Chapter 5 Seismic Design of Coupled Composite Plate Shear Walls / Concrete Filled (C-PSW/CF)

2020 NEHRP Provisions Training Materials
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Topics Covered

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

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)
A New Chapter in Composite Construction

Rainier Square, Seattle

- Client
- Architect
- Structural & Civil
- GC/GM

Steel Fabricator:

Steel Erector:

Rebar Fabricator:

Concrete Supplier:

Courtesy of Magnusson Klemencic Associates
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 Magnuson Klemencic Associates*
A New Chapter in Composite Construction

200 Park Avenue, San Jose, CA
- High seismic region
- 937,000 square foot
- 19 stories
- Under construction

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(Modern Steel Construction, February 2021)

Section Detailing, Limits, Requirements
Key Components of C-PSW/CF (SpeedCore System)

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

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
Local buckling, Plate Slenderness, Axial Compression

Seismic Design:

\[
\frac{b}{b_p} \leq 1.05 \frac{E_s}{R_p F_y} \\
F_{ct} \geq F_y
\]

\[P_{ud} = A_s F_y + 0.85 f'c' A_c\]
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:

  \[
  \frac{S}{t_p} < 1.2 \sqrt{\frac{E_s}{F_y}}
  \]

Tie Bar Size, Spacing, and Stability of Empty Modules

- Empty steel module flexibility governed by effective shear stiffness \((G A)_{\text{eff}}\) associated with Vierendeel truss / frame action

\[
\Delta_{\text{total}} = \frac{5 \times w L^4}{384 \times E I_{\text{total}}} + \frac{w E^2}{8 \times G A_{\text{eff}}} \quad \text{dominates}
\]

(Varma et al., 2019)
Tie Bar Size, Spacing, and Stability of Empty Modules

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

\[
\frac{S}{t_p} < 1.0 \sqrt[2]{\frac{E_s}{2\alpha + 1}}
\]

Where, \( \alpha = 1.7 \left( \frac{h_{tx}}{t_p} - 2 \right) \left( \frac{t_f}{d_{tx}} \right)^4 \)

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

\[
EI_{\text{eff}} = E_s I_s + 0.35 E_c I_c
\]
Effective flexural stiffnesses (AISC Design Guide 37, 2021)

\[
EA_{\text{eff}} = E_s A_s + 0.45 E_c A_c
\]
Effective axial stiffnesses (AISC Design Guide 37, 2021)

\[
GA_{\text{eff}} = G_s A_{\text{wall}} + G_c A_c
\]
Effective shear stiffnesses (AISC Design Guide 37, 2021)

Recommendations for Flexural Strength

Plastic stress distribution over composite cross-section

- Steel in compression & tension → \( f_y \)
- Compression concrete → 0.85\( f_c' \)
- Equilibrium to calc. plastic neutral axis location, \( c \)
- Plastic moment \( M_p \)

Nominal flexural strength of planar C-PSW/CF:

\[
M_p = C \left( \frac{c - t_e}{2} \right) + C_1 \left( \frac{c - t_e}{2} \right) + C_2 \left( \frac{c - t_e}{2} \right) + C_3 \left( \frac{c - t_e}{2} \right) + C_4 \left( \frac{c - t_e}{2} \right) + C_5 \left( \frac{c - t_e}{2} \right)
\]
(AISC Design Guide 37, 2021)
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:

\[ V_{n,wall} = \frac{K_s + K_{sc}}{\sqrt{3K_s^2 + K_{sc}^2}} A_{s,wall} f_y \]  

(AISC Design Guide 37, 2021)

where, \( K_s = G_s A_{s,wall} \)  

(AISC Design Guide 37, 2021)

where, \( K_{sc} = \frac{0.7 (E_s A_d) (E_s A_{s,wall})}{(4E_s A_{s,wall}) (E_c A_c)} \)  

(AISC Design Guide 37, 2021)

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Seismic Design of Coupled Composite Plate Shear Walls / Concrete Filled (Capacity Design)
Seismic Design of Coupled C-PSW/CF


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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 $C_d = 5.5$
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

2D Finite Element Model (Pushover Response)

(accounting for Seismic Design Guide 37, 2021)
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.

