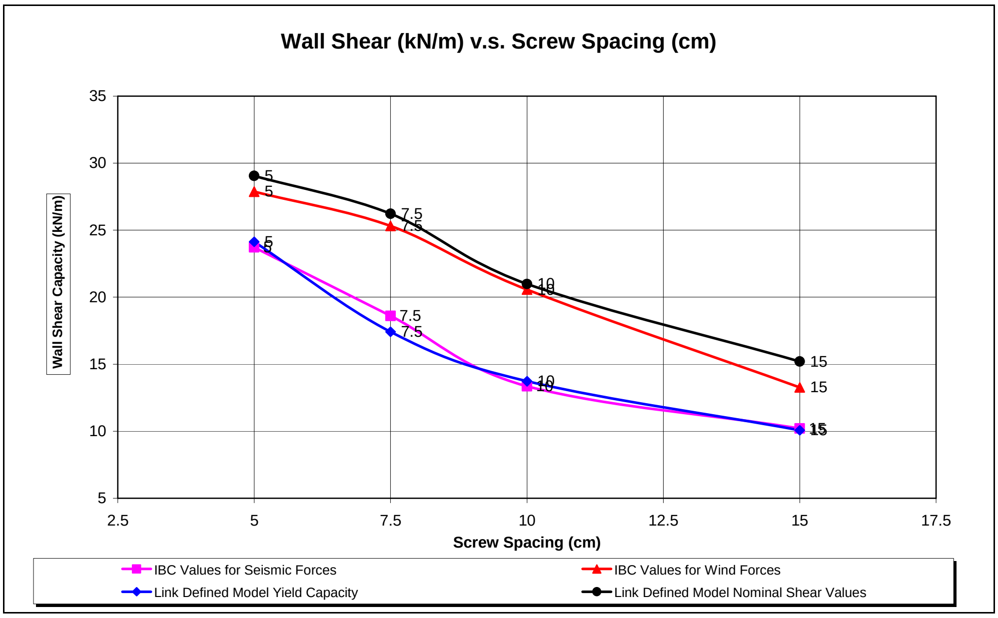

Figure 4.14 Comparison of Nonlinear Link defined Model yield and Nominal Shear Capacity with IBC nominal shear values for wind and seismic forces

- The nominal shear capacities of walls are determined for the deflection performance criteria and the effect of the aspect ratio to deflection is investigated. The lengths of the frames are 1.22 m, 2.44 m and 4.88 m, which have aspect ratios of two, one and 0.5 respectively. All the frames are analyzed for the unit wall shear of frame with aspect ratio one for each screw spacing and the lateral deflections are compared. All the lateral deflections are within the deflection limit of IBC 2003. Frames with aspect ratio one makes the least deflection and frames with aspect ratio two makes the maximum deflection among others (Figure 4.15).

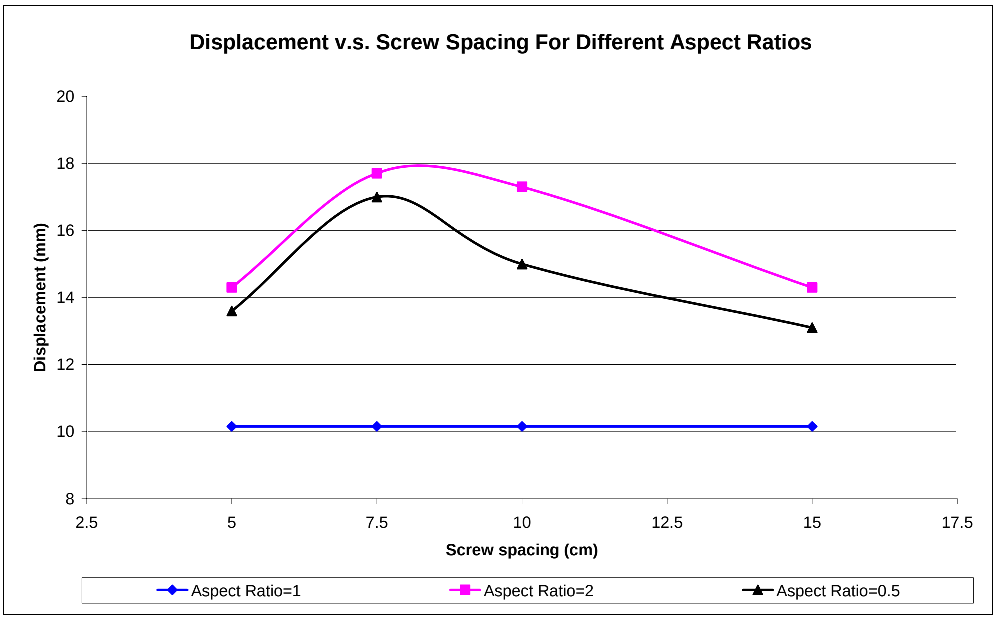

Figure 4.15 Displacement values of frames for different aspect ratios for four different screw spacing

- All the analyses results are compared with the values of tables in International Building Code which are the values approved to be used in the design of structures all over the world. The rows defined as 7/16 inch OSB one side and 7/16 inch rated sheathing OSB one side are used to compare the results. (Table 4.10 and 4.11)

Table 4.10 IBC 2003 Table 2211.2 (3) Nominal shear Values for seismic Forces for Shear Walls with cold Formed Steel Studs.

TABLE 2211.2(3)
NOMINAL SHEAR VALUES FOR SEISMIC FORCES IN POUNDS PER FOOT FOR SHEAR WALLS FRAMED WITH COLD-FORMED STEEL STUDS[a]

ASSEMBLY DESCRIPTION	MAXIMUM HEIGHT/LENGTH RATIO *h/w*	FASTENER SPACING AT PANEL EDGES[b] (inches)				MAXIMUM FRAMING SPACING (inches o.c.)
		6	4	3	2	
$^{15}/_{32}$-inch Structural 1 Sheathing (4-ply) plywood one side	2:1[c]	780	990	1,465	1,625	24
$^{15}/_{32}$-inch Structural 1 Sheathing (4-ply) plywood one side; end studs 0.043 inch minimum thickness	2:1	—	—	1,775	2,190	24
$^{15}/_{32}$-inch Structural 1 Sheathing (4-ply) plywood one side; all studs and track 0.043 inch minimum thickness	2:1	890	1,330	1,775	2,190	24
$^{7}/_{16}$-inch OSB one side	2:1[c]	700	915	1,275	1,625	24
$^{7}/_{16}$-inch OSB one side end studs, 0.043 inch minimum thickness	2:1	—	—	1,520	2,060	24
0.018-inch minimum thickness steel sheet one side	2:1	390	—	—	—	24
0.027-inch minimum thickness steel sheet one side	2:1[c]	—	1,000	1,085	1,170	24

For SI: 1 inch = 25.4 mm, 1 pound per foot = 14.5939 N/m.
a. Nominal shear values shall be multiplied by the resistance factor (ϕ) to determine design strength or divided by the safety factor (Ω) to determine allowable shear values as set forth in Section 2211.2.1.
b. Screws shall be attached to intermediate supports at 12 inches o.c. unless otherwise shown.
c. In Seismic Design Category A, B and C the aspect ratio (h/w) is permitted to be 4:1 where the design shear is reduced as required by Section 2211.2.2, Item 5.

Table 4.11 IBC 2003 Table 2211.2 (3) Nominal shear Values for Wind Forces for Shear Walls with cold Formed Steel Studs.

