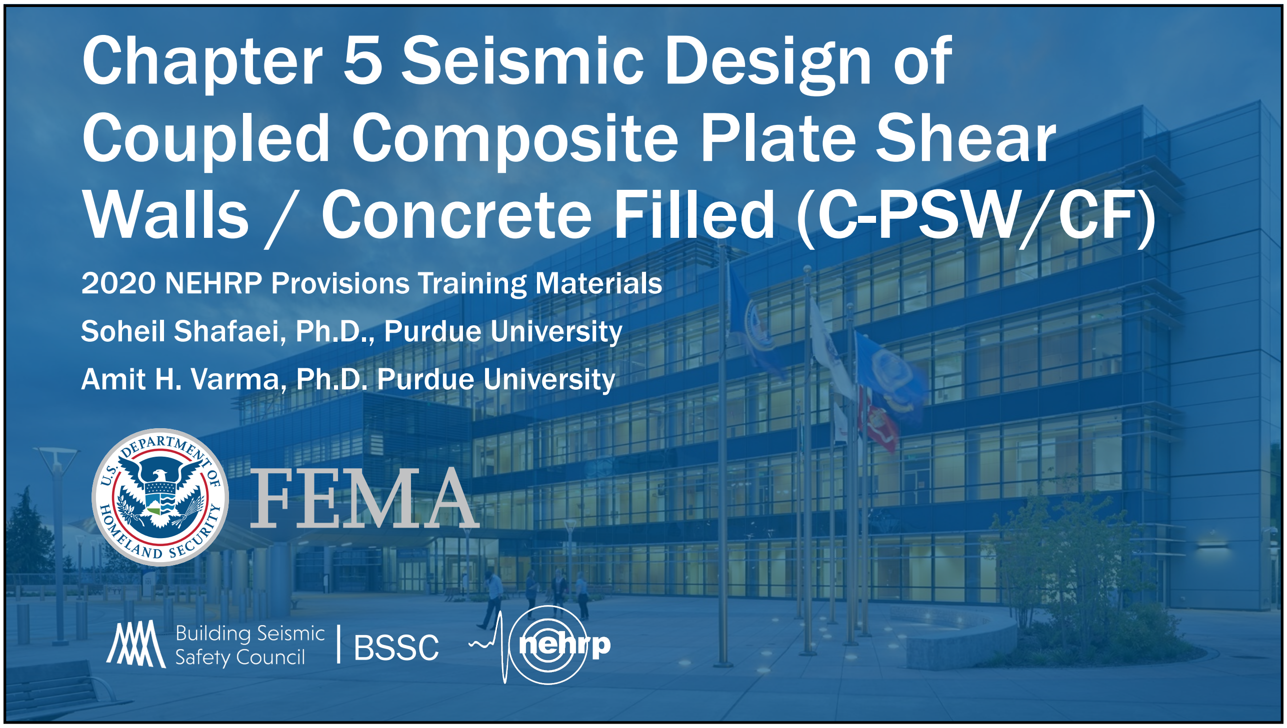

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Topics Covered

- Introduction to Coupled C-PSW/CFs (SpeedCore System)
- Section Detailing, Limits, Requirements
- Seismic Behavior & Capacity Design
- Design Example

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Introduction to Coupled C-PSW/CFs (SpeedCore System)

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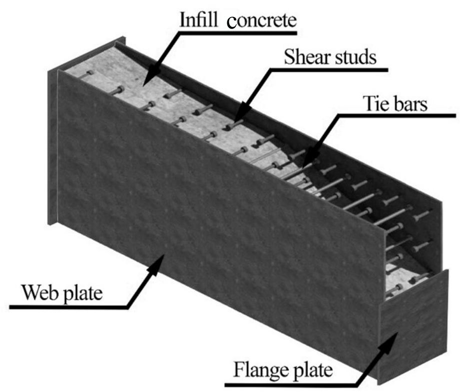
C-PSW/CF (SpeedCore System)

Composite Plate Shear Walls – Concrete Filled (C-PSW/CF)

- Steel plates
- Concrete infill
- Tie bars
- Shear studs
- No rebars or formwork

- Shear walls and/or elevation core walls



(Shafaei et al., 2021)

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A New Chapter in Composite Construction

Rainier Square, Seattle

- Client — WRIGHT RUNSTAD & COMPANY
- Architect — nbbj
- Structural & Civil — MAGNUSSON KLEMENCIC ASSOCIATES Structural + Civil Engineers
- GC/GM — LEASE CRUTCHER LEWIS

Courtesy of Magnusson Klemencic Associates

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A New Chapter in Composite Construction

- Steel Fabricator: SUPREME GROUP
- Steel Erector: THE ERECTION CO., INC.
- Rebar Fabricator: Farwest Steel Corporation
- Concrete Supplier: STONEWAY CONCRETE

100 YEARS 1920·2020 MAGNUSSON KLEMENCIC ASSOCIATES

Courtesy of Magnusson Klemencic Associates

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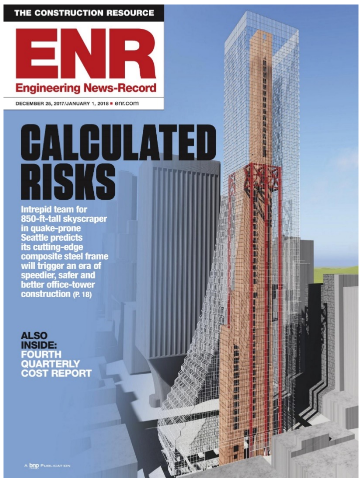
A New Chapter in Composite Construction



Cover of ENR Magazine

Constructed in 10 months

Eight months savings as compared to conventional RC construction

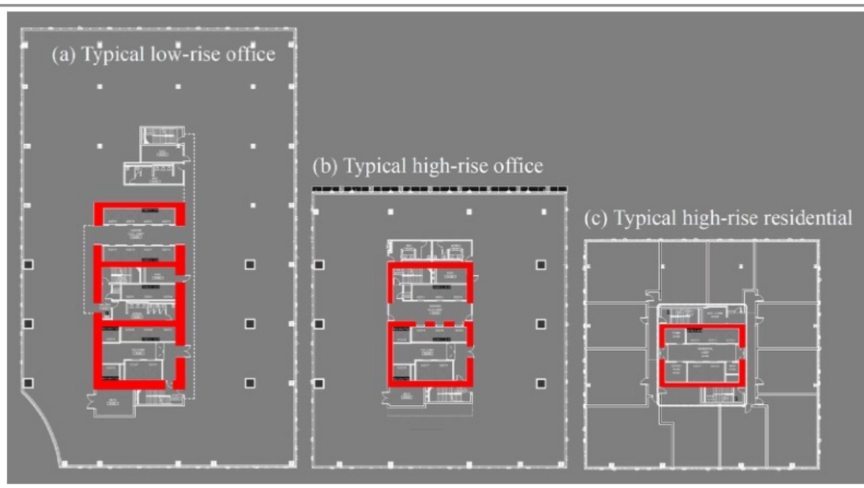
1.4 million square feet

850-feet tall

58-story office + residential

7 levels below-grade parking

Coupled Composite Plate Shear Walls – Core Walls



Courtesy of Magnusson Klemencic Associates

A New Chapter in Composite Construction

Copyright Gensler
(Modern Steel Construction, February 2021)

200 Park Avenue, San Jose, CA

- High seismic region
- 937,000 square foot
- 19 stories
- Under construction

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Section Detailing, Limits, Requirements

