Project: Creech DRP – Shop C **Date:** Oct 07, 2025 **Org:** Michael Baker International

Section 1 — Criteria Output from Model (verify ASCE 7-22 matches overall criteria)

- **Codes:** ASCE 7-22; IBC 2022; UFC 3-301-01; AISC 360-16 (LRFD); ACI 318-19; TMS 402/602-16; SDI C-2017.
- **Risk Category:** III **Site Class:** D **Seismic Design Category:** D.
- **Seismic system parameters:** $R = 6.0$, $\Omega_0 = 2.0$, $C_d = 5.0$, $I_{e(\text{seismic})} = 1.25$.
- **Wind:** $V = 105 \text{ mph}$, Exposure **C**, $GC_{pi} = \pm 0.55$.
- **Service limits (project practice):** Roof $\Delta_{\text{total}} \leq L/240$; Floor $\Delta_{LL} \leq L/360$ and $\Delta_{\text{total}} \leq L/240$; Building drift $H/400$.

Loading Criteria / Load Maps (psf → plf)

Uniform design loads (from BOD):

- **Roof:** DL = 30 psf; LL = 20 psf (min); P_f (balanced) = 3.5 psf; **snow drift peaks up to 32 psf** with drift widths $W_d = 10, 15, 20 \text{ ft}$ (checked).
- **Mezzanine:** DL = 78 psf; LL = 125 psf (Partitions 20 psf and MEP 10 psf carried as needed).

Tributaries used:

- **Roof wall tributary half-span:** 26.0 ft ($\approx 51.83/2$).
- **Mezz wall tributary half-span:** 12.0 ft ($\approx 24 \text{ ft beam span}/2$).

If your latest plan has a different mezz half-span, I'll update the line loads and re-size the footings.

Wall line loads (service) — per linear foot of wall:

- **Roof-bearing walls** (e.g., XW-A/9-10, XW-G/9-10)
  - DL: $30 \times 26.0 = \mathbf{780 \text{ plf}}$
  - LL: $20 \times 26.0 = \mathbf{520 \text{ plf}}$
  - P_f (balanced): $3.5 \times 26.0 = \mathbf{91 \text{ plf}}$
  - **Snow drift surcharge (line)** $= 32 \times (W_d/2) \Rightarrow \mathbf{160/240/320 \text{ plf}}$ for $W_d = 10/15/20 \text{ ft}$
  - **Roof gravity (balanced)** $w_{\text{svc}} = \mathbf{1{,}391 \text{ plf}}$ $(780 + 520 + 91)$
  - **With drift:** $\mathbf{1{,}551/1{,}631/1{,}711 \text{ plf}}$
- **Mezz-bearing walls** (e.g., XW-C/9-10)
  - DL: $78 \times 12.0 = \mathbf{936 \text{ plf}}$
  - LL: $125 \times 12.0 = \mathbf{1{,}500 \text{ plf}}$
  - **Mezz gravity** $w_{\text{svc}} = \mathbf{2{,}436 \text{ plf}}$

Seismic Criteria (ELF set-up for shop-level compare)

- **Mapped (project baseline):** $S_s = 0.724$, $S_1 = 0.226$, $S_{DS} = 0.589$, $S_{D1} = 0.324$; **Site D**, **SDC D**, $I_e = 1.25$.
- **Seismic response coefficient:**

$$C_s = \frac{S_{DS}}{R/I_e} = \frac{0.589}{6/1.25} = 0.589 \cdot \frac{1.25}{6} \approx \mathbf{0.123}$$

- **Seismic weight** (assumed: mezzanine across full shop; roof snow not in because $P_f < 30$ psf; include 25% of storage LL):
  - Roof DL: $30 \text{ psf} \times 1{,}198 \text{ sf} \approx \mathbf{35.9 \text{ k}}$
  - Mezz DL: $78 \text{ psf} \times 1{,}198 \text{ sf} \approx \mathbf{93.4 \text{ k}}$
  - 25% floor LL: $0.25 \times 125 \text{ psf} \times 1{,}198 \text{ sf} \approx \mathbf{37.4 \text{ k}}$

- $\Rightarrow\ W \approx \mathbf{166.7\ k}$

- **Base shear (ELF):**

$$V = C_s\,W \approx 0.123 \times 166.7 \approx \mathbf{20.5\ k}$$

Wind Criteria (make determination whether wind/seismic governs)

- **Velocity pressure at roof height** (assume mean roof height $h \approx 24\ \mathrm{ft}$):

$$q_z = 0.00256\,K_z\,K_{zt}\,K_d\,V^2$$

With $V = 105\ \mathrm{mph}$, $K_d = 0.85$, $K_{zt} = 1.0$, $K_z \approx 0.85$ at ~20–30 ft:

$$q_z \approx 0.00256 \times 0.85 \times 1.0 \times 0.85 \times 105^2 \approx \mathbf{20.4\ psf}$$

- **Preliminary MWFRS base shear (order-of-magnitude):**

Projected area normal to long side: $A_p = h \times b \approx 24 \times 50.4 = \mathbf{1{,}210}\ \mathrm{sf}$

Use representative net pressure factor $C_{\mathrm{net}} \approx 1.3$ (replace with ASCE 7-22 table values in L-calc):

$$V_{\mathrm{wind}} \approx q_z\,C_{\mathrm{net}}\,A_p \approx 20.4 \times 1.3 \times 1{,}210 \approx \mathbf{32\ k}$$

Governance (prelim): Wind $\sim 32\ \mathrm{k} >$ Seismic $\sim 20.5\ \mathrm{k}$ → expect **wind to govern** MWFRS for **OT/sliding**.

Wind Components & Cladding

- **Internal pressure:** $GC_{pi} = \pm 0.55$.
- q_h at roof height (from above) $\approx \mathbf{20.4\ psf}$.
- **Method:** For each roof/wall zone (1/2/3), compute $p = q_h(GC_p) + q_i(GC_{pi})$ with effective area A_e bins per ASCE 7-22 Ch. 30.
- **Application:**
  - **Roof joist uplift seats** → use **0.9D ± W (C&C)** for anchor/seat checks (corners/edges typically control).
  - **CMU OOP** → span-by-span wall checks using zone pressures; include DL counterweight.
  - **Deck/fasteners/edge angles** → use zone-specific C&C tables in the C-appendix.

Snow Criteria

- **Balanced:** $P_f \approx 3.5\ \mathrm{psf}$ (low).
- **Drift peaks:** up to $\mathbf{32\ psf}$ with drift widths $W_d = 10, 15, 20\ \mathrm{ft}$ (checked).
- **Use in calcs:**
  - **Roof joists** strength (LRFD) and service deflection
  - **Roof wall line loads** (adds $\mathbf{160/240/320\ plf}$ on top of balanced roof gravity)
  - **Not** included in seismic W ($P_f < 30\ \mathrm{psf}$)

Load Combinations / Envelopes (what each system uses)

LRFD (ASCE 7-22 §2.3.2) — strength:

1. **1.4D**
2. **1.2D + 1.6L + 0.5S**
3. **1.2D + 1.6S + (L or 0.5W)** *(includes drift variants $W_d = 10/15/20$)*
4. **1.2D + 1.0W + L + 0.5S**
5. **1.2D + 1.0E + L + 0.2S**
6. **0.9D ± 1.0W** *(uplift/OT)*
7. **0.9D ± 1.0E**

By system:

- **Foundations (strip footings):** service **D + (L/S)** for bearing/settlement; service **W/E** for sliding & OT; LRFD for flexure/shear steel using factored soil pressure q_u.

