Behaviour of Composite Framed Shear Wall System

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ABSTRACT

This research investigated the behaviour of a novel form of composite framed shear wall system (CFSWS) under lateral loading. The CFSWS consisted of a composite wall (made of two skins of profiled steel sheeting and an infill of concrete) connected to pinned steel or fixed concrete filled steel tube (CFST) frame. The experimental investigations on one and two-storey four CFSWS models of 1/6th scale provided information on shear load-deformation response, shear strength/stiffness, energy absorbing capacity, stress-strain characteristics and failure modes. The failure of CFSWS was associated with buckling of steel sheets and development of diagonal concrete core cracking as well as the wall-frame fastener and CFST frame joint failure. Overall, the failure was governed by wall failure rather than frame. Analytical models for the shear strength of CFSWS were developed and found to be in close agreement with experiments. This research confirmed the viability of using novel CFSWS in practical construction.

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LIST OF SYMBOLS

A	Larger dimension of a rectangular panel
a_g	Specified nominal maximum size of coarse aggregate
A_b	Cross sectional areas of the storey beam
A_c	Cross-sectional area of steel column
b_p and a_p	Dimensions of profile steel plate between two intermediate fasteners (Fig. 5.2)
a and b	Dimensions of a profiled steel sheet / shorter dimension of a rectangular panel
d	Original length of one repeating profiled steel sheet
d_v	Effective shear depth
E	Transferred energy from a projectile to a structure / Young's modulus
E_c	Initial tangent modulus of concrete
E_c	Tangent modulus of concrete
E_{FD}	Energy absorbed by frictional dissipation
E_s	Modulus of elasticity of steel
f_c	Compressive stress of concrete
f'_c	Cylinder compressive strength of concrete f'_t Splitting tensile strength of concrete
f_1	Principal tensile stress on average
f_2	Principal compressive stress on average
$f_{2, \max}$	Maximum average principal compressive stress that can be resisted
F_u	Ultimate tensile strength of profiled steel sheet
F_y	Yield strength of steel plate or profiled steel sheet
G	Shear modulus of the material
h	Frame storey height / height of profiled steel sheet
I_c	Moment of inertia of the column
I_y	Moment of inertia of one repeating profiled steel sheet about "y" axis
k_v	Shear buckling coefficient
l_w	Overall width of a profiled steel sheet
M	Mass of a projectile
$P_{c/d}$	The load associated with the initiation of concrete cracking and debonding of the profiled steel sheet from the concrete core

s_{θ}	Diagonal crack spacing
s_z	Crack spacing parameter
s_{ze}	Effective crack spacing accounting for aggregate size effect
t_c	Equivalent thickness of the profiled concrete panel
t_{eq}	Average thickness of profiled concrete panel
t_s	Thickness of a profiled steel sheet
v_c	Shear stress resistance provided by the concrete
v_{ci}	Shear stresses on concrete cracked interface resisted by aggregate interlock
V_c	Shear resistance attributed to the concrete panel
V_{cr}	Shear buckling capacity of a profiled steel sheet
V_f	Shear stress resistance provided by the frame
V_{CFSWS}	Shear resistance of the CFSWS
V_s	Shear resistance provided by one profiled steel sheet
w	Crack width
α	Angle of inclination of the tension field / ratio of extended length of one repeating profile to its original length
β	Factor accounting for shear resistance of cracked concrete / coefficient depends on connection type of steel sheet to the boundary frame
γ_{max}	Maximum shear strain in profiled steel sheet
ϵ_y	Longitudinal strain at mid-depth of the wall (positive when is tensile)
ϵ_c	Concrete strain
ϵ'_c	The apex strain in the compressive strain (ϵ_2) – stress (f_2) curve
ϵ_{nom}	Nominal strain
ϵ_{p1}	Maximum principal strain in profiled steel sheet
ϵ_{p2}	Minimum principal strain in profiled steel sheet
ϵ_1	Principal tensile strain on average in cracked concrete
ϵ_2	Principal compressive strain on average in cracked concrete
ρ	Mass density
σ	Effective normal stress
σ_c	Compressive stress in concrete

σ_{nom}	Nominal stress
σ_{p1}	Maximum principal stress in profiled steel sheet
σ_{p2}	Minimum principal stress in profiled steel sheet
σ_t	Tensile stress in concrete
τ	Shear stress capacity of material
τ_{cr}	Critical buckling shear stress for a flat steel plate
τ_{max}	Maximum shear stress in the steel sheet before failure due to von-Mises yield criterion
ϕ_c	Resistance factor for concrete ($\phi_c = 0.65$)
ϕ_s	Resistance factor for structural profiled steel sheet ($\phi_s = 0.90$)

CHAPTER ONE

INTRODUCTION

1.1 General

The structural elements commonly used in tall buildings to resist loads are moment resisting frames, braced frames, shear walls and tabular structures. A combination of two or more of these elements is often used in the structural system. Moment resisting frames become uneconomical as the height of the building is increased and recourse has to be made to the braced frames or shear walls. In a framed shear wall system, the wall panel is considered as an element to stiffen the shear resistance of the surrounding frame and can attract a larger portion of the horizontal loads due to its added stiffness.

Reinforced Concrete (RC) shear walls, traditionally, were used in building structures to resist lateral load applied by wind and earthquake forces for many years to brace the building. The main disadvantage of a RC shear wall is the development of tension cracks in the tension zones and compressive crushing in the localized compression areas during large cyclic loadings. Recently, steel plate shear walls have also been used as lateral load resisting systems in mid-rise and tall buildings (Astaneh-Asl 2001a, b). The disadvantage of a steel shear wall is associated with the buckling of the compression zone, which results in reduction of shear stiffness, strength, and energy dissipation capacity (Zhao et al. 2004).

In less than 40 years, other types of shear walls were developed to brace mostly steel structures from lateral loads. More recently, a novel form of double skin composite walling (DSCW) system (shown in Figure 1.1a) has been introduced which comprises vertically aligned profiled steel sheeting and an infill of concrete (Wright et al. 1992; Wright et al. 1994; Wright and Gallocher 1995; Hossain and Wright 1995). The concept of composite wall was originated from the floor structure using profiled steel deck and concrete (Wright et al. 1992). Profiled steel sheeting is widely used in composite construction and composite slabs known as "fast-track construction" (Wright et al. 1987). The proposed Framed Shear Wall Systems (CFSWS) consisting of DSCW connected to steel/RC/composite frames as shown in Figure 1.1 has many advantages (Hossain et al. 2004a, c, d).

Figure 1.1: Components of composite framed shear wall system (CFSWS)

First, during construction, profiled steel sheeting acts as a formwork for in-fill concrete (Wright and Gallocher 1995). Second, the steel sheet acts as a bracing system to the building frame against wind and destabilizing forces in the construction stage (Hossain and Wright 1995). Third, in the service stage, profiled steel sheets act as reinforcement. Finally, the added confinement of the concrete by the steel sheeting will both increase the load resistance compared to traditional reinforced concrete (RC) structures. Fire resistance and corrosion of these walls may be the causes of concern. However, such performances can be improved by providing adequate fire proofing protection and paint finishing (Hossain 1999; Taormina and Hossain. 2012).

In such construction, the shear bond and interaction between sheeting and concrete plays an important role in the composite action of the system and will govern the type of failure of the wall. The interface shear bond failure may be limiting criteria for the design of such system (Hossain and Wright 2004a,b). The bond between steel and concrete can be improved by

embossments installed in the commercial profiled steel sheets available in North America or by installing other forms of connectors such as fasteners and other mechanical devices to connect pair of sheeting. The mechanical interlock at the sheet-concrete interface may govern the ductile and brittle failure of such composite walls.

The sheet-concrete interface behavior in a composite wall is complex as profiled ribs play an important role in providing mechanical bond when steel tends to slide over the concrete after the failure of chemical bond. By using adequate intermediate fasteners (Figure 1.1a), global buckling has been controlled to ensure that the profiled sheet reaches its yield stress under shear loading. In-fact that concrete core provides bracing to the profiled sheet that prevents buckling prior to yielding.

1.2 Significance of the Research

Previous research has been conducted on double skin composite walling (DSCWs) subjected to axial loading as well as in-plane monotonic lateral, cyclic and impact loadings. Experimental, analytical, and numerical investigations were conducted to develop design equations/guidelines for strength and stiffness of DSCWs under these loading conditions. However, little or no research has been conducted on the behavior of CFSWS connected to different types of building frames under lateral loading. Use of concrete filled steel tube (CFST) frame is a special feature of the study. Use of such frames can improve the strength and ductility performance of the CFSWS (Hossain 2005; Sakino et al. 2004; Lachemi. et al. 2005; Hossain 2003).

1.3 Scope of the Research

The primary focus of this research is to study the behaviour of composite framed shear wall system (CFSWS) consisting of double skins of profiled steel sheeting with an infill of concrete connected to frames (as shown in Figure 1.1) under lateral loading.

To examine such behaviour, four (one and two storey) CFSWS models of 1/6th scale have been manufactured with overall dimensions of 640 mm x 640 mm and 640 mm x 1280 mm with

pinned steel frame and fixed concrete filled steel tube (CFST) building frame, respectively. In-fill double skin composite wall and building frame in CFSWS are attached together by innovative connection systems. The heavily instrumented four model tests under monotonic lateral loading will provide information on shear strength/stiffness, stress-strain development and failure modes of CFSWS. In addition, two tests were also conducted under monotonic lateral loading on one and two storey fixed CFST frames to investigate shear strength contribution of these frame in CFSWS. Moreover, analytical models were developed for the shear resistance of CFSWS. The main objectives of this research are to:

- 1) Perform experimental work on the composite framed shear wall system under monotonic loading to obtain comprehensive information on strength, deformation, stress-strain development and possible modes of failure.
- 2) Carryout analytical and design oriented analyses to develop design equations for the shear resistance of the CFSWS.

1.4 Outline of the Thesis

This thesis consists of six chapters which may be outlined as follows:

Chapter 1 presents brief introduction about the composite framed shear wall system, research significance and objectives of the study.

Chapter 2 presents a comprehensive literature review on different types of composite shear walls namely steel plate shear wall, profiled steel sheet shear wall, composite shear wall, and double skin profiled composite shear wall.

Chapter 3 provides detailed outline of the experimental program, objectives of the experiment, description of the model specimens, fabrication procedures, development of experimental test setup and material properties. The instrumentation and data acquisition as well as the loading procedure are discussed in detail in this chapter.

Chapter 4 presents the experimental results on four one and two storey CFSWS and two bare one and two CFST frames under lateral loading. The observations during the test and the test results are presented in the format of load versus displacement, principal strains & stresses versus load, shear strains & stresses versus load and failure modes of model specimens. Summary of the results and comparison between one and two storey (pinned and fixed framed) CFSWS are also presented.

Chapter 5 presents analytical models to predict the shear resistance of the composite framed shear wall system. The results are compared with the experiment findings.

Chapter 6 presents the conclusions of the study along with some recommendations for future work on the composite framed shear wall system.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Shear walling systems are used to resist lateral loads which are capable of effectively bracing a building against both wind and earthquake forces. Reinforced concrete shear walls have been used in building structures for many years to brace the building from lateral loads. In less than 40 years, other types of shear walling system were developed to brace steel structures from lateral loads. Different types of framed shear wall system consisting of an infill panel connected to the surrounding beam-column frame were developed. The infill panel can be made of flat steel plate, profiled steel plate, reinforced concrete wall, or double skin profiled composite wall. The infill panels are installed in one or more bays for the full height of a building to form a stiff cantilever wall. In this chapter, recent investigations and developments on different types of shear walls are presented.

2.2 Steel Plate Shear Wall

Since the early 1970s, many researchers investigated the behaviour and design of steel plate shear walls. The primary design of steel plate shear walls in early 1970s were based on the concept of preventing shear buckling of infill plates under lateral loads, and therefore using heavily stiffened steel plate and neglecting any post-buckling strength. The first major work on the post-buckling behaviour of shear panels was performed by Wagner (1931) based on some tests on thin aluminum shear panels used in aircraft and that research led to the development of the pure diagonal tension field theory. When steel shear panels are designed accurately, the load resisting mechanism changes from in-plane shear to an inclined tension field after buckling. In recent years, the idea of utilizing the post-buckling strength with the use of thin un-stiffened steel plates has been widely accepted by many researchers in Canada (Driver et al. 1998a,b; Timler et al.1998; Sabouri-Ghomi et al. 2005) and in the United States (Elgaaly et al. 1993; Caccese et al. 1993; Xue and Lu 1994; Astanek-Asl 2001a, b; Bruneau et al. 2007).

Research on the walls with openings in the infill plates has gained attention from researchers since perforations in the infill plates can significantly weaken the infill plates and hence reduce the design forces on boundary columns (Roberts and Sabouri-Ghomi 1992; Vian 2005).

The following paragraphs are about the previous research on thin un-stiffened steel plate shear walls. The development of the strip model is presented first and followed by analytical and experimental research on single/multi-storey steel plate shear walls under monotonic and cyclic loading.

Thorburn et al. (1983) studied thin un-stiffened steel plate shear wall to develop an analytical model based on the theory of pure diagonal tension for shear resistance of the shear wall. The shear strength of the panel before buckling was neglected and the tension field action was assumed as the load resisting mechanism. They proposed the “strip model” where the infill plate was modeled with a series of inclined pin-ended strips.

The orientation of each strip was in the same direction as the principal tensile stresses in the infill plate and each strip was assigned an area equal to the width of the strip multiplied by the plate thickness as shown in Figure 2.1.

Figure 2.1: Strip model for a typical panel (Thorburn et al. 1983)

The interior beams were assumed infinitely stiff and the beams are pin connected to the columns. The inclination angle of the tension field was obtained using the principle of least work. Only the energy from the tension field and axial energy in the beams and columns were considered. The angle of inclination of the tension field, α , can be derived from the following equation:

$$\tan^4 \alpha = \left[\frac{1 + \frac{Lt_w}{2A_c}}{1 + \frac{ht_w}{A_b}} \right] \quad (2.1)$$

where L is the frame bay width, t_w is the infill plate thickness, h is the frame storey height and A_c and A_b are cross-sectional areas of the storey column and beam, respectively. Thorburn et al. (1983) also performed analytical studies to determine the number of required strips to efficiently model the behaviour of the infill plate, and they concluded that 10 strips would be enough to represent an infill panel.

Timler and Kulak (1983) performed tests on a pair of single storey large-scale steel plate shear wall to verify the strip model proposed by Thorburn et al. (1983). The major area of interest was the tension field development in the steel plate. The detail of the specimen is shown in Figure 2.2.

Figure 2.2: Detail of large-scale steel plate shear wall specimen (Timler and Kulak 1983)

The specimen was tested to both service and ultimate load. A cyclic loading up to the allowable deflection limit of $(h/400)$ was also used. It was found that the bending stiffness of the columns affects the value of the angle of inclination of the tension field. Eq. (2.1) which was originally developed by Thorburn et al. (1983), was revised as follows:

$$\tan^4 \alpha = \left[\frac{1 + \frac{Lt_w}{2A_c}}{1 + ht_w \left(\frac{1}{A_b} + \frac{h^3}{360I_cL} \right)} \right] \quad (2.2)$$

where I_c is the moment of inertia of the column and the other variables were defined earlier.

The strip model proposed by Thorburn et al. (1983) and the modified angle of inclination of the tension field proposed by Timler and Kulak (1983) have been adopted by the Canadian standard, CAN/CSA S16-09 as a simple approach for the analysis of unstiffened steel plate shear walls (CSA-S16, 2009). Strip model has also been applied to determine strength and stiffness of plain steel plate shear wall (Hossain 1995).

Roberts and Sabouri-Ghomi (1992) conducted a series of quasi-static cyclic loading tests on steel plate shear panels with circular opening at the center of the panel. The specimens had a constant depth (d) of 300 mm, width (b) of either 300 mm or 450 mm (aspect ratio was 1 or 1.5) and panel thickness of either 0.83 mm or 1.23 mm. The diameter of the central circular openings (D) varied from 0 to 150 mm. The edges of the shear panel were clamped between pairs of pin-ended, steel plate frame members by two rows of 8 mm diameter, high-tensile bolts as shown in Figure 2.3. The corners of the plates were cut away in a circular arc.

Figure 2.3: Test setup; (a) Perforated shear panel, (b) Hinge detail (Roberts and Sabouri-Ghomi 1992)

Two diagonally opposite pinned corners were connected to the hydraulic grips of a 250 kN servo hydraulic testing machine, where the loading was applied diagonally. Based on the quasi-static test results, the researchers recommended that strength and stiffness of a perforated panel can be conservatively approximated by applying a linear reduction factor $(1-D/d)$ to the strength and stiffness of a similar imperforated (solid) panel.