TABLE 2211.2(1)
NOMINAL SHEAR VALUES FOR WIND FORCES IN POUNDS PER FOOT FOR SHEAR WALLS FRAMED WITH COLD-FORMED STEEL STUDS[a]

ASSEMBLY DESCRIPTION	MAXIMUM HEIGHT/LENGTH RATIO *h/w*	FASTENER SPACING AT PANEL EDGES[b] (inches)				MAXIMUM FRAMING SPACING (inches o.c.)
		6	4	3	2	
$^{15}/_{32}$-inch structural 1 sheathing (4-ply) plywood one side	2:1	1,065[c]	—	—	—	24
$^{7}/_{16}$-inch rated sheathing (OSB), one side	2:1	910[c]	1,410	1,735	1,910	24
$^{7}/_{16}$-inch rated sheathing (OSB), one side, oriented perpendicular to framing	2:1	1,020[c]	—	—	—	24
$^{7}/_{16}$-inch rated sheathing (OSB), one side	4:1[d]	—	1,025	1,425	1,825	24
0.018-inch steel sheet, one side	2:1	485	—	—	—	24
0.027-inch steel sheet, one side	4:1[d]	—	1,000	—	—	24

For SI: 1 inch = 25.4 mm, 1 pound per foot = 14.5939 N/m.
a. Nominal shear values shall be multiplied by the resistance factor (ϕ) to determine design strength or divided by the safety factor (Ω) to determine allowable shear values as set forth in Section 2211.2.1.
b. Screws shall be attached to intermediate supports at 12 inches on center unless otherwise shown.
c. Where fully blocked gypsum board is applied to the opposite side of this assembly, in accordance with Table 2211.2(2) with screw spacing at 7 inches o.c. edge and 7 inches o.c. field, these nominal values are permitted to be increased by 30 percent.
d. Where aspect ratio (*h/w*) is greater than 2:1, the design shear shall be reduced as required by Section 2211.2.2, Item 5.

4.3 Yield Pattern

Yield pattern of joints is obtained from the non-linear analyses (Figure 4.16). It is clear that first the joints on the upper and bottom track yields, then the joints on the end studs and at last the joints on the field. This theoretical yield sequence is very consistent with the observations in experiments reported by the researchers. It is observed from this pattern is that the most important part of the shear wall is the perimeter and the shear is first transferred by the screws on the perimeter.

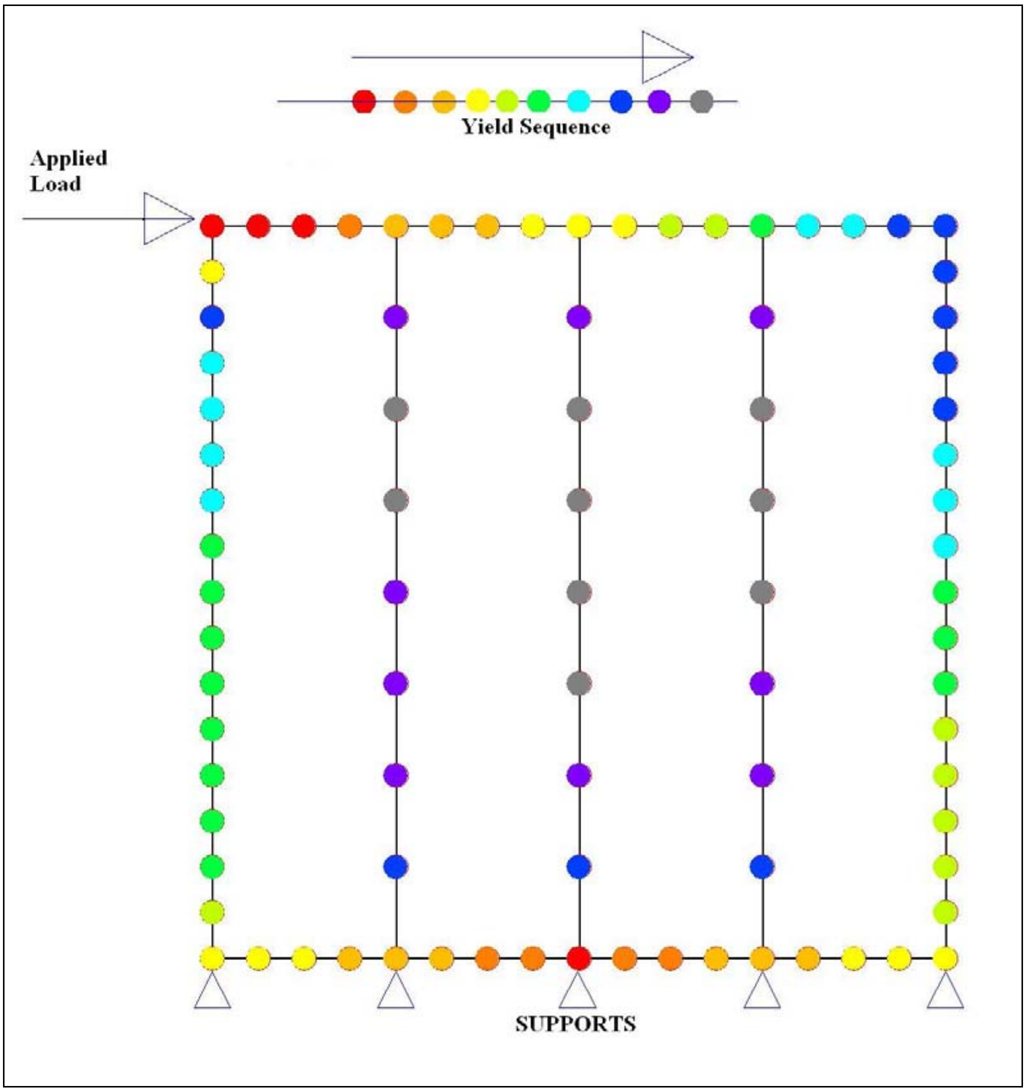

Figure 4.16 Yield pattern for the shear wall with 15 cm screw spacing on center at perimeter.

CHAPTER V

ANALYTICAL MODEL OF X-BRACED SHEAR WALLS

5.1 General Description

One of the most widely used bracing method in cold formed steel framing construction is X-type flat strap diagonal bracing (Figure 5.1). The flat strap bracings are too weak in compression and they only carry tension. The bracing-stud connection is established by self drilling screw (Figure 5.2) and the number of screws has to be determined to satisfy required connection strength. The corner of the frame that flat strap bracing connected to the frame must be anchored to foundation by hold-down member to transfer the tension load (Figure 5.3). The anchorage has to be designed strong enough to carry bracing tension force.

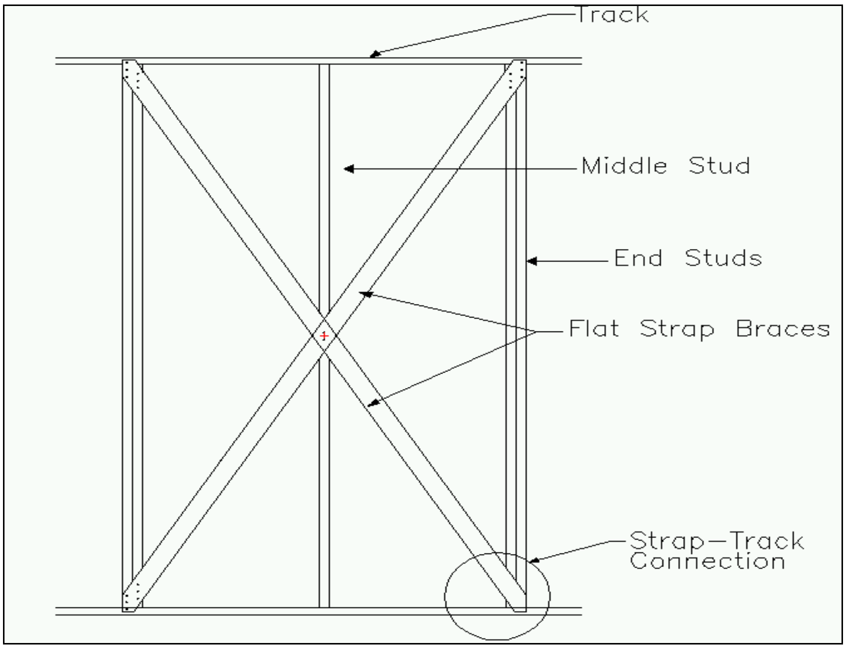

Figure 5.1 General Drawing of Frame with X-bracing.