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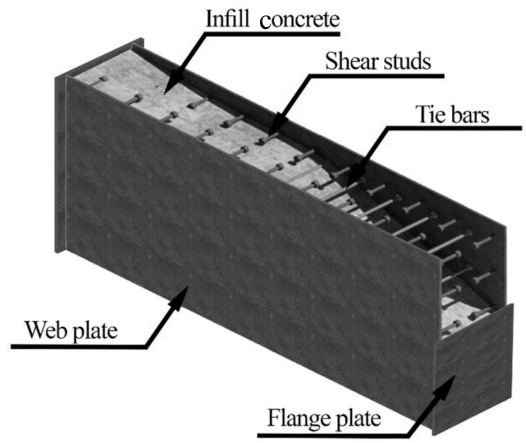
Key Components of C-PSW/CF (SpeedCore System)

- Steel plates
- Concrete infill
- Tie bars
- Shear studs



(Shafaei et al., 2021)

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Steel Plates

- Reinforcement ratio limits:

  Minimum = 1% Maximum = 10%

- Two steel plates must be connected to each other using ties
- Ties can consist of bars, steel shapes, or built-up shapes
- Steel plates must be anchored to concrete infill using stud anchors or ties or combination of ties and studs

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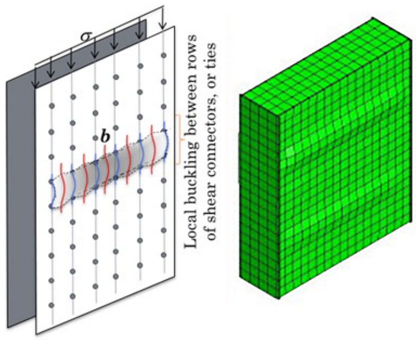
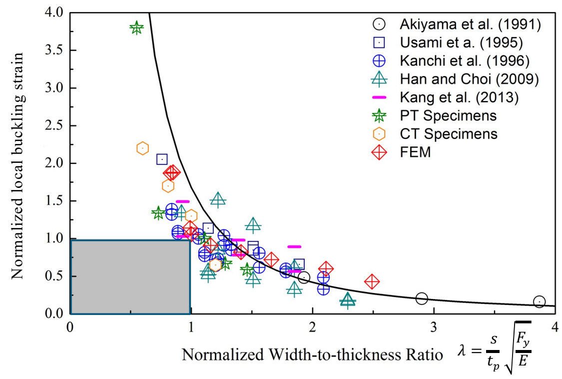
Local buckling, Plate Slenderness, Axial Compression



(Zhang et al. 2014, 2020)



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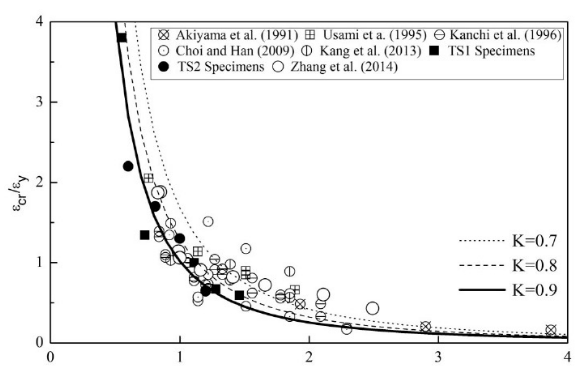
Local buckling, Plate Slenderness, Axial Compression

Seismic Design:

$$\frac{b}{t_p} \leq 1.05\sqrt{\frac{E_s}{R_y F_y}}$$

$$F_{cr} \geq F_y$$

$$P_{no} = A_s F_y + 0.85 f_c' A_c$$



(Zhang et al. 2020)

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Local buckling, Plate Slenderness, Axial Compression

- In accordance with AISC 341-22 Section H7.5s, steel plate slenderness ratio at the base of C-PSW/CF (protected zones) should be limited as follows:

$$\frac{S}{t_p} < 1.05\sqrt{\frac{E_s}{R_y\,F_y}}$$

- Steel plate slenderness ratio at regions, which are protected zones should be limited as follows:

$$\frac{S}{t_p} < 1.2\sqrt{\frac{E_s}{F_y}}$$

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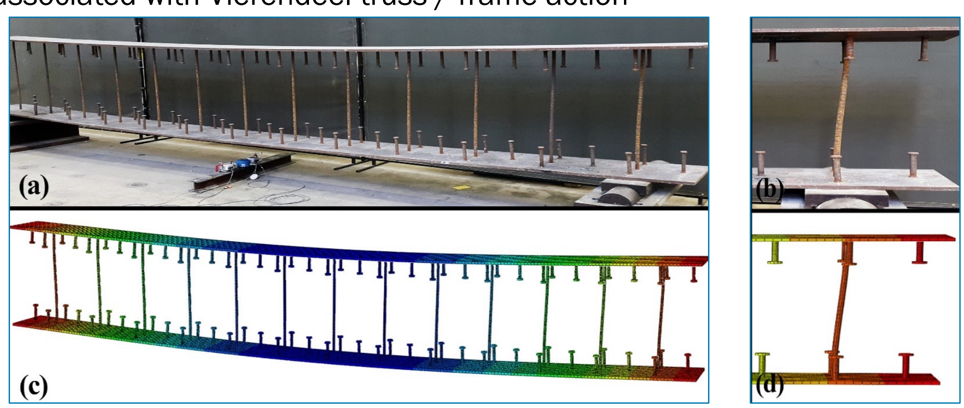
Tie Bar Size, Spacing, and Stability of Empty Modules

- Empty steel module flexibility governed by effective shear stiffness $(GA)_{eff}$ associated with Vierendeel truss / frame action



(Varma et al., 2019)

$$\Delta_{total} = \frac{5\times wL^4}{384\times EI_{total}} + \boxed{\frac{wL^2}{8\times GA_{eff}}}$$ dominates

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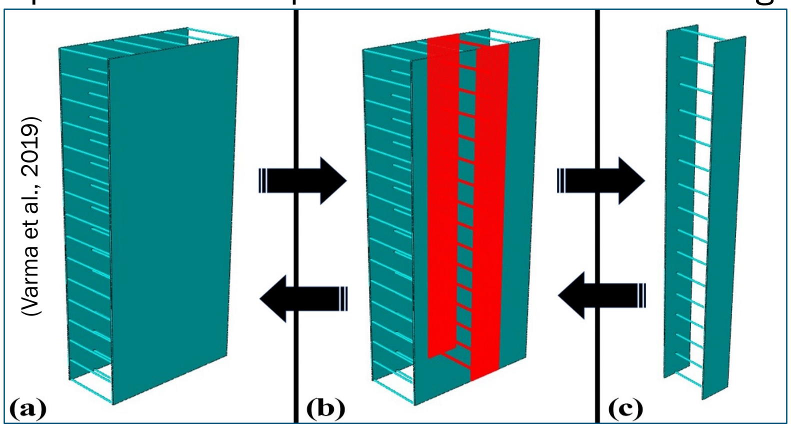
Tie Bar Size, Spacing, and Stability of Empty Modules

- Stability of empty modules during erection, construction and concrete placement → important consideration for design



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Tie Bar Size, Spacing, and Stability of Empty Modules

- Minimum $(GA)_{eff}$ of empty module for transportation, erection, and stability during construction, concrete casting
- Refined calculations can be made using theory
- Recommendations for tie bar size

$$\frac{S}{t_p} < 1.0\sqrt{\frac{E_s}{2\propto\ +1}}$$

Where, $\propto = 1.7\left(\frac{t_{sc}}{t_p} - 2\right)\left(\frac{t_p}{d_{tie}}\right)^4$

- α is the ratio of plate flexural stiffness to tie flexural stiffness
- α governs the value of $(GA)_{eff}$, and thus the tie spacing S/t_p requirement
- Still need to meet plate slenderness req.

