

University of Alberta

**Steel Plate Shear Walls for Low and Moderate Seismic Regions and
Industrial Plants**

by

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A thesis submitted to the Faculty of Graduate Studies and Research
in partial fulfillment of the requirements for the degree of

Doctor of Philosophy

in

Structural Engineering

Department of Civil and Environmental Engineering

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Fall 2013

Edmonton, Alberta

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ABSTRACT

Steel plate shear walls have traditionally been perceived to be suitable mainly for high seismic regions due to their great ductility and cyclic energy dissipation capacity. Therefore, design and detailing requirements have become increasingly onerous in an attempt to maximize their performance, effectively making the system uneconomical in other regions. Developing applications specifically for low and moderate seismic regions has largely been neglected by researchers. Moreover, despite unique advantages of the system in terms of inherent high ductility and redundancy, its performance under accidental blast has not been investigated systematically. The objective of this research is to examine these neglected areas.

Different practical details are investigated to reduce the force demands on the boundary frame of the wall system and ultimately reduce the construction cost in low seismic regions. A seismic zone-independent performance-based design method is developed and the efficiency of each detail is studied using comprehensive finite element simulations. It was found that suitable details for low seismic applications include simple beam-to-column connections, modular construction, and adopting a more liberal design philosophy for the columns.

A large-scale two-story steel plate shear wall test specimen was designed based on the efficient details for the limited-ductility performance application and

tested under gravity load concurrent with cyclic lateral loads. The test results are used to assess its overall seismic performance and verify the efficiency of the proposed design philosophy and selected details. The specimen, overall and in its details, showed excellent performance with high ductility.

The nature of the infill plate forces applied to the boundary frame members is discussed in detail, and the reasons for achieving conservative column design forces in current capacity design methods are described. A performance-based capacity design method for the wall system is proposed and the target performance level is defined in terms of ductility and redundancy. Based on new and previous experimental data, a holistic and sound set of principles for capacity design of steel plate shear walls for three different performance levels—including limited-ductility, moderately ductile, and ductile—along with their design provisions, are developed. The method is applied to design examples and verified against experimental results.

Another objective of this research was to explore the possible application of steel plate shear walls as a protective structure in industrial plants. Advanced and comprehensive numerical models that take into account important issues affecting the blast design are developed. The blast performance of the system is investigated by means of iso-response curves for both in-plane and out-of-plane blast orientations and different response parameters. An analytical normalization method is proposed that produces dimensionless iso-response curves.

ACKNOWLEDGEMENTS

Funding for this research was provided by the Natural Sciences and Engineering Research Council of Canada. The SPSW test specimen was donated by Supreme Steel, Edmonton. Financial support was provided through scholarships from the Steel Structures Education Foundation, Alberta Heritage Foundation, University of Alberta, Canadian Society for Civil Engineering, and Canadian Institute of Steel Construction. All supports are gratefully acknowledged.

Many individuals contributed to the success of the physical test conducted in the I.F. Morrison Laboratory of the University of Alberta. Significant contributions are made by Dr. Tim Joseph (data acquisition interface), Michael Leitch (construction), Sean Hudson (instrumentation), and Christopher Auger (testing).

I would like to express my deep gratitude to Professor Robert G. Driver for his consistent support and guidance during the course of my study. Without him this research would not have been possible.

Finally, I would like to express my deepest gratitude to my wife, Raheleh, who has unselfishly supported me and shared the ups and downs of my PhD study. Her understanding, patience and companionship made this research possible.

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LIST OF ABBREVIATIONS AND SYMBOLS

Abbreviations

BSE	basic safety earthquake
C	compression
D	ductile
ESDOF	equivalent single-degree-of-freedom
LD	limited-ductility
LT	lateral load transfer pattern from the diaphragm
MCE	maximum considered earthquake
MD	moderately ductile
MDOF	multi-degree-of-freedom
MRF	moment resisting frame
SDOF	single-degree-of-freedom
SPSW	steel plate shear wall
T	tension

Symbols

a	site class factor; half of a crack length
A	cross-section area of column cross-section
b	beam
c	column; directionally-dependent material damage parameters
C_r	front wall blast reflection coefficient
C_p	correction factor that takes into account the effects of pinched hysteresis cycles and cyclic strength and stiffness degradation on the maximum lateral deformation of a system
C_y	correction factor that takes into account the effects of yielding on the maximum lateral deformation of a system
d	scalar material damage parameters
d_b	beam depth

d_c	column depth
dw_p	incremental plastic work done by the incremental plastic strain tensor over the stress state at a point
$d\epsilon_p^*$	incremental effective plastic strain
$d\epsilon_p$	incremental plastic strain tensor
d_0	scalar material damage parameters
D	hole diameter
E	modulus of elasticity
f	scalar material damage parameters
f_{iR}	compression-column design yield mechanism loads at story i
f_R	total compression-column base shear
F	dynamic load amplitude
F_i	system yield mechanism force at story i
$F_{i,C}$	cumulative equivalent static design story shear force
F_L	system yield mechanism force applied to the left column
F_R	system yield mechanism force applied to the right column
F_s	total base shear
F_{si}	total seismic force at story i
F_y	nominal yield stress of steel material
F_{yw}	infill plate nominal yield stress
g	acceleration of gravity
G_I	strain energy release rate (fracture energy per unit area) for a crack in the first mode of opening in the plain stress condition
h	overall height of wall
h_c	clear height of a story
h_i	center-to-center height of story i
H_i	height of the story i from the base
i	number of story
I	total impulse (the area under blast load pressure) of a blast load
I_r	reflected impulse
I_x	moment of inertia of column cross-section about strong axis

I_0	ideal impulse with zero duration that produces the yield displacement in an elasto-plastic SDOF system
J_2	second principal invariant of the stress deviatoric tensor
k	elastic stiffness of SDOF
K_d	effective lateral stiffness of a system based on force level method
K_e	effective lateral stiffness of a system
K_i	elastic lateral stiffness
K_I	stress intensity factor for the first mode of opening
K_L	load transformation factor
K_{LM}	load–mass transformation factor
K_M	mass transformation factor
L	left (tension column) side; characteristic length of an element
L_c	clear distance between the columns
L_h	distance between the plastic hinges in beams
m, m_b	distributed couple about the beam centerline due to horizontal component of the unbalanced infill plate forces above and below the beam, mass of a SDOF system
m_c	distributed couple about the column centerline due to vertical component of the infill plate forces
M	total effective seismic mass of the system
M_c	resultant of the distributed couple m_c about the column centerline at each story
M_{CD}	compression column design moment from capacity design
$M_{CD,0}$	compression column moment from capacity design with pinned-base assumption for the column
M_{e1}	modal effective mass for the fundamental mode
$M_{FE,N}$	compression column moment from nonlinear finite element simulation
M_{max}	maximum moment at midspan of the beam
$M_{max,E}$	maximum moment at midspan of the equivalent beam
$M_{max,FE}$	maximum moment in the beam span from finite element results

M_{pb}^*	nominal plastic moment capacities of the beam ends reduced to account for the effect of axial force in the beam
M_{pc}	nominal plastic moment capacities of the column bases cross-section
M_{pc}^*	nominal plastic moment capacities of the column bases reduced to account for the effect of axial force in the column
M_r	factored moment strength for the compression column from S16
M_V	maximum moment at midspan of the beam due to unbalanced infill plate force
n	total number of story
$p(t)$	air blast wave
P	maximum blast load pressure; resultant force of the total applied blast pressure
P_{CD}	design axial force for the compression column from capacity design
P_e	equivalent dynamic load value for a distributed system
$P_{FE,N}$	axial force for the compression column from nonlinear finite element model
P_L, P_{cL}	axial force reaction at the left end of beam
P_r	factored axial strength for the compression column from S16
P_r	reflected blast pressure
P_R, P_{cR}	axial force reaction at the right end of beam
P_0	ideal dynamic load with an instantaneous rise time with infinite duration
Q	base shear
Q_y	base shear corresponding to yield lateral displacement (ordinate of point of significant yield)
R	response modification factor (product of R_d and R_o); right (compression column) side; standoff distance
R_o	overstrength-related force modification factor
R_d	ductility-related force modification factor
R_y	ratio of expected-to-nominal yield stress
S_a	design spectral acceleration
S_d	spectral displacement

S_{diag}	center-to-center distance between infill plate perforations
S_h	plastic hinge location in beams' ends from the face of the column
t	time in second
t_d	triangular pulse load duration
T	equivalent loading duration for a rectangular pulse with the same peak and impulse value as the actual decaying load
T_n	natural period of vibration of a SDOF system
T_1	fundamental period
T_{1e}	effective fundamental period
u_f^{pl}	equivalent plastic displacement at failure
V_b	base shear
V_{bL}	total shear force reaction at the left end of a beam due to infill plate forces
V_{bR}	total shear force reaction at the right end of a beam due to infill plate forces
V_{cL}	sum of the left column shear forces above and below the beam
V_{cR}	sum of the right column shear forces above and below the beam
V_D	design base shear
V_e	elastic design base shear
V_F	shear forces in columns due to frame action
V_G	shear forces in beams due to gravity forces
V_h	shear forces in beams due to frame action
V_I, V_{Ib}	beam shear force reaction due to unbalanced infill plate force
V_{Ic}	column shear force reaction due to horizontal component of the infill plate force
$V_{I,E}$	shear force reaction due to unbalanced infill plate force in the equivalent beam
V_L	shear force reaction at the left end of beam
V_M, V_{Mb}	beam shear force reaction due to distributed moment along the length of the beam (m)

V_{Mc}	column shear force reaction due to distributed moment along the length of the column (m_c)
V_R	shear force reaction at the right end of beam
V_y	effective yield strength; yield resistance (elastic limit) of a SDOF system
V_{yo}	yield strength of SPSW under out-of-plane direction loading
V_1	maximum elastic base shear for the fundamental mode
w	infill plate thicknesses
W	effective seismic weight; explosive charge weight
W_E	external work
W_I	strain energy
x	abscissa of the response spectrum curve
x_s	maximum static displacement
X	dimensionless impulse in pressure-impulse diagram
X_{max}	maximum dynamic displacement
y	ordinate of the response spectrum curve
Y	dimensionless load in pressure-impulse diagram
Z	scaled distance
α	slope of the inclined asymptote of a response spectrum curve
α_i	tension field angle from vertical direction
α_1	effective mass ratio of the fundamental mode
β	beam's ends moment reduction factor; y-intercept of the horizontal asymptote of a response spectrum curve
β_c	reduction in the plastic moment capacity of the column at the base due to the presence of the axial force
Γ_1	fundamental mode modal mass participation factor
δ	first story lateral displacement; column lateral displacement between its ends; centerline deflection of a component under blast load
δ_m	maximum displacement of a SDOF system
δ_{m0}	maximum displacement of a SDOF system under ideal dynamic load with an instantaneous rise time and infinite duration

δ_y	first story yield lateral displacement; yield displacement of a SDOF system
δ_{yo}	yield displacement of SPSW under out-of-plane direction loading
δ_r	maximum roof lateral displacement
$\delta_{r,y}$	roof yield lateral displacement
δ_t	target displacement
Δ_{ax}	axial deformation demand of a component
Δ_e	roof displacement for a linear elastic MDOF system
Δ_y	roof displacement corresponding to yield strength
Δ_{yd}	yield axial deformation of a component
$\Delta\omega_{(b)x}$	horizontal component of the unbalanced infill plate forces on the beam
$\Delta\omega_{(b)y}$	vertical component of the unbalanced infill plate forces on the beam
Δ^*	lateral deflection reduction at the top of the second story
ϵ_h	axial strains in the height of the infill plate
ϵ_L	axial strains in the width of the infill plate
ϵ_p^*	effective plastic strain
ϵ^{pl}	plastic strain at a material point
ϵ^{pl}_0	effective plastic strain at the verge of damage
ϵ^{pl}_f	equivalent plastic strain at failure
$\underline{\epsilon}_p$	plastic strain tensor
ϵ_u	uniaxial plastic strain of the material where the failure is initiated
η	stress triaxiality parameter
θ	rotational demand of a component in degree
θ_y	yield rotation of a component
κ_s	empirical damage material parameter
λ	shear stress ratio parameter
μ	first story ductility ratio; ductility ratio of a SDOF system
μ_0	ductility ratio of a SDOF system under ideal dynamic load with an instantaneous rise time and infinite duration

μ_i	lateral load distribution over the wall height at story i as a fraction of the base shear
μ_r	roof ductility ratio
μ_t	ductility ratio at roof target displacement
ζ_1	critical damping ratio of the fundamental mode
σ	stress applied to a plate that initiated a crack
$\underline{\sigma}$	stress tensor
σ^*	effective stress
σ_m	mean stress
σ_1	major principal stress in the infill plate
σ_2	minor principal stress in the infill plate
$\sigma_{H,CD}$	infill plate average horizontal (normal) stress applied to the compression column based on capacity design method
$\sigma_{H,CD}^*$	infill plate average horizontal (normal) stress applied to the compression column based on capacity design method and modification to account for the two-dimensional stress state in the infill plate
$\sigma_{H,FE}$	infill plate average horizontal (normal) stress applied to the compression column from nonlinear finite element model
σ_y	uniaxial (tension strip) yield stress of the infill plate
σ_{θ}^{pl}	yield stress corresponding to the effective plastic strain at the verge of damage
$\tau_{,CD}$	infill plate average vertical (shear) stress applied to the compression column based on capacity design method
$\tau_{,CD}^*$	infill plate average vertical (shear) stress applied to the compression column based on capacity design method and modification to account for the two-dimensional stress state in the infill plate
$\tau_{,FE}$	infill plate average vertical (shear) stress applied to the compression column from nonlinear finite element model
τ_{max}	maximum shear stress
ϕ	ratio of maximum shear stress to effective stress
$\phi_{1,r}$	ordinate of the fundamental mode shape at the roof node

ψ	correction factor that takes into account the effects of nonlinear response on the maximum lateral deformation of SPSW system
Ψ	absolute ratio of minor to major principal stresses in the infill plate
ω_h	column flexibility parameter
ω_L	end-panel flexibility parameter
ω_n	natural angular frequency
ω_x	horizontal components of the infill plates force
ω_y	vertical components of the infill plates force

1. INTRODUCTION

1.1 Foreword

Steel plate shear walls (SPSWs) are an efficient lateral load-resisting system capable of bracing a building against different lateral loads including wind, earthquake, and blast. The system consists of thin steel plates installed between the building beams and columns, and the beam-to-column connections can be shear (simple) or moment (rigid) connections. The system possesses exceptional properties to resist severe cyclic lateral loadings, including a high level of lateral stiffness, shear resistance, and ductility, which results in significant energy dissipation capacity. In the case that beam-to-column moment connections are used, the system will have a high level of redundancy and robust resistance against cyclic degradation.

SPSWs have traditionally been perceived as being a system suitable only for structures located in zones of high seismicity, and most of the previous research has focussed on this application. As a result, design and detailing requirements have become highly onerous, and the system is economical only for as a ductile system in high seismicity regions. On the contrary, developing applications for low and moderate seismic regions—with a focus on economics—has largely been neglected.

Current design methods for SPSWs, such as those in S16-09 (CSA 2009) and ANSI/AISC 341-10 (AISC 2010), are primarily based on the capacity design method. Conventional SPSW systems implement moment-resisting beam-to-column connections and consequently constitute a dual system. The seismic energy is dissipated through yielding of the infill plates and formation of plastic hinges at the ends of the beams and bases of the columns. To maximize the ductility and energy dissipation capacity of the system, no yielding is allowed in other locations of the system.

Although the limited-ductility SPSW option exists in S16-09 (CSA 2009), there is

a general lack of understanding about the performance of this system and its design provisions are rarely applied in practice. Moreover, despite the unique advantages of SPSW systems in industrial buildings in terms of ease of installation and mobility characteristics, the performance of this system under accidental blast loading has never been investigated.

This research aims to provide more economical solutions for buildings, along with convenience in design and construction, while improving safety and reliability. The seismic design method proposed for low and moderate seismic regions is verified by a large-scale two-story SPSW test specimen with simple beam-to-column connections.

1.2 Scope and Objectives

The overall objective of this project is to develop new applications for SPSW systems, which includes their application in low and moderate seismic regions and as protective structures in industrial plants.

Most previous research on SPSW systems has been conducted with the primary aim of maximising ductility and robustness under severe seismic loading. This design philosophy has resulted in onerous capacity design rules and a relatively high cost of the system in comparison with other options. One driving force behind this research was the fact that when the SPSW system is designed based on current capacity design methods, it renders large column sections. This, along with the application of moment-resisting beam-to-column connections, makes the system uneconomical for low and moderate seismic regions. The research aims to develop an economical SPSW system suitable for these region by introducing some modifications in system configuration and design methods.

Since the dual performance and added redundancy from moment-resisting beam-to-column connections is not necessary for achieving acceptable performance for SPSWs in low and moderate seismic regions, the use of simple beam-to-column

connections is proposed. This type of connection is also suitable for modular construction, which facilitates its erection and improves the quality of construction by eliminating field welds. Also, since the system possesses a high level of inherent ductility, minor yielding in the columns could be acceptable for low and moderate seismic applications.

A large-scale two-story test specimen using double-angle beam-to-column connections and readily-available hot-rolled infill plates is designed and tested under lateral cyclic loading concurrent with constant gravity loads. Modular construction was used in the specimen. One objective of the test was to verify the proposed design method and philosophy for low and moderate seismic regions, and observe the overall seismic performance of the designed wall. The other objective was to evaluate the performance of the details and modular construction used in the specimen, including the performance of the conventional double angle connections and using an economical single-sided infill plate splice.

Using the data from the test, as well as previous research on ductile SPSW systems, a three-tier performance-based framework of capacity design provisions for SPSW systems is proposed. The design provisions are applied to two design examples and the design methods are verified against the available physical test data.

Although all the special characteristics of SPSW systems in seismic applications—especially the high levels of redundancy, ductility, and energy dissipation capacity—are desirable for protective structures, their application in this regard has been largely neglected. Another objective of this research was to explore the possible application of SPSWs as a protective structure in industrial plants by means of iso-response curves for both in-plane and out-of-plane blast orientations. Since development of iso-response curves requires advanced and comprehensive numerical models and simulations, an analytical normalization method is proposed, which produces dimensionless iso-response curves.

1.3 Outline of the Research

In order to make the SPSW system more economical, this research focuses on the most critical and costly areas, which are the columns and beam-to-column connections. To make the columns more economical, two approaches are conceivable. First, the design demands on a column can be reduced by introducing special details such as simple beam-to-column joints. Second, a smaller column section can be justified by proposing an appropriate capacity design approach that acknowledges lower ductility demands in low and moderate seismic regions. The use of simple beam-to-column connections also facilitates the adoption of modular construction techniques, which has the potential to reduce the cost of the system greatly.

Another method that has been proposed for reducing demands on the columns of SPSWs is using perforated infill plates to reduce the lateral shear resistance of the wall when the infill plate thickness is larger than that required to resist the seismic design forces. It was assumed implicitly that since the method reduces the lateral shear resistance of the system, it also reduces the force demands on the columns. Chapter 2 studies the efficiency of this method to reduce the column design force demands. In order to compare these demands in different SPSW systems including solid and perforated infill plates, a seismic zone-independent performance-based design method is proposed. The method estimates the target displacement based on the ductility-related force modification factor, R_d . This performance-based method is used as a tool to evaluate the effectiveness of using a perforated infill plates to reduce the force demands on SPSW columns.

Since SPSWs in low and moderate seismic regions can be designed to experience relatively low ductility demands, the required redundancy is smaller than for a ductile wall. As such, simple beam-to-column connections are selected for this application. Based on the performance-based method developed in Chapter 2 and detailed numerical studies, an appropriate SPSW system suitable for low and moderate seismic regions is proposed and designed in Chapter 3. The seismic

performance of the proposed modular system and its standard double-angle beam-to-column connections were verified by large-scale test under vertical gravity loads concurrent with reversing lateral loads at each floor level. While the test specimen was designed with economics as a priority, its performance outcome is evaluated and discussed.

Chapter 4 studies the internal force distribution in the beams of SPSWs with simple beam-to-column connections. A simple and powerful analysis method, based on capacity design principles and extensive nonlinear finite element simulations, is presented for beam's axial force demand evaluation. The effect of lateral force transfer from the roof and floor diaphragms on the beam's axial force demand is studied. The various components of the shear and bending moment demands on the beams of the system are also investigated. Where the infill plates above and below an intermediate beam have similar thicknesses, a method that is complementary to the traditional capacity design assumption of full yielding in both plates is presented to estimate the bending moment and shear force demands on the beam. The internal force demand estimation methods are verified against the experimental results from Chapter 3.

Based on new and previous experimental data and the design methods developed in Chapters 3 and 4, a holistic and sound basis for capacity design of SPSWs for three different performance levels—including limited-ductility, moderately ductile, and ductile—has been provided in Chapters 5 and 6. Two target yield mechanisms associated with two key performance levels, namely ductile and limited-ductility, are identified in Chapter 5. The capacity design principles associated with these two performance levels are discussed. A complementary performance level for moderately ductile walls is also proposed, which results in performance between ductile and limited-ductility walls. The nature of the forces from the infill plate applied to the boundary frame elements is discussed in detail, and the reasons for achieving conservative column design forces in current capacity design methods are described. Based on the development principles in

Chapter 5, the capacity design provisions for limited-ductility walls are presented in Chapter 6. Recommended modifications to current capacity design methods for ductile walls are also described. Capacity design provisions for moderately ductile walls are rationalized based on the proposed ductile and limited-ductility walls provisions, and two different approaches for achieving the moderately ductile performance level are offered. The new design provisions are then applied to design examples and discussed in the context of available test results for multi-story walls.

The application of the SPSW system as a protective structure to mitigate the effects of accidental blast loading has received little research attention. Chapter 7 studies the performance of SPSWs as a potential protective structure in industrial plants. A comprehensive numerical model capable of capturing key parameters in blast design is developed. The constitutive model for the steel material includes mixed-hardening, strain rate effects, and damage initiation and evolution. Pressure-impulse diagrams for both in-plane and out-of-plane blast orientations, along with corresponding weight-standoff distance diagrams, are developed using the numerical model. By transforming the wall system into a generalized single-degree-of-freedom system, an analytical method is developed that produces dimensionless iso-response curves.

Chapter 8 provides a summary of the research and a discussion of the design recommendations. Also, areas for further research are highlighted.

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2. COLUMN DEMANDS IN STEEL PLATE SHEAR WALLS WITH REGULAR PERFORATION PATTERNS USING PERFORMANCE-BASED DESIGN METHODS¹

2.1 Introduction

Steel plate shear wall (SPSW) systems have demonstrated excellent performance as a lateral force resisting system. Experimental and numerical studies have shown high shear strength and outstanding ductility and redundancy. When the system is designed with steel grades commonly available in North America, the high inherent shear strength of the system often results in the need for only very thin infill plates that may cause handling and welding considerations to govern the thickness chosen. According to the capacity design concept, the selection of thicker infill plates imposes larger shear, moment, and axial force demands on the columns of SPSWs.

Several methods have been proposed to reduce the excessive demands on the columns due to the panel over-strength. These methods can be classified into two main approaches. In the first, the share of the column demands arising from the effect of frame action is reduced by using semi-rigid beam-to-column connections, using reduced beam sections with rigid beam-to-column connections, and/or using a horizontal strut at the mid-height of each story. In the second approach, the share of the column demands arising from the effect of tension field action in the infill plates is reduced by decreasing the strength of the infill plates themselves. The use of light-gage or low-yield-point steel for the infill plates is one method. Another is to reduce the strength of the infill plate by introducing a regular pattern of perforations in the form of circular holes. In the latter method, the decrease in shear strength of the system depends on the diameter of the holes and their center-to-center spacing. It has generally been

¹ A version of this chapter will be submitted for publication in the *Journal of Structural Engineering*, ASCE.

intuitively assumed that there is a direct correlation between the reduction of lateral strength of the wall associated with the infill plate perforations and the alleviation of column force demands. However, in most research programs on SPSW systems with perforated infill plates, the lateral performance of the wall has been investigated but little attention is given to the internal force demands on the columns. In this study, the focus is specifically on the nature of the demands on the columns of multi-story SPSWs with regular perforation patterns in their infill plates.

2.2 Background

Roberts and Sabouri-Ghomi (1992) conducted a series of tests on small-scale steel shear panels, each with a centrally-placed circular opening. The infill plates, referred to as shear panels, all had a 300 mm depth with either a 300 mm or a 450 mm width. Two different shear panel thicknesses, equal to 0.83 mm or 1.23 mm, were selected for each of the panel sizes, making four different panel types. The thinnest panels were aluminum alloy and the remainder of the materials were steel. Only the modulus of elasticity and 0.2% offset yield stress were reported for each infill panel material. Four different diameters for the central circular opening were used in each panel type, making 16 different specimens in total. The selected hole diameters were equal to 0 (no opening), 60, 105, or 150 mm. Each panel was installed in a stiff, pin-ended boundary frame and the frames were loaded at two opposite corners in the direction of the panel diagonal. Each specimen was tested under quasi-static cyclic loading by applying at least four complete cycles of loading with gradually increasing peak lateral displacements. All the specimens exhibited ductile behavior with stable hysteresis loops. Based on the test results, the shear strength and lateral stiffness of the perforated panels were approximated by applying a reduction factor, which is a linear function of the ratio of the hole diameter to the panel depth, to the corresponding properties of a solid panel.

A quasi-static cyclic test on a single-story SPSW specimen with a perforated infill

plate was conducted by Vian (2005). The frame centerline dimensions of the specimen were 2 000 mm in height and 4 000 mm in width. Reduced beam sections were used with rigid beam-to-column connections. The infill plate was fabricated from low-yield-strength steel, with measured yield and ultimate stresses of 165 MPa and 305 MPa, respectively, and was 2.6 mm in thickness. The yield strength of the frame members was nominally 345 MPa. A total of 20 holes of 200 mm diameter were used in the infill plate in a staggered pattern of four horizontal rows of five holes each. The holes' centers were arranged at a 45° angle on a rectangular coordinate system with grids 300 mm apart along both the horizontal and vertical directions. The specimen was tested under cyclic loading. Beam plastic hinges occurred at the reduced section, and the specimen exhibited ductile behavior with stable hysteresis curves.

Purba and Bruneau (2009) studied the behavior of perforated SPSWs with a series of numerical models subjected to monotonic loading. Models of individual perforated strips were studied and their load versus elongation response was compared to the behavior of complete single-story models with the same overall dimensions and number of holes as the wall tested by Vian (2005). Perforation diameters in the infill plate from 10 mm to 300 mm were considered, and the plate thickness for all models was 5.0 mm. They found that the results from an individual strip analysis based on the perforation pattern geometry can accurately predict the behavior of the perforated panel provided that the hole diameter is less than 60% of the overall strip width. It was suggested that the shear strength of the perforated SPSW panel can be evaluated by applying a “regression factor” of 0.7 to the equation proposed by Roberts and Sabouri-Ghomi (1992) for centrally-placed circular openings.

Driver and Moghimi (2011) studied the feasibility of different SPSW arrangements to be used in lower seismic regions. Three modular construction schemes, all with simple beam-to-column connections, were proposed that can potentially result in a considerable reduction in construction costs. In order to

identify the best possible system for lower seismic regions, the lateral performance of different two-story SPSW systems were studied with comprehensive finite element models. As part of the system optimization, different infill plate details were studied, including the potential inclusion of a uniform pattern of circular perforations. In order to evaluate the demand on the compression column of the different configurations studied, the applied horizontal and vertical forces from the infill plate adjacent to the first-story compression column and the total shear and axial force demands at the compression-column base were evaluated. As expected, the lateral shear strength of the wall with perforated infill plates was reduced considerably compared to one with a continuous plate and the forces applied directly to the columns by the infill plates also reduced. However, the total shear force demand at the column base, and possibly the moment action, in the columns actually increased. As a result of this apparently paradoxical outcome, the current study aims to investigate the performance of such systems—specifically in terms of column demands—in greater depth.

2.3 Scope and Objectives

One major parameter affecting the cost of SPSW systems is the column cross-section required. The panel over-strength due to the selection of infill plates that are thicker than required to resist the design story shear imposes large capacity force demands on the columns, which leads to a negative impact on the economic competitiveness of the system. This research investigates the appropriateness of the perforation method for the purpose of decreasing the demand on the columns.

Performance-based design principles are used to compare different systems. First, a standardized seismic hazard-independent method has been proposed to evaluate the target displacement. The method evaluates the target displacement based on the yield displacement of the system and the ductility-related force modification factor.

A numerical model is developed and validated against the available test results. Three different four-story steel plate shear wall systems have been considered. One system has no perforations, while the other two have different perforation patterns in the infill plates. Each wall is analysed for two different types of beam-to-column connection: simple and rigid. The target displacement corresponding to ductility-related force modification factors equal to 2 and 5 are evaluated for each wall based on the proposed method. The axial force and bending moment demands and the design beam-column interaction capacity for the columns are calculated. By comparing these values, the efficiency of perforation patterns on the internal force reduction are investigated.

2.4 Target Displacement Evaluation

This paper aims to investigate the lateral performance of SPSWs with perforations in their infill plates by comparing them with similar SPSW systems with no perforations. In order to compare the two systems, criteria are needed to make this comparison possible. Although capacity design provides a reliable and rational design method, the performance-based design approach has been used in this study.

The capacity design approach is a strength-based method. As such, it does not provide any information regarding the lateral deformation distribution over the height of the system when it is under the effect of the design earthquake. Moreover, it cannot be used when different performance objectives are to be satisfied for different seismic hazard levels. On the other hand, the performance-based design method eliminates these shortcomings. In this method, the performance objectives define the status of the building following a design earthquake that represents a desired seismic hazard level. Hence, this method provides a design procedure that allows for the design of lateral force resisting systems for different performance objectives under different seismic hazard levels.

In performance-based design, the target displacement, which is the maximum roof displacement under the design earthquake, δ_r , must be evaluated. Based on the Coefficient Method (ASCE 2007), the target displacement, δ_t , can be approximated as follows, in terms of the fundamental natural mode of vibration in the direction under consideration:

$$\delta_t = \delta_r = \Delta_e \psi = \varphi_{1,r} \Gamma_1 S_d(T_{1e}, \xi_1) \psi \quad (2.1)$$

where Δ_e is the roof displacement for a linear elastic multi-degree-of-freedom system, considering only the effect of the fundamental mode, $\varphi_{1,r}$ is the ordinate of the fundamental mode shape at the target (roof) node (which is equal to unity for a normalized mode shape), Γ_1 is the modal mass participation factor for the fundamental mode, $S_d(T_{1e}, \xi_1)$ is the spectral displacement at the effective fundamental period (T_{1e}) and its corresponding critical damping ratio (ξ_1) in the direction under consideration, and ψ is a correction factor that takes into account the effects of nonlinear response on the maximum lateral deformation of the system. The subscript 1 signifies fundamental mode properties of the multi-degree of freedom system.

The effective fundamental period is the fundamental period of the system, modified to account for yielding based on the idealized pushover curve. It can be evaluated from $T_{1e} = T_1 \sqrt{K_i / K_e} = 2\pi \sqrt{M / K_e}$, where T_1 is the fundamental period, M is the total effective seismic mass of the system, K_i is the elastic lateral stiffness, and K_e is the effective lateral stiffness, which is a secant stiffness calculated at a base shear force of 60% (ASCE 2007) of the effective yield strength of the system.

The correction factor ψ takes into account the effects of yielding (coefficient C_y) and pinched hysteresis cycles and cyclic strength and stiffness degradation (coefficient C_p) of the system on the maximum lateral displacement. ASCE

(2007) defines this correction factor as the product of two factors as follows:

$$\psi = C_y \times C_p = \left[1 + \frac{R_d - 1}{aT_{1e}^2}\right] \times \left[1 + \frac{1}{800} \left(\frac{R_d - 1}{T_{1e}}\right)^2\right] \quad (2.2)$$

where R_d is the ductility-related force modification factor, as defined in the National Building Code of Canada (NRCC 2010) and FEMA (NEHRP 2004), and is a measure of the extent of nonlinearity in the system. This factor represents its capability to dissipate energy through nonlinear behavior, and can be defined as the ratio of elastic base shear demand to the yield strength of the system. A site class factor, a , is defined to take into account the effect of soil conditions at the site of the building. Based on site soil properties, ASCE (2010) and NRCC (2010) have separated potential site conditions into six different classes. It is recommended that where the soil properties are not known in enough detail to determine the site class, Site Class D be assumed. ASCE (2007) defines $a = 60$ for Site Class D. The correction factors C_y and C_p may be selected as 1.0 for periods greater than 1.0 s and 0.7 s, respectively. Also, for systems with no degradation of stiffness and strength, the coefficient C_p can be selected as 1.0.

Normally, for a given design case the spectral displacement for the effective fundamental mode of vibration is evaluated based on following equation:

$$S_d(T_{1e}, \xi_1) = S_a(T_{1e}, \xi_1) g \left(\frac{T_{1e}}{2\pi}\right)^2 \quad (2.3)$$

where $S_a(T_{1e}, \xi_1)$ is the design spectral acceleration for the effective fundamental period and critical damping ratio of the structure in the direction considered, and g is the acceleration of gravity.

The target displacement can be evaluated by substituting Eqs. (2.2) and (2.3) into (2.1). This method is similar to the method proposed by ASCE (2007) for the

nonlinear static procedure. However, the method is site-dependent; that is, it depends on the seismic hazard and site class at each location and is not appropriate for general comparisons of different systems. In this study, a standardized seismic hazard-independent target displacement is proposed as a method of comparing different systems. Fig. 2.1 shows the pushover curve (roof displacement vs. base shear) of a multi-degree-of-freedom system under monotonically increasing lateral loading. The system is designed for a base shear of V_D , and the effective yield strength and corresponding roof displacement are V_y and Δ_y , respectively.

ASCE (2007) defines the ductility-related force modification factor, R_d , based on a strength ratio equal to:

$$R_d = \frac{V_1}{V_y} = \frac{\alpha_1 S_d W}{V_y} \quad (2.4)$$

where V_1 is the maximum elastic base shear for the fundamental mode, $\alpha_1 = M_{e1}/M$ is effective mass ratio of the fundamental mode, M_{e1} is the modal effective mass, and $W = Mg$ is the effective seismic weight. However, defining R_d as this strength ratio does not guarantee that the displacement ratio will be proportional to the effective lateral stiffness K_e (Fig. 2.1). In this study, R_d is defined as the following strength ratio, so that the effective stiffness of the system is maintained:

$$R_d = \frac{V_e}{V_y} = \frac{\Delta_e}{\Delta_y} = \frac{\phi_{1,r} \Gamma_1 S_d}{\Delta_y} \quad (2.5)$$

Substituting Eq. (2.5) into (2.1), the following equations for the target displacement, δ_t , and associated ductility ratio, μ_t , are obtained:

$$\delta_t = R_d \Delta_y [C_y \times C_p] \quad (2.6)$$

$$\mu_t = \frac{\delta_t}{\Delta_y} = R_d [C_y \times C_p] \quad (2.7)$$

where C_y and C_p are defined in Eq. (2.2).

Defining R_d based on Eq. (2.4) (ASCE 2007) leads to a conceptually different target displacement. Substituting Eq. (2.3) into (2.1) and replacing the term $S_d g$ with its equivalent from Eq. (2.4), the term $T_{1e} / 2\pi$ with $\sqrt{M/K_e}$, and the term V_y/K_e with Δ_y (Fig. 2.1), the following equations would be achieved for the target ductility ratio:

$$\mu_t = R_d \left(\frac{\Gamma_1}{\alpha_1} \right) [C_y \times C_p] \quad (2.8)$$

For a typical SPSW, the values of Γ_1 and α_1 are fairly constant and can be selected as 1.3 and 0.7, respectively, as shown later in the Table 2.1. This makes the coefficient Γ_1/α_1 equal to about 1.8. As such, Eq. 8 yields about 80% larger values for the target displacement than Eq. 7. Hence, the definition of R_d based on the force ratio (Eq. (2.4)) results in a decrease in effective lateral stiffness from K_e (R_d based on deformation ratio from Eq. (2.5)) to K_d (R_d based on force ratio from Eq. (2.4)) in Fig. 2.1.

Eq. (2.7) defines the ductility ratio mainly as a function of R_d . In this study, for any desired level of ductility-related force modification factor, the target displacement is evaluated from Eq. (2.6) and then the performance of the wall is studied for the ductility ratio obtained from Eq. (2.7).

2.5 Numerical Model of Steel Plate Shear Wall System

In order to study the performance of different SPSW systems under pushover analyses, a system that was tested previously and proved to have very good response under cyclic loading has been selected and studied. The system is a four-

story SPSW (Driver et al. 1998) that was tested under vertical load concurrent with increasing cyclic lateral displacement. The columns were spaced at 3.05 m center-to-center and the story height was 1.83 m for the top three stories and 1.93 m for the first story. The columns were W310×118 sections, the intermediate beams were W310×60 sections, and the top beam was a W530×82 section. The frame members met Class 1 (CSA 2009) and Highly Ductile Member (AISC 2010a) compactness requirements, and they satisfy the stiffness requirements for boundary members of SPSWs (ω_h and ω_L requirements (CSA 2009)). The mean measured infill plate thicknesses were 4.54 mm, 4.65 mm, 3.35 mm, and 3.40 mm, respectively, for stories 1 (base) to 4 (top). Moment connections were used at the beam-to-column joints and all the materials were hot-rolled steel. In the early stages of the experiment force control was used and, after achieving the yield lateral displacement for the first story, δ_y , the experiment switched to the displacement control method. Three full cycles were conducted at each deformation stage up to a deformation of $3\delta_y$, and two cycles were used at each deformation step afterward. The system achieved its maximum lateral strength at $5\delta_y$ and its lateral resistance then declined gradually to about 85% of the maximum value at $9\delta_y$. The roof lateral displacement at yield, maximum base shear, and maximum lateral deformation occurred at 32.0 mm, 92.5 mm, and 158 mm, respectively.

The commercial general-purpose finite element code ABAQUS is selected to model this wall. In order to verify the model, first the wall is analysed under the same conditions as those used in the experiment, but subjected to monotonic loading. As-built dimensions are used; however, the fish plates are not modeled and the infill plates are connected directly to the surrounding boundary elements.

All infill plates of the wall are modeled using four-node shell elements with reduced integration (S4R element), and frame elements (beams and columns) are modeled using two-node linear beam elements (B31). Material and geometric nonlinearities are both considered. The measured material properties for the

beams, columns, and infill plates are used. The von Mises yield criterion, along with the isotropic hardening rule for monotonic loading and kinematic hardening rule for dynamic loading, is used. The effect of residual stresses in the frame members is not considered in the analyses.

The story shear versus story deflection relationships of panel 1 (first story) from the analyses are compared with the experimental results in Fig. 2.2. Fig. 2.2(a) shows the response under monotonic loading, where it can be seen that the model provides acceptable results up to the ultimate strength. However, it over-predicts the strength slightly at the early stage of yielding, which is mainly because the monotonic analysis disregards the effect of softening of material due to the cyclic nature of the loading applied to the test specimen. Also, the model does not account for the gradual degradation of strength exhibited by the test specimen after a story drift of $5\delta_y$ (lateral displacement 42.5 mm) because localized material failure and low-cycle fatigue failure of the system due to cyclic response is not captured in the model.

Fig. 2.2(b) compares experimental results with the cyclic loading analysis results for two excursions at $4\delta_y$ and two at $5\delta_y$. The ultimate load is accurately predicted by the model at each displacement level and even at each excursion. Also, the model predicts the loading and unloading stiffnesses and the deflection at which the tension field in the panel becomes effective well. However, it over-predicts the shear strength of the panel slightly around the zero-displacement region. In general, the model predicts the monotonic and cyclic responses of the system up to its ultimate strength very well, and a similar model is used for the pushover analyses discussed in the next section.

2.6 Steel Plate Shear Wall Systems Investigated

In this study, three different steel plate shear wall systems have been considered. Wall A1, shown in Fig. 2.3(a), is the SPSW tested by Driver et al. (1998) and studied in the previous section numerically. Wall A2 is similar to Wall A1, but

with simple shear beam-to column connections. Fig. 2.3(b) shows Wall B1, for which all the properties are the same as Wall A1 except that it contains a regular perforation pattern in all four panels, as permitted by CSA standard S16 (CSA 2009) and AISC 341 standard (AISC 2010a). There are 23 holes in panels 1, 2 and 3, and 18 holes in panel 4. Wall C1, shown in Fig. 2.3(c), is the same as Wall B1, but with a different arrangement of holes; there are 22 holes in panels 1, 2 and 3, and 18 holes in panel 4. Walls B2 and C2 are the same as Walls B1 and C1, respectively, but with simple shear beam-to-column connections.

In all perforated walls, the holes are of equal diameter ($D = 230$ mm) and regularly spaced vertically and horizontally (at a distance of 280 mm) over the entire area of the infill plates. The regular grid of staggered holes allows the development of a diagonal tension field at 45° . The shortest center-to-center distance between perforations is $S_{\text{diag}} = 396$ mm, which makes the ratio of $D/S_{\text{diag}} = 0.58 < 0.6$. The distance between the first hole and the surrounding frame members is between $D = 230$ mm and $D+0.7 S_{\text{diag}} = 507$ mm. Hence, the two perforation patterns satisfy all the requirements of CSA S16 (CSA 2009) and AISC 341 (AISC 2010a) for perforated infill plates. The factored shear resistance of the perforated wall is equal to that of a conventional SPSW times the reduction factor $1-0.7D/S_{\text{diag}}$, which for the patterns selected is equal to 0.6, except that the clear distance between columns is used instead of center-to-center.

2.7 Performance Criteria

2.7.1 Classification of structural components and actions

The acceptability of force or deformation in each element depends on the action and component types. Each component is classified as primary or secondary. A structural component that is designed to resist the earthquake effects is classified as a primary component, while one that is not designed for the seismic forces is classified as secondary (ASCE 2007). As such, all components of a SPSW system are primary components.

Each action is classified as deformation-controlled (ductile) or force-controlled (non-ductile) based on the component strength versus internal deformation curve for that action. If the elastic part of the curve up to the component strength is followed by plastic response in the form of a yield plateau with non-negligible residual strength, the action shows ductile behavior, and it is classified as a deformation-controlled action. If the elastic part of the curve up to the component strength is followed by a loss of strength, the action shows non-ductile or brittle behavior and is classified as force-controlled (ASCE 2007).

Since the behaviors of the two types of action are inherently different, different component strengths are considered for each action in the component strength versus internal deformation curve. The component strength for a deformation-controlled action is the expected strength, which is the mean value of the component resistance for the action at the deformation level anticipated. The expected component strengths can be evaluated by code design equations using expected material properties, and taking the resistance factor equal to 1.0. The strain hardening may be incorporated in the component strength versus deformation curve. The component strength for a force-controlled action is a lower-bound strength, which is taken as the mean value minus one standard deviation of the component strength of the component for the action (ASCE 2007). It can be estimated by code design equations using lower-bound material properties and a resistance factor equal to 1.0.

Lower-bound material properties are defined based on mean values minus one standard deviation of tested material properties, while expected material properties can be established based on mean values of tested properties. Nominal material properties can be taken as the lower-bound properties. ASCE (2007) has introduced factors that translate the lower-bound steel properties to the expected-strength steel properties.

2.7.2 Capacities for different actions and components

When a nonlinear procedure is used, the component capacities are checked against component demands calculated at the target displacement. The component capacity for deformation-controlled actions is taken as the permissible inelastic deformation capacity considering all coexisting forces and deformations. The deformation capacity is defined in chapter 5 of the ASCE (2007) standard for different steel structural systems and is usually expressed in terms of the yield displacement. Conversely, the component capacity for force-controlled actions is taken as the lower-bound strength, considering all coexisting forces and deformations, and can be evaluated as mentioned above.

If the axial force demand is less than 10% of the axial strength, the component is considered a beam; otherwise, it is a column. The beam-to-column connections can be fully-restrained or partially-restrained. Walls A1, B1, and C1 all use fully-restrained moment connections, while Walls A2, B2, and C2 use partially-restrained moment connections.

Flexure of beams is considered to be a deformation-controlled action. Axial compression of columns is considered a force-controlled action, with the lower-bound axial compression capacity P_{CL} . Flexural loading of columns with an axial load present at the target displacement of less than $0.5P_{CL}$ is considered deformation-controlled; for greater axial loads, it is considered force-controlled. Steel columns subjected to axial tension or combined axial tension and bending moment are considered deformation-controlled (ASCE 2007).

The component expected strength of beams and other flexural deformation-controlled members is the lowest value obtained from the limit states of yielding, lateral-torsional buckling, local flange buckling, or shear yielding of web. The lower-bound axial compressive strength of steel columns is the lowest value obtained for the limit states of column buckling, local flange buckling, or local web buckling. The expected axial strength of columns in tension is equal to the

cross-sectional area times the expected yield stress of the material (ASCE 2007). For steel members subjected to the combined actions of axial compression and bending moment, the lower-bound strength is expressed in the form of a strength interaction ratio and is equal to the lowest value for cross-sectional strength, overall in-plane member strength, and lateral-torsional buckling strength, which is discussed in the next section.

All components of SPSWs can be considered under deformation-controlled actions except the connection of the infill plates to the surrounding frame, axial compressive load in the beams, axial compressive load in the columns, and axial compressive load and bending moment in the columns when the axial load at the target displacement is not less than $0.5P_{CL}$.

The infill plate connections to the surrounding frame members are designed based on the capacity design method to transfer forces developed by the expected yield strength of the infill plate. For the top beam or an intermediate beam with different infill plate thicknesses above and below, the axial force varies from tension to compression along its length, and the action may not be considered force-controlled. When the infill plate thicknesses above and below an intermediate beam are equal, the beam is under a relatively constant compressive force and bending moments (if the infill plate yield strain above and below the intermediate beam is not uniform), but the axial force level is normally smaller than $0.5P_{CL}$ and therefore the action would be deformation-controlled. Among the force-controlled components of SPSWs, therefore, the columns—and especially the compression column—are the most critical elements. The compressive force in the critical column is usually very large and accounts for more than 50% of the axial compressive capacity of the member. In such cases, the combined action of axial compressive force and bending moment is considered force-controlled. In the following section, the bending moment and axial compressive forces of both columns are investigated.

2.7.3 Comparison of ASCE (2007) with design codes for new buildings

ASCE (2007) defines two Basic Safety Earthquake (BSE) hazard levels, namely BSE-1 and BSE-2. The 5%-damped spectral response acceleration for the BSE-2 earthquake hazard level has a 2% probability of exceedance in 50 years. This hazard level is consistent with the Maximum Considered Earthquake (MCE) in NEHRP (2004). The 5%-damped spectral response acceleration for BSE-1 is defined as the smaller of the spectral response for the earthquake hazard level with a 10% probability of exceedance in 50 years and two-thirds of the spectral response for BSE-2. Design codes for new buildings in the U.S. usually utilize the two-thirds of the 5%-damped spectral response acceleration for the MCE (BSE-2) hazard level as their seismic design loads (NEHRP 2004). (NRCC (2010) defines the MCE as the seismic design event.) However, when the criteria of ASCE (2007) are used for new buildings, slightly higher than the seismic rehabilitation goal of life safety performance levels for a BSE-1 hazard level are needed in order to achieve a system comparable with a similar system designed based on design codes.

2.8 Pushover Analysis Results

Pushover curves are developed assuming a uniform lateral load distribution pattern over the height of each wall. The deformation-control loading scheme is used, and the effects of gravity loads are considered on the columns of each wall. The pushover curves for all SPSWs investigated are shown in Fig. 2.4. The difference between the curves of Wall A1 and Wall B1 or C1 indicates that the proposed reduction factor based on S16 (CSA 2009) and AISC 341 (AISC 2010a), $1 - 0.7 \cdot D/S_{diag} \approx 0.6$, provides a conservative evaluation for shear resistance of the perforated walls. At the target displacement corresponding to $R_d = 5$, the shear resistance of Walls B1 and C1 to Wall A1 are 0.76 and 0.79, respectively, and the same ratio for the Walls B2 and C2 to Wall A2 are 0.67 and 0.71, respectively.

2.8.1 Column internal forces demands

Previous research has shown that the demand on columns of SPSWs is complex

and notably large. Column shear, moment, and axial force demands result from the frame action of the boundary frame and tension field action in the infill plates. The level of the demand depends on the geometry of the system (such as bay width and story height, number of stories, and thickness of the infill plate), type of beam-to-column connection (rigid, semi-rigid, or simple), and mechanical properties of the material, especially the infill plate. The effects of infill plate perforation on the column demands are studied in the context of the performance-based design method. For this purpose, by assuming a desired level of R_d , the target displacement is evaluated from Eq. (2.6). The appropriate values for R_d could be 2, 3.5, and 5, which correspond to three ductility levels, such as limited-ductility (or Ordinary), moderately ductile (or Intermediate), and ductile (or Special) SPSWs, respectively. At the target displacement, the deformation demand on members under deformation-controlled actions must not exceed the expected deformation capacity, and the force demand on members under force-controlled actions must not exceed the lower-bound strength.

Table 2.1 shows the dynamic properties (α_1 and T_1), yield strength and displacement, and target (roof) ductility ratio, displacement, and displacement ratio (h = overall height of wall) for both $R_d = 2$ and 5 for the walls under investigation. The results for Walls B1 and B2 in Table 2.1 are applicable to Walls C1 and C2, respectively, since they are similar but except for the perforation pattern. The table shows that the effective mass ratio and modal mass participation factor of the fundamental mode are similar for all the walls and equal to 0.7 and 1.3, respectively. Also, they have similar target displacement ductility ratios for each level of the ductility factor, R_d . However, since the perforated walls have a smaller yield displacement compared with the wall with solid infill plates, they have a smaller target displacement for the same level of the ductility factor. It can be seen from the table that the target displacements for $R_d = 2$ and 5 are approximately 50 mm and 150 mm, respectively, for all the walls. Based on the test results of Driver et al. (1998), the roof lateral displacement at yield, maximum base shear, and maximum displacement occurred

at 32.0 mm, 92.5 mm, and 158 mm, respectively. As such, the 50 mm roof displacement is between the Immediate Occupancy and Life Safety performance levels, and the roof displacement of 150 mm is close to the Collapse Prevention performance level. However, the test results show the performance of the wall under cyclic loading, which causes considerably more tearing and low-cycle fatigue failure in the system than monotonic loading.

The axial force and moment demands on both columns of each wall for $R_d = 2$ and 5 are shown in Figs. 2.5 and 2.6, respectively. As can be seen from Figs. 2.5(a), 2.5(b), 2.6(a), and 2.6(b), the axial force demands for the perforated walls are generally lower than for the solid walls. However, Figs. 2.5(c), 2.5(d), 2.6(c), and 2.6(d) show that the column moment demands for the perforated walls for both connection types tend to be larger than for the solid walls. Moreover, these figures show that although the differences between Walls B and C are apparently minor and they have a similar target displacement and shear resistance, the differences in the right (compression) column moment demands near the intermediate connections of the walls with simple connections are considerable and up to 20%, suggesting an inherent uncertainty in the response of perforated SPSWs that warrants further study.

The higher moment demand in the columns of walls with perforated infill plates is attributed chiefly to two effects. First, the perforations cause an increase in the participation of frame action in the response of SPSWs with moment connections. Second, in both walls with simple and rigid connections, the perforations decrease the lateral stiffness of the infill plate and the wall in total, and that changes the lateral deformation of both columns over the height of the walls, having a significant impact on the moment demands in both columns at different stories. All in all, the column design demands for perforated walls could potentially increase in some cases—where much of the increased demand is located in the connection regions—as compared to an equivalent wall with solid infill plates, which is in contrast to the main idea behind the development of the system.

2.8.2 Column design checks

In order to study the actual change in the column demands in detail, design equations are used that take into account the interaction of axial compression and moment. The lower-bound strengths of steel members subjected to the combined actions of axial compression and bending moment are calculated for doubly-symmetric members based on two design standards. S16 (CSA 2009) defines the beam-column strength ratio as the lowest value obtained from cross-sectional strength, overall in-plane member strength, and lateral-torsional buckling strength. For doubly-symmetric rolled compact members subjected to flexure and compression, with moments primarily about their major axis, AISC 360 (AISC 2010b) defines the strength ratio as a combined approach by checking an interaction equation or the lower value for two independent limit states of in-plane instability and out-of-plane buckling or lateral-torsional buckling.

Tables 2.2 and 2.4 show the compression force–moment interaction ratios for $R_d = 2$ and 5, respectively, based on both S16 and AISC 360. The strength ratios presented are the maximum of the above-mentioned limit states, which in almost all cases is that of the cross-sectional strength. The first two stories of the left columns are subjected to tensile loads and, as such, they are considered as fully deformation-controlled elements so no strength ratio is given. The strength ratios for the upper two stories of this column are typically very low. For the first two stories of the right columns, some of the beam-column strength ratios are greater than 1.0. The reason is attributed to the fact that the design equations are a conservative approximation of true capacities, and also the material models consider strain hardening, while all the interaction equations are checked using yield values. The results show that the perforations increase the strength interaction values in the upper stories of both columns and in the lower stories the improvement is mostly negligible.

The results of this research are consistent with the results reported by Driver and Moghimi (2011) on different wall configurations. They showed that although the

perforated infill plate applies a smaller level of horizontal force to the column, the column base shear is actually larger than in a similar wall with solid infill plates. The reason for this is attributed to the augmented frame action and change in lateral deformation of the columns due to the perforations in the infill plates, as mentioned previously.

Tables 2.3 and 2.5 show the deformation ductility ratios and performance levels of each column at each story for $R_d = 2$ and 5, respectively. For the first two stories of the left columns, which are subjected to tension, the axial deformation ductility ratios are shown. All columns with an axial compressive force at the target displacement that exceeds 50 % of the lower-bound axial compressive strength (first two stories of the right column in all cases) are considered force-controlled for both axial load and flexure, and are identified in the table by the symbol FC. The rest of the columns (both left and right columns in the top two stories) are subjected to deformation-controlled actions, and the rotational ductility ratio of the member is calculated for the performance check. The rotational demand, θ , is calculated as a chord rotation by dividing the relative lateral deformation in each story by the story height. The yield rotations of the columns are also defined based on the chord rotation. Similar to what is proposed in ASCE (2007), the point of contraflexure is assumed at the mid-length of the elements, and the calculated rotation is then reduced linearly for the effect of axial force in the column members.

The deformation-controlled actions define the performance level of the system by checking the deformation demand against the expected deformation capacity. However, force-controlled actions must always satisfy the strength criteria for any performance level and seismic hazard level. As such, the performance level of each column at each story is evaluated in Tables 2.3 and 2.5, wherever the columns are subjected to deformation-controlled columns actions (either axial or rotational). ASCE (2007) specifies the expected deformation capacities at each performance level (Immediate Occupancy, Life Safety, and Collapse Prevention)

for stiffened steel plate shear walls, where the infill plates are sufficiently stiffened to prevent buckling. Since the behavior of such a system is inherently different from unstiffened SPSWs, in the current study the acceptance criteria for steel moment frames are used to evaluate the performance levels of each column. Since the infill plate reduces the deformation demands at the beam-to-column joints, the moment frame expected ductility capacities provide conservative performance evaluation metrics for SPSW systems.

Table 2.3 shows that at $R_d = 2.0$, the bottom two stories of the right columns of the different walls are all subjected to force-controlled actions, while the upper two stories resist deformation-controlled actions at Life Safety performance level. The left columns are subjected to deformation-controlled actions, mainly at the Life Safety performance level. Table 2.5 shows that at $R_d = 5.0$, again the bottom two stories of the right columns of the different walls are subjected to force-controlled actions, but the third and fourth stories resist deformation-controlled actions at the Collapse Prevention and Life Safety performance levels, respectively. The left columns are subjected to deformation-controlled actions, mainly at the Life Safety performance level, although other regions of the columns meet either Collapse Prevention or Immediate Occupancy. The results show that the perforations provide either limited or no improvements to the performance of deformation-controlled elements. The performance levels for R_d equal to 2 (Table 2.3) are essentially the same, since the perforations have only a small effect on the deformation ductility value in the different columns. However, the perforations may increase the rotational ductility demand at $R_d = 5.0$ (Table 2.5) of the walls with simple joints for both the right and left columns, and, as a result, it makes the performance levels worse in the upper stories at those cases.

2.9 Conclusion

In this paper, a standardized method has been presented to evaluate the target displacement of any lateral force resisting system in terms of the yield

displacement of the system and the ductility-related force modification factor, R_d . Also, it provides a suitable method for comparing different systems with the same value of R_d , since the method is not seismic-zone dependent.

The concept of perforated SPSWs was introduced primarily to alleviate the severe force demands on the columns of SPSWs by degrading the lateral shear resistance of the system. This study has shown that although the perforations reduce the shear capacity of the infill plate considerably, they may not reduce the force demands on the columns. The results indicate that although the perforated infill plate consistently reduces the axial force demands in the columns, it increases the contribution of frame action in the system and changes the lateral deformation distribution of the columns. These effects increase the moment demands on the columns, especially in the upper stories. The increase in the moment is such that the perforations in some cases increase the design strength ratios based on the combined actions of axial compression and bending, which is critical for the force-controlled columns. Also, in general the perforations provide no improvements to the performance of elements under deformation-controlled actions.

The results of this research show that a minor change in perforation pattern may cause up to 20% differences in column moment demand in some regions. This suggests an inherent uncertainty in the response of perforated SPSWs. However, the effect of the perforations needs to be investigated at different performance levels in walls with different geometries, numbers of stories, panel aspect ratios, perforation patterns, and boundary frame cross-sections.

