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Behaviour of double skin profiled composite shear wall system under in-plane monotonic, cyclic and impact loadings

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Behaviour of Double Skin Profiled Composite Shear Wall System under In-plane Monotonic, Cyclic and Impact Loadings

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A Dissertation

Presented to Ryerson University

In Partial Fulfillment of the Requirements for the
Doctor of Philosophy degree in the Program of Civil Engineering

Toronto, Ontario, Canada, 2011

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Behaviour of Double Skin Profiled Composite Shear Wall System under In-plane Monotonic, Cyclic and Impact Loadings

Shahryar Rafiei, Doctor of Philosophy, 2011

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ABSTRACT

This research investigated the behaviour of a new form of composite shear wall system consisting of two skins of profiled steel sheeting and an infill of concrete under in-plane monotonic, cyclic and impact loading. The extensive experimental, analytical and numerical investigations of composite shear walls provided information on strength, stiffness, load-deformation response, steel sheet-concrete interaction, stress-strain characteristics and failure modes.

Eight composite wall specimens with overall dimensions of 1626 mm (height) x 720 mm (width) were tested under monotonic, cyclic and impact loading. Steel sheet-concrete connections were provided by intermediate fasteners to generate composite action. Two types of steel sheets classified based on strength as mild and high strength and also, two types of concrete-infill namely Self-Consolidating Concrete (SCC) and Engineered Cementitious Composites (ECC) were used to construct the walls. An analytical model for shear resistance of the composite wall was developed 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 performance of developed numerical models was validated against experimental results and then the models were utilized to carry out an extensive parametric study to understand the influence of material and steel-concrete interaction on the structural behaviour of the walls.

The advantage of using ECC over SCC and mild over high strength steel was exhibited through more ductile wall behaviour. The intermediate fasteners provided sufficient steel-concrete composite action to prevent early elastic buckling of the profiled steel sheets. No reduction in the shear strength of the composite wall was observed after the application of about 3150 J impact energy. The developed finite element models were capable of simulating the behaviour of the composite shear wall under monotonic and impact loading with reasonable accuracy. Experimental and analytical shear resistance of composite walls showed very good agreement and therefore, the analytical model can be used for predicting shear resistance of the composite walls. This research confirms the viability of using this type of composite shear wall system in 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	Width of a profiled steel sheet / shorter dimension of a rectangular panel
C_d	Wave speed of material
d	Original length of one repeating profiled steel sheet (Fig. 2.9)
d_c	Damage variable in compression (Fig. 7.4)
d_t	Damage variable in tension (Fig. 7.3)
d_v	Effective shear depth
D	Diameter of the central circular openings
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_{fe}	Energy transmitted into structure at maximum deflection
E_{final}	Kinetic energy of a projectile at the time of separation from a structure
E_{fu}	Ultimate absorption energy
E_I	Internal energy of a structure
$E_{initial}$	Kinetic energy of a projectile before impact
E_{KE}	kinetic energy of a structure
E_p	Energy dispersed in the projectile by plastic deformation
E_s	Modulus of elasticity of steel
$E_{s,f}$	Energy lost by sound and friction
E_{TOT}	Total energy in a structure
E_V	Energy absorbed by viscous dissipation
E_W	Work applied by the external forces
E_0	Initial elastic stiffness of concrete (Figs. 7.3 and 7.4)
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_0	Constant pulse load
$F(\tau)$	Impact load
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_e	impulse transmitted during the impact collision at maximum deflection
I_u	Amount of impulse at ultimate failure
I_y	Moment of inertia of one repeating profiled steel sheet about “y” axis (Fig. 2.9)
k	Stiffness of a SDOF structure
K	A factor to control the slop of the stress-strain curve (Eq. 7.10)
k_v	Shear buckling coefficient
l_w	Overall width of a profiled steel sheet
L	Frame bay width / width of a profiled steel sheet
L_e	Shortest element length in FE model
m	Mass of a SDOF structure
M	Mass of a projectile
M_f	Bending moment due to factored loads
M_p	Smaller of the plastic moment capacity of columns or beams
n	Curve-fitting factor (Eq. 7.10)
N_f	Factored axial load normal to the cross-section of the wall
$P_{c/d}$	The load associated with the initiation of concrete cracking and debonding of the profiled steel sheet from the concrete core
s_0	Diagonal crack spacing (Fig. 6.9)
s_z	Crack spacing parameter

s_{ze}	Effective crack spacing accounting for aggregate size effect
S_{diag}	Diagonal spacing of the perforations in a steel plate
t	Time (sec)
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
t_w	Infill plate thickness
U	Impact energy capacity of a structure
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 force due to factored loads
V_{final}	Velocity of the projectile after separation from the structure
$V_{initial}$	Velocity of the projectile before impact collision
V_r	Total shear resistance of the composite wall
V_s	Shear resistance provided by one profiled steel sheet
V_{yp}	Shear strength of the shear wall
w	Crack width
w_c	Stiffness recovery in compression (Fig. 7.5)
w_t	Stiffness recovery in tension (Fig. 7.5)
$x(t)$	Displacement response of a SDOF structure
x_{max}	Maximum displacement of the composite wall at top
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
Δt_{stable}	Stability time limit
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 (Fig. 6.7b)
ε_{nom}	Nominal strain
ε_{p1}	Maximum principal strain in profiled steel sheet
ε_{p2}	Minimum principal strain in profiled steel sheet
ε_t	Total strain (Fig. 7.3)
ε_1	Principal tensile strain on average in cracked concrete
ε_2	Principal compressive strain on average in cracked concrete
$\tilde{\varepsilon}_c^{in}$	Inelastic strain (Fig. 7.4)
$\tilde{\varepsilon}_c^{pl}$	Plastic compressive strain in concrete (Fig. 7.4)
ε_{ln}^{pl}	Logarithmic plastic strain (Eq. 7.12)
$\tilde{\varepsilon}_t^{ck}$	Cracking strain (Fig. 7.3)
$\tilde{\varepsilon}_t^{pl}$	Plastic tensile strain in concrete (Fig. 7.3)
θ	Angle of inclination of diagonal compressive stresses to the longitudinal axis of the wall
μ_Δ	Displacement ductility ratio
ν_s	Poisson's ratio of steel, can be taken as 0.3
ρ	Mass density
σ	Effective normal stress
σ_c	Compressive stress in concrete
σ_{cu}	Ultimate compressive stress in concrete (Fig. 7.4)
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
σ_{t0}	Failure stress in tension (Fig. 7.3)
σ_{true}	True stress (Eq. 7.11)
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 ($\phi_s = 0.90$)
ω	Natural circular frequency

CHAPTER 1

INTRODUCTION

1.1 General

Reinforced concrete (RC) shear walls have been traditionally used as lateral load resisting systems in many structures. Recently steel plate shear walls have been also used as lateral load resisting system in mid-rise and tall buildings (Astaneh-Asl 2001a, b). Despite many economical and structural advantages, both steel and RC shear walls have some disadvantages. 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. The disadvantage of a steel shear wall is associated with the buckling of the compression zone, which results in reduced shear stiffness, strength, and energy dissipation capacity (Zhao et al. 2004). A steel-concrete composite shear wall can have the benefits of both steel and RC shear walls and yields the best traits of concrete and steel.

The concept of composite wall was originated from the floor structure using profiled steel deck and concrete (Wright et al. 1992). Composite walling as shear or core walls in steel frame buildings has many advantages (Hossain et al. 2004a, c, d). In building construction stage, profiled steel sheeting can act as a bracing system to the steel frame against lateral loads and also can act as a permanent formwork for infill concrete. During the in-service stage, profiled steel sheets and infill concrete work together to resist lateral loads (Wright et al. 1994; Hossain 1995). A typical composite walling system consisting of two skins profiled steel sheets and a concrete infill is shown in Fig. 1.1.

The interaction between the profiled steel sheet and concrete has an important role in the composite action of the system (Hossain et al. 1998a). The interface shear bond failure is a limiting criterion for designing this kind of system. The bond between the steel sheet and concrete can be improved by embossments or using other forms of shear connector. The

mechanical interlock at the sheet-concrete interface may govern the brittle or ductile mode of failure of such composite wall (Hossain et al. 2004c).

Figure 1.1: Application of composite walling system in steel frame building

Previous work on this form of composite walling system under in-plane shear has been conducted without shear connectors or embossments at the steel-concrete interface. To date, no research has been conducted to study the behaviour of such walls under in-plane cyclic or impact loadings. In this research, the behaviour of profiled composite walls (with mechanical shear connectors) under in-plane monotonic, cyclic and impact loadings are studied.

In order to increase the shear resistance of the composite wall, the global buckling of the profiled steel sheets should be prevented. The proposed study prevents the global buckling of the sheets by using adequate intermediate fasteners (providing sheet-concrete interface connection) along the height and width of the composite wall. The reason for preventing the global buckling of the sheets is to ensure that the profiled steel sheet reaches almost its yield stress under shear loading. This can be achieved in composite wall due to the fact that the concrete core provides bracing to the profiled steel sheet and hence, prevents its buckling prior to reaching yielding. In other words, the concrete core acts as a stiffener and prevents buckling of the sheet.

In this study, two types of steel strength namely; mild and high strength and also two types of infill concrete namely; Self-Consolidating Concrete (SCC) and Engineered Cementitious Composites (ECC) are used to construct the double skin profiled composite shear walls. SCC is very flowable (Khayat 1999) and can flow into place between two profiled steel sheets without vibration. On the other hand, ECC is also flowable and very ductile concrete which exhibits a strain capacity up to several hundred times larger than normal concrete due to its micro-cracking characteristics (Sahmaran et al. 2009a, b). It is expected that utilizing ECC in the double skin profiled composite shear wall will improve ductility and structural performance of the composite wall.

1.2 Significance of the Research

Numerous studies were conducted in the past on steel plate shear walls and steel plate composite shear walls. Experimental data and design methods, especially for steel plate shear walls, are available (CSA-S16 2009). The concept of a double skin profiled composite shear wall using mechanical shear connectors is new and design methods are not currently available. This research provides information on the strength, stiffness, load-deformation response, interaction between profiled steel sheets and concrete, and also the possible failure modes, based on the experimental, analytical and numerical investigations. The results of this research contribute significantly to the understanding of the behaviour of this type of structural element and might be of help to the Canadian Standard Association CSA-S16 regarding the design of double skin profiled composite shear wall.

1.3 Scope and Objectives

The primary focus of this research is to study the behaviour of double skin profiled composite shear wall when subjected to in-plane monotonic, cyclic and impact loading. To examine such behaviour, six tests were conducted on large-scale, one storey composite shear wall under monotonic and cyclic loading conditions. In addition, two tests were also conducted under impact loading. A nonlinear finite element (FE) model was developed in order to reasonably simulate the monotonic and cyclic behaviour of the double skin profiled composite shear wall and the accuracy of the FE model was validated against experimental results. The finite element model was then used for a parametric study to investigate the influence of steel strength, steel-concrete interface and concrete type/strength on the ultimate load capacity, stress-strain characteristics and failure modes of the composite shear walls. A nonlinear finite element model was also developed in order to simulate the wall behaviour under impact loading. Moreover, an analytical model was developed for the shear resistance of double skin profiled composite walls. Therefore, the main objectives of this research are to:

- 1) Perform experimental work on double skin profiled composite shear wall system under monotonic and cyclic loading to obtain comprehensive information on strength, deformation, possible modes of failure, and ductility of the system. Also compare the results for two different types of concrete (Self-Consolidating Concrete “SCC” and Engineered Cementitious Composites “ECC”) and investigate the possible advantages of using ECC versus SCC.
- 2) Perform impact tests on double skin profiled composite shear wall system to obtain comprehensive information on strength/stiffness degradation of the wall, deformation, energy absorption and possible failure modes of the wall.
- 3) Obtain the residual shear strength of wall specimen subjected to impact loading by post-testing to failure under monotonic shear. This will allow for establishing correlations between impact energy, residual strength and associated damage.

- 4) Carryout analytical and design oriented analyses to develop design equations for the shear resistance of the composite walls.
- 5) Perform numerical analysis (finite element modeling) of the wall using ABAQUS software to develop numerical model for simulating the behaviour of the shear wall system under monotonic/cyclic loading validated by experimental results. Then, utilized the model to carryout extensive parametric numerical investigations to study the influence of various parameters related to materials and steel-concrete interface connection on wall behaviour.
- 6) Perform finite element (FE) modeling using ABAQUS software to develop numerical model to simulate the composite wall behaviour under impact loading.

1.4 Outline of Thesis

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

Chapter 1 presents brief introduction about the novel form of the composite wall, research significance and objective of the study.

Chapter 2 presents a comprehensive literature review on different types of shear wall namely; steel plate shear wall, profiled steel sheet shear wall, composite shear wall, and double skin profiled composite shear wall. The properties of two types of concrete namely; Self-Consolidating Concrete (SCC) and Engineered Cementitious Composites (ECC) used as an infill in the composite shear wall are also reviewed. Moreover, behaviour of structural elements such as slabs, beams and shear walls subjected to impact loading is presented.

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

Chapter 4 presents the experimental test results on six specimens under monotonic and cyclic loading. The observations during the test and the test results for monotonic loading are presented in the format of load versus displacement, principal strains & stresses versus displacement, shear strains & stresses versus displacement and failure modes of specimens. For cyclic loading, the stiffness degradation versus number of cycles and envelopes of load-displacement are presented in addition to those results described under monotonic loading.

Chapter 5 provides some background on structural behavior under impact and the test results of two wall specimens tested under this type of loading. The results from the impact loading are presented in the format of acceleration versus time, Fast Fourier Transform (FFT) of the acceleration data, speed of the projectile before and after impact, period of the composite shear wall after impact, load versus displacement from monotonic loading after the impact test, and pictures showing failure modes of specimens.

Chapter 6 presents an analytical model to predict the shear resistance of the double skin profiled composite shear walls.

Chapter 7 describes in detail the finite element model developed to predict the behavior of the double skin profiled composite shear wall under monotonic and cyclic loading. Some strategies in nonlinear dynamic explicit Finite Element Method (FEM) for simulating a quasi-static response are discussed in detail. The results from Finite Element Analysis (FEA) and experimental results are also compared. Moreover, the parametric study results from the double skin profiled composite shear wall using FE model are presented at the end of Chapter. Some of the investigated parameters are steel sheet strength, concrete strength and location of intermediate steel-concrete interface connectors.

Chapter 8 outlines the detail of the finite element model developed to predict the behavior of the double skin profiled composite shear wall under impact loading. The results from FEA and experimental data are compared.

Chapter 9 presents the main conclusions of the study along with some suitable recommendations for future work on the double skin profiled composite shear wall system.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Shear walls are lateral load-resisting systems capable of effectively bracing a building against both wind and earthquake forces. Traditionally reinforced concrete shear walls were used in reinforced concrete building structures for many years to brace the building from lateral loads. In less than 40 years, other types of shear walls were developed to brace mostly steel structures from lateral loads. The system for this new type of shear wall consists of vertical infill panel one story high and one bay wide connected to the surrounding beam-column frame. The infill panel can be made of flat steel plate, profiled steel plate, composite plate, or double skin profiled steel plate. The infill panels are installed in one or more bays for the full height of a building to form a stiff cantilever wall. This type of shear walls is well-suited for new construction and it also offers relatively simple mean for the seismic upgrading of existing steel or concrete structures. In this chapter, the development and recent investigations on different types of this innovative shear walls are presented and some current researches on the effect of impact loading on structural elements are reviewed.

2.2 Steel Plate Shear Wall

Many researchers studied the behaviour and design of steel plate shear walls since the early 1970s. The primary designs 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; Adam et al. 2000; Sabouri-Ghomi et al. 2005) and in the United States (Elgaaly et al. 1993; Caccese et al. 1993; Xue and Lu 1994; Astaneh-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).

This section summarizes some of the previous research on thin unstiffened 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 unstiffened 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 Fig. 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:

$$\tan^4 \alpha = \frac{1 + \frac{L t_w}{2 A_c}}{1 + \frac{h t_w}{A_b}} \quad (2.1)$$

where, L is the frame bay width, t_w is the infill plate thickness, h is the frame storey height and A_c & 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 Fig. 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:

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

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 Fig. 2.3. The corners of the plates were cut away in a circular arc.

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.

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

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.

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.

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

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.

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 Bauschinger 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.

Rezai (1999) conducted shake-table testing on a 4-storey, quarter-scale steel plate shear wall to study the seismic performance of multi-storey shear walls. The 4-storey shake table specimen is shown in Fig. 2.5. All the infill plates were 1.5 mm thick and all beam-to-column connections were moment-resistant connections. At each storey, steel plates were placed on the beams to provide a storey mass of 1700 kg. The specimen was braced in the out-of-plane direction.

Figure 2.5: Four-storey steel plate shear wall test specimen (Rezai 1999)

Rezai (1999) performed a low-amplitude vibration test on the 4-storey specimen before doing the shake table testing to determine its frequency. The fundamental frequency of the shake table specimen was obtained as 6.1 Hz in the longitudinal direction. The test specimen was subjected to a number of site-recorded and synthetically generated ground motions at varying intensities.

The limited capacity of the shake table prevented a significant inelastic response from the specimen. Thus, the interpretations and recommendations by the researcher were based on the response of the system mainly in the elastic range. From load-deformation plots of all four storey levels, it was found that the majority of the energy was dissipated by the first storey and also, the first storey was dominated mostly by shear deformations.

Lubell et al. (2000) performed a quasi-static cyclic testing for a single bay; two single-storey and one four-storey steel plate shear wall specimens. All the specimens were a quarter-scale model of a typical office building core. The four storey shear wall is depicted in Fig. 2.6.

Figure 2.6: Four-storey steel plate shear wall test specimen (Lubell et al. 2000)

All the specimens were tested under quasi-static cyclic loading. The beam-to-column connections were moment resisting connections in all the specimens. The load history followed the procedures recommended in ATC-24 (ATC 1992) guidelines. All the specimens showed well-defined load-deformation envelopes, high initial stiffness, and stable hysteresis behaviour. Lubell et al. (2000) also studied a series of numerical analyses of the test specimens using a nonlinear frame analysis program. The researchers concluded that the design standard at that time, CAN/CSA S16.1-M94, did not consider some design-related issues like significant over prediction of a structure elastic stiffness under certain conditions, the influence of aspect ratio,

the effect of large overturning moments, and the potential for undesirable yielding sequences of the shear wall components for multi-storey steel plate shear walls.

Behbahanifard (2003) conducted a quasi-static cyclic testing on a large scale, three-storey unstiffened steel plate shear wall specimen, as shown in Fig. 2.7.

Figure 2.7: Three-storey steel plate shear wall test specimen (Behbahanifard 2003)

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 Bauschinger 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.

Vian et al. (2009a) experimentally investigated three half scale, single-storey, single bay detailed perforated steel plate shear wall specimens under quasi-static cyclic loading up to their maximum displacement and strength capacity. The researchers compared the experimental test

results of two proposed perforation methods for allowing passage of utilities through the steel plate shear walls. The first method altered a solid panel by having reinforced, quarter circle cut-out from each upper corner of the steel plate shear wall. The second proposed method introduced multiple circular perforations which were arranged in regularly spaced within the perimeter of the steel plate shear wall. Both types of the shear wall are shown in Fig. 2.8. A shear wall with solid steel plate was also tested as a reference.

Figure 2.8: Single-storey, single bay perforated steel plate shear wall specimens; (a) Quarter circle cut-out from each upper corner, (b) Multiple circular perforations (Vian et al. 2009a)

The tests were performed using displacement-controlled quasi-static cyclic loading. The loading protocol was a combination of the ATC-24 (ATC 1992) and the AISC Seismic Provision requirements for loading sequence in cyclic loading (AISC 2005).

Vian et al. (2009a) implemented low yield strength (LYS) steel for the first time into a steel plate shear wall. It was observed that the perforated shear wall with multiple regularly spaced holes has more ductility and there is no need for stiffeners around the perforations. This system was also recommended for use in steel plate shear wall applications where the minimum available plate thickness is larger than that required to reduce the effective strength of the solid steel plate.

Vian et al. (2009b) also conducted finite element analyses using ABAQUS/Standard on the three tested specimens. Both the infill steel panel and boundary frame members were modeled using the four-nodes, general purpose shell element with reduced integration (S4R). In general, good agreement was observed between the test results and the hysteresis curves obtained from the

analysis. The analytical model of the perforated steel plate shear wall (SPSW) was further extended to consider perforations in the infill steel plates with diameters of 50 mm to 300 mm with the increment of 50 mm. The researchers suggested an equation to evaluate the strength of a perforated panel by applying a linear reduction factor $(1 - 0.7 D/S_{diag})$ to the strength and stiffness of a similar solid steel panel. The D and S_{diag} are the diameter and diagonal spacing of the perforations, respectively.

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):

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

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 Fig. 2.9;

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 (Fig. 2.9).

Figure 2.9: Neutral axis and length of one repeating corrugation of profiled steel sheet

This section summarizes some of the previous research on thin profiled steel sheet shear walls as follows:

El-Dakhakhi (1976) was the first 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.

Wright and Hossain (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 2.10. 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 2.10: Shear rig frame and test setup (Hossain 1995)

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's 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:

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

Berman and Bruneau (2005) conducted quasi-static testing of three large scale one storey, single bay shear wall specimens using light gauge cold-formed steel for the infill steel plate. The width and storey height of the specimens were 3660 mm and 1830 mm, respectively. The experimental test setup of the shear wall is shown in Fig. 2.11.

Two of the specimens had flat infill plates (thickness of 0.9 mm) and the third was a corrugated infill plate with a thickness of 0.7 mm. The infill plate connection to the boundary frames was achieved through the use of bolts in combination with industrial strength epoxy for the first and third specimen and welding for only the second specimen. The experimental testing was performed under quasi-static cyclic loadings.

Figure 2.11: Experimental test setup and detail of corrugated metal deck connections (Berman and Bruneau 2005)

A steel deck, with the corrugations orientated at 45° from the horizontal, was used for the corrugated infill steel sheet. Because of the available 910 mm width for the corrugated metal deck, four sheets were connected to each other using 1.6 mm diameter steel pop rivets spaced at 100 mm on centre. From coupon tests of the infill material, yield stresses of 152, 214, and 330 MPa were obtained for first, second and third specimens, respectively. The hysteresis curve for corrugated panel specimen, along with pushover curves obtained from strip models of the specimen is shown in Fig. 2.12. The corrugated infill panel exhibited un-symmetric hysteresis loops, since tension field action only developed in the direction of the corrugations.

Figure 2.12: Hysteresis and pushover curves for corrugated steel sheet shear wall (Berman and Bruneau 2005)

In spite of the hysteretic curve was pinched, It was stable and provided significant energy dissipation. It was recommended that two walls with opposed orientation of corrugations be used in a given structural line to achieve symmetric system behaviour.

Botros (2006) conducted a nonlinear finite element analysis on corrugated steel plate shear walls using ADINA software. The shear wall models were studied under monotonic and cycling loadings. The variable parameters were: corrugation angle, shear panel width, plate thickness. It was observed that shear walls with corrugated infill panels exhibited 6% higher shear strength than that with flat infill panels. Also the corrugation angle has minimal effect on the strength of the corrugated steel plate shear walls.

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 compare 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.

Stojadinovic and Tipping (2009) experimentally studied the behaviour of 44 full scale corrugated steel sheet shear wall specimens with cold-formed steel boundary elements under cyclic loadings. The experimental test variables were thickness of corrugated steel sheets, the gauge of the cold-formed steel framing and the spacing and size of the sheet to the frame fasteners. Three gauges (22, 20 and 18) of steel decking were tested. The average corrugated steel sheet strength from coupon test was $F_y = 345$ MPa (50 ksi) and $F_u = 415$ MPa (60 ksi) (it was higher for 18 gauge and lower for 22 gauge steel sheet) and the minimum steel sheet elongation was 20%. The authors recommended a Response Modification Factor (R) of 5.5 for these shear wall systems. Based on the experimental results also a design table for the nominal shear strength values for the corrugated steel sheet shear wall was proposed.

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 (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).

In this study, the focus is on 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 plate with reinforced concrete which can be precast or cast in place as shown in Fig. 2.13. 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).

Figure 2.13: Types of studied composite shear wall (Astaneh-Asl 2002)

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 stiffness, strength and energy dissipation capacity (Alinia and Dastfan 2007). In a composite shear wall, concrete panel also can resist shear by developing compression diagonal 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 (7 ft). The thickness of the reinforced concrete shear panel and the flat steel plate were 76 mm (3 inch) and 4.8 mm (3/16 inch), 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 “Traditional” composite shear wall. The components of the test setup as shown in Fig. 2.14 are actuator, loading beam at top, reaction beam at bottom, R/C reaction blocks, and the specimen.

Figure 2.14: Experimental test setup and detail of the test specimen (Zhao and Astaneh-Asl 2004)

Cyclic displacements were applied at the top of the specimen according to the specifications for qualifying cyclic tests of beam to column connections in seismic provisions for structural steel buildings (AISC 1997). The loading sequence and failure points for both specimens are shown in Fig. 2.15.

Figure 2.15: Time history of drift applied to two specimens (Zhao and Astaneh-Asl 2004)

It was observed that both innovative and traditional composite shear walls had high ductility and good lateral-load resisting properties. The maximum inter-storey drift was more than 5% for both specimens and they can withstand up to 4% inter-storey drift without and reduction in the shear strength. The reinforced concrete panel by itself in both specimens did not contribute much (less than 20%) in increasing the shear strength, but it was crucial to prevent the steel plate panel from buckling. The reinforced concrete panel had less damage in first specimen (innovative) than second specimen (traditional) and also the first specimen behaved in a more ductile manner compare to the second 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.

The authors 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 slab (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 first 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), 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 acts as a formwork for the infill concrete and also as a bracing system to the building frame and in the service stage, it acts also 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 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 (Fig. 2.10). The 0.45 mm thick profiled steel sheets with no embossment were manufactured in-house from plain sheets. The maximum thickness of concrete panel was 30 mm as shown in Fig. 2.16. 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.

Figure 2.16: Detail of profiled composite panel (Hossain and Wright 2004d)

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.

The authors 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* which the strength of the concrete core was controlled by diagonal tension failure,
- *Steel sheet limit state* which is mostly governed by 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 chemical bond between the sheets and concrete, and
- *Wall-frame connections limit state* into 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 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 simulation 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.

Hossain and Wright (2004a) performed experimental tests on composite wall specimens under cyclic loading. The test setup was explained before as shown on Fig. 2.10. The cyclic loads were applied diagonally. The load increment was 6 kN in each cycle up to a load of 60 kN. Beyond that load, an increment of 30 kN was used until failure of the panel. The crack patterns in concrete are shown in Fig. 2.17.

Figure 2.17: Crack patterns in concrete core of composite wall; (a) Monotonic shear loading, (b) Cyclic loading (Hossain and Wright 2004a)

The authors 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 which 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 actual concrete-steel interaction in a composite wall. The results of 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.

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 h and then cured in air.

2.5.1.1 Self-Consolidating Concrete (SCC)

Self-Consolidating Concrete (SCC) is a highly flowable concrete that can flow into place under its own weight. 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. Commercial VMA currently available in the market is costly and increases the price of such a concrete (Lachemi et al. 2003). 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. 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.

2.5.1.2 Engineered Cementitious Composites(ECC)

Engineered Cementitious Composites (ECC) is a class of ultra-ductile fiber reinforced cementitious composites which was originally invented at the University of Michigan in the early 1990s (Li 1993). ECC is characterized by high ductility in the range of 3–7%, tight crack width of around 60 μm and relatively low fiber content of 2% or less by volume. The typical fiber used in ECC is the polyvinyl alcohol (PVA) fiber with a diameter of 39 μm and a length of 6–12 mm.

One of the most important characteristic of ECC is the high tensile ductility represented by a uniaxial tensile stress-strain curve with strain capacity as high as 5% as shown in Fig. 2.18. Unlike other concrete materials, ECC demonstrates a strain capacity up to several hundred times greater than normal concrete.

Figure 2.18: Typical tensile stress-strain curve of ECC (Sahmaran et al. 2009a)

In order to have tight micro crack width and high tensile ductility while keeping the fiber content low (2% or less by volume), ECC has been optimized through the use of micromechanics (Li 1993, Sahmaran et al. 2009b). Micromechanics is a branch of mechanics applied at the material constituent level that captures the mechanical interactions among the fiber, mortar matrix, and fiber matrix interface. Usually, fibers are of the order of millimeters in length and tens of microns in diameter, and they may have a surface coating on the nanometer scale. Matrix heterogeneities in ECC, including cement grains, sand particles, and mineral admixture particles, with the size range from nanometer up to millimeter scale (Nawy 2008). A typical ECC mix design using Poly-Vinyl-Alcohol (PVA) fiber reinforcements is shown in Table 2.1.

Table 2.1: Typical mix design of ECC (Sahmaran and Li 2009)

Material	Proportion
Cement	1.00
Water	0.54
Fly ash	1.20
Sand	0.54
Fiber (Vol.) %	2.00
Superplasticizer	0.013

2.6 Research on Impact Resistance of Structural Elements

Understanding the behaviour of a structure under impact loading is important due to the possibility of impact during the service life of the structure. For building structures, impact loading can be accidental explosions, hitting objects by hurricane, striking airplanes, impact vehicles, etc. The major difference between ‘impact’ and ‘static loading’ is the structure under impact must be designed to absorb energy, but the structure under static loading should be designed strictly to carry the loads.

2.6.1 Impact Studies on Slabs

Uchida et al. (1985) conducted impact test using free falling mass on 30 RC slabs with the dimension of 1800 mm x 1800 mm. The variable parameters were slab thickness, amount of reinforcement, impact load and mixture ratio of steel fibre. The energy absorption capacity

between slab under impact and static loading was compared. It was found that under static loading, cracks occur only on the rear face of the slab and the bending deformation is dominant. On the other hand, under impact loading circular cracks occur on the front face and diagonal cracks occur on the rear face due to the vibration of slab and the shock wave propagation. The dominant failure mode was punching shear for most of specimens. It was observed that having mixture of steel fibre was very effective to reduce the formation of the cracks under impact loading. The energy absorbed in slabs increases in proportion to the impact loading rate up to almost two times as compared to static loading. Using the steel fibre, increase this tendency up to four times.

Miyamoto and King (1996) proposed a design procedure for RC slabs under impact loads based on absorbed energy criterion. Impact design method can be simplified to static design method by magnifying the static load but would not describe sufficiently the behaviour of the structures under impact. During impact, higher modes of vibration of a structure can be excited. Design RC slabs using the equivalent dynamic force can be adequate for bending but not for punching shear and concrete scabbing. The most efficient method for designing RC structures is the energy criterion (Miyamoto and King 1996). For the design of RC structures under impact loads, the safety factor can be determined based on the energy criterion.

The amount of transferred energy (E) from a projectile of mass ' M ' to a RC structure can be obtained from Eq. 2.9:

$$E = E_{initial} - E_{final} - E_p - E_{s,f} = 0.5 M V_{initial}^2 - 0.5 M V_{final}^2 - E_p - E_{s,f} \quad (2.9)$$

where $E_{initial}$ is the kinetic energy of projectile before impact, E_{final} is the kinetic energy of the projectile at the time of separation from the structure, E_p is the energy dispersed in the projectile by plastic deformation, $E_{s,f}$ is the energy lost by sound, friction, etc., $V_{initial}$ is the velocity of the projectile before impact collision and V_{final} is the velocity of the projectile after separation from the structure.

If the projectile is relatively rigid, the energy of plastic deformation of projectile (E_p) can be ignored. Also energy lost by sound and friction are usually small and is typically neglected.

If the RC structures can absorb all transferred energy during the impact, the structural failure would not happen. Miyamoto and King (1996) suggested an equation for the safety factor of the RC structures under impact as follows:

$$\gamma_i = \frac{E_{fu} / I_u}{E_{fe} / I_e} = \frac{E_{fu} \times I_e}{E_{fe} \times I_u} \quad (2.10)$$

where, E_{fe} is energy transmitted (absorbed) into structure at maximum deflection, I_e is the amount of impulse transmitted during the impact collision at maximum deflection, E_{fu} is ultimate absorption energy, and I_u is the amount of impulse at ultimate failure. The definition of calculation of these parameters is shown in Fig 2.19.

Figure 2.19: Calculation of ultimate absorption energy and energy absorbed by the structure (Miyamoto and King 1996)

Zineddin et al. (2007) studied the effects of different types of slab reinforcements and different impact loads on the behaviour and dynamic response of reinforced concrete slabs to improve the state of the art protective design. In this study, the impact loading was performed by a dropping

weight and particular attention was paid to the drop height on the slabs. The impact test setup is shown in Fig. 2.20.

Figure 2.20: The impact test setup for reinforced concrete slab (Zineddin et al. 2007)

The test setup was fully instrumented as follow:

- Strain gauges were mounted on the steel reinforcement of the concrete slabs,
- A load cell was attached to the impacting plate of the hammer,
- Accelerometers were mounted on the back surface of the slabs at different locations to the global response of the slabs,
- Two deflection gages were placed on the back surface of the slab at different locations.

It was found that the response of a slab is affected by the amount of steel reinforcement and also the drop height. Slabs with heavier reinforcement experienced higher accelerations compare to slabs with lighter reinforcement and the reinforcement quantities affected the slab failure modes. The failure mode of the reinforced concrete slab at low height drops was ductile (flexural failure) and at higher drops was brittle (punching and direct shear failure).

2.6.2 Impact Studies on Beams

Banthia et al. (1987 and 1989) performed impact tests on simply supported concrete beams with 100 mm by 125 mm cross section and 1.4 m length. The beams at their midpoints were struck by a 345 kg mass dropped from heights of up to 2.4 m. Three accelerometers mounted along the beam at different locations as shown in Fig 2.21. They tested three different types of concrete: fibre reinforced, normal and high strength concrete. The maximum aggregate size was 10 mm and average strength of normal and high strength concrete were 42 and 82 MPa, respectively. They used two types of fibre. The steel fibre reinforced concrete contained 1.5% steel fibre by volume and the fibres were 0.6 mm diameter, 50 mm length and both ends hooked. The polypropylene fibre reinforced concrete contained 0.5% polypropylene fibre (37 mm long) by volume.

Figure 2.21: (a) Location of the accelerometers and (b) Distribution of acceleration (Banthia et al.1987)

The steel and polypropylene fibre reinforced concrete improved in fracture energy on average about 300% and 30%, respectively, over the plain concrete. They found that concrete, either high

or normal strength, is strain rate sensitive and based on static testing, prediction of concrete behaviour under impact, is not possible. Also fibre reinforced concrete has more impact resistance and ductility compared to plain concrete. In general, concrete is more energy absorbing and stronger under impact loading than under static loading.

May et al. (2005) investigated finite element modeling and experimental low velocity impact on reinforced concrete beams and slabs using drop weight with 200 kg mass and falling height of up to 4 m. The specimens were RC beams with a maximum 3.0 m span and 2.3 m square RC slabs. They used a high speed video to record the impact tests at a rate of 4,500 frames per second. They used ELFEN finite element software to simulate the test as shown in Fig. 2.22. The reinforcement rebar is modeled as elasto-plastic material. The results from the test were used to validate the finite element model.