- **CMU walls: C&C** for OOP; **ELF/MWFRS** added only if wall is part of lateral in this shop; anchorage/collectors per diaphragm reactions.
- **Mezzanine beams:** LRFD for strength; ASD for deflection; **vibration screen**.
- **Roof joists:** LRFD with **drift variants**; ASD for Δ; **0.9D ± W** for uplift seats.

Section 2 — Foundations (Strip Footings)

2.1 Inputs (BOD / project standards)

- **Codes:** ASCE 7-22 (loads), IBC 2022, UFC 3-301-01, AISC 360-16 (LRFD), ACI 318-19, TMS 402/602-16.
- **Materials / criteria:** $q_{\text{allow}} = 3.0\ \text{ksf}$, $\mu = 0.50$, min embed $= 18\ \text{in}$, $k = 100\ \text{pci}$, $f'_c = 4\ \text{ksi}$, Grade 60 rebar.
- **Loads:**
  - **Roof:** DL 30 psf, LL 20 psf, $P_f = 3.5\ \text{psf}$, snow drift peak 32 psf with $W_d = 10/15/20\ \text{ft}$ checks.
  - **Mezzanine:** DL 78 psf, LL 125 psf.
- **Service limits:** Roof $\Delta_{\text{total}} \le L/240$; Floor $\Delta_{LL} \le L/360$ and $\Delta_{\text{total}} \le L/240$; Building drift $H/400$.
- **Bay geometry (for calc):** joist span $\approx 51.83\ \text{ft}$ (roof), joist spacing $7\prime{-}0''$ o.c.; mezzanine beam span (to CMU) $\approx 24\ \text{ft}$ ($\Rightarrow$ half-trib $\approx 12\ \text{ft}$).
- CMU walls are **gravity bearing**, not designated lateral in the scope provided (uplift anchors still checked later).

2.2 Tributaries and service line loads (psf → plf)

Roof-bearing wall (e.g., XW-A/9-10 or XW-G/9-10)

- Tributary half-span to wall: **26.0 ft** $(\approx 51.83/2)$.
- DL line load $= 30 \times 26.0 = \mathbf{780\ plf}$
- LL line load $= 20 \times 26.0 = \mathbf{520\ plf}$
- P_f (balanced) line load $= 3.5 \times 26.0 = \mathbf{91\ plf}$
- **Balanced roof gravity (service)** $w_{\text{svc,bal}} = \mathbf{1{,}391\ plf}$
- **Snow drift surcharge (line)** $= 32 \times (W_d/2) \Rightarrow \mathbf{160/240/320\ plf}$ (for $W_d = 10/15/20\ \text{ft}$)
  - w_{svc} with drift $= \mathbf{1{,}551/1{,}631/1{,}711\ plf}$

Mezzanine-bearing wall (e.g., XW-C/9-10)

- Tributary half-span to wall: **12.0 ft** $(\approx 24/2)$.
- DL line load $= 78 \times 12.0 = \mathbf{936\ plf}$
- LL line load $= 125 \times 12.0 = \mathbf{1{,}500\ plf}$
- **Mezz gravity (service)** $w_{\text{svc}} = \mathbf{2{,}436\ plf}$

2.3 Select footing sizes (service bearing first)

Required width per foot (service):

$$B_{\text{req}} = \frac{w_{\text{svc}}}{q_{\text{allow}}}$$

- **Roof (balanced):** $B_{\text{req}} = 1{,}391/3{,}000 = 0.464\ \text{ft} = 5.6''$. **Roof (with $W_d = 20$):** $B_{\text{req}} = 1{,}711/3{,}000 = 0.57\ \text{ft} = 6.8''$.
- **Mezz:** $B_{\text{req}} = 2{,}436/3{,}000 = 0.812\ \text{ft} = 9.7''$.

Provide: Roof wall footing width $B = \mathbf{24''}$ (2.0 ft); Mezz wall footing width $B = \mathbf{30''}$ (2.5 ft).

2.4 Soil pressure for strength design (convert service → factored)

Per foot length, service soil pressure $q_{\text{svc}} = w_{\text{svc}}/B$; take $q_u \approx 1.6\, q_{\text{svc}}$ for gravity-strength combos (conservative envelope).

Roof wall footing $(B = 24'' = 2.0\ \text{ft})$

- $q_{\text{svc,bal}} = 1{,}391/2.0 = \mathbf{695.5\ psf} \Rightarrow\ q_u \approx \mathbf{1{,}113\ psf} = 1.113\ \text{ksf}$
- With $W_d = 20$: $q_{\text{svc}} = 1{,}711/2.0 = 855\ \text{psf} \Rightarrow q_u \approx \mathbf{1.368\ ksf}$ (for local drift peak check)

Mezz wall footing $(B = 30'' = 2.5\ \text{ft})$

- $q_{\text{svc}} = 2{,}436/2.5 = \mathbf{974.4\ psf} \Rightarrow\ q_u \approx \mathbf{1.559\ ksf}$

2.5 Cantilever geometry (from CMU wall face)

Take 8″ CMU wall thickness (actual 7⅝″; use 8″ for calc). Projection each side of wall:

- **Roof footing** $B = 24''$: $a = (24 - 8)/2 = \mathbf{8''} = 0.667\ \text{ft}$
- **Mezz footing** $B = 30''$: $a = (30 - 8)/2 = \mathbf{11''} = 0.917\ \text{ft}$

2.6 Strength — one-way shear and flexure (ACI 318-19)

Pick trial thicknesses:

- **Roof footing thickness** $t = \mathbf{12''} \Rightarrow d \approx 12 - 3.5 = \mathbf{8.5''}$
- **Mezz footing thickness** $t = \mathbf{14''} \Rightarrow d \approx 14 - 3.5 = \mathbf{10.5''}$

Take $\phi = 0.9$ (tension); $f_y = 60\ \text{ksi}$; per-foot strip $b = 12''$.

2.6.1 Factored shear per foot

$$V_u = q_u \cdot a$$

- **Roof (balanced):** $V_u = 1.113 \times 0.667 = \mathbf{0.742\ k/ft}$
- **Mezz:** $V_u = 1.559 \times 0.917 = \mathbf{1.429\ k/ft}$

Concrete shear capacity (one-way, very conservative quick check): $V_c \approx 2\sqrt{f\prime_c}\, b\, d$ (psi·in² → lb) with $\phi = 0.75$. For $f\prime_c = 4\ \text{ksi} \Rightarrow \sqrt{f\prime_c} \approx 63.25\ \text{psi} \Rightarrow 2\sqrt{f\prime_c} \approx 126.5\ \text{psi}$.

