Ankapura Remodel – Calculations (partial remodel)
Mercer Island, Exp B

| Load Type | Parameter | Value | Notes |
|---|---|---|---|
| **Snow Load** | Ground Snow Load (Pg) | 25 psf | Per IRC Table R301.2(1) for Mercer Island; adjust for roof slope and drift. |
| | Rain-on-Snow Surcharge | 5 psf | Added for roof slopes < 5° (approx. 1:12 pitch). |
| | Roof Live Load (Lr) | 20 psf (minimum) | Per IRC R301.6; use greater of snow load or minimum roof live load. |
| **Wind Load** | Basic Wind Speed (V) | 110 mph (3-second gust) | Risk Category II (residential), per IRC Figure R301.2(4)A and ASCE 7-16. |
| | Exposure Category | B or C | B (suburban) typical; C (open terrain) near lake; site-specific per map. |
| | Topographic Factor (Kzt) | 1.0 (default) | Adjust per Mercer Island wind exposure map if site-specific effects apply. |
| **Earthquake Load** | Seismic Design Category (SDC) | D | Assumes Site Class D (stiff soil); confirm with geotechnical report. |
| | Spectral Response, Short (Ss) | 1.5g (150% of gravity) | MCE, per USGS maps for Puget Sound region; adjust for site class. |
| | Spectral Response, 1-sec (S1) | 0.6g | MCE, per USGS maps; adjust for site class. |
| | Site Class | D (default) | Stiff soil typical; A–F possible with geotechnical analysis. |

LRFS Changes the back wall, verify capacity of new wall with nanowall opening.

**Simplified Procedure for Low-Rise Buildings** (ASCE 7-16, Chapter 28, Part 2)

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

Where:

- $K_z$: Velocity pressure exposure coefficient, based on height and exposure.
- $K_{zt}$: Topographic factor = 1.0.
- $K_d$: Wind directionality factor = 0.85 (for buildings, per Table 26.6-1).
- $V$: Basic wind speed = 110 mph.

For Exposure B at a height of 6 ft (midpoint of the exposed 4–8 ft zone):

- From ASCE 7-16 Table 28.3-1 (simplified), $K_z$ is interpolated between 0.57 (at 15 ft) and ground level. For low heights (<15 ft), $K_z \approx 0.57$ is conservative for Exposure B.
- $q_z = 0.00256 \cdot 0.57 \cdot 1.0 \cdot 0.85 \cdot (110)^2$
- $q_z = 0.00256 \cdot 0.57 \cdot 0.85 \cdot 12,100$
- $q_z = 14.98\,\mathrm{psf}$ (round to 15 psf for simplicity).

**Internal Pressure Consideration**

Internal pressure (±GC_pi) must be considered per ASCE 7-16 Section 28.3.2:

- For an enclosed building (typical house), $GC_pi = \pm 0.18$ (Table 26.13-1).
- Combined pressure:
  - Positive (outward): $p = q_z \cdot (GC_p - GC_pi) = 15 \cdot (0.85 - (-0.18)) = 15 \cdot 1.03 = 15.45\,\text{psf}$ (acting outward).
  - Negative (inward): $p = q_z \cdot (0.85 - 0.18) = 15 \cdot 0.67 = 10.05\,\text{psf}$ (acting inward).

However, the simplified method often uses a net pressure for design:

- From Figure 28.3-1, Zone 4 (wall zone), $p_s = 17.8\,\text{psf}$ (positive) and $-10.7\,\text{psf}$ (negative) for 110 mph, Exposure B, adjusted for height ≤ 15 ft.

### Step 3: Adjusted Design Pressure

Since the wall is only 4 ft high (4–8 ft range), and we're ignoring the bottom 4 ft:

- Use **Zone 4 (wall)** pressures from ASCE 7-16 Table 28.3-1, adjusted for 110 mph:
  - Positive pressure (windward): **17.8 psf**.
  - Negative pressure (suction, leeward or side): **-10.7 psf**.
- These are net pressures including internal effects, suitable for components and cladding (C&C) design up to 60 ft height.

18 psf Controls

Check roof loads

From Figure 28.3-1 (Exposure B, 90 mph, slope 7°–27°):

- **Zone 2 (Windward Edge):** +13.0 psf (downward), -16.7 psf (uplift).
- **Zone 1 (Windward Interior):** +8.5 psf, -11.2 psf.
- **Zone 1 (Leeward Interior):** -11.2 psf.
- **Zone 2 (Leeward Edge):** -16.7 psf.