Figure 2.22: Comparison between numerical model and experimental test (May et al. 2005)

2.6.3 Impact Studies on Shear Walls

Rezai et al. (1997) conducted vibration/impact tests on a 30% scale one bay, four storey steel frame with flat steel plate infill shear panels to detect and characterize structural damages. The structure was tested under quasi-static cyclic loading until failure. The measurement of vibration was performed on the frame after and before the test. The capture of the first longitudinal mode using accelerometers was the main goal. Two types of vibration testing, namely ambient and forced vibration were conducted on the structure. The ambient vibration test (which is vibration responses due to unknown forces like machines vibration) was performed first and followed by the impact test using sledge hammer to validate the results from the ambient vibration. Impact test is one type of forced vibration test which has been used to determine the dynamic response of small structures. An integral piezoelectric force sensor was attached to the sledge hammer to measure the applied force. A shift in longitudinal fundamental frequencies, after and before the test was detected. The natural frequencies of the structure are proportional to the square root of generalized stiffness and any decreases in natural frequencies represent the structural stiffness degradation. It was observed a 3% reduction in the first longitudinal natural frequency of the structure which represents about 6% structural degradation.

2.7 Summary and Conclusions

In this chapter the development of four types of structural shear wall (steel plate shear wall, profiled steel sheet shear wall, composite shear wall, and double skin profiled composite shear wall) to resist lateral loads were 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. By considering the fact that the shear yield capacity of a steel plate is

significantly greater than its capacity to resist shear in yielding due to diagonal tension field, the advantage of utilizing the composite shear wall system will be realized.

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. SCC can flow into place between two profiled steel sheets under its own weight without internal or external vibration and also without defects due to bleeding and segregation. Engineered Cementitious Composites (ECC) is very ductile and also flowable concrete. Unlike other concrete materials, ECC exhibits a strain capacity up to several hundred times larger than normal concrete. It is expected that utilizing ECC in the double skin profiled composite shear wall will improve structural performance of the composite wall.

Based on literature review, the concept of using double skin profiled composite shear wall is found to be relatively new and no research has been conducted on the behaviour of this structural element: (i) with the presence of intermediate fasteners as steel-concrete interface connectors along height and width of the wall, (ii) with the use of SCC and ECC and (iii) under both cyclic and impact loading conditions. Therefore, the study of double skin profiled composite shear wall (DSCW) under monotonic, cyclic and impact loading with interface connections and high performance concrete (SCC and ECC) is warranted to have better understanding of the structural behaviour of this type of shear wall and to develop design guidelines. This research is aimed at contributing significantly to these needs in the area of DSCW technology. In addition, two types of infill concrete namely; SCC and ECC will be used to investigate the possible advantages of using ECC over SCC as concrete infill in improving strength, stiffness, ductility and energy absorbing capacity of DSCW system.

CHAPTER 3

EXPERIMENTAL PROGRAM

3.1 Introduction

In order to test a shear panel, a shear rig (perimeter frame) is needed which can transfer uniform shear forces to the specimen. If the axial stiffness of the columns of the shear rig is relatively stiffer than the panel, the panel can be assumed to be in pure shear. The beam-column connection of the shear rig and also the connection of the shear rig to the shear panel can be welded or bolted as required by the nature of investigation. In this study, two types of steel sheets and two types of concrete were utilized to make eight specimens.

This chapter describes how the experimental test setup was developed in order to perform tests under monotonic/cyclic and impact loading under in-plane shear. The composite wall fabrication is explained in details. Three major modifications were performed to improve the performance of the experimental test setup and reduce the excessive deformation of the specimen. At the end of the chapter, material properties for each test specimen are presented.

3.2 Components of Composite Wall and Shear Test Setup

The main components of the experimental test setup are the composite shear wall, a base frame attached to the floor, an end plate structure and a MTS hydraulic actuator model 243.35T with attached load cell as shown in Fig. 3.1. The dimensions of the wall panels were chosen based on the capacity of the existing loading facilities and the feasibility of specimen fabrications considering availability of materials. The composite wall consists of a composite wall panel fastened to a pair of shear rig frame by bolts (Fig. 3.1).

Figure 3.1: Main components of the experimental test setup

3.2.1 Composite Wall Panel Dimensions and Description

Composite wall panel consists of two profiled steel sheets and an infill concrete where composite action between steel sheeting and concrete was provide by intermediate fasteners. It was decided to use commercial P-3012 Steel Deck manufactured by Canam Group to fabricate the composite 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 Fig. 3.2. 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.

Figure 3.2: Dimensions of P-3012 steel deck (Canam Group 2005)

Two types of concrete were used as concrete core in the composite wall panels namely; Self-Consolidating Concrete (SCC) and Engineered Cementitious Composites (ECC). Based on load capacity of the existing hydraulic actuator in the structures lab, the thickness and other dimensions of the composite wall panels were determined. The thickness of the composite panel is 30 mm at the troughs and 58 mm at the crests (Fig. 3.3). It was decided to have an adjustable shear test setup for monotonic and cyclic loading to accommodate for panel thickness of 30 ± 8 mm at the troughs. The 720 mm width of the composite panel was chosen to have the option of fastening the steel frame to each or alternate flutes of the wall panel and also to have a symmetrical wall with respect to fasteners (the total number of crests should be odd).

Figure 3.3: Composite wall panel cross section

Eleven rows of intermediate fasteners having 4 at each row at 128 mm c/c providing a total of 44 fasteners were used to attach the profiled steel sheet crests to each other as shown in Fig. 3.4. Five mm diameter bolt was used as intermediate fasteners prevent the global buckling of profiled steel sheets and also maintain the accurate distance between the sheets during fabrication, installation and casting of concrete.

Figure 3.4: Four 5 mm diameter bolt at each row as intermediate fasteners

3.2.2 Steel Frame for the Shear Test Setup

Two sets frame (shear rig) with each frame consisting of two flat steel plate beams and columns, were fabricated. The composite wall panel was mounted between the two frames by bolts. The thickness of the steel plate (made of Grade 350W) for beams and columns was 30 mm. The thickness was reduced to 15 mm at the end to accommodate hinge connections at the corners of the beam-column frame as shown in Fig. 3.5.

Figure 3.5: View of the frames and the composite wall panel assembly from the top

The pinned beam to column connections of the boundary frame allow uniform shear force transfer from the frame to the composite wall panel boundaries.

The composite panel was connected in between the two sets of steel frames (one set of the frame at each side of the composite panel) by 9, M12 bolts along each beam and 23, M12 bolts along each column of the frame. Four M30 bolts were used to simulate hinge connection at the four corners of the frame as shown in Fig 3.6.

The width of the beam and column of the boundary frame was limited to 80 mm. The 1616 mm length of the columns was chosen to keep the top of the wall in alignment with the level of actuator and also have either options of fastening the steel frame to each holes (64 mm c/c distance from each other) or alternate holes. The total height of the wall was 1626 mm to have 10 mm distance for columns rotation at the bottom of the wall.

Figure 3.6: Schematic diagram showing detail of the perimeter steel frame of shear rig

Under the lateral in-plane shear load, the frame should act as a mechanism allowing free rotation of beams and columns at the corner hinges. A 10 mm gap between steel beams and columns was provided to have free rotation between them at the hinges. Fig. 3.7 shows shear deformed mechanism of the frame. The boundary frame is designed to provide a maximum lateral displacement of 175 mm which is more than 10% of the column height (Fig. 3.8).

(a)

(b)

Figure 3.7: Free rotation of the steel boundary frame due to lateral movement; (a) Original position of the frame, (b) The final position of the frame after rotation

Figure 3.8: Maximum lateral columns displacement (175 mm) after frame rotation

3.3 Fabrication of Composite Wall

The following steps were taken to fabricate each composite wall specimen:

a) Making perimeter holes on two profiled steel sheets:

Each profiled steel sheet should have 64, 12 mm diameter holes along the frame boundary and 4, 30 mm diameter corner holes. The following steps were taken to make those holes in two profile sheets:

- A flat wooden board bigger than the dimension of the steel sheet was positioned underneath the assembly on the structures lab.

- Two profiled steel sheets were positioned face to face as shown in Fig. 3.9 and were aligned and fixed all four corners by using four vice-grip pliers (locking pliers).

Figure 3.9: Using vice grip pliers at each corner to keep holding the sheets

- One set of frame either front or back (consisting of two beams and columns) was positioned at the top of the sheets and was fixed to the steel sheets using small clamps (Fig. 3.10).

Figure 3.10: Position of frame at top of the sheets

- A 12 mm diameter drill bit was used to make 12 mm holes while steel hole saw was used to make 30 mm holes (Fig. 3.11).

Figure 3.11: Using steel hole saw to make 30 mm holes

- b) **Attaching the two sets of frames to the profiled steel sheets.**
- c) **Making intermediate holes and fastening 5 mm bolts to the both sides of the profiled steel sheets.**
- d) **Installing and tightening the formwork to the both vertical sides of the wall:** In order to seal the gap between profiled steel sheets and the formwork, flexible foam and flatted bike tube were used. Also two wooden fasteners were used to hold the formwork to the composite wall as shown in Fig. 3.12.

(a)

(b)

Figure 3.12: Sealing and holding the formwork; (a) Using flatted bike tube and foam between profiled steel sheets and formwork, (b) Using two handmade fasteners for holding the formwork

- e) **Casting of concrete infill:** Composite wall specimens were cast with two types of concrete (SCC and ECC) and detail casting process is explained in section 3.6.1.
- f) **Testing of composite wall:** Walls were tested at least 28 days after casting. Control concrete cylinders cast with the wall specimens were also tested for compressive and tensile strength at the same time.

3.4 Development of Experimental Test Setup for Monotonic and Cyclic Loading

In order to perform the monotonic and cyclic loading tests, the composite wall-frame assembly was mounted to the strong floor of structures lab vertically and connected to a horizontal actuator through an end plate structure as shown in Fig. 3.13.

Figure 3.13: The experimental test setup (Dimensions are in mm)

An I-shaped base frame consists of three HSS 203 x 203 x 9.5 was utilized to mount the composite wall-frame assembly to the floor as shown in Fig. 3.14. The three hollow sections were welded to each other to make the I-shaped base frame. After making the I-shaped base frame, 4 holes with 46 mm diameter and 12 holes with 22 mm diameter were made on the frame in order to mount the frame to the floor and also to the composite wall. A Unibor magnetic drill was utilized for making holes on the I-shaped base frame. This type of drills has a very strong electrical magnet at the base to attach tightly to the steel parts.

Figure 3.14: The I-shaped base frame consist of three welded HSS 203 x 203 x 9.5

Four 42 mm diameter threaded rods were used to fasten the frame to the floor (Fig. 3.15). Each nut at the bottom of the floor was tightened to 500 N.m torque by using special click-type torque wrench with an attached socket, adjusted by turning the knurled handle.

Figure 3.15: Fastening the base frame to the floor

Four angles (L 89 x 89 x 13) as shown in Fig. 3.16 were utilized to mount the composite wall to the base frame. Three 22 mm holes on each angle was slotted for future increase or decrease in the thickness of the composite wall (the panel thickness can be 30 ± 8 mm at the troughs).

The MTS hydraulic actuator model 243.35T, which is available at Ryerson structures lab, was used for monotonic and cyclic shear loading. This actuator can impose a maximum compression force of 365 kN and a maximum tension force of 240 kN to the specimen and has an attached load cell. This actuator can perform well for static and low frequency (3 to 4 Hz) cyclic loading. The stroke length of the actuator is 250 mm. The hydraulic actuator with attached load cell was mounted from one side to the strong wall and from another side to the top corner of the composite wall through a pin connection end plate. The function of the end plate is to connect the actuator to the composite wall (Fig. 3.18).

Figure 3.18: The connection of the actuator to the composite wall through the end plate structure

During the pilot test (first test), it was observed that some modification to the test setup is needed to reduce the deformation/rotation of the composite wall at the base and also prevent the movement of the actuator at the top of the wall. The test setup modifications were performed mainly in three phases namely: Phase I, Phase II and Phase III. In Phase I, the end plate structure and four angles were modified to prevent excessive deformation at the connection of the actuator to the composite wall and the angles at the base, respectively. In Phase II, a steel plate was welded to each modified angle for further strengthening and all four M30 threaded rods at the corners of the composite wall were replaced with high strength structural bolts. In Phase III, a steel plate was welded to each steel column at the base. The detail of each test setup modification is presented in Appendix A.

3.4.1 Instrumentation and data acquisition

A data acquisition system (VISHY System 5000, Model 5100B Scanner) which was suitable to read the input data from the electronic devices during the monotonic and cyclic tests, had 40 input channels for strain gauges and 10 input channels for LVDT's. The maximum sampling rate of the data acquisition system was 50 samples per second.

All the parameters which seemed to be important either in controlling the test or in interpreting the results were measured. The following parameters were monitored and recorded during each test:

- a) Horizontal applied displacements/loads from the actuator. The load cell attached to the actuator gave this information.
- b) Horizontal displacement of the composite wall at top at two locations for all tests (Fig. 3.19) using LVDTs. For monotonic loading vertical displacement (uplift) of the wall at the base at the actuator side and also horizontal displacement of the composite wall at the base (Figs. 3.20 & 3.21). For cyclic loading vertical displacement (uplift) of the wall at the base at each side of the wall (Fig. 3.22).
- c) The strains at two locations on the column and maximum 4 locations within the profiled steel sheet as shown in Fig. 3.23. Two single strain gauges were mounted on the column at one-third and two-third height of the column (No. 5 & No. 6) to capture the distribution of axial strain in the column, and maximum 4 rosette strain gauges were mounted on the face of the profiled steel sheet at center, right corner at bottom, left corner at bottom (all three at the crest) and center at the trough (No. 1 to No. 4) to capture the distribution of principal and shear strains along two diagonals of the profiled steel sheet.

Figure 3.19: The location of two LVDT's at top of the composite shear wall

Figure 3.20: The location of two LVDT's at bottom of the composite shear wall in the monotonic tests to measure possible uplift and rocking

Figure 3.21: The overall location of total four LVDT's in each monotonic tests

Figure 3.22: The location of two LVDT's at bottom of the composite shear wall in the cyclic tests to measure possible uplift

Figure 3.23: Location of four rosettes and two single strain gauges

Table 3.1 shows the test matrix for the strain gauges. For the first test, one rosette and 16 single strain gauges were mounted on the profiled steel sheet.

Table 3.1: The test matrix for the strain gauges

Tests No.	Strain Gauges						Comments
	1(Rosette)	2 (Rosette)	3 (Rosette)	4 (Rosette)	5 (Single)	6 (Single)	
CSW-1	√	-	-	-	-	-	Also 16 single strain gauges were attached to the profiled steel sheets
CSW-2	√	√	√	-	-	-	-
CSW-3	√	√	√	-	√	√	-
CSW-4	√	√	√	√	-	-	-
CSW-5	√	√	√	√	-	-	Rosette strain gauge No. 2 was damaged
CSW-6	√	-	√	√	-	-	-

All the strain gages and LVDTs were connected via bridge amplifiers to the data acquisition system. The actuator displacement along with load cell data was recorded in another data recorder. The data which was collected from the actuator and the data acquisition system had the same sampling rate but was saved at two different computers. The sampling rate in the monotonic loading was one scan per second and in the cyclic loading was 10 scan per second. The triggering of the collected data was performed manually at the beginning of each test.

3.4.2 Loading Procedures

The cyclic tests were conducted based on the recommendations outlined in AISC Seismic provisions (AISC 2002) and Applied Technology Council (ATC 24, 1992). The loading sequence was adjusted based on the capacity of the hydraulic actuator. The documents provide guidance on loading history for slow cyclic loading tests. The load has been applied in groups of cycles and each group consists of three cycles. The detail of the loading for each cyclic test is described in chapter 4. The change of direction of actuator stroke at each push and pull was performed manually.

The monotonic tests were performed based on displacement control by pushing top of the composite wall at a constant rate of 3 mm per minute until the failure of the wall.

3.5 Development of Experimental Test Setup for Impact Loading

The test set-up for the impact testing is shown in Fig. 3.24. The main components of the test set-up are: pneumatic actuator attached to a charging tank, base frame, railing frame that guides the projectile, 1-inch thick (25.4 mm) base plate, four angles welded to each other, composite wall with attached frames, the end plate structure (consisting of three plates), guided frame and a projectile.

Figure 3.24: The main components of the impact test setup

The detail of the pneumatic actuator, railing frame that guides the projectile charging tank and controller of the actuator are presented in Figs. 3.25 to 3.27.

Figure 3.25: The pneumatic actuator

Figure 3.26: The railing frame mounted on top of the base frame

Figure 3.27: Charging tank and controller of the actuator

20 nuts were welded to a 1-inch thick (25.4 mm) base plate as shown in Fig. 3.28. The base plate had a 45 degree cut edge in longitudinal direction for groove welding (Fig. 3.29). The base plate was welded to bottom flange of two I beams of the base frame at each side (Fig. 3.30).

Figure 3.28: Two sides of 1-inch thick (25.4 mm) base plate with the 20 holes and welded nuts

Figure 3.29: Forty five-degree cut edge of the base plate in longitudinal direction for groove welding

Figure 3.30: The location of the base plate between the lower flanges of two I beams

The composite panel with attached frames was mounted to four welded angles (Fig. 3.31) by two M30 and nine M12 high strength bolts. The composite wall mounted to the welded angle assembly was seated on a 25.4 mm base plate and was fastened by 20 high strength bolts (25 mm diameter) as shown in Fig. 3.32.

Figure 3.31: Welded angle assembly to fasten the composite wall to the base plate

Figure 3.32: Fastening the welded angle assembly to the base plate using 20 high strength bolts

The end plate structure was mounted on top of the composite wall by two M30 high strength bolts. The height of the composite shear wall (1626 mm) was matched to the level of the projectile so that the projectile could hit at the center of the end plate (Fig. 3.33).

Figure 3.33: The projectile hits to the center of the end plate structure

A guided frame was mounted on the base frame to prevent lateral displacement of the specimen during impact testing. The guided frame was mounted to the base frame using five clamps at each side. It was used only for the first impact test to prevent the possible damage to the boundary/perimeter steel frame of the composite shear wall. During the first impact test, it was observed that the perimeter steel frame did not contact the guided frame during impact testing and it was decided not to use the guided frame for the rest of the impact tests.

3.5.1 Instrumentation and Data Acquisition

One of the most important aspects of the impact test is the data collection. In the impact test on the composite wall, two types of data were collected: the data from accelerometers and high speed video camera.

a) Accelerometers

Four accelerometers (Model 3200B6 from Dytran Instrument Inc.) were used to capture acceleration of the specimen at four locations. Model 3200B6 accelerometer can capture acceleration up to 2,500 G's with nominal sensitivity of 2.00 mV/g and frequency up to 15 kHz the calibration certificate for each accelerometer is presented in Appendix B. Four Angles were drilled and tapped to hold accelerometers. Each accelerometer was screwed to an angle from 1/4-28 mounting stud and from other end to a 10-32 micro-coaxial cable.

For the first test, two angles on the top and two angles on the bottom at each side of the composite shear wall frame were glued as shown in Figs. 3.34 and 3.35. Then accelerometers were screwed to the angles. For the second impact test, two angles on top of the frame were welded and the other two angles on the top were embedded in the concrete as shown in Fig. 3.36.

Figure 3.34: Location of two angles at the top of the steel frame

Figure 3.35: Location of two angles at the bottom of the steel frame (only one angle is shown)

Figure 3.36: Location of four angles at top the composite shear wall

The data acquisition system to capture acceleration was imc Type CRONOS – PL 2 as shown in Fig. 3.37. The accelerometers were connected to the data acquisition system by using ACC/DSUB-ICP2 connectors (one connector for two accelerometers) and 9 meter micro-coaxial cables. From the preliminary impact tests and also from the finite element model, it was found that the sampling rate of the system should be $20\ \mu\text{s}$ (50,000 samples per second) which was the maximum collecting the data from the imc. All the acceleration data was collected as a function of time. The start of collecting data was triggered manually and the time of collecting data for each impact test was about few seconds. The collected data was saved on a Personal Computer.

Figure 3.37: Data acquisition system imc Type CRONOS – PL 2

The data from accelerometers were processed using FAMOS software version 6.0 from imc (<http://www.imcdataworks.com/famos/>). The FAMOS software was capable of transferring the data from time domain to frequency domain by using Fast Fourier Transformation (FFT) algorithm. Due to collection of data with high sampling rate and having large amount of data (about 250,000 samples), the FFT was performed only on the useful portion of data from the impact (which was fraction of a second).

b) High speed camera

The exact speed of impact was determined by utilizing a high-speed video camera of maximum 16,000 frames per second (fps) recording rate with single shutter speed. The high-speed digital camera is Fastec TroubleShooter model TSHRMS from Fastec Imaging Corporation (<http://www.fastecimaging.com>) with up to 1280 x 1024 pixels resolution as shown in Fig. 3.38. The images were dynamically stored in the camera memory on a FIFO (First-In, First-Out) basis until the camera was triggered manually. Upon triggering, the camera recorded the desired number of post-triggered frames and held the data in its memory.

Figure 3.38: High-speed digital video recording camera and two lights

The camera was manually triggered simultaneously with the data acquisition system to synchronize the video and numerical data. The video data were stored on a PC. In order to have a bright and clear recording video, two powerful lights (1000 watt each) were used. The speed of camera which was used in the impact tests were 500 frame per second (0.002 sec per frame), with total capturing duration of 4 second (2000 total frame) and resolution of 1280 x 512. This time was sufficient to determine the speed of the projectile before impact and also to capture the bounce back speed. These two speeds gave the clear measure of the exact amount of transferred kinetic energy to the specimen.

The speed measuring was achieved by using the software MiDAS 4.0 Express for Fastec TroubleShooter from Xcitex Inc (<http://www.xcitex.com>). The program enables the user to measure the distance of two points by the number of the monitor pixels. The equivalency of each monitor pixel with the actual distance close to the specimen was achieved by measuring pattern attached to the impact apparatus.

3.5.2 Impact Projectile

During the first test (pilot test) a 36 kg projectile was used to perform the impact test but it was damaged. A new projectile with the mass of 64.8 kg (Fig. 3.39) was replaced in order to apply high amount of energy to the specimen.

Figure 3.39: Final projectile (64.8 kg)

3.5.3 Test Procedures

The pneumatic actuator of the impacting apparatus accelerated the projectile. The projectile gained the designed speed and decoupled from the accelerating mechanism. At this point, the projectile traveled freely on a railing system and hit the end plate attached to the composite wall. The maximum speed of projectile (depends on the projectile mass) before impact was up to 60 kilometres/hour (16.7 meters/sec). The impact energy was transferred from the end plate to the frame of the composite wall through two high strength bolts with diameter of 30 mm. The frame transferred the energy to the composite panel through the M12 bolts.

3.6 Materials

3.6.1 Concrete

Two types of concrete, namely; Self-Consolidating Concrete (SCC) and Engineered Cementitious Composites (ECC) were used to fill the gap between two profiled steel sheets and having composite steel and concrete. The mix design and ingredients of SCC are shown in Table 3.2.

Table 3.2: Mixture proportions of SCC

SCC ingredients, kg/m ³								SP mL/100 Kg of binder	W/B
Cement	Slag	Binder	Water	Coarse	Fine	AEA	SP		
400	90	490	172	750	910	0.1068	2.77	538.4	0.351

The cement was type GU (type 10) from St Marys Cement and the slag was type “S” slag cement from GranCem. The coarse aggregate was crushed limestone from Munroe quarry with maximum size of 10 mm and the fine aggregate was from Edgar sand. The Air-Entraining Admixture (AEA) and Superplasticizer (SP) were “Micro air” and SP-1466, respectively and both from Basf.

The volume of SCC which needed for casing one composite wall, one time slump test and 6 cylinder samples was 70 litres. The maximum capacity of a concrete mixer in the concrete lab was 60 litres, so two batches of 35 litres concrete were made.

For making SCC, the following mix sequences were carried out:

- The slag was added to the cement bucket and mixed with the cement (Fig. 3.40a).
- Sand and Coarse aggregate were added to the concrete mixer and mixed for 15 second (Fig. 3.40b).
- The Cement and Slag was poured in the concrete mixer and mixed for 20 second.
- 60% of total water was added to the concrete mixer and mixed for 30 second.
- AEA and SP were added to the rest of the water. The water were poured to the mixer and mixed for 60 second.
- Rest (waiting) for 30 second.
- Resume mixing for 60 second.

Figure 3.40: Mixing the SCC ingredients; (a) Slag and cement, (b) Sand and coarse aggregate

The weight of the air-entraining admixture and superplasticizer were measured by using digital scale with the accuracy of 0.01 grams. The slump of the SCC was measured to insure the flow ability of the concrete as shown in Fig. 3.41.

Figure 3.41: Measuring the SCC slump

A special custom made funnel was made for casting concrete due to a narrow distance between two parallel profiled sheets. The V shape funnel was made with 20 mm throat as shown in Fig. 3.42. The mix design and ingredients of ECC are shown in Table 3.3.

Figure 3.42: Custom made wooden funnel for casting concrete

Table 3.3: Mixture properties of ECC

ECC ingredients, kg/m ³						FA/C	W/CM
Cement	FA	Water	PVA	Sand	SP		
375	823	318	26	435	2.00	2.20	0.27

The cement was type GU (type 10) from St Marys Cement and the Fly Ash was class F which was supplied by Boral Materials Technologies. The Polyvinyl alcohol fibre (8mm length) was supplied by Okayama Plant, Kuraray Company, LTD. The sand was silica sand with $110\mu\text{m}$ average grain size and Superplasticizer (SP) was ADVA® CAST 575, a polycarboxylate based high range water reducer from Grace Canada Inc.

The volume of the ECC which needed for composite wall and control specimens was 65 litres and two batches of 32.5 litres ECC were made. Solid ingredients, including cement, fly ash (FA), and sand, were first mixed for a minute. Water and chemical admixtures (SP) were then added into the dry mixture and mixed for two minutes and then polyvinyl alcohol fibre was added at the end (Fig. 3.43) and mixed for an additional three minutes. The amount of SP was adjusted during the mix to have a uniform mixture and flow able ECC. The ECC was poured to a small bucket for casting as shown in Fig. 3.43.

Figure 3.43: Making ECC

After casting concrete, the wall specimen and concrete control samples were covered with plastic sheets. The control samples were demolded after 24 hours. The wall specimen and control samples were left to air cure under uncontrolled conditions of humidity and temperature until the testing date.

The total of six ECC control samples were taken from each casting the composite shear wall, 3 cube samples (50 mm x 50 mm x 50 mm) for compression test at the time of specimen testing and 3 cylinder samples (100 x 200mm) for tensile test (Splitting test) at the time of testing. Also 2 cylinder samples were cast and tested for measuring the concrete modulus of elasticity at the time of testing. Three single strain gauges were attached to the longitudinal direction of the cylinder. The length of the strain gauges was bigger than five times of the maximum aggregate size of concrete. The secant modulus of elasticity of ECC was determined from the compression test of two cylinder ECC samples. The average slope of the three strain-stress curves at the linear stage ($0.4 f'_c$) for the first and second sample was 25,100 MPa (Fig. 3.44) and 23,900 MPa (Fig. 3.45), respectively.

Figure 3.44: The result of the first test for the modulus of elasticity (Average $E = 25,100$ MPa)

Figure 3.45: The result of the second test for the modulus of elasticity (Average $E=23,900$ MPa)

The ECC modulus of elasticity from the two tests and the average ($E = 24,500$ MPa) is presented in Table 3.4.

Table 3.4: Modulus of elasticity (E) for the ECC

Specimen No.	The average E (MPa) based on the results of three strain gauge
1	25,100 MPa
2	23,900 MPa
Average	24,500 MPa

The splitting (Brazilian) tensile test was conducted to measure the tensile strength of concrete (both SCC and ECC). The compressive and tensile strength of the two concrete types for eight specimens are shown in Table 3.5.

Table 3.5: Type and strength of concrete for each specimen

Specimen No.	Concrete Type	Concrete strength (Mean Value) MPa	
		Compressive strength	Tensile strength
CSW-1	SCC	51.9	4.92
CSW-2	SCC	48.1	4.68
CSW-3	ECC	40.2	4.75
CSW-4	ECC	39.4	4.71
CSW-5	SCC	35.7	3.28
CSW-6	SCC	34.6	3.15
CSW-7	SCC	38.7	3.44
CSW-8	SCC	34.3	2.98

3.6.2 Steel

The profiled steel sheet used in the composite shear wall specimens was Cold-Formed Steel (CSA-S136 2007). The profiled steel sheets in the first four specimens was made of high strength steel which caused brittle failure and low ductility during composite shear wall testing. For the rest of the wall specimens, mild strength steel was utilized to investigate the wall behaviour.

Tensile coupon test were performed to verify the steel mechanical properties. The tension coupon results gave detailed information on the stress versus strain and modulus of elasticity for

the profiled steel sheets. This information is important in the finite element modeling to enable the software to simulate the material behaviour which was presented in the experimental test.

A total of 6 tension coupons for the two different profiled steel sheets (cold-formed high and mild strength steel) were tested in uniaxial tension to determine the stress versus strain behaviour. Cutting, machining and testing of the tension coupons, followed the requirements of ASTM standards A370–08a (ASTM 2008a). Each coupon had an overall length of 203.2 mm (8 inch), 12.7 mm (0.5 inch) width at mid length and 19.05 mm (0.75 inch) width at each end.

A United testing machine with attached load cell was used to perform tensile coupon test. Before the test, the middle part of the coupon was marked by two lines with a 50.4 mm (2 inch) distance. Both ends of each coupon were gripped by apparatus in the United Testing Machine. An electro-mechanical extensometer was attached to the coupon and placed to the location of the two lines. The machine applied tension load to the coupon with a uniform rate until it ruptured. The strain was recorded using the electro-mechanical extensometer with a gauge length of 50.4 mm (2 inch) up to the 0.01 strain. At the point of steel strain equal to 0.004 for high strength steel and 0.01 for mild strength steel, the extensometer was removed and the test was continued till rupture by only capturing the load from the load cell. The ultimate elongation was determined by dividing the final distance of the two marked lines to the original length of 50.4 mm. The electro-mechanical extensometer attached to the coupon and rupture of the coupons is shown in Fig. 3.46.

Figure 3.46: The electro-mechanical extensometer and rupture of the coupons

The result of the coupon test for high and mild strength steel are shown in Figs. 3.47 and 3.48, respectively. For each type of steel, the result of three coupon tests is shown. The stress-strain model is the average of the three results which was simplified with just few points and was used

to simulate the steel behavior in the finite element modeling. The high strength steel has yield stress of 552 MPa, ultimate strength of 577 MPa, low ductility (elongation = 9.7%) and modulus of elasticity of 202,940 MPa.

Figure 3.47: The stress versus strain curves from the coupon tests (High strength steel)

Figure 3.48: The stress versus strain curves from three tension coupon tests (Mild strength steel)

The mild steel has yield stress of 354 MPa, ultimate strength of 420 MPa, high ductility (elongation = 24%) and modulus of elasticity of 206,980 MPa. The usual range of modulus of elasticity for steel sheets is from 200 GPa to 207 GPa (Yu 2000). Compared to high strength steel, the mild steel has 64% less yield stress, 73% less ultimate strength but 2.5 times more elongation that can improve ductility of the composite wall. A summary of the results from the coupon tests is shown in Table 3.6.

Table 3.6: Tensile coupon test results

Steel type	Stress & strain	Yield stress & strain	Stress & strain at disconnecting extensometer	Ultimate stress & strain	Stress & Strain at rupture	Modulus of elasticity (MPa) & elongation
High strength steel	Stress (MPa)	Fy = 552	570	577	569	202,940 & 9.7%
	Strain	0.00272	0.004	0.01	0.097	
Mild strength steel	Stress (MPa)	Fy = 354	375	420	390	206,980 & 24%
	Strain	0.00171	0.01	0.1	0.24	

3.7 Summary

In this chapter, the components of composite wall panel and shear rig were described and the geometry of the composite wall panel and steel shear rig was also detailed. Moreover, the composite wall panel fabrication process was presented. The development of experimental test setups both for monotonic/cyclic and impact loading was discussed. Several modifications were performed to minimize the rotation of the specimen at the base, rotation of the actuator at the top in the monotonic and cyclic loading. Also the columns of the shear rig were strengthened by welding 10 mm plates at the bottom of the columns. For the impact test, the test setup and the instrumentation were discussed. The mix design of concrete for SCC and ECC, casting, sampling and testing of concrete were presented. Moreover, the mechanical properties for the two different types of steel used to manufacture the profiled sheets were documented.

CHAPTER 4

EXPERIMENTAL INVESTIGATION OF COMPOSITE WALLS UNDER IN-PLANE MONOTONIC & CYCLIC LOADING

4.1 Introduction

The results of the experimental test program on composite walls subjected to in-plane monotonic/cyclic loading are presented. Six tests on monotonic/cyclic loading were conducted. The results for monotonic loading tests are presented and discussed by illustrating load versus displacement, principal strains/stresses versus displacement and shear strains/stresses versus displacement responses throughout the loading history and failure mode of the specimens. For cyclic loading, the stiffness degradation versus the number of load cycles and analysis of shear load-displacement hysteretic envelopes are presented in addition to those presented for monotonic testing.

In this chapter, the results of the monotonic loading tests followed by the cyclic loading tests are depicted and at the end of chapter, summary of results and main conclusions are presented.

4.2 Test Matrix

The test matrix for the six composite wall specimens tested under monotonic and cyclic loading is shown in Table 4.1. These six tests were divided into three sets where each set consisted of two tests: one monotonic and one cyclic. The type of concrete (either ECC or SCC) and material properties for each set were identical. The variable parameters in the tests are: type of loading (monotonic or cyclic), type of concrete (ECC or SCC) and steel strength while geometric dimensions remain the same. For the first four specimens the yield strength of steel was 552 MPa and the rest of specimens, the yield strength of steel were 354 MPa. The type of concrete for the third and fourth specimens was ECC while as SCC was used for the rest of specimens.

Table 4.1: Test matrix for monotonic and cyclic loading

Set No.	Specimen No.	Height x width mm x mm	Type of Loading	Type of concrete	f'_c MPa	f'_t MPa	Yield strength of profiled steel sheet MPa	Ultimate strength of profiled steel sheet MPa
1	CSW-1	1626 x 720	Monotonic	SCC	51.9	4.9	552	577
	CSW-2	1626 x 720	Cyclic	SCC	48.1	4.7	552	577
2	CSW-3	1626 x 720	Monotonic	ECC	40.2	4.8	552	577
	CSW-4	1626 x 720	Cyclic	ECC	39.4	4.7	552	577
3	CSW-5	1626 x 720	Monotonic	SCC	35.7	3.3	354	420
	CSW-6	1626 x 720	Cyclic	SCC	34.6	3.2	354	420
f'_c : Compressive strength of concrete; f'_t : Tensile strength of concrete; SCC : Self-Consolidating Concrete; and ECC : Engineered Cementitious Composites								

4.3 Behaviour of Composite Walls under Monotonic Loading

Monotonic tests on all three specimens were performed by applying in-plane load as a prescribed displacement with a rate of 3 mm per minute. The main reason to adopt this loading rate was to capture detailed information during the tests as well as apply quasi-static loading condition to the specimens. Sampling rate of data acquisition was one scan per second in order to eliminate collecting the unnecessary data and also reduce the size of data file. The shear load-displacement response and strain development on the steel sheet were monitored throughout the loading history.

The behaviour of the three walls were studied based on two types of concrete (SCC and ECC) and also profiled steel sheets of two yield strength under monotonic loading. The behaviour of the walls are described based on experimental results on load-displacement response, stress-strain characteristics, failure modes, concrete cracking, and buckling of steel sheets.