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)
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.
  - $1.4D$
  - $1.2D + 1.6L$
  - $1.2D + 0.5L \pm 1.0E$
  - $0.9D \pm 1.0E$

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.
Seismic Forces

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

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

Period of the structure
- $T_a = C_u h_n^2 = (0.020) (238 \text{ ft})^{0.75} = 1.21 \text{ seconds}$
- $C_u = 1.4$ (ASCE/SEI 7 Table 12.8-1)
- $T = C_u T_a = (1.4) (1.21) = 1.70 \text{ seconds}$
- $T = 1.87$ (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:
- $V = C_s W$
- $C_s = \frac{S_{DS}}{R/R_e} = \frac{1.101}{1} = 0.138$ (ASCE/SEI 7 12.8-2)
- $C_s, Max = \frac{S_{DS}}{T(R/R_e)} = \frac{1.101}{1.7(0/1)} = 0.048$ (ASCE/SEI 7 12.8-3)
- $C_s, Min = 0.44 S_{DS} h_e = (0.44)(1.101)(1) = 0.048$ (ASCE/SEI 7 12.8-5)
- $C_s = \frac{0.5 S_1}{(R/R_e)} = \frac{(0.5)(0.65)}{(0/1)} = 0.041$ (ASCE/SEI 7 12.8-6)
- $V = C_s W = (0.048)(25844) = 1,238 \text{ kips}$
- $OTM = \sum_{i=1}^{n} F_i h_i = 217,217 \text{ kip-ft}$
C-PSW/CFs and Coupling Beam Dimensions

C-PSW/CF:
- \( L_w = 12 \text{ ft} \)
- \( t_{sc} = 16 \text{ in.} \)
- \( t_p = \frac{1}{2} \text{ in.} \)

Coupling beams:
- \( L_{CB} = 10 \text{ ft} \)
- \( b_{CB} = 16 \text{ in.} \)
- \( h_{CB} = 24 \text{ in.} \)
- \( t_{CB,f} = \frac{1}{2} \text{ in.} \)
- \( t_{CB,w} = \frac{3}{8} \text{ in.} \)
- \( L_{CB} / h_{CB} = 5 \)

2D Modeling of Coupled C-PSW/CF

C-PSW/CF:
(Report Design Guide 37, 2021)
- \( E_{L_{eff}} = E_s I_s + 0.35 E_c I_c \)
- \( E_{A_{eff}} = E_s A_s + 0.45 E_c A_c \)
- \( G_{A_{eff}} = G_s A_{wall} + G_c A_c \)

Coupling beams:
(Report Design Guide 37, 2021)
- \( 0.64 E_{L_{eff,CB}} \)
- \( 0.8 E_{A_{eff,CB}} \)
- \( G_{A_{eff,CB}} \)
- \( L_{eff} = 323.8 \text{ in.} \)
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%.

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

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<th>(#)</th>
<th>Story Elevation (ft.)</th>
<th>Disp. (in.)</th>
<th>Amplified Disp. (in.)</th>
<th>Inter-story Drift (%)</th>
<th>CB Shear Force (kips)</th>
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Design Of Coupling Beams

Flexure-Critical Coupling Beams:

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

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

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

Minimum Area of Steel:

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

Steel Plate Slenderness Requirement for Coupling Beams:

- \( \frac{b_{CB}}{t_{CB,f}} = 30.5 < 2.37 \frac{E_l}{R_y f_y} = 2.37 \frac{29000}{(1.1)(50)} = 54.4 \) (AISC 360–22 Table I1.1b)
- \( \frac{h_{CB}}{t_{CB,w}} = 61.3 \geq 2.66 \frac{E_l}{R_y f_y} = 2.66 \frac{29000}{(1.1)(50)} = 61.1 \) (AISC 360–22 Table I1.1b)

Flexural Strength (\( M_{\text{p,CB}} \)):

- \( M_{n, \text{CB}} = M_{p, \text{CB}} = 1,407 \text{ kip-ft} \) (AISC Design Guide 37, 2021)
- \( \phi_p M_{n, \text{CB}} = 1,266 \text{ kip-ft} > M_{U, \text{CB}} = 835 \text{ kip-ft} \)
- \( \frac{M_{r, \text{CB}}}{\phi_p M_{n, \text{CB}}} = 0.66 \quad \frac{M_{U, \text{CB}, \text{Max}}}{\phi_p M_{n, \text{CB}}} = 0.88 \)
Design Of Coupling Beams