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Recommendations for Stiffness

In-Plane Flexural Stiffness

- Account for concrete cracking corresponding to the required strength level
- Section moment-curvature response → secant stiffness corresponding to 60% of moment capacity
- Extent of concrete cracking, if drift governs or walls are overdesigned

$EI_{eff} = E_s I_s + 0.35\, E_c I_c$	Effective flexural stiffnesses (AISC *Design Guide* 37, 2021)
$EA_{eff} = E_s A_s + 0.45\, E_c A_c$	Effective axial stiffnesses (AISC *Design Guide* 37, 2021)
$GA_{v.eff} = G_s A_{s.wall} + G_c A_c$	Effective shear stiffnesses (AISC *Design Guide* 37, 2021)

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Recommendations for Flexural Strength

Plastic stress distribution over composite cross-section

- Steel in compression & tension → f_y
- Compression concrete → $0.85f'_c$
- Equilibrium to calc. plastic neutral axis location, c
- Plastic moment M_p

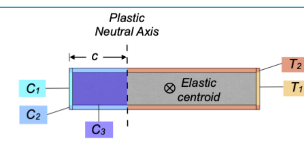

Fig. A-2. Plastic stress distribution and component forces in planar C-PSW/CF

$c = \dfrac{2t_p L_{wall} F_y + (t_{sc} - 2t_p)t_p 0.85 f'_c}{4t_p F_y + (t_{sc} - 2t_p) 0.85 f'_c}$	Plastic neutral axis location
$C_1 = (t_{sc} - 2t_p)t_p F_y$	Flange plate, compression force
$C_2 = 2t_p c F_y$	Web plate, compression force
$C_3 = (t_{sc} - 2t_p)(c - t_p) 0.85 f'_c$	Concrete compression force
$T_1 = 2t_p (L_{wall} - c) F_y$	Web plate, tension force
$T_2 = (t_{sc} - 2t_p)t_p F_y$	Flange plate, tension force

Nominal flexural strength of planar C-PSW/CF:

$$M_p = C_1\left(c - \frac{t_p}{2}\right) + C_2\left(\frac{c}{2}\right) + C_3\left(\frac{c - t_p}{2}\right) + T_1\left(\frac{L_{wall} - c}{2}\right) + T_2\left(L_{wall} - c - \frac{t_p}{2}\right)$$

(AISC *Design Guide* 37, 2021)

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Recommendations for Shear Strength

- In accordance with AISC 360-22 Section I4.4, nominal in-plane shear strength of L-shaped C-PSW/CFs is determined considering the steel section and infill concrete contributions as follows:

$$V_{n.wall} = \frac{K_s + K_{sc}}{\sqrt{3\,K_s^2 + K_{sc}^2}} A_{s.wall} F_y$$ (AISC *Design Guide* 37, 2021)

where, $K_s = G_s\, A_{s.wall}$ (AISC *Design Guide* 37, 2021)

where, $K_{sc} = \frac{0.7\,(E_c\,A_c)\,(E_s\,A_{s.wall})}{(4E_s\,A_{s.wall})\,(E_c\,A_c)}$ (AISC *Design Guide* 37, 2021)

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Seismic Design of Coupled Composite Plate Shear Walls / Concrete Filled (Capacity Design)

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Seismic Design of Coupled C-PSW/CF

- Seismic design can be performed using ASCE/SEI 7-22, AISC 341-22 *Seismic Provisions*, and AISC *Design Guide* 37 (2021).
- Design procedure for coupled C-PSW/CF is presented in FEMA P-2082 *NEHRP Provisions* (2020) and AISC *Design Guide* 37 (2021).

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Seismic Design of Coupled C-PSW/CF

The 2020 Edition of the *NEHRP Recommended Seismic Provisions*:

- Response modification factor $R = 8$
- Over-strength factor $\Omega_0 = 2.5$
- deflection amplification factor $C_d = 5.5$

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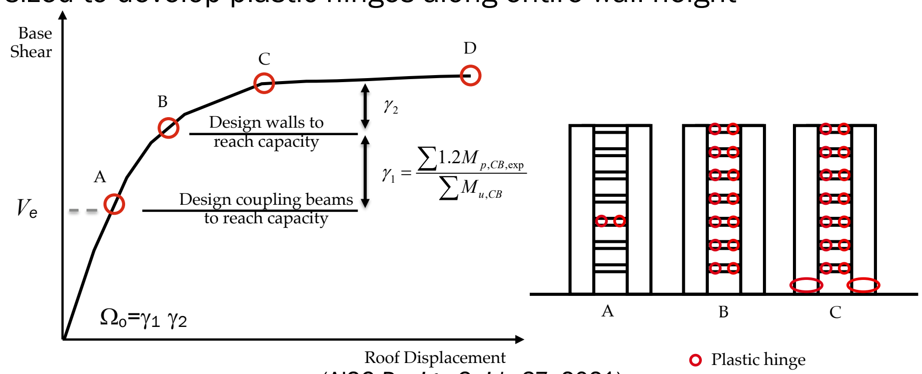
Seismic Design Philosophy for Coupled C-PSW/CF

- Coupling beams form plastic hinges and distributed plasticity along structure height
- Walls sized to develop plastic hinges along entire wall height



(AISC *Design Guide* 37, 2021)

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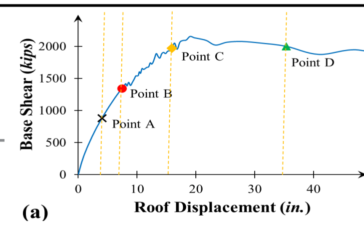
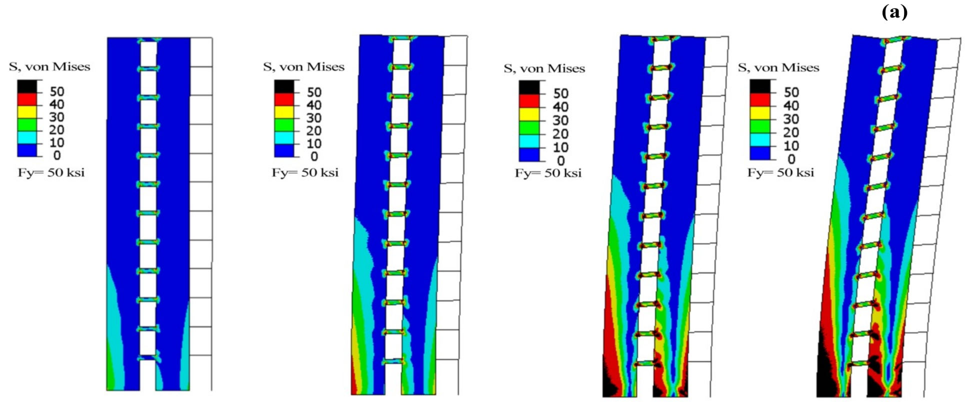
Seismic Design Philosophy

2D Finite Element Model (Pushover Response)



(a)



(Point A) (Point B) (Point C) (Point D)

(Shafaei et al., 2022)

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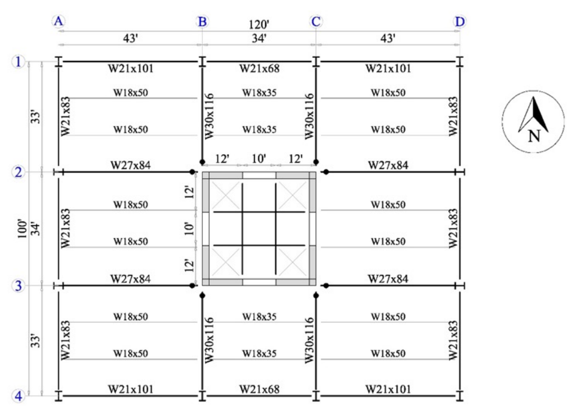
Design Example