4.3.1 Specimen CSW-1

The load- displacement response as shown in Fig. 4.1 showed a very small stiffness up to 20 mm displacement and the wall was tested to the maximum load of 185 kN at 97 mm displacement. The reason for small stiffness at the beginning was attributed to the deformation of the four corner bolts (M30) of the loading frame and also to the vertical movement of the actuator. This

test was discontinued at this phase (phase 1) and conducted again with modification of test set-up (in Phase 2) to avoid these problems.

Figure 4.1: Load-displacement response of composite wall (CSW-1) under monotonic loading

The maximum and minimum principal strains at the center (crest) of the profiled steel sheet (calculated from the rosette strains during the loading history) and corresponding stresses are shown in Figs. 4.2 and 4.3, respectively. The maximum and minimum principal stresses are calculated from 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 maximum and minimum principal stresses, ϵ_{p1} and ϵ_{p2} are maximum and minimum principal strains, E_s is modulus of elasticity of steel plate ($E_s = 202,940$ MPa) and ν_s is Poisson's ratio of steel ($\nu_s = 0.3$).

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):

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

Figure 4.2: Principal strains versus displacement at center of the wall under monotonic loading

Figure 4.3: Principal stresses versus displacement at center of the wall under monotonic loading

At 29 mm displacement, a sudden increase in the principal stresses was observed possibly due to initiation of cracks in concrete and debonding 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):

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

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

where, γ_{max} is the maximum shear strain, τ_{max} is the maximum shear stress, ε_{p1} and ε_{p2} are the maximum and minimum principal strains, and the other variables were defined earlier.

The τ_{max} is limited based on von-Mises yield criterion (Megson 2005) with the Eq. 4.6:

$$\tau_{max} \leq \frac{F_y}{\sqrt{3}} \quad (4.6)$$

The variation of calculated experimental shear strain and stress at the center (crest) of the profiled steel sheet are shown in Fig. 4.4. The maximum experimental shear stress was 193 MPa which was lower than the maximum yield shear stress for the steel based on Eq. 4.6 (318.7 MPa) and no visible damage was observed up to that stage of loading. It was decided to carry out the second test set-up modification and then perform the test on the same specimen again.

Figure 4.4: Shear strain and stress versus displacement at center of the wall (CSW-1)

After modifying the test set-up in Phase 2, monotonic loading test on the composite shear wall was performed. The experimental shear load versus displacement response of wall (Phase 2) is shown in Fig. 4.5. This figure shows the improvement of the test setup in Phase 2 compared to Phase 1. The test continued up to a maximum load of 240 kN at 96 mm displacement and then stopped due to test set-up problem, which happened due to local yielding of the connecting angles and columns of the steel frame at the bottom of the composite wall. This problem was fixed in the third phase (Phase 3) of the test set-up modification.

Figure 4.5: Load versus displacement under monotonic loading

The maximum/minimum principal strains and stresses at the center (crest) of the profiled steel sheet are shown in Fig. 4.6. The absolute values of the principal strains (stresses) remained almost same up to 67 mm displacement (181 kN). Beyond that the absolute values were not the same possibly due to the development of cracks in the concrete.

Figure 4.6: Principal strains and stresses versus displacement at center of the wall

The variation of shear strain and stress with displacement at the center (crest) of the profiled steel sheet are shown in Fig. 4.7. At this test the maximum shear stress was reached to 202 MPa which was lower than yield shear of 318.7 MPa for steel sheet. At 240 kN load, the test was stopped to perform the third phase of test set-up modification. No visible local buckling of the profiled steel sheet was observed.

Figure 4.7: Shear strain and stress versus displacement at center of the wall

In phase 1, the test was conducted by pulling while by pushing in phase 2 (after test set-up modification), the direction of cracks in the profiled concrete core reversed. This was evident from the development of diagonal cracks at almost 35 degrees in two directions as shown in Fig. 4.8. The diagonal cracks indicated the progress of diagonal tension-compression development in the wall which is the basic feature of the shear test.

Figure 4.8: The crack pattern in the concrete panel after the final test on the specimen

In order to improve the test setup in phase 3, all four steel columns from composite wall were disassembled to weld a steel plate to each column. The reassembling of the all parts was not possible and the failure mode of specimen CSW-1 was not detected.

4.3.2 Specimen CSW-3

This specimen was made of ECC and high strength profiled steel sheet. The main reason for this test was to compare the behaviour of the wall made ECC over SCC. ECC as a concrete material is very ductile and its use was expected to improve the ductility of the wall. The monotonic loading on this composite shear wall specimen was applied through prescribed displacement until failure. The load-displacement response is shown in Fig. 4.9. A drop in load from 159 kN (at 36 mm displacement) to 148 kN was observed possibly due to the formation of major cracks in the profiled concrete. The stiffness of the composite wall before and after load dropping did not change. The ultimate shear capacity and stiffness of the wall under monotonic loading was 281 kN (at 66 mm displacement) and 4.3 kN/mm, respectively. The stiffness of the wall up to failure was constant possibly due to high stiffness of the profiled steel sheets compare to concrete and sudden buckling of the sheets.

Figure 4.9: Load-displacement response under monotonic loading

The variation of maximum/minimum principal strains and stresses at three locations of the wall are presented in Figs. 4.10 and 4.11, respectively. The absolute maximum/minimum values of the principal stresses at the center of the wall (rosette 1) were almost equal and showed the pure shear condition at the center of the composite shear wall.

Figure 4.10: Principal strain versus displacement at three rosette locations of the wall

Figure 4.11: Principal stress versus displacement at three rosette locations of the wall

Those principal stresses at the bottom-left corner of the wall (rosette 3) were 230 MPa and 400 MPa, respectively. The values of strains in rosette 2 strain gauge were correct only up to 10 mm displacement due to debonding of this rosette from the profiled steel sheet after that displacement.

The calculated shear strain and stress at center and the bottom corners of the wall are shown in Figs. 4.12 and 4.13, respectively. The values of shear strain in rosette locations were close to each other. The maximum shear stress reached to 310 MPa which was very close to the maximum theoretical shear yield stress in the steel sheet based on von-Mises yield criterion (318.7 MPa). This indicates that the failure load of the wall is very close to that of shear yielding of steel.

Figure 4.12: Shear strain versus displacement in monotonic loading at three locations of the wall

Figure 4.13: Shear stress versus displacement in monotonic loading at three locations of the wall

The axial stress development in the boundary frame column is presented in Fig. 4.14. The strain gauges 5 and 6 were located at one-third and two-third height of the wall from the bottom, respectively. Gauge 5 near the bottom of the column had shown higher stress than gauge 6. The axial deformation of the column was less than 0.5 mm and showed good rigidity of the frame.

Figure 4.14: Variation of axial stress in the boundary column

The composite shear wall failed due to yielding of steel and subsequent buckling of both profiled steel sheets as shown in Fig. 4.15.

Figure 4.15: Buckling of profiled steel sheets at failure at two sides of the wall

The crack pattern in the profiled concrete core is shown in Fig. 4.16. The diagonal cracks in the ECC were not continuous and the angle of cracks was almost 39 degrees with respect to the profiled ribs.

Figure 4.16: The crack pattern in the concrete (ECC) core of composite wall at failure

4.3.3 Specimen CSW-5

This specimen was made of SCC and mild strength steel. The purpose of this test was to investigate the behaviour of the wall made of mild strength steel. The elongation and ductility of high strength steel was very low which contributed to the brittle failure and low ductility to the composite shear wall. It was expected that the replacement of high to mild strength steel improves the ductility of the wall. The load was applied as prescribed displacement on this composite shear wall specimen until failure of the wall. The experimental load versus displacement curve is shown in Fig. 4.17. The load increased with the increase of displacement until 199 kN (at 48 mm displacement) and then dropped to 175 kN possibly due to formation major continuous line of cracks in the profiled concrete. The ultimate load capacity and stiffness of the wall specimen were 212 kN (at 59 mm displacement) and 4.0 kN/mm, respectively.

Figure 4.17: Load versus displacement in monotonic loading

The maximum/minimum principal strains and stresses at three locations of the wall are presented in Figs. 4.18 and 4.19. The amount of principal strains at the center of the wall at the crest (rosette 1) and trough (rosette 4) had two jumps at 12 mm and 21 mm displacement which might be associated with initiation of concrete cracking and debonding of steel sheet-concrete interface. The strain at the rosette 3 showed only one jump at 21 mm displacement. The initiation of major

line of cracks on the concrete panel associated with steel-concrete debonding was the main reason for the sudden increase of the strains. The limiting principal stresses for steel yielding ($F_y = 354 \text{ MPa}$) based on von-Mises yield criterion as per Eq. 4.3 showed that the profiled steel sheet yielded at the three locations before the failure of the wall. The absolute values of the principal stresses at the center of the wall (rosette 1) were almost equal and showing the development of pure shear condition up to 30 mm displacement.

Figure 4.18: The principal strains versus displacement at three locations of the wall

Figure 4.19: The principal stresses versus displacement at three locations of the wall

The shear strain and stress at the three locations of the wall are shown in Figs. 4.20 and 4.21. The shear strain in rosette 4 (trough) was higher than the strain in rosette 1 (crest) at the linear stage before cracking in concrete (12 mm displacement). The lower thickness of the concrete panel at trough (25 mm) compared to crest (58 mm) caused higher shear strain/stress in the profiled steel sheet at trough. After initiation of cracks in the concrete core, the difference of shear strain/stress at trough and crest became insignificant. The maximum shear stress for steel yielding was limited to 204.4 MPa, based on von-Mises yield criterion (Eq. 4.6).

It was found that the profiled steel sheet yielded at the trough earlier than the crest (at 55 mm displacement compared to 62 mm).

Figure 4.20: The shear strain versus displacement at three locations of the wall

Figure 4.21: The shear stress versus displacement at three locations of the wall

The analysis of stress/strain development and experimental observation confirmed that the failure of composite shear wall was due to yielding of steel associated with the buckling of both profiled steel sheets as shown in Figs. 4.22 and 4.23. The initiation of local buckling, subsequent development of diagonal tension band bounded by the profiled ribs and loss of profiled geometry were the main aspects of sheet failure at the failure stage of the wall.

Figure 4.22: Buckling of the profiled steel sheets at two sides of the wall

Figure 4.23: Detailed buckling pattern of the profiled steel sheet

The crack pattern in the profiled concrete core is shown in Fig. 4.24. The angle of diagonal cracks was almost 31 degrees with respect to the profiled ribs.

Figure 4.24: The crack pattern in the concrete core after the test

4.4 Behaviour of Composite Walls under Cyclic Loading

The cyclic shear loading was applied following the sequence suggested in the AISC Seismic Provisions (AISC, 2002) and Applied Technology Council (ATC 24, 1992) for all three specimens. The loading sequence was adjusted based on the capacity of the hydraulic actuator and the limitations of the experimental test set-up. Sampling rate of data acquisition for the first cyclic loading (specimen 2) was ten scans per second. For the second and third cyclic loadings (specimens 4 and 6) the sampling rate was reduced from 10 to 5 scan per sec in order to eliminate collecting the unnecessary data and also reduce the size of data file. The shear load-displacement response and strain development on the steel sheet were monitored throughout the loading history. The displacement ductility ratio was calculated as a ratio of maximum displacement to the displacement at which significant yielding took place (Driver et al. 1998).

The cyclic behaviour of the three walls are studied based on types of concrete (SCC and ECC) and also the steel sheet strength. The behaviour of the walls are described based on experimental results on load-displacement response, stress-strain characteristics, failure modes, concrete cracking, buckling of steel sheets and stiffness degradation.

4.4.1 Specimen CSW-2

The specimen CSW-2 was manufactured using SCC and high strength steel. The test was conducted after phase 3 (final) modification of the test set-up. This test was performed to investigate the cyclic behaviour of the wall including displacement ductility ratio. The load was applied in groups of cycles (total of five groups) with each group consisted of three cycles except the last group (one cycle). The first three groups of cycles were load controlled and the rests were displacement controlled. The amount of maximum load ($V_y = 240$ kN) in the elastic range and the corresponding horizontal displacement at the top of the wall specimen ($\Delta_y = 52$ mm) were derived from the monotonic loading test. The loading sequences consisted of three initial cycles of load control with maximum load of 80 kN (cyclic group 1), three cycles of load controlled with maximum load of 160 kN (cyclic group 2), three cycles of load controlled with maximum load of 240 kN (cyclic group 3), three cycles of displacement controlled (cyclic group 4) with 1.5 times the displacement at load 240 kN ($1.5\Delta_y$) and finally, three cycles of

displacement controlled with $2 \Delta_y$ (cyclic group 5). The drift of the specimen was calculated by dividing the horizontal displacement of the top of the specimen by the total height of specimen which was 1586 mm. The cyclic loading detail is shown in Table 4.2.

Table 4.2: The detailed cyclic loading sequence for wall specimen CSW-2

Group of cycles	Number of cycles	Frequency (Hz)	Cumulative number of cycles	Load ratio (V/V_y) $V_y = 240$ kN	Disp. ratio (Δ/Δ_y) $\Delta_y = 52$ mm	Maximum load (kN)	Maximum Displacement (mm)	Drift (%)
1	3	0.1	3	0.33		80	17.1	1.11
2	3	0.05	6	0.67		160	34.3	2.23
3	3	0.0333	9	1.00		240	52	3.39
4	3	0.0167	12		1.5	294	78	5.08
5	1	0.0222	13		2.0	205	104	6.77

V: Maximum lateral load at top of the wall; V_y : Lateral load at which significant yielding took place; Δ : Maximum horizontal displacement at the top of the wall; and Δ_y : Horizontal displacement at which significant yielding took place

The hysteric loops of the load versus displacement for composite wall under cyclic loading is presented in Figure 4.25. A maximum load of 294 kN was reached during testing in the cyclic group 4.

Figure 4.25: Load-displacement response of wall specimen under cyclic loading

The failure of the wall was associated with the tearing of the profiled steel sheets and subsequent local buckling at the first cycle of the cyclic group 5 (13th cycle) as shown in Fig. 4.26. It should

be noted that before tearing of the steel sheets, no local buckling was observed which illustrated good composite action between steel and concrete provided through sheet-concrete intermediate shear fasteners in the wall. The displacement ductility of the wall was 1.5 which is low compared to the ductility of steel plate shear wall (Driver et al. 1998).

Figure 4.26: Failure modes of composite wall under cyclic loading

The average stiffness of the wall at each of the cyclic groups is shown in Table 4.3. The stiffness of the wall was reduced from 6.57 kN/mm in the first three cycle of the cyclic loading to 2.09 kN/mm in the 13th cycle showing a reduction of by 68.2%. The wall stiffness degradation versus number of cycles in the cyclic loading is presented in Fig. 4.27.

Table 4.3: The stiffness degradation of the wall due to cycling loading

Cyclic loading group No.	1	2	3	4	5
Average stiffness of the wall (kN/mm)	6.57	5.23	4.63	3.83	2.09
Stiffness degradation (%)	0	20.4	29.5	41.7	68.2

Figure 4.27: Stiffness degradation versus number of cycles

The envelope of load versus displacement in cyclic loading is shown in Fig. 4.28 as adopted by other researchers (Kabir and Nassira 2005). This envelope shows the maximum load and the displacement at each cycle. The results show that for the first four groups of cycles, the maximum load and displacement are almost the same and stable for each group. Also an increase in the load is observed except for the 13th cycle, where the load dropped to 205 kN.

Figure 4.28: Envelope of load-displacement in cyclic loading

The variation of shear strain at the centre of the wall is shown in Fig. 4.29. The shear strain gradually reached 3,300 micro-strains at the 12th cycle but at the 13th cycle, it jumped to 12,000 micro-strains due to tearing of a part of the profiled steel sheet close to the rosette location (Fig. 4.26).

Figure 4.29: The shear strain versus displacement at the center of the wall (rosette 1)

The shear stress versus displacement at the three locations of the wall is shown in Figs. 4.30 to 4.32. The calculated shear stress in the rosette 1 and 3 was limited to 318.7 MPa (this is the maximum yield shear stress for the steel) which indicated that shear yielding of steel sheets at failure. The shear stress hysteretic curves in the rosette 1 and 3 showed a brittle behaviour (narrow hysteresis loops) of the steel sheet without having sufficient ductility (elongation) which caused sudden failure of the wall at 13th cycle.

Figure 4.30: The shear stress versus displacement response at the center of the wall (rosette 1)

Figure 4.31: The shear stress versus displacement response at rosette 2

Figure 4.32: The shear stress versus displacement response at rosette 3

4.4.2 Specimen CSW-4

The specimen CSW-4 consisted of high strength profiled steel sheets and ECC. The type of concrete was ECC to investigate the advantage of ECC over SCC in cyclic loading. For this specimen, the cyclic shear loading was applied in total ten cyclic groups with each group consisting of three cycles. The loading sequences were three cycles of load controlled with maximum load of 80 kN (cyclic group 1), three cycles of load controlled with maximum load of 160 kN (cyclic group 2), three cycles load controlled with maximum load (V_y) of 240 kN (cyclic group 3), three cycles of displacement controlled (cyclic group 4) with 1.1 times of top displacement at the load of 240 kN ($1.1 \Delta_y$) and three cycles of displacement controlled with 1.2 Δ_y (cyclic group 5) up to 1.7 Δ_y (cyclic group 10). The difference in the cyclic loading pattern between this test and the previous test was 10% increment in displacement compared to 50% increment after the displacement at which significant yielding took place. The reason was to capture more detailed information before failure of the specimen. The cyclic loading scheme is shown in Table 4.4.

Table 4.4: The detailed cyclic loading sequence for wall specimen CSW-4

Group of cycles	Number of cycles	Frequency (Hz)	Cumulative number of cycles	Load ratio (V/V_y) $V_y = 240$ kN	Disp. ratio (Δ/Δ_y) $\Delta_y = 58$ mm	Maximum load (kN)	Maximum displacement (mm)	Drift (%)
1	3	0.1	3	0.333		80	17.5	1.14
2	3	0.05	6	0.667		160	35.3	2.30
3	3	0.0333	9	1.00		240	58	3.78
4	3	0.0303	12		1.1	226	63.8	4.15
5	3	0.0278	15		1.2	227	69.6	4.53
6	3	0.0256	18		1.3	228	75.4	4.91
7	3	0.0238	21		1.4	236	81.2	5.29
8	3	0.0222	24		1.5	246	87	5.66
9	3	0.0207	27		1.6	249	92.8	6.04
10	3	0.0194	30		1.7	250	98.6	6.42
V: Maximum lateral load at top of the wall; V_y : Lateral load at which significant yielding took place; Δ : Maximum horizontal displacement at the top of the wall; and Δ_y : Horizontal displacement at which significant yielding took place								

The experimental hysteric loops of the load- displacement response for this composite wall under cyclic loading presented in Figure 4.33, showing that a maximum load of 250 kN was reached in the cyclic group 10.

Figure 4.33: Load-displacement response of wall specimen under cyclic loading

The failure of the wall was associated with the initiation of local buckling of the profiled steel sheets at the bottom corners during the cyclic group of cycle 10 (30th cycle) as shown in Fig. 4.34.

Figure 4.34: Local buckling of the profiled steel sheets at failure under cyclic loading

The wall stiffness degradation versus number of cycles in the cyclic loading is presented in Fig. 4.35. The reason for the sudden stiffness degradation from cyclic group 3 to 4 and increase in stiffness from cyclic group 4 to 5 was attributed to the nature of cyclic loading sequence. After the load controlled sequence in the cyclic group 3, the displacement controlled was started with very small amount of displacement increment which caused a reduction of maximum load from 240 kN to 226 kN as shown in Table 4.5 and Fig. 4.33. The stiffness of the wall was reduced from 4.73 kN/mm in the first cyclic group of the cyclic loading to 2.47 kN/mm in the 10th cyclic group showing a reduction of 47.8%. Stiffness reduction of the specimen CSW-4 made of ECC was lower than specimen CSW-2 made of SCC (68.2%) in spite of having more cyclic groups. The displacement ductility of the wall was also 1.7 compared to 1.5 from specimen CSW-2. This showed better behaviour of the wall made of ECC compared to SCC.

Figure 4.35: Stiffness degradation versus number of cycles

The envelope of load- displacement response in cyclic loading regime is shown in Fig. 4.36. This envelope shows the maximum load and displacement at each load cycle. The maximum load and displacement for each group was almost the same and stable for the 10 cyclic groups. However, there was a sudden drop in load from cyclic group 3 to 4 for the reason explained before.

Figure 4.36: Envelope of load-displacement response in cyclic loading

The shear strain and stress versus displacement response at the centre of the wall at crest location of the profile (rosette 1) are illustrated in Figs. 4.37 and 4.38, respectively. The shear strain gradually increased with the increase of load and reached to 3,870 micro-strains (equivalent to a shear stress of 302 MPa) at the last cycle. The shear strain and stress at the centre of the wall at trough location (rosette 4) was higher than the crest location (reached to 310 MPa) as shown in Figs. 4.39 and 4.40, respectively at the last cycle.

Figure 4.37: Shear strain versus displacement for rosette 1

Figure 4.38: Shear stress versus displacement for rosette 1

Figure 4.39: Shear strain versus displacement for rosette 4

Figure 4.40: Shear stress versus displacement for rosette 4

The variations of shear strain and stress with displacement at the right corner of the wall (rosette 2) are shown Figs. 4.41 and 4.42, respectively. The shear strain reached 6,580 micro-strains at the last group of cycles. The shear stress was limited to the shear yield stress (318.7 MPa) as per criteria for yielding of the steel based on Eq. 4.6. The rosette 2 was located very close to the location where the local buckling was observed in the last cycle. This illustrated that the profiled steel sheet buckled after yielding of the steel sheet and showed effectiveness and adequacy of intermediate steel-concrete interface fasteners. The variations of shear strain and stress with displacement at the left corner of the wall (rosette 3) are shown Figs. 4.43 and 4.44, respectively. The shear strain gradually reached 3,630 micro-strains at the last cycle of loading which was equivalent to a stress of 283 MPa.

Figure 4.41: Shear strain versus displacement for rosette 2

Figure 4.42: Shear stress versus displacement for rosette 2

Figure 4.43: Shear strain versus displacement for rosette 3

Figure 4.44: Shear stress versus displacement for rosette 3

4.4.3 Specimen CSW-6

The specimen CSW-6 was manufactured using mild strength profiled steel sheets and SCC. In this test mild strength steel was utilized to investigate the advantage of mild strength steel over high strength steel in cyclic loading. The cyclic shear loading was applied in total six cyclic groups with each group consisting of three cycles. The loading sequences were three cycles of load controlled with maximum load of 60 kN (cyclic group 1), three cycles of load controlled with maximum load of 120 kN (cyclic group 2), three cycles of load controlled with maximum load of 180 kN (cyclic group 3), three cycles of displacement controlled (cyclic group 4) with 1.2 times of displacement at the load of 180 kN ($1.2 \Delta_y$), and three cycles of displacement controlled with $1.3 \Delta_y$ (cyclic group 5) and $1.4 \Delta_y$ (cyclic group 6). The difference in the cyclic loading pattern between this test and the previous test was the increment of displacement by 20% from the displacement at which significant yielding took place and 10% thereafter. The reason was to prevent a sudden drop in load from cyclic group 3 to 4 as explained in the previous test. The cyclic loading in detail is shown in Table 4.5.

Table 4.5: The detailed cyclic loading sequence for wall specimen CSW-6

Group of cycles	Number of cycles	Frequency (Hz)	Cumulative number of cycles	Load ratio (V/V_y) $V_y = 180$ kN	Disp. ratio (Δ/Δ_y) $\Delta_y = 46.9$ mm	Maximum load (kN)	Maximum displacement (mm)	Drift (%)
1	3	0.1	3	0.33		60	13.5	0.88
2	3	0.05	6	0.67		120	29.2	1.90
3	3	0.0333	9	1.00		180	46.9	3.05
4	3	0.0303	12		1.2	209	56.3	3.67
5	3	0.0278	15		1.3	199	60.9	3.96
6	3	0.0256	18		1.4	170	65.7	4.28
V: Maximum lateral load at top of the wall; V_y : Lateral load at which significant yielding took place; Δ : Maximum horizontal displacement at the top of the wall; and Δ_y : Horizontal displacement at which significant yielding took place								

The hysteric loops of load-displacement response for this composite wall under cyclic loading is presented in Figure 4.45. The maximum load reached in this test was 209 kN in the cyclic group 4 and then gradually dropped to 170 kN in the cyclic group 6. The failure of the wall started with the formation of scattered minor local buckling of the profiled steel sheets along the wall diagonal during the cyclic group 5. The major deformation in the steel sheets as shown in Fig. 4.46 was observed at the 18th cycle when the test was stopped.

Figure 4.45: Load-displacement response of wall specimen under cyclic loading

Figure 4.46: Buckling of the profiled steel sheets at two sides of the wall under cyclic loading

The wall stiffness degradation versus number of cycles during cyclic loading is presented in Fig. 4.47. The stiffness of the wall was reduced from 5.37 kN/mm in the first three cycles of loading to 2.23 kN/mm in the 18th cycle showing a reduction of 58.5%. Sudden stiffness degradation from the cyclic group 3 to 4 (similar to specimen CSW-4) was not observed. This is due to the fact that the displacement controlled loading regime was started with 20% increment of displacement at the load of 180 kN. The stiffness degradation in general was uniform and showed a good wall behaviour regarding reduction of stiffness in cyclic loading before failure. The displacement ductility of the wall was 1.4.

Figure 4.47: Stiffness degradation versus number of cycles

The envelope of load versus displacement in cyclic loading regime is shown in Fig. 4.48. The maximum load and displacement at each cycle are presented in this Figure. The result shows that in each cyclic group, the maximum load and displacement are almost the same for all the 6 cyclic groups.

Figure 4.48: Envelope of load-displacement response in cyclic loading regime

During performing the test, it was noticed that the rosette 2 (at the right corner of the wall) was disconnected. The variation of shear strain with displacement at the centre of the wall at crest, trough and the left corner (rosettes 1, 4 and 3) are shown in Figs. 4.49 to 4.51, respectively. The shear strain gradually reached to about 2,000 micro-strains at the cyclic loading group 4 and then dramatically increased to 14,000 micro-strains at the last cycle. The reason for this large strain increase was attributed to the location of rosette strain gauge very close to the formation of major deformation in the profiled steel sheet. The shear stress versus displacement at the centre of the wall at crest, trough and the left corner (rosettes 1, 4 and 3) are shown in Figs. 4.52 and 4.54, respectively. The maximum steel shear stress has been limited to 204.4 MPa as calculated using Eq. 4.8. The shear stress at the centre of the wall at trough (rosette 4) was higher than the crest in the first three groups of cycles before yielding of the steel sheets (Fig. 4.53).

Figure 4.49: Shear strain versus displacement for rosette 1

Figure 4.50: Shear strain versus displacement for rosette 4

Figure 4.51: Shear strain versus displacement for rosette 3

Figure 4.52: Shear stress versus displacement for rosette 1

Figure 4.53: Shear stress versus displacement for rosette 4

Figure 4.54: Shear stress versus displacement for rosette 3

4.5 Discussions and Conclusions

In this chapter, the experimental results from six composite wall specimens tested under monotonic and cyclic loading are presented. The summary of the test results are presented in Tables 4.6 to 4.8.

Table 4.6: Experimental test results for monotonic and cyclic loading

Specimen No.	Type of loading	Type of concrete	f'_c (MPa)	f'_t (MPa)	F_y (MPa)	F_u (MPa)	Ultimate load capacity (kN)	μ_Δ	Mode of failure/ Comments
CSW-1	Monotonic	SCC	51.9	4.92	552	577	240	-	Did not failed
CSW-2	Cyclic	SCC	48.1	4.68	552	577	294	1.5	Brittle failure
CSW-3	Monotonic	ECC	40.2	4.75	552	577	281	-	Inelastic buckling
CSW-4	Cyclic	ECC	39.4	4.71	552	577	250	1.7	Minor inelastic buckling
CSW-5	Monotonic	SCC	35.7	3.28	354	420	212	-	Inelastic buckling
CSW-6	Cyclic	SCC	34.6	3.15	354	420	209	1.4	Inelastic buckling
f'_c : Compressive strength of concrete; f'_t : Tensile strength of concrete; F_y : Yield strength of profiled steel sheet; F_u : Ultimate strength of profiled steel sheet; SCC : Self-Consolidating Concrete ECC : Engineered Cementitious Composites; and μ_Δ : Displacement ductility ratio									

The test on wall specimen CSW-1 was regarded as the pilot monotonic test to study the performance of the shear test set-up and to do some modifications on the test setup. In general, the ultimate cyclic load capacity of the wall specimens was not less than 12% of the monotonic load capacity. The advantage of using ECC over SCC in the CSW-3 and CSW-4 specimens was exhibited through more ductile behaviour. The advantage of using mild over high strength steel in the CSW-5 and CSW-6 specimens was demonstrated through prevention of brittle failure associated with tearing of the steel sheets.

The main difference of crack patterns in the profiled concrete core between SCC and ECC was crack continuity. In SCC the cracks were continuous along the width of the concrete core but in ECC, the cracks were scattered and no major continuous line of cracks was observed. The number of steel-concrete intermediate fasteners along the height and width of the specimens

provided sufficient steel-concrete composite action to prevent elastic buckling of the profiled steel sheets and initiated failure due to yielding of steel sheets.

The load associated with the initiation of concrete cracking and debonding of the profiled steel sheet from the concrete core ($P_{c/d}$) was observed from the shear strain response at the first sudden increase. The $P_{c/d}$ was very clear for specimens made of SCC but not for ECC at the location of rosette strain gauges. Additional experimental test results for monotonic loading are summarized in Table 4.7.

Table 4.7: Experimental test results for monotonic loading

Specimen No.	Type of concrete	Steel sheet strength	Concrete core		Profiled steel sheet at the center		
			Cracks type	Cracks angle	Maximum shear strain (10^{-6})	Maximum shear stress (MPa)	$P_{c/d}$ (kN)
CSW-1	SCC	High	continuous	35°	2633	202	33
CSW-3	ECC	High	scattered	39°	4016	310	-
CSW-5	SCC	Mild	continuous	31°	2913	204	49
$P_{c/d}$: Load associated with the initiation of concrete cracking and debonding							

In cyclic loading, the specimen made of ECC (CSW-4) subjected to ten cyclic groups. The stiffness of the wall was reduced from 4.73 kN/mm in the first cyclic group to 2.47 kN/mm in the 10th cyclic group, showing a stiffness degradation of 47.8% (Table 4.8). This stiffness degradation is the lowest among other specimens in spite of having more cyclic groups. The displacement ductility of the wall was also the highest (1.7) compared to 1.5 and 1.4 for specimens CSW-2 and CSW-6, respectively. This showed better behaviour of the ECC wall compared to SCC counterpart.

Table 4.8: Experimental test results for cyclic loading

Specimen No.	Type of concrete	Steel sheet strength	Wall stiffness (kN/mm)			
			First cyclic group	Last cyclic group	Number of cyclic groups	Stiffness degradation
CSW-2	SCC	High	6.57	2.09	5	68.2%
CSW-4	ECC	High	4.73	2.47	10	47.8%
CSW-6	SCC	Mild	5.37	2.23	6	58.5%

CHAPTER 5

EXPERIMENTAL INVESTIGATION OF COMPOSITE WALL UNDER IN-PLANE IMPACT LOADING

5.1 Introduction

The results of the experimental test program on composite walls subjected to in-plane impact loading are presented in this chapter. Two tests on impact loading were conducted. A complete impact test on each specimen consisted of three parts namely; Impact Phase I, Impact Phase II, and Post-impact Monotonic Loading. In Phase I, the speed or mass of projectile was kept low intentionally in order to get the natural period and maximum displacement at the top of the almost undamaged wall. In Phase II, the impact test was performed with the maximum speed that the impact apparatus could produce. In post-impact monotonic loading, the residual shear strength of the specimen after the impact loading was captured. The results of the impact loading tests are presented and discussed by illustrating the maximum wall displacement at the top, the fundamental period of the wall, and also analysing the acceleration of the wall at four locations. The results of monotonic loading tests are presented by illustrating load versus displacement, stiffness degradation of the specimen and failure modes of the specimens.

In this chapter, the behaviour of a structure under in-plane impact loading is discussed and the results of the tests on two composite wall specimens are presented and at the end of chapter, a summary of all the results and conclusions are demonstrated.

5.2 Structure under Impact Loading

Understanding the behaviour of a structure under impact loading is important due to the possibility of impact during the service life of the structure. For building structures, impact loading can be accidental explosions, hitting objects by hurricane, striking airplanes, impact vehicles, etc. The major difference between ‘impact’ and ‘static loading’ is that the structure

under impact must be designed to absorb energy, but the structure under static loading should be designed strictly to carry the loads.

When a projectile impacts a specimen, the specimen will absorb energy until the material deforms elastically up to its yield strength. After yield strength, the specimen will begin to have plastic deformation and will absorb energy in the plastic zone. Fracture will occur, when the specimen cannot absorb more energy. The amount of transferred energy (E) from a projectile of mass ' M ' to an object can be obtained from Eq. 2.9 in Chapter 2.

The time duration of the loading processes is called *strain rate* and describes the rapidness of deformation processes. The strain rate measures the time rate of change, usually taken in units of s^{-1} (Hiermaier 2008). The strength properties of steel usually change with the speed of applied load (Liew 2004). Generally, both the yield and ultimate strengths tend to increase with the speed of loading. The variation of strain rate versus tensile property of steel is shown in Fig. 5.1.

Figure 5.1: The effect of strain rate on tensile properties of mild steel at room temperature (Juvinal 2006)

As shown in Fig. 5.1, the ratio of yield strength to ultimate strength varies from 0.5 (for static loading) to 1 (for very high strain rate) for the mild steel. Also the amount of steel elongation increases with the increase of the strain rate.

The behavior of concrete subjected to high strain rate loading is different compared with that under quasi-static loading (Xiao et al. 2008; Bentur et al. 1986; Soroushian et al. 1986). The effect of a strain rate for concrete in tension based on experimental data is shown in Fig. 5.2. The Dynamic Increase Factor (DIF) is a coefficient that magnifies the tensile strength of concrete based on different strain rates. As shown in Fig. 5.2, the value of DIF for very low strain rate (static loading) is 1 and increases to 8 for high strain rate.

Figure 5.2: The variation of strain rate versus DIF for concrete in tension (Leppanen 2006)

The strain rate sensitivity of concrete in compression is less than concrete in tension (Rose et al. 1995, Tedesco et al. 1994). Similar to conventional concrete, Engineered Cementitious Composites (ECC) has mechanical properties which demonstrate rate dependency. Fig. 5.3 shows the tensile stress-strain curve of typical ECC with static tensile strength of 4 to 5 MPa, subjected to different strain rates (Yang 2008). The strain rate ranges from 10^{-5} to 10^{-1} s^{-1} , corresponding to quasi-static loading to low speed impact.

Figure 5.3: The effect of strain rate on tensile stress-strain curve for typical ECC (Yang 2008)

Many experimental testing and numerical impact analyses have been performed on reinforced concrete slabs (Zineddin 2002, Harris et al.1999, Nilsson 1973, Uchida 1985) and on reinforced concrete beams (Banthia et al. 1989, May et al. 2005) to investigate the behaviour of the structure and different type of concrete under striking dropping weight. From the experimental tests (Banthia et al. 1987), it was determined that in impact loading, fibre reinforced concrete can absorb more impact energy compared to plain concrete due to its ductility.

This research study was intended to investigate the behaviour of profiled composite shear wall with two types of concrete namely Self-Consolidating Concrete (SCC) and ECC, under in-plane impact shear loading. It was expected that the energy absorbing capacity from impact testing of composite shear wall made of ECC would be higher compared with that made with SCC because of strain capacity of ECC. Due to complexity and some unforeseen difficulties during the impact tests as well as the time limitation for completing the research, just two specimens with SCC were tested. The residual load carrying capacity of the composite wall after the impact was investigated by applying monotonic load to the wall up to the failure.