Xue and Lu (1994) conducted a finite element analysis using ADINA software on a twelve-storey, three-bay frame where the middle bay was infilled by steel plate shear panel. The frame wall system was designed for code specified seismic loading. Four different frame configurations having identical frame members with different connection arrangements were studied. For each case, the exterior bays had moment resisting beam-to-column connections and the interior bay had infill plates in every storey. It was found that the behaviour of a frame-shear wall system is significantly affected by the steel plate panel connections to the frame members due to the influence of the connections in developing the tension field action in the panels. Based on all the analysis of results, Xue and Lu (1994) recommended that the full moment frame with the steel plate shear panel connected to the girders is the most suitable structural system that can provide higher ductility and more stable strength. No experimental tests were performed to verify such conclusion.

Figure 2.4: Steel plate shear wall test specimen (Driver et al. 1998a)

Driver et al. (1998a) performed a quasi-static cyclic testing on a single bay, four-storey, large-scale, unstiffened steel plate shear wall specimen to evaluate the overall in-plane performance of the wall under extreme cyclic loading. The specimen had beam-to-column moment resisting connections and the infill steel plates were welded to the boundary members using fish plates. The test specimen had the overall height of 7.4 m (first storey height of 1.93 m and a typical storey height of 1.83 m) and width of 3.05 m (between column centrelines) as shown in Fig 2.4. The columns at base were welded to a 3800 x 800 x 90 mm steel base plate using fillet welds at the webs and full penetration groove welds at the flanges. The fish plate for the lower story was also welded to the base plate using fillet welds.

The infill steel plates for the first and second storey were nominally 4.8 mm thick and for the third and fourth storey were nominally 3.4 mm thick. Equivalent cyclic lateral loads were applied at each floor level, as per the requirements of ATC-24 (ATC 1992). A total of 30 load cycles were applied to the test specimen prior to failure, and the last 20 cycles were in the inelastic range. The maximum deflection was achieved nine times the deflection at the yield

deflection. The test specimen was found to have a good ductility and high energy dissipation capability.

Figure 2.5: Three-storey steel plate shear wall test specimen (Driver et al. 1998a)

Driver et al. (1998b) also developed a finite element model using ABAQUS software to analyse their experimental test specimen. The infill steel plates were modelled with shell elements and beams and columns were modelled with beam elements. The residual stresses for the boundary members which were measured experimentally also incorporated in the model. The material properties were assumed to be identical in tension and compression and the von-Mises yield surface was adopted as the yield criterion. Both static pushover and cyclic analyses were conducted. A kinematic hardening rule was invoked to simulate the effect in the cyclic loading analysis. An excellent prediction of the ultimate strength was observed for the monotonic analysis, but the pinching of the hysteresis loops due to buckling and redevelopment of the tension field in the load versus displacement curves were not captured for the cyclic analysis.

Behbahanifard (2003) conducted a quasi-static cyclic testing on a large scale, three-storey un-stiffened steel plate shear wall specimen, as shown in Figure 2.5. The specimen was under

gravity load and lateral cyclic loading. The cyclic loading sequence was based on the requirements of ATC-24 (ATC 1992) which was similar to that used in the test by Driver et al. (1997). 14 cycles out of 24 cycles were in inelastic range. Characteristic pinching of hysteresis loops was observed in the inelastic range. The specimen showed high initial stiffness, very good ductility and energy absorption capacity, and stable hysteresis loops.

A nonlinear finite element model based on explicit formulation was developed in ABAQUS to simulate the monotonic and cyclic behaviour of both Behbahanifard (2003) and Driver et al. (1997) specimens. A four node shell element with reduced integration (ABAQUS element S4R) was used for steel plates. Residual stresses were not included in the finite element model for simplicity. For the cyclic analyses, a kinematic hardening material model was included to simulate the effect. The capacity of the shear wall based on the finite element analyses was 12% for three-storey specimen and 7.8% for four-storey specimen less than experimental results. A parametric study was conducted with the validated finite element model to investigate some non-dimensional parameters affecting the behaviour of a single storey steel plate shear wall. It was found that an increase in the aspect ratio would generally decrease the capacity of the wall.

Berman and Bruneau (2003) derived equations based on the concept of plastic analysis and the strip model, to calculate the ultimate strength of single and multi-storey steel plate shear walls. Both simple and rigid beam-to-column connections were studied. For a single storey shear wall with simple beam-to-column connections, the storey shear strength, V_{yp} can be derived as follows:

$$V_{yp} = 0.5 F_y t_w L \sin 2\alpha \quad (2.3)$$

where F_y is the yield strength of the steel plate and all other parameters have been defined earlier. Equation 2.3 was modified (Berman and Bruneau 2003) for a frame with rigid beam-column connections by adding the components of internal work from plastic moments in the columns or beams as follows:

$$V_{yp} = 0.5 F_y t_w L \sin 2\alpha + 4 M_p / h \quad (2.4)$$

where M_p is the smaller of the plastic moment capacity of the columns or beams and h is the frame storey height.

2.3 Profiled Steel Sheet Shear Walls

Profiled steel sheets have been used in floor construction since the early 1940s (Wright et al. 1987). The profiled steel sheet compared to flat steel plate with the same thickness have a significant out of plane stiffness in the direction of the corrugations which can carry gravity loads as well as in-plane shear loads. Many researchers have conducted studies on the rigidity, shear strength, diaphragm behaviour and types of connection of the profiled steel deck to the top flange of floor/roof beams (Bryan and El-Dakhakini 1968; Hussain and Libove 1976; Davies and Lawson 1978; Davies 1977). The shear buckling capacity of a profiled steel sheet with simply supported edges can be expressed by the following equation (Easley and McFarland 1969; Easley 1975):

$$V_{cr} = 36L \frac{D_x^{0.25} D_y^{0.75}}{h^2} \quad (2.5)$$

where h , L = Height and width of profiled steel sheet, respectively;

$D_x = \frac{E_s t_s^3}{12\alpha}$, $D_y = \frac{E_s I_y}{d}$ are the orthotropic constants;

E_s , t_s = Modulus of elasticity and thickness of profiled steel sheet, respectively;

d = Original length of one repeating profiled steel sheet as shown on Figure 2.6;

α = Ratio of extended length of one repeating profile to its original length (d);

I_y = Moment of inertia of one repeating profiled steel sheet about “y” axis (Figure 2.6)

Figure 2.6: Neutral axis and length of one repeating corrugation of profiled steel sheet

El-Dakhakhi (1976) was one of the earlier researcher that proposed the application of the profiled steel sheets as vertical partitions in tall buildings to resist shear load. Five equations for shear flexibility of partitions were derived based on shear deformation of sheet, axial deformation of edge members, bending of corrugation profiles, crimping at edge fasteners and crimping at seam fasteners.

Hossain and Wright (1997) studied the in-plane shear behaviour of a profiled steel sheet panel. Small scale panels (560 mm x 560 mm) were tested to investigate the behaviour of the profiled steel sheet panels under pure shear forces along a diagonal of the panel. The experimental test setup is shown in Fig 2.7. Analytical models for shear strength and stiffness of the profiled steel sheeting was developed and validated by small scale model test results and finite element analysis.

Figure 2.7: Shear rig frame and test setup (Hossain and Wright, 1997)

The stiffness and strength of the profiled sheeting is found to be dependent on the manner of attachment of the sheeting to the boundary frame. Due to unstable and very rapid post buckling behaviour of the panel, it was suggested that it is unwise to use the post buckling shear reserves. The values of several factors related to the mode of attachment of the sheeting to practical building frames were also studied to verify the suitability of design equations. Same as Bergmann and Reissner's works, the researchers suggested an equation for the critical shear buckling load of a profiled steel sheet as follows:

$$V_{cr} = 36\beta L \frac{D_x^{0.25} D_y^{0.75}}{h^2} \quad (2.6)$$

where β , is a coefficient depends on connection type of sheet to the boundary frame ($1 \leq \beta \leq 1.9$) and the other variables were defined earlier.

Also based on Davies and Bryan works on floor diaphragm design (Davies and Bryan 1982), the researchers suggested for the design purpose a 25% reserve of safety due to the sudden buckling failure. Therefore, the Eq. (2.6) for design purpose was modified as follows:

$$V_{cr} = 28\beta L \frac{D_x^{0.25} D_y^{0.75}}{h^2} \quad (2.7)$$

Vora and Yu (2008) investigated the behaviour and shear strength of cold-formed steel framed shear wall assemblies with 0.027 inch (0.69 mm) corrugated steel sheet thickness and 9/16 inch rib. The parameters which were studied in the test program were the fastener size and spacing and the framing member thickness under both monotonic and cyclic loading. The experimental test results showed that with appropriate fastener configuration and the framing members, the corrugated steel sheet can have more than two times higher strength compared to the same framed walls with 0.027 inch flat steel plate. The researcher recommended utilizing the 0.027 inch corrugated sheet in the lateral resisting system of buildings.

2.4 Composite Shear Wall

The term “composite shear wall” can be referred to shear wall which consists of reinforced concrete panel cast between steel columns (as defined by Furlong 1996) or composite shear panel attached between steel columns. Recently many researchers from all over the world mostly from United States, China, South Korea, Britain and Canada pay attention to composite shear wall (Astaneh- Asl 2002; Eom et al. 2009; Liang et al. 2004).

Figure 2.8: Types of studied composite shear wall (Astaneh-Asl 2002)

In this study, the focus is on the composite shear wall made of steel and concrete as a shear panel attached to steel columns. The composite shear panel consist of one or two steel plates with reinforced concrete which can be precast or cast in place as shown in Figure 2.8. The steel plate can be connected to a precast reinforced concrete panel by using mechanical connectors like bolts and also can be attached to cast in place reinforced concrete by

using shear studs or bonding a fiber reinforced polymer sheet (Rahai and Hatami 2009).

In a composite shear wall, the concrete part restrains the steel plate and prevents the steel plate from buckling before yielding of steel. Therefore, the steel plate shear wall can resist the storey shear by yielding in shear. It should be noted that the shear yield capacity of the steel plate is significantly greater than its capacity to resist shear in yielding of diagonal tension field (Astaneh-Asl 2002). The main disadvantage of an unstiffened steel plate shear wall is the buckling of the compression zone of the shear wall, which results in reduction of the stiffness, strength and energy dissipation capacity (Alinia and Dastfan 2007). In a composite shear wall, concrete panel also can resist shear by developing diagonal compression field. Thus, the composite shear wall can have the most benefits of both concrete panel and steel plate shear wall.

Zhao and Astaneh-Asl (2004) carried out experimental studies on two half-scale, one bay, three- storey composite shear wall specimens. The composite shear wall consisted of a reinforced concrete shear panel bolted to one side of a steel plate shear wall. The bay span and storey height of the specimens were 2.1m. The thickness of the reinforced concrete shear panel and the flat steel plate were 76 mm and 4.8 mm, respectively.

The properties of the specimens were identical except that in the first specimen there was a 32 mm gap between the reinforced concrete panel and the surrounding steel frame and in the second specimen there was not any gap. The authors considered the first specimen as “innovative” composite shear wall while the second specimen represents “the Traditional” composite shear wall. The components of the test setup as shown in Figure 2.9 are the actuator, loading beam at the top, reaction beam at the bottom, R/C reaction blocks, and the specimen.

Rahai and Hatami (2009) conducted experimental tests on the effects of shear connector spacing variation on the composite shear wall behaviour. The experimental tests were performed on three small scale, single bay, and one storey composite shear wall specimens consisting of 3.3 mm thick steel plate and 50 mm thick reinforced concrete panel attached to one side of the

steel plate by bolts. The steel plate was connected to reinforced concrete panel by 9, 12, 24 bolts which the vertical bolts spacing varied from 222 mm to 777 mm and the horizontal bolts spacing was 222 mm for all three specimen. Cyclic loadings were applied at top of the specimens. It was observed that the decreasing distance between the bolts increases the absorbed energy in the shear wall and reduces the value of the out of plane displacement of steel.

Figure 2.9: Experimental test setup and detail of the test specimen
(Zhao and Astanesh Asl 2004)

Rahai and Hatami (2009) also performed numerical modeling of the composite shear wall using the ANSYS finite element software. The model was calibrated and verified based on the two laboratory models at Berkeley and Alberta Universities. The shear connectors were modeled using a three dimensional beam element with 6 degrees of freedom. The steel plates, beams and columns were modeled by shell element with four nodes and six degrees of freedom per node. A solid element with eight nodes, three degrees of freedom per node for translation was used for the reinforced concrete.

The following assumptions were made to investigate the effect of the shear connector spacing on the shear wall behaviour:

- the friction between steel and concrete was neglected, and
- the steel was considered to have a bilinear behaviour.

A good agreement was observed in the hysteresis curves between numerical model and experimental results.

2.5 Double Skin Profiled Composite Shear Wall

Composite slab consisting of a profiled steel sheet and cast in place concrete have been used in floor construction for many years. In the building flooring system, the profiled sheet in composite slab can act as both permanent formwork and also as tensile reinforcement to a cast in place concrete slab. Also because of a significant out of plane stiffness of the profiled steel sheet in the direction of the corrugations, the distance between the shoring for casting concrete will be minimized. Many researchers have conducted studies on the rigidity, shear strength, diaphragm behaviour of the profiled composite slabs (Schuster 1976, Davies and Fisher 1979, Easterling and Porter 1994a, b).

The idea of application of double skin profiled composite wall in steel structure building to carry gravity loads was earlier proposed by Wright et al. (1992). Double skin profiled composite wall comprises of two layers of embossed profiled steel sheeting and in-fill of concrete. The profiled steel sheets act as a permanent formwork for the concrete. Once the concrete hardens, the profiled steel plate and the concrete act as a composite element. The concept of this new walling system came about as a natural extension to the use of profiled steel sheeting in floor construction. Wright et al. (1992) stated some of the advantage of using this system as follows:

- temporary formworks are avoided,
- lateral stability of the steel frame can be achieved when the profiled steel sheets are fixed,
- bar reinforcement and its fixing are avoided.

Many studies were conducted experimentally and analytically on axial behaviour of the double skin profiled composite wall (Wright and Evans 1995, Wright and Gallocher 1995,

Wright 1998a), axial behaviour of pierced profiled composite walls (Hossain 2000), and axial and bending behaviour of the composite walls (Wright 1998b).

Wright et al. (1994) extended the application of the double skin profiled composite wall to resist the lateral loads in tall structures. In the building construction stage, profiled steel sheets act as a formwork for the infill concrete and also as a bracing system to the building frame. In the service stage, it also acts as reinforcement. Analytical equations for shear flexibility of the profiled steel sheet and concrete core were derived and combined to get shear flexibility and stiffness for the composite panel. Also the analytical results were compared with the linear three dimensional finite element analysis results and a very good agreement was observed.

Hossain and Wright (2004b, c, d) conducted an experimental work on small scale tests on double skin profiled composite shear wall to obtain information on the load-deflection response, stiffness, strength, sheet-concrete interaction and failure modes. The overall dimension of one-sixth scale specimen was 620 mm x 620 mm which provide the effective dimension of 560 mm x 560 mm. The experimental test setup was similar to the test setup used for profiled steel sheet panels (Figure 2.10). The 0.45 mm thick profiled steel sheets with no embossment were manufactured in-house from plain sheets.

Figure 2.10: Detail of profiled composite panel (Hossain and Wright 2004d)

The maximum thickness of concrete panel was 30 mm as shown in Figure 2.10. Tests on composite walls, concrete panels and profiled steel panels were conducted. The composite panel were connected to the test frame through intermediate bolts which also provided the mechanical connection between the pair of sheeting and concrete core. The specimens were loaded diagonally and monotonically which cause pure shear condition within the panel.

It was found that the behaviour of the composite walls were dependent on the interaction between concrete core and steel sheets. The interface connection between concrete and steel was mainly due to the chemical bond due to the absence of embossments in the sheeting. The composite wall exhibited higher ductility, strength and stiffness compare to its separate components.

Hossain and Wright (2004d) derived analytical equations for shear stiffness and shear strength and also developed a finite element model for the composite wall. If the boundary frames were strong enough compared to the composite panel, the shear strength of the composite wall can be controlled by four failure limit states as follows:

- Diagonal tension concrete limit state where the strength of the concrete core was controlled by diagonal tension failure,
- Steel sheet limit state which is mostly governed by the overall shear buckling of the sheets if the connections between frame and sheet are strong enough,
- Sheet-concrete shear transfer limit state which in this study was only relied on the chemical bond between the sheets and concrete, and
- Wall-frame connections limit state by which a connection failure can limit the strength of the composite panel.

In general for practical design consideration, the wall-frame connections should be strong enough to induce failure to the composite panel. The lower bound ultimate shear capacity of the composite wall was conservatively proposed by the summation of individual shear resistance of the sheets (two times of Eq. 2.6) and shear resistance of concrete panel as per Kupfer and Gerstle (1973) model:

$$V_u = 72\beta L \frac{D_x^{0.25} D_y^{0.75}}{h^2} + \frac{f'_c f'_t}{f'_c + f'_t} t_{eq} L \quad (2.8)$$

where f'_c is the cylinder compressive strength of concrete, f'_t is the splitting tensile strength of concrete, t_{eq} is average thickness of concrete panel and the other variables were defined earlier.

The authors suggested that β value can be assumed equal to 1.72 and also in the above equation the interaction between steel and concrete (composite action) was not considered.

