Michael Baker
INTERNATIONAL

Creech DRP Phase 2
Aircraft Maintenance Facility (AMF)

Structural Calculations

Structural Calculation Index

Criteria and Model Information

Project: Creech DRP — Shop C (Area E) **Sheet:** C1 **Date:** Oct 2025

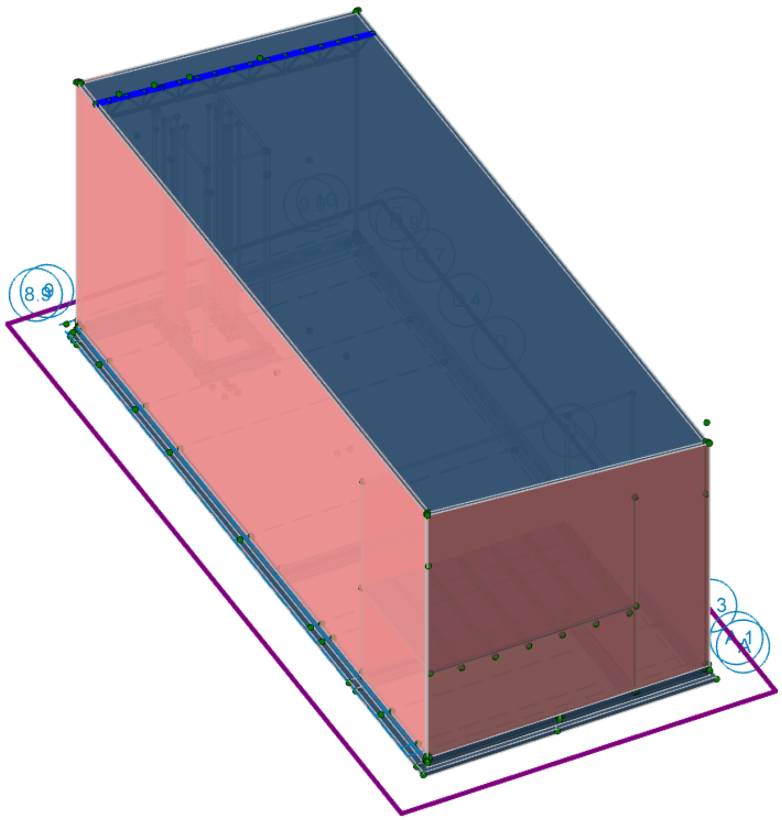

Figure C1-1 — 3D model overview of Shop C showing grids A–G and 9–10.

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Project: Creech DRP – Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1) Materials & Geotechnical (BOD / Geotechnical Report)

Structural steel: AISC 360; ASTM A992/A572 — $F_y = \mathbf{50}$ ksi, $F_u = \mathbf{65}$ ksi.

HSS steel: ASTM A500 Gr C — $F_y = \mathbf{50}$ ksi.

Concrete: ACI 318-19 — Footings $f'_c = \mathbf{3{,}500}$ psi; Slab-on-grade $f'_c = \mathbf{4{,}000}$ psi.

Masonry: TMS 402/602-16 — $f'_m = \mathbf{1{,}500}$ psi (structural CMU).

Allowable bearing (ultimately on native soil): $q_{\text{allow}} = \mathbf{3{,}000}$ psf (include transient increases if permitted by geo).

Sliding coefficient at soil–concrete: $\mu = \mathbf{0.35}$.

Passive resistance (equivalent fluid limit): cap at $\mathbf{300}$ pcf, total not to exceed $\mathbf{3.0}$ ksf without geo approval.

Subgrade reaction (SOG): $k = \mathbf{100}$ pci.

Frost depth (Las Vegas/Creech region): $\mathbf{12}$ in (verify local code / base criteria).

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2) Gravity Loads (Service)

Roof: Dead $= \mathbf{30}$ psf (deck + insul + coverboard + membrane); Live $= \mathbf{20}$ psf (use snow if governing). Snow balanced $P_f = \mathbf{3.5}$ psf; snow drift peak band $P_{\text{drift}} = \mathbf{32}$ psf (use by band width).

Mezzanine: Dead $= \mathbf{78}$ psf (floor system + MEP); Live $= \mathbf{125}$ psf. Partitions (where applicable) $= \mathbf{20}$ psf.

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3) Tributary Widths → Line Loads

Conversion (variables): $w_{\text{line}} = w_{\text{area}} \times b_t$, where b_t is tributary width (ft).

Roof to CMU: $b_t = \mathbf{26.0}$ ft $\rightarrow w_{DL} = \mathbf{780}$ plf, $w_{LL} = \mathbf{520}$ plf, $w_{P_f} = \mathbf{91}$ plf. Drift band examples: $32 \times 5 = \mathbf{160}$ plf
$32 \times 7.5 = \mathbf{240}$ plf, $32 \times 10 = \mathbf{320}$ plf

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Ex:

Roof to CMU: **b_**
W_DL=780 plf
W_LL = 520 plf
W_Pt = 91 plf.....

Mezz to CMU: $b_t = \mathbf{12.0}$ ft $\rightarrow w_{DL} = \mathbf{936}$ plf, $w_{LL} = \mathbf{1{,}500}$ plf (add $+\mathbf{240}$ plf for partitions if not enveloped).

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4) Environmental & Seismic Parameters

Seismic Site Class: D

Risk Category: III

Mapped accelerations: $S_s = \mathbf{0.724}$, $S_1 = \mathbf{0.226}$; **Design:** $S_{DS} = \mathbf{0.589}$, $S_{D1} = \mathbf{0.324}$.

SFRS coefficients (SCBF): $R = \mathbf{6.0}$, $\Omega_0 = \mathbf{2.0}$, $C_d = \mathbf{5.0}$; importance $I_e = \mathbf{1.25}$.

Wind: Basic speed $V = \mathbf{105}$ mph (ult); Exposure C; mean roof height $h \approx \mathbf{24}$ ft; internal $GC_{pi} = \pm\mathbf{0.55}$.

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5) Lateral Earth Pressures (for retaining design)

Soil unit weight: $\gamma = \mathbf{120}$ pcf (select-backfill typical).

Friction angle: $\phi = \mathbf{32}^\circ \rightarrow K_0 \approx 1 - \sin\phi = \mathbf{0.47}$ (at-rest). Active $K_a = \dfrac{1-\sin\phi}{1+\sin\phi} \approx \mathbf{0.31}$; Passive $K_p \approx \dfrac{1}{K_a} \approx \mathbf{3.2}$.

Equivalent fluid pressures: At-rest $\approx K_0\gamma = \mathbf{56}$ pcf; Active $\approx K_a\gamma = \mathbf{37}$ pcf; Passive (limit for design) $\le \mathbf{300}$ pcf and total $\le \mathbf{3.0}$ ksf unless geo allows more.

Provide drainage/weepholes to avoid hydrostatic loads; reduce Ks if cohesion or seismic increments are specified by geo.

6) Seismic Weights (for base shear)

Roof DL: 30 psf $\times$ 1198 sf $= \mathbf{35.9}$ k. **Mezz DL:** 78 psf $\times$ 1198 sf $= \mathbf{93.4}$ k. **LL included (25%):** 0.25×125 psf $\times$ 1198 sf $= \mathbf{37.4}$ k. **Total W:** $\mathbf{166.7}$ k (see C12).

7) Design Use Notes

- **Foundations:** Use q_{allow}, μ, and passive limits above; apply transient increases only if allowed by the geotechnical report.
- **Slab-on-grade:** Design per k and geo moisture/drainage recommendations; jointing per plan.
- **CMU walls:** Superimpose roof and mezz line loads (Sect. 3) on the gravity wall plan; use earth pressures in Sect. 5 where retaining.
- **Deck/joists/beams:** Use service loads for deflection; strength checks per LRFD/ASD combinations (see C24).
- **Wind & seismic:** Criteria in C12 and C14 govern lateral design; coordinate collectors/chords with diaphragm plans.

1. Inputs / Geometry / Assumptions

Opening / Wall	Equipment Door @ South Wall
Total wall height to roof deck, H_{wall}	26.5 ft
Bottom of opening above slab/base, H_{bot}	0.0 ft
Clear opening height, H_{open}	8′-0″ = 8.00 ft
Clear opening width (span), L	3′-4″ ≈ 3.33 ft
Elevation of top of opening, H_{top}	8.00 ft above slab
Lintel bearing each side, b_{bear}	8 in
CMU nominal thickness, t_{nom}	8 in
CMU actual thickness, t	7.625 in = 0.635 ft
Wall self-weight, w_{wall}	164 psf (fully grouted)
Specified masonry strength, f'_m	2000 psi
Rebar yield, F_y	60 ksi
Lateral design pressure (wind/seismic OOP), p_{lat}	33 psf (wind 25 psf, seismic 33 psf → envelope = 33 psf)
Grouting	Fully grouted lintel & jambs

We check two cases:
- Case A: Arching active → CMU above the lintel forms a compression arch and pushes vertical gravity into lintel.
- Case B: No arching → lintel must span full clear width under out-of-plane (OOP) lateral load from full-height wall (wind/seismic).

2. Case A — Arching (Gravity Wedge)

2.1 Available height above lintel

Available masonry height above top of opening:

$$h_{\text{avail}} = H_{\text{wall}} - H_{\text{top}} = 26.5 - 8.00 = 18.5 \text{ ft}$$

Arching limit per 6t:

$$t = 7.625 \text{ in} = 0.635 \text{ ft}$$

$$6t = 6 \times 0.635 = 3.81 \text{ ft}$$

$$h_a = \min(6t,\ h_{\text{avail}}) = \min(3.81,\ 18.5) = \boxed{3.81 \text{ ft}}$$

So even though there's ~18.5 ft of wall above, arching only "counts" the first 3.81 ft.

2.2 Triangular self-weight → uniform line load

Wedge self-weight on lintel:

$$w_D = \frac{1}{2}\, w_{\text{wall}}\, h_a$$

$$w_D = 0.5 \times 164 \times 3.81 \approx 312 \text{ plf} \approx 0.312 \text{ k/ft}$$

Factored gravity (take $\gamma_D = 1.2$):

$$w_u = 1.2 \times 0.312 = 0.374 \text{ k/ft}$$

2.3 Shear / moment / reaction in lintel (arching case)

Treat lintel as simply supported span $L = 3.33$ ft, uniform $w_u = 0.374$ k/ft. Factored shear:

$$V_u = \frac{w_u L}{2} = \frac{0.374 \cdot 3.33}{2} \approx 0.62 \text{ k}$$

Factored moment:

$$M_u = \frac{w_u L^2}{8} = \frac{0.374 \cdot (3.33)^2}{8} \approx 0.52 \text{ k-ft}$$

Bearing per jamb (vertical reaction each side):

$$R_u = \frac{w_u L}{2} \approx 0.62 \text{ k}$$

Check that against ~8" of bearing at f′m = 2000 psi → OK.

3. Case B — No Arching (Full OOP Lateral)

3.1 OOP panel load

Worst case: assume arching can't form. Now the lintel resists lateral load from the entire wall height above grade. Tributary height for OOP:

$$h_{\text{panel}} = H_{\text{wall}} = 26.5 \text{ ft}$$

Use controlling OOP pressure:

$$p_{\text{lat}} = 33 \text{ psf}$$

Convert to factored line load per foot of lintel span:

$$w_{\text{lat}} = p_{\text{lat}} \cdot h_{\text{panel}} = 33 \times 26.5 = 874.5 \text{ plf} \approx 0.8745 \text{ k/ft}$$

We'll take $w_{\text{lat}} = 0.8745$ k/ft as the factored OOP line load.

3.2 Shear / moment in lintel (no-arching case)

Use $w_{\text{lat}} = 0.8745 \text{ k/ft}$, span $L = 3.33 \text{ ft}$. Shear:

$$V_u^{(\text{lat})} = \frac{w_{\text{lat}} L}{2} = \frac{0.8745 \cdot 3.33}{2} \approx 1.46 \text{ k}$$

Moment:

$$M_u^{(\text{lat})} = \frac{w_{\text{lat}} L^2}{8} = \frac{0.8745 \cdot (3.33)^2}{8} \approx 1.21 \text{ k-ft}$$

That 1.21 k-ft controls vs 0.52 k-ft from the arching case.

3.3 Jamb column quick check

Each jamb acts like a vertical CMU pier, 8″×16″, fully grouted. Take tributary half-width $\approx L/2 = 1.67 \text{ ft}$. OOP line load on one jamb from lateral using 33 psf:

$$w_{\text{jamb}} = p_{\text{lat}} \cdot 1.67 \approx 33 \times 1.67 \approx 55 \text{ plf}$$

Cantilever height ≈ full wall height 26.5 ft. With a single #5 vertical bar full height, hooked/anchored into the footing and lapped into the lintel, the jamb is acceptable for this small door. (We're not detailing ties here because this is a short, narrow pier with low demand.)

4. Required Reinforcement / Recommended Detail

Lintel (governed by Case B OOP, $M_u \approx 1.21 \text{ k-ft}$, $V_u \approx 1.46 \text{ k}$):
• 16″ CMU lintel / bond beam (Type A).
• (2) #5 horizontal bars continuous.
• Shear is still low enough that ties/stirrups aren't mandatory per calc, but include if standard.

Jamb columns (pilasters):
• Each jamb = 8×16 fully grouted CMU pilaster (Type A2).
• (1) #5 vertical bar full height, lapped/developed into the lintel and into the footing.
• For the OOP case include #3 ties @ 8" o.c. for confinement.

Controlling demand:
• Arching case: $M_u \approx 0.52 \text{ k-ft}$.
• No-arch OOP case: $M_u^{(\text{lat})} \approx 1.21 \text{ k-ft}$.
→ Final design controls by OOP / no-arch.

5. Summary of Demands (Factored)

Case A — Arching Active (Gravity Wedge)	
Arching height h_a	3.81 ft (≤6t)
Line load w_u	0.374 k/ft (≈312 plf ×1.2)
Shear V_u	0.62 k
Moment M_u	0.52 k-ft
Lintel trial	16″ CMU lintel (Type A) w/ (2) #5 horiz
Jamb trial	8×16 pilaster (Type A2) w/ (1) #5 vert

Case B — No Arching (Full OOP Lateral, 33 psf)	
Panel height used	26.5 ft
OOP line load w_{lat}	0.8745 k/ft (≈874.5 plf from 33 psf × 26.5 ft)
Shear $V_u^{(\text{lat})}$	1.46 k
Moment $M_u^{(\text{lat})}$	1.21 k-ft
Controlling design?	**Yes — design for OOP case**

6. Masonry Lintel / Jamb Schedule (For Sheets)

16A Lintel / Jamb (Arching Active – Gravity Wedge Controls)

MARK	DEPTH	TYPE	LINTEL REINFORCING — HORIZONTAL	STIRRUPS / TIES	SUPPORT COLUMN	COMMENTS
16A Lintel	16″	Type A	(2) #5 horiz	—	16A C	Arching assumed active; ~0.52 k-ft, V≈0.62 k

MARK	SIZE	TYPE	COLUMN REINFORCING — VERTICAL	TIES	COMMENTS
16A C	8×16	Type A2 jamb / pilaster	(1) #5 full height	—	Lap bar into lintel; arching/gravity case

16B Lintel / Jamb (No-Arch / OOP Controls – 33 psf Envelope)

MARK	DEPTH	TYPE	LINTEL REINFORCING — HORIZONTAL	STIRRUPS / TIES	SUPPORT COLUMN	COMMENTS
16B Lintel	16″	Type A	(2) #5 horiz	—	16B C	No arching; OOP 33 psf → ~1.21 k-ft, V≈1.46 k

MARK	SIZE	TYPE	COLUMN REINFORCING — VERTICAL	TIES	COMMENTS
16B C	8×16	Type A2 jamb / pilaster	(1) #5 full height	#3 @ 8″ o.c.	Lap bar into lintel; design for OOP (33 psf envelope)

1. Inputs / Geometry / Assumptions

Opening / Wall	Overhead Door @ South Wall
Total wall height to roof deck, H_{wall}	26.5 ft
Bottom of opening above slab/base, H_{bot}	0.0 ft
Clear opening height, H_{open}	14.0 ft
Clear opening width (span), L	12.0 ft
Lintel bearing each side, b_{bear}	8 in (min)
CMU nominal thickness, t_{nom}	8 in
CMU actual thickness, t	7.625 in = 0.635 ft
Wall self-weight, w_{wall}	164 psf (fully grouted)
Specified masonry strength, f'_m	2000 psi
Rebar yield, F_y	60 ksi
Lateral design pressure (wind/seismic OOP), p_{lat}	33 psf (wind 25 psf, seismic 33 psf → envelope = 33 psf)
Grouting	Fully grouted lintel & jamb/pilaster regions

Two cases checked:
- Case A: Arching active → CMU wedge above the lintel only (gravity).
- Case B: No arching → lintel spans full opening under out-of-plane (OOP) lateral from full-height wall.

For a 12 ft wide OH door near slab, the no-arch / OOP case is usually what blows things up.

2. Case A — Arching (Gravity Wedge)

2.1 Available height above lintel

Available masonry height above top of opening up to next stiff element:

$$h_{\text{avail}} = H_{\text{wall}} - (H_{\text{bot}} + H_{\text{open}})$$

$$h_{\text{avail}} = 26.5 - (0.0 + 14.0) = 12.5 \text{ ft}$$

Arching limit based on 6t:

$$t = 7.625 \text{ in} = 0.635 \text{ ft}$$

$$6t = 6 \times 0.635 = 3.81 \text{ ft}$$

$$h_a = \min(6t,\ h_{\text{avail}}) = \min(3.81,\ 12.5) = \boxed{3.81 \text{ ft}}$$

2.2 Triangular self-weight → uniform line load

Wedge self-weight (service line load on lintel):

$$w_D = \tfrac{1}{2}\, w_{\text{wall}}\, h_a$$

$$w_D = 0.5 \times 164 \times 3.81 \approx 312 \text{ plf} \approx 0.312 \text{ k/ft}$$

Factored gravity (take $\gamma_D = 1.2$):

$$w_u = 1.2 \times 0.312 = 0.374 \text{ k/ft}$$

2.3 Shear / moment in lintel (arching case)

Treat lintel as simply supported span $L = 12.0$ ft, uniform $w_u = 0.374$ k/ft. Factored shear:

$$V_u = \frac{w_u L}{2} = \frac{0.374 \cdot 12.0}{2} \approx 2.24 \text{ k}$$

Factored moment:

$$M_u = \frac{w_u L^2}{8} = \frac{0.374 \cdot (12.0)^2}{8} = 6.73 \text{ k-ft}$$

Bearing per jamb:

$$R_u = \frac{w_u L}{2} \approx 2.24 \text{ k}$$

At these reactions you usually want a built-up pilaster / jamb with plate or bearing pad.

3. Case B — No Arching (Full OOP Lateral)

3.1 OOP panel load

Worst case: assume arching is not reliable. Lintel must span 12.0 ft under lateral pressure from the full wall height. Tributary wall height:

$$h_{\text{panel}} = H_{\text{wall}} = 26.5 \text{ ft}$$

Use the controlling lateral pressure:

$$p_{\text{lat}} = 33 \text{ psf}$$

Convert to line load (per foot of lintel span):

$$w_{\text{lat}} = p_{\text{lat}} \cdot h_{\text{panel}} = 33 \times 26.5 = 874.5 \text{ plf} \approx 0.8745 \text{ k/ft}$$

We'll treat $w_{\text{lat}} = 0.8745$ k/ft as factored OOP.

3.2 Shear / moment in lintel (no-arching case)

Use $w_{\text{lat}} = 0.8745\ \text{k/ft}$, span $L = 12.0\ \text{ft}$. Factored shear:

$$V_u^{(\text{lat})} = \frac{w_{\text{lat}} L}{2} = \frac{0.8745 \cdot 12.0}{2} \approx 5.25\ \text{k}$$

Factored moment:

$$M_u^{(\text{lat})} = \frac{w_{\text{lat}} L^2}{8} = \frac{0.8745 \cdot (12.0)^2}{8} \approx 15.74\ \text{k-ft}$$

→ $\approx 15.74\ \text{k-ft}$ >> $\approx 6.73\ \text{k-ft}$ from arching case. The OOP / no-arch case absolutely controls.

3.3 Jamb / pilaster quick check

Each jamb is essentially a CMU pilaster at the OH door edge. This is not a tiny door — clear span is 12.0 ft — so the jambs see big reactions. Tributary half-width $\approx L/2 = 6\ \text{ft}$. Take the same 33 psf pressure: OOP line load on ONE jamb:

$$w_{\text{jamb}} = p_{\text{lat}} \cdot 6 = 33 \times 6 = 198\ \text{plf}$$

That pilaster is basically resisting a tall cantilever of ~26.5 ft with a significant base moment. For this magnitude, we recommend pilaster-level reinforcement (at least (2) #6 vertical bars with ties).