Figure 5.2 Frame with X-bracing

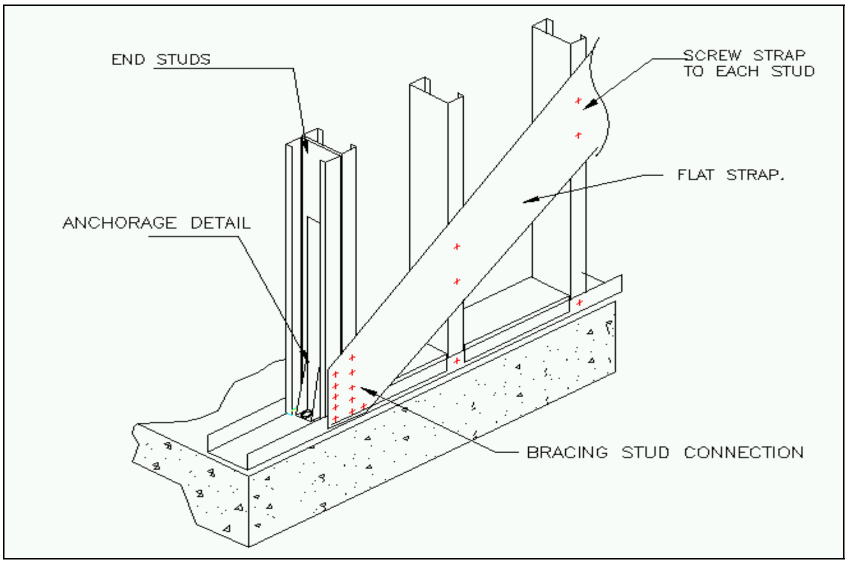

Figure 5.3 Bracing-frame connection detail

5.2 Design Parameters

There are many parameters that affect the lateral load capacity and stiffness of a wall. The most important parameters are frame section geometry, diagonal area, aspect ratio and number of bracings. These parameters are investigated in this study and result are tabled and compared below.

5.2.1 Frame Section Dimensions

The thickness of studs changes from 0.88 mm to 1.81 mm and the thickness of the tracks are 0.84 mm and 1.15 mm (Table 5.1).

Table 5.1 Dimensions of sections used in analyses.

Section Dimensions(mm)	Stud-33	Stud-43	Stud-54	Stud-68	Track-33	Track-43
Height	89	89	89	89	89	89
Flange	41.2	41.2	41.2	41.2	32	32
Lip	12.5	12.5	12.5	12.5	0	0
Thickness	0.88	1.15	1.44	1.81	0.84	1.15
Radius	1.9	1.8	2.15	2.71	1.9	1.9

5.2.2 Flat Strap X-Bracing Dimensions:

Flat straps having six different areas are used in the analyses with different widths and thickness (Table 5.2).

Table 5.2 Dimensions of X-bracings used in analyses.

Strap Dimensions (mm)	Bracing Type-1	Bracing Type-2	Bracing Type-3
Width (mm)	114	114	114
Thickness (mm)	0.88	1.15	1.44
Area(mm2)	100.32	131.1	164.16
Strap Dimensions (mm)	**Bracing Type-4**	**Bracing Type-5**	**Bracing Type-6**
Width (mm)	190	190	190
Thickness (mm)	1.15	1.44	1.81
Area(mm2)	218.5	273.6	343.9

5.3 Material Properties:

Mechanical properties of steel used in analyses.

Yield Strength of Steel: Fy=33ksi (Fy=228 MPa)

Tensile Strength of Steel: Fu=45 ksi (Fu=310 MPa).

5.4 Analysis Method

All the supports are modeled as pin and moments are released at the frame member connections. To simplify the calculations, the diagonal subjecting to compression is neglected in the model as flat strap is very weak under compression. The lateral load applied to frame is increased slightly and when one of the members (studs, tracks, bracing) fails, the corresponding load is taken as lateral load capacity of the frame. Stiffness is calculated by dividing lateral load capacity to corresponding displacement at node where diagonal brace, upper track and end stud joins. All the analyses and member checks are done by SAP2000 software. The factor of safety values are assigned as one in the software so nominal capacities of members are used in member stress checks.

5.5 CASE STUDIES

5.5.1 Case-1: Same Frame and Varying Flat Strap Diagonal Area.

In this case, the 1.22x2.44 m frame (Figure 5.4) is analyzed with six different type flat strap bracing (Table 5.3) and the effect of flat strap areas to stiffness is investigated. In each analysis, the wall shear capacity is same because the axial load capacity of stud governs the wall capacity (Table 5.4). As the diagonal area increases the displacement decreases and stiffness increases. The area of flat strap increases 3.43 times and the stiffness increases 2.77 times.

In this case it is observed that (Figure 5.5) if the axial load capacity governs the design, increasing the diagonal bracing only increases the stiffness but no effect on wall shear capacity.

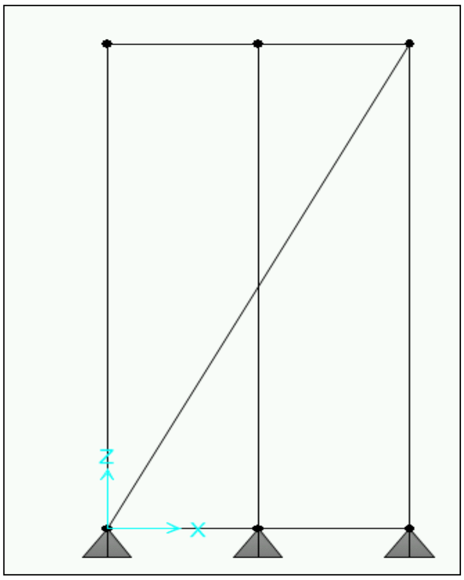

Figure 5.4 Sap2000 model of frame.

Frame Height: 2.44 m

Frame Width: 1.22 m

Stud Type: Stud- 68

Track Type: Track-43

Diagonal Type: Bracing Type-1, 2, 3,4,5,6

Table 5.3 Frame member properties of different models

CASE-1				
	Stud	**Track**	**Diagonal**	**Diagonal Area**
Model-1	Stud-68	Track-43	Bracing Type-1	100.32
Model-2	Stud-68	Track-43	Bracing Type-2	131.1
Model-3	Stud-68	Track-43	Bracing Type-3	164.16
Model-4	Stud-68	Track-43	Bracing Type-4	218.5
Model-5	Stud-68	Track-43	Bracing Type-5	273.6
Model-6	Stud-68	Track-43	Bracing Type-6	343.9

Table 5.4 Shear Capacities, Displacement and Stiffness values of different Models

	Shear Capacity (kN)	**Displacement**	**Stiffness (kN/mm)**
Model-1	11.18	8.28	**1.350**
Model-2	11.18	6.52	**1.715**
Model-3	11.18	5.37	**2.082**
Model-4	11.18	4.24	**2.637**
Model-5	11.18	3.55	**3.149**
Model-6	11.18	2.99	**3.739**