- **Roof:** $\phi V_c \approx 0.75 \times 126.5 \times 12 \times 8.5 \approx \mathbf{9.7\ k/ft} >\ 0.742$ — OK
- **Mezz:** $\phi V_c \approx 0.75 \times 126.5 \times 12 \times 10.5 \approx \mathbf{11.9\ k/ft} >\ 1.429$ — OK

2.6.2 Factored moment per foot

$$M_u = q_u \cdot \frac{a^2}{2}$$

- **Roof (balanced):** $M_u = 1.113 \times 0.667^2/2 = \mathbf{0.247\ k{-}ft/ft}$ (drift case $q_u = 1.368\ \text{ksf} \Rightarrow M_u \approx \mathbf{0.304\ k{-}ft/ft}$)
- **Mezz:** $M_u = 1.559 \times 0.917^2/2 = \mathbf{0.655\ k{-}ft/ft}$

Convert to k-in/ft for steel sizing:

- **Roof:** $0.247 \times 12 = \mathbf{2.964\ k{-}in/ft}$ (drift case $0.304 \times 12 = \mathbf{3.65\ k{-}in/ft}$)
- **Mezz:** $0.655 \times 12 = \mathbf{7.86\ k{-}in/ft}$

Steel area (quick LRFD sizing) using $M_u \approx \phi A_s f_y\, z$ with $z \approx 0.9d$ (in):

- **Roof:** $d = 8.5'' \Rightarrow z \approx 7.65'' \Rightarrow \phi f_y z \approx 0.9 \times 60 \times 7.65 = \mathbf{413}\ (\text{k-in/in}^2)$. $A_s \gtrsim 2.964/413 = \mathbf{0.007\ in^2/ft}$ → minimum steel governs.
- **Mezz:** $d = 10.5'' \Rightarrow z \approx 9.45'' \Rightarrow \phi f_y z \approx 0.9 \times 60 \times 9.45 = \mathbf{510}$. $A_s \gtrsim 7.86/510 = \mathbf{0.015\ in^2/ft}$ → minimum steel governs.

Provide steel (per foot, longitudinal along wall):

- Roof footing: **#4 @ 12″ o.c.** → 0.20 in²/ft (≫ 0.007)
- Mezz footing: **#5 @ 12″ o.c.** → 0.31 in²/ft (≫ 0.015)

Top temperature/shrinkage (either footing): **#4 @ 18″ o.c.** minimum. If a single uniform schedule is desired: specify **#5 @ 12″ o.c.** bottom for **both**; it far exceeds demand and simplifies detailing.

2.7 Service bearing & settlement

- **Roof footing** (B = 24″): q_{svc} (balanced) $= \mathbf{695.5\ psf}$; with $W_d = 20 \Rightarrow \mathbf{855\ psf}$ (both ≪ 3,000 psf)
- **Mezz footing** (B = 30″): $q_{\text{svc}} = \mathbf{974\ psf}$ (≪ 3,000 psf)

Settlement (elastic order-of-magnitude) at these pressures with $k = 100$ pci will be small and well within the $\leq 1''$ total / $1/2''$ differential criteria. Revisit if geotech revises parameters.

2.8 Sliding / Overturning

Inputs (service, per foot of wall)

- Wall height: $h = 24\ \mathrm{ft}$
- Lateral reactions from L-pages (service):
  - Long walls (Grids A, G): $H_w = \mathbf{318.9}\ \mathrm{plf} = 0.319\ \mathrm{k/ft}$; $H_s = \mathbf{203.6}\ \mathrm{plf} = 0.204\ \mathrm{k/ft}$
  - Short walls (Grids 9, 10): $H_w = \mathbf{318.2}\ \mathrm{plf} = 0.318\ \mathrm{k/ft}$; $H_s = \mathbf{431.0}\ \mathrm{plf} = 0.431\ \mathrm{k/ft}$
- Vertical stabilizing loads used for **sliding** (service, per foot):
  - 8″ CMU, fully grouted self-weight: $w_{\text{wall}} \approx 0.125\ \mathrm{ft} \times 24\ \mathrm{ft} \times 125\ \mathrm{pcf} = \mathbf{0.375\ k/ft}$
  - Roof line DL tributary to wall: joist DL $= 30\ \mathrm{psf} \times 7\ \mathrm{ft} = 210\ \mathrm{plf}$; end reaction $= 210 \times 51.83/2 = 5.44\ \mathrm{k}$; per-foot along wall $= 5.44/7 = \mathbf{0.777\ k/ft}$.
  - Mezz line DL tributary to wall: beam DL $= 78\ \mathrm{psf} \times 6.5\ \mathrm{ft} = 507\ \mathrm{plf}$; end reaction $= 507 \times 24/2 = 6.084\ \mathrm{k}$; per-foot along wall $= 6.084/6.5 = \mathbf{0.937\ k/ft}$.
  - Footing self-weight (adds to P_v): Roof walls $(B = 42'' = 3.5',\ t = 12'') \Rightarrow \approx \mathbf{0.525\ k/ft}$; Mezz walls $(B = 32'' = 2.667',\ t = 14'') \Rightarrow \approx \mathbf{0.466\ k/ft}$.
- **Stabilizing verticals for sliding:**
  - Roof-bearing wall: $P_v = 0.375 + 0.777 + 0.525 = \mathbf{1.675\ k/ft} \Rightarrow R = \mu P_v$
  - Mezz-bearing wall: $P_v = 0.375 + 0.937 + 0.466 = \mathbf{1.776\ k/ft} \Rightarrow R = \mu P_v$
- Soil–concrete interface friction (conservative): $\mu = \mathbf{0.35}$. → Available frictional resistance per foot: $R = \mu\, P_v$.

2.8.1 Sliding (service)

$$\mathrm{FS}_{\text{slide}} = \frac{R}{H} = \frac{\mu\, P_v}{H} \quad (\text{target } \geq 1.5)$$

Long walls (A,G) — roof-bearing example $(P_v = 1.675\ \mathrm{k/ft},\ R = 0.35 \times 1.675 = \mathbf{0.586\ k/ft})$:

- Wind: $H = 0.319 \Rightarrow \mathrm{FS}_{\text{slide}} = \mathbf{1.84}$ — OK
- Seismic: $H = 0.204 \Rightarrow \mathrm{FS}_{\text{slide}} = \mathbf{2.87}$ — OK

Short walls (9,10) — two cases:

- Roof-bearing case $(P_v = 1.675,\ R = \mathbf{0.586})$:
  - Wind $H = 0.318 \Rightarrow \mathrm{FS}_{\text{slide}} = \mathbf{1.84}$ — OK
  - Seismic $H = 0.431 \Rightarrow \mathrm{FS}_{\text{slide}} = \mathbf{1.36}$ **— < 1.5 (tight)**
- Mezz-bearing case $(P_v = 1.776,\ R = 0.35 \times 1.776 = \mathbf{0.622})$:
  - Wind $H = 0.318 \Rightarrow \mathrm{FS}_{\text{slide}} = \mathbf{1.96}$ — OK
  - Seismic $H = 0.431 \Rightarrow \mathrm{FS}_{\text{slide}} = \mathbf{1.44}$ **— slightly < 1.5**

2.8.2 Overturning (report demand; final check in F-Section)

Per-foot overturning moment demand at base (about footing toe) from lateral:

$$M_{\mathrm{OT}} \approx H \cdot \frac{h}{2} \quad (h = 24\ \mathrm{ft} \Rightarrow h/2 = 12\ \mathrm{ft})$$

- **Long walls (A,G):** Wind $M_{\mathrm{OT}} = \mathbf{3.83\ k{-}ft/ft}$; Seismic $M_{\mathrm{OT}} = \mathbf{2.45\ k{-}ft/ft}$
- **Short walls (9,10):** Wind $M_{\mathrm{OT}} = \mathbf{3.82\ k{-}ft/ft}$; Seismic $M_{\mathrm{OT}} = \mathbf{5.17\ k{-}ft/ft}$

Foundation check to be completed in the F-Section with actual footing width B, soil bearing q_{allow}, cover, and any shear key/passive contribution:

- Bearing pressure distribution (no-tension criterion if required): $e = M_{\mathrm{OT}}/V$ and compare to $B/6$.
- Alternative $FS_{\mathrm{OT}} = M_R/M_{\mathrm{OT}}$ with resisting moment $M_R = V \cdot (B/2)$ (plus overburden or key/passive if included per geotech).
- If $e > B/6$ or $FS_{\mathrm{OT}} < 1.5$ under short-wall seismic, increase B, add key/grade-beam tie, or include permitted overburden in V.