Table 2.1 Wall properties

Wall	α_1	Γ_1	V_y	Δ_y	$R_d = 2.0$			$R_d = 5.0$		
					μ_t	δ_t	δ_t/h	μ_t	δ_t	δ_t/h
			kN	mm	—	mm	%	—	mm	%
A1	0.72	1.32	2646	24.1	2.2	51.8	0.72	6.5	156	2.2
A2	0.72	1.32	2638	24.3	2.2	52.3	0.73	6.5	157	2.2
B1/C1	0.74	1.33	1908	22.8	2.2	49.1	0.69	6.6	149	2.1
B2/C2	0.74	1.33	1815	22.0	2.2	47.2	0.66	6.5	143	2.0

Table 2.2 Column strength ratios for $R_d = 2.0$

	Story	Wall A1		Wall A2		Wall B1		Wall B2		Wall C1		Wall C2	
		S16	AISC	S16	AISC	S16	AISC	S16	AISC	S16	AISC	S16	AISC
Left Column	1	—	—	—	—	—	—	—	—	—	—	—	—
	2	—	—	—	—	—	—	—	—	—	—	—	—
	3	0.20	0.13	0.21	0.13	0.32	0.27	0.34	0.29	0.30	0.24	0.32	0.27
	4	0.20	0.14	0.22	0.17	0.25	0.21	0.34	0.30	0.24	0.19	0.31	0.26
Right Column	1	1.14	1.17	1.12	1.14	1.11	1.13	1.01	0.97	1.11	1.14	1.01	0.97
	2	0.84	0.85	1.00	0.89	0.85	0.87	0.76	0.69	0.81	0.83	0.83	0.74
	3	0.62	0.64	0.58	0.60	0.67	0.69	0.62	0.64	0.64	0.66	0.61	0.62
	4	0.36	0.33	0.49	0.44	0.39	0.37	0.56	0.52	0.37	0.37	0.53	0.50

Table 2.3 Column deformation (ductility) ratios for $R_d = 2.0$

	Story	Wall A1		Wall A2		Wall B1		Wall B2		Wall C1		Wall C2	
		θ/θ_y	Δ_{ax}/Δ_{yd}	θ/θ_y	Δ_{ax}/Δ_{yd}	θ/θ_y	Δ_{ax}/Δ_{yd}	θ/θ_y	Δ_{ax}/Δ_{yd}	θ/θ_y	Δ_{ax}/Δ_{yd}	θ/θ_y	Δ_{ax}/Δ_{yd}
Left Column	1	—	0.87 ^c	—	0.81 ^c	—	0.60 ^c	—	0.39 ^c	—	0.63 ^c	—	0.45 ^c
	2	—	0.09 ^b	—	0.09 ^b	—	0.03 ^b	—	0.00 ^b	—	0.04 ^b	—	0.02 ^b
	3	2.12 ^c	—	2.17 ^c	—	2.20 ^c	—	2.52 ^c	—	2.20 ^c	—	2.46 ^c	—
	4	2.00 ^c	—	1.97 ^c	—	1.49 ^c	—	1.52 ^c	—	1.54 ^c	—	1.57 ^c	—
Right Column	1	FC ^a	—	FC ^a	—	FC ^a	—	FC ^a	—	FC ^a	—	FC ^a	—
	2	FC ^a	—	FC ^a	—	FC ^a	—	FC ^a	—	FC ^a	—	FC ^a	—
	3	3.53 ^c	—	3.71 ^c	—	3.27 ^c	—	3.67 ^c	—	3.26 ^c	—	3.59 ^c	—
	4	2.32 ^c	—	2.31 ^c	—	1.70 ^c	—	1.76 ^c	—	1.74 ^c	—	1.82 ^c	—

^a FC: Force-controlled element^b Immediate occupancy performance level^c Life safety performance levelTable 2.4 Column strength ratios for $R_d = 5.0$

	Story	Wall A1		Wall A2		Wall B1		Wall B2		Wall C1		Wall C2	
		S16	AISC	S16	AISC	S16	AISC	S16	AISC	S16	AISC	S16	AISC
Left Column	1	—	—	—	—	—	—	—	—	—	—	—	—
	2	—	—	—	—	—	—	—	—	—	—	—	—
	3	0.25	0.19	0.35	0.31	0.67	0.72	0.49	0.47	0.64	0.66	0.47	0.45
	4	0.23	0.18	0.35	0.31	0.44	0.43	0.53	0.52	0.42	0.40	0.51	0.50
Right Column	1	1.34	1.37	1.21	1.24	1.21	1.24	1.05	1.04	1.22	1.25	1.08	1.11
	2	1.06	1.08	1.09	1.01	1.11	1.14	0.91	0.84	1.12	1.15	0.98	0.89
	3	0.76	0.78	0.83	0.82	1.03	1.06	0.84	0.87	1.02	1.05	0.81	0.83
	4	0.43	0.39	0.69	0.64	0.56	0.56	0.78	0.75	0.55	0.55	0.73	0.70

Table 2.5 Column deformation (ductility) ratios for $R_d = 5.0$

	Story	Wall A1		Wall A2		Wall B1		Wall B2		Wall C1		Wall C2	
		θ/θ_y	Δ_{ax}/Δ_{yd}	θ/θ_y	Δ_{ax}/Δ_{yd}	θ/θ_y	Δ_{ax}/Δ_{yd}	θ/θ_y	Δ_{ax}/Δ_{yd}	θ/θ_y	Δ_{ax}/Δ_{yd}	θ/θ_y	Δ_{ax}/Δ_{yd}
Left Column	1	—	2.71 ^c	—	1.90 ^c	—	1.68 ^c	—	1.10 ^c	—	1.84 ^c	—	1.19 ^c
	2	—	0.03 ^b	—	0.06 ^b	—	$\approx 0^b$	—	$\approx 0^b$	—	$\approx 0^b$	—	$\approx 0^b$
	3	5.57 ^c	—	5.82 ^c	—	5.73 ^c	—	7.25 ^d	—	5.46 ^c	—	6.97 ^d	—
	4	5.74 ^c	—	5.25 ^c	—	3.42 ^c	—	5.57 ^c	—	3.60 ^c	—	5.56 ^c	—
Right Column	1	FC ^a	—	FC ^a	—	FC ^a	—	FC ^a	—	FC ^a	—	FC ^a	—
	2	FC ^a	—	FC ^a	—	FC ^a	—	FC ^a	—	FC ^a	—	FC ^a	—
	3	9.96 ^d	—	10.81 ^d	—	9.59 ^d	—	10.97 ^d	—	9.29 ^d	—	10.70 ^d	—
	4	6.71 ^c	—	6.34 ^c	—	4.04 ^c	—	6.59 ^c	—	4.24 ^c	—	6.57 ^c	—

^a FC: Force-controlled element^b Immediate occupancy performance level^c Life safety performance level^d Collapse prevention performance level

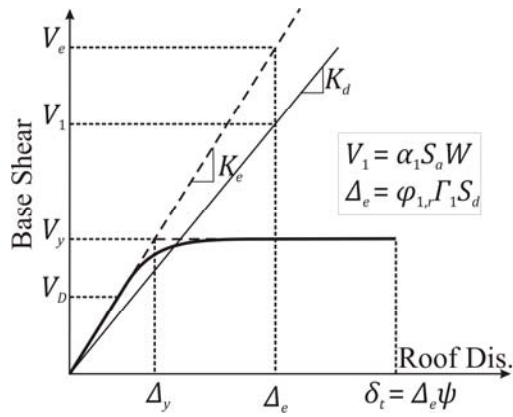


Fig. 2.1 Pushover curve of a general multi-degree of freedom system

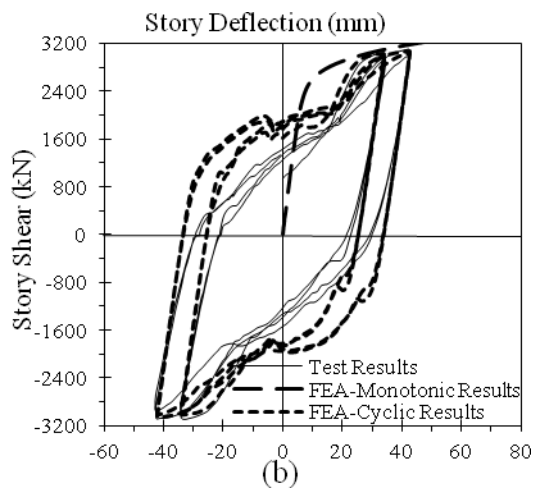
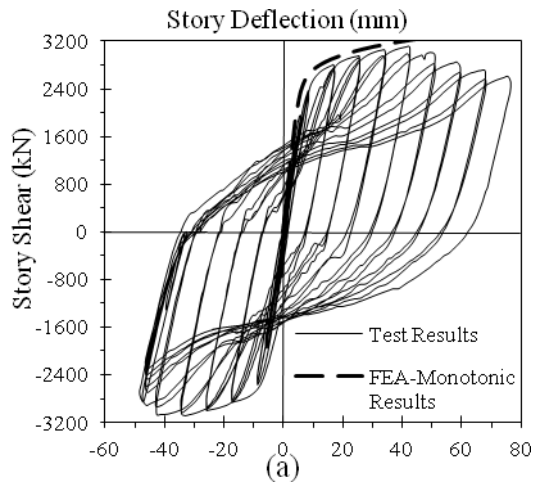


Fig. 2.2 Comparison of test results for panel 1 with FEA results:(a) Monotonic loading; (b) Cyclic loading

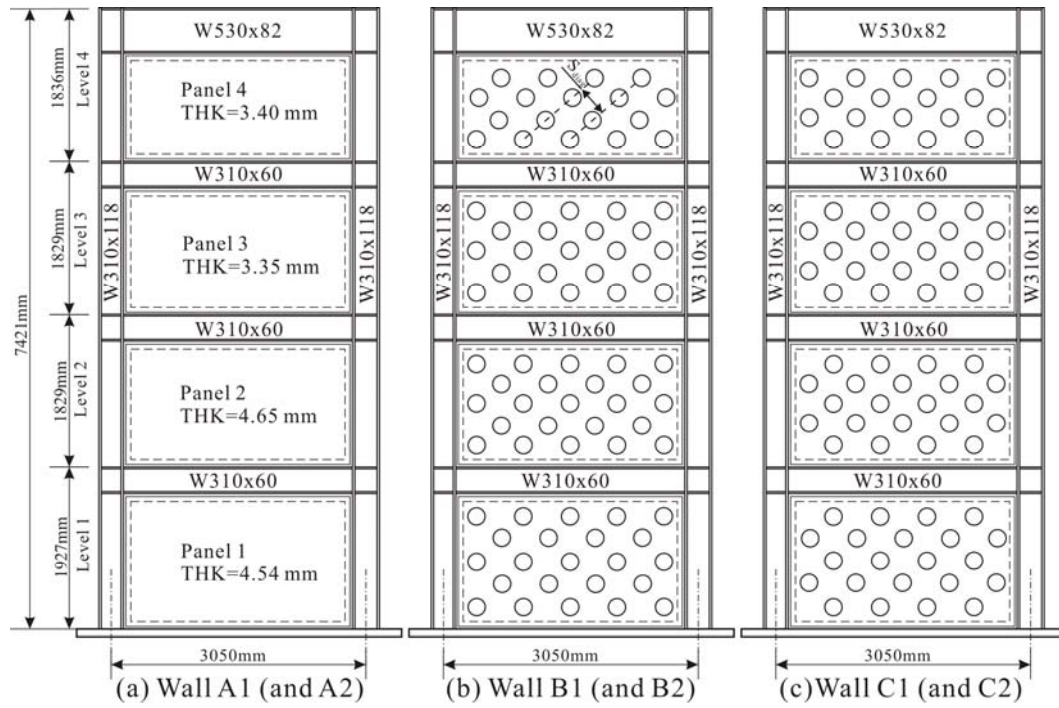


Fig. 2.3 Different SPSW systems: (a) solid panels; (b,c) perforated panels with two hole patterns

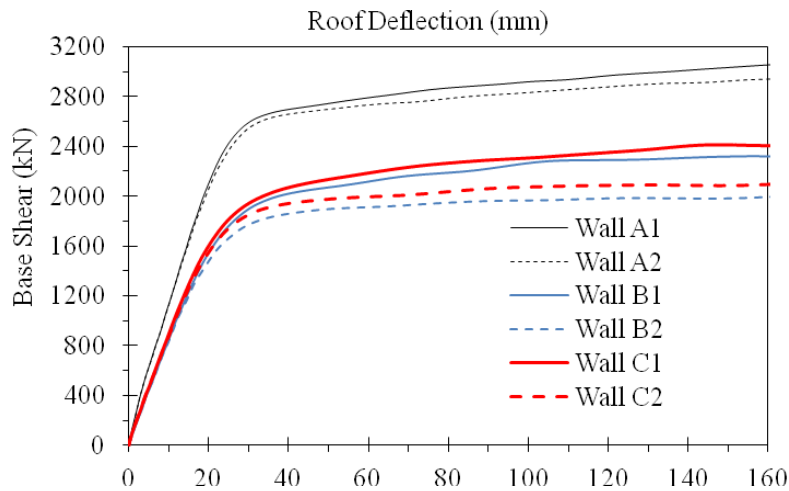


Fig. 2.4 Pushover curves of the SPSW systems

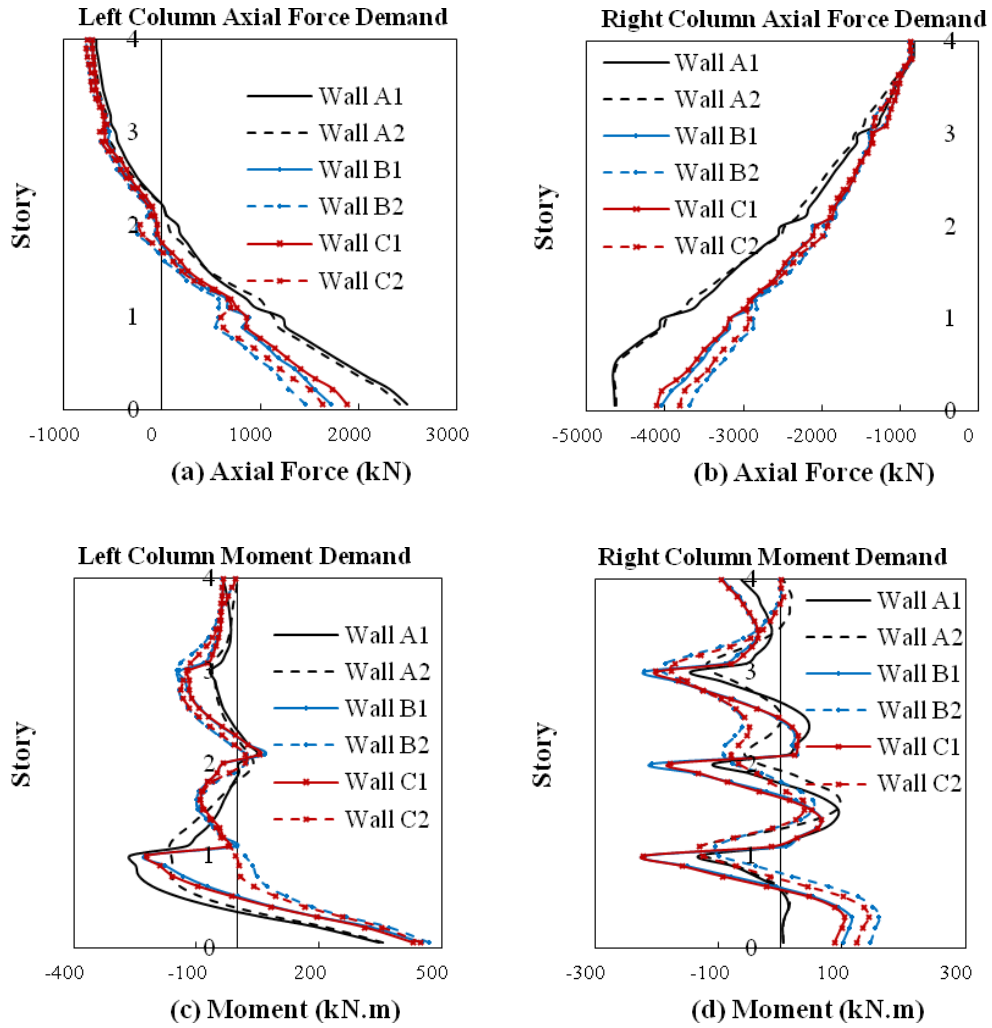


Fig. 2.5 Column internal force demands for $R_d = 2$: (a) and (c) axial force and bending moment in tension column, (b) and (d) axial force and bending moment in compression column

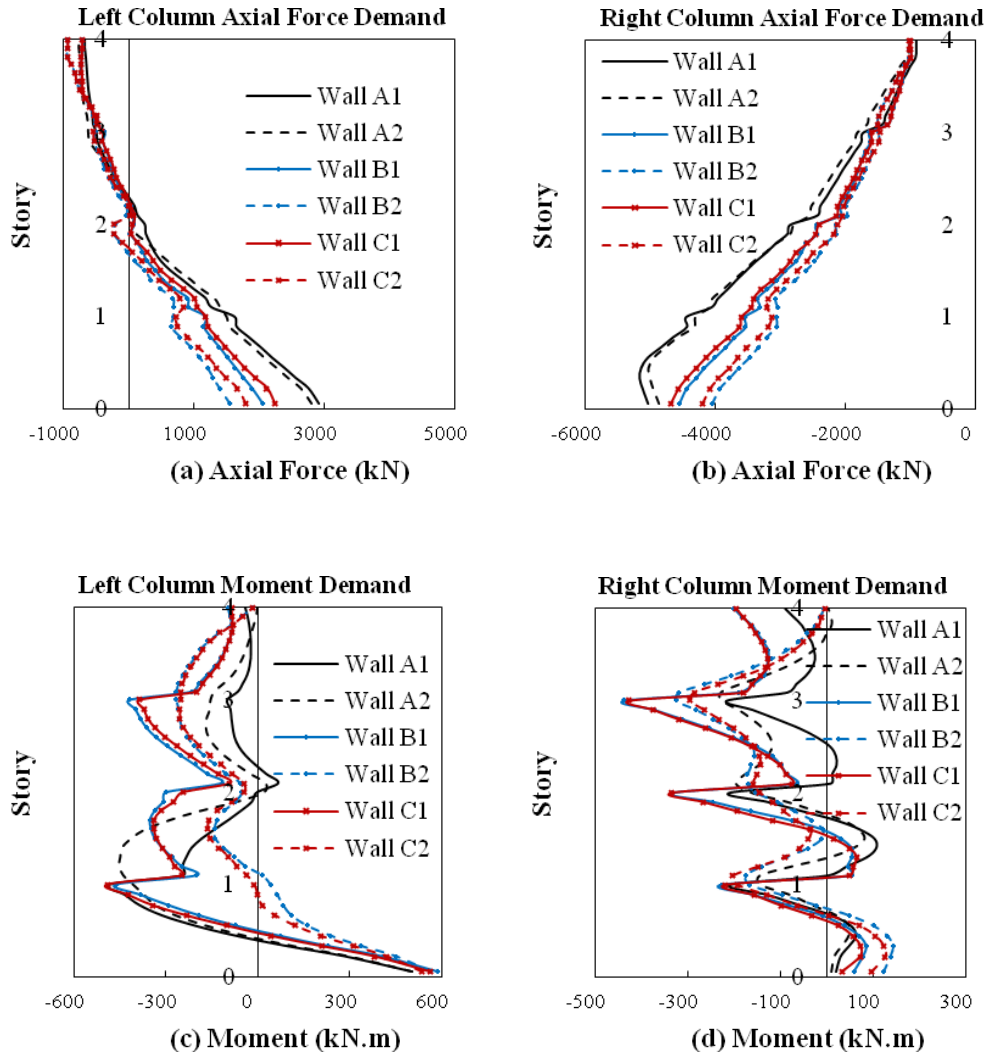


Fig. 2.6 Column internal force demands for $R_d = 5$: (a) and (c) axial force and bending moment in tension column, (b) and (d) axial force and bending moment in compression column

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3. ECONOMICAL STEEL PLATE SHEAR WALLS FOR LOW SEISMIC REGIONS²

3.1 Introduction

The steel plate shear wall (SPSW) system has become a viable lateral load resisting system for multi-story buildings, with beneficial properties for seismic applications such as high ductility, robust resistance to cyclic degradation, and resilient redundancy. These properties have been demonstrated mostly through tests on walls that contain moment-resisting beam-to-column connections and, as such, requirements for connections in ductile moment-resisting frames have significantly influenced the evolution of detailing and fabrication practice for SPSWs. The current international research thrust is aimed at improving the performance of SPSWs even further and optimizing their behavior under the overarching capacity design requirements that pervade modern seismic design provisions. As a result, the design and detailing requirements are tending to become more and more onerous, and increasingly the system is being limited economically to high seismic regions where the cost of maximizing performance can be justified. However, the lower demands on seismic force resisting systems in low and moderate seismic regions, which collectively encompass the majority of North America, may permit the use of much more economical SPSW detailing options that would make them competitive with systems that are more commonly used in these regions.

The Canadian steel design standard, CAN/CSA S16-09 (CSA 2009), hereafter referred to as S16, has adopted two SPSW performance levels: Type D (ductile) and Type LD (limited-ductility). These performance levels are associated with different force modification factors used to reduce the seismic load effects to account for both the capability of the structure to dissipate seismic energy through

² A version of this chapter has been published in the *Journal of Structural Engineering*, ASCE, 139(3), 379-388.

stable inelastic response and the dependable overstrength. As such, the factor is defined as the product of two separate coefficients, R_d (ductility-related force modification factor) and R_o (overstrength-related force modification factor). Although not used explicitly in S16, for convenience in this paper the product of these two factors is denoted simply as R , rather than $R_d R_o$. The *Seismic Provisions for Structural Steel Buildings*, ANSI/AISC 341-10 (AISC 2010a), hereafter referred to as AISC 341, adopted only one SPSW performance level: Special Plate Shear Walls. ASCE/SEI 7-10 (ASCE 2010) defines the associated response modification coefficient, R , which also accounts for both inelastic system response and overstrength. In neither standard is an intermediate-ductility option specified. Table 3.1 summarizes the R -factors specified by both S16 and ASCE 7 for the SPSW system.

Although the limited-ductility (Type LD) SPSW option exists in S16, there is a general lack of understanding of what constitutes this type of wall and these provisions are rarely applied in practice. The Type LD category was originally introduced into S16 based primarily on the experimental research of Timler and Kulak (1983) and Tromposch and Kulak (1987) and was intended to permit the use of shear connections between the beams and columns. The tests by Kulak and his co-workers predate the now well-established seismic loading test protocols and, although they confirmed that the SPSW system without moment-resisting beam-to-column connections as a feasible option, this concept is seldom tested specifically for seismic performance. The current S16 provisions (CSA 2009) for achieving $R_d = 2.0$ (Type LD) state that the requirements for $R_d = 5.0$ (Type D) must be met, with certain relaxations; i.e., these provisions were not developed independently for the $R_d = 2.0$ case.

3.2 Scope and Objectives

The major objective of this research is the rational development of SPSW concepts that would make them competitive with other systems and materials in low seismic regions, and to confirm the efficacy of such a system experimentally.

Therefore, the need for economical fabrication and erection procedures is considered of fundamental importance. At the early stages of the project, an assessment of major cost implications of the various design and detailing requirements for Type D/Special SPSWs was made with a view to maximizing the economic benefits of switching to the low-seismic concept for regions where such a system would suffice. Whereas most research on seismic force resisting systems aims to develop design and detailing requirements based on anticipated demands, or simply attempts to maximize overall seismic performance through high-performance detailing, this research has taken a fundamentally different developmental approach. In the traditional approach, the cost of the system is largely an uncontrolled outcome of the research. However, since it was anticipated that simple and relatively inexpensive detailing can be used in SPSWs and still achieve good seismic behavior due to the nature of the system itself, the SPSW test specimen was developed with the main emphasis on minimizing the in-place cost in a real structure rather than imposing detailing that is known to be highly robust under cyclic loading. In other words, the performance level of the low-seismic concept, rather than the cost of the system needed to maximize performance, was the principal outcome of the research.

A large-scale test was conducted in order to assess the performance that could be expected from the proposed low-seismic SPSW concept. The test specimen was developed for use with common and economical fabrication methods and simple erection procedures, with input from the steel industry. Selective use of the large body of knowledge available from previous research on highly-ductile SPSWs permits the design of a wall that is expected to perform well enough for low seismic regions, but be much less costly to construct.

3.3 Previous Tests on SPSWs with Simple Connections

Although there are many SPSW tests discussed in the literature, few have incorporated simple beam-to-column connections in the boundary frame. Since the use of conventional shear connections is such a key component of the low-

seismic wall being proposed, research on similar walls is reviewed briefly here. Timler and Kulak (1983) tested two single-story SPSWs simultaneously by testing a two-panel arrangement of vertically-oriented beams, horizontally-oriented columns, and 5 mm thick infill plates. Two different beam-to-column connections were implemented. At the four extreme corners, pinned joints were used, whereas at the centerline of the test specimen, rigid connections were used. Hence, the specimen is similar to two one-story SPSWs, each with pinned beam-to-column connections and rigid column bases. Axial loads were not applied to the columns. Three cycles of loading were applied to the allowable serviceability displacement limit (drift ratio of 0.25%, or 6.25 mm) and then a final monotonic loading excursion was applied until failure of the assembly occurred. The test specimen responded elastically during the first three cycles and the maximum capacity was reached in the final excursion when a weld tear occurred at the infill plate-to-fish plate connection, followed by failure at a pin connection.

Tromposch and Kulak (1987) tested a twinned single-story shear wall arrangement similar to the one tested by Timler and Kulak (1983), but with some modifications. The beam-to-column connections were double shear tabs welded to the column flange and bolted to the adjacent beam web. In order to provide better anchorage for the tension field, a thinner infill plate (3.25 mm) and stiffer beams were selected. Simulated gravity loads were applied to each column through full-length prestressing bars. The specimen was tested under fully-reversed cyclic lateral load, with amplitudes that were gradually increased up to a drift ratio of 0.78% (17 mm, which corresponded to 67% of the ultimate load). This sequence consisted of 28 cycles. Beyond this point, the test setup was able to apply the load in one direction only, and without the column prestressing rods due to the curvature of the columns. The final phase was completed as a monotonic loading sequence to the ultimate capacity of the specimen, which corresponded to a drift ratio of 3.23% (71 mm). The test specimen showed ductile behavior with severely pinched hysteresis curves due to the thin infill plate and flexible boundary frame.

Caccese et al. (1993) conducted a series of tests on one-quarter-scale SPSWs subjected to cyclic loading to study the effect of two main parameters: the beam-to-column connection type and the infill plate thickness. The test specimens were three stories high and one bay wide and had infill plate thicknesses that ranged from 0.76 mm to 2.66 mm. They reported test results from six specimens, including one moment-resisting frame, three SPSWs with moment-resisting beam-to-column connections and varying infill plate thicknesses, and two SPSWs with shear beam-to-column connections (beam web fillet-welded directly to the column flange) and varying plate thicknesses. Each test specimen underwent 24 cycles of a single in-plane lateral load applied at the roof level with gradually increasing roof displacements up to a drift ratio of between 1.8% and 2.0%. Subsequently, the specimen was loaded monotonically to the displacement limit of the actuator or failure. The effect of gravity loads was not included in the test. The authors suggest that the beam-to-column connection type has only a minor effect on the overall performance of the SPSW system.

Berman and Bruneau (2005) tested three single-story SPSW specimens using light-gauge cold-formed steel for the infill plates, with thicknesses ranging from 0.75 mm to 1.0 mm. The research aimed to study the lateral performance of a prototype designed as a seismic retrofit for a hospital in a zone of high seismicity. Two specimens had a flat infill plate (each with a different method of fastening to the boundary frame) and the third had a corrugated infill plate. The flat infill plates were lap-connected to the stem of a WT section by welds or epoxy and the WT flange was bolted to the boundary frame. The corrugated infill plate was connected to an angle on each side by epoxy, and the angles were bolted to the boundary frame. Double-angle beam-to-column connections were used in the boundary frame, which was designed to remain elastic with a safety factor of 2.5. Each test was conducted under cyclic lateral loading and the effect of gravity loads was excluded. Only the specimen with a flat (and 1.0 mm thick) infill plate that was fastened by welding showed stable and highly ductile performance. The yield displacement of the wall was assumed to be 5.3 mm, at a drift ratio 0.29%,

and the wall exhibited its maximum resistance at a ductility ratio of about 10, or a drift ratio of 3.07%. It reached the maximum ductility ratio of about 12 at a drift ratio 3.65%, at which time fractures propagated in all four corners of the infill plate from the endpoint of the welds connecting it to the WT sections.

Dastfan (2011) tested a two-story modular SPSW with partially encased composite columns. In this specimen, the 3 mm thick infill plate modules were connected to the columns and beams through bolts to the fish plates that were welded to the surrounding frames in the shop in advance. A double lap splice at the mid-height of each panel was used to connect the infill plate modules together. The beam-to-column connections were a customized double shear tab connection (on one side of the beam web a conventional shear tab was used, and on the other side was a continuous fish plate, augmented locally to accommodate more bolts). The gravity loads were applied as a constant axial compression to each column and the specimen was tested under cyclic lateral loading. A total of 27 load cycles were applied. Very good local and global performance was reported. The first story lateral deformation was chosen as the controlling parameter, and the yield displacement of the specimen was assumed to be 8.5 mm, at a drift ratio of 0.48%. The specimen reached its maximum strength at a ductility ratio of 5, or a drift ratio of 2.40%, and achieved a maximum ductility ratio of 8, corresponding to a drift ratio of 3.83%. The test was terminated at this point since the columns of the first story were damaged at mid-height and at the base, and the tears in the infill plate started to grow rapidly. The beam-to-column connections showed no significant damage. The authors observed that the simple connection rotation appeared to improve the distribution of yielding in the infill plates over the height of the wall, thereby increasing the total amount of energy dissipation in the system as compared to a similar wall they tested with rigid beam-to-column connections.

Chen and Jhang (2011) tested two one-quarter-scale SPSWs with stiffened infill plates, each representing two intermediate stories of a multi-story wall, under

cyclic loading. One had simple (shear tab) beam-to-column frame connections and the other used rigid connections. A low-yield-point steel (with yield and ultimate stresses of 95 MPa and 279 MPa, respectively) with a 3.5 mm thickness was used for the infill plates, while ASTM A572 Gr. 50 steel was used for the boundary frames. The infill plate stiffener arrangement was selected such that the width-to-thickness ratio of each sub-panel was 71. A total of 24 cycles of lateral load were applied to the top of each specimen. The yield displacement was assumed to be 0.50% and both walls showed good performance with a maximum story drift ratio of 6.0%, corresponding to a ductility ratio of 12. The specimen with rigid beam-to-column connections showed a slightly higher energy dissipation capacity.

3.4 Modular Construction of SPSWs

Modular construction concepts, with emphasis on repetitiveness of fabrication, ease and speed of erection, and elimination of field welding, can improve the competitiveness of SPSWs in low seismic regions. As buildings normally have a constant story height and bay width over their height, implementing simple beam-to-column connections in the SPSW makes them a perfect candidate for the use of modular construction. In this method, modular components with repetitive fabrication processes are produced in the shop and assembled completely by bolting in the field, eliminating the need for field welds. This not only tends to decrease costs, but can also enhance the quality of the finished structure due to the improved control that can be maintained in shop conditions. Although the modular construction method can be cost-effective (by reducing the costs of both fabrication and erection) with very rapid assembly time, the resulting shear wall has several distinct differences from most of the walls that have been tested in the laboratory, and the performance of these walls requires verification.

Driver and Moghimi (2011) describe several potential modular options and discuss their advantages and disadvantages. In particular, three concepts for SPSWs are considered that are designed specifically to reduce in-place cost.

Several main characteristics distinguish these concepts from conventional SPSWs. Most notably, the beam-to-column connections are simple shear connections and all concepts constitute modular systems that require no field welding. Also, the infill plates are spliced with single-sided lap plates, and all bolts are intended to be pre-tensioned to resist slip at design loads as per conventional practice in bolted seismic force resisting systems.

The first concept (Fig. 3.1(a)) is intended to maximize the number of similar pieces and minimize the piece sizes that need to be handled on site. The angles for the simple beam-to-column connections are connected to the columns in the shop (by bolting or welding), and bolted to the beams in the field. The fish plates are welded to the column and beam centerlines in the shop, and then after assembling the frame on site, the infill plates are bolted to the fish plates. Vertical and horizontal lap splices are used to connect the infill plate modules to each other. The field assembly and erection of numerous infill plate pieces could potentially increase construction time in comparison with other concepts, but may have applications in small projects or rehabilitation work.

The second concept (Figs. 3.1(b) and 3.1(c)) is intended to minimize the number of pieces to be handled during erection by maximizing shop assembly. In this concept, the bay of the wall is divided vertically into two parts, permitting installation in relatively wide bays. Therefore, each module could be up to about three stories in height (depending on the maximum practical size for shipping and lifting) and one-half bay wide. The connections (simple or rigid) of the beams to the columns are fabricated in the shop. The fish plates are omitted and the infill plates are connected directly to the surrounding frames in the shop. Vertical lap splices and beam splices are used to connect the modules together. Although the vertical splice may create erection and plumbing challenges, this method has some potential advantages, including speed of construction and the high out-of-plane stiffness of each module during handling, as compared to individual infill plates.

The objective of the third concept (Fig. 3.1(d)) is to provide internal modules of a single story in height (mid-story to mid-story) with effectively no limit to the bay width imposed by shipping concerns, while not limiting the height of the adjacent column tiers. Therefore, this concept consists of three module types: base, top, and intermediate story modules. As long as the heights of the intermediate stories and the associated beam and column sizes are consistent, all intermediate modules will be identical. In the base and top modules, the infill plates are welded to the base plate/beam and the top beam, respectively, in the shop. In intermediate modules, the infill plates above and below the beam are welded to the beam's flange in the shop, which eliminates the need for horizontal fish plates. Fish plates are welded only to the column flanges in the shop. The fish plates could be continuous or interrupted at the connections, and in either case the simple connection can be accommodated accordingly, as shown in Fig. 3.1(d). Shear wall modules (including the beams) are connected to the column fish plates on site by bolts, and then horizontal lap splices are used to connect the modules to each other. A disadvantage of this system is the lack of out-of-plane stiffness of the modules during handling. Therefore, some consideration is needed regarding erection of the system, such as providing temporary perimeter stiffening to the infill plates until installation of the panel is completed. Further details on these modular concepts and their performance under lateral monotonic forces in comparison with other construction systems are provided by Driver and Moghimi (2011).

3.5 Current SPSW Design Methods

Standard S16 (CSA 2009) requires using capacity design principles for any structure designed for seismic loads using $R > 1.3$ ($R \leq 1.3$ can be interpreted as essentially elastic design). For Type D SPSWs, it is assumed that significant inelastic deformation can be developed in the system by yielding of the infill plates and the formation of plastic hinges at the ends of the beams, at a short distance from the faces of the columns, and at the bases of the columns. The moment-resisting boundary frame alone must have sufficient capacity to resist at

least 25% of the factored story shear at each level, and minimum stiffness requirements are provided for the columns and the top and base beams to ensure the development of reasonably uniform tension fields in the infill plates. In general, the beam-to-column connections must comply with the requirements for Type LD (limited-ductility) moment-resisting frames and the column joint panel zones with those for Type D (ductile) frames. All areas that may develop significant yielding—ends of beams, column bases, and infill plates—are designated as protected zones, which prohibits most attachments or discontinuities that may cause stress concentrations.

The AISC 341 (AISC 2010a) provisions stipulate requirements for special plate shear walls that are similar to those for Type D walls in S16 (CSA 2009). They also use the capacity design philosophy and specify that, in general, the beam-to-column connections must comply with the requirements for ordinary moment frames and the panel zones next to the top and base beams and the boundary member cross-sectional compactness must satisfy the special moment frame requirements. Boundary member minimum stiffness requirements are also specified. Protected zones include the infill plates, boundary frame connections, and potential hinging regions in the beams.

For SPSW column design, S16 (Type D) and AISC 341 both require that the internal forces from frame action (beam hinging at both ends), including the effects of material overstrength and strain hardening, be added to the gravity forces and the distributed forces from the yielded infill plate (including material overstrength). Whereas S16 explicitly specifies the beam end moments for which the columns must be designed, AISC 341 instead stipulates that the column-beam moment ratio must comply with the associated requirement for special moment frames. Both requirements serve to ensure strong column-weak beam behavior. Berman and Bruneau (2008) have presented a detailed procedure for capacity design of columns in Type D/Special SPSWs.

Design requirements for SPSWs with a lower seismic force modification factor are included in S16 (CSA 2009), but not in AISC 341 (AISC 2010a). Capacity design requirements in S16 for Type LD SPSWs use Type D wall provisions as a starting point, with a few relaxations of the rules as deemed appropriate by the committee. These relaxations include reduced beam compactness requirements and beam-to-column connections other than rigid being permitted. As such, the requirement that the boundary frame be capable of resisting 25% of the factored story shear at each level does not apply. It is also recognized that the column panel zone and connection requirements specified for Type D walls need not be applied if shear connections are used. However, notably, there is no reduction of the column design moment arising from plastic hinging in the beams. That is, even for the Type LD case the beam end moment applied to the adjacent column is based on the beam's plastic moment increased to account for material overstrength and strain hardening. The presence of this requirement leaves the case of walls with shear connections somewhat ambiguous with respect to column design moments, although it is stated explicitly that shear forces that develop from these plastic moments need not be considered in this case. In the interest of clarity of intent and technical rigor it is imperative that Type LD wall provisions be developed within their own context, rather than simply being a modified version of those used to obtain Type D performance, and be based on observations from research specifically attuned to Type LD objectives.

3.6 Developmental Philosophy for Low-Seismic Wall

As the central goal of this research is to develop a SPSW concept suitable for use in low seismic regions, it is necessary to capitalize on the inherent ductility of these walls so that emphasis can instead be on fabrication economics.

Specifically, SPSWs should not require the costly connection detailing and stringent column design requirements that are specified in design provisions for highly ductile walls in order to perform well in low seismic regions.

The SPSW system proposed for low seismic regions has two main differences

from the most common SPSWs. First, it uses simple (shear) beam-to-column connections, giving rise to several advantages, both economic and technical. Clearly, it reduces the cost of the system, since simple beam-to-column connections are considerably less costly to fabricate than connections that comply with Type D/Special or even Type LD/Ordinary moment frame requirements. Also, the moment and shear forces that develop at the beam ends are greatly reduced in this system due to the flexibility at the frame joint, which in turn reduces the moment and axial force demands on the columns significantly.

A drawback of the shear connection application is that it reduces the redundancy of the system as a whole, although the redundancy of the infill plates as a distributed bracing system remains. Using shear connections instead of moment connections also tends to cause more pinching in the hysteresis curves, decreasing the total energy dissipated. However, since the shear connection allows rotation at the beam-to-column interface, the deformed shapes of the beams and columns are less affected by frame action. As a result, a more uniform yielding distribution develops in the infill plates over the height of the wall, potentially increasing the total energy dissipated by the system. This phenomenon has been demonstrated in both experimental and numerical investigations. For example, Dastfan (2011) compared the energy dissipation capacity of two two-story SPSWs with partially encased composite columns, one with simple and the other with rigid (and reduced beam section) beam-to-column connections that were identical in all other respects. They found that the total energy dissipated during the test of the system with simple connections was higher. To support these experimental observations, finite element pushover analyses of multi-story SPSWs conducted as part of the current research project have also shown that changing the beam-to-column connection type from rigid to simple makes the distribution of yielding in the infill plates over the height of the wall more uniform and increases the energy dissipation capacity of the system. While there are some advantages of using simple connections in SPSWs, any anticipated improved performance is predicated on the connection behaving in a robust manner under cyclic loading,

and this must be demonstrated through physical testing of such a SPSW.

The second main difference from conventional high-ductility SPSWs is the idea that some yielding in the columns can be tolerated in low seismic regions, as long as it does not cause the formation of a yield mechanism in the system. In the capacity design of Type D/Special SPSWs, columns are designed to remain elastic (except that S16 recognizes the formation of plastic hinges at the column bases), while the infill plates develop the yielded tension field in each panel and the beams develop a plastic hinge at each end. In addition, allowances are included for potential overstrength of these yielded regions. As a result, and especially in cases where beams or infill plates are oversized, large internal force demands are imposed on the columns of the system. This is particularly severe for the column under the maximum compression, and frequently extremely heavy column sections are needed to satisfy the design criteria.

Allowing partial yielding in the columns in low seismic regions, where lower ductility systems are typically used, is not without precedent, and this philosophy has been adopted in design provisions (S16 and AISC 341) for other lateral force resisting systems. For instance, columns in Type D/Special moment frames are designed for the plastic moments at the ends of each beam, amplified by both material overstrength and strain hardening factors; conversely, in the design of columns in Type LD/Ordinary moment frames, the plastic end moments of the beams are not amplified, implicitly permitting some column yielding to occur. This yielding is permitted even though the columns do not rely on an integrated direct bracing system for stability, as is the case in SPSWs or braced frames.

3.7 Test Specimen

A laboratory test of a two-story modular SPSW was conducted to assess the performance of the low-seismic SPSW concept discussed in the preceding sections. Instead of attempting to adapt the current provisions in S16 (CSA 2009) for Type LD walls, which are themselves under scrutiny, the wall was designed

using the performance-based methodology described by Moghimi and Driver (2011) and performance criteria specified in ASCE 41 (ASCE 2007). Frame components subjected to deformation-controlled actions use moment-frame acceptance criteria, in the absence of suitable SPSW criteria, and meet the life-safety performance level. Components subjected to force-controlled actions meet the strength design provisions of both S16 (CSA 2009) and AISC 360 (AISC 2010b). Of particular note, the goal was to select a configuration that, according to these performance-based design criteria, barely achieves the ductility level consistent with Type LD walls, i.e., $R_d = 2.0$. While this method does not result in a system that complies in all respects with the Type LD SPSW provisions of S16, it was believed that good performance would still be achieved at lower cost. The test results provide evidence of the performance that can be expected from such a system.

Discussions with steel industry personnel have led to the conclusion that modular Concept 3 (discussed previously) is the most promising in terms of practicality and economics, and it forms the basis of the specimen tested. Double web angle beam-to-column connections were used, which are common in practice and at the same time provide rotational freedom at the connection. The short legs of the angles were welded with 8 mm fillet welds to the beam web in the shop and the long legs were bolted to the column flanges during module assembly. Connecting the long legs to the columns increases the rotational capacity of the joint and consequently reduces the demand on the columns. The test specimen was constructed using normal industry procedures.

Fig. 3.2 shows the elevation of the specimen tested. The story height was 1900 mm and the center-to-center dimension between columns was 2440 mm, approximately representative of a half-scale wall for an office building. The story aspect ratio (story height/center-to-center distance between columns) was 0.78 for both stories. The columns were continuous W250×101 sections (W10×68), the intermediate beam was a W250×58 section (W10×39), the top beam was a

W460×67 section (W18×45), and the double-angle connections were L102×76×11 sections (L4×3×7/16) with 170 mm and 360 mm lengths for the first and second stories, respectively. (It should be noted that the current design procedures in S16 result in a W310x202 column section—two times the cross-sectional area and more than three times the moment of inertia of the one selected.) The infill plates were 4.8 mm thick, and the fish plates were 6.35 mm thick and 115 mm wide. The infill plates in both stories were spliced horizontally with a single lap plate of the same thickness as the infill plates. Since plastic hinges were expected to develop at the column bases, stiffener plates at both sides of the columns were provided to increase the ductility in this region.

A 4.8 mm (3/16 in.) thick infill plate was selected as a readily available thickness in the market and representing a plate that would be easy to work with at full scale. The infill plate connections to the beams and fish plates, as well as the fish plate connections to the columns, were designed to resist the expected tension yield force (including material overstrength) in the infill plates. For the design of the infill plate splices, the same expected yield force of the infill plate was considered; however, since the splice plates were cut from the same plate as the infill plates themselves, for consistency the expected yield stress was also used in the splice plate resistance equations. When the cutting of splice and infill plates from the same source plate cannot be assured, a thicker splice plate may be needed.

The beams were designed based on the expected tension yield force in the infill plate. Since the infill plates in the two stories were of the same thickness and the tension field orientations are similar, the intermediate beam was designed mainly for the compressive force due to the inward pull from the infill plates on the columns. In addition to this compressive force, the top beam was subjected to significant shear, flexural, and axial loads directly from the tension field in the infill plate below. Therefore, a relatively deep and stiff beam was needed. Both beams were checked against lateral torsional buckling in order to eliminate the

need for intermediate lateral bracing. The double-angle connections were designed for the axial force and shear present at the beam ends.

The measured material properties of the infill plates, angles, and columns are shown in Table A.1 in Section 1 of the Appendix. Since yielding in the beams was limited to a small area in the web around the double angle connections, this nonlinear behavior was not critical to the lateral performance of the system so the beam material properties are not reported in Table A.1. All frame members were fabricated from Grade 350W steel, and the angles and infill plates were from Grade 300W steel (CSA 2004). All frame members met Class 1 (S16) and Highly Ductile Member (AISC 341) compactness requirements. ASTM A325 bolts were used and all bolted connections in shear were designed to be of bearing-type. However, the bolts were pretensioned to meet the requirement (S16 and AISC 341) that all bolts resisting cyclic loading be pretensioned high-strength bolts to avoid slip at design loads. The infill plates were connected to the fish plates and splice plates by A325 19.1 mm (3/4 in.) diameter bolts with a 60 mm spacing between the centerlines of the fasteners. The same bolt size and spacing were used to fasten the connection angles to the column flanges. The infill plates (adjacent to the beams only) and fish plates were connected to the surrounding frame by 5 mm (infill plates) or 6 mm (fish plates) fillet welds on both sides. The electrode classification was E70XX. The structural drawings of the modular test specimen are shown in Section 2 of the Appendix, in Figs. A.1 through A.8.

3.8 Loading Scheme

The distribution of inertial loads on a seismic force resisting system depends on the earthquake ground motion characteristics and severity and the properties of the system itself, including geometry, distribution of mass, stiffness, strength, and damping. These properties influence the relative magnitudes of the deformations and internal forces within the structure, which can vary significantly during an earthquake as the stiffness distribution changes due to progressive yielding. Moreover, changes in the seismic acceleration history and frequency content

excite different mode shapes of the system, causing changes in force distribution. Hence, the distribution of inertial forces has been an issue of debate in recent decades and the use of more than one lateral load pattern has been recommended for nonlinear static design to bound the range of design actions that may occur during a seismic event (FEMA 1997). However, ASCE 41 (ASCE 2007) suggests the use of a single pattern based on the first mode shape, since recent research has shown that using multiple lateral load patterns is not particularly effective in improving the accuracy of a nonlinear static analysis. The first mode pattern is most appropriate for taller structures and it emphasizes a gradual increase in inertial force from the lower to the upper stories, which underscores the influence of story overturning moment over shear force in comparison with a uniform load pattern.

Based on the discussion in the preceding paragraph, the first mode load distribution was selected for the test. From eigenvalue analyses of the wall using several assumptions of story masses, the first mode shape has normalized lateral deformations of 1 and about 0.55 at the roof and top of the first story, respectively. However, the significant difference between these deformations comes from the fact that the specimen is two stories tall; for a taller structure, the difference between two adjacent floors would be smaller. As a result, an imaginary first mode shape with normalized deformations of 1 and 2/3 for the roof and first story, respectively, was selected for use throughout the test and it is believed to adequately represent a range of intermediate-height structures. To simulate the location of inertial forces induced by floor masses, the lateral loads were applied through two sets of twin-actuators (supported by a reaction wall) positioned in line with the top flanges of the intermediate and top beams. The SPSW was loaded through each beam top flange to simulate the delivery of load to the wall through a horizontal diaphragm.

In order to study the P- Δ effect on the overall behavior of the SPSW system subjected to cyclic lateral loading, reasonable unfactored gravity loads must be

applied. As such, a constant gravity load of 600 kN was applied to the top of each column by two sets of independent hydraulic jacks connected to a cross-shaped distributing beam supported at the top of the specimen by the columns. Four gravity load simulators, designed so that the gravity loads remain in a vertical orientation throughout the cyclic lateral deformation, were employed in conjunction with these jacks. An articulated bracing system that prevents out-of-plane deformations but provides no restraint to lateral and vertical deformations was affixed to each column at each floor level. Fig. A.9, in Section 3 of the Appendix, shows the test setup scheme.

The loading history for the test specimen was selected based on the methodology outlined by the Applied Technology Council (ATC 1992). The two stories of the test specimen had the same infill plate and column, but the lateral shear force and overturning moment resisted by the first story were 67% and 167%, respectively, larger than the corresponding values in the second story. Therefore, the majority of deformation, yielding, and energy dissipation was expected to take place in the first story. As such, the lateral deformation of the first story was selected as the “deformation control parameter” (δ) and the base shear was selected as the “force quantity” (Q)—or the force corresponding to this deformation—and these two parameters constitute the test control parameters. The point of significant yield (δ_y , Q_y), which is essential information needed for controlling the test, was first estimated by finite element analysis and then adjusted during the early stage of the test based on the observed behavior. It was found that the yield displacements of the first story in the push (north) and pull (south) directions were equal to 11 mm and 13 mm, respectively. Hence, the first story lateral yield displacement was selected as $\delta_y = 12$ mm, the average value of the two directions. The roof yield displacement was found to be equal to $\delta_{r,y} = 22$ mm for both the push and pull directions.

A large number of data collection devices, including load cells, LVDTs, cable transducers, clinometers, strain gauges, and rosettes, were used to control the test

and monitor and record the important structural responses. The first story and roof lateral displacements were measured at heights of 1845 mm and 3755 mm, respectively, from the top surface of the base plate. Fig. A.10, in Section 3 of the Appendix, shows the test specimen instrumentation scheme.

3.9 Test Results

Table 3.2 shows the loading/displacement regime throughout the test in the push direction. (The corresponding values for the pull direction differ only slightly from those in the table.) The hysteresis curves based on the test control parameters (first story) are shown in Fig. 3.3 (and repeated in Fig. A.11 in Section 3 of the Appendix without the hysteresis curves of the comparison test). From cycles 1 to 7, the test was conducted in force control to measure the elastic and initial inelastic behavior of the wall. From cycle 8 forward, the test was carried out in displacement control. Cycles 8 to 10 were completed with the yield displacement of $\delta_y = 12$ mm. For reference, the nominal shear capacity of the specimen of approximately 1730 kN (according to S16 and AISC 341, with 0.5 used as the coefficient to represent the fully yielded strength of the infill plate) is also shown in Fig. 3.3. This value is appreciably less than the base shear of 1920 kN resisted during cycles 8 to 10. The hysteresis curves show that the first story absorbed significant energy during and after cycle 8.

During the first half of cycle 11 (push direction) with a target lateral displacement of $2\delta_y$ (24 mm), a loud sound due to buckling of the infill plates was heard at a displacement of about 22 mm ($1.8\delta_y$) and the first story load cell began giving erroneous readings due to a break in the wiring. For safety reasons, the test specimen was unloaded without reaching the full target displacement and the load cell wiring was repaired. For the part of cycle 11 where load data are unavailable, the estimated curve is shown as a dashed line in Fig. 3.3. The second half of cycle 11 (pull direction) was done with the same lateral deformation level as the push half-cycle (22 mm) for symmetry. Cycle 12 was then carried out with a lateral deformation of 26 mm ($2.2\delta_y$) in order to compensate for the smaller lateral

deformation in the previous cycle. The last cycle of the $2\delta_y$ lateral deformation level (cycle 13) was done with the targeted displacement of 24 mm. The hysteresis curves for the second story (shown in Figs. A.12 and A.13 in Section 3 of the Appendix) indicates that this story started to absorb a considerable amount of energy from cycle 11 forward. Cycles 17 and 18 consisted of a lateral deformation of $4\delta_y$ (48 mm), and in the first half of cycle 18 (push direction) the first tear in the system was observed at the top-north corner of the lower infill plate (Tear 1 in Fig. 3.4, with a length of 50 mm).

Cycle 19 was completed with a lateral deformation of $5\delta_y$ (60 mm) and the peak base shear in the push direction of 2625 kN occurred in this cycle. In this cycle, the first tear had grown slightly under push loading and a new tear at the center of the infill plate above the splice plate occurred in the pull-loading condition (Tear 2 in Fig. 3.4, with a length of 30 mm and not intersecting the splice region). Toward the end of the push loading in cycle 20, at a lateral deformation of 55 mm, the welds connecting the column base stiffeners to the south flange of the south column ruptured, followed by the initiation of fracture at the adjacent flange tips. This caused a reduction (about 11%) in the load-carrying capacity of the system. Also, another tear appeared at the middle of first infill plate, below the splice plate (Tear 3 in Fig. 3.4). However, no such fracture occurred while loading in the pull direction. Cycles 21 and 22 were completed with a lateral deformation of $6\delta_y$ (72 mm). In the push direction of cycle 21, although the fracture in the south column propagated across the whole south flange and almost through the web, the wall system maintained good shear capacity and ductility. The north flange of the south column remained intact and acted similar to a pin connection (up to the end of the test) and the column tensile load transferred to the foundation through the infill plate and the north flange. In the pull direction, the fracture in the south column closed and the wall reached its maximum shear capacity of 2660 kN. Also, another tear appeared near the bottom-North corner of the first infill plate (Tear 4 in Fig. 3.4). In this cycle, the double-angle connections of the intermediate beam showed some minor, but visible, permanent deformation.

Local buckling occurred during cycle 22 (push direction) in the north-east flange of the north column, right above the column base stiffeners. In the pull direction, at a lateral deformation of about 40 mm ($3.3\delta_y$), the north flange of the north column and half the web fractured, causing a decrease in the shear capacity of the system of about 20%, after which the capacity of the system increased considerably and the half-cycle was completed to the target displacement. From this point forward, the columns both behaved as though they were pinned at the base when in tension. In cycles 23 and 24, a lateral displacement of $7\delta_y$ (84 mm) was applied. In cycle 23, the wall response produced a stable and relatively wide hysteresis curve with good lateral strength (in excess of 2000 kN in both directions), while in cycle 24, the wall showed a similar response, but with about 10% less shear capacity. Cycle 25 was done with a lateral deformation of $8\delta_y$ (96 mm) and again the specimen showed a stable and relatively wide hysteresis curve with an average base shear for the two directions of 1900 kN, still greater than the nominal shear capacity indicated in Fig. 3.3. However, in the push direction, Tear 1 grew considerably to 600 mm and a new tear was detected near the bottom-South corner of the infill plate (Tear 5 in Fig. 3.4). In the pull direction, Tears 2 and 4 grew significantly and a new tear appeared at the South side of the infill plate above the splice plate (Tear 6 in Fig. 3.4).

Despite the fact that the wall could still resist considerable shear force (more than 70% of the maximum base shear achieved), the test was terminated after 25 cycles of load, since the tears in the first story infill plate had begun growing more rapidly. Fig. 3.4 shows all the tear locations and orientations in the first story infill plates and their lengths at the end of the test. Although the second story was subjected to substantial nonlinear behavior, no tears occurred in the second story infill plate. Comparing the drift ratios in Table 3.2 shows that the contribution of the second story to the lateral deformation ductility of the system decreased in the last cycle because of the growth of the tears in the first story infill plate. However, hysteresis curves for the second story (Fig. A.13 in Section 3 of the Appendix) show that it absorbed a considerable amount of energy, even in the last cycle. At

the end of the test, the second story had dissipated 21% of the total energy dissipated by the first story.

The behavior of the test specimen was studied by comparison to a comprehensive finite element simulation subjected to monotonically increasing lateral displacement. The finite element simulation details are similar to those described in Section 2.5. The simulation contains all the details of the physical test in terms of loading scheme, material properties, geometry of the system, etc. All the components of the system, including the splice plates for the infill plates and the double-angle connections, were modeled using shell elements. The results of the numerical model are shown in Section 4 of the Appendix. Figs. A.11 and A.12 show the pushover curves from the numerical results for the first and second stories, respectively. The numerical model predicts a larger initial stiffness, which demonstrates the difference between real cyclic loading and monotonic lateral load in the numerical simulation.

For a qualitative comparison with the response of the modular wall tested, the result of another SPSW test by Driver et al. (1997), commonly referenced by researchers as evidence of the excellent cyclic behavior that can be achieved using the traditional SPSW configuration with moment-resisting connections, is also shown in Fig. 3.3. It was a four-story SPSW with a total height of 7420 mm and a distance between column centerlines of 3050 mm. The columns were W310×118 sections and the infill plates in the first two stories were 4.8 mm thick and in the top two stories they were 3.4 mm thick, with no splices. While the modular test specimen has a smaller elastic stiffness, smaller yield strength, and larger yield displacement because of the effects of the simple beam-to-column connections, bolted infill plates, and geometrical differences, in terms of overall ductility and robustness the walls demonstrated remarkably similar behavior.

The performance of the test specimen in general is demonstrated pictorially at different stages of the test in Section 5 of the Appendix. Fig. A.14 shows the

elevation of the test specimen before, during, and after the test. Fig. A.15 shows all the beam-to-column connections before the test. Fig. A.16 shows the beam-to-column connections during the test at different cycles. Finally, Fig. A.17 shows the beam-to-column connections after the test.

3.10 Discussion

One interesting result of this test is the shear connection performance and its influence on the overall system behavior. As Table 3.2 shows, the first story has the maximum relative drift ratio. Therefore, connections in the first story were under maximum rotational demand. Fig. 3.5(a) shows the rotation of the column and beam at the North-side connection of the first story, and Fig. 3.5(b) demonstrates the same rotations for the roof level. The power supply for the clinometers was off during cycle 19, so the figures present no result for this cycle. The column rotations in Figs. 3.5(a) and 3.5(b) are in close agreements with the drift ratios in Table 3.2, for both stories. Fig. 3.6 shows the relative (column-minus-beam) rotations between the beam ends and the adjacent column at the north-side connections. A positive rotation is clockwise when looking from east towards west, so a positive relative rotation represents the closing of the joint in the story below the connection. The figure indicates that the double-angle connections provided very good rotational freedom at the beam-to-column connections during the inelastic cycles, especially at the intermediate beam where the beam depth and angle length were smaller. The connection angles in the first story showed the first sign of slight yielding (via the whitewashed surface) at the bolt line connecting the long legs to the column flanges during cycle 17 ($4\delta_y$), and towards the end of the test the yielding had spread to the short legs connected to the beam web. At the end of the test, limited plastic deformation existed in the connections in the first story, while the connections in the second story exhibited signs of only minor yielding. Fig. 3.7(a) shows the first-story connection as fabricated and Fig. 3.7(b) shows one of these connections at the end of the test, indicating that it underwent little inelastic response and did not deteriorate even when the specimen was loaded to its greatest deformations. Due to the importance

of the shear connection to the low-seismic SPSW concept, this result supports its use when moment-resisting frame connections are not needed to meet strength criteria. The application of simple beam-to-column connections in the test specimen resulted in robust connection performance, limited the demand on the columns, and enhanced the total energy dissipation capacity of the system by pushing the second story far into nonlinear response. While their use may not be appropriate when extremely high ductility and maximum redundancy are needed, they appear to be well-suited for applications in low seismic regions.

As predicted by a finite element analysis of the test specimen, the column strain readings showed that partial yielding occurred in the first story columns right below the beam-to-column connections. The yielding was concentrated in the column webs and started from Cycle 15 ($3\delta_y$) (as shown in Figs. A.16(d) and (e) in Section 5 of the Appendix), and in later cycles extended downwards up to a distance of about 250 mm from the intermediate beam's lower flange by the end of the test ($8\delta_y$). Minor yielding also occurred in the internal column flanges and extended downwards a distance of about 120 mm by the end of the test, but no yielding occurred in the external column flanges. No collapse mechanism developed in the system, since the yielding was concentrated only in a very small area in the column webs as the wall reached its maximum base shear, and the plastic strains remained well below the strain-hardening value.

While the overall SPSW system performed well throughout the test, clearly rupture of a portion of the column cross-section at the base is not an outcome that is tolerable in practice. Nevertheless, detailing of the column bases to enhance ductility can only improve the SPSW system behavior observed. The columns performed as intended up to a first-story displacement of $5\delta_y$, when the first column tear occurred. Thereafter, they behaved as if pinned at the base and permitted investigation of the behavior of the modular SPSW system at very large displacement ductilities.

3.11 Summary and Conclusions

A SPSW concept consisting of a modular construction technique with simple fabrication details and shear connections at the frame joints is proposed for adoption in low seismic regions where extremely high ductility is not required. Based on the proposed scheme, a large-scale two-story SPSW was designed using performance-based criteria that resulted in columns considerably smaller than those that would have been required based on current seismic design provisions. The wall was tested under gravity and cyclic lateral loading and demonstrated very good performance and energy dissipation capacity under 25 cycles of loading to a story displacement of eight times the yield displacement.

The conventional double-angle shear connections showed remarkably good performance with no significant damage, even at the end of the test after many nonlinear cycles. They provide rotational freedom at the beam-to-column joints, which reduces the demand on the columns as compared to the use of moment-resisting connections. The rotation also tends to improve the distribution of yielding in the infill plates, potentially increasing the energy dissipated by the system. Neither the one-sided lap splices in the infill plates nor the bolted nature of the system contributed to any deterioration, and they permitted the full development of the infill plate capacity in the critical story. The wall reached its maximum shear capacity at a lateral drift ratio of 3.9%, which is well beyond the displacement ductilities expected from limited- and moderate-ductility seismic systems. Therefore, the SPSW concept discussed herein that aims primarily to achieve low in-place cost appears to be well-suited for use in low seismic regions. The development of appropriate capacity design provisions for these SPSWs is ongoing and they will be proposed in a future paper.

Table 3.1 Seismic force modification factors for SPSWs

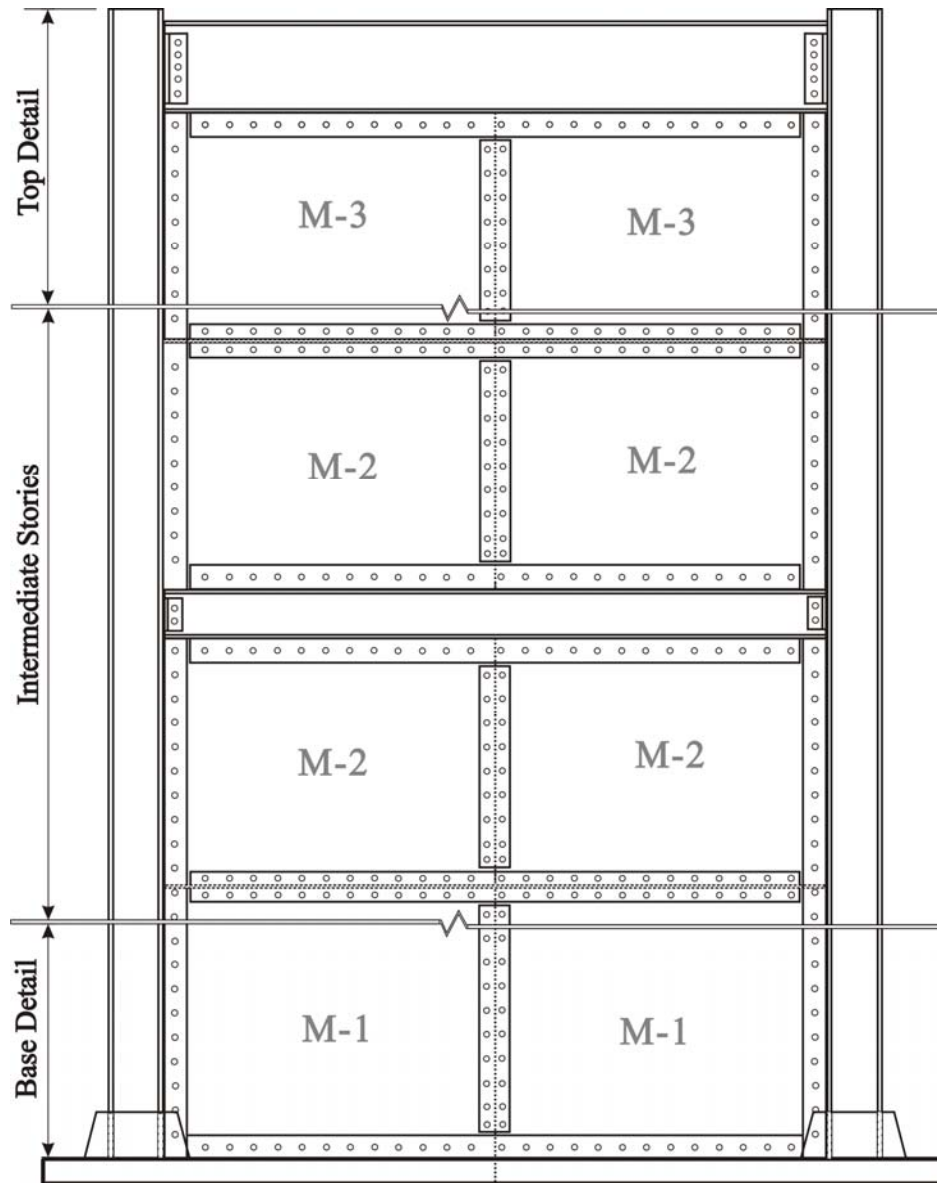
System	Performance level	CSA S16-09				ASCE 7-10	
		Designation	R_d	R_o	$R = R_d R_o$	Designation	R
SPSW	High	Type D	5	1.6	8	Special	7 (8 ^a)
	Medium	Type MD	-	-	-	Intermediate	-
	Low	Type LD	2	1.5	3	Ordinary	-

^a Dual system with Special Moment Frame capable of resisting at least 25% of prescribed seismic forces

Table 3.2 Cyclic base shear and displacement history

Cycle No.	Loading type ^a	Base shear	1 st story lateral displacement			Roof lateral displacement			2 nd story
		Q	δ	$\mu = \delta/\delta_y$	Drift ratio	δ_r	$\mu_r = \delta_r/\delta_{r,y}$	Drift ratio	Drift ratio
		kN	mm		%	mm		%	%
1	F	200	0.5	0.04	0.03	1.5	0.07	0.04	0.05
2	F	400	1.3	0.11	0.07	3.2	0.15	0.09	0.10
3	F	600	2.3	0.19	0.12	5.2	0.24	0.14	0.15
4	F	800	3.0	0.25	0.16	6.7	0.30	0.18	0.19
5	F	1000	4.4	0.37	0.24	9.2	0.42	0.25	0.25
6	F	1000	4.4	0.37	0.24	9.2	0.42	0.25	0.25
7	F	1000	4.4	0.37	0.24	9.2	0.42	0.25	0.25
8	D	1920	12	1	0.65	22	1	0.59	0.52
9	D	1920	12	1	0.65	22	1	0.59	0.52
10	D	1920	12	1	0.65	22	1	0.59	0.52
11	D	2320	22	1.8	1.19	36	1.6	0.96	0.73
12	D	2320	26	2.2	1.41	41	1.9	1.09	0.79
13	D	2150	24	2	1.30	38	1.7	1.01	0.73
14	D	2400	36	3	1.95	56	2.5	1.49	1.05
15	D	2350	36	3	1.95	56	2.5	1.49	1.05
16	D	2320	36	3	1.95	56	2.5	1.49	1.05
17	D	2500	48	4	2.60	74	3.4	1.97	1.36
18	D	2450	48	4	2.60	74	3.4	1.97	1.36
19	D	2625	60	5	3.25	97	4.4	2.58	1.94
20	D	2350	60	5	3.25	96	4.3	2.54	1.86
21	D	2200	72	6	3.90	112	5.1	2.98	2.09
22	D	2050	72	6	3.90	111	5.0	2.96	2.04
23	D	2060	84	7	4.55	126	5.7	3.36	2.20
24	D	1900	84	7	4.55	125	5.7	3.33	2.15
25	D	1830	96	8	5.20	137	6.2	3.65	2.15

^a F: force control, D: displacement control



(a)

Fig. 3.1 Modular SPSW concepts: (a) Concept 1, (b) Overview of Concept 2, (c) Assembled modules of Concept 2, (d) Concept 3 and two potential beam-to-column connections

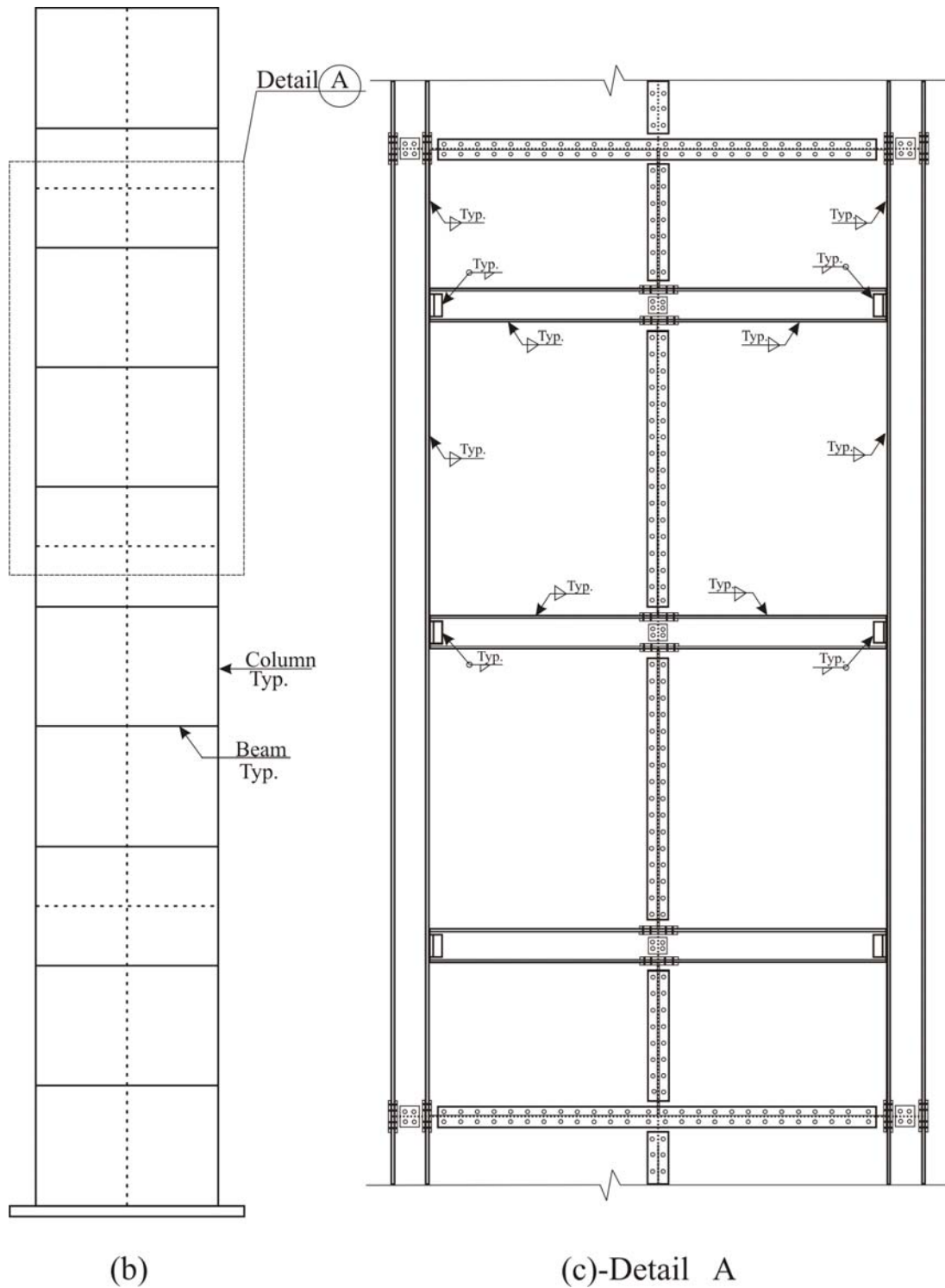
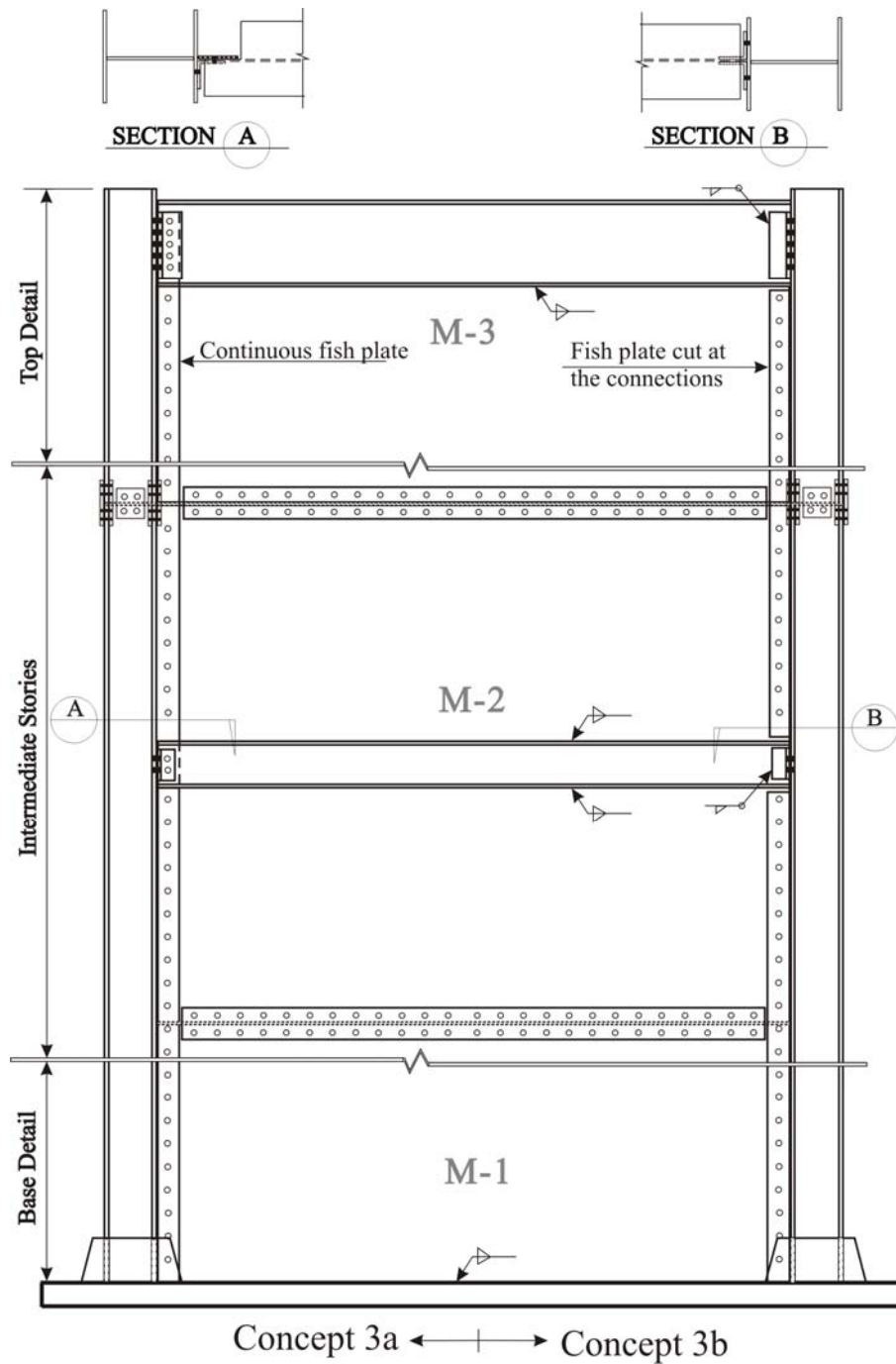
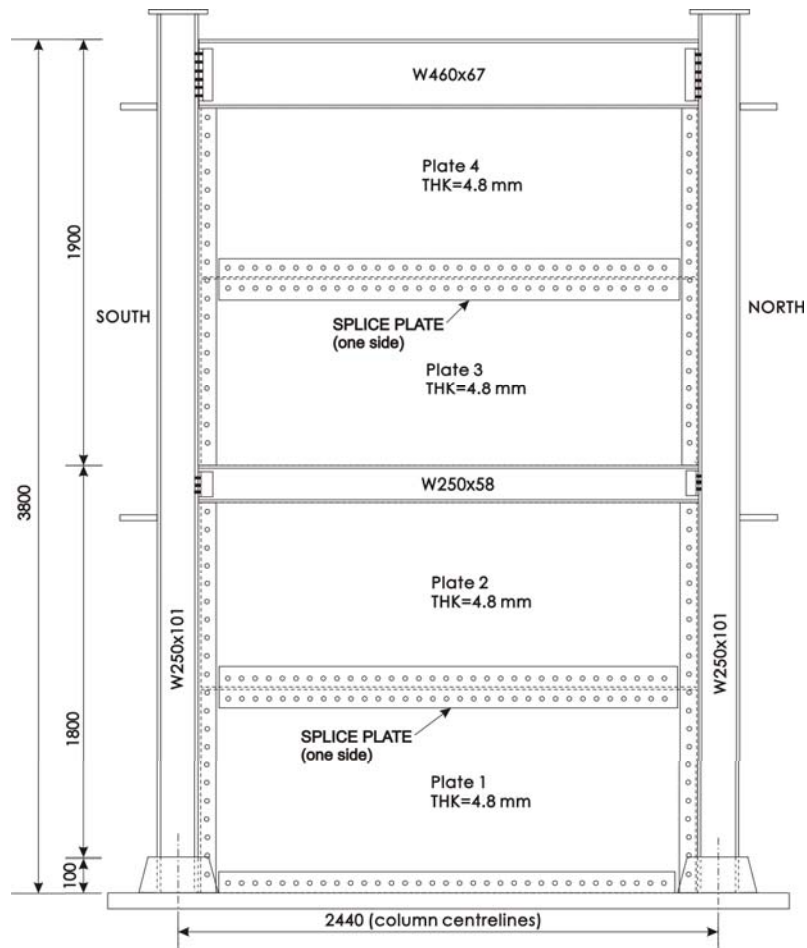


Fig. 3.1 Modular SPSW concepts: (a) Concept 1, (b) Overview of Concept 2, (c) Assembled modules of Concept 2, (d) Concept 3 and two potential beam-to-column connections (cont.)



