2720 Residence

2720 71st Avenue SE
Mercer Island, Washington 98040

Structural Engineering Calculations

By

Dihong Shao, SE

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GENERAL STRUCTURAL NOTES:

(THE FOLLOWING NOTES APPLIES TO THE PROPOSED PROJECT UNLESS OTHERWISE NOTED ON THE PLANS AND DETAILS)

ALL DESIGN AND CONSTRUCTION SHALL COMPLY WITH THE 2018 INTERNATIONAL BUILDING CODE AND SEATTLE BUILDING CODE

UNLESS INDICATED AS EXISTING OR (E), ALL OTHERS ARE NEW CONSTRUCTIONS OR AS INDICATED AS (N).

DESIGN LOADING CRITERIA:

1. DESIGN LOADS:	ROOF SNOW LOAD:	25 PSF
	FLOOR LIVE LOAD:	40 PSF
	DECK LIVE LOAD:	60 PSF
	WIND:	110-MPH (3-SECOND GUST), EXPOSURE B, Kzt= 1.90
	SEISMIC:	SEISMIC USER GROUP I, I=1.0, SITE CLASS Sd Ss=1.48; S_1=0.50, Fa=1.00; Fv=1.50 S_{DS}=0.98; S_{DI}=0.50 R=6.5 (WOOD SHEAR WALL) Ωo=3.0 Cd=4.0

DESIGN SOIL PRESSURE:

1500 PSF MAXIMUM DEAD+LIVE LOAD WITH ALLOWED INCREASE FOR DEPTH OF 110 PSF PER FOOT. CAST FOOTING ON NATIVE SITE SOILS OR STRUCTURAL FILL THAT EXTENDS DOWN TO THESE SOILS.

EQUIVALENT LATERAL FLUID PRESSURE FOR	
CANTILEVER WALLS:	35 PSF
BASEMENT WALLS:	55 PSF
PASSIVE SOIL PRESSURE:	350 PCF
SOIL FRICTION:	0.35

CONSTRUCTION REQUIREMENTS:

1. CONTRACTOR SHALL VERIFY DIMENSIONS AND CONDITIONS FOR COMPATIBILITY AND SHALL NOTIFY OWNER OF ANY DISCREPANCIES PRIOR TO CONSTRUCTION. ALL DIMENSIONS OF EXISTING CONSTRUCTION SHOWN IN THE DRAWINGS ARE INTENDED AS GUIDELINES ONLY AND MUST BE VERIFIED. THE CONTRACTOR SHALL BRING ALL DISCREPANCIES TO THE OWNER.

2. CONTRACTOR SHALL PROVIDE TEMPORARY SHORING AND BRACING FOR THE STRUCTURE AND STRUCTURAL COMPONENTS UNTIL ALL FINAL CONNECTIONS HAVE BEEN COMPLETED IN ACCORDING WITH THE PLANS AND DETAILS. THIS INCLUDES EXISTING STRUCTURE.

3. CONTRACTOR SHALL BE RESPONSIBLE FOR ALL SAFETY AND HEALTH PRECAUTIONS INCLUDING HAZARDOUS CONDITIONS AND MATERIALS EXISTED OR CREATED BY OTHER PARTIES THAT WORKING ON THE PROJECT. CONTRACTOR SHALL ALSO BE RESPONSIBLE FOR CONSTRUCTION METHODS, TECHNIQUES, AND SEQUENCES OR PROCEDURES REQUIRED TO PERFORM THE WORK.

4. CONTRACTOR SHALL BE RESPONSIBLE FOR KEEPING ALL EXISTING COMPONENTS, WHICH ARE REQUIRED TO REMAIN, IN THEIR ORIGINAL CONDITION. THIS INCLUDES WEATHER PROTECTIONS FOR THESE COMPONENTS UNTIL SUCH TIME THAT THE ENTIRE DWELLING INCLUDING THE NEW ADDITION ITSELF IS WEATHER PROTECTED.

5. CONTRACTOR INITIATED CHANGES SHALL BE SUBMITTED IN WRITING TO THE OWNER FOR APPROVAL PRIOR FABRICATION OR CONSTRUCTION. CHANGES SHOWN IN SHOP DRAWINGS ONLY WILL NOT SATISFY THIS REQUIREMENT.

6. CONTRACTOR SHALL VERIFY ALL EXISTING CONDITIONS BEFORE COMMENCING ANY DEMOLITION. SHORING SHALL BE INSTALLED TO SUPPORT EXISTING CONSTRUCTION AS REQUIRED AND IN A MANNER SUITABLE TO THE WORK SEQUENCES. DEMOLITION DEBRIS SHALL NOT BE ALLOWED TO DAMAGE OR OVERLOAD THE EXISTING STRUCTURAL. LIMIT CONSTRICTION LOADING (INCLUDING DEMOLITION DEBRIS) ON EXISTING CEILING FAMING TO 10 PSF AND ON EXISTING FLOOR FRAMING TO 40 PSF. PROVIDE TEMPORARY PLANKS OR STRUCTURAL SHEATHING OVER THE EXISTING CEILING JOISTS AS REQUIRED TO PROTECT THE EXISTING SOFFIT.

7. CONTRACTOR SHALL CHECK FOR DRY-ROT FOR ALL EXISTING STRUCTURAL COMPONENTS AT EXTERIOR WALLS, EXISTING TOILET ROOM FLOORS AND WALLS, AREAS SHOWN WATER STAINS, WOOD IN CONTACT WITH EARTH AND CONCRETE, AND ALL WOOD MEMBERS IN CRAW SPACES. ALL ROTTEN WOOD SHALL BE REMOVED AND DAMAGED MEMBERS SHALL BE REPLACED OR REPAIRED AS DIRECTED BY THE OWNER.

8. DRAWINGS INDICATE GENERAL AND TYPICAL DETAILS OF CONSTRUCTION. WHERE CONDITIONS ARE NOT SPECIFICALLY INDICATED BUT ARE OF SIMILAR CHARACTER TO DETAILS SHOWN, SIMILAR DETAILS OF CONSTRUCTION SHALL BE USED, SUBJECT TO REVIEW AND APPROVAL BY THE OWNER.

9. ALL STRUCTURAL SYSTEMS, WHICH ARE TO BE COMPOSED OF COMPONENTS TO BE FIELD ERECTED, SHALL BE SUPERVISED BY THE SUPPLIER DURING MANUFACTURING, DELIVERY, HANDLING, STORAGE, AND ERECTION IN ACCORDANCE WITH INSTRUCTIONS PREPARED BY THE SUPPLIER.

STRUCTURAL FRAMING REQUIREMENTS:

1. ALL LUMBER SHALL BE KILN DRIED OR MC-19 WITH WWPA GRADED OR APPROVED EQUAL. ALL STRUCTURAL FLOOR, ROOF, AND SHEAR WALL SHEARING SHALL BE APA RATED. ALL SPECIFIED INDUSTRIAL LUMBERS, NAMELY PARALLAM PSL, MICROLLAM LVL, TIMBERSTRAND LSL, AND TJI SHALL BE MADE BY TRUS-JOIST CORPORATION OR OWNER APPROVED EQUAL.

2. MINIMUM NAILING SHALL COMPLY WITH TABLE 2304.10.1 OF THE 2015 IBC.

3. ALL NAILS SIZES SPECIFIED ON DRAWINGS ARE BASED ON THE FOLLOWING SPECIFICATIONS:
NAIL SIZE, LENGTH, AND DIAMETER
6D 2” 0.113”   8D 2-1/2” 0.131   10D 2-1/2” 0.148   16D BOX 3” 0.131
THE FOLLOWING STAPLES MAY BE SUBSTITUTED FOR NAILING OF PLYWOOD
NAIL SIZE, EQUIVALENT STAPLE, AND MINIMUM LENGTH
6D 16GA 1-3/4”   8D 15GA 1-3/4”   10D 13GA 1-3/4”

4. GALVANIZED METAL TIMBER CONNECTORS CALLED OUT BY LETTERS AND NUMBERS SHALL BE “STRONG-TIE” BY SIMPSON COMPANY INCLUDING SIMPSON STRONG WALLS AND SIMPSON GARAGE PORTAL WALLS (WHERE OCCUR) OR OWNER APPROVED EQUAL. IF NO SPESIFIC HANGER IS CALLED OUT, ANY HANGER MADE FOR THE SPECIFIED BEAM OR JOIST CAN BE USED.

5. ALL EXTERIOR WALL STUDS ARE 2X6 HEM-FIR NO.2 STUDS AT 16” ON CENTER. ALL INTERIOR WALL STUDS ARE 2X4 HEM-FIR NO.2 STUDS AT 16” ON CENTER. PROVIDE ONE BEARING STUD AND ONE FULL HEIGHT STUD AT EACH SIDE OF DOOR AND WINDOW OPENINGS WHEN THEIR ROUGH OPENING WIDTH IS EQUAL OR LESS THAN 3’-0”. PROVIDE TWO BEARING STUDS AND TWO FULL HEIGHT STUDS AT EACH SIDE OF DOOR AND WINDOW OPENINGS WHEN THEIR ROUGH OPENING WIDTH IS GREATER THAN 3’-0” OR WALL IS FRAMED WITH (2)2X6 AT 16” ON CENTER. PROVIDE TRIPLE STUDS UNDER ALL BEAM AND KING-TRUSS BEARING LOCATIONS. THESE MULTIPLE STUDS NEED TO EXTEND DOWN TO THE TOP OF CONCRETE. PROVIDE TRIPLE VERTICAL BLOCKING AT JOIST SPACING AS NEEDED. FACE NAIL WALL TOP DOUBLE PLATE WITH 16D @ 12” AND LAP MINIMUM 4’-0” AT JOINTS AND PROVIDE (6) 16D @ 4” ON CENTER EACH SIDE OF JOINT. FACE NAIL WALL SILL PLATE THROUGH FLOOR SHEATHING TO DOUBLE PLATES, BEAM, OR SUPPORTING MEMBER BELOW WITH 16D @ 6” ON CENTER. MULTIPLE STUD SHALL BE NAILED TOGETHER WITH 16D @ 12” ON CENTER STAGGERED EACH FACE. PROVIDE SOLID BLOCKING BETWEEN STUDS AT MID-HEIGHT FOR ALL STUD WALLS OVER 10’ IN HEIGHT.

6. PROVIDE DOUBLE JOISTS UNDER ALL PARALLEL PARTITIONS THAT EXTEND OVER MORE THAN HALF THE JOIST LENGTH AND AROUND ALL OPENING IN FLOOR.

7. ALL FLOOR FRAMING LUMBERS: DOUGLAS FIR NO.2 OR HAM FIR NO.1
ALL HEADERS: DOUGLAS FIR NO.2 OR HAM FIR NO.1. TYPICAL HEADER 4X8 MINIMUM UNLESS OTHERWISE SHOWN ON THE PLANS.
ALL POSTS: DOUGLAS FIR NO.2 OR HAM FIR NO.1 UNLESS OTHERWISE SHOWN ON THE PLANS
STUDS, PLATES, AND MISCELLANEOUS LIGHT FRAMING: HEM-FIR NO.2

8. METAL PLATE CONNECTED WOOD TRUSSES: WOOD TRUSSES SHALL BE DESIGNED, MANUFACTURED AND INSTALLED PER TRUSS PLATE INSTITUTE (TPI) SPECIFICATIONS. TPI SPECIFICATIONS SHALL NOT REVISE TRUSS ENGINEER’S AND TRUSS MANUFACTURER’S RESPONSIBILITY NOTED BELOW. WEB AND CHORD SIZES INDICATED ON PLANS AND NOTES ARE MINIMUM ONLY. ROOF DESIGN DEAD LOAD 10 PSF MINIMUM TOP CHORD AND 7 PSF MINIMUM BOTTOM CHORD WITH 40 PSF MINIMUM AT ATTIC FLOOR WHERE APPLICABLE. USE 2X6 MINIMUM BOTTOM CHORD FOR ATTIC FLOOR. ROOF DESIGN WIND UPLIFT 15 PSF MINIMUM TYPICAL, EXCEPT USE 30 PSF MINIMUM WITHIN 10 FEET OF ROOF EAVES OR RAKES. DESIGN TRUSSES FOR SUPPORT OF DEAD, LIVE, SNOWDRIFT, AND WIND LOADS AND MECHANICAL/ELECTRICAL EQUIPMENT, PIPING, ETC AS REQUIRED. SNOW DRIFT LOADING LOCATIONS AND VALUES TO BE DETERMINED BY TRUSS ENGINEER. SUBMIT SHOP

DRAWINGS AND DESIGN CALCULATIONS SHOWING TRUSSES, TRUSS TO TRUSS AND TRUSS TO SUPPORTING STRUCTURE CONNECTIONS, ERECTION AND PERMANENT BRACING SIZES AND CONNECTIONS. PROVIDE STANDARD TRUSS CAMBER. PROVIDE ERECTION BRACING PER MANUFACTURE'S INSTRUCTIONS. PROVIDE AND INSTALL PERMANENT BRACING FOR LATERAL SUPPORT OF INDIVIDUAL WEB AND CHORD MEMBERS AS DESIGNED BY THE TRUSS ENGINEER. PROVIDE AND INSTALL ALL TRUSS TO TRUSS AND TRUSS TO SUPPORTING STRUCTURE CONNECTIONS.

9. VENT BLOCKINGS CALLED OUT IN THE DRAWINGS ARE 2X WOOD BLOCKING WITH (3) EQUAL SPACED 1-1/2" DIAMETER HOLES ON EACH BLOCKING WITH MASH INSTALLED.

10. ROOF SHEATHING: 15/32"(1/2") MINIMUM CDX PLYWOOD OR STRUCTURAL PANEL WITH SPAN RATING OF 32/16, UNBLOCKED, LAID UP WITH FACE GRAIN PERPENDICULAR TO FRAMING BELOW, STAGGER END JOINTS. INSTALL PLYCLIPS AS REQUIRED. NAILING IS AS FOLLOWS: 10D @ 6" DIAPHRAGM BOUNDARIES, OVER EXTERIOR WALLS, AND INTERIOR SHEAR WALLS, 10D @ 6 ALL SUPPORTED EDGES, AND 10D @ 12" FIELD.

11. FLOOR SHEATHING: 23/32"(3/4") MINIMUM CDX TONGUE AND GROOVE PLYWOOD WITH SPAN RATING OF 40/20, UNBLOCKED FOR FLOOR JOIST SPACED AT 16" ON CENTER; 7/8" MINIMUM CDX TONGUE AND GROOVE PLYWOOD WITH SPAN RATING OF 40/20 UNBLOCKED FOR FLOOR JOIST SPACED AT 24" ON CENTER; LAID UP WITH FACE GRAIN PERPENDICULAR TO FRAMING BELOW, STAGGER END JOINTS. GLUE FLOOR SHEATHING TO ALL SUPPORTS WITH A CONTINUOUS 3/16" DIAMETER BEAD MINIMUM. PROVIDE TWO BEADS AT PANEL JOINTS. NAILING IS AS FOLLOWS: 10D @ 6" DIAPHRAGM BOUNDARIES, OVER EXTERIOR WALLS, AND INTERIOR SHEAR WALLS, 10D @ 6" ALL SUPPORTED EDGES, AND 10D @ 10" FIELD.

12. EXTERIOR/INTERIOR/SHEAR WALL SHEATHING 15/32" (1/2") MINIMUM CDX PLYWOOD WITH SPAN RATING OF 24/0, EXTERIOR SIDE BLOCKED (BLOCK ALL UNSUPPORTED EDGES), NAIL WITH 10D @ 6" ALL EDGES AND 10D @ 12" FIELD. NAIL BOTTOM PLATE TO FRAMING BELOW WITH 16D @ 6".

13. WALL SILL PLATES OVER THE CONCRETE ARE TO BE 3X TREATED LUMBER WITH 1/2" DIAMETER ANCHOR BOLTS AT 4'-0" ON CENTER WITH EMBED IN CONCRETE OF 7" MINIMUM. ALL BOLTS SHALL HAVE 3X3X3/16 STEEL WASHER PLATE UNDER BOLT NUTS. MINIMUM OF TWO BOLTS PER PLATE WITH BOLT END DISTANCE OF 6" MINIMUM. SHEAR WALL BOTTOM PLATE NAILING AND ALL NAILING AT PRESSURE TREATED PLATE/MEMBERS SHALL BE HOT-DIPPED ZINC-COATED GALVANIZED STEEL OR STAINLESS STEEL NAILS

CONCRETE AND FOUNDATION CONSTRUCTIONS:

1. ALL CONCRETE f'c=2500 PSI, MAXIMUM WATER/CEMENT RATIO =0.45, MINIMUM 5-1/2 SACKS OF CEMENT PER CUBIC YARD. NO SPECIAL INSPECTION REQUIRED. CONCRETE BATCH TICKET OR DELIVERY RECEIPT FOR 2500 PSI CONCRETE ON SITE FOR BUILDING INSPECTOR VERIFICATION. CONCRETE SHALL BE AIR ENTRAINED. TOTAL AIR CONTENT (PERCENT BY VOLUME OF CONCRETE) SHALL NOT BE LESS THAN 5 PERCENT OR MORE THAN 7 PERCENT.

2. REINFORCING STEEL SHALL CONFORM TO ASTM A615, GRADE 60. SPECIAL INSPECTION REQUIRED. ASTM A706, GRADE 60, REINFORCING STEEL SHALL BE USED FOR WELDED OR FIELD-BENT BARS, SHEAR WALL BOUNDARY MEMBER REINFORCING, MAIN REINFORCING, SPIRALS, TIES AND STIRRUPS IN THE FRAME MEMBERS (BEAMS AND COLUMNS) COMPRISING THE LATERAL FORCE RESISTING SYSTEM.

3. WELDED WIRE FABRIC PER ASTM A185. FURNISH IN FLAT SHEETS, NOT ROLLS. LAP EDGES 1-1/2 MESH MINIMUM.

4. PROVIDE CONCRETE COVER AS FOLLOWS: FOOTINGS 3”, WALLS 1-1/2”, AND SLAB ON GRADE 1-1/2”.

5. PROVIDE 2#4 LONGITUDINAL BOTTOM BARS IN WALL FOOTINGS. PROVIDE CORNER BARS OF SAME SIZE AND NUMBER AT CORNERS AND INTERSECTIONS, 42 BAR DIAMETERS EACH LEG. PROVIDE VERTICAL DOWELS OF SAME SIZE, NUMBER AND SPACING AS CONCRETE STEM WALL VERTICAL BARS WITH A 90 DEGREE STANDARD HOOK AT THE BOTTOM OF THE FOOTING.

6. REINFORCING CONCRETE WALLS AS FOLLOWS”

6” WALLS, #4 @ 12 HORIZONTAL AND VERTICAL AT CENTER OF WALL,
8” WALLS, #5 @ 15 HORIZONTAL AND VERTICAL AT CENTER OF WALL,
10” WALLS, #4 @ 16 HORIZONTAL AND VERTICAL AT EACH FACE,
12” WALLS, #4 @ 12 HORIZONTAL AND VERTICAL AT EACH FACE.

AT OPENINGS OVER 12” SQUARE, PROVIDE 2#5 BARS AT CENTER OF WALL ALL FOUR SIDES, EXCEPT 10” WALLS OR OVER PROVIDE 1#6 BAR EACH FACE ALL FOUR SIDES, EXTENDING 42 BAR DIAMETERS PAST OPENING. PROVIDE 1#5X4’-0” DIAGONAL BAR AT CENTER OF WALL ALL FOUR CORNERS.

AT CORNERS, PROVIDE CORNER BARS IN OUTSIDE FACE OF SAME SIZE AND SPACING AS HORIZONTAL BARS, 42 BAR DIAMETER EACH LEG.

AT INTERSECTIONS, PROVIDE CORNER BARS OF SAME SIZE, NUMBER AND SPACING AS HORIZONTAL BARSOF INTERSECTING WALL, 42 BAR DIAMETER EACH LEG.

PROVIDE 2#4 LONGITUDINAL BARS AT TOP OF WALLS. PROVIDE KEY WAY OR ROUGHENED SURFACE AT CONSTRUCTION JOINTS.

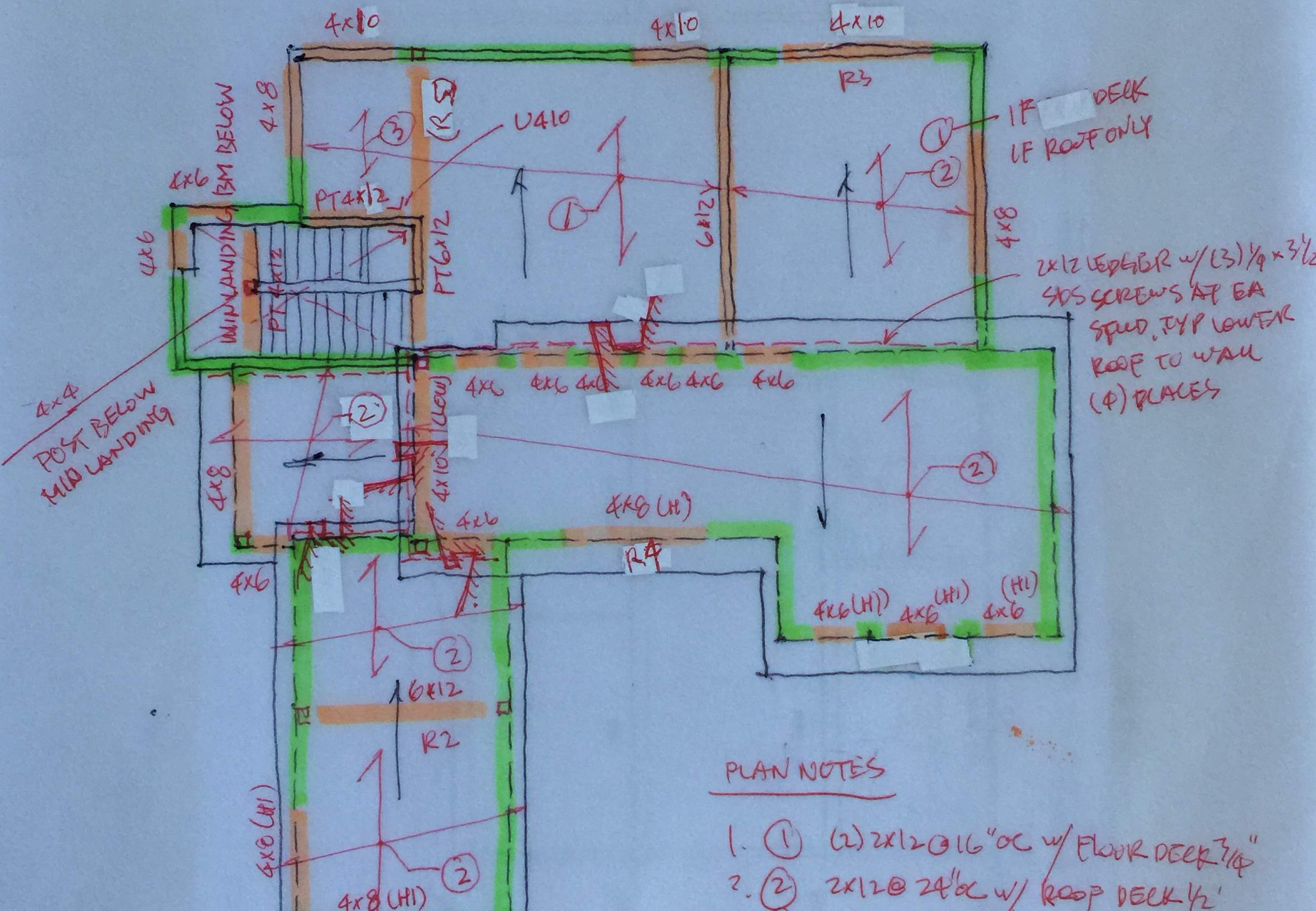
PROVIDE VERTICAL DOWELS OF SAME SIZE, NUMBER AND SPACING AS VERTICAL BARS.

7. GROUT – 5000 PSI MINIMUM 7-DAY CUBE STRENGTH PER ASTM C1157-00. GROUT TO BE PREMIXED, NON-SHRINK “MASTERFLOW 928 GROUT” BY MASTER BUILDERS OR APPROVED EQUAL. ICC CERTIFICATION REQUIRED. USE SPECIFIC GROUT MIX RECOMMENDED BY MANUFACTURER FOR EACH GROUT APPLICATION AND FOLLOW MANUFACTURER’S INSTRUCTIONS.

8. ANCHOR BOLTS, ASTM A307. SPECIAL INSPECTION REQUIRED. SET ALL ANCHOR BOLTS BY TEMPLATE WHEREVER POSSIBLE.

9. DRILL-IN EXPANSION BOLTS, “KWIK-BOLT TZ” BY HILTI FASTENING SYSTEMS BY HILTI FASTENING SYSTEM, OR APPROVED EQUAL. ICC CERTIFICATION REQUIRED (ERS-1917). SPECIAL INSPECTION REQUIRED.

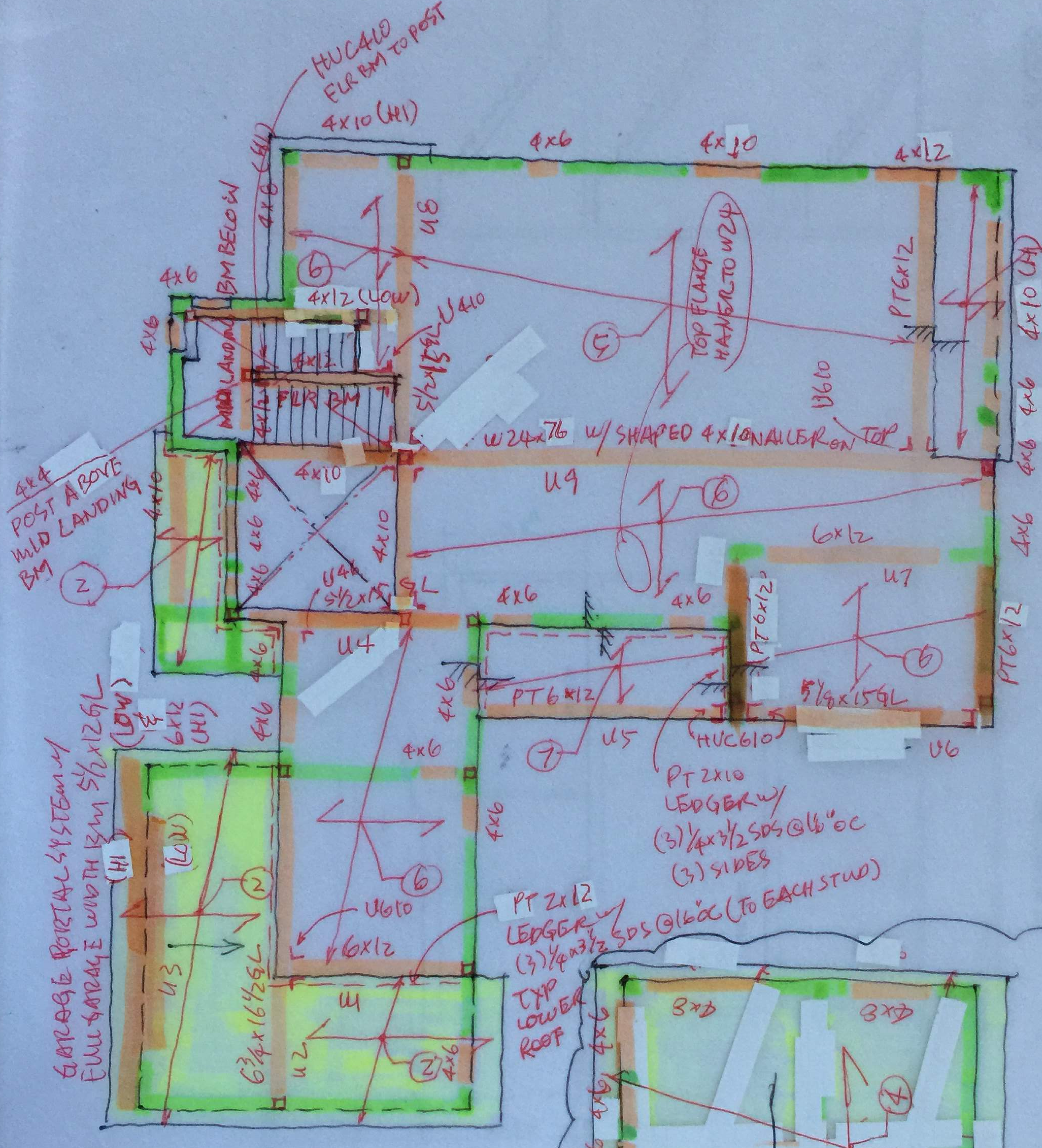
10. DRILL-IN ADHESIVE BOLTS, “HIT RE-500” ADHESIVE ANCHOR SYSTEM BY HILTI FASTENING SYSTEM, OR APPROVED EQUAL. ICC CERTIFICATION REQUIRED (ESR-2322). SPECIAL INSPECTION REQUIRED.

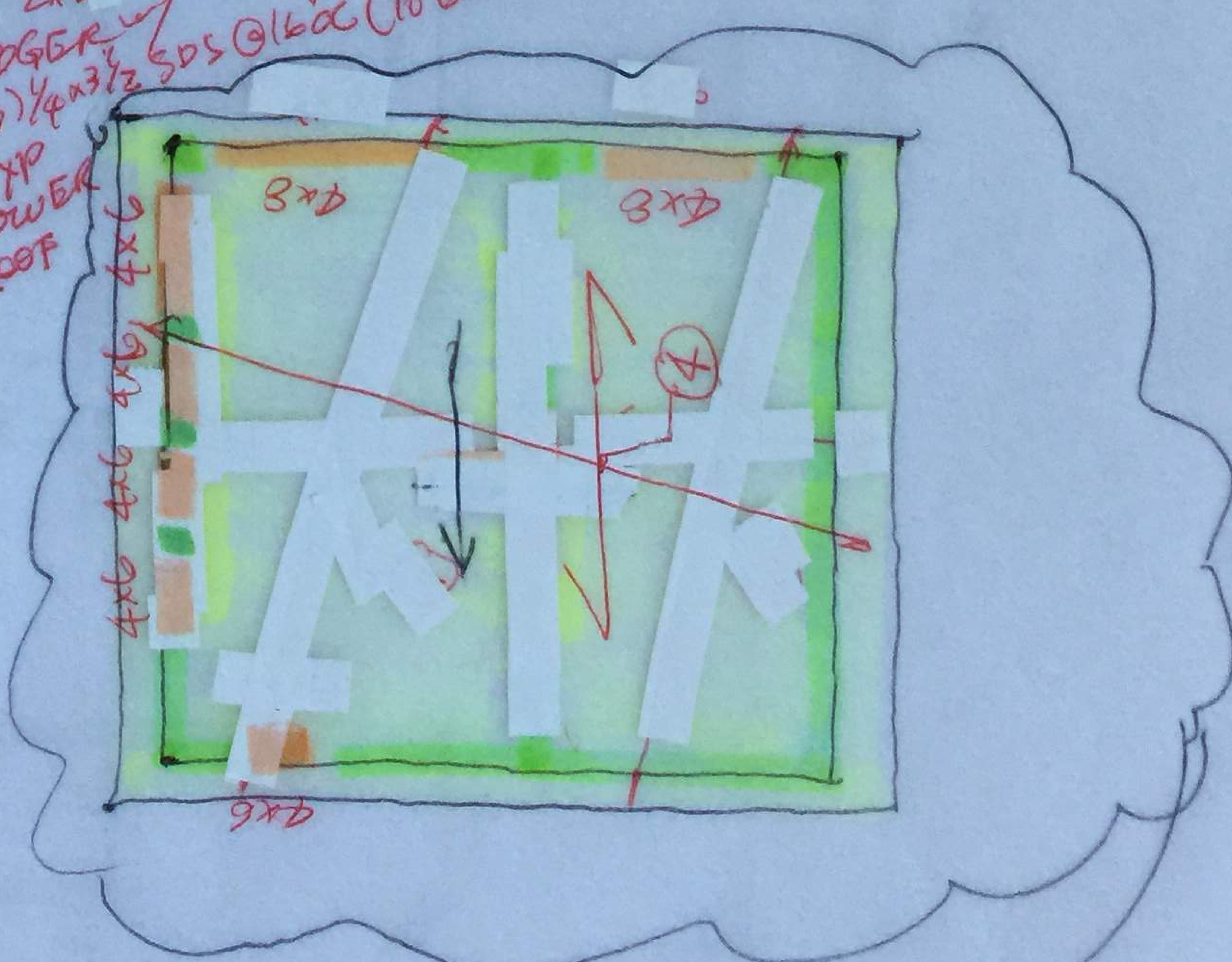


PLAN NOTES

1. ① (2) 2x12 @ 16" OC w/ FLOOR DECK 3/4"
2. ② 2x12 @ 24" OC w/ ROOF DECK 1/2"
3. ③ 2x12 @ 16" OC w/ FLOOR DECK 3/4"
4. ALL BM TOP FLUSH

ROOF FRAMING PLAN



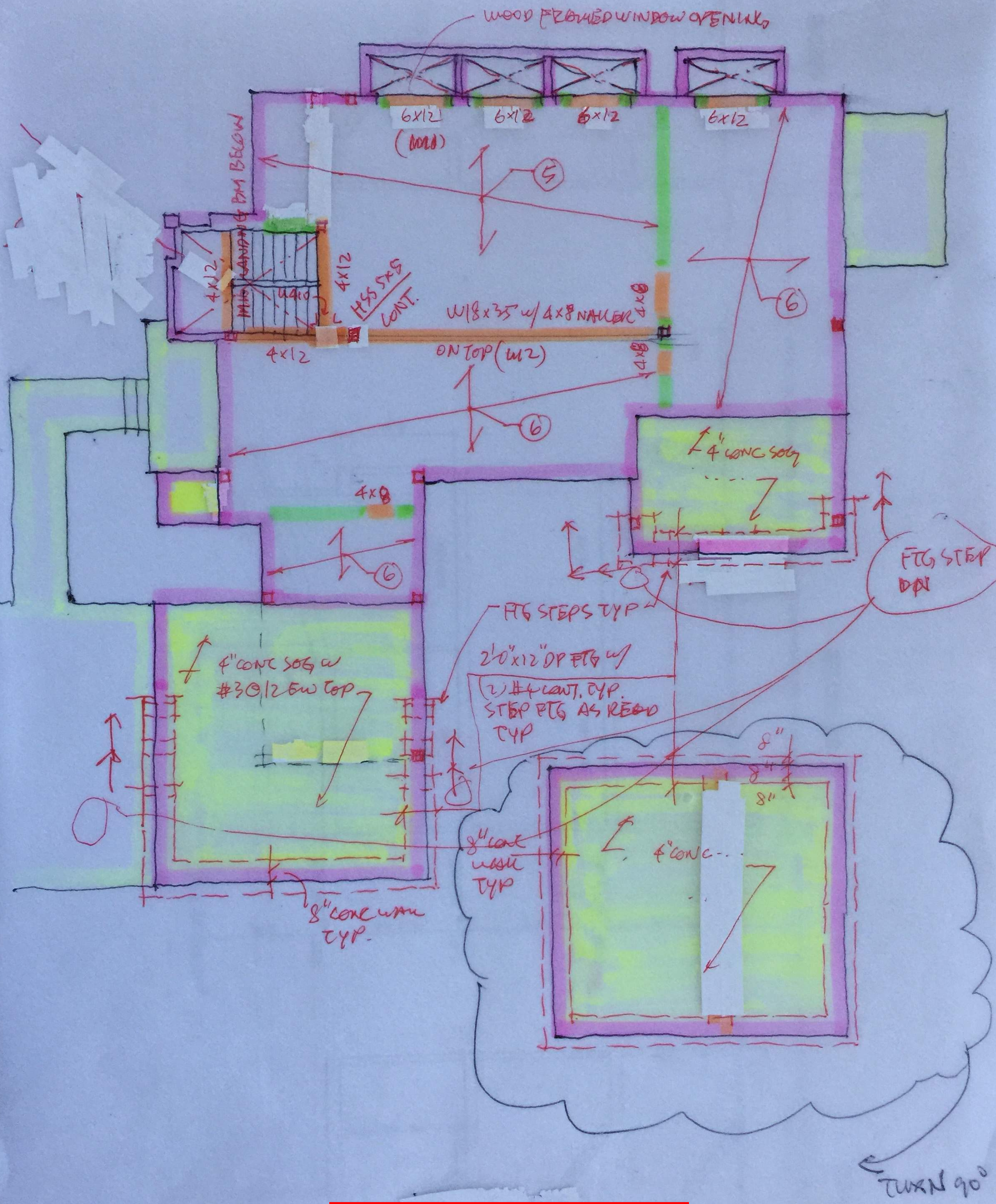


PLAN NOTES

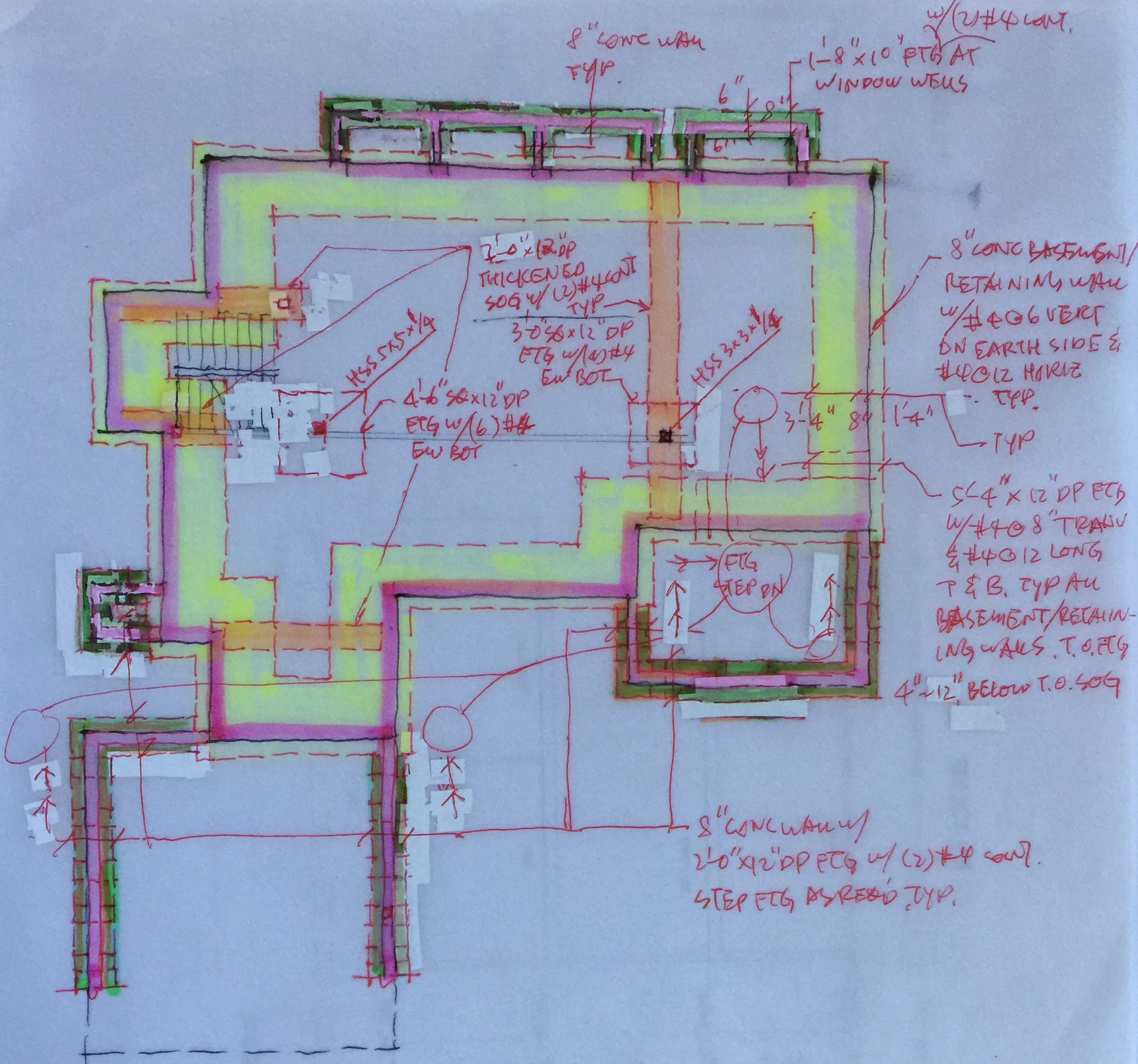
1. (2) 2x12@24"oc w/ ROOF DECK 1/2"
2. (4) 2x12@16"oc w/ ROOF DECK 1/2"
3. (5) 2x12@16"oc w/ FLOOR DECK 3/4"
4. (6) 2x10@16"oc w/ " "
5. (7) 2x10@16"oc w/ PT " "
6. ALL BM TOP FLUSH U.N.O.

TURN 90°

UPPER FLOOR & LOWER ROOF FRAMING PLAN



MAIN FLOOR FRAMING PLAN



BASEMENT & FOUNDATION PLAN

Joist, header, and beam calculations

*See Framing Plans for Beam Marks

Project Name				**Page No.**
Bm/Jst Location/Description:	**roof joists 2x12@24"**			
Roof				
dead load (psf)	15.00			
live load (psf)	25.00	additional total point load (kips)		0.00
tributary width (ft)	**2.00**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		0.00
tributary width (ft)	**0.00**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**0.00**			
Beam Span (ft)	**19.00**			
load duration/repetitive factor	**1.15**		**1.00**	
Beam Data Base Number	**5**		**2.0E PSL**	
tributary load (plf)	80.00		**#N/A**	**Beam No.61-88**
moment (kip-ft)	3.61		Provided M	#N/A
shear/reaction (kips)	0.76		Provided V	#N/A
			Provided I	#N/A
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	30.14	31.64	15.70	#N/A
Required I (in^4)	158.63	177.98	158.63	#N/A
Required A (in^2)	10.44	16.88	3.21	#N/A
Size	**2x12**	**Beam No.1-20**	**#N/A**	**Beam No.20-60**

Bm/Jst Location/Description:	**roof top deck (2)2x12@16"**			
Roof				
dead load (psf)	15.00			
live load (psf)	60.00	additional total point load (kips)		0.00
tributary width (ft)	**0.67**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		0.00
tributary width (ft)	**0.00**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**0.00**			
Beam Span (ft)	**19.00**			
load duration/repetitive factor	**1.00**		**1.00**	
Beam Data Base Number	**5**		**2.0E PSL**	
tributary load (plf)	50.25		**#N/A**	**Beam No.61-88**
moment (kip-ft)	2.27		Provided M	#N/A
shear/reaction (kips)	0.48		Provided V	#N/A
			Provided I	#N/A
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	21.77	31.64	11.34	#N/A
Required I (in^4)	99.64	177.98	99.64	#N/A
Required A (in^2)	7.54	16.88	4.34	#N/A
Size	**2x12**	**Beam No.1-20**	**#N/A**	**Beam No.20-60**

Project Name				Page No.
Bm/Jst Location/Description:	**roof top deck 2x12@16"**			
Roof				
dead load (psf)	15.00			
live load (psf)	60.00	additional total point load (kips)		0.00
tributary width (ft)	**1.33**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		0.00
tributary width (ft)	**0.00**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**0.00**			
Beam Span (ft)	**9.33**			
load duration/repetitive factor	**1.00**		**1.00**	
Beam Data Base Number	**5**		**2.0E PSL**	
tributary load (plf)	99.75		**#N/A**	**Beam No.61-88**
moment (kip-ft)	1.09		Provided M	#N/A
shear/reaction (kips)	0.47		Provided V	#N/A
			Provided I	#N/A
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	10.42	31.64	5.43	#N/A
Required I (in^4)	23.42	177.98	23.42	#N/A
Required A (in^2)	7.35	16.88	2.11	#N/A
Size	**2x12**	**Beam No.1-20**	**#N/A**	**Beam No.20-60**

Bm/Jst Location/Description:	**R1**			
Roof				
dead load (psf)	15.00			
live load (psf)	25.00	additional total point load (kips)		0.00
tributary width (ft)	**9.00**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		0.00
tributary width (ft)	**0.00**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**0.00**			
Beam Span (ft)	**7.00**			
load duration/repetitive factor	**1.00**		**1.00**	
Beam Data Base Number	**9**		**2.0E PSL**	
tributary load (plf)	360.00		**#N/A**	**Beam No.61-88**
moment (kip-ft)	2.21		Provided M	#N/A
shear/reaction (kips)	1.26		Provided V	#N/A
			Provided I	#N/A
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	21.17	30.66	11.03	#N/A
Required I (in^4)	35.70	111.15	35.70	#N/A
Required A (in^2)	19.90	25.38	5.73	#N/A
Size	**4x8**	**Beam No.1-20**	**#N/A**	**Beam No.20-60**

Project Name				Page No.
Bm/Jst Location/Description:	**R2**			
Roof				
dead load (psf)	15.00			
live load (psf)	25.00	additional total point load (kips)		0.00
tributary width (ft)	**12.00**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		0.00
tributary width (ft)	**0.00**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**0.00**			
Beam Span (ft)	**12.50**			
load duration/repetitive factor	**1.00**		**1.00**	
Beam Data Base Number	**17**		**2.0E PSL**	
tributary load (plf)	480.00		**#N/A**	**Beam No.61-88**
moment (kip-ft)	9.38		Provided M	#N/A
shear/reaction (kips)	3.00		Provided V	#N/A
			Provided I	#N/A
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	90.00	121.23	46.88	#N/A
Required I (in^4)	271.02	697.07	271.02	#N/A
Required A (in^2)	47.37	63.25	27.27	#N/A
Size	**6x12**	**Beam No.1-20**	**#N/A**	**Beam No.20-60**

Bm/Jst Location/Description:	**R3**			
Roof				
dead load (psf)	15.00			
live load (psf)	60.00	additional total point load (kips)		0.00
tributary width (ft)	**5.00**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		0.00
tributary width (ft)	**0.00**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**3.00**			
Beam Span (ft)	**9.00**			
load duration/repetitive factor	**1.00**		**1.00**	
Beam Data Base Number	**10**		**2.0E PSL**	
tributary load (plf)	405.00		**#N/A**	**Beam No.61-88**
moment (kip-ft)	4.10		Provided M	#N/A
shear/reaction (kips)	1.82		Provided V	#N/A
			Provided I	#N/A
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	39.37	49.91	20.50	#N/A
Required I (in^4)	85.35	230.84	85.35	#N/A
Required A (in^2)	28.78	32.38	8.28	#N/A
Size	**4x10**	**Beam No.1-20**	**#N/A**	**Beam No.20-60**

Project Name				Page No.
Bm/Jst Location/Description:	**R4**			
Roof				
dead load (psf)	15.00			
live load (psf)	25.00	additional total point load (kips)		0.00
tributary width (ft)	**7.00**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		0.00
tributary width (ft)	**0.00**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**0.00**	**both 2nd and 3r**		
Beam Span (ft)	**9.00**			
load duration/repetitive factor	**1.00**		**1.00**	
Beam Data Base Number	**9**		**2.0E PSL**	
tributary load (plf)	280.00		**#N/A**	**Beam No.61-88**
moment (kip-ft)	2.84		Provided M	#N/A
shear/reaction (kips)	1.26		Provided V	#N/A
			Provided I	#N/A
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	27.22	30.66	14.18	#N/A
Required I (in^4)	59.01	111.15	59.01	#N/A
Required A (in^2)	19.90	25.38	11.45	#N/A
Size	**4x8**	**Beam No.1-20**	**#N/A**	**Beam No.20-60**

Bm/Jst Location/Description:	**R5**			
Roof				
dead load (psf)	15.00			
live load (psf)	60.00	additional total point load (kips)		1.01
tributary width (ft)	**1.33**	point load location to farthest support (ft)		9.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		1.00
tributary width (ft)	**0.00**	point load location to farthest support (ft)		14.00
Wall				
wall weight (psf)	10.00			
height (ft)	**0.00**			
Beam Span (ft)	**18.50**			
load duration/repetitive factor	**1.00**		**1.00**	
Beam Data Base Number	**17**		**2.0E PSL**	
tributary load (plf)	99.75		**#N/A**	**Beam No.61-88**
moment (kip-ft)	12.35		Provided M	#N/A
shear/reaction (kips)	2.17		Provided V	#N/A
			Provided I	#N/A
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	118.58	121.23	61.76	#N/A
Required I (in^4)	528.50	697.07	528.50	#N/A
Required A (in^2)	34.30	63.25	9.87	#N/A
Size	**6x12**	**Beam No.1-20**	**#N/A**	**Beam No.20-60**