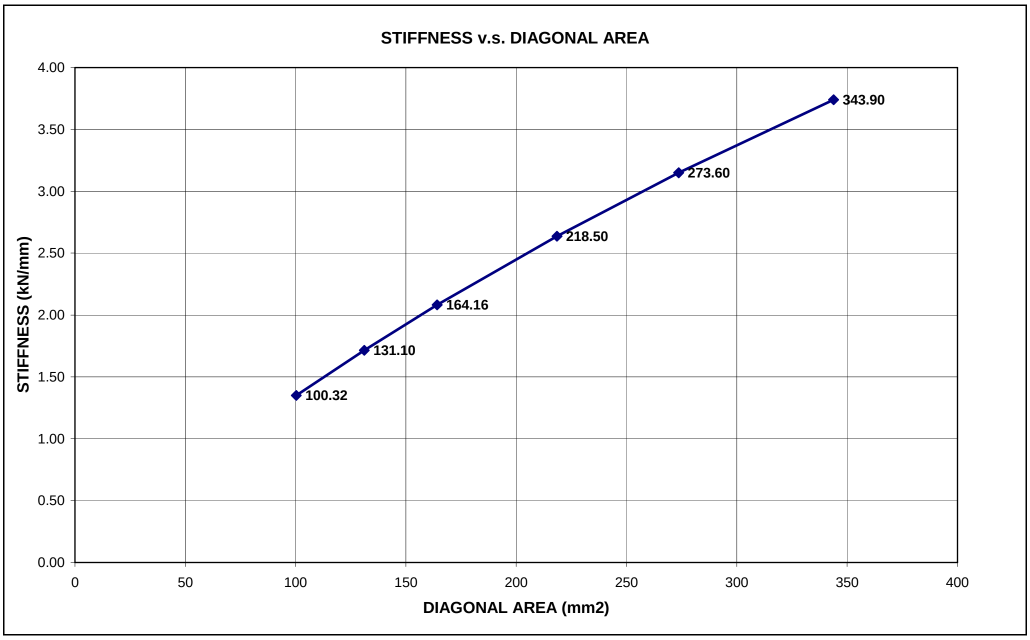

Figure 5.5 Stiffness vs. Diagonal Area curve

5.5.2 Case -2: Same Strap Diagonal Area and varying Stud thickness

In this case the 1.22x2.44 m frame is analyzed (Figure 5.6) with same diagonal bracing area and 4 different stud thicknesses (Table 5.5) and the effect of stud thickness to wall shear capacity is investigated. As the stud thickness increases the shear capacities of walls increase because the axial load capacity of stud governs the wall capacity. The stud thickness increases 2.06 times and the wall shear capacity increases 2.03 times (Figure 5.7).

In this case it is observed that increasing the stud thickness linearly increases the lateral shear capacity of the frame when axial load capacity of the stud governs the design.

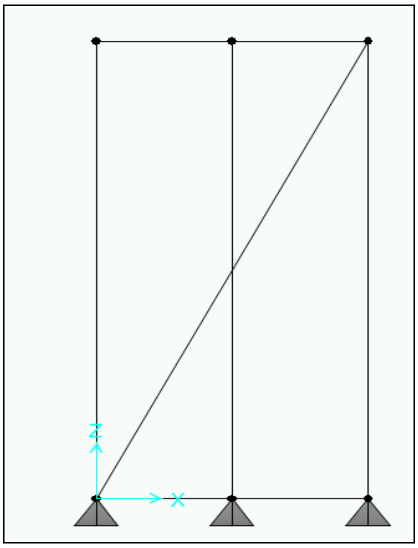

Figure 5.6 Sap2000 model of frame.

Frame Height: 2.44 m

Frame Width: 1.22 m

Stud Type: Stud- 33,43,54,68

Track Type: Track-33

Diagonal Type: Bracing Type-6

Table 5.5 Frame Member properties and Shear Capacities of different Models

	Stud	Track	Diagonal	Stud Thickness	Shear Capacity (kN)	Unit Shear Capacity (kN/m)
Model-1	Stud-33	Track-33	Bracing Type-6	0.88	5.51	4.59
Model-2	Stud-43	Track-33	Bracing Type-6	1.15	7.57	6.31
Model-3	Stud-54	Track-33	Bracing Type-6	1.44	9.29	7.74
Model-4	Stud-68	Track-43	Bracing Type-6	1.81	11.18	9.32

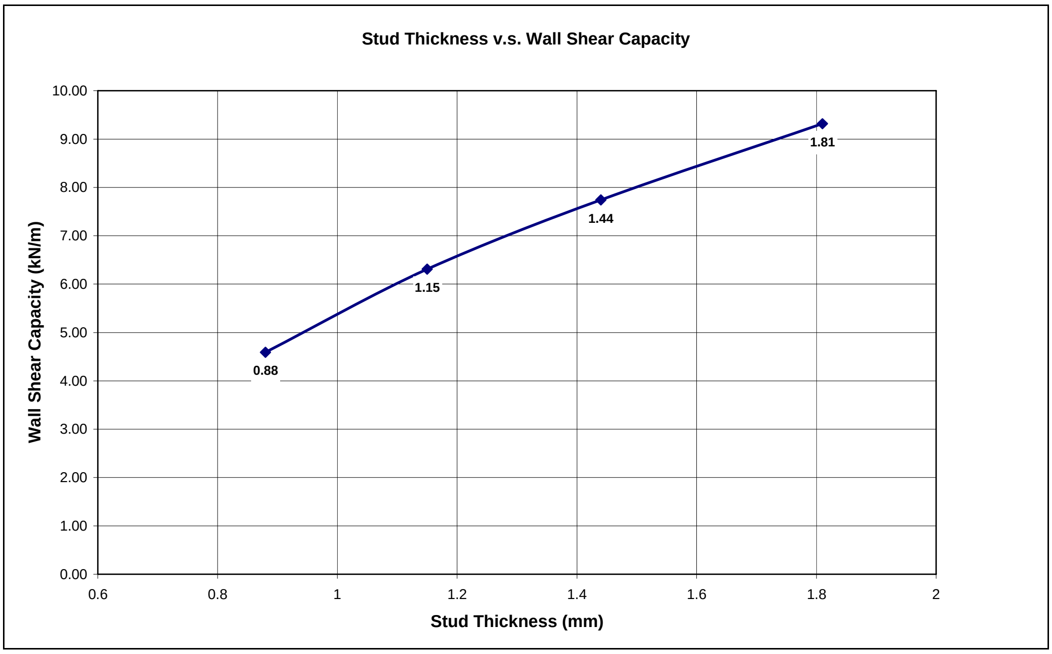

Figure 5.7 Stud Thickness vs. Wall Shear Capacity curve

5.5.3 Case -3: Same Frame Members and Varying Aspect Ratios

In this case, the frames with same sections but in different sizes (Table 5.6) are analyzed and the effect of aspect ratio to wall shear capacity is investigated (Figure 5.8). As the aspect ratio of walls increases there is nearly no change in unit shear capacity of walls (Table 5.7). The total shear capacity of walls increases but the shear capacity of unit length is nearly same, on the other side as the aspect ratio increases stiffness decreases. Stiffness decreases 2.82 times where aspect ratio changes from 0.67 to 2.0 (Figure 5.9).

In this case it is observed that aspect ratio of the wall does not have a significant effect on the shear capacity of unit wall but changes the stiffness significantly (Figure 5.10).