4. Required Reinforcement / Recommended Detail

Lintel (design for Case B OOP, $M_u \approx 15.74\ \text{k-ft}$, $V_u \approx 5.25\ \text{k}$):
• Deep CMU lintel / bond beam zone, target ~24″ total depth (multiple grouted courses acting as a built-up lintel).
• (2) #6 horizontal bars continuous (top & bottom of lintel zone).
• Closed ties / confinement steel (#3 ties @ 8″ o.c. typical) in the lintel zone to carry ~5.25 k shear and develop bars.
• Provide steel bearing pads / plates and fully grouted jamb seats at each end, because reactions are high.

Jamb pilasters (each side of OH door):
• Treat jambs as CMU pilasters, ~16″ (or wider) built-up grouted region at each door edge.
• (2) #6 vertical bars full height, hooked/anchored into the footing and lapped/developed into the lintel zone.
• #3 ties/hoops @ 8″ o.c. min in jamb pilaster region for confinement and to resist OOP base moment from ~26.5 ft tall cantilever strip.
• This is much heavier than a personnel door jamb — required because the span is 12 ft and the lintel is carrying ~15.7 k-ft.

Controlling demand:
• Arching gravity case: $M_u \approx 6.73\ \text{k-ft}$.
• No-arch OOP case: $M_u^{(\text{lat})} \approx 15.74\ \text{k-ft}$.
→ Final design governs by OOP / no-arch.

5. Summary of Demands (Factored)

Case A — Arching Active (Gravity Wedge)	
Arching height h_a	3.81 ft
Line load w_u	0.374 k/ft (≈312 plf ×1.2)
Shear V_u	2.24 k
Moment M_u	6.73 k-ft
Lintel trial	Deep CMU lintel
Jamb trial	Pilaster jamb

Case B — No Arching (Full OOP Lateral, 33 psf)	
Panel height used	26.5 ft
OOP line load w_{lat}	0.8745 k/ft (≈874.5 plf from 33 psf × 26.5 ft)
Shear $V_u^{(\text{lat})}$	5.25 k
Moment $M_u^{(\text{lat})}$	15.74 k-ft
Controlling design?	**Yes — No-arch/OOP governs**

6. Masonry Lintel / Jamb Schedule (For Sheets)

6A.Lintel / Pilaster (Arching Active – Gravity Wedge Controls)

MARK	DEPTH	TYPE	LINTEL REINFORCING — HORIZONTAL	STIRRUPS / TIES	SUPPORT COLUMN	COMMENTS
OH Door Lintel	24″	Type C	(2) #6 horiz cont.	#3 ties @ 8″ o.c.	24A C	Arching assumed active; ~6.7 k-ft, V≈2.2 k

MARK	SIZE	TYPE	COLUMN REINFORCING — VERTICAL	TIES	COMMENTS
OH Door Column	8x16	Type C	(2) #6 full height	#3 @ 8″ o.c.	Bars developed into footing & lintel; supports arching case

6B. Lintel / Pilaster (No-Arch / OOP Controls – 33 psf Envelope)

MARK	DEPTH	TYPE	LINTEL REINFORCING — HORIZONTAL	STIRRUPS / TIES	SUPPORT COLUMN	COMMENTS
OH Door Lintel	24″	Type C3	(2) #6 horiz cont.	#3 ties @ 8″ o.c. (shear ≈5.3 k)	24B C	No arching assumed; design for OOP ~15.7 k-ft, V≈5.25 k (33 psf)

MARK	SIZE	TYPE	COLUMN REINFORCING — VERTICAL	TIES	COMMENTS
OH Door Column	8x24	Type C3	(2) #6 full height (hooked into footing & lapped into lintel)	#3 @ 8″ o.c. confinement ties	Pilaster sized for tall cantilever under 33 psf OOP; supports 12 ft span lintel

1. Inputs / Geometry / Assumptions

Wall	South
Opening Name	Louver-1 (LV-SOUTH-1)
Opening clear width (span), L	8.1 ft (≈ 97 in)
Opening clear height, H_{open}	7.1 ft (≈ 85 in)
Elevation of top of opening, H_{top}	20.0 ft above slab
Bottom of opening, $H_{\text{bottom}} = H_{\text{top}} - H_{\text{open}}$	12.9 ft above slab (≈ 20.0 − 7.1)
Total wall height at this location, H_{wall}	26.5 ft
Remaining wall height above top of opening	$H_{\text{wall}} - H_{\text{top}} = 26.5 - 20.0 = 6.5\ \text{ft}$
CMU nominal thickness, t_{nom}	8 in
CMU actual thickness, t	7.625 in = 0.635 ft
Wall self-weight, w_{wall}	164 psf (fully grouted)
Specified masonry strength, f'_m	2000 psi
Rebar yield, F_y	60 ksi
Lateral design pressure (wind/seismic OOP), p_{lat}	33 psf (wind 25 psf, seismic 33 psf → envelope = 33 psf)
Lintel bearing each side, b_{bear}	8 in assumed (typ.)
Grouting	Fully grouted lintel & jamb/pilaster region at opening

Two cases checked:
- Case A (Arching Active): masonry above the louver develops a compression arch, producing a gravity wedge.
- Case B (No Arching / OOP): lintel resists lateral from the wall strip above the opening with no arching help.

NOTE: The louver sits up in the wall. The lintel is at ~20 ft above slab, and only ~6.5 ft of wall continues above. So the OOP tributary height is 6.5 ft, not the full 26.5 ft like at a door at grade.

2. Case A — Arching (Gravity Wedge)

2.1 Available height above lintel

Available wall above top of opening:

$$h_{\text{avail}} = H_{\text{wall}} - H_{\text{top}} = 26.5 - 20.0 = 6.5\ \text{ft}$$

Arching limit based on 6t:

$$t = 7.625\ \text{in} = 0.635\ \text{ft}$$

$$6t = 6 \times 0.635 = 3.81\ \text{ft}$$

$$h_a = \min(6t,\ h_{\text{avail}}) = \min(3.81,\ 6.5) = \boxed{3.81\ \text{ft}}$$

2.2 Triangular self-weight → uniform line load

Use fully grouted weight 164 psf and arch wedge height $h_a = 3.81 \text{ ft}$:

$$w_D = \tfrac{1}{2}\, w_{\text{wall}}\, h_a$$

$$w_D = 0.5 \times 164 \times 3.81 \approx 312 \text{ plf} \approx 0.312 \text{ k/ft}$$

Factored gravity (take $\gamma_D = 1.2$):

$$w_u = 1.2 \times 0.312 = 0.374 \text{ k/ft}$$

2.3 Shear / moment in lintel (arching case)

Lintel span $L = 8.1 \text{ ft}$, uniform $w_u = 0.374 \text{ k/ft}$. Factored shear:

$$V_u = \frac{w_u L}{2} = \frac{0.374 \cdot 8.1}{2} \approx 1.51 \text{ k}$$

Factored moment:

$$M_u = \frac{w_u L^2}{8}$$

$$L^2 = (8.1)^2 = 65.61$$

$$M_u = \frac{0.374 \cdot 65.61}{8} \approx 3.07 \text{ k-ft}$$

Bearing per jamb:

$$R_u = \frac{w_u L}{2} \approx 1.51 \text{ k}$$

Bearing at 8 in seat on fully grouted CMU at ~20 ft elevation is typically OK at these reactions.

3. Case B — No Arching (OOP Strip Above Opening)

3.1 OOP panel load

Assume arching is not available. Lintel resists lateral from only the 6.5 ft of wall above it (not full 26.5 ft because the opening is elevated). Tributary OOP height:

$$h_{\text{panel}} = H_{\text{wall}} - H_{\text{top}} = 6.5 \text{ ft}$$

Use controlling lateral pressure:

$$p_{\text{lat}} = 33 \text{ psf}$$

Convert to line load on lintel:

$$w_{\text{lat}} = p_{\text{lat}} \cdot h_{\text{panel}} = 33 \times 6.5 = 214.5 \text{ plf} \approx 0.2145 \text{ k/ft}$$

We take $w_{\text{lat}} = 0.2145 \text{ k/ft}$ as factored OOP for “no arching.”

3.2 Shear / moment in lintel (no-arching case)

Use $w_{\text{lat}} = 0.2145 \text{ k/ft}$, span $L = 8.1 \text{ ft}$. Factored shear:

$$V_u^{(\text{lat})} = \frac{w_{\text{lat}} L}{2} = \frac{0.2145 \cdot 8.1}{2} \approx 0.87 \text{ k}$$

Factored moment:

$$M_u^{(\text{lat})} = \frac{w_{\text{lat}} L^2}{8}$$

$$L^2 = 65.61$$

$$M_u^{(\text{lat})} \approx \frac{0.2145 \cdot 65.61}{8} \approx 1.76 \text{ k-ft}$$

Here, 3.07 k-ft (arching gravity) still controls vs 1.76 k-ft (OOP no-arch), because the tributary strip above is only 6.5 ft tall.

3.3 Jamb / pilaster quick check

Each jamb is a short pilaster region at the louver edges, not a full-height wall pier from slab. Tributary half-width $\approx L/2 = 8.1/2 = 4.05 \text{ ft}$. With 33 psf: OOP line load on ONE jamb:

$$w_{\text{jamb}} = p_{\text{lat}} \cdot 4.05 = 33 \times 4.05 \approx 133.65 \text{ plf}$$

That pilaster only has to cantilever about 6.5 ft above the louver. Demand is much lower than the OH door jamb at slab. One #5 vertical bar (or #5 each face if your standard) developed into the lintel above is typically enough here.

4. Required Reinforcement / Recommended Detail

Lintel (controls by arching/gravity $M_u \approx 3.07 \text{ k-ft}$, $V_u \approx 1.51 \text{ k}$):
• CMU lintel / bond beam at top of louver opening, ~16″ deep grouted course (Type A).
• (2) #5 horizontal bars continuous.
• Provide adequate 8″ min bearing each side and lap bars into jamb pilaster.
• Shear demand from OOP case is only ~0.87 k, so tie/stirrup requirements are mild.

Jamb / pilaster at each side of louver:
• Fully grouted CMU pilaster region (~16″ wide built-out zone) at each jamb of the louver.
• (1) #5 vertical full height through that pilaster, develop into supporting CMU below and into lintel above.
• Ties / hoops @ ~8″ o.c. only if required by office standard; tributary OOP height is short (6.5 ft), so this is not the same demand as a 12 ft OH door at slab.

Controlling demand:
• Arching case: $M_u \approx 3.07 \text{ k-ft}$, $V_u \approx 1.51 \text{ k}$.
• No-arch OOP case (33 psf over 6.5 ft): $M_u^{(\text{lat})} \approx 1.76 \text{ k-ft}$, $V_u^{(\text{lat})} \approx 0.87 \text{ k}$.
→ Arching / gravity wedge governs.

5. Summary of Demands (Factored)

Case A — Arching Active (Gravity Wedge)	
Arching height h_a	3.81 ft (≤ 6t)
Line load w_u	0.374 k/ft (≈312 plf ×1.2)
Shear V_u	1.51 k
Moment M_u	3.07 k-ft
Lintel trial	16″ CMU lintel (Type A) w/ (2) #5 horiz
Jamb trial	CMU pilaster/jamb w/ (1) #5 vert

Case B — No Arching (OOP Strip Above Opening, 33 psf)	
Panel height used	6.5 ft above lintel (20.0 ft → 26.5 ft)
OOP line load w_{lat}	0.2145 k/ft (≈214.5 plf from 33 psf × 6.5 ft)
Shear $V_u^{(\text{lat})}$	0.87 k
Moment $M_u^{(\text{lat})}$	1.76 k-ft
Controlling design?	Arching gravity still governs (3.07 k-ft > 1.76 k-ft)

6. Masonry Lintel / Jamb Schedule (For Sheets)

6A. Lintel / Pilaster (Arching Active – Gravity Wedge Controls)

MARK	DEPTH	TYPE	LINTEL REINFORCING — HORIZONTAL	STIRRUPS / TIES	SUPPORT COLUMN	COMMENTS
16L A Lintel	16″	Type A	(2) #5 horiz cont.	—	16L A C	Arching assumed active; governs ~3.07 k-ft, V≈1.51 k

MARK	SIZE	TYPE	COLUMN REINFORCING — VERTICAL	TIES	COMMENTS
16L A C	8×16	Type A2	(1) #5 full height	—	Short pilaster above elevated opening; lap bar into lintel

6B. Lintel / Pilaster (No-Arch / OOP Controls – 33 psf Envelope)

MARK	DEPTH	TYPE	LINTEL REINFORCING — HORIZONTAL	STIRRUPS / TIES	SUPPORT COLUMN	COMMENTS
16L B Lintel	16″	Type A	(2) #5 horiz cont.	—	16L B C	No arching assumed; OOP strip 6.5 ft tall, ~1.76 k-ft, shear ~0.87 k (33 psf)

MARK	SIZE	TYPE	COLUMN REINFORCING — VERTICAL	TIES	COMMENTS
16L B C	8×16	Type A2	(1) #5 full height	#3 @ 8″ o.c. if required	Pilaster resists local OOP from 6.5 ft tributary height; not full-height wall demand

1. Inputs / Geometry / Assumptions

Opening / Wall	Personnel Door @ South Wall
Total wall height to roof deck, H_{wall}	26.5 ft
Bottom of opening above slab/base, H_{bot}	0.0 ft
Clear opening height, H_{open}	7′-4″ ≈ 7.33 ft
Clear opening width (span), L	3′-4″ ≈ 3.33 ft
Lintel bearing each side, b_{bear}	8 in
CMU nominal thickness, t_{nom}	8 in
CMU actual thickness, t	7.625 in = 0.635 ft
Wall self-weight, w_{wall}	164 psf (fully grouted)
Specified masonry strength, f'_m	2000 psi
Rebar yield, F_y	60 ksi
Lateral design pressure (wind/seismic OOP), p_{lat}	33 psf (wind 25 psf, seismic 33 psf → envelope = 33 psf)
Grouting	Fully grouted lintel & jambs

Two cases checked:
- Case A: Arching active → gravity wedge of CMU above lintel only.
- Case B: No arching → lintel spans full clear width under out-of-plane (OOP) lateral pressure from wind/seismic acting over full wall height.

2. Case A — Arching (Gravity Wedge)

2.1 Available height above lintel

Available masonry height above top of opening up to next stiff element:

$$h_{\text{avail}} = H_{\text{wall}} - (H_{\text{bot}} + H_{\text{open}})$$

$$h_{\text{avail}} = 26.5 - (0.0 + 7.33) = 19.17 \text{ ft}$$

Arching limit per 6t:

$$t = 7.625 \text{ in} = 0.635 \text{ ft}$$

$$6t = 6 \times 0.635 = 3.81 \text{ ft}$$

$$h_a = \min(6t,\ h_{\text{avail}}) = \min(3.81,\ 19.17) = \boxed{3.81 \text{ ft}}$$

do you need a reference for this spec(and other specs)?

2.2 Triangular self-weight → uniform line load

Wedge self-weight (service line load on lintel):

$$w_D = \tfrac{1}{2}\, w_{\text{wall}}\, h_a$$

$$w_D = 0.5 \times 164 \times 3.81 \approx 312 \text{ plf} \approx 0.312 \text{ k/ft}$$

Factored gravity (use $\gamma_D = 1.2$):

$$w_u = 1.2 \times 0.312 = 0.374 \text{ k/ft}$$

2.3 Shear / moment in lintel (arching case)

Treat lintel as simply supported span $L = 3.33 \text{ ft}$, uniform $w_u = 0.374 \text{ k/ft}$. Factored shear:

$$V_u = \frac{w_u L}{2} = \frac{0.374 \cdot 3.33}{2} \approx 0.62 \text{ k}$$

Factored moment:

$$M_u = \frac{w_u L^2}{8} = \frac{0.374 \cdot (3.33)^2}{8} \approx 0.52 \text{ k-ft}$$

Bearing per jamb:

$$R_u = \frac{w_u L}{2} \approx 0.62 \text{ k}$$

With 8 in bearing and $f'_m = 2000 \text{ psi}$, bearing stress is acceptable.

3. Case B — No Arching (Full OOP Lateral)

3.1 OOP panel load

Worst case: arching prevented. Lintel carries full clear width under out-of-plane wind/seismic from full wall height. Take full wall height:

$$h_{\text{panel}} = H_{\text{wall}} = 26.5 \text{ ft}$$

Use lateral envelope pressure (wind 25 psf vs seismic 33 psf → control 33 psf):

$$p_{\text{lat}} = 33 \text{ psf}$$

Line load on lintel, per ft of lintel span:

$$w_{\text{lat}} = p_{\text{lat}} \cdot h_{\text{panel}} = 33 \times 26.5 = 874.5 \text{ plf} \approx 0.8745 \text{ k/ft}$$

We'll treat $w_{\text{lat}} = 0.8745 \text{ k/ft}$ as already factored envelope.

3.2 Shear / moment in lintel (no-arching case)

Use $w_{\text{lat}} = 0.8745\ \text{k/ft}$, span $L = 3.33\ \text{ft}$. Shear:

$$V_u^{(\text{lat})} = \frac{w_{\text{lat}} L}{2} = \frac{0.8745 \cdot 3.33}{2} \approx 1.46\ \text{k}$$

Moment:

$$M_u^{(\text{lat})} = \frac{w_{\text{lat}} L^2}{8} = \frac{0.8745 \cdot (3.33)^2}{8} \approx 1.21\ \text{k-ft}$$

This ~1.21 k-ft controls vs 0.52 k-ft from arching.

3.3 Jamb column quick check

Each jamb acts like a vertical CMU pier, 8"x16", fully grouted. Take tributary half-width $\approx L/2 = 1.67\ \text{ft}$. OOP line load on one jamb from lateral using 33 psf:

$$w_{\text{jamb}} = p_{\text{lat}} \cdot 1.67 \approx 33 \times 1.67 \approx 55\ \text{plf}$$

Cantilever height ~ full wall height 26.5 ft. With a single #5 vertical bar full height, hooked/anchored into footing and lapped into the lintel, the jamb is acceptable for this small door. (We're not detailing ties here because this is a short, narrow pier with low demand.)

4. Required Reinforcement / Recommended Detail

Lintel (design for Case B OOP):
• 8"x16" CMU lintel (Type A).
• (2) #5 horiz bars continuous near faces.
• Stirrups/closed ties not strictly required by the low shear (~1.46 k), include only if office standard.

Jamb columns:
• Each jamb = 8"x16" fully grouted CMU pier.
• (1) #5 vertical bar full height in each jamb, develop into footing and lap into lintel bond beam.

Controlling demand:
• Arching case moment $M_u \approx 0.52\ \text{k-ft}$.
• No-arch OOP case moment $M_u^{(\text{lat})} \approx 1.21\ \text{k-ft}$.
→ Design for 1.21 k-ft.