Project Name				**Page No.**
Bm/Jst Location/Description:	**garage roof joists 2x12@16"**			
Roof				
dead load (psf)	15.00			
live load (psf)	25.00	additional total point load (kips)		0.00
tributary width (ft)	**1.33**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		0.00
tributary width (ft)	**0.00**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**0.00**			
Beam Span (ft)	**21.00**			
load duration/repetitive factor	**1.00**		**1.00**	
Beam Data Base Number	**5**		**2.0E PSL**	
tributary load (plf)	53.20		**#N/A**	**Beam No.61-88**
moment (kip-ft)	2.93		Provided M	#N/A
shear/reaction (kips)	0.56		Provided V	#N/A
			Provided I	#N/A
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	28.15	31.64	14.66	#N/A
Required I (in^4)	142.43	177.98	142.43	#N/A
Required A (in^2)	8.82	16.88	5.08	#N/A
Size	**2x12**	**Beam No.1-20**	**#N/A**	**Beam No.20-60**

Bm/Jst Location/Description:	**typical floor joists**			
Roof				
dead load (psf)	15.00			
live load (psf)	25.00	additional total point load (kips)		0.00
tributary width (ft)	**0.00**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		0.00
tributary width (ft)	**1.33**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**0.00**			
Beam Span (ft)	**18.50**			
load duration/repetitive factor	**1.00**		**1.00**	
Beam Data Base Number	**5**		**2.0E PSL**	
tributary load (plf)	73.15		**#N/A**	**Beam No.61-88**
moment (kip-ft)	3.13		Provided M	#N/A
shear/reaction (kips)	0.68		Provided V	#N/A
			Provided I	#N/A
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	30.04	31.64	15.65	#N/A
Required I (in^4)	133.89	177.98	133.89	#N/A
Required A (in^2)	10.68	16.88	6.15	#N/A
Size	**2x12**	**Beam No.1-20**	**#N/A**	**Beam No.20-60**

Project Name				**Page No.**
Bm/Jst Location/Description:	**U1**			
Roof				
dead load (psf)	15.00			
live load (psf)	25.00	additional total point load (kips)		0.00
tributary width (ft)	**9.00**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		0.00
tributary width (ft)	**2.00**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**12.00**			
Beam Span (ft)	**12.50**			
load duration/repetitive factor	**1.00**		**1.00**	
Beam Data Base Number	**17**		**2.0E PSL**	
tributary load (plf)	590.00		**#N/A**	**Beam No.61-88**
moment (kip-ft)	11.52		Provided M	#N/A
shear/reaction (kips)	3.69		Provided V	#N/A
			Provided I	#N/A
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	110.63	121.23	57.62	#N/A
Required I (in^4)	333.13	697.07	333.13	#N/A
Required A (in^2)	58.23	63.25	16.76	#N/A
Size	**6x12**	**Beam No.1-20**	**#N/A**	**Beam No.20-60**

Bm/Jst Location/Description:	**U2**			
Roof				
dead load (psf)	15.00			
live load (psf)	25.00	additional total point load (kips)		0.00
tributary width (ft)	**0.00**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		3.69
tributary width (ft)	**10.50**	point load location to farthest support (ft)		13.00
Wall				
wall weight (psf)	10.00			
height (ft)	**12.00**			
Beam Span (ft)	**19.00**			
load duration/repetitive factor	**1.00**		**1.00**	
Beam Data Base Number	**50**		**2.0E PSL**	
tributary load (plf)	697.50		**#N/A**	**Beam No.61-88**
moment (kip-ft)	46.61		Provided M	#N/A
shear/reaction (kips)	9.15		Provided V	#N/A
			Provided I	#N/A
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	447.48	280.73	233.06	285.63
Required I (in^4)	2048.25	2456.38	2048.25	2576.82
Required A (in^2)	144.47	96.25	83.17	111.38
Size	**6x18**	**Beam No.1-20**	**6-3/4x16-1/2**	**Beam No.20-60**

Project Name				Page No.
Bm/Jst Location/Description:	**U3**			
Roof				
dead load (psf)	15.00			
live load (psf)	25.00	additional total point load (kips)		0.00
tributary width (ft)	**4.25**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		0.00
tributary width (ft)	**0.00**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**4.00**			
Beam Span (ft)	**16.00**			
load duration/repetitive factor	**1.00**		**1.00**	
Beam Data Base Number	**11**		**2.0E PSL**	
tributary load (plf)	210.00		**#N/A**	**Beam No.61-88**
moment (kip-ft)	6.72		Provided M	#N/A
shear/reaction (kips)	1.68		Provided V	#N/A
			Provided I	#N/A
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	64.51	73.83	33.60	#N/A
Required I (in^4)	248.66	415.28	248.66	#N/A
Required A (in^2)	26.53	39.38	7.64	#N/A
Size	**4x12**	**Beam No.1-20**	**#N/A**	**Beam No.20-60**

Bm/Jst Location/Description:	**U4**			
Roof				
dead load (psf)	15.00			
live load (psf)	25.00	additional total point load (kips)		0.00
tributary width (ft)	**7.00**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		1.00
tributary width (ft)	**5.00**	point load location to farthest support (ft)		12.00
Wall				
wall weight (psf)	10.00			
height (ft)	**12.00**			
Beam Span (ft)	**15.00**			
load duration/repetitive factor	**1.00**		**1.00**	
Beam Data Base Number	**35**		**2.0E PSL**	
tributary load (plf)	675.00		**#N/A**	**Beam No.61-88**
moment (kip-ft)	21.38		Provided M	#N/A
shear/reaction (kips)	5.86		Provided V	#N/A
			Provided I	#N/A
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	205.29	280.73	106.92	123.00
Required I (in^4)	741.84	2456.38	741.84	738.00
Required A (in^2)	92.57	96.25	26.65	61.50
Size	**6x18**	**Beam No.1-20**	**5-1/8x12**	**Beam No.20-60**

Project Name				Page No.
Bm/Jst Location/Description:	**U5**			
Roof				
dead load (psf)	15.00			
live load (psf)	25.00	additional total point load (kips)		0.00
tributary width (ft)	**0.00**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00			
live load (psf)	60.00	additional total point load (kips)		0.00
tributary width (ft)	**3.50**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**4.00**			
Beam Span (ft)	**17.00**			
load duration/repetitive factor	**1.00**		**1.00**	

Beam Data Base Number	**17**		**2.0E PSL**	
tributary load (plf)	302.50		**#N/A**	**Beam No.61-88**
moment (kip-ft)	10.93		Provided M	#N/A
shear/reaction (kips)	2.57		Provided V	#N/A
			Provided I	#N/A
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	104.91	121.23	54.64	#N/A
Required I (in^4)	472.61	697.07	472.61	#N/A
Required A (in^2)	40.60	63.25	23.37	#N/A
Size	**6x12**	**Beam No.1-20**	**#N/A**	**Beam No.20-60**

Bm/Jst Location/Description:	**U6**			
Roof				
dead load (psf)	15.00			
live load (psf)	25.00	additional total point load (kips)		0.00
tributary width (ft)	**11.00**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		0.00
tributary width (ft)	**6.00**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**12.00**			
Beam Span (ft)	**16.50**			
load duration/repetitive factor	**1.00**		**1.00**	

Beam Data Base Number	**36**		**2.0E PSL**	
tributary load (plf)	890.00		**#N/A**	**Beam No.61-88**
moment (kip-ft)	30.29		Provided M	#N/A
shear/reaction (kips)	7.34		Provided V	#N/A
			Provided I	#N/A
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	290.76	280.73	151.44	163.65
Required I (in^4)	1155.78	2456.38	1155.78	1050.79
Required A (in^2)	115.94	96.25	33.37	69.19
Size	**6x18**	**Beam No.1-20**	**5-1/8x13-1/2**	**Beam No.20-60**

Project Name				Page No.
Bm/Jst Location/Description:	**U7**			
Roof				
dead load (psf)	15.00			
live load (psf)	25.00	additional total point load (kips)		0.00
tributary width (ft)	**0.00**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		0.00
tributary width (ft)	**9.00**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**0.00**			
Beam Span (ft)	**14.00**			
load duration/repetitive factor	**1.00**		**1.00**	

Beam Data Base Number	**17**		**2.0E PSL**	
tributary load (plf)	495.00		**#N/A**	**Beam No.61-88**
moment (kip-ft)	12.13		Provided M	#N/A
shear/reaction (kips)	3.47		Provided V	#N/A
			Provided I	#N/A
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	116.42	121.23	60.64	#N/A
Required I (in^4)	392.67	697.07	392.67	#N/A
Required A (in^2)	54.71	63.25	31.50	#N/A
Size	**6x12**	**Beam No.1-20**	**#N/A**	**Beam No.20-60**

Bm/Jst Location/Description:	**U8**			
Roof				
dead load (psf)	15.00			
live load (psf)	25.00	additional total point load (kips)		1.01
tributary width (ft)	**0.00**	point load location to farthest support (ft)		9.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		1.00
tributary width (ft)	**4.00**	point load location to farthest support (ft)		14.00
Wall				
wall weight (psf)	10.00			
height (ft)	**0.00**			
Beam Span (ft)	**18.50**			
load duration/repetitive factor	**1.00**		**1.00**	

Beam Data Base Number	**35**		**2.0E PSL**	
tributary load (plf)	220.00		**#N/A**	**Beam No.61-88**
moment (kip-ft)	17.50		Provided M	#N/A
shear/reaction (kips)	3.28		Provided V	#N/A
			Provided I	#N/A
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	167.97	280.73	87.48	123.00
Required I (in^4)	748.60	2456.38	748.60	738.00
Required A (in^2)	51.86	96.25	14.93	61.50
Size	**6x18**	**Beam No.1-20**	**5-1/8x12**	**Beam No.20-60**

Project Name				Page No.
Bm/Jst Location/Description:	**U9 Steel Girder**			
Roof				
dead load (psf)	15.00			
live load (psf)	25.00	additional total point load (kips)		0.00
tributary width (ft)	**14.50**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		0.00
tributary width (ft)	**14.50**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**12.00**			
Beam Span (ft)	**38.50**			
load duration/repetitive factor	**1.00**		**1.00**	
Beam Data Base Number	**5**			
tributary load (plf)	1497.50			**W24x76**
moment (kip-ft)	277.46	Sr	141.50	176.00
shear/reaction (kips)	28.83	Ir	1986.88	2100.00
				bf=9"
			shaped 4x10 nailer on top	

Bm/Jst Location/Description:	**M1**			
Roof				
dead load (psf)	15.00			
live load (psf)	25.00	additional total point load (kips)		0.00
tributary width (ft)	**9.50**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00	**both main and upper floors**		
live load (psf)	40.00	additional total point load (kips)		0.00
tributary width (ft)	**19.00**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**24.00**			
Beam Span (ft)	**6.00**			
load duration/repetitive factor	**1.00**		**1.00**	
Beam Data Base Number	**17**		**2.0E PSL**	
tributary load (plf)	1665.00		**#N/A**	**Beam No.61-88**
moment (kip-ft)	7.49		Provided M	#N/A
shear/reaction (kips)	5.00		Provided V	#N/A
			Provided I	#N/A
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	71.93	121.23	37.46	#N/A
Required I (in^4)	103.97	697.07	103.97	#N/A
Required A (in^2)	59.15	63.25	45.40	#N/A
Size	**6x12**	**Beam No.1-20**	**#N/A**	**Beam No.20-60**

Project Name				Page No.
Bm/Jst Location/Description:	**M2**			
Roof				
dead load (psf)	15.00			
live load (psf)	25.00	additional total point load (kips)		0.00
tributary width (ft)	**0.00**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		0.00
tributary width (ft)	**14.50**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**12.00**			
Beam Span (ft)	**25.00**			
load duration/repetitive factor	**1.00**		**1.00**	
Beam Data Base Number	**5**			
tributary load (plf)	917.50			**W18x35**
moment (kip-ft)	71.68	Sr	36.56	57.60
shear/reaction (kips)	11.47	Ir	333.31	510.00
footing				bf=6"
			shaped 4x8 nailer on top	
reaction from U9	28.83			
reaction from M2	11.47			
4.5x4.5x2ksf=40.5 kips	**40.30**	**kips**		

Typical Wall Stud Calculations (see wind force calculations for lateral forces on stud)

INPUT DATA

MEMBER TYPE: 2

1	POST
2	WALL STUD
3	KING STUD

GEOMETRY DATA:

HEIGHT	h =	12	ft
UNBRACED LENGTH	Le x-x (H) =	12	ft
	Le y-y (B) =	2	ft

LOAD DATA:

DEAD LOAD		400	lbs
LIVE LOAD		1000	lbs
TOTAL		1,400	lbs
LATERAL LOAD x-x		30	plf
	M=	540	ft-lbs
	V=	180	lbs
LOAD DURATION		2	OCCUPANCY LIVE LOAD

DESIGN CRITERIA:

SECTION	1 pcs, B =	2	in
	H =	6	in
SPECIES (1 = DFL, 2 = SP)		1	DOUGLAS FIR-LARCH
GRADE (1, 2, 3, 4, 5, or 6)		4	No. 2
WET / DRY USE ? (1 = DRY, 2 = WE		1	DRY

DESIGN SUMMARY

USE: 1 - 2" x 6" DOUGLAS FIR-LARCH No. 2

1. CHECK VERTICAL LOADS : $f_c < F_c'$?

170 psi < 610 psi ok

2. CHECK BENDING LOADS : $f_b < F_b'$?

857 psi < 1346 psi ok

3. CHECK INTERACTION : $\left(\frac{f_c}{F_c'}\right)^2 + \left(\frac{1}{1 - f_c / F_{cEx}}\right)\frac{f_{bx}}{F_{bx}'} \leq 1$?

0.920 < 1 ok

4. CHECK SHEAR LOADS : $f_v < F_v'$?

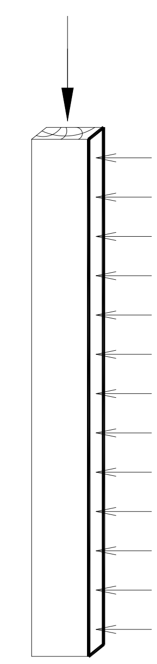
33 psi < 180 psi ok

5. HORIZONTAL DEFLECTION AT MIDDLE

$\Delta = 5wh^4 / (384EI) + 2.4wh^2 / (Ebd) =$ 3/7 in

(h / 335)

ANALYSIS



COLUMN BASIC DESIGN STRESSES:

COMPRESSIVE STRESS	F_c =	1350	psi
MODULUS OF ELASTICITY	E =	1600	ksi
BENDING STRESS (X-Axis)	F_{bx} =	900	psi
BENDING STRESS (Y-Axis)	F_{by} =	900	psi
SHEAR STRESS (X-Axis)	F_v =	180	psi

COLUMN PROPERTIES:

COLUMN SECTION	X-Dir	dx =	5.50	in
	Y-Dir	dy =	1.50	in
AREA		A =	8.25	in^2
SECTION PROPERTIES	Abt. xx	Sx =	7.56	in^3
		Ix =	20.80	in^4
	Abt. yy	Sy =	2.06	in^3
LENGTH-DEPTH RATIO		Le x-x / dx =	26.2	
		Le y-y / dy =	16.0	

ADJUSTMENT FACTORS:

		F_{bx}'	F_{by}'	F_c'	F_v'	E'
DURATION FACTOR	C_D	1.00	1.00	1.00	1.00	
MOISTURE FACTOR	C_M	1.00	1.00	1.00	1.00	1.00
TEMPERATURE FACTOR	C_t	1.00	1.00	1.00	1.00	1.00
INCISING FACTOR	C_i	1.00	1.00	1.00	1.00	1.00
SIZE FACTOR	C_F	1.30	1.30	1.10		1.00
FLAT USE FACTOR	C_{fu}		1.15			
COLUMN STABILITY	C_P			0.411		
REPETITIVE MEMBER	C_r	1.15	1.15			
BEAM STABILITY	C_L	1.00	1.00			

COLUMN PARAMETER	c =	0.80	
MODULUS OF ELASTICITY	E'_{min} =	580	ksi

CRITICAL EULER BUCKLING VALUES

F_{cE} =	696	psi
F_c^* =	1485	psi

ADJUSTED PROPERTIES:

MODULUS OF ELASTICITY	E' =	1600	ksi
BENDING STRESS (X-Axis)	F_{bx}' =	1346	psi
BENDING STRESS (Y-Axis)	F_{by}' =	1547	psi
AXIAL STRESS	F_c' =	610	psi
SHEAR STRESS	F_v' =	180	psi

ACTUAL STRESSES:

AXIAL STRESS	f_c =	169.7	psi
BENDING STRESSES	f_{bx} =	856.9	psi
SHEAR STRESS	f_v =	33	psi

Typical Wall Stud Calculations (where applicable to this project)

INPUT DATA

MEMBER TYPE: 2

1	POST
2	WALL STUD
3	KING STUD

GEOMETRY DATA:

HEIGHT	h =	21	ft
UNBRACED LENGTH	Le x-x (H) =	21	ft
	Le y-y (B) =	2	ft

LOAD DATA:

DEAD LOAD		300	lbs
LIVE LOAD		750	lbs
TOTAL		1,050	lbs
LATERAL LOAD x-x		30	plf
	M=	1654	ft-lbs
	V=	315	lbs
LOAD DURATION		2	OCCUPANCY LIVE LOAD

DESIGN CRITERIA:

SECTION	1 pcs, B =	2	in
	H =	10	in
SPECIES (1 = DFL, 2 = SP)		1	DOUGLAS FIR-LARCH
GRADE (1, 2, 3, 4, 5, or 6)		4	No. 2
WET / DRY USE ? (1 = DRY, 2 = WE		1	DRY

DESIGN SUMMARY

USE: 1 - 2" x 10" DOUGLAS FIR-LARCH No. 2

1. CHECK VERTICAL LOADS : $f_c < F_c'$?
   76 psi < 562 psi ok

2. CHECK BENDING LOADS : $f_b < F_b'$?
   928 psi < 1139 psi ok

3. CHECK INTERACTION : $\left(\frac{f_c}{F_c'}\right)^2 + \left(\frac{1}{1 - f_c/F_{cEx}}\right)\frac{f_{bx}}{F_{bx}'} \le 1$?
   0.942 < 1 ok

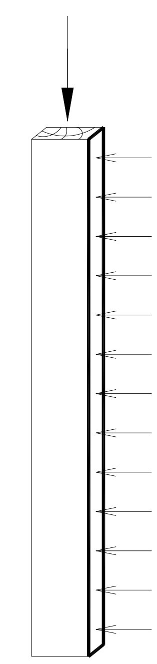
4. CHECK SHEAR LOADS : $f_v < F_v'$?
   34 psi < 180 psi ok

5. HORIZONTAL DEFLECTION AT MIDDLE
   $\Delta = 5wh^4 / (384EI) + 2.4wh^2 / (Ebd) =$ 6/7 in
   (h / 298)

ANALYSIS

COLUMN BASIC DESIGN STRESSES:

COMPRESSIVE STRESS	F_c =	1350	psi
MODULUS OF ELASTICITY	E =	1600	ksi
BENDING STRESS (X-Axis)	F_{bx} =	900	psi
BENDING STRESS (Y-Axis)	F_{by} =	900	psi
SHEAR STRESS (X-Axis)	F_v =	180	psi



COLUMN PROPERTIES:

COLUMN SECTION	X-Dir	dx =	9.25	in
	Y-Dir	dy =	1.50	in
AREA		A =	13.875	in^2
SECTION PROPERTIES	Abt. xx	Sx =	21.39	in^3
		Ix =	98.93	in^4
	Abt. yy	Sy =	3.47	in^3
LENGTH-DEPTH RATIO		Le x-x / dx =	27.2	
		Le y-y / dy =	16.0	

ADJUSTMENT FACTORS:

		F_{bx}'	F_{by}'	F_c'	F_v'	E'
DURATION FACTOR	C_D	1.00	1.00	1.00	1.00	
MOISTURE FACTOR	C_M	1.00	1.00	1.00	1.00	1.00
TEMPERATURE FACTOR	C_t	1.00	1.00	1.00	1.00	1.00
INCISING FACTOR	C_i	1.00	1.00	1.00	1.00	1.00
SIZE FACTOR	C_F	1.10	1.10	1.00		1.00
FLAT USE FACTOR	C_{fu}		1.20			
COLUMN STABILITY	C_P			0.416		
REPETITIVE MEMBER	C_r	1.15	1.15			
BEAM STABILITY	C_L	1.00	1.00			

COLUMN PARAMETER	c =	0.80	
MODULUS OF ELASTICITY	E'_{min} =	580	ksi
CRITICAL EULER BUCKLING VALUES			
	F_{cE} =	642	psi
	F_c^* =	1350	psi

ADJUSTED PROPERTIES:

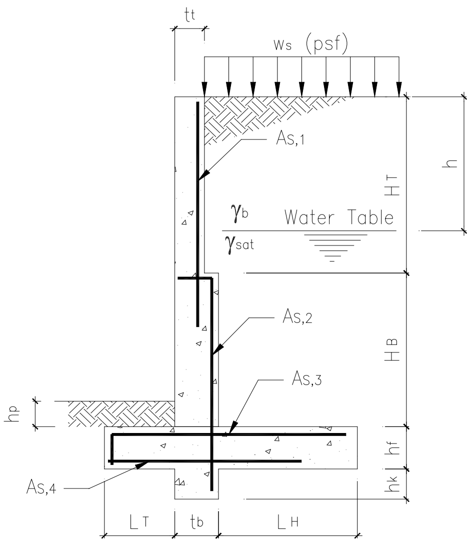
MODULUS OF ELASTICITY	E' =	1600	ksi
BENDING STRESS (X-Axis)	F_{bx}' =	1139	psi
BENDING STRESS (Y-Axis)	F_{by}' =	1366	psi
AXIAL STRESS	F_c' =	562	psi
SHEAR STRESS	F_v' =	180	psi

ACTUAL STRESSES:

AXIAL STRESS	f_c =	75.7	psi
BENDING STRESSES	f_{bx} =	927.7	psi
SHEAR STRESS	f_v =	34	psi

Basement/Retaining Wall Design

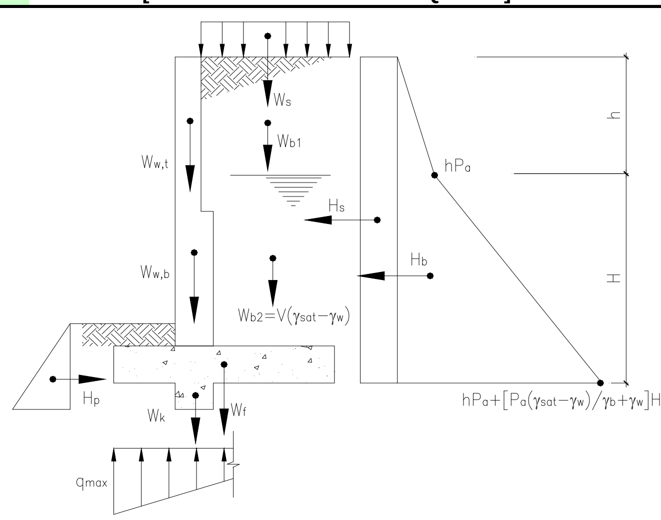
INPUT DATA & DESIGN SUMMARY



CONCRETE STRENGTH	f_c'	=	2.5	ksi		
REBAR YIELD STRESS	f_y	=	60	ksi		
LATERAL SOIL PRESSURE	$P_a = k_a \gamma_b$	=	35	pcf (equivalent fluid pressure)		
BACKFILL SPECIFIC WEIGHT	γ_b	=	110	pcf		
SATURATED SPECIFIC WEIGHT	γ_{sat}	=	118	pcf		
WATER TABLE DEPTH	h	=	10	ft		
PASSIVE PRESSURE	P_p	=	300	psf / ft		
SURCHARGE WEIGHT	w_s	=	40	psf		
FRICTION COEFFICIENT	μ	=	1	no sliding due to SOG		
ALLOW SOIL PRESSURE	Q_a	=	2.16	ksf =1.5ksf+0.11kcfx6ft		
THICKNESS OF TOP STEM	t_t	=	8	in		
THICKNESS OF KEY & STEM	t_b	=	8	in		
TOE WIDTH	L_T	=	3.33	ft		
HEEL WIDTH	L_H	=	1.33	ft		
HEIGHT OF TOP STEM	H_T	=	9.67	ft		
HEIGHT OF BOT. STEM	H_B	=	0.33	ft		
FOOTING THICKNESS	h_f	=	12	in		
KEY DEPTH	h_k	=	0	in		
SOIL OVER TOE	h_p	=	4	in		
TOP STEM REINF. ($A_{s,1}$)		#	4	@	6	in o.c.
$A_{s,1}$ LOCATION (0=at soil face, 1=at middle, 2=at each face)					0	at soil face
BOT. STEM REINF. ($A_{s,2}$)		#	4	@	6	in o.c.
$A_{s,2}$ LOCATION (0=at soil face, 1=at middle, 2=at each face)					0	at soil face
TOP REINF.OF FOOTING ($A_{s,3}$)		#	4	@	8	in o.c.
BOT. REINF.OF FOOTING ($A_{s,4}$)		#	4	@	8	in

[THE WALL DESIGN IS ADEQUATE.]

ANALYSIS



SERVICE LOADS

$H_b = 0.5\, P_a\, h^2 + h\, P_a\, H + 0.5\, [P_a\, (\gamma_{sat} - \gamma_w) / \gamma_b + \gamma_w]\, H^2$ = 2.14 kips

Where h = 10 ft , H = 1 ft

$H_s = w_s\, P_a\, (H_T + H_B + h_f) / \gamma_b$ = 0.14 kips

$H_p = 0.5\, P_p\, (h_p + h_f + h_k)^2$ = 0.27 kips

$W_s = w_s\, (L_H + t_b - t_t)$ = 0.05 kips

$W_b = W_{b1} + W_{b2}$ = 1.46 kips

Where W_{b1} = 1.46 kips , W_{b2} = 0.00 kips

$W_f = h_f\, (L_H + t_b + L_T)\, \gamma_c$ = 0.80 kips

$W_k = h_k\, t_b\, \gamma_c$ = 0.00 kips

$W_{w,t} = t_t\, H_T\, \gamma_c$ = 0.97 kips

$W_{w,b} = t_b\, H_B\, \gamma_c$ = 0.03 kips

FACTORED LOADS

$\gamma H_b = 1.6\, H_b$ = 3.42 kips

$\gamma H_s = 1.6\, H_s$ = 0.22 kips

$\gamma W_s = 1.6\, W_s$ = 0.09 kips

$\gamma W_b = 1.2\, W_b$ = 1.76 kips

$\gamma W_f = 1.2\, W_f$ = 0.96 kips

$\gamma W_k = 1.2\, W_k$ = 0.00 kips

$\gamma W_{w,t} = 1.2\, W_{w,t}$ = 1.16 kips

$\gamma W_{w,b} = 1.2\, W_{w,b}$ = 0.04 kips

OVERTURNING MOMENT

	H	γH	y	H y	γH y
H_b	2.14	3.42	3.63	7.77	12.43
H_s	0.14	0.22	5.50	0.77	1.23
Σ	2.28	3.65		**8.54**	13.67

RESISTING MOMENT

	W	γW	x	W x	γW x
W_s	0.05	0.09	4.66	0.25	0.40
W_b	1.46	1.76	4.66	6.82	8.18
W_f	0.80	0.96	2.66	2.13	2.55
W_k	0.00	0.00	3.66	0.00	0.00
$W_{w,t}$	0.97	1.16	3.66	3.54	4.25
$W_{w,b}$	0.03	0.04	3.66	0.12	0.15
Σ	3.32	4.00		**12.86**	15.53

OVERTURNING FACTOR OF SAFETY (1806.1)

$$SF = \frac{\Sigma Wx}{\Sigma Hy} = 1.51 > 1.5$$

[Satisfactory]

(cont'd)

CHECK SOIL BEARING CAPACITY (ACI 318-05 SEC.15.2.2)

$$L = L_T + t_b + L_H \quad = \quad 5.33 \text{ ft}$$

$$e = \frac{L}{2} - \frac{\Sigma W x - H y}{\Sigma W} \quad = \quad 1.36 \text{ ft}$$

$$q_{MAX} = \begin{cases} \dfrac{\Sigma W\left(1+\dfrac{6e}{L}\right)}{BL}, & for \quad e \le \dfrac{L}{6} \\ \dfrac{2\Sigma W}{3B(0.5L-e)}, & for \quad e > \dfrac{L}{6} \end{cases} \quad = \quad 1.70 \text{ ksf} \quad < \quad Q_a \quad \textbf{[Satisfactory]}$$

CHECK FLEXURE CAPACITY, AS,1 & AS,2, FOR STEM (ACI 318-05 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

h = 10 ft , H' = 0 ft

A = $w_s P_a / \gamma_b$ = 13 plf

B = $h P_a$ = 350 plf

C = $[P_a (\gamma_{sat} - \gamma_W) / \gamma_b + \gamma_W]$ H' = 0 plf

At base of top stem

M_u = 9.39 ft-kips

V_u = 2.82 kips

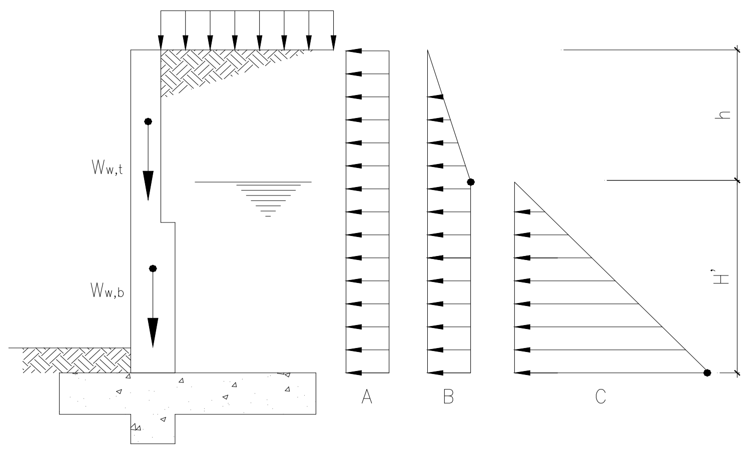
P_u = 1.16 kips

At base of bottom stem

M_u = 10.35 ft-kips

V_u = 3.00 kips

P_u = 1.20 kips



			At top stem		At base of bottom stem	
$\phi M_n = \phi\left[A_s f_y\left(d - \dfrac{A_s f_y - P_u}{1.7 b f'_c}\right)\right]$		=	10.44	ft-kips ,	10.45	ft-kips
			>	M_u	>	M_u
				[Satisfactory]		**[Satisfactory]**
where	d	=	6.25	in ,	6.25	in
	b	=	12	in ,	12	in
	ϕ	=	0.9	(ACI 318 Fig R9.3.2)	0.9	(ACI 318 Fig R9.3.2)
	A_s	=	0.4	in^2 ,	0.4	in^2
	ρ	=	0.005		0.005	
$\rho_{MAX} = \dfrac{0.85\beta_1 f'_c}{f_y}\dfrac{\varepsilon u}{\varepsilon u + \varepsilon t}$		=	0.013		0.013	
			>	ρ	>	ρ
				[Satisfactory]		**[Satisfactory]**
$\rho_{MIN} = 0.0018\dfrac{t}{d}$		=	0.002		0.002	
			<	ρ	<	ρ
				[Satisfactory]		**[Satisfactory]**

CHECK SHEAR CAPACITY FOR STEM (ACI 318-05 SEC.15.5.2, 11.1.3.1, & 11.3)

		At top stem		At base of bottom stem	
$V_{allowable} = 2\phi b d\sqrt{f'_c}$	=	5.63	kips ,	5.63	kips
		>	V_u	>	V_u
			[Satisfactory]		**[Satisfactory]**

where ϕ = 0.75 (ACI 318-05, Section 9.3.2.3)

CHECK HEEL FLEXURE CAPACITY, AS,3, FOR FOOTING (ACI 318-05 SEC.15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, & 12.5)

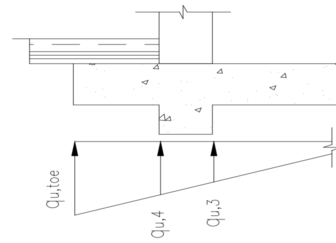
$$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon u}{\varepsilon u + \varepsilon t} \quad = \quad 0.013$$

$$\rho_{MIN} = \frac{0.0018}{2}\frac{h_f}{d} \quad = \quad 0.001$$

$$M_{u,3} = \begin{cases} \dfrac{L_H}{2}\left(\gamma w_s + w_b + \dfrac{L_H}{L} w_f\right) - \dfrac{\left(q_{u,3} + 2q_{u,heel}\right)b L_H^2}{6}, & for \quad e_u \le \dfrac{L}{6} \\ \dfrac{L_H}{2}\left(\gamma w_s + w_b + \dfrac{L_H}{L} w_f\right) - \dfrac{q_{u,3} b S^2}{6}, & for \quad e_u > \dfrac{L}{6} \end{cases} \quad = \quad 13.36 \text{ ft-kips}$$

(cont'd)

$$\rho = \frac{0.85 f_c^{'}\left(1-\sqrt{1-\frac{M_{u,3}}{0.383 b d^2 f_c^{'}}}\right)}{f_y} = 0.002$$



where d	=	10.25	in	$q_{u,\ toe}$	=	5.72	ksf
e_u	=	2.20	ft	$q_{u,\ heel}$	=	n/a	ksf
S	=	-2.60	ft	$q_{u,\ 3}$	=	-10.64	ksf

$(A_{S,\ 3})_{required}$ = 0.30 in^2 / ft **<** $A_{S,\ 3}$ **[Satisfactory]**

CHECK TOE FLEXURE CAPACITY, AS,4, FOR FOOTING (ACI 318-05 SEC.15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = \frac{0.85\beta_1 f_c^{'}}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t} = 0.013 \qquad \rho_{MIN} = MIN\left(\frac{4}{3}\rho,\ \frac{0.0018}{2}\frac{h_f}{d}\right) = 0.001$$

$$M_{u,4} = \frac{\left(q_{u,4}+2q_{u,toe}\right)bL_T^2}{6} - \frac{L_T^2}{2L}\gamma w_f = 5.54 \text{ ft-kips}$$

where	d	=	8.75	in
	$q_{u,\ 4}$	=	-7.91	ksf

$$\rho = \frac{0.85 f_c^{'}\left(1-\sqrt{1-\frac{M_{u,4}}{0.383 b d^2 f_c^{'}}}\right)}{f_y} = 0.001$$

$(A_{S,\ 4})_{required}$ = 0.14 in^2 / ft **<** $A_{S,\ 4}$ **[Satisfactory]**

CHECK SLIDING CAPACITY (IBC 1806.1)

1.5 (H_b + H_s) = 3.42 kips **<** H_p + μ ΣW = 3.58 kips
[Satisfactory]

Technical References:

1. Alan Williams: "Structural Engineering Reference Manual", Professional Publications, Inc, 2001.
2. Alan Williams: "Structural Engineering License Review Problems and Solutions", Oxford University Press, 2003.

Seismic Mass Calculation

Seismic Base at Main Floor with Concrete Base

Floor areas (sqft)

2nd	2180	
roof	1800	

Roof Framing Seismic Mass (psf)

roof framing	14.00	
roofing (4.00 psf future PV panels)	6.00	
wall framing to diaphragm	5.00	
total	**25.00**	psf

Floor Framing Seismic Mass (psf)

floor framing	15.00	
wall framing to diaphragm	10.00	
total	**25.00**	psf

2nd

seismic mass (area x floor framing seismic mass) **54.50 kips**

roof

seismic mass (area x roof framing seismic mass) **45.00 kips**

Seismic Forces

(per attached spreadsheet calculations)

roof	9.00	kips
2nd	6.20	
total	15.20	kips

ASD = Seismic Force/1.4

roof	6.43	
2nd	4.43	
total	10.86	kips

NS Cumulative	*EW* Cumulative
6.43 kips	**6.43 kips**
10.86 kips	10.86 kips

Wind Forces

(per attached spreadsheet calculations)

NS	17.77	kips	1.12
EW	19.83	kips	

NS

roof = ((3'+12'/2)/28') x 17.77 kips	5.71	
2nd = (((12'+13')/2)/28') x 17.77 kips	7.93	
total	13.64	kips

EW

roof = ((3'+12'/2)/28') x 19.83 kips	6.37	
2nd = (((12'+13')/2)/28') x 19.83 kips	8.85	
total	15.22	kips

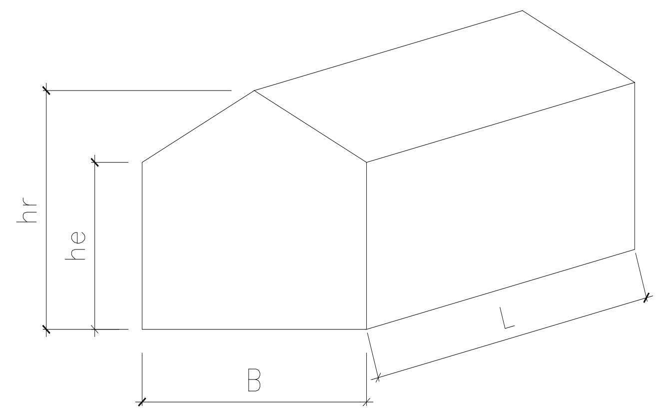
NS Cumulative	*EW* Cumulative
5.71 kips	6.37 kips
13.64 kips	**15.22 kips**

Lateral Force Summary (ASD)

NS Cumulative	*EW* Cumulative
6.43 kips	**6.43 kips**
13.64 kips	**15.22 kips**

Wind Force Calculations

INPUT DATA



Exposure category (B, C or D)		B	
Importance factor, pg 77, (0.87, 1.0 or 1.15)	I =	1.00	Category II
Basic wind speed (IBC Tab 1609.3.1V_{3S})	V =	85	mph 110 mph
Topographic factor (Sec.6.5.7.2, pg 26 & 45)	K_{zt} =	1.90	
Building height to eave	h_e =	28	ft
Building height to ridge	h_r =	28	ft
Building length	L =	62	ft
Building width	B =	55	ft
Effective area of components	A =	10	ft^2

DESIGN SUMMARY

Max horizontal force normal to building length, L, face	=	19.83 kips	27.76 kips at 110 mph
Max horizontal force normal to building length, B, face	=	17.77 kips	24.88 kips at 110 mph
Max total horizontal torsional load	=	163.62 ft-kips	
Max total upward force	=	39.57 kips	

ANALYSIS

Velocity pressure

$q_h = 0.00256\ K_h\ K_{zt}\ K_d\ V^2\ I$ = **15.19** psf

where: q_h = velocity pressure at mean roof height, h. (Eq. 6-15, page 27)

K_h = velocity pressure exposure coefficient evaluated at height, h, (Tab. 6-3, Case 1,pg 79) = **0.70**

K_d = wind directionality factor. (Tab. 6-4, for building, page 80) = **0.85**

h = mean roof height = **28.00** ft

< 60 ft, [Satisfactory]

Design pressures for MWFRS

$p = q_h\ [(G\ C_{pf}) - (G\ C_{pi})]$

where: p = pressure in appropriate zone. (Eq. 6-18, page 28).

$G\ C_{p\,f}$ = product of gust effect factor and external pressure coefficient, see table below. (Fig. 6-10, page 53 & 54)

$G\ C_{p\,i}$ = product of gust effect factor and internal pressure coefficient.(Fig. 6-5, Enclosed Building, page 47)

= **0.18** or **-0.18**

a = width of edge strips, Fig 6-10, note 9, page 54, MAX[MIN(0.1B, 0.4h), 0.04B,3] = **5.50** **ft**

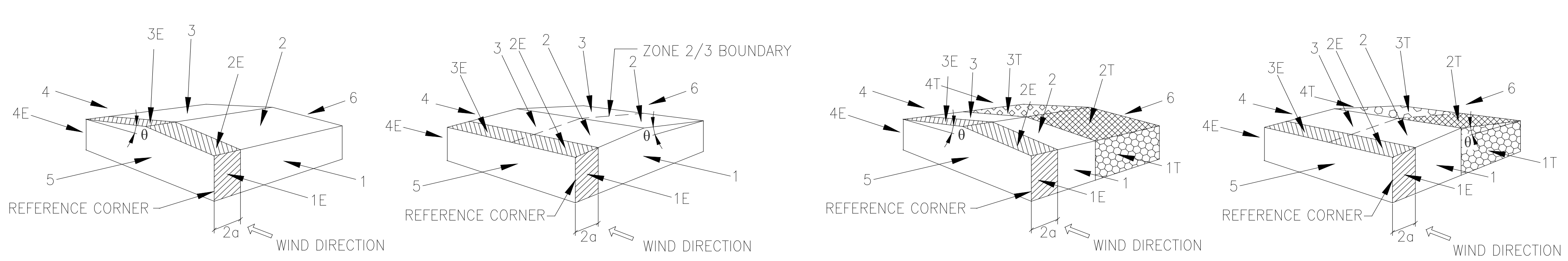
Net Pressures (psf), Basic Load Cases

Surface	Roof angle θ = 0.00			Roof angle θ = 0.00		
	$G\ C_{p\,f}$	Net Pressure with		$G\ C_{p\,f}$	Net Pressure with	
		$(+GC_{p\,i})$	$(-GC_{p\,i})$		$(+GC_{p\,i})$	$(-GC_{p\,i})$
1	0.40	3.34	8.81	0.40	3.34	8.81
2	-0.69	-13.21	-7.75	-0.69	-13.21	-7.75
3	-0.37	-8.35	-2.89	-0.37	-8.35	-2.89
4	-0.29	-7.14	-1.67	-0.29	-7.14	-1.67
1E	0.61	6.53	12.00	0.61	6.53	12.00
2E	-1.07	-18.98	-13.52	-1.07	-18.98	-13.52
3E	-0.53	-10.78	-5.32	-0.53	-10.78	-5.32
4E	-0.43	-9.26	-3.80	-0.43	-9.26	-3.80
5	-0.45	-9.57	-4.10	-0.45	-9.57	-4.10
6	-0.45	-9.57	-4.10	-0.45	-9.57	-4.10

Net Pressures (psf), Torsional Load Cases

Surface	Roof angle θ = 0.00		
	$G\ C_{p\,f}$	Net Pressure with	
		$(+GC_{p\,i})$	$(-GC_{p\,i})$
1T	0.40	0.84	2.20
2T	-0.69	-3.30	-1.94
3T	-0.37	-2.09	-0.72
4T	-0.29	-1.78	-0.42

Surface	Roof angle θ = 0.00		
	$G\ C_{p\,f}$	Net Pressure with	
		$(+GC_{p\,i})$	$(-GC_{p\,i})$
1T	0.40	0.84	2.20
2T	-0.69	-3.30	-1.94
3T	-0.37	-2.09	-0.72
4T	-0.29	-1.78	-0.42



Transverse Direction Longitudinal Direction Transverse Direction Longitudinal Direction

Basic Load Cases Torsional Load Cases

Basic Load Cases in Transverse Direction

Surface	Area (ft^2)	Pressure (k) with ($+GC_{pi}$)	Pressure (k) with ($-GC_{pi}$)
1	1428	4.77	12.58
2	1403	-18.53	-10.86
3	1403	-11.71	-4.05
4	1428	-10.19	-2.39
1E	308	2.01	3.70
2E	303	-5.74	-4.09
3E	303	-3.26	-1.61
4E	308	-2.85	-1.17
Σ	Horiz.	19.83	19.83
	Vert.	-39.25	-20.61
10 psf min.	Horiz.	17.36	17.36
Sec. 6.1.4.1	Vert.	-34.10	-34.10

Basic Load Cases in Longitudinal Direction

Surface	Area (ft^2)	Pressure (k) with ($+GC_{pi}$)	Pressure (k) with ($-GC_{pi}$)
1	1232	4.12	10.85
2	1364	-18.02	-10.56
3	1364	-11.39	-3.94
4	1232	-8.79	-2.06
1E	308	2.01	3.70
2E	341	-6.47	-4.61
3E	341	-3.68	-1.81
4E	308	-2.85	-1.17
Σ	Horiz.	17.77	17.77
	Vert.	-39.57	-20.92
10 psf min.	Horiz.	15.40	15.40
Sec. 6.1.4.1	Vert.	-34.10	-34.10

Torsional Load Cases in Transverse Direction

Surface	Area (ft^2)	Pressure (k) with ($+GC_{pi}$)	Pressure (k) with ($-GC_{pi}$)	Torsion (ft-k) ($+GC_{pi}$)	Torsion (ft-k) ($-GC_{pi}$)
1	560	1.87	4.93	24	63
2	550	-7.27	-4.26	0	0
3	550	-4.59	-1.59	0	0
4	560	-4.00	-0.94	51	12
1E	308	2.01	3.70	51	94
2E	303	-5.74	-4.09	0	0
3E	303	-3.26	-1.61	0	0
4E	308	-2.85	-1.17	73	30
1T	868	0.73	1.91	-11	-30
2T	853	-2.82	-1.65	0	0
3T	853	-1.78	-0.61	0	0
4T	868	-1.55	-0.36	-24	-6
Total Horiz. Torsional Load, M_T				164	164

Torsional Load Cases in Longitudinal Direction

Surface	Area (ft^2)	Pressure (k) with ($+GC_{pi}$)	Pressure (k) with ($-GC_{pi}$)	Torsion (ft-k) ($+GC_{pi}$)	Torsion (ft-k) ($-GC_{pi}$)
1	462	1.54	4.07	13	34
2	1023	-13.52	-7.92	0	0
3	1023	-8.55	-2.95	0	0
4	462	-3.30	-0.77	27	6
1E	308	2.01	3.70	44	81
2E	341	-6.47	-4.61	0	0
3E	341	-3.68	-1.81	0	0
4E	308	-2.85	-1.17	63	26
1T	770	0.64	1.70	-9	-23
2T	1364	-4.51	-2.64	0	0
3T	1364	-2.85	-0.98	0	0
4T	770	-1.37	-0.32	-19	-4
Total Horiz. Torsional Load, M_T				119.2	119.2

Design pressures for components and cladding

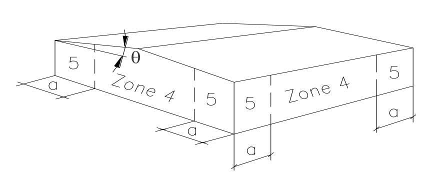
$p = q_h[(G C_p) - (G C_{pi})]$

where: p = pressure on component. (Eq. 6-22, pg 28)

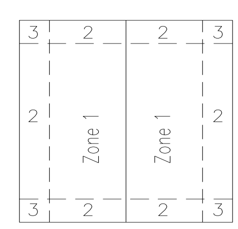
p_{min} = 10 psf (Sec. 6.1.4.2, pg 21)

$G C_p$ = external pressure coefficient.

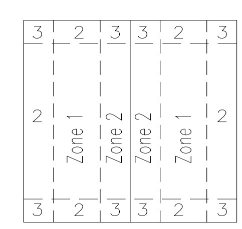
see table below. (Fig. 6-11, page 55~58)



Walls



Roof θ≤7°



Roof θ>7°

	Effective Area (ft^2)	Zone 1 GC_P	Zone 1 $-GC_P$	Zone 2 GC_P	Zone 2 $-GC_P$	Zone 3 GC_P	Zone 3 $-GC_P$	Zone 4 GC_P	Zone 4 $-GC_P$	Zone 5 GC_P	Zone 5 $-GC_P$
Comp.	10	0.30	-1.00	0.30	-1.80	0.30	-2.80	0.90	-0.99	0.90	-1.26

(Walls reduced 10 %, Fig. 6-11A note 5.)