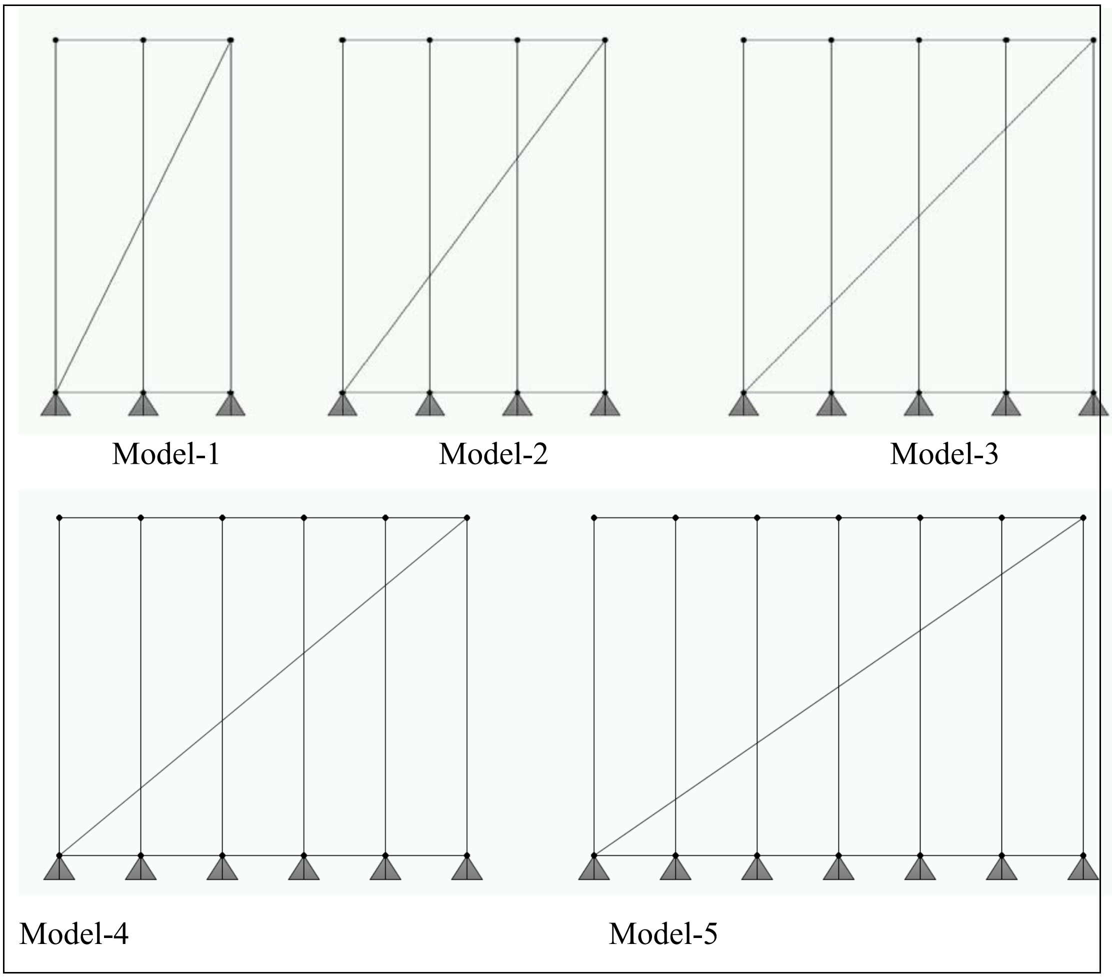

Figure 5.8 Analytical models of shear wall with X-bracing with different aspect ratios

Table 5.6 Frame Member properties and Aspect Ratios of different Models

	Frame Members	**Panel Width m (w)**	**Panel Height m (h)**	**Aspect Ratio(h/w)**
Model-1	Stud-54 Track-33 Bracing Type-6	1.22	2.44	2.00
Model-2		1.83	2.44	1.33
Model-3		2.44	2.44	1.00
Model-4		3.05	2.44	0.80
Model-5		3.66	2.44	0.67

Table 5.7 Shear capacities, displacements and stiffnesses of frames with different aspect ratios

	Total Shear Capacity (kN)	Unit Shear Capacity (kN/m)	Displacement (mm)	Stiffness (kN/mm)
Model-1	9.29	7.61	2.64	**3.519**
Model-2	14	7.65	2.25	**6.222**
Model-3	18.58	7.61	2.24	**8.295**
Model-4	23.33	7.65	2.47	**9.445**
Model-5	28	7.65	2.82	**9.929**