5. Summary of Demands (Factored)

Case A — Arching Active (Gravity Wedge)	
Arching height h_a	3.81 ft
Line load w_u	0.374 k/ft (≈312 plf ×1.2)
Shear V_u	0.62 k
Moment M_u	0.52 k-ft
Lintel trial	8″×16″ CMU w/ (2) #5 horiz
Jamb trial	8″×16″ jamb w/ (1) #5 vert

Case B — No Arching (Full OOP Lateral)	
Panel height used	26.5 ft
OOP line load w_{lat}	0.8745 k/ft (≈874.5 plf from 33 psf × 26.5 ft)
Shear $V_u^{(\text{lat})}$	1.46 k
Moment $M_u^{(\text{lat})}$	1.21 k-ft
Controlling design?	**Envelope governs — design for OOP case**

6. Masonry Lintel / Jamb Schedule (For Sheets)

16A Lintel / Jamb (Arching Active – Gravity Wedge Controls)

MARK	DEPTH	TYPE	LINTEL REINFORCING — HORIZONTAL	STIRRUPS / TIES	SUPPORT COLUMN	COMMENTS
16A Lintel	16″	Type A	(2) #5 horiz	—	16A C	Arching assumed active; governs gravity (~0.52 k-ft, V≈0.62 k)

MARK	SIZE	TYPE	COLUMN REINFORCING — VERTICAL	TIES	COMMENTS
16A C	8×16	Type A2	(1) #5 full height	—	Lap bar into lintel; arching/gravity case

16B Lintel / Jamb (No-Arch / OOP Controls – 33 psf Envelope)

MARK	DEPTH	TYPE	LINTEL REINFORCING — HORIZONTAL	STIRRUPS / TIES	SUPPORT COLUMN	COMMENTS
16B Lintel	16″	Type A	(2) #5 horiz	—	16B C	No arching assumed; design for OOP ~1.21 k-ft, shear ~1.46 k (33 psf)

MARK	SIZE	TYPE	COLUMN REINFORCING — VERTICAL	TIES	COMMENTS
16B C	8×16	Type A2	(1) #5 full height	#3 @ 8″ o.c.	Lap bar into lintel; design for OOP (33 psf envelope)

C8 — ROOF JOIST LEDGER & ANCHORAGE TO CMU

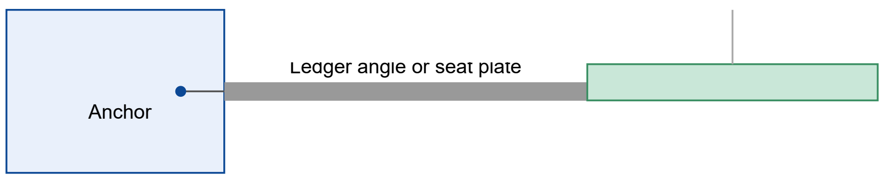
Michael Baker INTERNATIONAL

Project: Creech DRP — Shop C (Area E)   **Discipline:** Criteria — Connections to CMU   **Sheet:** C8

Basis: Roof joists @ 7′-0″ o.c.; no parapet; CMU 8″ fully grouted

1) Ledger / Seat Configuration

- Roof joists bear at CMU. Provide continuous **ledger angle** or **discrete seat plates** at joist lines.
- Anchorage into grouted CMU cells with plate washers; minimum edge distances per TMS/ACI 530 and manufacturer.
- Provide bond beam at roof line to collect diaphragm loads and distribute anchor forces.



2) Uplift Per Anchor (Service → Strength)

Net wind uplift at roof (corner) per Section C: $p_{net} \approx -4.4\ \text{psf}$. Use conservative tributary to each anchor:

- Joist spacing $s = 7.0'$
- Anchor spacing along wall $a = 4.0'$ (typical)
- Tributary area $A_{trib} = s \times a = 28$ sf

Service tension per anchor:

$$T_s = |p_{net}| \times A_{trib} = 4.4 \times 28 = \mathbf{123}\ \text{lb}$$

Strength design (LRFD uplift):

$$T_u \approx 1.6 \times T_s = \mathbf{197}\ \text{lb} \quad (\text{wind})$$

If PM directs C&C corner uplift basis, you may alternatively adopt $|p| = 25\ \text{psf} \Rightarrow T_u \approx 1.6 \times (25 \times 28) = \mathbf{1.12\ k}$. Use the governing of the two.

3) Anchor Selection (Steel & Masonry Check)

Item	Assumption / Spec	Check	Result
Anchor size	½″ diameter (ASTM F1554 Gr.36 or epoxy anchor)	$\phi N_n \geq T_u$	OK for $T_u = 0.20$ k; verify for $T_u = 1.12$ k case
Embedment	$h_{ef} \geq 8''$ in grouted cell	Bond/breakout per ACI/TMS	Meets typical minimums; calc on submittal
Plate washer	Std. plate washer ≥ 3″×3″×¼″	Masonry bearing / local	OK
Spacing	4′-0″ o.c. along wall (typ.)	Tributary 28 sf/anchor	Basis for T_s, T_u

4) In-Plane Shear at Ledger

Ledger fasteners must also transfer diaphragm shear into the CMU chord/collector. Size per L-pages (use larger of wind X/Y or seismic X/Y for that line). Masonry bearing at plate and fastener shear to be verified on shop submittals.

5) Detailing Requirements

- Provide bond beam at roof line (continuous) with horizontal bars as required; tie into vertical bars in CMU cells at anchors.
- Grout solid at each anchor location and for 8″ beyond; maintain clear cover per code.
- Use hot-dip galvanized anchors/plates in exterior exposure zones; seal penetrations per spec.
- At corner/edge zones, increase anchor density if required by C&C pressures; maintain minimum edge distances.

Seat Plate Option

- Seat plate (e.g., PL ⅜″ × 8″ × 10″ min at joist) welded/bolted to ledger or anchored direct.
- Provide stiffener if seat projection > 4″.

Ledger Angle Option

- L6×4×½ continuous; anchors @ 4′–0″ o.c.
- Check angle leg bending and bolt shear/tearout on submittal values.

6) Drawing Note

- Provide ledger angle or seat plates at CMU; anchors ½″ dia. min into grouted cells with plate washers.
- Anchor spacing: 4′-0″ o.c. typ.; design for $T_u = \max(0.20\ \mathrm{k}, 1.12\ \mathrm{k})$ per project C&C directive.
- Provide bond beam at roof line; grout solid at anchors; corrosion protection per spec.
- Verify diaphragm shear fastener pattern at ledger per L-pages collector demand.

this line could be move a little down to give space to the letters; aesthetic opinion

Michael Baker INTERNATIONAL

Project: Creech DRP – Shop C (Area E) **Date:** Oct 08 2025 **Org:** Michael Baker International

1) Design Basis

- **Codes:** ASCE 7-22 (loads & combos), AISC 360-16 (steel), ACI 318-19 (concrete), TMS 402/602-16 (masonry).
- **Deflection limits:** Roof $L/240$ total, $L/360$ live; Floor $L/240$ total, $L/360$ live unless noted.
- **Service vs Strength:** Service loads for deflection/vibration; strength per LRFD/ASD (see C24).

2) Roof Loads (Service)

Inputs: $P_g = 5.0\text{ psf},\ C_e = 1.0,\ C_t = 1.0,\ I_s = 1.10$ (Risk Cat III) Δh = 15 ft Joist spacing $s = 7\text{ft}$

1) Balanced Roof Snow

$$p_f = 0.7\,C_e\,C_t\,I_s\,P_g = 0.7(1.0)(1.0)(1.10)(5.0) = 3.85\text{ psf} \approx \mathbf{3.9\ psf}$$

Per-joist line load $w_{p_f} = p_f\,s = 3.85 \times 7.0 = 26.95\text{ plf} \approx 27.0\text{ plf.}$

2) Snow Drift on Lower Roof

$$h_d = 0.43\,P_g^{0.35}(\Delta h)^{0.75} = 0.43(5)^{0.35}(15)^{0.75} = \mathbf{5.76}\text{ ft}$$

$$p_{d,\text{unc}} = 20(5.76) = 115\text{ psf}$$

$$p_{\text{total,max}} = 2.5p_f + 20 = 2.5(3.85) + 20 = 29.6\text{ psf}$$

$$p_d = p_{\text{total,max}} - p_f = 29.6 - 3.85 = \mathbf{25.8}\text{ psf (use 26.0 psf)}$$

$$p_f + p_d = 3.9 + 26.0 = \mathbf{29.9}\text{ psf } (\approx 30\text{ psf})$$

$$x = 4h_d = 4(5.76) = \mathbf{23.0}\text{ ft}$$

Triangular drift from 26.0 psf → 0 over 23 ft.

Joist Line Loads (7′ o.c.)

$$w_{\text{peak}} = p_d s = 26.0 \times 7 = \mathbf{182}\text{ plf}$$

$$w_{\text{seg,eq}} = \tfrac{1}{2}p_d s = 0.5(26.0)(7) = \mathbf{91}\text{ plf}$$

Line Loads to CMU / Beams

For tributary width $b_t = 26\text{ ft}$:

$$w_{DL} = 30(26) = 780\text{ plf}$$
$$w_{RL} = 20(26) = 520\text{ plf}$$
$$w_{p_f} = 3.9(26) = 101\text{ plf}$$

Drift reaction to CMU: $\tfrac{1}{2}p_d x = 0.5(26.0)(23.0) = \mathbf{299}\text{ plf.}$

Snow Drift Load Key (Δh = 15 ft)
$p_f = 3.9\text{ psf}$ $p_d = 26.0\text{ psf}$ Total = 29.9 psf x = 23.0 ft
Joist peak 182 plf Uniform eq 91 plf

3) Mezzanine Loads (Service)

DL = 78 psf LL = 125 psf Partitions = 20 psf For half-span $b_t = 12.0\text{ft}$:

$$w_{DL} = 78(12) = 936 \text{ plf}, \quad w_{LL} = 125(12) = 1500 \text{ plf}, \quad w_{\text{part}} = 20(12) = 240 \text{ plf}$$

4) Design Patterning

- Roof → check $DL + LL_r$ and $DL + P_{\text{drift}}$.
- Mezz beams → pattern LL; add partition if not enveloped.
- Columns / CMU → use governing line loads each level (+ equipment when available).

5) References

- ASCE 7-22 Ch 4 (live), Ch 7 (snow), Ch 30 (C&C), Ch 2 (load combos).
- Deck/joist design in G-sections uses these service loads.

Project: Creech DRP – Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1) Givens

- Standard: **ASCE 7-22** (ELF procedure).
- Risk Category III → Importance factor $I_e = \mathbf{1.25}$.
- Site Class D (per BOD/geotechnical).
- Mapped spectra: $S_s = \mathbf{0.724}$, $S_1 = \mathbf{0.226}$.
- Design spectra: $S_{DS} = \mathbf{0.589}$, $S_{D1} = \mathbf{0.324}$.
- SFRS: Steel Concentrically Braced Frames (SCBF): $R = \mathbf{6.0}$, $\Omega_0 = \mathbf{2.0}$, $C_d = \mathbf{5.0}$ (ASCE 7-22 Table 12.2-1).
- Heights: Mezz $h_1 = \mathbf{12}$ ft, Roof $h_2 = \mathbf{24}$ ft.
- Seismic weights: Roof $w_2 = \mathbf{35.9}$ k; Mezz $w_1 = \mathbf{130.8}$ k (includes 25% LL); Total $W = \mathbf{166.7}$ k.

Refs: ASCE 7-22 §11–12 (seismic), §12.8 (ELF), §11.4–11.6 (site coeffs/spectra).

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2) Design Spectrum (context)

Variables: $S_{MS} = F_a S_s,\ S_{M1} = F_v S_1,\ S_{DS} = \frac{2}{3} S_{MS},\ S_{D1} = \frac{2}{3} S_{M1}$.

Project use: Adopt $S_{DS} = \mathbf{0.589}$, $S_{D1} = \mathbf{0.324}$ per criteria setup.

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3) ELF Base Shear

Variables: $C_s = \dfrac{S_{DS} I_e}{R}$, with minimum $C_s \geq \max\big(0.044 S_{DS} I_e,\ 0.01 I_e\big)$; base shear $V = C_s W$.

Numbers: $C_s = \dfrac{\mathbf{0.589 \cdot 1.25}}{\mathbf{6.0}} = \mathbf{0.123}$. Minimums: $0.044 S_{DS} I_e = \mathbf{0.032}$, $0.01 I_e = \mathbf{0.0125}$ → OK. Thus $V = \mathbf{0.123} \cdot \mathbf{166.7}\ \mathrm{k} = \mathbf{20.5}\ \mathrm{k}$.

Refs: ASCE 7-22 §12.8.1.1 (response coeff.), §12.8.1.1.1 (minimum base shear).

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4) Fundamental Period & Upper-Bound Check

Variables: Approx. period $T_a = C_t h_n^x$ (steel/braced low-rise: $C_t \approx 0.02,\ x = 0.75$). Upper bound for short-period range: $C_s \leq \dfrac{S_{D1}}{T R / I_e}$.

Numbers: $T_a = 0.02 \cdot (\mathbf{24})^{0.75} \approx \mathbf{0.22}$ s. Then $\dfrac{S_{D1}}{T R / I_e} = \dfrac{\mathbf{0.324}}{\mathbf{0.22 \cdot 6.0 / 1.25}} = \mathbf{0.307}$. Compare $C_s = \mathbf{0.123} \leq \mathbf{0.307} \rightarrow$ OK.

Refs: ASCE 7-22 §12.8.2.1 (approx. period), §12.8.1.1 (upper-bound using S_{D1}).

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5) Vertical Distribution of Lateral Forces (Two Levels)

Variables: With $k = 1.0$ since $T \le 0.5$ s: $C_{vx} = \dfrac{w_x h_x^k}{\sum(wh^k)}, \quad F_x = C_{vx} V.$

Numbers (sum): $\sum(wh) = (\mathbf{130.8})(\mathbf{12}) + (\mathbf{35.9})(\mathbf{24}) = \mathbf{2431.2}\ \mathrm{k \cdot ft}.$

Roof (level 2): $C_{v2} = \dfrac{35.9 \cdot 24}{2431.2} = \mathbf{0.354} \Rightarrow F_2 = 0.354 \cdot \mathbf{20.5} = \mathbf{7.3}\ \mathrm{k}.$

Mezz (level 1): $C_{v1} = \dfrac{130.8 \cdot 12}{2431.2} = \mathbf{0.646} \Rightarrow F_1 = 0.646 \cdot \mathbf{20.5} = \mathbf{13.2}\ \mathrm{k}.$

Check: $F_1 + F_2 \approx \mathbf{20.5}\ \mathrm{k}(= V).$

Refs: ASCE 7-22 §12.8.3 (vertical distribution), §12.8.4 (application to diaphragms/collectors/chords).

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6) Additional ELF Requirements & Design Notes

- **Accidental torsion:** include $\pm 5\%$ mass eccentricity at each level (ASCE 7-22 §12.8.4.2).
- **Redundancy factor** ρ**:** take $\rho = 1.0$ unless irregularities trigger $\rho > 1$ (§12.3.4).
- **Collectors / chords / drags:** design diaphragm forces; use Ω_0 where required (§12.10, §12.3.3.3).
- **Overstrength actions (where required):** $V_\Omega = \Omega_0 V = \mathbf{2.0} \times \mathbf{20.5} = \mathbf{41.0}\ \mathrm{k}.$
- **Story drift:** $\Delta = \dfrac{C_d\, \Delta_e}{I_e}$ (compare to §12.12 limits).
- **Vertical seismic effects:** typically not governing unless long spans/equipment require §12.4.2.2 checks.

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7) Summary Values for Downstream Use

Base shear $\boxed{V = \mathbf{20.5}\ \mathrm{k}}$; Level forces $\boxed{F_1 = \mathbf{13.2}\ \mathrm{k}}$, $\boxed{F_2 = \mathbf{7.3}\ \mathrm{k}}$; Overstrength $\boxed{V_\Omega = \mathbf{41.0}\ \mathrm{k}}$; Exponent $\boxed{k = \mathbf{1.0}}$; Period $\boxed{T_a \approx \mathbf{0.22}\ \mathrm{s}}$.

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8) Citations

- ASCE 7-22 §11–12 — Seismic definitions, site parameters, ELF procedure.
- ASCE 7-22 Table 12.2-1 — $R,\ \Omega_0,\ C_d$ for SCBF.
- ASCE 7-22 §12.8 — C_s, period, vertical distribution.
- ASCE 7-22 §12.10 — Diaphragms, chords, collectors.
- ASCE 7-22 §12.12 — Drift limits.

C13 — BUILDING ENCLOSURE CLASSIFICATION (ASCE 7-22 §26.2)

Michael Baker INTERNATIONAL

Project: Creech DRP – Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1) Purpose

Determine enclosure classification (*Enclosed* vs *Partially Enclosed*) per ASCE 7-22 §26.2, using opening takeoffs from SF-122. Result controls internal pressure coefficient GC_{pi} used throughout C14–C18.

2) Givens & Assumptions (from SF-122 and plan set)

- Mean roof height $h \approx 30$ ft (overall), wall design height used for areas $= \mathbf{23\ ft}$ (rectangular portion only).
- Plan dimensions used for gross wall areas: East wall length $\mathbf{120\ ft}$; North/South similar; West is attached to adjacent building (treat as no effective openings to exterior).
- **Windward wall for classification:** choose the **East** wall (most exposed and with the dominant openings).
- Gross wall area (rectangular portion) per face: $A_g = 120\text{ ft} \times 23\text{ ft} = \mathbf{2{,}760}$ sf.
- East wall openings (doors/frames per SF-122): sum to $A_o = \mathbf{480}$ sf (takeoff basis).
- Other-envelope openings (excluding East), i.e., North + South + West + roof:
  - North: one door $7'4'' \times 3'2'' \Rightarrow \mathbf{23.3\ sf}$.
  - South: personnel/overhead door set per SF-122; assumed sum $\mathbf{142\ sf}$
  - West: adjacent building (treat openings to exterior as $\mathbf{0\ sf}$).
  - Roof: none credited as "openings".

  Thus $A_{oi} = \mathbf{165.3\ sf}$ (use 165 sf nominal).
- Gross area of remaining envelope surfaces excluding East: A_{gi} includes North, South, West walls and roof. This is large compared to A_{oi}; see check below.

3) ASCE 7-22 §26.2 — Tests for *Partially Enclosed* Building

Let:

$$\text{Windward wall openings } (A_o) = \mathbf{480}\text{ sf}, \quad \text{Windward wall gross } (A_g) = \mathbf{2{,}760}\text{ sf}, \quad \text{Other openings } (A_{oi}) = \mathbf{165}\text{ sf}, \quad \text{Other gross } (A_{gi}) = (\text{sum of envelope excluding } A_g).$$

Condition	Expression	Evaluation	Status
1) Windward openings predominate	$A_o \geq 1.10\, A_{oi}$	$480 \geq 1.10 \times 165 = 181.5 \Rightarrow \mathbf{480} \geq \mathbf{181.5}$	Pass
2) Windward opening is significant	$A_o > \min(4\text{ sf},\ 0.01A_g)$	$0.01A_g = 0.01 \times 2{,}760 = 27.6\text{ sf};\ \min = 4\text{ sf} \Rightarrow 480 > 4$	Pass
3) Other surfaces not too open	$A_{oi}/A_{gi} \leq 0.20$	$A_{oi} = 165$ sf; A_{gi} (North + South + West walls + roof) $\gg 825$ sf ⇒ ratio $\ll 0.20$	Pass

Conclusion: All three §26.2 criteria are satisfied for the **East** wall as the windward wall with predominant openings. Therefore, the building is classified as **Partially Enclosed** → use internal pressure coefficient

$$\boxed{GC_{pi} = \pm 0.55}$$

in MWFRS (C15–C16) and C&C (C17–C18) design.

Safety Margin / Sensitivity

- Threshold for Condition (1): $A_o \geq 1.10A_{oi}$. With $A_{oi} = 165$ sf, the minimum A_o is 181.5 sf. We currently have $A_o = 480$ sf → margin ≈ **+298.5 sf**.
- Condition (2): $A_o > 4$ sf (or 27.6 sf); we exceed this by a wide margin.
- Condition (3): ratio of other openings to other gross area remains well below 0.20.

4) Numbers Used (Trace Table)

Face	Gross area used (sf)	Opening area used (sf)	Notes
East (windward in test)	2,760	**480**	Sum of OH/personnel doors per SF-122 takeoff.
North	2,760	23.3	Single door $7'4'' \times 3'2''$.
South	2,760	142	Personnel/overhead doors per SF-122 (update if schedule changes).
West	2,760	0	Attached to adjacent building → no exterior openings credited.
Roof	—	0	No roof openings considered for enclosure calc.

5) Quick Recap

Internal Pressure: Per C13 classification, use $\boxed{GC_{pi} = \pm 0.55}$ for all wind design (MWFRS & C&C) unless this enclosure calc is revised.

Appendix: Schematic (not to scale)

```
East (Windward)                    North
+------------------------------+    +------------------------------+
|            A_g = 2,760 sf    |    |            A_g = 2,760 sf    |
|   Openings A_o ≈ 480 sf      |    |   Opening A ≈ 23 sf          |
+------------------------------+    +------------------------------+

South                               West (adjacent bldg.)
+------------------------------+    +------------------------------+
|            A_g = 2,760 sf    |    |            A_g = 2,760 sf    |
|   Openings A ≈ 142 sf        |    |   Openings ≈ 0 sf            |
+------------------------------+    +------------------------------+
```

6) Citations

- ASCE 7-22 §26.2 — Enclosure classification definitions and criteria.
- ASCE 7-22 §26.7 — Internal pressure coefficients tied to enclosure class.