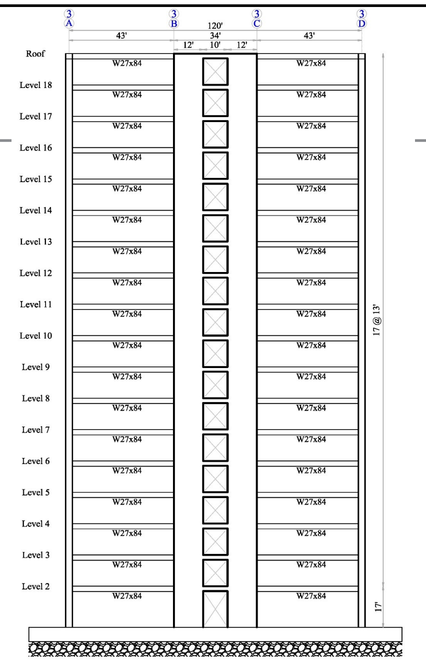
Building Description

- Coupled L-shaped Composite Plate Shear Walls / Concrete Filled (C-PSW/CFs) are used to resist seismic loads.
- Steel gravity frames are placed around the coupled C-PSW/CFs, and elevators and stairs are located inside the core walls



Building Description

- 18-story office building
- First story height = 17 *ft*
- Typical story height = 13 *ft*
- Total height = 238 *ft.*



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Material Properties

Steel:

- ASTM A572 Grade 50 steel (steel plates) & ASTM A992 Grade 50 steel (wide flange sections)
- $F_y = 50$ ksi
- $F_u = 65$ ksi
- $E_s = 29{,}000$ ksi
- $G_s = 11{,}500$ ksi
- $R_y = 1.1$ (ANSI/AISC 341-22 Table A3.1)

Concrete:

- Self-compacting concrete (SCC)
- $f'_c = 6$ ksi
- $E_c = 4{,}500$ ksi
- $G_c = 1{,}770$ ksi
- $R_c = 1.5$ (ANSI/AISC 341-16 H5-5)

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Loads & Load Combinations

Loads:

- Self-weight of structure (gravity frames and core walls) (dead load)
- Floor live load = 50 psf (Redactable)
- Partition = 15 psf
- Superimposed dead load (ceiling and floor finish) = 15 psf
- Curtain wall = 15 psf (wall surface area)

Load Combinations:

- Load combination provided in Chapter 2 of ASCE/SEI 7-16 are considered.
- $1.4D$
- $1.2D + 1.6L$
- $1.2D + 0.5L \pm 1.0E$
- $0.9D \pm 1.0E$

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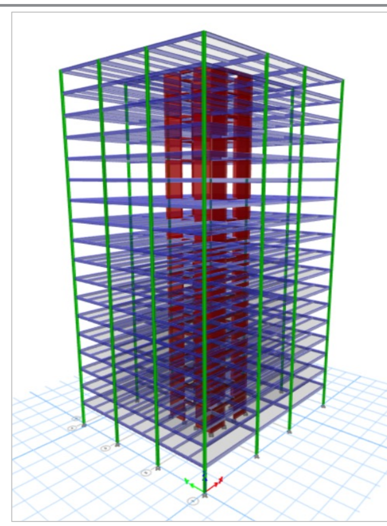
Building Description

- 3D computer model of the building was developed using a commercial software program for the design of steel gravity frames.
- Based on the preliminary design of gravity frames, the self-weight of structure is calculated.



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Seismic Forces

Building Seismic Weight:
- First Story = 1,555 kips
- Typical Story = 1,440 kips
- Roof = 1,263 kips

Seismic Design Parameters:
- $S_{DS} = 1.101\text{g}$
- $S_{D1} = 0.650\text{g}$
- Site Class D
- Risk Category II
- Seismic Design Category D

Period of the structure
- $T_a = C_t\, h_n^x \quad = (0.020)\,(238\, ft)^{0.75} = 1.21$ seconds
- $C_u = 1.4$ (ASCE/SEI 7 Table 12.8-1)
- $T = C_u\, T_a = (1.4)\,(1.21) = 1.70$ seconds
- $T = 1.87$ (3D ETABS model)
- The period of structure is considered to be the upper limit, $C_u T_a$=1.70

Design Base Shear

Equivalent Lateral Forces (ELF) procedure was used to calculate the seismic loads:

- $V = C_s\, W$
- $C_s = \frac{S_{DS}}{R/I_e} = \frac{1.101}{8/1} = 0.138$ (ASCE/SEI 7 12.8-2)
- $C_{s,Max} = \frac{S_{DS}}{T(R/I_e)} = \frac{1.101}{1.7(8/1)} = \mathbf{0.048}$ (ASCE/SEI 7 12.8-3)
- $C_{s,Min} = 0.44\, S_{DS}\, I_e = (0.44)(1.101)(1) = 0.048$ (ASCE/SEI 7 12.8-5)
- $C_s = \frac{0.5\, S_1}{(R/I_e)} = \frac{(0.5)(0.65)}{(8/1)} = \mathbf{0.041}$ (ASCE/SEI 7 12.8-6)
- $\mathbf{V = C_s\, W = (0.048)\,(25844) = 1{,}238}$ **kips**
- $\boldsymbol{OTM = \sum_{i=1}^{n} F_i\, h_i = 217{,}217}$ ***kip-ft***

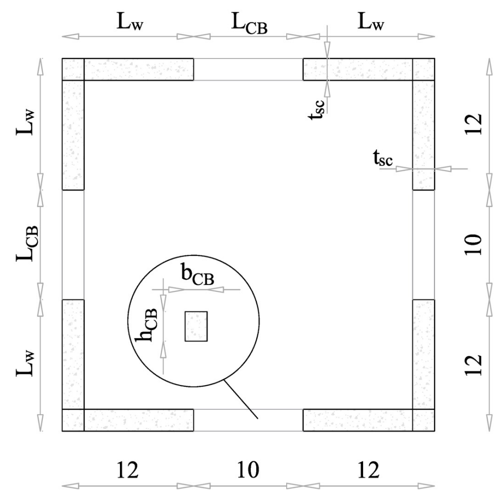
C-PSW/CFs and Coupling Beam Dimensions

C-PSW/CF:

- $L_w = 12$ ft
- $t_{sc} = 16$ in.
- $t_p = ½$ in.

Coupling beams:

- $L_{CB} = 10$ ft
- $b_{CB} = 16$ in.
- $h_{CB} = 24$ in.
- $t_{CB.f} = ½$ in.
- $t_{CB.w} = ⅜$ in.
- $L_{CB} / h_{CB} = 5$



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2D Modeling of Coupled C-PSW/CF

C-PSW/CF:

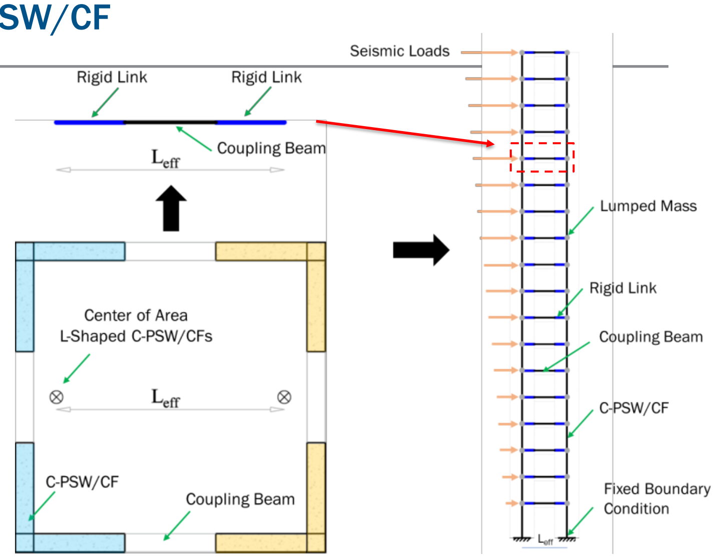
(AISC *Design Guide* 37, 2021)

- $EI_{eff} = E_s I_s + 0.35 E_c I_c$
- $EA_{eff} = E_s A_s + 0.45 E_c A_c$
- $GA_{v.eff} = G_s A_{s.wall} + G_c A_c$

Coupling beams:

(AISC *Design Guide* 37, 2021)

- $0.64\, EI_{eff.CB}$
- $0.8\, EA_{eff.CB}$
- $GA_{v.eff.CB}$
- $L_{eff} = 323.8$ in.