Adjusted for 110 mph:

- **Zone 2 (Windward Edge):** $13.0 \cdot 1.494 = 19.4\,\text{psf}$ (downward), $-16.7 \cdot 1.494 = -25.0\,\text{psf}$ (uplift).
- **Zone 1 (Windward Interior):** $8.5 \cdot 1.494 = 12.7\,\text{psf}$, $-11.2 \cdot 1.494 = -16.7\,\text{psf}$.
- **Zone 1 (Leeward Interior):** $-11.2 \cdot 1.494 = -16.7\,\text{psf}$.
- **Zone 2 (Leeward Edge):** $-16.7 \cdot 1.494 = -25.0\,\text{psf}$.

- **Windward Roof:**
  - **Edge (Zone 2):** +19.4 psf (downward), -25.0 psf (uplift).
  - **Interior (Zone 1):** +12.7 psf, -16.7 psf.
- **Leeward Roof:**
  - **Edge (Zone 2):** -25.0 psf.
  - **Interior (Zone 1):** -16.7 psf.
- **Corners (Zone 3, C&C):** -43.2 psf (highest uplift, for sheathing/attachments).

New walls:

**Windward Wall Pressure**

- Exposed Area: 34 ft × 4 ft (upper portion) = 136 sq ft.
- Pressure: 17.8 psf (from prior calculation, Zone 4, windward).
- Force: $F_{\text{wall}} = 17.8\,\text{psf} \cdot 136\,\text{sq ft} = 2,420.8\,\text{lb}.$

**Windward Roof Pressure (Horizontal Component)**

- Roof Slope: 5:12 (22.62°), horizontal projection factor = $\cos(22.62^\circ) \approx 0.923.$
- Area (projected vertically): Half-span = 17 ft, length = 34 ft, projected area = 17 ft × 34 ft = 578 sq ft.
- Pressure: 19.4 psf (Zone 2, downward), horizontal component = $19.4 \cdot \sin(22.62^\circ) \approx 19.4 \cdot 0.385 = 7.47\,\text{psf}$ (lateral force toward leeward side).
- Force: $F_{\text{roof, windward}} = 7.47\,\text{psf} \cdot 578\,\text{sq ft} = 4,317.7\,\text{lb}.$

Force = 2.2 K per wall

New shearwall (Nanowall) and existing garage wall share load. 1.1k each wall.

- **Shear Load:** $v = 1,100\,\text{lb}/6\,\text{ft} = 183.3\,\text{plf}.$
- **OSB Capacity:** Per APA Table 4A (1/2" OSB, 6d nails at 6" edge/12" field):
  - Nominal capacity = 720 plf (unblocked, Douglas Fir, G ≈ 0.5).
  - No adjustment for wind (same as seismic per IBC 2306.3).
- **Check:** 183.3 plf < 720 plf → **Shear capacity is adequate**.

Anchor Bolts

- **Bolt Spacing:** 48" o.c. over 6 ft = 2 bolts (0", 48").
- **Capacity per Bolt:** 5/8" bolt, 7" embedment (IBC Table 1911.2, f'c = 2,500 psi):
  - Shear ≈ 2,860 lb.
- **Shear Load per Bolt:** 1,100 lb / 2 = 550 lb.
- **Check:** 550 lb < 2,860 lb → **Anchor bolts are adequate for shear**.

Hold Downs (Not required, see next)

- **Overturning Moment:** $M = 1,100\,\text{lb} \cdot 8\,\text{ft} = 8,800\,\text{ft-lb}$ (load at top).
- **Resisting Moment from Dead Load:**
  - Dead Load (DL): Wall = 10 psf, Roof = 15 psf (same as before).
  - Wall DL: 6 ft × 8 ft × 10 psf = 480 lb.
  - Roof Tributary: 6 ft × 17 ft (half-span) × 15 psf = 1,530 lb.
  - Total DL = 480 + 1,530 = 2,010 lb.
  - 2/3 DL = $2,010 \cdot 2/3 = 1,340\,\text{lb}$.
  - Resisting Moment: $M_r = 1,340\,\text{lb} \cdot 3\,\text{ft} = 4,020\,\text{ft-lb}$.
- **Check:** 4,020 ft-lb < 8,800 ft-lb → **Overturning exceeds resisting moment**.
- **Hold-Down Force:**
  - Net tension = $(8,800 - 4,020)/6\,\text{ft} = 4,780/6 = 797\,\text{lb}$ (windward end).
- **Anchor Bolt Tension Capacity:** 5,510 lb/bolt > 797 lb → OK if tied to framing.