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

- $V_{n,CB} = 0.6 F_y A_{w,CB} + 0.06 K_c \sqrt{f_c^f} A_{c,CB} = 592$ kips (AISC Design Guide 37, 2021)
- $\phi V_{n,CB} = 532$ kips > $V_{u,CB} = 167$ kips

\[
\begin{align*}
\frac{V_{n,CB}}{\phi V_{n,CB}} &= \frac{167 \text{ kips}}{532 \text{ kips}} = 0.31 \\
\frac{V_{u,CB}}{\phi V_{n,CB}} &= \frac{223.5 \text{ kips}}{532 \text{ kips}} = 0.42
\end{align*}
\]

Flexure-Critical Coupling Beams (revisited):

- $V_{n,exp,CB} = 0.6 R_y F_y A_{w,CB} + 0.06 K_c \sqrt{R_c f_c^f} A_{c,CB} = 657$ kips
- $V_{n,exp,CB} = 657$ kips > $\frac{2.4 M_{p,exp,CB}}{I_{CB}} = 380$ kips (AISC Design Guide 37, 2021)

Design Of C-PSW/CFs

Minimum and Maximum Area of Steel:

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

  \[
  S_{tie} = 12 \text{ in. (the bottom two stories)}
  \]

- Steel plate slenderness ratio, \( b/t \), at regions which are not protected zones:

  \[
  S_{tie,top} = 14 \text{ in.}
  \]

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:

  \[
  d_{tie} = 3/4 \text{ in.}
  \]

- Tie spacing requires:

  \[
  \frac{S_{tie,bottom}}{t_p} = 24 < 1.0 \frac{E_s}{2 \times 1 + 1} = 1.0 \frac{29,000}{2(10.07)+1} = 37.0 \quad \text{(AISC Design Guide 37, 2021)}
  \]

  \[
  \frac{S_{tie,top}}{t_p} = 32 < 1.0 \frac{E_s}{2 \times 1 + 1} = 1.0 \frac{29,000}{2(10.07)+1} = 37.0 \quad \text{(AISC Design Guide 37, 2021)}
  \]
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)
- \( V_{r,wall} = \frac{4,952}{2} = 2,476 \text{ kips} \)

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.

- \( M_{p,\text{exp.CB}} = 1,583 \text{ kip-ft} \)  
  (Expected flexural capacity of CB)
- \( V_{n,\text{M,exp.CB}} = \frac{2A_{\text{M,exp.CB}}}{L_{CB}} = 380 \text{ kips} \)  
  (Expected shear strength of CB)
- \( \gamma_1 = \frac{\sum_{n=1}^{1.2} M_{p,\text{exp.CB}}}{\sum_{n} M_{U,CB}} = \frac{(18)(1)(1583)}{(18)(835)} = 2.27 \)  
  (Overstrength amplification factor)
- \( P_{CB} = 2 \sum_{n} V_{n,\text{M,exp.CB}} = 13,673 \text{ kips} \)  
  (Axial force due to coupling action)
- \( M_{r,\text{wall}} = \gamma_1 OTM - P_{CB} L_{eff} = 125,077 \text{ kip-ft} \)  
  (Required amplified OTM)
- \( P = -2 \sum_{n} V_{n,\text{M,exp.CB}} - (1.2 \sum_{n} F_{T_{\text{Tr,DLL}}} - (0.5 \sum_{n} F_{T_{\text{Tr,LL}}} = -20,644 \text{ kips} \)  
  (axial compression force)
- \( T = 2 \sum_{n} V_{n,\text{M,exp.CB}} - (0.9 \sum_{n} F_{T_{\text{Tr,DLL}}} = 9,219 \text{ kips} \)  
  (axial tension force)
Design Of C-PSW/CFs

Wall Tensile Strength:
- \( P_{n,T} = A_f F_y = (604)(50) = 30,200 \text{ kips} \)
- \( \phi_T P_{n,T} = 27,180 \text{ kips} \)
- \( \frac{\phi_T P_{n,T}}{T} = 0.35 \)