Comp. & Cladding Pressure (psf)	Zone 1 Positive	Zone 1 Negative	Zone 2 Positive	Zone 2 Negative	Zone 3 Positive	Zone 3 Negative	Zone 4 Positive	Zone 4 Negative	Zone 5 Positive	Zone 5 Negative
	10.00	-17.92	10.00	-30.07	10.00	-45.26	16.40	-17.77	16.40	-21.87

Seismic Force Calculations

INPUT DATA

Typical floor height	h =	**9.0**	ft
Typical floor weight	w_x =	**45**	k
Number of floors	n =	**2**	
Importance factor (ASCE 11.5.1)	I =	**1.00**	(IBC Tab.1604.5)
Building location	Zip Code	**98040**	
Site class (A, B, C, D, E, F)		**D**	(If no soil report, use D)
The coefficient (ASCE Tab 12.8-2)	C_t =	**0.02**	
The coefficient(ASCE Tab. 12.2.1)	R =	**6.50**	

DESIGN SUMMARY

Total base shear

V = 0.15 W, (SD) = **15** k, (SD)

= 0.11 W, (ASD) = **11** k, (ASD)

Seismic design category = **D**

Latitude: 47.562605

Longitude: -122.2254

S_S = 147.595 %g , S_{ms} = 1.476 g , F_a = 1.000

S_1 = 50.091 %g , S_{m1} = 0.751 g , F_v = 1.500

S_{DS} = **0.984** g , S_{D1} = **0.501** g

h_n = 23.0 ft k = 1.00 , (ASCE 12.8.3, pg 130) x = 0.75 , (ASCE Tab 12.8-2)

W = 100 k $\Sigma w_x h^k$ = 1,750 $T_a = C_t (h_n)^x$ = 0.21 Sec, (ASCE 12.8.2.1)

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Level Name	Floor to floor Height (ft)	Height h_x (ft)	Weight w_x (k)	$w_x h_x^k$	C_{vx}	Lateral force @ each level: F_x (k)	V_x (k)	O. M. (k-ft)	Diaphragm force: ΣF_i (k)	ΣW_i (k)	F_{px} (k)
2	Roof		23.0	45	1,035	0.591	9.0			9.0	45	9
		10.00						9.0				
1	2nd		13.0	55	715	0.409	6.2		90	15.1	100	11
		13.00						15.1				
	Ground		0.0						286			

6.43/52.75=122plf

CS16 STRAPS AT T & BOP WINDOW OPENING TYP

→ 1.16^k

BY INSP [1] NO HD

→ 1.27^k

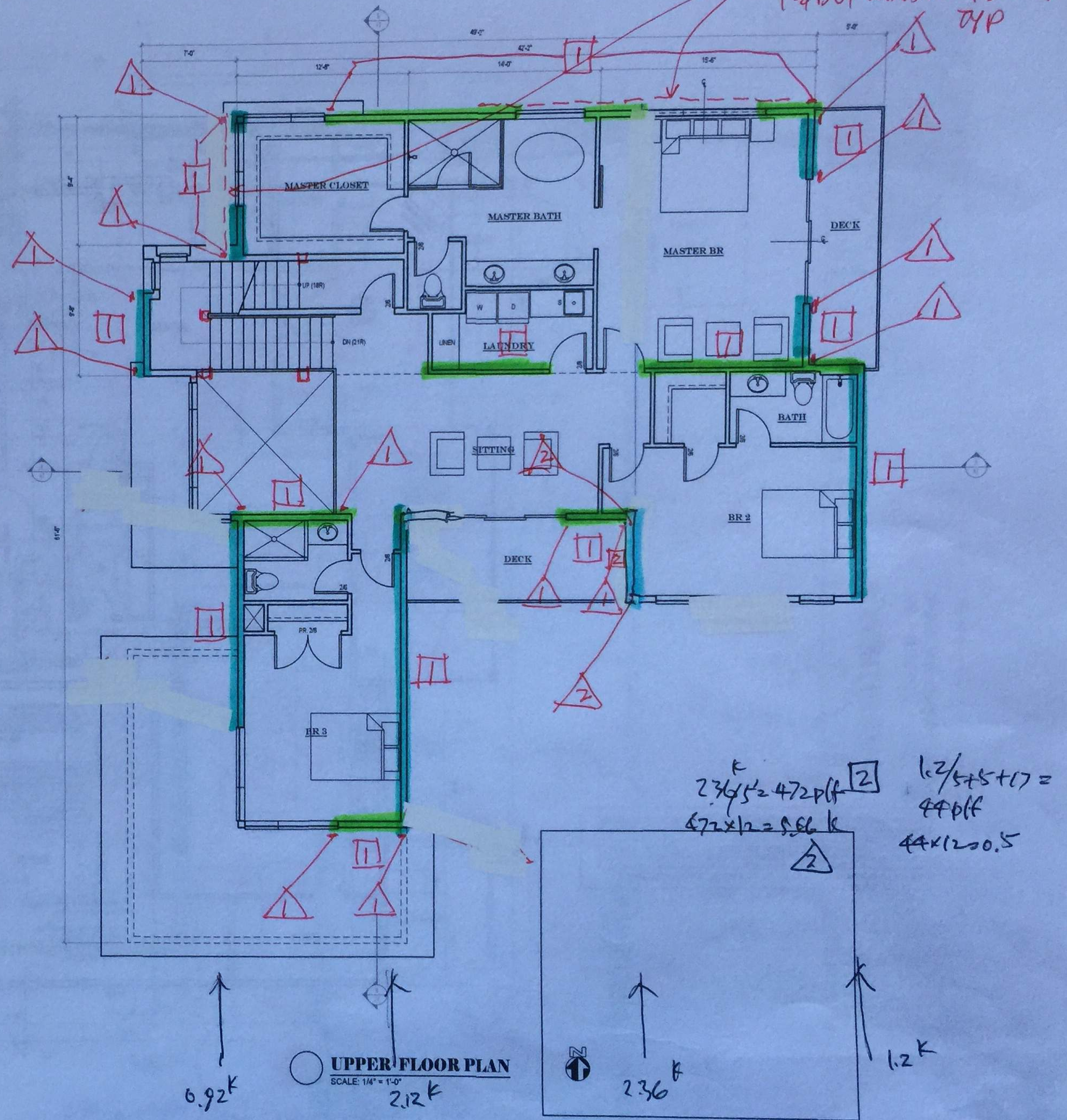
1.77/9+16 = 71 plf
71x12=0.85
NO HD

→ 2.04

2.04/9+3+5=120plf
120x12=1.44
1.44x 17/14 = 1.75^k

→ 1.40^k

1.40/5 = 280plf
280x12=3.36 k
wall 10x5x12=0.6
roof 15x8x5=0.6
1.2
3.36-1.2/2=2.76k [1]



2.36^k/5 = 472plf [2]
472x12=5.66 k

1.2/5+5+17 =
44plf
44x12=0.5

0.92^k

2.12^k

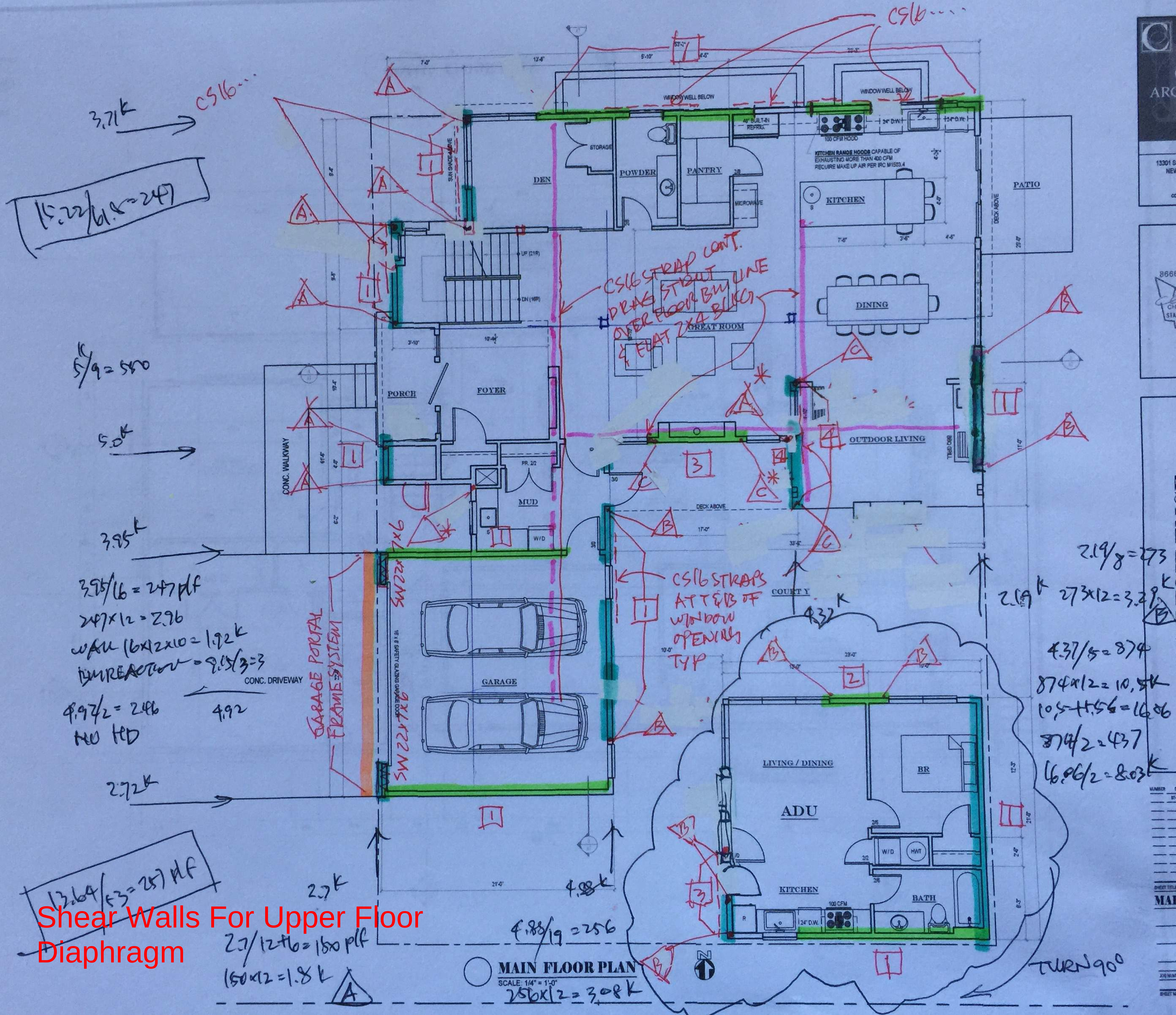
2.36^k

1.2^k

6.43/45.5=141plf

Shear Walls For Roof Diaphragm

2.12/23 =92plf
92x12=1.1k
NO HD



Shear Walls For Upper Floor Diaphragm

SHEARWALL SCHEDULE

LABEL	APA RATED SHEATHING (STRUC I)	NAILING AT PANEL EDGES	RIM JOIST OR BLOCK CONNECTION TO TOP PLATE	BOTTOM PLATE ATTACHMENT TO WOOD BELOW	SILL PLATE ATTACHMENT		CAPACITY (plf)
					ANCHOR BOLT	SILL PLATE SIZE	
1	15/32" PLYWOOD SHEATHING, ONE SIDE	10d @ 6" OC	LTP4 @ 18" OC	16d @ 6" OC	5/8" @ 48" OC (7" MIN EMBED)	2X	340
2	15/32" PLYWOOD SHEATHING, ONE SIDE	10d @ 4" OC	LTP4 @ 12" OC	⅜"ø LAG OR SIMPSON SDS @ 8" OC	5/8" @ 32" OC (7" MIN EMBED)	3X	510
3	15/32" PLYWOOD SHEATHING, ONE SIDE	10d @ 3" OC	LTP4 @ 9" OC	⅜"ø LAG OR SIMPSON SDS @ 6" OC	5/8" @ 24" OC (7" MIN EMBED)	3X	665
4	15/32" PLYWOOD SHEATHING, ONE SIDE	10d @ 2" OC	LTP4 AND A35 @ 8" OC	⅜"ø LAG OR SIMPSON SDS @ 4" OC	5/8" @ 16" OC (7" MIN EMBED)	3X	870
22	15/32" PLYWOOD SHEATHING, BOTH SIDES	10d @ 4" OC	LTP4 AND A35 @ 8" OC	⅜"ø LAG OR SIMPSON SDS @ 4" OC	5/8" @ 16" OC (7" MIN EMBED)	3X	1020
23	15/32" PLYWOOD SHEATHING, BOTH SIDES	10d @ 3" OC	LTP4 AND A35 @ 8" OC	(2) ⅜"ø LAG OR SIMPSON SDS @ 6" OC STAGGERED USE 4x RIM JOIST; TYP	5/8" @ 12" OC (7" MIN EMBED)	3X	1330
24	15/32" PLYWOOD SHEATHING, BOTH SIDES	10d @ 2" OC	LTP4 AND A35 @ 6" OC	(2) ⅜"ø LAG OR SIMPSON SDS @ 4" OC STAGGERED USE 4x RIM JOIST; TYP	5/8" @ 10" OC (7" MIN EMBED)	3X	1740

NOTES:

A. PANEL EDGES BACKED WITH 2" NOMINAL OR WIDER FRAMING. INSTALL PANELS EITHER HORIZONTALLY OR VERTICALLY. SPACE FASTENERS MAXIMUM 12" OC ON INTERMEDIATE SUPPORTS.

B. FRAMING AT ADJOINING PANEL EDGES SHALL BE 3" NOMINAL OR WIDER, AND NAILS SHALL BE STAGGERED WHERE NAILS ARE SPACED 2" OC.

C. FRAMING AT ADJOINING PANEL EDGES SHALL BE 3" NOMINAL OR WIDER, AND NAILS SHALL BE STAGGERED WHERE BOTH OF THE FOLLOWING CONDITIONS ARE MET: (1) 10d (3"x0.148") NAILS HAVING PENETRATION INTO FRAMING OF MORE THAN 1½" AND (2) NAILS ARE SPACED 3" OC.

D. WHERE PANELS APPLIED ON BOTH FACES OF A WALL AND NAIL SPACING IS LESS THAN 6" OC ON EITHER SIDE, PANEL JOINTS SHALL BE OFFSET TO FALL ON DIFFERENT FRAMING MEMBERS, OR FRAMING SHALL BE 3" NOMINAL OR THICKER AT ADJOINING PANEL EDGES AND NAILS ON EACH SIDE SHALL BE STAGGERED.

E. WHERE SHEAR DESIGN VALUES EXCEED 350 POUNDS PER LINEAR FOOT, ALL FRAMING MEMBERS RECEIVING EDGE NAILING FROM ABUTTING PANELS SHALL NOT BE LESS THAN A SINGLE 3" NOMINAL MEMBER, OR TWO 2" NOMINAL MEMBERS FASTENED TOGETHER IN ACCORDANCE WITH SECTION 2306.1 TO TRANSFER THE DESIGN SHEAR VALUE BETWEEN FRAMING MEMBERS. WOOD STRUCTURAL PANEL JOINT AND SILL PLATE NAILING SHALL BE STAGGERED IN ALL CASES. ANCHOR BOLTS FOR SHEAR WALLS SHALL INCLUDE STEEL PLATE AND NUT. THE HOLE IN THE PLATE WASHER IS PERMITTED TO BE DIAGONALLY SLOTTED WITH A WIDTH OF UP TO 3/16" LARGER THAN THE BOLT DIAMETER AND A SLOT LENGTH NOT TO EXCEED 1¾", PROVIDE A STANDARD CUT WASHER IS PLACED BETWEEN THE PLATE WASHER AND THE NUT. SILL PLATES RESISTING A DESIGN LOAD GREATER THAN 350 PLF USING ALLOWABLE STRESS DESIGN SHALL NOT BE LESS THAN A 3" NOMINAL SILL PLATE. (2) 20d BOX END NAILS SHALL BE SUITABLE FOR (2) 16D COMMON END NAILS FOUND IN LINE 8 OF TABLE 2304.9.1 (FASTENING SCHEDULE).

F. GALVANIZED NAILS SHALL BE HOT DIPPED OR TUMBLED.

1 SHEARWALL SCHEDULE

NTS

HOLDOWN SCHEDULE:

MARK	HOLDOWN STUDS	HOLDOWN STUD NAILING		SIMPSON HOLDOWN ANCHOR	REMARK
		NAIL SIZE	NAILING FROM WALL SHEATHING	ANCHOR TYPE & FASTENERS IN CONC.	
A	(2) 2x	10d	SHEARWALL EDGE NAILING ON BOTH 2x	HDU2-SDS2.5 w/ Ø5/8" ANCHOR ROD w/ 14" EMBED	
B	(2) 2x	10d	SHEARWALL EDGE NAILING ON BOTH 2x	HDU5-SDS2.5 w/ Ø5/8" ANCHOR ROD w/ 18" EMBED	

C 6X6 HDU8-SDS2.5 W/7/8" ANCHOR ROD W/18" EMBED

PLAN NOTES:

1) ALL POSTS ARE 6x6 U.N.O.

2) ALL POST-BEAM CONNECTION: SIMPSON PC/EPC OR AC/ACE POST CAPS OR EQUAL.

3) ALL POST-FOOTING CONNECTION: SIMPSON PB/PBS POST BASE.

4) ALL BEAM-BEAM CONNECTIONS: SIMPSON FACE MOUNTED HANGERS.

5) TYP. STRIP FTG. 24"W x 12"DP w/ (2) #5 CONT. BOT. U.N.O.

6) TYP. CONC. PAD 12"DP w/ #5 @ 12" BOT. E.W.

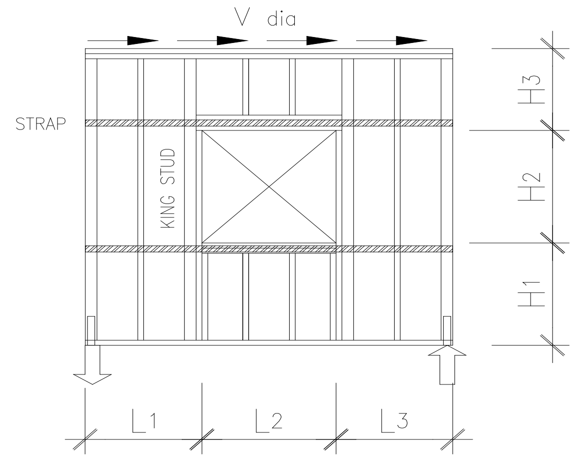
HOLDOWN SCHEDULE:

MARK	HOLDOWN STUDS	HOLDOWN STUD NAILING		SIMPSON HOLDOWN ANCHOR		REMARK
		NAIL SIZE	NAILING FROM WALL SHEATHING	ANCHOR TYPE	FASTENERS IN CONC.	
1	(2) 2x	10d	SHEARWALL EDGE NAILING ON BOTH 2x	MSTI 48	N/A	CENTER TO FLOOR JOIST
2	(2) 2x	10d	SHEARWALL EDGE NAILING ON BOTH 2x	MSTI- 60	N/A	CENTER TO FLOOR JOIST

HOLDOWN NOTES:

WHERE HOLDOWN STRAP IS REQUIRED TO TIE TO THE FLOOR BEAM BELOW, THE HOLDOWN STRAP SHOULD BE CENTERED TO THE FLOOR SHEATHING AND WRAP AROUND BEAM AS REQUIRED.

Shear Wall with Window Opening Reinforcing



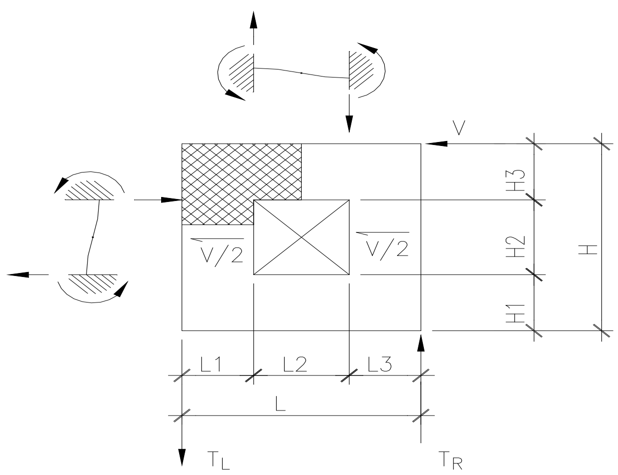
INPUT DATA

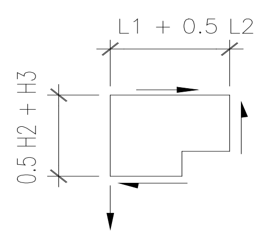
LATERAL FORCE ON DIAPHRAGM:			$v_{dia,\ WIND}$ =	170	plf,for wind			
(SERVICE LOADS)			$v_{dia,\ SEISMIC}$ =	170	plf,for seismic			
DIMENSIONS:	L_1 =	3	ft , L_2 =	6	ft , L_3 =	3	ft	
	H_1 =	2	ft , H_2 =	4	ft , H_3 =	2.5	ft	
KING STUD SECTION		1	pcs, b =	2	in , h =	6	in	
EDGE STUD SECTION		2	pcs, b =	2	in , h =	6	in	
PANEL GRADE (0 or 1)	=	1	<= Sheathing and Single-Floor					
MINIMUM NOMINAL PANEL THICKNESS			=	15/32	in			
COMMON NAIL SIZE (0=6d, 1=8d, 2=10d)				2	10d			
SPECIFIC GRAVITY OF FRAMING MEMBERS				0.5				
STORY OPTION (1=ground level, 2=upper level)				1	ground level shear wall			

DESIGN SUMMARY

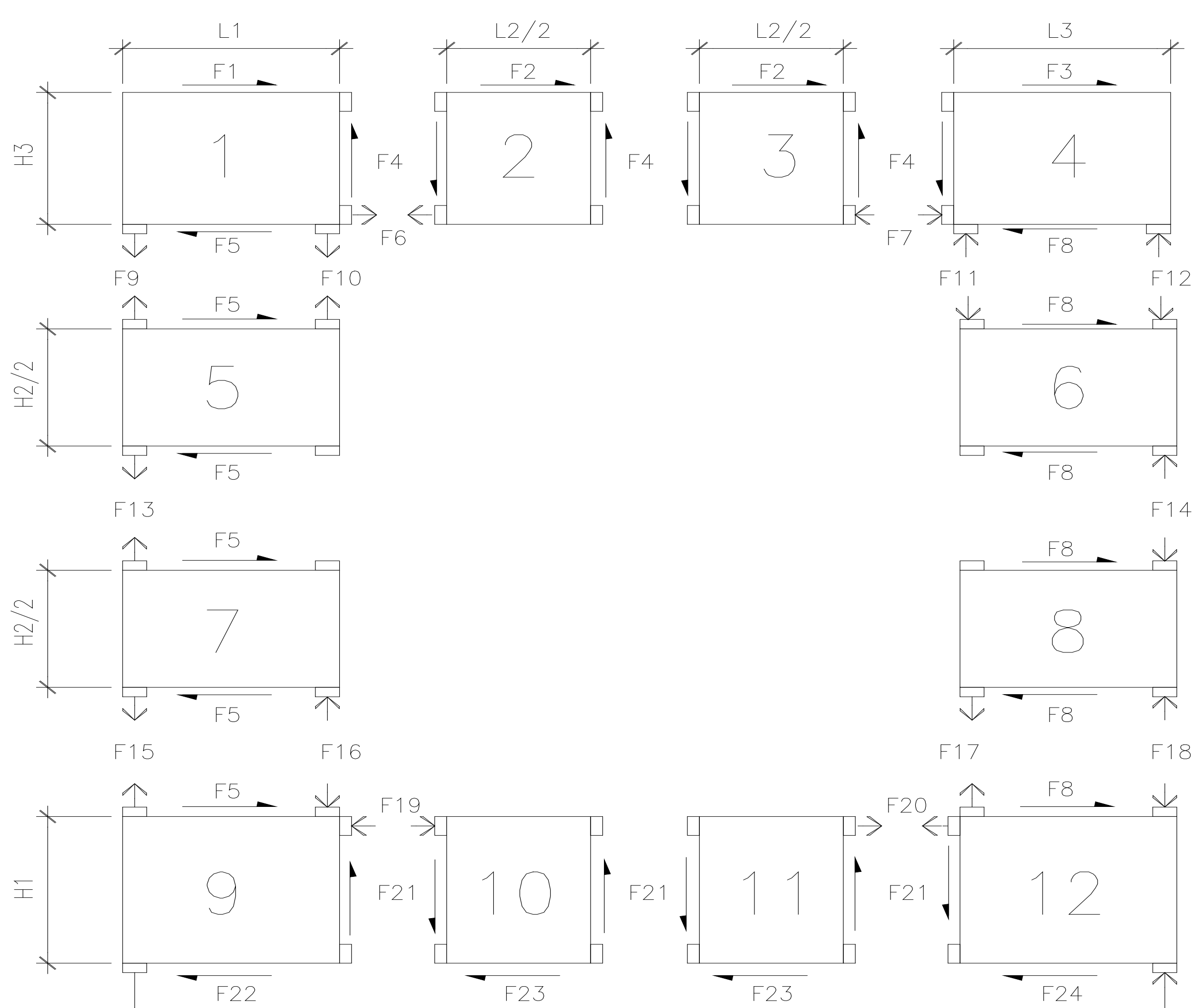
BLOCKED 15/32 SHEATHING WITH 10d COMMON NAILS
@ 4 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD,
5/8 in DIA. x 10 in LONG ANCHOR BOLTS @ 48 in O.C.

HOLD-DOWN FORCES: T_L = 1.50 k , T_R = 1.50 k (USE PHD2-SDS3 SIMPSON HOLD-DOWN)
MAX STRAP FORCE: F = 0.93 k (USE SIMPSON CS20 OVER WALL SHEATHING WITH FLAT BLOCKING)
KING STUD: 1 - 2" x 6" DOUGLAS FIR-LARCH No. 1, CONTINUOUS FULL HEIGHT.
EDGE STUD: 2 - 2" x 6" DOUGLAS FIR-LARCH No. 1, CONTINUOUS FULL HEIGHT.
SHEAR WALL DEFLECTION: Δ = 0.58 in





ASSUME INFLECTION POINT AT MIDDLE OF WINDOW



FREE-BODY INDIVIDUAL PANELS OF WALL

cont'd

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO h / w = 1.3 < 2 **[Satisfactory]**

DETERMINE FORCES & SHEAR STRESS OF FREE-BODY INDIVIDUAL PANELS OF WALL

INDIVIDUAL PANEL	W (ft)	H (ft)	MAX SHEAR STRESS (plf)	NO.	FORCE (lbf)	NO.	FORCE (lbf)
1	3.00	2.50	34	F1	102	F13	765
2	3.00	2.50	306	F2	918	F14	765
3	3.00	2.50	306	F3	102	F15	1445
4	3.00	2.50	34	F4	765	F16	680
5	3.00	2.00	340	F5	1020	F17	680
6	3.00	2.00	340	F6	918	F18	1445
7	3.00	2.00	340	F7	918	F19	933
8	3.00	2.00	340	F8	1020	F20	933
9	3.00	2.00	29	F9	85	F21	738
10	3.00	2.00	369	F10	680	F22	87
11	3.00	2.00	369	F11	680	F23	933
12	3.00	2.00	29	F12	85	F24	87

DETERMINE REQUIRED CAPACITY v_b = 369 plf, (1 Side Panel Required, the Max. Nail Spacing = 4 in)

THE SHEAR CAPACITIES PER IBC Table 2306.4.1 / UBCTable 23-II-I-1 :

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges			
				6	4	3	2
Sheathing and Single-Floor	10d	1 5/8	15/32	310	460	600	770

Note: The indicated shear numbers have reduced by specific gravity factor per IBC note a / UBC note1 of the table.

DETERMINE MAX SPACING OF 5/8" DIA ANCHOR BOLT (NDS 2005, Tab.11E)

5/8 in DIA. x 10 in LONG ANCHOR BOLTS @ 48 in O.C.

THE HOLD-DOWN FORCES:

	v_{dia} (plf)	Wall Seismic at mid-story (lbs)	Overturning Moments (ft-lbs)		Resisting Moments (ft-lbs)	Safety Factors	Net Uplift (lbs)		Holddown SIMPSON
SEISMIC	170	163	18034	Left	0	0.9	T_L =	1503	PHD2-SDS3
				Right	0	0.9	T_R =	1503	
WIND	170		17340	Left	0	2/3	T_L =	1445	
				Right	0	2/3	T_R =	1445	

(T_L & T_R values should include upper level UPLIFT forces if applicable)

DETERMINE MAXIMUM SHEAR WALL DEFLECTION: (IBC Section 2305.3.2)

$$\Delta = \Delta_{Bending} + \Delta_{Shear} + \Delta_{Nail\ slip} + \Delta_{Chord\ splice\ slip} = \frac{8 v_b h^3}{E A L_w} + \frac{v_b h}{G t} + 0.75 h e_n + \frac{h d_a}{L_w} = 0.576 \text{ in}$$

Where: v_b = 369 plf, L_w = 6 ft, E = 1.7E+06 psi

A = 16.50 in^2, h = 9 ft, G = 9.0E+04 psi

t = 0.298 in, e_n = 0.037 in, d_a = 0.15 in

CHECK KING STUD CAPACITY

P_{max} = 0.68 kips

F_c = 1500 psi C_D = 1.60 C_P = 0.48 A = 8.25 in^2

E = 1700 ksi C_F = 1.10 F_c' = 1255 psi **>** f_c = 82 psi

[Satisfactory]

CHECK EDGE STUD CAPACITY

P_{max} = 1.50 kips, (this value should include upper level DOWNWARD loads if applicable)

F_c = 1500 psi C_D = 1.60 C_P = 0.48 A = 16.50 in^2

E = 1700 ksi C_F = 1.10 F_c' = 1255 psi **>** f_c = 91 psi

[Satisfactory]

Technical References:

1. "National Design Specification, NDS", 2005 Edition, AF & PA, AWC, 2005.

Face-Mount Hangers HU/HUC/HUCQ/HGUS

Glulam Beam and Double-Shear Joist Hangers

See Hanger Options on pp. 121–123 for hanger modifications, which may result in reduced loads.

HU/HUC — Most models have triangle and round holes. To achieve maximum loads, fill both round and triangle holes with common nails.

HGUS — Face-mount hanger used for high load applications. All hangers in this series have double-shear nailing. This innovation distributes the load through two points on each joist nail for greater strength. It also allows the use of fewer nails, faster installation, and the use of common nails for all connections.

HUCQ — Heavy duty joist hangers that incorporate Simpson Strong-Tie® Strong Drive® SDS Heavy-Duty Connector screws (included).

Material: See tables

Finish: Galvanized. Some products available in ZMAX® or HDG coating; see Corrosion Information, pp. 15–18.

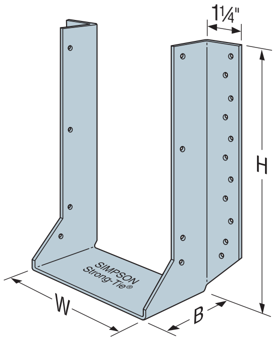
Installation:

- Use all specified fasteners; see General Notes.
- HU/HUC — can be installed filling round holes only, or filling round and triangle holes for maximum values.
- HGUS — Nails must be driven at an angle through the joist or truss into the header to achieve the table loads.
- HUCQ — Install ¼" x 2½" Strong-Drive SDS Heavy-Duty Connector screws (provided) in all round holes. Lag screws will not achieve the same load.
- With 3x carrying members, use 16d x 2½" (0.162" dia. x 2½" long) nails into the header and 16d commons into the joist with no load reduction. With 2x carrying members, use 10d x 1½" (0.148" dia. x 1½" long) nails into the header and 10d commons into the joist, and reduce the load to 0.64 of the table value.
- For installations to masonry or concrete, see pp. 279–281.

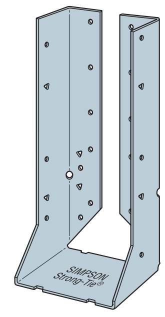
Options:

- HU hangers available with the header flanges turned in for 2⁵⁄₁₆" and larger widths, with no load reduction — order HUC hanger.
- See Hanger Options on pp. 121–123, for sloped and/or skewed HU models, and HUC (concealed flange) models.
- Concealed flanges are not available for HGUS.
- HGUS may be skewed only up to a maximum of 45°. See Hanger Options pp. 121–123 or load reductions.
- Other sizes available; contact Simpson Strong-Tie.
- See also HUS series.
- HUCQ hangers cannot be modified.

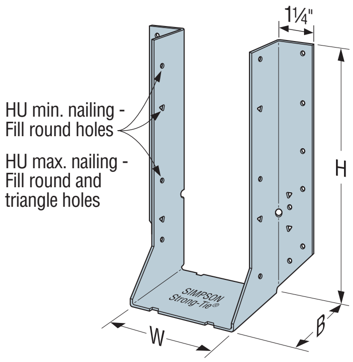
Codes: See p. 14 for Code Reference Key Chart



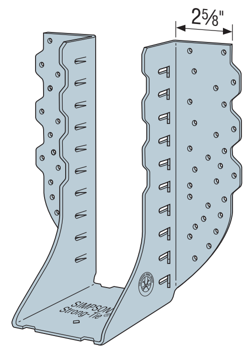
HU5.125/12



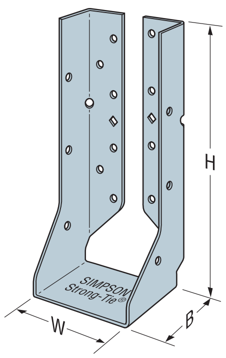
HUC210-2
Concealed Flanges
(HUCQ similar)



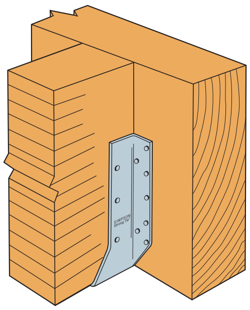
HU210-2



HGUS3.25/12



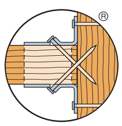
HUCQ410



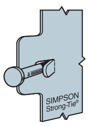
Typical HU Installation

Projection seat on most models for maximum bearing and section economy.

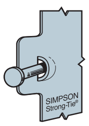
Model configurations may differ from those shown. Some HU models do not have triangle holes. Contact Simpson Strong-Tie for details.



Double-Shear Nailing Top View



Double-Shear Nailing Side View Do not bend tab



Dome Double-Shear Nailing Side View *(Available on some models)*
U.S. Patent 5,603,580

Face-Mount Hangers HU/HUC/HUCQ/HGUS

SIMPSON Strong-Tie®

Glulam Beam and Double-Shear Joist Hangers (cont.)

These products are available with additional corrosion protection. For more information, see p. 18.

	Carried Member Width (in.)	Model No.	Ga.	Dimensions (in.) W	H	B	Min./Max.	Fasteners Face	Joist	Allowable Loads DF/SP Species Header Uplift (160)	Floor (100)	Snow (115)	Roof (125)	SPF/HF Species Header Floor (100)	Snow (115)	Roof (125)	Code Ref.
	3⅛ Glulam	HU210-2 / HUC210-2	14	3⅛	8 13/16	2½	Min.	(14) 16d	(6) 10d	1,135	2,085	2,350	2,530	1,795	2,025	2,180	I9, FL, L17, L12
				3⅛	8 13/16	2½	Max.	(18) 16d	(10) 10d	1,895	2,680	3,020	3,250	2,305	2,605	2,800	
		HU212-2 / HUC212-2		3⅛	10 9/16	2½	Min.	(16) 16d	(6) 10d	1,135	2,380	2,685	2,890	2,050	2,315	2,490	
				3⅛	10 9/16	2½	Max.	(22) 16d	(10) 10d	1,895	3,275	3,695	3,970	2,820	3,180	3,425	
		HU3.25/10.5 / HUC3.25/10.5		3¼	10¼	2½	—	(22) 16d	(10) 10d	1,895	3,275	3,695	3,970	2,820	3,180	3,425	I9, L12, FL
		HU3.25/12 / HUC3.25/12		3¼	11¾	2½	—	(24) 16d	(12) 10d	2,015	3,570	4,030	4,335	3,075	3,470	3,735	
		HU216-2 / HUC216-2		3⅛	13⅞	2½	Min.	(20) 16d	(8) 10d	1,515	2,975	3,360	3,610	2,565	2,895	3,110	I9, FL, L17, L12
				3⅛	13⅞	2½	Max.	(26) 16d	(12) 10d	2,015	3,870	4,365	4,695	3,330	3,760	4,045	
		HU3.25/16 / HUC3.25/16		3¼	13 13/16	2½	Min.	(20) 16d	(8) 10d	1,515	2,975	3,360	3,610	2,560	2,890	3,105	I9, L12
				3¼	13 13/16	2½	Max.	(26) 16d	(12) 10d	2,015	3,870	4,365	4,695	3,330	3,755	4,040	
SS		HUCQ210-2-SDS		3¼	9	3	—	(12) ¼" x 2½" SDS	(6) ¼" x 2½" SDS	2,510	4,680	4,955	4,955	3,370	3,570	3,570	I9, FL, L12
		HGUS3.25/10	12	3¼	8⅝	4	—	(46) 16d	(16) 16d	4,095	9,100	9,100	9,100	7,825	7,825	7,825	
		HGUS3.25/12		3¼	10⅝	4	—	(56) 16d	(20) 16d	5,045	9,600	9,600	9,600	8,255	8,255	8,255	
	3½ Glulam	HU410	14	3 9/16	8⅝	2½	Max.	(18) 16d	(10) 10d	1,895	2,680	3,020	3,250	2,305	2,605	2,800	I7, I9, FL, L12
		HU414		3 9/16	12⅝	2½	Max.	(24) 16d	(12) 10d	2,015	3,570	4,030	4,335	3,075	3,470	3,735	
SS		HUCQ410-SDS		3 9/16	9	3	—	(12) ¼" x 2½" SDS	(6) ¼" x 2½" SDS	2,510	4,680	4,955	4,955	3,370	3,570	3,570	I9, FL, L12
SS		HUCQ412-SDS		3 9/16	11	3	—	(14) ¼" x 2½" SDS	(6) ¼" x 2½" SDS	2,510	5,460	5,560	5,560	3,930	4,005	4,005	
		HHUS410	12	3⅝	9	3	—	(30) 16d	(10) 16d	3,735	5,635	6,380	6,880	4,835	5,480	5,910	I7, I9, FL, L12
		HGUS410		3⅝	9 1/16	4	—	(46) 16d	(16) 16d	4,095	9,100	9,100	9,100	7,825	7,825	7,825	
		HGUS414		3⅝	12 7/16	4	—	(66) 16d	(22) 16d	5,515	10,100	10,100	10,100	8,685	8,685	8,685	
	5⅛ Glulam	HU310-2 / HUC310-2	14	5⅛	8⅞	2½	—	(14) 16d	(6) 10d	1,135	2,085	2,350	2,530	1,795	2,025	2,180	I9, FL, L17, L12
		HU5.125/12 / HUC5.125/12		5¼	10¼	2½	—	(22) 16d	(8) 16d	1,795	3,275	3,695	3,970	2,820	3,180	3,425	I9, FL, L12
		HU5.125/13.5 / HUC5.125/13.5		5¼	13¼	2½	—	(26) 16d	(12) 16d	2,695	3,870	4,365	4,695	3,330	3,760	4,045	
		HU5.125/16 / HUC5.125/16		5¼	13⅞	2½	—	(26) 16d	(12) 16d	2,695	3,870	4,365	4,695	3,330	3,760	4,045	
		HUCQ5.25/9-SDS		5¼	9	3	—	(12) ¼" x 2½" SDS	(6) ¼" x 2½" SDS	2,510	4,680	4,955	4,955	3,370	3,570	3,570	
		HUCQ5.25/11-SDS		5¼	11	3	—	(14) ¼" x 2½" SDS	(6) ¼" x 2½" SDS	2,510	5,460	5,560	5,560	3,930	4,000	4,000	
		HGUS5.25/10	12	5¼	9 1/16	4	—	(46) 16d	(16) 16d	4,095	9,100	9,100	9,100	7,825	7,825	7,825	
		HGUS5.25/12		5¼	10 9/16	4	—	(56) 16d	(20) 16d	5,045	9,600	9,600	9,600	8,255	8,255	8,255	
	5½ Glulam	HUCQ610-SDS	14	5½	9	3	—	(12) ¼" x 2½" SDS	(6) ¼" x 2½" SDS	2,520	4,680	5,380	5,715	3,370	3,875	4,115	
SS		HUCQ612-SDS		5½	11	3	—	(14) ¼" x 2½" SDS	(6) ¼" x 2½" SDS	2,520	5,315	5,315	5,315	3,825	3,825	3,825	
		HHUS5.50/10		5½	9	3	—	(30) 16d	(10) 16d	3,735	5,635	6,380	6,880	4,835	5,480	5,910	
		HGUS5.50/10	12	5½	8 15/16	4	—	(46) 16d	(16) 16d	4,095	9,100	9,100	9,100	7,825	7,825	7,825	
		HGUS5.50/14		5½	12½	4	—	(66) 16d	(22) 16d	5,515	10,100	10,100	10,100	8,685	8,685	8,685	
	6¾ Glulam	HGUS6.88/10	12	6⅞	8 13/16	4	—	(46) 16d	(16) 16d	4,095	9,100	9,100	9,100	7,825	7,825	7,825	
		HGUS6.88/12		6⅞	10 13/16	4	—	(54) 16d	(20) 16d	5,045	9,600	9,600	9,600	8,255	8,255	8,255	
		HGUS6.88/14		6⅞	12 13/16	4	—	(66) 16d	(22) 16d	5,515	10,100	10,100	10,100	8,685	8,685	8,685	
	7 Glulam	See HHUS and HGUS in 7" Structural Composite Lumber section, p. 186 or HGU / HHGU series on pp. 194–195.															
	8¾ Glulam	See HGU and HHGU on p. 173.															

1. Uplift loads based on Douglas Fir and have been increased 60% for wind or earthquake loading with no further increase allowed. Reduce where other loads govern.
2. Min. nailing quantity and load values — fill all round holes; Max. nailing quantity and load values — fill all round and triangle holes.
3. For SPF/HF uplift, use 0.86 x DF/SP uplift load for products requiring nails and 0.72 for products requiring screws.
4. **Nails:** 16d = 0.162" dia. x 3½" long, 10d = 0.148" dia. x 3" long. See pp. 26–27 for other nail sizes and information.

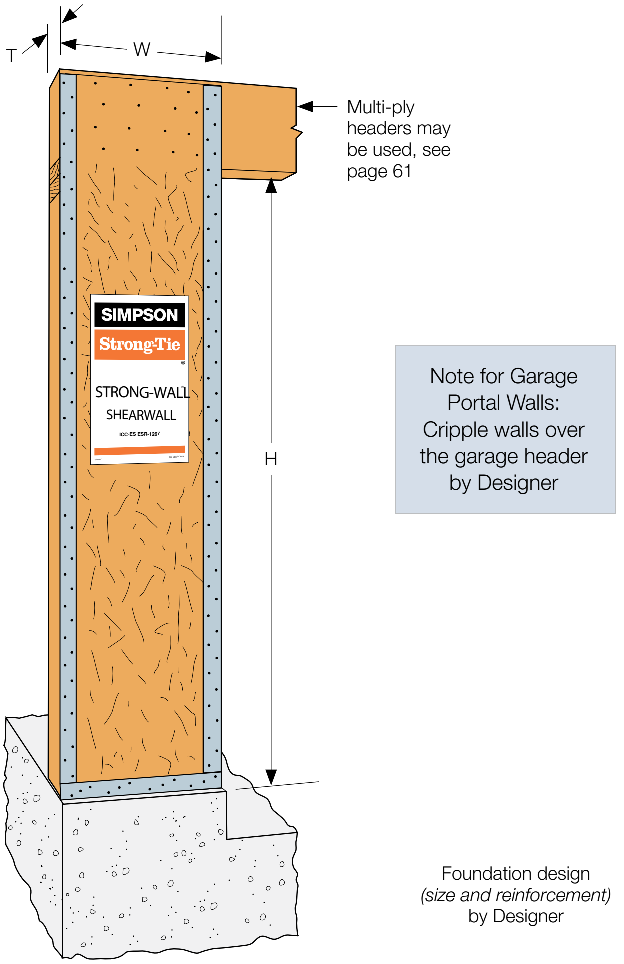
Glulam Beam Connectors

SIMPSON Strong-Tie

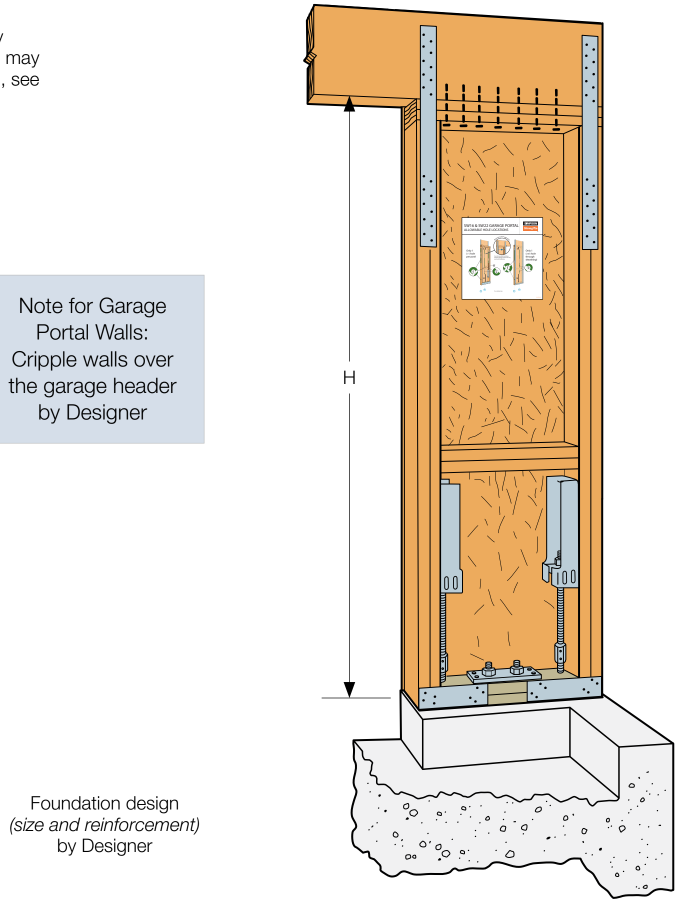
Garage Portal Systems on Concrete Foundations

Garage Portal systems provide increased lateral resistance over site-built shearwalls in locations where space is at a premium. Portal walls shall be installed with a minimum 12" nominal deep header for adequate shear nailing. Because the portal walls and header are tested as a system, the resulting portal frame offers superior engineered performance over site-built walls.

Codes: ICC-ES ESR-1267; City of L.A. RR 25427



U.S. Patents 5,706,626; 6,006,487; 6,109,850; 6,327,831; and 6,643,986



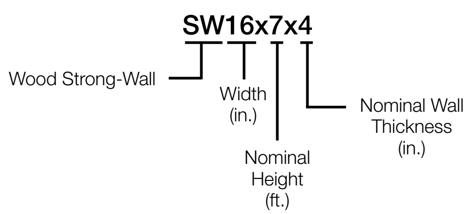
U.S. Patents 5,706,626; 6,006,487; 6,109,850; 6,327,831; and 6,643,986

Garage Portal Wall Product Data

Model No.	W (in.)	H (in.)	T (in.)	Number of Fasteners in Top of Wall	Mudsill Anchors[1]		Holdown Anchor Bolts	
					Qty.	Dia. (in.)	Qty.	Model[2,3]
SW16x7x4	16	78	4	8-SDS ¼"x6"	2	⅝	2	PAB7
SW16x7x6	16	78	5¾	8-SDS ¼"x6"	2	⅝	2	PAB7
SW16x8x4	16	90	4	8-SDS ¼"x6"	2	⅝	2	PAB7
SW16x8x6	16	90	5¾	8-SDS ¼"x6"	2	⅝	2	PAB7
SW22x7x4	22	78	4	10-SDS ¼"x6"	2	⅝	2	PAB7
SW22x7x6	22	78	5¾	10-SDS ¼"x6"	2	⅝	2	PAB7
SW22x8x4	22	90	4	10-SDS ¼"x6"	2	⅝	2	PAB7
SW22x8x6	22	90	5¾	10-SDS ¼"x6"	2	⅝	2	PAB7

1. Recommended minimum ⅝"x12" mudsill anchor.
2. PAB7 available in multiple lengths. See pages 66-68 for anchor bolt information and anchorage solutions.
3. SSTB28 anchor bolts may be specified when the anchor tension force does not exceed allowable anchor tension. See page 67 for SSTB anchor bolt information and anchorage solutions.