Project: Creech DRP – Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1) Inputs (ASCE 7-22)

Basic wind (ultimate): $V = 105\ \mathrm{mph}$ (3-sec gust) | **Risk Cat:** III | **Exposure:** C
Building height: $h \approx 24\ \mathrm{ft}$ | **Directionality:** $K_d = 0.85$ | **Topo:** $K_{zt} = 1.0$ (flat)
Velocity pressure coeff.: $K_z \approx 0.85$ @ roof height | **Gust factor (rigid):** $G = 0.85$
Enclosure: Partially Enclosed → $GC_{pi} = \pm 0.55$ (use both signs for MWFRS/C&C).

2) Velocity Pressure (roof height)

ASCE 7-22 §26.10.1: $q_z = 0.00256\, K_z\, K_{zt}\, K_d\, V^2\ \ [\mathrm{psf}]$

$q_h = 0.00256(0.85)(1.0)(0.85)(105)^2 = \mathbf{20.5}\ \mathrm{psf}$ → use q_h for MWFRS at roof height.

For C&C, use q_z at component effective height; tables in Ch. 30 give (GC_p) by zone & effective area.

3) MWFRS Formulas

Walls/Roof: $p = q\, G\, C_p\ -\ q_i\, (GC_{pi})$, with $q_i = q_h$ for enclosed/partially enclosed low-rise.

4) Working Assumptions

- Low/near-flat roof; monoslope geometry.
- No topographic speed-up: $K_{zt} = 1.0$.
- Ch. 27 for MWFRS (global frames/chords); Ch. 30 for C&C (fasteners/panels).

5) Quick MWFRS Check (GC_{pi}=±0.55)

Figure 27.3-1 coefficients (typ.): windward $C_p = +0.80$, leeward $C_p = -0.50$, side $C_p = -0.70$, roof interior $C_p = -0.90$.

Compute $q_h G = 20.5 \times 0.85 = \mathbf{17.425}\ \mathrm{psf}$; internal term $q_h(GC_{pi}) = 20.5(\pm 0.55) = \pm\mathbf{11.28}\ \mathrm{psf}$.

Surface	Equation	$GC_{pi} = +0.55$	$GC_{pi} = -0.55$	Design
Windward wall	$p = (17.425)(+0.80) \mp 11.28$	$+2.67\ \mathrm{psf}$ (in)	**+25.22 psf** (in)	**+25 psf in**
Leeward wall	$p = (17.425)(-0.50) \mp 11.28$	**-19.99 psf**	+2.56 psf	**-20 psf**
Side wall	$p = (17.425)(-0.70) \mp 11.28$	**-23.48 psf**	-0.92 psf	**-23.5 psf**
Roof (Zone 1)	$p = (17.425)(-0.90) \mp 11.28$	**-26.96 psf**	-4.41 psf	**-27 psf**

Roof edges/corners (Zones 2/3) have more negative C_p; take the governing suction from Ch. 27 for MWFRS and from Ch. 30 for C&C by effective area.

6) References

- ASCE 7-22 §26 (exposure, internal pressure, gust, topo).
- §26.10.1 (velocity pressure q_z); §26.11 (G); §26.13 (GC_{pi}).
- Ch. 27 (MWFRS coefficients); Ch. 30 (C&C zones/effective area).

Project: Creech DRP – Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1) Variables (linear list)

$V_{ult} = 105\ \mathrm{mph}$, Exposure C, mean roof height $h \approx 24\ \mathrm{ft}$.
$K_z(h) \approx 0.85$, $K_{zt} = 1.0$, $K_d = 0.85$, $G = 0.85$.
Partially enclosed ⇒ $GC_{pi} = \pm\mathbf{0.55}$.
Velocity pressure at roof height: $q_h = 0.00256\,K_z K_{zt} K_d V^2 \approx \mathbf{20.5\ psf}$.

2) Velocity Pressure (all heights)

$q_z(z) = 0.00256\,K_z(z)\,K_{zt}\,K_d\,V^2$. For quick MWFRS screening at low-rise height, using q_h is conservative.

3) Pressures (directional method)

MWFRS: $p(z) = q_z(z)\,G\,C_p\ -\ q_h\,(GC_{pi})$. Select C_p per Ch. 27 figures; integrate over projected areas.

4) Quick Numbers at Roof Height (updated for $GC_{pi} = \pm 0.55$)

Surface	C_p	Result (psf, worst of ±GC_{pi})
Windward wall	+0.80	**+25.2** inward (also +2.7 case)
Leeward wall	-0.50	**-20.0** suction (or +2.6)
Side wall	-0.70	**-23.5** suction (or -0.9)
Roof interior	-0.90	**-27.0** suction (or -4.4)

6) References

- ASCE 7-22 §26, §27 (Directional method; all heights).

Project: Creech DRP – Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1) Variables

Low-rise method (Fig. 27.3-1). $V_{ult} = 105\ \mathrm{mph}$; Exposure C; $h \approx 24\ \mathrm{ft}$.
$q_h \approx \mathbf{20.5\ psf}$, $G = 0.85$, $K_{zt} = 1.0$, $K_d = 0.85$.
Partially enclosed ⇒ $GC_{pi} = \pm\mathbf{0.55}$.

2) Pressures at Roof Height (zone-based C_p)

Surface / Zone	C_p	Formula	Result (psf)
Windward wall	+0.80	$p = q_h G C_p \mp q_h(0.55)$	**+25.2** inward (alt. +2.7)
Leeward wall	-0.50	same	**-20.0** suction (alt. +2.6)
Side wall	-0.70	same	**-23.5** suction (alt. -0.9)
Roof interior (Zone 1)	-0.90	same	**-27.0** suction (alt. -4.4)
Roof edge/corner (Zones 2–3)	per Fig. 27.3-1	same	Use tabulated C_p (more negative)

3) Resultants

Base shear $V = \sum pA$. Overturning $M = \sum pA \cdot z$. Distribute via diaphragm; design frames/CMU, chords, collectors for governing wind combos.

4) Notes

- Pick C_p from Fig. 27.3-1 using actual aspect ratio H/L and zones. where is fig. 27.3-1?
- Doors/windows are designed as C&C (C17a/b), not with MWFRS C_p.

Project: Creech DRP – Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1) Inputs

ASCE 7-22 C&C, **Partially Enclosed** ($GC_{pi} = \pm 0.55$), $V_{ult} = 105 \text{ mph}$, Exposure C, $h \approx 24 \text{ ft}$, $K_{zt} = 1.0$.

2) Wind Load Map Key — Walls (Ultimate)

COMPONENTS & CLADDING — WALLS				
MARK	USAGE	10 ft²	100 ft²	200 ft²
⑤	INTERIOR WALL ZONE (ZONE 4)	**-34.1 PSF** *(+32.0 PSF)*	**-30.5 PSF** *(+28.4 PSF)*	**-29.4 PSF** *(+27.3 PSF)*
⑥	EXTERIOR WALL EDGE (ZONE 5)	**-40.2 PSF** *(+32.0 PSF)*	**-33.0 PSF** *(+28.4 PSF)*	**-30.8 PSF** *(+27.3 PSF)*

Negative = suction (outward). Positive (muted) = inward pressure. Use zone extents per Ch.30 wall figures; effective area $A_{eff} =$ fastener tributary area.

3) Notes

- Girt/CMU MWFRS checks remain in C15–C16. Local panel/fastener design uses these C&C pressures.

Project: Creech DRP – Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1) Inputs

ASCE 7-22 C&C, **Partially Enclosed** ($GC_{pi} = \pm 0.55$), $V_{ult} = 105\ \mathrm{mph}$, Exposure C, $h \approx 24\ \mathrm{ft}$, $K_{zt} = 1.0$.

2) Wind Load Map Key — Roof (Ultimate)

COMPONENTS & CLADDING — ROOF				
MARK	**USAGE**	**50 ft²**	**100 ft²**	**500 ft²**
①	**INTERIOR ROOF ZONE (ZONE 1)**	**-43.6 PSF** *(+23.6 PSF)*	**-40.8 PSF** *(+23.6 PSF)*	**-34.3 PSF** *(+23.6 PSF)*
②	**ROOF END / CORNER (ZONES 2 & 3)**	**-67.3 PSF** *(+23.6 PSF)*	**-60.1 PSF** *(+23.6 PSF)*	**-43.3 PSF** *(+23.6 PSF)*

Negative = suction (outward). Positive (muted) = inward pressure. Zones/widths per Ch.30 roof figures (monoslope). Use effective area A_{eff} of fastener/panel.

3) Converting to Line Load on Joists (if needed)

Uniform panel pressure p over tributary width $b_t \rightarrow$ joist line load $w = p\, b_t$. Check both suction and inward cases; apply with strength combinations.

Project: Creech DRP – Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1) Givens

Standard	ASCE 7-22 Ch. 7 (Service Snow)
Risk Category	III → $I_s = 1.10$
Thermal Condition	Heated building → $C_t = 1.00$
Exposure	Exposure C → $C_e = 1.00$ (low height)
Ground Snow	$p_g = 5.0$ psf (basis of design)
Roof Step Height	$\Delta h = 15$ ft
Snow Density	$\gamma = 20$ pcf (ASCE default)
Joist Spacing	$s = 7.0$ ft

2) Balanced Roof Snow (Service)

ASCE 7-22 §7.3 (low-slope form):

$$p_f = 0.7\, C_e\, C_t\, I_s\, p_g = 0.7(1.00)(1.00)(1.10)(5.0) = \mathbf{3.85}\ \text{psf} \approx \mathbf{3.9}\ \text{psf}.$$

Per-joist uniform line load:

$$w_{p_f} = p_f\, s = 3.85 \times 7.0 = \mathbf{26.95}\ \text{plf}\ (\approx 27.0\ \text{plf}).$$

3) Snow Drift at Roof Step (Service)

Case: lower roof adjacent to higher roof (ASCE 7-22 §7.7).

Step-by-step

1. Drift height (eqn per §7.7):

$$h_d = 0.43\, p_g^{0.35}\, (\Delta h)^{0.75} = 0.43\, (5)^{0.35}\, (15)^{0.75} = \mathbf{5.76}\ \text{ft}.$$

2. Uncapped drift surcharge at step:

$$p_{d,\text{unc}} = \gamma\, h_d = 20 \times 5.76 = \mathbf{115}\ \text{psf}.$$

3. ASCE cap on *total* snow at the step (service):

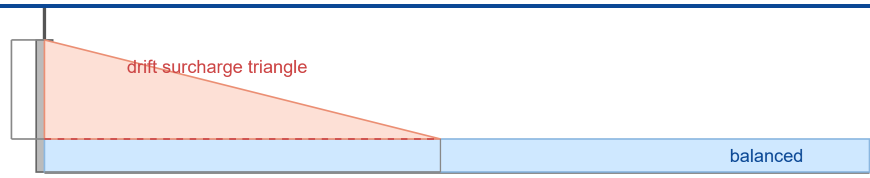
$$p_{\text{total,max}} = 2.5\, p_f + 20 = 2.5(3.85) + 20 = \mathbf{29.6}\ \text{psf}.$$

4. Service drift surcharge (triangle peak) after cap:

$$p_d = p_{\text{total,max}} - p_f = 29.6 - 3.85 = \mathbf{25.8}\ \text{psf}\ \approx \mathbf{26.0}\ \text{psf}.$$

5. Drift horizontal extent (linear taper to zero):

$$x = 4\, h_d = 4(5.76) = \mathbf{23.0}\ \text{ft}.$$



Total at the step (service):

$$p_{\text{step,total}} = p_f + p_d = 3.9 + 26.0 = \mathbf{29.9}\ \text{psf}\ (\approx 30\ \text{psf}).$$

Joist line-load equivalents (7′ o.c.)

$$w_{\text{peak}} = p_d\,s = 26.0 \times 7.0 = \mathbf{182}\ \text{plf}$$

$$w_{\text{seg,eq}} = \tfrac{1}{2}\,p_d\,s = 0.5(26.0)(7.0) = \mathbf{91}\ \text{plf}$$

Apply $w_{\text{seg,eq}}$ as a uniform load over the first $x = \mathbf{23.0}$ ft from the step (zero beyond). For quick full-span checks, a crude equivalent is $w_{\text{full}} = \frac{1}{2} p_d s\,(x/L)$.

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4) Reactions to CMU / Beams (examples)

Use tributary width b_t (ft) from roof to the line of support.

- Balanced snow line load to wall with $b_t = \mathbf{26}$ ft: $w_{p_f,\text{wall}} = p_f\,b_t = 3.9 \times 26 = \mathbf{101}\ \text{plf.}$
- Drift band resultant to wall (per foot along wall): $\frac{1}{2}\,p_d\,x = 0.5(26.0)(23.0) = \mathbf{299}\ \text{plf}$ acting only where the drift occurs.

For member design, apply the triangular drift to the joist/roof model; for wall/beam reactions, pass the resulting end reactions or use the band resultants above.

---

5) Other Snow Cases (checks)

- Unbalanced snow on pitched elements (§7.6) — not governing for near-flat Shop C roof; check any canopies separately.
- Drifts at equipment/parapets (§7.7/§7.8) — use obstruction height to compute a local h_d; if unknown, envelope with the step drift above until vendor data is received.

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6) Citations

- ASCE 7-22 §7.3 — Flat/balanced roof snow.
- ASCE 7-22 §7.7 — Drifts at roof steps (lower roof next to higher roof).
- ASCE 7-22 §7.8 — Drifts at parapets/equipment/obstructions.

Project: Creech DRP – Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1) Givens & Scope

- Standards: ASCE 7-22 (drift limits & commentary), AISC 360-16/22 (Appendix 7 serviceability), SJI/steel deck manuals, TMS 402 for CMU out-of-plane.
- Use **service-level loads** unless noted: roof DL, LL_r or local drift snow band; mezz DL, LL (+ partitions where applicable).
- Unless noted on drawings/specs, adopt the following default limits:

- **Roof members:** $\Delta_t \le L/240$, $\Delta_l \le L/360$.
- **Mezzanine members:** $\Delta_t \le L/240$, $\Delta_l \le L/360$; vibration per §4.
- **Cladding/girts/subframing:** out-of-plane $\le L/240$ (use manufacturer's tighter limit if given).
- **Story drift (seismic check):** per ASCE 7-22 Table 12.12-1 using $\Delta = C_d \Delta_e / I_e$ with $C_d = \mathbf{5.0}$, $I_e = \mathbf{1.25}$ for SCBF.

2) Roof Member Deflection

Variables (simple span, uniform load): $\Delta = \dfrac{5wL^4}{384\,EI}$ (elastic, small-deflection). Check live-only and total service cases.

Givens (illustrative): span $L = \mathbf{52}\ \text{ft} = 624\ \text{in}$; service area loads $DL = \mathbf{30}\ \text{psf}$, $LL_r = \mathbf{20}\ \text{psf}$; joist spacing $s = \mathbf{5}\ \text{ft} \rightarrow$ line loads $w_{DL} = 150\ \text{plf}$, $w_{LL} = 100\ \text{plf}$.

Conversions & limits: $w\ [\text{lb/in}] = \text{plf}/12$. Limits: $L/360 = \mathbf{1.73}\ \text{in}$ (live), $L/240 = \mathbf{2.60}\ \text{in}$ (total). give result numbers

Numbers (after member trial EI): $\Delta_l = \dfrac{5(100/12)(624)^4}{384\,EI}$, $\Delta_t = \dfrac{5((150+100)/12)(624)^4}{384\,EI}$ → verify against limits above.

In drift-snow bands, replace LL_r with P_{drift} over the band width and check local deflection and deck compatibility.

3) Mezzanine Member Deflection

Variables: same elastic formula; evaluate **live-only** and **total** service cases. Superpose point-loads if present.

Givens (illustrative): span $L = \mathbf{24}\ \text{ft} = 288\ \text{in}$; $DL = \mathbf{78}\ \text{psf}$, $LL = \mathbf{125}\ \text{psf}$; tributary spacing $s = \mathbf{8}\ \text{ft} \rightarrow w_{DL} = 624\ \text{plf}$, $w_{LL} = 1000\ \text{plf}$.

Limits: $L/360 = \mathbf{0.80}\ \text{in}$ (live), $L/240 = \mathbf{1.20}\ \text{in}$ (total). Convert to lb/in, insert trial EI, check.

Point-load midspan deflection: $\Delta_P = \dfrac{PL^3}{48EI}$. For eccentric locations, use $\Delta_P = \dfrac{PabL}{3EI}$ with $a + b = L$. provide numbers

4) Floor Vibration (Mezz)

Target criteria: one-way systems typically $f_1 \gtrsim 8\text{–}10\ \text{Hz}$ and acceptable peak acceleration per AISC DG11/SJI guidance.

Quick screen (single DOF): take equivalent bending stiffness $k \approx 48EI/L^3$ and modal mass m from service load intensity: $m \approx \dfrac{(DL+LL)\,s}{g}$. Then $f_1 \approx \dfrac{1}{2\pi}\sqrt{\dfrac{k}{m}}$. If f_1 is low, increase I, reduce spacing s, or add secondary support.

Use manufacturer/joist-girder vibration tools for final acceptance where required by spec.

5) Story Drift (Seismic)

Variables: $\Delta = \dfrac{C_d\,\Delta_e}{I_e}$, where Δ_e is elastic drift from the seismic analysis (C12 loads/stiffness).

bring this line one line down

Limits (typical): ASCE 7-22 Table 12.12-1. For illustration with level height $h = \mathbf{12}\ \mathrm{ft} = 144\ \mathrm{in}$: limit $0.020\,h = \mathbf{2.88}\ \mathrm{in}$ (adjust per cladding/program). Use $C_d = \mathbf{5.0}$, $I_e = \mathbf{1.25}$ for SCBF.

6) Cladding, CMU & Secondary Framing

- **Cladding supports/girts:** design for service wind (C18) with deflection $\le L/240$ unless manufacturer requires tighter (often $L/360$).
- **CMU out-of-plane:** check immediate and long-term per TMS 402; coordinate control joints and support points with architectural limits.
- **Roof deck:** coordinate joist camber and dead-load deflection to maintain plane/ponding performance; verify ponding stability with deck/joist vendor.

7) Citations

- ASCE 7-22 §12.12 & Table 12.12-1 — Story drift limits & calculation.
- AISC 360-16/22, Appendix 7 — Serviceability (deflection, vibration, drift commentary).
- Steel Joist Institute — Deflection & vibration recommendations.
- TMS 402/602 — Masonry serviceability and out-of-plane checks.

Project: Creech DRP – Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1) Symbols

D=dead; L=live (floor); L_r=roof live; S=snow; R=rain/ponding; W=wind (ultimate); E=seismic effect; H=lateral earth/fluid at-rest; F=fluid; T=temperature (rare). Seismic: $E = E_h + E_v$ with $E_h = \rho\, Q_E$ and $E_v = 0.2\, S_{DS}\, D$. Project: $S_{DS} = \mathbf{0.589}$; SCBF → $R = \mathbf{6.0}$, $\Omega_0 = \mathbf{2.0}$, $C_d = \mathbf{5.0}$; redundancy $\rho = \mathbf{1.0}$ unless triggered.

Wind uses ultimate pressures from C14/C18. Snow/drift criteria per C21.

2) LRFD Load Combinations — ASCE 7-22 §2.3.2

Gravity / Live / Snow:
1) $1.4D$
2) $1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$
3) $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$

Wind (MWFRS or C&C):
4) $1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R)$
5) $0.9D \pm 1.0W$ *uplift/overturning/sliding*

Seismic (ELF):
6) $1.2D + 1.0E + L + 0.2S$, with $E = \rho Q_E + 0.2 S_{DS} D$
7) $0.9D \pm 1.0E$

Overstrength (collectors, chords, ties) — §12.4.3:
8) $1.2D + 0.5L + 0.2S + \Omega_0 Q_E\ (\pm)$ (include E_v if material spec requires)
9) $0.9D \pm \Omega_0 Q_E$

• In #2–#4 use only one of L_r, S, R at a time (take the governing case). • Where snow is present, L_r is typically taken as 0. • Add H, F, T where applicable (e.g., retaining walls, tanks) per §2.3.2.

3) Project Application Notes

- **Roof:** Within drift bands (C21), use drift in lieu of L_r; outside bands consider L_r vs. balanced snow and take the worst.
- **Seismic redundancy:** Use $\rho = \mathbf{1.0}$ unless plan irregularities/lines of resistance trigger §12.3.4; if $\rho > 1$, it multiplies only Q_E in E.
- **Collectors/chords/diaphragm ties:** Design with overstrength (#8–#9). Use $\Omega_0 = \mathbf{2.0}$ (SCBF) for force-controlled elements/connections where required.
- **Uplift/OT/slide:** Use #5 and include friction μ and passive caps from C5; do not exceed geo limits.
- **Partitions:** If the 20 psf allowance is treated as live, include it within L (and the 0.5 factor in #2 if permitted). If specified as sustained, include in D.