The finite element modelling of the composite wall was performed using proprietary LUSAS software to simulate the actual experimental conditions (Hossain and Wright, 2004d). The three dimensional 8-noded QSL8 semi-loof shell elements with the provision of different layers were used in the modeling. Only a symmetric half thickness of the wall was modeled. Concrete and steel were represented as different layers of the element. The load was applied as a prescribed displacement. The finite element results were found good in simulating the strength, stiffness and characteristics of the profiled steel sheet and the concrete panel. The FE models with interface layers and full composite action could reasonably predict the initial stiffness and strength of the composite wall.

Figure 2.11: Crack patterns in concrete core of composite wall

Hossain and Wright (2004a) performed experimental tests on composite wall specimens under cyclic loading. The test setup is shown on Figure 2.7. The cyclic loads were applied diagonally.

The load increment was 6 kN in each cycle up to a load of 60 kN. Beyond this load, an increment of 30 kN was used until failure of the panel. The crack patterns in concrete are shown in Figure 2.11.

Hossain and Wright (2004a) introduced a reduction factor (ϕ) to Eq. (2.7) to take into account the reduction in strength due to cyclic loading. The value of ϕ was suggested to be varied from 0.73 to 0.8.

Hossain et al. (1998a; 2005a, b) studied the shear interaction between the profiled steel sheets and concrete core in the composite shear wall. The interaction between the steel sheets and concrete plays important roles in the composite action of the wall. The shear composite action between concrete and the steel sheets was assessed through a comparative study of their individual action. From the experimental work, it was found that the chemical bond between steel sheets and concrete is not significant and can be neglected. Also the boundary connection between steel sheets and concrete have important role in the shear interaction when there is no mechanical shear bond at the middle of the composite panel. When the load is applied through both concrete and steel sheet with adequate boundary connection, the composite wall exhibits high shear resistance compared to the wall with flexible connection or if the load is applied through concrete only. The failure of the composite wall started after debonding of the steel sheets and concrete due to failure of chemical bond by buckling of the steel sheets from the concrete. In the final stage, the steel sheets slide over the profiled concrete panel and extended tension field caused the profiled steel sheets to twist and lose its profiled geometry.

The researchers also developed four finite element models using proprietary LUSAS software to simulate the complex non-linear shear interaction at the steel sheets-concrete interface with various interface elements. Non-linear interface/joint elements between concrete and steel were

introduced to simulate partial composite action that allows concrete-steel to have in-plane slip or out of plane separation. The properties of the interface/joint elements were optimized through an extensive parametric finite element analysis using experimental results to achieve accurate simulation of the actual concrete-steel interaction in a composite wall. The results of the developed finite element models were validated through testing one-sixth scale model. The FE models were found to simulate stiffness, strength and strain characteristics of the wall reasonably well. Moreover, the proposed finite element models were suggested to be used to simulate the shear behaviour of composite walls in practical situations.

Research consisting of experimental, analytical and finite element studies analysis has been conducted to investigate behaviour of double skin profiled composite shear wall system under in-plane monotonic, cyclic and impact loadings (Rafiei et al. 2013, Rafiei 2011). The researchers prepared eight composite wall specimens with overall dimensions of 1626 mm (height) x 720 mm (width) and tested. Steel sheet-concrete connections were provided by intermediate fasteners to generate composite action. Two types of steel sheets were classified based on the strength as mild and high strength. Two types of concrete-infill namely Self-Consolidating Concrete (SCC) and Engineered Cementitious Composites (ECC) were used to construct the walls. The researchers also developed an analytical model for the shear resistance of the composite wall based on existing models taking into account the shear capacity of the steel sheets, concrete core and steel sheet-concrete interaction. Moreover, two non-linear finite element models for the composite wall under monotonic/cyclic and impact loading were developed using proprietary ABAQUS/CAE software.

The advantages of using ECC over SCC as profiled concrete core were exhibited through demonstration of more ductile behaviour and better crack development characteristics. After testing in SCC core, continuous lines of cracks were developed compared with scattered and discontinuous lines of cracks in ECC core. It was also recommended that the load associated with the initiation of concrete cracking and de-bonding of the profiled steel sheet from the concrete core was identified from the first sudden increase of strain in the experimental shear strain response graphs (Rafiei et al. 2013, Rafiei 2011). This load was very clear for the

specimens made of SCC but not for ECC. The reason might be due to the composition of ECC, where fibres prevent brittle cracking of concrete.

Rafiei et al. (2013) derived shear resistance of the profiled concrete core based on the simplified modified compression field theory (SMCFT). The analytical model for the shear resistance of the composite wall was derived based on the combined strength of profiled concrete core and double skins of profiled steel sheets. The difference between the experimental and analytical shear resistance was found to be less than 10%. Based on experimental testing, the researchers also concluded that use of ECC compared to SCC in composite wall improved the wall displacement ductility by 13% but it was still low compared to the ductility of other types of shear wall such as steel plate shear wall (Rafiei et al. 2013, Rafiei 2011).

2.5.1 Infill Concrete in the Composite Shear Wall

In order to have a double skin profiled composite shear wall, an infill concrete is needed. Hossain and Wright (1994) and Hossain (1995) studied the use of micro concrete to simulate the in-plane shear behaviour of composite wall using small-scale models. Due to the 14 mm thickness of composite panel specimens, a gap-graded micro-concrete was used to manufacture in-fill concrete core. The micro-concrete was machine mixed and poured into the mould by hand using a spatula. During casting, it was compacted on a vibrating table in different layers. Control specimens in the form of cubes and cylinders were cast at the same time. The wall panels were removed from the moulds after 4 or 5 days and then cured in air until testing. The control specimens were taken out of the moulds after 24 hours and then cured in air.

2.5.1.1 Self-Consolidating Concrete (SCC)

Self-Consolidating Concrete (SCC) is a pre-blended, high performance, flowable concrete material containing Portland cement, silica fume, 10 mm (3/ 8 inch) stone and other carefully selected admixtures. The SCC has superior plastic properties provide a fluid mix with self-consolidating characteristics without bleeding or segregation and can easily consolidated without rodding or vibrating. SCC achieves good consolidation without external or internal vibration and

also without defects due to bleeding or segregation. SCC can be used to improve the productivity of casting congested sections and also to insure the proper filling of restricted areas with minimum or no consolidation (Khayat 1999). SCC can improve the working environment by eliminating the noise and pollution caused by vibrators and also reduces labour cost.

SCC was developed in Japan in the early 1980's (Hayakawa et al. 1993). Three major factors for developing SCC are as follows:

- Demand for flowable concrete to compensate proper filling within the heavily reinforced seismic members,
- Decrease the number of skilled construction workers,
- Reduce the time and cost of construction.

Several different approaches can be used to develop SCC. One method is to increase significantly the amount of fine materials such as fly ash and slag cement without changing the water content compared to common concrete. An alternative approach to design SCC is incorporating a viscosity modifying admixture (VMA) to improve the stability of the SCC. Viscosity modifying admixtures are water soluble polymers which enhance the ability of cement paste to retain its constituents in suspension and also increase the viscosity of the mixture. Using the VMA with super-plasticizers can ensure adequate workability without segregation.

Bouzoubaâ and Lachemi (2001) evaluated the SCC made of high volumes of Class F fly ash. Nine SCC mixtures and one control concrete were studied. The content of the cementitious materials was maintained constant (400 kg/m^3), while the ratio of water/cementitious material ranged from 0.35 to 0.45. The self-compacting mixtures had a cement replacement of 40%, 50%, and 60% by Class F fly ash. The tests were performed on all mixtures to obtain the properties of fresh concrete in terms of stability and viscosity. The SCCs developed 28-day compressive strengths ranging from 26 to 48 MPa. The results showed that an economical SCC could be successfully developed by incorporating high volumes of Class F fly ash.

Lachemi et al. (2003) studied twenty-one concrete mixtures to investigate the performance of

three types of SCC manufactured with Fly Ash, slag cement, and various VMAs based on mechanical properties and also on cost. Fly Ash SCC mixtures had cement replacement of 40, 50, and 60%, while slag cement SCC mixtures had 50, 60, and 70% replacement. The water-cementitious material ratios (w/cm) ranged from 0.35 to 0.45. Three different VMAs were used in VMA SCC mixtures with w/cm of 0.45. Tests were carried out on all mixtures to obtain mechanical properties such as compressive strength. The results indicated that an economical SCC with desired properties could be successfully developed by incorporating FA, slag cement, or VMA. It was found that these SCC could replace the control concrete and could be more economical (30 to 40% in case of FA and slag cement). Although the cost of VMA SCC was slightly higher than those with FA and slag cement, it had more resistance to segregation and had higher early strength development.

SCC is designed with limits on the nominal maximum size (NMS) of the aggregate, the amount of aggregate and aggregate grading to achieve a high workability and avoid obstruction by closely spaced reinforcing. However, when the workability is high, the potential for segregation and loss of entrained air voids increases. These problems can be alleviated by designing a concrete with a high fine-to-coarse-aggregate ratio, a low water–cementitious material ratio (w/cm), good aggregate grading, and a high-range water-reducing admixture.

2.6 Summary and Conclusions

The development of four types of structural shear walls (steel plate shear wall, profiled steel sheet shear wall, composite shear wall, and double skin profiled composite shear wall) to resist lateral loads are discussed. The disadvantage of a steel plate shear wall is the elastic buckling of the shear wall, which results in reduction of stiffness, strength and energy dissipation capacity. The profiled steel sheet compared to flat steel plate with the same thickness, has a significant out of plane stiffness in the direction of the corrugations which can resist higher in-plane shear load but still buckles before yielding. In a composite shear wall, the concrete core restrains the steel plate and prevents the steel plate from buckling before yielding so that the wall can resist the load by yielding in shear. Concrete core can also resist shear by developing compression diagonal field and therefore, the composite shear wall can have the most benefits of both concrete and steel.

Self-Consolidating Concrete (SCC) is very flowable and this remarkable property makes it one of the best choices which can be used in production of the double skin profiled composite shear wall models. SCC can flow into space between two profiled steel sheets under its own weight without internal or external vibration and also without defects caused by bleeding and segregation.

Based on literature review, the concept of using composite framed shear wall system (CFSWS) is found to be relatively new and no research has been conducted on the behaviour of this type of system associated with building frames. Therefore, the study of CFSWS under monotonic lateral loading with interface connections is warranted to have better understanding of the structural behaviour of this type of system and to develop design guidelines. This research is aimed at contributing significantly to these needs in the area of CFSWS technology. In addition, two types of one and two storey CFSWS (having pinned and fixed boundary frames) in-filled with double skin profiled composite wall with concrete core will be used to investigate strength, stiffness, ductility, failure modes and load transfer mechanisms.

CHAPTER THREE

EXPERIMENTAL PROGRAM

3.1 Introduction

This chapter describes the development of experimental test setup in order to perform tests under monotonic lateral loading. Total four models/specimens had been fabricated to analyze the behaviour of composite profiled shear wall bounded by pinned and fixed boundary connection frames. The fabrication of composite framed shear wall system (CFSWS) is explained in details. Two fixed (one and two storey) bare frames constructed to study the frame contribution in the CFSWS are also explained.

3.2 Geometric Dimensions and Material Properties CFSWS models

The dimensions of the CFSWS models were chosen based on the capacity of the existing loading facilities and the feasibility of fabrications. The CFSWS consists of a composite shear wall panel fastened to the pinned steel frames and fixed concrete filled steel tube (CFST) frames, respectively as shown in Figure 3.1. CFSWS consists of composite wall panel fastened to one and two storey frames (pinned and fixed) by bolted and welded connections as shown in Figure 3.2. The detail geometric and material (concrete and steel) properties with other characteristics of the CFSWS and bare frames are presented in Table 3.1.

3.2.1 Double Skin composite Wall Panel

Composite wall panel consists of two profiled steel sheets on both sides and an infill concrete. Composite action between steel sheeting and concrete was provided by intermediate fasteners. It was decided to use commercial P-3012 Steel Deck manufactured by Canam Group to fabricate the composite shear wall specimens. The flutes of the P-3012 deck are 14 mm deep and are spaced at 64 mm center to center as shown in Figure 3.2.

Figure 3.1: Components of one and two storey (pinned & fixed) CFSWS

Table 3.1: Details of geometric and material properties

Test Specimen Identification	Width X Height (mm X mm), (No. of Storey)	Type of Model Frame	SCC properties (mean values) MPa			Profile steel sheet properties (mean values) Stress and modulus of elasticity in MPa				
			f'_c	f'_t	E_c	Yield		Ultimate		Modulus of elasticity (E_s)
						Stress	Strain	Stress	Strain	
CFSWS1	640 X 640 (one Storey)	Steel pinned	46.77	3.2	25100	552	0.00272	577	0.01	202940
CFSWS2	640X1280 (two Storey)	Steel pinned	45.95	3.2	24500	552	0.00272	577	0.01	202940
CFSWS3	640 X 640 (one Storey)	CFST fixed	47.10	3.2	25010	552	0.00272	577	0.01	202940
CFSWS4	640X1280 (two Storey)	CFST fixed	46.50	3.2	25010	552	0.00272	577	0.01	202940
CFST-F1 (bare)	640 X 640 (one Storey)	CFST fixed	47.25	3.3	25010	552	0.00272	577	0.01	202940
CFST-F2 (bare)	640X1280 (two Storey)	CFST fixed	47.25	3.3	25010	552	0.00272	577	0.01	202940
f'_c = Cylinder strength of concrete, f'_t = Tensile strength of concrete, E_c = Modulus of elasticity of concrete and E_s = Modulus of elasticity of profiled steel sheet.										

The standard width and length of the steel deck are 762 mm and 6,200 mm, respectively. The thickness of the type 24 steel deck is 0.61mm. The yield strength of the profiled steel sheet measured from the mean value of three coupon test is 552 MPA as presented in Table 3.1. The profiled steel sheet used in the composite framed shear wall system models was Cold-Formed Steel (CSA-S136 2007). The profiled steel sheet was made of high strength steel which caused brittle failure and low ductility. The profiled steel sheet has yield stress (F_y) of 552 MPa, ultimate strength of 577 MPa, low ductility (elongation = 9.7%) and modulus of elasticity of 202,940 MPa.

It was decided to use Self-Consolidating Concrete (SCC) based on aggregate size and concrete flow ability due to demand of small scale ($1/6^{\text{th}}$) test models and other dimensions of the composite wall panels. The coarse aggregate were of under 10 mm in diameter. The thickness of the composite panel was average 20 mm at the troughs and 40 mm at the crests (Figure 3.2).

Figure 3.2: Wall cross sectional details with pinned steel frame section

Three rows of intermediate fasteners having four columns at each row at 128 mm c/c providing a total of twelve fasteners for each panel were used to attach the both profiled steel sheets in such a way to keep spacing for infilled concrete as shown in Figure 3.3. Six millimeter diameter bolts were used as intermediate fasteners to prevent the global buckling of profiled steel sheets and also maintain the accurate distance between the two sheets during fabrication, installation and casting of concrete. The plastic pipe had been used as spacer at different locations.

(a) Model CFSWS 4

(b) Cross section of model CFSWS 1

Figure 3.3: Details of frame-wall and intermediate fasteners

3.2.2 Pinned Steel Frame and Fixed Concrete Filled Steel Tube (CFST) Frame

Total six, one and two storey, two pinned steel frames and four fixed concrete filled steel tube (CFST) frames (1/6th scale) have been fabricated with overall dimensions of 640 mm x 640 mm and 640 mm x 1280 mm, respectively.

Pinned steel building frames were fabricated by using steel channels C76 x 6 as columns and beams. These channels were connected to each other by drilling and bolting by 13 mm bolts as shown in Figure 3.4. Due to pinned connection, it was expected that frames will not contribute to strength of CFSWS and will allow uniform shear force transfer from the frame to the composite wall panel boundaries.

(a) Steel channel frame (one storey)

(b) View of beam-column steel frame connection

(c) Two storey pinned steel frame with composite wall

Figure 3.4: Column and beam connection details for pinned steel frames

In order to have a CFSWS, composite walls and building frames were connected together by innovative connection systems. For pinned CFSWS models, the composite wall panels were connected to the frames by bolts as shown in Figure 3.4.

Fixed concrete filled steel tube (CFST) frames (two double and two single storey) were

fabricated by using hollow steel sections (HSS) of 75x75x3 mm and 75x50x3 mm as columns and beams, respectively. Double skin composite walls were connected to the CFST frames (one and two storey) by using 10 mm bolts with special steel plate attachments welded to the frame. Angle pieces were welded to CFST frame to make an innovative boundary connection by bolts running through both steel sheet and concrete to avoid failure due to tearing of steel sheet on application of horizontal lateral force.

Figure 3.5: Welded frames with profiled steel sheets before casting concrete

Concrete filled steel tube (CFST) column and beam members were welded together in the building frames as shown in Figure 3.5. Two bared CFST frames (one double and one single storey) were also fabricated and tested to investigate the contribution of frame strength in CFSWS. These CFST columns and beams were also filled by Self-Consolidating Concrete (SCC). It was expected that fixed building frame will transfer lateral load uniformly to the shear

wall panel and will also contribute to the composite action of CFSWS and overall load resistance.

