Hu Residence

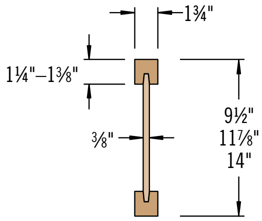
30XX 69th Avenue SE
Mercer Island, Washington 98040

Structural Engineering Calculations

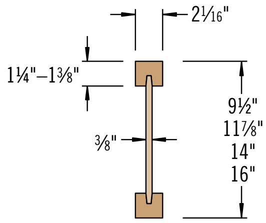
By

Dihong Shao, SE

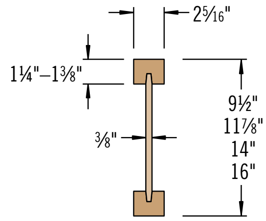
January 8, 2021



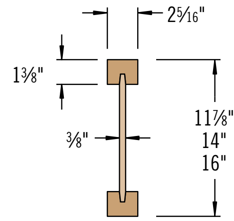
TJI® 110 Joists



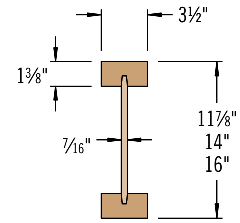
TJI® 210 Joists



TJI® 230 Joists



TJI® 360 Joists



TJI® 560 Joists

Some TJI® joist series may not be available in your region.
Contact your iLevel representative for information.

Design Properties (100% Load Duration)

Depth	TJI®	Basic Properties				Reaction Properties					
		Joist Weight (lbs/ft)	Maximum Resistive Moment(1) (ft-lbs)	Joist Only EI x 10⁶ (in.²-lbs)	Maximum Vertical Shear (lbs)	1¾" End Reaction (lbs)	3½" End Reaction (lbs)	3½" Intermediate Reaction (lbs)		5¼" Intermediate Reaction (lbs)	
								No Web Stiffeners	With Web Stiffeners	No Web Stiffeners	With Web Stiffeners
9½"	110	2.3	2,500	157	1,220	910	1,220	1,935	N.A.	2,350	N.A.
	210	2.6	3,000	186	1,330	1,005	1,330	2,145	N.A.	2,565	N.A.
	230	2.7	3,330	206	1,330	1,060	1,330	2,410	N.A.	2,790	N.A.
11⅞"	110	2.5	3,160	267	1,560	910	1,375	1,935	2,295	2,350	2,705
	210	2.8	3,795	315	1,655	1,005	1,460	2,145	2,505	2,565	2,925
	230	3.0	4,215	347	1,655	1,060	1,485	2,410	2,765	2,790	3,150
	360	3.0	6,180	419	1,705	1,080	1,505	2,460	2,815	3,000	3,360
	560	4.0	9,500	636	2,050	1,265	1,725	3,000	3,475	3,455	3,930
14"	110	2.8	3,740	392	1,860	910	1,375	1,935	2,295	2,350	2,705
	210	3.1	4,490	462	1,945	1,005	1,460	2,145	2,505	2,565	2,925
	230	3.3	4,990	509	1,945	1,060	1,485	2,410	2,765	2,790	3,150
	360	3.3	7,335	612	1,955	1,080	1,505	2,460	2,815	3,000	3,360
	560	4.2	11,275	926	2,390	1,265	1,725	3,000	3,475	3,455	3,930
16"	210	3.3	5,140	629	2,190	1,005	1,460	2,145	2,505	2,565	2,925
	230	3.5	5,710	691	2,190	1,060	1,485	2,410	2,765	2,790	3,150
	360	3.5	8,405	830	2,190	1,080	1,505	2,460	2,815	3,000	3,360
	560	4.5	12,925	1,252	2,710	1,265	1,725	3,000	3,475	3,455	3,930

(1) **Caution: Do not** increase joist moment design properties by a repetitive member use factor.

General Notes

- Design reaction includes all loads on the joist. Design shear is computed at the inside face of supports and includes all loads on the span(s). Allowable shear may sometimes be increased at interior supports in accordance with ICC ES ESR-1153, and these increases are reflected in span tables.
- The following formulas approximate the uniform load deflection of Δ (inches):

For TJI® 110, 210, 230, and 360 Joists

$$\Delta = \frac{22.5\,wL^4}{EI} + \frac{2.67\,wL^2}{d \times 10^5}$$

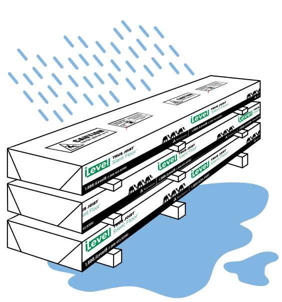
For TJI® 560 Joists

$$\Delta = \frac{22.5\,wL^4}{EI} + \frac{2.29\,wL^2}{d \times 10^5}$$

w = uniform load in pounds per linear foot
L = span in feet
d = out-to-out depth of the joist in inches
EI = value from table above

Protect products from sun and water

PRODUCT STORAGE



CAUTION: Wrap is slippery when wet or icy

Use support blocks at 10' on-center to keep products out of mud and water

TJI® joists are intended for dry-use applications

L/480 Live Load Deflection

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
		12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
9½"	110	16'-11"	15'-6"	14'-7"	13'-7"	16'-11"	15'-6"	14'-3"	12'-9"
	210	17'-9"	16'-3"	15'-4"	14'-3"	17'-9"	16'-3"	15'-4"	14'-0"
	230	18'-3"	16'-8"	15'-9"	14'-8"	18'-3"	16'-8"	15'-9"	14'-8"
11⅞"	110	20'-2"	18'-5"	17'-4"	15'-9"(1)	20'-2"	17'-8"	16'-1"(1)	14'-4"(1)
	210	21'-1"	19'-3"	18'-2"	16'-11"	21'-1"	19'-3"	17'-8"	15'-9"(1)
	230	21'-8"	19'-10"	18'-8"	17'-5"	21'-8"	19'-10"	18'-7"	16'-7"(1)
	360	22'-11"	20'-11"	19'-8"	18'-4"	22'-11"	20'-11"	19'-8"	17'-10"(1)
	560	26'-1"	23'-8"	22'-4"	20'-9"	26'-1"	23'-8"	22'-4"	20'-9"(1)
14"	110	22'-10"	20'-11"	19'-2"	17'-2"(1)	22'-2"	19'-2"	17'-6"(1)	15'-0"(1)
	210	23'-11"	21'-10"	20'-8"	18'-10"(1)	23'-11"	21'-1"	19'-2"(1)	16'-7"(1)
	230	24'-8"	22'-6"	21'-2"	19'-9"(1)	24'-8"	22'-2"	20'-3"(1)	17'-6"(1)
	360	26'-0"	23'-8"	22'-4"	20'-9"(1)	26'-0"	23'-8"	22'-4"(1)	17'-10"(1)
	560	29'-6"	26'-10"	25'-4"	23'-6"	***29'-6"***	***26'-10"***	25'-4"(1)	20'-11"(1)
16"	210	26'-6"	24'-3"	22'-6"(1)	19'-11"(1)	26'-0"	22'-6"(1)	20'-7"(1)	16'-7"(1)
	230	27'-3"	24'-10"	23'-6"	21'-1"(1)	***27'-3"***	23'-9"	21'-8"(1)	17'-6"(1)
	360	28'-9"	26'-3"	24'-8"(1)	21'-5"(1)	***28'-9"***	26'-3"(1)	22'-4"(1)	17'-10"(1)
	560	32'-8"	29'-8"	28'-0"	25'-2"(1)	***32'-8"***	***29'-8"***	26'-3"(1)	20'-11"(1)

L/360 Live Load Deflection (Minimum Criteria per Code)

Depth	TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
		12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
9½"	110	18'-9"	17'-2"	15'-8"	14'-0"	18'-1"	15'-8"	14'-3"	12'-9"
	210	19'-8"	18'-0"	17'-0"	15'-4"	19'-8"	17'-2"	15'-8"	14'-0"
	230	20'-3"	18'-6"	17'-5"	16'-2"	***20'-3"***	18'-1"	16'-6"	14'-9"
11⅞"	110	22'-3"	19'-4"	17'-8"	15'-9"(1)	20'-5"	17'-8"	16'-1"(1)	14'-4"(1)
	210	23'-4"	21'-2"	19'-4"	17'-3"(1)	***22'-4"***	19'-4"	17'-8"	15'-9"(1)
	230	24'-0"	21'-11"	20'-5"	18'-3"	***23'-7"***	20'-5"	18'-7"	16'-7"(1)
	360	25'-4"	23'-2"	21'-10"	20'-4"(1)	***25'-4"***	***23'-2"***	***21'-10"(1)***	17'-10"(1)
	560	28'-10"	26'-3"	24'-9"	23'-0"	***28'-10"***	***26'-3"***	***24'-9"***	20'-11"(1)
14"	110	24'-4"	21'-0"	19'-2"	17'-2"(1)	22'-2"	19'-2"	17'-6"(1)	15'-0"(1)
	210	26'-6"	23'-1"	21'-1"	18'-10"(1)	24'-4"	21'-1"	19'-2"(1)	16'-7"(1)
	230	27'-3"	24'-4"	22'-2"	19'-10"(1)	***25'-8"***	22'-2"	20'-3"(1)	17'-6"(1)
	360	28'-9"	26'-3"	24'-9"(1)	21'-5"(1)	***28'-9"***	***26'-3"(1)***	22'-4"(1)	17'-10"(1)
	560	32'-8"	29'-9"	28'-0"	25'-2"(1)	***32'-8"***	***29'-9"***	***26'-3"(1)***	20'-11"(1)
16"	210	28'-6"	24'-8"	22'-6"(1)	19'-11"(1)	26'-0"	22'-6"(1)	20'-7"(1)	16'-7"(1)
	230	30'-1"	26'-0"	23'-9"	21'-1"(1)	***27'-5"***	23'-9"	21'-8"(1)	17'-6"(1)
	360	31'-10"	29'-0"	26'-10"(1)	21'-5"(1)	***31'-10"***	***26'-10"(1)***	22'-4"(1)	17'-10"(1)
	560	36'-1"	32'-11"	31'-0"(1)	25'-2"(1)	***36'-1"***	***31'-6"(1)***	26'-3"(1)	20'-11"(1)

(1) Web stiffeners are required at intermediate supports of continuous-span joists when the intermediate bearing length is ***less*** than 5¼" and the span on either side of the intermediate bearing is greater than the following spans:

TJI®	40 PSF Live Load / 10 PSF Dead Load				40 PSF Live Load / 20 PSF Dead Load			
	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.	12" o.c.	16" o.c.	19.2" o.c.	24" o.c.
110	N.A.	N.A.	N.A.	15'-4"	N.A.	N.A.	16'-0"	12'-9"
210	N.A.	N.A.	21'-4"	17'-0"	N.A.	21'-4"	17'-9"	14'-2"
230	N.A.	N.A.	N.A.	19'-2"	N.A.	N.A.	19'-11"	15'-11"
360	N.A.	N.A.	24'-5"	19'-6"	N.A.	24'-5"	20'-4"	16'-3"
560	N.A.	N.A.	29'-10"	23'-10"	N.A.	29'-10"	24'-10"	19'-10"

- Long-term deflection under dead load, which includes the effect of creep, has not been considered. ***Bold italic*** spans reflect initial dead load deflection exceeding 0.33".