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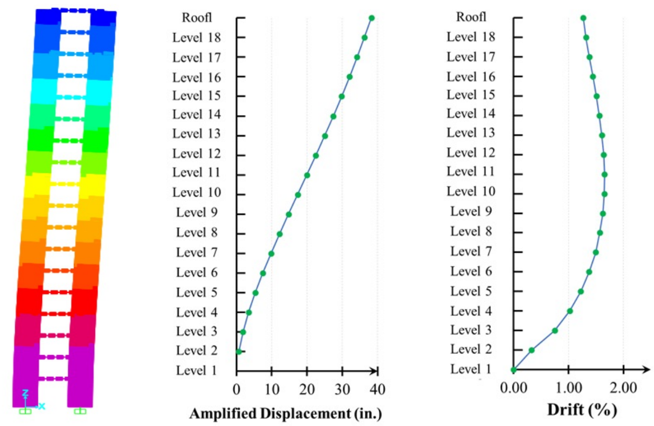
Inter-story Drift Limit

- Deformation shape, lateral displacement, and inter-story drift.
- Amplified displacement is calculated by multiplying story displacement value by the deflection amplification factor. Inter-story drift is calculated using the amplified displacement.
- Maximum inter-story is 1.65%.



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Linear Elastic Analysis

- $V_{r.CB} = 167$ kips (average)
- $V_{Max.CB} = 223.5$ kips (maximum)
- $M_{U.CB} = \frac{V_{r.CB}\, L_{CB}}{2} = 835$ kip−ft
- $M_{Max.CB} = \frac{V_{Max.CB}\, L_{CB}}{2} = 1{,}117$ kip−ft

(#)	Story Elevation (ft.)	Disp. (in.)	Amplified Disp. (in.)	Inter-story Drift (%)	CB Shear Force (kips)
Roof	238	6.95	38.24	1.32	89.2
Level 18	225	6.59	36.26	1.38	97.1
Level 17	212	6.22	34.20	1.44	110.2
Level 16	199	5.83	32.05	1.51	126.0
Level 15	186	5.42	29.80	1.56	129.4
Level 14	173	4.99	27.45	1.61	159.9
Level 13	160	4.55	25.01	1.64	176.0
Level 12	147	4.09	22.50	1.65	190.6
Level 11	134	3.63	19.94	1.65	203.1
Leve 10	121	3.16	17.36	1.63	213.1
Level 9	108	2.69	14.79	1.57	220.1
Level 8	95	2.23	12.25	1.49	$\boxed{223.5}$
Level 7	82	1.78	9.81	1.38	222.4
Level 6	69	1.36	7.47	1.22	216.0
Level 5	56	0.97	5.33	1.02	202.8
Level 4	43	0.62	3.42	0.75	180.9
Level 3	30	0.33	1.83	0.33	147.5
Level 2	17	0.12	0.67	0.00	98.7

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Design Of Coupling Beams

Flexure-Critical Coupling Beams:

- $V_{n.\text{exp.CB}} \geq \frac{2.4\, M_{P.exp.CB}}{L_{CB}}$ (AISC *Design Guide* 37, 2021)

Expected Flexural Capacity ($M_{p.exp.CB}$):

- $M_{p.exp.CB} = 1{,}582.6 \text{ kip−ft}$

Minimum Area of Steel:

- $A_{s.CB.min} = 0.01\, h_{CB}\, b_{CB} = (0.01)(24)(16) = 3.8 \text{ in.}^2$ (AISC Spec. I2.2a)
- $A_{s.CB} = 33.25 > A_{s.CB.min} = 3.8 \text{ in.}^2$

Design Of Coupling Beams

Steel Plate Slenderness Requirement for Coupling Beams:

- $\frac{b_{c.CB}}{t_{CB.f}} = 30.5 < 2.37\sqrt{\frac{E_s}{R_y\, F_y}} = 2.37\sqrt{\frac{29000}{(1.1)(50)}} = 54.4$ (AISC 360−22 Table I1.1b)
- $\frac{h_{c.CB}}{t_{CB.w}} = 61.3 \geq 2.66\sqrt{\frac{E_s}{R_y\, F_y}} = 2.66\sqrt{\frac{29000}{(1.1)(50)}} = 61.1$ (AISC 360−22 Table I1.1b)

Flexural Strength ($M_{p.CB}$):

- $M_{n.CB} = M_{p.CB} = 1{,}407 \text{ kip−ft}$ (AISC *Design Guide* 37, 2021)
- $\phi_b M_{n.CB} = 1{,}266 \text{ kip−ft} > M_{U.CB} = 835 \text{ kip−ft}$
- $\frac{M_{r.CB}}{\phi_b M_{n.CB}} = 0.66$ $\frac{M_{U.CB.Max}}{\phi_b M_{n.CB}} = 0.88$

Design Of Coupling Beams

Nominal Shear Strength ($V_{n.CB}$):

- $V_{n.CB} = 0.6\, F_y A_{w.CB} + 0.06\, K_c \sqrt{f_c'} A_{c.CB} = 592$ kips (AISC *Design Guide* 37, 2021)
- $\phi_v V_{n.CB} = 532\ kips \quad > \quad V_{U.CB} = 167\ kips$

$$\frac{V_{r.CB}}{\phi_v V_{n.CB}} = \frac{167\ kips}{532\ \text{kips}} = 0.31 \qquad \frac{V_{U.CB}}{\phi_v V_{n.CB}} = \frac{223.5\ kips}{532\ \text{kips}} = 0.42$$

Flexure-Critical Coupling Beams (revisited):

- $V_{n.exp.CB} = 0.6\, R_y\, F_y A_{w.CB} + 0.06\, K_c \sqrt{R_c\, f_c'} A_{c.CB} = 657$ kips
- $V_{n.exp.CB} = 657 \text{ kips} > \frac{2.4\, M_{P.exp.CB}}{L_{CB}} = 380$ kips (AISC *Design Guide* 37, 2021)

Design Of C-PSW/CFs

Minimum and Maximum Area of Steel:

- $A_{gross.wall} = (2)[(L_w\, t_{sc}) + (L_w\, - t_{sc}) t_{sc}\,] = 8{,}704 \text{ in.}^2$
- $A_{s.min} = 0.01\, A_{gross.wall} = (0.01)(8{,}704) = 87 \text{ in.}^2$ (ANSI/AISC 360-22 I2.2a)
- $A_{s.max} = 0.1\, A_{gross.wall} = (0.1)(8{,}704) = 870 \text{ in.}^2$ (ANSI/AISC 360-22 I2.2a)
- $A_s = (t_p)\left[8L_w + 4t_{sc} - 16t_p\,\right] = 604 \text{ in.}^2$
- $A_{s.min} = 87 \text{ in.}^2 \; < \; A_s = 604 \text{ in.}^2 \; < \; A_{s.max} = 870 \text{ in.}^2$

Design Of C-PSW/CFs

Slenderness Requirements:

- In accordance with ANSI/AISC 341-22 Section H8.4b, steel plate slenderness ratio, *b/t*, at the base of C-PSW/CF (protected zones) should be limited as follows:
- $S_{tie} = 12$ in. (the bottom two stories)
- $\frac{S_{tie}}{t_p} = 24 \quad < \quad 1.05\sqrt{\frac{E_s}{R_y\, F_y}} = 1.05\sqrt{\frac{29{,}000}{(1.1)(50)}} = 24.1$ (ANSI/AISC 341-22 H8.4b)
- Steel plate slenderness ratio, *b/t*, at regions which are not protected zones:
- $S_{tie.top} = 14$ in.
- $\frac{S_{tie.top}}{t_p} = 28 \quad < \quad 1.2\sqrt{\frac{E_s}{F_y}} = 1.2\sqrt{\frac{29{,}000}{(50)}} = 28.9$ (ANSI/AISC 360-22)