3.3 Fabrication, Casting and Curing of Composite Walls

The following steps were taken to fabricate each of the composite wall panels:

Cutting of steel sheets as per sizes of boundary frames: Two profiled steel sheets for each wall panel were cut by grinder as per size (640 mm x 640 mm, 640 mm x 1280 mm) of the prepared frames to fabricate double skin composite walls. Cutting of sheets is shown in Figure 3.6 (a).

(a) Cutting of Sheets
by grinder

(b) Drilling holes on Sheets

(c) Installing intermediate
fasteners

Figure 3.6: Cutting and drilling of steel sheet for intermediate fasteners.

Making perimeter and intermediate holes on profiled steel sheets: The holes were drilled in three rows and four columns for intermediate fasteners for each wall panel. External holes for frame-wall fasteners were also drilled to be used for mounting these double steel sheet wall panels in frames. Then both profiled steel sheets were positioned face to face and were aligned and fixed by using clamps to install the six mm bolts with spacer as shown in Figure 3.6. Eight holes of 13 mm diameter were drilled at the corners of columns and beams to connect each other for one storey pinned steel building frame. Twelve holes of 13 mm diameter were drilled at corners of columns and beams for two storey pinned steel building frame as shown in Figure 3.7. Total one hundred and eighty holes of 11 mm diameter (fourteen and eighteen holes for each pinned and fixed panels, respectively) were drilled along the perimeter of profiled wall steel sheets and to

mount wall panel in the frame as shown in Figure 3.7(a,b).

Figure 3.7: Holes drilled in pinned frame beam/column and at outer boundary of wall sheet.

Casting of concrete infill and curing of composite wall and CFST frame: Casting of concrete in small cross sectional size (wall thickness varies from 20 mm to 40 mm of these model specimens was difficult with ordinary Portland cement concrete. Therefore, SCC made of 10 mm maximum size aggregate, Portland cement, silica fume, and admixtures was selected as infill for both composite walls and CFST frames due to its superior flowability and self-compactability. The frames and steel sheets were connected together using all the bolts maintaining correct spacing with the help of spacers. The model assembly was then placed on a wooden platform. Opening around the wall and frame was sealed with the use of foam to control leakage of concrete during pouring as shown in Figure 3.8 (a). The SCC was machine mixed and poured into the wall, column and beam moulds from top of each model as shown in Figure 3.8(b). Holes of 13 mm diameter were drilled on the top of each beam (Figure 3.8c) to pour SCC mortar to fill the beam tube by use of a funnel. After completion of casting, specimens were covered by plastic sheets to control the escape of moisture contents as shown in Figure 3.8d. Control specimens in the form of cubes and cylinders were also cast at the same time to determine the properties of the SCC. After casting, CFSWS and bare CFST frames were cured in air at room temperature until tested. Control concrete cylinders were also cast for compressive and flexure strength at the same time. The cylinder strength (mean value of three specimens) of SCC at the age of the testing for each CFSWS and bare frame is presented in Table 3.1

(a) Wall-frame opening sealed by foam

(b) Pouring of concrete from top

(c) Beams filled by SCC mortar through drilled holes at top of beam

(d) Models covered with plastic sheets

Figure 3.8: CFSWS casting procedure with SCC

3.4 Experimental Test Setup

The main components of the experimental shear test setup are the model CFSWS, a base frame (W-Beam) attached to the floor, data acquisition system and load cell as shown in Figure 3.9.

Figure 3.9: Main components of the experimental test setup

In order to perform the monotonic lateral loading tests, the CFSWS and bare frames were mounted to the strong floor of Structures Laboratory vertically and connected to a horizontal load cell as shown in Figure 3.10.

A strong W-beam base frame was utilized to mount the composite wall-frame assembly to the floor as shown in Figure 3.10. For this purpose, two holes with 46 mm diameter and eight 25 mm diameter holes were made on the W-Beam in order to fix it with strong laboratory floor and CFSWS specimens, respectively. A Unibor magnetic drill was utilized for making these holes on the W-beam. Layout of holes drilled on I-Beam for assembly is shown in Figure 3.11 (a).

Figure 3.10: Typical experimental test setup for CFSWS

The hydraulic load cell was mounted between the strong vertical wall and CFSWS to apply monotonic shear loading at the top of the model specimens as shown in Figure 3.10.

Two 42 mm diameter threaded rods were used to fasten the I-beam to the floor (Figure 3.10). Two angles (L 75 x 75 x 13 mm) and two angles (L 75x 50x13 mm) were utilized to mount the CFSWS models. Gusset plates of 13 mm were also welded to both ends and centre of each angle to increase the stiffness of the angle plate. Each angle was fastened from one side to the I-beam by 10, 25 mm diameter bolts and other side by welding to the bottom both sides of loading column and beam of CFSWS as shown in Figure 3.11.

(a) Layout of holes on I-beam

Note: Not to scale

(b) 42 mm bolt on W-beam

(c) 25 mm high strength bolt on W-beam

Figure 3.11: Details of fastening of W-beam with floor and CFSWS.

3.4.1 Instrumentation and data acquisition system

A data acquisition system was utilized to record the data from the electronic devices during the tests. It had 40 input channels for strain gauges and ten input channels for Linear Variable Differential Transducers (LVDT's). The maximum sampling rate of the data acquisition system was 50 samples per second. The following data were recorded during the loading history for CFSWS and bare frame specimens:

- Horizontal applied load at the top through the load cell.

- Horizontal displacement of the CFSWS and bare frame at the top for the one storey and at the top and first storey level (mid-height) for two storey CFSWS/bare frame models through LVDTs as shown in Figure 3.12(a).
- Vertical displacements of the CFSWS/frame at loading/non-loading end were also monitored by installing LVDTs as shown in Figure 3.12(b).

(a) Two LVDT's for horizontal displacement

(b) LVDT for uplift of CFSWS

Figure 3.12: The location of LVDT's

The single and rosette strain gauges were mounted on the face (crest) of the profiled steel sheet of walls panel at different locations as shown in Figure 3.12 to capture the development of strain due to lateral loading until the failure of model.

One single strain gauge was also mounted at trough of the steel sheet to compare strain development at crest and trough of profiled steel sheet. The strain gauges were also installed at different locations of frames. Details of strain gauges for each CFSWS and bare frames are shown in Figure 3.13.

All the strain gauges, rosettes and LVDTs were connected via bridge amplifiers to the data acquisition system.

Figure 3.13: Location of rosettes and single strain gauges

3.4.2 Testing and Loading Procedure of CFSWS and Bare Frame

The tests were performed based on displacement control by pushing top of the CFSWS by load cell at a constant rate of 3 mm per minute until the failure of the specimens. During the loading history, buckling of steel in wall/frame, concrete cracking and overall failure modes were observed.

3.5 Testing of CFST Frame-Wall Fasteners and Intermediate Fasteners

Composite wall was connected to the pinned steel frame beams and column by 10 mm diameter bolts running through channel frame, steel sheet and in-filled concrete as shown in Figure 3.14. This will make the load transfer through both steel and concrete and avoid tearing of sheet as well as to ensure connection failure due to shearing of bolts only.

Figure 3.14:- Pinned frame-wall fastener with composite wall

It was expected that fixed CFST boundary frame would induce failure in connection or in the wall panel of CFSWS system as required in a typical framed shear wall systems in tall buildings under practical circumstances. Therefore, composite wall was mounted in fixed CFST frame by 10 mm bolts. First of all, angle pieces (L1.5x1.5x1/8 inches) were welded with HSS beams and columns at points to match with holes made in steel sheet as shown in Figure 3.15.

(a) Fixed frame with welded angle

(d) Fixed frame with profiled sheets before casting concrete

(b) View of bolt with welded angle with frame

(c) View of piece of angle with frame

Figure 3.15:- Details of wall connection with fixed frame

The pull out tests was conducted to determine the load-slip behaviour of wall-beam and wall-column fasteners 10 mm diameter and intermediate fasteners (6 mm diameter) with profiled steel sheet (0.61 mm thick). Test specimens prepared for pull out tests are shown in Figure 3.16.

Figure 3.16: Pull out test connection specimens

Figure 3.17:- Intermediate sheet fasteners pull-out load-slip curve

Load-slip curves for intermediate fasteners are shown in Figure 3.17. Average strength of fasteners and the crimping factor (slope of the initial load-slip curve) were 3.6 kN and 1.8 mm/kN, respectively. Table 3.2 summarises the fastener parameters and strength depicted from pull-out tests.

Table 3.2: Sheet-fasteners test results.

Test #	Fasteners diameter (mm)	Sheet thickness (mm)	Strength (kN)	Average crimping factor (mm/kN)	Mode of failure
1	6	0.61	3.83	1.8	Tearing of sheet
2	6	0.61	3.34	1.8	Tearing of sheet
3	10	3.2	8.84	0.85	Bolt run through hole
4	10	3.2	7.91	0.85	Bolt run through hole

Figure 3.18:- Failure due to tearing of sheet in intermediate fasteners.

Figure 3.19: Frame wall fasteners pull-out test load-slip curve

Load-slip curves for frame-wall fasteners are shown in Figure 3.19. The average strength of connections and the crimping factor derived from pull-out tests were 8.4 kN and 0.85 mm/kN, respectively. Results are summarised in Table 3.2. Pull-out test on welded angle-fastener assembly showed failure due to elongation of angle plate causing hole widening and subsequent bolt (10 mm dia) run through the tore hole as shown in Figure 3.20. Therefore, washer had been used to prolong the failure due to running of bolt through hole. Due to the presence of in-filled

concrete, it is expected that failure will be due to shearing-off the bolts on application of lateral loads.

Figure 3.20:- Angle with screw failure shapes -specimen failed as bolt run through hole

CHAPTER FOUR

EXPERIMENTAL INVESTIGATION: RESULTS AND DISCUSSION

4.1 Introduction

The results of the experimental test program on Composite Framed Shear Walls System (CFSWS) subjected to in-plane monotonic lateral loading are presented. Four tests were conducted on one and two storey CFSWS. The variable parameters in the tests were the type of boundary frame (pinned steel frame and fixed CFST frame), number of storeys (one and two) and wall-frame connections through profiled steel-concrete-frame (bolted pin connection in case of steel frame and discrete innovative pin connections in case of CFST frame) while concrete and steel properties remain the same. The results are presented and discussed by illustrating load versus displacement response, principal strain/stress responses, and shear strain/stress responses throughout the loading history of the specimens as well as overall failure modes. The behaviour of bare pinned steel frame and fixed CFST frame under shear loading based on above parameters are also presented. At the end of the chapter, summary of results and main conclusions are presented.

4.2 Behaviour of Composite Framed Shear Wall System (CFSWS)

The experimental behaviour of the CFSWS and bare frames is described in the following sections based on the load-displacement response, stress-strain characteristics, failure modes, concrete cracking, and buckling of steel sheets.

4.2.1 One story pinned frame CFSWS (CFSWS -1)

The shear load-displacement response shown in Figure 4.1 shows a steady increase up to the maximum load of 95 kN at approximately 26 mm displacement. Initial stiffness calculated from load-displacement response is found to be 0.23 mm/kN. The energy absorbed by the CFSWS calculated from load-slip curve is 2.98 kJ. Due to development of concrete cracks and

subsequent concrete failure at the loaded top edge (Figure 4.2), the post-peak load was dropped to 70 kN. With continued loading, load increased up to 93 kN at 38 mm displacement. At this loading, front (loaded side) column started to bend at approximately $1/3^{\text{rd}}$ from the top corner where concrete cracked and failed earlier. At this stage, a noise was heard and on inspection, it was noticed that bond between steel sheet and concrete failed accompanied by buckling, distortion and loss of profiled geometry of steel sheet at rear top corner similar to those observed before at the loaded top corner (Figure 4.2). Post-peak loading was continued to observe the response of frame-wall fasteners. At the last stage, top beam-wall fasteners were sheared off and broken at 57 mm displacement.

Figure 4.1: Shear load-displacement response of one storey CFSWS (CFSWS-1)

One storey pinned steel frame model CFSWS-1 is shown in Figure 4.2(a) along with different modes of failure during testing. The development of localized cracks and separation of concrete and sheet are shown in Figure 4.2(b, c) due to compression and tension diagonal zones. Final shapes of failure at loaded and non-loaded compression diagonal corners are shown in Figure 4.2(e, f).

(a) CFSWS-1

(d) Loading column failure

(b) Loading side top view

(c) Nonloading side top view

(e) Final failure (loading corner)

(f) Final failure (nonloading corner)

Figure 4.2: Details of failure modes of model CFSWS-1

Steel sheet distortion, buckling and concrete separation at compression diagonal at both corners were appeared while testing as shown in Figure 4.2(b-f) which shows localized failure was prior to start of column failure.

The maximum and minimum principal strains at the center (crest) of the profiled steel sheet and corresponding stresses are shown in Figures 4.3 and 4.4, respectively. The maximum and minimum principal stresses are calculated from the principal strains based on Eqs. 4.1 and 4.2 (Riley et al. 2006; Timoshenko and Goodier 1982):

$$\sigma_{p1} = \frac{E_s}{(1-\nu_s^2)} (\epsilon_{p1} + \nu_s \epsilon_{p2}) \quad (4.1)$$

$$\sigma_{p2} = \frac{E_s}{(1-\nu_s^2)} (\epsilon_{p2} + \nu_s \epsilon_{p1}) \quad (4.2)$$

where, σ_{p1} and σ_{p2} are the maximum and minimum principal stresses, ϵ_{p1} and ϵ_{p2} are the maximum and minimum principal strains, E_s is the modulus of elasticity of steel plate ($E = 202,940$ MPa) and ν_s is Poisson's ratio of steel (0.3). The σ_{p1} and σ_{p2} are limited based on von-Mises yield criterion ($F_y = 552$ MPa) with the equation 4.3 (Megson 2005):

$$F_y = \sqrt{(\sigma_{p1}^2 - \sigma_{p1}\sigma_{p2} + \sigma_{p2}^2)} \quad (4.3)$$

Figure 4.3: Load versus principal strains at the center of the wall

Figure 4.4: Load versus principal stresses at the center of the wall

Maximum principal strain and stress increased with the increase of load up to 80 kN and dropped suddenly possibly due to initiation of cracks in concrete and de-bonding of the steel sheet-concrete interface. The maximum shear strain and stress are derived from Eqs. 4.4 and 4.5 (Riley et al. 2006):

$$\gamma_{max} = \epsilon_{p1} - \epsilon_{p2} \quad (4.4)$$

$$\tau_{max} = \frac{E_s}{2(1+\nu_s)} \gamma_{max} \quad (4.5)$$

where, γ_{max} is the maximum shear strain and τ_{max} is the maximum shear stress, ϵ_{p1} and ϵ_{p2} are the maximum and the minimum principal strains, E_s is modulus of elasticity of steel plate ($E=202,940$ MPa) and ν_s is Poisson's ratio of steel (0.3). The τ_{max} is limited on von-Mises criterion (Megson 2005) with the Eq. 4.6:

$$\tau_{max} \leq \frac{F_y}{\sqrt{3}}$$

The variation of calculated experimental shear strain and stress at the center (crest) of the profiled steel sheet (wall panel) are shown in Figure 4.5. The maximum experimental shear yield stress was 87 MPa at approximately 45 mm displacement which is less than the limit $\tau_{max} = 319$ MPa which proves that steel sheet did not yield at mid of panel.

Figure 4.5: Load versus shear strain/stress at center of CFSWS-1

4.2.1.1 Analysis of strains within CFSWS panel

The development of tension–compression state along the diagonals is confirmed from the plot of diagonal strains as shown in Figure 4.6. The strain gauges g1 and g5 were installed along the off-loaded tensile diagonal at centre and bottom corner of the panel, respectively. Strain gauge g1 was showing higher development of tensile strain as compared to g5. On the loaded diagonal, results of strain gauges g3 and g4 showed almost same compressive strain values up to a displacement of 22 mm. It was observed that gauge g4 shows more strain development as compared to centre gauge g4 before the post cracking state. The situation was reversed at the post cracking stage.

Figure 4.6: Variation of diagonal strain showing tension-compression state (CFSWS-1)

4.2.1.2 Analysis of strains in columns

The development of axial micro strains at the centre of each column have confirmed that prior to the failure of wall, no significant strain has been noticed. The graphic lines are straight at 27 mm displacement for both gauges g6 and g7. Gauges g6 was in compression pre-failure stage, post localized failure at ends of compression diagonal (after 26 mm displacement) gauge g7 also starts to be in tension. On the other hand after failure of wall, gauge g7 shows higher strain development as compared to g6 (Figure 4.7). Bending of loaded column (as shown in Figure 4.2d) might have caused abrupt drop/reverse of strain determent for gauge g6. Axial strain in columns did not reach shear yielding strain at pre-failure stage. This proves that composite wall failure was prior to bending of loaded column. Steel column shear strain shows yielding at post failure stage.