5.3 Behaviour of a Structure under Impact Loading

The effect of impact on a structure can be considered in two different ways: the dynamic behaviour of the structure as a whole and the shock wave propagation in the structure. In solid body dynamics which is dealing with forces that cause the motion of a structure, it is assumed that if a load is applied to one point of the structure, then every other point in the structure responds to this load instantaneously. In other word, if a load is applied at one point and acceleration occurs, then every point in the structure attains its own acceleration instantaneously. The applied loads to a structure are propagated through the medium by waves and accordingly at the local speed of sound. In the case of quasi-static loads, wave effects are not investigated since the loading duration is long compared to the duration of multiple reflections throughout the structure. The shock wave (stress wave) propagation must be considered when loads are applied to the structure for short time durations and measurements are made in very small intervals of time after the application of the applied loads or impact (Tedesco et al. 1998).

5.3.1 Elastic Wave Propagation in Solids

Waves in solids are basically propagated through the continuum in different forms and at different but related velocities. The most important three forms of waves in solids are:

- *Longitudinal waves* of tensile or compressive type cause particle deflections along the propagation direction. Longitudinal waves also called *primary waves* are the fastest wave forms in solids. The longitudinal wave propagation speed in solids is related to the square root of the ratio of material stiffness (modulus of elasticity) to the material mass density. The approximate unbounded medium longitudinal wave velocity for steel is 5,770 m/sec.
- The next fastest waves propagating are the *secondary waves* or *shear* causing particle motion perpendicular to the wave propagation direction. The approximate unbounded medium shear wave velocity for steel is 3,180 m/sec (Tedesco et al. 1998).
- *Rayleigh waves* are propagated along the surfaces of solids which set surface particles into elliptic motion.

The structural deformation is not influenced by only single wave transitions. All the shock waves have two characteristics which are extremely short rise times and high wave amplitudes (Hiermaier 2008).

5.3.2 The Elastic Dynamic Behaviour of the Structure

The displacement response of a Single Degree of Freedom (SDOF) structure to any arbitrary (non-periodic) forms of excitation such as impact can be determined from the *Duhamel's integral*. For impact load due to its short duration of time, the response of the structure is not significantly affected by the presence of damping in the system. The effect of damping therefore, in this study is neglected. For an un-damped system, displacement response under impact can be derived by Eq. 5.1 (Tedesco et al. 1998):

$$x(t) = \frac{1}{m\omega} \int_0^t F(\tau) \sin \omega(t - \tau) d\tau \quad (5.1)$$

where, t = time (sec); $x(t)$ = displacement response (m); m = mass of the SDOF structure (kg); ω = natural circular frequency, $\omega = 2\pi f$ (rad/sec), $\omega = \sqrt{k/m}$; k = stiffness of the SDOF structure (N/m); and $F(\tau)$ = impact load (N).

The impact load can be simplified to a constant pulse load (F_0) starting from ($t = 0$) and ending at ($t = t_d$) or triangular pulse load starting from $F(t = 0) = F_0$ and ending at $F(t = t_d) = 0$. The t_d is called time of duration of the force and the ratio of t_d to natural period of the structure (T) has a significant effect on the dynamic response of a structure (Tedesco et al. 1998).

The response of the structure can be divided into two time intervals (t):

- 1) The time that the force is acting on the structure $0 \leq t \leq t_d$. This time interval is called the forced vibration phase.
- 2) The time after the forced has been removed, $t > t_d$. This time interval is called the transient vibration phase.

The maximum displacement response of the structure may occur in either the forced vibration phase or transient vibration phase, depending upon the ratio of t_d to natural frequency of the structure (T). The natural period of the undamaged composite wall in this study was 0.04 sec.

This natural period was measured by analysing the data from the high-speed video camera during the first impact test (20 frame = 20/500 = 0.04 sec). The duration of the impact was about 0.004 sec which shows that the ratio of t_d/T is 0.1.

With simplification of the impact load to a constant pulse load (F_0), the maximum displacement response occurs in the transient vibration phase when t_d/T is less than 0.5. Therefore, the maximum displacement of the composite wall (used in this research) occurs after the impact (by assuming the elastic behaviour of the wall). The displacement response of the wall at the transient vibration phase will be (Tedesco et al. 1998):

$$x(t) = \frac{F_0}{m \omega^2} [\cos \omega(t - t_d) - \cos \omega t] \quad \text{for } t > t_d \quad (5.2)$$

When $t_d/T = 0.1$, the maximum displacement response in the transient vibration phase is 0.618 times of the displacement if the load is applied in a quasi-static manner (static equilibrium position). The value of the constant pulse load (F_0) can be derived from the momentum equation:

$$F_0 = \frac{M_{\text{Projectile}} (V_{\text{Initial}} - V_{\text{Final}})}{t_d} \quad (5.3)$$

where, $M_{\text{Projectile}}$ = Mass of the projectile (kg); V_{Initial} = Velocity of the projectile before collision (m/s); and V_{Final} = Velocity of the projectile after separation from the object (m/s).

The above simplified calculation shows the elastic behaviour of the wall under impact load from which the top displacement of the wall can be derived. The behaviour of the wall will not be elastic, if the mass or speed of the projectile increases. The impact problems typically involve material and geometrical nonlinearities and unlike the case of the linear problems, very few exact, closed-form solutions are available (Szuladzinski 2010). The best solution for impact problems is numerical simulations which require the treatment of strain rate dependent non-linear material behavior, large deformations (Hiermaier 2008).

5.4 Theoretical Initial Impact Energy for the Composite Wall Test

It is very important to apply the right amount of energy to prevent the total damage to the boundary steel frame and also to the test setup. The impact energy capacity (U) of a linear system over its volume under a shear loading can be expressed as Eq. 5.4 (Juvinall 2006):

$$U = \int \frac{\tau^2}{2G} dV \quad (5.4)$$

where, τ = shear stress capacity of the material; and G = shear modulus, $G = \frac{E}{2(1+\nu)}$

For the composite wall specimen in this research, the total impact energy capacity of two profiled steel sheets and profiled concrete calculated based on Eq. 5.4 was 465 N.m. The non-linear behaviour of material was not considered in Eq. 5.4 and thus the calculated energy capacity of the composite shear wall is very conservative.

Other approach used to estimate preliminary energy capacity of the wall under impact was based on the area under the load-displacement from the monotonic testing which was about 7000 N.m.

Based on the above calculations, it was initially decided to have a projectile with 36.0 kg mass for the first impact test on the wall specimen. Based on the speed of the projectile before impact and the bounce back speed of 10 m/sec and 2 m/sec (20% of the projectile speed), respectively, the impact energy would be 1,728 N.m.

5.5 Impact Tests on Composite walls and Test Results

Two wall specimens were tested under impact loading as given by the test matrix shown in Table 5.1. The type of concrete, steel and material properties for the two specimens are almost same. The mild strength steel having yield strength of 354 MPa and SCC were used to construct both composite wall specimens.

Table 5.1: Test matrix for impact loading ($F_y = 354$ MPa and $F_u = 420$ MPa)

Specimen No.	Type of Loading	Type of concrete	f' _c MPa	f' _t MPa	Weight of the projectile kg	Speed of the projectile m/s	Number of repeating impacts
CSW-7	Impact	SCC	38.7	3.44	36	10.0	6
					64.8	10.0	1
CSW-8	Impact	SCC	34.3	2.98	64.8	6.4	1
					62.5	10.2	1
f' _c : Compressive strength of concrete; f' _t : Tensile strength of concrete; F _y : Yield strength of profiled steel sheet; F _u : Ultimate strength of profiled steel sheet and SCC: Self-Consolidating Concrete							

The results for the impact loading are presented and described in the format of acceleration versus time, Fast Fourier Transform (FFT) of the acceleration data, speed of the projectile before and after impact, period of the composite shear wall after impact, load versus displacement from monotonic loading test conducted after the impact test, and failure modes of the wall specimens.

5.5.1 Impact Test on Wall Specimen CSW-7

This test was performed in two phases, namely; Phase I and Phase II. The mass of projectile in Phase I was kept low (36 kg) intentionally to capture elastic dynamic characteristics of the wall. In Phase II, the impact test was performed with projectile mass of 64.8 kg and maximum speed of 10.2 m/s, which is the impact apparatus maximum speed. The natural period and maximum displacement at the top of the wall were captured in Phase II.

5.5.1.1 Impact Phase I

Analysis of wall displacement

The speed of the camera used during the test was 500 frame per second (0.002 sec per frame), with total capturing duration of 4 second (2000 total frame) and resolution of 1280 x 512 pixels. This time was sufficient to determine the speed of projectile before impact and also to capture the bounce back speed. These two speeds gave a clear measure of the exact amount of transferred kinetic energy to the wall specimen.

The speed measurement was achieved by calculating the distance of two points by number of the pixels. The equivalency of each pixel to the actual distance was performed by a measuring pattern attached to the impact apparatus. Each pixel is equal to 1 mm of the actual distance.

The projectile before the impact moved 20 pixel per frame which was 20 mm per frame, so the projectile speed was $20/0.002 = 10,000 \text{ mm/s} = 10 \text{ m/s}$. The bounce back speed of the projectile was 1.5 m/s. The mass of the projectile was 36 kg and the amount of transferred energy from the projectile to the composite wall was 1760 J ($0.5 \cdot 36 \cdot 10^2 - 0.5 \cdot 36 \cdot 1.5^2 = 1760 \text{ J}$).

The other information captured from the high speed camera was the maximum wall displacement at the top ($x_{\max}$) and also the fundamental period of the wall. The maximum wall displacement at the top was 18 mm and the fundamental period of the wall was 0.04 sec (20 frame = $20/500 = 0.04 \text{ sec}$).

The calculation of theoretical $x_{\max}$ obtained from Eqs. 5.2 and 5.3 is presented as follows:

$M_{\text{Projectile}} = 36 \text{ kg}$, $V_{\text{Initial}} = 10 \text{ m/sec}$ and $V_{\text{Final}} = -1.5 \text{ m/sec}$, therefore:

$$F_0 = \frac{36 [10 - (-1.5)]}{0.004} = 103,500 \text{ N} = 103.5 \text{ kN}$$

$m = 49 \text{ kg}$ (steel columns) + 22 kg (steel beams) + 51 kg (profiled concrete) + 6 kg (steel sheets) + 3 kg (steel bolts) + 23 kg (steel end plates) = 154 kg

$$\omega = 2\pi f = 2\pi / T = 2\pi / 0.04 = 157 \text{ rad/sec}$$

$$x_{\max} = \frac{0.618 \times F_0}{m \omega^2} = \frac{0.618 \times 103,500}{154 \times 157^2} = 0.0169 \text{ m} = 16.9 \text{ mm}$$

The maximum experimental displacement of the wall measured by analysing the data from the high-speed video camera was 18 mm which shows excellent correlation between analytical and experimental $x_{\max}$ at the linear behaviour of the composite wall.

The wall reached to its maximum displacement (18 mm) at about 0.01 s. The maximum shear strain, $\gamma_{\max}$ (calculated by dividing the 18 mm to the wall height of 1536 mm) and the maximum strain rate were 0.012 and 1.2 s^{-1} , respectively.

Analysis of wall acceleration

The acceleration of the wall specimen at four points was measured by mounting two accelerometers at the top and two accelerometers the bottom of the boundary frame as shown in Fig. 5.4.

Figure 5.4: The location of the accelerometers at top and bottom of the boundary steel frame

The shock wave amplitude and frequency of the wall at the location of the accelerometers were captured during the impact. A Fast Fourier Transform (FFT) was performed on the signal from the accelerometers to have a frequency domain solution for the composite wall response. The acceleration data and the related frequency content at four locations of the wall are presented in Figs. 5.5 to 5.8. At the bottom and top of the wall the maximum acceleration amplitude was 280 g's and 3810 g's, respectively. At the bottom of the wall, no dominant frequency content of the acceleration signal was observed. The dominant frequency content of the acceleration signal at the top-front of the wall was 8.3 kHz and the top-rear of the wall was 7.7 kHz. These frequencies are the shock (stress) wave frequencies which represent the shock wave speed.

Figure 5.5: The acceleration versus time and frequency domain (FFT) at bottom-front of the wall

Figure 5.6: The acceleration versus time and frequency domain (FFT) at bottom-rear of the wall

Figure 5.7: The acceleration versus time and frequency domain (FFT) at top-front of the wall

Figure 5.8: The acceleration versus time and frequency domain (FFT) at top-rear of the wall

The model 3200B6 accelerometers used in this research are specially designed for impact test to capture very high amplitude of acceleration and frequency (acceleration up to 2,500 g's, maximum shock of 50,000 g's, and frequency up to 15 kHz). These accelerometers are not sensitive to small amplitude of acceleration (the acceleration less than one percent of 2,500 g's

which is 25 g's). Therefore, due to the low sensitivity of the accelerometers, the natural frequency of the entire composite wall was not captured. It should be noted that if instead, other type of accelerometers with high sensitivity to the small amplitude of acceleration were used, there would be a high possibility to damage the accelerometer during the impact test.

The displacement response of the wall was not affected by the large amount of the acceleration at the top of the wall. If the acceleration signal is simplified to a harmonic sin function with maximum amplitude of 3810 g's and frequency of 8.0 kHz, the related maximum horizontal displacement would be:

$$a(t) = 3810 \text{ g} \sin(\omega t), \omega = 2 \pi f = 2 \pi (8,000 \text{ Hz}) = 50,265 \text{ rad/sec}$$

$$x(t) = -\frac{3810 \text{ g}}{\omega^2} \sin(\omega t) = -\frac{3810 \times 9.81 \times 1000}{50,265^2} \sin(\omega t) = -0.0148 \sin(\omega t) \text{ mm}$$

This displacement is very small compared to the maximum displacement response of 18 mm (less than 0.1 percent). This result shows that although the amount of the accelerations is very high but it does not affect the displacement response of the wall.

5.5.1.2 Impact Phase II

In this Phase, the impact test was performed 6 times with the 36 kg projectile mass and speed of 10 m/s. Each time, 1760 J energy was applied to the specimen but no visible damage on the specimen was observed. After the last impact test (6th impact) the projectile was damaged as shown in Fig. 5.9.

Figure 5.9: Damaged impact projectile with the mass of 36.0 kg

It was decided to replace the projectile with a higher mass to apply more impact energy to the specimen. The mass of the new projectile was 64.8 kg. The calculated speed of the projectile before the impact and the bounce back speed were 10 m/sec and 2 m/sec, respectively. The impact energy was 3,110 N.m which was almost twice that of the previous impact energy.

The speed measurement was achieved by calculating the distance of two points by using the number of pixels. The equivalency of each pixel to the actual distance was performed by a measuring pattern attached to the impact apparatus. Each pixel was equal to 1.052 mm of the actual distance. The projectile before the impact moved 19 pixel per frame which was 20 mm per frame, so the projectile speed was $20/0.002 = 10,000 \text{ mm/s} = 10 \text{ m/s}$. The bounce back speed of the projectile was 1.7 m/s. The amount of transferred energy from the 64kg projectile to the composite wall was 3146 J ($0.5*64.8*10^2 - 0.5*64.8*1.7^2 = 3146 \text{ J}$).

The maximum wall displacement at the top of the wall was 23 mm and natural period of the wall was 0.042 sec ($21 \text{ frame} = 21/500 = 0.042 \text{ sec}$) as per the data recorded by the high speed camera. The 5% ($0.042/0.04 = 1.05$) increase in the natural period of the wall results in almost 10% decrease in the stiffness of the wall from the impact loading in phase I to phase II. The wall reached to its maximum displacement (23 mm) in about 0.012 s. The maximum shear strain, $\gamma_{\max}$ and the maximum strain rate were 0.015 and 1.25 s^{-1} , respectively.

After the impact test in phase II, some cracks on the profiled panel along height and width of the wall were observed as shown in Fig. 5.10. The main cracks were at the upper part of the wall. Some local buckling of the profiled steel sheet at the lower part of wall was also observed as highlighted with red colour in Fig. 5.11.

(a)

(b)

Figure 5.10: Initial cracks on profiled concrete after the final impact test; (a) Along the wall height, (b) Along the wall width

Figure 5.11: Local buckling within the highlighted area after the final impact test (Phase II)

5.5.1.3 Post-impact Monotonic Loading to Failure

After the final impact test (Phase II), CSW-7 wall specimen was transferred to the structures lab to evaluate residual shear strength by applying monotonic loading to failure. The monotonic loading was applied as prescribed displacement at constant rate of 3 mm per minute until failure. The shear load versus displacement curve is shown in Fig. 5.12. The load increased with the increase of displacement up to a load of 153 kN (at 47 mm displacement) and then dropped to 142 kN possibly due to formation of major cracks (other than cracks due to impact loading) in the profiled concrete core and also due to deformation of the profiled steel sheets. The ultimate load capacity and stiffness of the wall were 229 kN (at 78 mm displacement) and 3.67 kN/mm, respectively. The stiffness of the wall specimen was about 9% lower than the equivalent CSW-5 specimen (4.0 kN/mm) tested purely under monotonic loading without impact. The 9% reduction in the wall stiffness also confirmed the results from the high speed camera which showed 5% increase in the wall natural period of vibration from the impact loading in phase I to phase II. The ultimate load capacity of the wall specimen was 8% higher than the equivalent CSW-5 specimen. This was due to the fact that the compressive strength of concrete (in CSW-7) was more than 9% higher than the CSW-5 specimen. It should be noted that the strength of concrete for specimen CSW-7 was measured after the first impact test but the final impact test which caused cracks in the concrete panel, was performed six months after the first impact test.

Figure 5.12: Load versus displacement in monotonic loading after impact test

The interesting point to note is that CSW-7 specimen after being subjected to six times of 1760 J and one time to 3146 J impact energy, it still had the full strength of a wall tested only under monotonic loading. The final failure of the wall associated with buckling of both profiled steel sheets and formation of the diagonal tension stress field as shown in Fig. 5.13.

Figure 5.13: The diagonal buckling shape of the profiled steel sheets from two sides

The crack pattern in the profiled concrete core after failure due to monotonic loading is shown in Fig. 5.14. The major differences in cracking pattern compared to the three other specimens tested without impact are: most cracks developed at upper part of the wall and the formation of major cracks at the trough are very close and parallel to the steel columns. These cracks may be associated with the cracks developed during the impact tests.

Figure 5.14: The crack pattern in the concrete panel after the test

5.5.2 Impact Test on Wall Specimen CSW-8

This test was performed in two phases, namely Phase I and Phase II. In Phase I, the speed of projectile was kept low (6.4 m/s) intentionally in order to get the natural period and maximum displacement at the top of the undamaged wall. In Phase II, the impact test was performed with the maximum speed of 10.2 m/s, the impact apparatus could produce.

During the test of the first specimen, as mentioned in chapter 3, the angles were mounted to the wall using superglue and during each impact test the angles got loose. To overcome this problem it was decided for the second specimen to mount the accelerometers at the top of the wall by welding two angles to the top beams and two angles to vertical stands which could be embedded to the concrete as shown in Fig. 5.15.

Figure 5.15: The position of each accelerometer at top of the wall

5.5.2.1 Impact Phase I

Analysis of wall displacement

The speed of the camera during the test was 500 frame per second (0.002 sec per frame) having a total capturing duration of 4 second (2000 total frame) with resolution of 1280 x 512 pixels. The speed measurement was achieved by calculating the distance of two points using the number of pixels. The equivalency of each pixel to the actual distance was performed by a measuring pattern attached to the impact apparatus. Each pixel was equal to 1.275 mm of the actual distance. The projectile before the impact moved 10 pixel per frame which was 12.75 mm per frame creating a projectile speed of $12.75/0.002 = 6,375 \text{ mm/s} = 6.4 \text{ m/s}$. The bounce back speed

of the projectile was 1.15 m/s. The amount of transferred energy from the 64.8 kg projectile to the composite wall was 1284 J.

The high speed camera also captured the maximum wall displacement (x_{max}) at the top and the natural period (T) of the wall. The recorded maximum wall displacement at the top was 14.3 mm and natural period of the wall was 0.04 sec (20 frame = 20/500 = 0.04 sec).

The maximum wall displacement at the top (x_{max}) calculated as per Eqs. 5.2 and 5.3 is as follows:

$$F_0 = 97,848 \text{ N}, m = 154 \text{ kg}, \omega = 157 \text{ rad/sec}$$

$$x_{max} = \frac{0.618 \times F_0}{m \omega^2} = \frac{0.618 \times 97,848}{154 \times 157^2} = 0.0159 \text{ m} = 15.9 \text{ mm}$$

The maximum displacement at the top of the wall from experimental data is 14.3 mm, which shows a good correlation with that obtained from analytical Eq. 5.3 within the linear behaviour of the wall.

Analysis of wall acceleration

The acceleration of the specimen at four points at the top of the wall was measured from data obtained from two accelerometers mounted on the top frame and two accelerometers embedded in concrete as described earlier and as shown in Fig. 5.16.

Figure 5.16: The name and location of each accelerometer at top of the wall

The shock wave amplitude and frequency of the wall at the locations of the accelerometers were captured. A Fast Fourier Transform (FFT) was performed on the signal from the accelerometers to have a frequency domain solution for the composite wall response. The acceleration data and the related frequencies content for the four locations are presented in Figs. 5.17 to 5.20.

At the top steel beams, the maximum acceleration amplitude was 1730 g's (steel-front) and 2375 g's (steel-rear). For concrete, at the top of the wall, the maximum acceleration amplitudes were 1787 g's (concrete-near) and 1435 g's (concrete-far). The accelerometer (concrete-near) was closer to the impact point and as a result exhibited higher amplitude (25 percent higher). From the acceleration signal of the "steel-rear", no dominant frequency content of the acceleration signal was observed. The frequency contents of the acceleration signals for the rest of the accelerometers were not consistent due to different location of the accelerometers.

Figure 5.17: The acceleration versus time and frequency domain (FFT) at steel-front

Figure 5.18: The acceleration versus time and frequency domain (FFT) at steel-rear

Figure 5.19: The acceleration versus time and frequency domain (FFT) at concrete-front

Figure 5.20: The acceleration versus time and frequency domain (FFT) at concrete-rear

5.5.2.2 Impact Phase II

The impact test in phase I was performed once with the 64.8 kg projectile having a speed of 6.4 m/s. The applied energy to the specimen was 1284 J but no visible damage on the specimen was observed. After the impact test, the projectile was damaged. A steel plate from back of the projectile was disconnected. The mass of the projectile was reduced to 62.3 kg. So the mass of the new projectile was reduced by 3.9%. The projectile speed calculated based on the number of pixels the projectile moves per frame as described before, so the projectile speed was $20.4/0.002 = 10,200 \text{ mm/s} = 10.2 \text{ m/s}$ with a bounce back speed of 2 m/s. The amount of transferred energy from the projectile to the composite wall was 3116 J.

The maximum wall displacement at the top was 23.1 mm and natural period of the wall was 0.042 sec (21 frame = $21/500 = 0.042 \text{ sec}$). The 5% increase in the natural period of the wall represents almost 10% decrease in the stiffness of the wall from the impact loading in phase I to phase II. The wall reached its maximum displacement (23.1 mm) at about 0.012 s. The maximum shear strain, $\gamma_{\max}$ and the maximum strain rate were 0.015 and 1.25 s^{-1} , respectively.

After the impact test, some cracks were observed on the profiled concrete core along the height and width of the wall as shown in Fig. 5.21. But no visible crack at the locations of the embedded accelerometers in concrete was observed (Fig. 5.22).

Figure 5.21: Initial cracks on profiled concrete after the final impact test; (a) Along the wall height, (b) Along the wall width

Figure 5.22: No cracks close to the embedded accelerometers in concrete

The acceleration data and the related frequencies for the four locations are presented in Figs. 5.23 to 5.26. At the top steel beams, the maximum acceleration amplitudes were 4052 g's (steel-front) and 4538 g's (steel-rear). At the top of the wall in concrete, the maximum acceleration amplitudes were 1954 g's (concrete-near) and 1676 g's (concrete-far). The accelerometer (concrete-near) was closer to the impact point and shown higher amplitude (17% higher). Compared to the results from Phase I, the average acceleration in the steel increased by 110% compared to just 13% increase in the concrete.

The reason could be the sequence of transferring impact energy from the end-plate structure to the composite wall panel. The projectile hit the end-plate structure, and then the energy was transferred to the steel frames through the 30 mm bolts and finally, to the composite wall panel from the frame through 12 mm bolts. From the signal of the “steel-rear” accelerometer, no dominant frequency content of the acceleration signal was observed. The frequency contents of the acceleration signals for the rest of the accelerometers were not consistent due to different location of the accelerometers.

Figure 5.23: The acceleration versus time and frequency domain (FFT) at steel-front

Figure 5.24: The acceleration versus time and frequency domain (FFT) at steel-rear

Figure 5.25: The acceleration versus time and frequency domain (FFT) at concrete-front

Figure 5.26: The acceleration versus time and frequency domain (FFT) at concrete-rear

5.5.2.3 Post-impact Monotonic Loading to Failure

After the final impact test (Phase II) the residual shear strength of CSW-8 specimen was evaluated by testing to failure under monotonic loading through prescribed displacement applied at a constant rate of 3 mm per minute. The load-displacement response is shown in Fig. 5.27. A drop in load was observed at 207 kN (at 58 mm displacement) possibly due to initiation of major continuous line of cracks formed due to the extension of the already existed cracks formed during impact loading or new cracks in the profiled concrete core as well as deformations in the profiled steel sheets. The ultimate load capacity and stiffness of the wall under monotonic loading were 219 kN (at 65 mm displacement) and 3.72 kN/mm, respectively. The stiffness of the specimen was reduced by 8% compared with the equivalent CSW-5 specimen (4.0 kN/m) tested solely under monotonic loading.

Figure 5.27: Load versus displacement in monotonic loading after impact test

The ultimate load capacity of the wall was 3% higher than the CSW-5 specimen which signified that no noticeable strength degradation occurred due to the application of about 3100 J impact energy on the wall.

The composite shear wall failed due to buckling of the profiled steel sheets at two sides of the wall accompanied by the formation of diagonal tension band and loss of profiled geometry as shown in Fig. 5.28 as well as major line of cracks in the concrete core.

Figure 5.28: The diagonal buckling shape of the profiled steel sheets from two sides

5.6 Conclusions and Discussions

At the initial stage of the research, it was intended to investigate the behaviour of profiled composite shear wall under impact loadings with two types of concrete namely Self-Consolidating Concrete (SCC) and Engineered Cementitious Composites (ECC). Due to high ductility of ECC, it was expected that the energy absorbing capacity of the walls could be improved compared to SCC. However, because of test set-up fabrications and instrumentation difficulties, and considering the time limit for the completion of the research, only two wall specimens with SCC were tested under impact loading. The summary of the test results from impact testing are presented in Table 5.2.

Table 5.2: Experimental test results for impact loading ($F_y = 354$ MPa and $F_u = 420$ MPa)

Specimen No.	Type of concrete	f' _c MPa	f' _t MPa	Type of loading	Weight of the projectile kg	Speed of the projectile (m/s) and applied energy (J)	Number of repeating impacts	Residual Ultimate load capacity (kN)
CSW-7	SCC	38.7	3.44	Impact (Phase 1)	36	10.0, 1760	6	—
				Impact (Phase 2)	64.8	10.0, 3146	1	—
				Post-impact Monotonic	—	—	—	229
CSW-8	SCC	34.3	2.98	Impact (Phase 1)	64.8	6.4, 1284	1	—
				Impact (Phase 2)	62.3	10.2, 3116	1	—
				Post-Impact Monotonic	—	—	—	219
f' _c : Compressive strength of concrete; f' _t : Tensile strength of concrete; F _y : Yield strength of profiled steel sheet; F _u : Ultimate strength of profiled steel sheet and SCC: Self-consolidating concrete								

It was observed that the shear strength of the composite wall did not reduce after the application of about 3100 J impact energy which is more than 6 times of impact energy capacity of the wall at the elastic range based on Eq. 5.4. This can be associated with the localized nature of the concrete damage in the proximity of the impact load as observed during the test. Such localized concrete damage is insufficient to degrade the overall integrity of the steel-concrete composite system in resisting monotonic loading besides degradation of wall stiffness. Stiffness degradation

of the wall after impact was 9% and 8% for specimens CSW-7 and CSW-8, respectively, compared to the equivalent CSW-5 specimen tested purely under monotonic loading without impact (Table 5.3).

The impact test for each specimen was performed in two phases, namely Phase I and Phase II. In Phase I, the amount of impact energy of the projectile was kept low intentionally to capture dynamic characteristics of the wall. In Phase II, the impact test was performed with maximum speed of the projectile, which the impact apparatus could produce. The behaviour of the wall in phase I of impact loading was almost elastic and difference in maximum displacement of the wall between experimental and theoretical was not significant. The change in natural period of the wall between phase I and phase II was about 5% which represent almost 10% stiffness degradation of the wall based on Single Degree of Freedom (SDOF) behaviour of the wall. This result was almost confirmed from the results of the post-impact monotonic loading of the wall (Table 5.3).

The differences in the cracking pattern in the impact loading compared to the post-impact monotonic loading were the development of cracks at upper part of the wall (close to the position of impact) and the formation of major cracks at the boundary columns at trough which may be an extension of the cracks developed during the impact testing.

Table 5.3: Experimental test results for impact loading

Specimen No.	Impact (Phase I)			Impact (Phase II) experimental		Post-impact monotonic loading	
	Experimental		Theoretical $X_{\max}(\text{mm})$	Natural period of the wall (s)	$X_{\max}$ (mm)	Wall stiffness (kN/mm)	Stiffness degradation compared to CSW-5 (4 kN/mm)
	Wall period (s)	$X_{\max}$ (mm)					
CSW-7	0.04	18.0	16.9	0.042	23.0	3.67	9%
CSW-8	0.04	14.3	15.9	0.042	23.1	3.72	8%
$X_{\max}$: Maximum displacement of the wall at top							

CHAPTER 6

DEVELOPMENT OF ANALYTICAL MODEL FOR THE SHEAR RESISTANCE OF COMPOSITE WALLS

6.1 General

The shear behaviour of a flat steel shear wall with boundary steel frame columns is similar to a web plate in a plate girder. In a thin web steel plate under shear loading initially principal compression and tension stresses develop in the plate and when the principal compression stresses reach a critical buckling stress, the plate buckles. After web plate buckling, the plate cannot effectively resist further compression, and resist the shear by developing a diagonal tension strip. Wagner (1929) described this behaviour through the tension field theory. If the thickness of the plate is relatively large or the plate has sufficient intermediate lateral supports along its height and width, the critical buckling stress value will be higher than the yield stress of the steel.

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. Concrete cracks when the principal tensile stresses exceed the tensile strength of the concrete and therefore, the concrete plate can resist shear by developing compression struts between the cracks and tensions in boundary steel frames. The *Compression Field Theory* (CFT) was developed by Collins and Mitchell to explain the behaviour and strength of the concrete panel. The general method for shear design in CSA A23.3 1984 was based on this theory which gave a good understanding of the behaviour of a cracked web, but did not consider the tension stresses in concrete between the cracks (Collins and Mitchell 1980, 1986). Vecchio and Collins developed the *Modified Compression Field Theory* or MCFT (Vecchio and Collins 1986) to include the effect of these stresses in CFT. The general method for shear design in CSA A23.3 1994 was based on this theory. In CSA A23.3 2004, the general method was developed by Bentz and Collins (Bentz et al. 2006) to simplify the method and make it significantly easier to use. The pioneers of the

simplified MCFT also suggested that the ACI 318 building code (ACI-318 2008) should adapt the simplified method to provide a more uniform level of safety for all types of the structural members (Collins et al. 2008a, b).

The shear capacity of the proposed composite shear wall (Fig. 6.1) should be derived from the shear capacity of profiled steel sheets, the shear capacity of concrete core and steel sheet-concrete interaction. In this chapter, the analytical equations (derived based on exiting models) for the shear capacity of both profiled steel sheet and concrete core is presented. The analytical shear capacity of the composite wall will also be compared with the experimental results.

Figure 6.1: Composite shear wall and a top view of the wall

6.2 Shear Resistance of the Profiled Steel Sheet Bounded by Frame

In general, most of theories for material failure are based on the assumption that failure occurs when some physical variable such as stress, strain, or energy reaches to a limiting value. The most famous three theories of failure for material are shown in Fig. 6.2 (Riley et al. 2006; Bauld 1982). For a brittle material, the maximum-normal-stress theory (solid lines) predicts material failure when the maximum normal stress in a specimen at any point reaches to its axial failure stress. The maximum-shear-stress or Tresca theory (dashed lines) predicts failure in a ductile material when the maximum shear stress at any point reaches the failure stress. For a ductile material like structural steel, von-Mises theory is one of the most well-known theories to predict failure. The major differences between the Tresca and von-Mises yield criteria are indicated in Fig. 6.3. The ellipse shown in the Fig. 6.3 is called Mises ellipse (Bauld 1982).

Figure 6.2: Three theories of material failure (Riley et al. 2006)

Figure 6.3: Comparison between Tresca and von-Mises material failure theories (Bauld 1982)

Based on von-Mises yield criterion for the plane stress condition, the relation between principal normal stresses (σ_{p1}, σ_{p2}) in two perpendicular directions for failure by yielding can be shown as:

$$\sigma_{p1}^2 - \sigma_{p1}\sigma_{p2} + \sigma_{p2}^2 = F_y^2 \quad (6.1)$$

where, F_y is yield strength of steel. The relation between normal stresses (σ_{11}, σ_{22}) and shear stress (τ_{12}) can also be shown as:

$$(\sigma_{11} - \sigma_{22})^2 + \sigma_{22}^2 + \sigma_{22}^2 + 6 \tau_{12}^2 = 2F_y^2 \quad (6.2)$$

In the Eq. 6.2 for pure shear condition, the normal stresses (σ_{11}, σ_{22}) can be assumed zero and in this way the maximum shear stress (τ_{max}) can be derived as:

$$\sigma_{11} = \sigma_{22} = 0 \rightarrow 6 \tau_{12}^2 = 2F_y^2 \rightarrow \tau_{12} = \tau_{max} = \frac{F_y}{\sqrt{3}} \quad (6.3)$$

The detail derivation of the von-Mises yield criterion equation, based on maximum distortion energy, can be found in any classic mechanics of materials books (Riley et al. 2006; Cook and Young 1985).

Based on the von-Mises yield criterion, the $F_y/\sqrt{3}$ is the maximum shear stress which the profiled steel sheet can resist. This is true when the elastic bulking of the sheet is prevented and the shear stress in the sheet can be reached to its maximum before failure.

The shear buckling capacity of profiled steel sheet depends on the type of connection of sheet to boundary frame, dimension of the sheet, sheet modulus of elasticity, thickness and shape of profile. The shear buckling capacity of a profiled steel sheet can be expressed by Eq. 2.6 in Chapter 2.

In order to increase the nominal shear capacity of the composite wall, the global buckling and elastic local buckling of the profiled steel sheets was prevented by using intermediate sheet-concrete interface connectors. The distance between the intermediate fasteners (along the height and width of composite shear wall) should be optimized to prevent elastic local buckling.