(d)

Fig. 3.1 Modular SPSW concepts: (a) Concept 1, (b) Overview of Concept 2, (c) Assembled modules of Concept 2, (d) Concept 3 and two potential beam-to-column connections (cont.)



(a)



(b)

Fig. 3.2 Test specimen: (a) Schematic diagram, (b) East elevation (splice plates installed on far side)

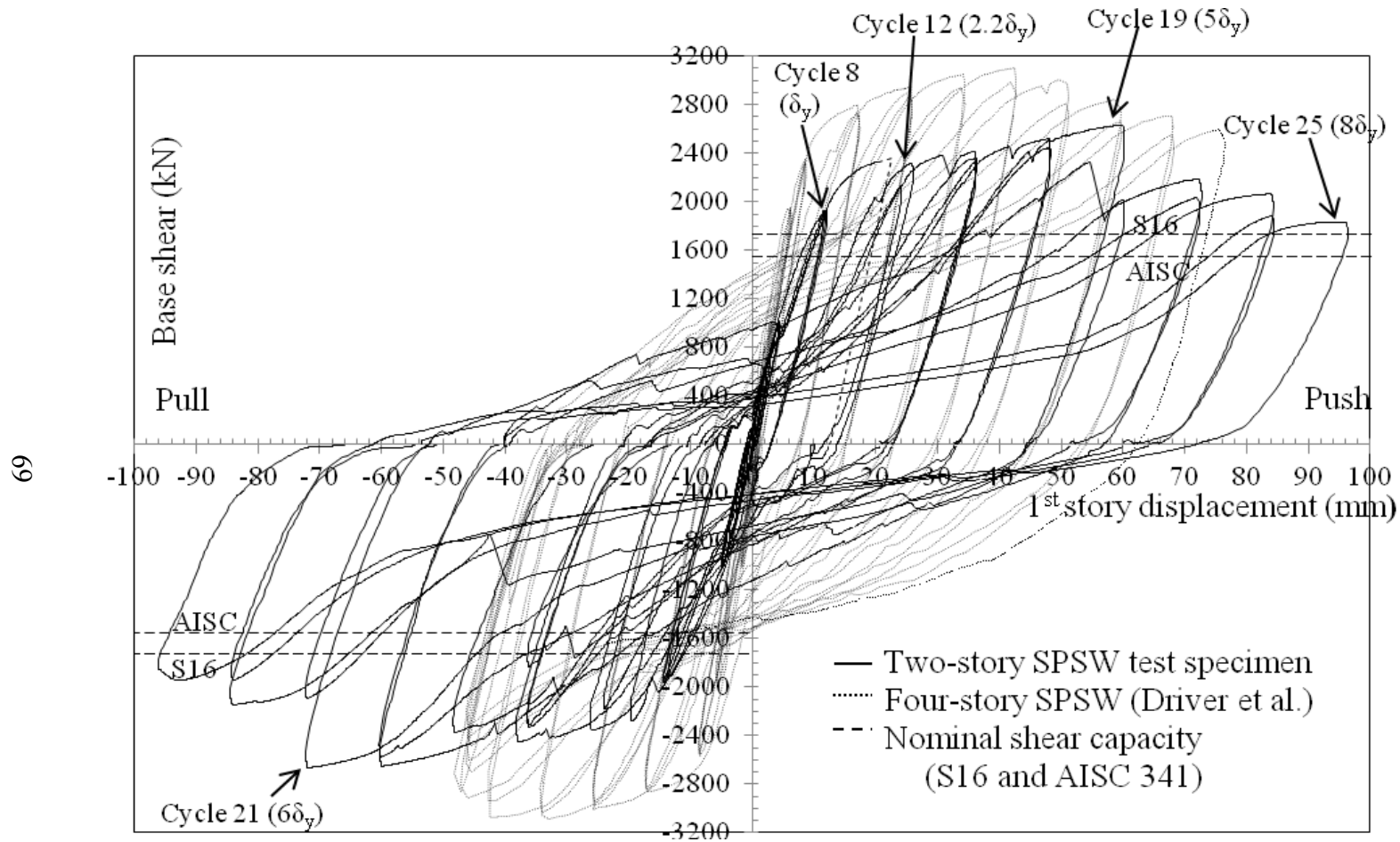


Fig. 3.3 Hysteresis curves for the first story lateral displacement versus base shear

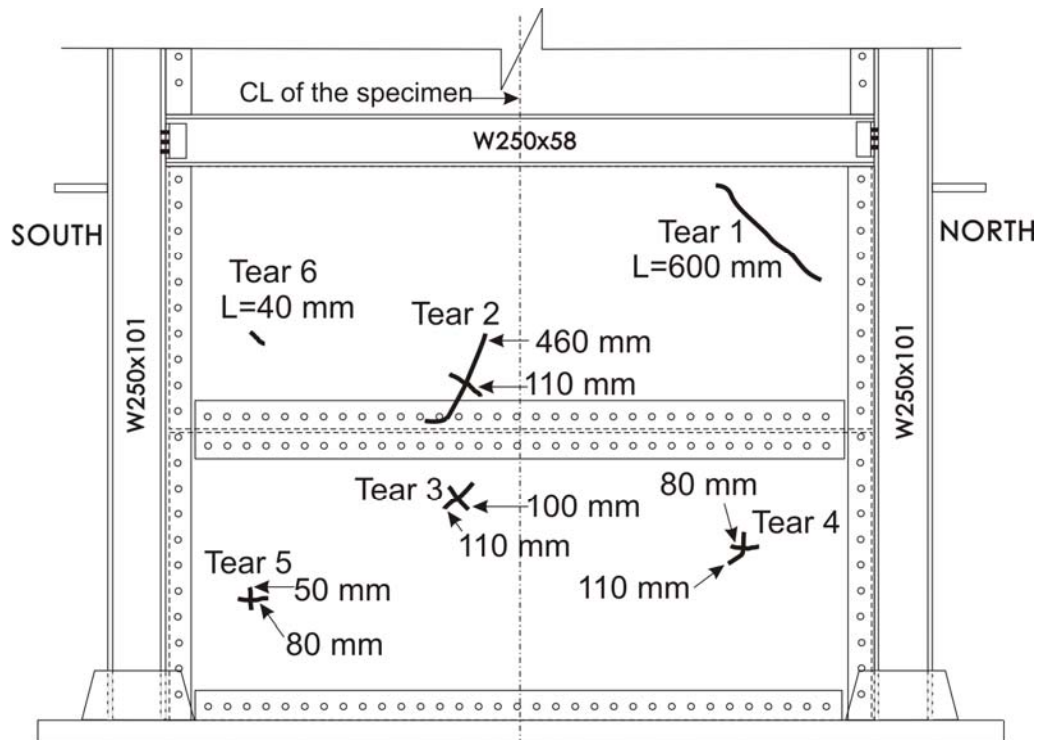
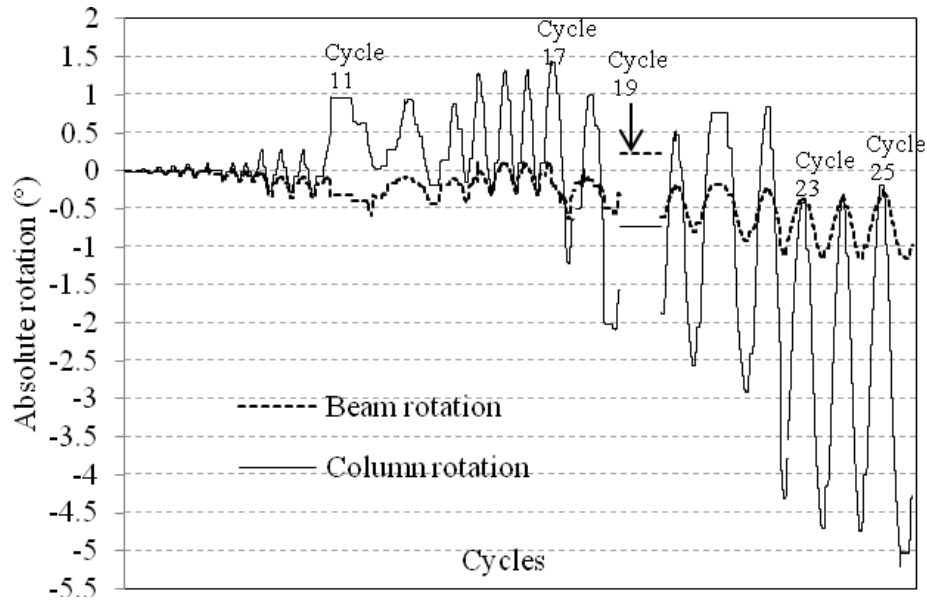
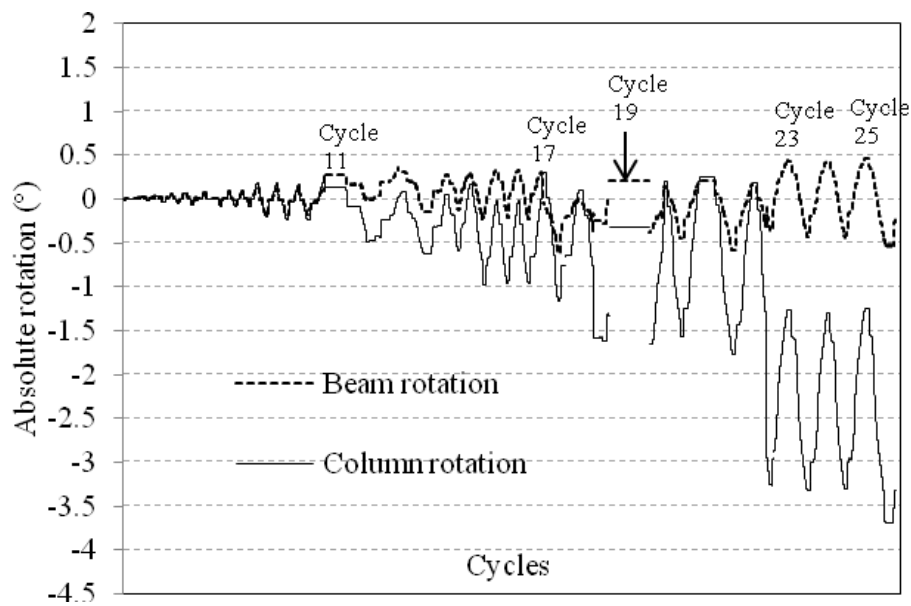


Fig. 3.4 Tearing in the first story infill plate at the end of the test



(a)



(b)

Fig. 3.5 Rotations of beams and columns at the North-side joints, (a) first story, (b) second story

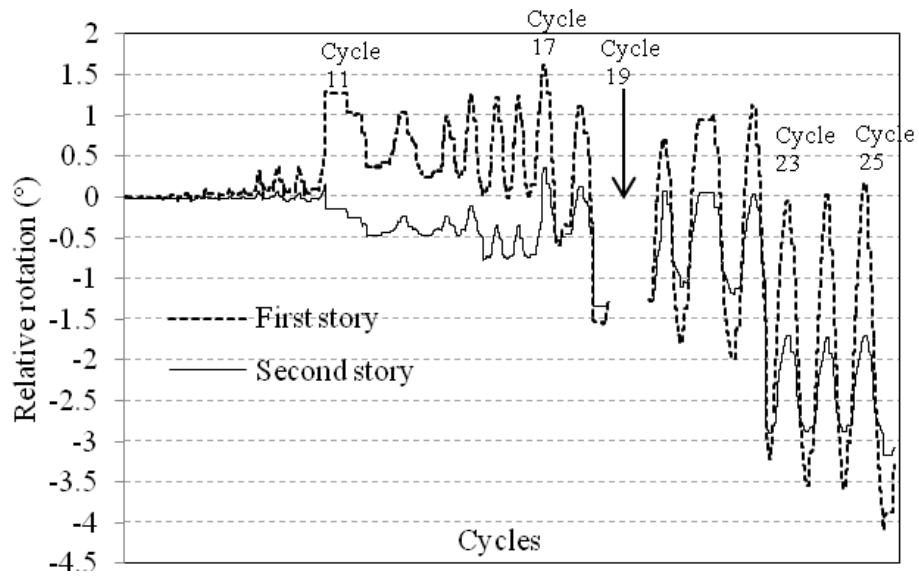


Fig. 3.6 Relative rotations between north end of beams and adjacent column



(a)



(b)

Fig. 3.7 Shear connection: (a) As fabricated, (b) At end of test

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4. BEAM DESIGN FORCE DEMANDS IN STEEL PLATE SHEAR WALLS WITH SIMPLE BOUNDARY FRAME CONNECTIONS³

4.1 Introduction

Steel plate shear walls (SPSWs) designed based on the capacity method have exhibited excellent performance when subjected to severe cyclic lateral loading in the laboratory. Many previous studies have shown that they possess a high level of initial stiffness, lateral force resistance, ductility, robustness, and redundancy. Since most previous research has investigated SPSWs with moment-resisting beam-to-column connections, they are generally considered to be a highly-ductile dual system that tends to be economical for high-seismic regions only. Because of the exceptional inherent redundancy and resilient performance of SPSWs, they should not need onerous beam-to-column connection detailing in lower seismic regions. Where extremely high ductility and redundancy is not required, the use of conventional simple beam-to-column connections in SPSWs has been proposed and their good performance verified (Moghimi and Driver 2013). Shear connections at the frame joints reduce the cost of the system by permitting significant reductions in both connection fabrication cost and column force demands, while still providing the desired level of seismic performance in regions of low or moderate seismicity.

The superior characteristics of a SPSW system are contingent on the proper performance of the beam members. In the case of simple connections, this objective is achieved in part through the flexibility of the connection elements. The design internal force demands on the beams should be evaluated accurately in the context of the capacity design approach. These demands affect not only the design of the beams themselves, but also the beam-to-column connections and the

³ A version of this chapter has been submitted for publication in the *Journal of Structural Engineering*, ASCE.

columns.

This research aims to develop a means of accurately evaluating the design actions—axial force, shear force, and bending moment—on the beams and beam-to-column connections of SPSWs with simple connections, based on capacity design principles and the results of detailed numerical studies. It is shown that the method described in the commentary of the AISC seismic provisions (AISC 2010) to evaluate the axial forces in the beams for the purpose of column design may not be appropriate for use in determining the forces in the beams and beam-to-column connections for walls with simple boundary frame connections. Also, it is shown that the shear in the intermediate beam is affected significantly by the infill plate yield pattern above and below the beam. Design methods are suggested specifically for axial and shear force estimation in the beams of SPSW systems with simple frame joints and the proposed methods are verified against experimental results.

4.2 Background

Due to the indeterminate nature of the SPSW system, evaluating the design force demands in the boundary frame members is a challenge. Aside from the plane frame analysis method, a number of analytical approaches have been proposed to define the design actions on the frame members that can be especially helpful at the preliminary design stage when the frame elements have not yet been selected. The commentary to the AISC seismic provisions (AISC 2010) recommends a number of such simplified analysis approaches based on capacity design principles. Among them is the method “combined plastic and linear analysis”, originally developed primarily to evaluate the design actions on the columns of SPSWs with rigid boundary frame connections (Berman and Bruneau 2008). In this method, the beams are designed for the larger effect of the yielded infill plate tension field stresses and the factored load combinations, and then the columns are designed based on the capacity method to withstand the tension field and frame actions. One component of the column design forces is imposed by the

axial forces that develop in the beams. The share of the axial force in each beam attributed to the inward pull of the infill plates on the columns is estimated by modeling the column as a continuous elastic flexural member that is pinned at the base and supported by linear elastic springs at the story beam locations. The spring stiffness at any story is equal to the elastic axial stiffness of the beam at that level, considering one-half of the beam span as the spring length. The column is then analyzed under the horizontal component of the infill plate yield force distributed uniformly over each story, and the axial force in each spring is evaluated. The total axial force in each beam at each end is taken as the corresponding spring force, plus or minus one-half of the net horizontal component of the infill plate yield forces applied above and below that beam. This method is a developed version of the one originally proposed by Sabelli and Bruneau (2006).

Qu and Bruneau (2010) discussed the capacity design of intermediate beams of SPSW systems with reduced beam sections and moment connections. Different sources of axial force in the beams were identified and examined. To evaluate the portion of the beam axial force arising from the inward pull on the columns from the infill plate tension field, the method “combined plastic and linear analysis” from the commentary to the AISC seismic provisions (AISC 2010), and described above, was used. Shear force demands in the intermediate beams and the moments at the faces of the columns were also discussed. A capacity design procedure for an intermediate beam with reduced beam sections and moment connections was presented.

4.3 Scope and Objectives

The SPSW system with simple beam-to-column connections has shown excellent overall cyclic performance, despite relatively inexpensive detailing (Moghimi and Driver 2013). The objective of this research is to develop a means of calculating reliable capacity design force demands on the beams and beam-to-column connections of such systems. This paper investigates the axial force, shear force,

and bending moment distributions in the beams to aid in the development of a comprehensive capacity design procedure for these walls.

The axial force distribution in the beams of SPSWs is highly indeterminate. As such, by combining the results of many and varied numerical models with capacity design principles, a reliable and simplified method is proposed that transforms the system to a determinate one for the sake of axial load analysis of the beams. Using this simplified approach, the upper bound axial forces in the beam are estimated.

Unlike the axial forces, the shear forces and bending moments in the beam are statically determinate under the conventional capacity design assumption of uniform and yielded stress distributions in the infill plates and the approximation of pinned end conditions. However, the factors that affect the shear force distribution in the beam have been the subject of only limited studies. Therefore, all components that have a significant effect on the shear force demands are investigated and verified using numerical simulations. For intermediate beams with the same infill plate thickness (and steel grade) above and below the beam, current practice underestimates the maximum shear force in SPSWs with simple frame connections and produces an incorrect shear distribution. A capacity method that provides appropriate shear design forces and distributions is presented. The results of the proposed methods for both axial and shear forces are verified with the results of a physical SPSW test.

4.4 Internal Force Demands on Beams of SPSW Systems with Simple Frame Connections

In general, three design actions are applied to the beams of SPSWs: axial force, shear force, and bending moment. When simple frame connections are employed, any moment transfer to the column can reasonably be neglected. Therefore, the beams are designed as simply-supported members to resist forces due to gravity load, tensile yielding of the infill plates, and the external design lateral loads. The

effect of gravity load should be considered in the design of every element of the boundary frames simultaneously with capacity design loads; however, they are simply additive to the beam capacity design actions, so in the interest of simplicity they are excluded from subsequent discussions.

Figure 4.1(a) shows the simply supported beam at the i^{th} story of a SPSW that is subjected to full tensile yielding of the infill plates above (denoted by subscript $i+1$) and below (denoted by subscript i), the external lateral mechanism loads, F_{iL} and F_{iR} , applied to the left (tension) and right (compression) column, respectively, and the net shear forces in the columns adjacent to the beam, V_{cL} and V_{cR} (sum of the shear forces above and below the beam). The axial forces and moments in the columns are not shown in the figure since they do not impact the beam design actions. The clear distance between the columns is L_c and the beam depth is d_b . The distributed forces from the infill plates in each story have been resolved in the figure into their horizontal and vertical components, ω_{xi} and ω_{yi} , respectively. In order to calculate these components, the infill plate material is assumed to be yielded for capacity design and the angle of inclination of the tension field can be determined according to established procedures (CSA 2009; AISC 2010). Figure 4.1(a) also shows the shear, V_{iL} and V_{iR} , and axial, P_{iL} and P_{iR} , reactions at the left and right ends of the beam, respectively.

The tensile yielding of the infill plates induces axial force, shear, and moment in the beams as a result of three mechanisms. First, when the infill plate thicknesses above and below an intermediate beam differ, there are distributed unbalanced forces on the beam. These distributed forces are denoted as $\Delta\omega_{xi}$ and $\Delta\omega_{yi}$ in Figs. 4.1(b) and Fig. 4. 1(c), respectively, and are defined as follows:

$$\Delta\omega_{xi} = \omega_{xi} - \omega_{xi+1} \quad (4.1a)$$

$$\Delta\omega_{yi} = \omega_{yi} - \omega_{yi+1} \quad (4.1b)$$

For the top beam, the yielding force is only applied below the beam, and the unbalanced forces in Eqs. (4.1a) and (4.1b) becomes $\Delta\omega_{xi} = \omega_{xn}$ and $\Delta\omega_{yi} = \omega_{yn}$, where n is the number of stories in the wall. Second, as discussed by Qu and Bruneau (2010), the horizontal components of the tension fields above and below each intermediate beam apply a distributed couple about the beam centerline. This force is shown as m_i in Fig. 4.1(c), and is defined as follows:

$$m_i = (\omega_{xi} + \omega_{xi+1}) d_b / 2 \quad (4.2)$$

At the top beam, the infill plate force above the beam, ω_{xi+1} , is zero. Third, the yielded infill plates pull the columns toward each other, and the beams work like compressive struts to keep the columns apart. The axial force distribution in the beams and, therefore, the design forces for the beam-to-column connections are also influenced by the means of transfer of the external lateral design load from the floor and roof diaphragms into the SPSW system. The axial force, shear force, and bending moment diagrams resulting from the various component actions are shown in Figs. 4.1(b) and Fig. 4.1(c).

4.5 Axial Force Evaluation

The net axial design force demands are shown on the beam centerline in Fig. 4.1(b). This force distribution is derived from the combined effects of the horizontal component of the unbalanced infill plate yielding force on the beam, the column reactions due to the inward pull from the yielded infill plates, and the external lateral mechanism loads. Under the first of these loads alone ($\Delta\omega_{xi}$ in the figure), the beam is statically-determinate if it is assumed that the infill plates above and below are fully yielded (and the axial support stiffnesses at the two ends are equal), but under the latter two it is indeterminate. As such, a main part of this paper is devoted to developing a method that evaluates the internal beam actions arising from these loads. The end reaction force from the inward pull of the infill plates on the columns is investigated based on the results of detailed numerical investigations and this aspect is discussed later in the paper.

The external lateral mechanism loads at each story (represented by F_{iL} and F_{iR} in Fig. 4.1(a)) need to be distributed to reflect the means of load transfer from the floor and roof diaphragms to the SPSW system; several possible cases are shown schematically in Fig. 4.2. The figure shows two-story SPSWs with possible seismic load transfer mechanisms, and the corresponding typical axial force distributions in both beams. The same infill plate thickness is assumed in both stories, so there is no unbalanced infill plate force for the intermediate beam, and the axial forces are therefore entirely due to the inward pull of the columns and the load transfer mechanism from the diaphragm. Conversely, the infill plate below the top beam causes a distributed axial force that varies from tension at the compression-column side to compression at the tension-column side, and the other two sources of axial force are additive to this.

For a real structure under earthquake loading, the SPSW would likely be subjected to some combination of load transfer distributions “a” to “d” shown in Fig. 4.2. The arrows show the means of seismic load transfer to the system, which in all cases is from left to right, and “C” and “T” represent compressive and tensile axial forces in the beam, respectively. The actual transmission of seismic loads into the SPSW depends on the layout of the building plan, the number and locations of SPSWs oriented in each direction, the nature of the collector elements in the surrounding floor structure, and the method of tying the floor and roof diaphragms directly to the SPSW, and therefore must be determined on a case-by-case basis. The differences among the axial force diagrams for distributions “a” to “d” indicate that the means of load transfer to the SPSW has a considerable influence on the axial forces that develop in the beams; nevertheless, their effects on other internal forces and deformations tend to be small enough that for design purposes they can normally be considered negligible. Specifically, the influence of the diaphragm load transfer mechanism on shear and moment in the beams, axial force, shear and moment in the columns, overall deformed shape of the wall, yielding pattern of the infill plates, and pushover curve of the entire wall tends to be small.

4.6 Shear Force and Bending Moment Evaluation

The net demands that induce shear force and bending moment in the beam are shown in their positive sense on the beam centerline in Fig. 4.1(c), along with the corresponding shear and bending moment diagrams. Although the shear diagram is not symmetric, the maximum moment, $M_{max\ i}$, occurs at midspan of the beam. This phenomenon can be explained by the presence of the distributed moment, m_i , which causes a constant shear in the beam, but no moment. Therefore, the moment arises solely from the unbalanced infill plate force, $\Delta\omega_{yi}$, and its maximum value is defined as follows:

$$M_{max\ i} = \Delta\omega_{yi} L_c^2/8 \quad (4.3)$$

The maximum shear reaction, V_{iR} , occurs at the face of the compression column and has two contributing actions: the unbalanced infill plate force, $\Delta\omega_{yi}$, and the distributed moment, m_i , defined in Eq. (4.2). The contribution to the maximum shear reaction of the unbalanced infill plate force, V_{Li} , is:

$$V_{Li} = -\Delta\omega_{yi} L_c/2 \quad (4.4)$$

and the contribution of the distributed moment, V_{Mi} , to the shear force along the length of the beam is:

$$V_{Mi} = -m_i = -d_b (\omega_{xi+1} + \omega_{xi})/2 \quad (4.5)$$

In a case where the infill plates above and below an intermediate beam have the same thickness and are fully yielded, the shear from the unbalanced infill plate force, V_{Li} , would be negligible, but the constant shear, V_{Mi} , still exists. When the infill plate thicknesses differ, the two sources of shear are additive:

$$V_{iR} = V_{Li} + V_{Mi} \quad (4.6)$$

The smaller shear reaction at the face of the tension column, V_{iL} , is also determined from Eq. 6, but the value of V_{iL} from Eq. 4 is taken positive. Equations 1 through 6 are equally applicable to the top beam by considering the fact that the force components above the beam are zero ($\omega_{y\ i+1} = \omega_{x\ i+1} = 0$).

4.7 Numerical Studies

In order to establish a simple and reliable tool for evaluating the axial force, shear force, and bending moment distributions in beams of SPSWs with simple frame connections, a series of numerical models were studied. Figure 4.3 shows the two wall configurations investigated, where the overall dimensions and boundary frame sizes were selected based on previous experimental programs at the University of Alberta (Driver et al. 1998, Moghimi and Driver 2013) involving approximately half-scale test specimens. While the wall tested by Moghimi and Driver (2013) had simple frame connections and the one tested by Driver et al. (1998) had rigid connections, for the purposes of this investigation all numerical models possess simple frame connections. The boundary frame members and column spacings shown in Fig. 4.3 apply to all models that have the same number of stories as depicted. For simplicity, the material properties for all components of the model walls are elasto-plastic, with a yield stress of 350 MPa, except that the models of the test specimens themselves use measured material properties including strain hardening and softening for all beams, columns, and infill plates. The infill plates are modeled using shell elements and the beams and columns are represented by beam elements to facilitate the extraction of boundary frame member forces for this study. The infill plates extended only to the surfaces of the beam and column flanges and were “attached” to the frame members, which were positioned at the member’s cross-sectional centroid, through the use of appropriate nodal constraints. These simplified models were validated against both the test results and more detailed models that use either solid or shell elements to simulate the boundary members. All walls are analyzed using a monotonic pushover analysis under displacement control.

Several properties of the walls were treated as variables: the height of each story (center-to-center), h_i , the infill plate thicknesses, w_i , the diaphragm load transfer pattern, and the lateral load distribution over the wall height as a fraction, μ_i , of the base shear, V_b . The values of the properties used in the study of the two-story wall (2SPSW), shown in Fig. 4.3(a), are summarized in Table 4.1. The table column “LT” represents the diaphragm load transfer pattern, for which the possible patterns “a” to “d” are depicted in Fig. 4.2. Walls 2SPSW-B01 to 2SPSW-B03 possess a constant infill plate thicknesses and a uniform lateral load distribution over the wall height; they differ only in the means of load transfer from the diaphragms at the floor and roof levels. Walls 2SPSW-B11 to 2SPSW-B13 and 2SPSW-B21 to 2SPSW-B23 are the same as walls 2SPSW-B01 to 2SPSW-B03, respectively, except that the infill plate thicknesses are different in the two stories. Wall 2SPSW-B14 is the same as walls 2SPSW-B11 to 2SPSW-B13, but with diaphragm load transfer type “d”. Walls 2SPSW-B31 and 2SPSW-B41 are the same as walls SPSW-B01 and 2SPSW-B21, respectively, but with a non-uniform lateral load distribution over the height of the wall. Wall 2SPSW-B51 is the same as wall SPSW-B41, but with a different panel aspect ratio (shorter story heights). The row “Test specimen” in Table 4.1 provides the properties of the physical test specimen (Moghimi and Driver 2013) and its base shear in the “push” direction from the cyclic test results. Wall “Test specimen model” is a numerical model of the test specimen and wall 2SPSW-BS1 is identical, but with diaphragm load transfer type “c”.

The other SPSW system investigated is the four-story wall (4SPSW) depicted in Fig. 4.3(b). The variables for this wall are similar to those for the two-story wall and are summarized in Table 4.2. Wall 4SPSW-B01 is the base model with varying infill plate thicknesses in each story, a uniform lateral load distribution over the wall height, and diaphragm load transfer type “c” at the floors and roof. Walls 4SPSW-B02 and 4SPSW-B03 are the same as wall 4SPSW-B01, but with different diaphragm load transfer patterns. Wall 4SPSW-B11 is identical to wall 4SPSW-B01, except with different infill plate thicknesses selected so that at each

story the infill plate thickness is proportional to the corresponding story shear. Walls 4SPSW-B12 and 4SPSW-B13 are similar to wall 4SPSW-B11, but with different diaphragm load transfer patterns. Wall 4SPSW-B21 is similar to wall 4SPSW-B11, but with a non-uniform lateral load distribution over the height of the wall, giving rise to different infill plate thicknesses so that they remain proportional to the corresponding story shear. Wall 4SPSW-B31 is the same as wall 4SPSW-B11, but with different infill plate thicknesses such that the thickness changes only once, which is considered to be a more practical design solution. Wall 4SPSW-B41 is the same as wall 4SPSW-B21, but with a uniform infill plate thickness. Wall 4SPSW-B51 is the same as wall 4SPSW-B11, but with a shorter story height (i.e., different panel aspect ratio).

4.7.1 Beam axial force results

Exploring the moment and shear distributions in the columns gives insight into the overall lateral behavior of SPSW systems. This, in turn, provides an understanding of the nature of the axial force that is transferred into the beams. Figures 4.4(a) and 4.4(b) show the moment and shear distributions, respectively, in both columns of the two-story SPSW systems studied when the maximum base shear is achieved. Since the diaphragm load transfer pattern affects only the axial force distributions within the beams themselves, all the walls with same properties other than this parameter (i.e., 2SPSW-B01 to 2SPSW-B03, 2SPSW-B11 to 2SPSW-B14, and 2SPSW-B21 to 2SPSW-B23) have essentially the same column moment and shear distributions, so only the first wall of each of these sets is represented in the figure. (Wall 2SPSW-B51 is not shown in the figure due to its different story height.)

It is significant to note that for all walls the moment and shear distributions in the compression column are nearly identical, while changes in parameters such as the infill plate thickness or lateral load distribution over the wall height have a considerable effect on the moment and shear distributions in the tension column. The major differences revealed between the compression and tension columns can

be explained largely by the process of tension field yielding development in the infill plates. Figure 4.5(a) shows the effective plastic strain contours of a typical SPSW under lateral loads at the drift ratio of 0.6%, which is at the early stage of nonlinear response. When the external lateral loads are applied to the SPSW system (with any type of load transfer pattern shown in Fig. 4.2, but type “c” is assumed in Fig. 4.5(a)), both the tension and compression columns tend to deform inward in each story relative to their ends due to their role in anchoring the tension field in the infill plates. The complex two-dimensional stress field in the infill plates develops a characteristic signature wherein yielding begins along a wide band between the panel corners in the direction of the tension field and, as the story shear increases, yielding progresses both from the compression column toward the tension column along the upper beam and from the top to the bottom of the panel along the compression column, as indicated in Fig. 4.5(a). For comparison, Fig. 4.5(b) shows the effective plastic strain distribution of the same wall at the lateral drift ratio of 2.5%. This characteristic progression of yielding results in the conventional capacity design approach producing more reliable design forces in the compression column than in the tension column for stories where the infill plate is only partially yielded. That is, the compression column is pulled more uniformly by the infill plate, and its shear force and bending moment diagrams therefore always have distributions similar to those of a classical uniformly-loaded continuous beam. Conversely, the loading on the tension column is more variable and is affected by the infill plate thickness, lateral load distribution over the height of the wall, and column size, etc. This phenomenon is particularly important for SPSWs with simple frame connections, which would typically be designed only for limited-to-moderate ductility demands.

Tables 4.3 and 4.4 show axial force results for the two-story and four-story wall systems, respectively, described in Tables 4.1 and 4.2. These results are representative of the point where the wall reaches its full capacity (the mechanism load). The “Beam axial forces” in Table 4.3 are the axial force demands at each end of the first and second story beams. “ P_L ” and “ P_R ” refer to the axial force

reaction at the left (tension column) and right (compression column) connections, respectively. Positive axial forces are in tension and negative forces are in compression. The “Beam axial force change” values in Tables 4.3 and 4.4 show the change in the axial force in each beam over its full length, normalized by the horizontal resultant of the yielding force in the infill plates above and below the intermediate beams, or below the top beam. (The effect of the distributed diaphragm load transfer is removed from the numerator for the two type “d” cases to isolate the effect of the infill plate forces.) The symbol “–” indicates that the normalizing force is zero (i.e., the infill plates above and below the beam have the same thickness). Ratios less than unity indicate that the infill plate has yielded partially and ratios less than about 0.5 suggest the infill plate has not yielded. Ratios of 1.0 and larger are considered to be indicative of fully-yielded infill plates.

Ratios larger than unity occur because of two effects. First, the expression used for the angle of inclination of the infill plate forces is approximate and represents an average value for the panel; it gives a value that is slightly smaller and larger, respectively, than the average of the corresponding angles in the numerical results adjacent to the beams and column. Therefore, the resulting distributed horizontal component of the tensile force on the beam is actually slightly larger than that estimated using conventional capacity design methods (represented by the denominator of the ratio). Second, despite the widely-used assumption that the compressive stresses in the infill plates are negligible, the stress state in the infill plate is indeed two-dimensional, with considerable compressive-to-tensile principal stress ratios in the models investigated of about 0.2~0.3 and 0.3~0.5 around the beams and columns, respectively. This stress state causes earlier yielding in the infill plates than is predicted by the “tension strip” analogy, and at the same time applies to the boundary elements a compressive force perpendicular to the tension field direction. These effects combine to cause the finite element models using nonlinear shell elements for the infill plate (instead of strip elements) to reveal a larger axial force and smaller shear/moment for both the

beams and columns compared with the classical capacity design method that approximates the infill plate as a series of independent strips. However, the differences between the demands obtained from the nonlinear finite element models and capacity design are much smaller in the beam than in the columns, and the capacity design approach seems to provide acceptable design forces for the beams in SPSWs with simple frame connections (Tables 4.3 to 4.7).

The “Column shear at connections” in Tables 4.3 and 4.4 are the sum of the column shear forces immediately above and below the beam-to-column joint for the intermediate stories, and below the joint for the top story. The net column shear forces for the left and right columns are “ V_{cL} ” and “ V_{cR} ”, respectively. These shear forces are normalized by the horizontal component of the total force applied from the yielded infill plates to the columns on a simplified tributary-width basis. That is, for the top beam the infill plate force on the top half of the column at that story is used, and for intermediate beams the infill plate force from mid-height of the story below to mid-height of the story above is considered. Positive ratios indicate inward column shear forces acting on the joint region and negative values represent outward shear forces.

4.7.2 Beam shear force and bending moment results

Since the lateral load transfer pattern from the diaphragm affects only the axial force distributions in the beams significantly, all the walls with the same properties other than this load transfer pattern (i.e., walls 2SPSW-B01 to 2SPSW-B03, 2SPSW-B11 to 2SPSW-B14, 2SPSW-B21 to 2SPSW-B23, 4SPSW-B01 to 4SPSW-B03, and 4SPSW-B11 to 4SPSW-B13) have essentially the same beam shear distributions. For this reason, only the first wall of each set is studied here.

Tables 4.5 and 4.6 show the shear force results for each beam of the two-story and four-story wall systems, respectively, described in Tables 4.1 and 4.2. The “FE Results” in the tables show the shear force demands at each beam end, where “ V_L ”

and “ V_R ” refer to the shears at the left (tension column) and right (compression column) connections, respectively. “ $M_{max,FE}$ ” is the maximum moment in the beam span, which usually occurs around the beam centerline. However, when the infill plates are only partially yielded, the resulting distributed forces are not uniform. As a result, the maximum moment shifts slightly from mid-span toward the tension-column side.

The values “FE results analysis” in Tables 4.5 and 4.6 show some further analysis on the numerical results. First, based on Eq. (4.6), the shear from the numerical results is resolved into the two main components of the beam end shears, $V_{li} = \pm(V_{iL} - V_{iR})/2$ and $V_{Mi} = -(V_{iL} + V_{iR})/2$. Since the resultant moment from the component “ V_M ” is zero (assuming that m_i is constant along the beam length), the “ V_I ” component is solely responsible for the moment distribution in the beam span and gives rise to the maximum moment $M_V = V_{li} L_c/4$. The values of the ratio “ $M_{max,FE}/M_V$ ” in most cases are close to unity, which suggests that the moments are derived mainly from the vertical component of the unbalanced infill plate tension field and that resolving the shear forces into its two components represents the true shear force distributions in the beams of the SPSW system. Only in wall 4SPSW-B41, which has the same infill plate thickness in every story of the wall, are the moment ratios considerably less than unity. In cases where the infill plate thicknesses above and below an intermediate beam are the same or similar, the moment from the “ V_I ” component may exist and can be significant. The effects of partial yielding in the infill plates on the beam demands are explained in the next section.

The “FE-to-CD ratios” in Tables 4.5 and 4.6 compare the numerical results with the outcome of a conventional capacity-design analysis. The numerical results (the numerators of the ratios) are all given in the first several columns of the same table. For the capacity design values (the denominators), the tension field forces are determined based on the infill plate yield stress, and the codified equation (CSA 2009; AISC 2010) is used to evaluate the angle of inclination of the tension

field. The horizontal and vertical components of the tension field actions on each beam (ω_{xi} and ω_{yi} , respectively) are then calculated. The shear force V_{li} is evaluated from Eq. (4.4), where $\Delta\omega_{yi}$ is defined in Eq. (4.1b). The shear force V_{Mi} is determined from the ω_{xi} and ω_{xi+1} components of the tension field actions and Eq. (4.5). The resultant shear at the left and right beam supports, V_{iL} and V_{iR} , respectively, are then obtained from Eq. (4.6). Also, from the V_{li} component of the shear, the maximum moment at mid-span of the beam is evaluated from Eq. (4.3). Among all the results ratios, the “ V_R ” component tends to be of highest importance since it determines the maximum shear force for design of the beams and their connections to the columns. Most internal force ratios are less than 1.0, which suggests that the capacity design method provides conservative design forces for the system. For both walls 2SPSW and 4SPSW, the only non-conservative results are for the intermediate beams where the infill plates above and below the beams have the same thickness and there is only partial yielding in the infill plates above the beam. This aspect is studied in detail in the next section.

4.8 Design Axial Force

4.8.1 Governing concepts

As discussed earlier, the shear force distribution in the columns has a direct effect on the axial force distributions in the beams. Also, the horizontal projection of the yielded infill plate forces above and below an intermediate beam causes a change in the axial force demand between the two ends of the beam. Both actions are influenced by the development of tension field yielding in the infill plates.

Therefore, as the results in Tables 4.3 and 4.4 show, the beam axial force demands depend partly on the progression of yield strain development in the infill plates of the system, which is a function of the column size, the choice of infill plate thicknesses, and the assumed lateral load distribution over the height of the wall.

The proposed method for determining design axial forces in the beams of SPSWs with simple frame connections is based on the assumption that all infill plates are yielded. Hence, only the results for the 17 walls that experienced general yielding

in all of the infill plates at the ultimate lateral load are considered. (Walls that developed little yielding in some or all of their infill plates constitute an important part of the development of the methods presented herein, but are not considered representative of walls designed using the capacity design method.) Assuming all the infill plates have yielded, the change in the axial force demand between the two ends of the beam is equal to the resultant horizontal projection of the yielded infill plate capacity above and below an intermediate beam, and below the top beam.

The compression column (column “R” in Tables 4.3 and 4.4) always has approximately the same shear force distribution shape, since for any lateral load transfer system at the floor, the infill plates pull the compression column against the beams, regardless of the infill plate thicknesses and lateral load distributions over the height of the wall. The total compression-column shear force at each connection (V_{CR}) is given in Tables 4.3 and 4.4 as a fraction of the horizontal projection of the fully-yielded infill plate tension field force on the columns above and below the connection based on a simplified tributary-width approach, as described previously. The net shear force in the compression column at the top beam is generally about 50~90% of the horizontal component of the infill plate force in the upper half of the top panel. The net shear force in the column at an intermediate beam of the four-story walls is typically about 70~100% of the horizontal force based on the tributary-width approach. For the two-story walls, the column shear force ratio at the first-story connection is about 100~125%, which is larger than 100% for two main reasons. First, the true tributary width for shear force estimation in the column is larger than that assumed between the mid-story points above and below the beam in two-story walls. Second, the effect of the progression of yielding in the infill plates, described earlier, is more pronounced in two-story walls such that the second story infill plate tends to bend the compression column against the first story beam (see Fig. 4.5(b)), which increases the shear demand in the compression column compared with the capacity design method. (Although models 4SPSW-B01 to 4SPSW-B03 were

excluded from the analysis due to the lack of infill plate yielding in the first story, it is worth noting that the high values of V_{cR} in the fourth story are due to the use of a very thin infill plate—present on one side of the beam only—resulting in the column shear from the limited frame action accounting for about half of the total value.)

Comparing the results of similar walls (such as 2SPSW-B01, 2SPSW-B11, and 2SPSW-B21) suggests that the significant variations in the compression-column shear forces arise not because of axial deformation of the beam, as implied by the “combined plastic and linear analysis” method (AISC 2010), wherein for the column analysis each beam is replaced by an axial spring having the same stiffness as the beam. Rather, the variations occur mainly because of the relative lateral deformations of the wall and the column at each story, which is a function of, for example, the infill plate thickness and lateral load distribution over the wall height. The elastic analysis of the columns recommended by AISC (2010) does not consider the relative lateral deformations of the individual stories.

The shear force and bending moment distributions in the tension column tend to be far more variable than in the compression column, as can be seen in Fig. 4.4. For instance, wall 2SPSW-B01 has a uniform plate thickness and a uniform lateral load distribution, giving characteristic shapes to both the moment (Fig. 4.4(a)) and shear (Fig. 4.4(b)) distributions in the tension column. Changing just the infill plate thickness in the top story (2SPSW-B11) or the lateral load distribution (2SPSW-B31) changes the relative lateral deformations of the tension column and the resulting distributions significantly.

4.8.2 Proposed method

Based on the concepts discussed in the previous section, the beams of SPSWs can be transformed into a statically determinate system for calculating the design forces in both the beam and beam-to-column connections. The total shear force in the compression column at each beam (i.e., the sum of the shear forces above and

below each beam-to-column connection) is estimated based on the range of potential values assuming full yielding of each infill plate. Regardless of the diaphragm load transfer mechanism at the floor, the maximum compression in the beams occurs when the shear in the compression column is maximum (i.e., using the 90% and 100%/125% factors for the top and intermediate beams, respectively) and the maximum tension happens when the shear in the compression column is minimum (i.e., using the 50% and 70%/100% factors for the top and intermediate connections, respectively). This gives the designer an idea about which value of the range should be selected for design force evaluation, and it is recommended that in general both cases be checked. In most cases the compression in the beam governs the design forces, and the maximum column shear should be selected. However, the axial tensile force may cause an increase in the end rotational stiffness of the connection, reducing its rotational capacity (Thornton 1997). This could have a negative effect on the lateral performance of the SPSW system, since rotational freedom of the simple joint reduces the demands on the columns and improves the uniformity of yielding of the infill plates at each story over the height of the wall (Moghimi and Driver 2013). Therefore, potential tension and compression both need to be assessed at the beam-to-column connections.

Having the total column shear (V_{cRi}) and design lateral load (F_{iR}) at each beam-to-column connection in the compression column, the axial force in the adjacent beam can easily be calculated from the free body diagram of the joint by subtracting the column shear force from the design lateral load at the connection, as follows:

$$P_{iR} = V_{cRi} - F_{iR} \quad (4.7)$$

as indicated in Fig. 4.1(a). The axial force demand at the other end of the beam (adjacent to the tension column) is then evaluated by adding the horizontal projection of the yielded infill plates above and below an intermediate beam or

below the top beam to the axial force demand in the beam at the end adjacent to the compression column:

$$P_{iL} = P_{iR} + \Delta\omega_{xi} L_c \quad (4.8)$$

The beam design axial forces are fully known at this stage. If required, the total shear force in the tension column (above and below the beam-to-column connection) can be evaluated from a free body diagram of that joint.

For the purpose of column design, the axial force in the beam can be evaluated based on other methods such as the one presented by Sabelli and Bruneau (2006) or the “combined plastic and linear analysis” method described in the commentary to the AISC seismic provisions (AISC 2010). As mentioned earlier, the means of lateral load transfer from the floor and roof diaphragms affects mainly the axial forces in the beams, and other structural behaviors, such as the column internal forces, are largely unaffected. As such, for the purpose of column design, the other methods can be used with the assumption of a type “c” load transfer (Fig. 4.2), regardless of the real lateral load transfer mechanism.

4.9 Design Shear Force and Bending Moment

4.9.1 Governing concepts

The methods for estimating V_{Li} , V_{Mi} , V_{iL} , V_{iR} , and $M_{max\ i}$ are described in Eqs. (4.3) to (4.6). For the top beam, the same method is used while assuming zero values for the infill plate tension field action components in the $i+1^{\text{th}}$ story. When the infill plates above and below an intermediate beam are fully yielded, the capacity design method often estimates the shear force distribution and maximum moment in the beams conservatively. However, in a case where the infill plates above and below an intermediate beam have the same (or nearly the same) thickness, the capacity method may not provide a conservative design shear and bending moment for the beam. This effect can be seen in walls 2SPSW-B01, 2SPSW-B31, and “Test specimen model” in Table 4.5, and 4SPSW-B31 and 4SPSW-B41 in

Table 4.6, where the “FE-to-CD ratios” for V_R of some intermediate beams are significantly greater than 1.0.

The capacity design method assumes that all the infill plates in the wall are yielded at the ultimate lateral load. Although this assumption provides conservative results for the column design forces, it may not be the case for the shear demands in intermediate beams with equal infill plate thicknesses above and below. In such cases, the typical capacity design approach results in a negligible theoretical value for the unbalanced infill plate force. Therefore, the shear component V_I and its corresponding moment are zero, and only the induced constant shear force, V_M , exists in the beam. In reality, however, different scenarios are possible. Assuming the same column profile runs through both stories, the shear resistances of consecutive stories with the same infill plate thickness are equal, while the force demand on the upper story is typically smaller. As such, depending on the column size and design seismic load distribution over the height of the wall, a chance exists that the upper infill plate is only partially yielded while the lower infill plate is fully yielded. Therefore, the upper infill plate applies a non-uniform force to the beam and the vertical component of the unbalanced infill plate force would not be negligible. Consequently, the V_I component of the shear and its corresponding moment are not zero.

The effect described above induces extra shear and moment demands on the intermediate beam in addition to the V_M shear component from capacity design with the assumption of fully-yielded infill plates. The presence of the V_I shear component can be confirmed by the results “ $M_{max, FE}/M_V$ ” in Tables 4.5 and 4.6, where the existence of moment implies that the V_I shear component exists. (While the moment induced by the V_M shear component is not zero when the distributed moment is non-uniform, it would be more than an order of magnitude smaller than that induced by the V_I shear component for typical SPSW proportions when the upper infill plate is only partially yielded.) It is worth noting that the presence

of the V_I component of shear is not because of the slight change in the tension field angle above and below the intermediate beam; the numerical results show that the major influence is related to the progression of infill plate yielding.

4.9.2 Proposed method

As demonstrated in Fig. 4.5, yielding of the infill plates progresses from the compression column toward the tension column and, therefore, when the infill plate is only partially yielded, it applies a non-uniform force to the beam. When the infill plate stress adjacent to the compression column is close to the yield value, the stress adjacent to the tension column could be much smaller.

Figure 4.6(a) shows an approximation of the non-uniform loading condition for shear force evaluation in an intermediate beam with the same infill plate thickness above and below. The infill plate in the lower story is considered fully yielded with the vertical stress component of ω_{y0} uniformly applied to the beam. The infill plate in the upper story is partially yielded with the tension field stress equal to $0.4\omega_{y0}$ adjacent to the tension column and ω_{y0} (yield stress) adjacent to the compression column. The vertical component of the unbalanced infill plate tension field is shown in Fig. 4.6(b). The shear at the faces of the right and left columns are $V_{IR} = -\omega_{y0} L_c/10$ and $V_{IL} = \omega_{y0} L_c/5 = -2V_{IR}$, respectively.

As Fig. 4.1(c) shows, the shear at the face of the compression column is most critical, since V_M is added to the V_I component. As such, the equivalent loading system depicted in Fig. 4.6(c) is selected such that the uniform unbalanced load, $\Delta\omega_{yi,E} = 0.2\omega_{y0}$, produces the same shear at the face of the compression column, $V_{IR,E} = V_{IR} = -\omega_{y0} L_c/10$. The equivalent system produces a beam shear at the tension column and maximum mid-span moment equal to $V_{IL,E} = V_{IR,E} = 0.5V_{IL}$ and $M_{max,E} = 0.65M_{max}$, respectively, where V_{IL} and M_{max} are the corresponding values in Fig. 4.6(b). As shown in Fig. 4.6(d), the equivalent unbalanced force, $0.2\omega_{y0}$, is analogous to using the conventional capacity design method, but taking the upper-story infill plate thickness as being 20% thinner than the thickness in the lower story, with both infill plates considered fully yielded. As a result, when

the infill plates above and below an intermediate beam have similar thicknesses and there is no guarantee that both infill plates will yield, it is recommended that the V_I shear component in the beam and its corresponding moment be calculated assuming the upper infill plate is 20% thinner than the lower one. While it would appear that the calculated moment should be increased by a factor of 1.5 ($= 1/0.65$), Tables 4.5 and 4.6 indicate that beam moments where this equivalent unbalanced force would be applied are particularly low. It is postulated that the unyielded infill plate actually serves to support the beam below and alleviate some of its moment. The actual force interactions are complex and, while this aspect warrants further study, in the interim it is suggested that $M_{max,E}$ can be taken as the design moment. The impact of this assumption is considered below. (Note that the 20% reduction in infill plate thickness is based on an assumption that both infill plates are produced from the same grade of steel. If different grades are used, the difference in thickness that would give rise to a 20% lower capacity design force in the upper plate is recommended.)

The shear and moment values corresponding to the intermediate beams in models 2SPSW-B01, 2SPSW-B31, and “Test specimen model” in Table 4.5, and 4SPSW-B31 and 4SPSW-B41 in Table 4.6 are re-calculated based on the design procedure recommended above. These are denoted as “CD values” in Table 4.7 and the new “FE-to-CD ratios” are also reported in the table. The method provides conservative shear forces at the compression-column (right) side, which is the most critical shear force for the design of the beams, columns, and the beam-to-column connections. It also provides a reasonable estimation of the total shear force at the tension-column end of the beam in that both the numerical model and proposed method predict a small force. The moments in these intermediate beams tend to be quite conservative, but are considered to be suitable for design.

The proposed method of assuming a 20% difference in infill plate thickness (when they are actually the same) for determining the V_I shear force component in

an intermediate beam is necessary only where the lower-story infill plate is fully yielded and the upper-story one is partially yielded. If both infill plates are fully yielded at the mechanism load, this refinement is theoretically not needed. For instance, the infill plate thicknesses in the second and first stories of wall 4SPSW-B21 are 4.32 and 4.80 mm, respectively, which is a 10% difference. However, as shown in Table 4.2, the lateral load distribution over the wall height is such that each story shear is proportional to the corresponding infill plate thickness. As such, under the mechanism load both infill plates are fully yielded. Table 4.6 shows that the proposed capacity design method provides a good, but conservative, estimation of the shear forces in the first-story beam of wall 4SPSW-B21, and there is no need for further refinement in the shear calculation given the uncertainty of seismic loading.

4.10 Experimental Verification

The performance of SPSWs designed for low and moderate seismic regions has been studied experimentally by testing a SPSW system under concurrent vertical and cyclic lateral loading (Moghimi and Driver 2013). The specimen dimensions and its lateral load distribution are defined in Table 4.1. Double-angle shear connections were used for the beam-to-column connections. The specimen survived seven elastic (force-controlled) and 18 inelastic (deformation-controlled) lateral load cycles, and the wall reached its maximum base shear resistance in the push and pull directions, in cycles 19 ($5\delta_y$) and 21 ($6\delta_y$), respectively. The wall was studied using numerical results of the models described in the previous section for monotonically-increasing lateral displacement, and good agreement with the overall behavior of the test specimen was achieved. Moreover, the finite element model shows good agreement between strain results at both ends of each beam and strain readings from the test itself. For instance, the strain gage at the middle of the top beam's web at the north side showed $-350 \mu\epsilon$ at the maximum lateral displacement in cycle 14 ($3\delta_y$), and the strain at the same location from the finite element model was $-307 \mu\epsilon$ at the same lateral displacement under monotonic loading. Similar agreement was confirmed at other points on the

beams. Based on the strain readings from the test specimen at selected points on the beams and the stress distribution in the beam cross-sections from the numerical model, the internal forces in each beam were extracted. Figures 4.7(a) and 4.7(b) show the axial and shear forces, respectively, in each beam at the maximum base shear.

Using the proposed method, the axial force demands in the beams of the test specimen were determined and are shown in Fig. 4.7(a) with dashed red lines. It can be seen that this method, which was developed specifically for SPSWs with simple beam-to-column connections based on capacity design principles and observations from a diverse suite of finite element analyses, gives excellent agreement with the experimental results, shown with solid black lines. For comparison, curves are also plotted assuming the total lateral force at each story is applied in equal measure at the two columns—i.e., load transfer pattern “c” in Fig. 4.2—with dashed blue lines, indicating the importance of selecting a pattern that is consistent with the expected behavior of the actual structure.

The shear forces in the beams were calculated based on the proposed capacity design method and are shown in the Fig. 4.7(b). The infill plates in the two stories had the same thickness and the upper infill plate experienced only partial yielding. As a result, the shear in the top beam determined from the proposed capacity design method is, as expected, quite conservative. The proposed modification to the capacity design method for intermediate beams with infill plates of the same thickness above and below has been implemented, so the infill plate thicknesses were assumed to differ by 20%. The figure shows an excellent match between the results from the proposed method and those from the experiment. For comparison, the curve corresponding to equal infill plate forces above and below the intermediate beam based on their equal thickness is also plotted with dashed blue lines; it is seen that neglecting this provision results in a considerably non-conservative estimate of the maximum shear in the beam.

4.11 Simple Beam-to-Column Connections in SPSWs

The beam-to-column connections in SPSWs are designed to resist the beam reaction forces. Since this study investigates only SPSW systems with simple beam-to-column connections, the frame actions are effectively null, and any moment transfer to the column at a typical shear connection can reasonably be neglected. Therefore, the connections are designed only for axial and shear reactions at the beam ends.

The shear connections are generally designed for strength requirements under capacity loads, considering the factored shear and axial force resistances of the connection elements and the associated fasteners. However, previous research (Moghimi and Driver 2013) has shown that rotational flexibility at the beam-to-column joints has several advantages that contribute to the performance of the SPSW system. Specifically, a flexible connection reduces the demands on the columns as compared to the use of moment-resisting connections and it tends to improve the distribution of plastic strain in the infill plates, potentially increasing the total energy dissipated by the system. As such, the designer should select a simple connection scheme that, besides providing the required strength, also maximizes the flexibility of the joint. For instance, angle, T-section, and end plate connections all incorporate cross-sectional elements perpendicular to the beam web that provide more flexibility for the connection compared to other types of shear connections such as shear tabs.

The rotational capacity of the three simple connection types mentioned above is governed by the deformation of the connection elements, and the fastener deformations play only a minor role. In order to provide flexibility for the connections, the thickness of the connection elements should be kept to the minimum value required to provide sufficient strength. For the same reason, where the connection elements are bolted to the columns a relatively large gage is preferred.

Thornton (1997) has studied the behavior of double angle, T-section, and end plate connections subjected to axial tensile and shear loads. Both strength and flexibility requirements are discussed. For double angle connections, a design table is provided that ensures the acceptable rotational flexibility of the connection. The table defines the minimum gage distance as a function of angle thickness and diameter of the bolts. While thicker connection elements and a smaller gage may increase the axial tensile resistance of the connection, it can simultaneously reduce its flexibility causing an amplified tensile load on the upper fasteners that link the connection elements to the column. This tensile load could lead to fracture of the upper bolts or welds and result in progressive failure of the connection. For all three connection types, Thornton (1997) provides design formulae for the minimum diameter of bolts or minimum fillet weld leg size, as a function of connection parameters, to prevent such failure. Although it is recommended that the principles of these design criteria be followed to promote connection flexibility in SPSWs, in many cases net tension in the connection would not be expected to occur. Moreover, the existence of the infill plates reduces any tensile demand on the connections and its fasteners, so this kind of progressive failure is not anticipated in a properly-designed SPSW system with simple frame connections.

Conventional bolted double-angle shear connections were used at the beam ends in the physical SPSW test by Moghimi and Driver (2013). The connections displayed highly-robust behavior and were virtually undamaged after the SPSW was loaded to failure. The connections at the ends of the top beam remained elastic, while those at the intermediate beam showed only minor plastic deformations. Further discussion about the good performance of these simple connections is provided by Moghimi and Driver (2013).

4.12 Conclusions and Design Recommendations

The internal forces in SPSW beams need to be estimated properly for the design of the beams, beam-to-column connections, and columns. Different sources of

shear force in the beams of SPSW systems with simple frame connections were studied, and their existences were verified by numerical results. The design shear force comes from the vertical component of the unbalanced infill plate yielding force and the induced constant shear from the horizontal component of the infill plate tension field that causes a distributed moment on the beam. The shear component due to the unbalanced infill plate force is in the same direction as the shear due to gravity loads, while the shear induced by the distributed moment is added to the other shear components at the compression-column end of the beam and subtracted from them at the tension-column end. As such, the shear reaction at the face of compression column is the critical shear force for design.

In cases where the infill plate in the story above an intermediate beam is thicker than 80% of the infill plate thickness in the story below, there is a chance that the upper infill plate will not yield fully under the capacity lateral loads. It is recommended that the share of the beam shear due to the unbalanced infill plate force be calculated assuming that the upper-story infill plate has a thickness of 80% of that in the lower story and both plates yield. This 20% difference in the infill plate thickness can be assumed to apply a uniform force to the intermediate beam that compensates for the non-uniform infill plate yielding if partial yielding occurs in the upper infill plate.

This study showed that the lateral load transfer mechanism at the floor and roof diaphragms can have a significant effect on the axial forces in the beams. It impacts the axial force distribution in the beam and may impose a large demand at the beam connections.

The traditional method of representing the infill plates as a series of pure tension strips in the direction of the tension field cannot consider the real stress state and the tension field angle in the vicinity of the boundary frame members. As such, it tends to underestimate the axial force and overestimate the shear and bending moments in the frame elements. However, the levels of capacity design force

obtained by using the methods proposed herein tend to be acceptable and conservative for the beams in most cases.

The proposed methods provide a useful tool for the preliminary design of SPSW systems with simple beam-to-column connections, and it is believed that they provides a good estimation of the final design forces. These provisions consider the lateral load transfer mechanism from the diaphragms and provide a rational estimation of the upper-bound tensile and compressive axial forces that can be applied to the beams of the system and their connections to the columns. The method was also shown to provide reasonable but conservative capacity design shear forces and bending moments for the beams. In principle, the methods presented to estimate axial forces in the beams of SPSWs with simple frame connections are also applicable to SPSWs with moment-resisting beam-to-column connections; however, the column shear forces transferred to the beams as axial force must be estimated specifically for such systems, since the connection rigidity has a considerable influence in that regard. For this application, the effect of the beam plastic moments and their induced shear forces would also need to be added to the design actions described in this study.