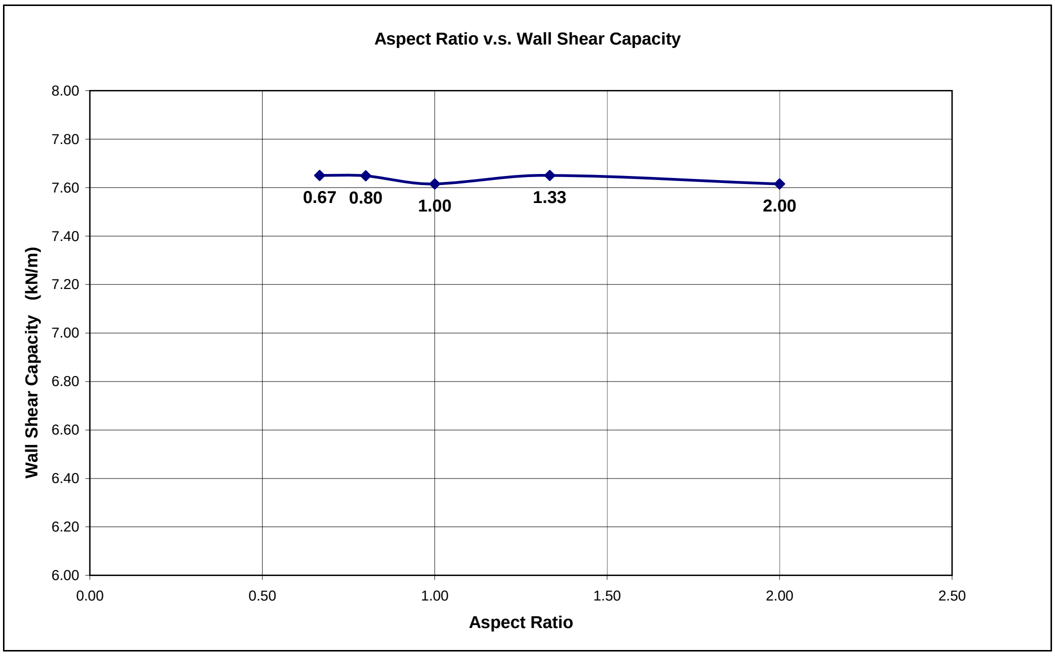

Figure 5.9 Change of Unit Wall Shear Capacity with Aspect Ratio curve

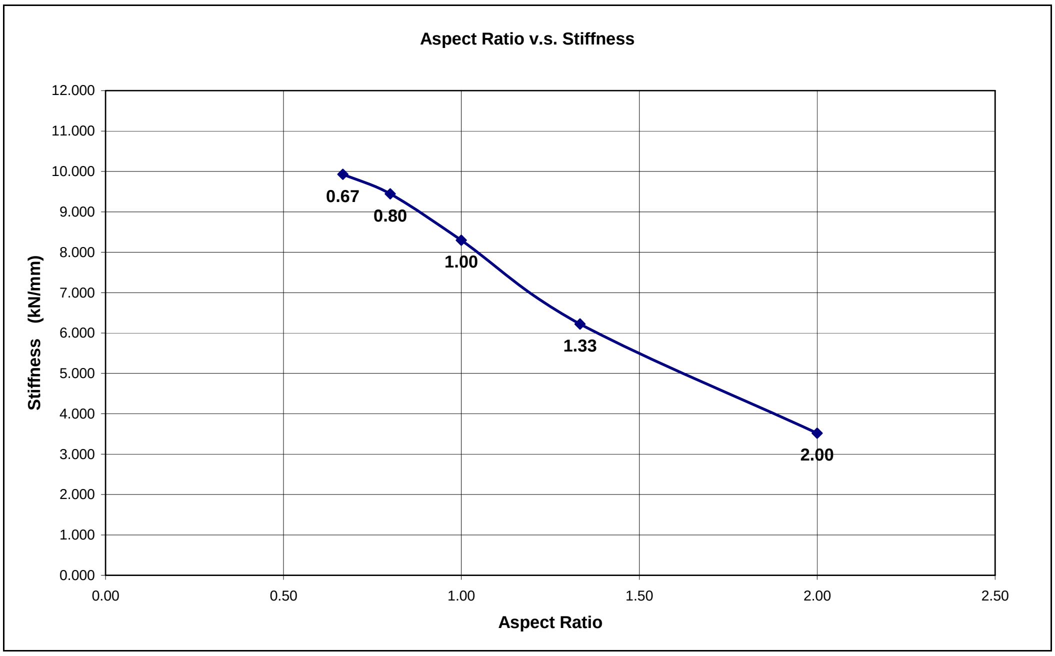

Figure 5.10 Change of Stiffness with Aspect Ratio curve

5.5.4 Case-4: Same Frame Members and Varying Number of Diagonal Bracing

In this case 2.44m x 2.44m frames with same sections but having different number of diagonal bracings (Table 5.8) are analyzed and the effect of bracing number to wall shear capacity is investigated (Figure 5.11). As the number of bracing increases there is nearly no change in shear capacity of walls (Table 5.9). On the other side as the number of bracings increases, stiffness decreases. Stiffness decreases 2.01 times where bracing number changes from one to four (Figure 5.12).

In this case it is observed that diagonal number of the wall does not have a significant effect on the shear capacity but changes the stiffness significantly (Figure 5.13).

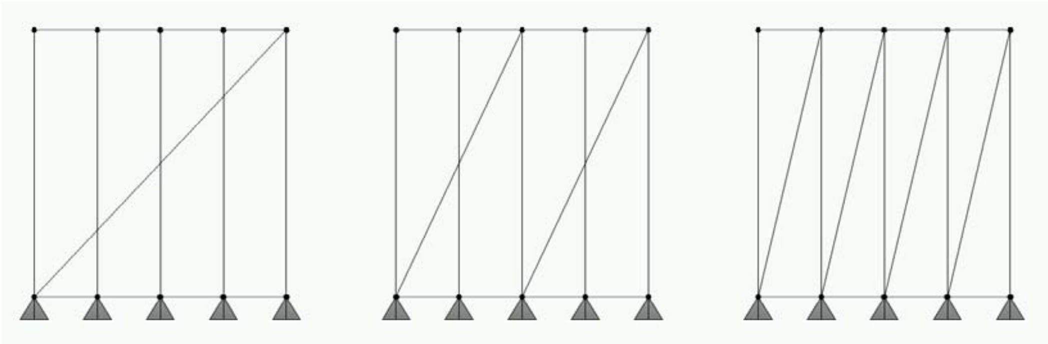

Figure 5.11 Analytical models of shear wall with X-bracing with different number of bracings

Table 5.8 Frame Member properties and Number of Bracings of different Models

	Stud	Panel Width (w)	Panel Height (h)	Number of Bracing
Model-1	Stud-54	2.44	2.44	1.00
Model-2	Track-33	2.44	2.44	2.00
Model-3	Bracing Type-6	2.44	2.44	4.00

Table 5.9 Shear capacities, displacements and stiffnesses of frames with different number of bracings

	Shear Capacity (kN)	Shear Capacity (kN/m)	Displacement(mm)	Stiffness (kN/mm)
Model-1	18.58	7.61	2.24	**8.295**
Model-2	18.62	7.63	2.64	**7.053**
Model-3	18.56	7.61	4.5	**4.124**

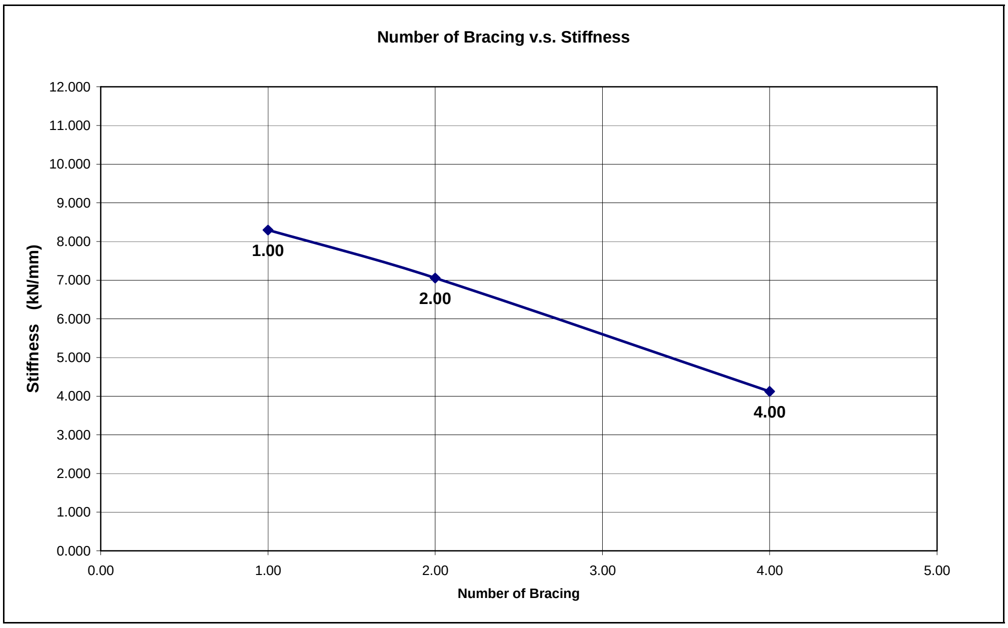

Figure 5.12 Change of Stiffness with Number of Bracing curve

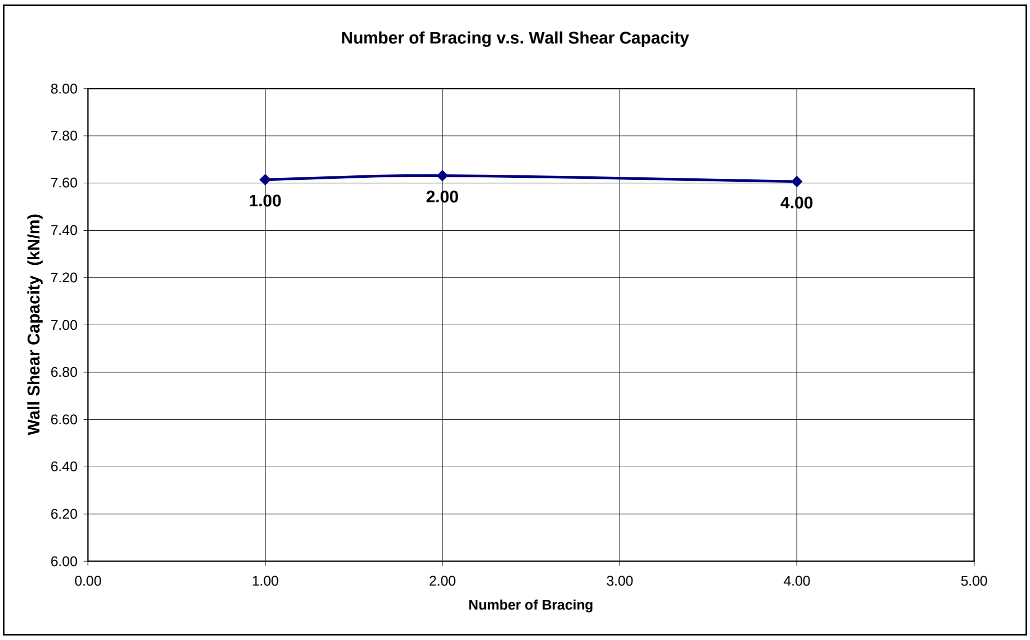

Figure 5.13 Change of Wall Shear Capacities with Aspect Ratio curve

CHAPTER VI

COMBINATION OF X-TYPE DIAGONAL BRACING and OSB SHEATHED WALLS

The shear walls designed according to X-type flat strap diagonal bracing are mostly sheathed by OSB, even if OSB is not included in the structural design because to make the external siding, the construction should be completely covered and this covering material is mostly OSB. The effect of OSB sheathing together with X-bracing is investigated in this part of the study.