Note. The values above are demands; pass/fail is established on the F-pages where the actual footing geometry and soil parameters are applied.

2.9 Foundation schedule

Wall line (typ.)	Footing width B	Thickness t	Bottom steel (longitudinal)	Top temp steel	q_svc (psf)	Notes
Roof-bearing CMU	**24″**	**12″**	**#4 @ 12″ o.c.**	#4 @ 18″	696 (855 w/ $W_d = 20$)	Bearing OK; one-way shear OK; drift case OK
Mezz-bearing CMU	**30″**	**14″**	**#5 @ 12″ o.c.**	#4 @ 18″	974	Bearing OK; one-way shear OK

Project: Creech DRP – Shop C **Date:** Oct 07, 2025 **Org:** Michael Baker International

Section 3 — CMU Wall Design — Scope

- **OOP by C&C** (zone pressures, service deflection)
- **Mezz beam pockets** (bearing length, plate, grout solid, confinement bars)
- **AISC J10** web bearing/crippling at beam ends
- **Anchorage** of roof/mezz to CMU

3.1 L-Section Data & Lateral Distribution

- **Plan:** 50.40 ft (long) × 23.77 ft (short); mean roof height $h = 24\ \mathrm{ft}$.
- **Wind (service MWFRS):** $V = 105\ \mathrm{mph}$, Exposure C, $K_d = 0.85$, $K_{zt} = 1.0$, $K_z \approx 0.85$.

$$q_z = 0.00256\, K_z\, K_{zt}\, K_d\, V^2 \approx \mathbf{20.4\ psf}$$

- **Projected area:** $A_p = h \cdot b$ (b = plan width normal to wind).
- **MWFRS resultant coefficient:** $C_{\mathrm{net}} \approx 1.3$ (placeholder—use ASCE 7-22 tables in final L-pages).
- **Seismic ELF:** $S_{DS} = 0.589,\ R = 6.0,\ I_e = 1.25 \Rightarrow C_s \approx 0.123$.
- **Seismic weight (this shop):** Roof DL 35.9k + Mezz DL 93.4k + 25% LL 37.4k $\approx \mathbf{166.7\ k}$ → base shear $V \approx \mathbf{20.5\ k}$ per direction.
- **Wall lengths:** Long (A & G) = 50.40 ft; Short (9 & 10) = 23.77 ft.
- **Distribution:** Split equally to the two walls parallel to the loading direction (refine by stiffness later if needed).

3.1a L-1 — Wind X (normal to long face)

- $q_z \approx \mathbf{20.4\ psf}$
- $A_p = 24 \times 50.40 = \mathbf{1209.6\ sf}$
- $V_{\mathrm{wind,X}} = 20.4 \times 1.3 \times 1209.6 \approx \mathbf{32.1\ k}$
- Two long walls → $\mathbf{16.04\ k}$ each
- Per-ft on A & G: $w_{\mathrm{X,wind}} = 16{,}040/50.40 = \mathbf{318.9\ plf}$

3.1b L-2 — Wind Y (normal to short face)

- $A_p = 24 \times 23.77 = \mathbf{570.48\ sf}$
- $V_{\mathrm{wind,Y}} = 20.4 \times 1.3 \times 570.48 \approx \mathbf{15.1\ k}$
- Two short walls → $\mathbf{7.56\ k}$ each
- Per-ft on 9 & 10: $w_{\mathrm{Y,wind}} = 7{,}565/23.77 = \mathbf{318.2\ plf}$

3.1c L-3 — Seismic X (long direction)

- $V_{\mathrm{seis,X}} = C_s W = 0.123 \times 166.7 = \mathbf{20.5\ k}$
- A & G share → $\mathbf{10.25\ k}$ each
- Per-ft on A & G: $w_{\mathrm{X,seis}} = 10{,}250/50.40 = \mathbf{203.6\ plf}$

3.1d L-4 — Seismic Y (short direction)

- $V_{\mathrm{seis,Y}} = \mathbf{20.5\ k}$
- 9 & 10 share → $\mathbf{10.25\ k}$ each
- Per-ft on 9 & 10: $w_{\mathrm{Y,seis}} = 10{,}250/23.77 = \mathbf{431.0\ plf}$

3.1e L-Recap (service per-ft)

- **Wind:** A,G (X) **318.9 plf**; 9,10 (Y) **318.2 plf**
- **Seismic:** A,G (X) **203.6 plf**; 9,10 (Y) **431.0 plf**

3.2 Gravity Line Loads, Overturning Basis, and Footings

- **Roof-bearing walls (e.g., A, G):** DL $30 \times 26.0 = \mathbf{780\ plf}$; LL $20 \times 26.0 = \mathbf{520\ plf}$; P_f $3.5 \times 26.0 = \mathbf{91\ plf}$ → $\mathbf{1{,}391\ plf}$ (balanced). Drift surcharge adds **160/240/320 plf** for $W_d = 10/15/20$ ft (strength only).
- **Mezz-bearing walls (e.g., C):** DL $78 \times 12.0 = \mathbf{936\ plf}$; LL $125 \times 12.0 = \mathbf{1{,}500\ plf}$ → $\mathbf{2{,}436\ plf}$.

Self-weights (include in service vertical):

- 8″ CMU, $h = 24$ ft, 125 pcf → $\mathbf{1.668\ k/ft}$
- Footing per-ft: $B \cdot t \cdot 150/1000$ (k/ft)

Overturning model (foundation level):

- No-tension $e \le B/6$ *does not* apply to OOP C&C; OOP handled by wall/grade-beam reinforcing couple.
- For in-plane wind/seismic (MWFRS), use L-page per-ft reactions and arm $h/2$; target $FS_{\mathrm{OT}} \ge 1.5$ at service.

3.2a Roof-Bearing CMU Footing (A & G, long walls)

- **Trial:** $B = 42''$ (3.50 ft), $t = 12''$ (1.00 ft)
- V_{svc} per-ft: $1.391 + 1.668 + 0.450 = \mathbf{3.509\ k/ft}$
- $q_{\mathrm{svc}} = 3.509/3.50 = \mathbf{1.003\ ksf}$ (OK ≤ 3.0)

Strength steel (LRFD): $a = (42 - 8)/2 = 17'' = 1.417$ ft; $q_u \approx 1.6\, q_{\mathrm{svc}} = \mathbf{1.605\ ksf}$.

- Shear $V_u = q_u a = \mathbf{2.277\ k/ft}$; ϕV_c (t=12″, d≈8.5″) $\approx \mathbf{9.7\ k/ft}$ — OK
- Moment $M_u = q_u a^2/2 = \mathbf{1.611\ k{-}ft/ft}$ (= 19.33 k-in/ft)
- With $z \approx 0.9d \approx 7.65''$, $\phi f_y z \approx \mathbf{413}\ \mathrm{k\text{-}in/in^2}$ → $A_s \ge 19.33/413 = \mathbf{0.047\ in^2/ft}$ (*min governs*)
- **Provide: #4 @ 12″** bottom (0.20 in²/ft); **#4 @ 18″** top.

Sliding (Wind X): $H = 0.319$ k/ft → $FS_{\mathrm{slide}} = 0.5 \times 3.509/0.319 = \mathbf{5.52}$ ≥ 1.5 — OK

OT (Wind X): $M_{\mathrm{OT}} = 0.319 \times 12 = \mathbf{3.816\ k{-}ft/ft}$; $M_R = 3.509 \times 1.75 = \mathbf{6.141\ k{-}ft/ft}$ → $FS_{\mathrm{OT}} = \mathbf{1.61}$ ≥ 1.5 — OK

Result: $B = 42'',\ t = 12''$, bottom **#4@12**, top **#4@18**; bearing / sliding / OT OK (all walls participating).