Naming Legend

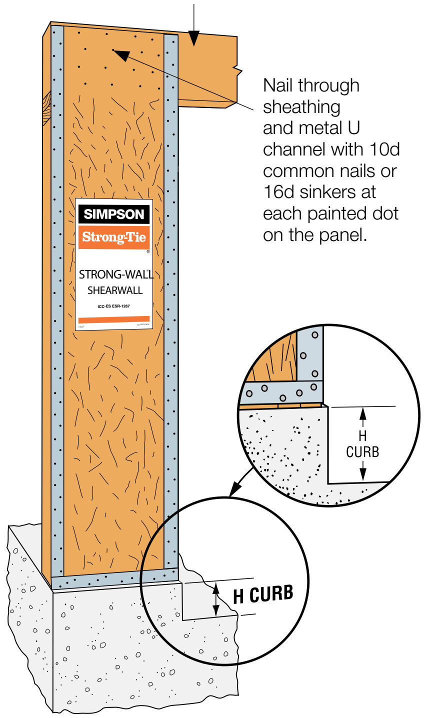


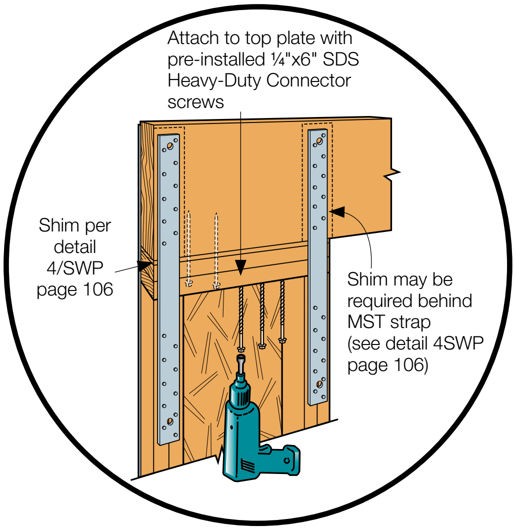
Garage Portal Systems on Concrete Foundations

Installation:

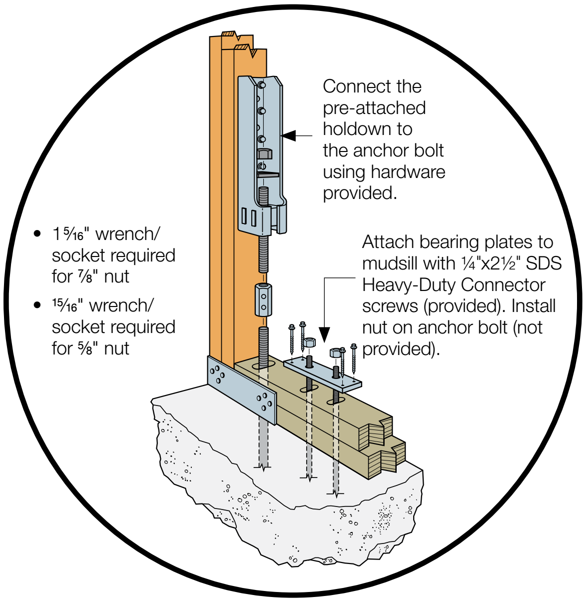
- Typical shim thickness between the Strong-Wall® and top plates or header is ⅞" or less using Simpson Strong-Tie® Strong-Drive® ¼"x6" SDS Heavy-Duty Connector screws. For additional shim thickness, contact Simpson Strong-Tie.
- For holdowns, per ASTM standards, anchor bolt nuts should be finger-tight plus ⅓ to ½ turn with a hand wrench, with consideration given to possible future wood shrinkage. Care should be taken to avoid over-torquing the nut, an impact wrench should not be used.

CAUTION:
Drilling or cutting holes in Wood Strong-Wall® is not allowed except as shown on page 107 and the Allowable Hole Chart attached to each wall.





Top-of-Wall Connection

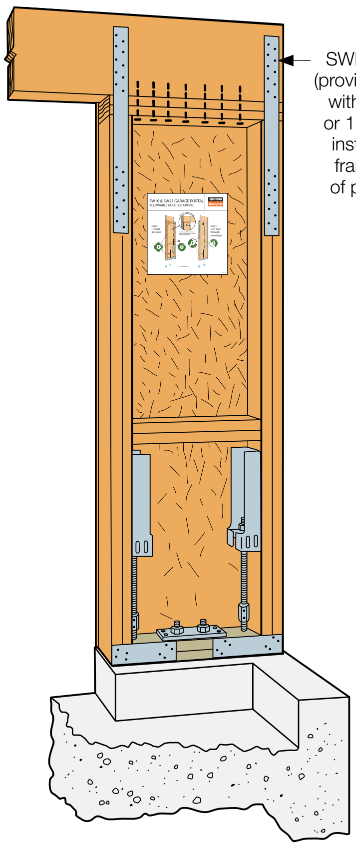


Bottom-of-Wall Installation

Garage Header Rough Opening Height

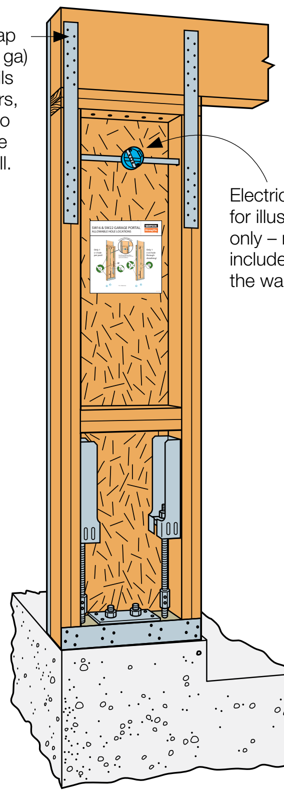
Model No.	H Curb	Without ½" Shim	With ½" Shim
SW16x7x4 SW22x7x4	6"	7'	7'-½"
SW16x7x6 SW22x7x6	7"	7'-1"	7'-1½"
SW16x8x4 SW22x8x4	6"	8'	8'-½"
SW16x8x6 SW22x8x6	7"	8'-1"	8'-1½"

1. The height of the garage curb above the garage slab is critical for rough header opening on portal walls.
2. One ½" shim is provided with Garage Portal walls.



Standard Portal Wall Installation

U.S. Patents 5,706,626; 6,006,487; 6,109,850; 6,327,831 and 6,643,986



Alternate Exterior Installation

For easy electrical installation position wall with sheathing inside garage.

U.S. Patents 5,706,626; 6,006,487; 6,109,850; 6,327,831 and 6,643,986

Garage Portal Systems on Concrete Foundations

Model No.	Double-Wall Garage Portal[1]				Single-Wall Garage Portal[2]			
	Seismic		Wind		Seismic		Wind	
	Allowable ASD Shear Load V (lbs.)	Drift at Allowable Shear (in.)	Allowable ASD Shear Load V (lbs.)	Drift at Allowable Shear (in.)	Allowable ASD Shear Load V (lbs.)	Drift at Allowable Shear (in.)	Allowable ASD Shear Load V (lbs.)	Drift at Allowable Shear (in.)
SW16x7x4	2670	0.36	3500	0.53	1335	0.36	1750	0.53
SW16x7x6	2670	0.36	3500	0.53	1335	0.36	1750	0.53
SW16x8x4	2350	0.40	3105	0.60	1175	0.40	1555	0.60
SW16x8x6	2350	0.40	3105	0.60	1175	0.40	1555	0.60
SW22x7x4	4160	0.37	5420	0.53	2080	0.37	2710	0.53
SW22x7x6	4160	0.37	5420	0.53	2080	0.37	2710	0.53
SW22x8x4	3730	0.42	4880	0.60	1865	0.42	2440	0.60
SW22x8x6	3730	0.42	4880	0.60	1865	0.42	2440	0.60

1. A double-wall garage portal system consists of 2 walls with a header spanning over the top and connected as shown.
2. A single-wall garage portal system consists of 1 wall with a header spanning over the top and connected as shown.
3. Recommended header moisture content is 19% or less at time of installation.
4. The minimum header sizes listed are the minimum required for lateral rigidity of the portal system. Larger headers may be required due to vertical loading.
5. Portal walls may be installed with sheathing facing inside or outside.
6. Typical shim thickness between the Strong-Wall® shearwall and header is ⅞" or less using Simpson Strong-Tie® Strong-Drive® ¼"x6" SDS Heavy-Duty Connector screws.
7. See allowable vertical load table on page 65 for Wood Strong-Wall shearwall maximum compression and tension capacities.
8. Allowable shear capacities must be reduced as limited by anchor bolt capacities for installations on CMU.
9. Anchor tension forces may be calculated using the following formula:

$$\text{Anchor Tension} = \frac{\text{Shear per Panel x Height}}{\text{Width - 5.25"}} \qquad K = 0.80 \text{ (SW16 Portal); } 0.67 \text{ (SW22 Portal)}$$

See page 66 for PAB anchorage solutions.

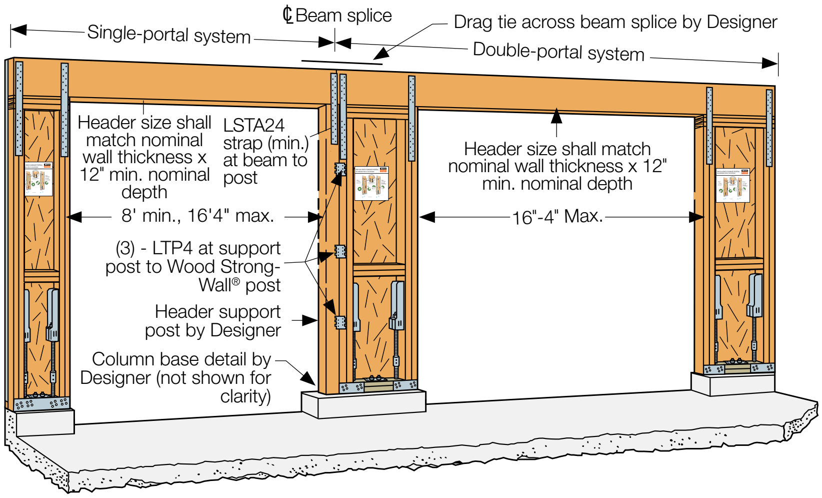
Multi-Ply Headers for Garage Portal Walls

Model No.	Allowable Load per Portal Wall (lbs.)			
	Seismic		Wind	
	Double 2x Header	Double LVL Header	Double 2x Header	Double LVL Header
SW16x7x4	1145	1200	1500	1575
SW16x8x4	940	1100	1245	1455
SW22x7x4	1500	1660	1950	2165
SW22x8x4	1400	1585	1830	2075

1. Garage portal walls listed above may be used with double 2x12 minimum or double 1¾"x11⅞" minimum LVL headers.
2. Headers shall be face nailed to each other with minimum 16d nails at 32" on center staggered along the top and bottom.
3. Double 2x12 require ½" ply or OSB shim to make the header assembly flush with Wood Strong-Wall®. The shim shall match the header depth and Wood Strong-Wall width minimum. It may be placed on either face of header or between plies directly over the Wood Strong-Wall.

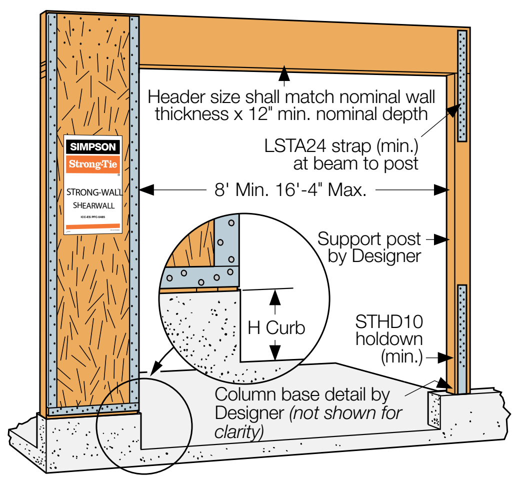
Detail 1 – Single- and Double-Wall Garage Portal

1. Beam to support post and support post to foundation uplift connectors may be reduced where justified by calculations.
2. This detail reflects lateral load requirements of a Single- and Double-Wall Portal system. It is the Designer's responsibility to provide a complete load path for all loads in accordance with the governing codes.
3. System rating equals the sum of the Single- and Double-Wall Portal values.
4. Alternate Installation: A single-piece header *(no camber)* may be substituted for the two headers shown. The design rating for this condition may then be evaluated as the sum of the individual single-wall ratings.
5. Longer header spans can be accommodated if larger headers are used such that equivalent stiffness is equal to or greater than that provided by the minimum header and maximum length indicated.
6. Simpson Strong-Tie® LTP4 and LSTA24 *(by Designer)* are minimum requirements to achieve the allowable loads.



Detail 2 – Single-Wall Garage Portal

1. Beam to support post and support post to foundation uplift connectors may be reduced where justified by calculations.
2. This detail reflects lateral load requirements of a Single-Wall Portal system. It is the Designer's responsibility to provide a complete load path for all loads in accordance with the governing codes.
3. Longer header spans can be accommodated if larger headers are used such that equivalent stiffness is equal to or greater than that provided by the minimum header and maximum length indicated.
4. Simpson Strong-Tie® STHD10 and LSTA24 *(by Designer)* are minimum requirements to achieve the allowable loads.



Wood Strong-Wall®

HRS/ST/PS/HST/HTP/LSTA/LSTI/MST/MSTA/MSTC/MSTI *Strap Ties*

Straps are designed to transfer tension loads in a wide variety of applications.

HRS—A 12 gauge strap with a nailing pattern designed for installation on the edge of 2x members. The HRS416Z installs with Simpson Strong-Tie® Strong-Drive® SDS Heavy-Duty Connector screws.

LSTA and MSTA—Designed for use on the edge of 2x members, with a nailing pattern that reduces the potential for splitting.

LSTI—Light straps that are suitable where pneumatic-nailing is necessary through diaphragm decking and wood chord open web trusses.

MST—Splitting may be a problem with installations on lumber smaller than 3½"; either fill every nail hole with 10dx1½" nails or fill every-other hole with 16d common nails. Reduce the allowable load based upon the size and quantity of fasteners used.

MSTC—High Capacity strap which utilizes a staggered nail pattern to help minimize wood splitting. Nail slots have been countersunk to provide a lower nail head profile.

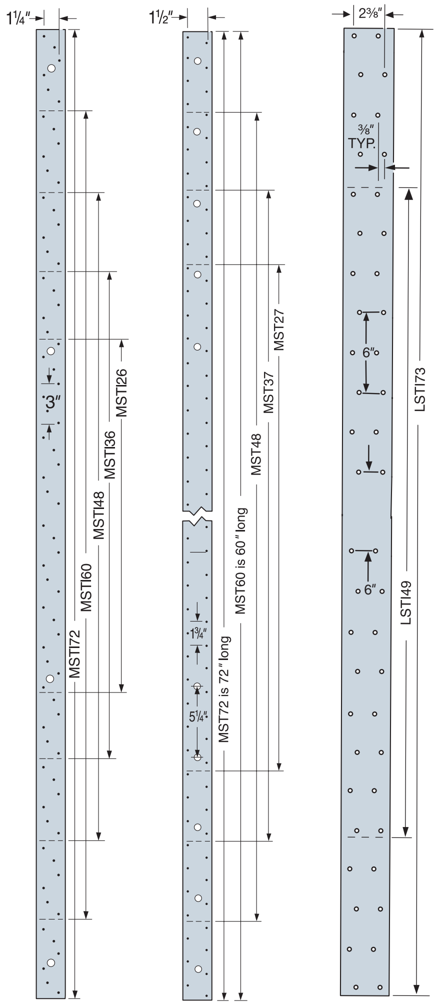
FINISH: PS–HDG; HST3 and HST6–Simpson Strong-Tie® gray paint; all others–galvanized. Some products are available in stainless steel or ZMAX® coating; see Corrosion Information, pages 13-15.

INSTALLATION: Use all specified fasteners. See General Notes.

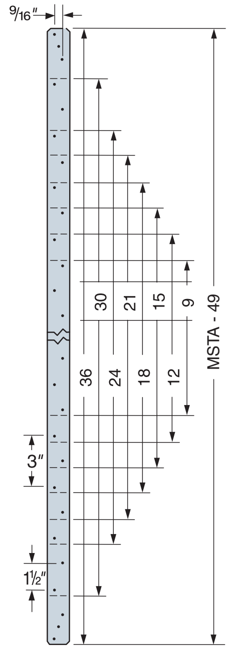
OPTIONS: Special sizes can be made to order. Contact Simpson Strong-Tie.

CODES: See page 12 for Code Reference Key Chart.

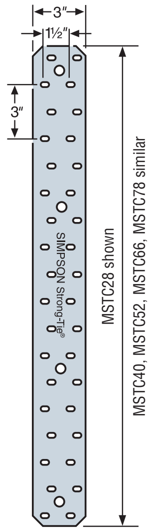
MSTC and RPS meet code requirements for reinforcing cut members (16 gauge) at top plate and RPS at sill plate. International Residential Code®– 2000/2006 R602.6.1 International Building Code®– 2000/2006 2308.9.8 *(For RPS, refer to page 223.)*



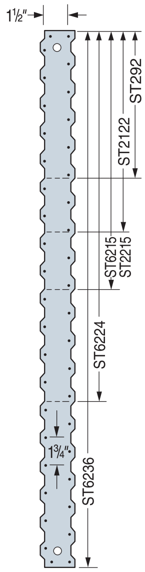
MSTI **MST** **LSTI**



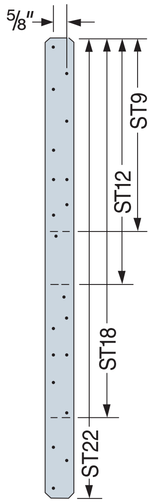
LSTA and MSTA
(Pilot holes not shown)



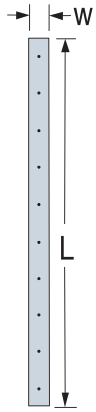
MSTC



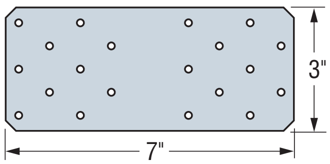
ST



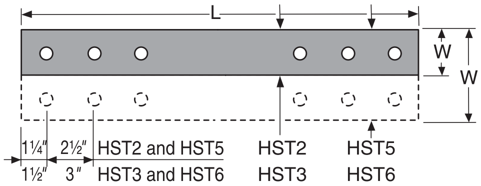
ST9, ST12, ST18, ST22



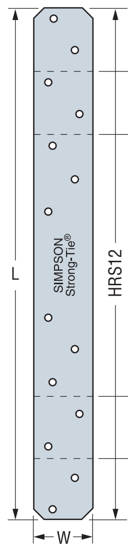
ST2115



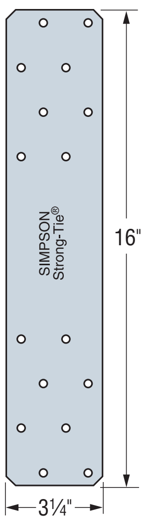
HTP37Z



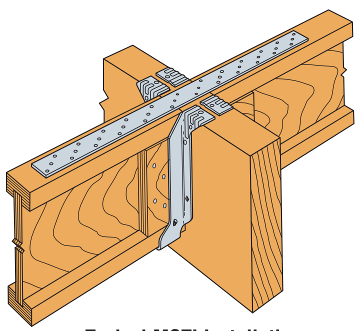
HST



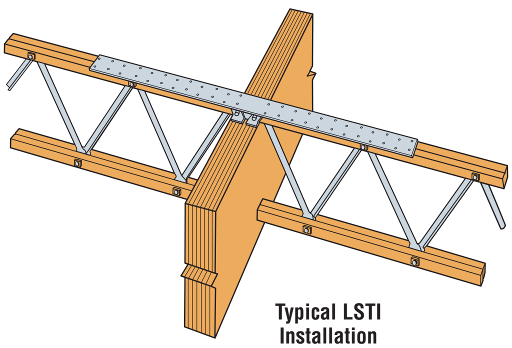
HRS



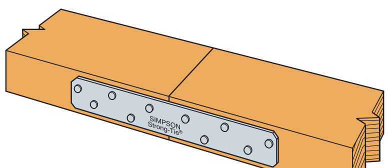
HRS416Z



Typical MSTI Installation
(MIT hanger shown)
LSTI similar



Typical LSTI Installation



Typical HRS Installation

Straps & Ties

HRS/ST/PS/HST/HTP/LSTA/LSTI/MST/MSTA/MSTC/MSTI *Strap Ties*

SIMPSON Strong-Tie®

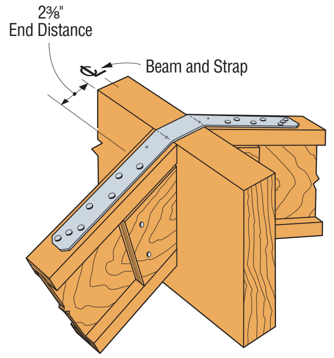
CODES: See page 12 for Code Reference Key Chart.

These products are available with additional corrosion protection. Additional products on this page may also be available with this option, check with Simpson Strong-Tie for details.

These products are approved for installation with the Strong-Drive® SD Connector screw. See page 27 for more information.

Model No.	Ga	Dimensions W	Dimensions L	Fasteners (Total)	Allowable Tension Loads (DF/SP) (160)	Allowable Tension Loads (SPF/HF) (160)	Code Ref.
LSTA9	20	1¼	9	8-10d	740	635	I4, L3, L5, F2
LSTA12	20	1¼	12	10-10d	925	795	I4, L3, L5, F2
LSTA15	20	1¼	15	12-10d	1110	950	I4, L3, L5, F2
LSTA18	20	1¼	18	14-10d	1235	1110	I4, L3, L5, F2
LSTA21	20	1¼	21	16-10d	1235	1235	I4, L3, L5, F2
LSTA24	20	1¼	24	18-10d	1235	1235	I4, L3, L5, F2
ST292	20	2¹⁄₁₆	9⁵⁄₁₆	12-16d	1265	1120	I4, L3, L5, F2
ST2122	20	2¹⁄₁₆	12¹³⁄₁₆	16-16d	1530	1505	I4, L3, L5, F2
ST2115	20	¾	16⁵⁄₁₆	10-16d	660	660	I4, L3, L5, F2
ST2215	20	2¹⁄₁₆	16⁵⁄₁₆	20-16d	1875	1875	I4, L3, L5, F2
LSTA30	18	1¼	30	22-10d	1640	1640	I4, L3, L5, F2
LSTA36	18	1¼	36	24-10d	1640	1640	I4, L3, L5, F2
LSTI49	18	3¾	49	32-10dx1½	2975	2555	I4, L3, L5, F2
LSTI73	18	3¾	73	48-10dx1½	4205	3830	I4, L3, L5, F2
MSTA9	18	1¼	9	8-10d	750	645	I4, L3, L5, F2
MSTA12	18	1¼	12	10-10d	940	810	I4, L3, L5, F2
MSTA15	18	1¼	15	12-10d	1130	970	I4, L3, L5, F2
MSTA18	18	1¼	18	14-10d	1315	1130	I4, L3, L5, F2
MSTA21	18	1¼	21	16-10d	1505	1290	I4, L3, L5, F2
MSTA24	18	1¼	24	18-10d	1640	1455	I4, L3, L5, F2
MSTA30	16	1¼	30	22-10d	2050	1820	I4, L3, L5, F2
MSTA36	16	1¼	36	26-10d	2050	2050	I4, L3, L5, F2
MSTA49	16	1¼	49	26-10d	2020	2020	F2, L3, L5
ST6215	16	2¹⁄₁₆	16⁵⁄₁₆	20-16d	2095	1900	I4, IL14, L3, L5, F2
ST6224	16	2¹⁄₁₆	23⁵⁄₁₆	28-16d	2540	2540	I4, L3, L5, F2
ST9	16	1¼	9	8-16d	885	760	I4, L3, L5, F2
ST12	16	1¼	11⅝	10-16d	1105	950	I4, L3, L5, F2
ST18	16	1¼	17¾	14-16d	1420	1330	I4, L3, L5, F2
ST22	16	1¼	21⅝	18-16d	1420	1420	I4, L3, L5, F2
MSTC28	16	3	28¼	36-16d sinkers	3455	2980	I4, L3, L5, F2
MSTC40	16	3	40¼	52-16d sinkers	4745	4305	I4, L3, L5, F2
MSTC52	16	3	52¼	62-16d sinkers	4745	4745	I4, L3, L5, F2
HTP37Z	16	3	7	20-10dx1½	1850	1600	L5
MSTC66	14	3	65¾	76-16d sinkers	5860	5860	I4, L3, L5, F2
MSTC78	14	3	77¾	76-16d sinkers	5860	5860	I4, L3, L5, F2
ST6236	14	2¹⁄₁₆	33¹³⁄₁₆	40-16d	3845	3845	I4, L3, L5, F2
HRS6	12	1⅜	6	6-10d	605	525	L5, F26
HRS8	12	1⅜	8	10-10d	1010	880	L5, F26
HRS12	12	1⅜	12	14-10d	1415	1230	L5, F26
MSTI26	12	2¹⁄₁₆	26	26-10dx1½	2745	2325	I4, L3, L5, F2
MSTI36	12	2¹⁄₁₆	36	36-10dx1½	3800	3220	I4, L3, L5, F2
MSTI48	12	2¹⁄₁₆	48	48-10dx1½	5065	4290	I4, L3, L5, F2
MSTI60	12	2¹⁄₁₆	60	60-10dx1½	5080	5080	I4, L3, L5, F2
MSTI72	12	2¹⁄₁₆	72	72-10dx1½	5080	5080	I4, L3, L5, F2
HRS416Z	12	3¼	16	16-SDS ¼"x1½"	2835	2305	170

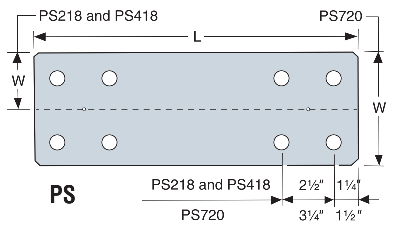
1. Loads include a 60% load duration increase on the fasteners for wind or earthquake loading.
2. 10dx1½" nails may be substituted where 16d sinkers or 10d are specified at 100% of the table loads except where straps are installed over sheathing.
3. 10d commons may be substituted where 16d sinkers are specified at 100% of table loads.
4. 16d sinkers (0.148" dia. x 3¼" long) or 10d commons may be substituted where 16d commons are specified at 0.84 of the table loads.
5. Use half of the nails in each member being connected to achieve the listed loads.
6. Tension loads apply for uplift when installed vertically.
7. **NAILS:** 16d = 0.162" dia. x 3½" long, 16d Sinker = 0.148" dia. x 3¼" long, 10d = 0.148" dia. x 3" long. 10dx1½ = 0.148" dia. x 1½" long. See pages 22-23 for other nail sizes and information.



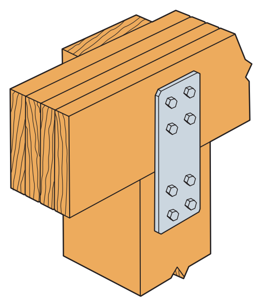
Typical LSTA Installation
(Hanger not shown)
Bend strap one time only, max 12/12 joist pitch.

Model No.	Material Thickness Gauge	Dim. W	Dim. L	Bolts Qty	Bolts Dia	Code Ref.
PS218	7 ga	2	18	4	¾	180
PS418	7 ga	4	18	4	¾	180
PS720	7 ga	6¾	20	8	½	180

1. PS strap design loads must be determined by the Designer for each installation. Bolts are installed both perpendicular and parallel-to-grain. Hole diameter in the part may be oversized to accommodate the HDG. Designer must determine if the oversize creates an unacceptable installation.
2. For allowable tension loads, see page 230.



PS



Typical PS720 Installation

HST/MST/MSTC/MSTA *Strap Ties*

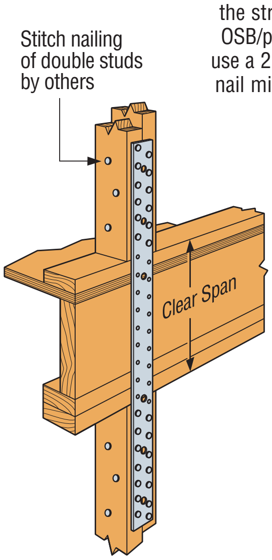
SIMPSON Strong-Tie®

CODES: See page 12 for Code Reference Key Chart.

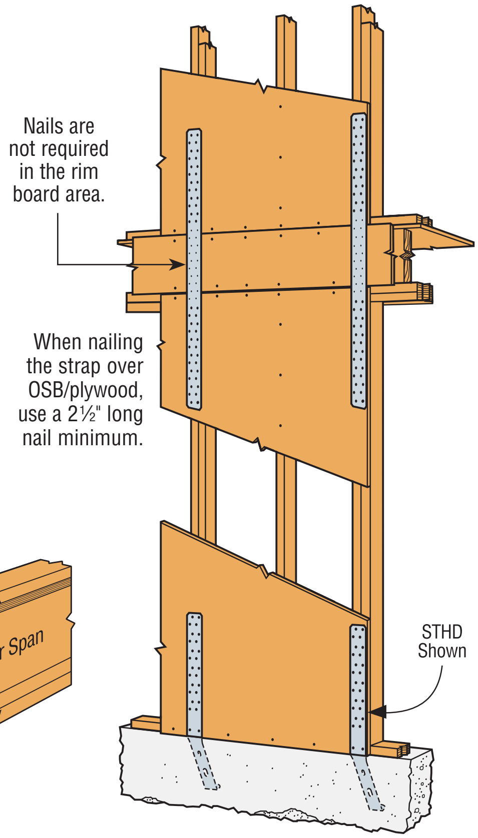
These products are approved for installation with the Strong-Drive® SD Connector screw. See page 27 for more information.

Floor-to-Floor Clear Span Table

Model No.	Clear Span	Fasteners (Total)	Allowable Tension Loads (DF/SP) (160)	Allowable Tension Loads (SPF/HF) (160)
MSTA49	18	26-10d	2020	2020
	16	26-10d	2020	2020
MSTC28	18	12-16d sinkers	1155	995
	16	16-16d sinkers	1540	1325
MSTC40	24	20-16d sinkers	2310	1985
	18	28-16d sinkers	2695	2320
	16	32-16d sinkers	3080	2650
MSTC52	24	36-16d sinkers	3465	2980
	18	44-16d sinkers	4235	3645
	16	48-16d sinkers	4620	3975
MSTC66	30	48-16d sinkers	4780	4120
	24	54-16d sinkers	5380	4640
	18	64-16d sinkers	5860	5495
	16	68-16d sinkers	5860	5840
MSTC78	30	64-16d sinkers	5860	5495
	24	72-16d sinkers	5860	5860
	18	76-16d sinkers	5860	5860
MST37	24	14-16d	1725	1495
	18	20-16d	2465	2135
	16	22-16d	2710	2345
MST48	24	26-16d	3215	2780
	18	32-16d	3960	3425
	16	34-16d	4205	3640
MST60	30	34-16d	4605	3995
	24	40-16d	5240	4700
	18	46-16d	6235	5405
MST72	30	48-16d	6505	5640
	24	54-16d	6730	6345
	18	62-16d	6730	6475



Floor-to-Floor Tie Installation showing a Clear Span



Typical Detail with Strap Installed Over Sheathing

These products are available with additional corrosion protection. Additional products on this page may also be available with this option, check with Simpson Strong-Tie for details.

Model No.	Ga	Dimensions W	Dimensions L	Fasteners (Total) Nails	Fasteners (Total) Bolts Qty	Fasteners (Total) Bolts Dia	Allowable Tension Loads (DF/SP) Nails (160)	Allowable Tension Loads (DF/SP) Bolts (160)	Allowable Tension Loads (SPF/HF) Nails (160)	Allowable Tension Loads (SPF/HF) Bolts (160)	Code Ref.
MST27	12	2¹⁄₁₆	27	30-16d	4	½	3700	2165	3200	2000	I4, L3, F2
MST37		2¹⁄₁₆	37½	42-16d	6	½	5080	3025	4480	2805	
MST48		2¹⁄₁₆	48	50-16d	8	½	5310	3675	5190	3410	
MST60	10	2¹⁄₁₆	60	68-16d	10	½	6730	4485	6475	4175	
MST72		2¹⁄₁₆	72	68-16d	10	½	6730	4485	6475	4175	
HST2	7	2½	21¼	—	6	⅝	—	5220	—	4835	
HST5		5	21¼	—	12	⅝	—	10650	—	9870	
HST3	3	3	25½	—	6	¾	—	7680	—	6660	
HST6		6	25½	—	12	¾	—	15470	—	13320	

1. Loads include a 60% load duration increase on the fasteners for wind or earthquake loading.
2. Install bolts or nails as specified by Designer. Bolt and nail values may not be combined.
3. Allowable bolt loads are based on parallel-to-grain loading and these minimum member thicknesses: MST–2½"; HST2 and HST5–4"; HST3 and HST6–4½".
4. Use half of the required nails in each member being connected to achieve the listed loads.
5. When installing strap over wood structural panel sheathing, use 2½" long nail minimum.
6. Tension loads apply for uplift as well when installed vertically.
7. **NAILS:** 16d = 0.162" dia. x 3½" long, 16d Sinker = 0.148" dia. x 3¼" long, 10dx1½ = 0.148" dia. x 1½" long. See pages 22-23 for other nail sizes and information.

SIMPSON Strong-Tie®

HDU/DTT *Holdowns*

VALUE ENGINEERED

This product is preferable to similar connectors because of a) easier installation, b) higher loads, c) lower installed cost, or a combination of these features.

HDU holdowns are pre-deflected during the manufacturing process, virtually eliminating deflection under load due to material stretch. They use Simpson Strong-Tie® Strong-Drive® SDS Heavy-Duty Connector screws which install easily, reduce fastener slip and provide a greater net section when compared to bolts.

The HDU series of holdowns are designed to replace previous versions of the product such as PHDs as well as bolted holdowns. The HDU2, 4 and 5 are direct replacements for the PHD2, 5 and 6, respectively.

The DTT tension ties are designed for lighter-duty holdown applications on single or 2x posts. The new DTT1Z is installed with nails or Simpson Strong-Tie Strong-Drive SD Connector screws and the DTT2Z installs easily with the Strong-Drive SDS Heavy-Duty Connector screws *(included)*. The DTT1Z holdowns have been tested for use in designed shearwalls and prescriptive braced wall panels as well as prescriptive wood-deck applications *(see page 209 for deck applications)*.

For more information on holdown options, contact Simpson Strong-Tie.

HDU SPECIAL FEATURES:

- Holdown designs virtually eliminate deflection due to material stretch.
- Uses Strong-Drive SDS Heavy-Duty Connector screws which install easily, reduce fastener slip, and provide a greater net section area of the post compared to bolts.
- Strong-Drive SDS Heavy-Duty Connector screws are supplied with the holdowns to ensure proper fasteners are used.
- No stud bolts to countersink at openings.

MATERIAL: See table

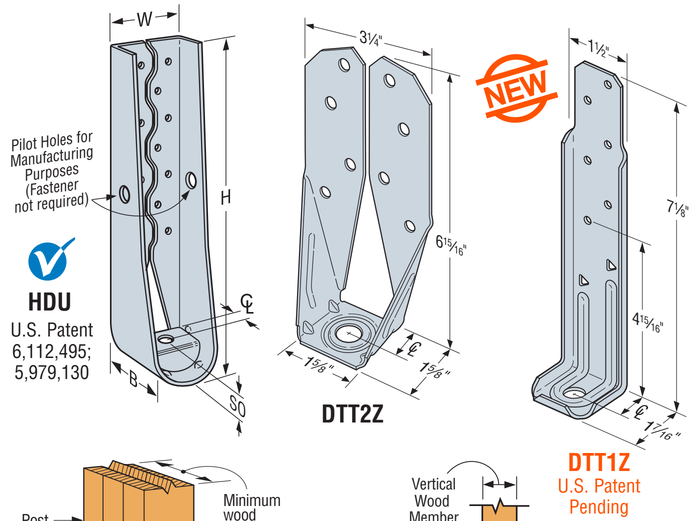
FINISH: HDU – Galvanized; DTT1Z and DTT2Z – ZMAX® coating; DTT2SS – stainless steel

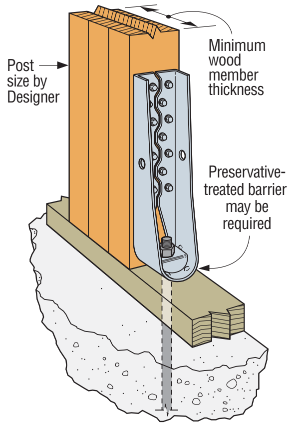
INSTALLATION: • See General Notes on page 45.

- The HDU requires no additional washer, the DTT requires a standard cut washer *(included with DTT2Z)* be installed between the nut and the seat.
- Strong-Drive SDS Heavy-Duty Connector screws install best with a low speed high torque drill with a ⅜" hex head driver.

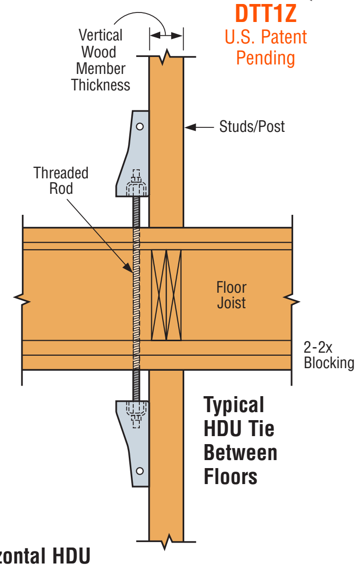
CODES: See page 12 for Code Reference Key Chart.

These products are available with additional corrosion protection. Additional products on this page may also be available with this option, check with Simpson Strong-Tie for details.

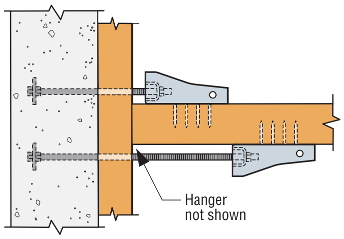




Vertical HDU Installation



Typical HDU Tie Between Floors



Horizontal HDU Offset Installation
(Plan View)

See Holdown and Tension Tie General Notes on page 45.

Model No.	Ga	Dimensions (in.)					Fasteners		Minimum Wood Member Thickness[4] (in.)	Allowable Tension Loads (160)[1]			Code Ref.
		W	H	B	℄	SO	Anchor Bolt Dia. (in.)	Post Fasteners		DF/SP	SPF/HF	Deflection at Allowable Load (in.)	
DTT1Z	14	1½	7⅛	1$^{7}/_{16}$	¾	$^{3}/_{16}$	⅜	6-SD #9x1½	1½	840	840	0.170	160
								6-10dx1½		910	640	0.167	
								8-10dx1½		910	850	0.167	
DTT2Z	14	3¼	6$^{15}/_{16}$	1⅝	$^{13}/_{16}$	$^{3}/_{16}$	½	8-SDS ¼"x1½"	1½	1825	1800	0.105	I6, L8, F5
								8-SDS ¼"x1½"	3	2145	1835	0.128	
DTT2Z-SDS2.5								8-SDS ¼"x2½"	3	2145	2105	0.128	
HDU2-SDS2.5	14	3	8$^{11}/_{16}$	3¼	1$^{5}/_{16}$	1⅜	⅝	6-SDS ¼"x2½"	3	3075	2215	0.088	
HDU4-SDS2.5	14	3	10$^{15}/_{16}$	3¼	1$^{5}/_{16}$	1⅜	⅝	10-SDS ¼"x2½"	3	4565	3285	0.114	
HDU5-SDS2.5	14	3	13$^{3}/_{16}$	3¼	1$^{5}/_{16}$	1⅜	⅝	14-SDS ¼"x2½"	3	5645	4065	0.115	
HDU8-SDS2.5	10	3	16⅝	3½	1⅜	1½	⅞	20-SDS ¼"x2½"	3	6765	4870	0.084	
									3½	6970	5020	0.116	
									4½	7870	5665	0.113	
HDU11-SDS2.5	10	3	22¼	3½	1⅜	1½	1	30-SDS ¼"x2½"	5½	9535	6865	0.137	
									7¼	11175	8045	0.137	
HDU14-SDS2.5	7	3	25$^{11}/_{16}$	3½	1$^{9}/_{16}$	1$^{9}/_{16}$	1	36-SDS ¼"x2½"	4x6[3,4]	10770	7755	0.122	170
									7¼[3]	14390	10435	0.177	I6, L8, F5
									5½[2,3]	14445	10350	0.177	

1. See page 45 for Holdown and Tension Tie General Notes.
2. Noted HDU14 allowable loads are based on a 5½" wide post *(6x6 min.)*.
3. HDU14 requires heavy hex anchor nut to achieve tabulated loads *(supplied with holdown)*.
4. Loads are applicable to installation on either narrow or wide face of post.

SB *Anchor Bolt*

SIMPSON Strong-Tie®

This product is preferable to similar connectors because of a) easier installation, b) higher loads, c) lower installed cost, or a combination of these features.

The SB⅝x24 anchor bolt offers a load-tested anchorage solution that exceeds the capacity of all of our holdowns that call for a ⅝" dia. anchor. Similarly, the SB1x30 covers holdowns utilizing a 1" diameter anchor that exceed the capacity of our SSTB bolts. The SB⅞x24 is designed to maximize performance with minimum embedment for holdowns utilizing a ⅞" dia. anchor.

SB anchor bolts are code listed by ICC-ES under the 2009 and 2012 IBC and IRC to meet the requirements of ICC-ES acceptance criteria – AC 399. ICC-ES ESR-2611 is the industry's first code report issued for proprietary anchor bolts evaluated to the criteria of AC 399.

Special Features:

- Indentification on the bolt head showing embedment angle and model
- Sweep geometry to optimize position in form
- Rolled thread for higher tensile capacity
- Hex nuts and plate washer fixed in position
- Available in HDG for additional corrosion resistance

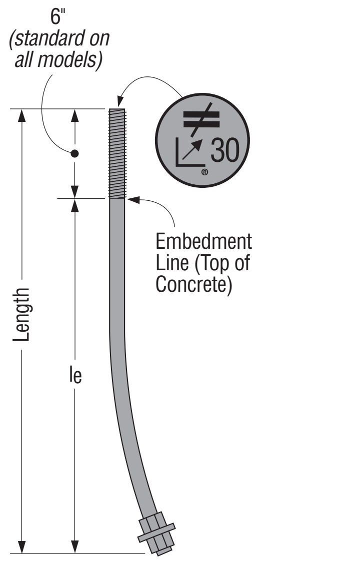
MATERIAL: ASTM F-1554, Grade 36

FINISH: None. May be ordered HDG. Contact Simpson Strong-Tie.

INSTALLATION:

- SB is only for concrete applications poured monolithically except where noted.
- Top nuts and washers for holdown attachment are not supplied with the SB; install standard nuts, couplers and/or washers as required.
- On HDG SB anchors, chase the threads to use standard nuts or couplers or use overtapped products in accordance with ASTM A563, for example Simpson Strong-Tie® NUT5/8-OST, NUT7/8-OST and NUT1-OST, CNW⅝-OST, CNW⅞-OST and CNW1-OST.
- Install SB before the concrete pour using AnchorMates®. Install the SB per the plan view detail.
- Minimum concrete compressive strength is 2500 psi.
- When rebar is required it does not need to be tied to the SB.

CODES: See page 12 for Code Reference Key Chart.



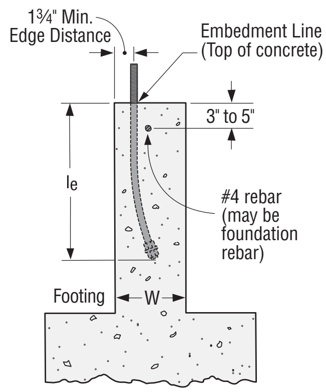
SB1x30
(Other models similar)

These products are available with additional corrosion protection. Additional products on this page may also be available with this option, check with Simpson Strong-Tie for details.

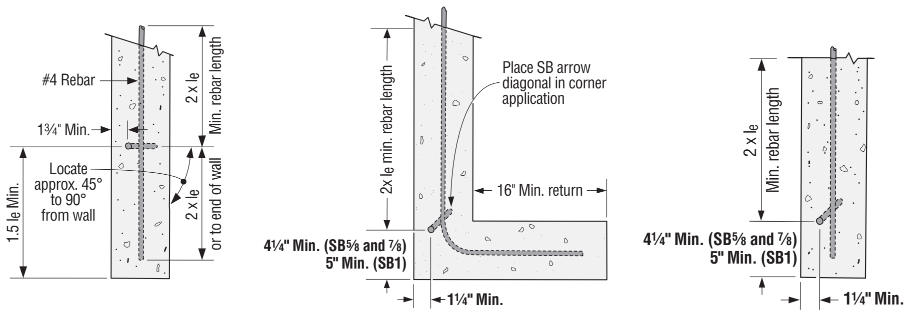
SB Bolts at Stemwall

Model No.	Dimensions (in.)				Allowable Tension Loads						Code Ref.
					Wind & SDC A&B			SDC C-F			
	Stemwall Width	Dia.	Length	Min. Embed. (le)	Midwall	Corner	End Wall	Midwall	Corner	End Wall	
SB⅝x24	6	⅝	24	18	6675	6675	6675	6675	5730	5730	I23, F30, L20
SB⅞x24	8	⅞	24	18	10470	9355	6820	8795	7855	5730	
SB1x30	8	1	30	24	13665	9905	7220	11470	8315	6065	

1. See page 34 for notes to the Designer.

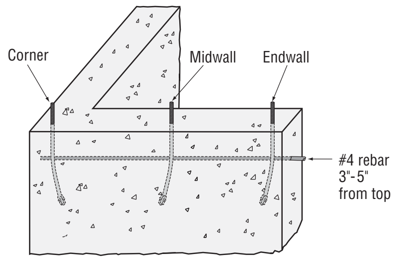


Typical SB Installation

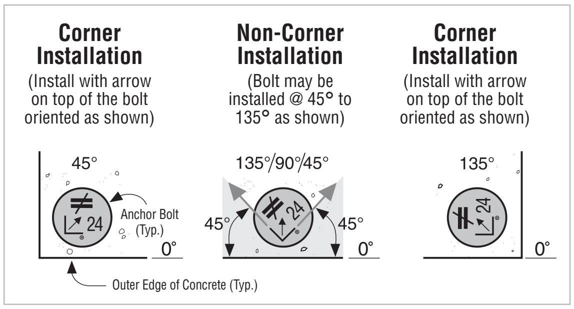


Midwall **Corner** **End Wall**

STEMWALL PLAN VIEWS



Perspective View

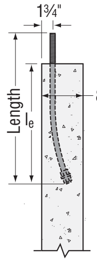


Plan View of SB Placement in Concrete

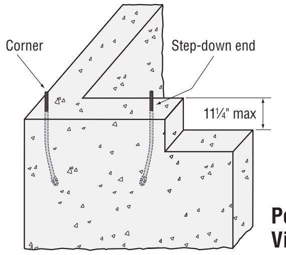
SIMPSON Strong-Tie®

SB Bolts at Stemwall: Garage Front

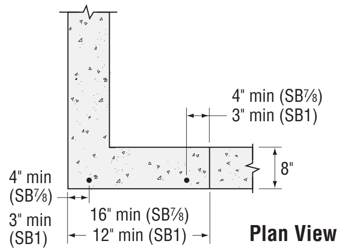
Model No.	Dimensions (in.)				Allowable Tension Loads				Code Ref.
	Stemwall Width	Dia.	Length	Min. Embed. (l_e)	Wind & SDC A&B		SDC C-F		
					Step-Down End	Corner	Step-Down End	Corner	
SB⅞x24	8	⅞	24	18	7225	7660	6070	6435	I23
SB1x30	8	1	30	24	11305	9635	9495	8030	



Stemwall Garage Front



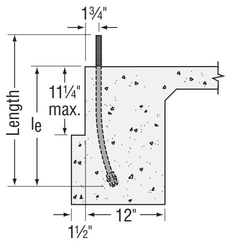
Perspective View



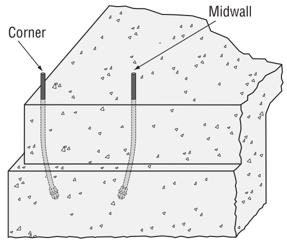
Plan View

SB Bolts at Slab on Grade: Edge

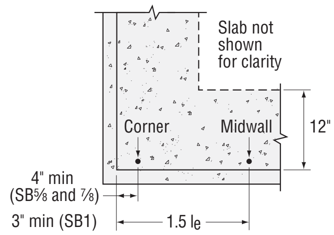
Model No.	Dimensions (in.)				Allowable Tension Loads				Code Ref.
	Footing Width	Dia.	Length	Min. Embed. (l_e)	Wind & SDC A&B		SDC C-F		
					Midwall	Corner	Midwall	Corner	
SB⅝x24	12	⅝	24	18	6675	6675	6675	5730	I23
SB⅞x24	12	⅞	24	18	13080	12135	12320	10190	
SB1x30	12	1	30	24	17080	15580	16300	13090	



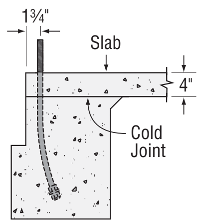
Slab Edge



Perspective View



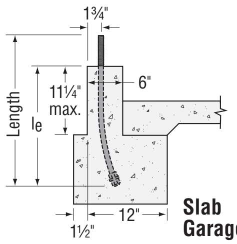
Plan View



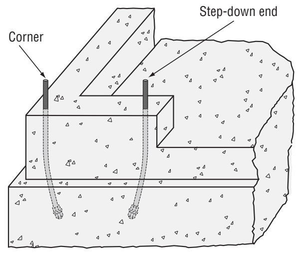
Two-Pour Installation

SB Bolts at Slab on Grade: Garage Curb

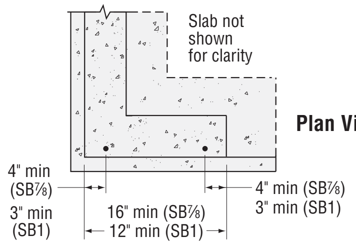
Model No.	Dimensions (in.)				Allowable Tension Loads				Code Ref.
	Curb Width	Dia.	Length	Min. Embed. (l_e)	Wind & SDC A&B		SDC C-F		
					Step-down End	Corner	Step-down End	Corner	
SB⅞x24	6	⅞	24	18	9175	11075	7705	9305	I23
SB1x30	6	1	30	24	15580	15580	13090	13090	



Slab Garage Curb



Perspective View



Plan View

Notes to the Designer:

1. Rebar is required at top of stemwall foundations but is not required for Slab-on-Grade Edge and Garage Curb, or Stemwall Garage Front installations.
2. Minimum end distances for SB bolts are as shown in graphics.
3. Multiply the tabulated ASD wind or seismic loads by 1.6 or 1.4, respectively, to obtain LRFD capacities.
4. Per Section 1613 of the IBC, detached one- and two-story dwellings in SDC C may use "Wind and SDC A&B" allowable loads.
5. See ESR-2611 for additional information.
6. Midwall loads apply when anchor is 1.5 l_e or greater from the end. For bolts acting in tension simultaneously, the minimum bolt center-to-center spacing is 3 l_e.
7. Full catalog loads apply for two-pour installation for slab-on-grade: edge.