6.1 Contribution of X-bracing to OSB sheathed Wall Shear Capacity

The change in the yield capacity, stiffness and nominal shear capacity is observed when X-bracing is added to an OSB sheathed shear wall for different screw spacing. Adding X-bracing mainly decreases the lateral displacement and increases the stiffness, and also it increases the yield and nominal capacities of the shear walls. The effects are analyzed below in detail

Using X-bracing together with OSB sheathed shear walls increases the yield capacity %4.2, %5.6, %8.5, %11.7 for 5 cm, 7.5 cm 10 cm, 15 cm screw spacing relatively (Figure 6.1).

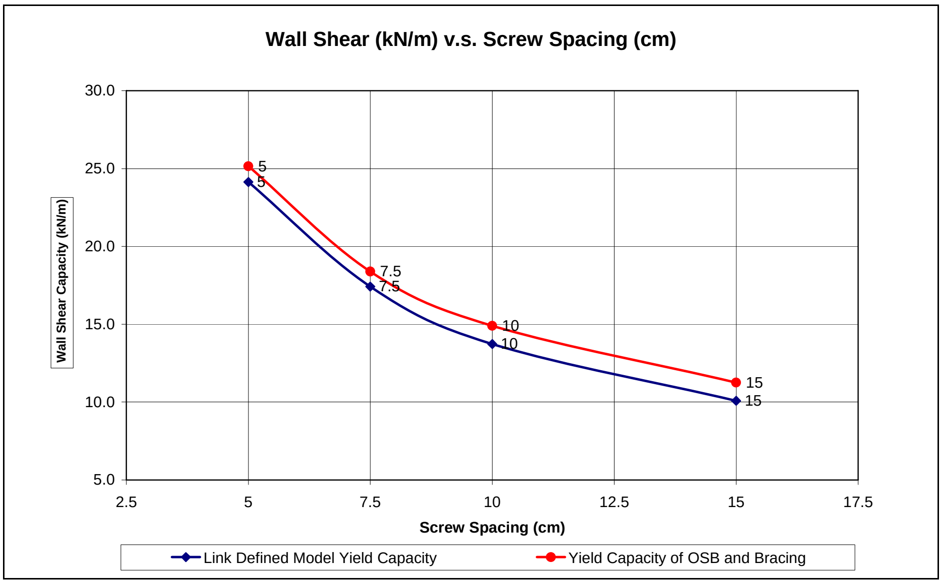

Figure 6.1 Yield Capacities of Walls vs. Screw Spacing with and without X-bracing

Using X-bracing together with OSB sheathed shear walls increase the stiffness %29.3, %21.6, %19.0, %14.9 for 15 cm, 10 cm, 7.5 cm, 5 cm screw spacing relatively since X-bracing decreases the displacement. The displacements are taken for corresponding yield capacities (Figure 6.2).

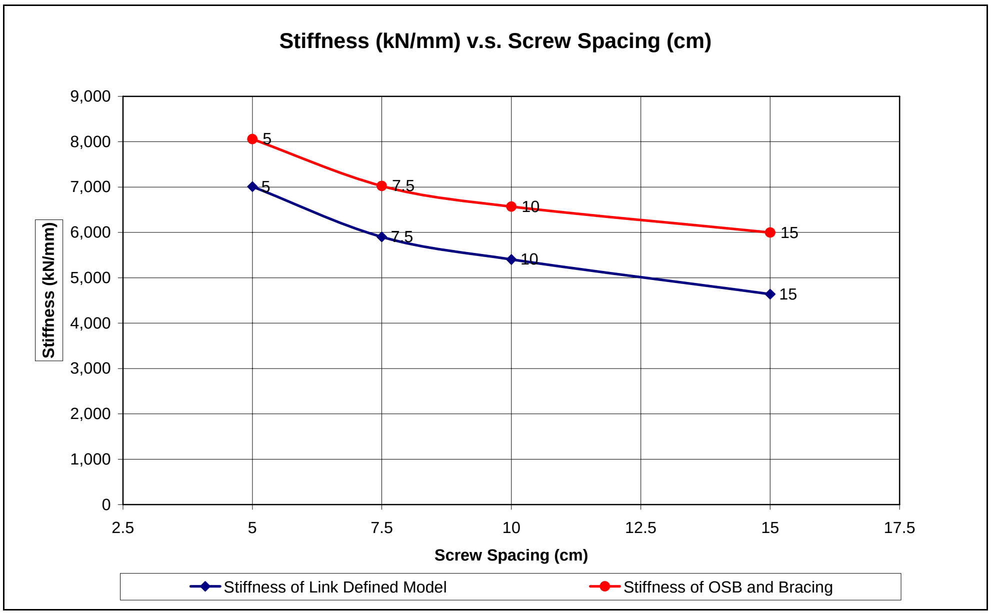

Figure 6.2 Stiffness of Walls vs. Screw Spacing with and without X-bracing

Adding X-bracing to OSB walls increases the Nominal Shear capacity %38.1, %19.5, %16.2, %13.9 for 15 cm, 10 cm 7.5 cm, 5 cm screw spacing relatively (Figure 6.3).

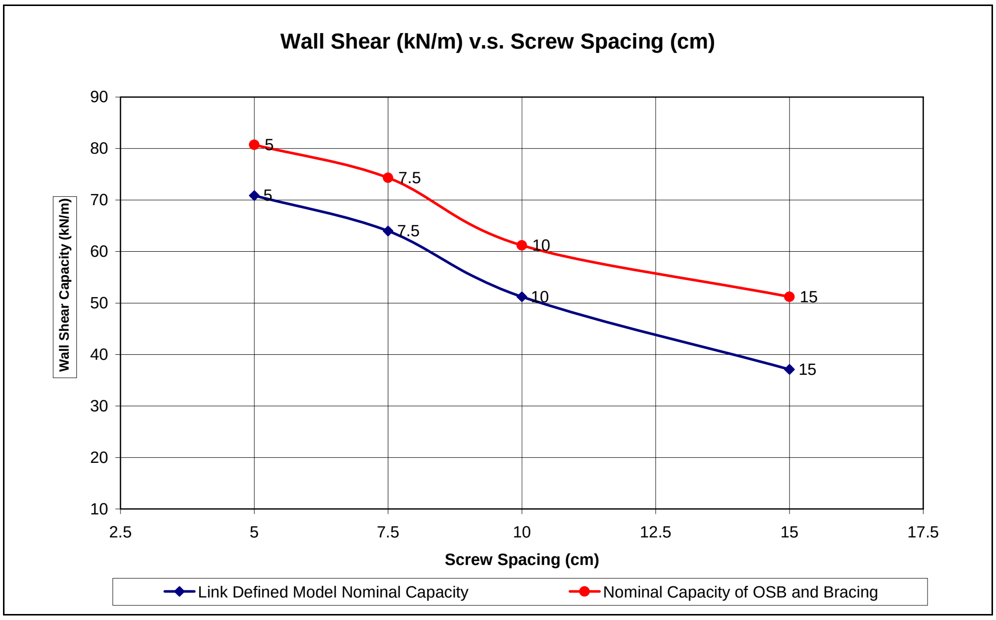

Figure 6.3 Nominal Capacities of Walls vs. Screw Spacing with and without X-bracing

6.2 Contribution of OSB Sheathing to Diagonal Bracing and End Stud Axial Force

The X-braced shear wall frames with four different perimeter screw spacings are analyzed for the nominal shear values and the change in the axial force of diagonal bracing and end studs with and without OSB sheathing is observed. Diagonal bracing on frames without OSB takes axial force proportional to applied force to frame but on the frames also sheathed with OSB, diagonal bracing takes nearly constant force and the remaining shear is transferred by plate. The axial force on the bracing decreases 2.7 to 4.2 times when the frame only with bracing is sheathed by OSB plates (Figure 6.4).