Check A35 between gable truss and new wall

  - Shear = 183.3 plf.
  - End clips: $183.3\,\text{plf} \cdot 0.67\,\text{ft} = 122.8\,\text{lb}$.
  - Interior clips: $183.3\,\text{plf} \cdot 1.33\,\text{ft} = 243.8\,\text{lb}$.
- **Alternative (Uniform Distribution):**
  - Total load = 1,100 lb / 5 clips = 220 lb/clip (simpler, assumes even distribution).

A35 in F1 direction = 260 lb/clip OK.

**Mark '1' Shearwalls OK** all new exterior walls (nanowall area)

**GRAVITY ANALYSIS.**

Most gravity systems are staying in place or lightly loaded controlled by deflection criteria. Replacement beam in the Basement and support is the only design feature.

Loads:

400 plf live (floor)

100 plf dead (floor)

Ignore the 2x Sleeper and web fillers.

**Span (L)**: 19 ft = 19 × 12 = 228 inches

**Loads**:

Total uniform load (w) = 400 plf (live) + 100 plf (dead) = 500 plf

Depth constraint: W8 nominal depth (actual depths around 8")

Deflection limit: **L/400** = 228 / 400 = 0.57 inches – User Defined Deflection

Assumptions: Simply supported, Fy = 50 ksi (A992 steel), ASD method, no bracing issues (assume laterally supported by sleeper attachment).

Calculate Reactions and Moments

Total load = w × L = 500 × 19 = 9,500 lbs

Reactions = 9,500 / 2 = 4,750 lbs each end

Maximum moment (M_max) = (w × $L^2$) / 8

M_max = (500 × $19^2$) / 8 = (500 × 361) / 8 = 180,500 / 8 = 22,562.5 ft-lbs

In in-lbs: 22,562.5 × 12 = 270,750 in-lbs

*Bending Stress Check*

Allowable stress (Fb) = 0.66 × Fy = 0.66 × 50 = 33 ksi

Required section modulus (S_x) = M_max / Fb

S_x = 270,750 in-lbs / 33,000 psi = 8.2 $in^3$

The beam needs S_x ≥ 8.2 $in^3$.

*Deflection Check*

Deflection formula: δ = ($5wL^4$) / (384EI)

$w = 500$ plf $= 500 / 12 = 41.67$ lb/in

$L = 228$ in

$E = 29{,}000$ ksi $= 29{,}000{,}000$ psi

Max $\delta = 0.57$ in

Rearrange for I:

$0.57 = (5 \times 41.67 \times 228^4) / (384 \times 29{,}000{,}000 \times I)$

$228^4 = 2{,}699{,}660{,}544$ in$^4$

Numerator: $5 \times 41.67 \times 2{,}699{,}660{,}544 \approx 562{,}327{,}500{,}000$ lb-in$^3$

Denominator: $384 \times 29{,}000{,}000 \times I \approx 11{,}136{,}000{,}000 \times I$

$562{,}327{,}500{,}000 / (11{,}136{,}000{,}000 \times I) = 0.57$

$I = 562{,}327{,}500{,}000 / (11{,}136{,}000{,}000 \times 0.57) \approx 88.6$ in$^4$

The beam needs $I_x \geq 88.6$ in$^4$.

Select a W8 Beam – User Defined depth for no show in ceiling.

From AISC W8 sections:

W8x18 (depth = 8.14"):

$S_x = 13.0$ in$^3$ > 8.2 ✓

$I_x = 53.0$ in$^4$ < 88.6 ✗

Deflection fails ($\delta \approx 0.94$" > 0.57").

W8x24 (depth = 7.93"):

$S_x = 17.4$ in$^3$ > 8.2 ✓

$I_x = 69.7$ in$^4$ < 88.6 ✗

Deflection $\delta \approx 0.72$" > 0.57", still fails.

W8x31 (depth = 8.00"):

S_x = 22.1 $in^3$ > 8.2 ✓

I_x = 110 $in^4$ > 88.6 ✓

Deflection δ ≈ 0.46" < 0.57" ✓

Step 6: **Verify W8x31**

Bending: Allowable moment = S_x × Fb / 12 = 22.1 × 33 / 12 = 60.8 ft-kips > 22.56 ft-kips ✓

Deflection: δ = (5 × 41.67 × $228^4$) / (384 × 29,000,000 × 110) ≈ 0.46 in < 0.57 in ✓

Depth: 8.00" < 9" constraint ✓

**W8 X 31 min**

Find reactions for footings.

**Step 1: Total Load**

- **Span (L):** 19 ft
- **Uniform load (w):** 500 plf
- **Total load:** w × L = 500 plf × 19 ft = 9,500 lbs

**Step 2: Reactions**
For a simply supported beam with a uniform load, the load is evenly split between the two supports:

- **Reaction at each end (R):** Total load / 2 = 9,500 lbs / 2 = 4,750 lbs

The reaction at each end is **4,750 lbs**. This assumes a perfectly uniform load and simple supports. Let me know if you need further details!