2720 Residence

2720 71st Avenue SE
Mercer Island, Washington 98040

Structural Engineering Calculations

Supplement Calculations for Architectural Design Revisions

By

Dihong Shao, SE

January 18, 2022

Project Name				Page No.
Bm/Jst Location/Description:	**ROOF DECK JOIST FOR SPAN OF 18'-0"**			
Roof				
dead load (psf)	0.00			
live load (psf)	0.00	additional total point load (kips)		0.00
tributary width (ft)	**0.00**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	25.00			
live load (psf)	60.00	additional total point load (kips)		0.00
tributary width (ft)	**1.33**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**0.00**			
Beam Span (ft)	**18.00**			
load duration/repetitive factor	**1.00**		**1.00**	
tributary load (plf)	113.05	**11.875 TJI-360@16"**		
moment (kip-ft)	**4.58**	6.18 OK		
shear/reaction (kips)	**1.02**	1.08 OK		
EI x 10^6 (in^2-lbs)	**419**			Total Deflection
Joist Depth (in)	**11.88**	Bending Deflection	0.64	
TJI 110, 210, 230, and 360		Shear Deflection	0.08	0.72
TJI 560		Shear Deflection	0.07	0.71
			Total Deflction Ratio "L" over	**305**
			LL Deflection Ratio: "L" over	**432**

Bm/Jst Location/Description:	**ROOF DECK JOIST FOR SPAN OF 16'-6"**			
Roof				
dead load (psf)	0.00			
live load (psf)	0.00	additional total point load (kips)		0.00
tributary width (ft)	**0.00**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	25.00			
live load (psf)	60.00	additional total point load (kips)		0.00
tributary width (ft)	**1.33**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**0.00**			
Beam Span (ft)	**16.50**			
load duration/repetitive factor	**1.00**		**1.00**	
tributary load (plf)	113.05	**11.875 TJI-230@16"**		
moment (kip-ft)	**3.85**	4.125 OK		
shear/reaction (kips)	**0.93**	1.06 OK		
16'TJI-560@16"				
EI x 10^6 (in^2-lbs)	**347**			Total Deflection
Joist Depth (in)	**11.88**	Bending Deflection	0.54	
TJI 110, 210, 230, and 360		Shear Deflection	0.07	0.61
TJI 560		Shear Deflection	0.06	0.60
			Total Deflction Ratio "L" over	**329**
			LL Deflection Ratio: "L" over	**465**

Project Name			Page No.
Bm/Jst Location/Description:	**ROOF JOIST FOR SPAN OF 22'-4"**		
Roof			
dead load (psf)	15.00		
live load (psf)	25.00	additional total point load (kips)	0.00
tributary width (ft)	**2.00**	point load location to farthest support (ft)	0.00
Floor			
dead load (psf)	15.00		
live load (psf)	40.00	additional total point load (kips)	0.00
tributary width (ft)	**0.00**	point load location to farthest support (ft)	0.00
Wall			
wall weight (psf)	10.00		
height (ft)	**0.00**		
Beam Span (ft)	**22.33**		
load duration/repetitive factor	**1.00**	**1.00**	
tributary load (plf)	80.00	**11.875 TJI-560@2'-0"**	
moment (kip-ft)	**4.99**	9.5 OK	
shear/reaction (kips)	**0.89**	1.265 OK	
EI x 10^6 (in^2-lbs)	**636**		Total Deflection
Joist Depth (in)	**11.88**	Bending Deflection 0.70	
TJI 110, 210, 230, and 360		Shear Deflection 0.09	0.79
TJI 560		Shear Deflection 0.08	0.78
		Total Deflction Ratio "L" over	**343**
		LL Deflection Ratio: "L" over	**472**

Bm/Jst Location/Description:	**ROOF JOIST FOR SPAN OF 15'-8"**		
Roof			
dead load (psf)	15.00		
live load (psf)	25.00	additional total point load (kips)	0.00
tributary width (ft)	**2.00**	point load location to farthest support (ft)	0.00
Floor			
dead load (psf)	15.00		
live load (psf)	40.00	additional total point load (kips)	0.00
tributary width (ft)	**0.00**	point load location to farthest support (ft)	0.00
Wall			
wall weight (psf)	10.00		
height (ft)	**0.00**		
Beam Span (ft)	**15.67**		
load duration/repetitive factor	**1.00**	**1.00**	
tributary load (plf)	80.00	**11.875 TJI-110@2'-0"**	
moment (kip-ft)	**2.46**	3.16 OK	
shear/reaction (kips)	**0.63**	.910 OK	
EI x 10^6 (in^2-lbs)	**267**		Total Deflection
Joist Depth (in)	**11.88**	Bending Deflection 0.41	
TJI 110, 210, 230, and 360		Shear Deflection 0.04	0.45
TJI 560		Shear Deflection 0.04	0.44
		Total Deflction Ratio "L" over	**423**
		LL Deflection Ratio: "L" over	**582**

Project Name				**Page No.**
Bm/Jst Location/Description:	**MAIN FLOOR JOIST FOR SPAN OF 27'-3"**			
Roof				
dead load (psf)	0.00			
live load (psf)	0.00	additional total point load (kips)		0.00
tributary width (ft)	**0.00**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	30.00			
live load (psf)	40.00	additional total point load (kips)		0.00
tributary width (ft)	**1.33**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**0.00**			
Beam Span (ft)	**27.25**			
load duration/repetitive factor	**1.00**		**1.00**	
tributary load (plf)	93.10	**14" TJI-560@16"**		
moment (kip-ft)	**8.64**	11.275 OK		
shear/reaction (kips)	**1.27**	1.265 OK		
EI x 10^6 (in^2-lbs)	**926**			Total Deflection
Joist Depth (in)	**14.00**	Bending Deflection	1.25	
TJI 110, 210, 230, and 360		Shear Deflection	0.13	1.38
TJI 560		Shear Deflection	0.11	1.36
			Total Deflction Ratio "L" over	**240**
			LL Deflection Ratio: "L" over	**421**

Bm/Jst Location/Description:	**MAIN FLOOR JOIST FOR SPAN OF 23'-9"**			
Roof				
dead load (psf)	0.00			
live load (psf)	0.00	additional total point load (kips)		0.00
tributary width (ft)	**0.00**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	30.00			
live load (psf)	40.00	additional total point load (kips)		0.00
tributary width (ft)	**1.33**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**0.00**			
Beam Span (ft)	**23.75**			
load duration/repetitive factor	**1.00**		**1.00**	
tributary load (plf)	93.10	**14" TJI-360@16"**		
moment (kip-ft)	**6.56**	7.335 OK		
shear/reaction (kips)	**1.11**	1.08 OK		
EI x 10^6 (in^2-lbs)	**612**			Total Deflection
Joist Depth (in)	**14.00**	Bending Deflection	1.09	
TJI 110, 210, 230, and 360		Shear Deflection	0.10	1.19
TJI 560		Shear Deflection	0.09	1.17
			Total Deflction Ratio "L" over	**243**
			LL Deflection Ratio: "L" over	**424**

Project Name				Page No.

Bm/Jst Location/Description:	**R1**			
Roof				
dead load (psf)	15.00			
live load (psf)	25.00	additional total point load (kips)		0.00
tributary width (ft)	**7.50**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		0.00
tributary width (ft)	**0.00**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**0.00**			
Beam Span (ft)	**9.00**			
load duration/repetitive factor	**1.15**		1.00	
Beam Data Base Number	**9**		**2.0E PSL**	
tributary load (plf)	300.00		**#N/A**	**Beam No.61-88**
moment (kip-ft)	3.04		Provided M	#N/A
shear/reaction (kips)	1.35		Provided V	#N/A
			Provided I	#N/A
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	25.36	30.66	13.21	#N/A
Required I (in^4)	63.22	111.15	63.22	#N/A
Required A (in^2)	18.54	25.38	8.00	#N/A
Size	**4x8**	**Beam No.1-20**	**#N/A**	**Beam No.20-60**

Bm/Jst Location/Description:	**R2**			
Roof				
dead load (psf)	25.00			
live load (psf)	60.00	additional total point load (kips)		0.00
tributary width (ft)	**9.50**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		0.00
tributary width (ft)	**0.00**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**0.00**			
Beam Span (ft)	**6.00**			
load duration/repetitive factor	**1.00**		1.00	
Beam Data Base Number	**10**		**2.0E PSL**	
tributary load (plf)	807.50		**#N/A**	**Beam No.61-88**
moment (kip-ft)	3.63		Provided M	#N/A
shear/reaction (kips)	2.42		Provided V	#N/A
			Provided I	#N/A
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	34.88	49.91	18.17	#N/A
Required I (in^4)	50.42	230.84	50.42	#N/A
Required A (in^2)	38.25	32.38	22.02	#N/A
Size	**4x10**	**Beam No.1-20**	**#N/A**	**Beam No.20-60**

Project Name				Page No.
Bm/Jst Location/Description:	**R3**			
Roof				
dead load (psf)	15.00			
live load (psf)	25.00	additional total point load (kips)		0.00
tributary width (ft)	**10.50**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		0.00
tributary width (ft)	**0.00**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**0.00**			
Beam Span (ft)	**9.50**			
load duration/repetitive factor	**1.15**		**1.00**	

Beam Data Base Number	**10**		**2.0E PSL**	
tributary load (plf)	420.00		**#N/A**	**Beam No.61-88**
moment (kip-ft)	4.74		Provided M	#N/A
shear/reaction (kips)	2.00		Provided V	#N/A
			Provided I	#N/A
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	39.55	49.91	20.60	#N/A
Required I (in^4)	104.10	230.84	104.10	#N/A
Required A (in^2)	27.39	32.38	15.77	#N/A
Size	**4x10**	**Beam No.1-20**	**#N/A**	**Beam No.20-60**

Bm/Jst Location/Description:	**R4**			
Roof				
dead load (psf)	25.00			
live load (psf)	60.00	additional total point load (kips)		0.00
tributary width (ft)	**9.50**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		0.00
tributary width (ft)	**0.00**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**0.00**			
Beam Span (ft)	**16.00**	**I ratio**	0.98	2.0/1.7=1.17 OK
load duration/repetitive factor	**1.00**		**1.00**	

Beam Data Base Number	**85**		**2.0E PSL**	
tributary load (plf)	807.50		**7x11-7/8**	**Beam No.61-88**
moment (kip-ft)	25.84		Provided M	39.81
shear/reaction (kips)	6.46		Provided V	16.07
			Provided I	975.00
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	248.06	280.73	129.20	1200.45
Required I (in^4)	956.17	2456.38	956.17	26244.00
Required A (in^2)	102.00	96.25	58.72	243.00
Size	**6x18**	**Beam No.1-20**	**6-3/4x36**	**Beam No.20-60**

Project Name				Page No.
Bm/Jst Location/Description:	U1			
Roof				
dead load (psf)	25.00			
live load (psf)	60.00	additional total point load (kips)		0.00
tributary width (ft)	15.00	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		0.00
tributary width (ft)	6.00	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	10.00			
Beam Span (ft)	11.50			
load duration/repetitive factor	1.00		1.00	
Beam Data Base Number	72		2.0E PSL	
tributary load (plf)	1705.00		3-1/2x14	Beam No.61-88
moment (kip-ft)	28.19		Provided M	27.16
shear/reaction (kips)	9.80		Provided V	9.48
			Provided I	800.00
	DF#2	Provided	24F-V4 or 24F-V8 DF GL	Provided
Required S (in^3)	270.58	280.73	140.93	1200.45
Required I (in^4)	749.64	2456.38	749.64	26244.00
Required A (in^2)	154.80	96.25	89.12	243.00
Size	6x18	Beam No.1-20	6-3/4x36	Beam No.20-60

Bm/Jst Location/Description:	U2			
Roof				
dead load (psf)	15.00			
live load (psf)	25.00	additional total point load (kips)		0.00
tributary width (ft)	11.00	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		9.80
tributary width (ft)	0.00	point load location to farthest support (ft)		12.00
Wall				
wall weight (psf)	10.00	S ratio	1.05	5.5/5.125=1.07
height (ft)	6.00	I ratio	0.98	
Beam Span (ft)	21.00	A ratio	0.87	
load duration/repetitive factor	1.05		1.00	
Beam Data Base Number	41		2.0E PSL	
tributary load (plf)	500.00		#N/A	Beam No.61-88
moment (kip-ft)	77.98		Provided M	#N/A
shear/reaction (kips)	10.85		Provided V	#N/A
			Provided I	#N/A
	DF#2	Provided	24F-V4 or 24F-V8 DF GL	Provided
Required S (in^3)	712.98	280.73	371.34	353.98
Required I (in^4)	3787.36	2456.38	3787.36	3855.72
Required A (in^2)	130.56	96.25	93.95	107.83
Size	6x18	Beam No.1-20	5-1/8x21	Beam No.20-60

Project Name				Page No.
Bm/Jst Location/Description:	**U3**			
Roof				
dead load (psf)	15.00			
live load (psf)	25.00	additional total point load (kips)		0.00
tributary width (ft)	**4.50**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		0.00
tributary width (ft)	**0.00**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**0.00**			
Beam Span (ft)	**16.50**			
load duration/repetitive factor	**1.00**		**1.00**	
Beam Data Base Number	**16**		**2.0E PSL**	
tributary load (plf)	180.00		**#N/A**	**Beam No.61-88**
moment (kip-ft)	6.13		Provided M	#N/A
shear/reaction (kips)	1.49		Provided V	#N/A
			Provided I	#N/A
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	58.81	82.73	30.63	#N/A
Required I (in^4)	233.75	392.96	233.75	#N/A
Required A (in^2)	23.45	52.25	9.56	#N/A
Size	**6x10**	**Beam No.1-20**	**#N/A**	**Beam No.20-60**

Bm/Jst Location/Description:	**U4**			
Roof				
dead load (psf)	15.00	average of 25 an		
live load (psf)	45.00	additional total point load (kips)		0.00
tributary width (ft)	**16.00**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		0.00
tributary width (ft)	**8.00**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**10.00**			
Beam Span (ft)	**10.00**			
load duration/repetitive factor	**1.00**		**1.00**	
Beam Data Base Number	**72**		**2.0E PSL**	
tributary load (plf)	1500.00		**3-1/2x14**	**Beam No.61-88**
moment (kip-ft)	18.75		Provided M	27.16
shear/reaction (kips)	7.50		Provided V	9.48
			Provided I	800.00
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	180.00	280.73	93.75	1200.45
Required I (in^4)	433.64	2456.38	433.64	26244.00
Required A (in^2)	118.43	96.25	68.18	243.00
Size	**6x18**	**Beam No.1-20**	**6-3/4x36**	**Beam No.20-60**

Project Name				Page No.
Bm/Jst Location/Description:	**U5**			
Roof				
dead load (psf)	25.00	average of 25 and 60 psf		
live load (psf)	60.00	additional total point load (kips)		0.00
tributary width (ft)	**9.50**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		0.00
tributary width (ft)	**9.50**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**10.00**			
Beam Span (ft)	**10.00**			
load duration/repetitive factor	**1.00**		**1.00**	

Beam Data Base Number	**71**		**2.0E PSL**	
tributary load (plf)	1430.00		**3-1/2x11-7/8**	**Beam No.61-88**
moment (kip-ft)	17.88		Provided M	19.90
shear/reaction (kips)	7.15		Provided V	8.04
			Provided I	490.00
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	171.60	280.73	89.38	1200.45
Required I (in^4)	413.40	2456.38	413.40	26244.00
Required A (in^2)	94.08	96.25	64.99	243.00
Size	**6x18**	**Beam No.1-20**	**6-3/4x36**	**Beam No.20-60**

Bm/Jst Location/Description:	**U6**			
Roof				
dead load (psf)	20.00			
live load (psf)	45.00	additional total point load (kips)		0.00
tributary width (ft)	**16.00**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		0.00
tributary width (ft)	**8.00**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**10.00**			
Beam Span (ft)	**16.00**	**I ratio**	1.17	
load duration/repetitive factor	**1.00**		**1.00**	

Beam Data Base Number	**86**		**2.0E PSL**	
tributary load (plf)	1580.00		**7x14**	**Beam No.61-88**
moment (kip-ft)	50.56		Provided M	54.33
shear/reaction (kips)	12.64		Provided V	18.95
			Provided I	1600.00
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	485.38	280.73	252.80	1200.45
Required I (in^4)	1870.90	2456.38	1870.90	26244.00
Required A (in^2)	149.69	96.25	114.90	243.00
Size	**6x18**	**Beam No.1-20**	**6-3/4x36**	**Beam No.20-60**

Project Name				Page No.
Bm/Jst Location/Description:	U6 STEEL BM			
Roof				
dead load (psf)	20.00			
live load (psf)	45.00	additional total point load (kips)		0.00
tributary width (ft)	16.00	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		0.00
tributary width (ft)	8.00	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	10.00			
Beam Span (ft)	16.00			
load duration/repetitive factor	1.00		1.00	
Beam Data Base Number			STEEL BEAM	W12x26
tributary load (plf)	1580.00		Required	Provided
moment (kip-ft)	50.56	S	25.79	33.40
shear/reaction (kips)	12.64	BxH		6.5" x 12.25"
		I	150.47	204.00
	DF#2	Provided	24F-V4 or 24F-V8 DF GL	Provided
Required S (in^3)	485.38	#N/A	252.80	#N/A
Required I (in^4)	1870.90	#N/A	1870.90	#N/A
Required A (in^2)	199.59	#N/A	76.60	#N/A
Size	#N/A	Beam No.1-20	#N/A	Beam No.20-60

Bm/Jst Location/Description:	U7 STEEL BM			
Roof				
dead load (psf)	25.00			
live load (psf)	60.00	additional total point load (kips)		0.00
tributary width (ft)	9.50	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		6.46
tributary width (ft)	14.00	point load location to farthest support (ft)		15.00
Wall				
wall weight (psf)	10.00			
height (ft)	10.00			
Beam Span (ft)	21.00			
load duration/repetitive factor	1.00		1.00	
Beam Data Base Number			STEEL BEAM	W14x53
tributary load (plf)	1677.50		Required	Provided
moment (kip-ft)	120.16	S	61.28	77.80
shear/reaction (kips)	22.23	BxH		8.06" x 13.92"
		I	469.34	541.00
	DF#2	Provided	24F-V4 or 24F-V8 DF GL	Provided
Required S (in^3)	1153.52	#N/A	600.79	#N/A
Required I (in^4)	5835.74	#N/A	5835.74	#N/A
Required A (in^2)	350.98	#N/A	134.70	#N/A
Size	#N/A	Beam No.1-20	#N/A	Beam No.20-60

Project Name				Page No.
Bm/Jst Location/Description:	**U8 STEEL BM**			
Roof				
dead load (psf)	15.00			
live load (psf)	25.00	additional total point load (kips)		12.64
tributary width (ft)	**15.00**	point load location to farthest support (ft)		10.50
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		22.23
tributary width (ft)	**2.00**	point load location to farthest support (ft)		11.50
Wall				
wall weight (psf)	10.00			
height (ft)	**10.00**			
Beam Span (ft)	**18.50**			
load duration/repetitive factor	**1.00**		**1.00**	
Beam Data Base Number			**STEEL BEAM**	**W18x55**
tributary load (plf)	810.00		Required	Provided
moment (kip-ft)	188.77	S	96.27	98.30
shear/reaction (kips)	28.48	BxH		7.5" x 18.25"
		I	649.55	890.00
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	1812.17	#N/A	943.84	#N/A
Required I (in^4)	8076.49	#N/A	8076.49	#N/A
Required A (in^2)	449.76	#N/A	172.61	#N/A
Size	**#N/A**	**Beam No.1-20**	**#N/A**	**Beam No.20-60**

Bm/Jst Location/Description:	**NOT USED STEEL BM**			
Roof				
dead load (psf)	15.00			
live load (psf)	25.00	additional total point load (kips)		0.00
tributary width (ft)	**15.00**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		10.00
tributary width (ft)	**6.00**	point load location to farthest support (ft)		15.50
Wall				
wall weight (psf)	10.00			
height (ft)	**19.50**			
Beam Span (ft)	**22.00**			
load duration/repetitive factor	**1.00**		**1.00**	
Beam Data Base Number			**STEEL BEAM**	**W14x53**
tributary load (plf)	1125.00		Required	Provided
moment (kip-ft)	113.86	S	58.07	77.80
shear/reaction (kips)	19.42	BxH		8.06" x 13.92"
		I	465.91	541.00
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	1093.04	#N/A	569.29	#N/A
Required I (in^4)	5793.09	#N/A	5793.09	#N/A
Required A (in^2)	306.65	#N/A	117.69	#N/A
Size	**#N/A**	**Beam No.1-20**	**#N/A**	**Beam No.20-60**

Project Name				Page No.
Bm/Jst Location/Description:	**U9**			
Roof				
dead load (psf)	25.00			
live load (psf)	60.00	additional total point load (kips)		0.00
tributary width (ft)	**3.00**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		0.00
tributary width (ft)	**0.00**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**4.00**			
Beam Span (ft)	**14.50**			
load duration/repetitive factor	**1.00**		**1.00**	
Beam Data Base Number	**16**		**2.0E PSL**	
tributary load (plf)	295.00		**#N/A**	**Beam No.61-88**
moment (kip-ft)	7.75		Provided M	#N/A
shear/reaction (kips)	2.14		Provided V	#N/A
			Provided I	#N/A
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	74.43	82.73	38.76	#N/A
Required I (in^4)	259.99	392.96	259.99	#N/A
Required A (in^2)	33.77	52.25	15.55	#N/A
Size	**6x10**	**Beam No.1-20**	**#N/A**	**Beam No.20-60**

Bm/Jst Location/Description:	**M1**			
Roof				
dead load (psf)	15.00			
live load (psf)	25.00	additional total point load (kips)		0.00
tributary width (ft)	**11.00**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00	**BOTH MAIN AND UPPER FLOORS**		
live load (psf)	40.00	additional total point load (kips)		0.00
tributary width (ft)	**11.00**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**22.00**			
Beam Span (ft)	**15.50**			
load duration/repetitive factor	**1.00**		**1.00**	
Beam Data Base Number	**86**		**2.0E PSL**	
tributary load (plf)	1265.00		**7x14**	**Beam No.61-88**
moment (kip-ft)	37.99		Provided M	54.33
shear/reaction (kips)	9.80		Provided V	18.95
			Provided I	1600.00
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	364.70	280.73	189.95	1200.45
Required I (in^4)	1361.82	2456.38	1361.82	26244.00
Required A (in^2)	154.80	96.25	89.12	243.00
Size	**6x18**	**Beam No.1-20**	**6-3/4x36**	**Beam No.20-60**

Project Name				Page No.
Bm/Jst Location/Description:	**M2**			
Roof				
dead load (psf)	25.00			
live load (psf)	60.00	additional total point load (kips)		0.00
tributary width (ft)	**9.50**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		0.00
tributary width (ft)	**9.50**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**22.00**			
Beam Span (ft)	**6.00**			
load duration/repetitive factor	**1.00**		**1.00**	

Beam Data Base Number	**69**		**2.0E PSL**	
tributary load (plf)	1550.00		**3-1/2x9-1/2**	**Beam No.61-88**
moment (kip-ft)	6.98		Provided M	13.06
shear/reaction (kips)	4.65		Provided V	6.43
			Provided I	250.00
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	66.96	280.73	34.88	1200.45
Required I (in^4)	96.79	2456.38	96.79	26244.00
Required A (in^2)	48.95	96.25	42.27	243.00
Size	**6x18**	**Beam No.1-20**	**6-3/4x36**	**Beam No.20-60**

Bm/Jst Location/Description:	**M3**			
Roof				
dead load (psf)	15.00			
live load (psf)	25.00	additional total point load (kips)		0.00
tributary width (ft)	**0.00**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		0.00
tributary width (ft)	**14.00**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**0.00**			
Beam Span (ft)	**6.00**			
load duration/repetitive factor	**1.00**		**1.00**	

Beam Data Base Number	**10**		**2.0E PSL**	
tributary load (plf)	770.00		**#N/A**	**Beam No.61-88**
moment (kip-ft)	3.47		Provided M	#N/A
shear/reaction (kips)	2.31		Provided V	#N/A
			Provided I	#N/A
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	33.26	49.91	17.33	#N/A
Required I (in^4)	48.08	230.84	48.08	#N/A
Required A (in^2)	30.40	32.38	21.00	#N/A
Size	**4x10**	**Beam No.1-20**	**#N/A**	**Beam No.20-60**

Project Name				Page No.
Bm/Jst Location/Description:	**M4**			
Roof				
dead load (psf)	15.00			
live load (psf)	25.00	additional total point load (kips)		0.00
tributary width (ft)	**0.00**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00			
live load (psf)	40.00	additional total point load (kips)		0.00
tributary width (ft)	**14.00**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**22.00**			
Beam Span (ft)	**11.00**	**DOUBLE THE CANTILEVER SPAN**		
load duration/repetitive factor	**1.00**		**1.00**	

Beam Data Base Number			**STEEL BEAM**	**W10x22**
			Required	Provided
tributary load (plf)	990.00			
moment (kip-ft)	14.97	**S**	7.64	23.20
shear/reaction (kips)	5.45	**BxH**		5.75" x 10.125"
		I	30.64	118.00

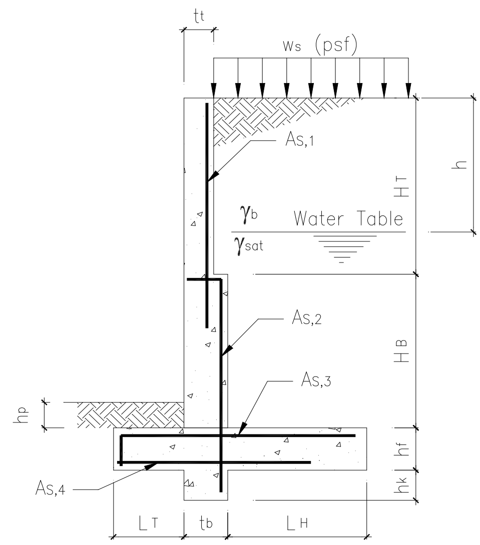
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	143.75	#N/A	74.87	#N/A
Required I (in^4)	380.93	#N/A	380.93	#N/A
Required A (in^2)	85.98	#N/A	33.00	#N/A
Size	**#N/A**	**Beam No.1-20**	**#N/A**	**Beam No.20-60**

Retaining Wall Design Based on ACI 318 — 4ft Retaining Wall

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c'	=	2.5	ksi		
REBAR YIELD STRESS	f_y	=	60	ksi		
LATERAL SOIL PRESSURE	P_a=$k_a\,\gamma_b$	=	35	pcf (equivalent fluid pressure)		
BACKFILL SPECIFIC WEIGHT	γ_b	=	110	pcf		
SATURATED SPECIFIC WEIGHT	γ_{sat}	=	118	pcf		
WATER TABLE DEPTH	h	=	10	ft		
PASSIVE PRESSURE	P_p	=	400	psf / ft		
SURCHARGE WEIGHT	w_s	=	40	psf		
FRICTION COEFFICIENT	μ	=	0.4			
ALLOW SOIL PRESSURE	Q_a	=	1.5	ksf		
THICKNESS OF TOP STEM	t_t	=	8	in		
THICKNESS OF KEY & STEM	t_b	=	8	in		
TOE WIDTH	L_T	=	0.67	ft		
HEEL WIDTH	L_H	=	1.16	ft		
HEIGHT OF TOP STEM	H_T	=	3.5	ft		
HEIGHT OF BOT. STEM	H_B	=	0.5	ft		
FOOTING THICKNESS	h_f	=	12	in		
KEY DEPTH	h_k	=	0	in		
SOIL OVER TOE	h_p	=	6	in		
TOP STEM REINF. ($A_{s,1}$)		#	5	@	15	in o.c.
$A_{s,1}$ LOCATION (0=at soil face, 1=at middle, 2=at each face)					1	at middle
BOT. STEM REINF. ($A_{s,2}$)		#	5	@	15	in o.c.
$A_{s,2}$ LOCATION (0=at soil face, 1=at middle, 2=at each face)					1	at middle
TOP REINF.OF FOOTING ($A_{s,3}$)		#	5	@	18	in o.c.
BOT. REINF.OF FOOTING ($A_{s,4}$)		#	5	@	18	in

[THE WALL DESIGN IS ADEQUATE.]



ANALYSIS

SERVICE LOADS

$H_b = 0.5\,P_a\,h^2 + h\,P_a\,H + 0.5\,[P_a\,(\gamma_{sat} - \gamma_w)\,/\,\gamma_b + \gamma_w]\,H^2$ = 0.44 kips

Where h = 5 ft , H = 0 ft

$H_s = w_s\,P_a\,(H_T + H_B + h_f)\,/\,\gamma_b$ = 0.06 kips

$H_p = 0.5\,P_p\,(h_p + h_f + h_k)^2$ = 0.45 kips

$W_s = w_s\,(L_H + t_b - t_t)$ = 0.05 kips

$W_b = W_{b1} + W_{b2}$ = 0.51 kips

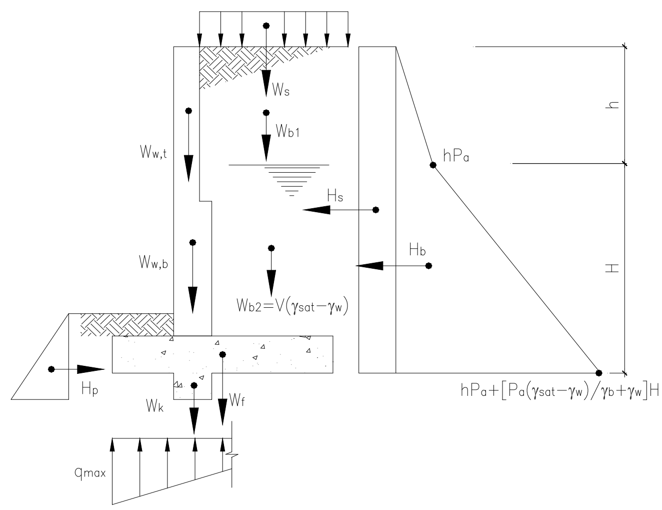
Where W_{b1} = 0.51 kips , W_{b2} = 0.00 kips

$W_f = h_f\,(L_H + t_b + L_T)\,\gamma_c$ = 0.37 kips

$W_k = h_k\;t_b\;\gamma_c$ = 0.00 kips

$W_{w,t} = t_t\;H_T\;\gamma_c$ = 0.35 kips

$W_{w,b} = t_b\;H_B\;\gamma_c$ = 0.05 kips



FACTORED LOADS

$\gamma H_b = 1.6\,H_b$ = 0.70 kips

$\gamma H_s = 1.6\,H_s$ = 0.10 kips

$\gamma W_s = 1.6\,W_s$ = 0.07 kips

$\gamma W_b = 1.2\,W_b$ = 0.61 kips

$\gamma W_f = 1.2\,W_f$ = 0.45 kips

$\gamma W_k = 1.2\,W_k$ = 0.00 kips

$\gamma W_{w,t} = 1.2\,W_{w,t}$ = 0.42 kips

$\gamma W_{w,b} = 1.2\,W_{w,b}$ = 0.06 kips

OVERTURNING MOMENT

	H	γH	y	H y	γH y
H_b	0.44	0.70	1.67	0.73	1.17
H_s	0.06	0.10	2.50	0.16	0.25
Σ	0.50	0.80		**0.89**	1.42

RESISTING MOMENT

	W	γW	x	W x	γW x
W_s	0.05	0.07	1.92	0.09	0.14
W_b	0.51	0.61	1.92	0.98	1.17
W_f	0.37	0.45	1.25	0.47	0.56
W_k	0.00	0.00	1.00	0.00	0.00
$W_{w,t}$	0.35	0.42	1.00	0.35	0.42
$W_{w,b}$	0.05	0.06	1.00	0.05	0.06
Σ	1.33	1.62		**1.94**	2.36

OVERTURNING FACTOR OF SAFETY (1806.1)

$$SF = \frac{\Sigma Wx}{\Sigma Hy}$$

2.18 **>** 1.5

[Satisfactory]

(cont'd)

CHECK SOIL BEARING CAPACITY (ACI 318-05 SEC.15.2.2)

$L = L_T + t_b + L_H$ = 2.50 ft

$e = \frac{L}{2} - \frac{\Sigma W x - H y}{\Sigma W}$ = 0.46 ft

$$q_{MAX} = \begin{cases} \frac{\Sigma W\left(1+\frac{6e}{L}\right)}{BL}, & for \quad e \le \frac{L}{6} \\ \frac{2\Sigma W}{3B(0.5L-e)}, & for \quad e > \frac{L}{6} \end{cases}$$

= 1.13 ksf < Q_a **[Satisfactory]**

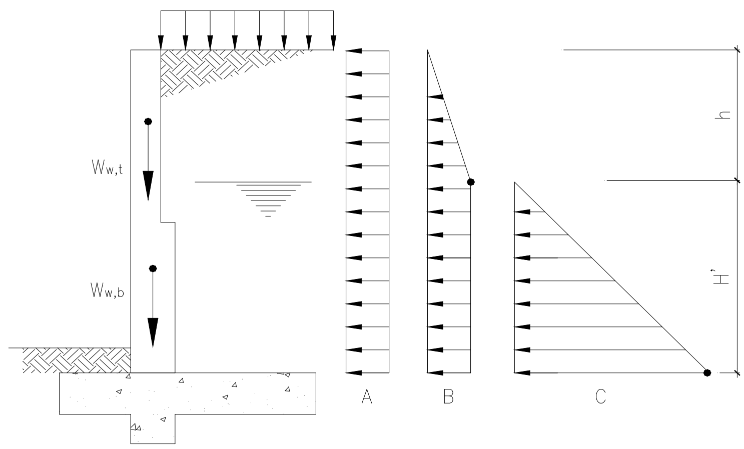
CHECK FLEXURE CAPACITY, AS,1 & AS,2, FOR STEM (ACI 318-05 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

h = 4 ft , H' = 0 ft

A = $w_s P_a / \gamma_b$ = 13 plf

B = $h P_a$ = 140 plf

C = $[P_a (\gamma_{sat} - \gamma_w) / \gamma_b + \gamma_w]$ H' = 0 plf



At base of top stem

M_u =	0.52	ft-kips
V_u =	0.41	kips
P_u =	0.42	kips

At base of bottom stem

M_u =	0.76	ft-kips
V_u =	0.53	kips
P_u =	0.48	kips

		At top stem		At base of bottom stem	
$\phi M_n = \phi \left[A_s f_y \left(d - \frac{A_s f_y - P_u}{1.7 b f'_c} \right) \right]$	=	4.15	ft-kips ,	4.15	ft-kips
		>	M_u	>	M_u
			[Satisfactory]		**[Satisfactory]**
where d	=	4.00	in ,	4.00	in
b	=	12	in ,	12	in
ϕ	=	0.9	(ACI 318 Fig R9.3.2)	0.9	(ACI 318 Fig R9.3.2)
A_s	=	0.248	in^2 ,	0.248	in^2
ρ	=	0.005		0.005	
$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y} \frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t}$	=	0.013		0.013	
		>	ρ	>	ρ
			[Satisfactory]		**[Satisfactory]**
$\rho_{MIN} = 0.0018 \frac{t}{d}$	=	0.004		0.004	
		<	ρ	<	ρ
			[Satisfactory]		**[Satisfactory]**

CHECK SHEAR CAPACITY FOR STEM (ACI 318-05 SEC.15.5.2, 11.1.3.1, & 11.3)

		At top stem		At base of bottom stem	
$V_{allowable} = 2\phi b d \sqrt{f'_c}$	=	3.60	kips ,	3.60	kips
		>	V_u	>	V_u
			[Satisfactory]		**[Satisfactory]**

where ϕ = 0.75 (ACI 318-05, Section 9.3.2.3)

CHECK HEEL FLEXURE CAPACITY, AS,3, FOR FOOTING (ACI 318-05 SEC.15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, & 12.5)

$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y} \frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t}$ = 0.013

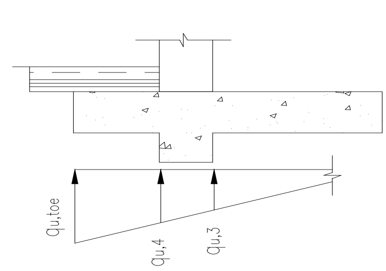
$\rho_{MIN} = \frac{0.0018}{2} \frac{h_f}{d}$ = 0.001

$$M_{u,3} = \begin{cases} \frac{L_H}{2}\left(\gamma_s + w_b + \frac{L_H}{L} w_f\right) - \frac{(q_{u,3} + 2q_{u,heel}) b L_H^2}{6}, & for \quad e_u \le \frac{L}{6} \\ \frac{L_H}{2}\left(\gamma_s + w_b + \frac{L_H}{L} w_f\right) - \frac{q_{u,3} b S^2}{6}, & for \quad e_u > \frac{L}{6} \end{cases}$$

= 0.51 ft-kips

(cont'd)

$$\rho = \frac{0.85 f_c^{'}\left(1-\sqrt{1-\frac{M_{u,3}}{0.383 b d^2 f_c^{'}}}\right)}{f_y} \quad = \quad 0.000$$



where	d	=	10.19	in	$q_{u,\ toe}$	=	1.86	ksf
	e_u	=	0.67	ft	$q_{u,\ heel}$	=	n/a	ksf
	S	=	0.40	ft	$q_{u,\ 3}$	=	0.43	ksf

$(A_{S,\ 3})_{required}$ = 0.13 in^2 / ft **<** $A_{S,\ 3}$ **[Satisfactory]**

CHECK TOE FLEXURE CAPACITY, AS,4, FOR FOOTING (ACI 318-05 SEC.15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = \frac{0.85\beta_1 f_c^{'}}{f_y}\frac{\varepsilon u}{\varepsilon u + \varepsilon t} \quad = \quad 0.013 \qquad \rho_{MIN} = MIN\left(\frac{4}{3}\rho,\ \frac{0.0018}{2}\frac{h_f}{d}\right) \quad = \quad 0.000$$

$$M_{u,4} = \frac{\left(q_{u,4} + 2q_{u,toe}\right) b L_T^2}{6} - \frac{L_T^2}{2L}\gamma w_f \quad = \quad 0.32 \text{ ft-kips}$$

where	d	=	8.69	in
	$q_{u,\ 4}$	=	1.14	ksf

$$\rho = \frac{0.85 f_c^{'}\left(1-\sqrt{1-\frac{M_{u,4}}{0.383 b d^2 f_c^{'}}}\right)}{f_y} \quad = \quad 0.000$$

$(A_{S,\ 4})_{required}$ = 0.01 in^2 / ft **<** $A_{S,\ 4}$ **[Satisfactory]**

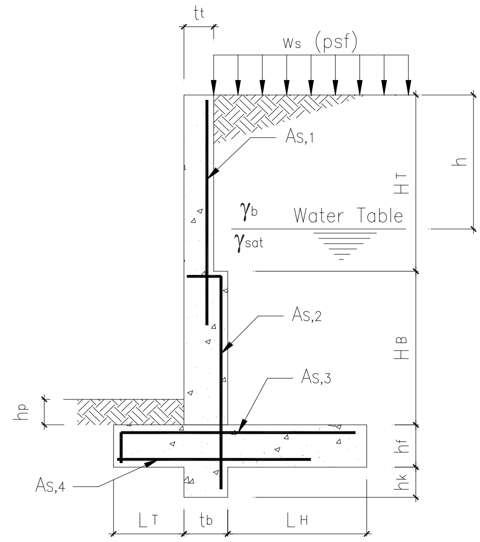
CHECK SLIDING CAPACITY (IBC 1806.1)

1.5 (H_b + H_s) = 0.75 kips **<** $H_p + \mu\ \Sigma W$ = 0.98 kips
[Satisfactory]

Technical References:
1. Alan Williams: "Structural Engineering Reference Manual", Professional Publications, Inc, 2001.
2. Alan Williams: "Structural Engineering License Review Problems and Solutions", Oxford University Press, 2003.

Retaining Wall Design Based on ACI 318 — 6ft Retaining Wall

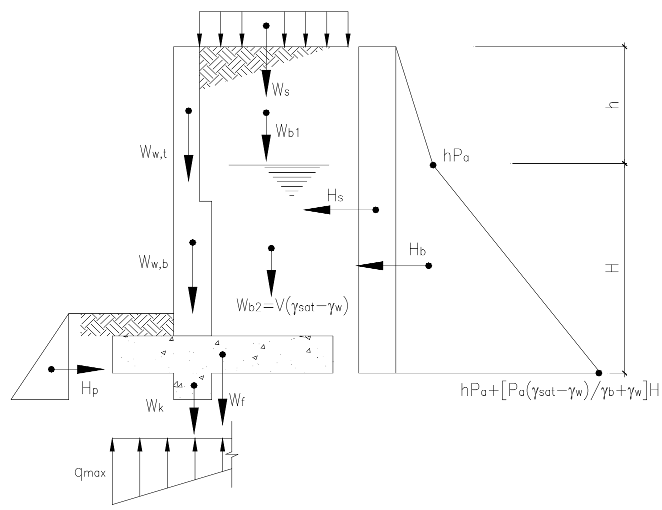
INPUT DATA & DESIGN SUMMARY



CONCRETE STRENGTH	f_c'	=	2.5	ksi		
REBAR YIELD STRESS	f_y	=	60	ksi		
LATERAL SOIL PRESSURE	$P_a=k_a\,\gamma_b$	=	35	pcf (equivalent fluid pressure)		
BACKFILL SPECIFIC WEIGHT	γ_b	=	110	pcf		
SATURATED SPECIFIC WEIGHT	γ_{sat}	=	118	pcf		
WATER TABLE DEPTH	h	=	10	ft		
PASSIVE PRESSURE	P_p	=	400	psf / ft		
SURCHARGE WEIGHT	w_s	=	40	psf		
FRICTION COEFFICIENT	μ	=	0.4			
ALLOW SOIL PRESSURE	Q_a	=	1.5	ksf		
THICKNESS OF TOP STEM	t_t	=	8	in		
THICKNESS OF KEY & STEM	t_b	=	8	in		
TOE WIDTH	L_T	=	1	ft		
HEEL WIDTH	L_H	=	1.83	ft		
HEIGHT OF TOP STEM	H_T	=	5.5	ft		
HEIGHT OF BOT. STEM	H_B	=	0.5	ft		
FOOTING THICKNESS	h_f	=	12	in		
KEY DEPTH	h_k	=	0	in		
SOIL OVER TOE	h_p	=	6	in		
TOP STEM REINF. ($A_{s,1}$)		#	5	@	12	in o.c.
$A_{s,1}$ LOCATION (0=at soil face, 1=at middle, 2=at each face)					1	at middle
BOT. STEM REINF. ($A_{s,2}$)		#	5	@	12	in o.c.
$A_{s,2}$ LOCATION (0=at soil face, 1=at middle, 2=at each face)					1	at middle
TOP REINF.OF FOOTING ($A_{s,3}$)		#	5	@	18	in o.c.
BOT. REINF.OF FOOTING ($A_{s,4}$)		#	5	@	18	in

[THE WALL DESIGN IS ADEQUATE.]