Material Weights

(Include TJI® weights in dead load calculations— see **Design Properties** table on page 3 for joist weights)

Floor Panels

Southern Pine

½" plywood . . . . 1.7 psf
⅝" plywood . . . . 2.0 psf
¾" plywood . . . . 2.5 psf
1⅛" plywood . . . . 3.8 psf
½" OSB . . . . 1.8 psf
⅝" OSB . . . . 2.2 psf
¾" OSB . . . . 2.7 psf
⅞" OSB . . . . 3.1 psf
1⅛" OSB . . . . 4.1 psf

Based on: Southern pine – 40 pcf for plywood, 44 pcf for OSB

Roofing

Asphalt shingles . . . . 2.5 psf
Wood shingles . . . . 2.0 psf
Clay tile . . . . 9.0 to 14.0 psf
Slate (⅜" thick) . . . . 15.0 psf

Roll or Batt Insulation (1" thick):

Rock wool . . . . 0.2 psf
Glass wool . . . . 0.1 psf

Floor Finishes

Hardwood (nominal 1") . . . . 4.0 psf
Sheet vinyl . . . . 0.5 psf
Carpet and pad . . . . 1.0 psf
¾" ceramic or quarry tile . . . . 10.0 psf

Concrete:

Regular (1") . . . . 12.0 psf
Lightweight (1") . . . . 8.0 to 10.0 psf
Gypsum concrete (¾") . . . . 6.5 psf

Ceilings

Acoustical fiber tile . . . . 1.0 psf
½" gypsum board . . . . 2.2 psf
⅝" gypsum board . . . . 2.8 psf
Plaster (1" thick) . . . . 8.0 psf

How to Use These Tables

1. Determine the appropriate live load deflection criteria.
2. Identify the live and dead load condition.
3. Select on-center spacing.
4. Scan down the column until you meet or exceed the span of your application.
5. Select TJI® joist and depth.

Live load deflection is not the only factor that affects how a floor will perform. To more accurately predict floor performance, use our TJ-Pro™ Ratings.

General Notes

- Tables are based on:
  - Uniform loads.
  - More restrictive of simple or continuous span.
  - Clear distance between supports (1¾" minimum end bearing).
- Assumed composite action with a single layer of 24" on-center span-rated, glue-nailed floor panels for deflection only. **Spans shall be reduced 6" when floor panels are nailed only.**
- Spans generated from iLevel® software may exceed the spans shown in these tables because software reflects actual design conditions.
- For multi-family applications and other loading conditions not shown, refer to iLevel® software or to the load table on page 5.

Floor—100% (PLF)

Depth	TJI®	8' Live Load L/480	8' Total Load	10' Live Load L/480	10' Total Load	12' Live Load L/480	12' Total Load	14' Live Load L/480	14' Total Load	16' Live Load L/480	16' Total Load	18' Live Load L/480	18' Total Load	20' Live Load L/480	20' Total Load	22' Live Load L/480	22' Total Load	24' Live Load L/480	24' Total Load
9½"	110	*	190	140	152	85	127	56	99	38	76								
	210	*	210	161	169	99	141	65	119	45	90								
	230	*	236	175	190	108	158	71	133	49	99								
11⅞"	110	*	190	*	152	*	127	92	109	63	95	45	76						
	210	*	210	*	169	*	141	106	121	74	106	53	92						
	230	*	236	*	190	*	158	116	136	80	119	58	102	43	83				
	360	*	241	*	193	*	162	136	139	95	121	69	108	51	97	39	78		
	560	*	294	*	236	*	197	*	169	138	148	101	132	76	119	58	108	45	91
14"	110	*	190	*	152	*	127	*	109	91	95	66	85						
	210	*	210	*	169	*	141	*	121	*	106	76	94	57	85				
	230	*	236	*	190	*	158	*	136	115	119	83	106	62	95	47	81		
	360	*	241	*	193	*	162	*	139	*	121	98	108	73	97	56	88	44	81
	560	*	294	*	236	*	197	*	169	*	148	*	132	107	119	83	108	65	99
16"	210	*	210	*	169	*	141	*	121	*	106	*	94	76	85	58	77		
	230	*	236	*	190	*	158	*	136	*	119	*	106	83	95	64	87	50	78
	360	*	241	*	193	*	162	*	139	*	121	*	108	*	97	75	88	59	81
	560	*	294	*	236	*	197	*	169	*	148	*	132	*	119	*	108	86	99

Joist Clear Span

* Indicates that **Total Load** value controls.

How to Use This Table

1. Calculate actual total and live load in pounds per linear foot (plf).
2. Select appropriate **Joist Clear Span**.
3. Scan down the column to find a TJI® joist that meets or exceeds actual total and live loads.

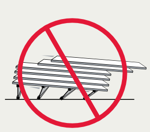
General Notes

- Table is based on:
  - Uniform loads.
  - No composite action provided by sheathing.
  - More restrictive of simple or continuous span.
- **Total Load** limits joist deflection to L/240.
- **Live Load** is based on joist deflection of L/480.
- If a live load deflection limit of L/360 is desired, multiply value in **Live Load** column by 1.33. The resulting live load shall not exceed the **Total Load** shown.
- Table does not account for safe loading. Use iLevel software when this condition applies.

PSF to PLF Conversions

O.C. Spacing	Load in Pounds Per Square Foot (PSF) 20	25	30	35	40	45	50	55	60
	Load in Pounds Per Linear Foot (PLF)								
12"	20	25	30	35	40	45	50	55	60
16"	27	34	40	47	54	60	67	74	80
19.2"	32	40	48	56	64	72	80	88	96
24"	40	50	60	70	80	90	100	110	120

DO NOT walk on joists until braced. INJURY MAY RESULT.



DO NOT stack building materials on unsheathed joists. Stack only over beams or walls.

DO NOT walk on joists that are lying flat.

WARNING

Joists are unstable until braced laterally

Bracing Includes:
- Blocking
- Hangers
- Rim Board
- Sheathing
- Rim Joist
- Strut Lines

WARNING NOTES: *Lack of proper bracing during construction can result in serious accidents. Observe the following guidelines:*

1. All blocking, hangers, rim boards, and rim joists at the end supports of the TJI® joists must be completely installed and properly nailed.
2. Lateral strength, like a braced end wall or an existing deck, must be established at the ends of the bay. This can also be accomplished by a temporary or permanent deck (sheathing) fastened to the first 4 feet of joists at the end of the bay.
3. Safety bracing of 1x4 (minimum) must be nailed to a braced end wall or sheathed area (as in note 2) and to each joist. Without this bracing, buckling sideways or rollover is highly probable under light construction loads—such as a worker or one layer of unnailed sheathing.
4. Sheathing must be completely attached to each TJI® joist before additional loads can be placed on the system.
5. Ends of cantilevers require safety bracing on both the top and bottom flanges.
6. The flanges must remain straight within a tolerance of ½" from true alignment.

Project Name				Page No.
Bm/Jst Location/Description:	**R1 OR GIRDER TRUSS ON GRID B**			
Roof				
dead load (psf)	15.00			
live load (psf)	25.00	additional total point load (kips)		0.00
tributary width (ft)	**20.00**	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)	**20.00**			
load duration/repetitive factor	**1.15**		**1.00**	
Beam Data Base Number	**38**		**2.0E PSL**	
tributary load (plf)	800.00		**#N/A**	**Beam No.61-88**
moment (kip-ft)	40.00		Provided M	#N/A
shear/reaction (kips)	8.00		Provided V	#N/A
			Provided I	#N/A
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	333.91	280.73	173.91	224.46
Required I (in^4)	1850.18	2456.38	1850.18	1918.51
Required A (in^2)	109.84	96.25	63.23	84.56
Size	**6x18**	**Beam No.1-20**	**5-1/8x16-1/2**	**Beam No.20-60**

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)	**6.00**	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)	**12.00**			
load duration/repetitive factor	**1.15**		**1.00**	
Beam Data Base Number	**15**		**2.0E PSL**	
tributary load (plf)	240.00		**#N/A**	**Beam No.61-88**
moment (kip-ft)	4.32		Provided M	#N/A
shear/reaction (kips)	1.44		Provided V	#N/A
			Provided I	#N/A
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	36.06	51.56	18.78	#N/A
Required I (in^4)	119.89	193.36	119.89	#N/A
Required A (in^2)	19.77	41.25	8.85	#N/A
Size	**6x8**	**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)	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)	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	11		**2.0E PSL**	
tributary load (plf)	600.00		**#N/A**	**Beam No.61-88**
moment (kip-ft)	6.08		Provided M	#N/A
shear/reaction (kips)	2.70		Provided V	#N/A
			Provided I	#N/A
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	50.71	73.83	26.41	#N/A
Required I (in^4)	126.45	415.28	126.45	#N/A
Required A (in^2)	37.07	39.38	21.34	#N/A
Size	**4x12**	**Beam No.1-20**	**#N/A**	**Beam No.20-60**