Table 4.1 Properties of two-story SPSWs

Wall No.	L_c	h_1	h_2	w_1	w_2	LT	μ_1	μ_2	V_b
	mm	mm	mm	mm	mm	–	–	–	kN
2SPSW-B01	2176	1775	1800	4.8	4.8	c	0.5	0.5	2622
2SPSW-B02	2176	1775	1800	4.8	4.8	a	0.5	0.5	2659
2SPSW-B03	2176	1775	1800	4.8	4.8	b	0.5	0.5	2661
2SPSW-B11	2176	1775	1800	4.8	2.4	c	0.5	0.5	2185
2SPSW-B12	2176	1775	1800	4.8	2.4	a	0.5	0.5	2219
2SPSW-B13	2176	1775	1800	4.8	2.4	b	0.5	0.5	2224
2SPSW-B14	2176	1775	1800	4.8	2.4	d	0.5	0.5	2222
2SPSW-B21	2176	1775	1800	4.8	3.6	c	0.5	0.5	2557
2SPSW-B22	2176	1775	1800	4.8	3.6	a	0.5	0.5	2533
2SPSW-B23	2176	1775	1800	4.8	3.6	b	0.5	0.5	2560
2SPSW-B31	2176	1775	1800	4.8	4.8	c	0.4	0.6	2573
2SPSW-B41	2176	1775	1800	4.8	3.6	c	0.33	0.67	2310
2SPSW-B51	2176	1000	1000	4.8	3.6	c	0.33	0.67	2623
Test specimen	2176	1775	1800	4.66	4.66	d	0.4	0.6	2660
Test specimen model	2176	1775	1800	4.66	4.66	d	0.4	0.6	2746
2SPSW-BS1	2176	1775	1800	4.66	4.66	c	0.4	0.6	2693

Table 4.2 Properties of four-story SPSWs

Wall No.	L_c	h_1	h_2	h_3	h_4	w_1	w_2	w_3	w_4	LT	μ_1	μ_2	μ_3	μ_4	V_b
	mm	mm	mm	mm	mm	mm	mm	mm	mm	–	–	–	–	–	kN
4SPSW-B01	2736	1776	1829	1829	1724	4.8	2.4	1.2	0.6	c	0.25	0.25	0.25	0.25	1938
4SPSW-B02	2736	1776	1829	1829	1724	4.8	2.4	1.2	0.6	a	0.25	0.25	0.25	0.25	1938
4SPSW-B03	2736	1776	1829	1829	1724	4.8	2.4	1.2	0.6	b	0.25	0.25	0.25	0.25	1938
4SPSW-B11	2736	1776	1829	1829	1724	4.8	3.6	2.4	1.2	c	0.25	0.25	0.25	0.25	2716
4SPSW-B12	2736	1776	1829	1829	1724	4.8	3.6	2.4	1.2	a	0.25	0.25	0.25	0.25	2718
4SPSW-B13	2736	1776	1829	1829	1724	4.8	3.6	2.4	1.2	b	0.25	0.25	0.25	0.25	2716
4SPSW-B21	2736	1776	1829	1829	1724	4.8	4.32	3.36	1.92	c	0.10	0.20	0.30	0.40	2670
4SPSW-B31	2736	1776	1829	1829	1724	4.8	4.8	2.4	2.4	c	0.25	0.25	0.25	0.25	3105
4SPSW-B41	2736	1776	1829	1829	1724	4.8	4.8	4.8	4.8	c	0.10	0.20	0.30	0.40	2854
4SPSW-B51	2736	1500	1500	1500	1388	4.8	3.6	2.4	1.2	c	0.25	0.25	0.25	0.25	2786

Table 4.3 Beam axial forces and normalized axial force results for two-story SPSWs

Wall No.	Beam axial forces				Beam axial force change		Column shear at connections			
	1 st Story		2 nd Story		1 st	2 nd	1 st Story		2 nd Story	
	P_L	P_R	P_L	P_R			V_{cL}	V_{cR}	V_{cL}	V_{cR}
	kN	kN	kN	kN	—	—	—	—	—	—
2SPSW-B01	-437	-439	-824	395	—	0.67 ^a	0.15	1.05	-0.34	0.48
2SPSW-B02	-1104	-1040	-1437	-252	—	0.65 ^a	0.16	1.00	-0.29	0.50
2SPSW-B03	125	133	-214	930	—	0.63 ^a	0.11	1.11	-0.40	0.70
2SPSW-B11	-1690	-512	-446	468	1.22	1.00	-1.25	1.22	0.37	0.34
2SPSW-B12	-2143	-1029	-948	-67	1.23	0.96	-1.19	1.21	0.36	0.36
2SPSW-B13	-1033	10	69	998	1.16	1.01	-1.18	1.26	0.32	0.35
2SPSW-B14	-1034	-1031	83	-61	1.20	1.00	-1.19	1.22	0.38	0.34
2SPSW-B21	-1092	-580	-1009	445	1.14	1.06	-0.48	1.25	-0.93	0.50
2SPSW-B22	-1777	-1269	-1608	-153	1.13	1.06	-0.56	1.31	-0.95	0.50
2SPSW-B23	-412	43	-397	1047	1.01	1.05	-0.41	1.24	-0.96	0.54
2SPSW-B31	-769	-799	-1395	475	—	1.03	-0.32	1.14	-1.09	0.73
2SPSW-B41	-1385	-870	-840	606	1.15	1.06	-1.02	1.27	-0.20	0.43
2SPSW-B51	-751	-222	-661	777	1.15	1.04	-0.59	1.20	1.03	0.58
Test specimen model	-34	-984	-449	-285	—	0.96	-0.03	0.96	-0.86	0.63
2SPSW-BS1	-606	-547	-1199	487	—	0.96	-0.08	1.04	-0.81	0.63

^a Second story infill plate partially yielded

Table 4.4 Normalized axial force results for four-story SPSWs

Wall No.	Beam axial force change				Column shear at connections							
	1 st	2 nd	3 rd	4 th	1 st Story		2 nd Story		3 rd Story		4 th Story	
					V_{cL}	V_{cR}	V_{cL}	V_{cR}	V_{cL}	V_{cR}	V_{cL}	V_{cR}
4SPSW-B01	1.19	1.05	1.01	1.00	-1.04	0.06 ^a	-1.17	0.93	-0.03	0.91	0.87	2.00
4SPSW-B02	1.19	1.04	1.04	1.03	-1.03	0.06 ^a	-1.14	0.91	-0.01	0.85	0.77	2.05
4SPSW-B03	1.20	1.04	1.01	1.04	-1.05	0.07 ^a	-1.12	0.89	-0.11	1.01	0.98	1.76
4SPSW-B11	1.13	1.07	1.05	1.02	-0.91	0.96	-0.66	0.75	-0.64	0.82	-0.23	0.89
4SPSW-B12	1.12	1.07	1.10	1.01	-0.90	0.95	-0.60	0.70	-0.83	0.94	-0.03	0.74
4SPSW-B13	1.15	1.07	1.03	1.02	-0.95	0.98	-0.55	0.65	-0.78	0.98	-0.07	0.75
4SPSW-B21	0.97	1.07	1.12	1.02	-0.77	0.83	-0.75	0.80	-0.83	0.87	-0.15	0.70
4SPSW-B31	—	1.05	—	0.71 ^b	0.05	0.75	-1.13	0.63	0.37	1.14	-0.38	0.13 ^b
4SPSW-B41 ^c	—	—	—	0.41	0.00	0.38	-0.15	0.68	0.14	0.30	0.09	0.20
4SPSW-B51	1.14	1.05	1.03	1.00	-0.92	0.98	0.40	0.56	0.63	0.94	0.24	0.95

^a First story infill plate partially yielded

^b Fourth story infill plate partially yielded

^c Infill plates in all four stories are partially yielded

Table 4.5 Shear force and bending moment results for two-story SPSWs

Wall No.	Story	FE Results			FE results analysis				FE-to-CD ratios				
		Shear		$M_{max,FE}$	V_I	V_M	M_V	$M_{max,FE}/M_V$	Shear		M_{max}	V_I	V_M
		V_L	V_R						V_L	V_R			
		kN	kN	kN.m	kN	kN	kN.m	—	—	—	—	—	—
2SPSW-B01	2	246	-694	317	470	-224	318	1.00	0.28	0.57	0.45	0.45	1.33
	1	7	-296	76	152	-144	103	0.74	-0.04	1.58 ^a	—	—	0.77
2SPSW-B11	2	395	-490	320	443	-48	299	1.07	0.92	0.82	0.92	0.86	0.56
	1	126	-399	169	262	-136	177	0.95	0.34	0.61	0.49	0.51	0.97
2SPSW-B21	2	484	-707	426	595	-111	402	1.06	0.74	0.78	0.81	0.76	0.88
	1	154	-422	177	288	-134	195	0.91	1.61	1.00	1.01	1.11	0.82
2SPSW-B31	2	524	-851	473	688	-163	465	1.02	0.60	0.70	0.67	0.66	0.97
	1	32	-341	103	187	-155	126	0.82	-0.17	1.83 ^a	—	—	0.83
2SPSW-B41	2	495	-720	432	607	-113	411	1.05	0.76	0.80	0.82	0.78	0.89
	1	122	-398	160	260	-138	175	0.91	1.27	0.94	0.91	1.00	0.84
2SPSW-B51	2	443	-677	376	560	-117	379	0.99	0.68	0.75	0.71	0.72	0.92
	1	87	-384	142	235	-148	159	0.89	0.91	0.91	0.81	0.91	0.91
Test specimen model	2	381	-814	434	598	-216	404	1.07	0.45	0.69	0.63	0.59	1.33
	1	-26	-355	83	165	-191	111	0.75	0.14	1.97 ^a	—	—	1.06

^a Infill plates in the two stories have the same thickness

Table 4.6 Shear force and bending moment results for four-story SPSWs

Wall No.	Story	FE Results			FE results analysis				FE-to-CD ratios				
		Shear		$M_{max,FE}$	V_I	V_M	M_V	$M_{max,FE}/M_V$	Shear		M_{max}	V_I	V_M
		V_L	V_R						V_L	V_R			
		kN	kN	kN.m	kN	kN	kN.m	—	—	—	—	—	—
4SPSW-B01	4	79	-125	95	102	-23	86	1.11	0.62	0.70	0.74	0.67	0.92
	3	73	-151	103	112	-39	94	1.10	0.66	0.77	0.80	0.73	0.91
	2	146	-286	200	216	-70	181	1.10	0.66	0.73	0.78	0.71	0.82
	1	-91	-223	77	66	-157	56	1.39	-0.21	0.29	0.15	0.11	0.92
4SPSW-B11	4	178	-271	206	224	-47	189	1.09	0.68	0.75	0.79	0.72	0.94
	3	144	-283	200	214	-70	180	1.11	0.64	0.72	0.76	0.69	0.82
	2	82	-344	200	213	-131	179	1.12	0.49	0.76	0.76	0.69	0.92
	1	17	-384	176	200	-184	169	1.04	0.15	0.75	0.67	0.64	0.92
4SPSW-B21	4	301	-445	346	373	-72	314	1.10	0.71	0.76	0.82	0.74	0.91
	3	99	-326	203	213	-113	179	1.14	0.39	0.65	0.64	0.56	0.91
	2	-13	-342	157	164	-178	138	1.14	-0.19	0.79	0.74	0.65	0.98
	1	-56	-328	107	136	-192	114	0.93	0.62	0.96	1.01	1.08	0.89
4SPSW-B31	4	159	-398	242	278	-120	234	1.03	0.30	0.55	0.46	0.44	1.21
	3	22	-192	91	107	-85	90	1.01	-0.20	1.69 ^a	—	—	0.75
	2	93	-371	196	232	-139	195	1.01	0.20	0.46	0.37	0.37	0.82
	1	-73	-342	106	134	-208	113	0.94	0.32	1.50 ^a	—	—	0.91
4SPSW-B41	4	18	-154	28	86	-68	72	0.39	0.02	0.10	0.03	0.07	0.34
	3	-19	-221	36	101	-120	85	0.42	0.08	0.98 ^a	—	—	0.53
	2	-59	-286	55	113	-173	95	0.58	0.26	1.26 ^a	—	—	0.76
	1	-159	-289	45	65	-224	55	0.82	0.70	1.27 ^a	—	—	0.99
4SPSW-B51	4	175	-267	207	221	-46	186	1.11	0.67	0.74	0.79	0.71	0.93
	3	153	-296	204	225	-72	189	1.08	0.68	0.75	0.78	0.72	0.84
	2	78	-339	194	208	-131	175	1.11	0.46	0.75	0.74	0.67	0.92
	1	13	-372	163	192	-180	162	1.01	0.11	0.73	0.62	0.62	0.90

^a Infill plates above and below the intermediate beam have the same thickness

Table 4.7 Shear force and bending moment results for intermediate beams
assuming upper infill plate is 20% thinner than lower

Wall No.	Story	CD values				FE-to-CD ratio			
		Shear		M_{max} kN.m	V_I kN	Shear		M_{max} —	V_I
		V_L kN	V_R kN			V_L —	V_R —		
2SPSW-B01	1	23	-396	142	209	~1	0.75	0.54	0.72
2SPSW-B31	1	23	-396	142	209	~1	0.86	0.73	0.89
Test specimen model	1	22	-384	137	203	~1	0.93	0.61	0.81
4SPSW-B31	3	12	-239	106	126	~1	0.80	0.86	0.85
	1	24	-478	211	251	~1	0.71	0.50	0.53
4SPSW-B41	3	28	-482	215	255	~1	0.46	0.17	0.40
	2	28	-482	215	255	~1	0.59	0.26	0.44
	1	28	-482	215	255	~1	0.60	0.21	0.26

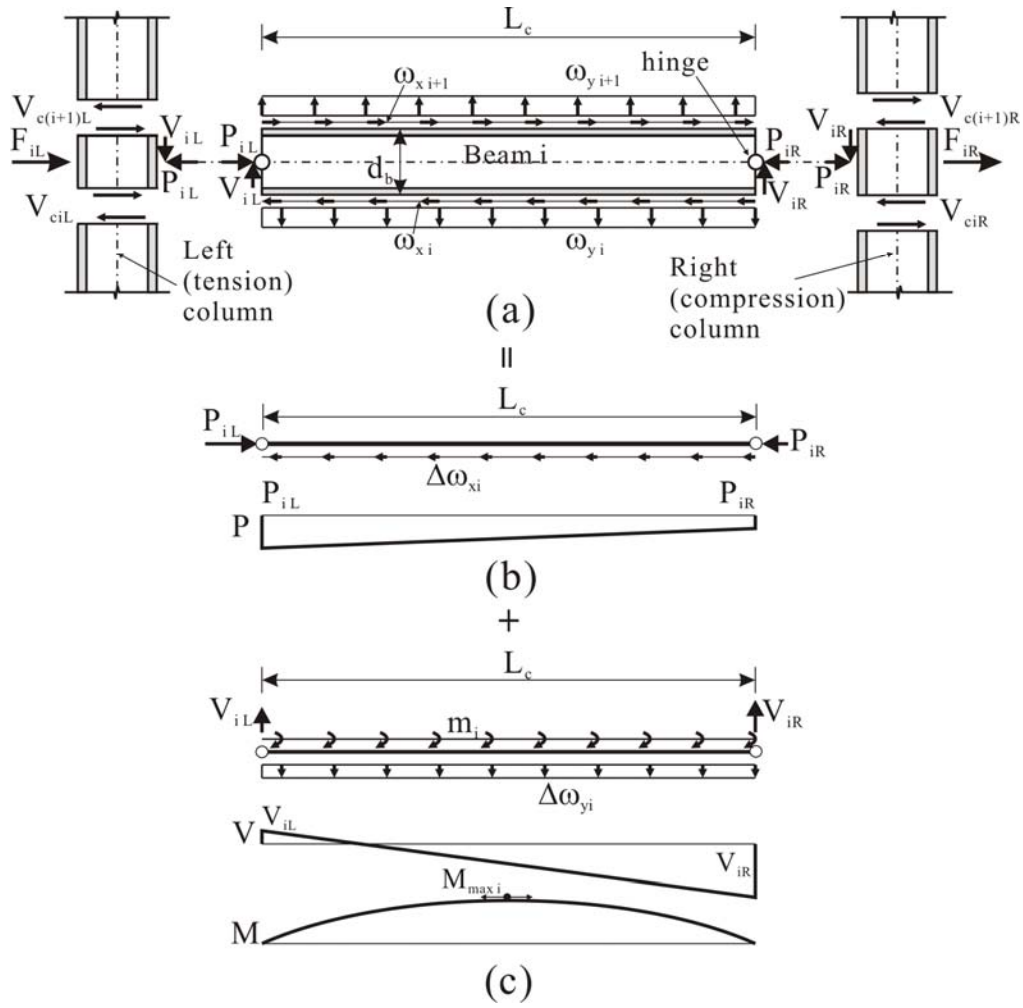


Fig. 4.1 Forces on the i^{th} intermediate beam of SPSW system: (a) Applied forces, (b) Net axial forces applied to beam centerline, (c) Net shear forces and moments applied to beam centerline

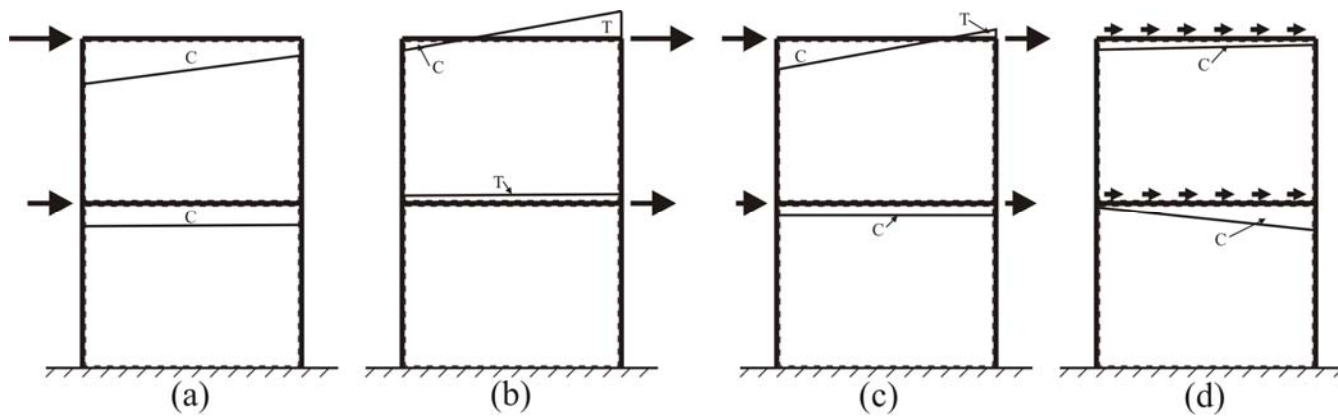


Fig. 4.2 Effect on beam axial forces of diaphragm load transfer pattern from floor and roof diaphragms

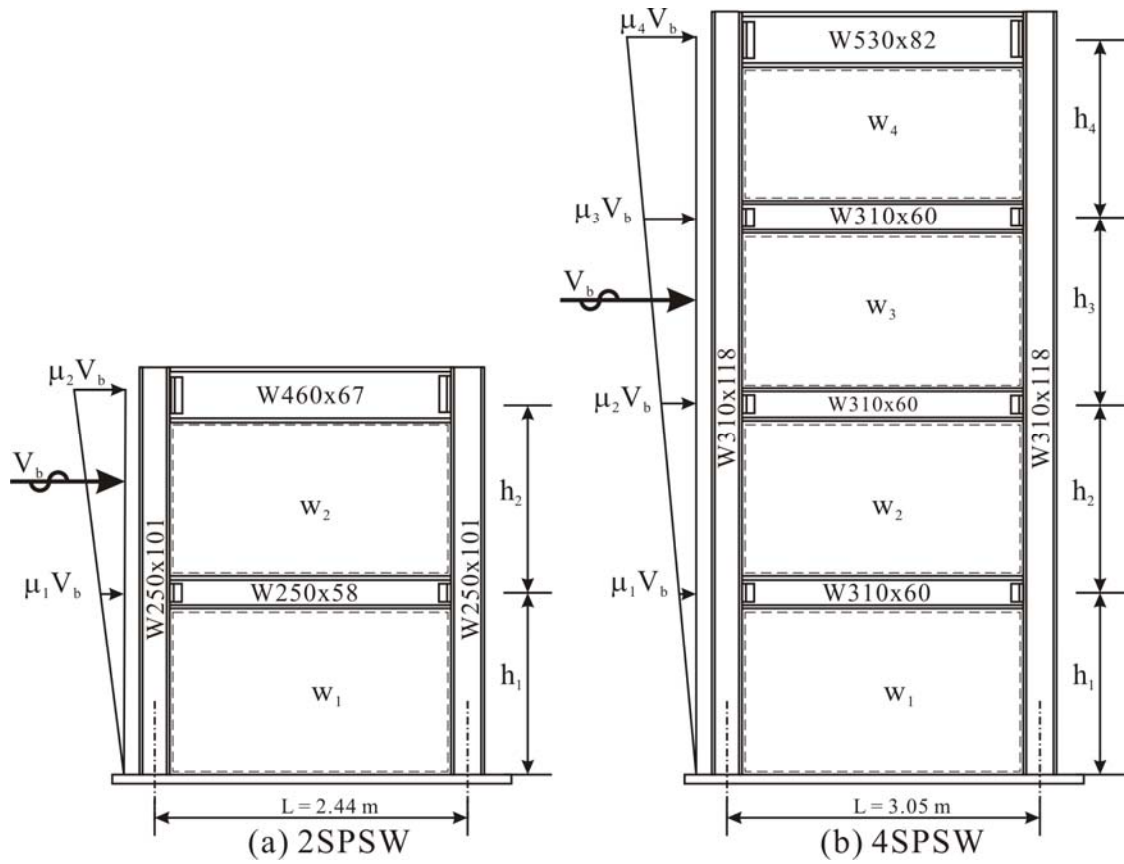


Fig. 4.3 Numerical models: (a) Two-story wall (2SPSW), (b) Four-story wall (4SPSW)

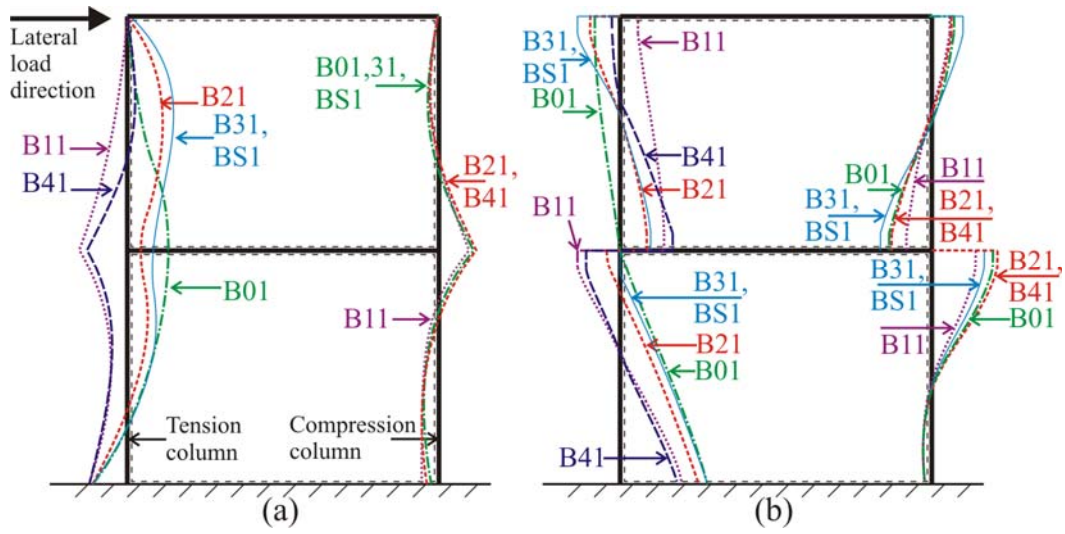


Fig. 4.4 Internal column force distributions in two-story SPSWs: (a) Moment, (b) Shear

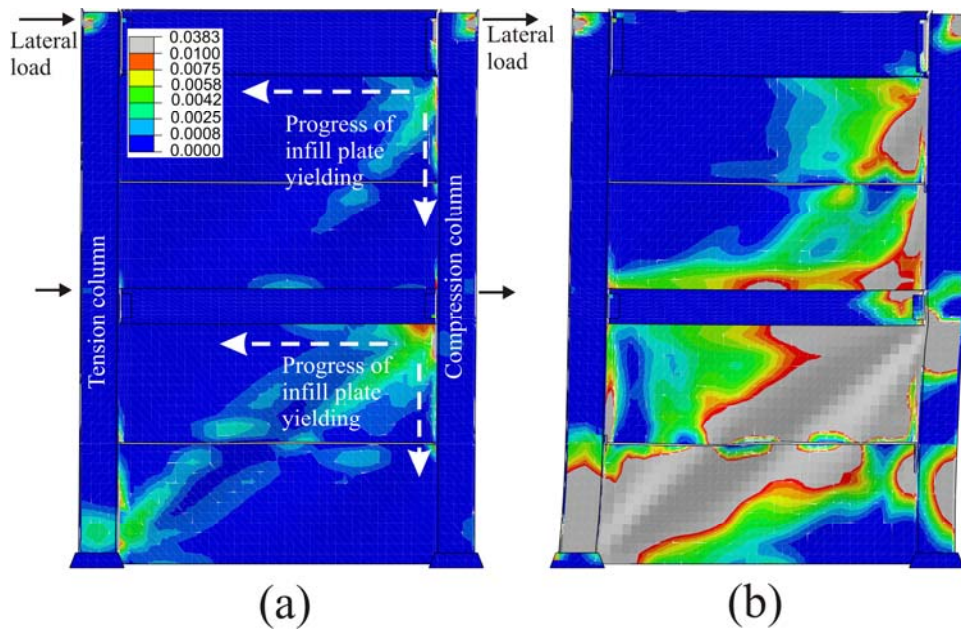


Fig. 4.5 Effective plastic strain distribution in a typical SPSW system: (a) at 0.6% drift ratio, (b) at 2.5% drift ratio

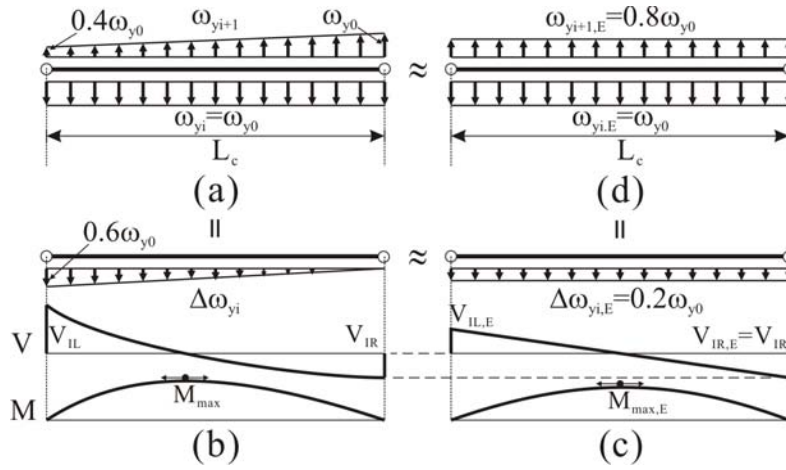


Fig. 4.6 Intermediate beam in a SPSW: (a) Full and partial yielding at lower- and upper-story infill plates, (b) Unbalanced infill plate force and shear and moment diagrams, (c) Equivalent unbalanced infill plate force and shear and moment diagrams, (d) Equivalent system with fully yielded infill plates at both stories

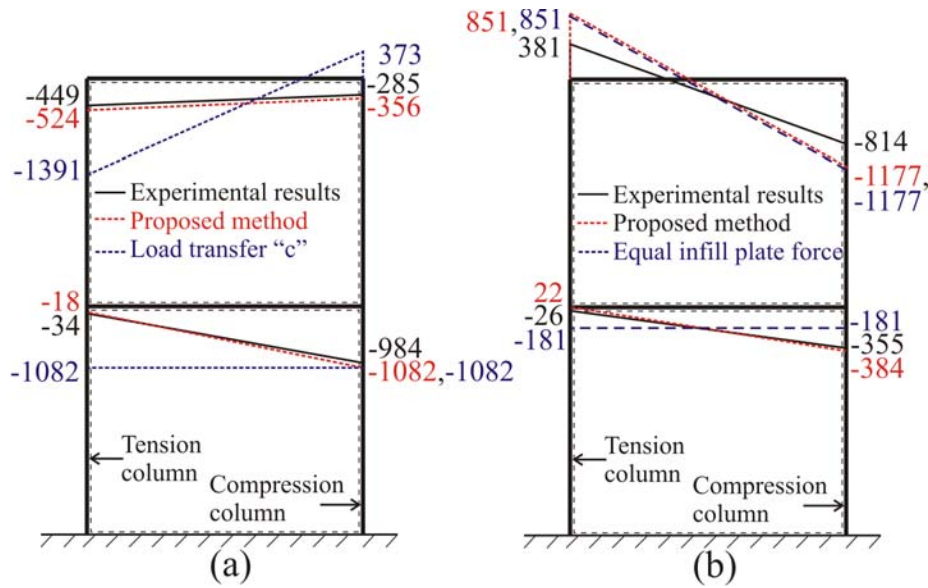


Fig. 4.7 Beam internal force demands (kN): (a) Axial force, (b) Shear force

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5. PERFORMANCE-BASED CAPACITY DESIGN OF STEEL PLATE SHEAR WALLS. I: DEVELOPMENT PRINCIPLES⁴

5.1 Introduction

Steel plate shear walls (SPSWs) are a lateral load resisting system that has been developed to an advanced stage primarily based on research that focuses on its expected performance under severe earthquake loading. Although the system is undeniably well-suited for high seismic regions, its potential applications in zones of low and moderate seismicity, encompassing the majority of the North American continent, have largely been neglected. Research on SPSWs for high-seismic applications is focused on maximizing the system ductility and overall cyclic robustness by incorporating high-performance detailing. Therefore, the relatively high cost of the system is a direct outcome. However, by focusing instead on lower-cost details and construction economy, SPSWs suitable for low-seismic applications can be developed and their performance verified under the lower demands associated with these regions using a combination of physical tests and numerical simulations.

Comparing SPSW systems with the treatment of moment resisting frames (MRFs) in current design standards gives a perspective on where the former system stands in the evolution of its design provisions in North America. The Canadian Standards Association steel design standard, S16-09 *Design of Steel Structures* (CSA 2009), hereafter referred to as S16, has adopted three performance levels for MRFs: Type D (ductile), Type MD (moderately ductile), and Type LD (limited-ductility). In the case of SPSWs, only two performance levels are recognized: Type D and Type LD. Each performance level is associated with a force modification factor used to reduce the seismic load effects to account for both the capability of the structure to dissipate seismic energy through stable inelastic

⁴ A version of this chapter has been submitted for publication in the *Journal of Structural Engineering*, ASCE.

response and the dependable overstrength. As such, this factor is defined as the product of two separate coefficients, R_d (ductility-related force modification factor) and R_o (overstrength-related force modification factor). Although not explicitly used in S16, for convenience in this paper the product of these two factors is denoted simply as R , rather than $R_d R_o$. ANSI/AISC 341-10 *Seismic Provisions for Structural Steel Buildings* (AISC 2010), hereafter referred to as AISC 341, also provides for three different MRF performance levels: Special, Intermediate, and Ordinary. Conversely, only one performance level was adopted for SPSWs: Special. ASCE/SEI 7-10 *Minimum Design Loads for Buildings and Other Structures* (ASCE 2010), hereafter referred to as ASCE 7, defines the associated response modification factors, R , which also account for both inelastic system response and overstrength. Table 5.1 summarizes the R -factors specified currently by S16 and ASCE 7 for both the MRF and SPSW systems.

The current S16 provisions (CSA 2009) for achieving $R_d = 2.0$ (Type LD) state that the requirements for $R_d = 5.0$ (Type D) must be met, with certain relaxations; i.e., these provisions were not developed independently for the $R_d = 2.0$ case. Therefore, new requirements—developed from the ground up to optimize designs for low-seismic regions—are needed for limited-ductility SPSWs that comply with the intent of the capacity design principles stated in S16 and AISC 341. With ductile and limited-ductility design provisions both available, a moderately ductile option could then be rationalized to achieve performance levels between the two extremes. Under the resulting three-tier system (analogous to the current three-tier system for MRFs), several technical and economic benefits will accrue. Most importantly, it will give designers additional options in low and moderate seismic regions (versus braced frames, MRFs, concrete shear walls, etc.) and, in the same way that for economic reasons highly-ductile MRFs are unlikely to be selected in low seismic zones, the lower ductility demands in these regions would make limited-ductility SPSWs economically superior to those designed to be highly ductile, while still exhibiting the required performance. Lower-ductility options also increase opportunities for utilizing SPSWs for seismic upgrades where parts

of the existing structure do not themselves possess high ductility.

Because of the efficiency of the SPSW infill plate in carrying story shear, plate thickness requirements are generally very low. In fact, handling and welding considerations are likely to govern the selection of infill plate thickness in the vast majority of cases for ductile walls. As such, a thicker-than-required infill plate may be used, creating additional demand on the boundary frame members under capacity design requirements. The use of lower ductility walls, with a concomitant lower value of the force modification factor, R , leads to higher story shear forces that utilize the thickness of the infill plate more fully, while at the same time reducing some of the expensive detailing requirements of the ductile category. In other words, despite the higher design forces applied to limited-ductility or moderately ductile systems, no increase in infill plate thicknesses or member sizes may be needed, and savings will be possible due to the significantly reduced ductility demand. Similar efficiencies can arise for taller SPSWs whose design is governed by deflection limits.

5.2 Scope and Objectives

To achieve compliance with current seismic design provisions, such as those in S16 and AISC 341, the fabrication of ductile SPSWs tends to be expensive due to the necessity of high-ductility and cyclically-robust connection detailing, combined with the requirements to meet capacity design objectives. This makes current design methods generally uneconomical for low and moderate seismic regions. This research aims to develop reliable and economical design methods for limited-ductility and moderately ductile walls that are suitable principally for these regions, and to set them within the context of a three-tier performance-based capacity design methodology that is capable of addressing SPSWs with a range of performance objectives. This paper proposes a target yield mechanism concept for limited-ductility walls that departs from the usual capacity design treatment, and introduces two new classifications of SPSWs designed for achieving moderate ductility. An additional objective of the paper is to meld the proposed

performance-based design methods and simplified analysis techniques into an efficient, but sufficiently accurate, design process. Finally, the paper characterizes and discusses implications of the proposed design and modeling approaches on the accuracy of the resulting boundary member design forces.

5.3 Literature Review

5.3.1 Capacity design of SPSWs

Standard S16 (CSA 2009) stipulates that capacity design principles must be implemented in the seismic design of any structure with $R > 1.3$ (utilizing $R \leq 1.3$ is thereby interpreted as being essentially elastic design). It is assumed that the Type D SPSW system can develop significant inelastic deformation in its protected zones—infill plates, ends of beams, and column bases—where most attachments or discontinuities that can cause stress concentrations are prohibited. To ensure the development of reasonably uniform tension fields in the infill plates, minimum flexural stiffness requirements are provided for the columns, as well as the top and base (if present) beams. Type D SPSWs are assumed to constitute a dual system and the beam-to-column connections and the column joint panel zones must comply with the requirements for Type LD and Type D MRFs, respectively. The AISC 341 (AISC 2010) provisions are also based upon capacity design philosophy and they stipulate requirements for special plate shear walls that are similar to those for Type D walls in S16. In general, the beam-to-column connections must comply with the ordinary moment frame requirements, and both the panel zones next to the top and base beams and the boundary member cross-sectional compactness must satisfy the special moment frame requirements. For Type D/Special SPSW column design, S16 and AISC 341 both require that strong column–weak beam behavior be ensured in the boundary frame.

Design requirements for SPSWs with lower ductility are included in S16, but not in AISC 341. S16 introduced the Type LD provisions by adopting the capacity design requirements of Type D walls as a starting point, with a few relaxations as

deemed appropriate by the committee. These relaxations include a reduction in the beam compactness requirements, permission to use other than rigid beam-to-column connections, and elimination of the dual-system requirement.

For Type D/Special SPSW column design, S16 and AISC 341 both require that strong column–weak beam behavior be ensured in the boundary frame. To satisfy this requirement, S16 explicitly specifies that the internal column forces induced by beam plastic hinging at both ends—increased for the effects of material overstrength and strain hardening—must be added to those induced by the distributed forces from the fully-yielded infill plate—also accounting for material overstrength—and the gravity forces. AISC 341 instead stipulates that the column–beam moment ratio must comply with the strong column–weak beam provision for special moment frames.

Berman and Bruneau (2003) evaluated plastic collapse loads for SPSWs using the concepts of the strip model and plastic analysis. Two types of mechanism were considered for multi-story walls: soft story and uniform yielding of all infill plates and beam ends simultaneously. Based on the more desirable latter mechanism, Berman and Bruneau (2008) presented a detailed procedure for capacity design of columns in ductile SPSWs. The collapse lateral load is calculated based on a uniform mechanism. A linear-elastic column model resting on linear springs (representing the beams) was proposed for evaluating the beam axial compressive loads due to the inward pull on the columns of the yielding infill plates. Each column is then designed for the actions caused by applying the lateral mechanism loads and internal forces to the column free body diagram.

Qu and Bruneau (2010) discussed the capacity design of intermediate beams of SPSW systems with reduced beam sections and moment connections. Various sources of axial force in the beams were identified, and for the portion arising from the column reaction (i.e., the reaction to the inward forces on the columns caused by the infill plate tension field) the method presented by Berman and

Bruneau (2008) was used. Sources of shear force demand in the intermediate beams and moment demand at the faces of the columns were also discussed. A capacity design procedure for intermediate beams was presented.

5.3.2 Experimental studies on SPSWs

Due to the extensive body of literature available on SPSWs, only research used directly in Chapters 5 and 6 is summarized. Previous research on SPSW systems has been reviewed comprehensively by Driver et al. (1997) and Sabelli and Bruneau (2006). Also, tests on SPSWs with simple beam-to-column connections are reviewed by Moghimi and Driver (2013a).

Driver et al. (1998) tested a four-story SPSW with rigid beam-to-column connections under concurrent vertical column loads of 720 kN and cyclic lateral loads distributed equally to the four floor levels. The test specimen elevation and its normalized hysteresis curves are shown in Figs. 6.7(b) and 6.8, respectively. The system was tested under increasing cyclic lateral displacement, and a total of 30 cycles—with 20 in the inelastic range—were applied. The first story yield displacement occurred during cycle 11 at the lateral deflection of $\delta_y = 8.5$ mm (corresponding to a drift ratio of 0.44%). The system achieved its maximum base shear of 3080 kN in cycle 22, corresponding to a first-story displacement ductility ratio of 5 (drift ratio of 2.21%), and its lateral resistance then declined gradually to about 85% of the maximum value at a displacement ductility ratio of 9 (drift ratio of 3.97%) in cycle 30. The specimen showed a high initial stiffness, large energy dissipation capacity, and excellent ductility and redundancy.

Qu et al. (2008) performed a two-phase experiment on a two-story SPSW with composite floors and beams with reduced beam sections and rigid connections to the columns. Each story was 4.0 m high and the columns were spaced at 4.0 m center-to-center. The infill plate thicknesses for the first and second stories were 3.0 and 2.0 mm, respectively, in phase 1 of the experiment, and 3.2 and 2.3 mm in phase 2. In the first phase, the specimen was subjected to three pseudo-dynamic

load histories. The specimen survived all the simulated ground motions with only moderate damage. In the second phase, the infill plates were replaced and the specimen first was subjected to another pseudo-dynamic loading sequence equivalent to the largest scaling from phase 1. The repaired specimen had a smaller initial stiffness, but survived the test and dissipated a significant amount of energy. The boundary frame was generally in good condition after the test, with plastic deformation evident at the column bases and reduced beam sections at both levels. The specimen suffered a small fracture along the bottom of a shear tab in the intermediate beam at the first story drift ratio of 2.6%, as well as small fractures at the infill plate corners. After replacing the damaged shear tab, the specimen was subjected to quasi-static cyclic loading to failure by imposing the first mode shape on the lateral deflections at the two stories. The specimen reached its maximum base shear in cycle 5 at the first story drift ratio of 3.0%. The shear tabs at both ends of the intermediate beam fractured completely in cycle 7, at the first story drift ratio of 2.8%, and in cycle 9, at the first story drift ratio of 3.3%, the bottom flange of the intermediate beam fractured at the face of a column and the connection of the infill plate to the adjacent boundary frame unzipped throughout the remainder of the test.

Moghimi and Driver (2013a) tested a two-story SPSW with simple beam-to-column connections and a modular construction system under cyclic displacement concurrent with gravity column loads of 600 kN. The test specimen elevation and its normalized hysteresis curves are shown in Figs. 6.7(a) and 6.8, respectively. Double angle beam-to-column shear connections were used to provide rotational flexibility at the joint. As a key component of the modular concept, the infill plates in both stories were spliced horizontally at mid-height with a bolted single-sided lap plate of the same thickness as the infill plates. A lateral load ratio for the two levels was selected for the cyclic test to represent the first mode in two central stories of a multi-story building. In cycle 8, the specimen reached the first-story yield displacement of $\delta_y = 12$ mm (corresponding to a drift ratio of 0.65%). The specimen reached its maximum base shear of 2625 kN in cycle 19,

corresponding to a displacement ductility ratio of 5 (drift ratio of 3.25%). The specimen demonstrated very good performance and energy dissipation capacity under 25 cycles of loading. The shear connections showed only nominal plastic deformation at the end of the test and their flexibility reduced the moment demand on the columns compared to rigid connections.

5.4 Yield Mechanisms

Lateral loads on conventional SPSWs are resisted by a combination of tension field action in the infill plates and frame action of the boundary members. The infill plates are the primary elements for dissipating seismic energy; however, the surrounding frame also undergoes inelastic behavior and must support the gravity loads throughout the seismic event for any performance level. As such, the yield mechanism of a SPSW develops in part by tension yielding of the infill plates, but it is mostly the inelastic behavior of the boundary frame that determines the seismic performance level of the system. Tests on different multi-story SPSW systems have shown that distinctly different yield mechanisms are conceivable for ductile and limited-ductility SPSWs, while still providing satisfactory performance for the design objectives.

The performance level of a seismic system is often defined by the deformation limits of the deformation-controlled actions of its components, while the strength capacity of force-controlled actions are treated essentially the same for all performance levels. Since capacity design is in effect a force method, and therefore does not directly provide the deformation demands for the components of the system, the seismic performance level is instead defined in terms of the system ductility and system redundancy. The ductility of a SPSW design is influenced by the yield mechanism that develops, and the redundancy level is distinguished mainly by the beam-to-column connection type. Using this approach, the “ductile” performance level is assigned to SPSWs that possess both high system ductility and the redundancy enabled through rigidly-connected frame joints. The “limited-ductility” performance level is assigned to SPSWs that

are permitted to develop a less ductile yield mechanism and exhibit reduced redundancy through the use of ductile shear connections at the frame joints. The “moderately ductile” performance level is assigned to hybrids of these two extremes, where the SPSW is designed for either the higher-ductility yield mechanism and the lower redundancy, or the lower-ductility yield mechanism and the higher redundancy.

5.4.1 Ductile SPSW system

In ductile walls, the columns are designed according to capacity design procedures to remain elastic above the base under any potential seismic loading, and the resulting yield mechanism of the system is ideally similar to that shown in Fig. 5.1(a) for a typical four-story wall. The figure shows a uniform yielding mechanism wherein all infill plates are fully yielded, along with the formation of plastic hinges at the column bases and all beam ends. With minor modifications to the assumptions of Berman and Bruneau (2003), the system yield mechanism load of a SPSW can be calculated by equating external to internal virtual work, as follows:

$$\sum_{i=1}^n F_i H_i = \sum_{i=1}^n 0.5(w_i - w_{i+1})R_y F_{yw} L_c H_i \sin(2\alpha_i) + \beta \sum_{i=1}^n 1.1R_y (M_{pbLi}^* + M_{pbRi}^*) + 1.1R_y (M_{pcL}^* + M_{pcR}^*) \quad (5.1)$$

where the subscript i represents the i^{th} story, subscripts L and R indicate left and right, respectively, n is the total number of stories, F_i is the system yield mechanism force at each level, w and F_{yw} are the infill plate thickness and nominal yield stress, respectively, H_i is the height of the story from the base, L_c is the clear distance between columns, and α_i is the tension field angle from vertical. M_{pb}^* and M_{pc}^* are the nominal plastic moment capacities of the beam ends and column bases, respectively, where the superscript $*$ indicates a capacity reduction to account for the effect of axial force in the member. R_y is the ratio of expected-to-nominal yield stress in the associated element, and the coefficient 1.1 represents the effect of material strain hardening at the point when the complete

mechanism forms. For any given lateral load distribution over the height of the wall, Eq. (5.1) renders the system yield mechanism force at each story, F_i . The coefficient β , which is a positive variable less than or equal to unity, accounts for the fact that it is unlikely that all beam ends in the system develop plastic hinges under the design event, as discussed in Chapter 6. Taking β equal to unity results in upper-bound values for the yield mechanism forces.

5.4.2 Limited-ductility SPSW system

Limited-ductility walls are expected to provide lower levels of inelastic deformation capacity compared with ductile walls. This reduced ductility, along with reduced redundancy due to the use of simple connections at the frame joints, results in larger seismic design forces because of the lower associated R-factor (primarily due to the component R_d). Therefore, a thicker infill plate—which is the main source of lateral force capacity of the SPSW system—is required compared to that in a ductile wall in a similar seismic region. The relatively thick infill plates in limited-ductility walls with simple frame connections tend to disrupt the yield mechanism of the system from the ideal pattern of Fig. 5.1(a) toward that of Fig. 5.1(b), if minor yielding is permitted in specific regions of the compression column and it is assumed that only partial tensile yielding develops in some of the infill plates. This latter phenomenon is particularly relevant when the same infill plate thickness is used in multiple stories, as would commonly be the case for economy. Nevertheless, in principle this modified pattern should be acceptable due to the greatly reduced ductility demands compared with those of a ductile wall design. While the beam-to-column joints are flexible in limited-ductility SPSWs, hinges are not explicitly shown in Fig. 5.1(b) to reflect the fact that rotations at these locations tend to be small.

Two scenarios contribute to the modification of the yield pattern for the limited-ductility case toward that represented by Fig. 5.1(b). First, the relatively thick infill plates impose internal force demands on the surrounding frame as the yield mechanism condition is approached and they also tend to restrain the free rotation

of the simple frame connections. These behaviors are liable to change the yield pattern of the system so that limited yielding takes place locally where the maximum shear demand occurs in the frame, as depicted in Fig. 5.2(a). The existence of a large shear force simultaneously with the axial force in the compression column (which is larger than the axial force in the tension column in the same story) and in the beam end adjacent to this column, can cause partial web yielding in the corresponding frame member, as shown schematically in Fig. 5.2(b). Depending on the relative sizes of the frame members and the infill plate thickness, either column or beam partial web yielding could occur. As such, a yield pattern similar to that shown in Fig. 5.1(b) forms, with only small rotations occurring at the beam-to-column connections themselves.

The second, and perhaps more significant, scenario that contributes to the formation of the modified yield pattern of Fig. 5.1(b) occurs when an infill plate at a given story experiences only partial yielding, while the infill plate in the story below is fully yielded. This would be common in multi-story walls when either the column flexural stiffness is insufficient to ensure a uniform tension field in the upper infill plate or the infill plate thickness distribution over the wall height is such that the story shear distribution cannot yield the plates at every story. To illustrate this concept, Fig. 5.3(a) shows a typical two-story wall with the same infill plate thickness in each story. While the base shear is large enough to yield the infill plate relatively uniformly in the first story, the second story shear is only sufficient to yield parts of the associated infill plate. Therefore, the plastic deformation of the second story is much smaller than that of the first story, as shown in Fig. 5.3(b), and the second story infill plate in effect restrains the lateral deflection of the compression column. The dashed column outline in Fig. 5.3(b) represents the position the compression column would take if the second story infill plate were fully yielded. To account for the restraint afforded by the partially-yielded infill plate, a lateral deflection reduction at the top of the second story, Δ^* , is applied. This action bends the compression column against the first-story beam, causing partial yielding in the column immediately below the frame

joint (both the shear and axial forces are higher below the joint than above), and also in the beam web adjacent to the connection where the reactive force opposing this action is concentrated. This scenario contributes to the localized yield regions shown in Fig. 5.2(b) and in the compression column it consists primarily of web yielding and minor internal-flange flexural yielding. The formation of this type of partial yielding pattern in the column was observed during a physical test on a two-story SPSW specimen (Moghimi and Driver 2013a) designed specifically for limited-ductility applications. The test showed that the partial yielding in the frame members poses no threat to the reliability of the SPSW system, even at drift ratios up to 5.2% (corresponding to a displacement ductility ratio of 8). The wall achieved its maximum base shear at a displacement ductility ratio of 5, which is well beyond what is normally required of limited-ductility—and even moderately ductile—seismic systems.

While a yield mechanism similar to the one depicted in Fig. 5.1(a) could be assumed by again enforcing elastic column behavior during design, the significantly reduced ductility demands placed on limited-ductility SPSWs permits the spirit of capacity design to be upheld, while accounting explicitly for the somewhat lower mean infill plate stresses consistent with the yield mechanism shown in Fig. 5.1(b). If this philosophy is taken, the yield mechanism lateral forces for a limited-ductility SPSW with simple connections are reasonable estimated as follows:

$$\sum_{i=1}^n F_i H_i = \sum_{i=1}^n 0.5(w_i - w_{i+1}) F_{ywt} L_c H_i \sin(2\alpha_i) + 1.1 R_y (M_{pcL}^* + M_{pcR}^*) \quad (5.2)$$

It is noted that in Eq. (5.2), the nominal infill plate yield stress is used, which is contrary to conventional capacity design philosophy. The implications of this are discussed later in the paper.

5.4.3 Moderately ductile SPSW system

Although the moderately ductile SPSW system is a new concept, clearly these walls need a combination of ductility and redundancy that is lower than in ductile walls and higher than in limited-ductility walls. Based on the ductile and limited-ductility systems discussed above, two different moderately-ductile system concepts are envisioned.

The first moderately ductile SPSW concept has a redundancy consistent with limited-ductility walls and relies on the high-ductility design philosophy for the columns. By utilizing simple beam-to-column connections and designing for the uniform yield mechanism (Fig. 5.1(a)), a moderately ductile wall can be achieved. In effect, real hinges exist at both ends of each beam, and when the mechanism loads are applied, plastic hinges form at the bases of columns and the infill plates are fully yielded.

The second moderately ductile SPSW concept incorporates greater redundancy than the first, but permits limited yielding in the columns above the base. That is, rigid beam-to-column connections are incorporated, but the partial yield mechanism (Fig. 5.1(b)) is permitted. Under this mechanism, the connections experience relatively small rotations and, as a result, the strain hardening factor need not be applied when calculating the beam plastic moment. As such, the coefficient 1.1 is omitted from the evaluation of the beam plastic moments in Eq. (5.1). Moreover, the factor β can reasonably be selected as 0.5, since numerous finite element pushover analyses of a variety of wall geometries conducted as part of this research have demonstrated that the beam ends tend to undergo far less yielding than when the system is designed for the uniform mechanism.

5.5 Performance-based Capacity Design Approach for SPSWs

A SPSW system is designed primarily for external actions arising from gravity and seismic loads (design load combinations that include wind are beyond the

scope of this research). The seismic loads are resisted by a combination of the post-buckling tension field that develops in the infill plates and frame action of the boundary frames. The infill plates are the primary means of resisting the lateral loads and dissipating energy, and they are sized for the design seismic forces while utilizing their full yield capacity for any performance level. The rest of the system is designed based on capacity design principles to support the gravity loads concurrent with full tensile yield forces from the infill plates. In the case of SPSWs with moment-resisting beam-to-column connections, it is also assumed that the beams develop full plastic hinges at each end, creating additional demands on the columns under capacity design.

The effects of gravity load are treated the same for all performance levels, and hereafter are not mentioned unless required to underscore specific points. These effects can be superimposed onto the effects of the seismic loads described below. The terms “tension column” and “compression column”, based on seismic lateral loading effects, are used throughout Chapters 5 and 6 for expediency, even though with the addition of gravity loads both columns could ultimately be in compression. Moreover, these terms are used to describe specific behaviours even though they belie the fact that each column will act as the compression column at different points in time during an earthquake.

To ensure that the SPSW system is able to develop the full tensile yield capacity of the infill plates, the external seismic design loads are replaced with yield mechanism forces, F_i , as demonstrated in Fig. 5.4(a) for an n -story building. The figure also highlights the yielded infill plate diagonal tension fields in stories i and $i+1$, below and above beam i , which can readily be decomposed into their component vertical and horizontal uniformly distributed forces on the adjacent columns and beam, as shown in Figs. 5.4(b) and (c), respectively. The inclination angle, α_i , can be estimated based on the provisions of standard S16 or AISC 341.

To evaluate the yield mechanism forces, F_i , the lateral load distribution over the

wall height, which is defined by the coefficients μ_i in Fig. 5.4(a), is assumed. In this regard, a lateral load pattern similar to that of the seismic design loads from the appropriate building code (e.g., ASCE 2010, NRCC 2010) or the first mode distribution (ASCE 2007) can be selected. By setting $F_i = \mu_i F_s$, Eq. (5.1) or (5.2), as appropriate, then returns the total base shear, F_s . The yield mechanism force at each story, F_i , is then distributed to each side of the wall as in Figs. 5.4(b) and (c) such that $F_i = F_{iL} + F_{iR}$. (The selection of appropriate force components, F_{iL} and F_{iR} , is discussed below.) For any performance level, every non-fuse element of the system is designed to resist the expected tensile yield stress in the infill plates and, if rigid frame connections are present, the expected plastic moment capacity of the beams at their anticipated hinge locations and the resulting shears, while the system is subjected to the yield mechanism forces.

The boundary frame flexibility limits, as stated in the relevant design standard, should be satisfied regardless of the performance level to prevent excessive pull-in of the boundary members as the mechanism load is approached. Other parts of the SPSW are designed differently based on the target seismic performance level, as explained in Chapter 6.

5.5.1 Applied forces from infill plates on boundary frames

The stress state in the infill plates of SPSWs at the ultimate capacity of the system is considerably more complex than typically assumed in design. It is influenced by the stiffness of the boundary elements, rigidity of the frame connections, panel aspect ratio, continuity of the columns from story to story, distribution of the lateral loads over the height of the wall, demand-to-capacity ratio of the panel—as well as the panels above and below—and thickness of the infill plate itself. Not only does the stress field vary in character over the infill plate surface and through its thickness, the principal stress orientation also varies along the lengths of the boundary frame elements. A typical stress field taken at the mid-surface of the infill plate of a SPSW under the lateral mechanism load is depicted in Fig. 5.5, showing the variation of the principal stress vectors in terms of both magnitude

and orientation. While the variations over the entire panel area may appear dramatic, fortunately several key aspects can be identified to assist in interpreting the consequences of using conventional analytical idealizations.

In the analysis and design of SPSW systems, the infill plate is often replaced by a series of parallel tension strips. The “tension strip analogy” implies that in the principal stress orientations in the infill plate, shown in Fig. 5.4(a) where σ_1 and σ_2 are the major and minor principal stresses, respectively, $\sigma_2 = 0$. As a result, any yield criterion—such as the von Mises criterion—is satisfied only when σ_1 reaches the uniaxial yield stress. This assumption is reasonable for the central region of the infill plate, which is susceptible to buckling under a small compressive force that typically does not exceed about 5% of the yield stress. As such, the assumption tends to provide a good estimation of the lateral shear capacity of the overall SPSW system. However, adjacent to the boundary frame members, the analogy breaks down because of the creation of a stiffened band around the periphery of the panel that is far less susceptible to instability. In fact, in these regions considerable compressive principal stresses develop such that when the mechanism load is reached the mean principal stress ratio along each boundary member, $\psi = |\sigma_2/\sigma_1|$ tends to be 0.2 to 0.3 (usually close to 0.2) for beams and 0.3 to 0.5 (usually close to 0.4) for columns, depending the wall configuration, applied load over the height of the wall, etc.

The phenomenon described above has three major implications for determining accurate applied forces (stresses) on the boundary frame members that are consistent with capacity design principles. First, the non-zero value of ψ results in earlier yielding of the infill plate around the boundary frame than is predicted by the tension strip analogy. For instance, by applying the von Mises yield criterion the acceleration of yielding can be described as follows:

$$\sigma_1/\sigma_y = (1+\psi+\psi^2)^{-1/2} \quad (5.3)$$

where σ_y is the uniaxial (tension strip) yield stress. Local yielding therefore occurs when σ_1 reaches only $0.9\sigma_y$ or $0.8\sigma_y$ for ψ ratios equal to 0.2 (beams) or 0.4 (columns), respectively. Second, the presence of σ_2 increases the stresses applied parallel to the boundary member and decreases those perpendicular (for both beams and columns). Third, the angle from vertical of the major principal stress, α , deviates from the codified value (S16 and AISC 341), which provides a good approximation of the average value only for the middle region of the infill plate. Experience has shown that for common SPSW configurations based on capacity design principles, the mean value of the angle α tends to be close to 39° and 51° adjacent to the beam and compression column, respectively (considering analogous stress states adjacent to the beam and column, the two values of α constitute complementary angles).

By and large, the tension strip analogy provides conservative capacity design forces for both the beams and columns, but the degree of conservatism varies and the simplifications of the method rely on certain compensating factors. The codified value of α tends to be fairly close to the actual mean value adjacent to the beams and, as a result, axial stresses are slightly underestimated and transverse stresses overestimated, both mainly because σ_2 is neglected. Any underestimation of axial stress is mitigated by the slight acceleration of yielding due to the presence of σ_2 , as defined by Eq. (5.3), although this same phenomenon increases the conservatism of the transverse design stresses on the beam. In general, the tension strip analogy provides acceptable and conservative results for the beam, since the net demands on the beam are derived from the differences in stresses in the infill plates above and below. However, the method is quite conservative for the top beam, where there is no infill plate above to alleviate the overestimates of shear and moment in the beam. The inability of the system to yield the top infill plate under the mechanism loads, as would likely occur if the plate in this story is thicker than required to resist its story shear, increases this conservatism further under capacity design methods.

For the compression column, the same phenomena cause the axial force to be underestimated and the shear force and bending moment to be overestimated at each story. However, the axial design force in the compression column in general would be conservative since, as shown in Chapter 6, a considerable portion of column axial compression comes from the shear reaction of the beam, which is itself overestimated by the tension strip analogy, as discussed above. In total, the axial compression in the column is moderately conservative (depending on the wall aspect ratio and number of stories), while the design shear and bending moment at each storey are highly conservative due mainly to the σ_2 effect that both accelerates yielding of the material and reduces the transverse stresses on the column. This effect is only slightly mitigated by the underestimate of the angle α adjacent to the column. Although the conservatism in the column design shears and moments exists for all performance levels, a less conservative method is proposed in Chapter 6 for limited-ductility SPSWs only.

5.5.2 Design of members with plastic hinges

Whenever a plastic hinge develops in a member under the mechanism load, that member cross-section need not be designed explicitly for its own plastic moment as long as the member satisfies the following three conditions to ensure ductile behavior at the hinge. The member should meet Class 1 (S16) or Highly Ductile Member (AISC 341) compactness requirements in order to be able to develop the plastic hinge without buckling locally. Also, appropriate seismic lateral bracing should be provided near the plastic hinge. Finally, the member must be designed for the shear induced at the plastic hinge location, considering the plastic moment reduced for the effect of any axial force present (M_p^*). However, adjacent elements—such as the foundation at the column base or the beam-to-column connections for frames with rigid joints—should be designed for the expected plastic moment at the influential hinge location(s) and any actions induced directly therefrom (increased for the effect of strain hardening for a ductile system). The location of plastic hinges can be evaluated from AISC (2005). Overall member stability limit states must be assessed considering the internal

forces along the length of the member, including at the hinge.

When a member is designed for the shear induced by the development of plastic moments in that same member, the shear force is determined using the nominal moment and the resistance factor need not be applied in evaluating the shear strength. In other words, when a member is designed for the effect of secondary actions (e.g., shear) induced by a primary action (e.g., plastic moment) in that member, the secondary action need not be increased by the expected yield stress factor (R_y) and the secondary design capacity need not be decreased by the resistance factor. This is because the material resisting the secondary action is the same as that causing the primary action; i.e., a higher or lower material strength affects both the demand and the capacity. An exception to this treatment is if the secondary action pertains to a limit state that is not a linear function of material strength, such as instability.

The rationale above can be used for any fuse element, such as an infill plate when designing a splice, as described in Section 5.5.5.

5.5.3 Design of beams and their connections

The beams in a SPSW are designed to resist forces due to tensile yielding of the infill plates and the external seismic design loads. When rigid connections are used, the shear and moments from frame action also contribute to the design forces. As such, three design actions—axial force, shear, and moment—are applied to the beam. Since the moment demand is easy to evaluate from the shear force, only the effects of axial and shear forces are discussed below.

5.5.3.1 Evaluation of axial forces

The beam axial design force can be evaluated by independently considering three constituents: the change in the axial force between the beam ends, and the magnitudes of the axial forces at each end. Each of these constituents itself can be induced from different sources. When the infill plate thicknesses above and below

an intermediate beam differ, there is a distributed unbalanced force on the beam. The horizontal component of the unbalanced forces (the difference between $\omega_{bx\ i}$ and $\omega_{bx\ i+1}$ in Fig. 5.4(c)) causes a change in axial force along the beam's length. Since the lower-story infill plate would not be thinner than that in the upper story, this effect imposes tension at the compression-column end and compression at the tension-column end of the beam. In a case where the infill plates above and below an intermediate beam have the same thickness (and material grade), this unbalanced force would be negligible. The yielded infill plates also apply distributed inward forces on both columns ($\omega_{cx\ i}$ and $\omega_{cx\ i+1}$ in Fig. 5.4(b)), inducing a uniform compressive force in the beam. In other words, in a laterally-loaded SPSW the columns are pulled toward each other by the internal forces in the infill plates, and the beams act as struts that keep the columns apart.

The seismic design load distributions in the floor and roof diaphragms, and the means of transferring these loads to the SPSW, also influence the axial force distribution in the beams, primarily by determining the proportions of the forces F_{iL} and F_{iR} in Fig. 5.4(b). In a case where the seismic design load at a floor is distributed equally to each side of the wall, the beam can be designed conservatively for the compressive force at the tension-column side. However, depending on the layout of the building plan and the means of tying the diaphragm to the SPSW, the seismic design loads can be transmitted into the SPSW in different ways, as shown schematically in Fig. 5.6. This figure shows two-story SPSWs with four possible diaphragm load transfer mechanisms, and the corresponding axial force distributions in both beams. In order to highlight the effect of the diaphragm force transfer mechanism, the same infill plate thickness is assumed for both panels. As such, the horizontal component of the unbalanced infill plate force on the intermediate beam is zero, while it is a significant contributor to the axial force in the top beam. Considering the fact that the inward reactive forces from the columns induce a uniform compression in the beams, the differences in the axial force distributions of the intermediate beams in Fig. 5.6 are associated exclusively with the means of load transfer from the diaphragms.

In a real design case, the SPSW would be subjected to some combination of the lateral load distributions in Figs. 5.6(a) to (d). For instance, in the case of a SPSW perpendicular and adjacent to the edge of a building, a combination of distributions (a) and (d) or (b) and (d) would likely occur, while for a system at the middle of the building plan, a combination of distributions (c) and (d) would exist. As such, a designer must consider possible combinations and design the beam for the envelope of potential axial force demands, since any combination could lead to the maximum tension or compression in the beam. It is important to note that although the seismic load transfer pattern from the diaphragms to the SPSW has a considerable effect on the axial force demands in the beams, its effects on other internal forces and deformations—such as shear and moment in the beams, internal forces in the columns, deformed shape and yielding pattern, and pushover curve of the entire wall—are typically small. As such, it generally only needs to be considered in the design of the beams and their connections (in addition to the diaphragm-force tributary members and their connections, as needed).

Based on extensive numerical studies, Moghimi and Driver (2013b) proposed a method to evaluate the design axial forces in beams of SPSWs with simple beam-to-column connections, and verified the method against experimental results. This method builds on the observation that the shear and moment distributions in the compression column vary with SPSW geometry far less than those in the tension column. It also makes use of the foregoing considerations regarding the mechanism load distribution over the height of the wall and the lateral load transfer mechanism from the diaphragms. The axial forces applied to the intermediate beams by the compression column (i.e., the sum of the shear forces in the column above and below each beam-to-column joint) was found to be 70~100% (100~125% for two-story walls) of the horizontal component of the tributary infill plate yield forces (considering a half-story tributary width above and below the beam for simplicity) and the force applied to the top beam was 50~90% of the horizontal component of the tributary infill plate yield force

applied to the top-story column. The variations in the forces transferred to the beam occur mainly because of differences in the relative lateral story deflections and the occurrence of incomplete yielding in some panels, which are a function of infill plate thickness and lateral load distribution over the wall height, as discussed in relation to Fig. 5.3. Regardless of the diaphragm load transfer mechanism, the maximum compression in each beam occurs when the force transferred from the compression column is maximum (i.e., using the 100% (125% for two-story walls) and 90% factors for the intermediate and top beams, respectively) and the maximum tension happens when the force is minimum (i.e., using the 70% (100% for two-story walls) and 50% factors, respectively). Although it is recommended that both the maximum tension and compression cases be checked, in most instances the compression case governs the beam design since a portion of the tensile force tends to be transmitted to the surrounding frame through the infill plates. Due to the large beam normally required at the top of the wall to provide adequate flexural stiffness for anchoring the infill plate tension field below and the relatively low internal axial force compared to the those in the intermediate beams, the axial force may not have a significant influence on the top beam selected. Having evaluated both the net axial force applied to the beam by the compression column and the collapse mechanism force at each beam-to-compression-column joint, the axial force at the beam end (and in the adjacent connection) is calculated from the free body diagram of the joint by subtracting the force induced by the column from the mechanism force. The axial force demand at the other end of the beam (adjacent to the tension column) can then be evaluated by adding the horizontal component of the unbalanced infill plate tensile yield force to the beam axial force demand at the compression-column end. While the system is actually highly indeterminate, this simple method provides reasonable axial forces for designing the beams of SPSWs with simple frame connections.

For cases with rigid beam-to-column connections, the commentary to AISC 341 recommends the method “combined plastic and linear analysis”, originally

developed primarily to evaluate the design actions on the columns (Berman and Bruneau 2008). The method does not consider the lateral load transfer mechanism from the diaphragm to the wall, and as a result may need to be modified accordingly under certain circumstances (Moghimi and Driver 2013b). In the case where the lateral loads are distributed equally to the left and right sides of the wall (i.e., the case of Fig. 5.6(c)), the results obtained for the axial force in the beam would be similar to those from the proposed method above. In such a case, the compressive force in each beam from the column reaction is approximately equal to the infill plate yield forces on the columns above and below the beam based on tributary widths (i.e., using a factor of 100% for both the intermediate and top beams, as described in the method for SPSWs with simple frame connections). The total beam axial forces at the compression- and tension-column ends are evaluated, respectively, by subtracting from or adding to this force one-half of the horizontal component of the unbalanced infill plate tensile yield force. As mentioned earlier, the lateral load transfer pattern from the diaphragms mainly affects the axial force demand in the beams. As a result, regardless of the real diaphragm force transfer mechanism, the “combined plastic and linear analysis” method, assuming the lateral loads are distributed equally between the left and right sides of the SPSW, is expected to provide reasonable estimates of the column design forces.

5.5.3.2 Evaluation of shear forces

The shear forces in SPSW beams are induced by the tension field action in the infill plates combined with frame action, if present. Fig. 5.7(a) shows the net actions at the centerline of a beam with simple beam-to-column connections. The associated shear and moment diagrams are also shown, where the shear reaction forces at the faces of the left and right columns are V_{bL} and V_{bR} , respectively. The net actions shown in Fig. 5.7(a) can be separated into the constituents shown in Figs. 5.7(b) and (c). First, as shown in Fig. 5.7(b), the vertical component of the unbalanced infill plate tensile yield forces on the beam ($\Delta\omega_{by}$) causes internal shear and moment along the beam’s length, resulting in a shear reaction of

$V_{lb} = \Delta\omega_{by}L_c/2$ at the face of each column (the effect of the horizontal component, $\Delta\omega_{bx}$, was discussed in the previous section). Second, as discussed by Qu and Bruneau (2010), the horizontal components of the tension field forces above and below each beam apply a distributed moment, $m_b = d_b (\omega_{bxi+1} + \omega_{bxi})/2$, to the beam, where d_b is the depth of the beam. In simply-supported beams, this results in a constant shear force in the beam ($V_{Mb} = m_b$), but no internal moment, as indicated in Fig. 5.7(c). The net beam shear distribution in Fig. 5.7(a) is determined by superimposing the distributions in Figs. 5.7(b) and (c). When the infill plates above and below an intermediate beam differ, the beam shear is non-uniform with its maximum magnitude at the compression-column side, as shown in Fig. 5.7(a). The same force components and equations are applicable to the top beam, considering the fact that the force above the beam is zero.

In a case where the infill plates above and below an intermediate beam have the same thickness (and material grade), the unbalanced distributed forces in Fig. 5.7(b) would theoretically be negligible according to capacity design procedures, and only the constant shear force induced by the distributed moments (Fig. 5.7(c)) would exist. However, depending on the design seismic load distribution over the height of the wall, in practice the upper infill plate may not have yielded fully since the shear resistance of both stories is similar, while the shear demand on the upper story is generally smaller. In such a case, similar to the case where the infill plate thicknesses differ, there is an extra shear and moment demand on the beam because of the vertical component of the resulting unbalanced infill plate forces. As a result, when the infill plates above and below an intermediate beam have similar thicknesses, it is recommended that when determining the shear and moment demands in the beam, the upper infill plate force be taken as 80% of its nominal capacity design value to account for the possibility of incomplete yielding (Moghimi and Driver 2013b).

When rigid frame connections are used in a SPSW system, the shear induced in the beam by the plastic moments due to frame action are also considered in the

design and are additive to the shears discussed above for SPSWs with simple connections. The shear is applied at the plastic hinge locations at each end of the beam, located at the distance S_h from the faces of the columns (AISC 2005). As such, the distance between the plastic hinges is $L_h = L_c - 2S_h$, and the induced shear, V_h , is equal to the sum of the plastic moments at the ends divided by L_h . The induced shear forces from the plastic hinges near the beam ends are discussed further in Chapter 6.

By adding the shear forces due to frame action (V_h) and gravity loads (V_G) to those shown in Fig. 5.7 (V_{lb} and V_{Mb}), the resultant shear forces at each beam end can be found from Fig. 5.8 for the general case. The beam shear at the compression-column side is maximum, as indicated in Fig. 5.2(a) and highlighted by the dashed circle in Fig. 5.8.

5.5.4 Design of columns

As for the beams, the columns of SPSWs are designed to resist forces due to tensile yielding of the infill plates and the external seismic design loads. When rigid frame connections are used, the shear and moments from frame action are added to the design forces. In general, the compression column is critical for design, although special conditions such as uplift can impose critical design requirements on the tension column as well. Also, special attention needs to be paid to the compression-column base, where in a ductile SPSW inelastic demand is expected to be extensive in a design earthquake.

5.5.4.1 Critical column (compression column)

The vertical component of the infill plate tension field applies tension or compression on the left and right column, respectively, as shown in Fig. 5.4(b). However, at any given level in the wall the amount of axial force in the compression column is larger than that in the tension column for the following reasons. First, some of the tension from the overturning moment is transmitted directly through the infill plate, diminishing the force in the tension column, while

the compression component is applied entirely through the compression column, as illustrated schematically at the foundation in Fig. 5.4(a). Second, the gravity forces are additive to the compressive column force, while they reduce the tensile force. These combined effects may cause a substantial difference in the levels of tensile and compressive column load.

The P - δ effect in the compression column of a SPSW potentially has a greater detrimental effect than in columns of other systems. Fig. 5.9 shows the i^{th} story of a SPSW under story shear forces, and the resulting tension field force components are shown on the columns. It can be seen that the horizontal component of the tension field exacerbates the P - δ effect by applying an inward force to the compression column in the same direction as its deflection between floors. While this destabilizing condition must be evaluated during design, the minimum column stiffness criteria in most design standards ameliorate this situation considerably.