The interface fastener spacing of 128 mm c/c used in the experimental composite wall specimens were calculated based on Eq. 6.4 for a flat plate panel to prevent shear buckling (Timoshenko et al. 1961; Paik et al. 2003; Akesson 2007; Brush et al. 1975):

$$\tau_{cr} = k_v \frac{\pi^2 E_s}{12(1 - \nu_s^2)} \left(\frac{t_s}{b}\right)^2 \quad (6.4)$$

where, E_s is modulus of elasticity of steel, ν_s is Poisson's ratio of steel, can be taken as 0.3, and k_v is the shear buckling coefficient. For all edges simply supported, which is a conservative assumption for the steel sheet, the value of k_v will be $4\left(\frac{b}{a}\right)^2 + 5.34$ (CSA-S16 2009), a is larger dimension and b is shorter dimension of a rectangular panel, t_s is thickness of the steel sheet. For a flat part of the profiled steel sheet between two intermediate fastener as used in the experimental composite wall ($a = 128$ mm, $b = 27$ mm, and $t_s = 0.61$ mm), the value of k and τ_{cr} are found to be 5.52 and 509 MPa, respectively.

The maximum shear yield stress of the steel sheet with yield strength of 552 MPa (as used in the experimental tested walls), based on von-Mises yield criterion is $F_y/\sqrt{3} = 329$ MPa (Sabouri-Ghomi et al. 2005, Oehlers 1999), which is lower than the shear buckling stress ($\tau_{cr} = 509$ MPa). In this way, the profiled steel in the composite wall was designed to develop its yield shear stress before elastic local buckling.

The profiled steel sheets in the composite shear walls were able to reach their yield shear stress because of their contact with rigid concrete medium where the concrete panel provided bracing to the profiled steel sheet enhancing their buckling capacity and prevented steel sheet buckling prior to yielding. Therefore, it is possible to derive the shear resistance of the profiled steel sheet panel (V_s) based on shear yield stress as in Eq. 6.5:

$$V_s = \frac{F_y}{\sqrt{3}} t_s l_w \quad (6.5)$$

where, F_y is yield strength of profiled steel sheet, t_s is thickness of the profiled steel sheet, and l_w is width of the steel sheet (Fig. 6.1). The experimental test results of wall specimens confirmed the yielding of profiled steel sheet before buckling and justified the validity of Eq. 6.5.

6.3 Shear Resistance of the Profiled Concrete Core Panel

6.3.1 Concrete Strength under Biaxial Loadings

The strength and modes of failure for concrete subjected to biaxial stresses (σ_1 and σ_2) is presented in Fig. 6.4. The pear shaped envelop showing combination of the biaxial principal stresses causing failure of concrete can be divided to four zones depending on the value and direction of the principal stresses. The f'_c and f'_t are compressive and tensile strength of concrete, respectively. The failure pattern associated with each zone is shown in Fig. 6.5.

Figure 6.4: Concrete stresses at failure point under biaxial loading (MacGregor and Bartlett 1999)

Figure 6.5: Fracture patterns associated with each zone (MacGregor and Bartlett 1999)

In the shear loading condition associated with the development of the diagonal tension/compression, one principal stress is compressive and the other tensile, the concrete fails at lower stresses compared to its strength in tension when loaded uniaxially (Kupfer et al. 1969; Kupfer and Gerstle 1973). In such condition, concrete failure occurs due to tensile fractures on planes perpendicular to the principal tensile stress which is shown by regions A-B and A'-B' in Fig. 6.4 (MacGregor and Bartlett 1999).

6.3.2 Compressive Strength of Cracked Concrete

If concrete cracks occurs under a shear loading (biaxial compression-tension), the strength of concrete under compression parallel to the cracks is reduced. The free-body diagram of shaded area of zone 2 in Fig. 6.5 is re-presented in Fig. 6.6. The compressive stress acting on the top of the shaded portion has equilibrium with the shearing stresses along the cracks (τ) and compressive stresses on the bottom of the shaded portion.

Figure 6.6: Free-body diagram of the shaded area shown in Fig. 6.5 (Zone 2)

If the crack widths in the concrete are small enough the shearing stresses transfer sufficient load across the cracks, the compressive stress on the bottom of the shaded portion is not significantly larger than the compressive stress on the top. In this case, the strength is not affected by the cracks. If the crack widths increase, the ability to transfer shear along the cracks decreases. Vecchio and Collins proposed Eq. 6.6 based on experimental results on concrete panels under in-plane shear loading (Vecchio and Collins 1982; Collins et al. 1985) to determine maximum average principal compressive stress ($f_{2, \max}$) that concrete can be resisted:

$$f_{2,\max} = \frac{f'_c}{0.8 + 170 \epsilon_1} \leq f'_c \quad (6.6)$$

where, f'_c is cylinder compressive strength of concrete, and ϵ_1 is principal tensile strain on average in cracked concrete.

The maximum compressive strength of concrete decreases as the strain ϵ_1 increases. The Eq. 6.6 is plotted in Fig. 6.7a. The strain-stress relationship for cracked concrete is shown in Eq. 6.7 (Vecchio and Collins 1986; Vecchio and Collins 1993) and plotted in Fig. 6.7b.

$$f_2 = f_{2,\max} \left[2 \left(\frac{\epsilon_2}{\epsilon'_c} \right) - \left(\frac{\epsilon_2}{\epsilon'_c} \right)^2 \right] \quad (6.7)$$

where, f_2 is principal compressive stress on average, $f_{2,\max}$ is maximum average principal compressive stress that can be resisted by concrete, and ϵ_2 is principal compressive strain on average in cracked concrete, and ϵ'_c is apex strain in the compressive strain-stress curve.

Figure 6.7: The effect of transverse tensile strain on the compressive strength in cracked concrete (MacGregor and Bartlett 1999)

The apex strain (ϵ'_c) in the compressive strain-stress curve in Fig. 6.7b can be taken as 0.002. If ϵ_1 increases, the value of maximum compressive strength parallel to the cracks, $f_{2,\max}$, decreases and eventually the f_2 decreases.

6.3.3 Proposed Analytical Model for Shear Resistance of Profiled Concrete Core 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 in the concrete 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.

When the 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 (Collins et al. 1996):

$$f_1 = \frac{0.33\sqrt{f'_c}}{1 + \sqrt{500\epsilon_1}} \text{ MPa} \quad (6.8)$$

The nominator of Eq. 6.8 represents the cracking stress which was taken as $0.33\sqrt{f'_c}$ where f'_c is the concrete compressive strength. The coefficient of 500 at early stage of MCFT was 200 (Vecchio and Collins 1988) and was modified to 500 at a later time to represent more accurately the relation between f_1 and ϵ_1 (Bentz and Collins 2006). 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}). The stresses between and along inclined cracks is illustrated in Fig. 6.8. This shear stress across the cracked interface (v_{ci}) is a function of crack width (w) and the aggregate size (a_g). In Fig. 6.8, t_c is the average thickness of the equivalent plain concrete panel of profiled concrete core, d_v is effective shear depth of the concrete core, v_c is shear stress resistance provided by the concrete, and θ is angle of inclination of diagonal compressive stresses to the longitudinal axis of the wall. Based on the experimental data from Walraven on aggregate interlock (Walraven 1981), Vecchio and Collins proposed Eq. 6.9 for the v_{ci} (Collins et al. 1996):

$$v_{ci} \leq \frac{0.18\sqrt{f'_c}}{0.31 + \frac{24w}{(a_g + 16)}} \text{ MPa, mm} \quad (6.9)$$

Figure 6.8: Stresses between and along inclined cracks

If the size of aggregate reduces, the inclined shear stress between interfaces of cracks reduces and also if the crack width size increases, the (v_{ci}) value reduces.

The crack width, w , which was incorporated in Eq. 6.9 is the product of the mean spacing of the inclined cracks (s_θ) and the principal tensile strain (ϵ_1) as shown in Eq. 6.10:

$$w = s_\theta \epsilon_1 \text{ (mm)} \quad (6.10)$$

The diagonal crack spacing (s_θ) can be derived by Eq. 6.11 as follows:

$$s_\theta = \frac{s_{ze}}{\sin \theta}, \quad s_{ze} = s_z \frac{35}{15 + a_g} \geq 0.85 s_z \text{ (mm)} \quad (6.11)$$

The crack spacing parameter (s_z) can be taken as 300 mm in the Eq. 6.11. 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.

When crack begins to slip, the relation between horizontal shear stress, shear stress across the cracked interface and average tensile stress in concrete can be derived as (Collins et al. 1996):

$$v_c = v_{ci} = f_1 \cot \theta \quad (6.12)$$

In order to calculate the shear stress (v_c) from Eq. 6.12, as average tensile stress in concrete (f_1) is a function of the ε_1 , the value of ε_1 should be calculated. From Mohr's circle for strain, the relationship between ε_1 , ε_2 , and ε_y (Fig. 6.8) can be derived as follows (MacGregor and Bartlett 1999):

$$\varepsilon_1 = \varepsilon_y + (\varepsilon_y - \varepsilon_2) \cot^2 \theta \quad (6.13)$$

In the Eq. 6.13, the longitudinal strain at mid-depth of the concrete core wall (ε_y) can be derived as:

$$\varepsilon_y = \frac{M/d_v + 0.5N + V}{2A_c E_s} \quad (6.14)$$

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 (Fig. 6.1), E_s is modulus of elasticity of steel columns, and A_c is average cross-sectional area of steel columns.

The development of Eq. 6.14 for the longitudinal strain is illustrated in Fig. 6.9. 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 (Bentz and Collins 2006). The term $0.5 V \cot \theta$ (in Fig. 6.9) was replaced by V in the nominator of Eq. 6.14 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). This simplification is one of the main differences between MCFT and simplified MCFT which makes the calculation of the shear stress easier compared to MCFT. The absolute value of the ε_y should not be less than $2E-4$ (Bentz and Collins 2006).

Figure 6.9: Development of an equation for the longitudinal strain

The ε_2 , which is incorporated in Eq. 6.13 to find ε_1 value, can be derived by using Eq. 6.15:

$$\varepsilon_2 = \varepsilon'_c \left(1 - \sqrt{\left(1 - \frac{f_2}{f_{2,max}} \right)} \right), \quad \varepsilon'_c = -0.002 \quad (6.15)$$

This equation is basically a rearranged form of Eq. 6.7 to calculate principal compressive strain on average in cracked concrete (ε_2). The principal compressive stress on average (f_2) in Eq. 6.15 can be derived using Eq. 6.16 (Bentz et al. 2006):

$$f_2 = f_1 \cot^2 \theta \rightarrow f_2 = \frac{0.33 \sqrt{f'_c} \cot^2 \theta}{1 + \sqrt{500 \varepsilon_1}} \quad (\text{MPa}) \quad (6.16)$$

where, f_1 is principal tensile stress on average, and θ is angle of inclination of diagonal compressive stresses to the longitudinal axis of the wall (Fig. 6.9).

The shear capacity of the concrete panel can be limited based on the follow possible modes of failure:

- 1) Crushing of the struts between cracks due to the compressive stress,
- 2) When the average tensile stress transverse to the cracks reaches to its limit,
- 3) When slip occurs along the cracks.

The first failure mode happens when the principal compressive stress, exceeds the $f_{2, \max}$ from Eq. 6.6. Due to the fact that the compressive strength of concrete is much higher than its tensile strength, this mode of failure is very rare and will not be discussed in this chapter.

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 (Davies and Fisher 1979; Easterling and Porter 1994a, b; Hossain and Wright 1998b).

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 shear stress resistance provided by the cracked concrete (v_c) can be expressed by the product of a coefficient (which is called beta, β , shear retention factor) and the square root of compressive strength of concrete (f'_c):

$$v_c = v_{ci} = \beta \sqrt{f'_c} \quad (6.17)$$

The value of β for the second and third possible modes of failure can be derived by substituting Eq. 6.8 to Eq. 6.12 and using Eq. 6.9 and Eq. 6.17 as follows:

$$\beta = \frac{v_{ci}}{\sqrt{f'_c}} = \frac{f_1 \cot \theta}{\sqrt{f'_c}} = \frac{0.33 \cot \theta}{1 + \sqrt{500 \epsilon_1}} \leq \frac{0.18}{0.31 + \frac{24 w}{(a_g + 16)}} \text{ (MPa, mm)} \quad (6.18)$$

The shear resistance of the profiled concrete core panel in the composite wall (V_c) based on an equivalent plain concrete panel can be derived using Eq. 6.19:

$$V_c = \beta \sqrt{f'_c} t_c d_v \text{ (MPa, mm, N)} \quad (6.19)$$

where, t_c is the average thickness of the equivalent plain concrete panel of profiled concrete core and d_v is effective shear depth of the concrete core. In the experimental composite walls, t_c and d_v can be taken as 44 mm and 640 mm, respectively (Fig. 6.1).

6.3.3.1 Calculation of β Based on General Method in CSA A23.3-2004

In order to simplify Eq. 6.9, it is necessary to determine the diagonal crack width “ w ” in the concrete panel at shear failure. Bentz and Collins (2006) proposed a simple linear equation for calculation of crack width, when s_{ze} is equal to 300 mm, as follows:

$$w = 0.2 + 1000 \epsilon_y \text{ (mm)} \quad (6.20)$$

By substituting Eq. 6.20 to Eq. 6.9 for $s_{ze} = 300$ mm and $a_g = 20$ mm, β can be calculated:

$$\beta = \frac{0.18}{0.31 + \frac{24 w}{(a_g + 16)}} = \frac{0.18}{0.31 + \frac{24 (0.2 + 1000 \epsilon_y)}{(20 + 16)}} = \frac{0.4}{1 + 1500 \epsilon_y} \quad (6.21)$$

In order to take into account the size effect, Eq. 6.21 was multiplied by the size effect correction:

$$\beta = \frac{0.4}{1 + 1500 \epsilon_y} \times \frac{1300}{(1000 + s_{ze})} \text{ (mm)} \quad (6.22)$$

It should be noted that if the aggregate size is 20 mm or more, no size correction factor is necessary to apply.

The ϵ_y can be calculated from Eq. 6.14 utilizing the factored loads:

$$\varepsilon_y = \frac{M_f/d_v + 0.5N_f + V_f}{2A_c E_s} \quad (6.23)$$

6.4 Analytical Model for the Shear Resistance of the Composite Shear Walls

Shear resistance of the wall can be derived by adding the shear resistance of the profiled steel sheet (Eq. 6.5) and the shear resistance of the concrete panel (Eq. 6.19) as follows:

$$V_r = 2V_s + V_c = 2 \frac{f_y}{\sqrt{3}} t_s l_w + \beta \sqrt{f'_c} t_c d_v \quad (\text{MPa, mm, N}) \quad (6.24)$$

where, F_y is the yield strength of profiled steel sheet, t_s is thickness of the profiled steel sheet, and l_w is width of the steel sheet, f'_c is cylinder compressive strength of concrete panel, t_c is equivalent thickness of the profiled concrete panel, and d_v is effective shear depth as shown in Fig. 6.1.

Shear resistance of the wall for design purposes can be derived by applying resistance factor for structural steel, $\phi_s = 0.90$ (CSA-S16 2009) and concrete, $\phi_c = 0.65$ (CSA-A23.3 2004) as follows:

$$V_{r, \text{ design }} = 2 \phi_s \frac{f_y}{\sqrt{3}} t_s l_w + \phi_c \beta \sqrt{f'_c} t_c d_v \quad (\text{MPa, mm, N}) \quad (6.25)$$

6.5 Shear Resistance of Composite Walls - Comparison of Experimental and Analytical Model: Specimens 1 and 5 (CSW-1 and CSW-5)

Two experimental walls (specimen CSW-1 and specimen CSW-5) are used to validate the performance of analytical model (Eq. 6.24) in predicting the shear resistance of composite walls.

In the simplified MCFT, the value of β can be calculated based on several trial and error procedure when the values ε_y , crack spacing, aggregate size, concrete strength and angle of cracks is known for a specimen. For the experimental walls 1 and 5, these values are known from the tests. The trial and error procedure can be started by adding a nominal value for strain to the ε_y to obtain the first estimate for the ε_1 . Bentz and Collins suggested a nominal value of 0.0005 for adding to the ε_y (Bentz and Collins 2006). By using Eq. 6.16, Eq. 6.6, Eq. 6.15, and Eq. 6.13, the new value for the ε_1 can be achieved. If the difference between the new ε_1 and the old value

of ε_1 is not significant the procedure can be stopped, otherwise should be continued till the difference is negligibly small. Once the value of β is known, the shear resistance (V_r) of the wall is then calculated based on Eq. 6.24. The following sections illustrate the calculation of shear resistance of composite wall CSW-1 and CSW-5.

6.5.1 Shear Resistance of Composite Wall Specimen CSW-1

Material, experimental and geometric parameters: $F_y = 552$ MPa, $f'_c = 48$ MPa, $\theta = 35^\circ$,

$A_c = 2 (30 \times 80) = 4,800$ mm², Crack spacing $s_0 = 300$ mm, $a_g = 10$ mm

Calculation of ε_y :

$$\varepsilon_y = \frac{M/d_v + 0.5N + V}{2A_c E_s} = \frac{294,000 \times 1,536/640 + 294,000}{2 \times 4,800 \times 200,000} = 0.000521$$

Trial and error procedure to obtain ε_1 :

	Estimate of Principal Tensile Strain ε_1	Principal Compressive Stress f_2 (MPa)	Max. Compressive Stress $f_{2, \max}$ (MPa)	Principal Compressive Strain ε_2	New Estimate of ε_1
Equation	$\sim \varepsilon_y + 0.0005$ or new estimate	Eq. (6.16)	Eq. (6.6)	Eq. (6.15)	Eq. (6.13)
Iteration 1	0.001021	2.720	48	-5.75×10^{-5}	0.001701
Iteration 2	0.001701	2.426	44.07	-5.58×10^{-5}	0.001698
Iteration 3	0.001698	2.427	44.09	-5.58×10^{-5}	0.001698

Converged value of $\varepsilon_1 = 0.001698 > 0.0002$ OK

Crack width calculation:

$$w = s_0 \times \varepsilon_1 = 300 \times 0.001698 = 0.5094 \text{ mm}$$

Calculation of β :

$$\beta = \frac{0.33 \cot \theta}{1 + \sqrt{500\varepsilon_1}} = \frac{0.33 \cot 35^\circ}{1 + \sqrt{500 \times 0.001698}} = 0.245 \leq \frac{0.18}{0.31 + \frac{24 w}{(a_g + 16)}} = \frac{0.18}{0.31 + \frac{24 \times 0.5094}{(10 + 16)}} = 0.231$$

$$\rightarrow \beta = 0.231$$

Calculation of shear resistance of the wall based on Eq. 6.24:

$$V_r = V_c + 2 V_s = \beta \sqrt{f'_c} t_c d_v + 2 \frac{F_y}{\sqrt{3}} t_s l_w = 0.231 \times \sqrt{48} \times 44 \times 640 + 2 \times \frac{552}{\sqrt{3}} \times 0.61 \times 720$$

$$= 45,068 + 2 \times 139,972 = 325,012 \text{ N} = 325.0 \text{ kN}$$

The ultimate shear resistance of specimen CSW-1 under monotonic loading was not achieved due to the final test setup modifications which required disassembling and strengthening of steel columns. Shear resistance of 294 kN was obtained from wall CSW-2 under cyclic loading which is 10% less than the analytical load of 325 kN.

6. 5.2 Shear Resistance of Composite Wall Specimen CSW-5

Experimental shear resistance = 212 kN,

Material, experimental and geometric parameters: $F_y = 354 \text{ MPa}$, $f'_c = 36 \text{ MPa}$, $\theta = 31^\circ$,

$A_c = 2 (30 \times 80) = 4,800 \text{ mm}^2$, crack spacing $s_\theta = 300 \text{ mm}$, $a_g = 10 \text{ mm}$

Calculation of ε_y :

$$\varepsilon_y = \frac{M/d_v + 0.5N + V}{2A_c E_s} = \frac{212,000 \times 1,536/640 + 212,000}{2 \times 4,800 \times 200,000} = 0.000375$$

Trial and error procedure to obtain ε_1 :

	Estimate of Principal Tensile Strain ε_1	Principal Compressive Stress f_2 (MPa)	Max. Compressive Stress $f_{2, \max}$ (MPa)	Principal Compressive Strain ε_2	New Estimate of ε_1
Equation	$\sim \varepsilon_y + 0.0005$ or new estimate	Eq. (6.16)	Eq. (6.6)	Eq. (6.15)	Eq. (6.13)
Iteration 1	0.000875	3.301	36	-9.39×10^{-5}	0.001674
Iteration 2	0.001674	2.864	33.19	-8.82×10^{-5}	0.001658
Iteration 3	0.001658	2.871	33.28	-8.82×10^{-5}	0.001658

Converged value of $\varepsilon_1 = 0.001658 > 0.0002$ OK

Crack width calculation:

$$w = s_\theta \times \varepsilon_1 = 300 \times 0.001658 = 0.4974 \text{ mm}$$

Calculation of β :

$$\beta = \frac{0.33 \cot \theta}{1 + \sqrt{500 \epsilon_1}} = \frac{0.33 \cot 31^\circ}{1 + \sqrt{500 \times 0.001658}} = 0.287 \leq \frac{0.18}{0.31 + \frac{24 w}{(a_g + 16)}} = \frac{0.18}{0.31 + \frac{24 \times 0.4974}{(10 + 16)}} = 0.234$$

$$\rightarrow \beta = 0.234$$

Calculation of shear resistance of the wall based on Eq. 6.24:

$$\begin{aligned} V_r &= V_c + 2 V_s = \beta \sqrt{f'_c} t_c d_v + 2 \frac{F_y}{\sqrt{3}} t_s l_w = 0.234 \times \sqrt{36} \times 44 \times 640 + 2 \times \frac{534}{\sqrt{3}} \times 0.61 \times 720 \\ &= 39,537 + 2 \times 89,765 = 219,067 \text{ N} = 219.1 \text{ kN} \end{aligned}$$

The 219.1 kN from the Eq. 6.25 is just 3% more than experimental shear resistance of 212 kN.

6.5.3 Comparison between Analytical and Experimental Shear Resistance

Table 6.1 compares the shear resistance of composite walls obtained from experiments and analytical model.

Table 6.1: Comparison of analytical and experimental shear resistance of composite walls

Composite Wall	Shear Resistance (kN)				Ratio
	Analytical			Experimental	
	Profiled concrete core (V_c)	Profiled steel sheet (V_s)	Composite wall (V)	Composite wall (V)	Expt./Analy.
CSW-1	45.1	140	325	294	0.91
CSW-5	39.5	89.8	219	212	0.97

Analytical shear resistances of CSW-1 and CSW-5 are found to be 10% and 3% lower than those from experiments, respectively. However, it should be noted that the shear resistance of 294 kN was for wall CSW-2 which was tested under cyclic loading. It is expected that the wall shear resistance under monotonic loading should be higher than 294 kN reducing the difference between analytical and experimental values. However, the ratio experimental to analytical values ranges between 0.91 and 0.97 showing a good agreement.

6.6 Summary

In this chapter, analytical models for the shear resistance of the profiled steel sheet, profiled concrete core and composite wall are developed based on existing theoretical models. For the steel sheet, the model is developed based on the assumption of failure due to shear yielding of steel (based on von-Mises failure criterion, the shear stress in steel can reach up to $F_y/\sqrt{3}$ where F_y is steel yield strength of steel) before buckling as a result of increased steel buckling capacity due to contact with concrete, the presence of sheet-concrete interface connections and composite confinement.

The shear resistance of the profiled concrete core was derived based on the simplified modified compression field theory (MCFT). The analytical model for the shear resistance of the composite wall is 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 for the two composite walls tested in this study was found to be less than 10%. The proposed analytical model can therefore, be used for the prediction of shear resistance of composite walls with reasonable accuracy.

CHAPTER 7

FINITE ELEMENT MODELING OF COMPOSITE WALL UNDER MONOTONIC AND CYCLIC LOADING

7.1 Introduction

Finite element (FE) modeling of composite wall was performed to develop numerical models based on experimental program. One of the objectives of this study is to develop reliable finite element models which can simulate the behaviour of the composite shear wall. In this research, three models were developed using ABAQUS/CAE (Dassault Systèmes Simulia Corp. 2008) designated as basic model, sophisticated model and practical model.

This chapter describes various modelling features of ABAQUS/CAE, development/performance evaluation of first two numerical/FE models and the third model development and performance evaluation in more details. Based on the third model (practical model), a parametric study was performed to investigate the behaviour of the wall with variable steel strength, concrete strength and location/orientation of intermediate steel-concrete connectors.

7.2 General Information about ABAQUS/CAE

The finite element modeling was conducted using ABAQUS/CAE Version 6.8–3. ABAQUS/CAE provides an interactive and graphical environment that allows for easy modeling and generating complex geometry into meshable regions. Material properties, loads and boundary conditions can be assigned to the geometry. This software is well suited for highly nonlinear engineering problems and its visualization module is an efficient option to interpret the results.

7.2.1 Selection of Finite Element Analysis Technique

ABAQUS/CAE consists of two main analysis products namely ABAQUS/Standard and ABAQUS/Explicit. ABAQUS/Standard is a general-purpose analysis software which solves a system of equations implicitly at each solution increment. It can solve a wide range of linear and nonlinear problems involving static, dynamic, thermal, and electrical response of components. For the implicit method, equilibrium is achieved through an iterative procedure.

ABAQUS/Explicit is a special-purpose analysis module which uses an explicit dynamic finite element formulation. It uses central difference method to integrate the equations of motion explicitly through time. The explicit time integration method uses the kinematic conditions at one increment to calculate the kinematic conditions at the next increment. It is suitable for modeling brief, transient dynamic events, such as impact problems, and is also very efficient for highly nonlinear problems involving contact conditions. The explicit method requires the use of small time increments. Since there are no simultaneous equations to solve, each increment is inexpensive.

Table 7.1 shows the difference between ABAQUS/Standard and ABAQUS/Explicit which was excerpted from the ABAQUS documentations (Dassault Systèmes Simulia Corp. 2008).

Table 7.1: Comparison between ABAQUS/Standard and ABAQUS/Explicit (Dassault Systèmes Simulia Corp. 2008)

Parameter	ABAQUS/Standard	ABAQUS/Explicit
Element library	Offers an extensive element library	Offers an extensive library of elements well suited for explicit analyses. The elements available are a subset of those available in ABAQUS/Standard
Analysis procedures	General and linear perturbation procedures are available	General procedures are available
Material models	Offers a wide range of material models	Similar to those available in ABAQUS/Standard; a notable difference is that failure material models are allowed

Contact formulation	Has a robust capability for solving contact problems	Has a robust contact functionality that readily solves even the most complex contact simulations
Solution technique	Uses a stiffness-based solution technique that is unconditionally stable	Uses an explicit integration solution technique that is conditionally stable
Disk space and memory	Due to the large numbers of iterations possible in an increment, disk space and memory usage can be large	Disk space and memory usage is typically much smaller than that for ABAQUS/Standard

For analyzing a nonlinear problem with a large number of degrees of freedoms (DOFs) such as composite walls, using ABAQUS/Standard will be impractical and time consuming. Since, for each iteration, ABAQUS/Standard requires solution of a large set of linear equations through iteration process. On the other hand, ABAQUS/Explicit determines the solution without iterating by explicitly advancing the kinematic state from the previous increment. Although for a large number of the DOFs the analysis may require a large number of time increments using the explicit method, the analysis can be more efficient in ABAQUS/Explicit compared to ABAQUS/Standard. The explicit method has great cost savings over the implicit method as the model size increases. Figure 7.1 shows the variation of the computational cost versus number of DOF for ABAQUS/Explicit and ABAQUS/Standard. The relation between the computational costs and the number of DOF in ABAQUS/Explicit is almost linear.

Figure 7.1: Cost versus model size in using the explicit and implicit methods (Dassault Systèmes Simulia Corp. 2008)

The four main reasons that ABAQUS/Explicit is used in this study are as follows:

- 1) Concrete degradation can cause severe convergence difficulties in ABAQUS/Standard (implicit) analysis program but in the ABAQUS/Explicit can perform well,
- 2) Reduce the computational cost (time) for the composite wall model,
- 3) Contact conditions are formulated more easily using ABAQUS/Explicit, and
- 4) Analyzing high-speed dynamic events such as impact can be extremely costly using ABAQUS/Standard. Because the load is applied rapidly and the response of the structure changes rapidly. Accurate tracking of stress waves through the composite wall is important for capturing the dynamic response. The stress waves are associated with the highest frequencies of the model and accurate solution requires many small time increments. ABAQUS/Explicit is a better option for a wave propagation analysis.

7.2.2 Conditional Stability and Time Increment in the Explicit Method

The maximum time increment is a critical factor for the performance and stability of the ABAQUS/Explicit. If the time increment is larger than a specified value, a numerical instability may happen. The maximum time increment used by the ABAQUS/Explicit solver has a significant effect on the reliability and accuracy of the results. The material properties and the finite element mesh size can change the critical time increment. The wave speed depends on both density and stiffness of the material. The higher material density results in a reduction of the wave speed and an increase in the critical time increment. On the other hand, the higher material stiffness results in an increase of the wave speed and a reduction in the critical time increment. The wave speed is constant for a specific material when the analysis is in the linear portion and, therefore, the critical time depends only on the smallest element size in the finite element domain.

ABAQUS/Explicit automatically controls the time increment size throughout the analysis to maintain stability. ABAQUS/Explicit calculates the stable time increment based on the highest frequency in the model which can be estimated by individual elements. Based on the element-by-element estimate, the stability time limit (Δt_{stable}) can be defined using the shortest element length (L_e) and the wave speed of the material (C_d) (Dassault Systèmes Simulia Corp. 2008):

$$\Delta t_{stable} = \frac{L_e}{C_d} \quad (7.1)$$

The wave speed is a property of the material. For a linear elastic material with a Poisson's ratio of zero the wave speed can be derived from the following equation:

$$C_d = \sqrt{\frac{E}{\rho}} \quad (7.2)$$

where E is Young's modulus and ρ is the mass density.

In a nonlinear material, such as a metal with plasticity or concrete, the wave speed changes as the material yields and the stiffness of the material changes. In the nonlinear portion of the analysis, the modulus of elasticity decreases which reduces the wave speed and increases the critical time increment. ABAQUS/Explicit monitors the effective wave speeds in the model throughout the analysis, and the current material state in each element is used for stability estimates.

The shortest element dimension has direct effect on the stability time limit (Eq. 7.2) and it is recommended to keep the element size as large as possible. On the other hand, for accurate analyses a fine mesh is needed. The best approach for meshing a FE model is to have a mesh which is as uniform as possible and also to make a balance in size of mesh that will not compromise the analysis accuracy or may result in very small stable time limit.

7.2.3 Quasi-static Analysis with ABAQUS/Explicit

The explicit solution method is a true dynamic procedure which was originally developed to model high-speed impact events when inertia playing a dominant role in the solution. Applying the explicit dynamic procedure to quasi-static problems needs some special considerations. The static or long-time solution is often computationally impractical to analyze the simulation in its natural time scale, which would require an excessive number of small time increments. In order to have an economical solution, the event should be accelerated. The goal is to model the process in the shortest time period in which inertial forces remain insignificant. In this study, the following three techniques were used to simulate the quasi-static condition in the monotonic loading:

- 1) **Smooth amplitude curves:** For accuracy and efficiency quasi-static analyses require the application of displacement that is as smooth as possible. Sudden movements cause stress waves, which can induce noisy or inaccurate solutions. Applying the displacement in the smoothest possible manner requires that the displacement changes only a small amount from one increment to the next. If the displacement is smooth, it follows that the changes in velocity and acceleration are also smooth. ABAQUS/CAE has a built-in smooth step amplitude curve that automatically creates smooth displacement amplitude, which is very efficient way to prescribe displacements in a quasi-static analysis.
- 2) **Loading rates:** It is safe to assume that performing an analysis in the natural time (which is the actual time taken for a physical process) for a quasi-static process will produce accurate static results. The prescribed displacement rate should be increased in the way that the same physical event occurs in less time as long as the solution remains nearly the same as the true static solution and dynamic effects remain insignificant. The time step used in the developed composite wall model was one second. It is worth mentioning that a series of analyses at varying displacement rates was performed to determine an acceptable displacement rate.
- 3) **Mass scaling:** This option enables an analysis to be performed economically without increasing significantly the prescribed displacement rate. As discussed before the stability limit for a model is the minimum stable time increment of all elements and the stable time increment is related to the material density. Mass scaling artificially increases the material density and therefore increases the stable time increment. When the global stability limit is increased, fewer increments are required to perform the same analysis.

One of the best ways to evaluate whether or not a simulation is producing an appropriate quasi-static is the energy method. The energy history for a quasi-static simulation is shown in Fig 7.2. If a simulation is quasi-static, the work applied by the external forces (E_W) is nearly equal to the internal energy (E_I) (both elastic and plastic strain energy) of the system. Other types of energy such as the kinetic energy (E_{KE}), the energy absorbed by frictional dissipation (E_{FD}), the energy absorbed by viscous dissipation (E_V) and the total energy in the system (E_{TOT}) in the composite wall can be considered small.

The energy absorbed by viscous dissipation is not small in visco-elastic materials, discrete dashpots, or when material damping is used. Usually the kinetic energy of the deforming material should not exceed a small fraction (typically 5% to 10%) of its internal energy (Dassault Systèmes Simulia Corp. 2008).

Figure 7.2: Energy history in quasi-static simulation (Dassault Systèmes Simulia Corp. 2008)

7.3 Material Properties

7.3.1 Concrete Model

There are three different models to simulate the concrete behaviour in ABAQUS/CAE Version 6.8–3 namely; the smeared cracking model, brittle cracking model and the concrete damaged plasticity model (Dassault Systèmes Simulia Corp. 2008). Each model has its limitations and is good for certain types of structures and loading conditions.

The Concrete Damage Plasticity (CDP) model (Lubliner et al. 1989, Lee et al. 1998, and Li et al. 2005) is the most comprehensive model and was used in the composite wall simulation of the current study. The CDP is a continuum, plasticity-based damage model for concrete. It assumes two concrete failure mechanisms: tensile cracking and compressive crushing of the material. The CDP model in ABAQUS/CAE is designed for applications in which concrete is subjected to monotonic, cyclic, and/or dynamic loading under low confining pressures for all types of structures (beams, trusses, shells, and solids). The model uses concepts of isotropic damaged elasticity in combination with isotropic tensile and compressive plasticity to represent the inelastic behaviour of concrete. The user can control the stiffness recovery effects during cyclic load reversals. The concrete behaviour in tension, compression and cyclic loading was mostly

excerpted from the ABAQUS documentations (Dassault Systèmes Simulia Corp. 2008) and presented in the following sections.

Fig. 7.3 shows the concrete behaviour in uniaxial loading in tension. The stress-strain response follows a linear elastic relationship until the failure stress (σ_{t0}). Beyond the failure stress, there will be a softening stress-strain response. When the concrete specimen is unloaded from any point on the strain softening branch of the stress-strain curve, the unloading response is weakened and the elastic stiffness of the material is degraded or damaged. The degradation of the elastic stiffness is characterized by damage variable in tension (d_t), which can take value from zero, representing the undamaged material, to one, representing the total loss of strength.