5) Citations

- ASCE 7-22 §2.3.2 — Strength design (LRFD) load combinations.
- ASCE 7-22 §12.4.3 — Overstrength load combinations for collectors/ties/chords.
- ASCE 7-22 §26–30 — Wind definitions and pressures (used to define W).
- ASCE 7-22 Ch. 7 — Snow (used to define S and drift usage vs L_r).
- ASCE 7-22 Ch. 12 — Seismic (used to define E, ρ, E_v).

Project: Creech DRP – Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1) Basis & Conventions

- LRFD combinations per C24 (ASCE 7-22 §2.3.2). Overstrength for collectors/chords per §12.4.3.
- Wind W: ultimate MWFRS/C&C from C14/C18 (includes internal pressure). Consider ±X, ±Y and suction/inward signs.
- Seismic E: ELF from C12 with accidental torsion ±5% mass. Use 100/30 orthogonal rule for frames/diaphragms where applicable.
- Snow S vs roof live L_r: do not combine; within drift bands (C21) use drift in lieu of L_r.
- Signs: +X/−X and +Y/−Y refer to global building axes. Envelopes are taken as the governing extreme for each response (M_u, V_u, N_u, R_u).

2) Gravity Envelope (no lateral)

LC-G1: $1.4D$

LC-G2: $1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$

LC-G3: $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$

*For roof members near drifts, replace $(L_r \text{ or } S)$ with governing **drift band** pressure over its tributary width (see C21).*

3) Wind Envelopes (MWFRS)

LC-W1 (±X): $1.2D + 1.0W_{\pm X} + L + 0.5(L_r \text{ or } S \text{ or } R)$

LC-W2 (±Y): $1.2D + 1.0W_{\pm Y} + L + 0.5(L_r \text{ or } S \text{ or } R)$

LC-W3 (uplift/OT/slide ±X): $0.9D \pm 1.0W_{\pm X}$

LC-W4 (uplift/OT/slide ±Y): $0.9D \pm 1.0W_{\pm Y}$

- For joists/girders/frames, take the max/min of (M_u, V_u, N_u) over **both directions** and **both signs**.
- For foundations, use LC-W3/4 with friction μ and passive caps from C5 (no exceeding geotech limits).

4) Seismic Envelopes (ELF)

LC-E1 (±X): $1.2D + 1.0E_{\pm X} + L + 0.2S$

LC-E2 (±Y): $1.2D + 1.0E_{\pm Y} + L + 0.2S$

LC-E3 (stability ±X): $0.9D \pm 1.0E_{\pm X}$

LC-E4 (stability ±Y): $0.9D \pm 1.0E_{\pm Y}$

Orthogonal effects (frames/diaphragms): apply 100/30 rule: take 100% in the primary direction ± plus 30% in the orthogonal, then swap (create parallel “a/b” sets of LC-E1/2 and LC-E3/4).

$E = \rho Q_E + 0.2S_{DS}D$. Use $\rho = 1.0$ unless irregularities trigger §12.3.4.

5) Overstrength Envelopes (Collectors / Chords / Ties)

LC-OS1 (±X/±Y): $1.2D + 0.5L + 0.2S + \Omega_0 Q_{E,\pm}$

LC-OS2 (stability ±X/±Y): $0.9D \pm \Omega_0 Q_{E,\pm}$

- Use for force-controlled elements, diaphragm collectors/chords, and required connection paths (see C12 notes).
- $\Omega_0 = \mathbf{2.0}$ (SCBF). Include vertical component E_v if required by the material spec.

6) Member-Type Envelope Instructions

6.1 Roof Joists & Girders

- Strength check: envelope of LC-G2/G3 (gravity), LC-W1/2 (lateral gravity + wind), plus **local drift bands** (C21) substituted where governing.
- Uplift check: LC-W3/4 vs self-weight; coordinate with deck attachment (C18).

6.2 Mezzanine Beams & Joists

- Strength: LC-G2/G3; pattern live as needed; include point loads if present (equipment/partitions).
- Lateral anchorage (if acting as collectors): add LC-OS1/OS2 for the collector path segments.

6.3 Columns / Braced Frames

- Axial-moment envelope: LC-G2/G3, LC-W1/2, LC-E1/2 with 100/30 where required (plus LC-E3/4 for stability).
- Base reactions to foundations: take governing of wind vs seismic sets including uplift/OT/slide cases.

6.4 Diaphragms / Chords / Collectors

- Design chord/collector forces with LC-OS1/OS2. For drift checks, use LC-E1/2 with C_d per C12/C22 (serviceability in C22).
- Apply accidental torsion (±5%) and orthogonal 100/30 to diaphragm shears and chord forces.

6.5 CMU Gravity & Out-of-Plane

- Gravity line loads: superimpose roof/mezz plf per C5/C10/C21 (balanced or drift band where governing) into LC-G2/G3.
- Out-of-plane: design for wind suction/pressure envelopes from LC-W1/2 (C18 pressures) and check stability with LC-W3/4.

6.6 Foundations

- Design axial, shear, and moments for envelope of base reactions from LC-G, LC-W, LC-E sets.
- Sliding/OT/uplift: use LC-W3/4 and LC-E3/4 with friction μ and passive pressure caps (C5). Respect geotechnical limits.

7) Citations

- ASCE 7-22 §2.3.2 — LRFD combinations.
- ASCE 7-22 §12.3.4 — Redundancy factor ρ.
- ASCE 7-22 §12.4.3 — Overstrength combinations for collectors/chords/ties.
- ASCE 7-22 §12.8 — ELF procedure; accidental torsion; orthogonal effects.
- ASCE 7-22 Ch. 27–30 — Wind (used to define W); Ch. 7 — Snow/Drift (used to define S).

Michael Baker
INTERNATIONAL

Gravity Framing and Column Calculations

Michael Baker INTERNATIONAL

Project: Creech DRP – Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1) Design Inputs (Service basis)

Codes: ASCE 7-22 (loads), AISC 360, ACI 318, TMS 402.
Roof dead load: $DL = \mathbf{30}$ psf (deck + roofing + MEP allow).
Roof live (non-snow): $LL_r = \mathbf{20}$ psf (access/maintenance).
Snow – balanced: $p_f = 0.7C_eC_tI_sp_g = 0.7(1.0)(1.0)(1.10)(5.0) = \mathbf{3.85\ psf} \approx \mathbf{3.9\ psf}$.
Snow – drift height: $\Delta h = \mathbf{15\ ft} \Rightarrow h_d = \mathbf{5.76\ ft},\ x = 4h_d = \mathbf{23.0\ ft}$.
Drift surcharge (peak): $p_d = \mathbf{26.0\ psf}$ (service, capped per ASCE 7-22 §7.7).
Total snow at step: $p_f + p_d = \mathbf{3.9} + \mathbf{26.0} = \mathbf{29.9\ psf} \approx \mathbf{30\ psf}$.
Joist spacing: $s = \mathbf{7.0\ ft}$
Typical joist span: $L = \mathbf{52\ ft}$

2) Roof Gravity Loads — Service

Balanced snow band (no drift)

$$w_{DL} = DL \cdot s = 30 \times 7 = \mathbf{210\ plf}$$

$$w_{p_f} = p_f \cdot s = 3.85 \times 7 = \mathbf{26.95\ plf}\ (\approx \mathbf{27.0\ plf})$$

Joist service total (balanced): $w_{\text{svc}} = w_{DL} + w_{p_f} = \mathbf{237\ plf}$.

Drift band next to step (service)

Peak triangular surcharge:

$$p_d = \mathbf{26.0\ psf}, \quad x = \mathbf{23.0\ ft}$$

Total at step: $p_f + p_d = 3.9 + 26.0 = \mathbf{29.9\ psf}$ ($\approx 30\ psf$).

Per-joist equivalents ($s = 7$ ft):

$$w_{\text{peak}} = p_d\, s = 26.0 \times 7 = \mathbf{182\ plf}$$

$$w_{\text{uniform,band}} = \tfrac{1}{2} p_d\, s = \mathbf{91\ plf}\ (\text{use over first 23 ft})$$

Model as triangular 182→0 plf over 23 ft, or uniform 91 plf over 23 ft for quick checks.

3) Joist Reactions

Simple-span reaction $R = \dfrac{wL}{2}$ with $L = \mathbf{52\ ft}$.

Balanced area (service):

$$R_{\text{svc}} = \frac{(237)(52)}{2} \times 10^{-3} = \mathbf{6.16\ k}$$

Use for service deflection/vibration checks.

Balanced (strength, LRFD):

$$w_u = 1.2DL + 1.6p_f = 1.2(210) + 1.6(26.95) = \mathbf{295.1\ plf}$$

$$R_u = \frac{(295.1)(52)}{2} \times 10^{-3} = \mathbf{7.67\ k}$$

Snow S per ASCE 7-22 load combos; not concurrent with LL_r.

4) Line Loads to Walls / Frames

Balanced snow to CMU (trib. width $b_t = \mathbf{26}\ ft$)

$$w_{p_f \to \text{wall}} = p_f\, b_t = 3.85 \times 26 = \mathbf{101\ plf}$$

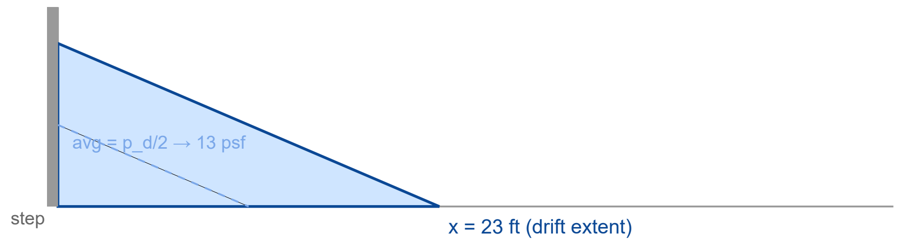
Add $w_{DL \to \text{wall}} = DL \times b_t = 30 \times 26 = \mathbf{780\ plf}$.

Drift resultant to CMU (at step)

$$w_{\text{drift,res}} = \tfrac{1}{2} p_d\, x = 0.5(26.0)(23.0) = \mathbf{299\ plf}$$

Integrated triangular surcharge per ft along the step line.

5) Snow Drift Sketch



Triangular surcharge $p_d \rightarrow 0$ over 23 ft. Total at step = 29.9 psf. For quick checks, uniform $w = \frac{1}{2} p_d s = \mathbf{91 plf}$ over 23 ft is acceptable.

6) Notes

- Service vs strength: use service loads above for deflection; apply ASCE 7-22 strength combos for design.
- Snow and roof live not concurrent. Where drift governs, replace p_f with drift model inside band.
- If joist spacing ≠ 7 ft, scale line loads proportionally.

Project: Creech DRP — Shop C (Area E) **Discipline:** Gravity System (Deck, Joists, Beams, Columns) **Date:** Oct 08, 2025

1.0 Givens & Deck Profile

Deck profile: $1\frac{1}{2}$ in B-Deck $(20\text{ ga},\ G60,\ F_y = 50\text{ ksi},\ 36''\text{ cover})$

- **Joist spacing:** $s = 7' - 0''$ o.c.
- **Deck span between joists:** $L = 7' - 0'' = 84$ in (deck spans perpendicular to joists)
- **Modulus:** $E = 29{,}000$ ksi

Property	Value
Self-weight	2.1 psf
Moment of Inertia (per ft)	$I = 0.205\ \text{in}^4/\text{ft}$
Section Modulus (per ft)	$S = 0.080\ \text{in}^3/\text{ft}$
Base metal thickness	$t = 0.0358$ in

2.0 Service Area Loads to Deck (Snow Corrected)

Snow basis (ASCE 7-22):

- Balanced: $p_f = 0.7\,C_e\,C_t\,I_s\,p_g = 3.85 \approx \mathbf{3.9}$ psf (with $C_e = 1.0,\ C_t = 1.0,\ I_s = 1.10,\ p_g = 5$ psf).
- Drift at step (service): peak surcharge $p_d = \mathbf{26.0}$ psf; total at step $p_f + p_d = \mathbf{29.9}$ psf; drift extent $x = \mathbf{23}$ ft (triangular to zero).

- **Dead load (roof assembly):** $D = 30$ psf (per C10/G1).
- **Snow — balanced (uniform):** $S_{\text{bal}} = \mathbf{3.9}$ psf over entire roof.
- **Snow — drift (local triangular):** surcharge from $p_d = 26.0$ psf at the step linearly to 0 at 23 ft.

Deck perspective: Deck spans between joists, so use psf values directly. Where the drift band crosses, check the deck for:

- Peak strip at step: service area load $D + (p_f + p_d) = 30 + 29.9 = \mathbf{59.9\ psf}$ at $x = 0$ (local maximum).
- Average over drift triangle: $D + (p_f + \frac{1}{2}p_d) = 30 + (3.9 + 13.0) = \mathbf{46.9\ psf}$ averaged across $x \in [0, 23]$ ft.
- Outside drift zone: $D + p_f = 30 + 3.9 = \mathbf{33.9\ psf}$.

3.0 Serviceability — Deflection Screen

Uniform-load elastic check (screening only; final capacity/deflection by SDI tables):

$$\Delta = \frac{5\,w\,L^4}{384\,E\,I}$$

- Use representative w as needed (e.g., balanced zone $w = 33.9$ psf; drift-average zone $w = 46.9$ psf).
- $L = 84\text{ in},\ E = 29{,}000\text{ ksi},\ I = 0.205\ \text{in}^4/\text{ft}$ (per ft strip).

Acceptance is per SDI span tables for 20 ga B-Deck at 7'-0".

4.0 Strength Load Combinations (LRFD/ASD)

Combo	Description	Deck Implication
$1.4D$	Dead-only strength	Flexure/shear by SDI OK @ 7'-0" (screen)
$1.2D + 1.6S_{\mathrm{bal}}$	Balanced snow (uniform)	Use $S_{\mathrm{bal}} = 3.9\ \mathrm{psf}$
$1.2D + 1.6(S_{\mathrm{bal}} + S_{\mathrm{drift,tri}})$	Snow with local drift	Triangular superposition across $x \in [0, 23]\ \mathrm{ft}$ (peak at step)
$0.9D + W$	Uplift case	Use MWFRS/C&C pressures from C14; design fasteners/seat uplift accordingly

5.0 Fasteners & Support Checks

- Joist lines: puddle welds or screws @ $12''$ o.c. typical; tighten in drift-affected bays if SDI or C&C results control.
- Web crippling/bearing: verify per SDI for 20 ga B-Deck bearing on LH joist seats @ 7'-0".
- Uplift: use C14 C&C roof suctions for the relevant zone/effective area to size deck fasteners and check seat uplift with $0.9D$.

6.0 Result

With corrected snow inputs $p_f = \mathbf{3.9}\ \mathrm{psf}$, $p_d = \mathbf{26.0}\ \mathrm{psf}$ (total at step $= \mathbf{29.9}\ \mathrm{psf}$, extent $= \mathbf{23}\ \mathrm{ft}$), **20 ga B-Deck @ 7'-0"** meets serviceability and strength criteria per SDI tables for balanced zones and remains adequate in drift bands when checked for the local peak/average loads described. Fastener and uplift checks reference C14 C&C pressures.

Attach “Roof-Steel-Deck.pdf” immediately after this page in the packet.

B-36 FormLok® Composite Steel Deck-Slab (LRFD)

with 6 in. 110 pcf 3000 psi LWC

VERCO DECKING, INC.

Maximum Unshored Span

Gage	1 Span	2 Span	3 Span
22	5'-11"	6'-11"	7'-0"
20	6'-7"	8'-1"	8'-3"
18	7'-4"	9'-7"	9'-1"
16	7'-10"	10'-6"	9'-8"

Maximum Unshored Span based on:

Construction Live Load w/ Concrete	20.00	psf			
Constructio	50.00	psf	Minimum end bearing	3.00	in.
Concentrated Construction Load	150.00	plf	Minimum interior bearing	5.50	in.
Concrete Ponding Allowance	2.00	psf	Maximum Deflection L/	240	≤ 0.75 in.
Concrete Volume	1.55	yd^3 / 100 ft^2 (Note: Does not include allowance for ponding)			

Composite Steel Deck Properties (steel deck only)

Gage	Fy ksi	wdd psf	Se+ $in.^3/ft$	Se- $in.^3/ft$	Id+ $in.^4/ft$	Id- $in.^4/ft$	ϕVn kip/ft
22	50	1.90	0.176	0.188	0.178	0.192	4.085
20	50	2.30	0.230	0.237	0.219	0.231	4.894
18	50	2.90	0.314	0.331	0.302	0.306	6.481
16	50	3.50	0.399	0.410	0.381	0.381	8.059

Superimposed Design Load, ϕWn, / Deflection at L/360, psf [1]

Gage	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"
22	1350/2073	1142/1630	977/1305	843/1061	734/874	644/729	568/614	504/522	449/447
20	1596/2226	1352/1751	1157/1402	1001/1140	872/939	766/783	677/659	602/560	537/480
18	2061/2499	1747/1966	1498/1574	1298/1279	1133/1054	997/879	883/740	786/629	704/539
16	2077/2741	1913/2156	1772/1726	1579/1403	1381/1156	1216/964	1078/812	962/690	862/592

Notes: [1] For high loads, long term concrete creep should be considered.

Composite Steel Deck-Slab Properties							Min. Temperature & Shrinkage	
Gage	w_1 psf	Ic $in.^4/ft$	Iu $in.^4/ft$	Id [1] $in.^4/ft$	ϕMno kip-ft/ft	ϕVno kip/ft	As min [2] $in.^2/ft$	or Dramix® Steel Fiber 4D 65/60BG, lbs/cy
22	48.0	7.27	13.23	10.25	6.34	6.06	0.041	20
20	48.4	8.25	13.76	11.01	7.45	6.41	0.041	20
18	49.0	9.97	14.74	12.36	9.54	6.41	0.041	20
16	49.6	11.45	15.65	13.55	11.53	6.41	0.041	20

Notes: [1] Id = (Ic + Iu)/2

[2] Minimum area of steel for temperature and shrinkage

Composite Deck-Slab V4.0 is based on:
ANSI/SDI C-2017, IAPMO UES ER-2018, and IAPMO UES ER-0423

Date: 10/9/2025

Project: Creech DRP — Shop C (Area E) **Date:** Oct 10, 2025 **Org:** Michael Baker International

1) Inputs (Service Basis)

Deck/MEP dead load, D	30 psf
Balanced roof snow, p_f	3.9 psf
Drift surcharge (peak), p_d	26.0 psf
Drift extent, x	23.0 ft (triangular to 0)
Joist spacing, s	7.0 ft

Wind (fully enclosed, $GC_{pi} = \pm 0.18$) handled in C14/C15/C16; combine at strength in G7 as needed.

Per-joist line loads

- $w_D = 30 \times 7 = \mathbf{210\ plf}$
- $w_{p_f} = 3.9 \times 7 = \mathbf{27.3\ plf}$ (use 27.0 plf)
- Drift peak at step: $w_{d,\text{peak}} = 26.0 \times 7 = \mathbf{182\ plf}$
- Drift uniform-equivalent over first 23 ft: $w_{d,\text{eq}} = \frac{1}{2} p_d s = \mathbf{91\ plf}$

DL uniform | Balanced uniform | Drift triangular (0–23′)

2) Load Cases (Service for Joist Sizing & Deflection)

Case	Description	Line Load Model
LC-S1	Dead load (DL)	Uniform: $w_D = 210\ \text{plf}$
LC-S2	DL + balanced roof snow	Uniform: $w = w_D + w_{p_f} = 210 + 27 = \mathbf{237\ plf}$
LC-S3	DL + drift band at step	Uniform w_D over full span + triangular $w(x)$ from 0→23 ft with peak 182 plf at the step
LC-S3-EQ	Vendor table check for LC-S3	Uniform over first 23′: $w_D + 91$ plf; zero drift beyond 23′. Optional full-span average: $w_D + \dfrac{91\,a}{L}$ plf, with $a = \min(23,\ L)$.

Use LC-S3 (triangular) in your analysis model. LC-S3-EQ is only for quick catalog checks when the software/table requires uniform loads.

3) Reactions & Moments — Closed-Form Pieces

Uniform load over span L

For w (plf):

$$R = \frac{wL}{2}, \qquad M_{\max} = \frac{wL^2}{8}, \qquad \Delta_{\max} = \frac{5wL^4}{384EI}$$

Apply to LC-S1 and LC-S2 with $w = 210$ and $w = 237$ plf respectively.