Wall Compression Strength:
- A simplified unit 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)
- \( L_{wall,\text{total}} = 48 \text{ ft} \) (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)
- \( P_e = \frac{\pi^2 E l_{eff} \text{min}}{l_{cr}^4} = 1797 \text{ kips} \)
- \( \frac{P_{no}}{P_e} = 0.84 < 2.25 \) (ANSI/AISC 360-22)
- \( P_{n.C} = P_{no} \left(0.685 \frac{P_{no}}{P_e}\right) = 1,066 \text{ kips} \)
- \( P_{n.C,\text{total}} = P_{n.C} n_{unit-width} = (1,066 \text{ kips})(48) = 51,168 \text{ kips} \)
- \( \phi_C \frac{P_{n.C,\text{total}}}{P} = (0.9)(51,168 \text{ kips}) = 46,051 \text{ kips} \) > \( P = 20,644 \text{ kips} \)
- \( \phi_C P_{n.C,\text{total}} = 0.45 \)
Design Of C-PSW/CFs (Flexural Strength)

Plastic Stress Distribution:

\[ M_{P.T.wall} = M_{n.T.wall} = 1,598,236 \text{ kip-in.} \]
\[ M_{P.C.wall} = M_{n.T.wall} = 1,761,166 \text{ kip-in.} \]

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.

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

Ratio of demand to capacity:

- \( \frac{M_{U.T.wall}}{\Phi IM_{n.T.wall}} = 0.45 \)
- \( \frac{M_{U.C.wall}}{\Phi IM_{n.C.wall}} = 0.54 \)
P-M Interaction of C-PSW/CFs

![Compression Walls Diagram](image1)

![Tension Walls Diagram](image2)

Design Of C-PSW/CFs (Shear Strength)

Wall Shear Strength:

- \( A_{S,wall} = 4 \left( L_w t_p \right) + 2 \left( t_{SC} t_p \right) = (4)(144)(0.5) + (2)(16)(0.5) = 304 \text{ in.}^2 \)
- \( K_s = G_c A_{S,wall} = (11200)(304) = 3.39 \times 10^6 \text{ kips} \)
- \( K_{sc} = \frac{0.7 (E_c A_c) (E_S A_{S,wall})}{(4E_S A_{S,wall})(E_c A_c)} = 3.14 \times 10^6 \text{ kips} \)
- \( V_{n,wall} = \frac{K_s + K_{sc}}{\sqrt{3 K_s^2 + K_{sc}^2}} A_{S,wall} F_y = 14906 \text{ kips} \)
- \( \phi_V V_{n,wall} = 13416 \text{ kips} > V_{U,wall} = 2476 \text{ kips} \)
- \( \frac{V_{U,wall}}{\phi_V V_{n,wall}} = 0.19 \)
Coupling Beam-to-Wall Connection

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

- C-PSW/CF Flange
- C-PSW/CF Web
- Coupling Beam Flange
- Coupling Beam Web
- CJP Weld
- Fillet Weld
- Slots in C-PSW/CF Web
Coupling Beam-to-Wall Connection

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

Flange Plate Connection Demand:

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

Required Length of CJP Welding:

- \( \frac{T_{\text{flange}}}{2} \leq \phi_d 0.6 F_y t_{CB,f} L_{\text{req.}} \) \( \phi_d = 1.0 \)
- \( \phi_n = 0.9 \)
- \( L_{\text{req}} \geq \frac{T_{\text{flange}}}{2\phi_d 0.6 F_y t_{CB,f}} = \frac{504}{2(1.0)(0.6)(50)(0.5)} = 19.8 \) in.
- \( L_{\text{weld,f}} = 20 \) in.
Check Shear Strength of Coupling Beam Flange Plate

Shear yielding of coupling beam flange plate:
- \( A_{fy} = t_{CB} f L_{weld} = (0.5)(20) = 10 \text{ in.}^2 \)
- \( \phi_d 0.6 F_y A_{fy} = 300 \text{ kips} \geq \frac{T_{flange}}{2} = 297 \text{ kips} \)

Shear rupture of coupling beam flange plate:
- \( A_{f,SR} = t_{CB} f L_{weld} = (0.5)(20) = 10 \text{ in.}^2 \)
- \( \phi_n 0.6 F_u A_{f,SR} = 351 \text{ kips} > \frac{T_{flange}}{2} = 297 \text{ kips} \)

Check Shear Strength of Wall Web Plates

Shear yielding of wall web plates:
- \( A_{w,SY} = 2 t_p L_{weld} = 2(0.5)(20) = 20 \text{ in.}^2 \)
- \( \phi_d 0.6 F_y A_{w,SY} = 600 \text{ kips} \geq \frac{T_{flange}}{2} = 297 \text{ kips} \)