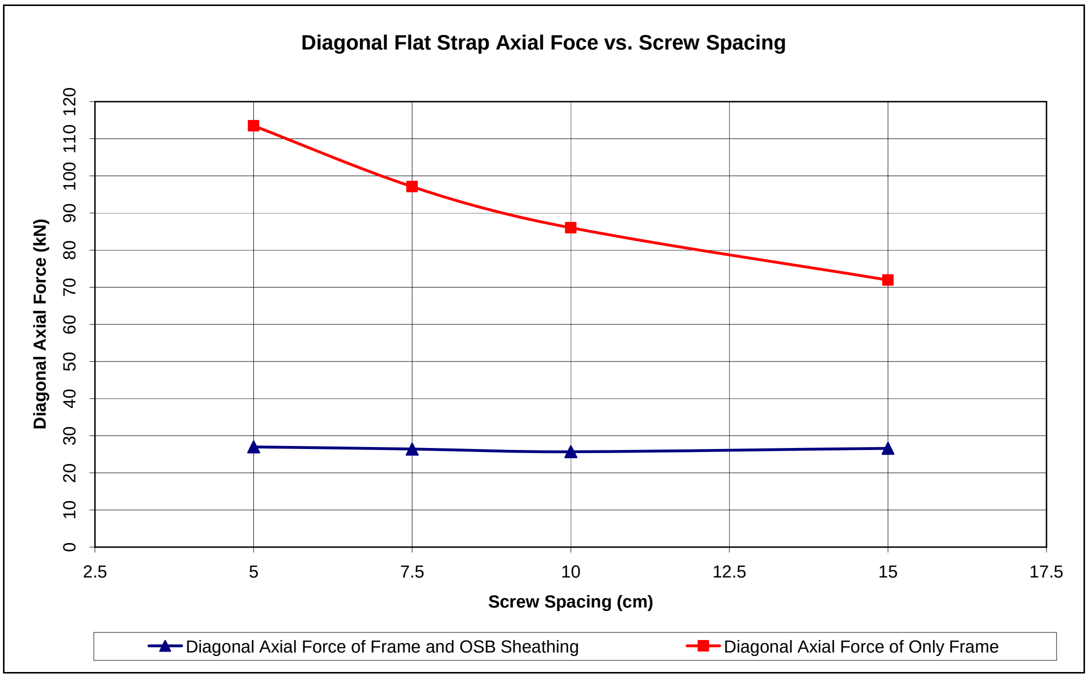

Figure 6.4 Diagonal Flat Strap Axial force vs. Screw Spacing with and without X-bracing

6.3 Contribution to End Stud Axial Force

End studs on X-braced frames without OSB takes around %20 more axial force than the X-braced frames sheathed with OSB. The ratio is nearly constant for models with different screw spacings (Figure 6.5).

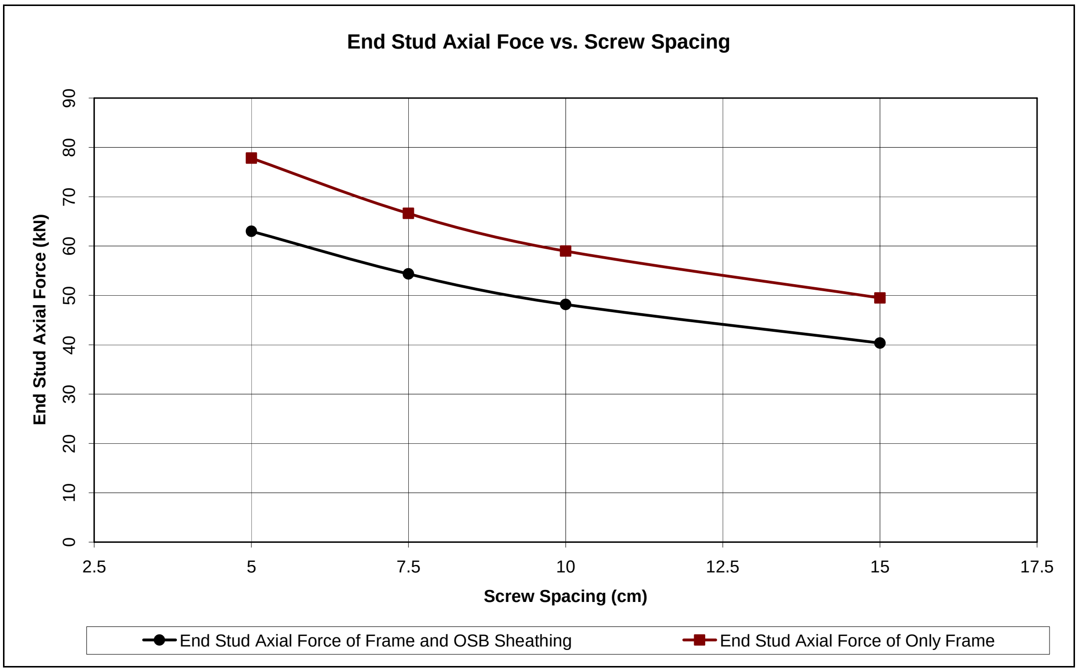

Figure 6.5 End Stud Axial Force vs. Screw Spacing with and without X-bracing

CHAPTER VII

CONCLUSIONS

Following conclusions were drawn from the study

- Oriented Strand boards sheathed shear walls have significant shear capacity to resist against lateral forces caused by earthquake and wind.

- The shear wall capacity can be increased by increasing the screw spacing on perimeter. The shear capacity increases 2.28 times if the perimeter screw spacing decreases from 15 cm to 5 cm, assuming that all the other parameters are constant.

- A consistency has been achieved with the analytical model results and International Building Code (IBC 2003) table values for nominal shear values for shear walls framed with cold-formed steel studs.

- While performing nonlinear analyses it is observed that first perimeter screws yield and than the joints on the field yield that is the case observed in the experiments done by researchers.

- Stiffness of X-braced wall frames increase if the flat strap bracing with larger area is used but it does not make significant change in the lateral load capacity of the frame because the axial compression capacities of the end studs govern the capacity.

- Aspect ratio of the X-braced shear walls does not make any significant change in the lateral load capacity of unit length but as the aspect ratio increases the stiffness decreases significantly.

- Number of bracings for a shear wall with constant dimensions does not change the lateral load capacity but stiffness decreases as number of bracing increases.

- Using X-bracing together with OSB sheathed walls both increases the nominal shear capacity and also increases the stiffness significantly.

- Using X-bracing together with OSB sheathed walls both decreases the axial forces on the diagonal bracing and end studs.

The contribution of gypsum board cladding on the interior side of the wall to the lateral load capacity, effect of openings in long shear walls and the performance of horizontal floor diaphragms can be studied in further studies.

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