Figure 4.7: Variation of strain development in columns (CFSWS-1)

The direction of developed cracks in the profiled concrete core are found continuous diagonal at 45 degrees as shown in Figure 4.8. The cracks developed along the compression diagonal were mostly continuous as shown in Figure 4.8. The cracks width varies from 0.3 mm to 0.6 mm and crack average spacing was 200 mm. Vertical cracks were also developed along the line of column-wall fasteners which indicates that at some stage concrete failed along column-wall fasteners due to less fasteners cover. The cracks indicate the progress of diagonal tension-compression development in the wall panel which is the basic feature of the shear test. Localized

failure at the loaded top compression corner and bottom of non-loaded side was found. Due to localized failure at loaded top corner of wall, loaded column was bent due to the absence of wall support at the edge. The column was not strong enough to transfer load to the middle part of wall panel.

Figure 4.8: The crack pattern in the concrete panel CFSWS-1

4.2.2 Two Storey pinned steel frame CFSWS (CFSWS -2)

The shear load-displacement responses at floor levels for CFSWS-2 showed a steady increase of load of up 84 kN at approximately 40 mm displacement (Figure 4.9). The initial stiffness calculated from load-displacement response for the top and mid level are 0.49 mm/kN and 0.29 mm/kN, respectively. The energy absorbed calculated from load-displacement curve is 2.8 kJ.

During loading, no significant failure or any noise had been recorded up to a load of 60 kN. At 67 kN, a loud noise was heard and but on inspection, no fastener failure had been seen - noise might be due to concrete cracking. At 79 kN, specimen was thoroughly inspected but no

significant failure of fasteners found. Yielding/buckling of steel sheeting was not also seen. At load 84 kN, a big cracking/failure noise was heard that was might be due to initiation of cracks in wall panel. After the displacement was started to increase faster on loading which shows that concrete core was not taking further load. On excessive displacement, frame-wall fasteners sheared off (Figure 4.10c) and a dust cloud around centre beam-frame fasteners and loaded column-frame fasteners was appeared.

Figure 4.9: Load-displacement response at each level for (CFSWS-2)

Different states of failure for CFSWS-2 are shown in Figure 4.10(a-d). Due to limited space around holes for frame-wall fasteners, some of the fasteners failed due to tearing of profiled steel sheet as concrete cracked along the fasteners as shown in Figure 4.10d. Steel sheet distortion and concrete cracking were noticed after testing as shown in Figure 4.10a-b. At the failure stage, fasteners at different locations (as shown in Figure. 4.10c) along with the fasteners at bottom beam were sheared off (broken) due to large deformation. The fasteners at intermediate beam-wall panel were also sheared off. Both wall panels (1st and 2nd storeys) were cast as one unit. The presence of intermediate beam prevented crack development in the concrete core between the two floors. Therefore, localized failures at top edges were generated during loading and unloading.

Figure 4.10: Details of failure modes of CFSWS-2 model

The variation of the maximum and minimum principal strains at both 1st and 2nd story wall panel centres during loading history calculated from strains in rosettes (R1) and (R2) is shown in Figure 4.11. The corresponding principal stresses at the wall centres are shown in Figure 4.12. The bottom (1st storey) panel showed higher strain/stress development compared to top (2nd storey) wall panel. The principal stresses did not exceed the yield stresses of steel sheet.

Figure 4.11: Load versus principal strain development in CFSWS (CFSWS-2)

Figure 4.12: Load versus principal stress development in CFSWS (CFSWS-2)

The variation of calculated experimental shear strains and stresses at the centre (crest) of each wall panel are shown in Figure 4.13. The maximum experimental shear stresses at the centre of the bottom and top panels were 34 MPa and 41 MPa at maximum loading which did not reach the yield stress of the steel sheet.

Figure 4.13: Load versus shear strain/stress development (CFSWS-2)

4.2.2.1 Analysis of Strains within CFSWS-2

The development of tension–compression state along the diagonals in both storey levels is confirmed from the plotting of diagonal strains in Figure 4.14 and Figure 4.15. The strain gauges g2, g3 and g6 installed along the tensile diagonal exhibited the development of tensile strain while gauges g1, g5 and g7 installed along the compression diagonal developed compressive strains. Strain gauge g1 (at the corner) showed higher compressive strain development as expected due to stress concentration at the loaded corner. Similar development of tensile and compressive strains along the off-loaded diagonal (gauges g9, g10 and g13) and loaded diagonal (g8, g12 and g14) in the 1st storey wall panel were also observed. Diagonal strains in both storey levels did not reach the yield strain of steel sheet.

Figure 4.14: Variation of diagonal strains at 2nd level shear panel (CFSWS-2)

Figure 4.15: Variation of diagonal strains at bottom shear panel (CFSWS-2)

To compare the strain development at crest and trough of the profiled sheet, strain gauge g21 was installed in line with gauge g3 at the trough on top storey wall panel. Figure 4.16 shows higher strain development in the crest (gauge g3) compared to that in the trough (gauge g21) due to lower overall thickness of the wall panel at crest.

Figure 4.16: Comparison of strain development in trough and crest (CFSWS-2)

4.2.2.2 Analysis of strains in columns

Strain developments at the centre of each column and beam during testing history are plotted in Figure 4.17. As expected, the strain gauges g16 and g18 confirmed the development of tension in top and bottom storey columns. The strain development at gauge g17 changed from compression to tension after failure of the model at approximately 47 mm displacement. Similar strain development was also observed in gauge g15. Strain development pattern in columns and beams looks identical in the pre-failure stage up to about 40 mm displacement. In post failure stage, the initiation of cracks and failure of fasteners caused change in strain development. The strain in tension columns (g16 & g18) on the loaded side reached the shear yield strain which can be associated with the failure due to frame column.

Figure 4.17: Variation of strain development in columns and beams (CFSWS-2)

The cracks developed at compressive diagonal corners in the profiled concrete core are at approximately 45 degrees to horizontal as shown in Figure 4.18. The profiled steel sheet tore off around column-wall fastener holes causing consequent failure of the system (due to fastener failure) without fully transferring the load to the wall panel. The concrete failure along column-frame connections can be seen in Figure 4.18. Subsequently, with the increasing displacement, all loads were taken by beam/column-wall fasteners leading to the shearing failure of fasteners before the development of concrete cracking in the middle part (at 1st storey level). As same wall panel covered both 1st and 2nd storeys, the intermediate beam at the 1st storey did not allow the load to be transferred between two storeys and prevented wall panel failure before the failure of fasteners hence no continuous cracks was seen. Therefore, ultimate failure of this model was due to localized concrete core failure and failure of column/beam fasteners. The scattered cracks developed near the bottom of the compression diagonal at off-loaded edge showed localized failure too.

Figure 4.18:- The crack pattern in the concrete panel of CFSWS-2

4.2.3 One Storey fixed CFST frame (Model CFST-F1)

One storey composite fixed CFST frame (Model CFST-F1) was fabricated and tested along with CFSWS model with CFST frame (CFSWS-3) to investigate the contribution of building frame to the overall system. The load-displacement response of CFST-F1 shows a steady increase in load up to the maximum load of 49 kN at approximately 44 mm displacement as shown in Figure 4.19. Initial stiffness of the frame calculated from shear load-displacement response is 1.12 mm/kN. The energy absorbed by the system was calculated from load-displacement curve is 3.04 kJ. The ultimate failure of the frame was due to opening of HSS column plates at joints and failure of joint welding (Figure 4.20).

A noise was heard during testing at 28 kN (17.11 mm) but no significant failure was observed on inspection. At maximum load of 49 kN, cracks appeared at loading top beam-column joint and load dropped to 45 kN. On continuing load, again increased to 48 kN when failure occurred with a big sound accompanied by the failure of other column-beam joints. At this stage, wide opening and tearing of HSS plate were appeared at column-beam joints as shown in Figure 4.20 (b-d).

Figure 4.19: Load-displacement response of bare fixed one storey CFST frame (CFST-F1)

Both CFST beam and column including concrete in-fill did not fail. It is therefore, assumed that frames will be sufficiently strong to transfer the load from the frame to the wall panel through frame-wall fasteners.

Figure 4.20:- Failure modes of one Storey CFST frame (CFST-F1)

Variation of strain development in top beam and both columns are shown in Figure 4.21. Maximum strain was observed at bottom of loading column (g3). Other strain developments are not significant as frame joint failed earlier.

Figure 4.21:- Variation of strain development in Columns and Beams (CFST-F1)

4.2.4 One storey CFSWS with fixed supported CFST frame (Model CFSWS-3)

The load- displacement response showed a smooth line up to maximum load of 135 kN up to 21 mm displacement as shown in Figure 4.22. The maximum load carried by the system was recorded as 146.84 kN at 82 mm displacement. The initial stiffness for the specimen, calculated from load-displacement curve, is 0.14 mm/kN. The energy absorbed by the system calculated from the results is 6 kJ.

Figure 4.22: Shear load-displacement response of one story CFST frame based CFSWS (CFSWS-3)

At 110 kN and 130 kN, noise were heard but no significant failure was noticed. At 133 kN, a loud noise was heard and load dropped to 118 kN but connections were still in good shape. On applying further load, it was observed that sheet had been buckled out at top loading corner (Figure 4.23b-d, bottom non-loading corner Figure 4.23c and at bottom beam-sheet fasteners as shown in Figure 4.23(e). Column/beam-wall fasteners failed at 144 kN (at 94 mm displacement). At final stage of testing (maximum displacement approximately 96 mm), the frame column-beam joint (top nonloading) was also failed.

(a) CFSWS-3

(b) Distortion and buckling out steel sheet at loading side

(c) Sheet buckling at non loading side

(d) Sheet buckling at loading side

(e) Sheet buckling along centre of bottom beam

Figure 4.23: Failure modes of one story CFST framed CFSWS (CFSWS-3)

The variation of the maximum and minimum principal strains at wall panel centre of the profiled steel sheet were calculated from the results obtained by rosette (R1) as shown in Figure 4.24. The variation of corresponding calculated principal stresses are shown in Figure 4.25. The tensile principal stress at the centre was about 1750 micro-strain, which signified that the steel sheet may be yielded at the loaded corner where stress concentrations were expected.

Figure 4.24: Load versus principal strains for CFSWS-3

Figure 4.25: Load versus principal stresses for CFSWS-3

The variation of calculated experimental shear strains and stresses at the centre (crest) is shown in Figure 4.26. The maximum experimental shear stress at maximum load was 90 MPa at centre of the wall panel but it reached after post failure to 145.3 MPa (at 95 mm displacement) which was also less than the shear yield stress.

Figure 4.26: Load versus shear strain and stress (CFSWS-3)

4.2.4.1 Analysis of strains within CFSWS-3 panel

The development of tension–compression state along the diagonals is confirmed from the plot of diagonal strains as shown Figure 4.27. The strain gauges g2, g3 and g6 installed along the tensile diagonal exhibited the development of tensile strain while gauges g1, g5 and g7 installed along the compression diagonal prior to failure stage (40 mm displacement) and reversed afterward. The strain gauge g1 shows compression prior to steel sheet distortion stage. The buckling of steel sheet, loss of profile shape and separation of concrete-sheet (bond failure) along loaded diagonal caused reversal tension-compression state at the post failure stage. The diagonal strain in gauge g7 reached the yield strain of profiled steel sheet and other diagonal strains in panel did not reach the yield strain of the steel sheet.

Figure 4.27: Variation of diagonal strains (CFSWS-3)

To compare the results at crest and trough of the profile sheet, a strain gauge g13 was installed in line with gauge g5 at trough on wall panel. Figure 4.28 shows higher strain development in strain gauge g5 (crest) as compared to strain gauge g13 (trough) prior failure stage due to lower overall thickness of the wall panel at crest.

Figure 4.28: Comparison of strain development in trough and crest (CFSWS-3)

4.2.4.2 Analysis of strains in columns

The strain developments at top and bottom of both columns and at non-loaded edge of top beam during testing history are plotted in Figure 4.29. As expected, the strain gauges g8 and g10 confirmed the development of tension at top and bottom of loaded columns and gauge g9 confirms the development of compression. The strain development in the gauge g12 installed at non-loaded edge of top beam shows the development of tension but insignificant. The strain in gauge g10 installed at bottom of tension column and gauge g11 installed at bottom of compression column reached to the yield strain after post failure stage of test model.

Figure 4.29: Variation of strain development in columns and beams (CFSWS-3)

The directions of developed cracks in the profiled concrete core are found diagonal at 45 degrees as shown in Figure 4.30. Major cracks are found continuous along compression diagonal and minor developed cracks at loaded edge are found scattered due to stress concentration. The major cracks width varies from 0.4 mm to 0.6 mm and crack average spacing is found 150 mm. The failure of the sheeting was associated with the formation of local buckling at loaded and non-loaded edge. The direction of the tension field followed the direction of principal stress. Most of the buckles formed at failure of test models were found due to few heavily distorted crest or trough lines as observed in the model test. Most of the cracks were seen at the loaded and non-loaded edges of compression zone.

Figure 4.30: The crack pattern in the concrete panel CFSWS-3

4.2.5 Two Storey Fixed CFST Frame (CFST-F2)

Two storey composite fixed CFST frame (Model CFST-F1) was fabricated and tested along with CFSWS model with CFST frame (CFSWS-4) to investigate contribution of building frame to the composite framed shear wall system. The load-displacement response of CFST-F2 shows a steady increase in load up to the maximum load of 26.2 kN at 44 mm displacement as shown in Figure 4.31. The initial stiffness of the frame calculated from shear load-displacement response are 1.84 mm/kN and 0.86 mm/kN at top and mid levels, respectively. The energy absorbed by the system calculated from shear load-displacement response is 1.35 kJ. The ultimate failure of the frame is (similar as one storey CFST frame (CFST-F1)) due to the opening of HSS column plates at joints and failure of joint welding (Figure 4.32).

Figure 4.31: Load-displacement response of bare fixed two storey CFST frame (CFST-F2)

Figure 4.32:- Failure modes of two storey fixed CFST Frame (CFST-F2)

Some noise were heard during testing at 22 kN (41 mm) and 24 kN (49 mm) but no significant failure was observed. At maximum load of 26.20 kN (66 mm displacement), cracks appeared at loaded top beam-column joint and column plate opening started at off loaded intermediate beam-column joint as shown in Figure 4.32e. At post failure loading, wide joint opening was seen on all beam-column joints causing failure of CFST bare frame model. Therefore, final failure of the CFST frame was due to HSS plate tearing off or welding causing joint failure as shown in Figure 4.32.

Variation of strain development in beams and columns is shown in Figure 4.33. Maximum strain was developed at the centre of 1st storey loaded and off-loaded columns as recorded by g17 and g18 as shown in Figure 4.33. The strain gauges g17 and g18 are showed the development of compression and tension at the centre of 1st storey columns, respectively as expected but reached yield strain at failure. The strain gauges installed on top and intermediate beams are showing low strain development as compare to columns.

Figure 4.33:- Variation of strain development in columns and beams (CFST-F2)

4.2.6 Two storey fixed CFST frame based CFSWS (Model CFSWS-4)

The load-displacement response for CFSWS-4 showed a steady increase in load up to 105 kN and approximately 45 mm horizontal top storey displacement. Maximum shear strength of the system was 121.24 kN at approximately 88.52 mm displacement (Figure 4.34). The initial stiffness calculated from load-displacement response for the top and mid-level are 0.82 mm/kN and 0.50 mm/kN, respectively. The energy absorbed by model CFSWS-4 calculated from load-displacement curve is 6.9 kJ.

During testing, first and second noise were heard at loads of 73 kN (37 mm displacement) and 93 kN (49 mm displacement), respectively but no significant failure was noticed. Load dropped a few times such as from 105 kN to 95 kN which were associated with the buckling of steel sheet at the corners of compression diagonal. The distortion and change of geometry of steel sheet at compressive diagonal edges loads confirmed the failure of filled concrete core as shown in Figure 4.35(e, g) prior to failure of CFST frame.

Figure 4.34: Load-displacement response at each level for (CFSWS-4)

(b) Distortion and buckling out steel sheet at loading side

(a) CFSWS-4 model

(f) Fasteners failure

(c) Beam-wall fasteners failure

(d) Fasteners yielding

(e) Distortion and buckling out steel sheet at loading side

(g) Change of geometry of profiled sheet

Figure 4.35: Failure modes of two storey CFST framed CFSWS (CFSWS-4)

The CFSWS systems showed ductile behaviour with increased displacement as can be seen from the multiple peaks in the load-displacement response. Displacement started to increase more rapidly as compared to load in the ultimate stage due to progressive concrete core and steel sheet failure. At the final stage, column/beam-wall fasteners and frame itself were taking all the lateral loads and ultimately CFSWS system failed.

The CFSWS model and its different types of failure modes showing buckling/distortion of steel sheet and fastener failures are shown in Figure 4.35(a-f).

The variation of the maximum and minimum principal strains at both 1st and 2nd story wall panel centres during loading history calculated from strains in rosettes (R1) and (R2) is shown in Figure 4.36. The corresponding principal stresses at the wall centres are shown in Figure 4.37. The bottom (1st story) panel showed higher strain/stress development as compared to top (2nd storey) wall panel. The principal stresses at both panel centres (1st and 2nd story) did not exceed the yield stresses of steel sheet.