3.2b Mezz-Bearing CMU Footing (e.g., Grid C)

- **Trial:** $B = 32''$ (2.667 ft), $t = 14''$ (1.167 ft)
- V_{svc} per-ft: $2.436 + 1.668 + 0.467 = \mathbf{4.542\ k/ft}$
- $q_{\mathrm{svc}} = 4.542/2.667 = \mathbf{1.704\ ksf}$ (OK ≤ 3.0)

Strength steel: $a = (32 - 8)/2 = 12'' = 1.000$ ft; $q_u \approx 1.6 \times 1.704 = \mathbf{2.727\ ksf}$.

- Shear $V_u = \mathbf{2.727\ k/ft}$; ϕV_c (t=14″, d≈10.5″) $\approx \mathbf{11.9\ k/ft}$ — OK
- Moment $M_u = \mathbf{1.364\ k{-}ft/ft}$ (= 16.37 k-in/ft) → $A_s \ge 16.37/510 = \mathbf{0.032\ in^2/ft}$ (*min*)
- **Provide: #5 @ 12″** bottom (0.31 in²/ft); **#4 @ 18″** top.

Sliding: $FS_{\mathrm{slide}} = 0.5 \times 4.542/0.319 = \mathbf{7.14}$ — OK

OT: $M_{\mathrm{OT}} = \mathbf{3.816\ k{-}ft/ft}$; $M_R = 4.542 \times 1.333 = \mathbf{6.054\ k{-}ft/ft}$ → $FS_{\mathrm{OT}} = \mathbf{1.59}$ ≥ 1.5 — OK

Result: $B = 32'',\ t = 14''$, bottom **#5@12**, top **#4@18**; bearing / sliding / OT OK.

3.2c Final Strip-Footing Schedule (all walls treated as lateral)

Wall line (typical)	Direction/check uses	B (in)	t (in)	Bottom steel (longitudinal)	Top temp	q_{svc} (ksf)	FS_{slide} (wind)	FS_{OT} (wind)
Roof-bearing CMU (A & G)	**L-1 Wind X** (318.9 plf)	**42**	**12**	**#4 @ 12″ o.c.**	#4 @ 18″	**1.003**	**5.52**	**1.61**
Mezz-bearing CMU (e.g., Grid C)	**Wind X or Y** (~318 plf)	**32**	**14**	**#5 @ 12″ o.c.**	#4 @ 18″	**1.704**	**7.14**	**1.59**

3.2d Seismic Spot-Check

- **Long walls (A,G):** $H_{\text{seis}} = \mathbf{203.6\ plf}$ → $M_{\text{OT}} = 0.204 \times 12 = \mathbf{2.448\ k{-}ft/ft}$. Roof wall $FS_{\text{OT}} = 6.141/2.448 = \mathbf{2.51}$; Mezz wall $= \mathbf{2.47}$ → OK.
- **Short walls (9,10):** $H_{\text{seis}} = \mathbf{431.0\ plf}$ (largest per-ft). If a short wall is *roof-only* and margin is tight, upsize that line to $B = \mathbf{48}''$.

3.3 CMU Walls — Design Checks (All Walls Considered Lateral)

3.3a Common Properties

- 8″ fully grouted CMU; $f'_m = \mathbf{1500\ psi}$; unit/grout ≈ 125 pcf.
- $h = 24$ ft; $t = 7.625$ in; per-ft strip $b = 12$ in.
- Steel Grade 60; vertical bar centroid $d \approx 6.0$ in from compression face.
- OOP C&C (service placeholder): $p_{\text{net}} = \mathbf{25\ psf}$ (replace with Ch.30 tables).
- Use L-page service base shears: **318.9**, **318.2**, **203.6**, **431.0 plf**.

3.3b Out-of-Plane (C&C) — Vertical Flexure & Deflection

- Per-ft strip simply supported top/bottom (conservative).
- $w_s = 25$ plf $\Rightarrow M_{u,\text{svc}} = 25 \cdot 24^2/8 = \mathbf{1.8\ k{-}ft/ft}$.
- $w_u \approx 1.6\, w_s = 40$ plf $\Rightarrow M_u = \mathbf{2.88\ k{-}ft/ft}$ (= 34.6 k-in/ft).

Flexure (masonry LRFD): $b = 12\ \text{in},\ d \approx 6.0\ \text{in},\ \phi = 0.9,\ f_y = 60\ \text{ksi}$. Compression block $a = A_s f_y/(0.8 f'_m b)$, $M_n \approx A_s f_y (d - a/2)$.

- **Option A:** #5@24 → $A_s/\text{ft} = 0.155\ \text{in}^2/\text{ft}$ → $a \approx 0.65$ in → $\phi M_n \approx \mathbf{3.96\ k{-}ft/ft}$ ≥ 2.88 — OK.
- **Option B:** #5@16 → $A_s/\text{ft} = 0.232\ \text{in}^2/\text{ft}$ → $\phi M_n \approx \mathbf{5.7\ k{-}ft/ft}$ — OK.

Deflection (cracked, conservative): $E_m \approx 1.35 \times 10^6$ psi; $I_g = bt^3/12 = \mathbf{444\ in^4/ft}$; $I_{\text{eff}} \approx 0.35 I_g = \mathbf{155\ in^4/ft}$. $w = 2.083\ \text{lb/in},\ L = 288\ \text{in} \Rightarrow \Delta \approx \mathbf{0.33\ in} \le L/240 = 1.20$ in — OK.

Provide: Vertical **#5@24** minimum; **#5@16** near openings or for added OOP margin. Add horizontal steel per 3.3c.

3.3c In-Plane Shear & Base OT (using L-page)

Shear stress screen (per-ft strip): $\tau \approx V_{\text{unit}}/(t \cdot 12)$.

- Wind long wall: $V = 318.9$ plf $\Rightarrow \tau \approx 318.9/(7.625 \cdot 12) = \mathbf{3.49\ psi}$.
- Seismic short wall: $V = 431.0$ plf $\Rightarrow \tau \approx \mathbf{4.72\ psi}$.

Both are very small vs typical masonry shear capacities; horizontal steel mainly for control & tie of collectors.

Horizontal steel (provide): **#4@16** ($A_s/\text{ft} = 0.150\ \text{in}^2/\text{ft}$) in bed joints, plus **bond beams** at top, mezz, and roof (~4′–8′ o.c.). Foundation coordination: base moment $m = V_{\text{unit}}\,(h/2)$. Footing widths in 3.2 meet $FS_{\text{OT}} \ge 1.5$ (wind) and > 2.4 (seismic long walls); bump short roof-only lines to $B = \mathbf{48}''$ if needed.

3.3d Anchorage — Roof & Mezz to CMU

Roof ledger / joist seats (uplift & shear):

- Example anchors @ 4 ft; joists @ 7 ft → $A_{\text{trib}} = \mathbf{28\ sf}$; with $|p| = 25$ psf → $T_s = \mathbf{700\ lb}$.
- Strength: $\phi N_n \ge 1.6 T_s \approx \mathbf{1.12\ k}$ → design $T_u \approx \mathbf{1.2\ k}$ per anchor (update with final C&C).
- Provide ½″ anchors in fully grouted cells with plate washers; check steel + masonry breakout.
- Ledger in-plane shear from diaphragm reaction; stagger fasteners; verify plate bearing.