Shear rupture of wall web plates:
- \( A_{w,SR} = 2 t_p L_{weld} = 2(0.5)(20) = 20 \text{ in.}^2 \)
- \( \phi_n 0.6 F_u A_{w,SR} = 702 \text{ kips} > \frac{T_{flange}}{2} = 297 \text{ kips} \)
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.

\[ A_{LM,f,g} = (b_{CB} + 2\text{ in.}) \cdot t_{CB,f} = (16 + 2)(0.5) = 9 \text{ in.}^2 \] (Gross area)

\[ A_{LM,f,n} = b_{CB} \cdot t_{CB,f} = (16)(0.5) = 8 \text{ in.}^2 \] (Net area)

\[ R_y \cdot F_y \cdot A_{LM,f,g} = (1.1)(50)(9) = 495 \text{ kips} \] (Available tension yielding capacity)

\[ R_t \cdot F_u \cdot A_{LM,f,n} = (1.1)(65)(8) = 572 \text{ kips} \] (Available tension rupture capacity)

\[ R_t \cdot F_u \cdot A_{LM,f,n} = 572 \text{ kips} > R_y \cdot F_y \cdot A_{LM,f,g} = 495 \text{ kips} \]

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 \( M_{CB,exp} \))

\[ T_{web} = 1.2 \left( T_{2,exp} - C_{2,exp} \right) = 667 \text{ kips} \] (CB web plates tension force)

\[ M_{web} = 1.2 \left( T_{2,exp} \frac{C_{CB,exp}}{2} + C_{2,exp} \frac{h_{CB} \cdot C_{CB,exp}}{2} \right) = 407 \text{ kip-ft} \] (CB web plates moment)

\[ V_{web} = 2 \left( \frac{1.2 \cdot M_{p,exp,EB}}{L_{CB}} \right) = 380 \text{ kips} \] (CB web plates shear force)
Calculate Force Demand on C-Shaped Weld

\[ T_{C,\text{weld}} = \frac{T_{\text{web}}}{2} = 333 \text{ kips} \]
\[ M_{C,\text{weld}} = \frac{M_{\text{web}}}{2} = 203 \text{ kip-ft} \]
\[ V_{C,\text{weld}} = \frac{V_{\text{web}}}{2} = 190 \text{ kips} \]

\[ D_{\text{min}} = \frac{3}{16} \text{ in.} \]
\[ D_{\text{max}} = \frac{5}{16} \text{ in.} \]
\[ D = \frac{5}{16} \text{ in.} \]
\[ D_{\text{min}} \leq D \leq D_{\text{max}} \]

Calculate Capacity of C-Shaped Weld

**Eccentricity** = \( \frac{M_{C,\text{weld}}}{V_{C,\text{weld}}} = 12.85 \text{ in.} \)
\[ c.\ g. = \frac{L_{H,\text{weld,w}}}{2L_{H,\text{weld,w}} + L_{V,\text{weld,w}}} = \frac{30^2}{2(36) + (22)} = 10.98 \text{ in.} \]
\[ e_x = \text{Eccentricity} + (L_{H,\text{weld,w}} - c.\ g.) = 31.88 \text{ in.} \]
\[ k = \frac{L_{H,\text{weld,w}}}{L_{V,\text{weld,w}}} = \frac{30}{22} = 1.36 \]
\[ a = \frac{e_x}{L_{V,\text{weld,w}}} = \frac{11}{22} = 1.45 \]
\[ P_{V,\text{weld}} = \phi P \frac{C_{8.8} \cdot C_{1-8.3} \cdot (16D)L_{V,\text{weld,w}}}{} \]
\[ P_{V,\text{weld}} = 334 \text{ kips} \]
\[ V_{C,\text{weld}} = 190 \text{ kips} \]
\[ \frac{V_{C,\text{weld}}}{P_{V,\text{weld}}} = 0.62 \]
Calculate Capacity of C-Shaped Weld

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

\[ P_{T,\text{weld}} = 501 \text{ kips} \quad \Rightarrow \quad T_{c,\text{weld}} = 333 \text{ kips} \]

\[ \frac{T_{c,\text{weld}}}{P_{T,\text{weld}}} = 0.67 \]

\[ \text{Capacity} = \sqrt{\frac{N_{c,\text{weld}}^2}{P_{V,\text{weld}}}} = \sqrt{(0.64)^2(0.67)^2} = 0.91 \leq 1 \]

Questions
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