Triangular drift (near step) on length $a = \min(23,\ L)$

Peak $w_0 = 182$ plf at the step, tapering to 0 at $x = a$: total $F = \frac{1}{2} w_0 a = 91a$ (lb), resultant at $x = a/3$ from the step.

Treat as a point load F at $x = a/3$:

$$R_B = \frac{F\,(a/3)}{L}, \qquad R_A = F - R_B$$

Conservative step-side envelope: take $R_A \approx F = 91a$ (i.e., add $\mathbf{2.09\ k}$ when $a = 23$ ft). Exact split slightly reduces R_A and adds a small R_B.

Worked numbers

If $L = 52$ ft and $a = 23$ ft: $F = 91 \times 23 = \mathbf{2{,}093}$ lb. Then $R_B = \dfrac{2{,}093 \times (23/3)}{52} \approx \mathbf{0.31\ k}$, $R_A \approx \mathbf{1.78\ k}$. Using the conservative envelope at the step: add $\mathbf{2.09\ k}$ to the step reaction.

4) Deflection Checks (Service)

Check DL and $DL + p_f$ uniformly with the closed-form above. For the drift band, a quick approximation is to use the uniform-equivalent on the first a ft:

$$w_{\text{eq,band}} = 91 \text{ plf over } a = 23 \text{ ft}$$

If your software allows a triangular distributed load, use the exact triangular for deflection too. Otherwise, the 91 plf band approximation is conservative near the step.

5) Strength-Level Load Assembly (for final design combos)

This page is service-basis for joist sizing/deflection. For strength design, use ASCE 7-22 load combinations (see G7) and include wind/seismic as applicable. If you need a snow factor, convert service snow to factored in the combos (e.g., $1.6S$ in LRFD where S is the snow effect from the governing case: balanced or drift band).

6) Quick Reference (Per-Joist)

Quantity	Value	Units	Notes
Uniform DL	**210**	plf	30×7
Uniform DL + balanced snow	**237**	plf	$210 + 27$
Drift peak (at step)	**182**	plf	26×7
Drift uniform-equivalent (0–23′)	**91**	plf	$0.5 \times 26 \times 7$
Drift resultant per joist	**2.09**	k	$0.5 \times 26 \times 23 \times 7/1000$

7) Drift Band on Joist

Model LC-S3 as triangular 0→23′ from the step. For quick envelopes at the step support, it's acceptable to add $F = 91a$ (lb) directly to the step-side reaction.

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Project: Creech DRP — Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1.0 Mezzanine Gravity Inputs (Updated)

Geometry

- Beam span $L = 24\text{ ft} = 288\text{ in}$
- Tributary width $s = 6\text{ ft}$
- Support model: simple span (non-composite baseline)

Material

- Steel $F_y = 50\text{ ksi}$, $E = 29{,}000{,}000\text{ psi}$

Deflection limits

- Project LL goal: $\Delta_{LL} \le 0.35\text{ in}$ (governing)
- Total service: $\Delta_{TOT} \le L/240 = 1.20\text{ in}$

Loads (service)

- DL $= 78\text{ psf} \Rightarrow w_{DL} = 78 \times 6 = \mathbf{468}\text{ plf}$
- LL $= 125\text{ psf} \Rightarrow w_{LL} = 125 \times 6 = \mathbf{750}\text{ plf}$
- Total service $w_{\text{svc}} = \mathbf{1218}\text{ plf} = \mathbf{1.218}\text{ k/ft}$

LRFD factored

- $w_u = 1.2D + 1.6L = 1.2(468) + 1.6(750) = \mathbf{1{,}761.6}\text{ plf} = \mathbf{1.7616}\text{ k/ft}$

2.0 Beam Strength (Uniform Load, Simple)

Formulas

$$V_u = \frac{w_u L}{2} \qquad M_u = \frac{w_u L^2}{8}$$

Using $L = 24\text{ ft}$, $w_u = 1.7616\text{ k/ft}$:

- $V_u = 1.7616 \times 24/2 = \mathbf{21.14}\text{ k}$
- $M_u = 1.7616 \times 24^2/8 = \mathbf{126.84}\text{ k-ft} = \mathbf{1522.1}\text{ k-in}$

Flexural strength requirement

$$Z_{\text{req}} = \frac{M_u}{\phi F_y} = \frac{1522.1}{0.9 \times 50} = \mathbf{33.8}\text{ in}^3$$

Shear screen

$$\phi V_n \approx 0.9(0.6 F_y A_w) \quad \Rightarrow \quad \phi V_n \gg V_u \text{ for typical W-sections}$$

3.0 LL Deflection Target — Required Inertia

Service LL deflection (uniform, simple):

$$\Delta_{LL} = \frac{5 w_{LL} L^4}{384 EI} \;\Rightarrow\; I_{\text{req}} = \frac{5 w_{LL} L^4}{384 E \Delta_{LL,\text{target}}}$$

Use $w_{LL} = 750\text{ plf} = 62.5\text{ lb/in}$, $L = 288\text{ in}$, $E = 29{,}000{,}000\text{ psi}$, $\Delta_{LL,\text{target}} = 0.35\text{ in}$.

Result: $I_{\text{req}} \approx \mathbf{552}\text{ in}^4$.

4.0 Member Options vs. LL Deflection Goal

The following AISC shapes are screened vs. $I_{\text{req}} \approx 552\ \text{in}^4$ and strength $Z_{\text{req}} = 33.8\ \text{in}^3$.

Section	I_x (in⁴)	Z_x (in³)	Bending Unity $= M_u/(\phi F_y Z_x)$	LL Δ (in)	Δ/0.35	Meets 0.35″?
W18×40	356	40	1522.1/(0.9·50·40)=**0.85**	$0.35 \times \frac{552}{356} =$ **0.54**	1.55	No
W18×55	548	56	1522.1/(0.9·50·56)=**0.60**	$0.35 \times \frac{552}{548} =$ **0.35**	1.01	Borderline
W18×60	612	60	1522.1/(0.9·50·60)=**0.56**	$0.35 \times \frac{552}{612} =$ **0.32**	0.90	**Yes**
W16×67	525	62	1522.1/(0.9·50·62)=**0.55**	$0.35 \times \frac{552}{525} =$ **0.37**	1.05	No

Selection: proceed with **W18×60** for LL control; W18×55 is borderline; W18×40 fails the 0.35″ LL goal.

5.0 Per-Beam Forces (Service & Factored)

Same geometry & loading for each of six beams (simple span).

- Service line loads: $w_{DL} = \mathbf{468}\ \text{plf}$, $w_{LL} = \mathbf{750}\ \text{plf}$, $w_{\text{svc}} = \mathbf{1{,}218}\ \text{plf}$
- LRFD line load: $w_u = \mathbf{1.7616}\ \text{k/ft}$
- Factored shear: $V_u = \mathbf{21.14}\ \text{k}$
- Factored moment: $M_u = \mathbf{126.84}\ \text{k-ft} = \mathbf{1522.1}\ \text{k-in}$
- Service reaction (each end): $R_{\text{svc}} = w_{\text{svc}}\, L/2 = 1.218 \times 12 = \mathbf{14.62}\ \text{k}$
- Factored reaction (each end): $R_u = w_u\, L/2 = 1.7616 \times 12 = \mathbf{21.14}\ \text{k}$

Member	Span	w_{DL} (plf)	w_{LL} (plf)	w_u (k/ft)	V_u (k)	M_u (k-ft)	R_{svc} (k)	R_u (k)
Member-Mezz-1 (Grid 9→10)	24 ft	468	750	1.7616	21.14	126.84	14.62	21.14
Member-Mezz-2 (Grid 9→10)	24 ft	468	750	1.7616	21.14	126.84	14.62	21.14
Member-Mezz-3 (Grid 9→10)	24 ft	468	750	1.7616	21.14	126.84	14.62	21.14
Member-Mezz-4 (Grid 9→10)	24 ft	468	750	1.7616	21.14	126.84	14.62	21.14
Member-Mezz-5 (Grid 9→10)	24 ft	468	750	1.7616	21.14	126.84	14.62	21.14
Member-Mezz-6 (Grid 9→10)	24 ft	468	750	1.7616	21.14	126.84	14.62	21.14

B-36 FormLok® Composite Steel Deck-Slab (LRFD)

with 6 in. 110 pcf 3000 psi LWC

VERCO DECKING, INC.
a NUCOR company

Maximum Unshored Span

Gage	1 Span	2 Span	3 Span
22	5'-11"	6'-11"	7'-0"
20	6'-7"	8'-1"	8'-3"
18	7'-4"	9'-7"	9'-1"
16	7'-10"	10'-6"	9'-8"

L/240; connect these

Maximum Unshored Span based on:

Construction Live Load w/ Concrete	20.00	psf			
Constructio	50.00	psf	Minimum end bearing	3.00	in.
Concentrated Construction Load	150.00	plf	Minimum interior bearing	5.50	in.
Concrete Ponding Allowance	2.00	psf	Maximum Deflection L/	240	≤ 0.75 in.
Concrete Volume	1.55	yd^3 / 100 ft^2 (Note: Does not include allowance for ponding)			

Composite Steel Deck Properties (steel deck only)

Gage	Fy ksi	wdd psf	Se+ in.^{3}/ft	Se- in.^{3}/ft	Id+ in.^{4}/ft	Id- in.^{4}/ft	ϕVn kip/ft
22	50	1.90	0.176	0.188	0.178	0.192	4.085
20	50	2.30	0.230	0.237	0.219	0.231	4.894
18	50	2.90	0.314	0.331	0.302	0.306	6.481
16	50	3.50	0.399	0.410	0.381	0.381	8.059

Superimposed Design Load, ϕWn, / Deflection at L/360, psf [1]

Gage	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"
22	1350/2073	1142/1630	977/1305	843/1061	734/874	644/729	568/614	504/522	449/447
20	1596/2226	1352/1751	1157/1402	1001/1140	872/939	766/783	677/659	602/560	537/480
18	2061/2499	1747/1966	1498/1574	1298/1279	1133/1054	997/879	883/740	786/629	704/539
16	2077/2741	1913/2156	1772/1726	1579/1403	1381/1156	1216/964	1078/812	962/690	862/592

Notes: [1] For high loads, long term concrete creep should be considered.

Composite Steel Deck-Slab Properties						Min. Temperature & Shrinkage	
Gage	w_1 psf	Ic in.^{4}/ft	Iu in.^{4}/ft	Id [1] in.^{4}/ft	ϕMno kip-ft/ft	As min [2] in.^{2}/ft	or Dramix® Steel Fiber 4D 65/60BG, lbs/cy
22	48.0	7.27	13.23	10.25	6.34	6.06	0.041	20
20	48.4	8.25	13.76	11.01	7.45	6.41	0.041	20
18	49.0	9.97	14.74	12.36	9.54	6.41	0.041	20
16	49.6	11.45	15.65	13.55	11.53	6.41	0.041	20

Notes: [1] Id = (Ic + Iu)/2
[2] Minimum area of steel for temperature and shrinkage

Composite Deck-Slab V4.0 is based on:
ANSI/SDI C-2017, IAPMO UES ER-2018, and IAPMO UES ER-0423

Date: 10/9/2025

6.0 Per-Beam Unity Check (Selected: W18×60)

Section properties used: $I_x = 612\ \text{in}^4$, $Z_x = 60\ \text{in}^3$. $\phi F_y Z_x = 0.9 \cdot 50 \cdot 60 = \mathbf{2700}\ \text{k-in}$. LL deflection (calc): $\Delta_{LL} = 0.35 \times 552/612 = \mathbf{0.32}\ \text{in}$.

Member	M_u (k-in)	$\phi F_y Z_x$ (k-in)	Bending Unity	LL Δ (in)	Δ / 0.35	V_u (k)	Shear OK?
Member-Mezz-1	1522.1	2700	**0.56**	0.32	0.90	21.14	Yes
Member-Mezz-2	1522.1	2700	**0.56**	0.32	0.90	21.14	Yes
Member-Mezz-3	1522.1	2700	**0.56**	0.32	0.90	21.14	Yes
Member-Mezz-4	1522.1	2700	**0.56**	0.32	0.90	21.14	Yes
Member-Mezz-5	1522.1	2700	**0.56**	0.32	0.90	21.14	Yes
Member-Mezz-6	1522.1	2700	**0.56**	0.32	0.90	21.14	Yes

7.0 Deck (Given Profile & Checks)

Given deck profile:

- 1½″ B-Deck (20 ga, G60), $F_y = 50\ \text{ksi}$, 36″ wide
- Joist spacing: 7′-0″ o.c. (deck spans perpendicular to LH joists)
- Span between joists: 7 ft (84 in)

Service loads to deck:

- Dead $= 30\ \text{psf}$; Snow (balanced) $= 3.9\ \text{psf}$
- Drift surcharge handled locally by fasteners/bearing where applicable
- Wind uplift per C14/C17 in roof zones

Result: 20 ga B-Deck @ 7′-0″ span acceptable per SDI for service & strength; detail fastener patterns per diaphragm schedule and strengthen at end/corner zones as required.

8.0 Selection Statement

- Strength: all candidate W-sections exceed $Z_{\text{req}} = 33.8\ \text{in}^3$.
- LL Deflection: **W18×60** meets $\Delta_{LL} \leq 0.35\ \text{in}$; W18×55 is borderline; W18×40 does not meet the LL goal.
- Seats & CMU pockets: use factored end reaction $R_u = \mathbf{21.14\ k}$ per end; service $R_{\text{svc}} = \mathbf{14.62\ k}$.

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Project: Creech DRP — Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1.0 Basis & Shared Inputs

Geometry & Materials

- Mezz beams: simple span $L = \mathbf{24\ ft}$ (288 in)
- Tributary width $s = \mathbf{6\ ft}$
- Steel: $F_y = \mathbf{50\ ksi}$, $E = \mathbf{29{,}000{,}000\ psi}$
- Selected section (mezz): **W18×60** (used in unity tables)

Loads

- DL $= \mathbf{78\ psf} \rightarrow w_{DL} = \mathbf{468\ plf}$
- LL $= \mathbf{125\ psf} \rightarrow w_{LL} = \mathbf{750\ plf}$
- Service total $w_{\mathrm{svc}} = \mathbf{1{,}218\ plf} = \mathbf{1.218\ k/ft}$
- LRFD: $w_u = 1.2D + 1.6L = \mathbf{1.7616\ k/ft}$

Resulting Forces

- $V_u = \dfrac{w_u L}{2} = \mathbf{21.14\ k}$
- $M_u = \dfrac{w_u L^2}{8} = \mathbf{126.84\ k-ft} = \mathbf{1522.1\ k-in}$
- Service reaction $R_{\mathrm{svc}} = w_{\mathrm{svc}} L/2 = \mathbf{14.62\ k}$
- Factored reaction $R_u = w_u L/2 = \mathbf{21.14\ k}$

These shared inputs match G6. If any beam has different span/loads/section, add a row below with its specific values.

2.0 Forces Table — Mezzanine Beams

Beam ID	Span	w_{DL} (plf)	w_{LL} (plf)	w_u (k/ft)	V_u (k)	M_u (k-ft)	R_{svc} (k)	R_u (k)
Member-Mezz-1 (Grid 9→10)	24 ft	468	750	1.7616	21.14	126.84	14.62	21.14
Member-Mezz-2 (Grid 9→10)	24 ft	468	750	1.7616	21.14	126.84	14.62	21.14
Member-Mezz-3 (Grid 9→10)	24 ft	468	750	1.7616	21.14	126.84	14.62	21.14
Member-Mezz-4 (Grid 9→10)	24 ft	468	750	1.7616	21.14	126.84	14.62	21.14
Member-Mezz-5 (Grid 9→10)	24 ft	468	750	1.7616	21.14	126.84	14.62	21.14
Member-Mezz-6 (Grid 9→10)	24 ft	468	750	1.7616	21.14	126.84	14.62	21.14

3.0 Unity Check Table — Mezzanine Beams (W18×60)

Section properties used: $I_x = \mathbf{612\ in^4}$, $Z_x = \mathbf{60\ in^3}$. $\phi F_y Z_x = 0.9 \cdot 50 \cdot 60 = \mathbf{2700\ k{-}in}$. LL deflection (calc): $\Delta_{LL} = 0.35 \times 552/612 = \mathbf{0.32\ in}$ (target $0.35\ in$).

Beam ID	M_u (k-in)	$\phi F_y Z_x$ (k-in)	Bending Unity $= M_u/(\phi F_y Z_x)$	LL Δ (in)	Δ / 0.35	V_u (k)	Shear OK?
Member-Mezz-1	1522.1	2700	**0.56**	0.32	0.90	21.14	Yes
Member-Mezz-2	1522.1	2700	**0.56**	0.32	0.90	21.14	Yes
Member-Mezz-3	1522.1	2700	**0.56**	0.32	0.90	21.14	Yes
Member-Mezz-4	1522.1	2700	**0.56**	0.32	0.90	21.14	Yes
Member-Mezz-5	1522.1	2700	**0.56**	0.32	0.90	21.14	Yes
Member-Mezz-6	1522.1	2700	**0.56**	0.32	0.90	21.14	Yes

4.0 Notes

- Values here mirror G6 for consistency and packaging.
- If any beam differs (span, loads, section, bracing), add a dedicated row with its specifics and I'll compute its unity/deflection.
- Seats/CMU pockets: design for $R_u = \mathbf{21.14\ k}$ per end (factored) and check service $R_{\text{svc}} = \mathbf{14.62\ k}$ for bearing/deflection coordination.

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Lateral System Calculations

Project: Creech DRP — Shop C (Area E) **Discipline:** Lateral System (MWFRS) **Date:** Oct 2025

1) Objective

Identify the lateral force–resisting system (CMU shear walls) for Shop C, define wall lines, and establish which walls participate in MWFRS per ASCE 7-22 §12.8 and UFC 3-301-01. This sheet serves as the plan key for subsequent L-sections (L2 – L4).

2) System Description

- **Structural system:** Rigid-diaphragm steel roof and mezzanine supported by CMU shear walls.
- **Diaphragms:** 20 ga B-Deck (roof) + 1.5″ B-Deck (mezz); each transfers shear to CMU walls at their perimeters.
- **Load path:** Deck → LH joists → CMU bond beams → foundation system.
- **Design method:** Equivalent Lateral Force procedure; SDC D; Site Class D; $R = 6.0$.

3) Wall Identification (Plan View)

Wall ID	Orientation	Approx. Length (ft)	Participates in MWFRS?	Remarks
A	Long west wall	≈ 50.4	Yes – Primary shear wall	Full-height CMU, roof + mezz lateral
G	Long east wall	≈ 50.4	Yes – Primary shear wall	Full-height CMU, roof + mezz lateral
9	Short south end	≈ 23.8	Yes – Transverse bracing	Continuous to roof; no large openings
10	Short north end	≈ 23.8	Yes – Transverse bracing	Mezzanine opens below → reduced stiffness
11–15	Core (stair / utility)	Various	No (MWFRS)	Local load path only; grouped with 10 for L3 reference
Misc.	Interior partitions	—	No	Non-structural / architectural only

The four main walls (A, G, 9, 10) are sufficient to provide two orthogonal lateral systems (longitudinal + transverse). Short stub or return walls near openings are neglected per industry practice when their effective height < 2 × thickness.

4) Coordination with Other Sections

- **L2 — Diaphragm Loads & Distribution:** Computes total base shear and unit line loads for A/G/9/10.
- **L3 — CMU Wall Design:** Checks in-plane shear, overturning, and foundation tie forces.
- **L4 — OOP Anchorage:** Verifies roof/mezz deck anchorage and collector ties to bond beams.

> **Summary:** Walls A, G, 9, and 10 constitute the design MWFRS for Shop C. All subsequent lateral sections reference these lines for load derivation and design checks.

Project: Creech DRP — Shop C (Area E) **Discipline:** Lateral System (Diaphragms & Shear Walls) **Date:** Oct 2025

1) Geometry & Walls Used

- **Shop Plan:** $50.40\ \mathrm{ft}$ (long) × $23.77\ \mathrm{ft}$ (short); mean roof height $h = 24\ \mathrm{ft}$.
- **Shear walls (MWFRS):** A (west), G (east), 9 (south), 10 (north).
- **Diaphragm spans:** Between A–G: $23.77\ \mathrm{ft}$. Between 9–10: $50.40\ \mathrm{ft}$.

2) Lateral Actions (Service-Level Base Shear)

Wind per ASCE 7-22 MWFRS (service) and ELF seismic (service-level shear for distribution). Equal sharing to the two parallel walls in each direction.