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)	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)	0.00	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	0.00			
Beam Span (ft)	6.50			
load duration/repetitive factor	1.15		1.00	
Beam Data Base Number	9		**2.0E PSL**	
tributary load (plf)	440.00		**#N/A**	**Beam No.61-88**
moment (kip-ft)	2.32		Provided M	#N/A
shear/reaction (kips)	1.43		Provided V	#N/A
			Provided I	#N/A
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	19.40	30.66	10.10	#N/A
Required I (in^4)	34.93	111.15	34.93	#N/A
Required A (in^2)	19.63	25.38	11.30	#N/A
Size	**4x8**	**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)	**11.00**	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)	**3.00**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**9.00**			
Beam Span (ft)	**21.00**	**I RATIO**	0.93	5.5/5.125=1.07OK
load duration/repetitive factor	**1.15**		**1.00**	

Beam Data Base Number	**39**		**2.0E PSL**	
tributary load (plf)	755.00		**#N/A**	**Beam No.61-88**
moment (kip-ft)	41.62		Provided M	#N/A
shear/reaction (kips)	7.93		Provided V	#N/A
footing 2x4x1.5=12kips			Provided I	#N/A
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	347.43	280.73	180.95	264.56
Required I (in^4)	2223.47	2456.38	2223.47	2400.75
Required A (in^2)	108.85	96.25	62.66	82.25
Size	**6x18**	**Beam No.1-20**	**5-1/8x18**	**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)		0.00
tributary width (ft)	**16.50**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**9.00**			
Beam Span (ft)	**11.00**			
load duration/repetitive factor	**1.00**		**1.00**	

Beam Data Base Number	**71**		**2.0E PSL**	
tributary load (plf)	997.50		**3-1/2x11-7/8**	**Beam No.61-88**
moment (kip-ft)	15.09		Provided M	19.90
shear/reaction (kips)	5.49		Provided V	8.04
			Provided I	490.00
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	144.84	280.73	75.44	1200.45
Required I (in^4)	383.82	2456.38	383.82	26244.00
Required A (in^2)	64.97	96.25	49.87	243.00
Size	**6x18**	**Beam No.1-20**	**6-3/4x36**	**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)	**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)		5.49
tributary width (ft)	**11.00**	point load location to farthest support (ft)		3.50
Wall				
wall weight (psf)	10.00			
height (ft)	**9.00**			
Beam Span (ft)	**6.00**			
load duration/repetitive factor	**1.00**		**1.00**	
Beam Data Base Number	**69**		**2.0E PSL**	
tributary load (plf)	775.00		**3-1/2x9-1/2**	**Beam No.61-88**
moment (kip-ft)	11.49		Provided M	13.06
shear/reaction (kips)	5.53		Provided V	6.43
			Provided I	250.00
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	110.29	280.73	57.44	1200.45
Required I (in^4)	159.42	2456.38	159.42	26244.00
Required A (in^2)	72.70	96.25	50.23	243.00
Size	**6x18**	**Beam No.1-20**	**6-3/4x36**	**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)	**14.50**	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)	**3.00**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**9.00**			
Beam Span (ft)	**20.00**			
load duration/repetitive factor	**1.15**		**1.00**	
Beam Data Base Number	**39**		**2.0E PSL**	
tributary load (plf)	895.00		**#N/A**	**Beam No.61-88**
moment (kip-ft)	44.75		Provided M	#N/A
shear/reaction (kips)	8.95		Provided V	#N/A
			Provided I	#N/A
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	373.57	280.73	194.57	264.56
Required I (in^4)	2276.88	2456.38	2276.88	2400.75
Required A (in^2)	122.89	96.25	37.84	82.25
Size	**6x18**	**Beam No.1-20**	**5-1/8x18**	**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)	**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)		8.95
tributary width (ft)	**10.50**	point load location to farthest support (ft)		3.50
Wall				
wall weight (psf)	10.00			
height (ft)	**9.00**			
Beam Span (ft)	**6.00**			
load duration/repetitive factor	**1.00**		**1.00**	
Beam Data Base Number	**71**		**2.0E PSL**	
tributary load (plf)	747.50		**3-1/2x11-7/8**	**Beam No.61-88**
moment (kip-ft)	16.42		Provided M	19.90
shear/reaction (kips)	7.46		Provided V	8.04
footing 2x4x1.5=12kips			Provided I	490.00
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	157.59	280.73	82.08	1200.45
Required I (in^4)	227.79	2456.38	227.79	26244.00
Required A (in^2)	98.21	96.25	67.84	243.00
Size	**6x18**	**Beam No.1-20**	**6-3/4x36**	**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)	**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)	**10.50**	point load location to farthest support (ft)		0.00
Wall				
wall weight (psf)	10.00			
height (ft)	**9.00**			
Beam Span (ft)	**5.50**			
load duration/repetitive factor	**1.00**		**1.00**	
Beam Data Base Number	**9**		**2.0E PSL**	
tributary load (plf)	747.50		**#N/A**	**Beam No.61-88**
moment (kip-ft)	2.83		Provided M	#N/A
shear/reaction (kips)	2.06		Provided V	#N/A
			Provided I	#N/A
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	27.13	30.66	14.13	#N/A
Required I (in^4)	35.95	111.15	35.95	#N/A
Required A (in^2)	24.34	25.38	18.69	#N/A
Size	**4x8**	**Beam No.1-20**	**#N/A**	**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)	12.50	point load location to farthest support (ft)		0.00
Floor		HD FORCExOVER-STRENGTH FACTOR/DURATION FACTRO		
dead load (psf)	15.00	0.9 KIPSx3.5/1.6=1.97 KIPS		
live load (psf)	40.00	additional total point load (kips)		1.97
tributary width (ft)	2.00	point load location to farthest support (ft)		6.50
Wall				
wall weight (psf)	10.00			
height (ft)	9.00			
Beam Span (ft)	11.00			
load duration/repetitive factor	1.00		1.00	
Beam Data Base Number	71		**2.0E PSL**	
tributary load (plf)	700.00		3-1/2x11-7/8	Beam No.61-88
moment (kip-ft)	15.83		Provided M	19.90
shear/reaction (kips)	5.01		Provided V	8.04
			Provided I	490.00
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	151.93	280.73	79.13	1200.45
Required I (in^4)	402.61	2456.38	402.61	26244.00
Required A (in^2)	59.38	96.25	30.39	243.00
Size	**6x18**	Beam No.1-20	**6-3/4x36**	Beam No.20-60

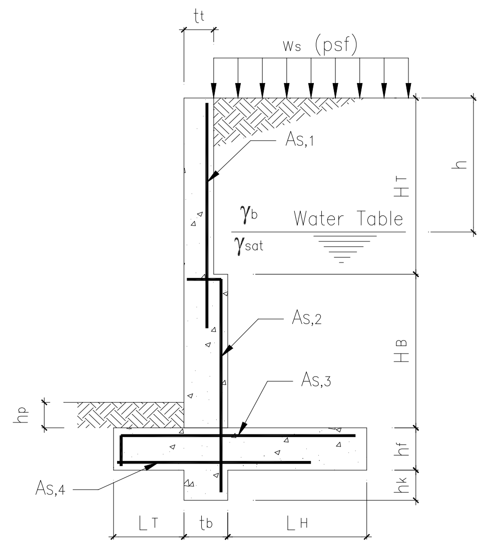
Bm/Jst Location/Description:	**M1**			
Roof				
dead load (psf)	15.00			
live load (psf)	25.00	additional total point load (kips)		0.00
tributary width (ft)	12.50	point load location to farthest support (ft)		0.00
Floor		HD FORCExOVER-STRENGTH FACTOR/DURATION FACTRO		
dead load (psf)	15.00	2.76 KIPSx3.5/1.6=3.85 KIPS		
live load (psf)	40.00	additional total point load (kips)		3.85
tributary width (ft)	4.50	point load location to farthest support (ft)		10.50
Wall				
wall weight (psf)	10.00			
height (ft)	20.00			
Beam Span (ft)	16.50			
load duration/repetitive factor	1.00		1.00	
Beam Data Base Number	39		**2.0E PSL**	
tributary load (plf)	947.50		#N/A	Beam No.61-88
moment (kip-ft)	46.94		Provided M	#N/A
shear/reaction (kips)	10.27		Provided V	#N/A
			Provided I	#N/A
	DF#2	Provided	**24F-V4 or 24F-V8 DF GL**	Provided
Required S (in^3)	450.67	280.73	234.72	264.56
Required I (in^4)	1791.41	2456.38	1791.41	2400.75
Required A (in^2)	121.59	96.25	93.33	82.25
Size	**6x18**	Beam No.1-20	**5-1/8x18**	Beam No.20-60

Retaining Wall Design Based on ACI 318	4ft;passive sf=2;friction sf=1.5; no slide

INPUT DATA & DESIGN SUMMARY

CONCRETE STRENGTH	f_c'	=	2.5	ksi		
REBAR YIELD STRESS	f_y	=	60	ksi		
LATERAL SOIL PRESSURE	P_a=$k_a\,\gamma_b$	=	40	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	=	0.8			
ALLOW SOIL PRESSURE	Qa	=	2	ksf		
THICKNESS OF TOP STEM	t_t	=	8	in		
THICKNESS OF KEY & STEM	t_b	=	8	in		
TOE WIDTH	L_T	=	1.5	ft		
HEEL WIDTH	L_H	=	0.33	ft		
HEIGHT OF TOP STEM	H_T	=	4	ft		
HEIGHT OF BOT. STEM	H_B	=	0.01	ft		
FOOTING THICKNESS	h_f	=	10	in		
KEY DEPTH	h_k	=	0	in		
SOIL OVER TOE	h_p	=	6	in		
TOP STEM REINF. ($A_{s,1}$)		#	4	@	12	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	@	12	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	@	16	in o.c.
BOT. REINF.OF FOOTING ($A_{s,4}$)		#	5	@	16	in

[THE WALL DESIGN IS ADEQUATE.]