Check Footing
Per Geotech

**Table 1. Design Parameters for Shallow Foundations**

| |
|---|
| Allowable soil bearing pressure = 3,000 psf |
| Allowable coefficient of sliding resistance = 0.35 |
| Allowable passive earth pressure = 300 pcf |
| Minimum foundation width = 18 inches (continuous), 24 inches (isolated) |

*Figure 1- Sage Geotech Memo*

- **Footing Area:** $2\,\mathrm{ft} \cdot 2\,\mathrm{ft} = 4\,\mathrm{sq\ ft}.$
- **Applied Pressure:** $q = 4,750\,\mathrm{lb}/4\,\mathrm{sq\ ft} = 1,187.5\,\mathrm{psf}.$
- **Allowable Pressure:** 3,000 psf.
- **Check:** 1,187.5 psf < 3,000 psf → **Footing size is adequate for bearing.**
    - Factor of Safety ≈ 3,000 / 1,187.5 ≈ 2.53 (well within typical soil design).

- **Moment Calculation:** Critical section at post face (ACI 318-19, 7.4.2).
  - Distance from edge to post face = (24" - 4") / 2 = 10".
  - Tributary area = (10" / 12) × 2 ft = 1.67 sq ft.
  - Load: $1,187.5\,\text{psf} \cdot 1.67\,\text{sq ft} = 1,983\,\text{lb}$.
  - Moment arm = 10" / 2 = 5" (centroid of pressure to post face).
  - $M_u = 1,983\,\text{lb} \cdot 5\text{"}/12 = 827\,\text{ft-lb} = 9,924\,\text{in-lb}$.
- **Flexural Capacity:**
  - $\phi M_n = \phi \cdot A_s \cdot f_y \cdot (d - a/2), \phi = 0.9$.
  - Try 3 #4 bars (minimum practical): $A_s = 3 \cdot 0.20 = 0.60\,\text{sq in.}$
  - $f_y = 60,000\,\text{psi}$ (Grade 60 rebar).
  - $a = (A_s \cdot f_y)/(0.85 \cdot f\text{'}c \cdot b) = (0.60 \cdot 60,000)/(0.85 \cdot 3,000 \cdot 24) = 0.588\text{"}$.
  - $M_n = 0.60 \cdot 60,000 \cdot (4.75 - 0.588/2) = 36,000 \cdot 4.456 = 160,416\,\text{in-lb}$.
  - $\phi M_n = 0.9 \cdot 160,416 = 144,374\,\text{in-lb}$.
- **Check:** 9,924 in-lb < 144,374 in-lb → **Flexural capacity is adequate**.
- **Minimum Reinforcement (ACI 318-19, 7.6.1.1):**
  - $A_{s,min} = 0.0018 \cdot b \cdot h = 0.0018 \cdot 24 \cdot 8 = 0.346\,\text{sq in.}$
  - 3 #4 = 0.60 sq in > 0.346 sq in → **OK**.

- **Minimum Bars:** 3 #4 bars each way (6 total, grid pattern).
  - Spacing: 24" - 2 × 3" (cover) = 18", 2 spaces ≈ 9" o.c.
  - Place 3 #4 @ 9" o.c. in both directions, bottom of footing.

Pick Hangers for existing 2X10

**Option 1: JB210A (Light-to-Medium Duty)**

- **Description:** Galvanized Top-Flange Joist Hanger
- **Fit:** 2x10 (W = 1-9/16", H = 9-1/4")
- **Gauge:** 18-ga
- **Fasteners:**
  - Header: 6-10d nails (min) or 10-10d (max)
  - Joist: 2-10d x 1-1/2" (min) or 4-10d x 1-1/2" (max)

- **Allowable Loads (ASD, Douglas Fir/Southern Pine):**
  - Down: 1,220 lbs (min nailing), 2,030 lbs (max nailing)

  - Uplift: 290 lbs (min), 580 lbs (max)

**<End Calculations> Original**

**<Start Rev 1 Calculations>**

Deck Connection review, worst case new deck.