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Design Of C-PSW/CFs

Tie spacing requirements:

- In accordance with ANSI/AISC 360-22 Section I1.6b, the tie bar spacing to plate thickness ratio, S/t_p, should be limited as follows:
- $d_{tie} = 3/4\ in.$
- $\propto = 1.7\left(\frac{t_{sc}}{t_p} - 2\right)\left(\frac{t_p}{d_{tie}}\right)^4 = 1.7\left(\frac{16}{0.5} - 2\right)\left(\frac{0.5}{0.75}\right)^4 = 10.07$ (AISC *Design Guide* 37, 2021)
- $\frac{S_{tie.bottom}}{t_p} = 24 \quad < \quad 1.0\sqrt{\frac{E_s}{2\propto + 1}} = 1.0\sqrt{\frac{29{,}000}{2(10.07)+1}} = 37.0$ (AISC *Design Guide* 37, 2021)
- $\frac{S_{tie.top}}{t_p} = 32 \quad < \quad 1.0\sqrt{\frac{E_s}{2\propto + 1}} = 1.0\sqrt{\frac{29{,}000}{2(10.07)+1}} = 37.0$ (AISC *Design Guide* 37, 2021)

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Design Of C-PSW/CFs

Required Wall Shear Strength:

- A shear amplification factor of 4 is used to amplify the base shear.
- $V_{Amplified} = 4{,}952 \text{ kips}$ (AISC *Design Guide 37*, 2021)
- $V_{r.wall} = \frac{4{,}952}{2} = 2{,}476 \text{ kips}$

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Design Of C-PSW/CFs

Required Flexural Strength of Coupled C-PSW/CFs

- A shear amplification factor of 4 is used to amplify the base shear.
- $M_{p.exp.CB} = 1{,}583 \text{ kip-ft}$ (Expected flexural capacity of CB)
- $V_{n.Mp.exp.CB} = \frac{2.4\, M_{P.exp.CB}}{L_{CB}} = 380 \text{ kips}$ (Expected shear strength of CB)
- $\gamma_1 = \frac{\sum_n 1.2\, M_{p.exp.CB}}{\sum_n M_{U.CB}} = \frac{(18)(1.2)(1583)}{(18)(835)} = 2.27$ (Overstrength amplification factor)
- $P_{CB} = 2 \sum_n V_{n.Mp.exp.CB} = 13{,}673 \text{ kips}$ (Axial force due to coupling action)
- $M_{r.wall} = \gamma_1\, OTM - P_{CB}\, L_{eff} = 125{,}077 \text{ kip-ft}$ (Required amplified OTM)
- $P = -2 \sum_n V_{n.Mp.exp.CB} - (1.2 \sum_n \mathrm{F}_{Tri.DL}) - (0.5 \sum_n \mathrm{F}_{Tri.LL}) = -20{,}644 \text{ kips}$ (axial compression force)
- $T = 2 \sum_n V_{n.Mp.exp.CB} - (0.9 \sum_n \mathrm{F}_{Tri.DL}) = 9{,}219 \text{ kips}$ (axial tension force)

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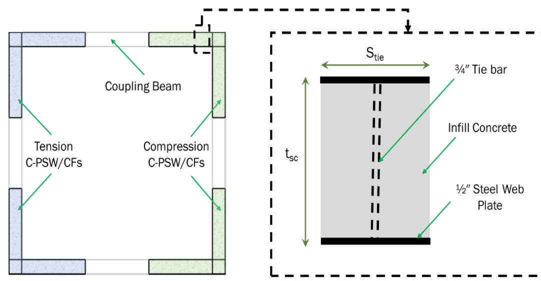
Design Of C-PSW/CFs

Wall Tensile Strength:

- $P_{n.T} = A_s\, F_y = (604)(50) = 30{,}200 \text{ kips}$
- $\phi_t\, P_{n.T} = 27{,}180 \text{ kips} \;>\; T = 9{,}219 \text{ kips}$
- $\frac{T}{\phi_t\, P_{n.T}} = 0.35$

Wall Compression Strength:

- A simplified unite width method is considered to calculate nominal compression strength.

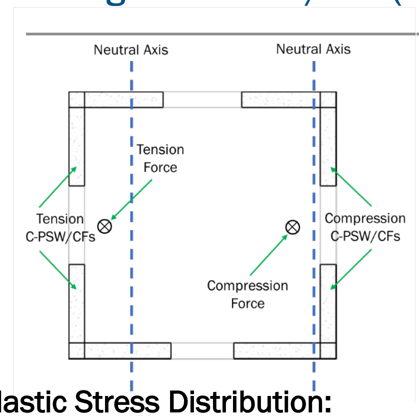


Design Of C-PSW/CFs

Wall compression Strength:

- $S_{tie} = 12 \text{ in} = 1 \text{ ft}$ (Length of selected unit width)
- $L_{wall.total} = 48 \text{ ft}$ (Total length of two C-PSW/CFs)
- $P_{no} = 2t_p\, S_{tie}\, F_y + 0.85\, f_c'\, (t_{sc} - 2t_p)\, S_{tie} = 1{,}518 \text{ kips}$ (ANSI/AISC 360-22)
- $P_e = \frac{\pi^2\, EI_{eff.min}}{L_{cr}{}^2} = 1797 \text{ kips}$
- $\frac{P_{no}}{P_e} = 0.84 \;<\; 2.25$ (ANSI/AISC 360-22)
- $P_{n.C} = P_{no}\left(0.685^{\frac{P_{no}}{P_e}}\right) = 1{,}066 \text{ kips}$
- $P_{n.C.total} = P_{n.C}\, n_{unit-width} = (1{,}066 \text{ kips})(48) = 51{,}168 \text{ kips}$
- $\phi_C\, P_{n.C.total} = (0.9)(51{,}168 \text{ kips}) = 46{,}051 \text{ kips} \;>\; P = 20{,}644 \text{ kips}$
- $\frac{\mathrm{P}}{\phi_C\, P_{n.C.total}} = 0.45$

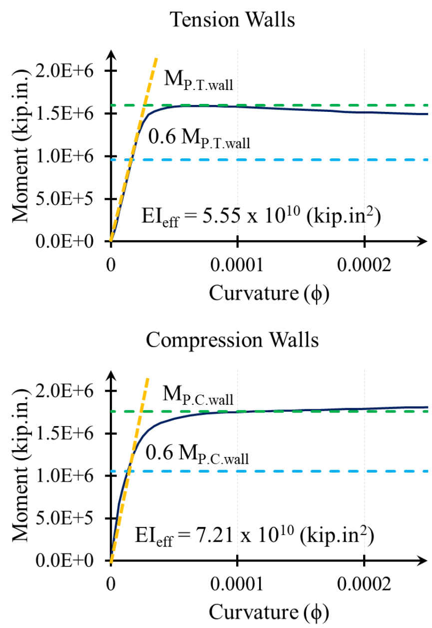
Design Of C-PSW/CFs (Flexural Strength



Plastic Stress Distribution:

$M_{P.T.wall} = M_{n.T.wall} = 1{,}598{,}236 \text{ kip−in.}$

$M_{P.C.wall} = M_{n.T.wall} = 1{,}761{,}166 \text{ kip−in.}$



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Design Of C-PSW/CFs (Flexural Strength)

The effective flexural stiffnesses of tension and compression ($EI_{T.wall}$ and $EI_{C.wall}$) L-shaped C-PSW/CFs are used to calculated required flexural strengths of tension and compression walls.