ANALYSIS

SERVICE LOADS

$H_b = 0.5\,P_a\,h^2 + h\,P_a\,H + 0.5\,[P_a\,(\gamma_{sat} - \gamma_w)\,/\,\gamma_b + \gamma_w]\,H^2$ = 0.47 kips

Where h = 4.84 ft , H = 0 ft

$H_s = w_s\,P_a\,(H_T + H_B + h_f)\,/\,\gamma_b$ = 0.07 kips

$H_p = 0.5\,P_p\,(h_p + h_f + h_k)^2$ = 0.36 kips

$W_s = w_s\,(L_H + t_b - t_t)$ = 0.01 kips

$W_b = W_{b1} + W_{b2}$ = 0.15 kips

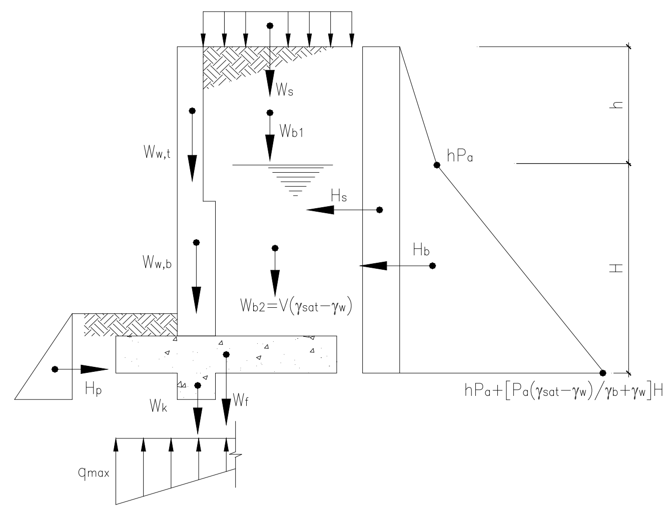
Where W_{b1} = 0.15 kips , W_{b2} = 0.00 kips

$W_f = h_f\,(L_H + t_b + L_T)\,\gamma_c$ = 0.31 kips

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

$W_{w,t} = t_t\,H_T\,\gamma_c$ = 0.40 kips

$W_{w,b} = t_b\,H_B\,\gamma_c$ = 0.00 kips



FACTORED LOADS

$\gamma H_b = 1.6\,H_b$ = 0.75 kips

$\gamma H_s = 1.6\,H_s$ = 0.11 kips

$\gamma W_s = 1.6\,W_s$ = 0.02 kips

$\gamma W_b = 1.2\,W_b$ = 0.17 kips

$\gamma W_f = 1.2\,W_f$ = 0.37 kips

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

$\gamma W_{w,t} = 1.2\,W_{w,t}$ = 0.48 kips

$\gamma W_{w,b} = 1.2\,W_{w,b}$ = 0.00 kips

OVERTURNING MOMENT

	H	γH	y	H y	γH y
H_b	0.47	0.75	1.61	0.76	1.21
H_s	0.07	0.11	2.42	0.17	0.27
Σ	0.54	0.86		**0.93**	1.48

RESISTING MOMENT

	W	γW	x	W x	γW x
W_s	0.01	0.02	2.33	0.03	0.05
W_b	0.15	0.17	2.33	0.34	0.41
W_f	0.31	0.37	1.25	0.39	0.47
W_k	0.00	0.00	1.83	0.00	0.00
$W_{w,t}$	0.40	0.48	1.83	0.73	0.88
$W_{w,b}$	0.00	0.00	1.83	0.00	0.00
Σ	0.87	1.05		**1.49**	1.81

OVERTURNING FACTOR OF SAFETY (1806.1)

$$SF = \frac{\Sigma Wx}{\Sigma Hy} = \quad 1.61 \quad > \quad 1.5$$

[Satisfactory]

(cont'd)

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

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

$e = \frac{L}{2} - \frac{\Sigma Wx - Hy}{\Sigma W}$ = 0.60 ft

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

= 0.89 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 = 4.01 ft , H' = 0 ft

A = $w_s P_a / \gamma_b$ = 15 plf

B = h P_a = 160 plf

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

At base of top stem

M_u = 0.87 ft-kips

V_u = 0.61 kips

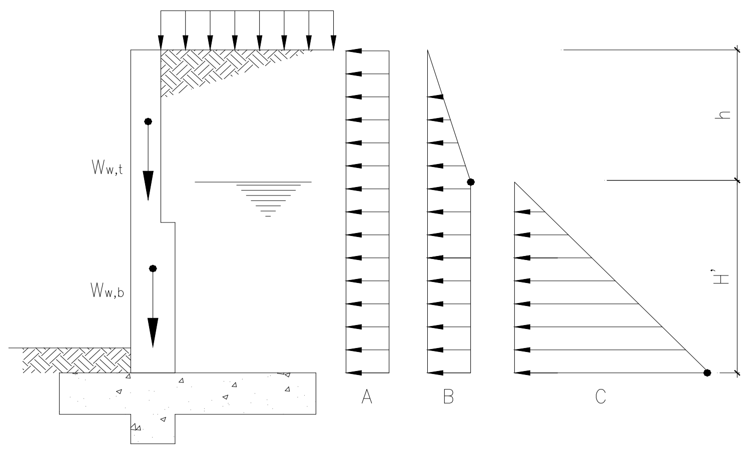
P_u = 0.48 kips

At base of bottom stem

M_u = 0.87 ft-kips

V_u = 0.61 kips

P_u = 0.48 kips



		At top stem		At base of bottom stem	
$\phi M_n = \phi\left[A_s f_y\left(d - \frac{A_s f_y - P_u}{1.7bf'_c}\right)\right]$	=	5.42	ft-kips ,	5.42	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	=	0.2	in^2 ,	0.2	in^2
ρ	=	0.003		0.003	
$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t}$	=	0.013		0.013	
		>	ρ	>	ρ
			[Satisfactory]		**[Satisfactory]**
$\rho_{MIN} = 0.0018\frac{t}{d}$	=	0.002		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	
$V_{allowable} = 2\phi bd\sqrt{f'_c}$	=	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)

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

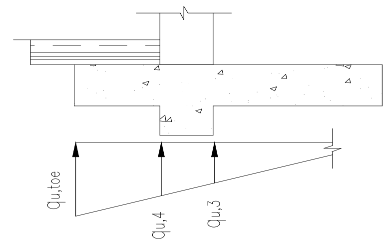
$\rho_{MIN} = \frac{0.0018}{2}\frac{h_f}{d}$ = 0.001

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

= 0.85 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	=	8.25	in	$q_{u,\ toe}$	=	2.29	ksf
	e_u	=	0.94	ft	$q_{u,\ heel}$	=	n/a	ksf
	S	=	-1.25	ft	$q_{u,\ 3}$	=	-3.13	ksf

$(A_{S,\ 3})_{required}$ = 0.11 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)

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

$$M_{u,4} = \frac{\left(q_{u,4} + 2 q_{u,toe}\right) b L_T^2}{6} \quad \frac{L_T^2}{2L} \gamma w_f \quad = \quad 1.00 \text{ ft-kips}$$

where	d	=	6.69	in
	$q_{u,\ 4}$	=	-1.46	ksf

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

$(A_{S,\ 4})_{required}$ = 0.04 in^2 / ft **<** $A_{S,\ 4}$ **[Satisfactory]**

CHECK SLIDING CAPACITY (IBC 1806.1)

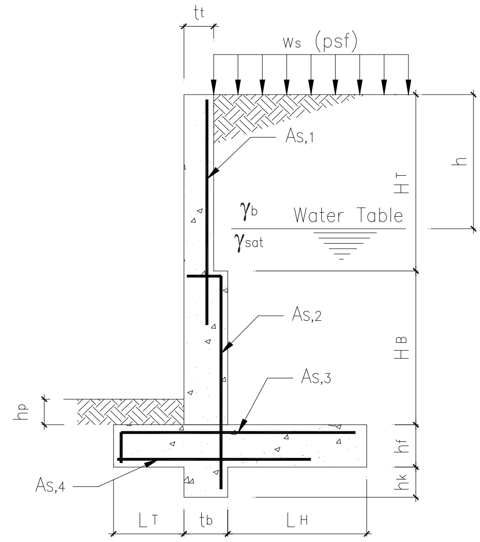
1.5 (H_b + H_s) = 0.81 kips **<** $H_p + \mu\ \Sigma W$ = 1.05 kips
[Satisfactory]

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;passive sf=2;friction sf=1.5; no slide

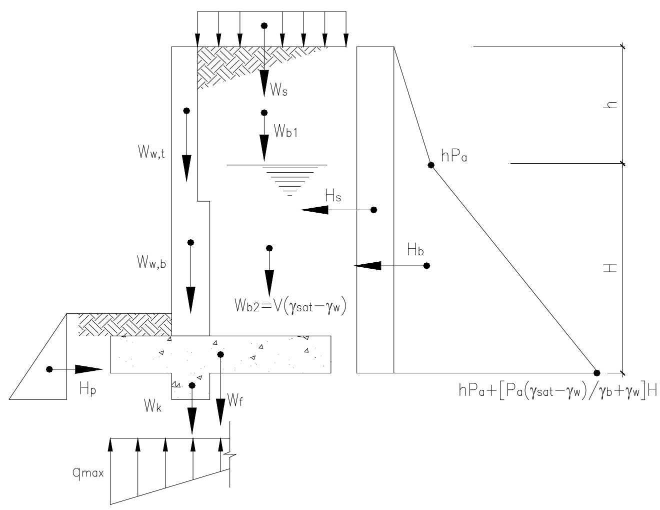
INPUT DATA & DESIGN SUMMARY



CONCRETE STRENGTH	f_c'	=	2.5	ksi		
REBAR YIELD STRESS	f_y	=	60	ksi		
LATERAL SOIL PRESSURE	$P_a=k_a \gamma_b$	=	40	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	=	0.8			
ALLOW SOIL PRESSURE	Q_a	=	2	ksf		
THICKNESS OF TOP STEM	t_t	=	8	in		
THICKNESS OF KEY & STEM	t_b	=	8	in		
TOE WIDTH	L_T	=	2	ft		
HEEL WIDTH	L_H	=	0.83	ft		
HEIGHT OF TOP STEM	H_T	=	6	ft		
HEIGHT OF BOT. STEM	H_B	=	0.01	ft		
FOOTING THICKNESS	h_f	=	10	in		
KEY DEPTH	h_k	=	0	in		
SOIL OVER TOE	h_p	=	6	in		
TOP STEM REINF. ($A_{s,1}$)		#	4	@	12	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	@	12	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	@	12	in o.c.
BOT. REINF.OF FOOTING ($A_{s,4}$)		#	5	@	12	in

[THE WALL DESIGN IS ADEQUATE.]