ANALYSIS



SERVICE LOADS

$H_b = 0.5\,P_a\,h^2 + h\,P_a\,H + 0.5\,[P_a\,(\gamma_{sat} - \gamma_w) / \gamma_b + \gamma_w]\,H^2$ = 0.86 kips

Where h = 7 ft , H = 0 ft

$H_s = w_s\,P_a\,(H_T + H_B + h_f) / \gamma_b$ = 0.08 kips

$H_p = 0.5\,P_p\,(h_p + h_f + h_k)^2$ = 0.45 kips

$W_s = w_s\,(L_H + t_b - t_t)$ = 0.06 kips

$W_b = W_{b1} + W_{b2}$ = 1.21 kips

Where W_{b1} = 1.21 kips , W_{b2} = 0.00 kips

$W_f = h_f\,(L_H + t_b + L_T)\,\gamma_c$ = 0.52 kips

$W_k = h_k\;t_b\;\gamma_c$ = 0.00 kips

$W_{w,t} = t_t\;H_T\;\gamma_c$ = 0.55 kips

$W_{w,b} = t_b\;H_B\;\gamma_c$ = 0.05 kips

FACTORED LOADS

$\gamma H_b = 1.6\,H_b$ = 1.37 kips

$\gamma H_s = 1.6\,H_s$ = 0.12 kips

$\gamma W_s = 1.6\,W_s$ = 0.10 kips

$\gamma W_b = 1.2\,W_b$ = 1.45 kips

$\gamma W_f = 1.2\,W_f$ = 0.63 kips

$\gamma W_k = 1.2\,W_k$ = 0.00 kips

$\gamma W_{w,t} = 1.2\,W_{w,t}$ = 0.66 kips

$\gamma W_{w,b} = 1.2\,W_{w,b}$ = 0.06 kips

OVERTURNING MOMENT

	H	γH	y	H y	γH y
H_b	0.86	1.37	2.33	2	3.20
H_s	0.08	0.12	3.50	0.27	0.44
Σ	0.94	1.50		**2.27**	3.64

RESISTING MOMENT

	W	γW	x	W x	γW x
W_s	0.06	0.10	2.58	0.17	0.26
W_b	1.21	1.45	2.58	3.12	3.74
W_f	0.52	0.63	1.75	0.92	1.10
W_k	0.00	0.00	1.33	0.00	0.00
$W_{w,t}$	0.55	0.66	1.33	0.73	0.88
$W_{w,b}$	0.05	0.06	1.33	0.07	0.08
Σ	2.40	2.90		**5.00**	6.07

OVERTURNING FACTOR OF SAFETY (1806.1)

$$SF = \frac{\Sigma Wx}{\Sigma Hy}$$ = 2.2 > 1.5

[Satisfactory]

(cont'd)

CHECK SOIL BEARING CAPACITY (ACI 318-05 SEC.15.2.2)

$$L = L_T + t_b + L_H \quad = \quad 3.50 \text{ ft}$$

$$e = \frac{L}{2} - \frac{\Sigma Wx - Hy}{\Sigma W} \quad = \quad 0.61 \text{ ft}$$

$$q_{MAX} = \begin{cases} \dfrac{\Sigma W\left(1+\dfrac{6e}{L}\right)}{BL}, & for \quad e \le \dfrac{L}{6} \\ \dfrac{2\Sigma W}{3B(0.5L-e)}, & for \quad e > \dfrac{L}{6} \end{cases} \quad = \quad 1.40 \text{ ksf} \quad < \quad Q_a \quad \textbf{[Satisfactory]}$$

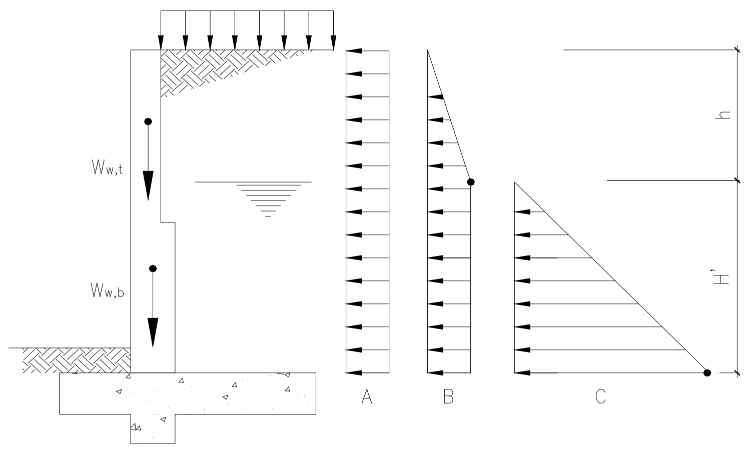
CHECK FLEXURE CAPACITY, AS,1 & AS,2, FOR STEM (ACI 318-05 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

h = 6 ft , H' = 0 ft

A = $w_s P_a / \gamma_b$ = 11 plf

B = h P_a = 210 plf

C = [P_a (γ_{sat} - γ_w) / γ_b + γ_w] H' = 0 plf



At base of top stem

M_u =	1.82	ft-kips
V_u =	0.95	kips
P_u =	0.66	kips

At base of bottom stem

M_u =	2.34	ft-kips
V_u =	1.11	kips
P_u =	0.72	kips

		At top stem		At base of bottom stem	
$\phi M_n = \phi\left[A_s f_y \left(d - \dfrac{A_s f_y - P_u}{1.7 b f'_c}\right)\right]$	=	5.09	ft-kips ,	5.09	ft-kips
		>	M_u	>	M_u
			[Satisfactory]		**[Satisfactory]**
where d	=	4.00	in ,	4.00	in
b	=	12	in ,	12	in
ϕ	=	0.9	(ACI 318 Fig R9.3.2)	0.9	(ACI 318 Fig R9.3.2)
A_s	=	0.31	in^2 ,	0.31	in^2
ρ	=	0.006		0.006	
$\rho_{MAX} = \dfrac{0.85\beta_1 f'_c}{f_y}\dfrac{\varepsilon_u}{\varepsilon_u+\varepsilon_t}$	=	0.013		0.013	
		>	ρ	>	ρ
			[Satisfactory]		**[Satisfactory]**
$\rho_{MIN} = 0.0018\dfrac{t}{d}$	=	0.004		0.004	
		<	ρ	<	ρ
			[Satisfactory]		**[Satisfactory]**

CHECK SHEAR CAPACITY FOR STEM (ACI 318-05 SEC.15.5.2, 11.1.3.1, & 11.3)

		At top stem		At base of bottom stem	
$V_{allowable} = 2\phi b d\sqrt{f'_c}$	=	3.60	kips ,	3.60	kips
		>	V_u	>	V_u
			[Satisfactory]		**[Satisfactory]**

where ϕ = 0.75 (ACI 318-05, Section 9.3.2.3)

CHECK HEEL FLEXURE CAPACITY, AS,3, FOR FOOTING (ACI 318-05 SEC.15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, & 12.5)

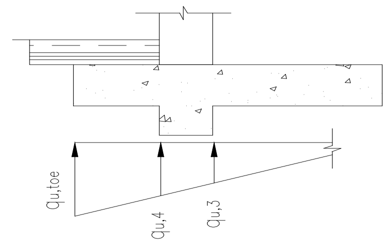
$$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t} \quad = \quad 0.013$$

$$\rho_{MIN} = \frac{0.0018}{2}\frac{h_f}{d} \quad = \quad 0.001$$

$$M_{u,3} = \begin{cases} \dfrac{L_H}{2}\left(\gamma H_s + w_s + w_b - \dfrac{L_H}{L}w_f\right) - \dfrac{\left(q_{u,3}+2q_{u,heel}\right)bL_H^2}{6}, & for \quad e_u \le \dfrac{L}{6} \\ \dfrac{L_H}{2}\left(\gamma H_s + w_s + w_b - \dfrac{L_H}{L}w_f\right) - \dfrac{q_{u,3}bS^2}{6}, & for \quad e_u > \dfrac{L}{6} \end{cases} \quad = \quad 1.63 \text{ ft-kips}$$

(cont'd)

$$\rho = \frac{0.85 f'_c \left(1 - \sqrt{1 - \frac{M_{u,3}}{0.383 b d^2 f'_c}}\right)}{f_y} \quad = \quad 0.000$$



where	d	=	10.19	in	$q_{u, toe}$	=	2.31	ksf
	e_u	=	0.91	ft	$q_{u, heel}$	=	n/a	ksf
	S	=	0.84	ft	$q_{u, 3}$	=	0.78	ksf

$(A_{S, 3})_{required}$ = 0.13 in^2 / ft **<** $A_{S, 3}$ **[Satisfactory]**

CHECK TOE FLEXURE CAPACITY, AS,4, FOR FOOTING (ACI 318-05 SEC.15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon u}{\varepsilon u + \varepsilon t} \quad = \quad 0.013$$

$$\rho_{MIN} = MIN\left(\frac{4}{3}\rho, \quad \frac{0.0018}{2}\frac{h_f}{d}\right) \quad = \quad 0.000$$

$$M_{u,4} = \frac{\left(q_{u,4} + 2q_{u,toe}\right) b L_T^2}{6} \quad \frac{L_T^2}{2L}\gamma w_f \quad = \quad 0.91 \text{ ft-kips}$$

where	d	=	8.69	in
	$q_{u, 4}$	=	1.39	ksf

$$\rho = \frac{0.85 f'_c \left(1 - \sqrt{1 - \frac{M_{u,4}}{0.383 b d^2 f'_c}}\right)}{f_y} \quad = \quad 0.000$$

$(A_{S, 4})_{required}$ = 0.03 in^2 / ft **<** $A_{S, 4}$ **[Satisfactory]**

CHECK SLIDING CAPACITY (IBC 1806.1)

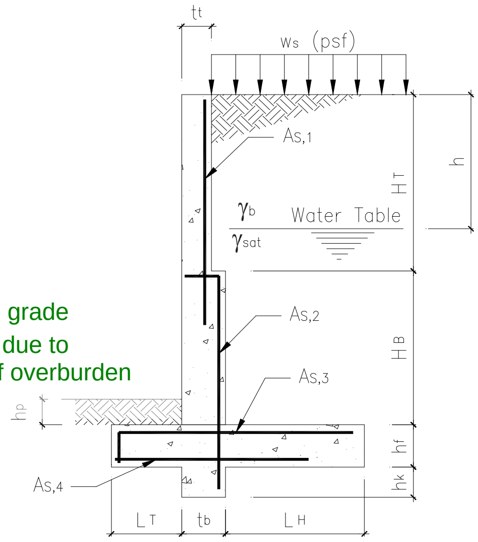
1.5 ($H_b + H_s$) = 1.4 kips **<** $H_p + \mu\, \Sigma W$ = 1.41 kips
[Satisfactory]

Technical References:

1. Alan Williams: "Structural Engineering Reference Manual", Professional Publications, Inc, 2001.
2. Alan Williams: "Structural Engineering License Review Problems and Solutions", Oxford University Press, 2003.

Retaining Wall Design Based on ACI 318 — 8ft Retaining Wall

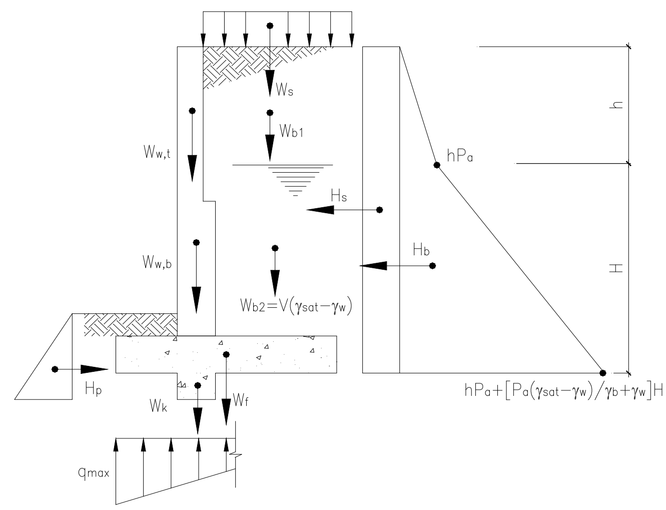
INPUT DATA & DESIGN SUMMARY



CONCRETE STRENGTH	f_c'	=	2.5	ksi		
REBAR YIELD STRESS	f_y	=	60	ksi		
LATERAL SOIL PRESSURE	$P_a = k_a\,\gamma_b$	=	35	pcf (equivalent fluid pressure)		
BACKFILL SPECIFIC WEIGHT	γ_b	=	110	pcf		
SATURATED SPECIFIC WEIGHT	γ_{sat}	=	118	pcf		
WATER TABLE DEPTH	h	=	10	ft		
PASSIVE PRESSURE	P_p	=	400	psf / ft		
SURCHARGE WEIGHT	w_s	=	40	psf		
FRICTION COEFFICIENT	μ	=	0.6	no sliding due to conc slab on grade		
ALLOW SOIL PRESSURE	Q_a	=	1.6	ksf bearing capacity increase due to excavation and removal of overburden existing soil		
THICKNESS OF TOP STEM	t_t	=	8	in		
THICKNESS OF KEY & STEM	t_b	=	8	in		
TOE WIDTH	L_T	=	1.75	ft		
HEEL WIDTH	L_H	=	2.08	ft		
HEIGHT OF TOP STEM	H_T	=	7.5	ft		
HEIGHT OF BOT. STEM	H_B	=	0.5	ft		
FOOTING THICKNESS	h_f	=	12	in		
KEY DEPTH	h_k	=	0	in		
SOIL OVER TOE	h_p	=	6	in		
TOP STEM REINF. ($A_{s,1}$)		#	5	@	10	in o.c.
$A_{s,1}$ LOCATION (0=at soil face, 1=at middle, 2=at each face)					1	at middle
BOT. STEM REINF. ($A_{s,2}$)		#	5	@	10	in o.c.
$A_{s,2}$ LOCATION (0=at soil face, 1=at middle, 2=at each face)					1	at middle
TOP REINF.OF FOOTING ($A_{s,3}$)		#	5	@	12	in o.c.
BOT. REINF.OF FOOTING ($A_{s,4}$)		#	5	@	12	in

[THE WALL DESIGN IS ADEQUATE.]

ANALYSIS



SERVICE LOADS

$H_b = 0.5\,P_a\,h^2 + h\,P_a\,H + 0.5\,[P_a\,(\gamma_{sat} - \gamma_w) / \gamma_b + \gamma_w]\,H^2$ = 1.42 kips

Where h = 9 ft , H = 0 ft

$H_s = w_s\,P_a\,(H_T + H_B + h_f) / \gamma_b$ = 0.11 kips

$H_p = 0.5\,P_p\,(h_p + h_f + h_k)^2$ = 0.45 kips

$W_s = w_s\,(L_H + t_b - t_t)$ = 0.08 kips

$W_b = W_{b1} + W_{b2}$ = 1.83 kips

Where W_{b1} = 1.83 kips , W_{b2} = 0.00 kips

$W_f = h_f\,(L_H + t_b + L_T)\,\gamma_c$ = 0.67 kips

$W_k = h_k\;t_b\;\gamma_c$ = 0.00 kips

$W_{w,t} = t_t\;H_T\;\gamma_c$ = 0.75 kips

$W_{w,b} = t_b\;H_B\;\gamma_c$ = 0.05 kips

FACTORED LOADS

$\gamma H_b = 1.6\,H_b$ = 2.27 kips

$\gamma H_s = 1.6\,H_s$ = 0.18 kips

$\gamma W_s = 1.6\,W_s$ = 0.13 kips

$\gamma W_b = 1.2\,W_b$ = 2.20 kips

$\gamma W_f = 1.2\,W_f$ = 0.81 kips

$\gamma W_k = 1.2\,W_k$ = 0.00 kips

$\gamma W_{w,t} = 1.2\,W_{w,t}$ = 0.90 kips

$\gamma W_{w,b} = 1.2\,W_{w,b}$ = 0.06 kips

OVERTURNING MOMENT

	H	γH	y	H y	γH y
H_b	1.42	2.27	3.00	4.25	6.80
H_s	0.11	0.18	4.50	0.52	0.82
Σ	1.53	2.45		**4.77**	7.63

RESISTING MOMENT

	W	γW	x	W x	γW x
W_s	0.08	0.13	3.46	0.29	0.46
W_b	1.83	2.20	3.46	6.33	7.59
W_f	0.67	0.81	2.25	1.52	1.82
W_k	0.00	0.00	2.08	0.00	0.00
$W_{w,t}$	0.75	0.90	2.08	1.56	1.88
$W_{w,b}$	0.05	0.06	2.08	0.10	0.13
Σ	3.39	4.10		**9.80**	11.87

OVERTURNING FACTOR OF SAFETY (1806.1)

$$SF = \frac{\Sigma Wx}{\Sigma Hy}$$ 2.05 > 1.5

[Satisfactory]

(cont'd)

CHECK SOIL BEARING CAPACITY (ACI 318-05 SEC.15.2.2)

$L = L_T + t_b + L_H$ = 4.50 ft

$e = \frac{L}{2} - \frac{\Sigma W x - H y}{\Sigma W}$ = 0.76 ft

$$q_{MAX} = \begin{cases} \frac{\Sigma W\left(1+\frac{6e}{L}\right)}{BL}, & for \quad e \le \frac{L}{6} \\ \frac{2\Sigma W}{3B(0.5L-e)}, & for \quad e > \frac{L}{6} \end{cases}$$

= 1.52 ksf < Q_a **[Satisfactory]**

CHECK FLEXURE CAPACITY, AS,1 & AS,2, FOR STEM (ACI 318-05 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

h = 8 ft , H' = 0 ft

A = $w_s P_a / \gamma_b$ = 13 plf

B = $h P_a$ = 280 plf

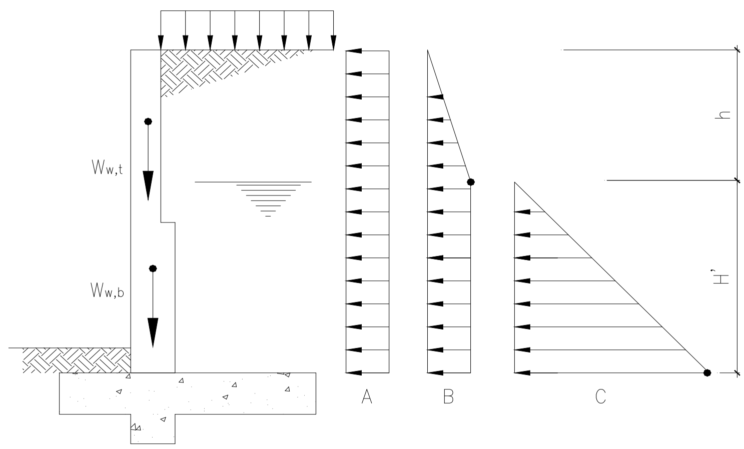
C = $[P_a (\gamma_{sat} - \gamma_w) / \gamma_b + \gamma_w]$ H' = 0 plf

At base of top stem

M_u =	4.51	ft-kips
V_u =	1.73	kips
P_u =	0.90	kips

At base of bottom stem

M_u =	5.43	ft-kips
V_u =	1.95	kips
P_u =	0.96	kips



$$\phi M_n = \phi\left[A_s f_y \left(d - \frac{A_s f_y - P_u}{1.7 b f'_c}\right)\right]$$

		At top stem		At base of bottom stem	
ϕM_n	=	5.99	ft-kips ,	5.99	ft-kips
		>	M_u	>	M_u
			[Satisfactory]		**[Satisfactory]**
where d	=	4.00	in ,	4.00	in
b	=	12	in ,	12	in
ϕ	=	0.9	(ACI 318 Fig R9.3.2)	0.9	(ACI 318 Fig R9.3.2)
A_s	=	0.372	in^2 ,	0.372	in^2
ρ	=	0.008		0.008	
$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t}$	=	0.013		0.013	
		>	ρ	>	ρ
			[Satisfactory]		**[Satisfactory]**
$\rho_{MIN} = 0.0018\frac{t}{d}$	=	0.004		0.004	
		<	ρ	<	ρ
			[Satisfactory]		**[Satisfactory]**

CHECK SHEAR CAPACITY FOR STEM (ACI 318-05 SEC.15.5.2, 11.1.3.1, & 11.3)

		At top stem		At base of bottom stem	
$V_{allowable} = 2\phi b d\sqrt{f'_c}$	=	3.60	kips ,	3.60	kips
		>	V_u	>	V_u
			[Satisfactory]		**[Satisfactory]**

where ϕ = 0.75 (ACI 318-05, Section 9.3.2.3)

CHECK HEEL FLEXURE CAPACITY, AS,3, FOR FOOTING (ACI 318-05 SEC.15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, & 12.5)

$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t}$ = 0.013

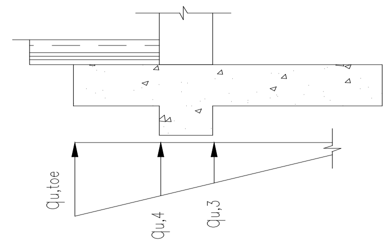
$\rho_{MIN} = \frac{0.0018}{2}\frac{h_f}{d}$ = 0.001

$$M_{u,3} = \begin{cases} \frac{L_H}{2}\left(\gamma w_s + w_b + \frac{L_H}{L} w_f\right) - \frac{\left(q_{u,3}+2q_{u,heel}\right)bL_H^2}{6}, & for \quad e_u \le \frac{L}{6} \\ \frac{L_H}{2}\left(\gamma w_s + w_b + \frac{L_H}{L} w_f\right) - \frac{q_{u,3} b S^2}{6}, & for \quad e_u > \frac{L}{6} \end{cases}$$

= 2.77 ft-kips

(cont'd)

$$\rho = \frac{0.85 f'_c \left(1 - \sqrt{1 - \frac{M_{u,3}}{0.383 b d^2 f'_c}}\right)}{f_y} \quad = \quad 0.000$$



where d	=	10.19	in	$q_{u,\ toe}$	=	2.64	ksf
e_u	=	1.21	ft	$q_{u,\ heel}$	=	n/a	ksf
S	=	0.69	ft	$q_{u,\ 3}$	=	0.59	ksf

$(A_{S,\ 3})_{required}$ = 0.13 in^2 / ft < $A_{S,\ 3}$ **[Satisfactory]**

CHECK TOE FLEXURE CAPACITY, AS,4, FOR FOOTING (ACI 318-05 SEC.15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = \frac{0.85 \beta_1 f'_c}{f_y} \frac{\varepsilon u}{\varepsilon u + \varepsilon t} \quad = \quad 0.013 \qquad \rho_{MIN} = MIN\left(\frac{4}{3}\rho,\ \frac{0.0018}{2}\frac{h_f}{d}\right) \quad = \quad 0.001$$

$$M_{u,4} = \frac{\left(q_{u,4} + 2q_{u,toe}\right) b L_T^2}{6} \quad \frac{L_T^2}{2L} \gamma w_f \quad = \quad 3.01 \text{ ft-kips}$$

where d	=	8.69	in
$q_{u,\ 4}$	=	1.15	ksf

$$\rho = \frac{0.85 f'_c \left(1 - \sqrt{1 - \frac{M_{u,4}}{0.383 b d^2 f'_c}}\right)}{f_y} \quad = \quad 0.001$$

$(A_{S,\ 4})_{required}$ = 0.10 in^2 / ft < $A_{S,\ 4}$ **[Satisfactory]**

CHECK SLIDING CAPACITY (IBC 1806.1)

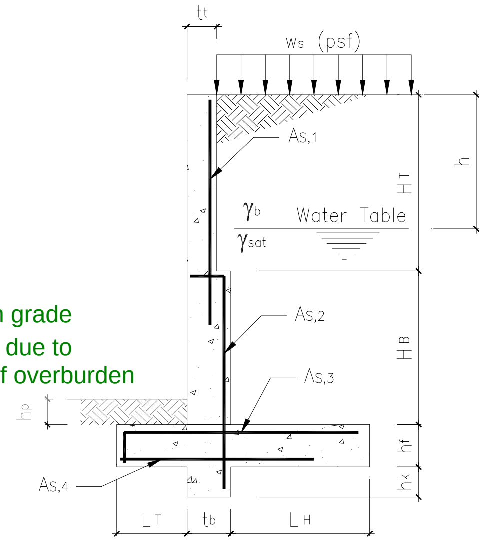
1.5 (H_b + H_s) = 2.3 kips < H_p + μ ΣW = 2.48 kips
[Satisfactory]

Technical References:

1. Alan Williams: "Structural Engineering Reference Manual", Professional Publications, Inc, 2001.
2. Alan Williams: "Structural Engineering License Review Problems and Solutions", Oxford University Press, 2003.

Retaining Wall Design Based on ACI 318 — 10ft Retaining Wall

INPUT DATA & DESIGN SUMMARY

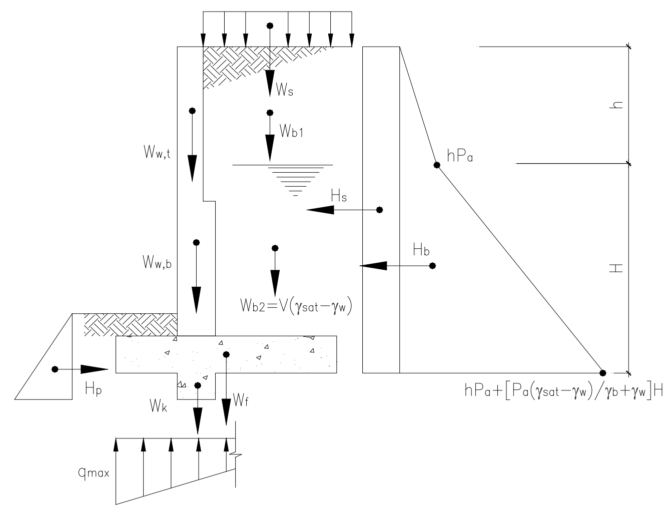


CONCRETE STRENGTH	f_c'	=	2.5	ksi		
REBAR YIELD STRESS	f_y	=	60	ksi		
LATERAL SOIL PRESSURE	$P_a = k_a\,\gamma_b$	=	35	pcf (equivalent fluid pressure)		
BACKFILL SPECIFIC WEIGHT	γ_b	=	110	pcf		
SATURATED SPECIFIC WEIGHT	γ_{sat}	=	118	pcf		
WATER TABLE DEPTH	h	=	20	ft		
PASSIVE PRESSURE	P_p	=	400	psf / ft		
SURCHARGE WEIGHT	w_s	=	40	psf		
FRICTION COEFFICIENT	μ	=	0.7	no sliding due to conc slab on grade		
ALLOW SOIL PRESSURE	Q_a	=	1.7	ksf bearing capacity increase due to excavation and removal of overburden existing soil		
THICKNESS OF TOP STEM	t_t	=	8	in		
THICKNESS OF KEY & STEM	t_b	=	8	in		
TOE WIDTH	L_T	=	2.5	ft		
HEEL WIDTH	L_H	=	2.33	ft		
HEIGHT OF TOP STEM	H_T	=	9.5	ft		
HEIGHT OF BOT. STEM	H_B	=	0.5	ft		
FOOTING THICKNESS	h_f	=	12	in		
KEY DEPTH	h_k	=	0	in		
SOIL OVER TOE	h_p	=	6	in		
TOP STEM REINF. ($A_{s,1}$)		#	6	@	9	in o.c.
$A_{s,1}$ LOCATION (0=at soil face, 1=at middle, 2=at each face)					0	at soil face
BOT. STEM REINF. ($A_{s,2}$)		#	6	@	9	in o.c.
$A_{s,2}$ LOCATION (0=at soil face, 1=at middle, 2=at each face)					0	at soil face
TOP REINF.OF FOOTING ($A_{s,3}$)		#	5	@	12	in o.c.
BOT. REINF.OF FOOTING ($A_{s,4}$)		#	5	@	12	in

[THE WALL DESIGN IS ADEQUATE.]

ANALYSIS

SERVICE LOADS



$H_b = 0.5\,P_a\,h^2 + h\,P_a\,H + 0.5\,[P_a\,(\gamma_{sat} - \gamma_w)\,/\,\gamma_b + \gamma_w]\,H^2$

= 2.12 kips

Where h = 11 ft , H = 0 ft

$H_s = w_s\,P_a\,(H_T + H_B + h_f)\,/\,\gamma_b$ = 0.14 kips

$H_p = 0.5\,P_p\,(h_p + h_f + h_k)^2$ = 0.45 kips

$W_s = w_s\,(L_H + t_b - t_t)$ = 0.09 kips

$W_b = W_{b1} + W_{b2}$ = 2.56 kips

Where W_{b1} = 2.56 kips , W_{b2} = 0.00 kips

$W_f = h_f\,(L_H + t_b + L_T)\,\gamma_c$ = 0.82 kips

$W_k = h_k\,t_b\,\gamma_c$ = 0.00 kips

$W_{w,t} = t_t\,H_T\,\gamma_c$ = 0.95 kips

$W_{w,b} = t_b\,H_B\,\gamma_c$ = 0.05 kips

FACTORED LOADS

$\gamma H_b = 1.6\,H_b$ = 3.39 kips

$\gamma H_s = 1.6\,H_s$ = 0.22 kips

$\gamma W_s = 1.6\,W_s$ = 0.15 kips

$\gamma W_b = 1.2\,W_b$ = 3.08 kips

$\gamma W_f = 1.2\,W_f$ = 0.99 kips

$\gamma W_k = 1.2\,W_k$ = 0.00 kips

$\gamma W_{w,t} = 1.2\,W_{w,t}$ = 1.14 kips

$\gamma W_{w,b} = 1.2\,W_{w,b}$ = 0.06 kips

OVERTURNING MOMENT

	H	γH	y	H y	γH y
H_b	2.12	3.39	3.67	7.76	12.42
H_s	0.14	0.22	5.50	0.77	1.23
Σ	2.26	3.61		**8.53**	13.65

RESISTING MOMENT

	W	γW	x	W x	γW x
W_s	0.09	0.15	4.33	0.40	0.65
W_b	2.56	3.08	4.33	11.10	13.32
W_f	0.82	0.99	2.75	2.27	2.72
W_k	0.00	0.00	2.83	0.00	0.00
$W_{w,t}$	0.95	1.14	2.83	2.69	3.23
$W_{w,b}$	0.05	0.06	2.83	0.14	0.17
Σ	4.48	5.41		**16.61**	20.09

OVERTURNING FACTOR OF SAFETY (1806.1)

$$SF = \frac{\Sigma Wx}{\Sigma Hy}$$ 1.95 > 1.5

[Satisfactory]

(cont'd)

CHECK SOIL BEARING CAPACITY (ACI 318-05 SEC.15.2.2)

$L = L_T + t_b + L_H$ = 5.50 ft

$e = \frac{L}{2} - \frac{\Sigma W x - \Sigma H y}{\Sigma W}$ = 0.95 ft

$$q_{MAX} = \begin{cases} \frac{\Sigma W\left(1+\frac{6e}{L}\right)}{BL}, & for \quad e \le \frac{L}{6} \\ \frac{2\Sigma W}{3B(0.5L-e)}, & for \quad e > \frac{L}{6} \end{cases}$$ = 1.66 ksf < Q_a **[Satisfactory]**

CHECK FLEXURE CAPACITY, AS,1 & AS,2, FOR STEM (ACI 318-05 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

h = 10 ft , H' = 0 ft

A = $w_s P_a / \gamma_b$ = 13 plf

B = h P_a = 350 plf

C = $[P_a (\gamma_{sat} - \gamma_w) / \gamma_b + \gamma_w]$ H' = 0 plf

At base of top stem

M_u = 8.92 ft-kips

V_u = 2.72 kips

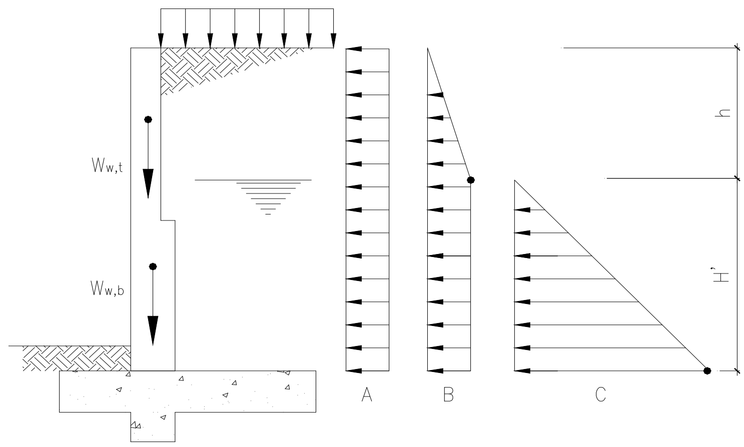
P_u = 1.14 kips

At base of bottom stem

M_u = 10.35 ft-kips

V_u = 3.00 kips

P_u = 1.20 kips



$$\phi M_n = \phi \left[A_s f_y \left(d - \frac{A_s f_y - P_u}{1.7 b f'_c} \right) \right]$$

		At top stem		At base of bottom stem	
ϕM_n	=	13.09	ft-kips ,	13.09	ft-kips
		>	M_u	>	M_u
			[Satisfactory]		**[Satisfactory]**
where d	=	5.63	in ,	5.63	in
b	=	12	in ,	12	in
ϕ	=	0.9	(ACI 318 Fig R9.3.2)	0.9	(ACI 318 Fig R9.3.2)
A_s	=	0.58667	in^2 ,	0.586667	in^2
ρ	=	0.009		0.009	
$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y} \frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t}$	=	0.013		0.013	
		>	ρ	>	ρ
			[Satisfactory]		**[Satisfactory]**
$\rho_{MIN} = 0.0018 \frac{t}{d}$	=	0.003		0.003	
		<	ρ	<	ρ
			[Satisfactory]		**[Satisfactory]**

CHECK SHEAR CAPACITY FOR STEM (ACI 318-05 SEC.15.5.2, 11.1.3.1, & 11.3)

		At top stem		At base of bottom stem	
$V_{allowable} = 2\phi b d \sqrt{f'_c}$	=	5.06	kips ,	5.06	kips
		>	V_u	>	V_u
			[Satisfactory]		**[Satisfactory]**

where ϕ = 0.75 (ACI 318-05, Section 9.3.2.3)

CHECK HEEL FLEXURE CAPACITY, AS,3, FOR FOOTING (ACI 318-05 SEC.15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, & 12.5)

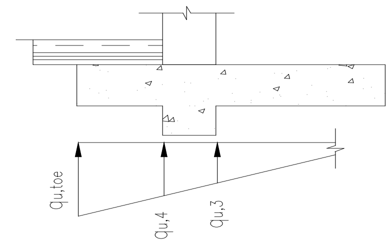
$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y} \frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t}$ = 0.013

$\rho_{MIN} = \frac{0.0018}{2} \frac{h_f}{d}$ = 0.001

$$M_{u,3} = \begin{cases} \frac{L_H}{2}\left(\ldots \gamma_s \ldots w_b \ldots \frac{L_H}{L} w_f\right) - \frac{\left(q_{u,3} + 2q_{u,heel}\right) b L_H^2}{6}, & for \quad e_u \le \frac{L}{6} \\ \frac{L_H}{2}\left(\ldots \gamma_s + \ldots w_b \ldots \frac{L_H}{L} w_f\right) - \frac{q_{u,3} b S^2}{6}, & for \quad e_u > \frac{L}{6} \end{cases}$$ = 4.24 ft-kips

(cont'd)

$$\rho = \frac{0.85 f'_c \left(1 - \sqrt{1 \quad \frac{M_{u,3}}{0.383 b d^2 f'_c}}\right)}{f_y} \quad = \quad 0.001$$



where	d	=	10.19	in	$q_{u, toe}$	=	3.04 ksf
	e_u	=	1.56	ft	$q_{u, heel}$	=	n/a ksf
	S	=	0.40	ft	$q_{u, 3}$	=	0.34 ksf

$(A_{S, 3})_{required}$ = 0.13 in^2 / ft < $A_{S, 3}$ **[Satisfactory]**

CHECK TOE FLEXURE CAPACITY, AS,4, FOR FOOTING (ACI 318-05 SEC.15.4.2, 10.2, 10.3.5, 10.5.4, 7.12.2, 12.2, & 12.5)

$$\rho_{MAX} = \frac{0.85 \beta_1 f'_c}{f_y} \frac{\varepsilon u}{\varepsilon u + \varepsilon t} \quad = \quad 0.013 \qquad \rho_{MIN} = MIN\left(\frac{4}{3}\rho, \quad \frac{0.0018}{2} \frac{h_f}{d}\right) \quad = \quad 0.001$$

$$M_{u,4} = \frac{\left(q_{u,4} + 2 q_{u,toe}\right) b L_T^2}{6} \quad \frac{L_T^2}{2L} \gamma w_f \quad = \quad 6.71 \text{ ft-kips}$$

where	d	=	8.69	in
	$q_{u, 4}$	=	0.91	ksf

$$\rho = \frac{0.85 f'_c \left(1 - \sqrt{1 \quad \frac{M_{u,4}}{0.383 b d^2 f'_c}}\right)}{f_y} \quad = \quad 0.002$$

$(A_{S, 4})_{required}$ = 0.18 in^2 / ft < $A_{S, 4}$ **[Satisfactory]**

CHECK SLIDING CAPACITY (IBC 1806.1)

1.5 (H_b + H_s) = 3.39 kips < H_p + μ ΣW = 3.59 kips
[Satisfactory]

Technical References:

1. Alan Williams: "Structural Engineering Reference Manual", Professional Publications, Inc, 2001.
2. Alan Williams: "Structural Engineering License Review Problems and Solutions", Oxford University Press, 2003.

2720 Residence

2720 71st Avenue SE
Mercer Island, Washington 98040

Structural Engineering Calculations

Supplement Calculations for DADU Lateral System Design

By

Dihong Shao, SE

April 28, 2022

Seismic Mass Calculation

Seismic Base at Main Floor with Concrete Base

Floor areas (sqft)

roof	425			
Roof Framing Seismic Mass (psf)				
roof framing	14.00			
roofing (4.00 psf future PV panels)	6.00			
wall framing to diaphragm	5.00			
total	**25.00** psf			
roof				
seismic mass (area x roof framing seismic mass)	**10.62 kips**			
Seismic Forces				
(per attached spreadsheet calculations)				
roof	1.90 kips			
			NS	***EW***
ASD = Seismic Force/1.4			Cumulative	Cumulative
roof	1.36		1.36 kips	1.36 kips
Wind Forces				
(per attached spreadsheet calculations)				
NS	6.87 kips		1.06	
EW	7.27 kips			
ASD = Wind Force/1.4				
NS	4.91 kips			
EW	5.19 kips			
			NS	***EW***
NS			Cumulative	
roof = ((1'+15.5'/2)/16.5') x 4.91 kips	2.61		**2.61 kips**	
EW				Cumulative
roof = ((1'+15.5'/2)/16.5') x 5.19 kips	2.76			**2.76 kips**
Lateral Force Summary (ASD)			***NS***	***EW***
			Cumulative	Cumulative
		WIND/WIND	**2.61 kips**	**2.76 kips**

INPUT DATA

Typical floor height	h =	**14.5**	ft	
Typical floor weight	w_x =	**11**	k	
Number of floors	n =	**1**		
Importance factor (ASCE 11.5.1)	I =	**1.00**		(IBC Tab.1604.5)
Building location	Zip Code	**98040**		
Site class (A, B, C, D, E, F)		**D**		(If no soil report, use D)
The coefficient (ASCE Tab 12.8-2)	C_t =	**0.02**		
The coefficient(ASCE Tab. 12.2.1)	R =	**6.50**		

DESIGN SUMMARY

Total base shear

V	=	0.17	W, (SD) =	**2**	k, (SD)
	=	0.12	W, (ASD) =	**1**	k, (ASD)

Seismic design category = **D**

Latitude: 47.562605

Longitude: -122.2254

S_{DS} = **1.119** g

h_n = 14.5 ft k = 1.00 , (ASCE 12.8.3, pg 130) x = 0.75 , (ASCE Tab 12.8-2)

W = 11 k $\Sigma w_x h^k$ = 160 $T_a = C_t (h_n)^x$ = 0.15 Sec, (ASCE 12.8.2.1)

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Level Name	Floor to floor Height (ft)	Height h_x (ft)	Weight w_x (k)	$w_x h_x^k$	C_{vx}	Lateral force @ each level: F_x (k)	V_x (k)	O. M. (k-ft)	Diaphragm force: ΣF_i (k)	ΣW_i (k)	F_{px} (k)
1	Roof		14.5	11	160	1.000	1.9			1.9	11	2
		14.50						1.9				
	Ground		0.0						27			

ATC Hazards by Location

Search Information

Address: 2720 71st Ave SE, Mercer Island, WA 98040, USA

Coordinates: 47.5861883, -122.2437783

Elevation: 276 ft

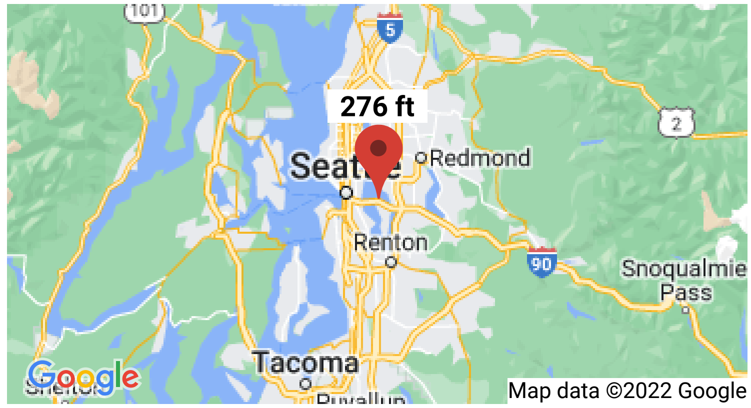
Timestamp: 2022-04-25T07:12:53.539Z

Hazard Type: Seismic

Reference Document: ASCE7-16

Risk Category: II

Site Class: D-default



Basic Parameters

Name	Value	Description
S_S	1.398	MCE_R ground motion (period=0.2s)
S_1	0.487	MCE_R ground motion (period=1.0s)
S_{MS}	1.678	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	1.119	Numeric seismic design value at 0.2s SA
S_{D1}	* null	Numeric seismic design value at 1.0s SA

* See Section 11.4.8

Additional Information

Name	Value	Description
SDC	* null	Seismic design category
F_a	1.2	Site amplification factor at 0.2s
F_v	* null	Site amplification factor at 1.0s
CR_S	0.902	Coefficient of risk (0.2s)
CR_1	0.896	Coefficient of risk (1.0s)
PGA	0.598	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.718	Site modified peak ground acceleration

T_L	6	Long-period transition period (s)
SsRT	1.398	Probabilistic risk-targeted ground motion (0.2s)
SsUH	1.55	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	3.234	Factored deterministic acceleration value (0.2s)
S1RT	0.487	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.543	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	1.319	Factored deterministic acceleration value (1.0s)
PGAd	1.116	Factored deterministic acceleration value (PGA)

* See Section 11.4.8

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Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.

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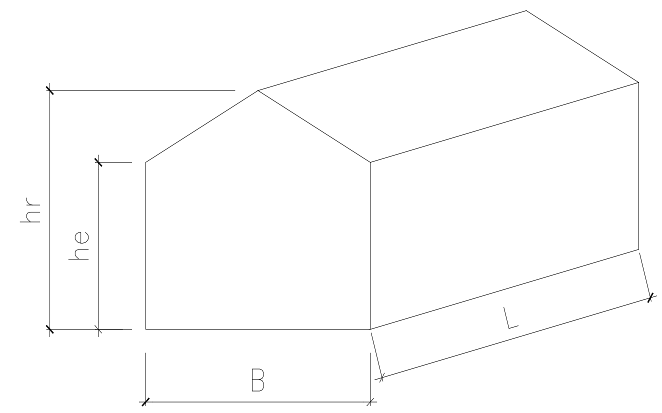
INPUT DATA



Exposure category (B, C or D)		B	
Importance factor, pg 77, (0.87, 1.0 or 1.15)	I =	1.00	Category II
Basic wind speed (IBC Tab 1609.3.1V_{3S})	V =	98	mph
Topographic factor (Sec.6.5.7.2, pg 26 & 45)	K_{zt} =	1.9	
Building height to eave	h_e =	14.5	ft
Building height to ridge	h_r =	16.5	ft
Building length	L =	19.75	ft
Building width	B =	21.5	ft
Effective area of components	A =	10	ft^2

DESIGN SUMMARY

Max horizontal force normal to building length, L, face	=	6.87	kips
Max horizontal force normal to building length, B, face	=	7.27	kips
Max total horizontal torsional load	=	20.34	ft-kips
Max total upward force	=	9.62	kips

ANALYSIS

Velocity pressure

$q_h = 0.00256\ K_h\ K_{zt}\ K_d\ V^2\ I$ = **27.79** psf

where: q_h = velocity pressure at mean roof height, h. (Eq. 6-15, page 27)

K_h = velocity pressure exposure coefficient evaluated at height, h, (Tab. 6-3, Case 1,pg 79) = **0.70**

K_d = wind directionality factor. (Tab. 6-4, for building, page 80) = **0.85**

h = mean roof height = **15.50** ft

< 60 ft, [Satisfactory]

Design pressures for MWFRS

$p = q_h\ [(G\ C_{pf}) - (G\ C_{pi})]$

where: p = pressure in appropriate zone. (Eq. 6-18, page 28).

$G\ C_{p\,f}$ = product of gust effect factor and external pressure coefficient, see table below. (Fig. 6-10, page 53 & 54)

$G\ C_{p\,i}$ = product of gust effect factor and internal pressure coefficient.(Fig. 6-5, Enclosed Building, page 47)

= **0.18** or **-0.18**

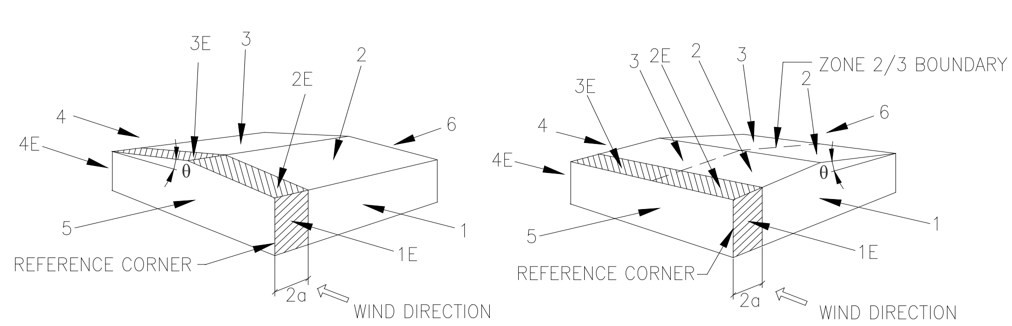
a = width of edge strips, Fig 6-10, note 9, page 54, MAX[MIN(0.1B, 0.4h), 0.04B,3] = **3.00 ft**

Net Pressures (psf), Basic Load Cases

Surface	Roof angle θ = 10.54			Roof angle θ = 0.00		
	$G\ C_{p\,f}$	Net Pressure with		$G\ C_{p\,f}$	Net Pressure with	
		$(+GC_{p\,i})$	$(-GC_{p\,i})$		$(+GC_{p\,i})$	$(-GC_{p\,i})$
1	0.45	7.45	17.46	0.40	6.11	16.12
2	-0.69	-24.18	-14.18	-0.69	-24.18	-14.18
3	-0.41	-16.42	-6.41	-0.37	-15.29	-5.28
4	-0.34	-14.50	-4.49	-0.29	-13.06	-3.06
1E	0.68	13.90	23.91	0.61	11.95	21.96
2E	-1.07	-34.74	-24.74	-1.07	-34.74	-24.74
3E	-0.59	-21.38	-11.37	-0.53	-19.73	-9.73
4E	-0.51	-19.11	-9.10	-0.43	-16.95	-6.95
5	-0.45	-17.51	-7.50	-0.45	-17.51	-7.50
6	-0.45	-17.51	-7.50	-0.45	-17.51	-7.50

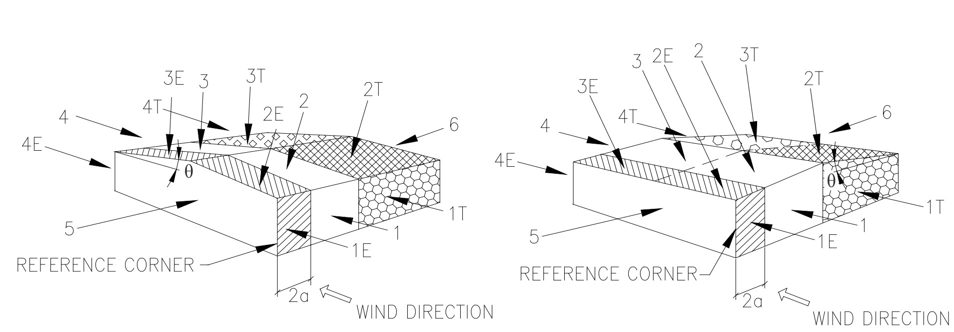
Net Pressures (psf), Torsional Load Cases

Surface	Roof angle θ = 10.54		
	$G\ C_{p\,f}$	Net Pressure with	
		$(+GC_{p\,i})$	$(-GC_{p\,i})$
1T	0.45	1.86	4.36
2T	-0.69	-6.05	-3.54
3T	-0.41	-4.10	-1.60
4T	-0.34	-3.63	-1.12
	Roof angle θ = 0.00		
Surface	$G\ C_{p\,f}$	Net Pressure with	
		$(+GC_{p\,i})$	$(-GC_{p\,i})$
1T	0.40	1.53	4.03
2T	-0.69	-6.05	-3.54
3T	-0.37	-3.82	-1.32
4T	-0.29	-3.27	-0.76



Transverse Direction Longitudinal Direction

Basic Load Cases



Transverse Direction Longitudinal Direction

Torsional Load Cases

Basic Load Cases in Transverse Direction

Surface	Area (ft^2)	Pressure (k) with $(+GC_{pi})$	Pressure (k) with $(-GC_{pi})$
1	199	1.49	3.48
2	150	-3.64	-2.13
3	150	-2.47	-0.96
4	199	-2.89	-0.90
1E	87	1.21	2.08
2E	66	-2.28	-1.62
3E	66	-1.40	-0.75
4E	87	-1.66	-0.79
Σ	Horiz.	6.87	6.87
	Vert.	-9.62	-5.37
10 psf min.	Horiz.	3.26	3.26
Sec. 6.1.4.1	Vert.	-4.25	-4.25

Basic Load Cases in Longitudinal Direction

Surface	Area (ft^2)	Pressure (k) with $(+GC_{pi})$	Pressure (k) with $(-GC_{pi})$
1	243	1.49	3.92
2	156	-3.76	-2.21
3	156	-2.38	-0.82
4	243	-3.17	-0.74
1E	90	1.08	1.98
2E	60	-2.09	-1.49
3E	60	-1.19	-0.59
4E	90	-1.53	-0.63
Σ	Horiz.	7.27	7.27
	Vert.	-9.27	-5.02
10 psf min.	Horiz.	3.33	3.33
Sec. 6.1.4.1	Vert.	-4.25	-4.25

Torsional Load Cases in Transverse Direction

Surface	Area (ft^2)	Pressure (k) with $(+GC_{pi})$	Pressure (k) with $(-GC_{pi})$	Torsion (ft-k) $(+GC_{pi})$	Torsion (ft-k) $(-GC_{pi})$
1	56	0.42	0.98	1	3
2	42	-1.02	-0.60	-1	0
3	42	-0.70	-0.27	0	0
4	56	-0.81	-0.25	3	1
1E	87	1.21	2.08	8	14
2E	66	-2.28	-1.62	-3	-2
3E	66	-1.40	-0.75	2	1
4E	87	-1.66	-0.79	11	5
1T	143	0.27	0.62	-1	-3
2T	108	-0.65	-0.38	1	0
3T	108	-0.44	-0.17	0	0
4T	143	-0.52	-0.16	-3	-1
Total Horiz. Torsional Load, M_T				19	19

Torsional Load Cases in Longitudinal Direction

Surface	Area (ft^2)	Pressure (k) with $(+GC_{pi})$	Pressure (k) with $(-GC_{pi})$	Torsion (ft-k) $(+GC_{pi})$	Torsion (ft-k) $(-GC_{pi})$
1	76	0.47	1.23	1	3
2	95	-2.31	-1.35	2	1
3	95	-1.46	-0.50	-1	0
4	76	-1.00	-0.23	2	1
1E	90	1.08	1.98	8	15
2E	60	-2.09	-1.49	2	1
3E	60	-1.19	-0.59	-1	-1
4E	90	-1.53	-0.63	12	5
1T	167	0.25	0.67	-1	-4
2T	156	-0.94	-0.55	-2	-1
3T	156	-0.60	-0.21	1	0
4T	167	-0.54	-0.13	-3	-1
Total Horiz. Torsional Load, M_T				20.3	20.3

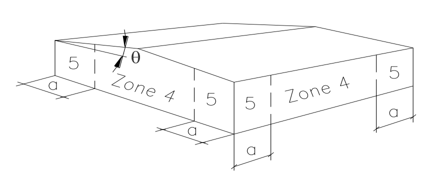
Design pressures for components and cladding

$$p = q_h[(G\,C_p) - (G\,C_{pi})]$$

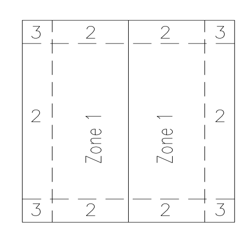
where: p = pressure on component. (Eq. 6-22, pg 28)

p_{min} = 10 psf (Sec. 6.1.4.2, pg 21)

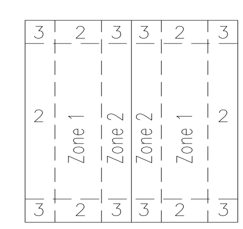
$G\,C_p$ = external pressure coefficient. see table below. (Fig. 6-11, page 55~58)



Walls



Roof θ≤7°



Roof θ>7°

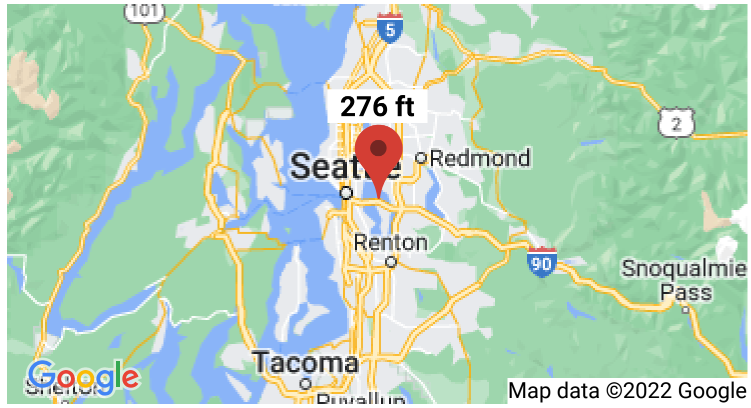
	Effective Area (ft^2)	Zone 1 GC_P	Zone 1 $-GC_P$	Zone 2 GC_P	Zone 2 $-GC_P$	Zone 3 GC_P	Zone 3 $-GC_P$	Zone 4 GC_P	Zone 4 $-GC_P$	Zone 5 GC_P	Zone 5 $-GC_P$
Comp.	10	0.50	-0.90	0.50	-1.70	0.50	-2.60	1.00	-1.10	1.00	-1.40

Comp. & Cladding Pressure (psf)	Zone 1 Positive	Zone 1 Negative	Zone 2 Positive	Zone 2 Negative	Zone 3 Positive	Zone 3 Negative	Zone 4 Positive	Zone 4 Negative	Zone 5 Positive	Zone 5 Negative
	18.90	-30.02	18.90	-52.25	18.90	-77.27	32.80	-35.58	32.80	-43.92

ATC Hazards by Location

Search Information

Address:	2720 71st Ave SE, Mercer Island, WA 98040, USA
Coordinates:	47.5861883, -122.2437783
Elevation:	276 ft
Timestamp:	2022-04-25T07:11:33.708Z
Hazard Type:	Wind



ASCE 7-16

MRI 10-Year	67 mph
MRI 25-Year	73 mph
MRI 50-Year	78 mph
MRI 100-Year	83 mph
Risk Category I	92 mph
Risk Category II	97 mph
Risk Category III	104 mph
Risk Category IV	108 mph

ASCE 7-10

MRI 10-Year	72 mph
MRI 25-Year	79 mph
MRI 50-Year	85 mph
MRI 100-Year	91 mph
Risk Category I	100 mph
Risk Category II	110 mph
Risk Category III-IV	115 mph

ASCE 7-05

ASCE 7-05 Wind Speed	85 mph

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Disclaimer

Hazard loads are interpolated from data provided in ASCE 7 and rounded up to the nearest whole integer. Per ASCE 7, islands and coastal areas outside the last contour should use the last wind speed contour of the coastal area – in some cases, this website will extrapolate past the last wind speed contour and therefore, provide a wind speed that is slightly higher. NOTE: For queries near wind-borne debris region boundaries, the resulting determination is sensitive to rounding which may affect whether or not it is considered to be within a wind-borne debris region.

Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

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DADU Shear Wall Design

shear wall location:	**NORTH**	roof diaphragm
shear force (kips)		1.38
floor height (ft)		16.00
wall length without opening (ft)		7.50
wall length with opening (ft)		3.50
wall segment length (ft)		7.50
shear flow (plf)		394.29
shear wall type per schedule on GN		**SW4**
dead loads from floor/roof framing (plf)		120.00
wall weight (plf)		160.00
hold down force (kips) with 0.6DL		4.68
hold down type per schedule on GN		**5**
shear wall location:	**SOUTH**	roof diaphragm
shear force (kips)		1.38
floor height (ft)		13.50
wall length without opening (ft)		11.00
wall length with opening (ft)		11.00
wall segment length (ft)		11.00
shear flow (plf)		125.45
shear wall type per schedule on GN		**SW6**
dead loads from floor/roof framing (plf)		105.00
wall weight (plf)		135.00
hold down force (kips) with 0.6DL		0.10
hold down type per schedule on GN		**NO HD**
shear wall location:	**EAST**	roof diaphragm
shear force (kips)		1.38
floor height (ft)		14.50
wall length without opening (ft)		8.00
wall length with opening (ft)		8.00
wall segment length (ft)		8.00
shear flow (plf)		172.50
shear wall type per schedule on GN		**SW6**
dead loads from floor/roof framing (plf)		60.00
wall weight (plf)		145.00
hold down force (kips) with 0.6DL		2.01
hold down type per schedule on GN		**2**
shear wall location:	**WEST**	roof diaphragm
shear force (kips)		1.38
floor height (ft)		14.50
wall length without opening (ft)		15.00
wall length with opening (ft)		15.00
wall segment length (ft)		15.00
shear flow (plf)		92.00
shear wall type per schedule on GN		**SW6**
dead loads from floor/roof framing (plf)		60.00
wall weight (plf)		145.00
hold down force (kips) with 0.6DL		-0.09
hold down type per schedule on GN		**NO HD**

2720 Residence

2720 71st Avenue SE
Mercer Island, Washington 98040

Structural Engineering Calculations

Supplement Calculations for HOUSE Lateral System Design

By

Dihong Shao, SE

April 28, 2022

Seismic Mass Calculation

Seismic Base at Main Floor with Concrete Base

Floor areas (sqft)

2nd	2388	
roof	1636	

Roof Framing Seismic Mass (psf)

roof framing	14.00	
roofing (4.00 psf future PV panels)	6.00	
wall framing to diaphragm	5.00	
total	**25.00**	psf

Floor Framing Seismic Mass (psf)

floor framing	15.00	
wall framing to diaphragm	10.00	
total	**25.00**	psf

2nd

seismic mass (area x floor framing seismic mass)	**59.70**	**kips**

roof

seismic mass (area x roof framing seismic mass)	**40.90**	**kips**

Seismic Forces

(per attached spreadsheet calculations)

roof	9.50	kips				
2nd	7.90	kips				
total	17.40	kips				
			NS		***EW***	
ASD = Seismic Force/1.4			Cumulative		Cumulative	
roof	6.79		6.79	kips	6.79	kips
2nd	5.64		12.43	kips	12.43	kips
total	12.43	kips				

Wind Forces

(per attached spreadsheet calculations)

NS	32.53	kips	1.12			
EW	36.29	kips				
ASD = Wind Force/1.4						
NS	23.24	kips				
EW	25.92	kips				
			NS		***EW***	
NS			Cumulative			
roof = ((3'+12'/2)/28') x 23.24 kips	7.47		**7.47**	**kips**		
2nd = (((12'+13')/2)/28') x 23.24 kips	10.38		**17.85**	**kips**		
total	17.85	kips				
EW					Cumulative	
roof = ((3'+12'/2)/28') x 25.92 kips	8.33				**8.33**	**kips**
2nd = (((12'+13')/2)/28') x 25.92 kips	11.58				**19.91**	**kips**
total	19.91	kips				

Lateral Force Summary (ASD)

	NS		***EW***	
	Cumulative		Cumulative	
WIND/WIND	**7.47**	**kips**	**8.33**	**kips**
WIND/WIND	**17.85**	**kips**	**19.91**	**kips**

INPUT DATA

Typical floor height	h =	**10.0**	ft	
Typical floor weight	w_x =	**60**	k	
Number of floors	n =	**2**		
Importance factor (ASCE 11.5.1)	I =	**1.00**	(IBC Tab.1604.5)	
Building location	Zip Code	**98040**		
Site class (A, B, C, D, E, F)		**D**	(If no soil report, use D)	
The coefficient (ASCE Tab 12.8-2)	C_t =	**0.02**		
The coefficient(ASCE Tab. 12.2.1)	R =	**6.50**		

DESIGN SUMMARY

Total base shear

V	=	0.17	W, (SD) =	**17**	k, (SD)
	=	0.12	W, (ASD) =	**12**	k, (ASD)

Seismic design category = **D**

Latitude: 47.562605

Longitude: -122.2254

S_{DS} = **1.119** g

h_n = 23.0 ft   k = 1.00 , (ASCE 12.8.3, pg 130)   x = 0.75 , (ASCE Tab 12.8-2)

W = 101 k $\Sigma w_x h^k$ = 1,723 $T_a = C_t (h_n)^x$ = 0.21 Sec, (ASCE 12.8.2.1)

VERTICAL DISTRIBUTION OF LATERAL FORCES

Level No.	Level Name	Floor to floor Height ft	Height h_x ft	Weight w_x k	$w_x h_x^k$	C_{vx}	Lateral force @ each level F_x k	V_x k	O. M. k-ft	Diaphragm force ΣF_i k	ΣW_i k	F_{px} k
2	Roof		23.0	41	943	0.547	9.5			9.5	41	10
		10.00						9.5				
1	2nd		13.0	60	780	0.453	7.9		95	17.4	101	13
		13.00						17.4				
	Ground		0.0						321			

ATC Hazards by Location

Search Information

Address: 2720 71st Ave SE, Mercer Island, WA 98040, USA

Coordinates: 47.5861883, -122.2437783

Elevation: 276 ft

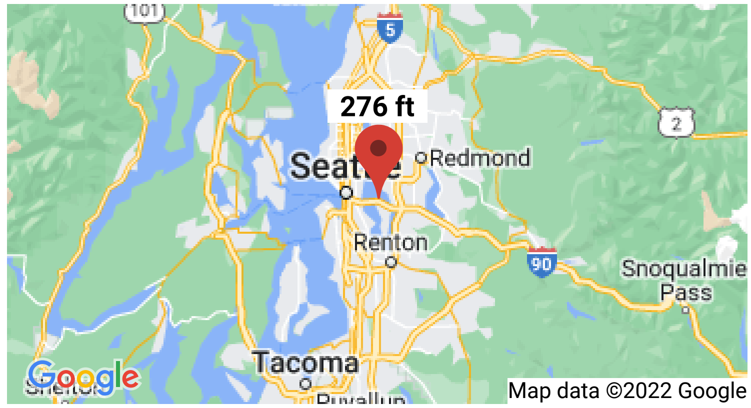
Timestamp: 2022-04-25T07:12:53.539Z

Hazard Type: Seismic

Reference Document: ASCE7-16

Risk Category: II

Site Class: D-default



Basic Parameters

Name	Value	Description
S_S	1.398	MCE_R ground motion (period=0.2s)
S_1	0.487	MCE_R ground motion (period=1.0s)
S_{MS}	1.678	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	1.119	Numeric seismic design value at 0.2s SA
S_{D1}	* null	Numeric seismic design value at 1.0s SA

* See Section 11.4.8

Additional Information

Name	Value	Description
SDC	* null	Seismic design category
F_a	1.2	Site amplification factor at 0.2s
F_v	* null	Site amplification factor at 1.0s
CR_S	0.902	Coefficient of risk (0.2s)
CR_1	0.896	Coefficient of risk (1.0s)
PGA	0.598	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.718	Site modified peak ground acceleration

T_L	6	Long-period transition period (s)
SsRT	1.398	Probabilistic risk-targeted ground motion (0.2s)
SsUH	1.55	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	3.234	Factored deterministic acceleration value (0.2s)
S1RT	0.487	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.543	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	1.319	Factored deterministic acceleration value (1.0s)
PGAd	1.116	Factored deterministic acceleration value (PGA)

* See Section 11.4.8

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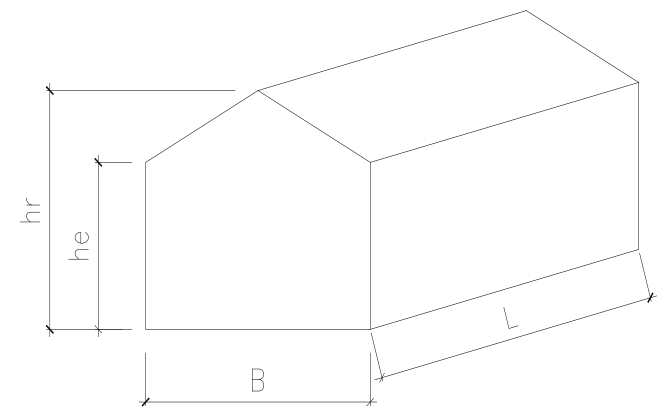
INPUT DATA



Exposure category (B, C or D)		B	
Importance factor, pg 77, (0.87, 1.0 or 1.15)	I =	1.00	Category II
Basic wind speed (IBC Tab 1609.3.1V_{3S})	V =	98	mph
Topographic factor (Sec.6.5.7.2, pg 26 & 45)	K_{zt} =	1.9	
Building height to eave	h_e =	28	ft
Building height to ridge	h_r =	28	ft
Building length	L =	62	ft
Building width	B =	55	ft
Effective area of components	A =	10	ft^2

DESIGN SUMMARY

Max horizontal force normal to building length, L, face	=	36.29 kips
Max horizontal force normal to building length, B, face	=	32.53 kips
Max total horizontal torsional load	=	299.46 ft-kips
Max total upward force	=	72.41 kips

ANALYSIS

Velocity pressure

$q_h = 0.00256\, K_h\, K_{zt}\, K_d\, V^2\, I$ = **27.79** psf

where: q_h = velocity pressure at mean roof height, h. (Eq. 6-15, page 27)

K_h = velocity pressure exposure coefficient evaluated at height, h, (Tab. 6-3, Case 1,pg 79) = **0.70**

K_d = wind directionality factor. (Tab. 6-4, for building, page 80) = **0.85**

h = mean roof height = **28.00** ft

< 60 ft, [Satisfactory]

Design pressures for MWFRS

$p = q_h\,[(G\,C_{pf}) - (G\,C_{pi})]$

where: p = pressure in appropriate zone. (Eq. 6-18, page 28).

$G\,C_{p\,f}$ = product of gust effect factor and external pressure coefficient, see table below. (Fig. 6-10, page 53 & 54)

$G\,C_{p\,i}$ = product of gust effect factor and internal pressure coefficient.(Fig. 6-5, Enclosed Building, page 47)

= **0.18** or **-0.18**

a = width of edge strips, Fig 6-10, note 9, page 54, MAX[MIN(0.1B, 0.4h), 0.04B,3] = **5.50 ft**

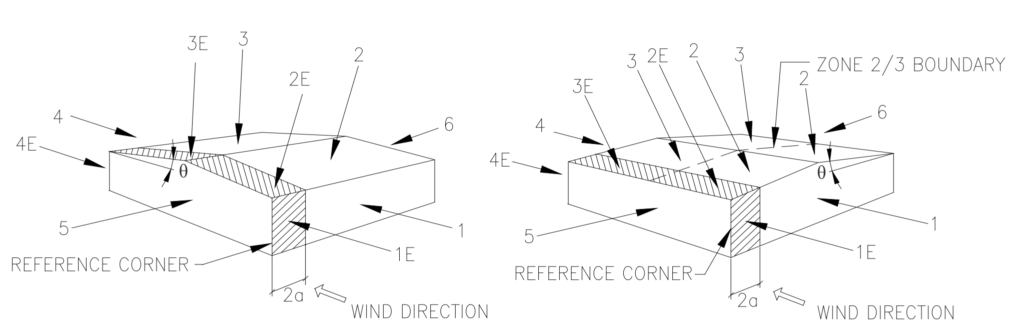
Net Pressures (psf), Basic Load Cases

Surface	Roof angle θ = 0.00			Roof angle θ = 0.00		
	$G\,C_{p\,f}$	Net Pressure with		$G\,C_{p\,f}$	Net Pressure with	
		$(+GC_{p\,i})$	$(-GC_{p\,i})$		$(+GC_{p\,i})$	$(-GC_{p\,i})$
1	0.40	6.11	16.12	0.40	6.11	16.12
2	-0.69	-24.18	-14.18	-0.69	-24.18	-14.18
3	-0.37	-15.29	-5.28	-0.37	-15.29	-5.28
4	-0.29	-13.06	-3.06	-0.29	-13.06	-3.06
1E	0.61	11.95	21.96	0.61	11.95	21.96
2E	-1.07	-34.74	-24.74	-1.07	-34.74	-24.74
3E	-0.53	-19.73	-9.73	-0.53	-19.73	-9.73
4E	-0.43	-16.95	-6.95	-0.43	-16.95	-6.95
5	-0.45	-17.51	-7.50	-0.45	-17.51	-7.50
6	-0.45	-17.51	-7.50	-0.45	-17.51	-7.50

Net Pressures (psf), Torsional Load Cases

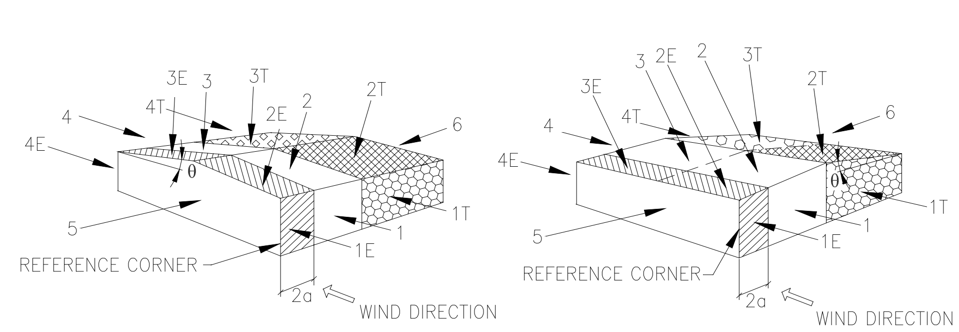
Surface	Roof angle θ = 0.00		
	$G\,C_{p\,f}$	Net Pressure with	
		$(+GC_{p\,i})$	$(-GC_{p\,i})$
1T	0.40	1.53	4.03
2T	-0.69	-6.05	-3.54
3T	-0.37	-3.82	-1.32
4T	-0.29	-3.27	-0.76

Surface	Roof angle θ = 0.00		
	$G\,C_{p\,f}$	Net Pressure with	
		$(+GC_{p\,i})$	$(-GC_{p\,i})$
1T	0.40	1.53	4.03
2T	-0.69	-6.05	-3.54
3T	-0.37	-3.82	-1.32
4T	-0.29	-3.27	-0.76



Transverse Direction   Longitudinal Direction

Basic Load Cases



Transverse Direction   Longitudinal Direction

Torsional Load Cases

Basic Load Cases in Transverse Direction

Surface	Area (ft^2)	Pressure (k) with (+GC_{pi})	Pressure (k) with (-GC_{pi})
1	1428	8.73	23.02
2	1403	-33.91	-19.88
3	1403	-21.44	-7.41
4	1428	-18.65	-4.37
1E	308	3.68	6.76
2E	303	-10.51	-7.48
3E	303	-5.97	-2.94
4E	308	-5.22	-2.14
Σ	Horiz.	36.29	36.29
	Vert.	-71.83	-37.71
10 psf min.	Horiz.	17.36	17.36
Sec. 6.1.4.1	Vert.	-34.10	-34.10

Basic Load Cases in Longitudinal Direction

Surface	Area (ft^2)	Pressure (k) with (+GC_{pi})	Pressure (k) with (-GC_{pi})
1	1232	7.53	19.86
2	1364	-32.98	-19.34
3	1364	-20.85	-7.20
4	1232	-16.09	-3.77
1E	308	3.68	6.76
2E	341	-11.85	-8.44
3E	341	-6.73	-3.32
4E	308	-5.22	-2.14
Σ	Horiz.	32.53	32.53
	Vert.	-72.41	-38.29
10 psf min.	Horiz.	15.40	15.40
Sec. 6.1.4.1	Vert.	-34.10	-34.10

Torsional Load Cases in Transverse Direction

Surface	Area (ft^2)	Pressure (k) with (+GC_{pi})	Pressure (k) with (-GC_{pi})	Torsion (ft-k) (+GC_{pi})	Torsion (ft-k) (-GC_{pi})
1	560	3.42	9.03	44	115
2	550	-13.30	-7.80	0	0
3	550	-8.41	-2.90	0	0
4	560	-7.32	-1.71	93	22
1E	308	3.68	6.76	94	172
2E	303	-10.51	-7.48	0	0
3E	303	-5.97	-2.94	0	0
4E	308	-5.22	-2.14	133	55
1T	868	1.33	3.50	-21	-54
2T	853	-5.15	-3.02	0	0
3T	853	-3.26	-1.13	0	0
4T	868	-2.83	-0.66	-44	-10
Total Horiz. Torsional Load, M_T				299	299

Torsional Load Cases in Longitudinal Direction

Surface	Area (ft^2)	Pressure (k) with (+GC_{pi})	Pressure (k) with (-GC_{pi})	Torsion (ft-k) (+GC_{pi})	Torsion (ft-k) (-GC_{pi})
1	462	2.83	7.45	23	61
2	1023	-24.74	-14.50	0	0
3	1023	-15.64	-5.40	0	0
4	462	-6.04	-1.41	50	12
1E	308	3.68	6.76	81	149
2E	341	-11.85	-8.44	0	0
3E	341	-6.73	-3.32	0	0
4E	308	-5.22	-2.14	115	47
1T	770	1.18	3.10	-16	-43
2T	1364	-8.25	-4.83	0	0
3T	1364	-5.21	-1.80	0	0
4T	770	-2.51	-0.59	-35	-8
Total Horiz. Torsional Load, M_T				218.2	218.2

<u>Design pressures for components and cladding</u>

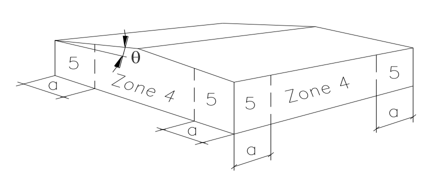
$p = q_h[(G C_p) - (G C_{pi})]$

where: p = pressure on component. (Eq. 6-22, pg 28)

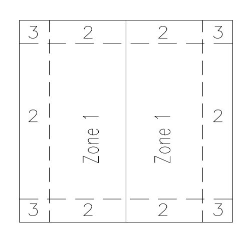
p_{min} = 10 psf (Sec. 6.1.4.2, pg 21)

G C_p = external pressure coefficient.

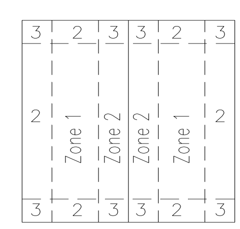
see table below. (Fig. 6-11, page 55~58)



Walls



Roof θ≤7°



Roof θ>7°

	Effective Area (ft^2)	Zone 1 GC_P	Zone 1 - GC_P	Zone 2 GC_P	Zone 2 - GC_P	Zone 3 GC_P	Zone 3 - GC_P	Zone 4 GC_P	Zone 4 - GC_P	Zone 5 GC_P	Zone 5 - GC_P
Comp.	10	0.30	-1.00	0.30	-1.80	0.30	-2.80	0.90	-0.99	0.90	-1.26

(Walls reduced 10 %, Fig. 6-11A note 5.)

Comp. & Cladding Pressure (psf)	Zone 1 Positive	Zone 1 Negative	Zone 2 Positive	Zone 2 Negative	Zone 3 Positive	Zone 3 Negative	Zone 4 Positive	Zone 4 Negative	Zone 5 Positive	Zone 5 Negative
	13.34	-32.80	13.34	-55.03	13.34	-82.83	30.02	-32.52	30.02	-40.02

Hazards by Location

Search Information

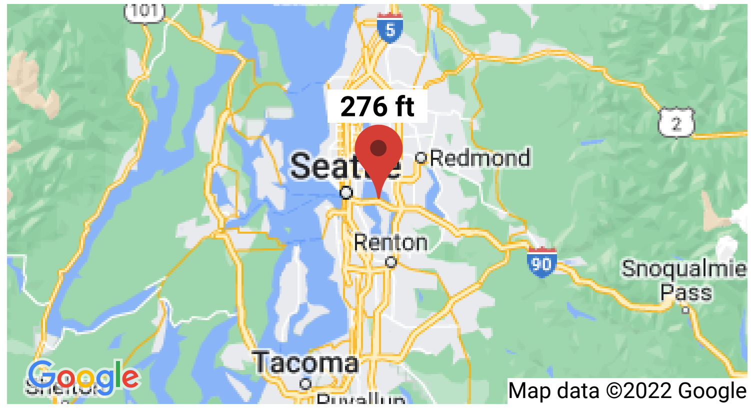
Address: 2720 71st Ave SE, Mercer Island, WA 98040, USA

Coordinates: 47.5861883, -122.2437783

Elevation: 276 ft

Timestamp: 2022-04-25T07:11:33.708Z

Hazard Type: Wind



ASCE 7-16

MRI 10-Year	67 mph
MRI 25-Year	73 mph
MRI 50-Year	78 mph
MRI 100-Year	83 mph
Risk Category I	92 mph
Risk Category II	97 mph
Risk Category III	104 mph
Risk Category IV	108 mph

ASCE 7-10

MRI 10-Year	72 mph
MRI 25-Year	79 mph
MRI 50-Year	85 mph
MRI 100-Year	91 mph
Risk Category I	100 mph
Risk Category II	110 mph
Risk Category III-IV	115 mph

ASCE 7-05

ASCE 7-05 Wind Speed	85 mph

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are interpolated from data provided in ASCE 7 and rounded up to the nearest whole integer. Per ASCE 7, islands and coastal areas outside the last contour should use the last wind speed contour of the coastal area – in some cases, this website will extrapolate past the last wind speed contour and therefore, provide a wind speed that is slightly higher. NOTE: For queries near wind-borne debris region boundaries, the resulting determination is sensitive to rounding which may affect whether or not it is considered to be within a wind-borne debris region.

Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

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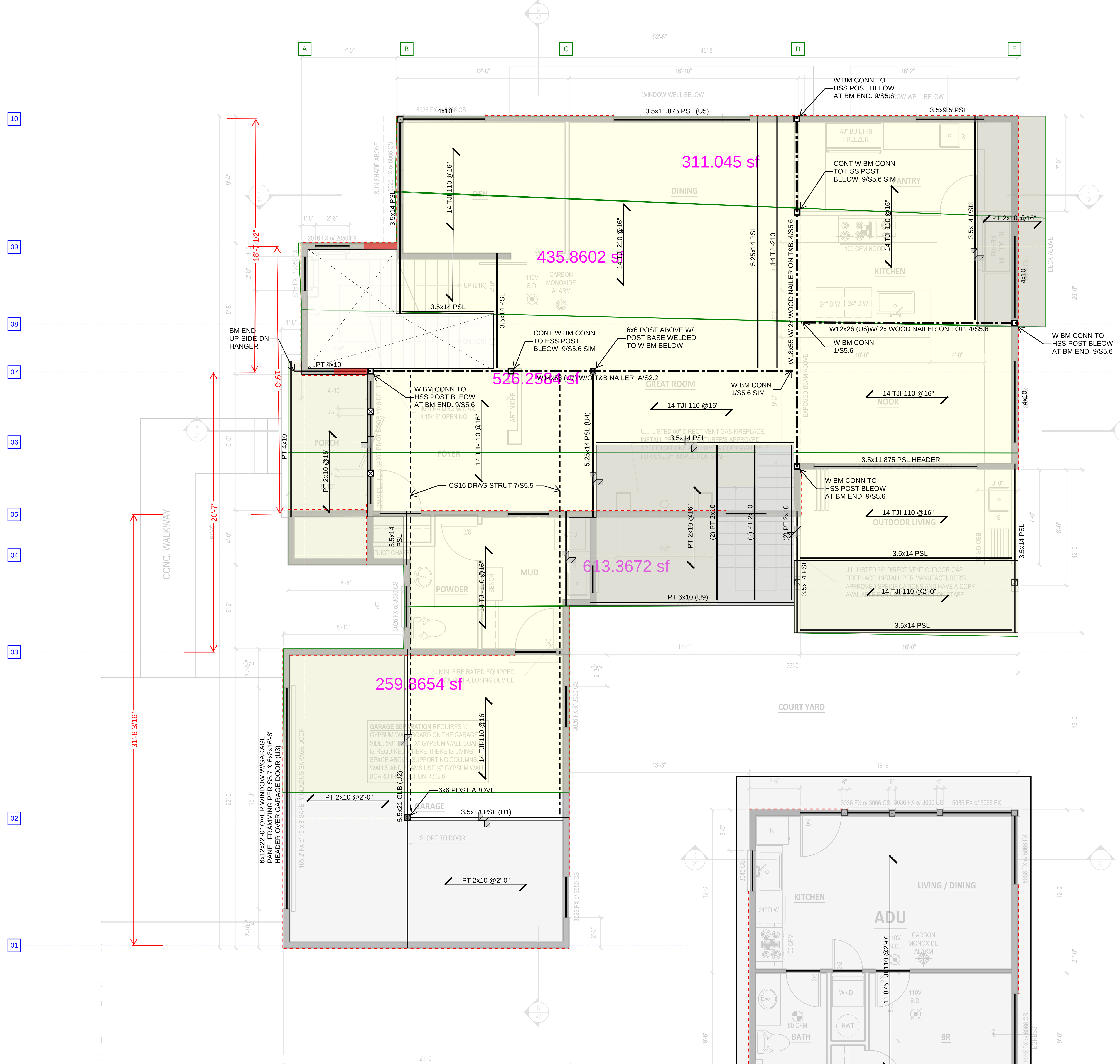


EW SHEAR WALL PLAN - UPPER FLOOR

1/4"=1'-0"

DHS ENGINEERS
1201 3RD AVE, 2200
SEATTLE, WA 98101
(206) 734-5858

DIHONG SHAO, STATE OF WASHINGTON, STRUCTURAL ENGINEER, 32032

2720 RESIDENCE

2720 71ST AVENUE SE
MERCER ISLAND WA 98040

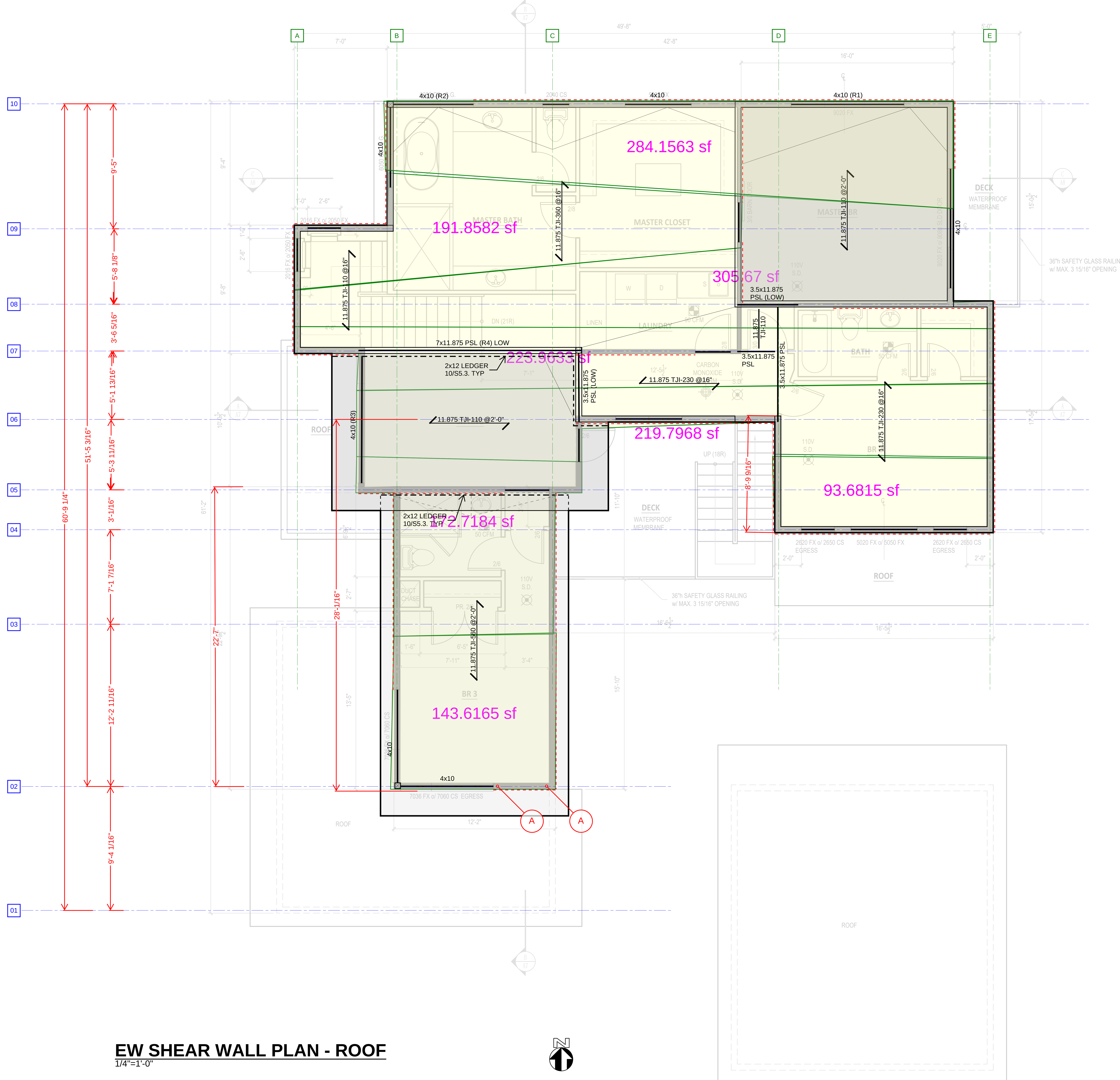
NUMBER	DATE	DESCRIPTION OF REVISIONS

SHEET TITLE
UPPER FLOOR & LOWER ROOF FRAMING PLAN

JOB NUMBER

SHEET NUMBER
S2.2

CITY STAMP



EW SHEAR WALL PLAN - ROOF

1/4"=1'-0"

DHS ENGINEERS

1201 3RD AVE, 2200
SEATTLE, WA 98101
(206) 734-5858

DIHONG SHAO
STATE OF WASHINGTON
32032
STRUCTURAL ENGINEER
PROFESSIONAL ENGINEER

2720 RESIDENCE

2720 71ST AVENUE SE
MERCER ISLAND WA 98040

NUMBER	DATE	DESCRIPTION OF REVISIONS

SHEET TITLE

ROOF FRAMING PLAN

JOB NUMBER

SHEET NUMBER

S2.3

CITY STAMP

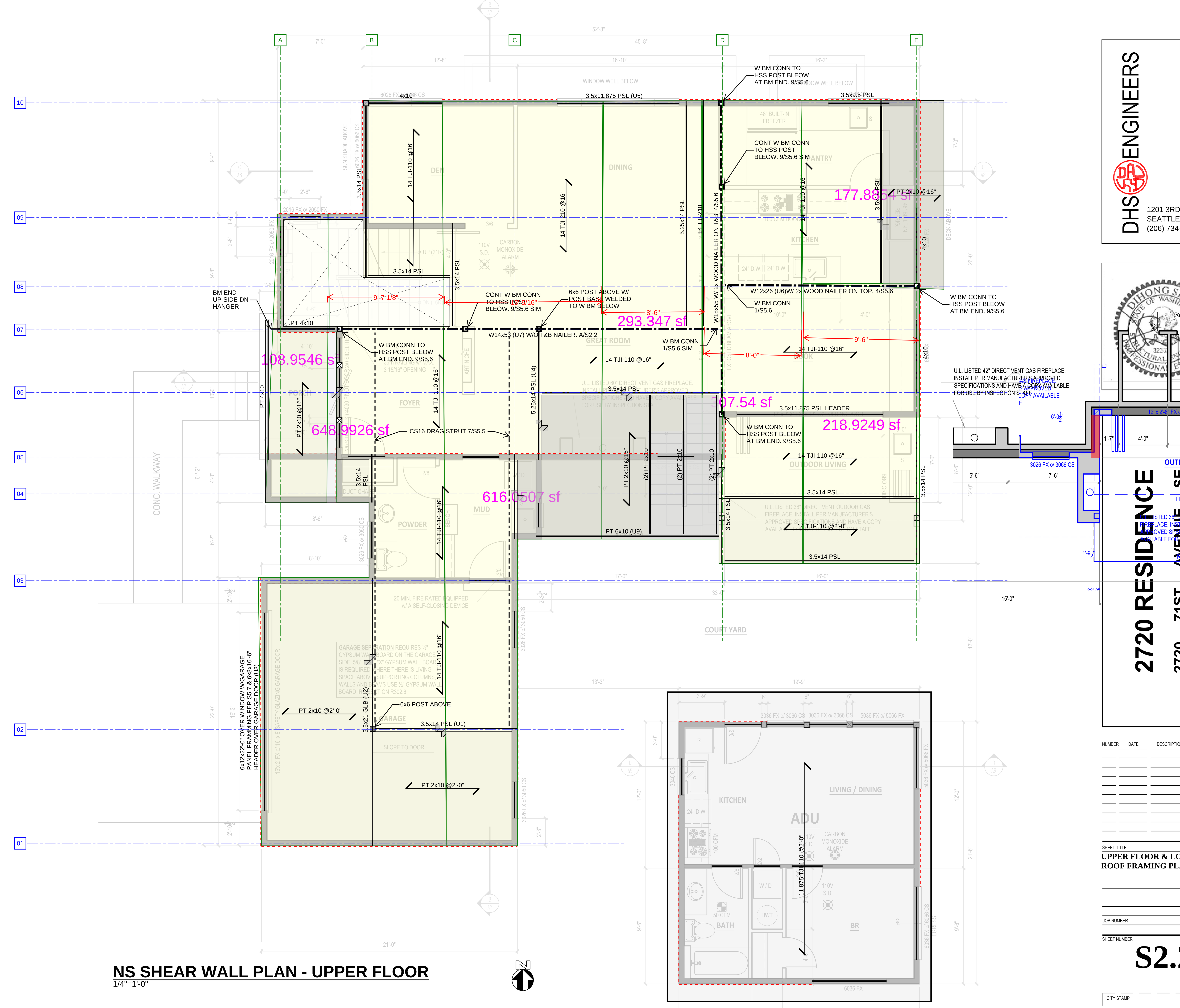
EW Shear Wall Design

shear wall location:	1	roof diaphragm	2nd flr diaphragm
shear force (kips)		0.00	3.18
floor height (ft)		10.00	13.00
wall length without opening (ft)		9.00	21.00
wall length with opening (ft)		9.00	21.00
wall segment length (ft)		9.00	21.00
shear flow (plf)		0.00	151.43
shear wall type per schedule on GN		**NA**	**SW6**
dead loads from floor/roof framing (plf)		30.00	30.00
wall weight (plf)		100.00	130.00
hold down force (kips) with 0.6DL		-0.35	0.61
hold down type per schedule on GN		**NA**	**4**
shear wall location:	2	roof diaphragm	2nd flr diaphragm
shear force (kips)		1.63	0.00
floor height (ft)		10.00	13.00
wall length without opening (ft)		4.50	5.00
wall length with opening (ft)		4.50	5.00
wall segment length (ft)		4.50	5.00
shear flow (plf)		362.22	0.00
shear wall type per schedule on GN		**SW4**	**NA**
dead loads from floor/roof framing (plf)		165.00	75.00
wall weight (plf)		100.00	130.00
hold down force (kips) with 0.6DL		3.26	2.96
hold down type per schedule on GN		**B**	**NA**
shear wall location:	3	roof diaphragm	2nd flr diaphragm
shear force (kips)		0.00	4.63
floor height (ft)		10.00	13.00
wall length without opening (ft)		22.00	16.50
wall length with opening (ft)		22.00	16.50
wall segment length (ft)		22.00	16.50
shear flow (plf)		0.00	280.61
shear wall type per schedule on GN		**NA**	**SW6**
dead loads from floor/roof framing (plf)		60.00	165.00
wall weight (plf)		100.00	130.00
hold down force (kips) with 0.6DL		-1.06	1.13
hold down type per schedule on GN		**NA**	**2**
shear wall location:	4	roof diaphragm	2nd flr diaphragm
shear force (kips)		0.58	0.00
floor height (ft)		10.00	13.00
wall length without opening (ft)		16.00	21.00
wall length with opening (ft)		6.00	21.00
wall segment length (ft)		16.00	21.00
shear flow (plf)		96.67	0.00
shear wall type per schedule on GN		**SW6**	**NA**
dead loads from floor/roof framing (plf)		135.00	2.00
wall weight (plf)		100.00	130.00
hold down force (kips) with 0.6DL		-0.16	-0.99
hold down type per schedule on GN		**NO HD**	**NA**

shear wall location:	5	roof diaphragm	2nd flr diaphragm
shear force (kips)		2.03	5.50
floor height (ft)		10.00	13.00
wall length without opening (ft)		11.00	13.00
wall length with opening (ft)		11.00	13.00
wall segment length (ft)		11.00	13.00
shear flow (plf)		184.55	423.08
shear wall type per schedule on GN		**SW6**	**SW4**
dead loads from floor/roof framing (plf)		165.00	90.00
wall weight (plf)		100.00	130.00
hold down force (kips) with 0.6DL		0.97	5.61
hold down type per schedule on GN		**A**	**8**
shear wall location:	6	roof diaphragm	2nd flr diaphragm
shear force (kips)		0.91	0.00
floor height (ft)		10.00	13.00
wall length without opening (ft)		15.00	10.00
wall length with opening (ft)		10.00	10.00
wall segment length (ft)		15.00	10.00
shear flow (plf)		91.00	0.00
shear wall type per schedule on GN		**SW6**	**NA**
dead loads from floor/roof framing (plf)		30.00	30.00
wall weight (plf)		100.00	130.00
hold down force (kips) with 0.6DL		0.03	-0.46
hold down type per schedule on GN		**NO HD**	**NA**
shear wall location:	7	roof diaphragm	2nd flr diaphragm
shear force (kips)		0.93	4.72
floor height (ft)		10.00	7.00
wall length without opening (ft)		13.50	4.50
wall length with opening (ft)		13.50	4.50
wall segment length (ft)		4.50	4.50
shear flow (plf)		68.89	1048.89
shear wall type per schedule on GN		**SW6**	**STRONG WALL**
dead loads from floor/roof framing (plf)		67.50	150.00
wall weight (plf)		100.00	70.00
hold down force (kips) with 0.6DL		0.06	7.11
hold down type per schedule on GN		**NO HD**	**STRONG WALL**
shear wall location:	8	roof diaphragm	2nd flr diaphragm
shear force (kips)		1.27	0.00
floor height (ft)		10.00	13.00
wall length without opening (ft)		11.50	3.50
wall length with opening (ft)		11.50	3.50
wall segment length (ft)		11.50	3.50
shear flow (plf)		110.43	0.00
shear wall type per schedule on GN		**SW6**	**NA**
dead loads from floor/roof framing (plf)		240.00	120.00
wall weight (plf)		100.00	130.00
hold down force (kips) with 0.6DL		-0.07	-0.33
hold down type per schedule on GN		**NO HD**	**NA**

shear wall location:	9	roof diaphragm	2nd flr diaphragm
shear force (kips)		1.09	3.91
floor height (ft)		10.00	7.00
wall length without opening (ft)		7.00	4.50
wall length with opening (ft)		7.00	4.50
wall segment length (ft)		7.00	4.50
shear flow (plf)		155.71	868.89
shear wall type per schedule on GN		**SW6**	**STRONG WALL**
dead loads from floor/roof framing (plf)		71.25	30.00
wall weight (plf)		100.00	70.00
hold down force (kips) with 0.6DL		1.20	7.14
hold down type per schedule on GN		**A**	**STRONG WALL**

shear wall location:	10	roof diaphragm	2nd flr diaphragm
shear force (kips)		1.20	2.79
floor height (ft)		10.00	13.00
wall length without opening (ft)		36.00	20.00
wall length with opening (ft)		20.00	15.00
wall segment length (ft)		36.00	20.00
shear flow (plf)		60.00	186.00
shear wall type per schedule on GN		**SW6**	**SW6**
dead loads from floor/roof framing (plf)		120.00	135.00
wall weight (plf)		100.00	130.00
hold down force (kips) with 0.6DL		-1.78	-0.95
hold down type per schedule on GN		**NO HD**	**NO HD**



NS SHEAR WALL PLAN - UPPER FLOOR

1/4"=1'-0"

DHS ENGINEERS

1201 3RD AVE, 2200
SEATTLE, WA 98101
(206) 734-5858

2720 RESIDENCE

2720 71ST AVENUE SE
MERCER ISLAND WA 98040

NUMBER	DATE	DESCRIPTION OF REVISIONS

SHEET TITLE

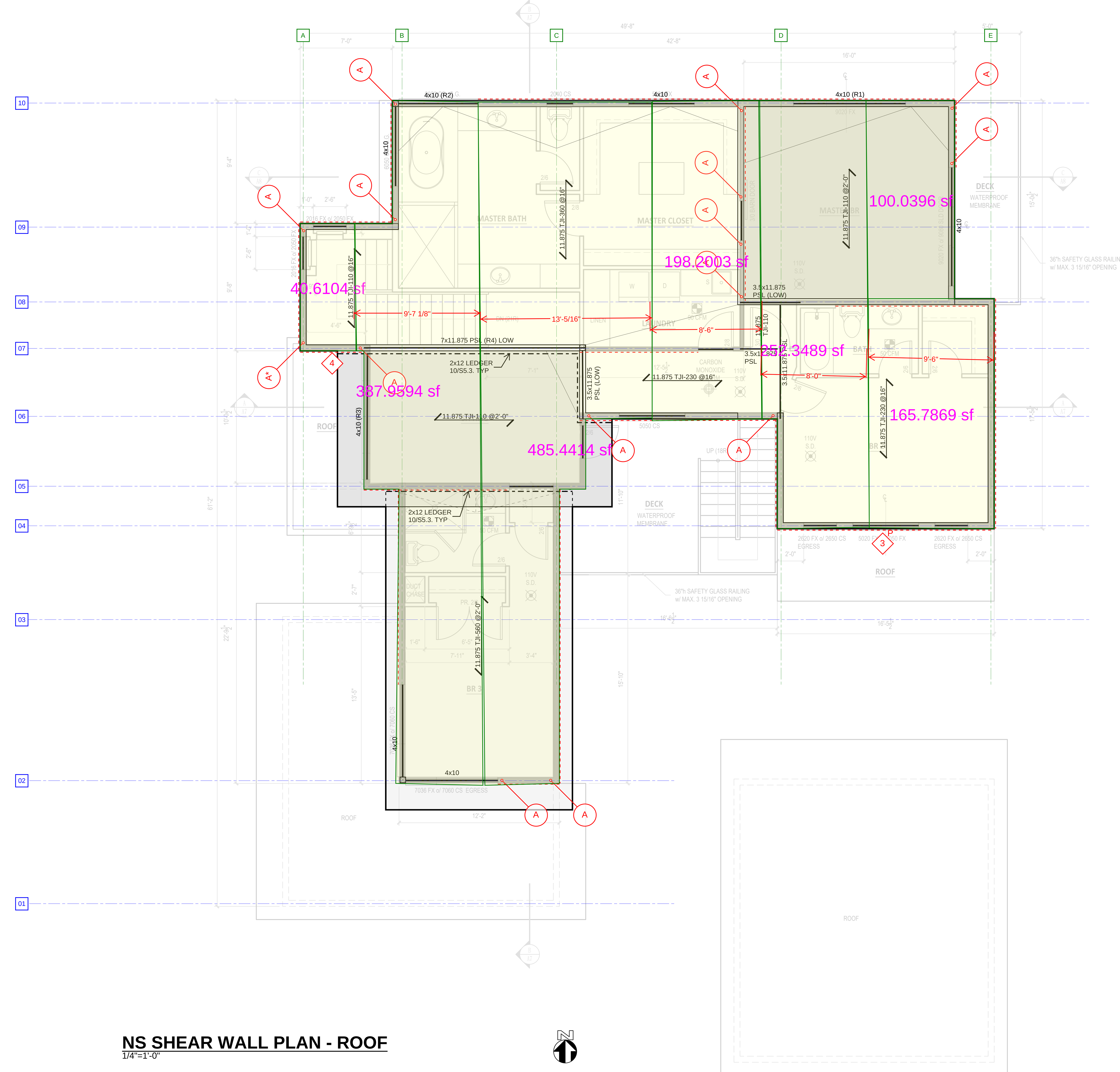
UPPER FLOOR & LOWER ROOF FRAMING PLAN

JOB NUMBER

SHEET NUMBER

S2.2

CITY STAMP



NS SHEAR WALL PLAN - ROOF
1/4"=1'-0"