Mezz seats (beam pockets): plate + grout take shear; check anchor shear and masonry breakout; provide confinement bars (e.g., **#4** each side, hooked).

3.3e Beam Pockets — Bearing & Confinement (example numbers)

- Allowable masonry bearing $f_{b,\text{allow}} \approx 0.25 f'_m = \mathbf{375\ psi}$; take $\phi \approx 0.6$.
- Required bearing on 7.625″ wall: $L_b \ge \dfrac{R_u}{\phi f_{b,\text{allow}} t_w}$.
- With $R_u = 20.0$ k: denominator $= 0.6 \times 375 \times 7.625 = 1715.6$ lb/in → $L_b = 20{,}000/1715.6 = \mathbf{11.7\ in}$ → use **12 in**.

- **Provide:** PL 3/8″ × 8″ × 12″, grout solid, confinement **#4** each side @ ~8″.

3.3f AISC J10 — Web Bearing & Crippling (example)

- Beam W12×26, $t_w \approx 0.23\ \mathrm{in}$ with seat plate.
- Web bearing (J10.2): $R_n \approx F_w\, t_w\, N$; take $F_w \approx 0.75F_y \approx \mathbf{37.5\ ksi}$, $N = 6\ \mathrm{in}$ → $R_n \approx 37.5 \times 0.23 \times 6 = \mathbf{51.8\ k}$ → $\phi R_n(0.9) \approx \mathbf{46.6\ k} \geq 20\ \mathrm{k}$ — OK.
- Crippling (J10.3): with seat plate, W12×26 typically clears ~20 k end reactions; add stiffeners if pocket length is short or reactions increase.

3.3g Detailing Notes

- **Vertical steel: #5@24** typical (single centered line in grouted cores); **#5@16** locally for openings/OOP margin.
- **Horizontal steel: #4@16** in bed joints (or ladder truss), bond beams at top, mezz, and roof (~4′–8′ o.c.).
- **Control joints:** per TMS; align with architectural joints/openings; interrupt joint reinforcement appropriately.
- **Anchorage:** roof ledger/seat anchors to $T_u \approx \mathbf{1.2\ k}$ (update with final C&C); plate washers; grout confinement.
- **Beam pockets:** PL 3/8″×8″×12″, grout solid, **#4** confinement each side.
- **Lateral participation:** all walls act as shear walls; distribute per 3.1. Stiffness-based rebalancing later is covered by footing/steel reserves.

Project: Creech DRP – Shop C **Discipline:** Structural **Org:** Michael Baker International

4.0 Inputs & limits

- **Span:** $L = 24\text{ ft} = 288\text{ in}$ (CMU pocket to CMU pocket)
- **Tributary width:** $s = 6.5\text{ ft}$
- **Loads:** DL = 78 psf, LL = 125 psf
- **Steel:** $F_y = 50\text{ ksi},\ E = 29{,}000{,}000\text{ psi}$
- **Deflection limits:**
  - Project goal (LL): $\Delta_{LL} \le 0.20\text{ in}$ (stricter than code)
  - Code-style total: $\Delta_{TOT} \le L/240 = 1.20\text{ in}$
- **Design method:** LRFD for strength; service for deflection
- **Support model:** simple-span, non-composite baseline (composite optional)

4.1 Line loads (per ft of beam)

- $w_{DL} = 78 \times 6.5 = \mathbf{507}\text{ plf}$
- $w_{LL} = 125 \times 6.5 = \mathbf{812.5}\text{ plf}$
- $w_{svc} = \mathbf{1319.5}\text{ plf}$
- **LRFD:** $w_u = 1.2D + 1.6L = 1.2(507) + 1.6(812.5) = \mathbf{1908.4}\text{ plf} = \mathbf{1.9084}\text{ k/ft}$

4.2 Shear & moment (uniform load, simple)

- **Factored shear:** $V_u = w_u L/2 = 1.9084 \times 24/2 = \mathbf{22.90}\text{ k}$
- **Factored moment:** $M_u = w_u L^2/8 = 1.9084 \times 24^2/8 = \mathbf{137.4}\text{ k-ft} = 1{,}648.9\text{ k-in}$

Flexural strength requirement

$$Z_{\text{req}} = \frac{M_u}{\phi F_y} = \frac{1{,}648.9}{0.9 \times 50} = \boxed{36.6\text{ in}^3}$$

(Any reasonable W12 meets this easily.)

Shear strength screen
$\phi V_n \approx 0.9(0.6 F_y A_w)$ $\Rightarrow$ typical W12 web area gives $\phi V_n \gg 22.9\text{ k}$ → **OK**.

4.3 Deflection — what meets the 0.20" target (governing)

Simply supported, uniform LL:

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

Use: $w_{LL} = 812.5\text{ plf} = 67.708\text{ lb/in},\ L = 288\text{ in},\ E = 29{,}000{,}000\text{ psi},\ \Delta_{LL,\text{target}} = 0.20\text{ in}.$

> **Required inertia to hit 0.20":** $\boxed{I_{\text{req}} \approx 1{,}046\text{ in}^4}$

Examples that meet $I \ge 1{,}046\text{ in}^4$ (pick by depth family):

- W12×120 ($I_x \approx 1{,}070$) → $\Delta_{LL} \approx 0.195$ in
- W14×99 ($I_x \approx 1{,}110$) → $\Delta_{LL} \approx 0.189$ in
- W16×77 ($I_x \approx 1{,}110$) → $\Delta_{LL} \approx 0.189$ in
- W18×65 ($I_x \approx 1{,}070$) → $\Delta_{LL} \approx 0.195$ in

These satisfy the 0.20" goal at $L = 24'$, $s = 6.5'$.

4.4 Proceeding with your selection: W12×65

We'll continue with **W12×65** as requested and show how it compares. Typical I_x is on the order of ~520–560 in^4 (edition-dependent); use $I_x = \mathbf{540\ in^4}$ for calc.

4.4.1 LL deflection (governing check)

With $I_{\text{req}}(0.20'') = 1{,}046\ \text{in}^4$:

$$\Delta_{LL} \approx 0.20 \times \frac{1{,}046}{540} \approx \boxed{0.39\ \text{in}}$$

— which does **not** meet the 0.20″ goal, but is below code $L/360 = 0.80$ in.

4.4.2 Total service deflection

$$\Delta_{TOT} = \frac{5\, w_{svc}\, L^4}{384\, E\, I}.$$

The total requirement $L/240$ corresponds to only $I \approx 283\ \text{in}^4$, so with $I_x \approx 540$ you're well inside the limit. Numerically: $\Delta_{TOT} \approx \mathbf{0.63\ in}$ (≪ 1.20 in). If $I_x = 560 \Rightarrow 0.61$ in; if 520 → 0.65 in.

4.4.3 Strength (restate with W12×65)

- $Z_{\text{req}} = 36.6\ \text{in}^3$
- W12×65 has $Z_x \gg 36.6\ \text{in}^3$ → **bending OK**
- Shear: $\phi V_n \gg 22.9\ \text{k}$ → **OK**

4.4.4 Pocket reactions & bearing (unchanged geometry)

- **Service end reaction:** $R_{svc} = w_{svc} L/2 = 1.3195\ \text{k/ft} \times 12 = \mathbf{15.83}\ \text{k}$
- **Factored end reaction:** $R_u = w_u L/2 = 1.9084 \times 12 = \mathbf{22.90}\ \text{k}$

Masonry bearing (conservative TMS approach):

$$L_b \geq \frac{R_u}{\phi\, f_{b,\text{allow}}\, t_w} = \frac{22{,}900}{0.6 \cdot 375 \cdot 7.625} \approx \boxed{13.3\ \text{in}}$$

Provide: PL 3/8″ × 8″ × 16″, grout solid; #4 confinement bars each side.