The deck's area is:

$$\text{Area} = 12\,\text{ft} \times 26\,\text{ft} = 312\,\text{ft}^2$$

The uniform load is given as 10 psf. The total vertical load (dead load) on the deck is:

$$W = 10\,\text{psf} \times 312\,\text{ft}^2 = 3120\,\text{lb}$$

Use a conservative Cs of .5

$$F = 0.5 \times 3120\,\text{lb} = 1560\,\text{lb}$$

3" Ledgerlok in tension

TABLE 2—REFERENCE WITHDRAWAL DESIGN VALUES FOR INSTALLATION INTO THE FACE OF THE WOOD MEMBER[1]

| FASTENER | MINIMUM EMBEDDED THREAD LENGTH (inches) | $W_g$ (lbf/in.) FOR SPECIFIC GRAVITIES OF: | | | | | |
|---|---|---|---|---|---|---|---|
| | | 0.57 | 0.55 | 0.5 | 0.46 | 0.43 | 0.42 |
| OlyLog/ TimberLOK | 1.25 | 270 | 260 | 220 | 200 | 180 | 170 |
| HeadLOK | 2.0 | 290 | 270 | 230 | 200 | 180 | 170 |
| LedgerLOK, VersaLOK, LogHog | 2.0 | 330 | 310 | 270 | 240 | 220 | 210 |
| ThruLOK | n/a | See Note 2 | | | | | |

For **SI:** 1 inch = 25.4 mm, 1 lbf/in = 0.175 N/mm.
[1]Tabulated reference withdrawal design values apply to fasteners installed at 90 degrees to the face of the wood member.
[2]For the ThruLOK screws, withdrawal resistance is provided by the nut installed on the backside of the connection, which is greater than the head pull-through design value shown in Table 3.

Assuming ½ of the deck is in tension – 3 pairs of LedgerLOK resist 1620 # which is more than the entire deck lateral pull away. **No tension ties required.**

IEBC LRFS

New loads to upper wall and roof diaphragm: ( overframing)

**Seismic** review of building front, rear of building has new walls per above:

Existing roof area = 2610

Existing roof system weight = 26,100 #

Overframing area= 350 sf

Remove roofing (2.5 psf) under overframe areas = 875#

Existing porch = 150 sf = 1500#

New Porch =- 42 sf = 420 #

Overframing weight = 350sf * 5 psf = 1750#

Change in dead load = 26100#-875#-1500#+420#+1750# = 25,895

Dead load at front of house is basically equal, IEBC Section 503.3 ok for seismic

**Wind** See above for wind loads. The gable vs hip roof has same profile but different wind results. (Zone 2 vs Zone 5 using GCpf)

18 psf (Gable) vs 18 psf X .7 = 12.6 psf

Area – 71.25 SF Load difference is: 906 # vs 1305# - 200 lb additional load per side. Existing walls ok by inspection. (worst case load scenario)

**<End Calculations> l**

References Used


1. **International Building Code (IBC) 2021** - International Code Council (ICC), effective March 15, 2024, in Washington State with local amendments.
2. **International Residential Code (IRC) 2021** - ICC, adopted by Mercer Island for residential design, effective March 15, 2024.
3. **ASCE/SEI 7-16: Minimum Design Loads and Associated Criteria for Buildings and Other Structures** - American Society of Civil Engineers, 2016 (referenced by IBC 2021).
4. **ACI 318-19: Building Code Requirements for Structural Concrete and Commentary** - American Concrete Institute, 2019 (referenced by IBC 2021 for concrete design).
5. **NDS 2018: National Design Specification for Wood Construction** - American Wood Council, 2018 (referenced by IBC 2021 for wood design).
6. **APA Engineered Wood Construction Guide, Form E30** - APA – The Engineered Wood Association, 2019 (current for wood shear wall capacities).
7. **Simpson Strong-Tie Wood Construction Connectors Catalog, C-C-2021** - Simpson Strong-Tie, 2021 (latest available for connector capacities as of 2025).
8. **Washington State Building Code (WSBC)** - Washington State Legislature, 2021 amendments to IBC/IRC, effective March 15, 2024.
9. **Mercer Island Building Code Amendments** - City of Mercer Island, adopting 2021 IBC/IRC with local modifications, accessed via mercerisland.gov, current as of 2025.
10. **USGS National Earthquake Hazards Reduction Program (NEHRP) Seismic Design Maps** - U.S. Geological Survey, 2023 update (for seismic parameters in Puget Sound region).
11. **AWC SDPWS 2021: Special Design Provisions for Wind and Seismic** - American Wood Council, 2021 (supplement to NDS for wind and seismic design of wood structures).
12. **SEAOC Wind Design Manual 2020** - Structural Engineers Association of California, 2020 (practical guide for ASCE 7 wind load applications).
13. **AISC Steel Construction Manual, 15th Edition** - American Institute of Steel Construction, 2017 (latest edition as of 2025 for steel design, referenced by IBC).