ANALYSIS



SERVICE LOADS

$H_b = 0.5\, P_a\, h^2 + h\, P_a\, H + 0.5\, [P_a\, (\gamma_{sat} - \gamma_w) / \gamma_b + \gamma_w]\, H^2$ = 0.94 kips

Where h = 6.84 ft , H = 0 ft

$H_s = w_s\, P_a\, (H_T + H_B + h_f) / \gamma_b$ = 0.10 kips

$H_p = 0.5\, P_p\, (h_p + h_f + h_k)^2$ = 0.36 kips

$W_s = w_s\, (L_H + t_b - t_t)$ = 0.03 kips

$W_b = W_{b1} + W_{b2}$ = 0.55 kips

Where W_{b1} = 0.55 kips , W_{b2} = 0.00 kips

$W_f = h_f\, (L_H + t_b + L_T)\, \gamma_c$ = 0.44 kips

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

$W_{w,t} = t_t\, H_T\, \gamma_c$ = 0.60 kips

$W_{w,b} = t_b\, H_B\, \gamma_c$ = 0.00 kips

FACTORED LOADS

$\gamma H_b = 1.6\, H_b$ = 1.50 kips

$\gamma H_s = 1.6\, H_s$ = 0.16 kips

$\gamma W_s = 1.6\, W_s$ = 0.05 kips

$\gamma W_b = 1.2\, W_b$ = 0.66 kips

$\gamma W_f = 1.2\, W_f$ = 0.52 kips

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

$\gamma W_{w,t} = 1.2\, W_{w,t}$ = 0.72 kips

$\gamma W_{w,b} = 1.2\, W_{w,b}$ = 0.00 kips

OVERTURNING MOMENT

	H	γH	y	H y	γH y
H_b	0.94	1.50	2.28	2.14	3.42
H_s	0.10	0.16	3.42	0.34	0.54
Σ	1.04	1.66		**2.48**	3.96

RESISTING MOMENT

	W	γW	x	W x	γW x
W_s	0.03	0.05	3.08	0.10	0.16
W_b	0.55	0.66	3.08	1.69	2.03
W_f	0.44	0.52	1.75	0.76	0.92
W_k	0.00	0.00	2.33	0.00	0.00
$W_{w,t}$	0.60	0.72	2.33	1.40	1.68
$W_{w,b}$	0.00	0.00	2.33	0.00	0.00
Σ	1.62	1.96		**3.96**	4.79

OVERTURNING FACTOR OF SAFETY (1806.1)

$$SF = \frac{\Sigma Wx}{\Sigma Hy} = 1.6 > 1.5$$

[Satisfactory]

(cont'd)

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

$L = L_T + t_b + L_H$ = 3.50 ft

$e = \frac{L}{2} - \frac{\Sigma Wx - Hy}{\Sigma W}$ = 0.83 ft

$$q_{MAX} = \begin{cases} \frac{\Sigma W\left(1+\frac{6e}{L}\right)}{BL}, & for \quad e \le \frac{L}{6} \\ \frac{2\Sigma W}{3B(0.5L-e)}, & for \quad e > \frac{L}{6} \end{cases}$$ = 1.18 ksf < Q_a **[Satisfactory]**

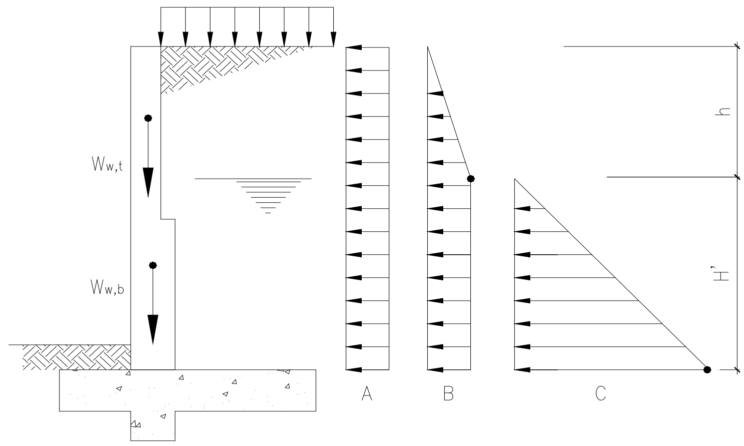
CHECK FLEXURE CAPACITY, AS,1 & AS,2, FOR STEM (ACI 318-05 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

h = 6.01 ft , H' = 0 ft

A = $w_s P_a / \gamma_b$ = 15 plf

B = h P_a = 240 plf

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



At base of top stem

M_u = 2.72 ft-kips

V_u = 1.29 kips

P_u = 0.72 kips

At base of bottom stem

M_u = 2.74 ft-kips

V_u = 1.30 kips

P_u = 0.72 kips

			At top stem		At base of bottom stem	
$\phi M_n = \phi\left[A_s f_y \left(d - \frac{A_s f_y - P_u}{1.7 b f'_c}\right)\right]$		=	5.43	ft-kips ,	5.43	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	=	0.2	in^2 ,	0.2	in^2
	ρ	=	0.003		0.003	
$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t}$		=	0.013		0.013	
			>	ρ	>	ρ
				[Satisfactory]		**[Satisfactory]**
$\rho_{MIN} = 0.0018\frac{t}{d}$		=	0.002		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)

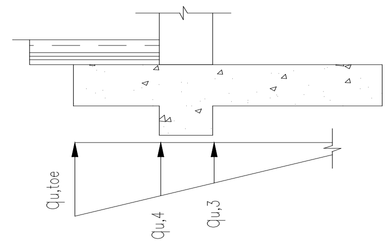
$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t}$ = 0.013

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

$$M_{u,3} = \begin{cases} \frac{L_H}{2}\left(\gamma h_s + w_b \frac{L_H}{L} w_f\right) - \frac{\left(q_{u,3}+2q_{u,heel}\right)bL_H^2}{6}, & for \quad e_u \le \frac{L}{6} \\ \frac{L_H}{2}\left(\gamma h_s + w_b \frac{L_H}{L} w_f\right) - \frac{q_{u,3}bS^2}{6}, & for \quad e_u > \frac{L}{6} \end{cases}$$ = 1.44 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	=	8.25	in	$q_{u,\ toe}$	=	3.08	ksf
	e_u	=	1.32	ft	$q_{u,\ heel}$	=	n/a	ksf
	S	=	-1.40	ft	$q_{u,\ 3}$	=	-3.38	ksf

$(A_{S,\ 3})_{required}$ = 0.11 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)

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

$$M_{u,4} = \frac{\left(q_{u,4} + 2q_{u,toe}\right) b L_T^2}{6} - \frac{L_T^2}{2L} \gamma w_f \quad = \quad 2.63 \text{ ft-kips}$$

where	d	=	6.69	in
	$q_{u,\ 4}$	=	-1.77	ksf

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

$(A_{S,\ 4})_{required}$ = 0.11 in^2 / ft **<** $A_{S,\ 4}$ **[Satisfactory]**

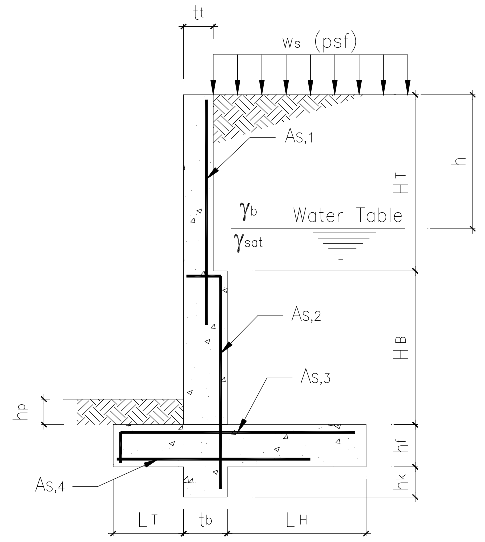
CHECK SLIDING CAPACITY (IBC 1806.1)

1.5 (H_b + H_s) = 1.55 kips **<** $H_p + \mu\ \Sigma W$ = 1.65 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;passive sf=2;friction sf=1.5; no slide

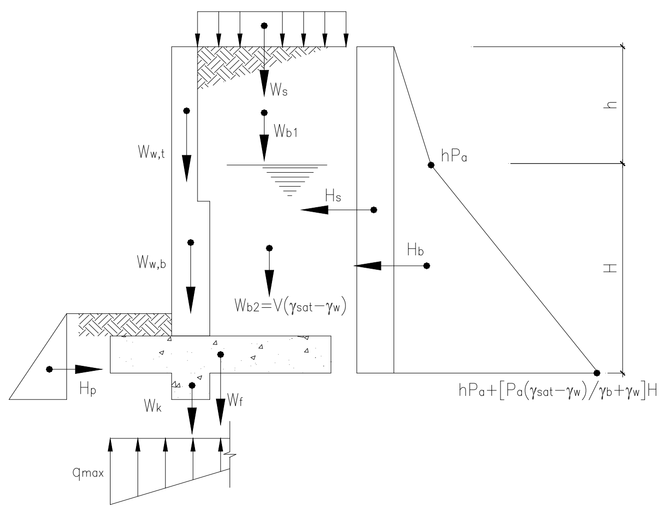
INPUT DATA & DESIGN SUMMARY



CONCRETE STRENGTH	f_c'	=	2.5	ksi		
REBAR YIELD STRESS	f_y	=	60	ksi		
LATERAL SOIL PRESSURE	P_a=$k_a\,\gamma_b$	=	40	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	=	0.8			
ALLOW SOIL PRESSURE	Q_a	=	2	ksf		
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	=	1.67	ft		
HEIGHT OF TOP STEM	H_T	=	8	ft		
HEIGHT OF BOT. STEM	H_B	=	0.01	ft		
FOOTING THICKNESS	h_f	=	10	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)					0	at soil face
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)					0	at soil face
TOP REINF.OF FOOTING ($A_{s,3}$)		#	5	@	16	in o.c.
BOT. REINF.OF FOOTING ($A_{s,4}$)		#	5	@	12	in

[THE WALL DESIGN IS ADEQUATE.]