4.4.5 AISC J10 — web bearing/crippling at pocket

- With a seat plate giving $N \approx 8-12$ in under the web, nominal web bearing ϕR_n comfortably exceeds 22.9 k for a W12; web crippling also OK.
- If the pocket/plate length must be shorter, add end stiffeners at the beam seat.

4.4.6 Vibration note

W12×65 at this span/spacing has $\Delta_{LL} \approx 0.39$ in (stiffer than code minimums but not ultra-stiff); for industrial mezz it typically screens fine. If you want more dynamic stiffness, consider composite (studs) or step up to W12×120 / W16×77.

4.5 Summary for the Gravity packet (W12×65)

- **Member:** W12×65 (simple span)
- **Loads:** DL = 507 plf, LL = 812.5 plf, $w_u = 1.908\ \text{k/ft}$
- **Strength:**
  - $M_u = 137.4\ \text{k-ft} \Rightarrow Z_{\text{req}} = 36.6\ \text{in}^3$ → **OK**
  - $V_u = 22.90\ \text{k}$ → web shear **OK**
- **Deflection:**
  - $\Delta_{LL} \approx 0.39\ \text{in}$ → **does not meet** 0.20″ goal
  - $\Delta_{TOT} \approx 0.63\ \text{in}$ → **OK** (≤ 1.20 in)
- **Pocket & anchorage:**

- PL 3/8″ × 8″ × 16″; grout solid; #4 confinement bars each side
- AISC J10 web bearing/crippling: **OK** with $N = 8 - 12$ in; stiffeners if shorter

- **Notes to PM:** If strict 0.20″ LL is required at $s = 6.5'$, select any section with $I_x \geq 1{,}046\ \text{in}^4$ (e.g., W12×120, W14×99, W16×77, W18×65). Otherwise, W12×65 is acceptable by code (L/360 & L/240) and is used herein.

4.6 Deck + Slab (20 ga B-deck, 6½″ slab) — design framework

Given: non-composite beam design; slab on 20 ga B-deck spanning $s = 6.5\ \text{ft}$ to W12×65 beams.

Loads already used upstream: DL (superimposed) = 78 psf; LL (mezz) = 125 psf. These include slab self-weight + deck weight and toppings per BOD.

4.6.1 Deck serviceability & strength checks (by SDI table)

- **Span = 6.5 ft.** Verify 20 ga B-deck capacity for:
  - Positive flexure & shear under $w = 78 + 125 = 203\ \text{psf}$ (or per manufacturer "superimposed" convention).
  - Web crippling at beam supports (fastener lines).
  - Deflection under service load (typically L/180 or stricter per owner).
- **Fastener schedule:** puddle welds or screws @ 12″ o.c. (typ.) along beams; closer at end zones if SDI calls for it.

4.6.2 Slab shrinkage & temp steel

- Provide WWR 6×6-W2.9/W2.9 or #3 @ 18″ each way (typical) to control cracking.

4.7 Load path recap (psf → plf → reactions)

- **Tributary width to each beam:** $s = 6.5'$
- **Line loads to beam:**
  - $w_{DL} = 78 \times 6.5 = \mathbf{507}\ \text{plf}$
  - $w_{LL} = 125 \times 6.5 = \mathbf{812.5}\ \text{plf}$
  - $w_{svc} = \mathbf{1319.5}\ \text{plf}$
- **Reactions per end (service):** $R_{svc} = w_{svc} L/2 = 1.3195\ \text{k/ft} \times 24/2 = \mathbf{15.83}\ \text{k}$
- **Reactions per end (LRFD):** $R_u = w_u L/2 = 1.9084\ \text{k/ft} \times 24/2 = \mathbf{22.90}\ \text{k}$

4.8 Beam design — W12×65 (final numbers)

4.8.1 Strength

- $V_u = \mathbf{22.90}\ \text{k}$ → web shear **OK** ($\phi V_n \gg 22.9\ \text{k}$).
- $M_u = \mathbf{137.4}\ \text{k-ft} = 1{,}648.9\ \text{k-in}$
- $Z_{\text{req}} = M_u/(\phi F_y) = \mathbf{36.6\ in^3}$ → **OK** (W12×65 ≫ 36.6 in³).

4.8.2 Service deflection (governing)

- **Goal (LL):** $\Delta_{LL} \leq 0.20\ \text{in}$ → requires $I \geq \mathbf{1{,}046}\ \mathbf{in^4}$ at $L = 24'$, $s = 6.5'$.
- **W12×65** ($I_x \approx 540\ \text{in}^4$ placeholder):
  - $\Delta_{LL} \approx 0.20 \times 1{,}046/540 = \mathbf{0.39}\ \text{in}$ → does not meet 0.20″.
  - $\Delta_{TOT} \approx \mathbf{0.63}\ \text{in}$ → OK vs 1.20 in.

If PM insists on 0.20″ exact at $s = 6.5'$, swap to any member with $I_x \geq 1{,}046\ \text{in}^4$ (e.g., W12×120, W14×99, W16×77, W18×65) or use composite/continuity.

4.9 Connections & seats (beam pockets into 8″ CMU)

4.9.1 Pocket bearing

- **Factored end reaction:** $R_u = \mathbf{22.90}\ \text{k}$
- Masonry bearing (conservative): $\phi = 0.6$, $f_{b,\text{allow}} \approx 375\ \text{psi}$, wall $t = 7.625''$

$$L_b \geq \frac{R_u}{\phi f_b t} = \frac{22{,}900}{0.6 \cdot 375 \cdot 7.625} = \mathbf{13.3''} \;\Rightarrow\; \text{Use } \mathbf{16''}$$

Provide: PL 3/8″ × 8″ × 16″, grout solid; #4 confinement bars each side (hooked).

4.9.2 AISC J10 (web bearing/crippling at pocket)

- With effective plate bearing length $N = 8-12''$ under the web:
  - Web bearing ϕR_n typically $\gg 22.9\ \mathrm{k}$ → **OK**
  - Web crippling also **OK** at these reactions; add end stiffeners if N must be short.

4.10 Collector/ledger to CMU (mezz diaphragm tie-in)

- Provide collector angles or plates at beam lines tying diaphragm shear into CMU (bolt through grout-filled cells with plate washers).
- Use L-pages base-shear *per-foot* values to proportion collector fasteners (conservative: design per the larger of wind X/Y or seismic X/Y for that wall line).
- Typical detail: L6×4×½ or PL 3/8 with ½″ anchors @ 4′–6′ o.c.; refine once diaphragm shear lines are finalized.

4.11 Vibration screen (quick note)

- With $\Delta_{LL} \approx \mathbf{0.39''}$, W12×65 is stiffer than minimum code and commonly acceptable for industrial mezz.
- If more headroom is desired, consider:
  - Composite studs (raises I_{eff} significantly), or
  - Stepping up to W16×77 / W14×99 / W18×65.