At any performance level, column bases might be stiffened such that the plastic hinge at the base forms above the base plate, but the stiffened height should be limited to alleviate the shear induced by the column plastic moment.

Common practice is to provide a fixed-support condition where the columns meet the foundation, and under capacity design procedures plastic hinges are expected to develop at the column bases. However, the column cross-section need not be designed explicitly at the base for the plastic moment (M_{pc}^*) as long as the stability and ductility of the region is assured. The actual plastic column-base moment at the yield mechanism load can converge to a small value (compared to the plastic moment capacity of the cross-section, M_{pc}) for the critical (compression) column in certain cases. Fig. 5.10(a) and (b) show moment distributions in the compression columns of SPSWs with simple and rigid beam-to-column connections, respectively, under the yield mechanism loading. The moments are normalised by the corresponding plastic moment capacity of the

column cross-section. For the wall with simple connections, the moment distribution resembles that of a continuous beam under distributed loading, but for the wall with rigid connections the plastic moments from the beam ends are added at the joint locations. For each type of wall depicted in Fig. 5.10, the moment distributions corresponding to two performance levels are illustrated based on the three-tier performance-based design concepts proposed in this paper. The higher and lower performance levels are associated, respectively, with moderately ductile and limited-ductility designs for walls with simple connections (Fig. 5.10(a)) and ductile and moderately ductile designs for walls with rigid connections (Fig. 5.10(b)).

For higher performance levels, where strong columns are needed, the cross-section at the base of the compression column can develop a significant percentage of its plastic moment, and the dashed lines in Fig. 5.10 show the moment diagrams for such a case. For lower performance levels, for which the moment diagrams are depicted in the figure by solid lines, a smaller column section can be justified for a given infill plate thickness. The reduction in cross-sectional area causes an increase in the axial compressive stresses in the column at the mechanism condition. Therefore, the compressive force uses up a greater proportion of the normal stress capacity of the cross-section and the remaining moment capacity of the column at the base is considerably reduced. The coefficient β_c in Fig. 5.10 accounts for the reduction in the plastic moment capacity of the column at the base due to the presence of the axial force. This coefficient is less than unity and its value depends on the axial force in the column under the yield mechanism loading. For higher performance levels, β_c tends to be in the range of 0.4 to 0.6, but for lower performance levels the factor converges to a small value, as shown in Fig. 5.10. This moment capacity reduction at the base affects mainly the moment distribution in the first story of SPSWs with simple frame connections and the first two stories of SPSWs with rigid frame connections. The differences in the moment distributions in the upper stories between the two performance levels is mainly caused by the differences in

the cross-sectional dimensions of the columns.

Due to the presence of the large axial column force that develops in the first story, the columns can be analyzed and designed conservatively for a pinned-base condition when elastic materials are assumed for the boundary members. The results shown in Fig. 5.10 suggest that the pinned-base assumption for the compression column in lower performance walls is fairly consistent with the real response, despite the fixed column-base detailing. Also, for higher performance walls, although some level of rotational fixity is likely to exist, the pin assumption is often conservative and acceptable providing appropriate cross-sectional compactness requirements are met and seismic bracing is provided.

5.5.4.2 Evaluation of shear forces

Similar to the beams of SPSWs, the columns have two main sources of shear force: tensile yielding of the infill plates and frame action. The horizontal component of the infill plate force applies a distributed load that causes a shear (V_{Ic}) in the column at the top and bottom of the story, as shown in Fig. 5.8. The vertical component of the infill plate force applies a distributed moment about the column centerline, which causes additional shear reactions (V_{Mc}). When rigid beam-to-column connections are used, the frame action induces further shear (V_F). The three shear reactions V_{Ic} , V_{Mc} , and V_F correspond, respectively, to shear reactions V_{Ib} , V_{Mb} , and V_h in the beams. The maximum net shear force in each column at each story occurs where the beam-to-column connection is in the closing mode, as shown in the deformed shape of the story depicted in Fig. 5.8 (locations distinguished by dashed rectangles) and also in Fig. 5.2(a).

5.5.5 Design of infill plate splices

Although the infill plates are among the protected zones specified in S16 and AISC 341, a previous test (Moghimi and Driver 2013a) has shown that infill plates with a single-sided lap splice exhibit excellent performance up to a lateral drift ratio of 4.6%, which is far greater than the ductility required of a limited-

ductility SPSW system. Similar results were obtained for double-sided infill plate splices used in a test by Dastfan and Driver (2010). The design of these splices must follow capacity design principles; the expected yield stress factor (R_y) is applied to the nominal yield stress of an infill plate to determine the forces on the splice plate if the splice material is different from that used for the infill plate. However, this factor may be omitted if it can be assured that the splice plates will be cut from the same source plate as the infill plates themselves. In the latter case, the splice plate capacity also need not be decreased by the resistance factor, since the material that produces the demand (infill plates) is the same as that of the designed element (splice plate). When the cutting of splice and infill plates from the same source plate cannot be assured, a thicker splice plate will be needed to meet capacity design objectives.

5.6 Summary and Conclusions

Previous research on SPSWs has focused on maximizing system ductility and seismic performance, effectively making them economically competitive only in high seismic regions. A new three-tier, performance-based capacity design framework for SPSWs has been proposed that will accrue several technical and economic benefits and increase the competitiveness of this system in low and moderate seismic regions. Besides the ductile SPSW format, a limited-ductility and two moderately ductile SPSW concepts have been defined and developed. In a SPSW system, it is primarily the behavior of the boundary frame—beams, columns, and beam-to-column connections—that defines the performance level, which is evaluated in this paper in terms of both redundancy and ductility of the overall system. The type of beam-to-column connection describes the redundancy of the wall, with simple and rigid connections delineating the system redundancy limits, and the ductility of the wall is classified in terms of the yield mechanism that develops. In this context, a system yield mechanism that is less ductile than the one assumed for highly-ductile walls has been defined, based on observations about SPSW behavior from both tests and numerical simulations, that targets performance considered adequate for limited-ductility applications.

Two main aspects that could result in limited-ductility yield patterns forming have been identified. The high level of compressive force in the critical column of a SPSW along with large shear forces can cause a yield mechanism in the system that exhibits somewhat lower ductility. Also, when the infill plate thickness distribution over the wall height is not proportional to the shear demands imposed by the seismic loads, a lower-ductility yield mechanism can occur due to incomplete yielding in some infill plates and non-uniform lateral deformation of the compression column. By selecting smaller columns, the performance of the wall may deviate from the ductile yield pattern, yet still be quite acceptable for walls under lower ductility demands.

Methods for determining the design forces in the components of ductile, moderately ductile, and limited-ductility SPSWs have been presented and discussed. It was found that although the conventional tension strip analogy, commonly used in design development, provides generally conservative design forces in the boundary members, this outcome arises due to a variety of compensating factors. These include the accelerated yielding caused by the two-dimensional stress state in the infill plate that is neglected by the strip analogy, the contribution of the minor principal stresses to the design actions on the frame, and the variability of the principal stress directions in the vicinity of the frame members. Even though the combination of these effects cause the axial stresses applied to the column from the infill plate to be underestimated, the axial design forces in the columns are reasonable since the same phenomena cause the shear reactions from the beams to be overestimated. Conversely, the design moments in the columns of SPSWs obtained using the tension strip analogy tend to be highly conservative.

Table 5.1 Current seismic response modification factors for SPSWs and MRFs

System	Performance level	ASCE 7-10		CSA S16-09			
		Designation	R	Designation	R _d	R _o	R
MRF	High	Special	8	Type D	5	1.5	7.5
	Medium	Intermediate	4.5	Type MD	3.5	1.5	5.25
	Low	Ordinary	3.5	Type LD	2	1.3	2.6
SPSW	High	Special	7 (8 ^a)	Type D	5	1.6	8
	Medium	Intermediate	–	Type MD	–	–	–
	Low	Ordinary	–	Type LD	2	1.5	3

^a Dual system with Special MRF capable of resisting at least 25% of the prescribed seismic forces

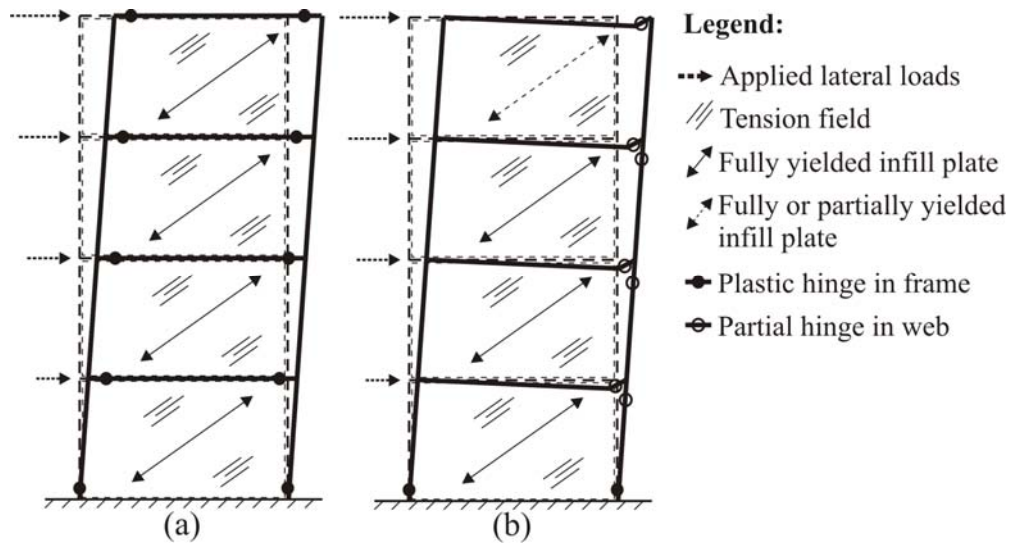


Fig. 5.1 Yield mechanisms: (a) Uniform yield mechanism for ductile wall, (b) Partial yield pattern for limited-ductility wall

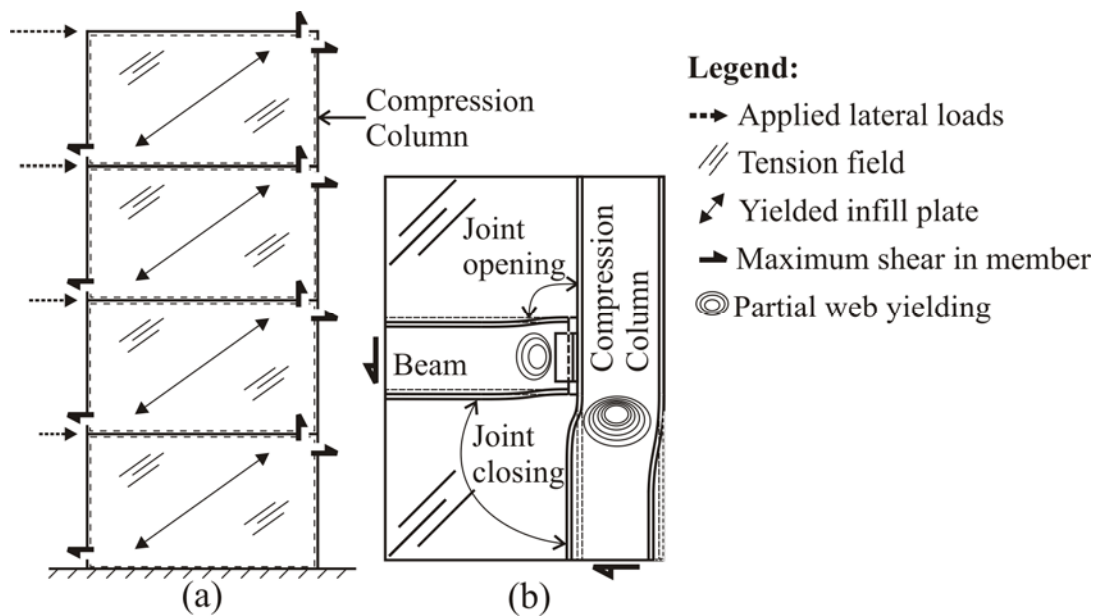


Fig. 5.2 (a) Locations of maximum shear forces in frame, (b) Beam-to-compression-column joint yielding for limited-ductility wall

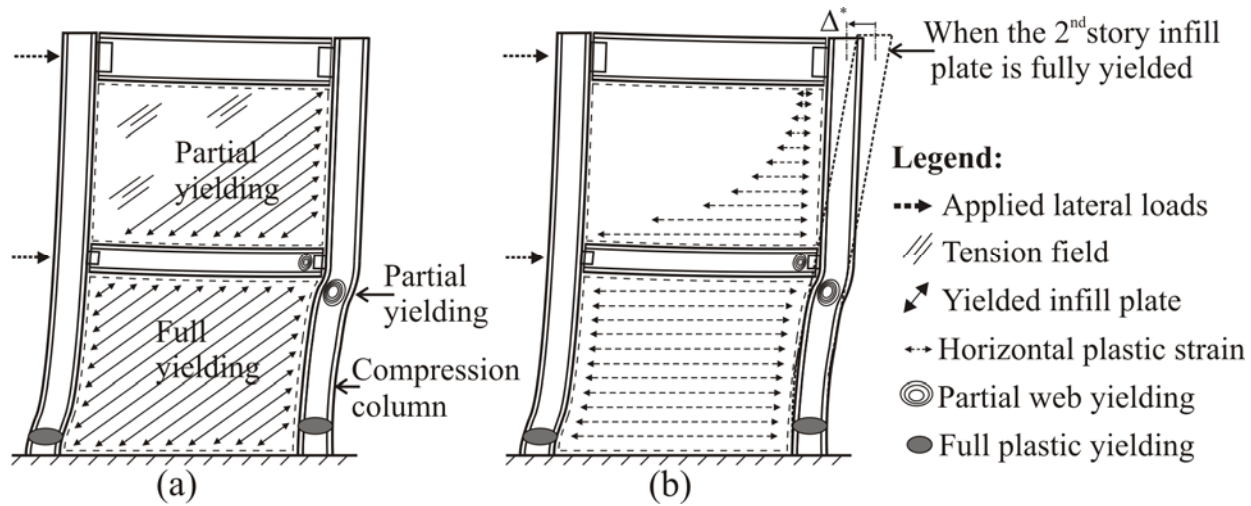


Fig. 5.3 Non-uniform infill plate yield pattern in a two-story limited-ductility wall: (a) Yielded tension field distribution, (b) Horizontal component of plastic strain distribution

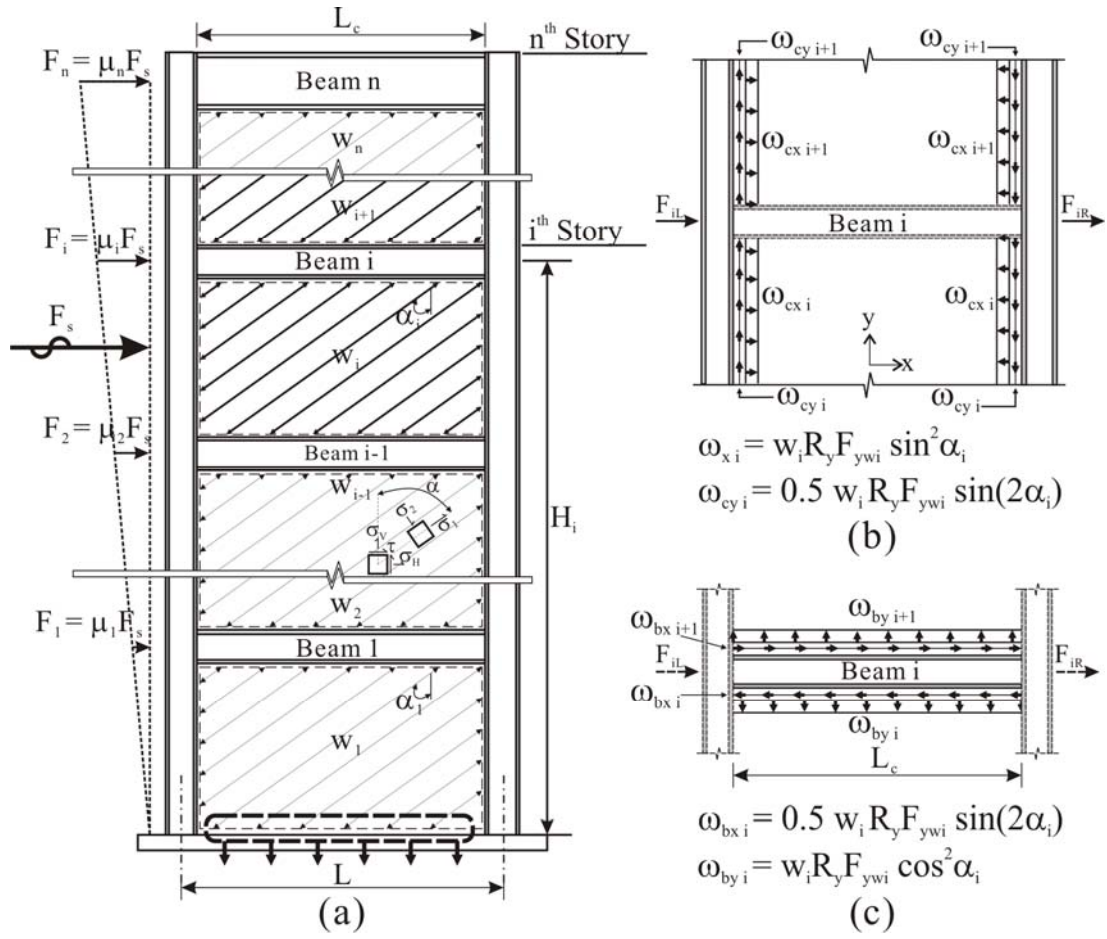


Fig. 5.4 (a) System yield mechanism loads of a multi-story SPSW with simple or rigid frame joints, (b) Full tensile yield forces on columns in stories i and $i+1$, (c) Full tensile yield forces on i^{th} story beam

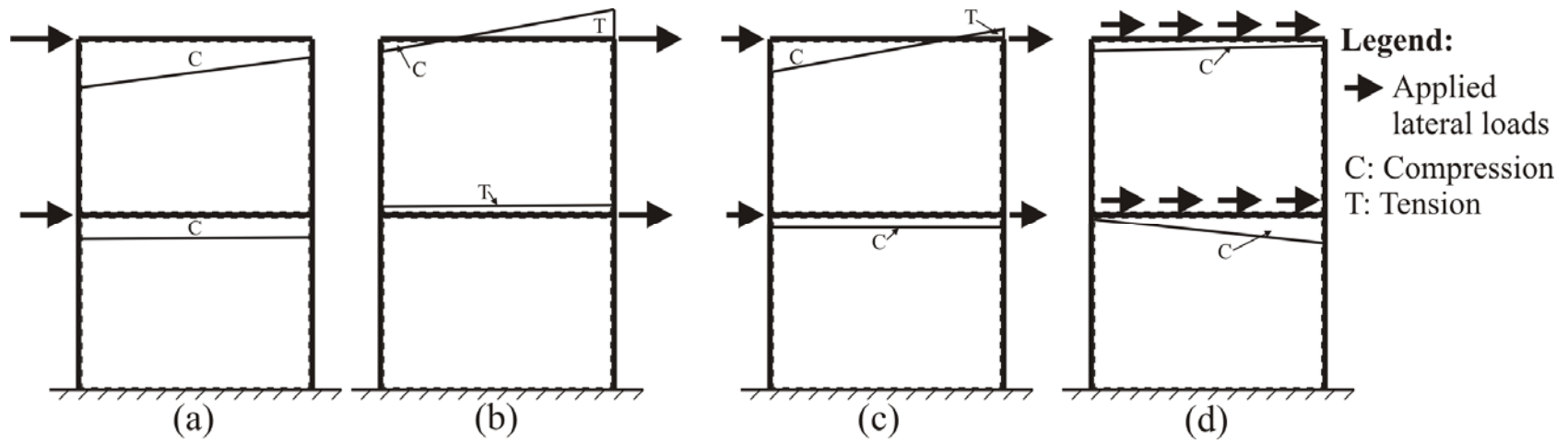


Fig. 5.6 Effect of lateral load transfer pattern from floor and roof diaphragms on beam axial forces

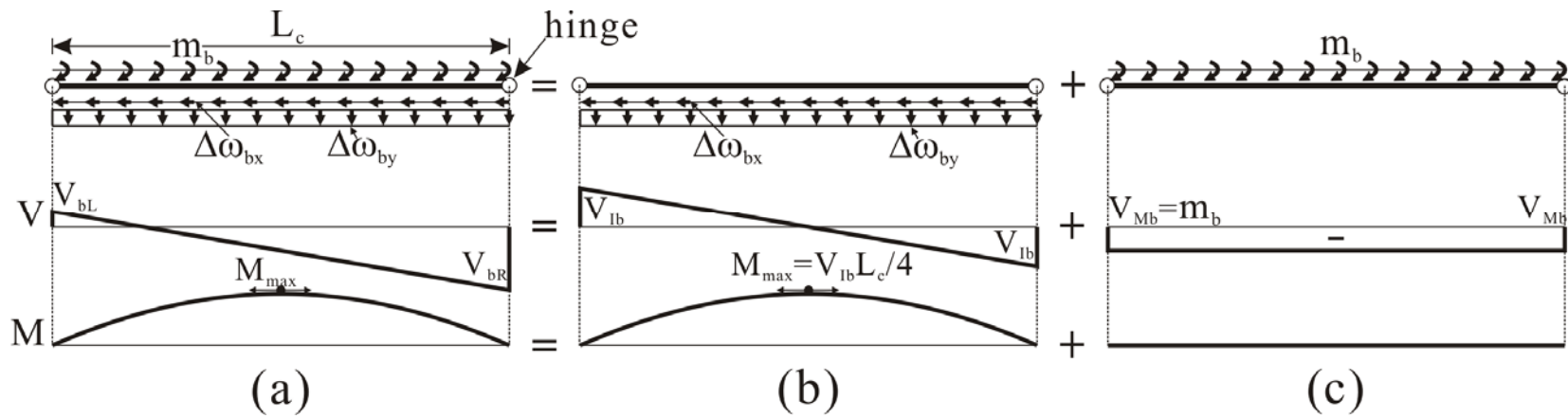


Fig. 5.7 Internal actions for beams with simple connections due to full tensile infill plate yielding: (a) Net actions, (b) Orthogonal components of unbalanced force, (c) Distributed moment

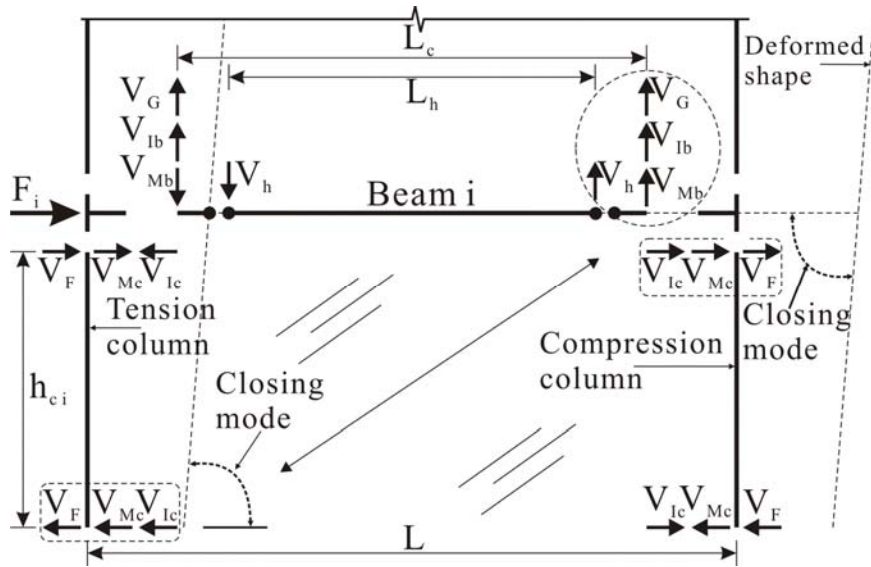


Fig. 5.8 Shear forces on beams and columns of SPSW system

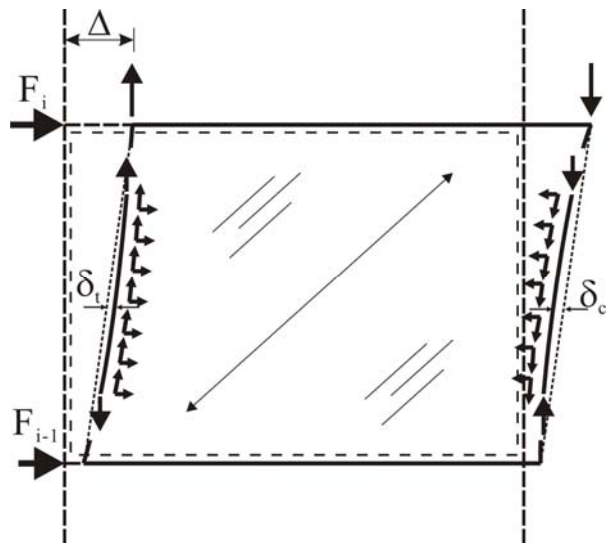


Fig. 5.9 P-Δ effect on columns of SPSW system

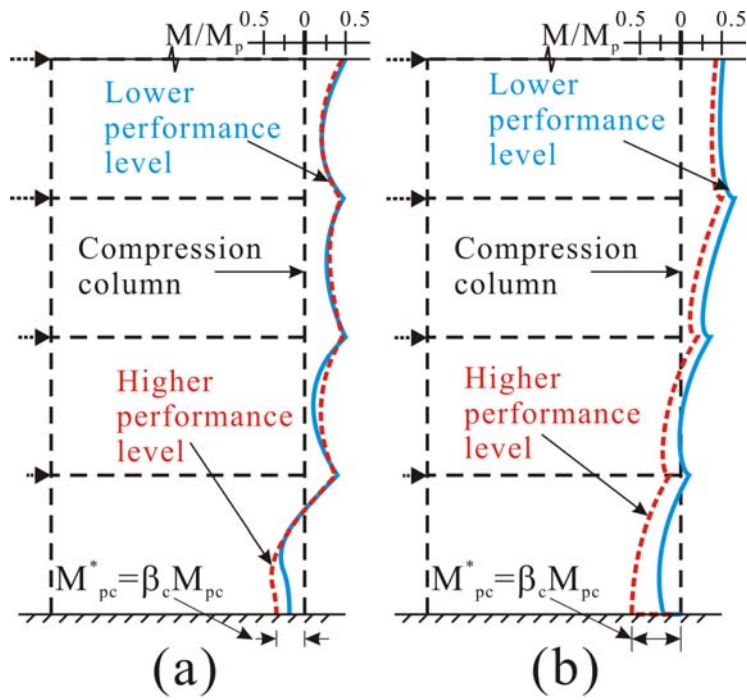


Fig. 5.10 Normalized moment distribution in compression column: (a) SPSW with simple frame connections, (b) SPSW with rigid frame connections

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6. PERFORMANCE-BASED CAPACITY DESIGN OF STEEL PLATE SHEAR WALLS. II: DESIGN PROVISIONS⁵

6.1 Introduction

This paper proposes provisions for performance-based capacity design of steel plate shear wall (SPSW) systems for different seismic performance levels, with emphasis on limited-ductility walls for low and moderate seismic regions. The Canadian steel design standard, S16-09 (CSA 2009), hereafter referred to as S16, has adopted two performance levels: Type D (ductile) and Type LD (limited-ductility). The *Seismic Provisions for Structural Steel Buildings*, ANSI/AISC 341-10 (AISC 2010a), hereafter referred to as AISC 341, adopted only the ductile SPSW concept: Special Plate Shear Walls. Although capacity design requirements for Type LD SPSWs have been included in S16, they were developed using the Type D wall provisions as a starting point, with a few relaxations of the rules. New requirements, developed independently, are needed for economical limited-ductility walls that comply with the intent of the capacity design principles stated in S16 and AISC 341. Similar to the treatment of moment-resisting frames, a three-tiered framework of performance levels for SPSWs is proposed: limited-ductility, moderately ductile, and ductile walls.

A main emphasis of Chapters 5 and 6 is on developing capacity design provisions for limited-ductility SPSWs. Therefore, it is important to distinguish this intent from the use of the design category “steel systems not specifically detailed for seismic resistance” (ASCE 2010), where $R = 3$ in seismic design category B or C and the system is designed and detailed in accordance with the *Specification for Structural Steel Buildings* (AISC 2010b). This latter procedure is demonstrated for SPSWs by Sabelli and Bruneau (2006), but they note that full compliance with the capacity design rules of AISC 341 (AISC 2010a) is not required. In both methods,

⁵ A version of this chapter has been submitted for publication in the *Journal of Structural Engineering*, ASCE.

the lower ductility capacity compared to ductile SPSWs is compensated by a reduced response modification factor (R -factor) assigned to the system, which makes the wall laterally strong enough to greatly limit the ductility demands. However, even though the system may be laterally strong, failure to impose capacity design constraints could lead to poor performance of its boundary elements, which are not necessarily proportioned and detailed for loads commensurate with yielding of the infill plates. Nevertheless, the new capacity design provisions must keep in view the much lower ductility needs when a low R -factor is used for determining the design loads.

6.2 Scope and Objectives

This research aims to establish a performance-based framework for designing SPSWs to allow for a range of design objectives. As they have received relatively little research attention, one major goal is to develop reliable and economical capacity design provisions for limited-ductility SPSWs within their own context, rather than simply being a modified version of those used to obtain highly-ductile performance. As such, the requirements are based on observations from research specifically attuned to limited-ductility objectives, with their efficacy being verified by results of a physical test designed based on this method (Moghim and Driver 2013a). With additional research data on limited-ductility wall performance now available, and heeding the extensive collective knowledge accumulated to date about ductile walls, design provisions for moderately ductile walls are rationalized as an additional option for designers between the two extremes. The proposed design provisions for limited-ductility and moderately ductile walls are applied to design examples, and the results are compared with numerical pushover analysis results. Also, the proposed methods are substantiated against experimental results on multi-story walls subjected to cyclic loading.

6.3 Capacity Design of Limited-Ductility SPSWs

Limited-ductility SPSWs have lower ductility and redundancy requirements as compared with ductile wall designs. Acceptance of lower redundancy allows the

use of relatively inexpensive simple beam-to-column connections, provided they behave in a robust manner under cyclic loading. The low ductility demand justifies minor yielding in the columns at the yield mechanism condition, which permits using smaller column sections compared with those in a ductile wall. As such, the system consists of a frame with simple beam-to-column connections and relatively thick infill plates (compared to the column size), which makes the yield pattern of the system similar to the one shown in Fig. 5.1(b). Therefore, hinges exist at all beam ends due to the simple connections, and plastic hinges form at the column bases, while the infill plates are assumed for design of the frame members to be yielded in all stories. The infill plate tensile yield forces on the boundary frame members are defined in Figs. 5.4(b) and (c), setting $R_y = 1$ in the equations for limited-ductility walls. The use of the nominal infill plate yield stress is an acknowledgement of the accelerated yielding caused by the development of significant minor principal stresses that are neglected by the equations, as discussed in Chapter 5. The system yield mechanism loads, F_i , are evaluated from Eq. (5.2).

6.3.1 Beam design

The beams of limited-ductility SPSWs are designed to resist forces resulting from tensile yielding of the infill plates and the external lateral mechanism loads, where the latter cause only axial force in the beam, while the former induces axial force, shear, and moment. Fig. 6.1(a) shows the design forces applied to the i^{th} intermediate beam (all forces are considered positive in the direction shown in the figures and gravity loads are excluded). The beam-to-column joints have simple connections and the external lateral mechanism load at level i (F_i) is distributed in some proportion to the two sides of the wall (F_{iL} , F_{iR}), as discussed in Chapter 5. Only the axial force reactions at the beam ends (P_{cL} , P_{cR}), which depend on the external lateral mechanism loads, the inward pull of the infill plates on the columns, and any unbalanced forces above and below the beam from the infill plates, are shown at the connection location. The vertical reaction forces are not shown for clarity, since they can readily be calculated from a free body diagram

of the beam. The uniformly distributed forces applied to the beam by the infill plates above and below are defined in Fig. 5.4(c) for the yield mechanism condition, with $R_y = 1$ for limited-ductility walls.

A summary of the beam design forces is provided in Fig. 6.1(b) as referenced to the beam centerline. The distributed vertical component of the unbalanced force ($\Delta\omega_{by} = \omega_{byi} - \omega_{byi+1}$) induces shear and moment in the beam, while the horizontal projections of the infill plate forces above and below the beam induce a constant transverse shear equal to:

$$V_{Mb} = d_b (\omega_{bxi+1} + \omega_{bxi})/2 \quad (6.1)$$

as well as a distributed axial force. (Note that in Fig. 6.1(b) the distributed moment described in Fig. 5.7(c) is replaced symbolically by end shears to emphasize the fact that this distributed moment produces a constant shear in the beam and no moment.) The column reaction on the beam due to the inward pull from the infill plates and the means of transfer of the external lateral mechanism load from the floor or roof diaphragm into the SPSW system at that level determines the axial force in the beam at the compression-column (right) side, P_{cR} . The distributed horizontal component of the unbalanced infill plate force ($\Delta\omega_{bx} = \omega_{bxi} - \omega_{bxi+1}$) causes the axial force magnitude to change along the beam's length, such that:

$$P_{cL} = P_{cR} + \Delta\omega_{bx} L_c \quad (6.2)$$

where P_{cL} is the axial force in the beam at the tension-column (left) side.

For beam design, the external lateral mechanism load (F_i) can be evaluated using M_{pc} instead of M_{pc}^* in Eq. (5.2) to simplify the calculations, since it is a conservative approach and has only a small effect on the beam axial design forces.

6.3.2 Beam-to-column connection design

The beam-to-column connections are designed for the beam shear and axial reaction forces at the faces of the column, which are induced by tensile yielding of the infill plates and the external lateral mechanism loads. The connection design forces can readily be evaluated from the net beam design forces shown in Fig. 6.1(b). While in principle any of a variety of rotationally-flexible simple connections could be used in a limited-ductility SPSW, double-angle shear connections bolted to the column flange are recommended (Moghimi and Driver 2013a) unless experimental evidence of good cyclic performance under loads consistent with usage in a SPSW is also developed for another connection type.

6.3.3 Column design

6.3.3.1 Applied loads

The columns are designed to resist internal forces resulting from the external lateral mechanism loads and tensile yielding of the infill plates. Fig. 6.2(a) shows the design forces applied to the columns of the i^{th} and $i+1^{\text{th}}$ stories. The uniformly distributed forces applied to the columns by the infill plates are defined in Fig. 5.4(b) for the yield mechanism condition of the system, with $R_y = 1$ for limited-ductility walls. The lower and upper stories shown in Fig. 6.2(a) have clear column heights of h_{ci} and $h_{c\ i+1}$, respectively, and the other geometrical parameters have been defined previously. The external lateral yield mechanism load at beam i (f_i), which differs from that used to design beam i itself, is distributed to each side of the wall (f_{iL}, f_{iR}) and is defined in the next section. The net column design forces are shown in Fig. 6.2(b) as referenced to the column centerlines.

The beam axial and shear force reactions are reflected onto the interior column faces at each level. The axial forces are described in the previous section, and the detailed method for determining them is provided in Chapter 5. The vertical component of the unbalanced infill plate force on the beam results in the following vertical force being applied to the columns:

$$V_{Ib} = \Delta\omega_{by}L_c/2 \quad (6.3)$$

As such, the total vertical forces applied to the columns at these locations are equal to:

$$V_{bL} = V_{Ib} - V_{Mb} \quad (6.4a)$$

and

$$V_{bR} = V_{Ib} + V_{Mb} \quad (6.4b)$$

These vertical reactions from the beam induce both axial force and moment in the column. As such, they are transmitted to the column centerline using an eccentricity of one-half the column depth, $d_c/2$.

Besides applying both horizontal and vertical distributed forces on the column in each story, the infill plates induce a distributed moment in the column equal to:

$$m_{ci} = \omega_{cyi} d_c/2 \quad (6.5)$$

which for the purpose of determining the external lateral yield mechanism loads can be replaced by a statically-equivalent concentrated moment of:

$$M_{ci} = m_{ci} h_{ci} \quad (6.6)$$

as shown dashed in Fig. 6.2(c).

6.3.3.2 Design yield mechanism loads

At the yield mechanism load, plastic hinges always form at both column bases. As such, the column can be analysed and designed by applying the expected plastic moment at the column base, increased for the presence of strain hardening but

reduced for the effect of axial load in the column, $1.1R_y M_{pc}^*$.

For designing the columns, it is necessary that the system yield mechanism loads be in equilibrium with the other loads applied to the columns. Therefore, the loads obtained from Eq. (5.2) are modified, since they do not account for the infill plate loads and beam reactions being eccentric to the column centerlines. As a result, the free body diagram of each column should be considered, and the yield mechanism load can be evaluated from moment equilibrium. Fig. 6.2(c) shows the free body diagram of the first three stories of the compression column. The consideration of moment equilibrium by summing moments about the column base results in the following equation for determining the compression-column design yield mechanism loads, f_{iR} :

$$\begin{aligned} \sum_{i=1}^n f_{iR} H_i &= \sum_{i=1}^n \omega_{cxi} h_{ci} [H_i - (h_{ci} + d_{bi}) / 2] \\ &+ \sum_{i=1}^n (V_{bRi} d_c / 2 + M_{ci}) - \sum_{i=1}^n P_{cRi} H_i + 1.1R_y M_{pcR}^* \end{aligned} \quad (6.7)$$

where $R_y = 1$ in determining ω_{cxi} , H_i is the height of f_{iR} from the base, and f_{iR} is distributed over the height of the wall in the same way as the system yield mechanism loads (F_i) defined in Eq. (5.2). Substituting f_{iR} with μf_R , where $f_R = \Sigma f_{iR}$ is the total compression-column base shear, Eq. (6.7) can be solved for f_R . The base moment M_{pcR}^* can be obtained by a trial-and-error method (if the column cross-section has yet to be designed); alternatively, for simplicity the column can be analysed and designed based on a pin-support condition by removing the expected plastic moment at the column base in Eq. (6.7). As shown in the design examples, the assumption of a pinned base does not affect the column design moments greatly, since they are typically small in the presence of a significant axial force, and releasing the moment at the base increases the moment at the top of the first story.

6.3.3.3 Design forces

To ensure that the columns have enough strength to withstand the yield mechanism loads, they are designed for the internal forces determined by considering the free body diagram of the entire column (Fig. 6.2(c)). The moment demand at each story is influenced mainly by the infill plate force applied to the column at that story, while the axial force is cumulative of the infill plate forces and beam shears applied at all levels above the story. Consequently, although the maximum moment demand may happen at a given story during an earthquake, it is unlikely that the maximum compressive force predicted using capacity design forces at all stories would occur, except perhaps near the top of the wall. As such, it is proposed that, with the exception of the top two stories, the axial design force be decreased by 10% for two reasons. First, as established during the discussion in Chapter 5 of the yield mechanism for limited-ductility walls, partial yielding in the infill plates is expected in some stories. As a result, the axial force from a nonlinear pushover analysis is always less than the capacity design force, which assumes all infill plates are fully yielded. Second, compared to a wall subjected to an actual seismic event, the capacity design method tends to produce conservative axial column forces by structuring the infill plate force distribution similar to that consistent with the first mode of vibration, whereas higher modes always have some effect on the overall seismic response of a wall (an effect that becomes more significant for taller walls). Further research is required to determine if greater reductions in column axial design force can be justified for taller structures.

Beam-column design equations generally assume a constant compressive force in the member for simplicity. Therefore, the average axial force in each story is selected as a rational design demand for limited-ductility columns.

6.3.4 Infill plate splice

Infill plate splices, if needed, are designed based on capacity design principles to resist the expected yield strength of the infill plates ($R_y F_{yw}$). As discussed in Chapter 5, a properly-designed, single-sided splice provides adequate

performance for the limited-ductility system. It is important that the splice be proportioned so as not to inhibit the formation of a uniform tension field in the panel.

6.4 Capacity Design of Ductile SPSWs

The ductile SPSW system has been developed for high seismic zones through the work of many researchers and has been shown repeatedly to possess a high ductility capacity under cyclic loading. It incorporates a moment-resisting frame, which adds redundancy to the system compared to limited-ductility walls. The requirement that the columns remain elastic above the base enhances the ductility capacity of the system, and the yield mechanism would be similar to that depicted in Fig. 5.1(a), with plastic hinges forming at the beam ends and column bases, while the infill plates are fully yielded at every story. Although the yield displacement of the SPSW system—in the range of 0.5% (Driver et al. 1998)—is smaller than that of the moment frame alone—in the range of 1.0% (AISC 2010a)—the relatively thin infill plates in the ductile wall provide only a small amount of restraint against the rotation of the beam-to-column connections and the connections can experience large rotations as the beam ends undergo plastic hinging. As a result, the presence of strain hardening has been accounted for in the beam plastic moments in ductile walls in both the following design provisions and in the evaluation of the system yield mechanism load in Eq. (5.1). Also, the infill plate tensile yield forces on the boundary frames are defined in Figs. 5.4(b) and (c), wherein the infill plate yield stresses are increased by the factor R_y .

6.4.1 Beam design

Beams in ductile SPSWs are designed against internal forces resulting from frame action, full tensile yielding of the infill plates, and external lateral design loads. Compared to the beam actions in limited-ductility walls, only the frame action is fundamentally different, inducing additional shear and moment in the beam. Fig. 6.3(a) shows the design forces applied to the i^{th} intermediate beam. The

plastic hinges form in the beams a distance S_h from the faces of the columns (AISC 2005), a distance L_h apart. The internal moment and shear forces at the plastic hinge locations are shown only on the middle part of the free body diagram in Fig. 6.3(a) to reduce congestion. Similar to the limited-ductility wall case (Fig. 6.1(a)), only the axial force reaction is shown at the column interface. A summary of the beam design forces is shown in Fig. 6.3(b) as referenced to the beam centerline.

M_{pbL}^* and M_{pbR}^* are the nominal plastic moments at the beam's left and right ends, respectively, reduced for the effect of axial force in the beam. Since the mechanism condition with a plastic hinge at each end is considered, the beam becomes statically determinate for shear and moment. The shear force arising from frame action, V_h , must be considered in design and it could be calculated using the nominal flexural hinge capacity of the beam, M_{pbL}^* and M_{pbR}^* , increased by a strain hardening factor of 1.1, with the associated shear capacity determined with no resistance factor, ϕ . However, in order for this share of total design shear force to be directly additive to other factored shear forces in the beam, V_h may be calculated based on the strain-hardened factored plastic moment capacity of the beam, $1.1\phi M_{pb}^*$, as follows:

$$V_h = 1.1\phi(M_{pbL}^* + M_{pbR}^*)/L_h \quad (6.8)$$

assuming the value of ϕ is the same for both moment and shear. Where the governing shear limit state is something other than full yielding of the beam web, V_h in Eq. (6.8) should be increased to reflect the fact that the expected material strength being higher than the nominal value affects the demand more than the capacity, and V_h can then conservatively be multiplied by R_y .

As discussed for the limited-ductility wall, when evaluating the external lateral mechanism load (F_i) for the beam design, M_{pc}^* can conservatively be replaced by M_{pc} in Eq. (5.1) for simplicity, without impacting the beam design greatly.

6.4.2 Beam-to-column connection design

The forces to be used for the connection and column panel zone designs can be evaluated from the reaction forces shown in Fig. 6.3(b), considering the expected plastic moment capacity of the beam, including the strain hardening effect. For example, Eq. (6.8) for determining V_h is replaced by:

$$V_h = 1.1R_y(M_{pbL}^* + M_{pbR}^*)/L_h \quad (6.9)$$

Effective means of connecting the frame members for seismic moment resistance are discussed widely in the literature and this aspect is beyond the scope of the research.

6.4.3 Column design

6.4.3.1 Applied loads

Columns of ductile SPSWs are designed to resist internal forces resulting from the external lateral mechanism loads, the expected strength-based forces from the beams due to frame action, and full tensile yielding of the infill plates. Fig. 6.4(a) shows the design forces applied to the i^{th} and $i+1^{\text{th}}$ stories. The column is designed for the expected level of the beam plastic moments reduced for the effect of axial load in the beam, $1.1R_yM_{pb}^*$. The induced shear forces at the beam plastic hinge locations are also calculated based on the expected moment level, as per Eq. (6.9).

A summary of the column design forces referenced to the column centerline is shown in Fig. 6.4(b), and most have been defined in the Section 6.3. The beam shear force induced by the plastic moments, V_h , can be transmitted to the column face, along with a total moment of:

$$M_{pRi} = 1.1R_yM_{pbR}^* + V_h S_h \quad (6.10)$$

where for most unreinforced connections, $S_h = d_b/2$ (AISC 2005).

6.4.3.2 Design yield mechanism loads

Upon reaching the yield mechanism, plastic hinges always form at both column bases and, accordingly, a moment of $1.1R_y M_{pc}^*$ is considered for design. Also, the system yield mechanism loads defined by Eq. (5.1) should be modified such that the estimated lateral forces are in equilibrium with the other loads applied to the columns. Fig. 6.4(c) shows the free body diagram of the first three stories of the compression column. Summing moments about the base of the column:

$$\begin{aligned} \sum_{i=1}^n f_{iR} H_i &= \sum_{i=1}^n \omega_{xi} h_{ci} [H_i - (h_{ci} + d_{bi}) / 2] \\ &+ \sum_{i=1}^n [V_{bRi} d_c / 2 + V_{hi} (S_{hi} + d_c / 2) + M_{ci}] \\ &- \sum_{i=1}^n P_{cRi} H_i + \beta \sum_{i=1}^n 1.1R_y M_{pbRi}^* + 1.1R_y M_{pcR}^* \end{aligned} \quad (6.11)$$

Note that Eq. (6.10) is decomposed into its constituent parts in Eq. (6.11) in order to include a factor, β , in the beam moment term, which will be discussed in the context of moderately ductile SPSWs. For ductile walls, $\beta = 1.0$.

While this method ensures that the column has enough strength to withstand forces up to the yield mechanism condition, any change in the internal force demands due to potential non-uniform relative lateral column deformations, mainly from incomplete infill plate yielding in some stories, is not captured. As such, the strong-column/weak-beam check, as defined in S16 and AISC 341, is also recommended to ensure achieving a uniform yield mechanism.

6.5 Capacity Design of Moderately Ductile SPSWs

Two distinctly different moderately ductile system concepts are proposed in Chapter 5. In the first concept (with simple joints and designed for the uniform yield mechanism), the design forces are similar to those for limited-ductility walls, except that the infill plate forces are calculated by applying the factor R_y to the nominal infill plate yield stress. Eq. (6.7) is used to determine the yield mechanism loads for evaluating the column design forces, and the 10% reduction

in axial compressive force recommended for limited-ductility SPSWs is not applied to moderately ductile walls proportioned for the uniform yield mechanism.

In the second moderately ductile SPSW concept (with rigid joints and designed for the partial yield mechanism), the connections would be expected to experience only small rotations and it is unlikely that any significant material strain hardening would be experienced. Therefore, the strain hardening factor (1.1) need not be applied to the beam plastic moments in Eq. (5.1), in the equations given in Figs. 6.3 and 6.4, or in Eq. (6.11). The factor β equal to 0.5 is applied at the beam plastic hinges because of the partial yield mechanism. The resulting beam shear, V_h , is accordingly calculated based on the lower level of moment. The axial design forces in the columns of each story below the top two are reduced by 10%, as for the limited-ductility case.

6.6 Design Examples of SPSWs with Simple Beam-to-column Connections

6.6.1 Limited-ductility SPSW

The proposed method for capacity design force estimation and design of limited-ductility SPSWs is applied to a four-story wall, and a numerical model is then developed based on the designed wall. The yield mechanism of the system under pushover loading is examined and the resulting internal forces are compared with those obtained by the proposed design method.

The building is a four-story hypothetical medical clinic in Montréal, Canada. The structure is a steel-framed building with plan dimensions of 60 m in both the north-south and east-west directions. The first-story height is 4.2 m and the other stories are 3.7 m high. The lateral load resisting system consists of four limited-ductility SPSWs ($R = 3.0$) oriented in each principal direction. A schematic elevation of the wall to be designed is depicted in Fig. 6.5(a). The geotechnical report indicates site class E (soft clay) and the unfactored design dead and live loads are, respectively, 4.2 kPa and 2.4 kPa for the roof and 4.6 kPa and 3.6 kPa

for the floors. The mass of each level is uniformly distributed over the roof or floor area. The building is a regular structure with low torsional sensitivity, and the design of an isolated wall with maximum accidental torsion is considered. All steel materials have a nominal yield stress of $F_y = 350$ MPa.

The equivalent static force procedure of the National Building Code of Canada (NRCC 2010) was used to evaluate the design base shear and distribute the forces over the height of the wall. The total lateral seismic force that is tributary to one wall is 3884 kN, from which 210 kN is applied at the roof level. The remainder of the seismic force is distributed over the full height of the wall by the factor μ_i , giving the story seismic forces, F_{si} , in Table 6.1. Accidental torsion and the notional load are added to these forces to determine the total lateral design loads. F_i and $F_{i,C}$ in Table 6.1 are the equivalent static design story shear forces and their cumulative values, respectively. The infill plate thicknesses, w in Table 6.1, are selected for the first and third story panels based on the design shear strength (CSA 2009, AISC 2010a), and a tension field inclination angle of 40° (Shishkin et al. 2009) is selected as an initial estimate. The inclination angles are re-calculated (α in Table 6.1) after the boundary frames are designed, and the infill plate thicknesses are re-checked. For design practicality, the second- and fourth-story panel thicknesses are selected equal to those of the first and third, respectively, and both are readily available in the North American market. The boundary frame members are selected to satisfy the flexural stiffness requirements and resist the capacity design forces (S16 and AISC 341), and are shown in Table 6.1. For comparison, the standard rolled wide-flange shape closest to the selected welded built-up column (WWF 500×456) that possesses adequate strength and stiffness is a W610×498 (W24×335) section. However, if no depth limit were imposed, W920×449 (W36×302) and W920×368 (W36×247) sections would be the lightest rolled sections adequate for the lower and upper tiers, respectively. Note that a significant reduction in mass is not obtained in the lower tier by removing the depth limit because of the dominance of axial load in determining the section size required in the first story.

Fig. 6.5(b) shows the moment (plotted on the tension side) and axial force distributions in the compression column of the wall. Table 6.2, in the section entitled “Capacity design”, shows the design moments and axial forces for the compression column (P_{CD} and M_{CD}) immediately above each floor level and below the top beam based on the proposed capacity design method for limited-ductility walls, and assuming a column-base flexural capacity of $1.1R_yM_{pc}^*$. (Since simple beam-to-column connections are used, the column moments immediately below floors 1 to 3 are essentially equal to those shown in the table for above the joint.) The section “FE-Nonlinear” in the table shows the corresponding forces based on a nonlinear finite element pushover analysis of the wall designed according to the capacity design method. The infill plates are modeled as elasto-plastic shell elements that are able to buckle in compression and shear, with a nominal yield strength of $R_yF_{yw} = 1.1 \times 350$ MPa. The boundary frame members are assigned a yield strength of $R_yF_y = 1.1 \times 350$ MPa, with linear hardening beginning at 10 times the yield strain up to a stress of $1.1R_yF_y$ at the ultimate strain of 20%. A fixed condition is selected for the column at the foundation. The plasticity model takes into account the interaction between the axial force and moment on the cross-section, so that the plastic moment capacity at the base of the columns reduces under axial forces. The system is analysed using the same relative lateral load distribution (F_i) as shown in Table 6.1.

The resulting yield mechanism is similar to that shown in Fig. 5.1(b), with full plastic hinges forming at both column bases and full yielding of the infill plates developing in the first two stories and partial yielding in the top two stories. The yield patterns of the infill plates are described in detail in the Section 6.7. Minor yielding occurred in the compression column in the first story (at one-half the beam depth below the lower beam flange) and the rest of the frame remained elastic. $P_{FE,N}$ and $M_{FE,N}$ in Table 6.2 are the axial forces and moments, respectively, for the compression column at the same locations as the corresponding capacity design forces (P_{CD} and M_{CD}). The internal forces are normalized in the table using their factored member strength counterparts (P_r and

M_r) based on S16. These ratios give an indication of what portions of the capacity are utilized for moment and axial compression individually. It can be seen that the moment at the column base is very small (compared with the capacity), despite the fixed support condition. Also, the ratio $P_{FE,N}/P_r$ at the base is slightly greater than 1.0; this will often be the case, since at the base the column always develops its full cross-sectional capacity at the mechanism load, as discussed in Chapter 5, and P_r and M_r are lower-bound estimates of member capacity. The next two columns in Table 6.2 show the pushover values normalized by the counterpart capacity design forces. These ratios demonstrate that the proposed capacity design method predicts the design forces with reasonable accuracy on the conservative side.

Common practice for preliminary design investigations is to conduct simple pushover analyses of SPSWs by modeling the infill plates as elasto-plastic strips, and using elastic boundary frame members and a fixed-support condition at the column bases. The model is then pushed to the point where the frame reaches its plastic moment capacity at some point (at the base of the compression column in this case). In order to check the appropriateness of this method, the SPSW finite element model discussed above is modified by assigning elastic material properties to the boundary frame. Both fixed and pinned support conditions are considered for the column base, and the results are shown in Table 6.3. In both analyses, all infill plates are fully yielded except the top panel, which yielded only partially. The compression-column forces for the pinned and fixed bases are indicated by the subscripts "FE,E0" and "FE,EF", respectively. For comparison to the pinned case, the capacity design moment with the pinned column-base assumption ($M_{CD,0}$) is also shown in the table. (The axial force is not affected by this assumption, and is the same as in Table 6.2). For the pinned-base assumption, the internal forces are normalized by the capacity design forces (P_{CD} and M_{CD}) and by the capacity design moments with the pinned-base assumption ($M_{CD,0}$). The internal forces for the fixed-base assumption are normalized by the capacity design forces. The results in Table 6.3 show that when using elastic boundary

elements in the pushover analysis of a limited-ductility SPSW, assuming a pinned base provides acceptable results, while the fixed-base assumption results in an unrealistic bending moment distribution, with column design moments grossly underestimated in the second story (and of the wrong sign) and grossly overestimated in the first story. In fact, the use of an elastic boundary frame and fixed base results in a moment at the base so large that the selected column (Table 6.1) does not come even close to satisfying the design strength requirements (while the nonlinear model shows that the column is indeed adequate for a limited-ductility wall). However, selecting a larger column simply results in a larger design moment using this method, and the design is unlikely to converge. As such, pushover analysis of SPSWs assuming an elastic frame with fixed column bases provides unreliable and unrealistic design moments for the columns.

6.6.2 Moderately ductile SPSW

The wall designed and discussed above is checked for moderately ductile criteria (with simple beam-to-column connections) by redesigning the beams and columns accordingly. Although thinner infill plates might be justifiable by selecting a moderately ductile wall within the same structure due to an associated higher R -factor, the same thickness was used to permit a direct comparison between the frame members for limited ductility and moderately ductile designs. The expected yield strength of the infill plates ($R_y F_{yw}$), along with column design yield mechanism loads from Eq. (6.7), are used to design the boundary frames. While the beam design forces differ for the two seismic performance levels, it resulted in no difference in the beam sizes selected. Therefore, this section focuses only on capacity design of the critical (compression) column. The new column sections for both tiers are shown in Table 6.4. The section “Capacity design” gives the capacity design forces in the compression column. The standard rolled wide-flange shapes closest to the selected columns, WWF 550×721 and WWF 550×620 for the lower and upper tiers, respectively, are W690×802 (W27×539) and W610×608 (W24×409) sections. However, the lightest rolled

sections would be W1000×748 (W40×503) and W1000×483 (W40×324) for the lower and upper tiers, respectively. Again, the column mass in the lower tier is not reduced by selecting a deeper member.

Similar numerical models to those used for the limited-ductility design example are developed for the moderately ductile wall, and the results are shown in Tables 6.4 and 6.5, where the table columns are analogous to the corresponding columns in Tables 6.2 and 6.3, respectively. The yield mechanism based on the nonlinear pushover analysis is similar to that shown in Fig. 5.1(a), with full plastic hinges forming at both column bases and fully yielded infill plates developing in the first three stories and partial yielding in the top panel, and the rest of the system remained elastic.

$P_{FE,N}$ and $M_{FE,N}$ in Table 6.4 are the internal column forces from the nonlinear analysis. When these forces are normalized by their design strength counterparts (P_r and M_r), the results show that the moment at the base is 44% of the member moment capacity, which is significantly larger than the 11% value for the limited-ductility wall (Table 6.2). This is because of the larger column cross-section in the moderately ductile wall, which alleviates the plastic moment reduction due to the axial compression force effect. The internal forces are also normalized by the corresponding capacity design forces (P_{CD} and M_{CD}), showing that the proposed capacity design method provides reliable but conservative design forces.

The capacity design method with pinned column bases and the pushover analysis results with elastic boundary elements and both pinned and fixed bases are investigated in Table 6.5. Comparing the results for the first-story column shows that the capacity design method with pinned bases (Table 6.5) provides a more critical moment distribution than with fixed bases (Table 6.4), since the former not only produces the higher design moment (2850 kN·m) but the column is also bent in single curvature, which potentially reduces the moment capacity of the beam-column. The pushover analysis with an elastic boundary frame and pinned

column bases provides design demands in the compression column that are perhaps acceptable, although it tends to be quite conservative in the lower stories (similar to the capacity design forces assuming a pinned base), while imposing fixed column bases results in an unrealistic moment distribution.

6.7 Design Performance Verification

The examples above result in two potential designs for SPSWs with different target performance levels. While both employ simple beam-to-column connections, the limited ductility wall is expected to perform differently from the moderately ductile wall—both in the yield patterns that develop in the infill plates and in the yielding that takes place in the columns—and, as discussed in the previous sections, this was found to be the case. To study the performance of the limited-ductility wall further, the infill plates' yield patterns are examined based on the results of the nonlinear pushover analysis of this design. The proposed methods are also applied to two physical SPSWs tested under vertical load and cyclic lateral displacement. The capacity design forces at different performance levels are used to re-design the columns for these walls. The designed sections are then compared with those chosen for the test specimens, and the compression column performance at the yield mechanism is assessed.

6.7.1 Infill plate yield patterns from limited-ductility SPSW design example

The performance of the limited-ductility SPSW is investigated by examining the yield patterns in the infill plates in Fig. 6.6. (The lateral displacement scales in the figure are exaggerated to better convey the deformed shape). The figure depicts the distribution of effective plastic strain, ϵ_p^* , which constitutes a monotonically increasing parameter at each point in the material defined as $\epsilon_p^* = \sqrt{2/3 (\epsilon_p)^T (\epsilon_p)}$, where ϵ_p is the plastic strain tensor at the point. Fig. 6.6(a) shows the effective plastic strain distribution in the infill plates at the lateral deformation consistent with the current ductility-related force modification factor for designing limited-ductility SPSWs ($R_d = 2.0$) from S16 (CSA 2010), where it can be seen that the infill plates have barely yielded ($\epsilon_p^* = 0.0$ indicates elastic

behavior). There is very little deformation in the boundary frame, and performance up to this (design) displacement level is expected to be excellent. Fig. 6.6(b) shows the yield pattern when the wall is subjected to its yield mechanism load, indicating that the infill plates in the first two stories and much of the panel in the third story are fully yielded. Since panel yielding is the primary means of energy dissipation in the system, and the boundary frame deformations are consistent with those of the target yield mechanism for limited-ductility walls, even when loaded beyond the design seismic forces to the full capacity of the system the performance is expected to be satisfactory.

One of the distinguishing features of the proposed limited-ductility SPSW design provisions compared to those of the other performance levels is the use of the nominal infill plate yield stress in determining capacity design forces. While the observation that the infill plates are generally yielded upon reaching the mechanism load might seem to imply that the expected yield stress $R_y F_y$ should in fact be used, it is instructive to examine instead the actual forces applied by the infill plates to the critical (compression) column.

Table 6.6 shows the mean stress orientations and values in each infill plate adjacent to the compression column of the limited-ductility SPSW. In the table, α is calculated in three ways: based on the formula in the design standards (S16 and AISC 341), as the average of 20 elements at the middle of each panel from the finite element model (FE_Mid), and as the average of all elements immediately adjacent to the compression column (FE_Col). The values of σ_1 and σ_2 are the average of the major and minor principal stresses, respectively, in the infill plate elements adjacent to the compression column over the height of each panel. The averaged values for the first-story panel do not consider the effect of the three elements immediately above the base, since these elements are hardly yielded due to compatibility of deformations with the compression column, which is fixed at the base.

The absolute values of the ratio of the smaller-to-larger principal stress, ψ , are calculated for each element adjacent to the compression column, and then averaged over the height of each panel. The early yielding of the infill plate as compared to the tension strip model (due to the minor principal stress) is reflected by the quantity $\sigma_1/R_y F_{yw}$ according to the von Mises yield criterion.

The quantities $\sigma_{H, FE}$ and $\tau_{, FE}$ are the average horizontal (normal) and vertical (shear) stresses, respectively, applied directly to the compression column by the infill plates, as obtained from the finite element model. Stresses $\sigma_{H, CD}$ ($= F_{yw} \sin^2(\alpha)$) and $\tau_{, CD}$ ($= 0.5 F_{yw} \sin(2\alpha)$) are the corresponding horizontal and vertical stresses, respectively, based on the capacity design method proposed for limited-ductility walls (i.e., setting $R_y = 1$, estimating the tension field angle by the codified method, and utilizing the tension strip analogy for the infill plate). Stresses $\sigma_{H, CD}^*$ and $\tau_{, CD}^*$ show similar values as $\sigma_{H, CD}$ and $\tau_{, CD}$, but the superscript $*$ indicates a modification to account for the two-dimensional stress state in the infill plate. The superscripted values use $\alpha = 51^\circ$ (as discussed in Chapter 5) and account for both early yielding and also the effect of the minor principal stress, and are calculated as:

$$\sigma_{H, CD}^* = R_y F_{yw} \times (\sigma_1/R_y F_{yw}) \times (\sin^2(\alpha) - \psi \cos^2(\alpha)) \quad (6.12)$$

$$\tau_{, CD}^* = 0.5 R_y F_{yw} \times (\sigma_1/R_y F_{yw}) \times \sin(2\alpha) \times (1 + \psi) \quad (6.13)$$

(Based on this study, in the absence of accurate values it would appear that reasonable estimates of $(\sigma_1/R_y F_{yw})$ and ψ would be 0.8 and 0.4, respectively.) The results in Table 6.6 show that the values of $\sigma_{H, CD}^*$ and $\tau_{, CD}^*$ are very close to the finite element results, but require the inclusion of the effects of σ_2 and an accurate estimate of α . The difference between $\sigma_{H, CD}$ and $\tau_{, CD}$ —which use the traditional tension strip model except with $R_y = 1$ —and the associated finite element values shows that the net consequence of neglecting the two-dimensional stress state and using the codified value of α , which is accurate only in the central region of the

plate, is a slight overestimate of the horizontal stress and an underestimate of the vertical stress applied to the column. Although the method may not provide a conservative vertical stress, the associated axial compressive force in the column is accurate and conservative, as shown in Tables 6.2 and 6.4, because the same mechanism that increases the vertical stress on the compression column decreases the shear stress on the beam. As such, the contribution of the beam shear reaction (especially at the top beam) to the column axial force is overestimated, causing the overall axial force in the compression column from capacity design to be slightly conservative. For tall and narrow walls, on the other hand, the axial force may not be conservative and more research is needed to confirm that the analysis of stresses discussed here applies also to such walls.

Figure 6.6(c) shows the horizontal normal infill plate stresses that develop at the yield mechanism load, and these can be compared directly to the horizontal component of the capacity design stress on the column, $\sigma_{H, CD}$ in Table 6.6. The figure shows that the compression column is actually subjected to horizontal stresses much lower than the capacity design value (using $R_y = 1.0$), despite the fact that in the numerical model the expected yield stress of $R_y F_y = 385$ MPa is used. This observation is critical because it demonstrates that even when the infill plates are fully yielded, the horizontal components of the capacity design forces determined using the tension strip analogy tend to be quite conservative, owing to the lack of consideration of the two-dimensional stress state.

If the infill plates utilize a steel grade for which R_y is significantly greater than 1.1, it would be prudent to use $R_y F_y / 1.1$ in lieu of F_y for the capacity design stress demand on the boundary frame members of limited-ductility SPSWs.

6.7.2 Two-story SPSW test

The proposed capacity design methods for limited-ductility and moderately ductile SPSWs are applied to a two-story SPSW system with simple connections (Moghimi and Driver 2013a). The elevation of the tested specimen is shown in

Fig. 6.7(a) and the test is described in the Section 5.3.2. The mean static yield stresses for the infill plates and column flanges were found equal to 338 MPa and 348 MPa, respectively. The frame members meet Class 1 (S16) and Highly Ductile Member (AISC 341) compactness requirements, and they satisfy the stiffness requirements for boundary members of SPSWs. The infill plate connections to the beams and fish plates, as well as the fish plate connections to the columns, were designed to resist the expected tensile yield stress ($R_y F_y$) in the infill plates. The boundary frame members were designed using conventional nonlinear performance-based methods to be just adequate for a ductility-related force modification factor, R_d , of 2.0 (limited-ductility SPSW).

The design of the beams, which is a similar process for all seismic performance levels, is explained by Moghimi and Driver (2013a, b). The design of the columns is investigated in Table 6.7, wherein “Test specimen section” shows the column cross-section selected for both stories in the test specimen. In the section “Capacity design, limited-ductility”, the axial compression (P_{LD}) and moment (M_{LD}) design forces are evaluated based on the proposed method for limited-ductility walls assuming the development of the plastic moment at the base, reduced for the effect of axial compression. (All axial forces in Tables 6.7 and 6.8 include the vertical loads applied in the associated test.) To be directly comparable with the test results, these design forces are determined using the mean measured yield stress of the infill plates, and the column is redesigned using the mean measured yield stress of the column flanges, with no resistance factor applied. The column is designed for 90% of the average design axial force in the first story and the full moment, even though the proposed capacity design method for limited-ductility SPSWs applies this reduction to the axial force only below the top two stories; this is considered appropriate here since the reduction is representative of an anticipated lack of full yielding in all the infill plates above, which is indeed the observed case in the test specimen. The resulting column cross-section is one profile larger than the one used in the test specimen. A_{LD}/A and $I_{x,LD}/I_x$ in Table 6.7 are the cross-sectional area and strong-axis moment of

inertia ratios for the designed section to the test specimen section for reference.