- $M_{U.T.wall} = \left[\frac{EI_{T.wall}}{(EI_{C.wall} + EI_{T.wall})}\right] M_{r.wall} = 652833 \; kip\text{−}in. = 54403 \text{ kip−ft}$
- $M_{U.C.wall} = \left[\frac{EI_{C.wall}}{(EI_{C.wall} + EI_{T.wall})}\right] M_{r.wall} = 848094 \; kip\text{−}in. = 70675 \text{ kip−ft}$

Ratio of demand to capacity:

- $\frac{M_{U.T.wall}}{\phi_t \, M_{n.T.wall}} = 0.45$
- $\frac{M_{U.C.wall}}{\phi_t \, M_{n.C.wall}} = 0.54$

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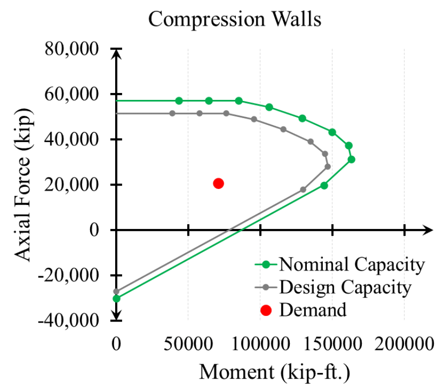
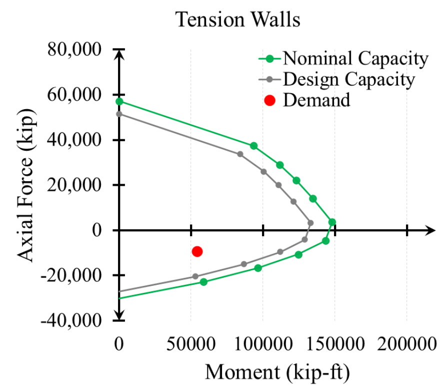
P-M Interaction of C-PSW/CFs





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Design Of C-PSW/CFs (Shear Strength)

Wall Shear Strength:

- $A_{s.wall} = 4\,(L_W\, t_p) + 2(t_{sc}\, t_p) = (4)(144)(0.5) + (2)(16)(0.5) = 304 \text{ in.}^2$
- $K_s = G_s\, A_{s.wall} = (11200)(304) = 3.39 \times 10^6 \text{ kips}$
- $K_{sc} = \dfrac{0.7\,(E_c\, A_c)\,(E_s\, A_{s.wall})}{(4E_s\, A_{s.wall})\,(E_c\, A_c)} = 3.14 \times 10^6 \text{ kips}$
- $V_{n.wall} = \dfrac{K_s + K_{sc}}{\sqrt{3\, K_s^{\,2} + K_{sc}^{\,2}}} A_{s.wall} F_y = 14906 \text{ kips}$
- $\phi_v V_{n.wall} = 13416 \text{ kips} \quad > \quad V_{U.wall} = 2476 \text{ kips}$
- $\dfrac{V_{U.wall}}{\phi_v V_{n.wall}} = 0.19$

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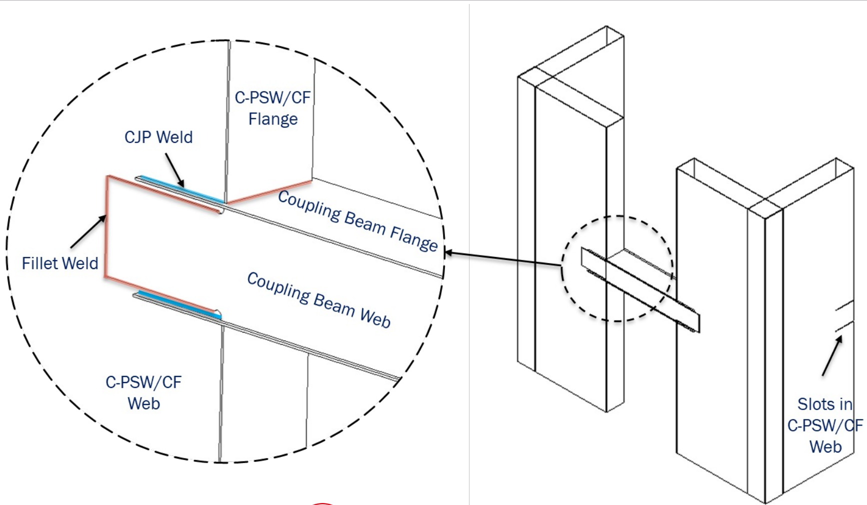
Coupling Beam-to-Wall Connection



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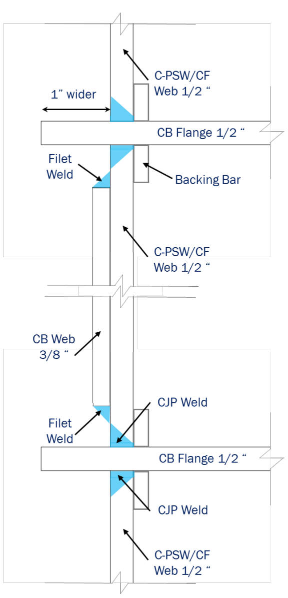
Coupling Beam-to-Wall Connection

- Coupling Beam-to-Wall Connection Details (scaled specimen)



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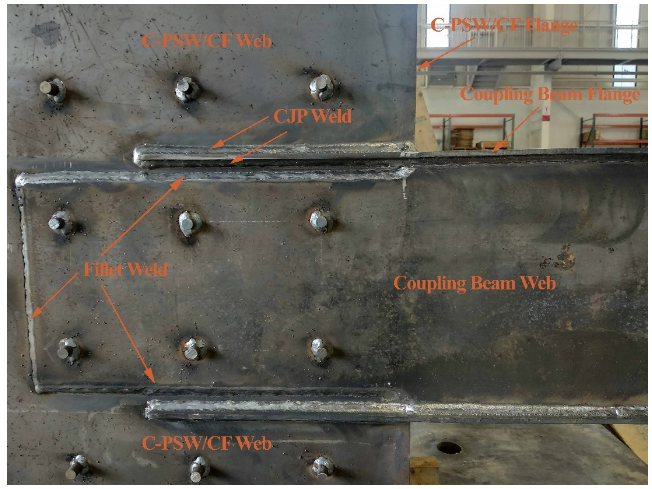
Coupling Beam-to-Wall Connection

- Coupling Beam-to-Wall Connection Details (scaled specimen)



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Coupling Beam-to-Wall Connection

Flange Plate Connection Demand:

- $T_{flange} = min\left(1.2\ R_y F_y A_{CB.f}, R_t F_u A_{CB.f},\right) = 594$ kips
- $\frac{T_{flange}}{2} = 297$ kips

Required Length of CJP Welding :

- $\frac{T_{flange}}{2} \leq \phi_d\ 0.6\ F_y t_{CB.f} L_{req.}$
- $L_{req.} \geq \frac{T_{flange}}{2(\phi_d\ 0.6\ F_y t_{CB.f})} = \frac{594}{2(1.0)(0.6)(50)(0.5)} =\ 19.8$ in.
- $L_{weld.f} = 20$ in.