Figure 4.36: Load versus Principal Strains for composite framed shear wall (CFSWS-4)

Figure 4.37: Load versus principal stresses for CFSWS-4

The variation of calculated experimental shear strains and stresses at the centre (crest) of each wall panel are shown in Figure 4.38. The maximum experimental shear stresses at the centre of the bottom and top panels were 88 MPa and 56 MPa at maximum loading which did not reach the shear yield stress of the steel sheet.

Figure 4.38: Load versus shear strain and stresses (CFSWS-4)

4.2.6.1 Analysis of Strains within CFSWS-4 Panels

The development of tension–compression state along the diagonals in both storey levels is confirmed from the plotting of diagonal strains in Figure 4.39 and Figure 4.40. The strain gauges g2, g3 and g6 installed along the tensile diagonal exhibited the development of tensile strain while gauges g1, g5 and g7 installed along the compression diagonal developed compressive strains. Strain gauge g1 showed higher tensile strain development due to stress concentration and distortion and damage of profiled geometry at the loaded corner (Figure 4.35b). The strain gauges g1 and g5 shows reached close to yield shear stress of steel sheet as shown in Figure 4.39. The development of tensile and compression strains along the off-loaded diagonal (gauges g9, g10 and g13) and loaded diagonal (g8, g12 and g14) in the 1st story wall panel were observed as expected. The strain gauges g10 and g12 are not showing considerable strains development as compared to other gauges and the tensile diagonal strain in g13 reached yield strain of the steel sheet.

Figure 4.39: Variation of diagonal strains at top wall panel (CFSWS-4)

Figure 4.40: Variation of diagonal strains at bottom wall panel (CFSWS-4)

To compare the results of strain development at crest and trough of the profile sheet, a strain gauge g21 was installed in line with gauge g5 at trough on wall panel. Figure 4.41 shows tensile strain development in strain gauge g5 (crest) and compressive strain development in gauge g21 (trough) prior to failure of model. Both did not reach to the shear yield strain of the steel sheet.

Figure 4.41: Comparison of strain development in trough and crest (CFSWS-4)

4.2.6.2 Analysis of strains in columns

The strain developments at centre of each column and beam during testing history are plotted in Figure 4.42. As expected, the strain gauges g16 confirmed the development of tension at top loaded column and gauge g15 confirms the compression development. The strain development in the gauges g19 and g20 installed at top and intermediate beams shows compression and tension development respectively. At post failure stage, only gauge g19 reached the shear yield.

Figure 4.42: Variation of strain development in columns and beams (CFSWS-4)

The directions of developed cracks in the profiled concrete core are found diagonal at 40 degrees as shown in Figure 4.43. Major cracks are found continuous along compression diagonal in both wall panels and minor developed cracks are found scattered in the bottom wall panel. The major cracks width varies from 0.4 mm to 0.6 mm and crack average spacing is found 150 mm. Most of the buckles of steel sheet at failure of test models were found due to few heavily distorted crest or trough lines as observed in the model test. Most of the cracks were seen at the loaded and non-loaded edges of compression zone. The cracks indicate the progress of diagonal tension-compression development in the both wall panels which is the basic feature of the shear test. Localized failure at the loaded top compression corner and bottom of non-loaded side was also found.

Figure 4.43:- The crack pattern in the concrete 2nd storey panel CFSWS-4

Figure 4.44: The crack pattern in the concrete core 1st storey panel CFSWS-4

4.3 Discussions and Conclusions

The summary of experimental results from four one and two storey CFSWS and two bare fixed CSFT composite frames in terms of ultimate shear load, ultimate shear displacement, failure modes, maximum diagonal/principal/shear stress/strain, absorbed energy and cracking characteristics are presented in Table 4.2.

Table 4.2: Summary of test results

Specimen (type of frame –storey level)	Ultimate shear load resistance and displacement		Concrete cracking			Shear stress in steel sheet (1 st /2 nd level) (MPa)	Energy absorbed (kJ)	Stiffness (mm/kN) Top and (Mid) level	Mode of failure/ Comments
	Peak (kN)	Peak (mm)	Crack Angle Θ	Crack Width w (mm)	Average crack spacing s_0 (mm)				
CFSWS-1 (pinned-one)	95	26	45°	0.3-.06	200	87	2.98	0.23	Localized failiure*
CFSWS-2 (pinned-two)	84	40	30°	--	--	41/31	2.8	0.49 (0.29)	Frame-wall fasteners failure**
CFSWS-3 (fixed-one)	146	21	45°	0.4-0.6	150	90	6	0.14	Inelastic buckling*
CFSWS-4 (fixed-two)	121	45	40°	0.4-0.6	150	88/56	6.9	0.82 (0.50)	Inelastic Buckling*
CFST-F1 (fixed-one)	49	44	Concrete filled in tubes didn't fail.			--	3.04	1.12	Joints and HSS plate tearing***
CFST-F2 (fixed-two)	26	66	Concrete filled in tubes didn't fail.			--	1.35	1.84 (0.86)	Joints and HSS plate tearing***
*wall failure; ** frame-wall connection failure; ***frame failure									

4.3.1 Comparison between one and two storey CFSWS

The one storey pinned and fixed CFSWS models had higher shear resistance (11.5% to 17%) and underwent lower shear displacement (53% to 114%), respectively compared to two storey

models. Similarly, one story CFST bare frame had 47% higher shear resistance and underwent 50% lower shear displacement as compared to two storey CFST frame. However, the shear strength in pinned steel frame CFSWS is less as frame did not contribute to the load resistance capacity.

The energy absorbed by the one storey pinned and fixed CFSWS models was lesser by 6% to 15% and higher stiffness by 100% to 185% as compared to two storey models. Similarly, one story CFST bare frame had absorbed less energy and higher stiffness capacity by 55% to 65% as compared to two storey CFST frame.

4.3.2 Comparison between pinned steel framed and fixed CFST framed CFSWS

The shear strength of the one storey fixed CFST frame CFST-F1 was 49 kN. Therefore, the shear strength contribution of in-fill wall to the strength of CFRSWS-3 is 97.84 kN (shear strength of CFSWS-3 – shear strength of CFST-F1) which is approximately equal to the shear strength (95 kN) of CFSWS-1 (pinned steel framed CFSWS-1). On the other hand, similarly, the shear strength of bare two storey fixed CFST frame CFST-F2 is 26.26 kN. Hence, the shear strength contribution of infill wall to the strength of CFSWS-4 is 94.98 kN (shear strength of CFSWS-4 – shear strength of CFST-F2) which is approximately the shear strength (84 kN) of pinned steel frame CFSWS-2. However, the shear strength of CFSWS-2 was affected by the failure of fasteners before the failure of wall. This proves that CFST boundary frame contributed to the shear strength of CFSWS but did not affect the strength of infill double skin composite wall.

CHAPTER FIVE

ANALYTICAL MODELS FOR COMPOSITE FRAMED SHEAR WALL SYSTEM

5.1 Introduction

Analytical models for the shear capacity of composite wall are derived and implemented in the shear resistance of composite framed shear wall system (CFSWS) with special reference to frame-wall connections. The shear capacity of the composite framed shear wall system will be the summation of the shear capacity of the frame and the shear capacity of infill double skin composite wall. In general, the shear strength of frame-wall fasteners should be strong enough to transfer load from frame to the composite wall. The performance of analytical model is validated with comparison of experimental results.

5.2 Shear Capacity of Frames

Pinned connected steel frame does not contribute to the shear capacity of CFSWS and such frames only transfer load to the infill composite wall panel through the frame-wall fasteners. However, fixed CFST frames have contributed to the shear strength of CFSWS. As per experimental results, the shear capacity of fixed one and two storey CFST building frames (V_{CFST}) were 49 kN and 26 kN, respectively (Table 4.2) which were added to overall shear strength of the CFSWS.

$$V_{CFST} = 49 \text{ kN (for one storey)} \quad \& \quad 26 \text{ kN (for two storey)}$$

5.3 Shear Capacity of double skin composite wall

The shear resistance of the double skin composite wall (shown in Figure 5.1) may be governed by:

- the failure at the line of seam fasteners,
- the failure of connections between sheet and frame (frame-wall fasteners),
- the shear yielding or buckling of steel sheeting,

- the failure of concrete core and
- the failure of steel sheet-concrete connections (intermediate fasteners) leading to the loss of steel sheet-concrete interaction.

Figure 5.1: Schematic diagram of one storey CFSWS showing connection details

The following assumptions can be made in case of composite wall (Hossain and Wright, 2004):

- At seams between adjacent steel sheets for practical construction, the concrete carries almost all the shear force. Therefore, failure of the seam fasteners may be ignored.
- The boundary connections should be provided through both concrete and steel to be strong enough to induce failure in the wall panel.
- The confining effect of the concrete eliminates the distortion and bending of the sheeting as infill concrete will act as stiffeners allowing the flat cross section of the steel sheets to remain flat and therefore sheet distortion before ultimate loading stage may be neglected.
- The full mobilization of sheet-concrete capacity and to achieve optimum sheet-concrete interaction, boundary connections to the frame should be provided through both steel and concrete.

Therefore, the shear capacity of the double skin composite wall can be derived from the shear strength of concrete core and shear strength of steel sheet due to yielding or buckling. The profiled concrete core of the composite shear wall can be simplified with a uniform flat concrete plate with an average thickness of crest and trough section. A concrete plate cracks due to principal tensile stresses and therefore, can resist shear by developing compression struts between the cracks and tensions in boundary steel frames.

5.3.1 Shear Capacity of Frame-Wall Fasteners

The frame-wall fasteners are playing a major role in the calculation of the shear capacity of the composite frames shear wall system (CFSWS). If these frame-wall fasteners will not strong enough to transfer shear force from frame to the shear wall, then full capacity of the system cannot be utilized. The shear strength due to sheet-column or sheet-beam fasteners can be calculated by following equation Eq.5.1 (Hossain and Wright, 2004).

$$V_f = F_{sb} \cdot \frac{a}{P_a} = F_{sc} \cdot \frac{b}{P_b} \quad (5.1)$$

where V_f is the shear strength of wall-frame fasteners; F_{sb} and F_{sc} are the strength of each beam-sheet and column-sheet fastener, respectively; a and b are width and length of wall, respectively; P_a and P_b are fasteners spacing along beam and column, respectively as shown in Figure 5.1.

The strength /shear capacity of fasteners depends mainly on the sheet thickness and the type of connections. For the experimental models, 10 mm diameter bolts have been used to connect composite wall with frame (CFST frame). The shear capacity of one frame-wall fastener was determined by simple pullout tests on sample of same material used for the preparation of CFSWS models. The load-slip relationship of connections (shown in Figure 3.19) depends on the type of fasteners and the thickness of steel sheet.

The average shear capacity of each fastener was recorded as approximately 8.5 kN (refer to Table 3.2) without using washers for single profiled steel sheet. Therefore, for pair of sheeting, it will be 17 kN for each fastener. Total strength of the frame-wall fasteners can be determined based on Eq. 5.1:

$$V_f = F_{sb} \cdot \frac{a}{P_a} = F_{sc} \cdot \frac{b}{P_b} = 2 \times 17 \times 640/128 = 170 \text{ kN}$$

5.3.2 Shear Resistance of the Profiled Steel Sheet Bounded by Frame

The profiled steel sheets in the composite shear wall should reach their yield shear stress because of their contact with rigid profiled concrete core and the presence of shear connectors preventing buckling. Therefore, it is possible to derive the shear resistance of the single profiled steel sheet (V_s) based on Eq. 5.2:

$$V_s = \frac{F_y}{\sqrt{3}} t_s d_v \quad (5.2a)$$

where F_y is yield strength of profiled steel sheet, t_s is thickness of the profiled steel sheet, and d_v is width of the steel sheet. Based on the von-Mises yield criterion, $F_y/\sqrt{3}$ is the maximum shear stress which the profiled steel sheet can resist (Megson 2005).

If the shear wall does not reach to yielding, it is possible to calculate the shear resistance of single profiled sheet (Paik and Thayamballi 2003) as follows:

$$\tau_{cr} = k_v \frac{\pi^2 E_s}{12(1 - \nu_s^2)} \left(\frac{t_s}{b}\right)^2 \quad (5.3)$$

where τ_{cr} is shear buckling stress, E_s is modulus of elasticity of steel, ν_s is Poisson's ratio of steel ($\nu_s = 0.3$), and k_v is the shear buckling coefficient.

Figure 5.2: Typical CFSWS diagram with flat sheet considering for buckling

Considering flat part (ABCD) of the profiled steel sheet between two intermediate fasteners used in the experimental composite wall as shown in Figure 5.2. For all edges simply supported, which is a conservative assumption for the steel sheet, the value of k_v is $4\left(\frac{b_p}{a_p}\right)^2 + 5.34$ as per CSA-S16 using b_p and a_p as larger and shorter dimensions of a rectangular steel plate, respectively.

The value of τ_{cr} for ABCD is found to be 39 MPa (calculated as per Eq. 5.3) which is significantly lower than the shear yield stress ($F_y/\sqrt{3}$ based on von-Mises yield criterion) of 319 MPa steel sheet. It proves that shear resistance calculations based on yielding criteria does not govern as observed in the experimental results.

Therefore, it is assumed that plate will buckle prior to shear yielding. However, the actual buckling stress of steel sheet should be much higher due to the presence of concrete core in contact with sheet as well as the action of ribs of the profiled steel sheet acting as stiffeners. At post buckling stage, tension field will be generated and the shear strength of steel sheet (V) will be greater than critical shear strength (V_{cr}) due to buckling but less than ultimate shear load (V_{ult}) at the time of yielding. When the compressive stress equals the critical buckling stress, the central part of the plate buckles. But the edges parallel to the x-axis cannot deflect in the z-direction and so the strips closer to these edges continue to carry the load without any instability.

Hence, a tensile field will be generated to take account of load more than the buckling load. Depending on the thickness or slenderness, the panel may be either in pure tension or in incomplete tension field or in pure shear. Panels which are not very thin, they will act partially in shear and partially in tension which is an intermediate case of two extreme cases of pure shear and pure tension.

Hence, it is possible that shear resistance of single steel sheet (V_s) can be obtained by post buckling tension field criteria considering pure tension (Shear stress, $\tau = 2\tau_{cr}$) based on Eq. 5.2a to consider post-critical shear reserves (Hossain and Wright 1998):

$$V_s = \tau t_s d_v = 2 \tau_{cr} t_s d_v \quad (5.2b)$$

The experimental test results of CFSWS specimens confirmed that yielding of profiled steel sheet could not be attained but buckling at loaded compression diagonal and other diagonal edges were noticed. Therefore, actual shear stress (f_s) of sheeting can also be used to calculate shear capacity (V_s) of single steel sheet instead of using shear yield/buckling stress as per Eq. 5.2c.

$$V_s = f_s t_s d_v \quad (5.2c)$$

5.3.3 Shear Resistance of the Profiled Concrete Core

Analytical model for shear resistance of profiled concrete core is developed using simplified modified compression field theory. In the composite wall with sufficient intermediate fasteners connecting the two profiled sheets and profiled concrete core, the crack widths can be assumed to be small enough due to confinement and composite action. In such case, the behaviour of concrete can be assumed to be similar to the reinforced concrete. The shear capacity of the concrete core panel can be limited based on the follow possible modes of failure:

- (a) crushing of the struts between cracks due to the compressive stress,
- (b) when the average tensile stress transverse to the cracks reaches to its limit,

(c) when slip occurs along the cracks.

The first failure mode happens when the principal compressive stress, exceeds the maximum average principal compressive stress ($f_{2, \max}$) from Eq. 5.4 that concrete can be resisted (Vecchio, and Collins 1986, 1988, 1993):

$$f_{2, \max} = \frac{f'_c}{0.8 + 170 \varepsilon_1} \leq f'_c \quad (5.4)$$

where f'_c is cylinder compressive strength of concrete, and ε_1 is principal tensile strain on average in cracked concrete. Due to the fact that the compressive strength of concrete is much higher than its tensile strength, this mode of failure is very rare.

Figure 5.3: Stresses and strains between and along inclined cracks in concrete core

For the last two modes of failure, the shear capacity of the panel is related to the area of inclined crack. For this reason, the profiled concrete core panel was simplified to an equivalent plain concrete panel with a rectangular cross-section, having an average uniform thickness of ' t_c ' (Hossain and Wright, 2004). Moreover, the concrete panel under shear load was assumed to behave like a concrete having minimum shear reinforcement. This assumption was based on the fact that the concrete panel was connected to the profiled steel sheets by intermediate fasteners along the width and height of the wall and the crack width of the concrete panel is limited by the steel sheet. The experimental observations on the cracks in the profiled concrete core of the tested composite wall also confirm this assumption.