Although the column in the test specimen was smaller than the one designed based on the proposed capacity design method, the wall performed as expected, with a yield pattern developing that is similar to the one depicted in Fig. 5.1(b). While the first-story infill plate was fully yielded, the one in the second story was partially yielded, and as a result partial yielding occurred in the first-story columns right below the beam-to-column connections (Moghimi and Driver 2013a), similar to the deformed state depicted in Fig. 5.3(a). That is, the yielding was concentrated in the column webs and extended downwards a distance of about a column-depth from the intermediate beam's lower flange. Minor yielding also occurred in the inner column flanges that extended downwards a distance of about a half-column-depth, but no yielding occurred in the outer column flanges. No collapse mechanism developed in the system and the strains in the yielded regions of the compression column remained well below the strain-hardening strain. The conventional double-angle shear connections used in the test specimen provided rotational freedom at the beam-to-column joints, reducing the demand on the columns as compared with moment-resisting connections. These connections showed remarkably good performance with no significant damage, even at the end of the test after many nonlinear cycles (Moghimi and Driver 2013a). The bolted, one-sided infill plate lap splices permitted the full development of the infill plate capacity in the critical story and contributed no deterioration to the system.

The normalized hysteresis curve for the base shear versus first-story lateral displacement is shown with a solid line in Fig. 6.8, where the push condition means lateral loads applied from the south to north (see Fig. 6.7(a)). The first-story lateral displacement, δ , is normalized by the first-story yield displacement, giving the inter-story displacement ductility ratio, μ , on the horizontal axis. The base shear, V , is normalized on the vertical axis by the maximum value achieved during the test. Although the specimen was designed for limited-ductility

performance, and even had a column section one size smaller than the one obtained using the proposed capacity design method, it showed excellent energy dissipation capacity, cyclic robustness, and ductility.

The test specimen is also re-designed as a moderately ductile system, and the results are shown in the section “Capacity design, moderately ductile” of Table 6.7. The capacity design forces in the compression column, P_{MD} and M_{MD} , are calculated based on the proposed method and assuming the formation of plastic moments at the column bases, reduced for the effect of axial compression. The designed profile is a W250×167 section with 1.7 and 1.8 times larger cross-sectional area and moment of inertia than the test specimen column section. The nonlinear numerical pushover analysis of the moderately ductile wall demonstrates a yield pattern with the new column that is similar to the one shown in Fig. 5.1(a). The boundary frame members remain entirely elastic, except for the plastic hinges at the column bases. Although the lateral load distribution over the wall height is similar to that used for the limited-ductility wall, increasing the strength and stiffness of the column section according to the proposed moderately ductile capacity design method induces fully yielded infill plates in both stories. This results in a more ductile yield mechanism that is commensurate with the use of a higher value of R_d in determining the seismic design forces.

6.7.3 Four-story SPSW test

The proposed capacity design methods for moderately ductile and ductile walls are applied to a previously-tested four-story SPSW with moment connections at the beam-to-column joints (Driver et al. 1998). The system is shown in Fig. 6.7(b) and described in the Section 5.3.2. The mean measured static yield stresses for the infill plates in panels 1-2, 3, and 4 were 341 MPa, 257 MPa, and 262 MPa, respectively, and for the column flanges it was 308 MPa. The frame member cross-sections meet Class 1 (S16) and Highly Ductile Member (AISC 341) compactness criteria, and they satisfy the stiffness requirements for boundary members of SPSWs.

The design of the columns is studied in Table 6.8, wherein “Test specimen sections” shows the column section used in all stories of the test specimen. The compression column design forces shown in the table section “Capacity design, moderately ductile” are evaluated based on the proposed method. The rigid beam-to-column connections impose concentrated moments and axial forces on the column at each floor level. As such, the design demands above and below each intermediate floor are different, and they are designated by superscript “a” and “b”, respectively, in Table 6.8. P_{MD} and M_{MD} are the capacity design axial compression and moment, respectively, in the column assuming the formation of a plastic moment at the base, reduced for the effect of axial compression, and using the mean measured yield stresses of the infill plates. Fully plastic hinges in each beam are not expected to form in the moderately ductile system (i.e., $\beta = 0.5$), and the stress increase factor (1.1) that accounts for strain hardening is omitted. Since moment-resisting beam-to-column connections are used, which provide redundancy to the moderately ductile system, the columns may be designed using the proposed limited-ductility capacity design method (i.e., 90% of the average axial design force is used for the bottom tier). The mean measured yield stress of the column flanges with no strength reduction factor is used in the column design to facilitate a direct comparison with the test results. The columns are designed as two, two-story tiers, and the profile selected for each tier is shown in Table 6.8. As expected, the nonlinear numerical model of the test specimen developed a yield pattern similar to the limited-ductile wall depicted in Fig. 5.1(b).

The section “Capacity design, ductile” in Table 6.8 demonstrates the column design using the ductile wall provisions. Plastic hinges at both beam ends are considered, along with factor (1.1) to account for strain hardening. P_D and M_D are the design axial compression and moment in the column assuming the formation of a plastic moment at the column base, reduced for the effect of axial compression. The designed column sections are shown in Table 6.8. The nonlinear numerical model of the test specimen with this column cross-section

shows a yield pattern very similar to that shown in Fig. 5.1(a), with the boundary frame remaining elastic except for the plastic hinges that form at the column bases and all beam ends, and with fully yielded infill plates in all stories.

The normalized base shear versus first-story displacement hysteresis curve is shown with a dotted line in Fig. 6.8, where the push condition means lateral loads applied from east to west (see Fig. 6.7(b)). Although the column section used in the test specimen was considerably smaller than those shown in Table 6.8 that were selected through capacity design, the tested wall demonstrated very ductile and robust performance. The effects of geometrical differences, simple beam-to-column connections, and bolted infill plates caused the two-story test specimen to exhibit a smaller elastic stiffness, lower yield strength, and larger yield displacement. However, the normalized hysteresis curves in Fig. 6.8 permit a direct comparison of the two tests and it can be seen that the walls demonstrated remarkably similar behavior in terms of overall ductility and robustness. Both SPSWs reach the maximum shear resistance at a ductility ratio of 5. Although the wall with simple connections reduces the demands on the columns and improves the uniformity of yielding in the infill plates, the wall with rigid connections increases the redundancy of the system and slows the strength degradation after the maximum shear strength is reached.

6.8 Summary and Conclusions

The research presented proposes a reliable and consistent capacity design methodology for SPSW systems, with a three-tier hierarchy of seismic performance levels. The performance level is defined primarily in terms of the redundancy and ductility of the system, which are characterized by the type of beam-to-column connection in the boundary frame and the type of yield mechanism pattern that develops at the ultimate load, respectively. The most critical force-controlled actions in the system are the moments and axial forces in the compression column. As such, specific methods developed for analysis and design of the compression column have been outlined, and their effectiveness

verified against test results and nonlinear numerical simulations.

Capacity design is in general an efficient and highly reliable ultimate-force-based approach, and the procedures presented are expected to produce good estimates of the maximum force demands in the system for different performance levels. However, the capacity design method is inherently incapable of considering the changes in force demands on the system due to non-uniform relative lateral deformations over the wall height. After the wall is designed based on the capacity design method, a pushover analysis can be used to check the overall performance of the system under a proper lateral load distribution.

The infill plates are the primary source of lateral force resistance and energy dissipation in SPSW systems. Therefore, a uniform and fully yielded tension field in the infill plates at each story under lateral loads consistent with the yield mechanism is a key indicator of good performance of the system. Two parameters that help to develop a more uniform tensile yield distribution in the infill plates are a smaller infill plate thickness compared to the size of the boundary members and rotationally-flexible boundary frame connections. When a high level of redundancy is required, a ductile wall with rigid beam-to-column connections is recommended. In such a case, the boundary frames are designed for the capacity forces from the infill plate and they need to remain elastic in all regions except the beam ends and column bases. This will result in flexurally stiff boundary frame members that help to induce uniform yielding in the infill plates. However, where lower redundancy is considered permissible, such as in limited-ductility or moderately ductile walls, simple beam-to-column connections are recommended to reduce the flexural demands on the columns.

While the recommendation for limited-ductility SPSWs of using the nominal yield stress of the infill plate to determine the capacity design forces on the boundary members apparently violates the spirit of capacity design, an examination of the stresses that actually develop adjacent to the critical column at

the yield mechanism load and the resulting net internal member design forces reveals that this approach is actually quite conservative. This is attributed chiefly to the widely-used assumption of uniform “tensile strip” behavior in the panels, wherein the yield point of the material is determined based on uniaxial tensile response and the contribution of the minor principal stress is neglected. The reality of a two-dimensional stress state with a complex field of stresses and attendant principal stress orientations tends to accelerate the yielding, resulting in smaller stresses—particularly horizontal stresses—being applied to the columns. The angle α in the case considered was approximately equal to 39° and 51° adjacent to the beams and compression column, respectively, while the code value is a good approximation for the middle of the infill plate at the mechanism load.

The performance-based capacity design methods proposed provide good estimates of member forces of SPSWs at the mechanism load, although the column moments tend to be quite conservative. The outcome of designs for all three performance levels—ductile, moderately ductile, and limited-ductility—were assessed and confirmed using nonlinear finite element simulations and the results of physical SPSW tests.

Table 6.1 Limited-ductility SPSW example: Geometric parameters and design forces

Floor	H _i m	w mm	Beam section	Column section	α °	μ_i –	F _{si} kN	F _i kN	F _{i,C} kN
4(R)	15.3	4.8	W610×455	WWF500×456	43.1	0.42	210+ 1538	2061	2061
3	11.6	4.8	W250×149		42.4	0.28	1045	1247	3308
2	7.9	6.4	W310×283	WWF500×456	41.8	0.19	712	859	4167
1	4.2	6.4	W250×149		39.8	0.10	379	470	4637
Ave. =					41.8	Σ =	1.00	3884	4637

Table 6.2 Limited-ductility SPSW example: Moment and axial force distributions in compression column

Floor	Capacity design		FE-Nonlinear					
	P_{CD}	M_{CD}	$P_{FE,N}$	$M_{FE,N}$	$P_{FE,N}/$	$M_{FE,N}/$	$P_{FE,N}/$	$M_{FE,N}/$
	kN	kN·m	kN	kN·m	P_r	M_r	P_{CD}	M_{CD}
4(R)	-2943	0	-1850	-94	0.11	0.02	0.63	≈ 1
3	-5638	3569	-5116	2053	0.30	0.54	0.91	0.58
2	-8990	2728	-8810	2109	0.51	0.55	0.98	0.77
1	-14233	2244	-13973	1934	0.81	0.51	0.98	0.86
0	-19464	-490	-18113	-434	1.05	0.11	0.93	0.89

Table 6.3 Limited-ductility SPSW example: Moment and axial force distributions with elastic boundary frame

Floor	Capacity design, pined base	FE-Elastic boundary frame, pinned base					FE-Elastic boundary frame, fixed base			
	$M_{CD,0}$	$P_{FE,E0}$	$M_{FE,E0}$	$P_{FE,E0}/$	$M_{FE,E0}/$	$M_{FE,E0}/$	$P_{FE,EF}$	$M_{FE,EF}$	$P_{FE,EF}/$	$M_{FE,EF}/$
	kN·m	kN	kN·m	P_{CD}	M_{CD}	$M_{CD,0}$	kN	kN·m	P_{CD}	M_{CD}
4(R)	0	-1770	-82	0.60	≈ 1	≈ 1	-2280	-49	0.77	≈ 1
3	3634	-5011	2272	0.89	0.64	0.63	-5491	3293	0.97	0.92
2	2902	-8732	2561	0.97	0.94	0.88	-9148	1699	1.02	0.62
1	2557	-13865	3054	0.97	1.36	1.19	-14332	-611	1.01	-0.27
0	0	-19337	0	0.99	—	≈ 1	-19776	-4200	1.02	8.58

Table 6.4 Moderately ductile SPSW example: Moment and axial force distributions in compression column

Floor	Column section	Capacity design		FE-Nonlinear					
		P_{CD}	M_{CD}	$P_{FE,N}$	$M_{FE,N}$	$P_{FE,N}/$	$M_{FE,N}/$	$P_{FE,N}/$	$M_{FE,N}/$
		kN	kN·m	kN	kN·m	P_r	M_r	P_{CD}	M_{CD}
4(R)	WWF550×620	-3205	0	-2390	-47	0.10	0.01	0.75	≈ 1
3		-6168	3717	-5600	3480	0.23	0.63	0.91	0.94
2		-9833	2336	-9276	2258	0.39	0.41	0.94	0.97
1	WWF550×721	-15592	1239	-14524	1041	0.53	0.17	0.93	0.84
0		-21301	-2516	-19823	-2642	0.73	0.44	0.93	1.05

Table 6.5 Moderately ductile SPSW example: Moment and axial force distributions with elastic boundary frame

Floor	Capacity design, pinned base	FE-Elastic boundary frame, pinned base					FE-Elastic boundary frame, fixed base			
	$M_{CD,0}$	$P_{FE,E0}$	$M_{FE,E0}$	$P_{FE,E0}/$	$M_{FE,E0}/$	$M_{FE,E0}/$			$P_{FE,EF}/$	$M_{FE,EF}/$
	kN·m	kN	kN·m	P_{CD}	M_{CD}	$M_{CD,0}$	$P_{FE,EF}$	$M_{FE,EF}$	P_{CD}	M_{CD}
4(R)	0	-2059	-51	0.64	≈ 1	≈ 1	-2389	-58	0.75	≈ 1
3	4051	-5296	2708	0.86	0.73	0.67	-5582	3198	0.91	0.86
2	3231	-8998	3031	0.92	1.30	0.94	-9237	1083	0.94	0.46
1	2850	-14171	2901	0.91	2.34	1.02	-14389	-1992	0.92	-1.61
0	0	-19633	0	0.92	—	≈ 1	-19904	-5988	0.93	2.38

Table 6.6 Mean infill plate stress orientations and values adjacent to compression column of limited-ductility SPSW

Story	α	α	α	σ_1	σ_2	$\psi =$	$\sigma_1/$	$\sigma_H,$	$\tau,$	$\sigma_H,$	$\tau,$	$\sigma^*_H,$	$\tau^*,$
	S16	FE_Mid	FE_Col	FE	FE	$ \sigma_2/\sigma_1 $	$R_y F_{yw}$	FE	FE	CD	CD	CD	CD
	°	°	°	MPa	MPa	—	—	MPa	MPa	MPa	MPa	MPa	MPa
4	43.1	42.9	49.2	296	-136	0.490	0.760	111	212	164	175	120	213
3	42.4	41.8	50.1	311	-119	0.389	0.806	134	211	159	174	139	211
2	41.8	45.3	51.3	315	-111	0.374	0.813	148	207	156	174	143	210
1	39.8	44.3	50.9	306	-123	0.435	0.785	134	209	143	172	130	212

Table 6.7 Column design for two-story SPSW specimen for different seismic performance levels

Floor	Test specimen section	Capacity design, limited-ductility					Capacity design, moderately ductile				
		P_{LD} kN	M_{LD} kN·m	Section	A_{LD}/A	$I_{x,LD}/I_x$	P_{MD} kN	M_{MD} kN·m	Section	A_{MD}/A	$I_{x,MD}/I_x$
2(R)		-1745	0				-1861	0			
1	W250×101	-2878	481	W250×115	1.13	1.15	-3105	379	W250×167	1.65	1.83
0		-4370	-22				-4744	-439			

Table 6.8 Column design for four-story SPSW specimen for different seismic performance levels

Floor	Test specimen sections	Capacity design, moderately ductile					Capacity design, ductile				
		P_{MD} kN	M_{MD} kN·m	Section	A_{MD}/A	$I_{x,MD}/I_x$	P_D kN	M_D kN·m	Section	A_D/A	$I_{x,D}/I_x$
4(R)		-1485	356				-2226	862			
3 ^a	W310×118	-2061	825	W310×179	1.52	1.62	-2859	1491	W310×313	2.66	3.26
3 ^b		-2321	972				-3257	1847			
2 ^a		-2981	609				-3983	1072			
2 ^b	W310×118	-3860	756	W310×202	1.72	1.89	-5063	1427	W310×313	2.66	3.26
1 ^a		-5062	665				-6382	749			
1 ^b		-5384	812				-6849	1105			
0		-6634	-260				-8221	-841			

^a Immediately above floor level^b Immediately below floor level

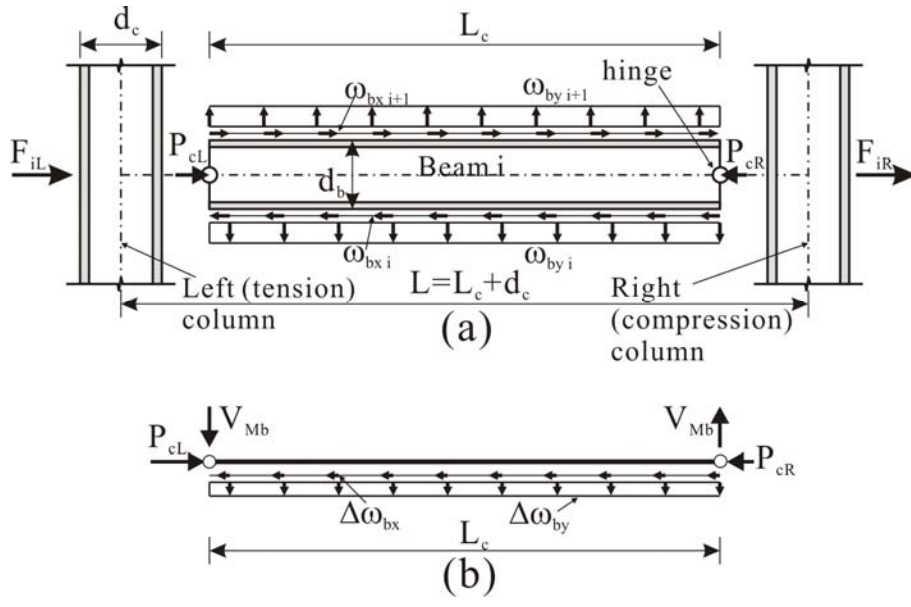


Fig. 6.1 Intermediate beam of limited-ductility SPSW system: (a) Applied forces, (b) Net design forces applied to beam centerline

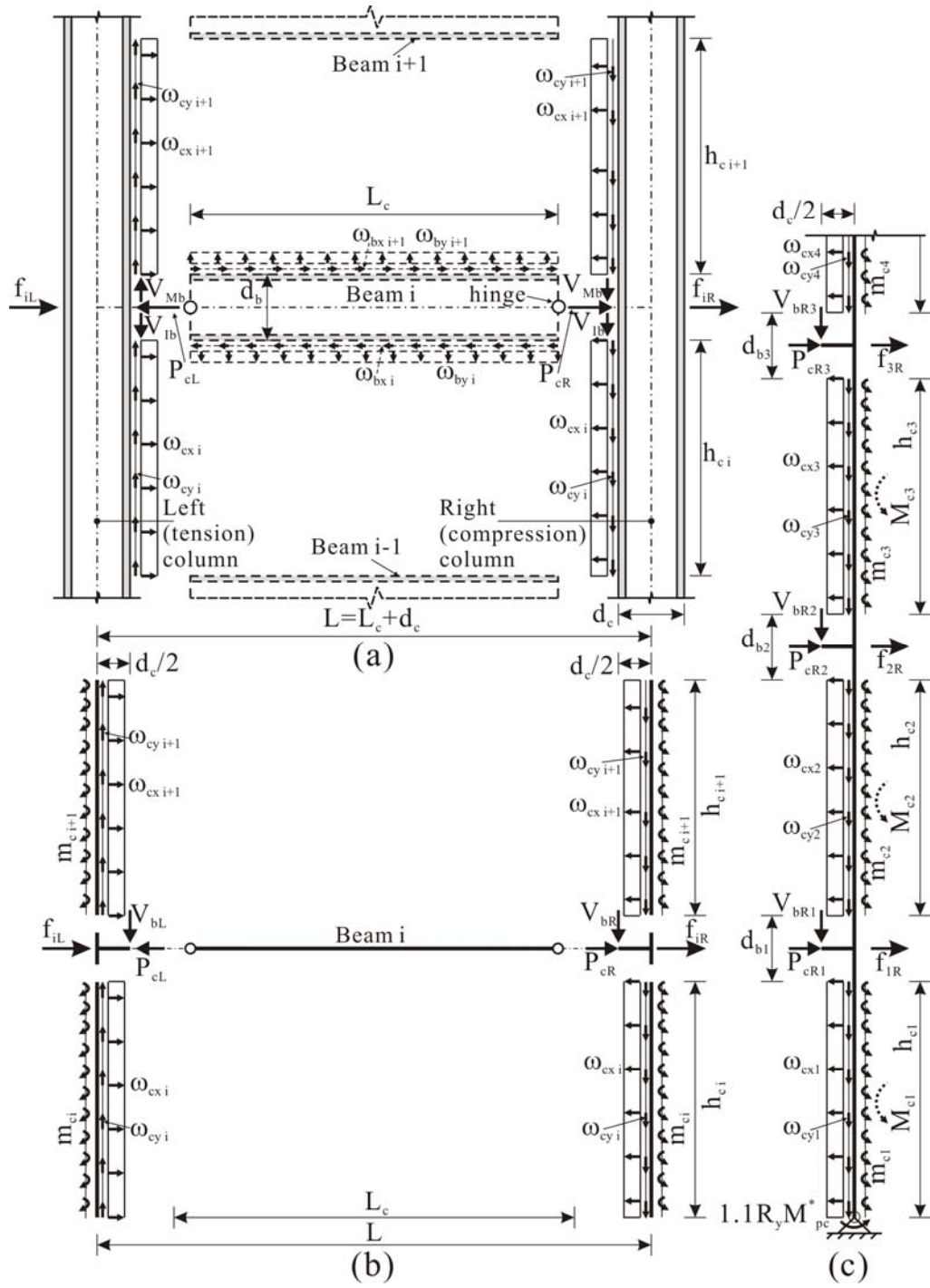


Fig. 6.2 Columns of limited-ductility SPSW system: (a) Applied forces, (b) Net design forces applied to column centerline, (c) Free body diagram of compression column

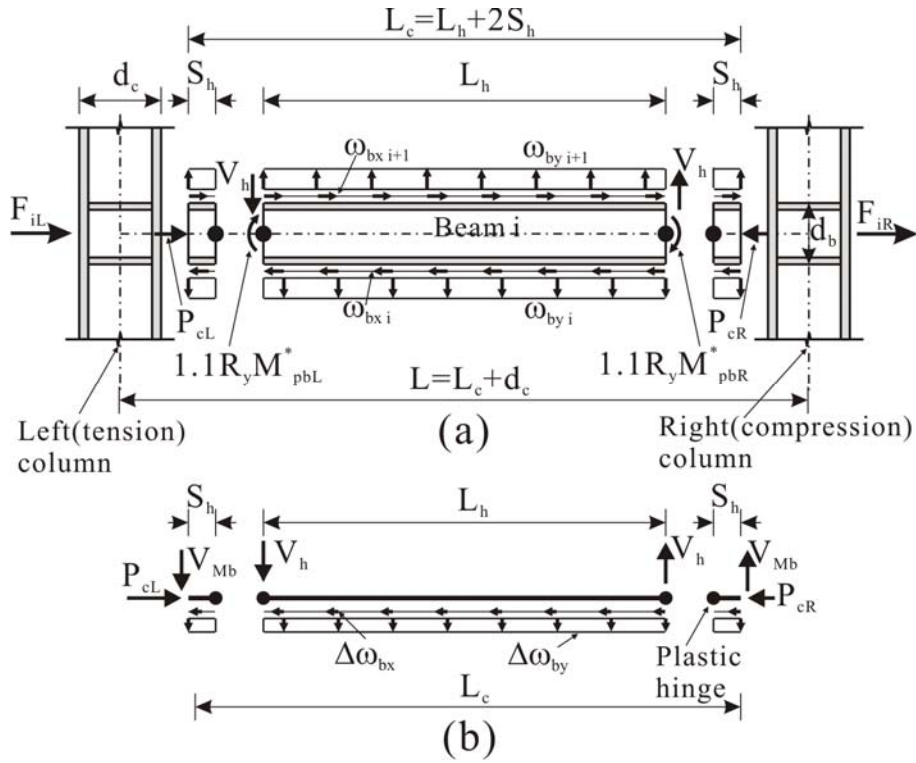


Fig. 6.3 Intermediate beam of ductile SPSW system: (a) Applied forces, (b) Net design forces applied to beam centerline

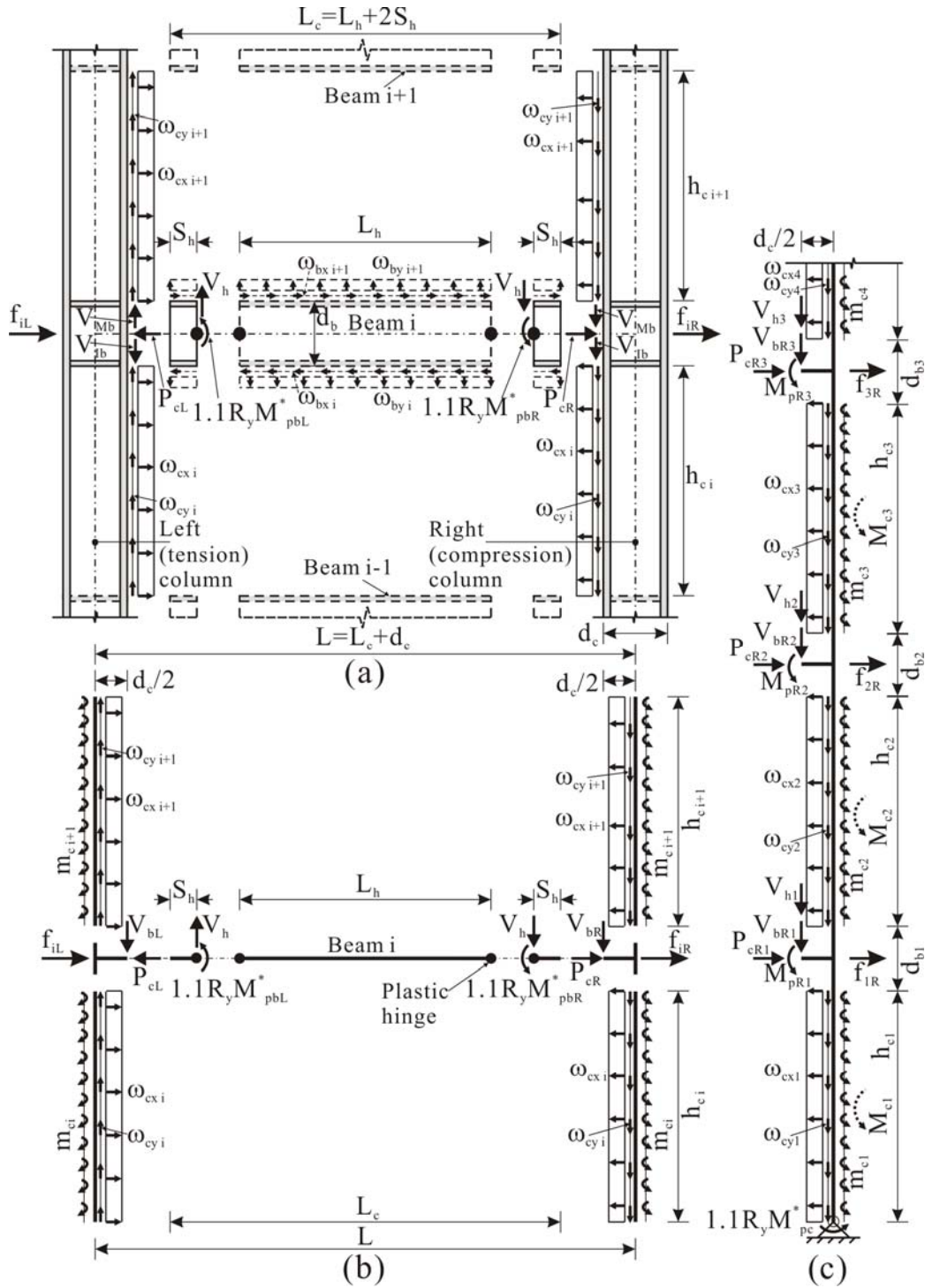


Fig. 6.4 Columns of ductile SPSW system: (a) Applied forces, (b) Net design forces applied to column centerline, (c) Free body diagram of compression column

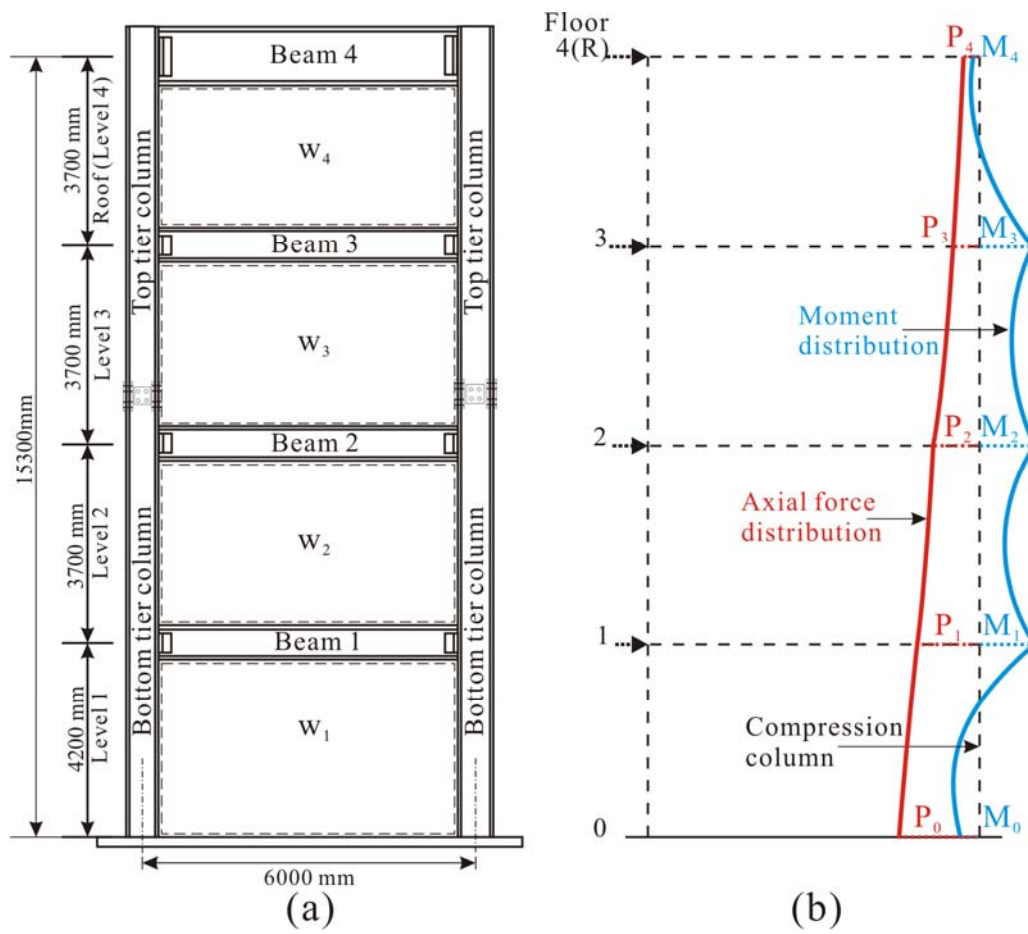


Fig. 6.5 (a) Elevation of example SPSW, (b) Axial force and moment distributions in compression column of limited-ductility wall

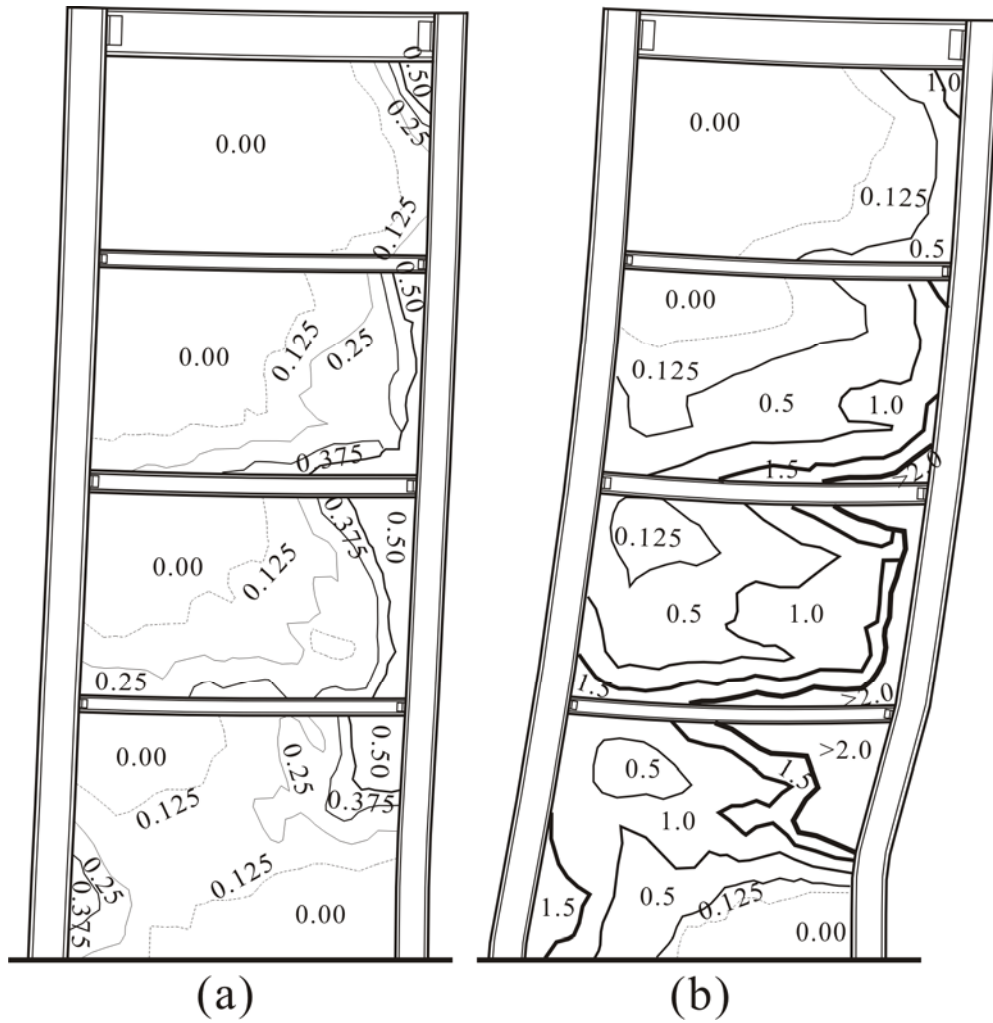


Fig. 6.6 Infill plate yield patterns in limited-ductility SPSW: (a) Effective plastic strain (%) at $R_d = 2$, (b) Effective plastic strain (%) at yield mechanism, (c) Horizontal normal stress (MPa) at yield mechanism

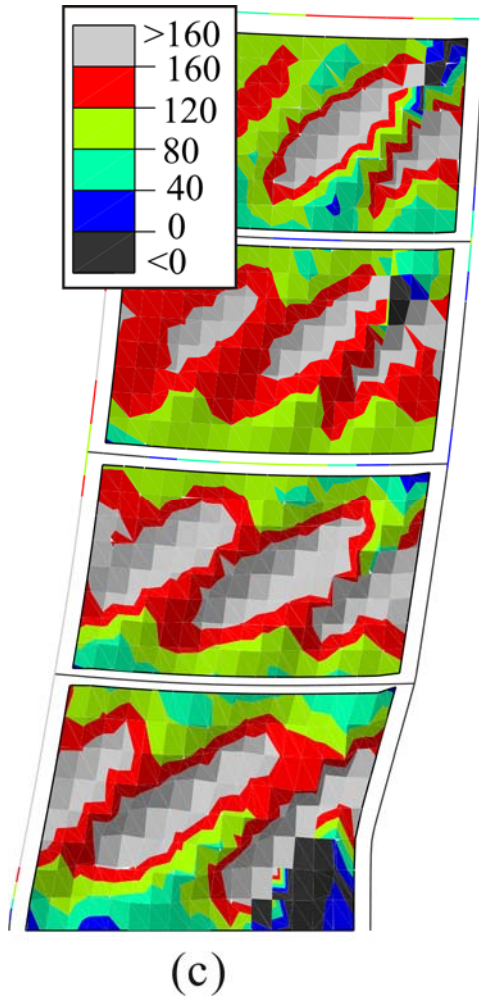


Fig. 6.6 Infill plate yield patterns in limited-ductility SPSW: (a) Effective plastic strain (%) at $R_d = 2$, (b) Effective plastic strain (%) at yield mechanism, (c) Horizontal normal stress (MPa) at yield mechanism (cont.)

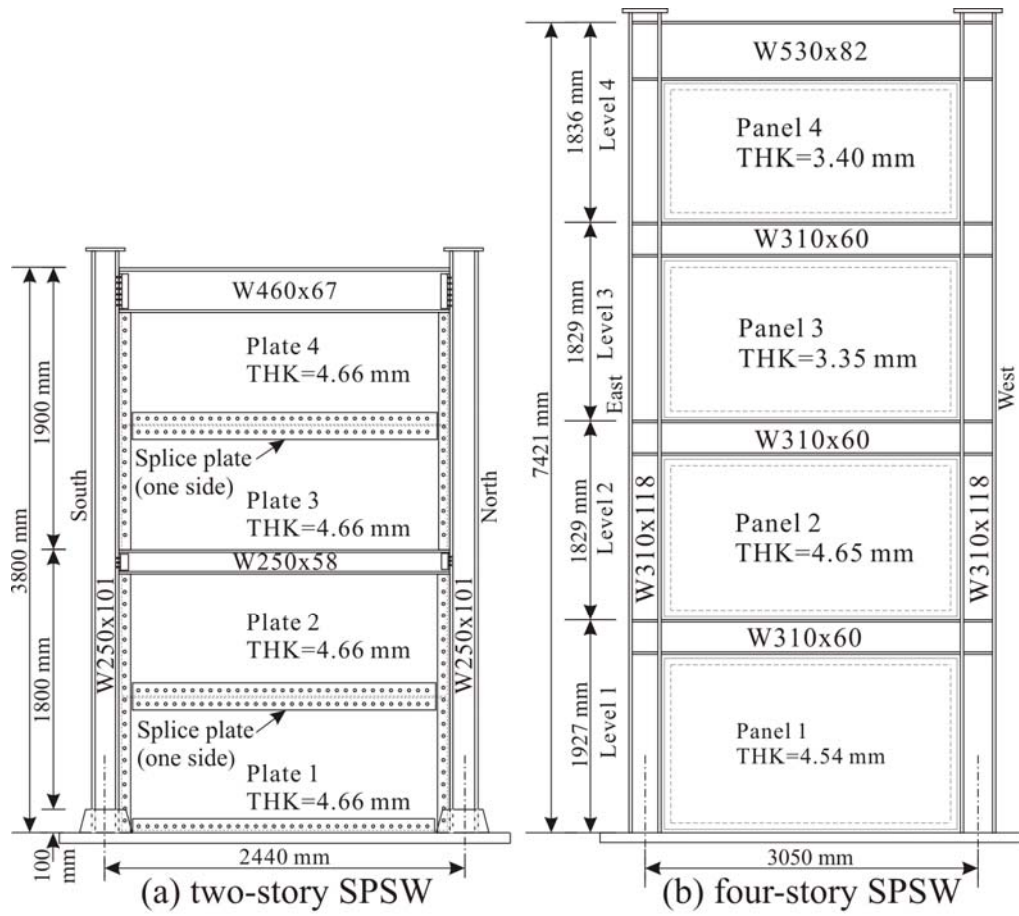


Fig. 6.7 Test specimen elevations: (a) Two-story SPSW (Moghimi and Driver 2013a), (b) Four-story SPSW (Driver et al. 1998)

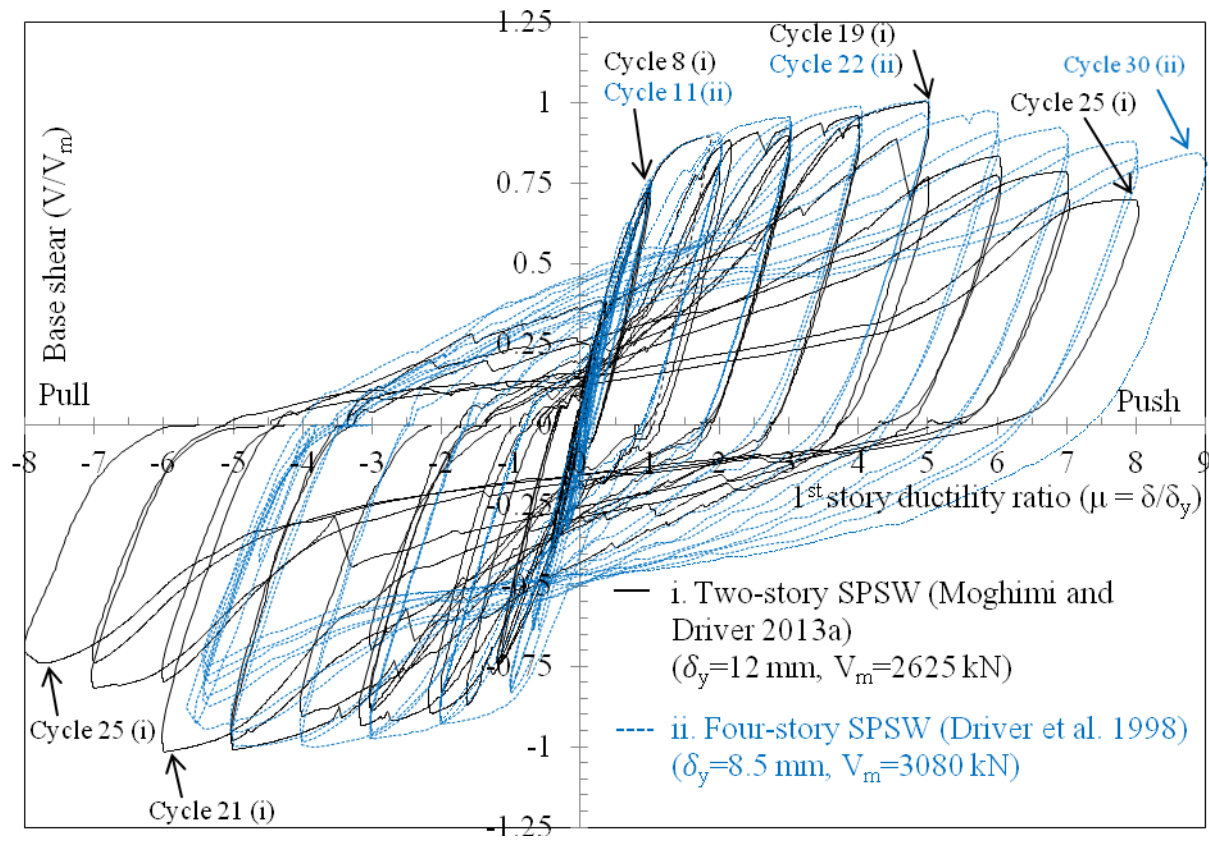


Fig. 6.8 Comparison of normalized base shear versus first-story lateral displacement for two-story and four-story SPSW tests

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7. PERFORMANCE ASSESSMENT OF SPSW SYSTEM UNDER ACCIDENTAL BLAST LOADS⁶

7.1 Introduction

Steel plate shear walls (SPSW) have been advanced through the last two decades primarily based on research that focuses on improving the performance of the system under severe earthquake loading. Previous research has shown that the system possesses exceptional ductility and lateral force resistance, with a high level of energy dissipation capacity without degrading under cyclic loading. As such, the system has reached a stage where design standards, such as Canadian Standard S16 (CSA 2009), have assigned it the highest ductility-related and overstrength-related force modification factors. Although the system is undeniably well-suited for high seismic regions, and similar properties are advantageous for resisting other types of dynamic loading such as blast, their potential applications as protective structures have received little attention.

Most of the structures in industrial plants are made up of steel systems, which normally have rapid erection times and tend to be more flexible than concrete construction in terms of future expansion and site rearrangement. Therefore, having a reliable protective structural steel option available would be advantageous. Through the process of site planning, protective structures in industrial plants are sited at a suitable distance from process equipment and any source of release of flammable and explosive material. As such, the blast loads that need to be considered in the design of industrial structures tend to be “accidental” far-range (low pressure) detonations and are less detrimental for slender steel members than near-range (high pressure) explosions. Protective structures are prone to localized damage and failures under blast loading, but their overall integrity must not be compromised if they are to fulfill their intended

⁶ A version of this chapter will be submitted for publication in the *International Journal of Impact Engineering*, ELSEVIER.

function. To limit the damage and improve the reliability of the system, a high level of redundancy is beneficial for blast-resistant systems to ensure the availability of alternative load paths. As such, the SPSW, which is a continuous system with a high level of ductility capacity, is potentially a good candidate as a primary component of protective structures in industrial plants.

This research is an exploration of the inherent qualities of conventional SPSWs for use as protective structures, with the additional goal of identifying where modifications are required for optimal performance in this new application. This is achieved through the development of pressure–impulse ($P-I$) diagrams. First, the $P-I$ curve is described in detail and is generalized (normalized) by transforming a wall system into a single-degree-of-freedom (SDOF) system. A comprehensive numerical model that is able to capture all critical aspects of the blast response is developed. The in-plane and out-of-plane responses are investigated separately. $P-I$ diagrams for two different-size walls have been developed and normalized. They are then converted to charge weight–standoff distance curves. The results show that a properly-designed and detailed SPSW may indeed be a viable protective system for accidental blast in industrial plants such as petrochemical facilities.

7.2 Literature Review

7.2.1. Performance criteria

The maximum dynamic responses of the structural components intended to resist the blast loading need to be limited against the desired blast protection levels or blast design objectives. These response limits are typically called “performance criteria,” and are defined in blast design guidelines. Generally, when components are under large shear or compressive forces, the response limits are small and barely reach the yield limits, while large deformation limit values are permitted for components loaded mainly in flexure. Additionally, other factors, such as the siting distance from the blast source, occupancy of the building, and the importance of the equipment protected by the building, affect the blast design

requirements.

The American Society of Civil Engineers' document for blast design of petrochemical facilities (ASCE 2011b) defines the allowable deformation of individual components based on the desired protection level and type of component for different construction material types. Three performance levels, or damage levels—namely, low, moderate, and high response ranges—have been considered. The performance levels are conceptually similar to the immediate occupancy, life safety, and collapse prevention performance levels, respectively, used in performance-based seismic design (NEHRP 2004). The low response range corresponds to a high degree of blast protection with only localized damage. The medium and high response ranges represent widespread damage and loss of structural integrity, respectively. Two dimensionless response parameters—namely, ductility ratio, μ , and support (chord) rotation in degrees, θ —have been defined at each performance level. The ductility ratio is the ratio of the maximum component deformation to its yield deformation, which is a measure of the capability of the component to experience inelastic deformation with no significant capacity loss. The support rotation is the ratio of the maximum component deflection to the distance between the support and the point on the component where this deflection is measured, and is a measure of both rotational ductility at the support and the degree of potential instability in the member. Building performance criteria are also defined according to the inter-story drift ratio. For example, the lateral drift ratios of moment-resisting structural steel frames are limited to 2.0%, 2.85%, and 4.0% for low, medium, and high response ranges, respectively. The response limits have been elaborated from the first edition of the document published in 1997, and the changes have been described in detail by Oswald (2008).

The response limit for an individual structural steel component based on various design guidelines are shown in Table 7.1. The column “LP” shows the component Level of Protection, where “H”, “M”, “L”, and “VL” represent High, Medium,

Low, and Very Low blast protection levels, respectively. The column “Resp. Par.” shows the different response parameters at each protection level, which are ductility ratio, μ , and support chord rotation, $\theta(^{\circ})$. Because of different definitions of damage or performance level in the various design guides, direct comparisons of the response limits shown may not be absolutely consistent; the table is intended for general comparisons only.

For the response limits proposed by the ASCE blast design manual (ASCE 2011b), the performance levels have been identified in table column “Perf. Level”, as “Low Resp.”, “Med. Resp.”, and High Resp.”, which correspond with the high, medium, and low levels of protection, respectively. The manual provides defined response limits for different hot-rolled steel components, including compact secondary flexural members such as beams, girts, and purlins (column “BM Sec.”), primary frame members with and without significant compression (column “Prim. Mem.”), and plates (column “PL”). Significant compression is defined as a force larger than 20% of the dynamic axial compressive capacity of the member, where the axial force is evaluated from a capacity method based on the ultimate resistance of the supported members exposed to the blast loads.

The UFC 3-340-02 document (DoD 2008) presents methods of design for protective construction against accidental explosion of high-explosive (military mainly) materials. Two protection levels have been considered for blast design. Structures designed to protect personnel against accidental blast are classified as Category 1, while structures provided to protect equipment are designated Category 2. The response criteria proposed by this document are shown in Table 7.1, where the column “Prot. Cat.” shows the protection categories and columns “BM” and “PL” show the response limits for beams and plates, respectively. Categories 1 and 2 (“Cat 1” and “Cat 2”, respectively, in the table) correspond to medium and low protection levels, respectively.

The PDC TR-06-08 document (PDC 2008) defines response criteria against explosive terrorist threats in terms of ductility ratio and support rotation for four different component damage levels, including Superficial, Moderate, Heavy, and Hazardous, as shown in Table 7.1. Different structural component types and characteristics have been considered for both primary and secondary elements. Table column “Comp. Dam.” shows the component damage levels, where columns “BM”, “CPR” and “PL” show response limits for primary compact beam elements, compression members, and plates, respectively. (The document also suggests response limits for non-compact, secondary, and non-structural components, which are not shown in the table.) The moderate and heavy component damage levels correspond roughly to the medium and high response performance levels, respectively, in the ASCE petrochemical design guidelines (ASCE 2011b). However, the superficial damage level represents more conservative design limits (i.e., lighter damage levels) than the low response performance level in the ASCE manual (ASCE 2011b), but both can be classified as high protection levels. Since the response limits have been developed primarily based on static test data, they are generally more conservative in comparison with other criteria (Oswald 2008). In order to enable the user to assess the overall building blast protection, the document (PDC 2008) also defines the hazardous damage level, corresponding to a very low level of protection, although it is not generally selected as a blast design protection level. The ASCE/SEI blast design document (ASCE 2011a) has adopted the same response limits as PDC TR-06-08 (PDC 2008).

The ASCE document on structural design for physical security (ASCE 1999) defines the response limits for light, moderate, and severe damage, and in Table 7.1 in the column “Dam. Level” these are indicated as “Light Dam.”, “Mod. Dam.”, and “Severe Dam.”, respectively. The columns “BM Flex.” and “BM Shr.” describe response limits for beams subjected to flexural/membrane and shear actions, respectively, and column “CPR” provides the limits for columns under compression. Since the response limit values in this document are presented

in such a way that they are not directly comparable with those in the other manuals, they are transformed accordingly as follows. The beam response limits have been defined in terms of the ratio of the centerline deflection to the span (δ/L). Assuming a symmetric response, the $2\delta/L$ value is the tangent of the support rotations. By doubling the limits and taking the inverse tangent (which is very close to the angle in radians for angles in the ranges under consideration) the support rotation is calculated and shown in Table 7.1. Also, the ratio of shortening of the column to its original height is provided for the compression-column response limit. This ratio is equal to the average compressive strain in the column. Assuming the application of steel with a yield stress of 400 MPa, the yield strain would be 0.2%. By dividing the column limit ratios by 0.2%, the strain ductility ratio is calculated and presented in the table. Compared with other design guides, this document suggests larger response limit values because they represent structural damage levels observed in experimental/numerical results. As such, these values are suggested for post-event assessment and should not be used for design.

7.2.2. *P–I curves*

In blast-resistant design, the first step is to evaluate the blast load history applied to the system. Next, the response history of each component is estimated, and then the maximum response is compared with corresponding response limits. The application of appropriate *P–I* curves allow the last two steps to be combined into one simple check. Abrahamson and Lindberg (1976) illustrated the characteristics of *P–I* curves for elastic and rigid-plastic undamped SDOF systems subjected to idealized rectangular, triangular, and exponential-shaped pulse loads with zero rise times. The results showed that the pulse shape has virtually no effect on the impulsive and quasi-static loading regimes, but it changes the response in the dynamic loading realm. Iso-damage curves were produced for uniformly-loaded, rigid-plastic beams and plates, and for dynamic buckling of cylindrical shells under uniform lateral loads.

Baker et al. (1983) describe three response regimes for undamped linear elastic SDOF systems subjected to an exponentially decaying load, and the results were compared with an analogous rigid-plastic system. The $P-I$ curves were produced for non-ideal explosions applied to an undamped elastic SDOF system. Two different cases for blast waves with finite rise time (e.g., confined gas or dust explosion) and blast waves with zero rise times and a negative phase (e.g., pressure vessel burst) were studied. The impulsive and quasi-static asymptotes were developed for different systems by means of energy solutions. It was shown that the dynamic response associated with any iso-response curve depends only on the maximum load and system stiffness on the quasi-static loading asymptote and only on the impulse and system mass and stiffness on the impulsive loading asymptote. In the latter case, the authors claim that any peak load and duration combination with equal impulse results in the same dynamic response.

Krauthammer et al. (2008) describe three different search algorithms to derive $P-I$ diagrams numerically. The methods are compared with closed-form solutions for undamped linear elastic SDOF systems subjected to rectangular, triangular, and exponential pulse loading types with instantaneous rise times, and good agreement was reported.

7.2.3. SPSW systems

SPSW systems have been the subject of very little blast research in the past. The out-of-plane blast resistances of two 40%-scale single-story SPSW specimens designed for seismic loads were studied by Warn and Bruneau (2009). The first specimen was subjected to a small charge weight in close proximity, with a scaled distance of Z^* (specific values were not reported). After the test, significant residual inelastic deformations were observed in the infill plate, but it remained attached to the boundary frame. The base beam sustained larger inelastic deformations than the top beam, with cracks forming in the weld connecting the top flange of the base beam to the column flanges. The second specimen was subjected to a larger charge with a longer standoff distance compared with the

first specimen, such that the scaled distance was $1.66Z^*$. Although this specimen was subjected to a smaller blast load effect due to the larger scaled distance, the infill plate unzipped around three sides of the panel because of the failure of the weld connecting the infill plate to the fish plates. The out-of-plane resistance of the wall was estimated using yield line theory and an approximate plastic analysis method. The former method underestimates the out-of-plane resistance of the wall greatly, while the latter one overestimates the strength of the system.

Moghimi and Driver (2010) studied the overall performance of a SPSW system subjected to in-plane and out-of-plane blast load using numerical methods. The blast loads were shock waves with an appropriate duration for the design of petrochemical facilities. Local and global damage indices were suggested for the system. Dynamic performances of the system in both blast load directions were studied. Blast resistance capacities were assessed according to both total absorbed strain energy and maximum structural displacement. The results showed that SPSWs could be a competent system for protective structures in industrial plants.

7.3. P–I Diagrams

7.3.1. P–I diagram versus response spectrum curve

For designing systems subjected to dynamic loading, the maximum response, rather than the response time history, is normally of most interest. As such, response spectra or dynamic load factors—the ratio of maximum dynamic to static response as a function of dynamic load duration—have been used in dynamic design of structures for several decades. As shown by Baker et al. (1983), the typical response spectrum diagram for an undamped linear SDOF system subjected to an exponentially decaying pulse load with instantaneous rise time and infinite duration would be similar to Fig. 7.1(a). The system has a mass and stiffness of m and k , respectively, with a resulting natural angular frequency of $\omega_n = \sqrt{k/m} = 2\pi/T_n$, where T_n is the natural period of vibration of the system. As shown in Fig. 7.1(a), the load, which represents an air blast wave, is defined by the equation $p(t) = P_0 \exp(-t/T)$, and the impulse of the load would be

$I = P_0 T$, where T can be described as an equivalent loading duration for a rectangular pulse with the same peak value as the actual decaying load. For the described system, the ordinate of the response spectrum curve is the ratio of maximum dynamic displacement (X_{max}) to the maximum static displacement ($x_s = P_0/k$), which is $y = X_{max}/(P_0/k)$, and the abscissa is the scaled time, or the ratio of equivalent loading duration to the natural period of vibration, which (scaling by the constant 2π for convenience) is $x = 2\pi T/T_n = \omega_n T$.

The response spectrum curve shown in Fig. 7.1(a) has two asymptotes. The first is an inclined asymptote ($y = \alpha x$) that passes through the origin with the slope $\alpha = 1$ and the second is a horizontal asymptote ($y = \beta$) with a y-intercept of $\beta = 2$. As described by Baker et al. (1983), the diagram consists of three loading regimes: impulsive ($x = \omega_n T < 0.4$), dynamic ($0.4 \leq x = \omega_n T \leq 40$), and quasi-static ($x = \omega_n T > 40$). The curve provides useful information in the dynamic and quasi-static domains, while it suppresses the system response in the impulsive loading regime where the duration of pulse load is short relative to the response time of the system (x converges to zero).

Since blast design deals mostly with the impulsive loading regime, the response spectrum is normally not an appropriate tool. As such, the response spectrum needs to be transformed into a different coordinate system for use in blast design, which is called a peak load (or peak pressure)–impulse (P – I) diagram. A typical P – I curve is shown in Fig. 7.1(b), which has a horizontal and a vertical asymptote, both with unity intercepts. Two mappings are required to derive Fig. 7.1(b) from Fig. 7.1(a), and Figs. 7.1(c) and (d) show the ordinate and abscissa transformations, respectively. The ordinate is transformed such that the impulsive loading regime response, which passes through the origin in the original response spectrum curve, is retrieved. While the transformation $1/y$ does the job, in order to have the horizontal (quasi-static loading regime) asymptote with a unity intercept, the transformation $Y = (1/y) \beta$, as shown in Fig. 7.1(c), can be used. As defined in Fig. 7.1(b), the new ordinate, $Y = P_0/[(kX_{max})/2]$, is a normalized or

nondimensional load for a linear SDOF system where the numerator is the maximum pulse load and the denominator is the average equivalent elastic static force due to the maximum dynamic deformation.

Fig. 7.1(c) still has an inherent restriction, since when Y (the dimensionless load) is a large value in the impulsive loading regime, x (the period ratio) converges to zero for all different systems and applied pulse loads. In order to change this characteristic of the curve, the inclined asymptote in the response spectrum curve in Fig. 7.1(a) needs to be mapped to a vertical asymptote. Since the original response spectrum curve has a linear inclined asymptote, the mapping x/y for the abscissa brings the inclined asymptote to a vertical asymptote with $1/\alpha$ intercept. As such, the mapping $X = (x/y) \alpha$ maps the inclined asymptote to the vertical (impulsive loading regime) asymptote with a unity intercept, as shown in Fig. 7.1(d). As defined in Fig. 7.1(b), the new abscissa, $X = I / [(X_{max} \omega_n) m]$, is a normalized or nondimensional impulse, where the numerator is the impulse, or area under the pulse load, and the denominator is an equivalent impulse or change in momentum per unit time for an elastic SDOF system under free vibration with a displacement amplitude of X_{max} .

The P – I and response spectrum curves encompass the same information and are essentially different representations of the same response. However, they emphasize different dynamic response features. The response spectrum curve displays the (nondimensional) dynamic response as a function of the (nondimensional) dynamic load duration. In other words, it shows how the dynamic response is affected by the load duration. The curve has two linear asymptotes. The quasi-static loading (horizontal) asymptote shows that any change in dynamic load duration within this regime does not have any effect on the maximum dynamic response. The P – I curve, on the other hand, is the locus of equivalent dynamic loads—a combination of maximum dynamic load (P) and associated impulse (I)—that produce the same maximum dynamic response in the system. In other words, for any selected response limit in the system, the curve

represents an iso-response as a function of maximum dynamic load and its corresponding impulse. The $P-I$ curve has both linear vertical (impulsive) and horizontal (quasi-static) asymptotes. The critical parameter in the impulsive loading regime is the impulse value. That is, any change in just the maximum load may not cause the response to deviate from the iso-response curve, while a change in only the impulse necessarily does. In the same way, the critical parameter in the quasi-static loading regime asymptote is the maximum load.

Historically, $P-I$ curves have been developed for damage assessment of structures under severe blast and transient dynamic loads to define the peak load and impulse combinations that result in a specific damage level. As such, they are commonly known as iso-damage curves. However, in this research the terms $P-I$ curve and iso-response curve are interchangeably used, instead of iso-damage curve, since they better convey the concept of the curve.

7.3.2. Trial-and-error approach

A $P-I$ curve can be developed for a simple linear elastic SDOF system subjected to well-defined dynamic loads by analytical methods or energy conservation principles. The former method, as represented by Fig. 7.1(b), can even result in a closed-form solution for simple cases, while the latter method is mainly used to derive the solution in the asymptotic regions (Baker et al. 1983). However, both methods are applicable to simple and well-defined cases. The numerical solution is the only reasonable means of deriving the iso-response curves for a general nonlinear system subjected to complex loading functions.

The $P-I$ curve corresponding to a given SPSW and a selected response limit is generated by curve-fitting on a sufficient number of points computed by numerical analysis. Each point on the curve is recovered by a series of numerical analyses of the system and a trial-and-error method (Krauthammer et al. 2008). Fig. 7.2 demonstrates the method to find two arbitrary points (I_i, P_i) and (I_q, P_q) on the impulsive and quasi-static asymptotes, respectively. By keeping the pressure

P_i constant and changing the impulse ($I_{i,j}$), where the index j represents the number of the iteration that varies from 0 to n , the threshold impulse in the impulsive asymptote that satisfies the response limit ($I_{i,n} = I_i$) is found. Similarly, for the quasi-static asymptote, the impulse I_q is kept constant and the pressure ($P_{q,j}$) changes until it satisfies the response limit criterion ($P_{q,n} = P_q$). As such, each point ($I_{i,j}, P_i$) or ($I_q, P_{q,j}$) in the figure represents the results from a single dynamic analysis. Among them, the two points (I_i, P_i) and (I_q, P_q) belong to the iso-response curve and they indicate two combinations of impulse and peak pressure that cause the maximum dynamic response of the system to just reach the selected response limit. In the dynamic response domain, either of the methods mentioned (i.e., constant peak pressure or constant impulse) can be implemented.

7.3.3. Nondimensional P – I diagram

Although iso-response curves are powerful and provide an effective blast design tool, they are inherently tied to the mechanical and dynamic properties of the system under consideration. Therefore, for any given response limit, a new P – I curve should be developed if the system is modified, which requires many nonlinear finite element analyses or blast tests. Since each P – I curve deals with one specific response, the system can be transformed into an equivalent single-degree-of-freedom (ESDOF) system for the degree-of-freedom (DOF) associated with the response limit (Biggs 1964). With the help of ESDOF system properties, a P – I curve developed for a given response limit and specific SPSW system can be normalized and used for different walls.

7.3.3.1 Nondimensional load/pressure

Considering an elasto-plastic SDOF system under a dynamic load with an amplitude of F , the external work is equal to $F \delta_m$ and the strain energy is equal to $V_y (\delta_m - 0.5 \delta_y) = 0.5 V_y \delta_y (2\mu - 1)$, where δ_y and V_y are the yield displacement and yield resistance (elastic limit) of the system with an elastic stiffness $k = V_y/\delta_y$, δ_m is the maximum displacement of the system, and $\mu = \delta_m/\delta_y$ is ductility ratio.

The ideal dynamic load (with an instantaneous rise time with infinite duration), P_0 , which produces the maximum displacement δ_{m0} or ductility μ_0 in the system can be calculated using conservation of energy, as follows:

$$P_0 = 0.5 V_y (2 - 1/\mu_0) \quad (7.1)$$

P_0 would be the quasi-static asymptote of the iso-response curve for the maximum response of $\delta_{m0} = \mu_0 \delta_y$ in the elasto-plastic SDOF system. The blast load applied to the system can be normalized with such a load. However, Eq. (7.1) is valid only for a SDOF system. For a multi-degree-of-freedom (MDOF) system such as a SPSW, it must be transformed into an equivalent SDOF system through transformation factors (Biggs 1964). As such, considering Eq. (7.1) the following equation for the nondimensional blast load on the system is used:

$$P_e/P_{e0} = K_L P/(K_L [0.5 V_y (2 - 1/\mu_0)]) = P/[0.5 V_y (2 - 1/\mu_0)] = P/P_0 \quad (7.2)$$

where the subscript e represents the equivalent value for the distributed system (real wall), P is the resultant force of the total applied blast pressure, and K_L is the load transformation factor. At the verge of yielding ($\mu_0 = 1$), the strain energy would be equal to $0.5 k \delta_y^2$, the ideal force that produces the yield displacement in the system would be equal to $P_0 = 0.5 V_y$, and the nondimensional force would be equal to P/P_0 (Eq. (7.2)).

7.3.3.2. Nondimensional Impulse

The ideal impulse (with zero duration), I_0 , that produces the yield displacement in an elasto-plastic SDOF system can be found by using conservation of energy. By equating the above-defined strain energy with the kinetic energy, which is equal to $I_0^2/(2m)$ in the elastic region for a SDOF system with mass m :

$$\begin{aligned} I_0 &= \sqrt{(k m) \delta_y} \sqrt{(2\mu_0 - 1)} = m \omega \delta_y \sqrt{(2\mu_0 - 1)} = \\ &\sqrt{(V_y m \delta_y)} \sqrt{(2\mu_0 - 1)} = (V_y/\omega) \sqrt{(2\mu_0 - 1)} \end{aligned} \quad (7.3)$$

where $\omega = \sqrt{k/m}$ is the natural frequency of the system. I_0 would be the impulsive asymptote of the iso-response curve for the maximum response of $\delta_{m0} = \mu_0 \delta_y$. Similarly, Eq. (7.3) is for a SDOF system, and for the distributed system the transformation factors should be applied to transform the system into an ESDOF:

$$\begin{aligned} I_{e0} &= \sqrt{(k_e m_e)} \delta_y \sqrt{(2\mu_0 - 1)} = \\ &\sqrt{(K_M K_L)} \sqrt{(k m)} \delta_y \sqrt{(2\mu_0 - 1)} = \sqrt{(K_M K_L)} I_0 \end{aligned} \quad (7.4)$$

where K_M is the mass transformation factor. Also, the equivalent applied impulse would be equal to:

$$I_e = 0.5 P_e t_d = K_L (0.5 P t_d) = K_L I \quad (7.5)$$

where t_d is the triangular pulse load duration. The nondimensional impulse would be as follows, using Eqs. (7.4) and (7.5):

$$\begin{aligned} I_e/I_{e0} &= K_L (0.5 P t_d) / [\sqrt{(K_M K_L)} I_0] = \\ &K_L I / [\sqrt{(K_M K_L)} I_0] = \sqrt{(K_L/K_M)} I/I_0 = 1/\sqrt{(K_{LM})} I/I_0 \end{aligned} \quad (7.6)$$

where I is the total impulse of the blast load and $K_{LM} = K_M/K_L$ is the load–mass transformation factor (Biggs 1964). At the end of the elastic region ($\mu_0 = 1$), the strain energy would be equal to $I_0^2/(2m)$ and the ideal impulse that produces the yield displacement in the system would be equal to $I_0 = \sqrt{(k m)} \delta_y$ and for a MDOF it would be $I_{e0} = \sqrt{(K_M K_L)} I_0$ and the nondimensional impulse would be equal to $1/\sqrt{(K_{LM})} I/I_0$ (Eq. (7.6)).

Eqs. (7.2) and (7.6) define the parameters for the nondimensional P – I curves.