Figure 7.3: Response of concrete to uniaxial loading in tension (Dassault Systèmes Simulia Corp. 2008)

The cracking strain ($\tilde{\varepsilon}_t^{ck}$) is defined as the total strain (ε_t) minus the elastic strain corresponding to the undamaged material; which is:

$$\tilde{\varepsilon}_t^{ck} = \varepsilon_t - \sigma_t/E_0 \quad (7.3)$$

where σ_t is tensile stress and E_0 is the initial elastic stiffness of concrete. Tension stiffening data are given in terms of the cracking strain, $\tilde{\varepsilon}_t^{ck}$. The unloading data are provided to ABAQUS in terms of tensile damage curves (d_t versus $\tilde{\varepsilon}_t^{ck}$). ABAQUS automatically converts the cracking strain values to plastic strain values ($\tilde{\varepsilon}_t^{pl}$) using the following relationship:

$$\tilde{\varepsilon}_t^{pl} = \tilde{\varepsilon}_t^{ck} - \frac{d_t}{(1-d_t)} \frac{\sigma_t}{E_0} \quad (7.4)$$

7.3.1.2 Behaviour of Concrete in Compression

Fig. 7.4 shows the concrete behaviour in uniaxial loading in compression. Under uniaxial compression the response is linear until the value of initial yield, σ_{c0} is reached. In the plastic regime the response is typically characterized by stress hardening followed by strain softening beyond the ultimate stress, σ_{cu} .

Figure 7.4: Response of concrete to uniaxial loading in compression
(Dassault Systèmes Simulia Corp. 2008)

When the concrete specimen is unloaded from any point on the strain softening branch of the stress-strain curves, the unloading response is weakened and the elastic stiffness of the material is degraded or damaged. The degradation of the elastic stiffness is characterized by damage variable, d_c , which can take values from zero, representing the undamaged material, to one, representing the total loss of strength.

Compressive stress data are provided as a tabular function of inelastic or crushing strain ($\tilde{\varepsilon}_c^{in}$). Positive (absolute) values should be given for the compressive stress and strain. The stress-strain curve can be defined beyond the ultimate stress, into the strain-softening regime.

Hardening data are given in terms of an inelastic strain, $\tilde{\varepsilon}_c^{in}$, instead of plastic strain, $\tilde{\varepsilon}_c^{pl}$. The compressive inelastic strain is defined as the total strain minus the elastic strain corresponding to the undamaged material,

$$\tilde{\varepsilon}_c^{in} = \varepsilon_c - \sigma_c/E_0 \quad (7.5)$$

where E_0 is the initial elastic stiffness of the concrete. Unloading data are provided to ABAQUS in terms of compressive damage curves (d_c versus $\tilde{\varepsilon}_c^{in}$). ABAQUS automatically converts the inelastic strain values to plastic strain values ($\tilde{\varepsilon}_c^{pl}$) using the following relationship:

$$\tilde{\varepsilon}_c^{pl} = \tilde{\varepsilon}_c^{in} - \frac{d_c}{(1 - d_c)} \frac{\sigma_c}{E_0} \quad (7.6)$$

7.3.1.3 Behaviour of Concrete in Cyclic Loading

The degradation mechanisms of concrete under uniaxial cyclic loading are quite complex, involving the opening and closing of previously formed micro-cracks and also their interaction. It is observed experimentally that there is some recovery of the elastic stiffness as the load changes sign during a uniaxial cyclic test. The stiffness recovery effect is an important aspect of the concrete behaviour under cyclic loading. The effect is usually more obvious as the load changes from tension to compression, causing tensile cracks to close, which results in the recovery of the compressive stiffness.

The concrete damaged plasticity model assumes that the reduction of the elastic modulus is given in terms of a scalar degradation variable (d) as:

$$E = (1 - d)E_0 \quad (7.7)$$

The degradation or damage variable in the strain-softening part can be calculated from the following equations (Jankowiak et al. 2005):

$$d_c = \frac{\sigma_{cu} - \sigma_c}{\sigma_{cu}} \quad (7.8)$$

where d_c is damage variable in compression, σ_{cu} is ultimate compressive stress (Fig. 7.4) and σ_c is compressive stress.

and

$$d_t = \frac{\sigma_{t0} - \sigma_t}{\sigma_{t0}} \quad (7.9)$$

where d_t is damage variable in tension, σ_{t0} is tensile stress at failure (Fig. 7.3) and σ_t is tensile stress.

The effect of stiffness recovery in compression (w_c) and in tension (w_t) on the behaviour of the concrete material is shown in Fig. 7.5. The value of w_c and w_t are between zero and one, and they control the recovery of the compressive and tensile stiffness upon load reversal. ABAQUS allows direct user specification of the stiffness recovery factors w_c and w_t .

Figure 7.5: Uniaxial load cycle (tension-compression-tension) assuming $w_t = 0$ and $w_c = 1$ (default values) for the stiffness recovery (Dassault Systèmes Simulia Corp. 2008)

The experimental observation in most quasi-brittle materials, including concrete, shows that the compressive stiffness is recovered upon crack closure as the load changes from tension to compression. On the other hand, the tensile stiffness is not recovered as the load changes from compression to tension once crushing micro-cracks have developed. This behaviour, which corresponds to $w_t = 0$ and $w_c = 1$, is the default used by ABAQUS. If $w_c = 1$, the material fully recovers the compressive stiffness which in this case is the initial undamaged stiffness, $E = E_0$. If $w_c = 0$, there is no stiffness recovery. Intermediate values of w_c result in partial recovery of the compressive stiffness. In the composite wall model, the default values for stiffness recovery ($w_t = 0$ and $w_c = 1$) were used.

7.3.1.4 Stress-strain Relationship in Concrete

For compressive stress-strain relationship in concrete, the model proposed by Popovics was incorporated (Popovics 1973, Yip 1998) to simulate the concrete behaviour in compression. In

this model, the value of compressive concrete stress (f_c) is a function of concrete strain (ϵ_c) which can be derived from the following equation:

$$\frac{f_c}{f'_c} = \frac{n (\epsilon_c / \epsilon_0)}{n - 1 + (\epsilon_c / \epsilon_0)^{nK}} \quad (7.10)$$

where,

f'_c = ultimate compressive strength of concrete

n = a curve-fitting factor (can be taken as $n = 0.8 + \frac{f'_c}{17}$ or $n = \frac{E_c}{E_c - E'_c}$)

E_c = initial tangent modulus (can be taken as $E_c = 6900 + 3300 \sqrt{f'_c}$)

ϵ_0 = strain when f_c reaches f'_c , $\epsilon_0 = \frac{f'_c}{E_c} \left(\frac{n}{n-1} \right)$

E'_c = tangent modulus of concrete at f'_c , $E'_c = f'_c / \epsilon_0$

K = a factor to control the slope of the stress-strain curve:

$$\text{if } (\epsilon_c / \epsilon_0) \leq 1 \Rightarrow K = 1.0$$

$$\text{if } (\epsilon_c / \epsilon_0) > 1 \Rightarrow K = 0.67 + \frac{f'_c}{62} \geq 1.0$$

Many researchers proposed a stress-strain relationship for concrete in tension. The strain-softening part (after cracking) of the model can be linear, bi-linear or reciprocal function (Wang et al. 2001 and Shima 1998). In this study, for SCC a bi-linear strain-softening model (Massicotte et al. 1988) and for ECC a linear strain-softening model (Bathe et al. 1989) were incorporated to simulate the concrete behaviour in tension.

The bi-linear strain-softening model decreases from point (ϵ_{cr}, f'_t) to $(5 \epsilon_{cr}, f'_t/3)$ with the slope of $(\frac{f'_t}{6 \epsilon_{cr}})$ and from point $(5 \epsilon_{cr}, f'_t/3)$ to $(16 \epsilon_{cr}, 0)$ with the slope of $(\frac{f'_t}{33 \epsilon_{cr}})$ as shown in Fig. 7.6.

Figure 7.6: Bi-linear model for strain-softening (Massicotte et al. 1988)

The linear strain-softening model for normal concrete assumes that the strain softening after failure reduces the stress linearly to zero at a total strain of about 10 times the strain at failure. The strain at failure (f'_t/E_c) in concrete is typically, the order of 10^{-4} . It was suggested the tension stiffening that reduces the stress to zero at a total strain of about 10 (f'_t/E_c) is of the order of 10^{-3} (Dassault Systèmes Simulia Corp. 2008). For the ECC, due to its ductility, the total strain was magnified by 100 (f'_t/E_c) to get the most comparable results with the experiment test results.

Table 7.2 shows the summary of the required parameters for calculating concrete properties where ultimate concrete compressive and tensile strengths were derived from experimental testing.

Table 7.2: The summary of the concrete parameters

Specimen No.	Type of concrete	f'_c MPa	n	E_c MPa	ϵ_0	E'_c MPa	K If $(\epsilon_c/\epsilon_0) > 1$	f'_t MPa
CSW-1	SCC	51.9	3.85	30,670	0.002286	22,700	1.51	4.92
CSW-2	SCC	48.1	3.63	29,790	0.002229	21,580	1.45	4.68
CSW-3	ECC	40.2	3.16	24,500	0.002401	16,740	1.32	4.75
CSW-4	ECC	39.4	3.12	24,500	0.002367	16,650	1.31	4.71
CSW-5	SCC	35.7	2.90	26,620	0.002047	17,440	1.25	3.28
CSW-6	SCC	34.6	2.84	26,310	0.002030	17,040	1.23	3.15

For prediction of experimentally obtained behaviour, the ordinates of the stress-strain curve were computed based on f'_c and then multiplied by a reduction coefficient of 0.9 (MacGregor et al. 2000).

As an example, the stress-strain curve for specimen CSW-4 based on Eq. 7.10 is presented in Fig. 7.7. The input data to define the compressive stress (tensile stress) in ABAQUS is based on tabular function of inelastic strain (cracking strain). The stress-inelastic strain curve for $0.9 f'_c = 35.5$ MPa from Eq. 7.5 is shown in Fig. 7.8.

Figure 7.7: The stress-strain curve for $f'_c = 39.4$ MPa

Figure 7.8: The stress-inelastic strain curve for concrete

Other parameters of the CDP model in the absence of the required information were taken as the default values in the ABAQUS (Sadjadi 2009).

7.3.2 Steel Model

The steel properties that were obtained from the tensile coupon tests are nominal (engineering) stress and strain values. These nominal values were defined in terms of the initial sectional area and gauge length of the coupon. The ABAQUS software for finite element analysis uses true (Cauchy) stress and logarithmic strain. The true stress (σ_{true}) and logarithmic plastic strain (ϵ_{ln}^{pl}) can be derived from the tensile coupon data using the following equations (Lubliner 1990):

$$\sigma_{true} = \sigma_{nom} (1 + \epsilon_{nom}) \quad (7.11)$$

and,

$$\epsilon_{ln}^{pl} = \ln(1 + \epsilon_{nom}) - \frac{\sigma_{true}}{E} \quad (7.12)$$

where, σ_{nom} is the nominal stress, ϵ_{nom} is the nominal strain and the E is the modulus of elasticity obtained from tensile coupon tests. A comparison between true and engineering stress-strain for high strength and mild strength steel are presented in Figs. 7.9 and 7.10, respectively.

Figure 7.9: Comparison of true and engineering stress-strain for high strength steel

Figure 7.10: Comparison of true and engineering stress-strain for mild strength steel

The von-Mises yield surface criterion was utilized in ABAQUS/Explicit to specify the start of plastic flow in the state of multi axial stress. It was defined by giving the value of the uniaxial true stress as a function of uniaxial equivalent plastic strain at five points.

The summary of the results from the tensile coupon tests which were incorporated in the finite element model are presented in Table 7.3.

Table 7.3: True stress-plastic strain based on tensile coupon test results for ABAQUS model

Steel type	True stress & plastic strain	Point 1 (yield)	Point 2	Point 3	Point 4	Point 5 (rupture)
High strength steel F _y = 552 MPa E = 202,940 MPa	True stress (MPa)	553.5	572.3	582.8	603.7	624.2
	Plastic strain	0.0	0.001184	0.007090	0.048682	0.089516
Mild strength steel F _y = 354 MPa E = 206,980 MPa	True stress (MPa)	354.6	375.9	378.8	462.0	483.6
	Plastic strain	0.0	0.000685	0.008125	0.093084	0.212781
F _y : Yield strength of profiled steel sheet; and E: Modulus of elasticity						

For monotonic loading, isotropic hardening model was defined in ABAQUS. The yield surface in isotropic hardening model changes size uniformly in all directions such that the yield stress increases (or decreases) in all stress directions as plastic straining occurs (Davis and Selvadurai 2002).

For cyclic loading, two kinematic hardening models are provided in ABAQUS to model the cyclic loading of metals. The first one, the linear kinematic model, approximates the hardening behavior with a constant rate of hardening using two pairs of data points (the yield stress at zero plastic strain and a yield stress at a finite plastic strain value). The second model, the nonlinear isotropic/kinematic (combined) model, gives better predictions but requires more detailed calibration. Linear kinematic hardening model was used to model the behavior of metals subjected to cyclic loading. The linear kinematic hardening model was selected by specifying the “Plastic”, and “Hardening = Kinematic” option. The linear kinematic hardening component takes the Bauschinger effect into consideration (Driver et al. 1998a).

Bauschinger effect is characterized by a reduced yield stress upon load reversal after plastic deformation has occurred during the initial loading as shown in Fig. 7.11. This phenomenon decreases with continued cycling.

Figure 7.11: Stress-strain curve for steel in cyclic loading illustrating the Bauschinger effect (Kent et al. 1973)

7.4 Description of the FE Model under Monotonic and Cyclic Loading

7.4.1 Basic FE Model

It was presumed that the profiled concrete and profiled steel sheet can be modeled as a solid element and shell element, respectively as shown in Figs. 7.12 and 7.13.

Figure 7.12: Modeling of profiled concrete as a linear 8-node brick element (solid element)

Figure 7.13: Modeling of profiled steel sheet as a linear 4-node shell element

Steel can be considered as a homogeneous material and its material properties can be well defined. Concrete is, on the other hand, a heterogeneous material and its mechanical properties scatter widely and cannot be defined easily, but for the convenience of analysis, concrete is considered a homogeneous material in this study.

The composite action was achieved by modeling steel sheet with shell element as a skin, an option available in ABAQUS/CAE version 6.8–3 (Dassault Systèmes Simulia Corp. 2008) on the profiled concrete. Boundary test frame was not included in the model. The following assumptions were considered to simplify the finite element model:

- The beam-column frame does not resist any lateral loads (the beam to column connection is pinned) so the frame allows shear transfer from the frame to the wall maintaining the wall in pure shear;
- The beams and columns of the frame are rigid enough (axial and flexural) to transfer uniform shear stress to the wall boundaries;
- The wall is attached to the frame continuously.

The boundary condition and prescribed lateral displacements of the wall are shown in Fig. 7.14. The out-of-plane deformation was restrained along all four edges (Displacement U_2 (U_y) = 0). At the base, all the three displacement degrees of freedom (DOF) (U_1 , U_2 , U_3) were restrained. A linear prescribed lateral displacements were applied (along Z-axis) in X and Z direction (zero at the base and maximum at the top of wall) as shown in Fig. 7.14 to simulate shear deformation of the wall associated with shear loading.

A uniform displacement in X and Z directions at top of the wall (Fig. 7.14) using an Amplitude function versus time was defined for each direction namely Amp-1 and Amp-2, respectively. At the left and right side of the wall, a linear displacement distribution in X and Z directions multiplied by the amplitude function versus time was defined for each direction.

$U1 (U_x) = \text{Uniform displacement in X direction using Amplitude function (Amp-1)}$

$U2 (U_y) = 0$

$U3 (U_z) = \text{Uniform displacement in Z direction using Amplitude function (Amp-2)}$

Figure 7.14: The boundary condition of the wall

It should be noted that in theory, the pure shear condition can be achieved when the vertical displacement at top of the wall is constrained (Alinia et al. 2006). This condition also was examined on rectangular flat steel plate in monotonic shear loading in ABAQUS.

In experimental test, vertical displacement at the top of the wall is a function of horizontal displacement as shown in Fig. 7.15. The relation of vertical displacement versus horizontal displacement at the top of the wall can be derived from the following equation:

$$d_z = h - \sqrt{(h^2 - d_x^2)} \quad (7.13)$$

where,

d_z = vertical drop at top of the wall

h = height of the wall (1536 mm)

d_x = horizontal displacement at top of the wall

Figure 7.15: Shear deformation of the wall

The variation of d_z versus d_x for $h = 1536$ mm (as in composite wall test) is presented in Fig. 7.16. The vertical drop increases exponentially by increasing horizontal displacement. For horizontal displacement equal to 25 mm the amount of d_z will be very small ($d_z = 0.2$ mm) and this causes a vertical strain equal to $0.2/1536 = 1.3e-4$ which is less than 5 percent of the yield

strain for steel. It was observed that assuming $d_z = 0$ for horizontal displacement less than 25 mm, does not change the FEA results significantly.

Figure 7.16: Vertical drop versus horizontal displacement of the wall

The Amp-1 and Amp-2 are function of horizontal and vertical displacement versus time, which can be defined as a tabular form in ABAQUS/CAE. The Amp-1 and Amp-2 for monotonic prescribed displacement with maximum horizontal displacement of 25 mm is shown in Figs. 7.17 and 7.18.

Figure 7.17: Horizontal displacement at top of the wall versus time (Amp-1)

Figure 7.18: Vertical drop at top of the wall versus time (Amp-2)

The analysis was performed with two steps namely ‘initial’ and ‘loading’. The time period for loading step was assumed the same value as horizontal displacement (25s for monotonic prescribed displacement).

In this model, the wall was assumed to have rigid connections at the base and also, the actual rigidity of the steel frame was ignored. This model was very rigid and hence, it was decided to have a model which represents actual stiffness of the experimental specimens.

7.4.2 Sophisticated FE Model

In this model, almost all the parts of the composite wall with the boundary test frame were simulated as shown in Fig. 7.19. The model consisted of 64 (23 + 9 + 23 + 9), 12 mm bolts, 4 bolts with the size of 30 mm at corners, 4 columns, 4 beams, 4 supports at the base and a profiled composite shear wall panel (consisted of two profiled steel sheets and a profiled concrete core).

To simulate the behaviour of two surfaces which are in contact to each other, contact surfaces should be detected and appropriate surface properties should be assigned. Abacus/CAE has a feature that detects the contact surface pairs automatically. In the sophisticated model, the contact surface names and the number of contact surface pairs is shown in the Table 7.4. The number of contact surfaces (pair) was 242 in this model and refine meshing was needed in the

location of circular holes. The running time for analyzing the model, because of large numbers of contact surface and refine meshing, reached to ten days for a monotonic loading. It was decided to simplify the model in order to reach a reasonable time for running.

Figure 7.19: Sophisticated model showing; (a) Pair of steel frames and bolts, (b) Profiled composite wall panel and (c) Assembly of the profiled composite wall panel, frames and bolts

Table 7.4: Number of contact surface pairs in the sophisticated model

Contact surface name	Number of contact surface pairs
Steel sheet to concrete panel	$2 \times 1 = 2$
Columns to beams	$4 \times 2 = 8$
Columns to the composite wall	$4 \times 1 = 4$
Columns to the supports	$4 \times 1 = 4$
Beams to the composite wall	$4 \times 1 = 4$
30M bolts to other parts	$4 \times 5 = 20$
12M bolts to the supports	$4 \times 2 = 8$
12M bolts to the other parts	$64 \times 3 = 192$
	Total = 242

7.4.3 Practical FE Model

This model is a simplified version of the previous model. Instead of using 64, 12 mm bolts to connect the frame to the composite shear wall, the frame was tied (using tie constraints, see section 7.4.3.4) to the profiled composite shear wall as shown in Fig.7.20.

Figure 7.20: Practical model showing; (a) Pair of steel frames and bolts, (b) Profiled composite shear wall panel and (c) Assembly of the whole profiled composite shear wall

7.4.3.1 Parts, Meshing and Element Assignment

In order to make the FE model, 8 parts were created. These parts, meshing and assigned elements for each part are presented in Figs. 7.21 to 7.26. Two steel beams with different width at the top (80 mm) and bottom (90 mm) of the wall were generated. For all parts except the profiled steel sheet, the linear hexahedral element (C3D8R) was used. For the profiled concrete core, only at the corners where the automated meshing was not capable to use C3D8R, the linear tetrahedral element (C3D4) was employed. The assigned element for the profiled steel sheet was the linear quadrilateral shell element (S4R) and similar to the profiled concrete core, only at the corners where the automated meshing was not capable to utilize S4R, the linear triangular shell element (S3R) was employed.

Figure 7.21: Steel beam meshing using C3D8R elements

Figure 7.22: Steel column meshing which consist of 1097 elements (C3D8R)

Two 30M steel bolt with different lengths at the top (138 mm length) and the bottom (214 mm length) of the wall were generated.

Figure 7.23: Steel 30M bolt meshing using C3D8R elements

Figure 7.24: Profiled concrete panel meshing which consists of 9022 elements (C3D4) at the corners and 7184 elements (C3D8R) at the body (total 16206 elements)

Figure 7.25: Profiled steel sheet meshing which consists of 52 elements (S3R) at the corners and 9334 elements (S4R) at the body (total 9386 elements)

Figure 7.26: Steel support meshing which consists of 256 elements (C3D8R)

Reduced-integration elements used in this model had one fewer integration point in each direction than the fully integrated elements. These elements have just a single integration point located at the element's centroid. The locations of the integration points for linear elements are shown in Fig. 7.27. Reduced integration uses a lower-order integration to form the element stiffness and reduces running time.

Figure 7.27: The integration points for linear elements

Hourglassing is a numerical problem in the linear reduced-integration elements which tend to be too flexible. In ABAQUS, a small amount of artificial “hourglass stiffness” is introduced in first-order reduced integration elements to limit the propagation of hourglass modes. This stiffness is more effective at limiting the hourglass modes when more elements are used in the model, which means that linear reduced integration elements can give acceptable results as long as a

reasonably fine mesh is used. The description of the elements which are utilized in the FE practical model is presented in Table 7.5. 42,444 elements were utilized to generate the whole composite wall model. The breakdown of the type and number of elements for each part is shown in Table 7.6

Table 7.5: Description of stress/displacement elements

Element	Description
Linear hexahedral element (C3D8R)	An 8-node linear brick with reduced integration and hourglass control. 3D stress family with uniform strain and linear geometric order. This element has only one integration point (instead of $2*2*2=8$ integration points)
Linear tetrahedral element (C3D4)	A 4-node linear tetrahedron. 3D stress family with uniform strain and linear geometric order. This element has only one integration point. Note: This element provides accurate results only with very fine meshing
Linear quadrilateral shell element (S4R)	A 4-node shell with reduced integration and hourglass control. Shell family with uniform strain and linear geometric order. This element has only one integration point (instead of $2*2=4$ integration points)
Linear triangular shell element (S3R)	A 3-node triangle shell with reduced integration and hourglass control. Shell family with uniform strain and linear geometric order. This element has only one integration point (instead of 3 integration points)

Table 7.6: The type and number of elements for each part after meshing

Part's Name	Element Type	Total number of elements
Profiled concrete core	C3D8R & C3D4	$16206 \times 1 = 16206$
Profiled steel sheets	S4R & S3R	$9386 \times 2 = 18772$
Steel columns	C3D8R	$1097 \times 4 = 4388$
Steel beams at the top	C3D8R	$425 \times 2 = 850$
Steel beams at the bottom	C3D8R	$510 \times 2 = 1020$
Steel 30M bolts at the top	C3D8R	$36 \times 2 = 72$
Steel 30M bolts at bottom	C3D8R	$56 \times 2 = 112$
Supports	C3D8R	$256 \times 4 = 1024$
		Total = 42444

Mesh refinement study was performed in shear buckling study of profiled steel sheet (Rafiei et al. 2010) to verify if the results vary with further mesh refinement. It was observed that the mesh refinement was not sensitive after having the maximum size of the element less than 50 mm. In the practical model, the number of elements is more than twice the number needed to minimize the analysis warnings. In the practical model, only 87 elements from profiled concrete core had analysis warnings.

Mass scaling was performed only for the concrete elements at the corner of the wall (C3D4 elements) to increase the stable time increment for analyzing the model.

7.4.3.2 Assembling of Parts

From the original 8 parts, 19 parts in total were generated. Position constraints were utilized to assemble the whole model. The meshing of the assembled model is shown in Fig. 7.28.

Figure 7.28: Meshing for the whole practical FE model

7.4.3.3 Contact Surfaces

When two surfaces are in contact, a force normal to the contacting surfaces acts on the two bodies at points of contact. If there is friction between the surfaces, shear forces may be created to resist the tangential motion (sliding) of the bodies. In ABAQUS/CAE contact conditions are a special class of discontinuous constraint, which can allow forces to be transmitted from one part of the model to another. The constraint is discontinuous due to the fact that it is applied only when the two surfaces are in contact. When the two surfaces separate, no constraint is applied. The analysis has to be able to detect whether the two surfaces are in contact or separate to apply or remove the appropriate contact constraints.

In the Practical FE model, in order to simulate a free rotation between beams and columns at the wall corners, and between different parts and 30M bolts the contact surface in ABAQUS/CAE was used. For the simulation of the interface between profiled steel sheets and concrete core, the option of having contact surface and also surface tie constraint was studied. In the Practical FE model with contact surface between the profiled steel sheets and profiled concrete panel, the total number of contact surface pairs was 34 as shown in the Table 7.7.

Table 7.7: Number of contact surface pairs in the practical model

Contact surface name	Number of contact surface pairs
Between columns and beams	$4 \times 2 = 8$
Between columns and supports	$4 \times 1 = 4$
Between 30M bolts and beams	$4 \times 2 = 8$
Between 30M bolts and columns	$4 \times 2 = 8$
Between 30M bolts at bottom and supports	$2 \times 2 = 4$
Between profiled steel sheets and profiled concrete core	$2 \times 1 = 2$
	Total = 34

For the 34 pairs of contact surfaces the following properties were defined:

- For normal contact behaviour, the default option for “Constraint enforcement method” was considered and the “Hard” contact option for Pressure-over closure was chosen.

- b) For tangential contact behaviour, the frictionless option for friction formulation was considered.

Also separation of surfaces after contact was allowed.

7.4.3.4 Tie Constraints

Tie constraints are used to tie two nodes or surfaces together for the duration of a simulation. In tie constraint for surfaces, each node on the slave surface is constrained to have the same motion as the point on the master surface. In other word, this means all translational degrees of freedom of two surfaces are constrained.

Two types of constraint namely; surface tie and node tie constraints were utilized to connect the parts of the assembly to each other. In order to take into account the effect of 12M bolts in the practical model surface tie constrains were used.

Two FE models are incorporated to simulate interface between the profiled steel sheets and profiled concrete core using surface ties and contact surface. The results are compared and it was decided to use the contact surface instead of surface tie constraints (See Fig. 7.33). The number of surface tie constrains is shown in the Table 7.8.

Table 7.8: Number of surface tie constrains in the practical model

Surface tie constraints name	Number
Between Columns and profiled steel sheets	$4 \times 1 = 4$
Between Profiled steel sheet and concrete panel at location of the columns	$4 \times 1 = 4$
Between Beams and profiled steel sheets	$4 \times 1 = 4$
Between Profiled steel sheet and concrete core at location of the beams	$4 \times 1 = 4$
Between 30M bolts and concrete panel	$4 \times 1 = 4$
	Total = 20

The profiled steel sheets, were tied to the concrete panel at 88 nodes (at the location of the 44 intermediate fasteners) using tie constraints to simulate the effect of intermediate fasteners.

7.4.3.5 Boundary Conditions

The out of plane displacement (Z direction) of the steel frames at both sides were restrained to take into the account the concrete confinement. Also, all three displacement degrees of freedom (DOFs) of the bottom side of the steel supports were restrained as shown in Fig. 7.29.

Figure 7.29: Boundary condition of the model; (a) Z direction displacement for the columns and beams were restrained, (b) The bottom side of the steel supports were restrained

In order to take into account the concrete confinement due to 44 intermediate fasteners in actual test, 88 node restraints were utilized to restrain the nodes against out of plane displacement in Z direction) as shown in Fig. 7.30.

Figure 7.30: Restraining the nodes against the out of plane direction (Z direction)

7.4.3.6 Displacement Control Analysis

The loading was achieved by defining a Reference Point (RP) and assigning a prescribed displacement to the RP. The RP position was aligned by the center line of the 30M corner bolts located at the top of the specimen and the mid thickness of the composite wall. The RP was constrained (Coupling, coupling type: Kinematic) in the X direction to the cross section of the 30 M bolts (four areas) as shown in Fig. 7.31. The prescribed displacement was achieved by using amplitude function (smooth step) and uniform distribution in the boundary condition module.

Figure 7.31: Location of the Reference Point and constraining the PR in X direction to the cross section of 30M bolts at both sides

The analysis was performed in step-1 (Dynamic, Explicit), a step after initial step. Automatic increment and one second total “Time period” were chosen. The total horizontal displacement of the reference point (X direction) at the end of the analysis was 80 mm for the third test and 70 mm for the fifth test.

A complete run of the software for monotonic loading (1s time period) using one of the fastest available Personal Computers (Intel Quad CPU Q9550 @ 2.83 GHz, 8.00 GB RAM memory and 64-bit Operating System) took 47 hours. The average stable time increment during analysis was 6.76×10^{-7} second.

7.5 Results from the Finite Element Model

In this study three FE models were developed namely; Basic, Sophisticated and Practical FE model. It was found that the Practical FE model was the best model to simulate the experimental composite wall with respect to load-displacement response and also computational running time. The results from the FE Practical Model under monotonic and also cyclic loadings are presented in the following sections.

7.5.1 Monotonic Loading

The results from FE Practical Model under monotonic are presented in this section. Due to several modification of the test setup for the first specimen (CSW-1), the results of from the FE model were compared only with the experiment results obtained from the specimens CSW-3 and CSW-5.

7.5.1.1 FE Modeling of Third Test (Specimen CSW-3 using ECC)

The results of two FE Practical Models for simulation of concrete core-steel sheet interface were studied to finalize the model. For the concrete core-steel sheet interface two options of surface tie constrains and also contact surface were compared. The load versus displacement at the top of the wall from the experiment and two Practical FE models using surface tie constraints and contact surface is presented in Figures 7.32. It was observed that the simulation of the steel sheets and concrete panel interface using contact surface with “Hard contact” for normal behaviour and “Frictionless contact” for tangential behaviour was a better option. For the rest of all analysis, the FE Practical Model with contact surface was utilized. The maximum load capacity of the wall was 246 kN compared to 281 kN from the experiment which was 12% less.

The maximum/minimum principal stresses at the center of the profiled steel sheet are presented in Fig. 7.33. The principal stresses from the FE model and experiment were very close to each other up to a displacement of 65 mm.

Figure 7.32: Comparison of load versus displacement curve for experimental and FEA results

Figure 7.33: Comparison of FEA and experimental principal stress development in steel at the center of the wall at crest (Rosette 1)

The shear stress from the FE model and experiment at the center and also at the bottom left corner of the profiled steel sheet are shown in Figs. 7.34 and 7.35. The maximum FEA shear stress at the center reached 277 MPa which is 11% less than the maximum shear stress from experiment (310 MPa). Also the maximum shear stress from FEA at the bottom left corner was very close to the experimental results.

Figure 7.34: The shear stress in steel versus displacement form FEA and the experiment for middle of the wall at crest (Rosette 1)

Figure 7.35: The shear stress in steel versus displacement form FEA and the experiment at the location of Rosette 3

The development of axial stress in the steel column (part of the frame attached to the composite wall panel) from FE model and experiment is presented in Fig. 7.36. The maximum axial stress from FEA was 67 MPa compared to 71 MPa of test (only 6% less than the experimental).

Figure 7.36: The axial stress in steel versus displacement from FEA and the experiment at the location of strain gauge #5

The inelastic out of plane deformation of the profiled steel sheets at both sides of the composite wall is presented in Figs. 7.37 and 7.38. The out-of-plane deformation of the wall reached 52 mm when the horizontal displacement at top of the wall was 80 mm. Also the buckling of the steel sheet was diagonal and very similar to those observed in experiment.

Figure 7.37: Inelastic out of plane deformation of the profiled steel sheets at both sides of the wall

Figure 7.38: Out-of plane deformation of the composite shear wall

The contour diagram of the shear stress in the profile steel sheet before and after buckling is presented in Figs. 7.39 and 7.40. The shear stress at the middle of the wall decreased after buckling of the profile steel sheet. The failure mode of the FE model was due to buckling of the steel sheets after yielding of steel which was similar to the experimental failure mode.

Figure 7.39: Contour of shear stress (MPa) in the profiled steel sheet before buckling at load of 246 kN (64.4 mm horizontal displacement at top of the wall)

Figure 7.40: The contour of shear stress (MPa) in the profiled steel plate after buckling at 80 mm horizontal displacement at top of the wall

7.5.1.2 FE Modeling of Fifth Test (Specimen CSW-5 using SCC)

The load versus top displacement response from FE model is presented in Fig. 7.41. The first crack in the concrete core started at 35 kN and the wall failed at an ultimate load of 179 kN, exactly before the buckling of profiled steel sheets. The FEA load-displacement response is also compared with that obtained from experiment in Fig. 7.42. The FEA ultimate load was 15% less than experimental.

Figure 7.41: The load versus displacement curve from the FEA

The maximum/minimum principal stresses at the center of the profiled steel sheet for crest and trough locations are presented in Figs. 7.43 and 7.44. The principal stresses from the FEA and experiment were close to each other especially before the buckling. It should be noted that the value of the stresses was very sensitive to the location of the node in the profiled steel sheets, specially the nodes at the steel sheet buckling zone.

Figure 7.42: Comparison of load versus displacement curve from FEA and experimental results

Figure 7.43: The principal stresses in steel versus displacement from FEA and the experiment for middle of the wall at crest (Rosette 1)

Figure 7.44: The principal stresses in steel versus displacement from FEA and the experiment for middle of the wall at trough (Rosette 4)

The shear stresses at the center of the profiled steel sheet for crest and trough locations are shown in Fig. 7.45 and Fig. 7.46. The shear stresses from FEA and experiment were close to each other especially before the steel sheet buckling. A maximum shear stress of 194 MPa at the crest and 188 MPa at the trough were observed in FEA.

Figure 7.45: The shear stress in steel versus displacement form FEA and the experiment for center of the wall at crest (Rosette 1)

Figure 7.46: The shear stress in steel versus displacement form FEA and the experiment for center of the wall at trough (Rosette 4)

The maximum/minimum principal stresses from FEA at the center of the concrete core at crest and trough are presented in Fig. 7.47 and 7.48. The maximum and minimum principal stresses at the crest were 0.6 MPa (tension) and 4.0 MPa (compression) at 19 mm displacement. The minimum principal stress was 17 MPa (compression) at 55 mm displacement. The maximum/minimum principal stresses were higher at the trough compared to those at the crest before concrete cracking. The principal stresses were reached at 2.1 MPa (tension) and 5.0 MPa (compression) at 19 mm displacement. The minimum principal stress was reached at 13.3 MPa (compression) at 55 mm displacement. The relationship between minimum and maximum principal stresses at concrete cracks (19 mm displacement) can be explained based on pear shaped envelop of combination of the biaxial principal stresses causing failure of concrete (Fig. 6.4, zone 2). After this point, the tensile stress in concrete gradually drops to zero based on strain-softening behaviour of concrete (Fig. 7.6). The relationship between minimum and maximum principal stresses after concrete cracks was found complex due to cracks surfaces in concrete core was not perpendicular to the wall (3-dimensional behaviour) and also changing the angle of the principal stresses.

Figure 7.47: The principal stresses versus displacement form FEA for center of the concrete panel at crest

Figure 7.48: The principal stresses versus displacement from FEA for center of the concrete panel at trough

The out-of plane deformation at the top level of the profiled steel sheet at 70 mm horizontal displacement is presented in Fig. 7.49. The buckling of the steel sheet was predominantly diagonal and similar to those observed in the experiment.