The stresses between and along inclined cracks of the concrete core are illustrated in Figure 5.3. The shear stress resistance provided by the cracked concrete (v_c) can be expressed by Eq. 5.5:

$$v_c = v_{ci} = f_1 \cot \theta = \beta \sqrt{f'_c} \quad (5.5)$$

where f_1 is the average principal tensile stress in the concrete, β is the shear retention factor, and θ is the angle of diagonal cracks to the longitudinal axis of the concrete core wall. When concrete cracks, the relationship between average principal tensile stress (f_1) in the concrete and the principal tensile strain (ϵ_1) can be expressed as follows (Bentz and Collins, 2006):

$$f_1 = \frac{0.33\sqrt{f'_c}}{1 + \sqrt{500\epsilon_1}} \text{ MPa} \quad (5.6)$$

where $0.33\sqrt{f'_c}$ represents concrete cracking stress. When the ϵ_1 have a large value, the cracks become wider and the f_1 reduces. In this condition, the shear can be transmitted by the shear stress across the cracked interface (v_{ci}) as shown in Figure 5.3.

In order to calculate the shear stress (v_c) from Eq. 5.4, the value of ϵ_1 should be calculated. The relationship between ϵ_1 , ϵ_2 , and ϵ_y (Figure 5.3) can be derived as follows (MacGregor and Bartlett, 1994):

$$\varepsilon_1 = \varepsilon_y + (\varepsilon_y - \varepsilon_2) \cot^2 \theta \quad (5.7)$$

The longitudinal strain at mid-depth of the concrete core wall (ε_y) can be derived as:

$$\varepsilon_y = \frac{M/d_v + 0.5N + V}{2A_{sc}E_{sc}} = \frac{Vh/d_v + 0.5N + V}{2A_{sc}E_{sc}} \quad (5.8)$$

where, M is bending moment about strong axis of the wall, N is axial load normal to the cross-section of the wall, V is shear load, d_v is effective shear depth/width of the wall, h is the height of the frame, E_{sc} is modulus of elasticity and A_{sc} is average cross-sectional area of steel frame columns.

The mid-depth strain is conservatively approximated as one half the strain (ε_t) in the frame steel column under tensile force, as the strain in flexural compression (ε_c) tends to be small due to the high stiffness of concrete in compression. The term $0.5 V \cot \theta$ (Figure 5.3) was replaced by V in Eq. 5.8 to simplify and also decouple the relation of ε_y with the angle of diagonal cracks to the longitudinal axis of the concrete core wall (θ). The ε_2 , which is incorporated in Eq. 5.7 to find ε_1 value, can be derived by using Eq. 5.9:

$$\varepsilon_2 = \varepsilon'_c \left(1 - \sqrt{\left(1 - \frac{f_2}{f_{2,max}} \right)} \right), \quad \varepsilon'_c = -0.002 \quad (5.9)$$

The principal compressive stress on average (f_2) in Eq. 5.9 can be derived using Eq. 5.10 (Bentz and Collins, 2006):

$$f_2 = f_1 \cot^2 \theta \rightarrow f_2 = \frac{0.33\sqrt{f'_c} \cot^2 \theta}{1 + \sqrt{500\varepsilon_1}} \quad (\text{MPa}) \quad (5.10)$$

The shear stress across the cracked interface (v_{ci}) as shown in Figure 5.3 can be calculated as a function of crack width (w) and the aggregate size (a_g) as per Eq. 5.11 (Collins et al. 1996):

$$v_{ci} \leq \frac{0.18\sqrt{f'_c}}{0.31 + \frac{24w}{(a_g + 16)}} \text{ MPa, mm} \quad (5.11)$$

The value of β for the second and third possible modes of failure can be derived by substituting Eq. 5.6 and Eq. 5.11 into Eq. 5.5:

$$\beta = \frac{v_{ci}}{\sqrt{f'_c}} = \frac{f_1 \cot \theta}{\sqrt{f'_c}} = \frac{0.33 \cot \theta}{1 + \sqrt{500}\epsilon_1} \leq \frac{0.18}{0.31 + \frac{24w}{(a_g + 16)}} \text{ (MPa, mm)} \quad (5.12)$$

The crack width, w , can be calculated using the diagonal crack spacing (s_θ) as per Eq. 5.13:

$$w = s_\theta \epsilon_1 \text{ (mm); } s_\theta = \frac{s_{ze}}{\sin \theta}, \quad s_{ze} = s_z \frac{35}{15 + a_g} \geq 0.85 s_z \text{ (mm)} \quad (5.13)$$

Crack spacing parameter (s_z) can be taken as 300 mm and if the maximum aggregate size in concrete is equal or greater than 20 mm, the effective crack spacing (s_{ze}) also can be taken as 300 mm. The shear resistance of the profiled concrete core panel in the composite wall (V_c) based on an equivalent plain concrete panel of thickness (t_c) can be derived using Eq. 5.14:

$$V_c = \beta \sqrt{f'_c} t_c d_v \quad (5.14)$$

5.4 Analytical Model for Shear Resistance of the Composite Walls

Shear resistance of the composite wall (V_w) can be derived by adding the shear resistance of the profiled steel sheets (Eq. 5.2a) and the shear resistance of the concrete panel (Eq. 5.14), considering shear yield stress of profiled steel sheet as per Eq. 5.15a:

$$V_w = 2V_s + V_c = 2 \frac{F_y}{\sqrt{3}} t_s d_v + \beta \sqrt{f'_c} t_c d_v \quad (5.15a)$$

Shear resistance of the composite wall (V_w) can be derived from Eq. 5.15b based on the post buckling shear stress of profiled steel sheet, considering pure tension field.

$$V_w = 2V_s + V_c = 2 (2 \tau_{cr} t_s d_v) + \beta \sqrt{f'_c} t_c d_v \quad (5.15b)$$

Shear resistance of the composite wall can be derived based on actual shear stress (f_s) observed at peak experimental load as follows:

$$V_w = 2V_s + V_c = 2f_s t_s d_v + \beta \sqrt{f'_c} t_c d_v \quad (5.15c)$$

5.5 Analytical Model for Composite Framed Shear Wall System (CFSWS)

Shear resistance of the CFSWS (V_{cfsws}) can be derived by adding the shear resistance of the CFST frame and the shear resistance of profiled composite wall (Eq. 5.15) considering interaction, between frame and wall, is insignificant as frame should not take any shear load as follows:

$$V_{CFSWS} = V_w + V_{CFST} = 2V_s + V_c + V_{CFST} \quad (5.16)$$

Shear resistance based on shear yield stress:

$$V_{CFSWS} = 2 \frac{F_y}{\sqrt{3}} t_s d_v + \beta \sqrt{f'_c} t_c d_v + V_{CFST} \quad (5.17a)$$

Shear resistance of the CFSWS can be derived based on post buckling shear stress:

$$V_{CFSWS} = 2 (2 \tau_{cr} t_s d_v) + \beta \sqrt{f'_c} t_c d_v + V_{CFST} \quad (5.17b)$$

Shear resistance of the CFSWS based on actual shear stress developed at peak experimental load:

$$V_{CFSWS} = 2f_s t_s d_v + \beta \sqrt{f'_c} t_c d_v + V_{CFST} \quad (5.17c)$$

Experimental shear resistance of composite framed shear wall system is used to validate the performance of analytical models presented in Eqs. 5.17. Accuracy of Eqs. 5.17(b-c) is validated as the failure of the tested CFSWSs in this study was governed by the buckling of steel sheet associated with tension field development. However, the ideal case would be the failure due to shear yield stress of profiled steel sheet which can be achieved by increasing the number of intermediate steel sheet-concrete fasteners.

The value of β can be calculated based on two procedures:

Iterative procedure: can be used when the values ε_y , crack spacing, aggregate size, concrete strength and angle of cracks are known. For the experimental walls, these values are known from the tests. The trial and error procedure can be started by adding a nominal value for strain (such as 0.005 as suggested by Bentz and Collins 2006) to the ε_y to obtain the first estimate for the ε_1 . By using Eq. 5.11 (associated with Eqs. 5.4 and 5.8-5.10), the new value for the ε_1 can be calculated. If the difference between the new ε_1 and the old value of ε_1 is not significant, the procedure can be stopped. The value of β can then be calculated by using Eq. 5.12 using Θ , ε_1 , w and a_g . Once the value of β is known, the shear resistance (V_r) of the wall is then calculated based on Eq. 5.15. The above procedure for calculation of β is applicable for ordinary concrete or SCC.

Procedure using experimental crack width: optimum value of β is calculated based on Eq. 5.13 using experimental values of w (at failure) and a_g . Eq. 5.17 is then used for the determination of shear resistance.

5.5.1 Illustration of shear resistance calculation for CFSWS-1

Material, geometric and other experimental parameters: $f'_c = 46$ MPa, $\theta = 45^\circ$, $A_c = 2$ (781) = 1,562 mm², crack spacing $s_\theta = 200$ mm, $a_g = 10$ mm, $d_v = 640$ mm, $h = 640$ mm, $t_c = 25$ mm, $t_s = 0.61$ mm; $F_s = 87$ MPa, experimental shear resistance (V) = 95 kN, $N = 0$

- **Iterative procedure:**

- Calculation of ε_y (Eq.5.8):

$$\varepsilon_y = \frac{M/d_v + 0.5N + V}{2A_c E_s} = \frac{Vh/d_v + 0.5N + V}{2A_c E_s} = \frac{95,000 \times 640/640 + 95,000}{2 \times 11250 \times 200,000} = 0.000042$$

Table No. 5.1: Calculation of ε_1 for CFSWS-1

Equation	Estimate of principal tensile strain ε_1 (Eq. 5.8) ε_y (Eq. 5.9)+0.0005 or new estimate	Principal compressive stress (f_2) Eq. (5.10)	Max, compressive stress ($f_{2 \max}$) Eq. (5.4)	Principal compressive strain (ε_2) Eq. (5.9)	New Estimate Eq. (5.7)
Iteration 1	0.0005422	1.471817884	46	-3.22562E-05	0.000117
Iteration 2	0.000117	1.802709799	46	-3.9581E-05	0.000124
Iteration 3	0.000124	1.791934911	46	-3.93421E-05	0.000124

Converged value of $\varepsilon_1 = 0.000124 > 0.0002$

- Crack width calculation: $w = s_0 \times \varepsilon_1 = 200 \times 0.0002 = 0.04 \text{ mm}$
- Calculation of β (Eq. 5.12): $\beta = \frac{0.33 \cot \theta}{1 + \sqrt{500\varepsilon_1}} = \frac{0.33 \cot 45^\circ}{1 + \sqrt{500 \times 0.0002}} = 0.251$

$$\beta \leq \frac{0.18}{0.31 + \frac{24w}{(a_g + 16)}} = \frac{0.18}{0.31 + \frac{24 \times 0.04}{(10 + 16)}} = 0.519; \rightarrow \beta = 0.251 \text{ (governs)}$$

- Calculation of shear resistance of the CFSWS based on Eq. 5.17a:

$$\begin{aligned} V_{\text{CFSWS}} &= 2 \frac{F_y}{\sqrt{3}} t_s d_v + \beta \sqrt{f'_c} t_c d_v + V_{\text{CFST}} \\ &= 2 \times \frac{552}{\sqrt{3}} \times 0.61 \times 640 + 0.211 \times \sqrt{46} \times 25 \times 640 + 0 \\ &= 235,315 + 27,207 = 262,522 \text{ N} = 262.52 \text{ kN} \end{aligned}$$

- Calculation of shear resistance of the CFSWS based on Eq. 5.17b:

$$\begin{aligned} V_{\text{CFSWS}} &= 2 (2 \tau_{cr} t_s d_v) + \beta \sqrt{f'_c} t_c d_v + V_{\text{CFST}} \\ &= 2 \times 2 \times 78 \times 0.61 \times 640 + 0.251 \times \sqrt{46} \times 25 \times 640 + 0 \\ &= 60,902 + 27,207 = 88,109 \text{ N} = 88.11 \text{ kN} \end{aligned}$$

- Calculation of shear resistance of the CFSWS based experimental maximum stress on peak load based on eq. 5.17c:

$$\begin{aligned} V_{\text{CFSWS}} &= 2 f_s t_s d_v + \beta \sqrt{f'_c} t_c d_v + V_{\text{CFST}} \\ &= 2 \times 87 \times 0.61 \times 640 + 0.251 \times \sqrt{46} \times 25 \times 640 + 0 \\ &= 67,930 + 27,207 = 95,137 \text{ N} = 95.14 \text{ kN} \end{aligned}$$

The 95.14 kN is just 0.15% higher than experimental shear resistance of 95 kN.

- **Procedure using experimental crack width:** Experimental crack width (w) = 0.3 to 0.6 mm; mean value = 0.40 mm

$$\beta = \frac{0.18}{0.31 + \frac{24 w}{(a_g + 16)}} = \frac{0.18}{0.31 + \frac{24 \times 0.45}{(10 + 16)}} = 0.248$$

$$\begin{aligned} V_{\text{CFSWS}} &= 2f_s t_s d_v + \beta \sqrt{f'_c} t_c d_v + V_{\text{CFST}} \\ &= 2 \times 87 \times 0.61 \times 640 + 0.248 \times \sqrt{46} \times 25 \times 640 + 0 \\ &= 67,930 + 26,928 = 94,858 \text{ N} = 94.86 \text{ kN} \end{aligned}$$

Use of experimental crack width also predicted the shear resistance close to experimental value.

5.5.2 Illustration of shear resistance calculation for CFSWS-2

Material, geometric and other experimental parameters: $f'_c = 46 \text{ MPa}$, $\theta = 30^\circ$, $A_c = 2 (781) = 1,562 \text{ mm}^2$, crack spacing $s_\theta = 1150 \text{ mm}$, $a_g = 10 \text{ mm}$, $d_v = 640 \text{ mm}$, $h = 1280 \text{ mm}$, $t_c = 30 \text{ mm}$, $t_s = 0.61 \text{ mm}$; $F_s = 41 \text{ MPa}$, Experimental shear resistance (V) = 85 kN, $N = 0$

- **Model CFSWS-2 was failed due to failure of frame-wall fasteners**
  - Calculation for fasteners shear strength

It was assumed that steel sheet will not tear off around the fasteners which are running through both concrete and steel sheet. But due to limited space for cover around fasteners, concrete could not give support to the sheet resulting tearing off profiled sheet during testing.

The each fastener with profiled sheet was tested and average strength was experienced 3.6 kN (Table 3.3). Therefore, total strength of the model CFSWS-2 was depending on lesser strength among shear wall or total strength of fasteners. As this model was failed due to failure of frame-wall fasteners, CFSWS-2 strength is equal to total strength of frame-wall fasteners. Total strength of the

frame-wall fasteners can be determined by multiplying one fastener's strength with total number of fasteners.

$$V_{\text{CFSWS}} = 14 \times 3.6 + 4 \times 8.5 = 84.4 \text{ kN}$$

The 84.4 kN is just 0.71% lower than experimental shear resistance of 85 kN.

5.5.3 Illustration of shear resistance calculation for CFSWS-3

Material, geometric and other experimental parameters: $f'_c = 47.1 \text{ MPa}$, $\theta = 48^\circ$, $A_c = 2 \times 1460 = 2920 \text{ mm}^2$, crack spacing $s_\theta = 150 \text{ mm}$, $a_g = 10 \text{ mm}$, $d_v = 640 \text{ mm}$, $h = 640 \text{ mm}$, $t_c = 25 \text{ mm}$, $t_s = 0.61 \text{ mm}$; $F_s = 90 \text{ MPa}$, Experimental shear resistance (V) = 146.84 kN, $N = 0$

- Iterative Procedure:**

Transformation of cross sectional Area:

The modular ratio (n) for the steel and concrete = $E_s / E_c = 200000 / 25000 = 8$

Transformed steel of composite column = Area of HSS section + Area of infilled concrete/ n

$$= (2 \times 75 + 2 \times 69) \times 3 + (69 \times 69) / 8$$

$$= 864 + 4761 / 8 = 1460 \text{ mm}^2$$

$$\varepsilon_y = \frac{M/d_v + 0.5N + V}{2A_c E_s} = \frac{Vh/d_v + 0.5N + V}{2A_c E_s}$$

$$= \frac{97,840 \times 640 / 640 + 97,840}{2 \times 2920 \times 200000} = 0.00001676$$

Table 5.2: Calculation of ε_1 for CFSWS-3

Equation	Estimate of principal tensile strain ε_1 (Eq. 5.8) ε_y (Eq. 5.9)+0.0005 or new estimate	Principal compressive stress (f_2) Eq. (5.10)	Max, compressive stress ($f_{2 \text{ max}}$) Eq. (5.4)	Principal compressive strain (ε_2) Eq. (5.9)	New Estimate Eq. (5.7)
Iteration 1	0.0006676	1.150069882	47.1	-2.45685E-05	0.000323
Iteration 2	0.000323	1.294110456	47.1	-2.76672E-05	0.000326
Iteration 3	0.000326	1.292673497	47.1	-2.76362E-05	0.000326

Converged value of $\varepsilon_1 = 0.000326 > 0.0002$ OK

- Crack width calculation: $w = s_\theta \times \varepsilon_1 = 150 \times 0.000326 = 0.0489 \text{ mm}$

○ Calculation of β (Eq. 5.12): $\beta = \frac{0.33 \cot \theta}{1 + \sqrt{500 \varepsilon_1}} = \frac{0.33 \cot 45^\circ}{1 + \sqrt{500 \times 0.000326}} = 0.235$

$$\leq \frac{0.18}{0.31 + \frac{24 w}{(a_g + 16)}} = \frac{0.18}{0.31 + \frac{24 \times 0.0489}{(10 + 16)}} = 0.507; \rightarrow \beta = 0.235 \text{ (governs)}$$

- Calculation of shear resistance of the CFSWS based on Eq. 5.17a:

$$\begin{aligned} V_{\text{CFSWS}} &= 2 \frac{F_y}{\sqrt{3}} t_s d_v + \beta \sqrt{f'_c} t_c d_v + V_{\text{CFST}} \\ &= 2 \times \frac{552}{\sqrt{3}} \times 0.61 \times 640 + 0.235 \times \sqrt{47.1} \times 25 \times 640 + 49,000 \\ &= 235,315 + 25,814 + 49,000 = 310,129 \text{ N} = 310.13 \text{ kN} \end{aligned}$$

- Calculation of shear resistance of the CFSWS based on Eq. 5.17b:

$$\begin{aligned} V_{\text{CFSWS}} &= 2 (2 \tau_{cr} t_s d_v) + \beta \sqrt{f'_c} t_c d_v + V_{\text{CFST}} \\ &= 2 \times 2 \times 78 \times 0.61 \times 640 + 0.235 \times \sqrt{47.1} \times 25 \times 640 + 49,000 \\ &= 60,902 + 25,814 + 49,000 = 135,716 \text{ N} = 135.72 \text{ kN} \end{aligned}$$

- Calculation of shear resistance of the CFSWS based experimental maximum stress on peak load based on eq. 5.17c:

$$\begin{aligned} V_{\text{CFSWS}} &= 2 f_s t_s d_v + \beta \sqrt{f'_c} t_c d_v + V_{\text{CFST}} \\ &= 2 \times 90 \times 0.61 \times 640 + 0.235 \times \sqrt{47.1} \times 25 \times 640 + 49,000 \\ &= 70,272 + 25,814 = 146,086 \text{ N} = 146.09 \text{ kN} \end{aligned}$$

The 146.09 kN is just 1.21% lower than experimental shear resistance of 146.84 kN.