7.4. Material Model for Blast Analysis of SPSWs

To study the performance of the SPSW system under accidental blast loading, a previously-tested SPSW designed for seismic applications has been selected. The system is a half-scale four-story SPSW tested by Driver et al. (1997) under vertical gravity load concurrent with cyclic lateral loads resembling the effect of a seismic event. The commercial general-purpose finite element code ABAQUS is used to model the test specimen. The material plasticity was represented by the von Mises yield criterion along with isotropic hardening for monotonic loading and kinematic hardening for cyclic loading, and the models were validated by Moghimi and Driver (2010).

To develop a reliable $P-I$ diagram from numerical study, a comprehensive finite element model capable of considering all important issues that affect the blast response is required. As such, the numerical model by Moghimi and Driver (2010) is enhanced further in the current study. The welds connecting the infill plate to the surrounding frame are modeled explicitly, using weld material properties based on the test results of transversely-loaded fillet welds by Ng et al. (2002). The selected yield and ultimate stress values are 470 and 630 MPa, respectively, while the strain corresponding to the ultimate stress and fracture strain are selected as 0.10 and 0.22, respectively. The weld material has a higher strength with lower ductility than the structural steel used for the infill plates and boundary frame. Moreover, the model includes a comprehensive hardening rule, stress dependency on strain rate, and damage accumulation in the steel materials, which are important properties to arrive at reliable iso-response curves. These properties are described in detail in the following section.

7.4.1. Constitutive material model

7.4.1.1 Hardening rules

Since the blast response of SPSWs involves yielding and cyclic response, a suitable hardening model is required for achieving acceptable results. The isotropic hardening rule assumes that as plastic deformation develops, the

subsequent yield surface experiences no translation and just expand its size symmetrically in the stress space from its initial yield surface. As such, it is appropriate for a monotonic loading condition. The kinematic hardening model assumes that as the plastic deformation develops, the subsequent yielding surface experiences rigid body translation with no change in the size of the yield surface. Although the kinematic hardening rule may lead to acceptable results for structures subjected to mild dynamic loading, it imposes two major limitations. First, the size and orientation of the initial yield surface remains unchanged. Second, it requires a modified bilinear stress–strain curve, which does not allow strain softening to occur if large strain is experienced in the material subjected to the blast load.

In this study, mixed hardening with full stress–strain properties, including softening, is used. This is in effect a combination of the isotropic and kinematic hardening rules. As plastic deformation advances, the subsequent yield surface can experience translation, while at the same time allowing growth in the size of the yield surface in the stress space. In the current study, a 10% increase in the size of the yield surface after initial yield is assumed, which is believed to be conservative (although the results are not particularly sensitive to this quantity), and the remaining increase in the yield stress is allocated to the translation of the yield surface.

7.4.1.2. Rate dependent yield

The high strain rate in the material resulting from the blast load causes an increase in the yield stress level compared to the static loading regime. Neglecting the strain rate effect results in a conservative flexural design for the components directly subjected to the blast load. However, it underestimates the induced design forces from the flexural action in the component, such as the design shear force of the member itself or the design forces of its connections and supporting members. As a result, the rate dependency of the material is considered in the current study, since it could have an unconservative design effect, especially in the out-of-plane

blast direction. The suggested values in the UFC document (DoD 2008) for the yield stress increase for structural steel are presented in Table 7.2. Different values are proposed for normal and high strength steels. As shown in the table, values slightly closer to those for A36 steel than A514 steel are selected in the current study to reflect the properties of the steel used in the test. Although the increase in the yield stress is larger than the increase in the ultimate stress, due to lack of information on the amount of increase in the ultimate stress, the whole stress-strain curve is increased by the strain rate values in Table 7.2. The effect of this approximation is considered negligible, since the strain values in the system are generally much less than the ultimate strain and barely pass the yield plateau.

In order to implement the stress increase factors defined in Table 7.2 at any Gauss point in a MDOF numerical model, the strain state at the material point needs to be transformed into an equivalent scalar value, and then the equivalent strain rate can be calculated. The most common strain transformation is the incremental effective plastic strain ($d\epsilon_p^*$) used in the theory of plasticity, and it can be defined by the strain hardening method ($d\epsilon_p^* = \sqrt{[2/3 (d\epsilon_p)^T (d\epsilon_p)]}$, where $d\epsilon_p$ is the incremental plastic strain tensor at the point) or the work hardening method ($d\epsilon_p^* = dw_p/\sigma^* = \underline{\sigma}^T d\epsilon_p/\sigma^*$, where dw_p is the incremental plastic work done by the incremental plastic strain tensor over the stress state at the point, $\underline{\sigma}$ and $\sigma^* = \sqrt{(3J_2)}$ are the stress tensor and the effective stress value at the point, respectively, and J_2 is the second invariant of the deviatoric stress tensor). Although the effective plastic strain does the transformation job, both definitions result in a non-negative value. As a result, it is not a proper parameter for calculation of the equivalent strain rate value, which can have a reversing sign. Therefore, in the current study, the volumetric strain is used to transform the strain state at any Gauss point to a scalar value. Since the steel materials are thin plate, only the in-plane strain components have been considered in the equivalent volumetric strain value calculation. The equivalent strain rate is defined by dividing the change in the equivalent volumetric strain at each Gauss point by the time increment, and the strain rate effect is incorporated into the analysis based on

the selected coefficients in Table 7.2 Linear interpolation is used for any strain rate that falls between the values in the table.

7.4.2. Damage model

As demonstrated in Table 7.1, the design guidelines define the structural performance in terms of support rotation and ductility ratio. The former and latter tend to limit and control the maximum deformation and the degree of nonlinearity (approximate measure of component uniaxial plastic strain), respectively, at the location of the component's maximum demand. However, they do not capture all the parameters that affect the blast damage in the component. For instance, when a biaxial stress condition exists or when the shear stress level is high at any point in the material, the steel could be prone to localized damage or failure other than strain softening under a large uniaxial strain value. Hence, the numerical model needs to capture all the parameters that affect the blast design and are not considered in the response limit values proposed in the design guides.

When the material subjected to the blast wave is damaged from breach, there is a potential for leakage pressure to enter the building, especially for far-field explosions with relatively longer durations. As such, the numerical model needs to capture the material damage properly, and identify any blast load levels that cause failure in material, even though the blast pressure itself induces acceptable ductility or support rotation response in the system.

Moghim and Driver (2010) showed that global damage indices, such as the change in dynamic properties of the system obtained from a model incorporating only material plasticity, are not necessarily competent tools for accurately estimating the extent of damage in the system when subjected to blast loads. Also, a simple fracture model based on maximum strain criteria is not an accurate method, since the fracture strain would be constant for all stress states. As such, considering the comprehensive damage models being used for the current numerical study, which take into account the stress state in the fracture model, is

necessary for developing the iso-response curves.

7.4.2.1. Damage initiation models

Hooputra et al. (2004) suggested that metal sheets may fail due to one or a combination of the different potential failure mechanisms, including ductile fracture due to void nucleation, growth, and coalescence, shear failure due to shear band localization, and instability due to localized necking. The study was carried out for specific types of aluminum alloy with yield strengths and strain hardening properties comparable with structural steel. The material damage models were validated by quasi-static three-point-bending tests and quasi-static and dynamic axial compression tests on the double-chamber extrusions. The predominant fracture modes in all tests were found to be shear and ductile failure, while instability failure did not govern due to the loading conditions. It was shown that for both quasi-static and dynamic loadings, where the ratio of minor to major principal strain rate is larger than about +0.35, the instability damage mode does not govern. For a SPSW system under medium- to far-field blast loads, the principal strain rate ratio is generally larger than 0.5, and as such, the ductile and shear damages are the governing damage modes and are considered in the numerical model. When the material properties for structural steel are not available, the suggested properties for the damage models from the original study (Hooputra et al. 2004) are used. Conservatively, the effect of strain rate on equivalent plastic strain at the onset of damage is ignored for both failure models, since test data are not available for structural steel.

The ductile damage criterion (Hooputra et al. 2004) takes into account the effect of the hydrostatic stress condition in material damage by introducing a stress triaxiality parameter, $\eta = \sigma_m / \sigma^*$, where σ_m is the mean stress and σ^* is the effective stress as defined earlier. For a given temperature and strain rate, the effective plastic strain at the verge of damage, ϵ^p_0 , can be defined as a monotonically decreasing function of the stress triaxiality parameter as $\epsilon^p_0 = d_0 \exp(-c \eta)$, where d_0 and c are scalar and directionally-dependent

material parameters (Hooputra et al. 2004). Assuming homogeneous material properties for steel, the parameter c becomes scalar and the value suggested by Hooputra et al. (2004) is used ($c = 5.4$). Substituting the uniaxial coupon tension test ($\eta = 1/3$) into the equation for ϵ_o^{pl} gives $d_o = 6.05 \epsilon_u$, where ϵ_u is the uniaxial plastic strain of the material where the failure is initiated. In this study, the plastic ultimate strain of the materials from the tension coupon tests is used as ϵ_u . As such, the effective plastic strain at the onset of ductile damage would be as follows:

$$\epsilon_o^{pl} = 6.05 \epsilon_u \exp(-5.4\eta) \quad (7.7)$$

The shear damage criterion (Hooputra et al. 2004) takes into account the effect of shear stress on the material failure by introducing the shear stress ratio parameter, $\lambda = (1 - \kappa_s \eta) / \varphi$, where κ_s is an empirical material parameter and $\varphi = \tau_{max} / \sigma^*$ is the ratio of maximum shear stress to effective stress. In this study $\kappa_s = 0.3$ is selected, as proposed by Hooputra et al. (2004). The effective plastic strain at the onset of damage, ϵ_o^{pl} , for a given temperature and strain rate can be defined as a monotonically increasing function of shear stress ratio as $\epsilon_o^{pl} = d_o \exp(f\lambda)$, where d_o and f are scalar material parameters (Hooputra et al. 2004). For the latter parameter, the value suggested by Hooputra et al. (2004) is used ($f = 4.04$). Substituting the uniaxial coupon tension test ($\lambda = 1.8$) into the equation for ϵ_o^{pl} gives $d_o = \epsilon_u / 1439$. As such, the effective plastic strain at the onset of shear damage would be as follows:

$$\epsilon_o^{pl} = (\epsilon_u / 1439) \exp(4.04\lambda) \quad (7.8)$$

7.4.2.2. Damage evolution

When a material point is under a loading condition and its strain increases, the stress state eventually reaches the plastic limit. From this instance forward, the plasticity model takes over the material behavior at the point and it defines any material softening and strain hardening up to the fracture strain. However, at the

time the effective plastic strain in the material reaches the effective plastic strain at the verge of damage, ϵ^p_0 , damage in material is initiated and the material point enters the damage evolution phase. In this phase, the plasticity model cannot accurately represent the material behavior since it may introduce a strong mesh dependency because of strain localization. As such, a damage evolution law is added to material behavior, which applies progressive degradation in material stiffness leading to complete material failure at the plastic strain equal to the equivalent plastic strain at failure, ϵ^p_f . A linear interpolation is used in the current study for the material stiffness degradation evolution from the plastic strain at the onset of fracture to the equivalent plastic strain at failure.

For both damage models, the same damage evolution law is used. To make the material response mesh-independent, the ABAQUS software formulates the damage evolution law based on stress–displacement (instead of stress–strain) response by introducing either fracture energy dissipation or the equivalent plastic displacement at failure, u^p_f . The latter parameter—which is defined with the evolution equation of $du^p = L d\epsilon^p$, where ϵ^p and L are the plastic strain at the material point and the characteristic length of the element—is used in the current study.

A fracture energy-based approach is implemented to rationally estimate the equivalent plastic displacement at failure. Based on classical fracture mechanics, the strain energy release rate (fracture energy per unit area) for a crack in the first mode of opening in the plane stress condition is equal to $G_I = K_I^2/E$, where E is the modulus of elasticity and K_I is the stress intensity factor for the first mode of opening. For an infinite plate with a crack with a total length of $2a$, the stress intensity factor would be equal to $K_I = \sigma\sqrt{(\pi a)}$, where σ is the stress applied to the plate that initiated the crack. In the current problem, the applied stress is selected equal to σ^p_0 , which is the yield stress corresponding to the effective plastic strain at the verge of damage. On the other hand, the fracture energy dissipation can also be defined based on the equivalent stress and displacement as

$G_I = \sigma^p_{l0} u^p_f/2$. Both definitions for the fracture energy dissipation result in the following equation for the equivalent plastic displacement at failure:

$$u^p_f = 2 \sigma^p_{l0} (\pi a)/E \quad (7.9)$$

Assuming the average crack length, yield stress at the onset of failure, and modulus of elasticity for structural steel are equal to 20 mm, 350 MPa, and 2E5 MPa, respectively, the equivalent plastic displacement at failure from Eq. (7.9) would be equal to $u^p_f = 0.1$ mm.

Although the damage evolution is formulated in terms of the equivalent plastic displacement (instead of strain), the method is still fairly mesh dependent. The analysis for the current study showed that the model provides acceptable results for global and member behavior, while it may not necessarily predict the local material behavior at a small discontinuity accurately, since plastic and damage response sensitivity to the mesh refinement exists.

The material models and laws described above are incorporated into the ABAQUS software by means of defining appropriate field variables and user subroutines. The damage models are not compared with the cyclic test results of Driver et al. (1997), since most of the reported material failures were due to low cycle fatigue rather than ductile and shear damage. A typical result for a large out-of-plane blast load is shown in Fig. 7.3. The pressure applied to the front wall is 1.66 MPa, which is a very severe blast pressure. The pressure applied to the top beam in the downward direction has a resultant equal to 1/2.5 times the resultant of the pressure applied to the infill plate, where the factor 2.5 is the reflection coefficient and is described in the next section. The numerical model shows the same failure pattern under out-of-plane blast overpressure as was reported by Warn and Bruneau (2009), where the infill plate unzipped along three weld lines.

7.5. Numerical Model of SPSW Systems

7.5.1. Blast effects on SPSWs

Positive-phase shock-wave-type blast loads are used in all analyses of this study; the negative phase is ignored as it contributes little to the overall dynamic response. Fig. 7.4(a) shows a typical shock load and its linearized triangular step-type load. P_0 is the maximum peak incident or reflected blast overpressure, which can be represented as a pressure or force. t_d is the positive-phase duration, or the duration of the linearized triangular step-type load. In this study, the linearized shock load is used, where P_0 and t_d are variables in developing P – I curves. The area under the P_0 – t_d triangle is the intensity (I) of the blast load. The blast loads on the wall and the roof are assumed to be in phase since the system is a single wall. Design blast loads (pressure and duration) for petrochemical facilities can be found in the literature (ASCE 2011b and Oswald 2008) and are summarized by Moghimi and Driver (2010). In most blast-resistant design cases, which are close to the impulsive asymptote, the maximum duration of the blast loads is less than one-quarter of the first natural period of the structure, and the pulse load shape has a negligible effect on the dynamic response.

SPSW systems in a protective structure could be under in-plane or out-of-plane blast load effects. An idealized, square-plan protective structure—representing a building such as a small control house—consisting of four SPSWs under reflected overpressure on the front wall and incident overpressure on the roof is shown in Fig. 7.4(b). These blast waves are separated into out-of-plane and roof overpressures on the front wall, as shown in Fig. 7.4(c), and in-plane and roof overpressures on the side walls, as shown in Fig. 7.4(d). The blast response of these two single walls—the side wall and front wall—are investigated.

The side wall is analyzed under uniform in-plane reflected blast overpressure acting on the left column (in the left-to-right or X direction) and uniform roof incident overpressure blast acting on the top beam (in the downward or $-Y$ direction) in Fig. 7.4(d). The effects of the front and rear walls are replaced by a

roller support at the center of each frame connection that prevents movement in the Z -direction. The resultant of the blast overpressure acting on each structural member (left column and beam) depends on the blast-load tributary area for the member. To make the results independent of the tributary area, the resultant reflected blast pressure applied to the left column is defined as a multiplier of the yield strength of the wall, V_y , derived from a bilinear curve taken from the pushover analysis result with the same spatial load distribution as the blast load and explained in Section 7.5.3.1. The resultant blast load acting on the top beam is equal to the same multiplier times V_y/C_r , where C_r is the reflection coefficient pertaining to the front wall, which is a function of peak overpressure and the angle of incidence. For the range of parameters used in petrochemical facilities, C_r can be selected as 2.5 (ASCE 2011b), implying a perpendicular blast applied to the front wall. The blast effect is localized to the vicinity of the reflected blast pressure applied to the system and maximizes the local ductility demand in the left column.

The front wall is analyzed under uniform out-of-plane reflected blast overpressure acting on the infill plate (in the $-Z$ direction) and uniform roof incident overpressure blast acting on the top beam (in the downward or $-Y$ direction) in Fig. 7.4(c). The out-of-plane translational movements (in the Z direction) of the top and bottom flanges of the beam at the connections are restrained. Similar to the side wall, the resultant blast pressure on the infill plate and top beam are different multipliers of V_{yo} and V_{yo}/C_r , respectively, where V_{yo} is defined in Section 7.5.3.2 for the out-of-plane response of the wall and C_r is selected as the same value as for the side wall (2.5).

7.5.2. Selected SPSW systems

The competency of SPSW systems under blast loads is investigated by developing $P-I$ curves for different walls and different response limits and comparing them with the results of relevant design examples. The selected base model is the first story of a multi-story SPSW specimen tested by Driver et al. (1997) and described

in Section 7.4. Since most industrial protective structures are one-story buildings for design and safety requirements, only the first story of the specimen is considered in this study. To allow the development of full tensile yielding of the infill plate, the deep top beam of the actual specimen was used for the analyzed system. As shown in Fig. 7.5(a), the columns are W310×118 sections spaced at 3.05 m center-to-center. The top beam is a W530×82 section and the total story height is 2.15 m. Moment connections are used at the beam-to-column joints, and the infill plate thickness is 4.54 mm.

The above-described system represents a half-scale wall for an industrial protective building. To study the effect of size on the response, a full-scale wall that has the same configuration but all the dimensions (including beam and column cross-sectional depth and element width and thickness) are doubled. The story height is therefore 4.30 m and the columns are spaced at 6.1 m center-to-center. These two walls are the subject of the blast-resistance study under in-plane and out-of-plane blast loads. The share of the roof's dead load taken by one wall is simulated by adding total masses of 3240 kg and 12 960 kg, for the half- and full-scale walls, respectively. The mass is added by means of point masses distributed to the top beam at the nodes on the beam-to-column connections and along the beam flange-to-web interface, effectively creating a uniform mass distribution over the beam length and a more concentrated mass at the beam-to-column connections.

7.5.3. Pushover responses of SPSW systems

The pushover analysis results are needed for the normalization process to transform each wall system into an ESDOF system. Also, different multipliers of the yield strength of each wall are used as the blast load intensity applied to the system. As such, the yield strength and yield displacement of the wall systems are needed, and are explained in the following sections.

7.5.3.1. In-plane response (side wall)

The lateral deflection at the roof level is selected as the displacement in the pushover curve and the generalized displacement for the ESDOF system. The dashed pushover curve in Fig. 7.5(b) shows the in-plane pushover analysis of the system under monotonically increasing lateral displacement at the roof level. The curve demonstrates a similar result to that of the base shear versus first-story displacement response of the four-story test specimen. However, in order for the results of the pushover curve to be applicable to a SDOF system, the lateral pushover force should have the same spatial distribution as the applied blast load. The solid curve in Fig. 7.5(b) shows the result of a pushover analysis under a uniform lateral pressure applied to the left column. The pushover load distribution is similar to the in-plane blast load and is identified as “Modified pushover analysis” in Fig. 7.5(a). Because of the distributed applied lateral loads, the modified pushover curve has a larger yield strength and displacement.

The yield shear strength and displacement of the side wall can be found from the equivalent bi-linear pushover curve representing the real nonlinear curve with the same energy absorption capacity. Bilinear representation of the pushover curve results in yield values equal to $V_y = 3300$ kN and $\delta_y = 7$ mm, respectively. The modified pushover analysis curve of the full-scale wall shows the same behaviour as half-scale wall in Fig. 7.5(b), but with yield strength and displacement equal to $V_y = 13\,200$ kN and $\delta_y = 14$ mm, respectively.

7.5.3.2. Out-of-plane response (front wall)

The out-of-plane pushover analysis of the SPSW demonstrates a distinctly different response. The out-of-plane deformation at the center of the infill plate is selected as the displacement in the pushover curve and the generalized displacement for the ESDOF system. Since the infill plate is the main element that contributes to the out-of-plane resistance of the front wall, it is a simple system and the pushover curve can be evaluated by an analytical approach in addition to the numerical method. Two analytical approaches—namely, the energy balance

and equilibrium methods—are used to derive the pushover curve. The out-of-plane flexural resistance of the infill plate is ignored in the total resistance of the infill plate in both analytical approaches, since membrane action dominates the response. Fig. 7.6(a) demonstrates a typical SPSW system with infill plate thickness of w and clear length and width of infill plate equal to L_c and h_c , respectively. The corresponding yield lines for the infill plate subjected to the out-of-plane blast pressure with intensity p are shown in the figure. For a given wall system, the deformed shape of the infill plate depends on the panel aspect ratio and the dynamic load rate and intensity. For high-speed loading, the deformed shape is close to linear deformation, as shown in Fig. 7.6(a), while for quasi-static loading the deformed shape becomes curved.

The external work in the energy balance method is equal to

$W_E = p h_c \delta (3L_c - 2x)/6$. The internal strain energy can be calculated by dividing the infill plate into two triangles with height x and two trapezoids, as shown in Fig. 7.6(a). The strain energy is equal to the axial tension in each element multiplied by its axial deformation, or $W_I = \sigma w h_c [x \epsilon_L + (L_c - x) \epsilon_h]$, where $\epsilon_h = \sqrt{[(2\delta/h_c)^2 + 1]} - 1$ and $\epsilon_L = \sqrt{[(\delta/x)^2 + 1]} - 1$ are axial strains in the h_c and L_c directions, respectively. The parameter x can be evaluated by equating the external work to the internal energy and solve for the pressure, p . Setting the first derivative of the pressure equation with respect to x equal to zero gives the maximum pressure applied to the system. However, the equation with respect to x would be nonlinear and does not have a closed-form solution. As such, a simplifying assumption is made for x based on the numerical response of the SPSW under blast load at different loading rates. The numerical results show that at a fast blast loading rate (impulsive asymptote), the deformed shape of the infill plate is such that the plate rotation along the boundary L_c $\theta_L = \theta$ or $x = h_c/2$ (Fig. 7.6(a)), which results in $\epsilon_L = \epsilon_h$. The assumption results in the following equation for the resultant force of the applied pressure to the infill plate:

$$pL_ch_c = 6 \sigma w L_c^2 h_c/(3L_c - h_c) (\sqrt{[4/h_c^2 + 1/\delta^2]} - 1/\delta) \quad (7.10)$$

where σ is the tensile stress in the infill plate and is equal to the elastic stress for strains less than the yield strain and equal to the yield stress for greater strain values. The pressure applied to the infill plate can be evaluated based on equilibrium and the same assumed deformed shape for the infill plate. Neglecting the flexural resistance of the infill plate, at every stage of loading the horizontal component of the membrane action of the infill plate resists the applied lateral loads. As such, the horizontal components of the infill plate membrane forces f_{th} and f_{tl} in Fig. 7.6(a), which are applied to the faces L_c and h_c , respectively, would be equal to the resultant of the pressure applied to the infill plate:

$$p h_c L_c = 2\sigma w [L_c + h_c] \sin (\theta) \quad (7.11)$$

where $\tan (\theta) = 2\delta/h_c$. Fig. 7.6(b) shows the pushover curves for the half-scale wall depicted in Fig. 7.5(a). The figure shows the pushover curves from the numerical model and the two analytical methods (Eqs. (7.10) and (7.11)). Also, the result of the method proposed by Warn and Bruneau (2009), which assumes a second-order polynomial for the infill plate deformation in both directions, is shown.

The results show that at the early stage of the pushover curve, the response is nonlinear. The flat (unloaded) infill plate subjected to out-of-plane load has a very small resistance because of its insignificant flexural stiffness, and it deforms rapidly under the load. As the infill plate deforms in the out-of-plane direction, most of the applied load is carried by membrane action. This effect increases the stiffness of the system considerably, and the upward curved shape of the pushover diagram is the region that the membrane action engages in the infill plate. In order to capture this effect in the numerical model, geometric non-linearities must be taken into account.

Although at large out-of-plane deformations the infill plate is fully yielded and the membrane strength remains fairly constant, the pushover curves do not indicate a

distinguished yield strength and yield plateau, similar to the side wall. This is attributed to the fact that with an increase in the applied force, the plate experiences a larger out-of-plane deformation, which makes the horizontal component of the infill plate membrane force larger. The phenomenon results in a fairly linear pushover curve even beyond the chord rotation of 12° for the infill plate connection to the boundary frames, which is considered as the largest acceptable out-of-plane deformation (or plate rotation) for the plates according to all blast design guides cited in Table 7.1.

The results in Fig. 7.6(b) show that the parabola assumption for the infill plate deformation overestimates the resistance of the infill plate, while the energy balance method underestimates the pushover resistance. The equilibrium method, with an assumed linear deformed shape, estimates the resistance of the infill plate close to the numerical result. Their actual difference is even smaller in reality for two reasons. First, the load is applied very slowly in the pushover numerical results, which forces the infill plate to deform into a curved shape. As explained before, the infill plate deformation would be nearly linear for a fast rate of loading (impulsive asymptote), which is the main subject of the current study. In such a case, the numerical result would be even closer to the equilibrium method result. Second, the pushover analysis was done by a displacement-controlled method, and the applied displacement was in the horizontal direction. As a result, with increasing out-of-plane deformation of the infill plate, the membrane component of the applied force increases. However, the blast pressure is always normal to the infill plate, which reduces the resistance of the infill plate at each displacement level slightly.

The analytical approach estimates the out-of-plane deformation associated with achieving the yield strain in the infill plate to be equal to 48 mm. The numerical model in Fig. 7.6(b) estimates the out-of-plane yield deformation and its corresponding base shear equal to $\delta_{yo} = 58$ mm and $V_{yo} = 986$ kN, respectively, where the subscript “o” represents the out-of-plane response. This point is

associated with the change in the curvature sign of the pushover curve, and that corresponds to a fully yielded infill plate in the h_c direction. The pushover analysis curve of the full-scale wall shows the same behavior as the half-scale wall, but with an out-of-plane yield displacement and strength equal to $\delta_{yo} = 117$ mm and $V_{yo} = 4030$ kN, respectively.

7.6. Results of Blast Design Study

7.6.1. Response limits

The goal of this study is to evaluate the blast performance of SPSW systems by developing iso-response curves. Any $P-I$ curve applies only to one single response limit in the system. In blast design, the members of structures are allowed to yield to achieve an economical design. The design guidelines specify the structural performance of the ductile elements based on maximum deformation limits instead of strength level, to provide controlled and ductile yielding. In an approximate way, these limits define the strain energy absorption capacity and amount of acceptable damage in the component and prevent component failure. The safety factor in design is also included in the deformation response limits.

As pointed out earlier in reference to Table 7.1, the deformation performance criteria are usually defined in terms of ductility ratio and support rotation for individual members. Both response parameters can be calculated from the maximum component deformation, which is determined from the nonlinear dynamic numerical model in this study. The response criteria are applicable to ductile members, and when brittle failures are prevented ductile flexural response governs the overall component behavior. The equivalent static method can be used to prevent brittle failure in the strength-controlled actions, such as axial compression, shear, and reaction forces at the connections. In this method, the internal force demands should be less than the lower-bound strengths for the corresponding actions.

The yield displacement is the effective displacement at which plastic strain begins in the same location as the maximum deformation occurs. The yield displacement and its corresponding yield strength of the system (δ_y and V_y and in Eqs. (7.2) and (7.4)) can be extracted from the static pushover resistance–displacement graph of the system, where the pushover load has the same spatial distribution as the applied blast load. The method results in an equivalent yield displacement that produces the equivalent ductility ratio. If the yield displacement is selected as the displacement at the beginning of the yielding process or the displacement at the mechanism condition, the resulting ductility ratio would be respectively larger and less than the equivalent ductility ratio. The support rotation can be calculated from the chord rotation of the member's overall (or global) flexural response rather than the actual local rotation at a point in the connection. The tangent of the support chord rotation is defined as the ratio of the maximum deflection in the member to the shortest distance from support to the location of maximum deflection.

The support rotation controls the maximum deformation (which usually occurs at midspan) of the member. The ductility ratio limits the extent of plastic response in the maximum deformation location of the component (which should be the same location as the initial yielding occurs). However, a ductile member may develop membrane action at a large ductility ratio. The axial tensile action may cause damage/failure in the connections before damage/failure at the midspan occurs. In such a case, the support rotation can limit the amount of membrane tensile force to an acceptable level that prevents the connection failure. Hence, it would be a better damage index for a ductile member.

Support rotation limits for the low response range of the structural steel system have been selected to develop the iso-response curves in this study. The rotation of the beam-to-column connection adjacent to the column under blast load (point E in Fig. 7.4(d)) is limited to 1° for the side wall study. Both local and global rotations are considered and their results are compared. The local rotation is the

amount of actual rotation at point E. The tangent of the global (overall or chord) rotation is calculated by dividing the maximum translation at the middle of beam or column to the distance between connection and maximum deformation point. Since the reflected blast pressure is applied to the left column in this study, the maximum deformation occurs at the middle of the left column, and the tangent of the global rotation at points E or F is calculated by dividing the U_x deformation of point D of Fig. 7.4(d) by the half-column height.

The rotation of the infill plate connection at the base or top beam at the middle of the bay (points B and C in Fig. 7.4(c)) is limited to 3° for the front wall study. The tangent of the infill plate rotation is calculated by dividing the out-of-plane deformation of the infill plate mid-point (point A in Fig. 7.4(c)) by the half-span height. For comparison and to check the extent of yielding in the infill plate at the support rotation of 3° , the iso-response curves for a ductility ratio of 1.0 are calculated. The yield displacement is selected from pushover curve, and the iso-response curve shows the pressure and impulse combinations that make the infill plate fully yielded.

7.6.2. *P–I curves*

7.6.2.1. *Iso-response curves*

The iso-response curves are developed for the four different walls and the response limits specified in the previous section using the numerical model described earlier and the trial-and error approach. In this study, all analyses are done with the explicit method, originally developed to analyse nonlinear high-speed dynamic systems. Conservatively, the effect of damping in the analysis is not considered. However, this effect is negligible since the peak dynamic responses are studied. For the SPSW systems under consideration, the shear damage is usually a little more critical than ductile damage in both blast loading directions.

The *P–I* curves for the half- and full-scale side walls are demonstrated in

Fig. 7.7(a). The response limits are local and chord (global) rotations (θ_L and θ_G , respectively) of the beam-to-left-column connection of 1° . As expected, the full-scale wall is considerably stronger than the half-scale wall. The chord support rotation results in a larger rotation than the actual local rotation at the joint, and it is a more conservative response limit. In other words, for a given rotation limit, the local rotation shows a larger applied pressure and impulse than the chord rotation.

Some extensions of the impulsive and quasi-static asymptotes for the local rotation are shown by dashed lines for both the half- and full-scale walls. Although they represent 1° connection rotations, ductile and/or shear failure (especially of the left column and infill plate adjacent to the left column base) occurs in the system in these branches. Since the damage is sustained by the system, the dashed lines are not acceptable regions for design. For instance, the maximum blast pressure resultant that can be applied to the full-scale wall is $7.05V_y = 93\,060$ kN. Any blast load with a larger resultant, and even with a smaller duration that would otherwise produce a 1° connection rotation, causes failure in the system. As it can be seen, any combination of P and I at the impulsive asymptote that produce the same impulse may not be acceptable in design, especially for the blast load with large incident overpressure. The same concept is valid for the quasi-static loading realm. The minimum blast load resultant that can be applied to the system and reach the monitored response limit is $2.6V_y = 34\,320$ kN. A smaller blast load with duration large enough to produce a 1° connection rotation causes failure in the system.

Fig. 7.7(b) shows the iso-response curves for both front walls and both response limits of 3° connection rotation and a ductility ratio of unity. The chord rotation curve also limits the maximum out-of-plane deformation of the infill plate, which is one of the limit state criteria in the front wall design. The figure shows that the 3° connection rotation imposes a more conservative response limit than ductility ratio of unity.

The efficiency of SPSW systems in petrochemical facilities is investigated by checking appropriate design examples for both half- and full-scale walls against the iso-response curves. The actual wall dimensions are used for the front wall design. However, an assumption for the tributary width is necessary for each side wall design. Widths of 7 m and 14 m are assumed for the half- and full-scale walls, respectively, constituting a relatively severe demand scenario. The blast-pressure design is based on the ASCE guideline (ASCE 2011b), which specifies a shock wave with 70 kPa side-on overpressure—which is scaled up by a reflection factor of 2.5 to represent the reflected pressure of 175 kPa—and a 20 ms duration. Since the design points all fall below the associated $P-I$ curves in Figs. 7.7, the selected SPSWs constitute a competent lateral force resisting system for accidental blast loading in petrochemical facilities, even for the low response limit and out-of-plane blast load.

7.6.2.2. Nondimensional curves

Using Eqs. (7.2) and (7.6), the iso-response curves for the side and front walls are normalized and shown in Figs. 7.8(a) and (b), respectively. The significant DOF of each wall, which is the DOF of the ESDOF system, has been selected as the roof displacement for the side wall and the out-of-plane displacement of the center of the infill plate for the front wall. The shape function for the ESDOF system is selected as the normalized deformed shape of the system under static application of the blast load. Unlike the fundamental mode shape, this method works for both elastic and plastic responses and provides better results for design force calculations.

Nevertheless, the ESDOF system is approximate, since the deformed shape of any system not only depends on the spatial load distribution but also on the rate of the applied load and interaction of the load with the structural response. Considering a given iso-response curve, the deformed shapes of the system in the quasi-static and impulsive loading regimes are different. Moreover, in the impulsive loading asymptote, local deformation exists in the system because of the fast application

of the blast load which cannot be captured with the static deformed shape of the system. However, the approximations have a negligible effect on the dimensionless curve results as long as the same shape function used to produce the dimensionless curves is also used to utilize them for design purposes.

Figs. 7.8(a) and (b) show that the normalization method brings each pair of curves close together. This can be an effective method for developing unified iso-response curves for different SPSWs. While in theory the normalized $P-I$ curves can be applied to any SPSW with the same type of system as the one used to develop the curves, the best accuracy will be achieved for walls with comparable values for the parameters that most influence the behavior, such as similar panel aspect ratios.

7.6.3. Charge weight–Standoff distance diagram

The information in an iso-response curve can also be represented in the form of combinations of explosive charge weight (W) and distance from the charge (standoff distance, R) that cause the response limit to be reached. The $W-R$ curve defines the system susceptibility to air explosions and shows all combinations of energy release amounts and standoff distance that cause the same maximum response in the system. Under sea level ambient conditions, the charge weight and standoff distance are uniquely determined based on the type of explosion. In this study, the positive phase of the shock wave parameters for a hemispherical TNT surface burst at sea level (CONWEP 1992) is selected, since it is similar to potential accidental industrial explosions.

Each point on a $P-I$ diagram associates the reflected blast pressure (P_r) to the impulse (I_r). From the reflected blast pressure, the corresponding scaled distance, $Z = R/W^{1/3}$, is determined from the selected blast chart. Having the scaled distance, the normalized reflected blast pressure, $I_r/W^{1/3}$, is obtained. The charge weight is evaluated by substituting the blast impulse from the $P-I$ curve into the normalized reflected blast pressure. The standoff distance is then determined by

substituting the charge weight into the scaled distance equation.

The conversion of the $P-I$ diagrams in Fig. 7.7 into $W-R$ curves is shown in Fig. 7.9. Logarithmic axes have been selected since the logarithmic representation of $W-R$ curves tends to be close to a straight line with a positive slope. $W-R$ curves represent iso-response curves similar to the $P-I$ diagrams. Any point above the curve implies the response limit has been passed, while a point below the curve is a safe condition with a lower response than the limit.

7.7 Design Recommendations

Blast load tests on steel systems often cause the fasteners to govern failure in the system. In the SPSW system, the side wall is a proper system for resisting blast load. However, the front wall is susceptible to failure of the fasteners connecting the infill plates to the surrounding frames, as a previous test shows (Warn and Bruneau 2009). Since loading of a fastener tends to be a force-controlled action due to a lack of ductility, using a fish plate thicker than the infill plate can improve the out-of-plane blast resistance of the wall. The fish plate can be welded to the surrounding frame with large, double-sided fillet welds and the infill plate connected to the fish plate by bolts or welding.

Also, the out-of-plane blast resistance capacity of SPSWs could be enhanced by adding some simple detailing to the system, such as adding a girt of cold-formed double-stud (back-to-back) steel sections to the infill plates that are properly connected to the surrounding frames by double angle connections. The cold-formed sections improve the out-of-plane response of the infill plate considerably by reducing the whole panel to smaller panels. Their involvement in the in-plane direction response of the system can be prevented by using proper connection (e.g., slotted holes) in the cold-formed sections in their connection to the infill plate. The choice of vertical and/or horizontal studs shall be studied by numerical or experimental studies.

7.8. Conclusion

It has been proven through past research that SPSWs make superior lateral force resisting systems for seismic applications. Despite the inherent slenderness of the steel members, this study shows that the SPSW system has potential to be an effective protective structure in industrial plants. The side wall subjected to in-plane blast load is a strong and reliable system, and the front wall subjected to out-of-plane blast load can be sized to provide acceptable design for industrial plant applications.

The proposed nondimensional $P-I$ diagram provides an efficient tool for preliminary design of SPSW systems to resist accidental blast loads. The study shows that to develop reliable iso-response curves, appropriate damage and failure criteria should be considered in the numerical study of the wall system. This is especially important in structural steel members, since their slenderness makes them vulnerable to heavy blast overpressure, even with very small duration.

Additional research is required to study the effects of various parameters not considered in the limited study presented in this paper. Low cycle fatigue failure has not been considered in the analysis. It may have some effect in the overall response of the SPSW system under far-field accidental blast load. To optimize the system and improve its reliability, improvements in detailing that are developed explicitly for blast-resistant applications are needed.

Table 7.1 Response limits for hot-rolled structural steel members in various design guides

LP	Resp Par		ASCE (2011b)				UFC (2008)			PDC (2008), ASCE/SEI (2011a)				ASCE (1999)			
			Perf. Level	BM Sec.	Prim. Mem. ^a	PL	Prot. Cat.	BM	PL	Comp. Dam.	BM ^b	CPR	PL	Dam. Level	BM Flex.	BM Shr.	CPR
H	μ	—	—	—	—	—	—	—	—	Super- ficial	1 (1)	0.9	4	—	—	—	—
	θ	°	—	—	—	—	—	—	—	—	— (—)	—	1	—	—	—	—
M	μ	—	Low Resp.	3	1.5(1.5)	5	—	—	—	—	—	—	—	Light Dam.	—	—	10
	θ	°	—	2	1 (1)	3	—	—	—	—	—	—	—	—	5.7	2.3	—
L	μ	—	Med. Resp.	10	3 (2)	10	Cat 1	10	10	Moder- ate	3 (3)	1.3	8	Mod. Dam.	—	—	20
	θ	°	—	6	2 (1.5)	6	—	2	2	—	3 (3)	—	2	—	13.5	4.6	—
VL	μ	—	High Resp.	20	6 (3)	20	Cat 2	20	20	Heavy	12 (3)	2	20	Severe Dam.	—	—	40
	θ	°	—	12	4 (2)	12	—	12	12	—	10 (3)	—	6	—	26.6	9.1	—
VL	μ	—	—	—	—	—	—	—	—	Hazard- ous	25 (3)	3	40	—	—	—	—
	θ	°	—	—	—	—	—	—	—	—	20 (3)	—	12	—	—	—	—

^a For member with significant compression, the values in parentheses should be used^b For combined flexure and compression, the values in parentheses should be used

Table 7.2 Strain rate effects on stress increase in structural steel

Strain rate (mm/mm/s)	Strength increase factor			
	ASTM A36 ^a	ASTM A514 ^b	Average	Selected value
0.1	1.29	1.09	1.19	1.2
1	1.45	1.17	1.31	1.4
10	1.6	1.27	1.44	1.5

^a Minimum yield stress of 250 MPa and ultimate tensile strength of 400-550 MPa^b Minimum yield stress of 690 MPa and ultimate tensile strength of 750-900 MPa

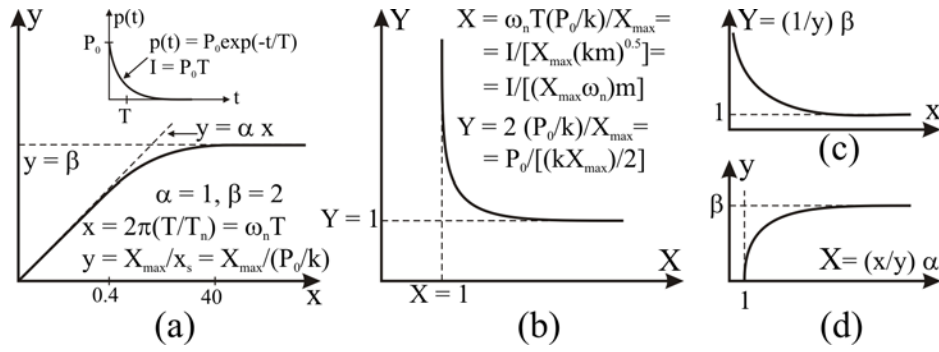


Fig. 7.1 (a) Typical response spectrum curve for SDOF under exponentially decaying pulse load, (b) Nondimensional mapped $P-I$ diagram, (c) Ordinate transformation (y to Y), (d) Abscissa transformation (x to X)

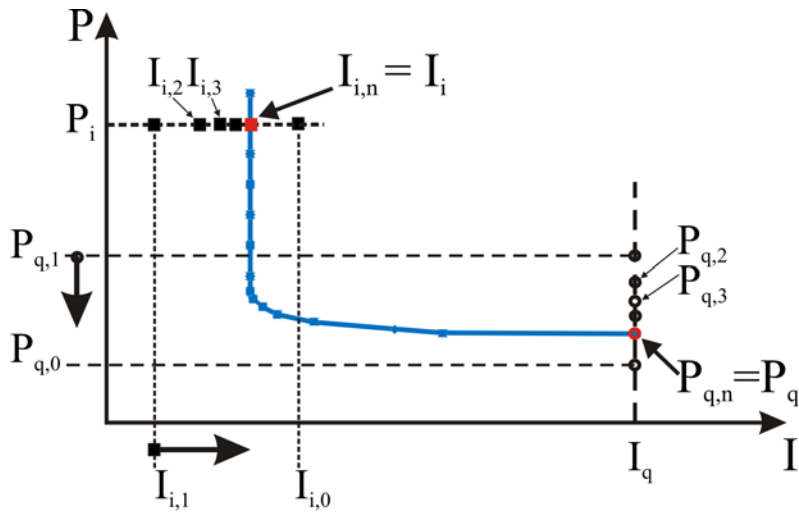


Fig. 7.2 Trial-and error method to find single points on iso-response curve

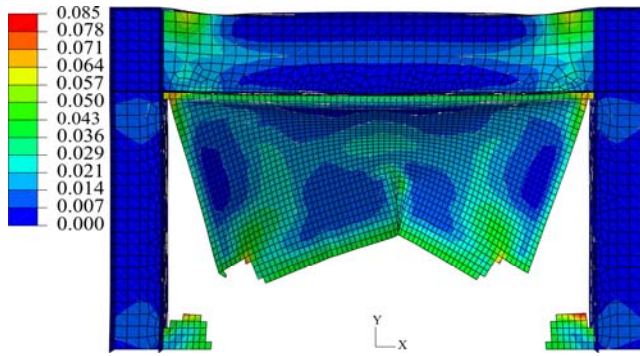


Fig. 7.3 Effective plastic strain and damage distribution in a front wall from large out-of-plane blast load

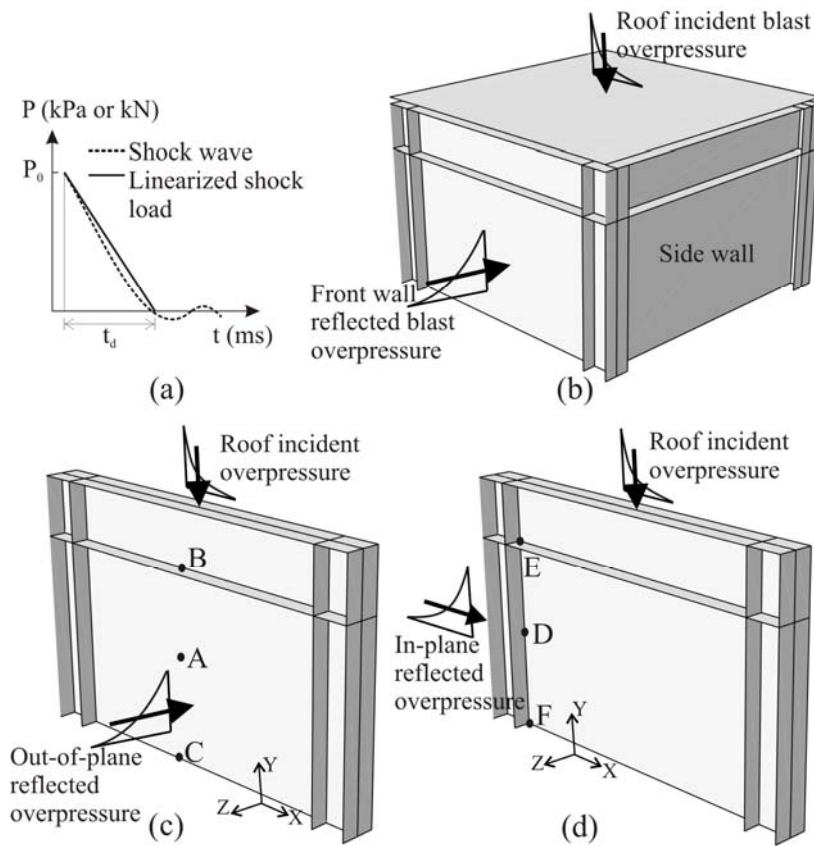


Fig. 7.4 (a) Shock wave blast load, (b) Protective structure consisting of four SPSWs under blast loading, (c) Front wall under out-of-plane and roof blast loads, (d) Side wall under in-plane and roof blast loads

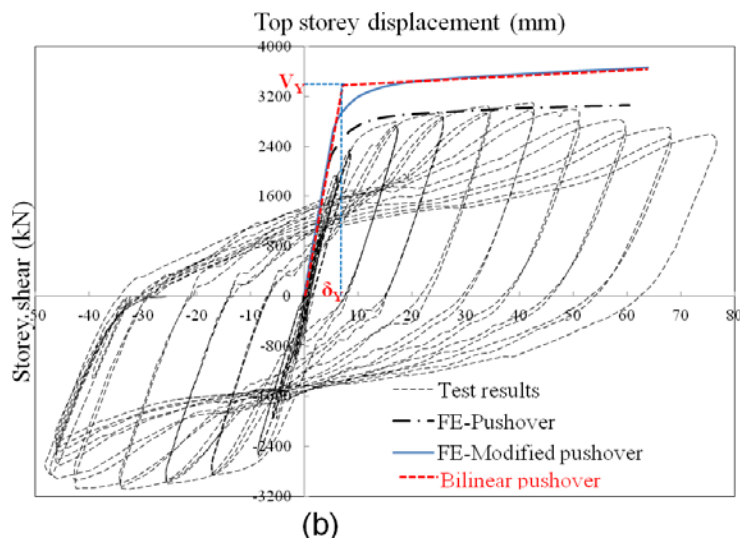
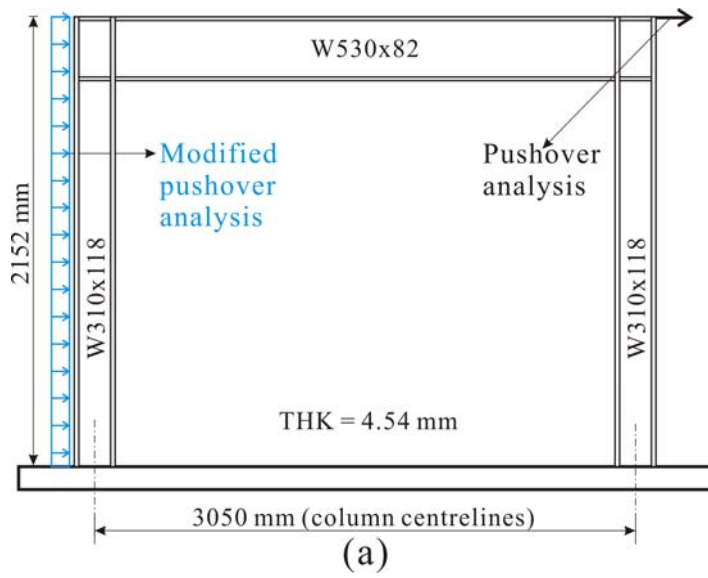


Fig. 7.5 Selected half-scale SPSW system, (a) Wall elevation and in-plane pushover loadings, (b) Pushover analysis results and comparison with test

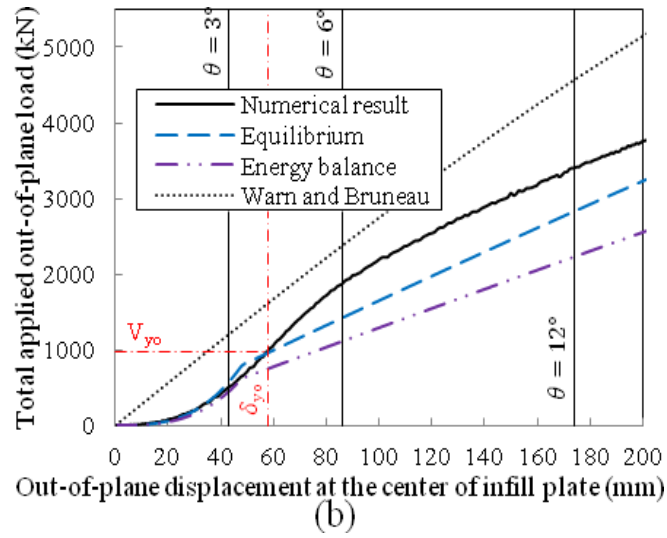
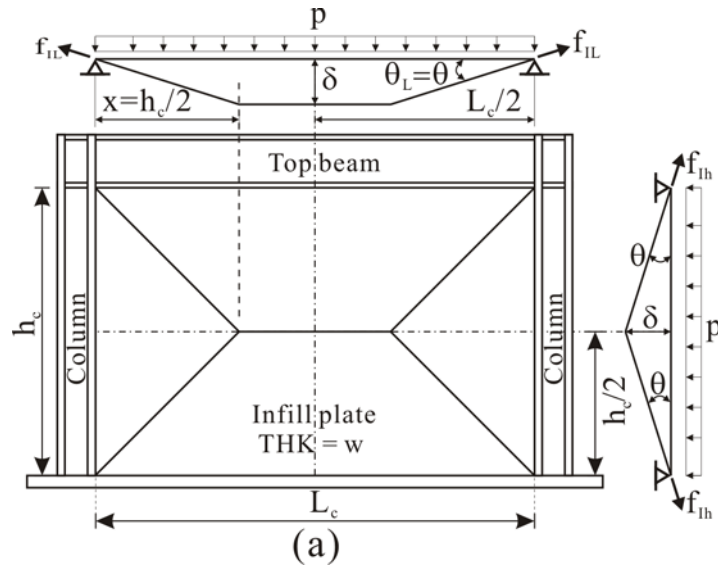


Fig. 7.6 (a) Infill plate subjected to out-of-plane blast load and its associated yield lines, (b) Pushover curve for out-of-plane resistance of infill plates versus displacement at center

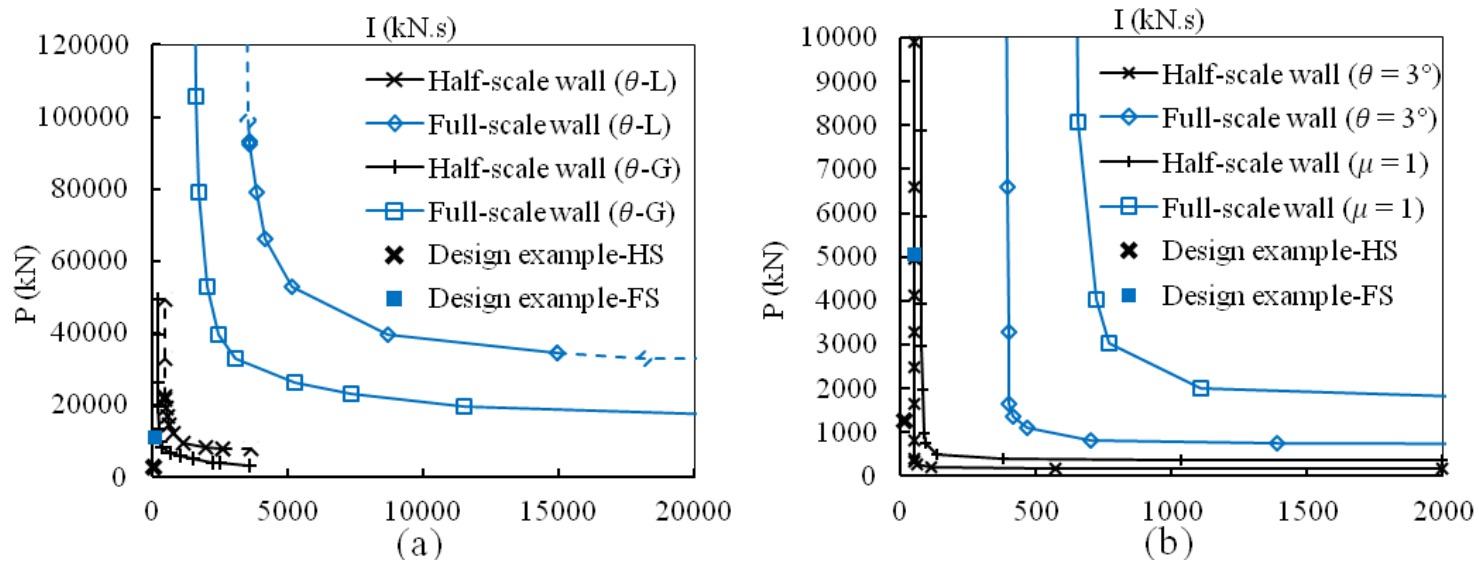


Fig. 7.7 P-I diagrams, (a) Side wall for 1° rotation, (b) Front wall

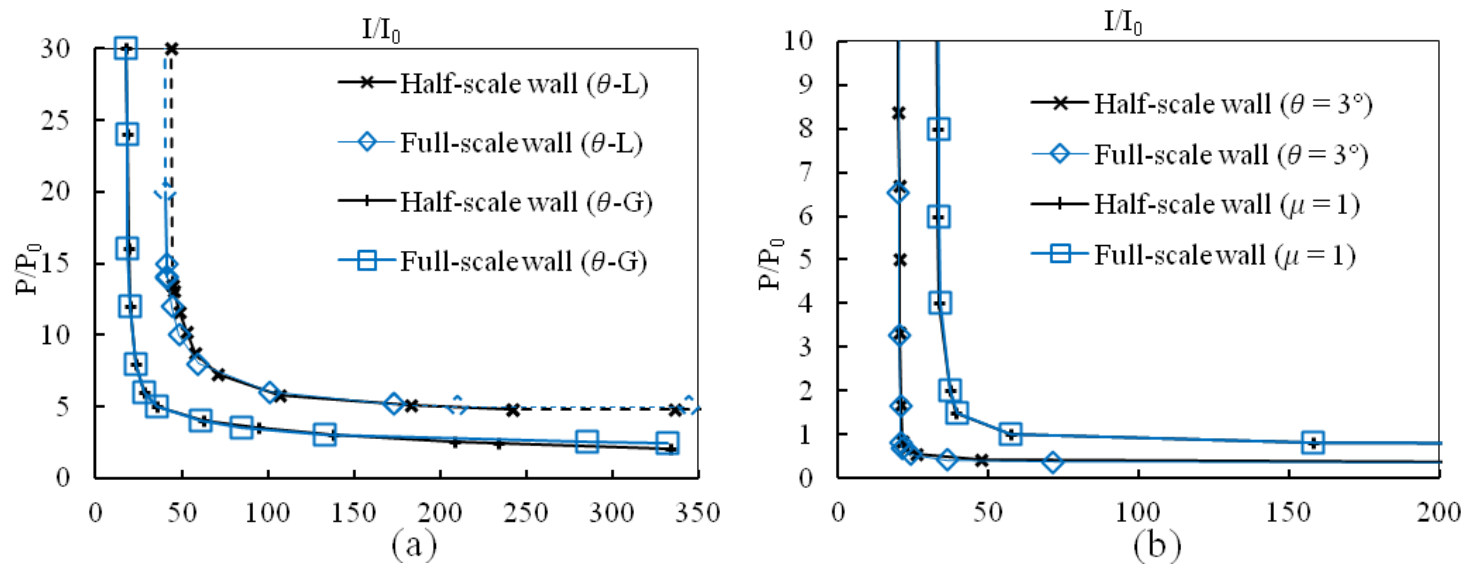


Fig. 7.8 Nondimensional P-I diagrams, (a) Side wall for 1° rotation, (b) Front wall

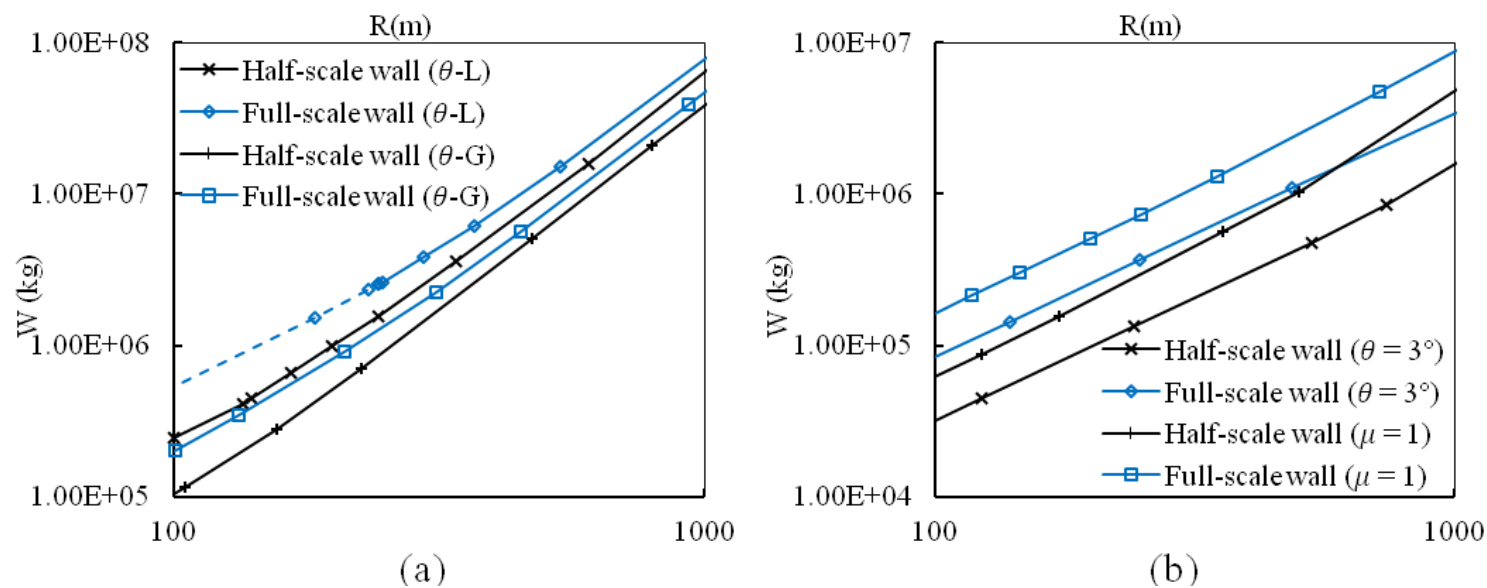


Fig. 7.9 Charge weight–Standoff diagram, (a) Side wall for 1° rotation, (b) Front wall

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8. SUMMARY, CONCLUSIONS AND DESIGN RECOMMENDATIONS

8.1 Summary

This research project consisted of two main parts: seismic and blast design of steel plate shear wall (SPSW) systems. In the seismic design, first, a new design philosophy for SPSW systems in low and moderate seismic regions was envisaged. The aim was to develop a design method that is independent from the current ductile wall design requirements. A standardized seismic hazard-independent method was proposed to evaluate the target displacement. The method was incorporated in a conventional performance-based design method, and a test specimen suitable for low-seismic applications was designed and tested under cyclic loading. The test results were used to develop performance-based capacity design provisions for SPSW systems for limited-ductility and moderately ductile applications. The blast performance of the SPSW system was studied by developing iso-response curves for different blast orientations.

One major parameter affecting the performance and cost of SPSW systems is the column cross-section required. In order to develop an independent new philosophy for lower ductility SPSW systems, all practical methods to reduce the column demand were explored. One such method is to reduce the strength of the infill plate by perforating the plate with a regular pattern of circular holes. In order to study the effect of the regular perforation on the column performance of the SPSW system, a standardized seismic hazard-independent method was proposed. The method evaluates the target displacement based on the yield displacement of the system and the ductility-related force modification factor. The method also takes into account the effect of pinched hysteresis cycles and cyclic strength and stiffness degradation on the target displacement. The SPSWs with perforated infill plates were then compared with similar walls with solid infill plates at an equal target displacement corresponding to the ductility-related force modification factors equal to 2.0 and 5.0. Both simple and rigid beam-to-column

connections were considered. In each case, both force-controlled actions (based on S16 and AISC 341 design requirements) and deformation-controlled actions were compared for the primary elements of the systems, with special emphasis on the compression column. The results showed that although introducing the perforations in the infill plates may reduce the lateral strength of the system, the net demand on the columns can in some cases increase, which was contrary to the intention of introducing the perforations. As such, different approaches for lower ductility walls were considered.

Due to the nature of the SPSW system, it was anticipated that simple and relatively inexpensive detailing can be used in lower ductility SPSWs and still achieve good seismic behavior. As such, a design philosophy was developed with the main emphasis on minimizing the in-place cost in a real structure, rather than providing detailing that is known to be highly robust under cyclic loading. Since the system in lower seismic regions can be designed to receive lower ductility demands, two main changes to the conventional SPSW were applied. First, minor levels of yielding in the columns were presumed to be acceptable in such applications. Second, a simple connection is proposed for the beam-to-column connections.

The simple frame joint serves the system in different ways. It reduces the cost of the system considerably by removing the detailing and inspection requirements for achieving a ductile and resilient moment-resisting connection. Also, it facilitates the application of different modular construction schemes for the SPSW system, which results in a reduction in cost of the construction. It also reduces the demand on the columns by providing hinges at the boundary frame joints instead of plastic hinges in the beams close to the face of the column when moment-resisting joints are used. The rotational flexibility of the simple connections also improves the distribution of yielding in the infill plates. These changes maximized the economic benefits of switching to the low-seismic concept for regions where such a system would suffice.

A large scale, two-story steel plate shear wall specimen was tested to evaluate the associated performance of the adapted new design philosophy for limited-ductility walls. The wall had modular construction, with no field welding and standard double-angle beam-to-column shear connections, and was tested under gravity load concurrent with reversing lateral loads. The distribution of the lateral load in the two stories represent the first mode lateral load distribution in two adjacent stories at an intermediate-height in the structure. The specimen survived 25 lateral load cycles, of which 18 were in the inelastic range. The test results indicate that excellent performance can be expected from this type of SPSW in low seismic regions, despite significantly reduced costs as compared to traditional designs.

The remainder of the seismic design part of this research was devoted to developing a performance-based capacity design method for SPSW systems. First, different methods were developed to provide reliable estimations of the boundary frame internal force demands. The axial force distribution in the beams of SPSWs is highly indeterminate. Based on the principle of capacity design and extensive nonlinear finite element simulations of wall systems, a simple and powerful analysis method was presented for evaluating the beam's axial force demand. The numerical models considered different numbers of stories, infill plate aspect ratios and thicknesses, and lateral load distributions. Also, different load transfer mechanisms to the system from the floor and roof diaphragms were considered. It was found that the axial force demands were highly dependent on the mechanism of load transfer to the system from the diaphragms and the shear force distribution in the compression column. The various components of shear force and bending moment demands on the beams of the system were also studied. A method for determining appropriate design moment and shear values was proposed for the case where the same infill plate thickness is used above and below an intermediate beam. The methods presented for moment, shear, and axial force determination were verified against experimental results for the two-story steel plate shear wall described earlier.

A new three-tier, performance-based capacity design framework for SPSWs was proposed that will accrue several technical and economic benefits and increase the competitiveness of this system in low and moderate seismic regions. Besides the ductile SPSW format, a limited-ductility and two moderately ductile SPSW concepts have been defined and developed. The performance of each wall system was evaluated in terms of redundancy (beam-to-column connection type) and ductility (yield pattern in the wall) of the overall system. As such, a new target yield mechanism concept for limited-ductility walls that departs from the usual capacity design treatment was proposed.

It is found that the traditional method of representing the infill plates as a series of pure tension strips in the direction of the tension field cannot consider the real stress state and the tension field angle in the vicinity of the boundary frame members. The method cannot consider the accelerated yielding caused by the two-dimensional stress state in the infill plate that is neglected by the strip analogy, the contribution of the minor principal stresses to the design actions on the frame, or the variability of the principal stress directions in the vicinity of the frame members. These effects tend to underestimate the axial force and overestimate the shear and bending moments in the frame elements. However, the levels of capacity design force obtained by using the methods proposed in this research tended to be acceptable and conservative for the boundary frames in most cases.

The most critical force-controlled actions in the system were identified as the moments and axial forces in the compression column. As such, specific methods developed for analysis and design of the compression column were outlined, and their effectiveness verified against test results and nonlinear numerical simulations. For the limited-ductility wall, the proposed capacity design method allows some yielding in the column and also uses the nominal yield stress of the infill plate to determine the capacity design forces on the boundary members.

The design outcomes for all three performance levels—ductile, moderately ductile, and limited-ductility—were assessed and confirmed using nonlinear finite element simulations and the results of physical SPSW tests. The performance-based capacity design methods proposed provided good estimates of member forces at the mechanism load for the different performance levels, although the column moments tended to be quite conservative.

The potential application of the SPSW as a protective system in industrial plants, when subjected to accidental explosions, was studied by means of iso-response curves. First, the relevant response limits for the SPSW system and the development of pressure-impulse ($P-I$) curves were described in detail. A comprehensive numerical model that is able to capture all critical aspects of the blast response was developed. The in-plane and out-of-plane responses were investigated separately. $P-I$ diagrams for two different-size walls were developed using the numerical model and a trial-and-error approach. They were then converted to charge weight–standoff distance curves. A method was also proposed to produce dimensionless iso-response curves by transforming a wall system into a single-degree-of-freedom system. The results showed that a properly-designed and detailed SPSW may indeed be a viable protective system for accidental blast in industrial plants such as petrochemical facilities.