ANALYSIS



SERVICE LOADS

$H_b = 0.5\,P_a\,h^2 + h\,P_a\,H + 0.5\,[P_a\,(\gamma_{sat} - \gamma_w) / \gamma_b + \gamma_w]\,H^2$ = 1.56 kips

Where h = 8.84 ft , H = 0 ft

$H_s = w_s\,P_a\,(H_T + H_B + h_f) / \gamma_b$ = 0.13 kips

$H_p = 0.5\,P_p\,(h_p + h_f + h_k)^2$ = 0.36 kips

$W_s = w_s\,(L_H + t_b - t_t)$ = 0.07 kips

$W_b = W_{b1} + W_{b2}$ = 1.47 kips

Where W_{b1} = 1.47 kips , W_{b2} = 0.00 kips

$W_f = h_f\,(L_H + t_b + L_T)\,\gamma_c$ = 0.60 kips

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

$W_{w,t} = t_t\,H_T\,\gamma_c$ = 0.80 kips

$W_{w,b} = t_b\,H_B\,\gamma_c$ = 0.00 kips

FACTORED LOADS

$\gamma H_b = 1.6\,H_b$ = 2.50 kips

$\gamma H_s = 1.6\,H_s$ = 0.21 kips

$\gamma W_s = 1.6\,W_s$ = 0.11 kips

$\gamma W_b = 1.2\,W_b$ = 1.77 kips

$\gamma W_f = 1.2\,W_f$ = 0.73 kips

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

$\gamma W_{w,t} = 1.2\,W_{w,t}$ = 0.96 kips

$\gamma W_{w,b} = 1.2\,W_{w,b}$ = 0.00 kips

OVERTURNING MOMENT

	H	γH	y	H y	γH y
H_b	1.56	2.50	2.95	4.61	7.38
H_s	0.13	0.21	4.42	0.57	0.91
Σ	1.69	2.71		**5.18**	8.29

RESISTING MOMENT

	W	γW	x	W x	γW x
W_s	0.07	0.11	4.00	0.27	0.43
W_b	1.47	1.77	4.00	5.89	7.07
W_f	0.60	0.73	2.42	1.46	1.75
W_k	0.00	0.00	2.83	0.00	0.00
$W_{w,t}$	0.80	0.96	2.83	2.27	2.72
$W_{w,b}$	0.00	0.00	2.83	0.00	0.00
Σ	2.94	3.56		**9.89**	11.97

OVERTURNING FACTOR OF SAFETY (1806.1)

$$SF = \frac{\Sigma Wx}{\Sigma Hy} = 1.91 > 1.5$$

[Satisfactory]

(cont'd)

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

$L = L_T + t_b + L_H$ = 4.84 ft

$e = \frac{L}{2} - \frac{\Sigma W x - H y}{\Sigma W}$ = 0.82 ft

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

= 1.23 ksf < Q_a **[Satisfactory]**

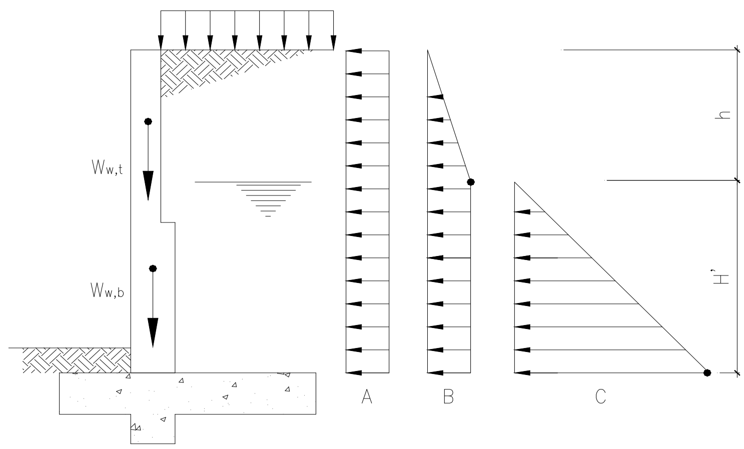
CHECK FLEXURE CAPACITY, AS,1 & AS,2, FOR STEM (ACI 318-05 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

h = 8.01 ft , H' = 0 ft

A = $w_s P_a / \gamma_b$ = 15 plf

B = h P_a = 320 plf

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



At base of top stem

M_u = 6.21 ft-kips

V_u = 2.23 kips

P_u = 0.96 kips

At base of bottom stem

M_u = 6.23 ft-kips

V_u = 2.24 kips

P_u = 0.96 kips

			At top stem		At base of bottom stem	
$\phi M_n = \phi\left[A_s f_y \left(d - \frac{A_s f_y - P_u}{1.7 b f'_c}\right)\right]$		=	8.15	ft-kips ,	8.15	ft-kips
			>	M_u	>	M_u
				[Satisfactory]		**[Satisfactory]**
where	d	=	6.19	in ,	6.19	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	=	0.004		0.004	
$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t}$		=	0.013		0.013	
			>	ρ	>	ρ
				[Satisfactory]		**[Satisfactory]**
$\rho_{MIN} = 0.0018\frac{t}{d}$		=	0.002		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	
$V_{allowable} = 2\phi b d\sqrt{f'_c}$	=	5.57	kips ,	5.57	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

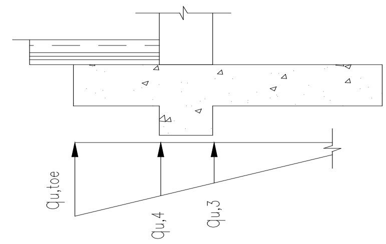
$\rho_{MIN} = \frac{0.0018}{2}\frac{h_f}{d}$ = 0.001

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

= 1.77 ft-kips

(cont'd)

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



where	d	=	8.19	in	$q_{u, toe}$	=	2.29	ksf
	e_u	=	1.38	ft	$q_{u, heel}$	=	n/a	ksf
	S	=	-0.06	ft	$q_{u, 3}$	=	-0.05	ksf

$(A_{S, 3})_{required}$ = 0.11 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)

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

$$M_{u,4} = \frac{(q_{u,4} + 2q_{u,toe}) b L_T^2}{6} \quad \frac{L_T^2}{2L}\gamma w_f \quad = \quad 4.77 \text{ ft-kips}$$

where	d	=	6.69	in
	$q_{u, 4}$	=	0.45	ksf

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

$(A_{S, 4})_{required}$ = 0.16 in^2 / ft **<** $A_{S, 4}$ **[Satisfactory]**

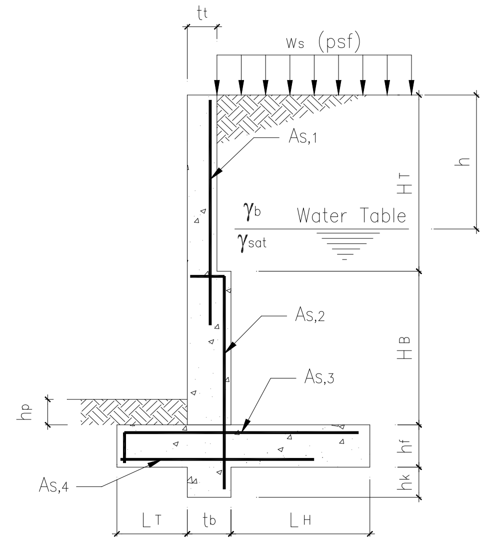
CHECK SLIDING CAPACITY (IBC 1806.1)

1.5 (H_b + H_s) = 2.54 kips **<** $H_p + \mu \Sigma W$ = 2.71 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;passive sf=2;friction sf=1.5; no slide

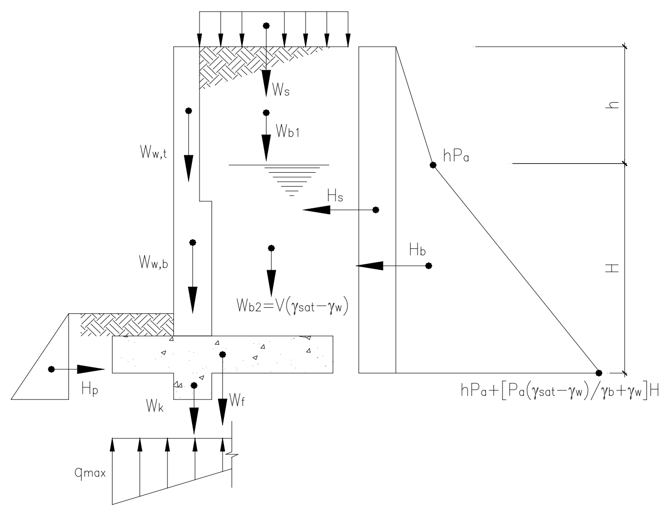
INPUT DATA & DESIGN SUMMARY



CONCRETE STRENGTH	f_c'	=	2.5	ksi		
REBAR YIELD STRESS	f_y	=	60	ksi		
LATERAL SOIL PRESSURE	P_a=k_a γ_b	=	40	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	=	0.8			
ALLOW SOIL PRESSURE	Q_a	=	2	ksf		
THICKNESS OF TOP STEM	t_t	=	8	in		
THICKNESS OF KEY & STEM	t_b	=	8	in		
TOE WIDTH	L_T	=	3.5	ft		
HEEL WIDTH	L_H	=	2	ft		
HEIGHT OF TOP STEM	H_T	=	10	ft		
HEIGHT OF BOT. STEM	H_B	=	0.01	ft		
FOOTING THICKNESS	h_f	=	14	in		
KEY DEPTH	h_k	=	0	in		
SOIL OVER TOE	h_p	=	6	in		
TOP STEM REINF. ($A_{s,1}$)		#	5	@	7	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. ($A_{s,2}$)		#	5	@	7	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