The contour diagrams of shear stress development within the profile steel sheet before and after buckling are presented in Figs. 7.50 and 7.51. The shear stress at middle of the wall decreased after the buckling of profile steel sheet.

Figure 7.49: Inelastic deformation of the profiled steel sheet at 70 mm horizontal displacement at top of the wall

Figure 7.50: Contour of shear stress (MPa) in the profiled steel plate before buckling at load of 179 kN (48 mm horizontal displacement at top of the wall)

Figure 7.51: The contour of shear stress (MPa) in the profiled steel plate after buckling at 70 mm horizontal displacement at top of the wall

The contour of in-plane principal plastic strain in the profile steel sheet after buckling is presented in Fig. 7.52. The contour shows the existence of plastic strain at the buckling areas. The failure mode of the FE model wall was due to buckling of the steel sheets after yielding of steel. This failure mode also was observed from experimental results.

The tensile damage sequence in the profiled concrete core is shown in Fig. 7.53. The damage started at the panel boundaries where the steel frames were tied to the panel. The most damage was occurred in the troughs of the panel as shown in Fig. 7.53a. The damage extended to the diagonal cracking at the load of 180 kN as shown in Fig. 7.53b. Most of the concrete core was found to be damaged when the displacement at top of the wall reached to 70 mm (Fig. 7.53c).

Figure 7.52: The contour of plastic strain in the profiled steel plate after buckling at 70 mm horizontal displacement at top of the wall

Figure 7.53: The contour of tensile damage in the profiled concrete core at: (a) 100 kN, (b) 180 kN and (c) 70 mm top displacement

7.5.2 Cyclic Loading

The cyclic loading in ABAQUS/CAE was implemented by introducing multiple steps and each step was represented as a monotonic displacement control. The optimum “Time period” in each step was verified from the monotonic loading results. For the monotonic displacement control analysis in ABAQUS/Explicit when the “Time period” is equal or less than 0.25 second, a numerical instability (oscillatory solution) was observed as shown in Fig. 7.54. The optimum “Time period” in each step was equal to one second.

Each one second “Time Period” in the model, took two days to run the software with stable time increment of 6.76×10^{-7} second. In general, for a complete cyclic loading analysis, 50 second “Time Period” was needed. By using one of the fastest available Personal Computers (Intel Quad CPU Q9550 @ 2.83 GHz, 8.00 GB RAM memory and 64-bit Operating System), it was estimated to take 100 days for a complete run.

Figure 7.54: Comparison of the effect of “Time Period” in monotonic loading

A complete cyclic loading analysis was very time consuming and was not practical for the scope of this study. The results of one and half cyclic loading analysis (only four steps) for the specimen CSW-4 is presented to demonstrate only the functionality of the developed FE model.

7.5.2.1 FE Modeling of Fourth Test (Specimen CSW-4 using ECC)

The displacement versus time response in cyclic loading was achieved using four steps as shown in Fig. 7.55. At each step, an amplitude function of “smooth step” was utilized.

Figure 7.55: Displacement versus time in cyclic loading using amplitude function of “smooth step”

The load versus time response in the cyclic loading is presented in Fig. 7.56. The load- simulated displacement response of the composite shear wall (as shown in Fig. 7.57) was linear (as expected) due to mostly elastic behavior of the wall at the load level that was one-third of the ultimate load. However, this FE model can be used to simulate behaviour under cyclic loading up to the failure load.

Figure 7.56: The load versus time in cyclic loading

Figure 7.57: The load versus displacement in cyclic loading

7.6 Parametric Study

A parametric study was performed based on the developed Practical FE model with contact surface to investigate the effect of the number of intermediate fasteners, concrete compressive strength and steel yield strength on the composite shear wall behaviour. The parameters in this study are: three different concrete compressive strength (20 MPa, 40 MPa and 60 MPa), two different steel tensile strength Grade 40 (280 MPa) and Grade 50 (350 MPa) and tie constraints at 0, 20, 44 and 207 nodes at each side of the wall representing steel-concrete intermediate fasteners as shown in Fig 7.58. A total of 24 FE model walls were created for this study as shown in Table 7.9.

The steel sheet was modeled as an isotropic, elasto-perfectly plastic material with a modulus of elasticity $E = 203,000$ MPa. The steel properties were identical in tension and compression and von-Mises yield surface was adopted as the yield criterion.

Figure 7.58: Location of the tie constraints at each side of the wall to simulate intermediate connections; (a) No constraint, (b) $4 \times 5 = 20$ constraints, (c) $4 \times 11 = 44$ constraints (128 mm c/c), and (d) $9 \times 23 = 207$ constraints (64 mm c/c).

Table 7.9: Variables in the parametric study of numerical model walls

FE Model No.	No. of Intermediate Fasteners (constraints)	Concrete strength f'_c (MPa)	Steel tensile strength F_y (MPa)
M-1	9 * 23 = 207	20	280
M-2	9 * 23 = 207	20	350
M-3	9 * 23 = 207	40	280
M-4	9 * 23 = 207	40	350
M-5	9 * 23 = 207	60	280
M-6	9 * 23 = 207	60	350
M-7	4 * 11 = 44	20	280
M-8	4 * 11 = 44	20	350
M-9	4 * 11 = 44	40	280
M-10	4 * 11 = 44	40	350
M-11	4 * 11 = 44	60	280
M-12	4 * 11 = 44	60	350
M-13	20	20	280
M-14	20	20	350
M-15	20	40	280
M-16	20	40	350
M-17	20	60	280
M-18	20	60	350
M-19	None	20	280
M-20	None	20	350
M-21	None	40	280
M-22	None	40	350
M-23	None	60	280
M-24	None	60	350

The concrete was modeled using ABAQUS concrete damage plasticity and compressive stress-strain curve proposed by Popovics (Popovics 1973). The concrete parameters used in the parametric study are presented in Table 7.10. The definition of all the parameters is given in Eq. 7.10. The concrete tensile strength was assumed ten percent of the compressive strength of concrete and the modulus of elasticity of:

$$E_c = 6900 + 3300 \sqrt{f'_c} \quad (7.14)$$

Table 7.10: The summary of the concrete parameters used in the parametric study

f'_c MPa	n	E_c MPa	ϵ_0	E'_c MPa	K If $(\epsilon_c/\epsilon_0) > 1$	f_t MPa
20	1.98	21,660	0.001866	10,720	0.99	2.0
40	3.15	27,770	0.002110	18,960	1.32	4.0
60	4.33	32,460	0.002404	24,960	1.64	6.0

7.6.1 Effect of Yield Strength of Steel (F_y)

The effect of steel tensile strength on the load capacity of the composite shear wall for three different concrete compressive strength using 44 intermediate fasteners are shown in Figs. 7.59 to 7.61. The load capacity of the wall increased with the increase of steel yield strength.

Figure 7.59: Displacement versus shear load response (44 intermediate fasteners)

Figure 7.60: Displacement versus shear load response (44 intermediate fasteners)

Figure 7.61: Displacement versus shear load response (44 intermediate fasteners)

The steel strength had minimum effect on the load capacity of the wall when there is no intermediate fastener as shown in Figs. 7.62 and 7.63. This is due to elastic buckling of the profiled steel sheets due to no steel-concrete interface connection.

Figure 7.62: Displacement versus shear load response (no intermediate fastener)

Figure 7.63: Displacement versus shear load response (no intermediate fastener)

7.6.2 Effect of Concrete Compressive Strength (f'_c)

The effect of concrete strength on load capacity of the composite shear wall for two different steel strength using 0, 44 and 207 intermediate fasteners is shown in Figs. 7.64 to 7.69. The increase of load capacity due to increasing concrete compressive strength was 41% when concrete varies from 20 MPa to 60 MPa (Fig. 7.65).

Figure 7.64: Displacement versus shear load response (no intermediate fastener)

Figure 7.65: Displacement versus shear load response (no intermediate fastener)

Figure 7.66: Displacement versus shear load response (44 intermediate fasteners)

Figure 7.67: Displacement versus shear load response (44 intermediate fasteners)

Figure 7.68: Displacement versus shear load response (207 intermediate fasteners)

Figure 7.69: Displacement versus shear load response (207 intermediate fasteners)

7.6.3 Effect of Intermediate Fasteners

The effect of intermediate fasteners on load capacity of the composite shear wall is presented in Figs. 7.70 to 7.73. The load capacity of the composite wall with no intermediate fastener was mostly related to the concrete strength and the steel strength had minimum effect. The load capacity of the wall increased with the increase number of intermediate fasteners due to enhanced composite action. The load capacity difference between 44 and 207 intermediate fasteners was not significant. However, strength enhancement between 0 and 20 intermediate fasteners were more than 30%.

Figure 7.70: Displacement versus shear load response (steel yield strength of $F_y=280$ MPa)

Figure 7.71: Displacement versus shear load response (steel yield strength of $F_y=350 \text{ MPa}$)

Figure 7.72: Displacement versus shear load response (steel yield strength of $F_y=280 \text{ MPa}$)

Figure 7.73: Displacement versus shear load response (steel yield strength of $F_y=350 \text{ MPa}$)

7.6.4 Buckling Deformed Shape and Shear Stress in the Profiled Steel Sheet

The buckling deformed shape of the sheets in the composite shear wall without any intermediate fastener is shown in Fig. 7.74. The buckling shape of the wall consisted of two predominant diagonal strips of steel buckling with 74 degrees angle of buckled line and maximum out of plane deflection of 77 mm when the displacement at top of the wall was 80 mm.

Figure 7.74: The buckling of the profiled steel sheet without any intermediate fastener

The maximum theoretical shear stress in the steel sheet based on von-Mises yield criterion is $F_y/\sqrt{3}$ (Sabouri-Ghomi et al. 2005). For $F_y = 350$ MPa, the maximum theoretical shear stress for yielding of steel sheet is 202.1 MPa. The variation of shear stress along the profiled steel sheet

panels before buckling and after buckling for a composite shear wall with concrete compressive strength of 20 MPa and steel yield strength (F_y) of 350 MPa for 207 intermediate fasteners and no intermediate fastener, are presented in Figs. 7.75 and 7.76, respectively. The shear stress in the steel sheet with 207 intermediate fasteners reached the maximum theoretical shear yield stress before buckling and buckling occurred at nonlinear stage (Fig. 7.75). For the profiled steel sheet without any intermediate fastener, the buckling occurred at linear stage as shown in Fig. 7.76.

Figure 7.75: Shear stress (MPa) contour in the profiled steel sheet; (a) Before sheet buckling at 41 mm top displacement, (b) After buckling at 80 mm for a composite shear wall with 207 intermediate fasteners, concrete compressive strength of 20 MPa and $F_y=350$ MPa

Figure 7.76: Shear stress (MPa) contour in the profiled steel sheet; (a) Before buckling at 41 mm top displacement, (b) After buckling at 80 mm for a composite shear wall without any intermediate fastener, concrete compressive strength of 20 MPa and $F_y=350$ MPa

7.6.5 Comparison of Parametric and Analytical Studies

In chapter 6, an analytical equation was derived for shear resistance of the composite wall. 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 was limited by the steel sheets. For this reason, the shear resistance of the FE model composite shear walls with only 44 and 207 intermediate fasteners from the parametric study is compared with that obtained from analytical Eq. 6.24 in Tables 7.11 and 7.12. For simplicity, it was assumed that the β factor is equal to 0.23 (from examples in the Chapter 6). The ultimate shear resistance from FE models was almost 10% less than that derived from analytical Eq. 6.24. The difference between shear resistance of the FE model composite

shear walls with 44 intermediate fasteners was less than 5% compared to 207 intermediate fasteners. This shows that the optimum number of intermediate fasteners for shear resistance of the wall should be higher but very close to 44 intermediate fasteners.

Table 7.11: Shear resistance of the composite shear wall with 44 intermediate fasteners

Steel yield strength F_y MPa	Concrete Strength f'_c MPa	FEA ultimate load kN	Analytical Model (Eq. 6.24)			Ratio (FEA/Analytical Eq. 6.24)
			Steel capacity $2V_s$ (kN)	Concrete capacity V_c (kN)	Shear resistance of wall $V_r = 2V_s + V_c$ (kN)	
280	20	152.7	142.0	29.0	171.0	0.893
280	40	161.3	142.0	41.0	183.0	0.881
280	60	179.2	142.0	50.2	192.2	0.932
350	20	178.0	177.5	29.0	206.5	0.862
350	40	185.9	177.5	41.0	218.5	0.851
350	60	208.6	177.5	50.2	227.7	0.916

Table 7.12: Shear resistance of the composite shear wall with 207 intermediate fasteners

Steel Yield strength F_y MPa	Concrete Strength f'_c MPa	FEA Ultimate load kN	Analytical Model (Eq. 6.24)			Ratio (FEA/Analytical Eq. 6.24)
			Steel capacity $2V_s$ (kN)	Concrete capacity V_c (kN)	Shear resistance of wall $V_r = 2V_s + V_c$ (kN)	
280	20	153.1	142.0	29.0	171.0	0.895
280	40	164.2	142.0	41.0	183.0	0.897
280	60	181.3	142.0	50.2	192.2	0.943
350	20	179.9	177.5	29.0	206.5	0.871
350	40	188.8	177.5	41.0	218.5	0.864
350	60	214.6	177.5	50.2	227.7	0.942

7.6.6 Comparison of the Shear Resistance of the FE Model of Walls

Composite action between profiled steel sheets and concrete core was assumed to be 100% (when yielding of sheets rather than buckling govern the failure) for the wall with 207 intermediate fasteners and zero percent when there is no intermediate fastener (buckling of sheets govern the failure). It should be noted that the number of 207 intermediate fasteners seems to be large to achieve the full composite action and optimum number should be much less than this number.

The composite action for 20 and 44 intermediate fasteners was calculated based on linear interpolation between shear resistances of the FE model walls. The results are shown in Table 7.13. The minimum composite action for the walls with 20 and 44 intermediate fasteners are 70% and 91%, respectively. The results show that use of 44 intermediate fasteners can provide reasonable composite action for the walls with different steel yield strength and concrete compressive strength.

Table 7.13: Shear resistance and composite action of the composite shear walls

F_y MPa	f'_c MPa	207 intermediate fasteners		44 intermediate fasteners		20 intermediate fasteners		No intermediate fastener	
		Ultimate load (kN)	Composite action (%)	Ultimate load (kN)	Composite action (%)	Ultimate load (kN)	Composite action (%)	Ultimate load (kN)	Composite action %
280	20	153.1	100	152.7	99.2	146.7	87.0	103.7	0
280	40	164.2	100	161.3	92.6	157.2	82.1	125.2	0
280	60	181.3	100	179.2	95.2	175.0	85.5	137.8	0
350	20	179.9	100	178.0	97.4	170.4	86.8	108.2	0
350	40	188.8	100	185.9	95.3	176.6	80.4	126.7	0
350	60	214.6	100	208.6	91.4	193.7	70.2	144.5	0
F_y : Steel yield strength and f'_c : Concrete compressive strength									

7.7 Summary and Conclusions

In this study three FE models were developed namely; Basic, Sophisticated and Practical FE model. It was found that the Practical FE model is a better option to simulate composite walls with respect to time of computation and reasonable accuracy of results.

The results of two FE Practical models using surface tie constraints and also contact surface to simulate the concrete core-steel sheet interface were compared. It was found that the contact surface is a better modeling technique to simulate the experimental walls.

The results from the FE Practical model and experimental for comparison and validity of the FE model are presented in Table 7.14. The ultimate load capacity of the walls based on the FE model was about 15% less than experimental walls. The 15% difference showed a good FE model simulation of the composite shear wall considering high degree of complexity of the problem. Also the simulated stresses, failure mode and buckling patterns in the profiled steel sheet were comparable with the experimental results.

Table 7.14: Comparison between the results from FE Practical model and experimental

Specimen No.	FE Practical Model			Experimental		
	Ultimate load (kN)	Maximum Shear stress at wall centre (MPa)	Failure mode of the wall	Ultimate load (kN)	Maximum Shear stress at wall centre R1 (MPa)	Failure mode of the wall
CSW-3	246	277	Inelastic buckling	281	310	Inelastic buckling
CSW-5	179	194	Inelastic buckling	212	177	Inelastic buckling
R1: Rosette 1						

One limitation of the FE Practical model is the time for a complete run. This time for monotonic loading was about 2 days and for cyclic loading was estimated to take 100 days for a complete run by using one of the fastest available Personal Computers. In general, the developed FE model was found to be capable of simulating the behaviour of the composite shear wall accurately.

The performed parametric study provided valuable information on the effects of number of intermediate fasteners along height and width of the wall, steel tensile strength and concrete compressive strength on the composite shear wall behaviour. An increase in the number of intermediate fasteners increased the shear load capacity of the wall and it would be possible to optimize the intermediate steel-concrete connections to obtain a failure due to the yielding of profiled steel sheets. The shear load capacity difference between 44 and 207 intermediate fasteners was very small (less than 5%) and therefore, 44 intermediate fasteners can provide full composite action (more than 90%) which can effectively increase the shear load capacity of the wall.

The steel strength has minimum effect on the shear load capacity of the wall without steel-concrete interface connections. This may be attributed to the failure due to elastic buckling of the profiled steel sheets before yielding as a result of the lack of steel-concrete composite action. Moreover, for adequate intermediate fasteners, the effect of the steel strength on load capacity for a given concrete compressive strength was higher than the effect of concrete compressive strength for a given steel strength. The increase in steel yield strength is more effective in enhancing the shear strength of composite wall compared to the increase in concrete compressive strength.

CHAPTER 8

FINITE ELEMENT MODELING OF COMPOSITE WALL SUBJECTED TO IMPACT LOADING

8.1 Introduction

The development of reliable finite element models simulating the behaviour of composite shear walls under impact loading can be very useful for engineers and designers. In this study, a numerical model has been developed using ABAQUS/CAE finite element analysis software (Dassault Systèmes Simulia Corp. 2008) to simulate composite shear wall behaviour under impact loading. Analyzing high-speed dynamic events under impact can be difficult using ABAQUS/Standard. As the load is instantaneous, usually very severe and rapid changes in the response of the structure occur. Accurate tracking of stress waves through the proposed composite shear wall is important for capturing the dynamic response. The stress waves are associated with the highest frequencies of the structural model and accurate solution requires behavior simulation in many small time increments. ABAQUS/Explicit module is the best option for such wave propagation analysis.

This chapter describes material models used in the ABAQUS/CAE, development of numerical models for the composite walls and the results illustrating the performance of the numerical simulation compared to experimental results.

8.2 Modeling of Material Properties

The concrete and steel models incorporated in the finite element model are presented. It should be noted that the strain rate (the rate of change in strain with respect to time) for the concrete and steel were not utilized in the models due to lack of experimental information. Moreover, the strain rate was in the order of 10^0s^{-1} in the experimental impact test (as discussed in Chapter 5) and for such order, the difference between material behaviour under impact and quasi-static loading is not significant (Figs. 5.1 and 5.2).

8.2.1 Concrete Model

Concrete Damaged Plasticity (CDP) model (Lubliner et al. 1989, Lee et al. 1998, and Li et al. 2005) was incorporated to simulate the concrete behaviour in ABAQUS/CAE Version 6.8–3 (Dassault Systèmes Simulia Corp. 2008). This comprehensive plasticity-based concrete model assumes mainly two failure mechanisms resulting from tensile cracking and compressive crushing of concrete. The model utilizes concepts of isotropic damaged elasticity in combination with isotropic tensile and compressive plasticity to represent the inelastic behavior of concrete. The user also can control the stiffness recovery effects during cyclic load reversals (Dassault Systèmes Simulia Corp. 2008). The bi-linear strain-softening model (Massicotte et al. 1988) was used to simulate the concrete behaviour in tension. For compressive stress-strain relationship in concrete, the model proposed by Popovics was incorporated (Popovics 1973, Yip 1998) to simulate the concrete behaviour in compression.

Table 8.1 shows the summary of key parameters for calculating concrete properties in the CDP model. It should be noted that the concrete compressive strength (f'_c) and tensile strength (f'_t) were derived from the experimental tests. The detailed definition of each of the parameters was described in Chapter 7.

Table 8.1: The summary of the concrete parameters for CDP

Test No.	Type of concrete	f'_c MPa	n	E_c MPa	ε_0	E'_c MPa	If $(\varepsilon_c/\varepsilon_0) > 1$	f'_t MPa
CSW-7	SCC	38.7	3.08	27,430	0.002089	18,530	1.29	3.44
CSW-8	SCC	34.3	2.82	26,230	0.002026	16,930	1.22	2.98

8.2.2 Steel Model

The profiled steel sheet properties obtained from the tensile coupon tests were nominal (engineering) stress and strain. These nominal values were defined in terms of an initial sectional area and gauge length of the coupon. The ABAQUS software for finite element analysis uses true (Cauchy) stress and logarithmic strain. The relationship between true stress (σ_{true}) and

nominal stress (σ_{nom}) and also logarithmic plastic strain (ε_{ln}^{pl}) and true stress were described in Chapter 7.

The von-Mises yield surface criterion was used in ABAQUS/Explicit to specify the start of plastic flow in the state of multi-axial stress. It was defined by giving the value of the uniaxial yield stress as a function of uniaxial equivalent plastic strain at five points as shown in Figs 7.9 and 7.10. The summary of the results from the tensile coupon tests which were incorporated in the finite element model are presented in Table 8.2.

Table 8.2: True stress-plastic strain based on tensile coupon test results for FE model

Steel type	Parameter	Point 1 (yield)	Point 2	Point 3	Point 4	Point 5 (rupture)
Mild strength steel $F_y = 354 \text{ MPa}$ $E = 206,980 \text{ MPa}$	True stress (MPa)	354.6	375.9	378.8	462.0	483.6
	Plastic strain	0.0	0.000685	0.008125	0.093084	0.212781
F_y : Yield strength of profiled steel sheet; and E : Modulus of elasticity						

8.3 Description of the Developed FE Model for Impact Loading

8.3.1 Parts, Meshing and Element Assignment

In order to have a model for impact loading, two parts were added to the developed FE Practical model described in chapter 7. The names of those parts were “End plate” and “Projectile”. Meshing and assigned elements for each part are presented in Figs. 8.1 to 8.2.

Figure 8.1: The end plate meshing with hexahedral elements (C3D8R)

Figure 8.2: The projectile meshing with hexahedral elements (C3D8R)

The “End plate” part was located at the top of the composite wall and was consisted of 2746 linear hexahedral elements (C3D8R, 8-node brick element). The “Projectile” part is the movable part that modeled the projectile which strikes the wall and consisted of 1428 linear hexahedral elements (C3D8R).

A total of 46,618 elements were utilized to generate the whole FE model. The breakdown of the type and number of elements for each of the 10 parts of the model is presented in Table 8.3.

Table 8.3: The type and number of elements for each part after meshing

Part's Name	Element Type	Number of nodes and element type	Total number of elements
Profiled concrete panel	C3D8R & C3D4	8 & 4, brick (3D)	$16,206 \times 1 = 16,206$
Profiled steel sheets	S4R & S3R	4 & 3, Shell (2D)	$9,386 \times 2 = 18,772$
Steel columns	C3D8R	8, brick (3D)	$1,097 \times 4 = 4,388$
Steel beams at top	C3D8R	8, brick (3D)	$425 \times 2 = 850$
Steel beams at bottom	C3D8R	8, brick (3D)	$510 \times 2 = 1,020$
Steel 30M bolts at top	C3D8R	8, brick (3D)	$36 \times 2 = 72$
Steel 30M bolts at bottom	C3D8R	8, brick (3D)	$56 \times 2 = 112$
Supports	C3D8R	8, brick (3D)	$256 \times 4 = 1,024$
End plate	C3D8R	8, brick (3D)	$2,746 \times 1 = 2,746$
Projectile	C3D8R	8, brick (3D)	$1,428 \times 1 = 1,428$
			Total = 46,6182

8.3.2 Assembling of Parts

The 'End plate' and 'Projectile' parts were assembled to the FE Practical model. Position constraints were utilized to locate these two parts at the top of the wall as shown in Fig. 8.3. The meshing of the whole assembled FE model is shown in Fig. 8.4.

Figure 8.3: Assembling of the 'End plate' and 'Projectile' parts to the existing assembly

Figure 8.4: Meshing of the whole FE model

8.3.3 Contact Surfaces

To simulate the behaviour of two surfaces which are in contact to each other, contact surfaces should be detected and appropriate surface properties should be assigned. Five contact surfaces additional to the previously developed contact surfaces as defined in Chapter 7 were added to the FE Practical model for impact simulation. One contact surface between the “Projectile” and “End plate” and four contact surfaces between the “end plate” and 30M bolts were defined. The total number of contact surface pairs is 39 as shown in Table 8.4.

Table 8.4: Number of contact surface pairs in the model for impact loading

Contact surface name	Number of contact surface pairs
Columns and beams	$4 \times 2 = 8$
Columns and supports	$4 \times 1 = 4$
30M bolts and beams	$4 \times 2 = 8$
30M bolts and columns	$4 \times 2 = 8$
30M bolts at bottom and supports	$2 \times 2 = 4$
Profiled steel sheets and profiled concrete	$2 \times 1 = 2$
Projectile and End plate	$1 \times 1 = 1$
30M bolts at top and End plate	$2 \times 2 = 4$
	Total = 39

The contact property for all the surfaces was defined as “Hard contact” for the normal contact behaviour and “Frictionless contact” for the tangential contact behaviour, similar to the FE model described on the Chapter 7.

8.3.4 Steps

Two steps other than initial step were defined in the model. In step-1(before impact), the projectile was freely able to move with a constant speed. The “time period” in step-1 was set to 0.02s, regardless of the projectile speed. The initial distance between the projectile and head of

the end plate was variable based on the speed of the projectile. Step-2 simulates the behaviour of the wall after impact. The “time period” in step-2 was set to 0.06s in order to capture the important data and also to minimize the running time and data file size. In this step, the number of acceleration data (in the field output request) was set to every 0.0001s of time (600 samples) and for other data was set to every 0.001s of time (60 samples).

8.3.5 Model Tree for the FE Practical Impact Model

The details of the FE modeling sequence in ABAQUS/CAE for impact loading in the form of a model tree is shown in Fig. 8.5. The model tree provided useful information of how the FE model in ABAQUS/CAE for impact loading was made. The main modules are parts, materials, assembly, steps, output requests, interactions, constraints, loads, boundary conditions and jobs.

The Part module allows the user to create individual parts by sketching their geometry directly in ABAQUS/CAE. The name of the ten parts in the FE practical model for impact loading is shown in Fig. 8.5. In the Interaction module the user can specify mechanical interactions between regions of a model such as contact. The Constraints module allows the user to define tie constraints for nodes or surfaces. In the Job module the user can interactively submit a job for analysis and monitor its progress.

Figure 8.5: The detail of the FE modeling sequence in ABAQUS/CAE for impact loading

8.4 Analysis of FE Model Results

8.4.1 Modeling of the Seventh Tested Specimen (CSW-7 using SCC)

The results from the finite element modeling of the composite wall (CSW-7) under impact loading are presented and described. The results are divided into two phases similar to the experimental results. In phase I, the mass of the projectile was 36 kg with a speed of 10 m/s and in phase II, the mass of the projectile was 64.8 kg with a speed of 10 m/s. The difference between these two phases in the FE model was only a change in the mass density of the “projectile part” to adjust the projectile mass. In both phases, the projectile hit twice to the end plate structure. The second strike happened when the specimen was returning back to its original position or passing from its original position (after the specimen reached to the maximum displacement). This double strike was also observed in the experimental test.

8.4.1.1 Impact Phase I

The displacement at the top of the wall versus time is presented in Fig. 8.6. The projectile was moving toward the wall without any strike from 0.0 s up to 0.02 s and at 0.02 s, it hit the wall.

Figure 8.6: Displacement-time response at the top of the wall

The simulated maximum displacement of the wall by FE model was 17.6 mm compared to 18.0 mm of experimental results. The fundamental period of the wall was 0.043. The maximum shear strain rate can be calculated roughly from the displacement response (Fig. 8.6). The FE model reached its maximum displacement (17.6 mm) at 0.0115 s. The maximum shear strain, $\gamma_{\max}$ (calculated by dividing the 17.6 mm to the wall height of 1536 mm) and the maximum strain rate were 0.0115 and 1.0 s^{-1} , respectively.

The location of accelerometers at top of the boundary steel frame is shown in Fig. 8.7. The numerical model predicted acceleration of the wall at the same location of the accelerometer on steel beam (used in experimental test) versus time is shown in Fig. 8.8.

Figure 8.7: The location of the accelerometers at the boundary steel frame

Figure 8.8: FE model predicted acceleration versus time at top of the wall

The maximum acceleration was $33,850 \text{ m/s}^2$ (equals to 3451 g's). The second strike happened at about 0.05 s , when the wall displacement at the top was 12 mm in opposite direction of the impact (-12 mm).

The amount of kinetic energy (E_{KE}) and internal energy (E_I) of the whole model versus time is presented in Figs. 8.9 and 8.10, respectively. The kinetic energy of the projectile before the impact was $1800 \text{ N} \cdot \text{m}$ ($0.5 \text{ m V}^2 = 0.5 \cdot 36 \cdot 10.0^2$) and after impact, most of the kinetic energy was converted to internal energy.

Figure 8.9: The amount of the kinetic energy (E_{KE}) versus time

Figure 8.10: The amount of the internal energy (E_I) versus time

The internal energy (E_I) is the sum of the recoverable elastic strain energy, the energy dissipated through inelastic processes such as plasticity and the artificial strain energy. The artificial strain energy includes energy stored in hourglass resistances and transverse shear in shell elements (profiled steel sheets).

Ideally, the summation of the kinetic energy (E_{KE}) and the internal energy (E_I) should be constant. In the FE model, it remains approximately constant, generally with an error of few percentages.

The maximum principal plastic strain and von-Mises stress distribution in the profiled steel sheet after impact in Phase I are shown in Figs. 8.11 and 8.12, respectively. The cracks in the concrete panel are mostly located at the troughs close to the steel frame. The location of concrete cracks can be detected when the maximum principal plastic strain is greater than zero as shown in the Fig. 8.11.

Figure 8.11: The maximum principal plastic strain in the profiled concrete after impact

Also the amount of von-Mises stress was higher in the areas close to the impact with most of von-Mises stress located at top-left side of the wall near the projectile impact, as expected (indicated by the concentration of red color shade, Fig. 8.12). This is due to the difference between quasi static and impact loading as most of the damaged areas are close to the impact and it is more localized with the increase of the impact speed. In the high speed impact, time is too short for the stress wave propagation and the local response under the impact load normally should dominate the wall behaviour as observed in both experimental and FEA (Zineddin and Krauthammer 2007).

Figure 8.12: The von-Mises stress (MPa) distribution in the profiled steel sheet after impact

8.4.1.2 Impact Phase II

The displacement at the top of the wall versus time is presented in Fig. 8.13. Similar to phase I, the projectile was moving toward the wall without any strike from 0.0 s up to 0.02 s then it hit the wall at 0.02 s.

Figure 8.13: Displacement-time response at the top of the wall

The maximum horizontal displacement of the wall predicted by the FE model was 27.1 mm compared to 23.0 mm obtained from the experiment (18% higher in FE model). The period of the FE model was 0.046s. The wall reached its maximum displacement (27.1 mm) at 0.0121 s. The maximum shear strain ($\gamma_{\max}$) and the maximum strain rate were 0.0176 and 1.45 s^{-1} , respectively.

The acceleration of the wall (at the same location of the accelerometer on steel beam in experimental test) versus time is shown in Fig. 8.14. The maximum acceleration was $53,650 \text{ m/s}^2$ (equal to 5469 g's). The second strike happened at about 0.036 s when the wall was moving back to its original position after reaching to the maximum displacement.

Figure 8.14: Predicted FE model acceleration versus time at top of the wall

The amount of kinetic energy (E_{KE}) and internal energy (E_I) of the whole model versus time are presented in Figs. 8.15 and 8.16, respectively. The kinetic energy of the projectile before the impact equals to 3240 N. m ($0.5 \text{ m V}^2 = 0.5 * 64.8 * 10.0^2$), almost twice the kinetic energy from the phase I. Most of the kinetic energy after the impact was converted to the internal energy as shown in Fig. 8.16.

Figure 8.15: The amount of the kinetic energy (E_{KE}) versus time

Figure 8.16: The amount of the internal energy (E_I) versus time

The maximum principal plastic strain in the profiled concrete and von-Mises stress distribution in the profiled steel sheet after impact are shown in Figs. 8.17 and 8.18, respectively.

Figure 8.17: The maximum principal plastic strain in the profiled concrete after impact

The cracks in the concrete core are mostly located at the troughs which are close to the steel frame. Also the von-Mises stress was higher at top areas of the wall, close to the impact. Compared to the results from phase I, the stress was about 5% higher and also, more distributed over the width of the wall at the top (Fig. 8.18). This is due to nature of impact which depends on speed of impact; the damage on structure could be localized. If the speed of projectile is high at impact, time is too short for stress wave propagation, and local response under the load could dominate behaviour of the structure (Zineddin and Krauthammer 2007).

Figure 8.18: The von-Mises stress (MPa) distribution in the profiled steel sheet after impact

8.4.1.3 Comparison of FEA and Experimental Results

The summary of the FEA results from the two phases are compared with those obtained from experiments in terms of maximum wall displacement/acceleration at the top and fundamental period as depicted in Table 8.5.

Table 8.5: Comparison of experimental and FEA results

Projectile mass & speed	Results from	Max. Wall displacement at the top (mm)	Wall fundamental period (s)	Max. Wall acceleration at the top (g)
W = 36.0 kg V = 10.0 m/s	FEA	17.6	0.043	3451
	Experimental	18.0	0.04	3810
	FEA / Exp.	0.978	1.075	0.906
W = 64.8 kg V = 10.0 m/s	FEA	27.1	0.046	5469
	Experimental	23.0	0.042	4436
	FEA / Exp.	1.178	1.095	1.233

In general, a reasonable agreement between FEA results and the experimental results is evident in the comparison shown in Table 8.5. Thus, the use of FE models to simulate composite wall behavior under impact loading can be robust.

8.4.2 Modeling of the Eighth Tested Specimen (CSW-8 using SCC)

The results from the finite element modeling of the composite wall (CSW-8) under impact loading are presented. The results are divided into two phases similar to the experimental results. In phase I, the mass of the projectile was 64.8 kg with a speed of 6.4 m/s and in phase II, the mass of the projectile was 62.3 kg with a speed of 10.2 m/s.

The difference between these two phases in the FE model is the change in the mass density of the “projectile part” and also initial projectile speed. To have a constant 0.02 s time (Step I) before impact for both phases, the initial distance between projectile and the end plate structure, was fixed to 128 mm and 204 mm for phase I and II, respectively. The projectile in both phases hit twice the end plate structure. The second strike was happened when the specimen was returning back to its original position after reaching to its maximum displacement. This double strike was also observed in the experimental test.

8.4.2.1 Impact Phase I

The displacement at the top of the wall versus time is presented in Fig. 8.19. The projectile was moving freely toward the wall without any strike from 0.0 s up to 0.02 s then at 0.02 s, it hit the wall.

Figure 8.19: Displacement-time response at the top of the wall

The maximum displacement of the wall was 17.6 mm compare to 14.3 mm of experimental results (23% more) and the fundamental period of the wall was 0.045. The FEA predicted maximum displacement of the wall was (17.6 mm) at 0.0116 s. The maximum shear strain, $\gamma_{\max}$ and the maximum strain rate were 0.0114 and 0.99 s^{-1} , respectively.