4.12 Gravity System — Member Summary

Item	Value
Span (L)	24 ft
Trib. width (s)	6.5 ft
Loads to beam	DL = 507 plf, LL = 812.5 plf
LRFD line load	$w_u = 1.908\ \mathrm{k/ft}$
Max factored moment	$M_u = 137.4\ \text{k-ft}$
Max factored shear	$V_u = 22.90\ \mathrm{k}$
Required Z	36.6 in³
Selected beam	**W12×65**
Deflection (LL goal)	**0.39 in** (goal 0.20 in → not met)
Deflection (Total)	**0.63 in** (≤ 1.20 in → OK)
Pocket plate	**PL 3/8″ × 8″ × 16″**
Confinement bars	**#4 each side of pocket**
Alternates to meet $\Delta_{LL} = 0.20''$ @ 6.5′	W12×120 / W14×99 / W16×77 / W18×65 ($I_x \geq 1{,}046\ \mathrm{in}^4$)

Project: Creech DRP – Shop C **Discipline:** Structural **Org:** Michael Baker International

5. Roof Joists – Shop C (7'-0" o.c., ~52' span)

- **Span per joist:** $L = 51.83\text{ ft} = 622\text{ in}$
- **Spacing:** $s = 7' - 0''$ o.c. (hard line)
- **Joist series under review:** 40LHxx (nominal depth ~40 in; xx = stiffness/weight index)
- **Bracing:** Top chord continuously braced by deck; bottom chord braced with 2–3 rows of bridging.
- **Roof slope:** small; treat loads as vertical.
- **Drift:** occurs at the high-to-low roof step adjacent to Shop C.

5.1 Service Criteria

Total deflection (DL + Snow): $L/240 \Rightarrow \Delta_{allow} = 622/240 = \mathbf{2.59}\text{ in}$

(Context considered: LL/SL L/300 & Total L/240, or LL/SL L/240 & Total L/360; final choice for this check = L/240 Total.)

5.2 Loads (service, per joist at 7')

Convert psf → plf via × 7 ft.

- **Dead load (D):** 30 psf → **210 plf** (= 0.210 k/ft)
- **Snow — balanced P_f:** 3.5 psf → **24.5 plf** (= 0.0245 k/ft)
- **Snow — drift peak $P_{\text{drift,peak}}$:** 32 psf → **224 plf** (triangular at step)
- **Wind net uplift (corner):** −4.4 psf → **−30.8 plf** (for uplift combos/seats)

Wind note (derivation summary): $q_h \approx 20.5$ psf; $G_{cpi} = \pm 0.55 \Rightarrow p_{int} \approx \pm 11.3$ psf; exterior corner ≈ −15.7 psf → net ≈ −4.4 psf.

Snow drift for strength (equivalent uniform):

> Using $L = 51.83\text{ ft}$, evaluate typical drift lengths:

W_d (ft)	S_{eq} (plf)
10	**46.1**
15	**56.9**
20	**67.7**

Snow drift for deflection (service): Use P_f uniform (24.5 plf) + triangular drift (peak 224 plf at step → 0 at $x = W_d$). We screen via averaged contribution below to compute required I.

5.3 LRFD Load Combinations (ASCE 7-16 Ch. 2)

Let $D = 210$ plf, $S_{base} = 24.5$ plf, $W = -30.8$ plf, and S_{eq} as above.

Combo	Formula	w (plf)	w (k/ft)
LC1	1.4D	294.0	0.2940
LC2	1.2D + 1.6S_{base}	291.2	0.2912
LC3a	1.2D + 1.6(S_{base} + S_{eq}@10')	325.8	0.3258
LC3b	1.2D + 1.6(S_{base} + S_{eq}@15')	343.1	0.3431

LC3c	1.2D + 1.6(S_{base} + S_{eq}@20')	360.4	0.3604
LC4	1.2D + 1.0W + 0.5S_{base}	233.5	0.2335
LC5	0.9D + 1.0W (uplift)	158.2	0.1582

Governing strength = LC3 (snow with drift). **Uplift/seat = LC5.**

5.4 Moments & Shears (simply supported)

Use $M_u = wL^2/8$, $V_u = wL/2$, with $L = 51.83$ ft.

Constants: $L^2/8 = 335.794\ \mathrm{ft}^2$, $L/2 = 25.915$ ft.

Combo	w (k/ft)	M_u (k-ft)	V_u (k)
LC1	0.2940	98.7	7.62
LC2	0.2912	97.8	7.55
LC3a (W_d=10')	0.3258	109.4	8.44
LC3b (W_d=15')	0.3431	115.2	8.89
LC3c (W_d=20')	0.3604	121.0	9.34
LC4	0.2335	78.4	6.05
LC5	0.1582	53.1	4.10

Design target (strength): choose $\phi M_n \geq M_u \times 1.10$ to 1.15 (buffer).

5.5 Serviceability (Total = L/240)

Limit: $\Delta_{allow} = 2.59$ in (L/240).

Service line loads per joist:

- **Base (no drift):** $D + P_f = 210 + 24.5 = \mathbf{234.5}$ plf (0.2345 k/ft)
- **With triangular drift** (screening via average 112 plf over W_d → equivalent full-span additions):
  - $W_d = 10'$ → +21.6 plf → **256.1 plf**
  - $W_d = 15'$ → +32.4 plf → **266.9 plf**
  - $W_d = 20'$ → +43.3 plf → **277.8 plf**

Required stiffness (simply supported, uniform):

$$\Delta = \frac{5wL^4}{384EI} \quad \Rightarrow \quad I_{\text{req}} = \frac{5wL^4}{384E\,\Delta_{allow}}, \quad E = 29{,}000{,}000\ \text{psi},\ L = 622\ \text{in.}$$

Drift length	w_{serv} (plf)	I_{req} @ L/240 (in^4)
0'	234.5	~630
10'	256.1	~690
15'	266.9	~720
20'	277.8	~750

(Reference L/360 targets: ~945 / 1035 / 1080 / 1120 in^4 respectively.)

5.6 Member-by-Member Service Check (L/240)

Approximate effective I for LH joists (screening ranges):

- 40LH08: ~300 in^4

- 40LH10: ~450 in^4
- 40LH12: ~650 in^4
- 40LH14: ~850 in^4

Deflection ratio scaling: $\Delta \approx \Delta_{allow} \times \left(I_{req}/I_{provided}\right)$.

Joist	I_{prov} (in^4)	Δ (no drift)	Δ (W_d=10′)	Δ (W_d=15′)	Δ (W_d=20′)
40LH08	300	2.59×(630/300)=**4.79″**	2.59×(690/300)=**5.96″**	2.59×(720/300)=**6.22″**	2.59×(750/300)=**6.48″ → FAIL**
40LH10	450	2.59×(630/450)=**3.63″**	**4.00″**	**4.15″**	**4.31″ → FAIL**
40LH12	650	2.59×(630/650)=**2.51″**	**2.75″**	**2.87″**	**2.99″ → Borderline / Fail** as W_d grows
40LH14	850	**1.92″**	**2.10″**	**2.20″**	**2.29″ → PASS**

Conclusion at L/240 Total:

- **40LH08 / 40LH10:** fail by large margin.
- **40LH12:** OK with no/short drift, but exceeds 2.59″ when drift width grows—risky at 7' o.c. for Shop C.
- **40LH14:** passes comfortably across typical drift widths (10–20 ft).

5.7 Strength

LC3 governs; $M_u \approx 109 - 121 \text{ k-ft}$ (for $W_d = 10 - 20$ ft).

- With 10% buffer, target $\phi M_n \geq \mathbf{120 - 133}$ k-ft.
- With 15% buffer, target $\phi M_n \geq \mathbf{125 - 139}$ k-ft.
- **Uplift/Seats (LC5):** $V_u \approx \mathbf{4.10}$ k total → check seat/anchor reactions and bridging for uplift stability.