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

= 2.50 kips

Where h = 11.2 ft , H = 0 ft

$H_s = w_s\, P_a\, (H_T + H_B + h_f) / \gamma_b$ = 0.16 kips

$H_p = 0.5\, P_p\, (h_p + h_f + h_k)^2$ = 0.56 kips

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

$W_b = W_{b1} + W_{b2}$ = 2.20 kips

Where W_{b1} = 2.2 kips , W_{b2} = 0.00 kips

$W_f = h_f\, (L_H + t_b + L_T)\, \gamma_c$ = 1.08 kips

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

$W_{w,t} = t_t\, H_T\, \gamma_c$ = 1.00 kips

$W_{w,b} = t_b\, H_B\, \gamma_c$ = 0.00 kips

FACTORED LOADS

$\gamma H_b = 1.6\, H_b$ = 4.00 kips

$\gamma H_s = 1.6\, H_s$ = 0.26 kips

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

$\gamma W_b = 1.2\, W_b$ = 2.64 kips

$\gamma W_f = 1.2\, W_f$ = 1.30 kips

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

$\gamma W_{w,t} = 1.2\, W_{w,t}$ = 1.20 kips

$\gamma W_{w,b} = 1.2\, W_{w,b}$ = 0.00 kips

OVERTURNING MOMENT

	H	γH	y	H y	γH y
H_b	2.50	4.00	3.73	9.31	14.89
H_s	0.16	0.26	5.59	0.91	1.45
Σ	2.66	4.26		**10.2**	16.35

RESISTING MOMENT

	W	γW	x	W x	γW x
W_s	0.08	0.13	5.17	0.41	0.66
W_b	2.20	2.64	5.17	11.38	13.65
W_f	1.08	1.30	3.08	3.33	3.99
W_k	0.00	0.00	3.83	0.00	0.00
$W_{w,t}$	1.00	1.20	3.83	3.83	4.60
$W_{w,b}$	0.00	0.00	3.83	0.00	0.00
Σ	4.36	5.27		**18.96**	22.91

OVERTURNING FACTOR OF SAFETY (1806.1)

$$SF = \frac{\Sigma Wx}{\Sigma Hy} = 1.86 > 1.5$$

[Satisfactory]

(cont'd)

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

$L = L_T + t_b + L_H$ = 6.17 ft

$e = \frac{L}{2} - \frac{\Sigma W x - H y}{\Sigma W}$ = 1.08 ft

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

= 1.45 ksf < Q_a **[Satisfactory]**

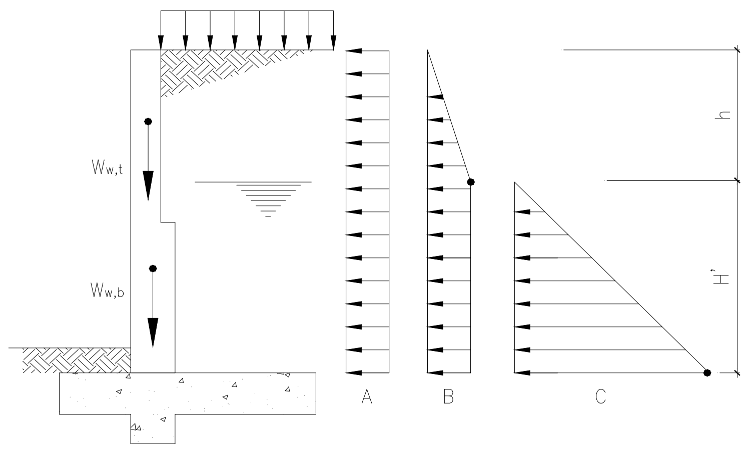
CHECK FLEXURE CAPACITY, AS,1 & AS,2, FOR STEM (ACI 318-05 SEC.15.4.2, 10.2, 10.5.4, 7.12.2, 12.2, & 12.5)

h = 10.01 ft , H' = 0 ft

A = $w_s P_a / \gamma_b$ = 15 plf

B = $h P_a$ = 400 plf

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



At base of top stem

M_u = 11.83 ft-kips

V_u = 3.43 kips

P_u = 1.20 kips

At base of bottom stem

M_u = 11.86 ft-kips

V_u = 3.44 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		=	13.36	ft-kips ,	13.36	ft-kips
			>	M_u	>	M_u
				[Satisfactory]		**[Satisfactory]**
where	d	=	6.19	in ,	6.19	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	=	0.53143	in^2 ,	0.531429	in^2
	ρ	=	0.007		0.007	
$\rho_{MAX} = \frac{0.85\beta_1 f'_c}{f_y}\frac{\varepsilon_u}{\varepsilon_u+\varepsilon_t}$		=	0.013		0.013	
			>	ρ	>	ρ
				[Satisfactory]		**[Satisfactory]**
$\rho_{MIN} = 0.0018\frac{t}{d}$		=	0.002		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	
$V_{allowable} = 2\phi b d\sqrt{f'_c}$	=	5.57	kips ,	5.57	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

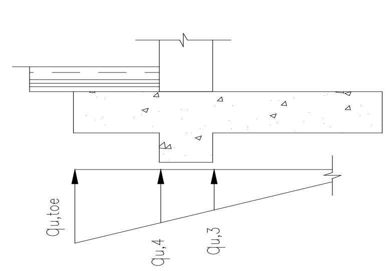
$\rho_{MIN} = \frac{0.0018}{2}\frac{h_f}{d}$ = 0.001

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

= 3.20 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	=	12.19 in	$q_{u, toe}$	=	2.82	ksf
	e_u	=	1.84 ft	$q_{u, heel}$	=	n/a	ksf
	S	=	-0.43 ft	$q_{u, 3}$	=	-0.32	ksf

$(A_{S,3})_{required}$ = 0.15 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)

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

$$M_{u,4} = \frac{\left(q_{u,4} + 2 q_{u,toe} \right) b L_T^2}{6} \quad \frac{L_T^2}{2L} \gamma w_f \quad = \quad 10.58 \text{ ft-kips}$$

where d = 10.69 in

$q_{u,4}$ = 0.18 ksf

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

$(A_{S,4})_{required}$ = 0.23 in^2 / ft **<** $A_{S,4}$ **[Satisfactory]**

CHECK SLIDING CAPACITY (IBC 1806.1)

1.5 (H_b + H_s) = 3.99 kips **<** $H_p + \mu \Sigma W$ = 4.05 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

Floor areas (sqft)		
2nd	2100	
roof	1900	
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)	**52.50**	**kips**
roof		
seismic mass (area x roof framing seismic mass)	**47.50**	**kips**

Seismic Forces

(per attached spreadsheet calculations)

			NS		EW	
roof	10.10	kips				
2nd	5.00					
total	15.10	kips				
			Cumulative		Cumulative	
ASD = Seismic Force/1.4						
roof	7.21		**7.21**	**kips**	**7.21**	**kips**
2nd	3.57		10.79	kips	10.79	kips
total	10.79	kips				

Wind Forces

(per attached spreadsheet calculations)

			NS		EW	
NS	15.65	kips	1.30			
EW	20.30	kips				
NS			Cumulative			
roof = ((3'+9'/2)/23') x 15.65 kips	5.10		5.10	kips		
2nd = (((9'+11')/2)/23') x 15.65 kips	6.80		**11.90**	**kips**		
total	11.90	kips				
EW					Cumulative	
roof = ((3'+9'/2)/23') x 20.30 kips	6.62				6.62	kips
2nd = (((9'+11')/2)/23') x 20.30 kips	8.82				**15.44**	**kips**
total	15.44	kips				

Lateral Force Summary (ASD)

	NS		EW	
	Cumulative		Cumulative	
SEISMIC/SEISMIC	**7.21**	**kips**	**7.21**	**kips**
WIND/WIND	**11.90**	**kips**	**15.44**	**kips**

INPUT DATA

Typical floor height	h =	**11.0**	ft	
Typical floor weight	w_x =	**48**	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 = 20.0 ft   k = 1.00 , (ASCE 12.8.3, pg 130)   x = 0.75 , (ASCE Tab 12.8-2)

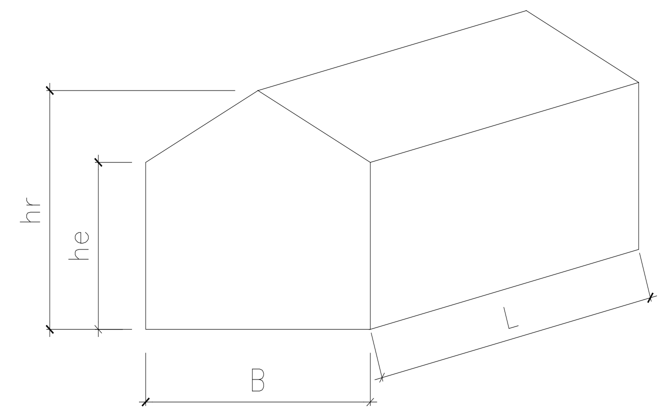
W = 100 k $\Sigma w_x h^k$ = 1,573 $T_a = C_t (h_n)^x$ = 0.19 Sec, (ASCE 12.8.2.1)

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		20.0	53	1,050	0.668	10.1			10.1	53	10
		9.00						10.1				
1	2nd		11.0	48	523	0.332	5.0		91	15.1	100	9
		11.00						15.1				
	Ground		0.0						257			