8.2 Conclusions and Design Recommendations

8.2.1 Seismic versus blast design

This research studies both seismic design, for low and moderate seismic regions, and blast design of SPSW systems. In seismic design, ground motions are typically assumed to be uniform over the foundation of the structure. The seismic forces imparted to the building are proportional to its mass, and are transmitted to the structural system via inertial forces at the floor and roof diaphragms. This assumption allows the application of different powerful analysis methods, such as spectrum analysis or capacity design methods, to evaluate an upper bound of design force demands in the system. In addition, the structure is designed so that

the ductility demand is fairly uniformly distributed at each story and the entire lateral load resisting system at each story is engaged in lateral load resistance. As such, a performance-based capacity design approach was proposed for the three performance levels of SPSW systems for seismic design.

However, blast loading applies dynamic forces over a very small duration to the system, which results in a localized large deformation to the system in the vicinity of the applied blast overpressure. Unlike the seismic forces imparted to a building, the blast response decreases as the mass of the structure increases. The short duration of blast loads, and especially the localized deformation, has a major impact on the lateral load resistance of the system. First, the story shear may not necessarily be distributed to all lateral load resisting systems through the roof and floor diaphragms in proportion to their stiffness. Second, the distribution of the ductility demands at each story is not uniform, and exterior elements subjected to blast pressure sustain much larger ductility demands than the rest of the structure. As such, the resistance to blast loading is typically concentrated at elements that are subjected to the blast pressure directly. The conventional seismic design approach for estimation of design demands does not apply to blast design, and a different design method is required. In this study, the application of SPSWs to protective structures was studied via the development of $P-I$ diagrams for the wall system.

8.2.2 Test specimen design and results for limited-ductility wall

As one way of reducing the column design demand, the perforated infill plate was studied. The study shows that although the perforation reduces the lateral shear resistance of the wall, the bending moment demand on columns may not be reduced because of increased frame action and in-plane flexibility of the infill plate and its effect on inter-story lateral deformation over the height of the wall. As such, the perforation could actually worsen the beam-column interaction ratio and performance level of the column. The perforated system was also sensitive to the pattern of holes selected and a small change in the arrangement of the holes can

have a significant effect on the moment demand on the columns. The sensitivity increases when the system has lower redundancy due to the use of simple beam-to-column connections.

Based on the proposed design philosophy for limited-ductility walls, a test specimen was developed for use with common and economical fabrication methods and simple erection procedures, with input from the steel industry. The wall was tested under vertical gravity load concurrent with reversing lateral loads at each floor level. It showed stable performance at large lateral deformation ratios with high levels of ductility and energy dissipation capacity. The wall reached its maximum shear capacity of 2660 kN at a lateral drift ratio of 3.9%, which is well beyond the displacement ductilities expected from limited- and moderately ductile seismic systems. The specimen survived the lateral inter-story deformation of $8\delta_y$ (96 mm) with a stable and relatively wide hysteresis curve with an average base shear of 1900 kN, still greater than the nominal shear capacity based on design codes (S16 and AISC 341) and more than 70% of the maximum base shear achieved. Neither the one-sided lap splices in the infill plates nor the bolted nature of the system contributed to any deterioration, and they permitted the full development of the infill plate capacity in the critical story.

The conventional double-angle shear connections used in the test specimen showed remarkably robust performance with no significant damage, even at the end of the test after many nonlinear cycles. They provide rotational freedom at the beam-to-column joints, which reduces the demand on the columns as compared to the use of moment-resisting connections. The rotation also tends to improve the distribution of yielding in the infill plates, potentially enhancing the total energy dissipation capacity of the system by pushing the second story far into nonlinear response. Partial yielding occurred in the first story columns right below the beam-to-column connections, as expected. No collapse mechanism developed in the system, since the yielding was concentrated only in a very small area in the column webs—below the intermediate beam's lower flange—as the wall reached

its maximum base shear, and the plastic strains remained well below the strain-hardening value. While the use of simple connections may not be appropriate when extremely high ductility and maximum redundancy are needed, they appear to be well-suited for applications in low and moderate seismic regions.

8.2.3 Performance-based capacity design of SPSW systems

A capacity design method was proposed to evaluate the design demands in the beams of SPSWs with simple connections based on following observations. First, it was found that in different wall systems, the moment and shear distributions in the compression column are nearly identical, while changes in parameters such as the infill plate thickness or lateral load distribution over the wall height have a considerable effect on the moment and shear distributions in the tension column. This major difference between the compression and tension columns was explained by the process of tension field yielding development in the infill plates. It was also found that the lateral load transfer mechanism at the floor and roof diaphragms can have a significant effect on the axial forces in the beams. This impacts the axial force distribution in the beam and may impose a large demand on its connections.

The axial load in the beams was estimated based on following two rules, and considering the lateral load transfer mechanism to the system from the floor and roof diaphragms. First, assuming all the infill plates have yielded, the change in the axial force demand between the two ends of the beam is equal to the resultant horizontal projection of the yielded infill plate capacity above and below an intermediate beam, and below the top beam. Second, the total compression-column shear force at each connection has been estimated based on a fraction of the horizontal projection of the fully-yielded infill plate tension field force on the columns above and below the connection. Considering the above rules, the axial load distribution in the beam can be estimated from a free-body diagram.

Different sources of shear force in the beams of SPSW systems with simple frame

connections were studied, and their existences were verified by numerical results. The design shear force comes from the vertical component of the unbalanced infill plate yielding force and the induced constant shear from the horizontal component of the infill plate tension field that causes a distributed moment on the beam. The shear component due to the unbalanced infill plate force is in the same direction as the shear due to gravity loads, while the shear induced by the distributed moment is added to the other shear components at the compression-column end of the beam and subtracted from them at the tension-column end. As such, the shear reaction at the face of the compression column is the critical shear force for design.

It was shown that the net shear forces on an intermediate beam depend on the pattern of yield progression in the infill plates above and below. As such, in a cases where the infill plate in the story above an intermediate beam is thicker than 80% of the infill plate thickness in the story below, there is a chance that the upper infill plate will not yield fully under the capacity lateral loads. It is recommended that the share of the beam shear force due to the unbalanced infill plate force be calculated assuming that the upper-story infill plate has a thickness of 80% of that in the lower story and both plates yield. This 20% difference in the infill plate thickness can be assumed to apply a uniform force to the intermediate beam, which compensates for the non-uniform infill plate yielding if partial yielding occurs in the upper infill plate.

A three-tier performance-based capacity design method for SPSW systems was proposed. It is primarily the behavior of the boundary frame—beams, columns, and beam-to-column connections—that defines the performance level of the wall. Since the capacity design approach is a force method, the performance level was evaluated in terms of both redundancy and ductility of the overall system. The type of beam-to-column connection describes the redundancy of the wall, with simple and rigid connections defining the system redundancy limits, and the ductility of the wall is classified in terms of the yield mechanism that develops. In

this context, a system yield mechanism that is less ductile than the one assumed for highly-ductile walls was defined, based on observations about SPSW behavior from both tests and numerical simulations. The yield mechanism targeted performance that was considered adequate for limited-ductility applications.

Two main aspects that could result in limited-ductility yield patterns forming were identified. The high level of compressive force in the critical column of a SPSW along with large shear forces can cause a yield mechanism in the system that exhibits somewhat lower ductility. Also, when the infill plate thickness distribution over the wall height is not proportional to the shear demands imposed by the seismic loads, a lower-ductility yield mechanism can occur due to incomplete yielding in some infill plates and non-uniform lateral deformation of the compression column. By selecting smaller columns, the performance of the wall may deviate from the ductile yield pattern, yet still be quite acceptable for walls under lower ductility demands.

It was found that although the conventional tension strip analogy, commonly used in design development, provides generally conservative design forces in the boundary members, this outcome arises due to a variety of compensating factors. In the conventional tension strip analogy, the yield point of the material is determined based on uniaxial tensile response and the contribution of the minor principal stress is neglected. The reality of a two-dimensional stress state with a complex field of stresses and attendant principal stress orientations tends to accelerate the yielding, resulting in smaller stresses—particularly horizontal stresses—being applied to the columns. Also, the method neglects the contribution of the minor principal stresses to the design actions on the frame. Lastly, the variability of the principal stress directions, angle α , in the vicinity of the frame members is not captured by the tension strip model. The angle α in the case considered was approximately equal to 39° and 51° adjacent to the beams and compression column, respectively, while the code value is a good approximation for the middle of the infill plate at the mechanism load.

Even though the combination of these effects caused the axial stresses applied to the column from the infill plate to be underestimated, the axial design forces in the columns were reasonable since the same phenomena caused the shear reactions from the beams to be overestimated. Conversely, the design moments in the beams and columns of SPSWs obtained using the tension strip analogy tended to be highly conservative.

Reliable and economical capacity design provisions for limited-ductility SPSWs were developed. The provisions were defined within the system context, rather than simply being a modified version of those used to obtain highly-ductile performance. The requirements were based on observations from research specifically attuned to limited-ductility objectives, with their efficacy being verified by results of a physical test designed based on this method.

While the recommendation for limited-ductility SPSWs of using the nominal yield stress of the infill plate to determine the capacity design forces on the boundary members apparently violates the spirit of capacity design, an examination of the stresses that actually develop adjacent to the critical column at the yield mechanism load and the resulting net internal member design forces revealed that this approach is actually quite conservative. This is attributed mainly to the inaccuracies of the tension strip analogy in representing the behavior of the panels adjacent to the boundary members.

With additional research data on limited-ductility wall performance now available, and the extensive collective knowledge accumulated to date about ductile walls, design provisions for moderately ductile walls are rationalized as an additional option for designers between the two extremes. The proposed design provisions for limited-ductility and moderately ductile walls were applied to design examples, and the results were compared with numerical pushover analysis results. Very good agreements were observed in all cases. Also, the proposed methods were substantiated against experimental results on multi-story walls

subjected to cyclic loading.

8.2.4 Blast design of SPSW systems

The inherent qualities of conventional SPSWs for use as protective structures, with the additional goal of identifying where modifications are required for optimal performance in this new application, were explored. This was achieved through the development of $P-I$ diagrams.

To capture all important aspects in blast response, a comprehensive numerical model was developed. The constitutive model for the steel material includes mixed-hardening, strain-rate effects, and damage initiation and evolution. The $P-I$ diagrams for both in-plane and out-of-plane blast orientations, along with the corresponding weight–standoff distance diagrams, were produced by the developed numerical model and a trial-and-error method. Different response criteria and wall sizes were considered.

By transforming a wall system into a single-degree-of-freedom system, a dimensionless iso-response curve was proposed. The transformation is based on pushover response of the wall in each blast direction. As such, required techniques and analytical methods for pushover analysis appropriate for blast design were provided for both in-plane and out-of-plane directions. The proposed nondimensional $P-I$ diagram provided an efficient tool for preliminary design of SPSW systems to resist accidental blast loads. The study showed that to develop reliable iso-response curves, appropriate damage and failure criteria should be considered in the numerical study of the wall system. This is especially important in structural steel members, since their slenderness makes them vulnerable to heavy blast overpressure, even with very small duration.

The results showed that despite the inherent slenderness of the steel members, the wall system had the potential to be an effective system for use in a protective structure for industrial plants, especially for the in-plane blast load condition. The

side wall subjected to in-plane blast load is a strong and reliable system, and the front wall subjected to out-of-plane blast load can be sized to provide acceptable design for industrial plant applications.

8.3 Recommendations for Further Research

The results for SPSWs with perforated infill plates showed an inherent uncertainty in the response such that a minor change in perforation pattern may cause considerable differences in column moment demand in some regions. The effect of the perforations needs to be further investigated at different performance levels in walls with different geometries, numbers of stories, panel aspect ratios, perforation patterns, and boundary frame cross-sections.

A method was proposed to evaluate the capacity-design-level axial force demands applied to the beams of SPSWs with simple beam-to-column connections. In principle, the method is also applicable to SPSWs with moment-resisting beam-to-column connections. However, the values of the net compression-column shear forces at the frame joints need to be established for such an application, as they are affected considerably by the moments at the beam-to-column connections.

It was shown that although the axial compressive force in the column of a SPSW due to the infill plate stresses is underestimated by the application of the tension strip analogy, the overall axial force in the compression column from capacity design is slightly conservative. This is due primarily to the fact that the contribution of the beam shear reaction to the column axial force is overestimated by the same phenomena. However, the overestimation of this component of the column compression force needs to be investigated for tall and narrow walls, where the beam shear force overestimation is reduced by the short beam span and the column axial force underestimation is increased by the larger number of stories.

The physical test on the limited-ductility wall conducted as part of this research

program showed very promising results for SPSWs with simple frame connections. As such, a moderately ductile wall option has been proposed based on simple beam-to-column connections, with the wall being designed to develop a uniform yield mechanism. It is believed that such a SPSW system has the potential to provide performance that rivals even that of ductile SPSWs that, based on current design standards, necessitate the use of rigid beam-to-column connections with ductile seismic detailing and the attendant fabrication and inspection requirements. Further experimental investigation is recommended in order to prove its competency as a fully-ductile system.

Additional research is required to study the effects of various parameters that were not considered in the blast design study presented. Low-cycle fatigue failure has not been considered in the analysis and may have some effect in the overall response of the SPSW system under far-field accidental blast load. To optimize the system and improve its reliability, improvements in detailing that are developed explicitly for blast-resistant applications are needed.

Appendix. Test Specimen Information

1 Tension Coupon Test Results

Table A.1 Infill plate, column flange, column web, and angle connection tension coupon test results

Coupon Mark	Elastic Modulus	Mean Static Yield Stress	Static Ultimate Stress	Failure Stress	Yield Strain	Hardening Strain	Ultimate Strain	Failure Strain (Elongation)	Upper Yield Stress	Lower Yield Stress	Static Yield Stress 1	Static Yield Stress 2	Static Yield Stress 3	Reduction of Area
	GPa	MPa	MPa	MPa	%	%	%	%	MPa	MPa	MPa	MPa	MPa	%
Infill Plate														
IP1 ^a	198.47	326.1	468.5	369.3	N/A	N/A	22.6	37.1	N/A	N/A	321.2	324.1	332.9	27.7
IP2	198.81	333.6	477.8	384.7	0.179	2.02	23.5	35.4	369.4	355.1	336.1	328	336.6	30.1
IP3	197.19	346.5	473.8	403.6	0.182	2.29	20.1	29.9	371.9	363.9	345.6	346.1	347.7	28.3
IP4	203.53	347.1	470.3	393.5	0.180	2.07	23.0	32.5	373.6	361.2	347.8	349.2	344.2	27.0
Mean	199.50	338.3	472.6	387.8	0.180	2.13	22.3	33.7	371.6	360.1	337.68	336.85	340.4	28.3
SD	2.78	10.26	4.11	14.54	0.00	0.14	1.51	3.21	2.13	4.48	12.10	12.64	6.79	1.32
COV	1.39	3.03	0.87	3.75	0.85	6.80	6.78	9.52	0.57	1.24	3.58	3.75	2.00	4.65
Column Flanges														
CF1	191.86	341.5	466.7	324.9	0.19	1.96	22.9	42.8	373.0	357.4	343.2	340.2	341.0	33.4
CF2	199.22	349.8	458.7	318.9	0.2	2.10	23.3	43.7	379.4	368.8	353.7	351.5	344.3	34.4
CF3	196.13	346.9	464.3	319.6	0.200	2.16	21.1	42.9	384.2	364.3	350.2	346	344.5	33.1
CF4	187.72	352.3	462.0	317.3	0.2	2.02	20.6	42.4	392.5	370.4	354.9	352.2	349.9	32.5
Mean	193.73	347.6	462.9	320.2	0.195	2.06	22.0	42.9	382.3	365.2	350.5	347.5	344.9	33.3
SD	5.02	4.67	3.41	3.29	0.01	0.09	1.33	0.56	8.22	5.82	5.26	5.59	3.68	0.78
COV	2.59	1.34	0.74	1.03	3.07	4.28	6.03	1.30	2.15	1.59	1.50	1.61	1.07	2.33
Column Web														
CW1	196.77	389.2	478.0	314.4	0.200	1.80	13.5	35.6	416.2	400.3	388.5	389.7	389.5	34.7
CW2	189.89	381.8	471.9	318.6	0.2	1.60	13.1	36.4	N/A	N/A	381.1	381.3	382.9	33.6
Mean	193.33	385.5	475.0	316.5	0.200	1.70	13.3	36.0	416.2	400.3	384.8	385.5	386.2	34.2
SD	4.86	5.28	4.31	2.97	0.00	0.14	0.25	0.56	N/A	N/A	5.23	5.94	4.67	0.83
COV	2.52	1.37	0.91	0.94	0.35	8.27	1.90	1.56	N/A	N/A	1.36	1.54	1.21	2.43

Angle														
Short leg														
SL2	196.68	342.8	511.8	378.8	0.19	1.46	17.2	35.8	375.2	356.3	341.5	342.8	344.0	31.8
Long leg														
LL1 ^b	196.40	N/A	504.1	374.7	N/A	N/A	17.8	36.2	N/A	N/A	N/A	N/A	N/A	30.9
LL2	198.52	338.5	499.3	374.6	0.18	1.50	20.1	37.8	354.2	353.0	338.2	338.6	338.8	26.8
Mean	197.20	340.7	505.1	376.0	0.185	1.48	18.4	36.6	364.7	354.7	339.9	340.7	341.4	29.8
SD	1.15	2.99	6.31	2.40	0.00	0.03	1.53	1.02	14.85	2.33	2.33	2.97	3.68	2.64
COV	0.58	0.88	1.25	0.64	1.92	1.82	8.35	2.78	4.07	0.66	0.69	0.87	1.08	8.84

^a Test results are not available at the yield point

^b Test results are not available at the yield point and yield plateau

2 Structural Drawings and Modular Construction of the Test Specimen

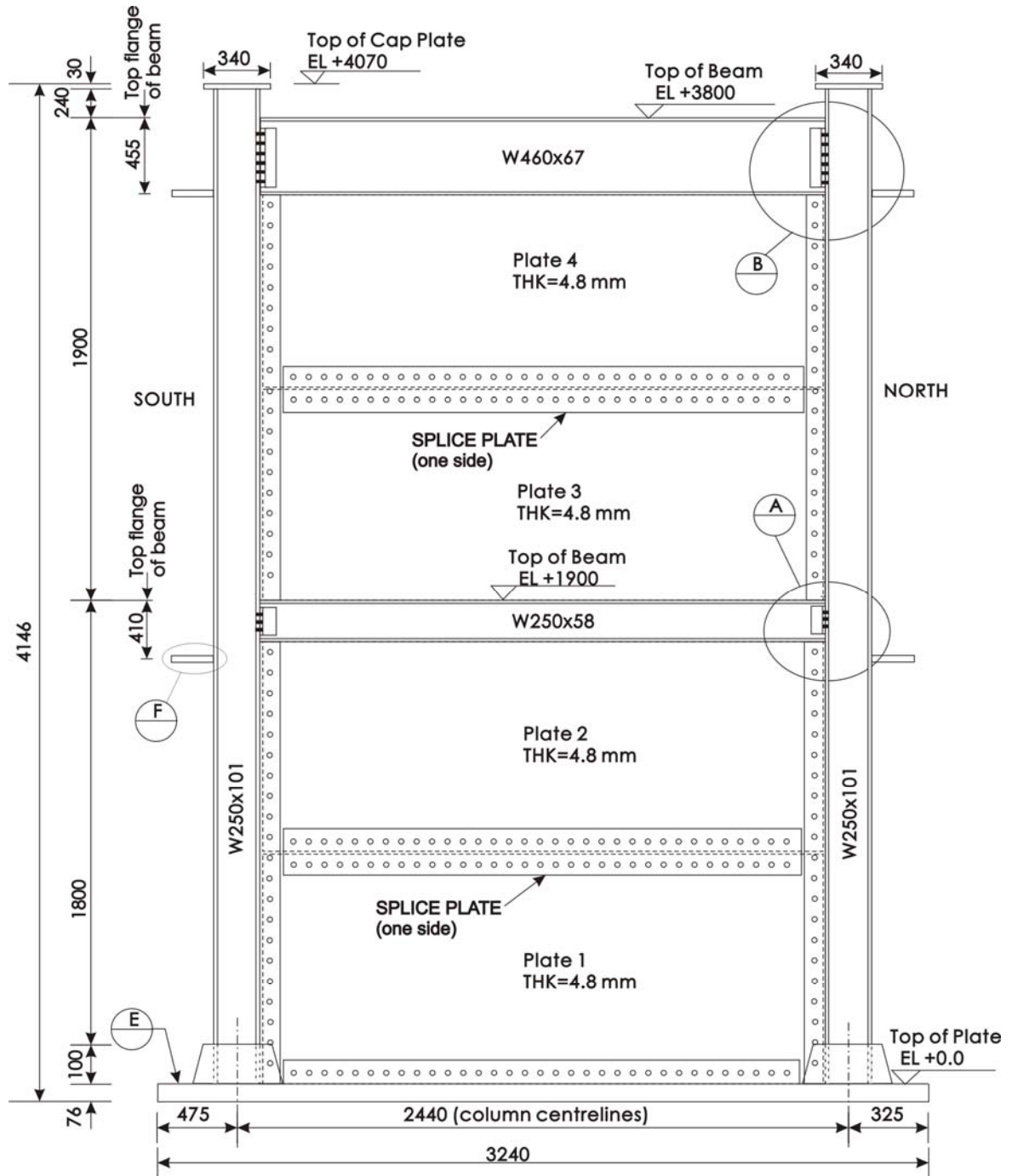


Fig. A.1 Elevation of the test specimen

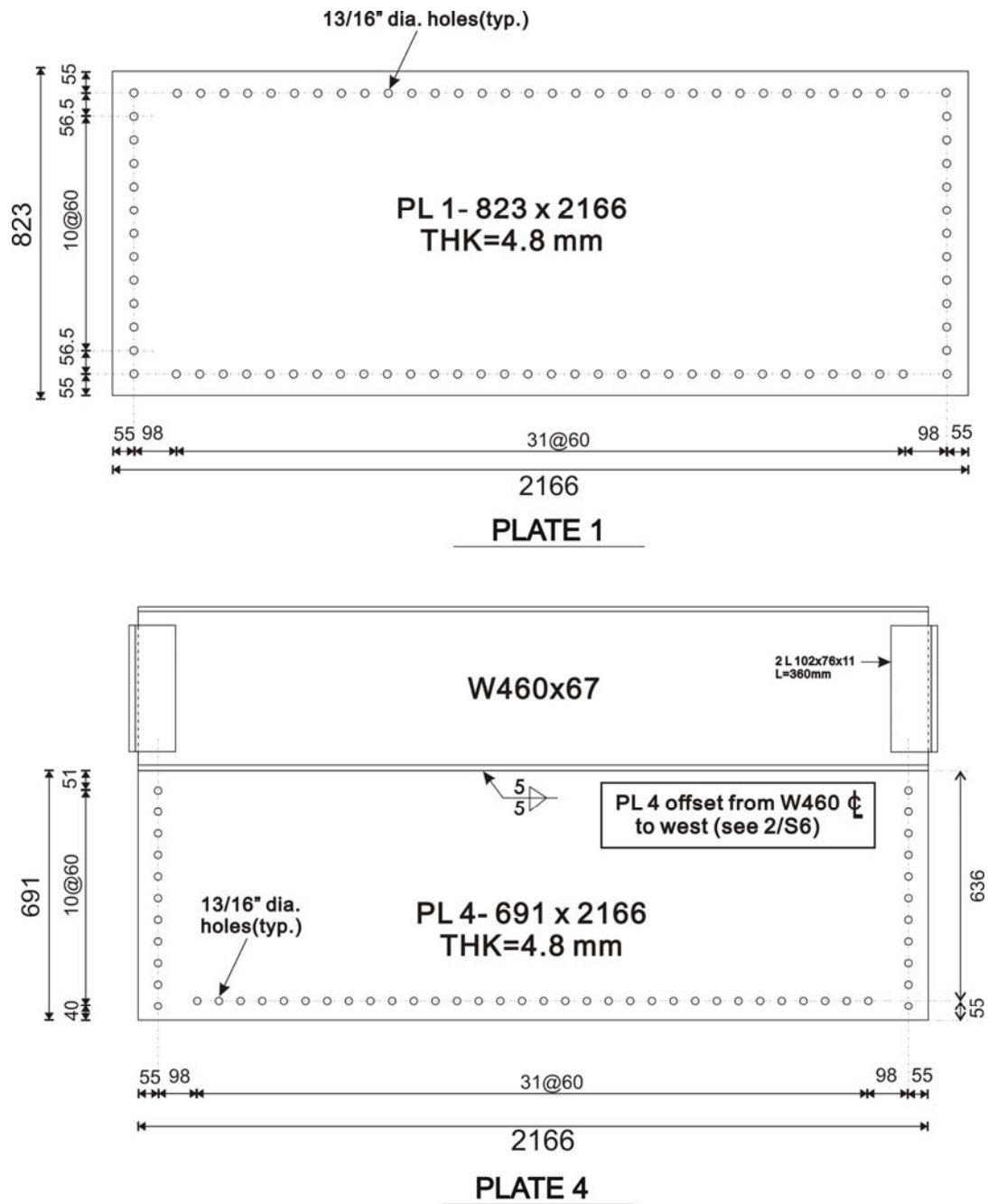


Fig. A.2 Top and bottom modules

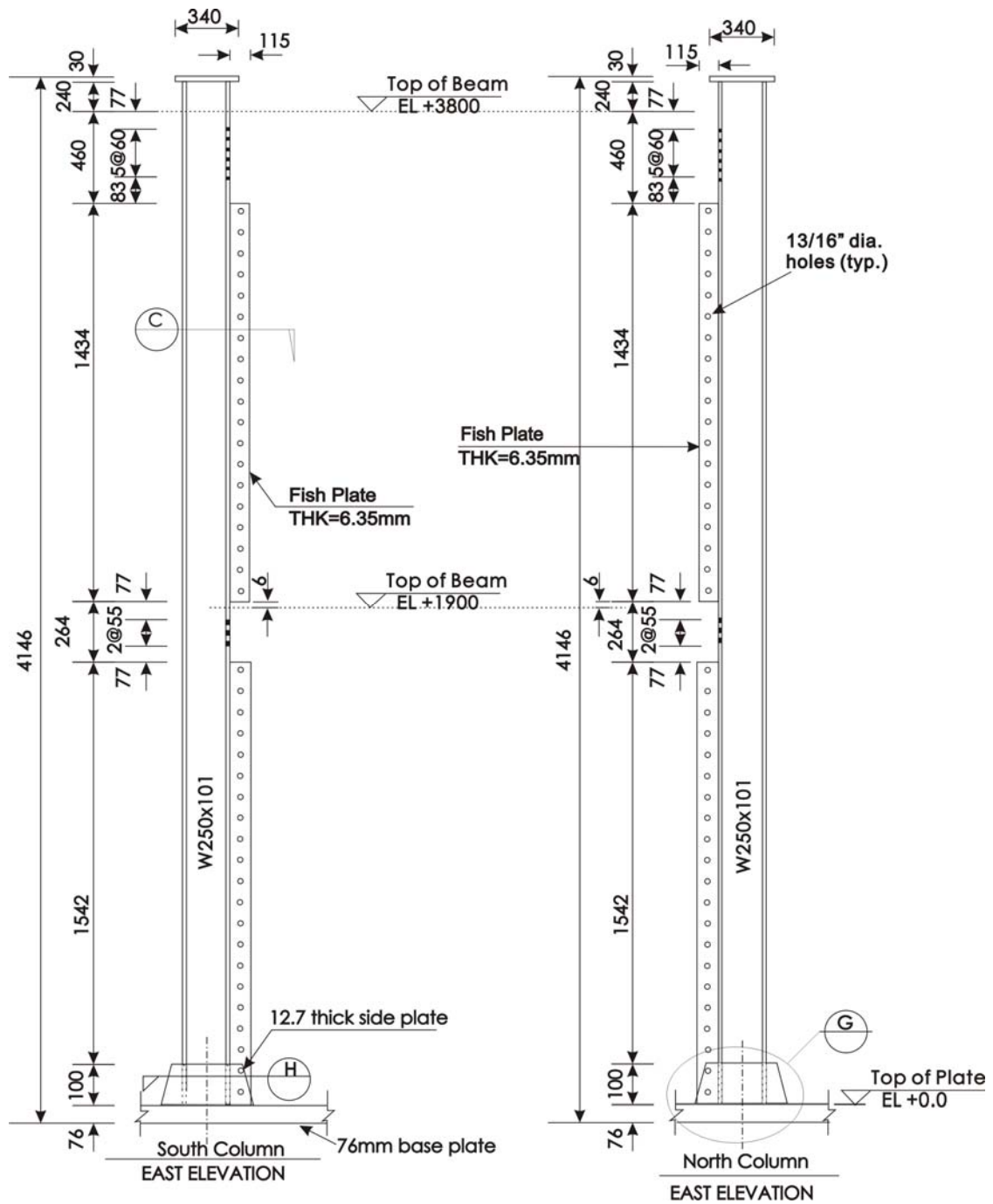


Fig. A.4 Column elevations

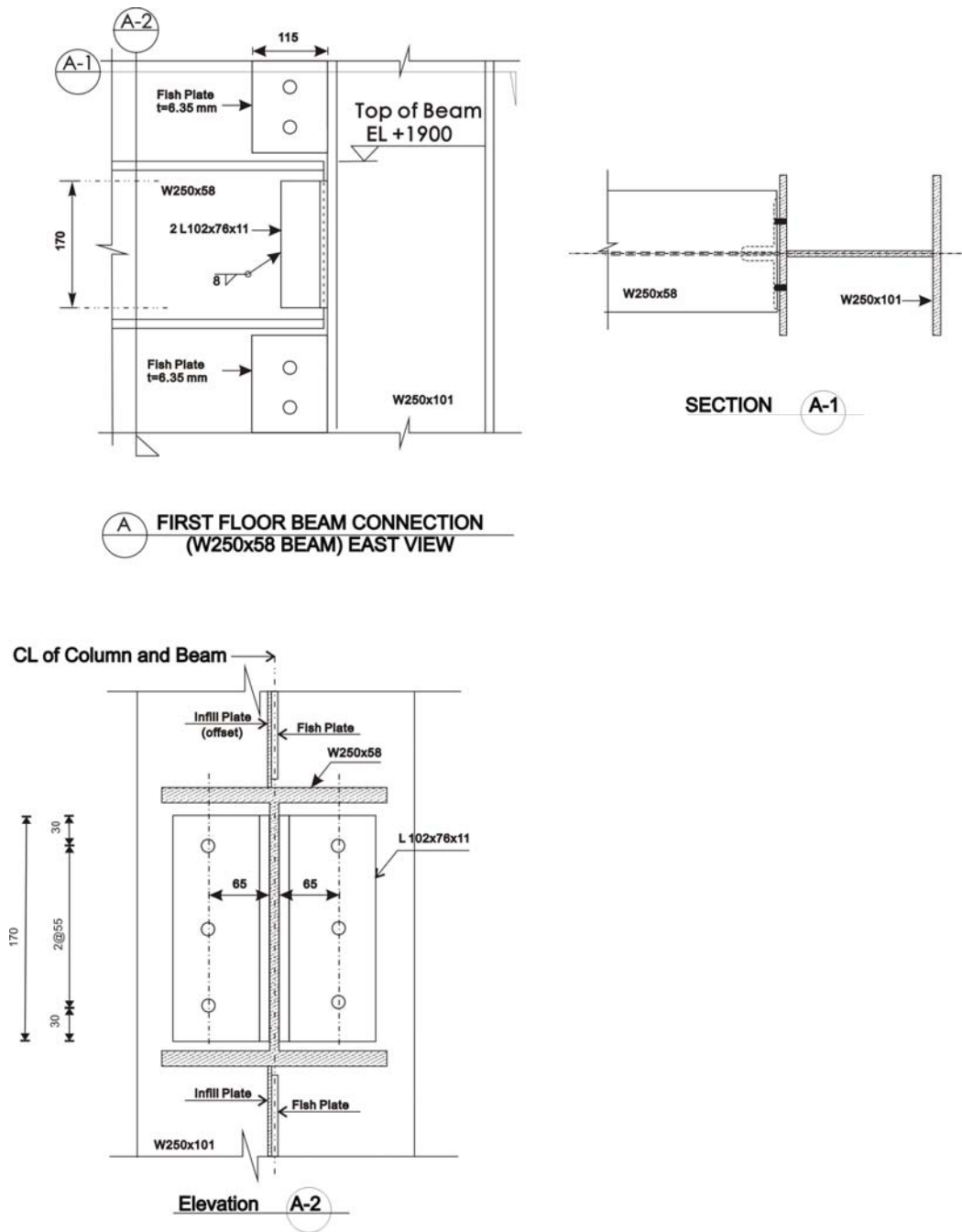


Fig. A.5 Intermediate beam-to-column connection details

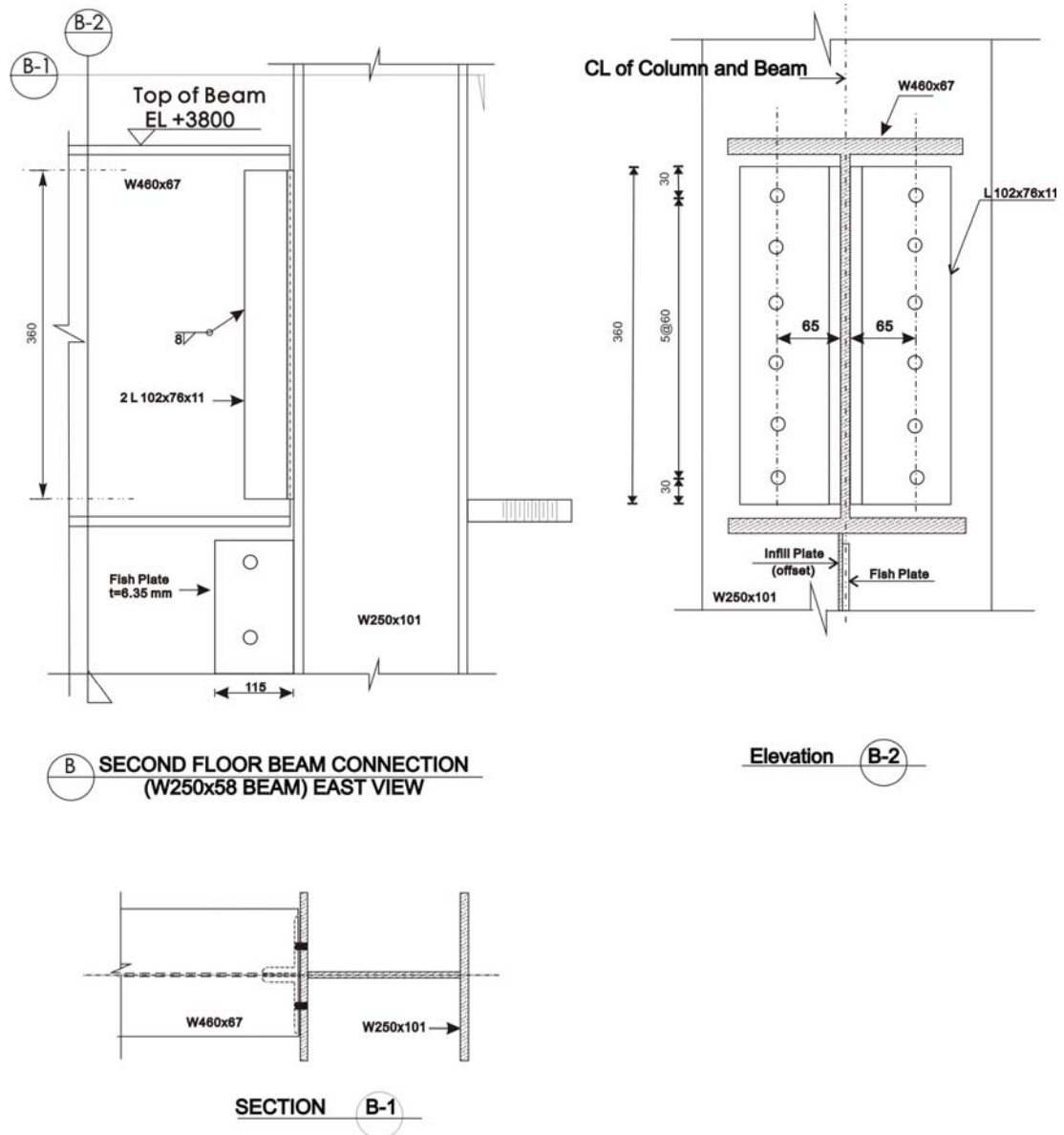


Fig. A.6 Top beam-to-column connection details

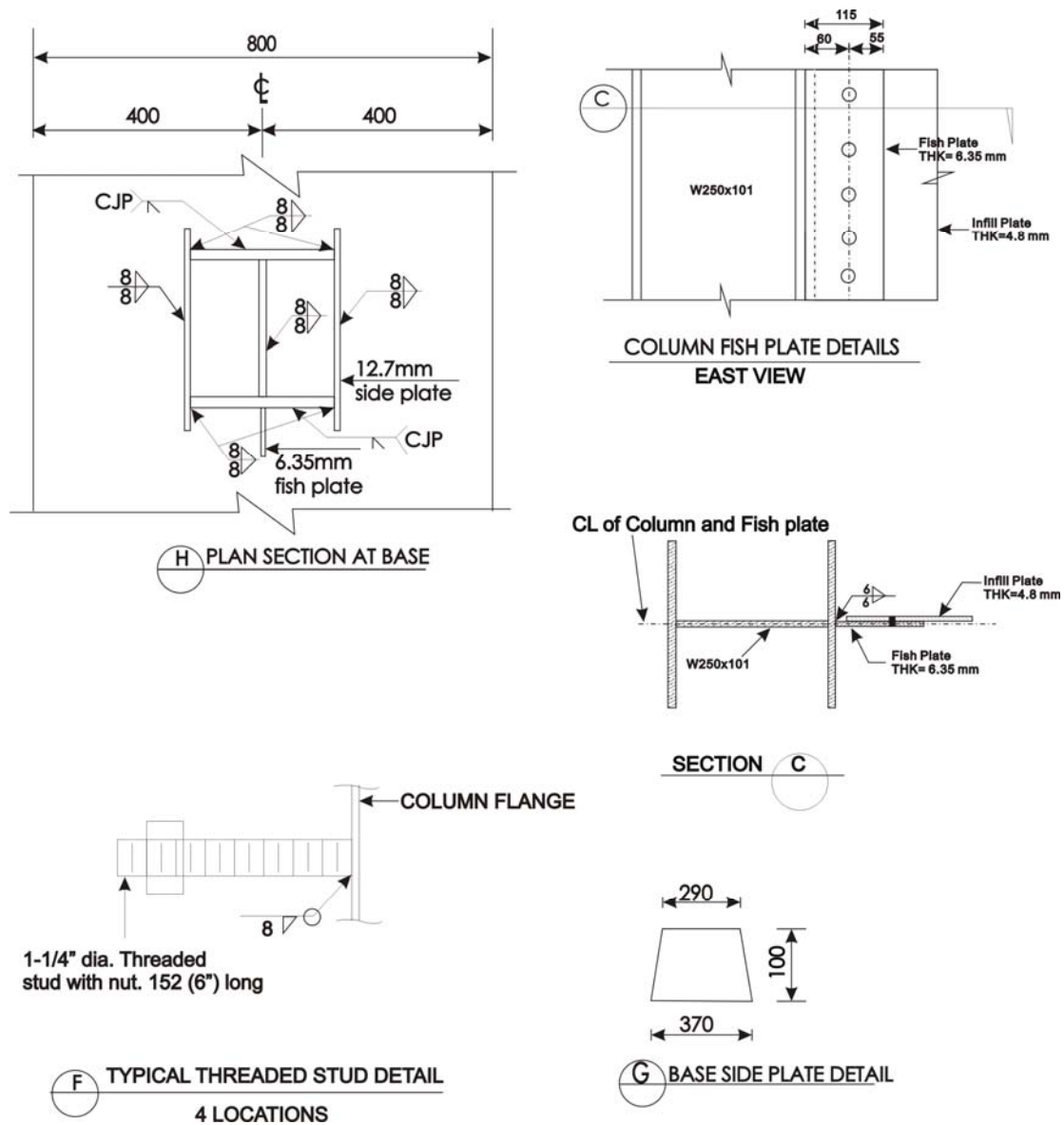


Fig. A.7 Details of column attachments

3 Test Setup and Instrumentation

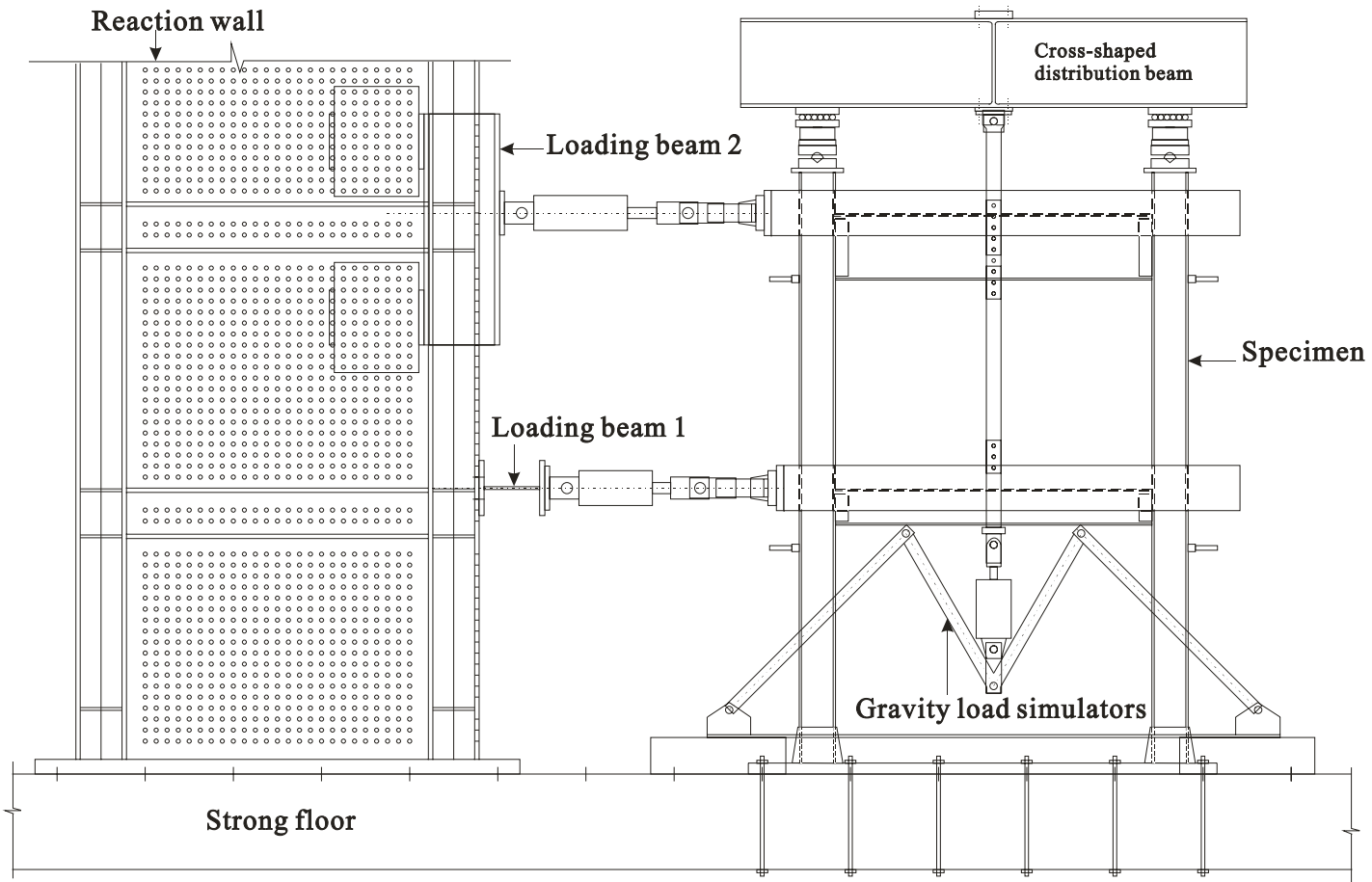


Fig. A.9 Test setup

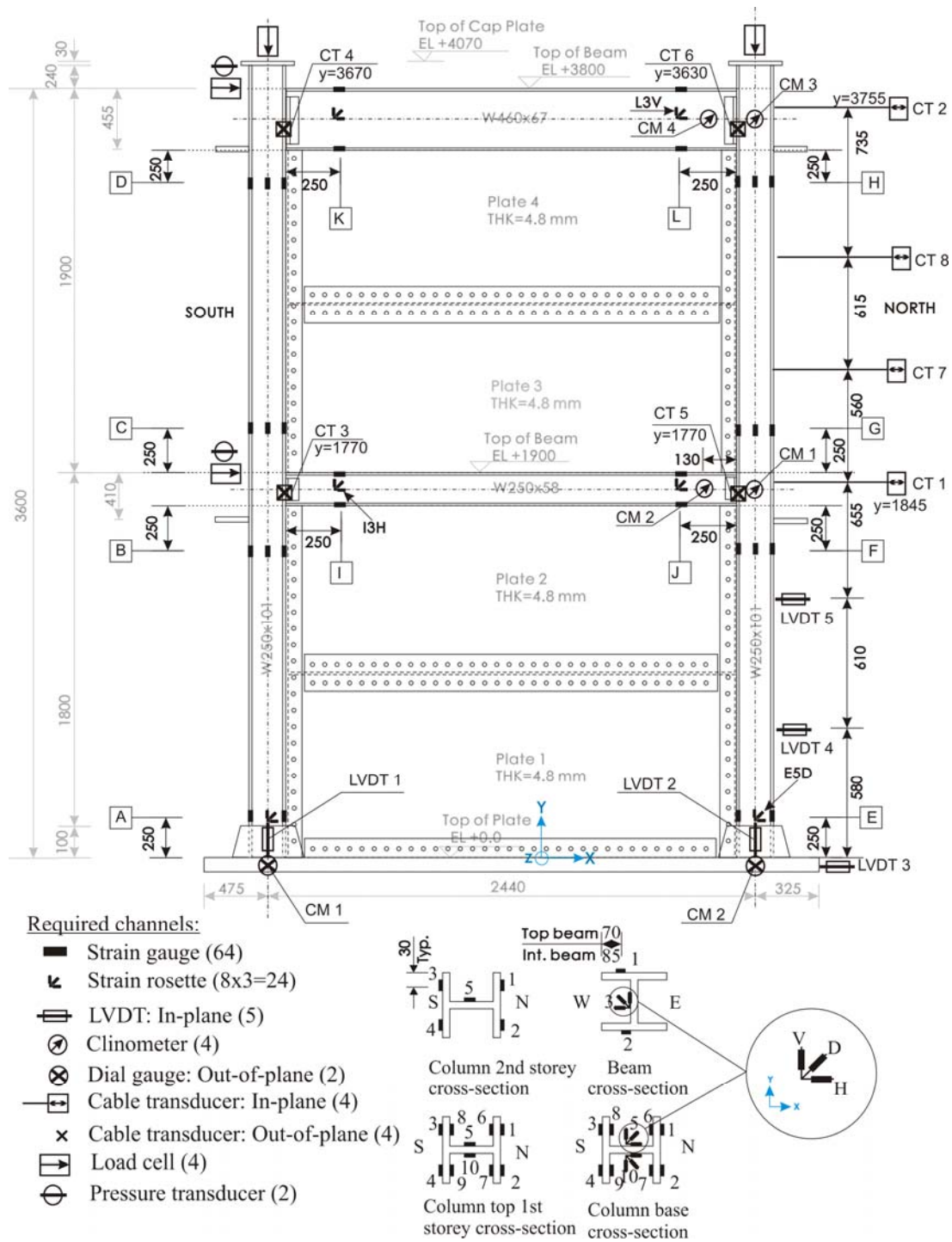


Fig. A.10 Test specimen instrumentation

4 Numerical Model Verification

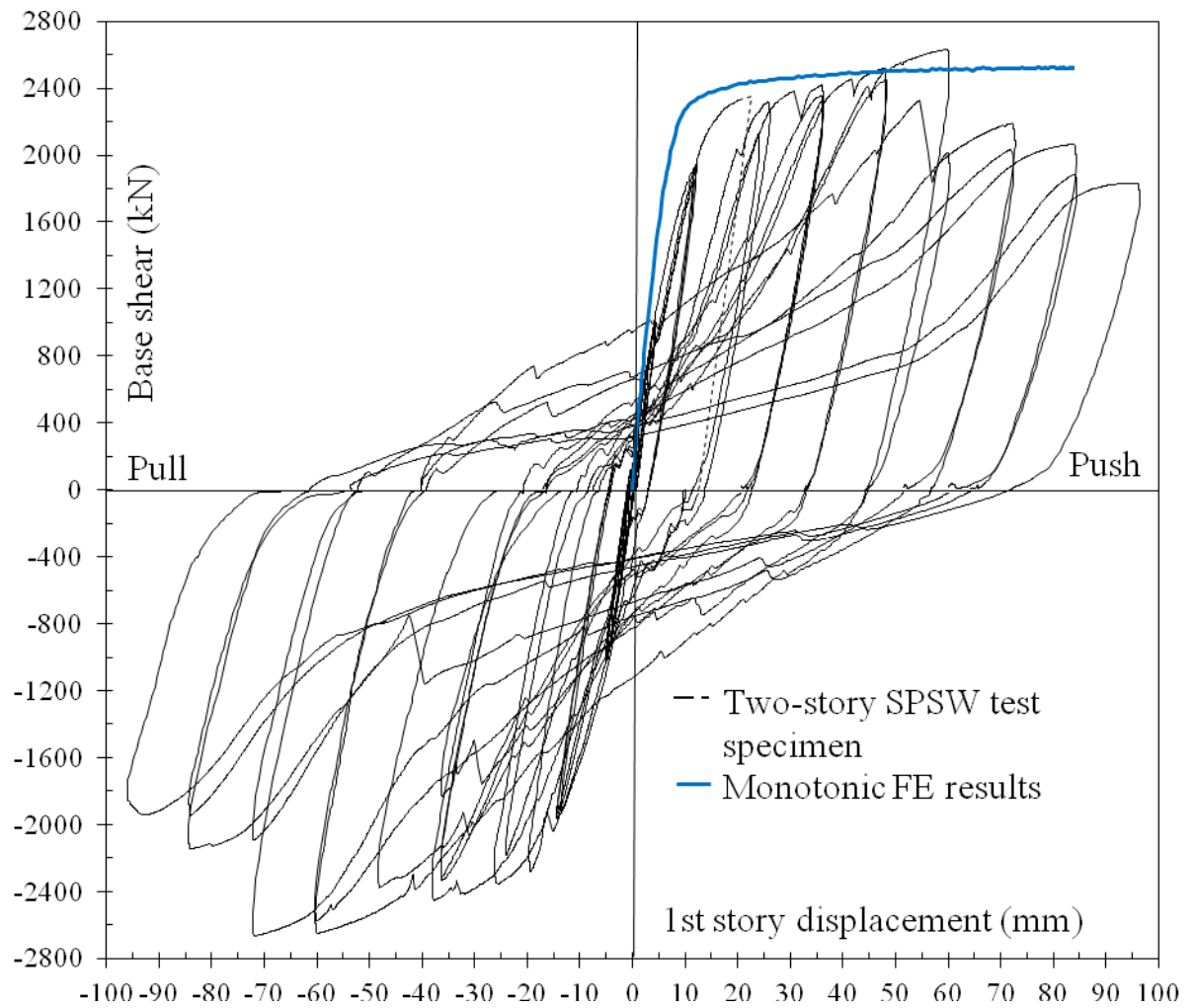


Fig. A.11 Hysteresis curves for first story lateral displacement versus base shear

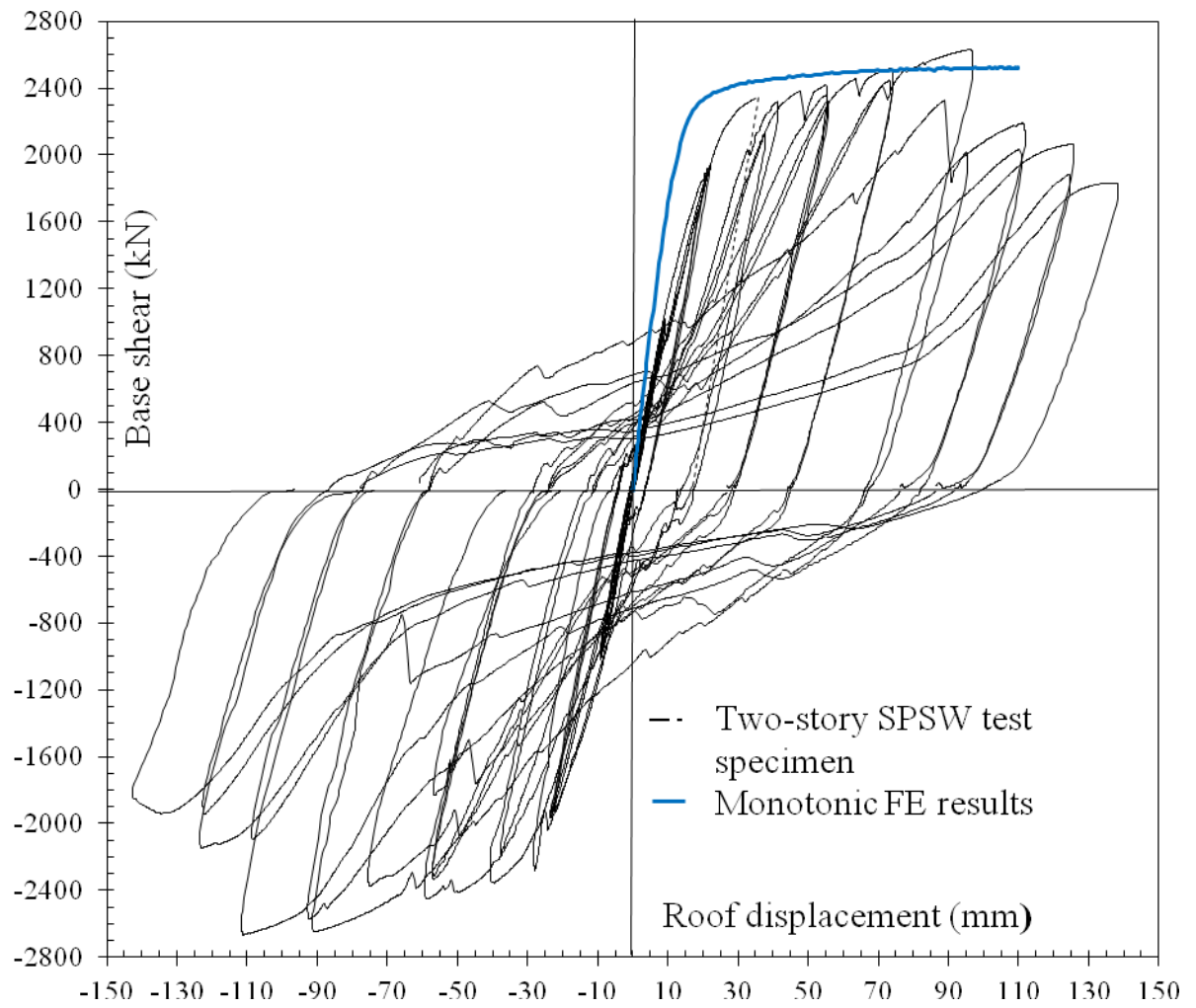


Fig. A.12 Hysteresis curves for roof lateral displacement versus base shear

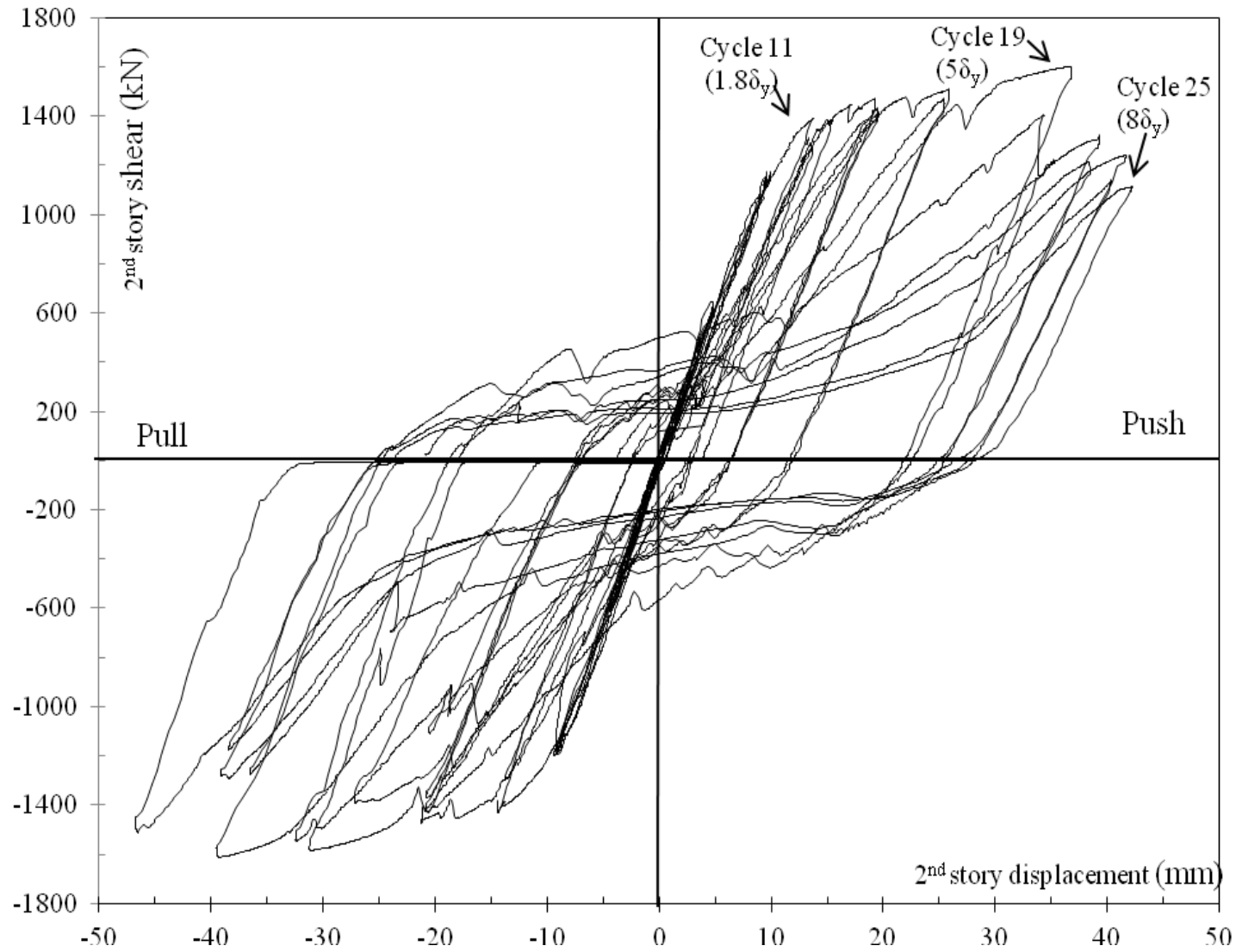


Fig. A.13 Hysteresis curves for second story lateral displacement versus second story shear

5 Test Photographs



(a) Before test (also shown in Chapter 3)

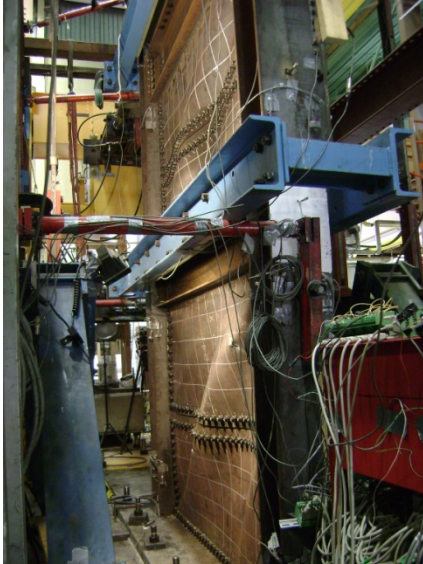


(b) Cycle 19-push



(c) Cycle 19-pull

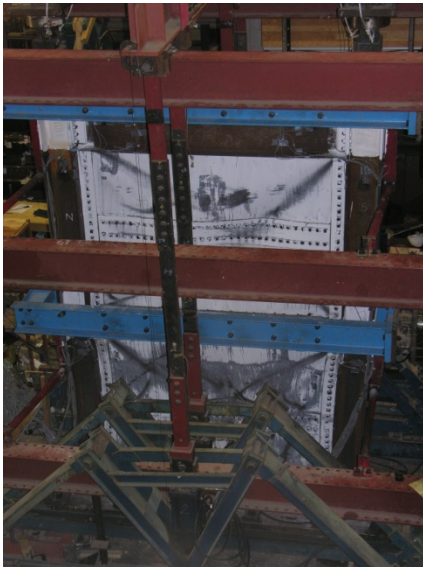
Fig. A.14 Elevation of test specimen at different stages of test



(d) Cycle 21-push



(e) Cycle 21-pull



(f) After test



(g) After test

Fig. A.14 Elevation of test specimen at different stages of test (cont.)



(a) North top connection



(b) South top connection

(also shown in Chapter 3↓)



(c) North intermediate connection



(d) South intermediate connection

Fig. A.15 Beam-to-column connections before the test



(a) North intermediate connection, Cycle 10-Pull



**(b) North top connection,
Cycle 15-Push**



**(c) South top connection,
Cycle 15-Push**

Fig. A.16 Beam-to-column connections during the test



**(d) North intermediate connection,
Cycle 15-Push**



**(e) South intermediate connection,
Cycle 15-Push**

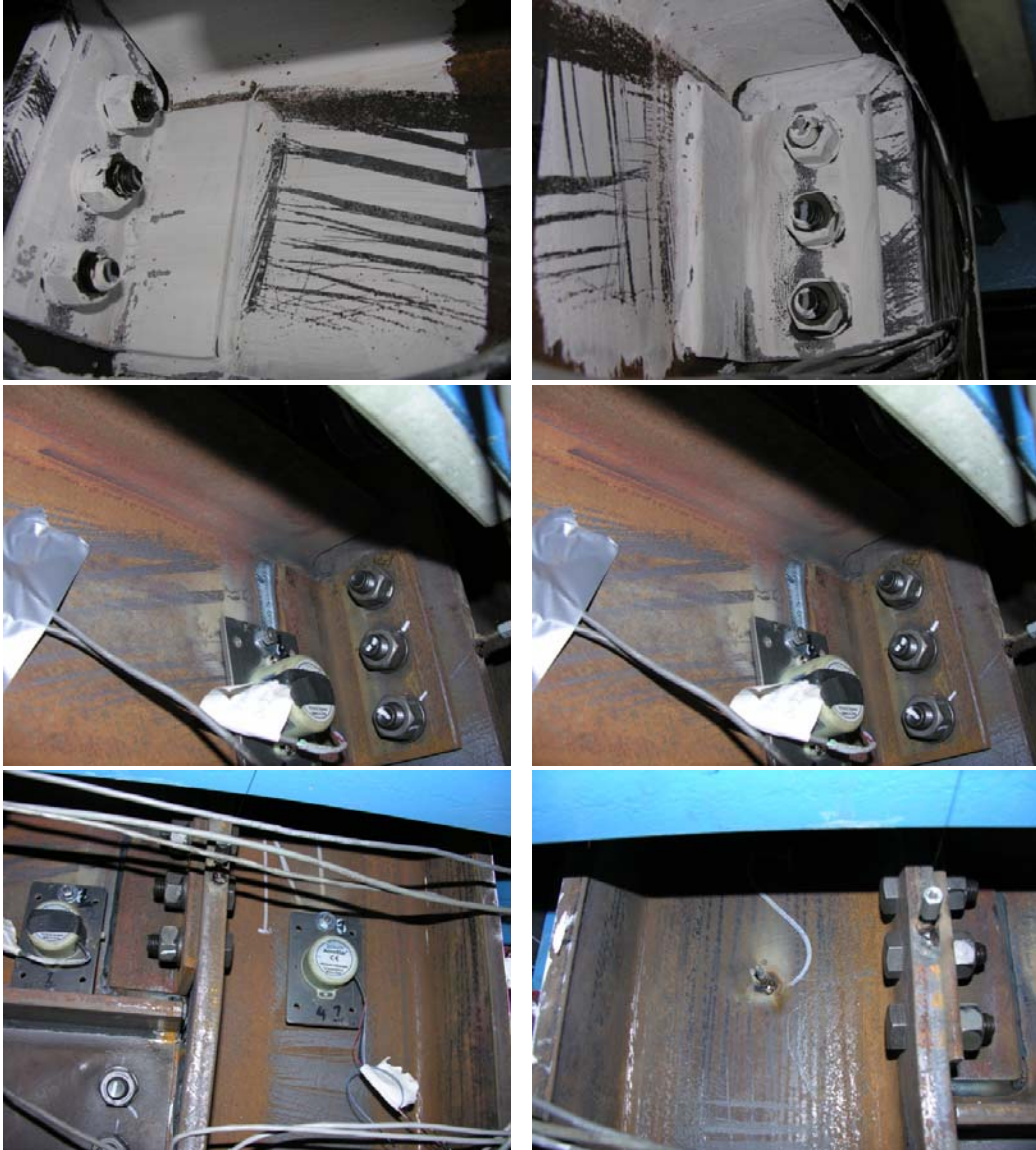


**(f) North top connection,
Cycle 18-Unload**



**(g) South top connection,
Cycle 18-Unload**

Fig. A.16 Beam-to-column connections during the test (cont.)



**(h) North intermediate connection,
Cycle 18-Unload**

**(i) South intermediate connection,
Cycle 18-Unload**

Fig. A.16 Beam-to-column connections during the test (cont.)



**(j) North top connection,
Cycle 20-Push**



**(k) South top connection,
Cycle 20-Push**



**(l) North intermediate connection,
Cycle 20-Push**



**(m) South intermediate connection,
Cycle 20-Push**

Fig. A.16 Beam-to-column connections during the test (cont.)



**(n) North intermediate connection,
Cycle 20-Pull**



**(o) South intermediate connection,
Cycle 20-Pull**



**(p) North top connection,
Cycle 22-Unload**



**(q) South top connection,
Cycle 22-Unload**

Fig. A.16 Beam-to-column connections during the test (cont.)



**(r) North intermediate connection,
Cycle 22-Unload**



**(s) South intermediate connection,
Cycle 22-Unload**



**(t) North intermediate connection,
Cycle 25-Push**



**(u) South intermediate connection,
Cycle 25-Push**

Fig. A.16 Beam-to-column connections during the test (cont.)



(v) North top connection, Cycle 25-Pull



**(w) North intermediate connection,
Cycle 25-Pull**



**(x) South intermediate connection,
Cycle 25-Pull**

Fig. A.16 Beam-to-column connections during the test (cont.)



(a) North top connection



(b) South top connection



(c) North intermediate connection



(d) South intermediate connection

(Also shown in Chapter 3↑)

Fig. A.17 Beam-to-column connections after the test