The location of accelerometers at top of the wall is shown in Fig. 8.20. The accelerations of the wall at three locations (same location of the acidometers in experimental test) versus time are shown in Figs. 8.21 to 8.23. The maximum acceleration at the steel beam was $24,200 \text{ m/s}^2$ (2467 g's). The maximum accelerations in the concrete close to impact and far from impact were $20,550 \text{ m/s}^2$ (2095 g's) and $16,850 \text{ m/s}^2$ (1718 g's), respectively. The maximum acceleration in the concrete close to impact was 22% higher than the maximum acceleration far from impact. The second strike happened at about 0.037 s when the wall was moving back to its original position after reaching its maximum displacement.

Figure 8.20: The name and location of each accelerometer at the top of the wall

Figure 8.21: Acceleration versus time at the top of the wall in the steel beam

Figure 8.22: Acceleration versus time at the top of the wall in concrete-near

Figure 8.23: Acceleration versus time at the top of the wall in concrete-far

The amount of kinetic energy (E_{KE}) and internal energy (E_I) of the whole model versus time are presented in Figs. 8.24 and 8.25, respectively. The kinetic energy of the projectile before the impact was 1327 N. m. Most of the kinetic energy after the impact was converted to the internal energy as shown in Fig. 8.25.

Figure 8.24: The amount of the kinetic energy (E_{KE}) versus time

Figure 8.25: The amount of the internal energy (E_I) versus time

The maximum principal plastic strain in the concrete core and von-Mises stress distribution in the profiled steel sheet after the impact are shown in Figs. 8.26 and 8.27, respectively. The cracks in the concrete panel are mostly located at the troughs close to the steel frame. The

amount of von-Mises stress was higher in the areas close to the impact and the stress concentration is more localized at the top-left side of the wall.

Figure 8.26: The maximum principal plastic strain in the profiled concrete after impact

Figure 8.27: The von-Mises stress (MPa) distribution in the profiled steel sheet after impact

8.4.2.2 Impact Phase II

The displacement at the top of the wall versus time is presented in Fig. 8.28. Similar to Phase I, the projectile was moving toward the wall without any strike from 0.0 s up to 0.02 s then it hit the wall at 0.02 s.

Figure 8.28: Displacement-time response at the top of the wall

The maximum horizontal displacement of the wall was 27.2 mm compare to 23.1 mm from experimental results (18% more) and the fundamental period of the wall was 0.046. The wall reached its maximum displacement (27.2 mm) at 0.0122 s. The maximum shear strain ($\gamma_{\max}$) and the maximum strain rate were 0.0177 and 1.45 s^{-1} , respectively.

The accelerations of the wall at three locations versus time are shown in Figs. 8.29 to 8.31. The maximum acceleration at the steel beam was $52,100 \text{ m/s}^2$ (5311 g's). The maximum accelerations in the concrete close to impact and far from impact were $29,800 \text{ m/s}^2$ (3038 g's) and $24,850 \text{ m/s}^2$ (2533 g's), respectively. The maximum acceleration in the concrete close to impact was 20% higher than the maximum acceleration far from impact. The second strike

happened at about 0.035 s when the wall was moving back to its original position after reaching to its maximum displacement.

Figure 8.29: Acceleration versus time at the top of the wall in the steel beam

Figure 8.30: Acceleration versus time at the top of the wall in concrete-near

Figure 8.31: Acceleration versus time at the top of the wall in concrete-far

The amount of kinetic energy (E_{KE}) and internal energy (E_I) of the whole model versus time are presented in Figs. 8.32 and 8.33, respectively. The kinetic energy of the projectile before the impact was equal to 3241 N. m, more than twice the kinetic energy from the phase I.

Figure 8.32: The amount of the kinetic energy (E_{KE}) versus time

Figure 8.33: The amount of the internal energy (E_I) versus time

The maximum principal plastic strain in the concrete core and von-Mises stress distribution in the profiled steel sheet after impact are shown in Figs. 8.34 and 8.35, respectively. The cracks in the concrete panel are mostly located at the troughs which are close to the steel frame similar to the results from phase I.

Figure 8.34: The maximum principal plastic strain in the profiled concrete after impact

The cracks in the concrete core are mostly located at the troughs which are close to the steel frame. Also the von-Mises stress is higher at top areas of the wall, close to the impact. Comparing the results from phase I and II, the value of the stress in phase II is about 3% higher and it is also more distributed over the width of the wall at the top.

Figure 8.35: The von-Mises stress (MPa) distribution in the profiled steel sheet after impact

8.4.2.3 Comparison of Experimental and FEA Results

The summary of the FEA results from the two phases with the experimental is presented in Table 8.6. In general, FE models simulated the composite wall behavior under impact loading very well (regarding displacement and fundamental period) as confirmed by the reasonable agreement with experimental results.

Table 8.6: Comparison of experimental and FEA results

Projectile mass and speed	Results	Max. wall disp. at the top (mm)	Wall period (s)	Max. wall acc. in steel (g)	Max. wall acc. in concrete-near (g)	Max. wall acc. in concrete-far (g)
W = 64.8 kg V = 6.4 m/s	FEA	17.6	0.045	2467	2095	1718
	Experimental	14.3	0.04	2375	1787	1435
	FEA / Exp.	1.23	1.13	1.04	1.17	1.20
W = 62.3 kg V = 10.2 m/s	FEA	27.2	0.046	5311	3038	2533
	Experimental	23.1	0.042	4538	1954	1676
	FEA / Exp.	1.18	1.10	1.17	1.56	1.51

8.5 Summary

In this chapter, the FE Practical model from chapter 7 was modified to simulate impact loading. The developed finite element model was capable of simulating the behaviour of the composite shear wall under impact loading reasonably well. Although the strain rate effect for material was not considered in the FE model due to lack of the material data and also difficulty to obtain this information from the lab test, the results were quite reasonable. The results from modified FE Practical model for impact and experimental for comparison and validity of the FE model are presented in Table 8.7.

Table 8.7: Comparison between the results from the FE model and experimental

Specimen No.		Modified FE Practical model				Experimental			
		Max. wall disp. (mm)	Wall Period (s)	Max. wall acc. in steel (g)	maximum shear strain rate (s^{-1})	Max. wall disp. (mm)	Wall Period (s)	Max. wall acc. in steel (g)	maximum shear strain rate (s^{-1})
CSW-7	Phase I	17.6	0.043	3451	1.0	18.0	0.04	3810	1.0
	Phase II	27.1	0.046	5469	1.45	23.0	0.042	4436	1.25
CSW-8	Phase I	17.6	0.045	2467	0.99	14.3	0.04	2375	-
	Phase II	27.2	0.046	5311	1.45	23.1	0.042	4538	1.25

The maximum horizontal displacement of the wall at impact was about 20% higher than the experimental value, showing a good agreement considering the high complexity of the problem. The fundamental period of the wall was 13% higher than the experimental value. The maximum shear strain rate for FE model and experiment was in the order of 10^0 s^{-1} and FEA result was 16% higher than the experimental value. It was found that the FEA results for acceleration was very sensitive to the location of the accelerometers and the difference between FEA and experimental results was more in concrete (55%) compared to steel (25%). This might be due to the high complexity of the wall which required defining several contact surfaces in the FE model.

One of the advantages of using ABAQUS/Explicit for the simulation of the composite wall under impact load is the reduction in computational time needed to complete a run. Compared to monotonic loading which took two days for a complete run, the impact loading model took only 4 hours. This is due to small time step as discussed in chapter 7.

Performing impact test on structures experimentally, can be quite expensive and time consuming. On the other hand, FE models can be reasonably accurate, economical and time efficient. Therefore, such models can be utilized to conduct parametric studies and for preliminary design consideration of many complex problems.

CHAPTER 9

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

9.1 Summary

This study was conducted to examine the behaviour of a novel form of composite shear wall consisting of two profiled steel sheets and an infill concrete under in-plane monotonic, cyclic and impact loadings. The interaction between steel and concrete is important in this type of walling system to prevent early elastic buckling of profile sheets to optimize structural capacity. The composite action between steel and concrete was achieved through steel-concrete interface connectors in the form of intermediate fasteners connecting two outer sheets with concrete core along the height and width of the wall.

This research consisted of experimental, analytical, and finite element investigations. The experimental investigation and numerical/finite element modeling of the composite shear wall were performed to obtain information on the strength, stiffness, load-deformation response, profiled steel sheet-concrete interaction, stress-strain characteristics, influence of steel-concrete material properties, concrete cracking/propagation, and possible failure modes. In addition, experimental test results were utilized to confirm the validity of the developed numerical models.

Eight composite shear wall specimens with overall dimensions of 1626 mm x 720 mm were tested under in-plane monotonic, cyclic and impact loading. Steel sheet-concrete interface connections were provided by intermediate fasteners along height and width of the wall to generate composite action. Two types of concrete namely Self-Consolidating Concrete (SCC) and Engineered Cementitious Composites (ECC), and a cold formed profiled steel sheet having same geometry but two different yield strengths were incorporated to investigate their influence on composite wall behaviour.

An analytical model for shear resistance of the composite wall was developed based on existing models taking into account the shear capacity of the steel sheet, concrete core and steel sheet-concrete interaction. Moreover, two non-linear finite element (FE) models for the composite shear wall under monotonic/cyclic and impact loading were developed using proprietary ABAQUS/CAE finite element software. The performance of the numerical FE models were validated against experimental results and the models were then utilized to carry out an extensive parametric study to understand the influence of material and steel-concrete interaction parameters on the structural behaviour of composite shear walls. This research confirmed the viability and the suitability of this new composite shear walling system to be used in steel framed building as lateral load resisting elements.

9.2 Conclusions

Based on experimental results, analytical modeling and finite element analyses of the composite walling system, the conclusions were drawn and presented in the following sections.

9.2.1 Experimental Investigation of Composite Walls under In-Plane Monotonic & Cyclic Loading

Six composite wall specimens were tested under monotonic and cyclic loading. The first test was considered as a pilot monotonic test and used to develop and validate the performance of the shear test set-up.

The advantages of using ECC over SCC as profiled concrete core were exhibited through demonstration of more ductile behaviour and better crack development characteristics. In SCC core, continuous lines of cracks were developed compared with scattered and discontinuous lines of cracks in ECC core. This can be associated with ECC's high strain capacity while maintaining low crack widths.

For cyclic loading, the wall specimen made of ECC (specimen CSW-4) was subjected to ten cyclic loading groups. The wall stiffness was reduced from 4.73 kN/mm (at the first cyclic group) to 2.47 kN/mm (at the 10th cyclic group), showing a stiffness degradation of 47.8%. This

stiffness degradation, in spite of having more cyclic groups in ECC wall, is the lowest compared with other SCC wall specimens. The displacement ductility (measured in terms of the ratio of maximum displacement to the displacement at which significant yielding took place) of the ECC wall was also the highest with a value of 1.7 compared to around 1.4 of SCC wall specimens (CSW-2 and CSW-6) showing a 13% higher displacement ductility. ECC wall exhibited better behaviour showing lower stiffness degradation and higher ductility compared to its SCC counterpart.

In general, the ultimate cyclic shear load capacity of wall specimens was not less than 12% of their monotonic load capacity. This was an indication of the superior strength retaining capacity of composite walls under cyclic loading and illustrated the viability of practical application of such walls as shear resisting elements in buildings.

The load associated with the initiation of concrete cracking and debonding 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. This load was very clear for 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.

The advantage of using mild strength over high strength steel was demonstrated through prevention of sudden and brittle failure, associated with tearing of the steel sheets. In general, the failure of composite walls was due to yielding of steel sheet associated with post-yield buckling. However, wall specimen in cyclic loading (in the second test) failed due to tearing of the profiled steel sheets. In this brittle failure, the wall stiffness dropped from 3.7 to 2.1 kN/mm in the last cycle showing 43% stiffness degradation. The number of steel-concrete intermediate fasteners along the height and width of the specimens provided sufficient steel-concrete composite action to prevent early elastic buckling of the profiled steel sheets and initiated failure due to steel yielding. The values of principle stress and shear stress at the location rosette strain gauges reached to the failure stress according to von-Mises yield criterion before wall failure showing adequacy and effectiveness of the provided intermediate steel sheet-concrete fasteners.

9.2.2 Experimental Investigation of the Walls under In-Plane Impact Loading

At the initial stage of the research, it was intended to investigate the behaviour of profiled composite shear wall under impact loadings with two types of concrete namely Self-Consolidating Concrete (SCC) and Engineered Cementitious Composites (ECC). Due to high ductility of ECC, it was expected that the energy absorbing capacity of the ECC walls could be improved compared to SCC. However, considering the time limit for the completion of the research, two wall specimens with SCC were tested.

The post impact shear strength of the composite wall did not reduce (compared to control wall tested under static monotonic loading without impact) after the application of about 3100 J impact energy which was more than 6 times of impact energy capacity of the wall at elastic range. This can be associated with the localized nature of the concrete damage in the proximity of the impact load as observed during the test. Such localized concrete damage was not able to degrade the overall integrity of the steel-concrete composite system in resisting monotonic loading besides the degradation of wall stiffness. Stiffness degradation of the wall after impact was around 8% compared to the equivalent wall specimen tested purely under monotonic loading without impact. This was an indication of better strength retaining capacity of the wall at impact and its suitability as impact resisting element in a building frame.

The impact test for each specimen was performed in two phases, namely Phase I and Phase II. In Phase I, the amount of impact energy of the projectile was kept low intentionally to capture dynamic characteristics of the wall. In Phase II, the impact test was performed with maximum speed of the projectile, which the impact apparatus could produce. The behaviour of the wall in Phase I of impact loading was almost elastic and difference in maximum displacement of the wall between experimental and theoretical was not significant. The change in natural period of the wall between phase I and phase II was about 5% which represented almost 10% stiffness degradation based on Single Degree of Freedom (SDOF) behaviour of the wall. This was confirmed from the results of the post-impact monotonic loading of the walls.

The differences in the cracking pattern in the impact loading compared to the post-impact monotonic loading were the development of cracks at upper part of the wall (close to the impact)

and the formation of major cracks close to the boundary columns at trough. It is expected that these cracks were associated with the cracks developed during the impact tests.

9.2.3 Development of Analytical Model for the Shear Resistance of Composite Walls

Analytical models for the shear resistance of the profiled steel sheet, profiled concrete core and composite wall were developed based on existing theoretical models. For the steel sheet, the model was developed based on the assumption of failure due to shear yielding of steel before buckling as a result of increased steel buckling capacity due to steel-concrete interaction, the presence of sheet-concrete interface connections and composite confinement.

The shear resistance of the profiled concrete core was derived based on the simplified modified compression field theory (MCFT). 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%. The proposed analytical model can therefore, be used for the prediction of shear resistance of composite walls with reasonable accuracy.

9.2.4 Finite Element Modeling of Composite Wall under Monotonic and Cyclic Loading

Three FE models were developed namely “Basic”, “Sophisticated” and “Practical” FE models. It was found that the Practical FE model is capable to simulate the behaviour of experimental composite walls in terms of load-displacement response, stress-strain characteristics and failure modes as well as efficient in computer processing/run time.

The results of two FE Practical models using “surface tie constraint” and “contact surface” to simulate the concrete core -steel sheet interface were compared. It was found that the FE model using the “contact surface” is the best to simulate the behavior of experimental composite walls.

The ultimate load capacities of the walls using FE model were about 15% less than those of experimental walls. A 15% difference was an indication of good FE model simulation of the composite shear wall considering the complexity of the experiments. Also the FE model simulated stresses, failure mode, buckling patterns in the profiled steel sheet and concrete cracking were comparable with those obtained from experiments.

One of the limitations of the FE Practical model was the computer time required for a complete run for cyclic loading. This time for monotonic loading was about 2 days and a complete run for cyclic loading was estimated to take about 100 days by using one of the fastest available Personal Computers. In general, the developed finite element (FE) model was found to be capable of simulating the behaviour of the composite shear wall accurately. However, for cyclic loading, use of faster computer is warranted to complete the Finite Element Analysis (FEA).

The performed FE parametric study provided valuable information on the effect of number of intermediate fasteners along the height and width of the wall, steel tensile strength and concrete compressive strength on the composite shear wall behaviour. An increase in the number of intermediate fasteners increased the shear load capacity of the wall and it would be possible to optimize the intermediate steel-concrete connections to obtain a failure due to the yielding of profiled steel sheets before buckling. The shear load capacity difference between 44 and 207 intermediate fasteners was very small (less than 5%) and therefore, 44 intermediate fasteners could provide effectively a load capacity of the wall equivalent to more than 90% of that derived based on full composite action.

The steel strength had minimum effect on the shear load capacity of the wall without steel-concrete interface connections. This was attributed to the failure due to elastic buckling of the profiled steel sheets before yielding as a result of the lack of steel-concrete composite action. Moreover, for adequate intermediate fasteners, the effect of the steel strength on load capacity for a given concrete compressive strength was higher than the effect of concrete compressive strength for a given steel strength. The increase in steel yield strength is more effective in enhancing the shear strength of composite wall compared to an increase in concrete compressive strength. For example by providing 44 intermediate fasteners, the increase in wall load capacity

due to increasing concrete compressive strength was 17% when concrete strength varied from 20 MPa to 60 MPa (representing 200% increase in concrete strength) but the same load capacity increase of 17% was observed when steel yield strength varied from 280 MPa to 350 MPa (representing a 25% increase in steel yield strength).

9.2.5 Finite Element Modeling of Composite Wall Subjected to Impact Loading

The FE Practical Model for monotonic and cyclic loading was modified to simulate impact loading. The developed finite element model was capable of simulating the behaviour of the composite shear wall under impact loading reasonably well. Although the strain rate effect for material was not considered in the FE model (due to lack of some of the material data and also difficulty to obtain this information from the lab test), the results were quite reasonable.

The maximum horizontal displacement of the wall at impact was about 20% higher than the experimental value, showing a good agreement considering the high complexity of the problem.

The fundamental period of the wall obtained from the FE model in Phase II was 0.046 s which was 10% higher than the experimental value (0.042 s). The maximum shear strain rate for FE model and experiment was in the order of 10^0s^{-1} and FEA result was 16% higher than the experimental value. It was found that the FEA results for acceleration were very sensitive to the location of the accelerometers and the difference between FEA and experimental results was more in concrete (55%) compared to steel (25%). This might be due to the high complexity of the wall which required defining several contact surfaces in the FE model.

One of the advantages of using ABAQUS/Explicit for simulation of the composite wall under impact load was short time required for a complete run. For impact loading run time was 4 hours compared to a run time of two days for monotonic loading. This is due to short time step required for impact loading. Performing impact test on structures experimentally, can be quite expensive and the developed FE model showed that the results are comparable with the experiment and it can be utilized to reduce the cost of future researches by decreasing the number of test specimens and also limiting required full-scale experimental tests.

9.2.6 General Conclusions

One of the main objectives of this research was to investigate the behavior of the composite wall under different types of in-plane loadings without early elastic buckling of the profiled steel sheets prior to wall failure. This goal was achieved reasonably well through providing intermediate steel sheet-concrete connections generating better composite action between steel and concrete. Although the composite walls exhibited high shear resistance in cyclic loading, low displacement ductility was observed during experimental test. The 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. The use of low-yield point (LYP) steel in the production of composite shear wall is recommended to enhance the ductility. It is expected that the composite shear wall with enhanced ductility will be a suitable alternative for other types of shear wall to construct buildings in areas of high seismic risk.

This composite walling system with excellent shear resistance 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 or less expensive to other lateral load resisting systems due to the fact that there is no need for formwork for infill concrete (as the profile steel sheet works as a permanent formwork).

The proposed analytical model is suitable for the calculation of the shear resistance of composite wall. The validity of the analytical model can be confirmed through performing more research on the behaviour of composite shear wall with different dimensions and configurations which can be useful to the Canadian Standard Association (CSA-S16 2009) to formulate design guidelines for the proposed double skin profiled composite shear wall system.

9.3 Recommendations for Future Research

The following recommendations are suggested for further research studies:

- 1) More experimental works on one story and also multi-story composite shear wall with different types of frame connection are suggested to investigate the behaviour of the wall under monotonic, cyclic and impact loading.
- 2) Understanding of the behaviour of composite perforated shear walls subjected to in-plane loads is required for building construction to allow for passage of electrical and mechanical services through the wall.
- 3) The proposed analytical model for shear resistance of the composite wall was derived based on full composite action between steel sheet and concrete. The need for analytical models to simulate partial composite action is warranted for better understanding of the composite wall behaviour.
- 4) In the numerical FE model, some parameters (such as “dilation angle” and “the ratio of biaxial compressive strength to uniaxial compressive strength”) in the concrete damage plasticity model were assumed as the default values due to lack of experimental material testing data. It is recommended to verify the effect of those parameters in further research.
- 5) Additional experimental investigations would be helpful to understand the behaviour of composite wall subjected to in-plane loading at the presence of gravity loads and fire.
- 6) Parametric studies under impact loading using the FE model would be a great help to investigate the behaviour of the composite shear wall to optimize the space between intermediate fasteners based on the amount of impact energy.
- 7) Work on design tables is recommended to provide the optimum distance between intermediate fasteners to prevent steel buckling based on different thickness, shape and yield strength of profiled steel sheets.

- 8) Work on the development of Code based design procedure for the walls is needed to derive equations for shear resistance of the composite wall based on partial composite action between steel and concrete. The partial composite action may improve the composite wall ductility and make it more suitable for the structures which are located in high seismic risk.
- 9) Prefabrication and marketization aspects of this walling system should be studied in association with construction and manufacturing industries to use this system as viable alternative to other traditional systems.

Appendix A - Test Setup Modification

The following modification was performed to the test setup for cyclic and monotonic loading in order to improve the rigidity of the connection of the composite wall to the I-shaped base frame and the actuator and also to strengthen the columns of the boundary frame at the bottom of the composite wall.

A.1 Modifying the test setup – Phase 1

At the beginning of the first (pilot) test, excessive deformation at the connection of the actuator to the composite wall was noticed. This was due to hinge end of the actuator and hinge connection of the end plate to the composite wall (two hinges in the horizontal direction). In order to fix the problem (only in the first test), it was decided to perform the test by pulling the composite wall instead of pushing. During the first test (pulling the composite wall), excessive displacement of the composite wall was observed because of the deformation of the angle connectors. The test was stopped intentionally to fix those problems. The following modifications were performed:

a) Modifying the end plate structure

In order to build a new end plate structure, the two existing vertical plates were cut (Fig. A.1) and two new long plates were welded to the base plate (Fig. A.2) which can be mounted at the top of the composite wall.

Figure A.1: Cutting the existing vertical plates from the base plate

Figure A.2: The modified end plate structure

Due to heavy weight of the end plate structure, it was lifted by a crane in the structures lab to be attached at the top of the composite wall (Fig. A.3).

Figure A.3: Lifting the end plate structure by the crane

The end plate structure was mounted to the top of composite wall by using two M30 threaded rods to split the load from actuator and also to prevent the hinge problem.

b) Modifying the four angles

One rectangular plate with 10 mm thickness (PL 250 x 150 x 10) was welded to each angle to prevent the deformation of the angle during the test. The modified strengthened angle is shown in Fig. A.4.

Figure A.4: Modified angle with a rectangular plate

In order to mount the modified angle to the I-shaped base frame, 16 holes (4 holes for each modified angle) on the sides of the base frame were made using Unibor magnetic drill. Each modified angle was mounted to the base frame using 7, 20M bolts (Fig. A.5).

Figure A.5: Mounting the modified angle to the base frame

A.2 Modification of test setup – Phase 2

After the first phase of modification of the test setup, test by applying monotonic loading (pushing) was performed for the second time on the first wall specimen. A small amount of the plastic deformation in the modified angle at the location of the 32 mm hole was observed. It was decided to increase the thickness of the modified angle at that location. A PL 80 x 65 x 10 with a 32 mm hole was welded to each modified angle for further strengthening (Fig. A.6).

Figure A.6: Increasing the thickness of the modified angles

The final shape of the modified angles is shown in the Fig. A.7 and mounting the composite shear wall to the base frame is shown in Fig. A.8.

Figure A.7: The final shape of the modified angle

Figure A.8: Mounting the composite wall to the base frame

It should be noted that all four M30 threaded rods at the corners of the composite wall were replaced with high strength structural (ASTM A490 or SAE Grade 8) M30 bolts as shown in Fig. A.9 to increase the effective area and strength of the bolts. The high strength structural bolt has minimum yield strength of 900 MPa and minimum ultimate tensile strength of 1040 MPa.

Figure A.9: Mounting the end plate structure to the top of composite wall

A.3 Modifying the test setup – Phase 3

A small amount of plastic deformation of steel columns at the bottom of the composite wall was observed after performing the final test on the first specimen. The first step to fix the problem was to weld a steel plate with 10 mm thickness to the column as shown in the Fig. A.10. The second step was to make holes to the welded plate. The final shape of the modified columns is shown in Fig. A.11. Also the lateral holes on the base frame were modified based on the thickness of the welded plate (10 mm).

Figure A.10: Strengthening of steel columns at the bottom of the wall by welded steel plate

Figure A.11: The final shape of the modified steel columns

A.4 Additional modification to the test setup

In order to prevent the movement of the actuator, two straps were used to tight the actuator to the lab floor (Fig. A.12). Also to prevent the horizontal movement (rocking) of the I-shaped base frame, the gap between the base frame and the existing rigid frame in the Structures laboratory was completely filled by stacking steel plates as shown in Fig. A.13.

Figure A.12: Tightening the actuator to the lab floor

Figure A.13: Filling the gap between the base frame and existing rigid frame

This remedy was good for the monotonic loading when the actuator pushed the composite wall. For the cyclic loading a “floor brake” was used. The “floor brake” consisted of a (hollow steel section) HSS and two steel shafts with diameter of 50 mm. The steel shafts were passed through the HSS and were welded to the HSS (Fig. A.14). The distance between two shafts was exactly 600 mm matching the distance of the spacing holes on the lab strong floor. The steel shafts can go through the lab floor and in this way; the “floor brake” can be fixed to the floor. Figure A.15 shows the I-shaped base frame which was prevented from rocking during cyclic loading. The gap between the base frame and the “floor brake” was filled with steel plates and small HSS.

Figure A.14: The “floor brake”

Figure A.15: Preventing the base frame from rocking in cyclic loading

Two V shaped handles were welded to the top beams of the composite wall as shown in Fig. A.16. These handles were very useful for lifting the composite wall by the crane.

Figure A.16: Welding the handles to the beams for lifting the composite shear wall

Appendix B - Calibration Certificate for Accelerometers

Dytran Instruments, Inc.
21592 Marilla St. Chatsworth, CA 91311 Ph: 818-700-7818 Fax 818-700-7880
www.dytran.com email: info@dytran.com

CALIBRATION CERTIFICATE LIVM SHOCK ACCELEROMETER

CUSTOMER: A-TECH INSTRUMENTS, LTD				TEST REPORT #: 5796		
PURCHASE ORDER #: 9021490		SALES ORDER #: 137704		PROCEDURE: TP3005		
MODEL: 3200B6			SERIAL #: 5796			
TEMPERATURE (°C): 23			HUMIDITY (%): 40			
NEW UNIT	☒	RE-CALIBRATION [1]	☐	AS RECEIVED CODE	AS RETURNED CODE	
MOUNTED RESONANT FREQUENCY (KHz): 131			RANGE, F.S. (g's): 2500			
G LEVEL (g's)			SENSITIVITY (mV/g) [2]:			
1000			2.16			
2000			2.17			
2500			2.17			
REMARKS: NONE						
TEST EQUIPMENT LIST - CALIBRATION STATION # 5						
DII #	MANUFACTURER	MODEL	SERIAL #	DESCRIPTION	CAL DATE	DUE DATE
692	FLUKE	45	6976018	MULTIMETER	12/26/08	12/26/09
017	NICOLET	310	84D00744	DIGITAL OSCILLOSCOPE	02/12/09	02/12/10
550	DYTRAN INST.	4165	175	CHARGE AMPLIFIER	08/20/08	08/20/09
055	DYTRAN INSTRUMENTS	5868A	55	DROP SHOCK FORCE SENSOR	06/02/09	06/02/10
<p>[1] AS RECEIVED / AS RETURNED CODES:</p> <p>1 = IN TOLERANCE, NO ADJUSTMENTS 4 = OUT OF TOLERANCE > 5% 7 = UNIT NON-REPAIRABLE, RECOMMEND REPLACEMENT</p> <p>2 = IN TOLERANCE, BUT ADJUSTED 5 = REPAIR REQUIRED 8 = UNIT SERVICEABLE WITH CURRENT CALIBRATION DATA</p> <p>3 = OUT OF TOLERANCE < 5% 6 = REPAIRED AND CALIBRATED</p> <p>[2] THE REFERENCE SENSITIVITY IS MEASURED AT 100 Hz, 1G RMS.</p> <p>[3] THIS CALIBRATION WAS PERFORMED IN ACCORDANCE WITH ANSI/NCSL Z540-1-1994, ISO 10012-1, ISO/IEC 17025 USING THE BACK-TO-BACK COMPARISON METHOD PER ISA RP37.2 AND IS TRACEABLE TO THE NIST THROUGH TEST REPORT # 11632-120H DUE 08-26-09</p> <p>ESTIMATED UNCERTAINTY OF CALIBRATION: 2% FROM 20-100 Hz, 1.5% FROM 100-2500 Hz, 2.8% FROM 2.5KHZ-10 kHz. APPLIES TO FREQUENCY RESPONSE ONLY.</p> <p>THIS CERTIFICATE SHALL NOT BE REPRODUCED EXCEPT IN FULL, WITHOUT THE WRITTEN PERMISSION FROM DYTRAN INSTRUMENTS, INC.</p>						
CALIBRATION TECHNICIAN: _____				TEST DATE: 07/27/09		
HUNG LE				RECALL DATE: 07/27/10		

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Appendix B - Calibration Certificate for Accelerometers

Dytran Instruments, Inc.
21592 Marilla St. Chatsworth, CA 91311 Ph: 818-700-7818 Fax 818-700-7880
www.dytran.com email: info@dytran.com

CALIBRATION CERTIFICATE LIVM SHOCK ACCELEROMETER

CUSTOMER: A-TECH INSTRUMENTS, LTD				TEST REPORT #: 5798		
PURCHASE ORDER #: 9021490		SALES ORDER #: 137704		PROCEDURE: TP3005		
MODEL: 3200B6			SERIAL #: 5798			
TEMPERATURE (°C): 23			HUMIDITY (%): 40			
NEW UNIT	X	RE-CALIBRATION [1]		AS RECEIVED CODE		AS RETURNED CODE
MOUNTED RESONANT FREQUENCY (KHz): 131			RANGE, F.S. (g's): 2500			
G LEVEL (g's)			SENSITIVITY (mV/g) [2]:			
1000			2.14			
2000			2.15			
2500			2.15			
REMARKS: NONE						
TEST EQUIPMENT LIST - CALIBRATION STATION # 5						
DII #	MANUFACTURER	MODEL	SERIAL #	DESCRIPTION	CAL DATE	DUE DATE
692	FLUKE	45	6976018	MULTIMETER	12/26/08	12/26/09
017	NICOLET	310	84D00744	DIGITAL OSCILLOSCOPE	02/12/09	02/12/10
550	DYTRAN INST.	4165	175	CHARGE AMPLIFIER	08/20/08	08/20/09
055	DYTRAN INSTRUMENTS	5868A	55	DROP SHOCK FORCE SENSOR	06/02/09	06/02/10
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CALIBRATION CERTIFICATE LIVM SHOCK ACCELEROMETER

CUSTOMER: A-TECH INSTRUMENTS, LTD				TEST REPORT #: 5799		
PURCHASE ORDER #: 9021490		SALES ORDER #: 137704		PROCEDURE: TP3005		
MODEL: 3200B6			SERIAL #: 5799			
TEMPERATURE (°C): 23			HUMIDITY (%): 40			
NEW UNIT	X	RE-CALIBRATION [1]		AS RECEIVED CODE		AS RETURNED CODE
MOUNTED RESONANT FREQUENCY (KHz): 131			RANGE, F.S. (g's): 2500			
G LEVEL (g's)			SENSITIVITY (mV/g) [2]:			
1000			2.17			
2000			2.17			
2500			2.18			
REMARKS: NONE						
TEST EQUIPMENT LIST - CALIBRATION STATION # 5						
DII #	MANUFACTURER	MODEL	SERIAL #	DESCRIPTION	CAL DATE	DUE DATE
692	FLUKE	45	6976018	MULTIMETER	12/26/08	12/26/09
017	NICOLET	310	84D00744	DIGITAL OSCILLOSCOPE	02/12/09	02/12/10
550	DYTRAN INST.	4165	175	CHARGE AMPLIFIER	08/20/08	08/20/09
055	DYTRAN INSTRUMENTS	5868A	55	DROP SHOCK FORCE SENSOR	06/02/09	06/02/10
<p>[1] AS RECEIVED / AS RETURNED CODES:</p> <p>1 = IN TOLERANCE, NO ADJUSTMENTS 4 = OUT OF TOLERANCE > 5% 7 = UNIT NON-REPAIRABLE, RECOMMEND REPLACEMENT</p> <p>2 = IN TOLERANCE, BUT ADJUSTED 5 = REPAIR REQUIRED 8 = UNIT SERVICEABLE WITH CURRENT CALIBRATION DATA</p> <p>3 = OUT OF TOLERANCE < 5% 6 = REPAIRED AND CALIBRATED</p> <p>[2] THE REFERENCE SENSITIVITY IS MEASURED AT 100 Hz, 1G RMS.</p> <p>[3] THIS CALIBRATION WAS PERFORMED IN ACCORDANCE WITH ANSI/NCSL Z540-1-1994, ISO 10012-1, ISO/IEC17025 USING THE BACK-TO-BACK COMPARISON METHOD PER ISA RP37.2 AND IS TRACEABLE TO THE NIST THROUGH TEST REPORT # 11632-120H DUE 08-26-09</p> <p>ESTIMATED UNCERTAINTY OF CALIBRATION: 2% FROM 20-100 Hz, 1.5% FROM 100-2500 Hz, 2.8% FROM 2.5KHZ-10 kHz. APPLIES TO FREQUENCY RESPONSE ONLY.</p> <p>THIS CERTIFICATE SHALL NOT BE REPRODUCED EXCEPT IN FULL, WITHOUT THE WRITTEN PERMISSION FROM DYTRAN INSTRUMENTS, INC.</p>						
CALIBRATION TECHNICIAN: _____				TEST DATE: 07/27/09		
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www.dytran.com email: info@dytran.com

CALIBRATION CERTIFICATE LIVM SHOCK ACCELEROMETER

CUSTOMER: A-TECH INSTRUMENTS, LTD				TEST REPORT #: 5800		
PURCHASE ORDER #: 9021490		SALES ORDER #: 137704		PROCEDURE: TP3005		
MODEL: 3200B6				SERIAL #: 5800		
TEMPERATURE (°C): 23				HUMIDITY (%): 40		
NEW UNIT	X	RE-CALIBRATION [1]		AS RECEIVED CODE		AS RETURNED CODE
MOUNTED RESONANT FREQUENCY (KHz): 131				RANGE, F.S. (g's): 2500		
G LEVEL (g's)				SENSITIVITY (mV/g) [2]:		
1000				2.09		
2000				2.10		
2500				2.10		
REMARKS: NONE						
TEST EQUIPMENT LIST - CALIBRATION STATION # 5						
DII #	MANUFACTURER	MODEL	SERIAL #	DESCRIPTION	CAL DATE	DUE DATE
692	FLUKE	45	6976018	MULTIMETER	12/26/08	12/26/09
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