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MERCER ISLAND WA 98040

NUMBER	DATE	DESCRIPTION OF REVISIONS

SHEET TITLE
ROOF FRAMING PLAN

JOB NUMBER

SHEET NUMBER
S2.3

CITY STAMP

NS Shear Wall Design

shear wall location:	A	roof diaphragm	2nd flr diaphragm
shear force (kips)		0.50	1.80
floor height (ft)		10.00	13.00
wall length without opening (ft)		9.00	9.00
wall length with opening (ft)		6.00	6.00
wall segment length (ft)		9.00	9.00
shear flow (plf)		83.33	300.00
shear wall type per schedule on GN		**SW6**	**SW6**
dead loads from floor/roof framing (plf)		30.00	30.00
wall weight (plf)		100.00	130.00
hold down force (kips) with 0.6DL		0.48	3.95
hold down type per schedule on GN		**A**	**4**
shear wall location:	**B/(GARAGE DOOR)**	roof diaphragm	2nd flr diaphragm
shear force (kips)		1.61	3.50
floor height (ft)		10.00	5.00
wall length without opening (ft)		17.00	5.00
wall length with opening (ft)		17.00	5.00
wall segment length (ft)		14.00	2.50
shear flow (plf)		94.71	700.00
shear wall type per schedule on GN		**SW6**	**GARAGE PANEL**
dead loads from floor/roof framing (plf)		60.00	75.00
wall weight (plf)		100.00	50.00
hold down force (kips) with 0.6DL		-0.02	3.38
hold down type per schedule on GN		**NO HD**	**GARAGE PANEL**
shear wall location:	**C**	roof diaphragm	2nd flr diaphragm
shear force (kips)		2.01	6.93
floor height (ft)		10.00	13.00
wall length without opening (ft)		22.00	20.00
wall length with opening (ft)		22.00	17.00
wall segment length (ft)		22.00	20.00
shear flow (plf)		91.36	407.53
shear wall type per schedule on GN		**SW6**	**SW4**
dead loads from floor/roof framing (plf)		60.00	60.00
wall weight (plf)		100.00	130.00
hold down force (kips) with 0.6DL		-0.14	4.02
hold down type per schedule on GN		**NO HD**	**4**
shear wall location:	**C.8**	roof diaphragm	2nd flr diaphragm
shear force (kips)		1.21	0.00
floor height (ft)		10.00	13.00
wall length without opening (ft)		11.50	21.00
wall length with opening (ft)		11.50	21.00
wall segment length (ft)		7.00	21.00
shear flow (plf)		105.22	0.00
shear wall type per schedule on GN		**SW6**	**NA**
dead loads from floor/roof framing (plf)		60.00	2.00
wall weight (plf)		100.00	130.00
hold down force (kips) with 0.6DL		0.72	-0.12
hold down type per schedule on GN		**A**	**NA**

shear wall location:	D	roof diaphragm	2nd flr diaphragm
shear force (kips)		1.14	5.55
floor height (ft)		10.00	7.00
wall length without opening (ft)		8.00	3.50
wall length with opening (ft)		8.00	3.50
wall segment length (ft)		8.00	3.50
shear flow (plf)		142.50	1585.13
shear wall type per schedule on GN		**SW6**	**STRONG WALL**
dead loads from floor/roof framing (plf)		30.00	120.00
wall weight (plf)		100.00	70.00
hold down force (kips) with 0.6DL		1.11	12.01
hold down type per schedule on GN		**A**	**STRONG WALL**

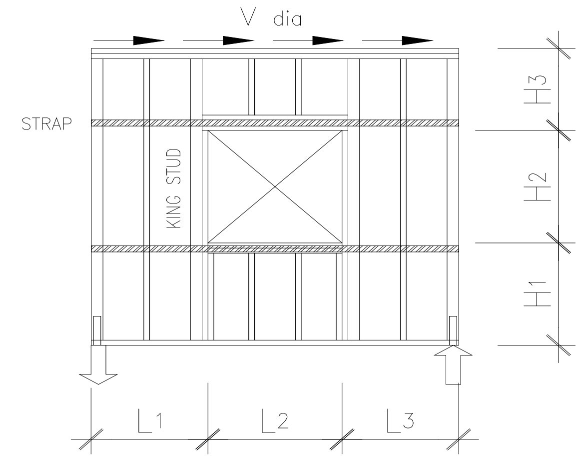
shear wall location:	D.8	roof diaphragm	2nd flr diaphragm
shear force (kips)		0.64	0.00
floor height (ft)		10.00	13.00
wall length without opening (ft)		5.00	10.00
wall length with opening (ft)		5.00	10.00
wall segment length (ft)		5.00	10.00
shear flow (plf)		128.00	0.00
shear wall type per schedule on GN		**SW6**	**NA**
dead loads from floor/roof framing (plf)		30.00	30.00
wall weight (plf)		100.00	130.00
hold down force (kips) with 0.6DL		1.09	0.61
hold down type per schedule on GN		**A**	**NA**

shear wall location:	E	roof diaphragm	2nd flr diaphragm
shear force (kips)		1.33	3.56
floor height (ft)		10.00	13.00
wall length without opening (ft)		17.50	26.00
wall length with opening (ft)		17.50	14.00
wall segment length (ft)		17.50	26.00
shear flow (plf)		75.72	254.48
shear wall type per schedule on GN		**SW6**	**SW6**
dead loads from floor/roof framing (plf)		30.00	120.00
wall weight (plf)		100.00	130.00
hold down force (kips) with 0.6DL		0.07	1.43
hold down type per schedule on GN		**NO HD**	**2**

Wood Shear Wall with an Opening Based on NDS	window width = total wall pier length = 340/2=170 plf

INPUT DATA

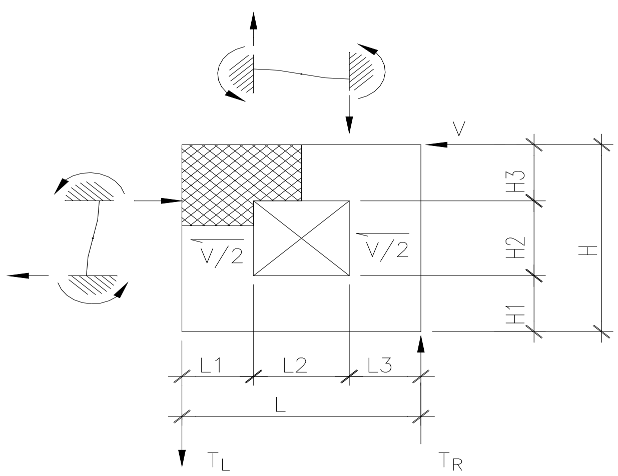
LATERAL FORCE ON DIAPHRAGM:	$v_{dia,\ WIND}$ =	170	plf,for wind			
(SERVICE LOADS)	$v_{dia,\ SEISMIC}$ =	170	plf,for seismic			
DIMENSIONS: L_1 =	3	ft , L_2 =	6	ft , L_3 =	3	ft
H_1 =	2	ft , H_2 =	5	ft , H_3 =	2	ft
KING STUD SECTION	1	pcs, b =	2	in , h =	6	in
EDGE STUD SECTION	2	pcs, b =	2	in , h =	6	in
PANEL GRADE (0 or 1) =	1	<= Sheathing and Single-Floor				
MINIMUM NOMINAL PANEL THICKNESS =		15/32	in			
COMMON NAIL SIZE (0=6d, 1=8d, 2=10d)		2	10d			
SPECIFIC GRAVITY OF FRAMING MEMBERS		0.5				
STORY OPTION (1=ground level, 2=upper level)		2	upper level shear wall			

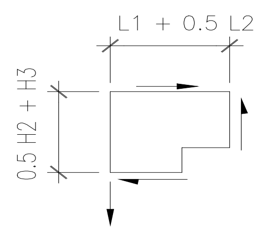


DESIGN SUMMARY

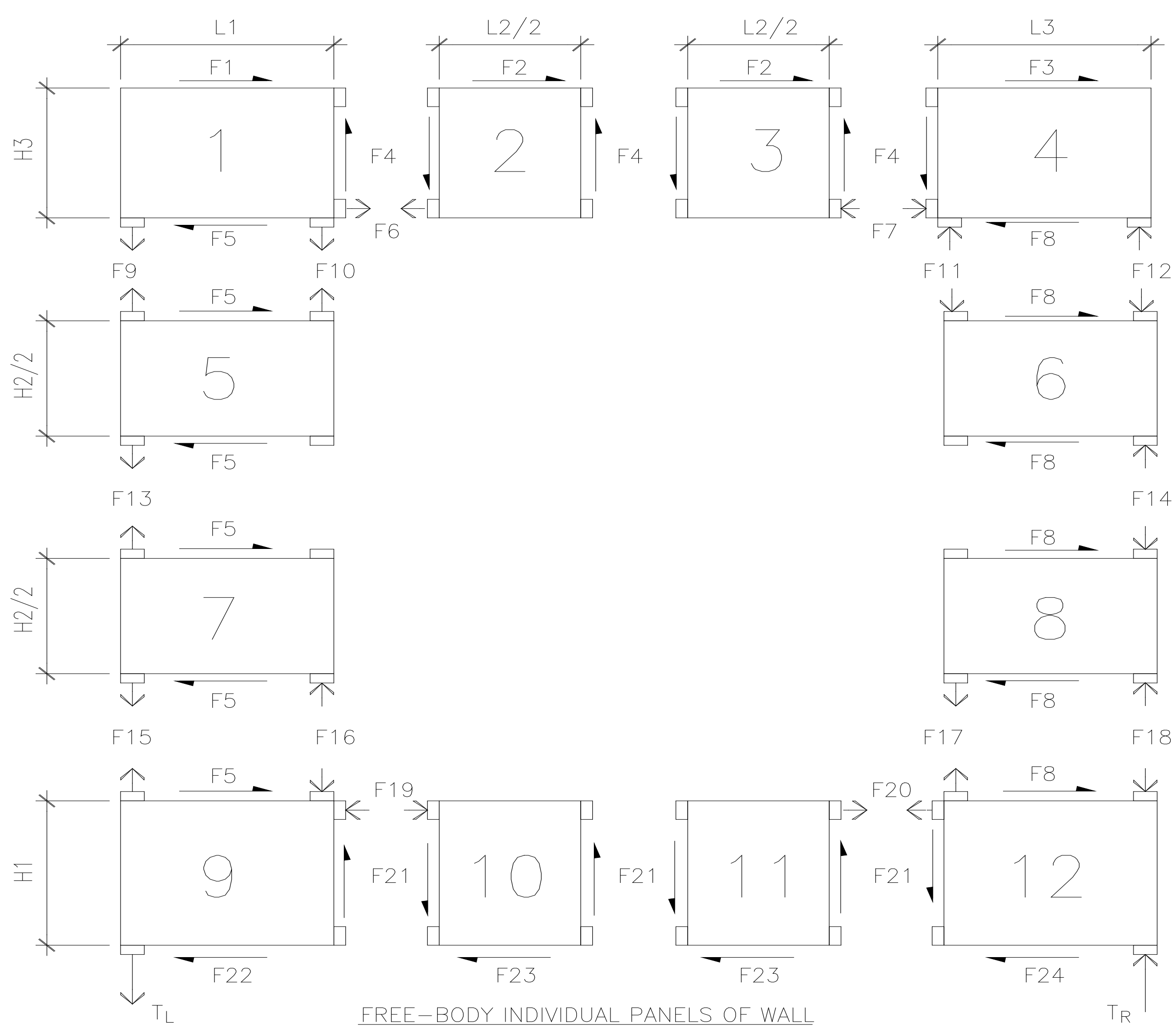
BLOCKED 15/32 SHEATHING WITH 10d COMMON NAILS
@ 4 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD,
SILL PLATE ATTACHMENT 16d AT 6" O.C.

HOLD-DOWN FORCES: T_L = 1.59 k , T_R = 1.59 k (USE CS16 SIMPSON HOLD-DOWN)
MAX STRAP FORCE: F = 1.15 k (USE SIMPSON CS18 OVER WALL SHEATHING WITH FLAT BLOCKING)
KING STUD: 1 - 2" x 6" DOUGLAS FIR-LARCH No. 1, CONTINUOUS FULL HEIGHT.
EDGE STUD: 2 - 2" x 6" DOUGLAS FIR-LARCH No. 1, CONTINUOUS FULL HEIGHT.
SHEAR WALL DEFLECTION: Δ = 0.63 in





ASSUME INFLECTION POINT AT MIDDLE OF WINDOW



FREE-BODY INDIVIDUAL PANELS OF WALL

cont'd

ANALYSIS

CHECK MAX SHEAR WALL DIMENSION RATIO h / w = 1.7 < 2 **[Satisfactory]**

DETERMINE FORCES & SHEAR STRESS OF FREE-BODY INDIVIDUAL PANELS OF WALL

INDIVIDUAL PANEL	W (ft)	H (ft)	MAX SHEAR STRESS (plf)	NO.	FORCE (lbf)	NO.	FORCE (lbf)
1	3.00	2.00	-43	F1	-128	F13	765
2	3.00	2.00	383	F2	1148	F14	765
3	3.00	2.00	383	F3	-128	F15	1615
4	3.00	2.00	-43	F4	765	F16	850
5	3.00	2.50	340	F5	1020	F17	850
6	3.00	2.50	340	F6	1148	F18	1615
7	3.00	2.50	340	F7	1148	F19	1050
8	3.00	2.50	340	F8	1020	F20	1050
9	3.00	2.00	-10	F9	-85	F21	830
10	3.00	2.00	415	F10	850	F22	-30
11	3.00	2.00	415	F11	850	F23	1050
12	3.00	2.00	-10	F12	-85	F24	-30

DETERMINE REQUIRED CAPACITY v_b = 415 plf, (1 Side Panel Required, the Max. Nail Spacing = 4 in)

THE SHEAR CAPACITIES PER IBC Table 2306.4.1 / UBCTable 23-II-I-1 :

Panel Grade	Common Nail	Min. Penetration (in)	Min. Thickness (in)	Blocked Nail Spacing Boundary & All Edges			
				6	4	3	2
Sheathing and Single-Floor	10d	1 5/8	15/32	310	460	600	770

Note: The indicated shear numbers have reduced by specific gravity factor per IBC note a / UBC note1 of the table.

DETERMINE FLOOR SILL PLATE ATTACHMENT (NDS 2005, Table 11Q & Table 11L)

SILL PLATE ATTACHMENT 16d AT 6" O.C.

THE HOLD-DOWN FORCES:

	v_{dia} (plf)	Wall Seismic at mid-story (lbs)	Overturning Moments (ft-lbs)		Resisting Moments (ft-lbs)	Safety Factors	Net Uplift (lbs)		Holddown SIMPSON
SEISMIC	170	173	19138	Left	0	0.9	T_L =	1595	CS16
				Right	0	0.9	T_R =	1595	
WIND	170		18360	Left	0	2/3	T_L =	1530	
				Right	0	2/3	T_R =	1530	

(T_L & T_R values should include upper level UPLIFT forces if applicable)

DETERMINE MAXIMUM SHEAR WALL DEFLECTION: (IBC Section 2305.3.2)

$$\Delta = \Delta_{Bending} + \Delta_{Shear} + \Delta_{Nail\ slip} + \Delta_{Chord\ splice\ slip} = \frac{8 v_b h^3}{E A L_w} + \frac{v_b h}{G t} + 0.75 h e_n + \frac{h d_a}{L_w} = 0.628 \text{ in}$$

Where: v_b = 415 plf; A = 16.50 in^2; t = 0.298 in; L_w = 6 ft; h = 9 ft; e_n = 0.037 in; E = 1.7E+06 psi; G = 9.0E+04 psi; d_a = 0.15 in

CHECK KING STUD CAPACITY

P_{max} = 0.85 kips

F_c = 1500 psi C_D = 1.60 C_P = 0.43 A = 8.25 in^2

E = 1700 ksi C_F = 1.10 F_c' = 1146 psi > f_c = 103 psi

[Satisfactory]

CHECK EDGE STUD CAPACITY

P_{max} = 1.59 kips, (this value should include upper level DOWNWARD loads if applicable)

F_c = 1500 psi C_D = 1.60 C_P = 0.43 A = 16.50 in^2

E = 1700 ksi C_F = 1.10 F_c' = 1146 psi > f_c = 97 psi

[Satisfactory]

Technical References:

1. "National Design Specification, NDS", 2005 Edition, AF & PA, AWC, 2005.

2720 Residence

2720 71st Avenue SE
Mercer Island, Washington 98040

Structural Engineering Calculations

Supplement Calculations for Review Comments (Round-5)

By

Dihong Shao, SE

August 30, 2022

Roof Framing with Rooftop Deck	material weight in psf			
paving system	9.00			
roofing	0.50			
roofing protective board over insulation	0.38			
4.5" average rigid insulation	0.97	0.216	psf/inch	
3/4" deck floor sheathing	2.50			
floor joists	3.00	4 plf	11.875 TJI-560 @16"	
R30 batt insulation (8.25")	0.33	0.04	psf/inch	
gyp sheathing ceiling	2.20			
misc. blockings and MEP system	1.12			
	20.00		**Used 25 psf**	
	roof deck area 970 sf			

Roof without Rooftop Deck	material weight in psf			
roofing	0.50			
roofing protective board over insulation	0.38			
4.5" average rigid insulation	0.97	0.216	psf/inch	
5/8" roof sheathing	1.80			
floor joists	2.00	4 plf	11.875 TJI-560 @2'-0"	average 15.8 psf
R30 batt insulation (8.25")	0.33	0.04	psf/inch	**seismic mass =**
gyp sheathing ceiling	2.20			15.8 psf + 5 pst = 20.8 psf
misc. blockings and MEP system	1.82			(5 psf walls to roof diaphragm)
	10.00		**Used 15 psf**	**Used 25 psf**
	roof area 700 sf			

Floor Framing	material weight in psf			
flooring and finishes	5.00			
3/4' floor sheathing	2.50			
floor joists	3.00	4 plf	11.875 TJI-560 @16"	**seismic mass =**
gyp sheathing ceiling	2.20			15 psf + 10 pst = 25 psf
misc. blockings and MEP system	2.30			(10 psf walls to floor diaphragm)
	15.00		**Used 15 psf**	**Used 25 psf**

Project Name				**Page No.**
Bm/Jst Location/Description:	**Wind Girts of (3) 2x6 w/ A35 top and bottom**			
Wind				
wind load (psf)	44.00	conservative	added point load (kips)	0.00
tributary width (ft)	**6.00**		point load location to farthest support (ft)	0.00
Beam Span (ft)	**10.00**	conservative		
load duration/repetitive factor	**1.60**		**1.00**	
Beam Data Base Number	**2**		**2.0E PSL**	
tributary load (plf)	264.00		**#N/A**	**Beam No.61-88**
moment (kip-ft)	3.30		Provided M	#N/A
shear/reaction (kips)	1.32		Provided V	#N/A
A35 capacity 670#/each	**1.34**		Provided I	#N/A
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	19.80	7.56	10.31	#N/A
Required I (in^4)	76.32	20.80	76.32	#N/A
Required A (in^2)	13.03	8.25	6.30	#N/A
Size	**2x6**	**Beam No.1-20**	**#N/A**	**Beam No.20-60**

Wood Column Design Based on NDS 2005

14ft tall wall studs @16"

INPUT DATA

MEMBER TYPE: 2

1	POST
2	WALL STUD
3	KING STUD

GEOMETRY DATA:

HEIGHT	h =	14	ft
UNBRACED LENGTH	Le x-x (H) =	14	ft
	Le y-y (B) =	1.33	ft

LOAD DATA:

DEAD LOAD		279	lbs
LIVE LOAD		466	lbs
TOTAL		745	lbs
LATERAL LOAD x-x		44	plf
	M=	1078	ft-lbs
	V=	308	lbs
LOAD DURATION	2	OCCUPANCY LIVE LOAD	

DESIGN CRITERIA:

SECTION	1 pcs, B =	2	in
	H =	6	in
SPECIES (1 = DFL, 2 = SP)		1	DOUGLAS FIR-LARCH
GRADE (1, 2, 3, 4, 5, or 6)		4	No. 2
WET / DRY USE ? (1 = DRY, 2 = WE		1	DRY

DESIGN SUMMARY

USE: 1 - 2" x 6" DOUGLAS FIR-LARCH No. 2

1. CHECK VERTICAL LOADS : $f_c < F_c'$?

90 psi < 486 psi ok

2. CHECK BENDING LOADS : $f_b < F_b'$?

1711 psi < 2153 psi ok

3. CHECK INTERACTION : $\left(\frac{f_c}{F_c'}\right)^2 + \left(\frac{1}{1 - f_c / F_{cEx}}\right)\frac{f_{bx}}{F_{bx}'} \le 1$?

1.000 < 1 ok

4. CHECK SHEAR LOADS : $f_v < F_v'$?

56 psi < 288 psi ok

5. HORIZONTAL DEFLECTION AT MIDDLE

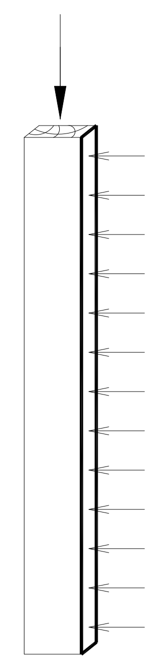
$\Delta = 5wh^4 / (384EI) + 2.4wh^2 / (Ebd) =$ 1 1/6 in

(h / 145)

ANALYSIS

COLUMN BASIC DESIGN STRESSES:

COMPRESSIVE STRESS	F_c =	1350	psi
MODULUS OF ELASTICITY	E =	1600	ksi
BENDING STRESS (X-Axis)	F_{bx} =	900	psi
BENDING STRESS (Y-Axis)	F_{by} =	900	psi
SHEAR STRESS (X-Axis)	F_V =	180	psi



COLUMN PROPERTIES:

COLUMN SECTION	X-Dir	dx =	5.50	in
	Y-Dir	dy =	1.50	in
AREA		A =	8.25	in^2
SECTION PROPERTIES	Abt. xx	Sx =	7.56	in^3
		Ix =	20.80	in^4
	Abt. yy	Sy =	2.06	in^3
LENGTH-DEPTH RATIO		Le x-x / dx =	30.5	
		Le y-y / dy =	10.6	

ADJUSTMENT FACTORS:

		F_{bx}'	F_{by}'	F_c'	F_V'	E'
DURATION FACTOR	C_D	1.60	1.60	1.60	1.60	
MOISTURE FACTOR	C_M	1.00	1.00	1.00	1.00	1.00
TEMPERATURE FACTOR	C_t	1.00	1.00	1.00	1.00	1.00
INCISING FACTOR	C_i	1.00	1.00	1.00	1.00	1.00
SIZE FACTOR	C_F	1.30	1.30	1.10		1.00
FLAT USE FACTOR	C_{fu}		1.15			
COLUMN STABILITY	C_P			0.205		
REPETITIVE MEMBER	C_r	1.15	1.15			
BEAM STABILITY	C_L	1.00	1.00			

COLUMN PARAMETER	c =	0.80	
MODULUS OF ELASTICITY	E'_{min} =	580	ksi
CRITICAL EULER BUCKLING VALUES			
	F_{cE} =	511	psi
	F_c^* =	2376	psi

ADJUSTED PROPERTIES:

MODULUS OF ELASTICITY	E' =	1600	ksi
BENDING STRESS (X-Axis)	F_{bx}' =	2153	psi
BENDING STRESS (Y-Axis)	F_{by}' =	2476	psi
AXIAL STRESS	F_c' =	486	psi
SHEAR STRESS	F_V' =	288	psi

ACTUAL STRESSES:

AXIAL STRESS	f_c =	90.3	psi
BENDING STRESSES	f_{bx} =	1710.5	psi
SHEAR STRESS	f_V =	56	psi

Project Name				Page No.
Bm/Jst Location/Description:	UA SUPPORTING A DISCOUNTINOUS SHEAR WALL			
Roof	SHEAR FORCE AND HD FORCE FOR LINE C.8 ARE 1.21K AND 0.72K RESPECTIVELY			
dead load (psf)	15.00			
live load (psf)	25.00	additional total point load (kips)		0.00
tributary width (ft)	2.00	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00	0.72K * (3.0 OVERSTRENGTH / 1.6 LOAD DURATION)=1.35 K		
live load (psf)	40.00	additional total point load (kips)		1.35
tributary width (ft)	2.00	point load location to farthest support (ft)		10.00
Wall				
wall weight (psf)	10.00			
height (ft)	16.00			
Beam Span (ft)	18.00			
load duration/repetitive factor	1.00		1.00	

Beam Data Base Number	79		2.0E PSL	
			5-1/4x14	Beam No.61-88
tributary load (plf)	350.00			
moment (kip-ft)	20.18		Provided M	40.74
shear/reaction (kips)	3.90		Provided V	14.21
			Provided I	1200.00
	DF#2	Provided	24F-V4 or 24F-V8 DF GL	Provided
Required S (in^3)	193.68	280.73	100.88	1200.45
Required I (in^4)	839.87	2456.38	839.87	26244.00
Required A (in^2)	47.90	96.25	35.45	243.00
Size	6x18	Beam No.1-20	6-3/4x36	Beam No.20-60

Bm/Jst Location/Description:	UB SUPPORTING A DISCOUNTIOUS SHEAR WALL			
Roof	SHEAR FORCE AND HD FORCE FOR LINE D.8 ARE 0.64K AND 1.09K RESPECTIVELY			
dead load (psf)	15.00			
live load (psf)	25.00	additional total point load (kips)		0.00
tributary width (ft)	2.00	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00	1.09K * (3.0 OVERSTRENGTH / 1.6 LOAD DURATION)=1.35 K		
live load (psf)	40.00	additional total point load (kips)		2.04
tributary width (ft)	2.00	point load location to farthest support (ft)		10.50
Wall				
wall weight (psf)	10.00			
height (ft)	16.00			
Beam Span (ft)	14.67			
load duration/repetitive factor	1.00		1.00	

Beam Data Base Number	79		2.0E PSL	
			5-1/4x14	Beam No.61-88
tributary load (plf)	350.00			
moment (kip-ft)	15.50		Provided M	40.74
shear/reaction (kips)	4.03		Provided V	14.21
			Provided I	1200.00
	DF#2	Provided	24F-V4 or 24F-V8 DF GL	Provided
Required S (in^3)	148.84	280.73	77.52	1200.45
Required I (in^4)	526.02	2456.38	526.02	26244.00
Required A (in^2)	49.46	96.25	36.61	243.00
Size	6x18	Beam No.1-20	6-3/4x36	Beam No.20-60

Project Name			**Page No.**	
Bm/Jst Location/Description:	**U6 SEISMIC**			
Roof	**SHEAR FORCE AND HD FORCE FOR LINE 8 ARE 1.27K AND 0.00 K RESPECTIVELY**			
dead load (psf)	20.00			
live load (psf)	45.00	additional total point load (kips)		0.00
tributary width (ft)	**16.00**	point load location to farthest support (ft)		0.00
Floor				
dead load (psf)	15.00	**POINT LOAD FROM U8 INCLUDING AMPLIFIED SEISMIC FORCE 4.0 K**		
live load (psf)	40.00	additional total point load (kips)		4.03
tributary width (ft)	**8.00**	point load location to farthest support (ft)		13.00
Wall				
wall weight (psf)	10.00			
height (ft)	**10.00**			
Beam Span (ft)	**16.00**			
load duration/repetitive factor	**1.00**		**1.00**	

Beam Data Base Number			**STEEL BEAM**	**W12x26**
tributary load (plf)	1580.00		Required	Provided
moment (kip-ft)	60.38	**S**	30.79	33.40
shear/reaction (kips)	15.91	**BxH**		6.5" x 12.25"
		I	179.68	204.00
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	579.62	#N/A	301.88	#N/A
Required I (in^4)	2234.16	#N/A	2234.16	#N/A
Required A (in^2)	251.25	#N/A	96.43	#N/A
Size	**#N/A**	**Beam No.1-20**	**#N/A**	**Beam No.20-60**

Standard and Balloon Framing on Concrete Foundations

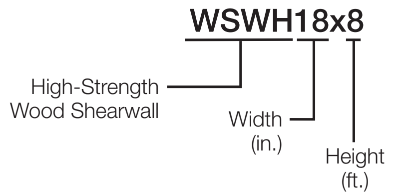
Simpson Strong-Tie® Strong-Wall® high-strength wood shearwalls combine design flexibility with performance. Field trimmable, they can be customized to accommodate varying heights or rake walls. They are evaluated to the 2018 IRC/IBC and are listed by ICC-ES.

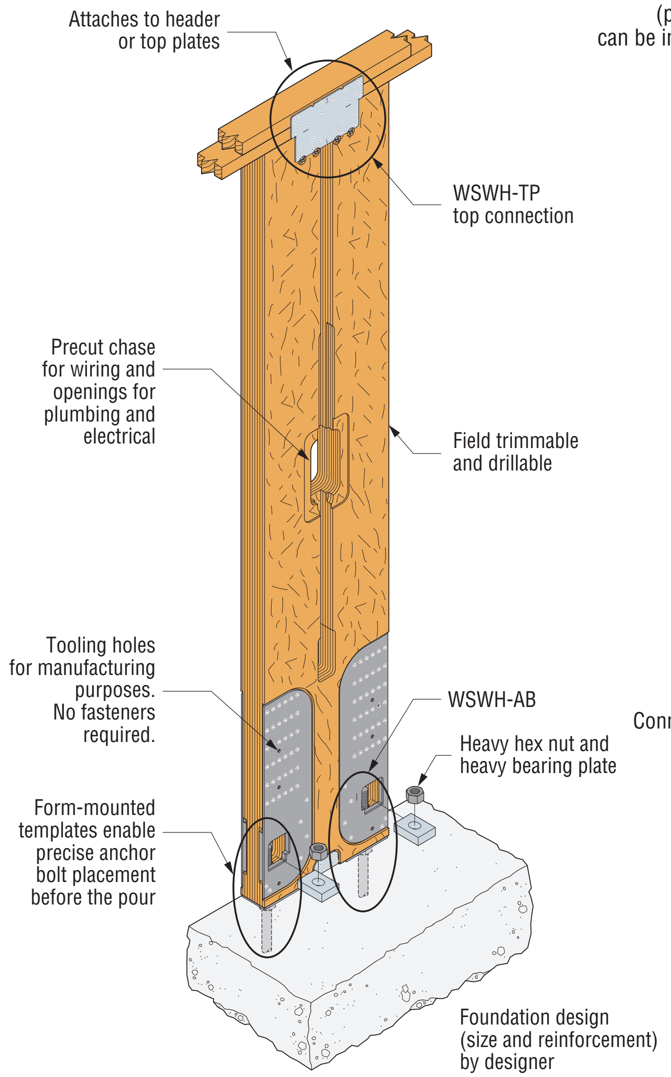
Installation

- All panels may be field trimmed to a minimum of 74½". Trim height from top of panel only, do not trim from sides or bottom. Drilling holes in the Strong-Wall high-strength wood shearwalls is not allowed except as shown on p. 36.
- Anchor bolt nuts should be finger tight plus ½ turn.
- Maximum shim thickness between the shearwall and top plates or header is ⅞". For additional shim thicknesses, see detail 9/WSWH2 on p. 35.
- Walls may also be used in 2x6 wall framing. Install the panel flush to the outside face of the framing and add furring to the opposite face as required to accommodate finish material. See detail 6/WSWH2 on p. 33.
- Top connection installs with a combination of SDS25600 Heavy-Duty Connector screws and SWS16150 Strong-Wall screws.

Codes: ICC-ES ESR-2652, City of LA Building Code Supplement and State of Florida FL5113

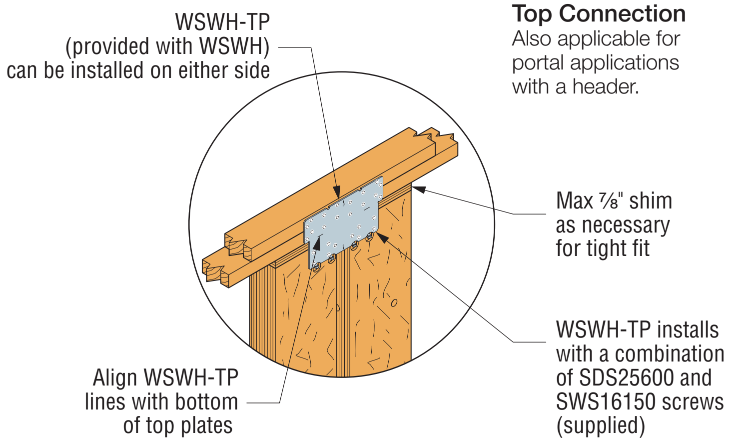
Strong-Wall High-Strength Wood Shearwall Naming Legend

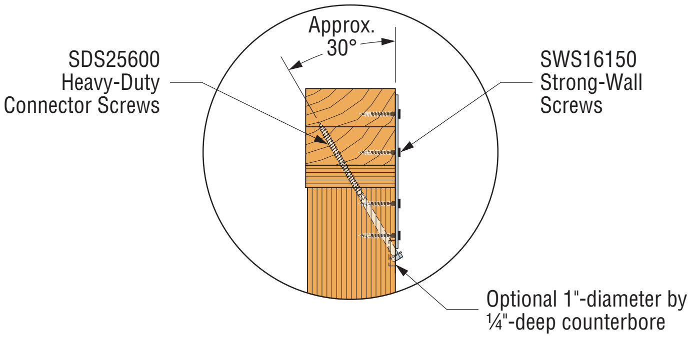




Standard Installation
US Patent 10,711,477

Top Connection
Also applicable for portal applications with a header.





Top Connection
Section View

Standard and Balloon Framing on Concrete Foundations

SIMPSON Strong-Tie

Strong-Wall High-Strength Wood Shearwall Model No.	Panel Evaluation Height, H_e (lb.)[6]	Allow Vertical Load, P (lb.)[4]	2,500 psi Concrete						3,000 psi Concrete					
			Seismic[3]			Wind			Seismic[3]			Wind		
			Allowable ASD Shear Load, V (lb.)	Drift at Allowable Shear, Δ (in.)[7]	Anchor Tension at Allowable Shear, T (lb.)[11]	Allowable ASD Shear Load, V (lb.)	Drift at Allowable Shear, Δ (in.)[7]	Anchor Tension at Allowable Shear, T (lb.)[11]	Allowable ASD Shear Load, V (lb.)	Drift at Allowable Shear, Δ (in.)[7]	Anchor Tension at Allowable Shear, T (lb.)[11]	Allowable ASD Shear Load, V (lb.)	Drift at Allowable Shear, Δ (in.)[7]	Anchor Tension at Allowable Shear, T (lb.)[11]
WSWH12x7	78	1,000	1,300	0.32	13,295	1,670	0.43	17,075	1,300	0.32	13,295	1,670	0.43	17,075
		4,000	1,300	0.32	13,295	1,670	0.43	17,075	1,300	0.32	13,295	1,670	0.43	17,075
		7,500	1,300	0.32	13,295	1,670	0.43	17,075	1,300	0.32	13,295	1,670	0.43	17,075
WSWH18x7	78	1,000	3,795	0.32	23,680	4,470	0.39	27,890	3,795	0.32	23,680	4,470	0.39	27,890
		4,000	3,795	0.32	23,680	4,365	0.38	27,245	3,795	0.32	23,680	4,470	0.39	27,890
		7,500	3,795	0.32	23,680	4,050	0.36	25,285	3,795	0.32	23,680	4,470	0.39	27,890
WSWH24x7	78	1,000	7,450	0.30	33,210	7,795	0.34	34,755	7,450	0.30	33,210	7,795	0.34	34,755
		4,000	7,450	0.30	33,210	7,565	0.33	33,715	7,450	0.30	33,210	7,795	0.34	34,755
		7,500	7,115	0.28	31,715	7,115	0.31	31,715	7,450	0.30	33,210	7,795	0.34	34,755
WSWH12x8	93.25	1,000	1,030	0.40	12,580	1,325	0.53	16,195	1,030	0.40	12,580	1,325	0.53	16,195
		4,000	1,030	0.40	12,580	1,325	0.53	16,195	1,030	0.40	12,580	1,325	0.53	16,195
		7,500	1,030	0.40	12,580	1,325	0.53	16,195	1,030	0.40	12,580	1,325	0.53	16,195
WSWH18x8	93.25	1,000	3,060	0.39	22,835	3,880	0.52	28,925	3,060	0.39	22,835	3,955	0.53	29,490
		4,000	3,060	0.39	22,835	3,650	0.49	27,245	3,060	0.39	22,835	3,955	0.53	29,490
		7,500	3,060	0.39	22,835	3,390	0.46	25,285	3,060	0.39	22,835	3,955	0.53	29,490
WSWH24x8	93.25	1,000	6,240	0.37	33,240	6,650	0.43	35,430	6,240	0.37	33,240	6,910	0.45	36,815
		4,000	6,240	0.37	33,240	6,330	0.41	33,715	6,240	0.37	33,240	6,910	0.45	36,815
		7,500	5,950	0.35	31,715	5,950	0.38	31,715	6,240	0.37	33,240	6,910	0.45	36,815
WSWH12x9	105.25	1,000	850	0.45	11,750	1,095	0.60	15,145	850	0.45	11,750	1,095	0.60	15,145
		4,000	850	0.45	11,750	1,095	0.60	15,145	850	0.45	11,750	1,095	0.60	15,145
		7,500	850	0.45	11,750	1,095	0.60	15,145	850	0.45	11,750	1,095	0.60	15,145
WSWH18x9	105.25	1,000	2,575	0.45	21,680	3,325	0.60	27,975	2,575	0.45	21,680	3,325	0.60	27,975
		4,000	2,575	0.45	21,680	3,235	0.58	27,245	2,575	0.45	21,680	3,325	0.60	27,975
		7,500	2,575	0.45	21,680	3,005	0.54	25,285	2,575	0.45	21,680	3,325	0.60	27,975
WSWH24x9	105.25	1,000	5,150	0.43	30,975	5,890	0.52	35,430	5,150	0.43	30,975	6,120	0.54	36,815
		4,000	5,150	0.43	30,975	5,605	0.50	33,715	5,150	0.43	30,975	6,120	0.54	36,815
		7,500	5,150	0.43	30,975	5,275	0.47	31,715	5,150	0.43	30,975	6,120	0.54	36,815
WSWH12x10	117.25	1,000	700	0.50	10,750	900	0.67	13,855	700	0.50	10,750	900	0.67	13,855
		4,000	700	0.50	10,750	900	0.67	13,855	700	0.50	10,750	900	0.67	13,855
		7,500	700	0.50	10,750	900	0.67	13,855	700	0.50	10,750	900	0.67	13,855
WSWH18x10	117.25	1,000	2,140	0.50	20,055	2,755	0.67	25,840	2,140	0.50	20,055	2,755	0.67	25,840
		4,000	2,140	0.50	20,055	2,755	0.67	25,840	2,140	0.50	20,055	2,755	0.67	25,840
		7,500	2,140	0.50	20,055	2,695	0.65	25,285	2,140	0.50	20,055	2,755	0.67	25,840
WSWH24x10	117.25	1,000	4,010	0.48	26,860	5,215	0.67	34,935	4,010	0.48	26,860	5,215	0.67	34,935
		4,000	4,010	0.48	26,860	5,030	0.64	33,715	4,010	0.48	26,860	5,215	0.67	34,935
		7,500	4,010	0.48	26,860	4,735	0.61	31,715	4,010	0.48	26,860	5,215	0.67	34,935
WSWH12x11	129.25	1,000	595	0.56	10,055	765	0.73	12,930	595	0.56	10,055	765	0.73	12,930
		4,000	595	0.56	10,055	765	0.73	12,930	595	0.56	10,055	765	0.73	12,930
		7,500	595	0.56	10,055	765	0.73	12,930	595	0.56	10,055	765	0.73	12,930
WSWH18x11	129.25	1,000	1,960	0.55	20,240	2,520	0.73	26,060	1,960	0.55	20,240	2,520	0.73	26,060
		4,000	1,960	0.55	20,240	2,520	0.73	26,060	1,960	0.55	20,240	2,520	0.73	26,060
		7,500	1,960	0.55	20,240	2,445	0.71	25,285	1,960	0.55	20,240	2,520	0.73	26,060
WSWH24x11	129.25	1,000	4,000	0.54	29,550	4,795	0.68	35,430	4,000	0.54	29,550	4,985	0.70	36,815
		4,000	4,000	0.54	29,550	4,565	0.64	33,715	4,000	0.54	29,550	4,985	0.70	36,815
		7,500	4,000	0.54	29,550	4,295	0.60	31,715	4,000	0.54	29,550	4,985	0.70	36,815

See foonotes on p. 15.

High-Strength Wood Shearwall Anchorage Solutions

SIMPSON Strong-Tie

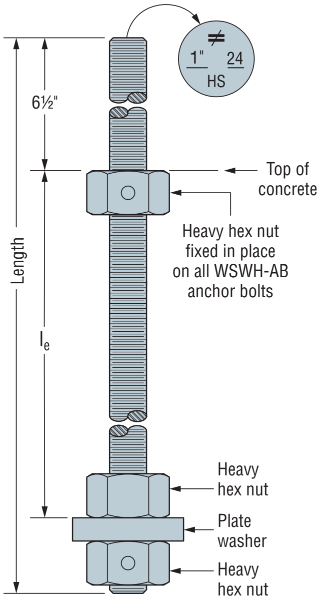
WSWH-AB Anchor Bolts

WSWH-AB anchor bolts in 1" diameters offer flexibility to meet specific project demands. Inspection is easy; the head is stamped with a No-Equal® symbol for identification, bolt length, bolt diameter, and optional "HS" for "High-Strength" if specified.

Material: ASTM F1554 Grade 36;
High-Strength (HS) ASTM A193 Grade B7

An additional nut for template installation is provided with each WSWH-AB.

Strong-Wall® High-Strength Wood Shearwall Model No.	Model No.	Dia. (in.)	Total Length (in.)	l_e (in.)
WSWH12 WSWH18 WSWH24	WSWH-AB1x24	1	24	15½
	WSWH-AB1x24HS	1	24	15½
	WSWH-AB1x30	1	30	21½
	WSWH-AB1x30HS	1	30	21½
	WSWH-AB1x36	1	36	27½
	WSWH-AB1x36HS	1	36	27½



WSWH-AB

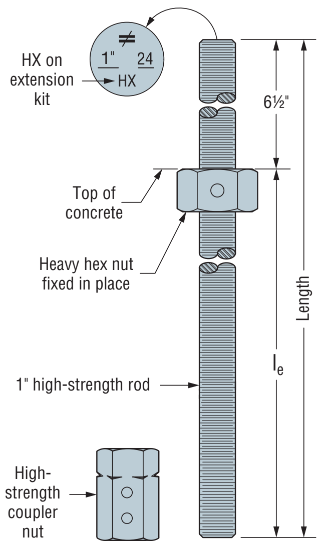
WSWH-HSR Extension Kit

WSWH-HSR allows for anchorage in tall stemwall applications where full embedment of a WSWH-AB into the footing is required. The head is stamped for identification like a WSWH-AB. Kit includes ASTM A193 Grade B7 high-strength rod with heavy hex nut fixed in place and high-strength coupler nut.

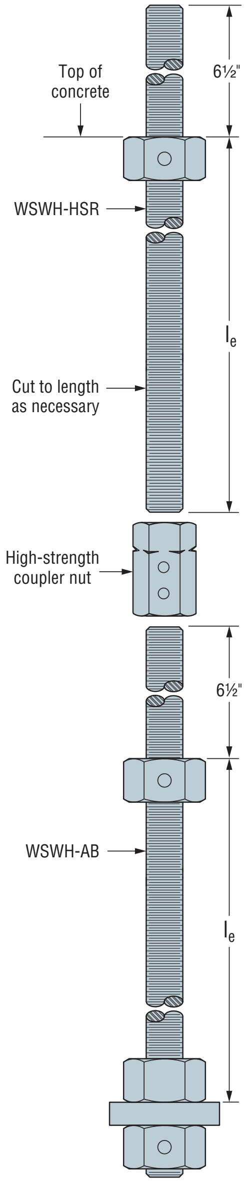
Strong-Wall High-Strength Wood Shearwall Model No.	Model No.	Dia. (in.)	Total Length (in.)	l_e (in.)
WSWH12 WSWH18 WSWH24	WSWH-HSR1x24KT	1	24	17½
	WSWH-HSR1x36KT	1	36	29½

Note: Do not use in place of WSWH-AB.

Total l_e = WSWH-HSR l_e + WSWH-AB l_e + 6½"



WSWH-HSRXXKT



WSWH-HSR and WSWH-AB Assembly

High-Strength Wood Shearwall Anchorage Solutions

SIMPSON Strong-Tie

Tension Anchorage Solutions — 2,500 psi Concrete[1,5,6]

Design Criteria	Concrete Condition	Anchor Strength[2]	WSWH-AB1 Anchor Bolt		
			ASD Allowable Tension (lb.)	W (in.)	d_e (in.)
Seismic[3]	Cracked	Standard	16,000	33	11
			17,100	35	12
		High-Strength	34,100	52	18
			36,800	55	19
	Uncracked	Standard	15,700	28	10
			17,100	30	10
		High-Strength	33,500	45	15
			36,800	48	16
Wind[4]	Cracked	Standard	6,200	16	6
			11,400	24	8
			17,100	32	11
		High-Strength	21,100	36	12
			27,300	42	14
			34,100	48	16
			36,800	51	17
	Uncracked	Standard	6,400	14	6
			12,500	22	8
			17,100	28	10
		High-Strength	22,900	33	11
			26,400	36	12
			34,200	42	14
			36,800	44	15

See footnotes on p. 23.

Tension Anchorage Solutions — 3,000 psi Concrete[1,5,6]

Design Criteria	Concrete Condition	Anchor Strength[2]	WSWH-AB1 Anchor Bolt		
			ASD Allowable Tension (lb.)	W (in.)	d_e (in.)
Seismic[3]	Cracked	Standard	16,000	31	11
			17,100	33	11
		High-Strength	33,900	49	17
			36,800	52	18
	Uncracked	Standard	16,300	27	9
			17,100	28	10
		High-Strength	34,000	43	15
			36,800	46	16
Wind[4]	Cracked	Standard	5,600	14	6
			10,200	21	7
			17,100	30	10
		High-Strength	20,000	33	11
			26,500	39	13
			33,600	45	15
			36,800	48	16
	Uncracked	Standard	6,200	13	6
			12,800	21	7
			17,100	26	9
		High-Strength	21,800	30	10
			28,900	36	12
			33,100	39	13
			36,800	42	14

See footnotes on p. 23.

(https://www.strongtie.com/)

Coiled Strap Designer

www.strongtie.com

Input Data:

DEMAND LOAD	COILED STRAP MODEL	NAIL DIAMETER	NAIL LENGTH
6000 lbs.	**CMST14** installed over sheathing	**0.148 in.**	**2.500 in.**

WOOD SPECIES
DFL

Calculation Results:

COILED STRAP MODEL	NAIL SIZE	END NAILS	TOTAL NAILS
CMST14	**0.148 in. x 2 1/2 in.**	**31**	**62**

NUMBER OF STUDS	ALLOWABLE STRAP CAPACITY
Double	**6085 lbs**

SIMPSON Strong-Tie (https://www.strongtie.com/)

Coiled Strap Cut Length Calculator

www.strongtie.com

Input Data:

COILED STRAP MODEL
CMST14

NUMBER OF INSTALLED STRAPS
1

QUANTITY OF NAILS SPECIFIED?
Yes

TOTAL NAILS PER STRAP
62

NAILS INSTALLED IN EVERY HOLE?
Yes

SOLE PLATE DEPTH
0

SUB-FLOOR THICKNESS
0

TOP PLATES DEPTH
0

FLOOR DEPTH
0

Calculation Results:

TOTAL LINEAR FEET OF STRAP REQUIRED
4.833 linear ft.

END LENGTH
29 in.

TOTAL NUMBER OF NAILS
62 Nails

CLEAR SPAN
0 in.

CUT LENGTH
58 in.