$\phi_d = 1.0$
$\phi_n = 0.9$

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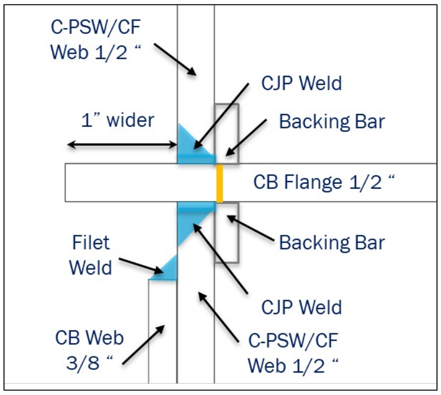
Check Shear Strength of Coupling Beam Flange Plate

Shear yielding of coupling beam flange plate:

- $A_{f.SY} = t_{CB.f}\ L_{weld.f} = (0.5)(20) = 10\ \text{in.}^2$
- $\phi_d\ 0.6\ F_y\ A_{f.SY} = 300\ kips\ \geq\ \frac{T_{flange}}{2} = 297\ \text{kips}$

Shear rupture of coupling beam flange plate:

- $A_{f.SR} = t_{CB.f}\ L_{weld.f} = (0.5)(20) = 10\ \text{in.}^2$
- $\phi_n\ 0.6\ F_u\ A_{f.SR} = 351\ kips > \frac{T_{flange}}{2} = 297\ \text{kips}$



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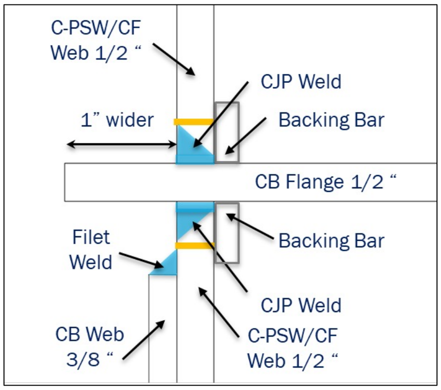
Check Shear Strength of Wall Web Plates

Shear yielding of wall web plates:

- $A_{w.SY} = 2\ t_p\ L_{weld.f} = 2(0.5)(20) = 20\ \text{in.}^2$
- $\phi_d\ 0.6\ F_y\ A_{w.SY} = 600\ kips\ >\ \frac{T_{flange}}{2} = 297\ \text{kips}$

Shear rupture of wall web plates:

- $A_{w.SR} = 2\ t_p\ L_{weld.f} = 2(0.5)(20) = 20\ \text{in.}^2$
- $\phi_n\ 0.6\ F_u\ A_{w.SR} = 702\ \text{kips} > \frac{T_{flange}}{2} = 297\ \text{kips}$



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Check Ductile Behavior of Flange Plates

In coupling beam flange plate to C-PSW/CF connection design, the available tensile rupture strength should be higher than the available tensile yield strength.

- $A_{CB.f.g} = (b_{CB}+2in.)\, t_{CB.f} = (16+2)(0.5) = 9 \text{ in.}^2$ (Gross area)
- $A_{CB.f.n} = b_{CB}\, t_{CB.f} = (16)(0.5) = 8 \text{ in.}^2$ (Net area)
- $R_y\, F_y\, A_{CB.f.g} = (1.1)(50)(9) == 495 \text{ kips}$ (Available tension yielding capacity)
- $R_t\, F_u\, A_{CB.f.n} = (1.1)(65)(8) = 572 \text{ kips}$ (Available tension rupture capacity)
- $R_t\, F_u\, A_{CB.f.n} = 572\ kips \;\; > \;\; R_y\, F_y\, A_{CB.f.g} = 495 \text{ kips}$

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Calculate Forces in Web Plates

$T_{2.exp} = 773 \text{ kips}$ (Expected tension force of CB web)

$C_{2.exp} = 217 \text{ kips}$ (Expected compression force of CB web)

$C_{CB.exp} = 5.26 \text{ in.}$ (Plastic neutral axis of CB considering $M_{CB.p.exp.}$)

$T_{web} = 1.2\,(T_{2.exp} - C_{2.exp}) = 667 \text{ kips}$ (CB web plates tension force)

$M_{web} = 1.2\left(T_{2.exp}\frac{C_{CB.exp}}{2} + C_{2.exp}\frac{h_{CB}\, C_{CB.exp}}{2}\right) = 407 \text{ kip−ft}$ (CB web plates moment)

$V_{web} = 2\left(\frac{1.2\, M_{p.exp.CB}}{L_{CB}}\right) = 380 \text{ kips}$ (CB web plates shear force)

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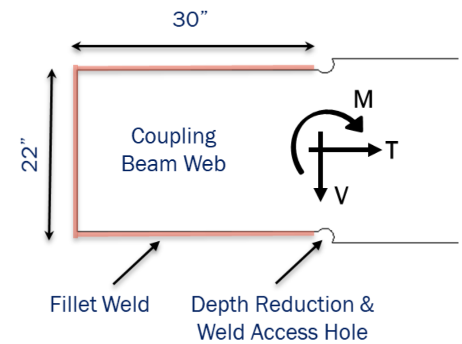
Calculate Force Demand on C-Shaped Weld

$$T_{C.weld} = \frac{T_{web}}{2} = 333 \text{ kips}$$

$$M_{C.weld} = \frac{M_{web}}{2} = 203 \text{ kip–ft}$$

$$V_{C.weld} = \frac{V_{web}}{2} = 190 \text{ kips}$$

$$D_{min} = 3/16 \text{ in.}$$
$$D_{max} = 5/16 \text{ in.}$$
$$D = 5/16 \text{ in.}$$
$$D_{min} \leq D \leq D_{max}$$



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Calculate Capacity of C-Shaped Weld

$$Eccentricity = \frac{M_{C.web}}{V_{C.web}} = 12.85 \text{ in.}$$

$$c.g. = \frac{{L_{H.weld.w}}^2}{2L_{H.weld.w} + L_{V.weld.w}} = \frac{30^2}{2(36)+(22)} = 10.98 \text{ in.}$$

$$e_x = Eccentricity + (L_{H.weld.w} - c.g.) = 31.88 \text{ in.}$$

$$k = \frac{L_{H.weld.w}}{L_{V.weld.w}} = \frac{30}{22} = 1.36$$

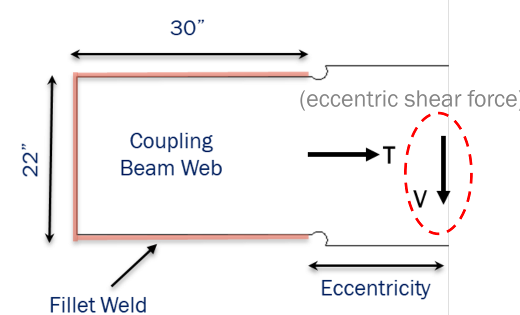
$$a = \frac{e_x}{L_{V.weld.w}} = \frac{11}{22} = 1.45$$

(AISC Steel Manual 15th Edition Table 8-8)

$$P_{V.weld} = \phi_n C_{8.8}\, C_{1-8.3}\, (16D) L_{V.weld.w}$$

$$P_{V.weld} = 334 \text{ kips} \quad > \quad V_{C.weld} = 190 \text{ kips}$$

$$\frac{V_{C.weld}}{P_{V.weld}} = 0.62$$



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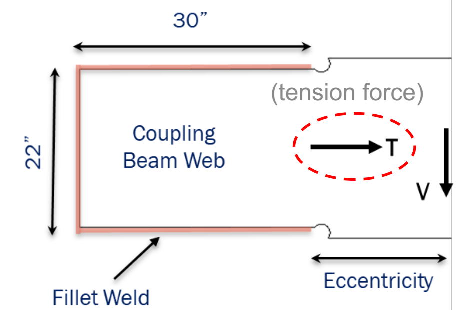
Calculate Capacity of C-Shaped Weld

$$P_{T.weld} = \phi_n \, 0.6 \, F_{XX} \, 2 \, L_{H.weld.w} 0.7071 D = (0.9)(0.6)(70)[2(30)](0.7071)(5/16)$$

$$P_{T.weld} = 501 \text{ kips} \quad > \quad T_{C.weld} = 333 \text{ kips}$$

$$\frac{T_{C.weld}}{P_{T.weld}} = 0.67$$



$$Capacity = \sqrt{\left(\frac{V_{C.weld}}{P_{V.weld}}\right)^2 \left(\frac{T_{C.weld}}{P_{T.weld}}\right)^2} = \sqrt{(0.64)^2 (0.67)^2} = 0.91 \quad \leq \quad 1$$

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Questions

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