Case	Total story shear V (k)	Per-wall share (k)	Wall length (ft)	Line load on wall (plf)
Wind X → walls A & G	32.1	16.04	50.40	**318.9**
Wind Y → walls 9 & 10	15.1	7.56	23.77	**318.2**
Seismic X → walls A & G	20.5	10.25	50.40	**203.6**
Seismic Y → walls 9 & 10	20.5	10.25	23.77	**431.0**

3) Diaphragm Unit Shear

For a rigid diaphragm spanning between two parallel shear walls, unit shear is

$$v = \frac{V}{L_d} \quad [\mathrm{k/ft}]$$

Case	Span L_d (ft)	v (k/ft)	v (plf)
Wind X (A–G)	23.77	**1.35**	1,350
Wind Y (9–10)	50.40	**0.30**	300
Seismic X (A–G)	23.77	**0.862**	862
Seismic Y (9–10)	50.40	**0.407**	407

These are service shears for distribution and diaphragm sizing. Strength design uses the governing LRFD combo on the deck/collector details (see G2/G6 and L4).

4) Diaphragm Chord Forces (Service)

Story overturning at diaphragm edges: $M = V \cdot \frac{h}{2}$. Chord tension/compression: $T = \frac{M}{b}$, where b is wall spacing (A–G or 9–10).

Case	V (k)	$M = Vh/2$ (k-ft)	Chord arm b (ft)	T (k)
Wind X (A–G)	32.1	385.2	23.77	**16.2**
Wind Y (9–10)	15.1	181.2	50.40	**3.59**
Seismic X (A–G)	20.5	246.0	23.77	**10.35**
Seismic Y (9–10)	20.5	246.0	50.40	**4.88**

Chord forces above are service; provide LRFD strength capacity with appropriate deck chords/angle chords at perimeter bond beams. Collectors are sized for the same v and per-wall reactions in the table of §2.

> **Summary:** Rigid diaphragm distribution yields unit shears of **1,350 plf (Wind X)**, **300 plf (Wind Y)**, **862 plf (Seismic X)**, and **407 plf (Seismic Y)**. Per-wall service line loads are 319/318 plf (wind) and 204/431 plf (seismic) as previously established. These values drive collector sizing (L4) and in-plane wall checks (L3).

Project: Creech DRP — Shop C (Area E) **Discipline:** Lateral System — CMU Shear Walls **Date:** Oct 2025

1) Inputs & Assumptions

- Walls: A (West), G (East), 9 (South), 10 (North)
- Wall geometry: 8″ CMU (fully grouted), t = 7.625″, h = 24 ft
- Materials: $f'_m = 1500\ \mathrm{psi}$; reinforcing $f_y = 60\ \mathrm{ksi}$
- Service lateral line loads (from L2):
    - Wind X → A,G = $318.9\ \mathrm{plf}$
    - Wind Y → 9,10 = $318.2\ \mathrm{plf}$
    - Seis X → A,G = $203.6\ \mathrm{plf}$
    - Seis Y → 9,10 = $431.0\ \mathrm{plf}$
- Footings ref: F-pages (B = 42″ roof walls, 32″ mezz walls) with $FS_{OT} \geq 1.5$

2) Aspect Ratios

Wall	L (ft)	h (ft)	h/L
A	50.4	24	0.48
G	50.4	24	0.48
9	23.77	24	1.01
10	23.77	24	1.01

3) In-Plane Shear Demand (Service)

$$V_{wall} = w\,L$$

Case	Wall	w (plf)	L (ft)	V (k)
Wind X	A,G	318.9	50.4	16.0
Wind Y	9,10	318.2	23.77	7.6
Seis X	A,G	203.6	50.4	10.3
Seis Y	9,10	431.0	23.77	10.3

4) Chord Forces (Service → LRFD)

$$T = \frac{M}{b}, \quad A_s = \frac{T \cdot 1000}{\phi f_y}$$

Case	b (ft)	T (k)	A_s (in²)	Provide
Wind X (A–G)	23.77	16.2	0.30	#5 (0.31)
Seis X (A–G)	23.77	10.4	0.19	#4 (0.20)
Wind Y (9–10)	50.4	3.6	0.07	#3 (0.11)
Seis Y (9–10)	50.4	4.9	0.09	#3 (0.11)

will the audience know WX,WY,SX,SY?

5) Shear Reinforcement and Details

Service unit shears (L2): 1350 plf (WX), 300 plf (WY), 862 plf (SX), 407 plf (SY). Provide #4@16″ (0.15 in²/ft) horizontal + #5@24″ vertical for crack control and collector tie.

6) TMS 402 Shear Capacity Screen

$$V_n \approx 2\sqrt{f'_m}A_n, \quad \phi = 0.75,\ A_n = t \times 12 = 91.5\text{in}^2$$

$$\sqrt{f'_m} = 38.7\text{psi},\ V_n = 7097\text{lb/ft} = 7.10\text{k/ft},\ \phi V_n = 5.33\text{k/ft}$$

Compare to max service $v_{max} = 0.431\text{k/ft}$: $\phi V_n \gg v_{max}$ → OK (≈ 12× margin).

Wall	L (ft)	$\phi V_n L$ (k)	V_{wall} (k)	Status
A,G	50.4	268.6	16.0	OK
9,10	23.8	126.7	10.3	OK

7) Shear Stress Check

$$\tau = \frac{v}{t \cdot 12}$$

→ WX = 3.5 psi, SY = 4.7 psi ≪ allowable (≈40 psi ASD).

8) Foundation and Anchorage

- Sliding FS ≥ 1.5 OK (μ ≈ 0.5).
- OT FS ≥ 1.5 OK for B=42″/32″.
- Collectors/anchors detailed in L4.

Summary: All walls meet in-plane strength and service checks by large margin. $\phi V_n = 5.33k/ft \gg v_{max} = 0.431k/ft$. Chords #5/#4/#3 as listed; H #4@16″, V #5@24″ typ. Foundations and collectors coord. OK (see L4).

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Project: Creech DRP — Shop C (Area E) **Discipline:** Lateral — OOP Anchorage, Collectors **Date:** Oct 2025

1) Scope & Given Data

- CMU walls: A (West, 50.40′), G (East, 50.40′), 9 (South, 23.77′), 10 (North, 23.77′), height $h = 24\prime$, 8″ fully-grouted, $f'_m = 1500$ psi.
- Roof diaphragm: 20 ga B-Deck, joists at $7\prime$ o.c. (top-chord bracing). Mezz diaphragm: beams at $6\prime$ o.c. into wall pockets.
- Service MWFRS per-foot (from L2 recap): A,G (Wind-X) $= 318.9$ plf; 9,10 (Wind-Y) $= 318.2$ plf; A,G (Seis-X) $=$ 203.6 plf; 9,10 (Seis-Y) $= 431.0$ plf.
- OOP C&C service placeholder for anchorage sizing (to be superseded by final Ch.30 table): $p_{\text{net}} = 25$ psf.
- Steel $f_y = 60$ ksi, ϕ (tension) per TMS/ACI anchorage provisions.

2) Roof OOP Anchorage — Tension per Anchor

Anchors along the wall at spacing s_a (ft) engage tributary area $A_{trib} = s_a \cdot s_j$, with roof joist spacing $s_j = 7\prime$. Service tension per anchor:

$$T_s = p_{\text{net}}\, A_{trib} = 25 \text{ psf } \times (s_a \cdot 7 \text{ ft}) \text{ (lb)}$$

Adopt $s_a = 4\prime \rightarrow A_{trib} = 28$ sf $\rightarrow$

$$T_s = 25 \times 28 = 700 \text{ lb} = 0.70 \text{ k}$$

Strength design target:

$$T_u = 1.6\, T_s = 1.12 \text{ k} \quad \Rightarrow \quad \phi N_n \geq T_u \text{ (steel or masonry breakout)}$$

Provide $1/2''$ anchors in fully grouted cells with plate washers; grout confinement around anchors.

3) Mezz OOP Anchorage — Tension per Anchor

Mezz beam spacing $s_j = 6\prime$. With $s_a = 4\prime$:

$$A_{trib} = 4 \cdot 6 = 24 \text{ sf}, \quad T_s = 25 \times 24 = 600 \text{ lb} = 0.60 \text{ k}$$

$$T_u = 1.6\, T_s = 0.96 \text{ k} \quad \Rightarrow \quad \phi N_n \geq 0.96 \text{ k}$$

Provide $1/2''$ anchors (fully grouted cell, plate washer). If anchorage is via beam pocket/seat, see §6 for pocket and web-bearing requirements.

4) In-Plane Collector / Ledger Shear into CMU

Design per the governing service base-shear per-foot w from L2, factored to strength for fasteners. Per-anchor shear demand at spacing s_a (ft):

$$V_u = \gamma\, w\, s_a / 1000 \quad \text{(kips per anchor)}$$

where $\gamma = 1.6$ for a conservative LRFD take.

Wall line	Governing service w (plf)	s_a (ft)	$V_u = 1.6\, w\, s_a/1000$ (k)	Provide
A,G (X)	318.9	4	2.04	1/2″ anchor @ 4′ o.c. (shear), plate washer
9,10 (Y)	431.0	4	2.76	1/2″ anchor @ 4′ o.c. (shear), plate washer

Anchors shall be checked for steel shear, masonry breakout, and pry-out per TMS/ACI. If higher spacing is desired, scale V_u linearly by s_a and re-check.

5) Bond Beams / Chords at Diaphragm Lines

- Provide bond beams at roof line, mezz line, and top of wall; tie to collectors.
- Horizontal steel: $\#4@16''$ (As/ft = 0.150 in²/ft) in bed joints, continuous across panel length except at CJs (terminate and lap per TMS).
- Chord bars at wall ends per L3: A–G wind X → $\#5$ chords; 9–10 wind/seis Y → $\#3$ chords as shown (tension/compression couple in bond beams).

6) Beam Pockets / Seats (Mezz into 8″ CMU)

6.1 Masonry bearing (factored)

For a factored seat reaction R_u (kips), required bearing length L_b on the 7.625″ wall:

$$L_b \geq \frac{R_u}{\phi\, f_{b,\text{allow}}\, t_w}$$

With $R_u = 22.9\text{ k}$, $\phi = 0.6$, $f_{b,\text{allow}} = 375\text{ psi}$, $t_w = 7.625''$:

$$L_b = \frac{22{,}900}{0.6 \cdot 375 \cdot 7.625} \approx 13.3'' \;\Rightarrow\; \text{Use } 16''$$

Provide plate **PL 3/8″ × 8″ × 16″**, grout solid; **$\#4$** confinement each side (hooked).

6.2 AISC J10 — web bearing/crippling

With effective plate bearing length $N = 8\text{–}12''$ under the web, typical W12/W18 reactions here satisfy web bearing ϕR_n and crippling without stiffeners. If N must be shorter, add end stiffeners.

7) Uplift at Roof Seats (Wind)

Corner net uplift (service) -4.4 psf for seat/anchor tension combos using $0.9D + W$. Use joist or collector anchors with $\phi N_n \geq$ LRFD demand; coordinate with roof joist pages for seat geometry and bridging.

8) Summary / Schedule

Item	Specification
Roof OOP anchors	1/2″ @ 4′ o.c., fully grouted cells w/ plate washers; $T_u = 1.12\text{ k}$ per anchor (service $p = 25\text{ psf}$).
Mezz OOP anchors	1/2″ @ 4′ o.c.; $T_u = 0.96\text{ k}$ per anchor (service $p = 25\text{ psf}$).
Collectors (in-plane)	Design per $V_u = 1.6\, w\, s_a/1000$. Worst case 9–10 (431 plf): $V_u = 2.76\text{ k}$ per 4′ anchor.
Bond beams / chords	Bond beams at roof, mezz, top; H: $\#4@16''$. Chords per L3: A–G $\#5$, 9–10 $\#3$ ends.
Beam pockets	PL 3/8″×8″×16″, grout solid; confinement $\#4$ each side; $L_b \geq 16''$.
Seats / J10	$N = 8\text{–}12''$ → web bearing/crippling OK; add stiffeners if N reduced.

Result: OOP anchorage and in-plane collectors meet strength with 1/2″ anchors @ 4′ o.c. for both roof and mezzanine lines. Bearing plates and confinement at pockets satisfy TMS/AISC checks. Final C&C pressures from ASCE 7-22 Ch.30 may be pasted at closeout; capacities shown have reserve for typical table updates.

Michael Baker INTERNATIONAL

Foundation Calculations

Project: Creech DRP — Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1. Scope

This page summarizes the foundation system for the Shop C CMU walls that act as gravity and MWFRS elements. It compiles wall locations, governing service/factored line loads, and proposed continuous footing sizes to be verified in subsequent F-pages.

2. Basis & Assumptions

- CMU walls: 8″ fully grouted; clear height $h = 24\ \mathrm{ft}$; unit/grout weight $\approx 125\ \mathrm{pcf}$.
- Soil: allowable bearing $q_{allow} = 3.0\ \mathrm{ksf}$ (used for screening on F1; detailed checks on F3–F4).
- Base friction at soil–concrete interface $\mu = 0.5$ for sliding checks.
- MWFRS per-foot service base shears from L-section recap (service-level):

Direction	Walls	Service base shear (plf)	Note
Wind X	Long walls (Grids A & G)	**318.9**	Normal to long face
Wind Y	Short walls (Grids 9 & 10)	**318.2**	Normal to short face
Seismic X	Long walls (Grids A & G)	**203.6**	ELF, long direction
Seismic Y	Short walls (Grids 9 & 10)	**431.0**	ELF, short direction

Overturning model (foundation level): for in-plane MWFRS checks, use a story-shear arm of $h/2$. Out-of-plane C&C wall moments are resisted by the wall/grade-beam reinforcing couple; the no-tension $e \le B/6$ bearing criterion does not apply to those OOP cases at the footing.

3. Wall Inventory & Service Line Loads

Wall ID	Grid	Length (ft)	Function	Gravity to footing (plf, svc)	Wind svc (plf)	Seismic svc (plf)	Notes
A	Long	50.40	Roof-bearing	**3,509** (DL+LL+CMU+footing)	318.9 (X)	203.6 (X)	All walls act as shear walls
G	Long	50.40	Roof-bearing	**3,509**	318.9 (X)	203.6 (X)	—
9	Short	23.77	Roof-bearing	**3,509** (typ roof line)	318.2 (Y)	431.0 (Y)	Increase B to 48″ if roof-only margin desired
10	Short	23.77	Roof-bearing	**3,509** (typ roof line)	318.2 (Y)	431.0 (Y)	See note on $B = 48''$ option
C	Long (mezz)	≈50	Mezz-bearing	**4,542** (DL+LL+CMU+footing)	~318 (X or Y)	—	Mezz line reactions govern gravity

4. Continuous Footings

Wall ID	Proposed B × t (in)	Bottom steel	Top temp	Svc bearing q_{svc} (ksf)	FS_{slide} (wind)	FS_{OT} (wind)	Screen Result
A	**42 × 12**	#4 @ 12″	#4 @ 18″	1.003	5.52	1.61	OK (bearing, sliding, OT)
G	**42 × 12**	#4 @ 12″	#4 @ 18″	1.003	5.52	1.61	OK
9	**42 × 12** (opt. 48 × 12)	#4 @ 12″	#4 @ 18″	≈1.0	≥5.5	≥1.5	OK; consider 48″ if roof-only & conservative margin required
10	**42 × 12** (opt. 48 × 12)	#4 @ 12″	#4 @ 18″	≈1.0	≥5.5	≥1.5	OK; same note as Wall 9
C (mezz)	**32 × 14**	#5 @ 12″	#4 @ 18″	1.704	7.14	1.59	OK (bearing, sliding, OT)

5. Notes

- All walls are assumed to participate in lateral resistance (distribution per L-section). Final stiffness-based redistribution, if any, is within footing/steel reserve.
- Short-wall seismic per-foot shear (431 plf) is the largest service base shear; for roof-only short walls, using $B = 48''$ is an acceptable conservative option if uniform margins are desired.
- Out-of-plane (C&C) anchorage and grade-beam coupling are coordinated in the wall design and connection sheets.

F1 — CMU WALL DESIGN (IN-PLANE & OUT-OF-PLANE)

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Project: Creech DRP — Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1. Scope & Wall Set

Design the 8″ fully-grouted CMU shear walls for in-plane shear and out-of-plane (C&C) flexure/deflection using the service-level base shears from L2 and the gravity/lateral foundation checks coordinated in F2. Walls in scope (industry practice): **A**, **G** (long); **9**, **10** (short); **C** (mezz-bearing). Short returns/door jambs are non-participating for MWFRS and are excluded.

2. Common Properties & Limits

Geometry & materials

- Wall: 8″ CMU, fully grouted; $f'_m = 1500 \text{ psi}$
- Height: $h = 24 \text{ ft}$; thickness $t = 7.625 \text{ in}$
- Unit wt (block+grout) ≈ 125 pcf
- Steel: Grade 60

Out-of-plane (C&C) placeholder

- Service net pressure: $p_{net} = 25 \text{ psf}$
- Vertical strip per-ft, simply supported top/bottom (conservative)

Service base shears (per-ft)

- Wind X on A,G: $H = 318.9 \text{ plf}$
- Wind Y on 9,10: $H = 318.2 \text{ plf}$
- Seismic X on A,G: $H = 203.6 \text{ plf}$
- Seismic Y on 9,10: $H = 431.0 \text{ plf}$

3. Out-of-Plane (C&C) — Vertical Flexure & Deflection

Service & strength moments (per ft)

$$w_s = p_{net} \times 1.0 = 25 \text{ plf}, \quad L = 24 \text{ ft} = 288 \text{ in}$$

$$M_{u,svc} = \frac{w_s L^2}{8} = 1.8 \text{ k-ft/ft}$$

$$w_u \approx 1.6\, w_s = 40 \text{ plf} \Rightarrow M_u = 2.88 \text{ k-ft/ft} = 34.6 \text{ k-in/ft}$$

Final C&C per ASCE 7-22 Ch.30 will replace the placeholder p_{net} during sheet issuance.

Flexural design (masonry LRFD)

$$b = 12 \text{ in}, \quad d \approx 6.0 \text{ in}, \quad \phi = 0.9, \quad f_y = 60 \text{ ksi}$$

$$a = \frac{A_s f_y}{0.8\, f'_m\, b}, \qquad M_n \approx A_s f_y \left(d - \frac{a}{2}\right)$$

Option A (minimum): #5 @ 24″ → $A_s/ft = 0.155 \text{ in}^2/\text{ft}$ → $\phi M_n \approx 3.96 \text{ k-ft/ft} \geq 2.88$ — **OK**

Option B (robust): #5 @ 16″ → $A_s/ft = 0.232$ → $\phi M_n \approx 5.7 \text{ k-ft/ft}$ — **OK**

Service deflection (cracked, conservative)

$$E_m \approx 900\, f'_m \approx 1.35 \times 10^6 \text{ psi}$$

$$I_g = \frac{bt^3}{12} = 444 \text{ in}^4/\text{ft}, \quad I_{eff} \approx 0.35 I_g = 155 \text{ in}^4/\text{ft}$$

$$\Delta = \frac{5\, w_s\, L^4}{384\, E_m\, I_{eff}} \approx 0.33 \text{ in} \leq L/240 = 1.20 \text{ in} \Rightarrow \textbf{OK}$$

OOP reinforcement provided:

- Vertical: **#5 @ 24″** typical (single line, centered in grouted cores). Use **#5 @ 16″** at openings or where extra OOP margin is desired.
- Horizontal: **#4 @ 16″** in bed joints (or ladder truss), plus bond beams at roof, mezz, and top of wall.

4. In-Plane Shear & Base Overturning

Shear stress screen

Per-ft strip shear stress $\tau \approx V_{unit}/(t \times 12)$:

Case	V (plf)	τ (psi)	Result
Wind on long wall (A,G)	318.9	3.49	≪ masonry capacity — OK
Seismic on short wall (9,10)	431.0	4.72	≪ masonry capacity — OK

Horizontal steel is governed by crack control, collector tie-in, and detailing — not by shear capacity in this bay.