- **Procedure using experimental crack width:** Experimental crack width (w) = 0.4 to 0.6 mm; mean value = 0.50 mm

$$\beta = \frac{0.18}{0.31 + \frac{24 w}{(a_g + 16)}} = \frac{0.18}{0.31 + \frac{24 \times 0.5}{(10 + 16)}} = 0.233$$

$$\begin{aligned} V_{\text{CFSWS}} &= 2 f_s t_s d_v + \beta \sqrt{f'_c} t_c d_v + V_{\text{CFST}} \\ &= 2 \times 90 \times 0.61 \times 640 + 0.233 \times \sqrt{47.1} \times 25 \times 640 + 49,000 \\ &= 70,272 + 25,618 + 49,000 = 144,890 \text{ N} = 144.89 \text{ kN} \end{aligned}$$

Use of experimental crack width also predicted only 1.35 % lower shear resistance as compare to experimental value 146.84 kN.

5.5.4 Illustration of shear resistance calculation for CFSWS-4

Material, geometric and other experimental parameters: $f'_c = 46.5$ MPa, $\theta = 45^\circ$, $A_c = 2 \times 1460 = 2920$ mm², crack spacing $s_0 = 150$ mm, $a_g = 10$ mm, $d_v = 640$ mm, $h = 1280$ mm, $t_c = 30$ mm, $t_s = 0.61$ mm; $f_s = 88$ MPa, Experimental shear resistance (V) = 121.24 kN, $N = 0$

- **Iterative Procedure:**

- Calculation of ε_y (Eq.5.8):

$$\varepsilon_y = \frac{M/d_v + 0.5N + V}{2A_c E_s} = \frac{Vh/d_v + 0.5N + V}{2A_c E_s}$$

$$= \frac{94,980 \times 1280/640 + 94,980}{2 \times 2920 \times 200000} = 0.000244$$

Table No. 5.3: Calculation of ε_1 for CFSWS-4

Equation	Estimate of principal tensile strain ε_1 (Eq. 5.8) ε_y (Eq. 5.9)+0.0005 or new estimate	Principal compressive stress (f_2) Eq. (5.10)	Max, compressive stress ($f_{2 \max}$) Eq. (5.4)	Principal compressive strain (ε_2) Eq. (5.9)	New Estimate Eq. (5.7)
Iteration 1	0.0007440	1.974544631	46.5	-4.29239E-05	0.000651
Iteration 2	0.000651	2.023819368	46.5	-4.40072E-05	0.000653
Iteration 3	0.000653	2.022951903	46.5	-4.39881E-05	0.000653

Converged value of $\varepsilon_1 = 0.000653 > 0.0002$ OK

- Crack width calculation: $w = s_0 \times \varepsilon_1 = 150 \times 0.000653 = 0.0979$ mm
- Calculation of β (Eq. 5.12): $\beta = \frac{0.33 \cot \theta}{1 + \sqrt{500\varepsilon_1}} = \frac{0.33 \cot 40^\circ}{1 + \sqrt{500 \times 0.000653}} = 0.250$

$$\leq \frac{0.18}{0.31 + \frac{24w}{(a_g + 16)}} = \frac{0.18}{0.31 + \frac{24 \times 0.0979}{(10 + 16)}} = 0.499; \rightarrow \beta = 0.250 \text{ (governs)}$$

- Calculation of shear resistance of the CFSWS based on Eq. 5.17a:

$$V_{\text{CFSWS}} = 2 \frac{F_y}{\sqrt{3}} t_s d_v + \beta \sqrt{f'_c} t_c d_v + V_{\text{CFST}}$$

$$= 2 \times \frac{552}{\sqrt{3}} \times 0.61 \times 640 + 0.250 \times \sqrt{46.5} \times 25 \times 640 + 26000$$

$$= 235,315 + 27,307 + 26000 = 288,622 \text{ N} = 288.62 \text{ kN}$$

- Calculation of shear resistance of the CFSWS based on Eq. 5.17b:

$$V_{\text{CFSWS}} = 2 (2 \tau_{cr} t_s d_v) + \beta \sqrt{f'_c} t_c d_v + V_{\text{CFST}}$$

$$\begin{aligned}
&= 2 \times 2 \times 78 \times 0.61 \times 640 + 0.250 \times \sqrt{46.5} \times 25 \times 640 + 26000 \\
&= 60,902 + 27,307 + 26000 = 114,061 \text{ N} = 114.06 \text{ kN}
\end{aligned}$$

- Calculation of shear resistance of the CFSWS based experimental maximum stress on peak load based on eq. 5.17c:

$$\begin{aligned}
V_{\text{CFSWS}} &= 2f_s t_s d_v + \beta \sqrt{f'_c} t_c d_v + V_{\text{CFST}} \\
&= 2 \times 88 \times 0.61 \times 640 + 0.235 \times \sqrt{46.5} \times 25 \times 640 + 26000 \\
&= 68,710 + 27,307 + 26000 = 122,017 \text{ N} = 122.02 \text{ kN}
\end{aligned}$$

The 122.02 kN is just 0.70% higher than experimental shear resistance of 121.24 kN.

- **Procedure using experimental crack width:** Experimental crack width (w) = 0.4 to 0.7 mm; mean value = 0.55 mm

$$\beta = \frac{0.18}{0.31 + \frac{24 w}{(a_g + 16)}} = \frac{0.18}{0.31 + \frac{24 \times 0.55}{(10 + 16)}} = 0.220$$

$$\begin{aligned}
V_{\text{CFSWS}} &= 2f_s t_s d_v + \beta \sqrt{f'_c} t_c d_v + V_{\text{CFST}} \\
&= 2 \times 88 \times 0.61 \times 640 + 0.220 \times \sqrt{46.5} \times 25 \times 640 + 26000 \\
&= 68,710 + 24,018 + 26000 = 118,728 \text{ N} = 118.73 \text{ kN}
\end{aligned}$$

Use of experimental crack width also predicted only 2.11% lower shear resistance as compare to experimental value 121.24 kN.

5.6 Discussions and Conclusions

The experimental and predicted parameters for analytical models are summarized in Table 5.4. These parameters are used in the calculation of shear resistance in the previous section. The comparison of analytical and experimental shear resistance of CFSWS is presented in Table 5.5. Model CFSWS-2 failed due to shearing off wall-frame fasteners; therefore, shear strength was equal to shear strength of wall-frame fasteners.

Table 5.4: Experimental and predicted model parameters

CFSWS Models	a_g (mm)	Θ	S_Θ (mm)	$\tau = 2 \tau_{cr}$ MPa	f_s MPa	f'_c MPa	Crack width (w), mm		β	
							Model predicted	Expt.	Iterative	Crack width
CFSWS-1 (pinned steel framed one storey)	10	45°	200	78	87	46	0.04	0.3- 0.6 (0.45*)	0.251	0.248
CFSWS-3 (fixed CFST framed one storey)	10	45°	150	78	90	47.1	0.0489	0.4- 0.6 (0.50*)	0.235	0.233
CFSWS-4 (fixed CFST framed two storey)	10	40°	150	78	88	46.5	0.0979	0.4-0.7 (0.55*)	0.250	0.220
a_g : Maximum aggregate size; Θ : Average angle of diagonal cracks; S_Θ : Average diagonal crack spacing; τ_{cr} = Shear buckling stress of sheet, τ = Post buckling shear stress of sheet, f_s : Actual experimental shear stress, f'_c : Cylinder concrete strength, β : Shear retention factor; *Mean value										

Table 5.5: Comparison of analytical and experimental shear resistance of CFSWS

CFSWS Models	Shear Resistance of CFSWS (kN)							Ratio Expt./Analy.	
	Analytical Model						Expt.	Iterative (crack width)	
	Profiled concrete core (V_c) Iterative (crack width)	Profiled steel sheets ($2V_s$)		Frame (V_{CFST})	Analytical CFSWS ($V_{CFSWS} = V_c + 2V_s + V_{CFST}$)		CFSWS (V_{CFSWS})		
		Buckling stress Eq. 5.15b	Expt. stress Eq. 5.15c		Buckling stress Eq. 5.17b	Expt. stress Eq. 5.17c		Buckling stress	Actual stress
1	2	3	4	5	6 (2+3+5)	7 (2+4+5)	8	9 (8/6)	10 (8/7)
CFSWS-1 (pinned steel framed one storey)	27.21 (26.93)	60.90	67.93	0	88.11	95.14 (94.89)	95	1.08	0.99 (1.01)
CFSWS-2 (pinned steel framed two storey)	--		--	0		84.4	85	--	1.01
CFSWS-3 (fixed CFST framed one storey)	25.81 (25.62)	60.90	70.27	49	138.17	145.09 (144.89)	146.84	1.06	1.01 (1.01)
CFSWS-4 (fixed CFST framed two storey)	27.31 (28.99)	60.90	68.71	26	114.06	122.02 (123.7)	121.24	1.06	0.99 (0.98)

Iterative and crack width procedures provided similar shear resistance of the concrete core, hence both of them can be used (Table 5.5). Analytical shear resistances of both one/two storey CFSWSs on post buckling stress and actual observed shear stress are found very close to those of experimental values (as ratio of experimental to analytical values ranges between 0.98 and 1.08). However, post buckling shear strength model (Eq. 5.17b) provided lower values compared to actual shear stress model based on actual experimental shear stress (Eq. 5.17c).

CHAPTER SIX

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

6.1 Summary

This study has focused on the behaviour of a novel form of composite framed shear wall system (CFSWS) consisting of pinned and fixed building frames (one and two storey), innovative frame-wall connections and double skin profiled steel sheet composite wall infilled with self-consolidating concrete (SCC) under in-plane monotonic lateral loading. The wall-frame fasteners and steel sheet-concrete interaction are important in this type of wall system to prevent failure due to boundary connections and elastic buckling of profile sheets for the optimization of structural system capacity. The intermediate fasteners connecting two outer sheets through infill concrete core provided steel sheet-concrete composite action.

This research consisted of experimental and analytical investigations on the CFSWS models which provided information on the strength, stiffness, load-deformation response, steel sheet-concrete interaction, stress-strain characteristics, influence of steel-concrete material properties, concrete cracking/propagation, and failure modes. In addition, experimental results were utilized to validate the performance of developed analytical models.

Tests were conducted on one and two storey CFSWS models using pinned steel frames and fixed concrete filled steel tube (CFST) frames. In addition, one and two storey bare fixed CFST frames were also tested to analyse shear resistance contribution of frame to the overall shear strength of CFSWS. SCC was used in both CFST beam-column frame and composite walls due to its flowability and self-consolidation properties.

6.2 Conclusions

The following conclusions are drawn from experimental and analytical investigations on the composite framed shear wall system:

- One storey pinned CFSWS model failed due to the development of cracks and steel-concrete separation along the zone of compression and tension diagonals. Localized failure associated with sheet distortion/buckling and steel-concrete separation at both compression diagonal corners occurred prior to the start of frame column bending at post peak failure stage.
- The failure of two storey pinned CFSWS model was associated with tearing of profiled steel sheet at fastener locations, development of diagonal concrete cracks and shearing-off of fasteners at different locations including those at the bottom and intermediate frame beams. The presence of 1st storey intermediate beam prevented crack development in the concrete core of the composite wall (same wall extended to both floors) between the two floors. Localized failures at the top corners were observed.
- One and two storey bare fixed CSFT frames failed due to column-beam joint failure associated with the tearing of column tube plate in the final stage.
- The failure of one and two storey CFSWS with fixed CFST frame was associated with buckling of steel sheet at the loaded corner, followed by the development of diagonal concrete cracking of concrete core, failure of wall-frame fasteners and finally CFST frame joint failure. Overall, failure was governed by wall failure rather than frame failure.
- Initial shear stiffness of one and two storey pinned steel frame CFSWS were 0.23 mm/kN and 0.49 kN/mm, respectively while corresponding energy absorbed (area under the shear load-deformation response) 2.98 kJ and 2.8 kJ, respectively. Initial stiffness of one and two storey CFST bare frames were 1.12 mm/kN and 1.84 mm/kN, respectively while the energy absorbed were 3.04 kJ and 1.35 kJ, respectively.
- Initial stiffness of one storey and two storey CFSWS with fixed CFST frame were 0.14 mm/kN and 0.82mm/kN while the energy absorbed were 6 kJ and 6.9kJ, respectively.
- As expected, initial stiffness of one storey CFSWSs was higher compared to their two storey counterparts – which signified the higher ductility of two storey CFSWSs compared to their one storey counterparts. The energy absorbed was lower in two storey pinned steel framed CFSWS compared to one storey. However, for CFSWS with CFST frames the energy absorbed was higher in two story compared to that of one story..
- One storey CFST bare frame had 47% higher shear resistance and underwent 50% lower shear displacement compared to their two storey counterpart. One storey pinned and

fixed CFSWS had higher shear resistance and underwent lower shear displacement, respectively compared to two storey CFSWS. The shear strength of CFSWS with pinned steel frame was lower compared to those with CFST frame as expected because pinned frame did not contribute to the shear resistance capacity. The CFST boundary frame had contributed to the shear strength of CFSWS only after the failure in-filled composite wall.

- Analytical models for the shear resistance of the CFSWS were developed based on existing theoretical models and their performance is validated through experimental results. Shear resistances of the concrete core calculated by using iterative and crack width methods were found to be identical. Both methods can be used to predict the shear resistance of concrete core.
- Shear resistances of CFSWS calculated from proposed analytical models based on post buckling stress (considering tension field development) and actual shear stress (derived from experimental results) are found to be in close agreement with those obtained from experiments. Therefore, the proposed analytical models can be used for the prediction of shear resistance of the composite walls.
- CFSWS exhibited high shear resistance with excellent ductility and energy absorbing capacity. Such novel system can be used in new construction and also in the strengthening of the lateral load resisting system in existing buildings. The construction cost of this walling system can be comparable to other traditional systems – as profiled steel sheet and steel tube act as permanent formwork (no formwork needed) and reinforcement in composite wall and CFST frame.

6.3 Recommendations for Future Research

The following recommendations are suggested for further research studies:

- More experiments with closer intermediate fasteners should be carried out to prevent buckling of steel sheet prior to yielding. In this way, full shear capacity of profiled steel sheets can be utilized.
- More investigations on one and two story CFSWS with different types of frame-wall

connection are to be carried out to investigate the behaviour under monotonic and cyclic shear as well as under impact loading.

- Investigations on the preparation of design tables are to be conducted to provide optimum spacing between intermediate fasteners to prevent steel sheet buckling based on different thickness, shape and yield strength of profiled steel sheets.
- Fire resistance of CFSWS with varying temperature and duration should be conducted.
- Research should be directed towards the development of Code based design procedures for the calculation of shear resistance of CFSWS using partial and full composite action between steel sheet and concrete. The partial composite action may improve the composite wall ductility and make it more suitable for structures located in high seismic risk.
- Efforts should be made in association with construction and manufacturing industries, to manufacture commercial prefabricated CFSWS as viable alternative to traditional systems.

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