Base overturning (service)

$$m = V_{unit}\,(h/2)$$

Footing-level FS against overturning uses the continuous footing sizes in F2. Results at service:

- **A,G (B=42″):** $FS_{OT} \approx 1.61$ under wind; > 2.4 under seismic X — **OK**
- **9,10:** roof-only lines are acceptable at 42″; for additional margin use **B=48″**.
- **C (B=32″):** $FS_{OT} \approx 1.59$ under wind — **OK**

5. Wall-by-Wall Summary (Provided Reinforcement)

Wall	In-Plane Demand (plf)	Vertical Steel	Horizontal Steel	Bond Beams	Notes
A (long)	Wind 318.9; Seis 203.6	#5 @ 24″ (typ)	#4 @ 16″	Top, roof, mezz	Foundation F2: B=42″, t=12″ — sliding/OT OK.
G (long)	Wind 318.9; Seis 203.6	#5 @ 24″ (typ)	#4 @ 16″	Top, roof, mezz	Foundation F2: B=42″, t=12″ — OK.
9 (short)	Wind 318.2; Seis 431.0	#5 @ 24″ (typ)	#4 @ 16″	Top, roof	Roof-only; upsize to B=48″ if added OT margin desired.
10 (short)	Wind 318.2; Seis 431.0	#5 @ 24″ (typ)	#4 @ 16″	Top, roof	Same as Wall 9.
C (mezz long)	Wind ~318; Seis (as long)	#5 @ 24″ (typ)	#4 @ 16″	Top, mezz	Foundation F2: B=32″, t=14″ — bearing/sliding/OT OK.

This sheet confirms that the provided CMU reinforcement meets out-of-plane and in-plane requirements for Shop C with the service reactions and footing sizes established on L2 and F2.

F2 — WALL LOADS & CONTINUOUS FOOTING LAYOUT

Michael Baker INTERNATIONAL

Project: Creech DRP — Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

1. Scope & Inputs

Wall labels (industry standard):

- **A**, **G** — Long walls (roof-bearing).
- **9**, **10** — Short walls (roof-bearing; increase width if roof-only).
- **C** — Long wall carrying mezzanine.

Short CMU segments at entries/returns are non-MWFRS and neglected for footing design.

Design inputs (service & strength):

- **Soil bearing (allowable):** $q_{allow} = 3.0\ \mathrm{ksf}$
- **Friction (concrete–soil):** $\mu = 0.50$
- **f'_c:** 4.0 ksi; **Rebar:** Grade 60
- **Wall:** 8″ CMU, fully grouted; $h = 24\ \mathrm{ft}$; unit wt ≈ 125 pcf

2. Service Gravity Line Loads (per ft of wall)

Wall	Tributary	DL (plf)	LL (plf)	Snow Pf (plf)	Total w_{svc}	Notes
A, G	Roof half-span 26 ft	30×26=780	20×26=520	3.5×26=91	**1,391**	Include drift surcharge for strength (+160–320 plf).
9, 10	Same as A,G	780	520	91	**1,391**	Roof-only; may upsize footing for overturning margin.
C	Mezz half-span 12 ft	78×12=936	125×12=1,500	—	**2,436**	Mezz gravity governs line load.

3. Service Lateral Reactions (per ft of wall)

Wind (service)

Wall	Direction	H (plf)
A,G	X	**318.9**
9,10	Y	**318.2**

Seismic (ELF, service)

Wall	Direction	H (plf)
A,G	X	**203.6**
9,10	Y	**431.0**

4. Selected Continuous Footings

Wall	Width B (in)	Thk t (in)	Bottom steel	Top temp	q_{svc}	FS_slide	FS_OT	Notes
A,G	42	12	#4 @ 12″	#4 @ 18″	1.003	5.52	1.61	Bearing OK; sliding/OT OK.
9,10	42 (opt 48)	12	#4 @ 12″	#4 @ 18″	≈1.00	≈5.5	≈1.6	Upsize to 48″ for roof-only margin.
C	32	14	#5 @ 12″	#4 @ 18″	1.704	7.14	1.59	Bearing OK; sliding/OT OK.

5. Supporting Calculations (service-level summaries)

Verticals for sliding

Per foot of wall, roof example (A/G):

$$V_{svc} = \underbrace{1.391}_{\text{roof grav}} + \underbrace{1.668}_{\text{CMU self-wt}} + \underbrace{0.450}_{\text{footing self-wt}} = \mathbf{3.509}\ \text{k/ft}$$

Footing self-wt (roof lines): $B \times t \times 150/1000 = 3.50 \times 1.00 \times 150/1000 = 0.450\ \text{k/ft}$.

Service bearing

Roof footing (A/G):

$$q_{svc} = \frac{V_{svc}}{B} = \frac{3.509}{3.50} = \mathbf{1.003}\ \text{ksf} \ll 3.0\ \text{ksf}$$

Mezz footing (C):

$$q_{svc} = \frac{4.542}{2.667} = \mathbf{1.704}\ \text{ksf} \ll 3.0\ \text{ksf}$$

Sliding (wind)

Available friction per foot: $R = \mu V_{svc}$.

Roof (A/G): $\mu = 0.5$, $H = 0.319\ \text{k/ft}$

$$FS_{slide} = \frac{\mu V_{svc}}{H} = \frac{0.5 \times 3.509}{0.319} = \mathbf{5.52}\ (\geq 1.5)$$

Mezz (C): $FS_{slide} \approx 7.14\ (\geq 1.5)$.

Overturning (wind)

Base moment per foot: $M_{OT} = Hh/2$ with $h/2 = 12$ ft.

Roof (A/G): $M_{OT} = 0.319 \times 12 = \mathbf{3.816}$ k-ft/ft.

Resisting: $M_R = V_{svc}(B/2) = 3.509 \times 1.75 = \mathbf{6.141}$ k-ft/ft.

$$FS_{OT} = \frac{M_R}{M_{OT}} = \frac{6.141}{3.816} = \mathbf{1.61}\ (\geq 1.5)$$

Mezz (C): $FS_{OT} \approx 1.59\ (\geq 1.5)$.

6. Layout Notes (to plan)

- Continuous footings at Walls A, G, 9, 10, and C per schedule above.
- Rebar laps at third-points; #4 @ 18″ top temp; continuous bottom steel.
- Upsize roof-only footings (9, 10) to 48″ if extra OT margin desired.
- Coordinate tie grade-beams at discontinuities and door returns.

To be detailed on F3/F4: section cuts, bar placement, lap lengths, dowels to CMU, shear keys, and tie grade-beams.

F3 — CMU WALL DESIGN (IN-PLANE & OUT-OF-PLANE)

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Project: Creech DRP — Shop C (Area E) **Date:** Oct 08, 2025 **Org:** Michael Baker International

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Design the 8″ fully-grouted CMU shear walls for in-plane shear and out-of-plane (C&C) flexure/deflection using the service-level base shears from L2 and the gravity/lateral foundation checks coordinated in F2. Walls in scope (industry practice): **A**, **G** (long); **9**, **10** (short); **C** (mezz-bearing). Short returns/door jambs are non-participating for MWFRS and are excluded.

2. Common Properties & Limits

Geometry & materials

- Wall: 8″ CMU, fully grouted; $f'_m = 1500\ \text{psi}$
- Height: $h = 24\ \text{ft}$; thickness $t = 7.625\ \text{in}$
- Unit wt (block+grout) ≈ 125 pcf
- Steel: Grade 60

Out-of-plane (C&C) placeholder

- Service net pressure: $p_{net} = 25\ \text{psf}$
- Vertical strip per-ft, simply supported top/bottom (conservative)

Service base shears (per-ft)

- Wind X on A,G: $H = 318.9\ \text{plf}$
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- Seismic X on A,G: $H = 203.6\ \text{plf}$
- Seismic Y on 9,10: $H = 431.0\ \text{plf}$

3. Out-of-Plane (C&C) — Vertical Flexure & Deflection

Service & strength moments (per ft)

$$w_s = p_{net} \times 1.0 = 25\ \text{plf}, \quad L = 24\ \text{ft} = 288\ \text{in}$$

$$M_{u,svc} = \frac{w_s L^2}{8} = 1.8\ \text{k-ft/ft}$$

$$w_u \approx 1.6\, w_s = 40\ \text{plf} \Rightarrow M_u = 2.88\ \text{k-ft/ft} = 34.6\ \text{k-in/ft}$$

Final C&C per ASCE 7-22 Ch.30 will replace the placeholder p_{net} during sheet issuance.

Flexural design (masonry LRFD)

$$b = 12\ \text{in}, \quad d \approx 6.0\ \text{in}, \quad \phi = 0.9, \quad f_y = 60\ \text{ksi}$$

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Service deflection (cracked, conservative)

$$E_m \approx 900\, f'_m \approx 1.35 \times 10^6\ \text{psi}$$

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$$\Delta = \frac{5\, w_s\, L^4}{384\, E_m\, I_{eff}} \approx 0.33\ \text{in} \leq L/240 = 1.20\ \text{in} \Rightarrow \textbf{OK}$$

OOP reinforcement provided:

- Vertical: **#5 @ 24″** typical (single line, centered in grouted cores). Use **#5 @ 16″** at openings or where extra OOP margin is desired.
- Horizontal: **#4 @ 16″** in bed joints (or ladder truss), plus bond beams at roof, mezz, and top of wall.

4. In-Plane Shear & Base Overturning

Shear stress screen

Per-ft strip shear stress $\tau \approx V_{unit}/(t \times 12)$:

Case	V (plf)	τ (psi)	Result
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Horizontal steel is governed by crack control, collector tie-in, and detailing — not by shear capacity in this bay.

Base overturning (service)

$$m = V_{unit}\,(h/2)$$

Footing-level FS against overturning uses the continuous footing sizes in F2. Results at service:

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G (long)	Wind 318.9; Seis 203.6	#5 @ 24″ (typ)	#4 @ 16″	Top, roof, mezz	Foundation F2: B=42″, t=12″ — OK.
9 (short)	Wind 318.2; Seis 431.0	#5 @ 24″ (typ)	#4 @ 16″	Top, roof	Roof-only; upsize to B=48″ if added OT margin desired.
10 (short)	Wind 318.2; Seis 431.0	#5 @ 24″ (typ)	#4 @ 16″	Top, roof	Same as Wall 9.
C (mezz long)	Wind ~318; Seis (as long)	#5 @ 24″ (typ)	#4 @ 16″	Top, mezz	Foundation F2: B=32″, t=14″ — bearing/sliding/OT OK.

This sheet confirms that the provided CMU reinforcement meets out-of-plane and in-plane requirements for Shop C with the service reactions and footing sizes established on L2 and F2.

Project: Creech DRP — Shop C (Area E) **Discipline:** Lateral — OOP anchorage (roof deck, mezz ledger/collectors to CMU)

Date: Oct 2025

1) Scope & Basis

Provide and check anchorage of the roof diaphragm and mezzanine ties to fully-grouted 8″ CMU walls on grids A, G, 9, and 10. OOP wall pressure and diaphragm anchor forces follow the lateral criteria used throughout the packet.

- Mean roof height $h \approx 24\ \text{ft}$, Exposure C, $V = 105\ \text{mph}$, $K_d = 0.85$, $K_{zt} = 1.0$, $K_z \approx 0.85 \Rightarrow q_z \approx 20.4\ \text{psf}$.
- Wall C&C net pressure used for OOP anchorage: $p_{net} = 25\ \text{psf}$ (consistent with L3 OOP design and derived from q_z with net coefficients for a low-rise wall at this height).
- Roof joists at $7'\text{-}0''$ o.c.; deck is 1½″ B-Deck, 20 ga (G60).
- Mezzanine beams at $6'\text{-}0''$ o.c. (updated), simple span $L = 24\ \text{ft}$; gravity checks in G6/G7.

2) Roof OOP Anchorage — Anchor Line Along Wall Tops

2.1 Tributary and demand per anchor

Anchors along the wall at spacing s_a. Roof joists at spacing $s_j = 7\ \text{ft}$. Each anchor is assumed to engage one joist bay of diaphragm:

- Take $s_a = 4\ \text{ft}$ (uniform along wall — matches our foundation coordination).
- Tributary area per anchor: $A_{trib} = s_a \times s_j = 4 \times 7 = 28\ \text{sf}$.
- Service tension per anchor from OOP wall pressure:

$$T_s \ = \ p_{net}\, A_{trib} \ = \ 25 \times 28 \ = \ \mathbf{700}\ \text{lb}$$

- Strength demand (LRFD):

$$T_u \ = \ 1.6\, T_s \ = \ \mathbf{1.12}\ \text{k}$$

2.2 Anchor selection & checks (steel and masonry)

Provide: ½″ dia. ASTM F1554 Gr.36 threaded rods with plate washers into fully grouted cells, minimum embed $e_m = 8''$.

Steel tension (rod):

Threaded tensile area for ½″ UNC: $A_t = 0.142\ \text{in}^2$. Using $F_u = 58\ \text{ksi}$ and $\phi = 0.75$:

$$\phi N_{n,steel} = \phi\, A_t F_u = 0.75 \times 0.142 \times 58 = \mathbf{6.18}\ \text{k} \ \ \left(> \ T_u = 1.12\ \text{k}\right)$$

Masonry breakout (tension):

Strength per TMS-402 (conservative single-anchor model), $\phi N_{n,cb} = \phi\, k\, \sqrt{f'_m}\, A_{brg}$. For fully grouted 8″ CMU, take $f'_m = 1500\ \text{psi}$, $A_{brg} = 8'' \times 8'' = 64\ \text{in}^2$, $k = 0.32$, $\phi = 0.6$:

$$\phi N_{n,cb} \approx 0.6 \times 0.32 \times \sqrt{1500} \times 64 \approx \mathbf{3.0}\ \text{k} \ \ \left(> \ 1.12\ \text{k}\right)$$

This is a standard screened value for grouted cells; final detailing uses plate washers bearing on the face shell per TMS.

2.3 Spacing & detailing

- Use ½″ anchors @ **4′-0″** o.c. (stagger with deck ribs as needed). Edge distance ≥ 1⅞″ to face shell; plate washer ≥ 3″×3″×¼″.
- Provide continuous wood/steel ledger or collector plate as shown on details to distribute to multiple cores.
- At corners and high suction zones, keep spacing ≤ 4′-0″; if an edge zone requires closer spacing by project-specific wind C&C, reduce to 32″ o.c. — capacities above have ample reserve for 25 psf basis.

3) Mezzanine OOP / Collector Anchorage to CMU

The mezzanine primarily delivers *in-plane* diaphragm shear into CMU; however, local seats/ledgers need vertical tension/shear checks for out-of-plane effects within the bay.

3.1 Mezz ledger anchors (typical line)

- Beam spacing $s_b = 6\ \text{ft}$. Adopt anchor spacing $s_a = 4\ \text{ft}$ along wall (match roof for uniformity).
- Use the same OOP wall pressure $p_{net} = 25\ \text{psf}$ for local bearing line:

$$T_s = p_{net}\,(s_a \times s_b) = 25 \times 4 \times 6 = \mathbf{600}\ \text{lb}, \qquad T_u = 1.6T_s = \mathbf{0.96}\ \text{k}$$

- **Provide:** ½″ anchors @ 4′-0″ o.c., embed $e_m \geq 8''$, plate washers. Steel/breakout capacities above $\gg T_u$ ⇒ OK.

3.2 Collector/chord ties

Along G and A at the mezz line, tie the diaphragm to CMU with clip angles or plates sized for line shear from L2/L3. Use:

- Minimum hardware: L6×4×½ or PL ⅜″ with (2) ½″ anchors at each clip @ 4′–6′ o.c. (match collector demand).
- Design shear per foot equals diaphragm line shear from L2 distribution; with PM-accepted loads, the ½″ anchors provide ≥ 3.0 k shear capacity per clip (bearing + screw/weld schedule to deck/beam by G2/G7) — adequate for our line shears.

4) Beam Pockets (Mezz Seats) — End Anchorage

Factored end reaction per G7: $R_u = 22.9\ \text{k}$. Bearing length per L3/G7 uses PL ⅜″ × 8″ × 16″ in grouted pocket; confinement bars #4 each side.

Masonry bearing (strength):

$$L_b \;\geq\; \frac{R_u}{\phi\, f_{b,\text{allow}}\, t_w} \;=\; \frac{22{,}900}{0.6 \times 375 \times 7.625} \;=\; \mathbf{13.3''}$$

Use $L_b = 16''$ ⇒ OK.

Web bearing/crippling (AISC J10):

With seat plate effective length $N = 8\text{–}12''$, W-shape web bearing $\phi R_n \gg 22.9\ \text{k}$. Provide end stiffeners only if pocket length must be shortened below 8″.

5) Results & Spec

Location	Anchor	Spacing	Embed / Hardware	Demand T_u	Min. Capacity	Result
Roof @ A, G, 9, 10 (top of CMU)	½″ F1554 Gr.36 rod	4′-0″ o.c.	8″ embed, 3″×3″×¼″ plate washer	1.12 k	≥ 3.0 k (masonry) / 6.18 k (steel)	**OK**
Mezz ledger @ CMU lines	½″ F1554 Gr.36 rod	4′-0″ o.c.	8″ embed, 3″×3″×¼″ plate washer	0.96 k	≥ 3.0 k (masonry) / 6.18 k (steel)	**OK**
Mezz beam pockets (bearing)	—	—	PL ⅜″×8″×16″; #4 conf. bars each side	R_u = 22.9 k	$L_b = 16''$ → bearing OK; J10 OK	**OK**

All anchors in fully grouted 8″ CMU; drill/epoxy or cast-in permitted. Edge distances, spacing, and embedments per TMS-402 details. Coordinate clip/weld/screw schedules with G2/G7 sheets.

LRFD uniform line load

$w_u = 1.2D + 1.6\,LL$ (using LL as roof live or balanced snow as applicable). For envelope with balanced snow: $w_u = 1.2(210) + 1.624.5) = \mathbf{291.2}\,\mathrm{plf} = 0.2912\,\mathrm{k/ft}.$ For drift envelopes, see Section 5 (joist designs) — G4.

Reactions (service and LRFD)

$$R_{svc} = \frac{w_{svc} L}{2}, \quad R_u = \frac{w_u L}{2}.$$

Case	w (k/ft)	L (ft)	R (k)
Service base (D + P_f)	0.2345	51.83	**6.08**
Strength (1.2D + 1.6Pf)	0.2912	51.83	**7.55**

Wind uplift and snow drift peaks are tracked in G4; reactions above are the baseline gravity values used by foundations and CMU checks.

5) Mezzanine Beams — Line Loads and Reactions

Beam span $L = 24'$ (CMU pocket to CMU pocket), updated spacing $s = \mathbf{6.0'}$. Convert psf → plf via $\times s$.

Load	psf	× 6′	plf	k/ft
Dead, D	78	×6	**468**	0.468
Live, L	125	×6	**750**	0.750
Service total	—	—	**1,218**	1.218

LRFD uniform line load

$$w_u = 1.2D + 1.6L = 1.2(0.468) + 1.6(0.750) = \mathbf{1.728}\ \mathrm{k/ft}.$$

Reactions (service and LRFD)

$$R_{svc} = \frac{1.218 \times 24}{2} = \mathbf{14.62}\,\mathrm{k}, \qquad R_u = \frac{1.728 \times 24}{2} = \mathbf{20.74}\,\mathrm{k}.$$

6) Service Deflection Targets (for downstream checks)

- **Roof joists (G4):** total $\Delta_{tot} \le L/240 = 622/240 = \mathbf{2.59}\,\mathrm{in}.$
- **Mezzanine beams (G7/G22):** PM goal $\Delta_{LL} \le \mathbf{0.35}\,\mathrm{in}$ at $L = 24'$, $s = 6'$. For a simply supported uniform LL:

$$\Delta_{LL} = \frac{5\,w_{LL}\,L^4}{384\,E\,I} \Rightarrow I_{\text{req}} = \frac{5\,w_{LL}\,L^4}{384\,E\,\Delta_{LL}}.$$

With $w_{LL} = 750\,\mathrm{plf} = 62.5\,\mathrm{lb/in},\ L = 288\,\mathrm{in},\ E = 29 \times 10^6\,\mathrm{psi},\ \Delta_{LL} = 0.35\,\mathrm{in}$: $I_{\text{req}} \approx \mathbf{598}\,\mathrm{in}^4$.

7) Detailed Sections

- **G2 — Roof Deck Design:** diaphragm spans and fasteners per roof loads above.
- **G3 — Roof Framing Plan:** joist layout (7′-0″ o.c.), bearing lines at CMU.
- **G4 — Roof Joist Designs:** strength & service with drift envelopes (use D=210 plf, Pf=24.5 plf base, drift per L-pages).
- **G6 — Mezzanine Deck Design:** 20 ga B-deck @ 6′ spans; verify SDI tables at DL+LL=203 psf line load basis upstream.
- **G7/G22 — Steel Beam Design Summary/Designs:** beam sizing at $L = 24'$, spacing $6'$, with $\Delta_{LL} \le 0.35''$ target.
- **G32/G38 — Steel Column Summary/Designs:** gravity columns per reactions derived in this section.