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
Topographic factor (Sec.6.5.7.2, pg 26 & 45)	K_{zt} =	2	
Building height to eave	h_e =	23	ft
Building height to ridge	h_r =	23	ft
Building length	L =	54	ft
Building width	B =	40.67	ft
Effective area of components	A =	10	ft^2



DESIGN SUMMARY

Max horizontal force normal to building length, L, face	=	20.30 kips
Max horizontal force normal to building length, B, face	=	15.65 kips
Max total horizontal torsional load	=	141.94 ft-kips
Max total upward force	=	36.93 kips

ANALYSIS

Velocity pressure

$q_h = 0.00256\, K_h\, K_{zt}\, K_d\, V^2\, I$ = **22.01** 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 = **23.00** 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**

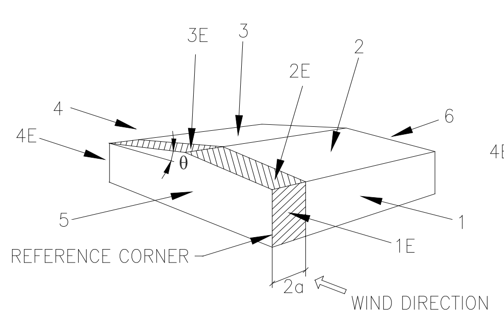
a = width of edge strips, Fig 6-10, note 9, page 54, MAX[MIN(0.1B, 0.4h), 0.04B,3] = **4.07** ft

Net Pressures (psf), Basic Load Cases

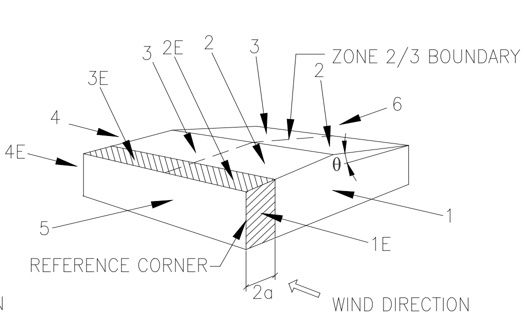
Surface	Roof angle θ = 0.00			Roof angle θ = 0.00		
	$G\,C_{p\,f}$	Net Pressure with		$G\,C_{p\,f}$	Net Pressure with	
		$(+GC_{p\,i})$	$(-GC_{p\,i})$		$(+GC_{p\,i})$	$(-GC_{p\,i})$
1	0.40	4.84	12.77	0.40	4.84	12.77
2	-0.69	-19.15	-11.23	-0.69	-19.15	-11.23
3	-0.37	-12.11	-4.18	-0.37	-12.11	-4.18
4	-0.29	-10.34	-2.42	-0.29	-10.34	-2.42
1E	0.61	9.46	17.39	0.61	9.46	17.39
2E	-1.07	-27.51	-19.59	-1.07	-27.51	-19.59
3E	-0.53	-15.63	-7.70	-0.53	-15.63	-7.70
4E	-0.43	-13.43	-5.50	-0.43	-13.43	-5.50
5	-0.45	-13.87	-5.94	-0.45	-13.87	-5.94
6	-0.45	-13.87	-5.94	-0.45	-13.87	-5.94

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	1.21	3.19
2T	-0.69	-4.79	-2.81
3T	-0.37	-3.03	-1.05
4T	-0.29	-2.59	-0.61
Surface	Roof angle θ = 0.00		
	$G\,C_{p\,f}$	Net Pressure with	
		$(+GC_{p\,i})$	$(-GC_{p\,i})$
1T	0.40	1.21	3.19
2T	-0.69	-4.79	-2.81
3T	-0.37	-3.03	-1.05
4T	-0.29	-2.59	-0.61

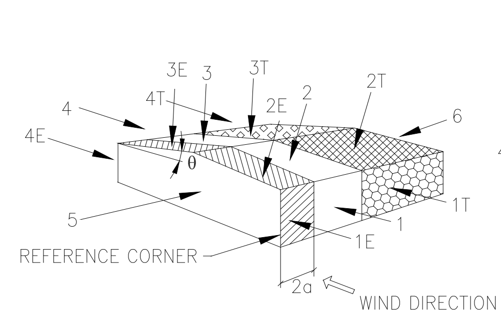


Transverse Direction

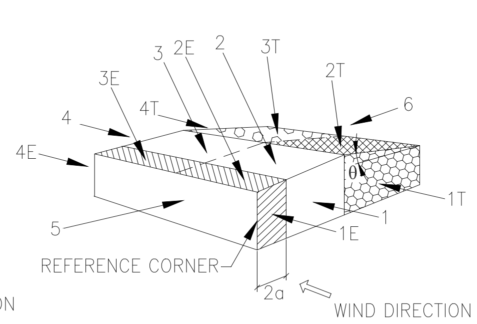


Longitudinal Direction

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	1055	5.11	13.47
2	933	-17.86	-10.47
3	933	-11.29	-3.90
4	1055	-10.91	-2.55
1E	187	1.77	3.25
2E	165	-4.55	-3.24
3E	165	-2.58	-1.27
4E	187	-2.51	-1.03
Σ	Horiz.	20.30	20.30
	Vert.	-36.29	-18.88
10 psf min.	Horiz.	12.42	12.42
Sec. 6.1.4.1	Vert.	-21.96	-21.96

Basic Load Cases in Longitudinal Direction

Surface	Area (ft^2)	Pressure (k) with (+GC_{pi})	Pressure (k) with (-GC_{pi})
1	748	3.62	9.55
2	878	-16.82	-9.86
3	878	-10.63	-3.67
4	748	-7.74	-1.81
1E	187	1.77	3.25
2E	220	-6.04	-4.30
3E	220	-3.43	-1.69
4E	187	-2.51	-1.03
Σ	Horiz.	15.65	15.65
	Vert.	-36.93	-19.53
10 psf min.	Horiz.	9.35	9.35
Sec. 6.1.4.1	Vert.	-21.96	-21.96

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	434	2.10	5.54	24	64
2	384	-7.35	-4.31	0	0
3	384	-4.64	-1.60	0	0
4	434	-4.49	-1.05	51	12
1E	187	1.77	3.25	41	75
2E	165	-4.55	-3.24	0	0
3E	165	-2.58	-1.27	0	0
4E	187	-2.51	-1.03	58	24
1T	621	0.75	1.98	-10	-27
2T	549	-2.63	-1.54	0	0
3T	549	-1.66	-0.57	0	0
4T	621	-1.61	-0.38	-22	-5
Total Horiz. Torsional Load, M_T				142	142

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	281	1.36	3.58	8	22
2	659	-12.62	-7.40	0	0
3	659	-7.98	-2.76	0	0
4	281	-2.90	-0.68	18	4
1E	187	1.77	3.25	29	53
2E	220	-6.04	-4.30	0	0
3E	220	-3.43	-1.69	0	0
4E	187	-2.51	-1.03	41	17
1T	468	0.57	1.49	-6	-15
2T	878	-4.21	-2.47	0	0
3T	878	-2.66	-0.92	0	0
4T	468	-1.21	-0.28	-12	-3
Total Horiz. Torsional Load, M_T				77.6	77.6

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

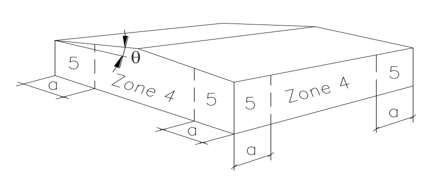
$p = q_h[(G C_p) - (G C_{pi})]$

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

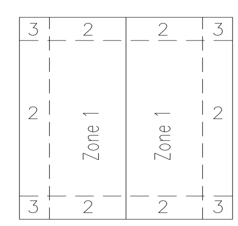
p_{min} = 10 psf (Sec. 6.1.4.2, pg 21)

$G C_p$ = external pressure coefficient.

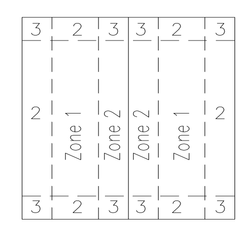
see table below. (Fig. 6-11, page 55~58)



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
	10.56	-25.97	10.56	-43.58	10.56	-65.59	23.77	-25.75	23.77	-31.69

EW Shear Wall Design

shear wall location:	NORTH	roof diaphragm	2nd flr diaphragm
shear force (kips)		2.70	5.79
floor height (ft)		9.00	11.00
wall length without opening (ft)		29.00	16.00
wall length with opening (ft)		29.00	10.00
wall segment length (ft)		29.00	16.00
shear flow (plf)		93.23	347.40
shear wall type per schedule on GN		**SW6**	**SW4 W/40% TO CONC**
dead loads from floor/roof framing (plf)		30.00	150.00
wall weight (plf)		90.00	110.00
hold down force (kips) with 0.6DL		-0.20	2.57
hold down type per schedule on GN		**NO HD**	**4**
shear wall location:	MIDDLE	roof diaphragm	2nd flr diaphragm
shear force (kips)		3.61	7.72
floor height (ft)		9.00	11.00
wall length without opening (ft)		16.00	7.00
wall length with opening (ft)		9.00	7.00
wall segment length (ft)		16.00	7.00
shear flow (plf)		400.56	1102.86
shear wall type per schedule on GN		**SW4**	**CONC/DRAG STRUT**
dead loads from floor/roof framing (plf)		30.00	60.00
wall weight (plf)		90.00	110.00
hold down force (kips) with 0.6DL		3.03	11.77
hold down type per schedule on GN		**4**	**NA**
shear wall location:	SOUTH	roof diaphragm	2nd flr diaphragm
shear force (kips)		0.90	1.93
floor height (ft)		9.00	11.00
wall length without opening (ft)		16.00	7.00
wall length with opening (ft)		13.00	7.00
wall segment length (ft)		16.00	7.00
shear flow (plf)		69.33	275.71
shear wall type per schedule on GN		**SW6**	**SW6**
dead loads from floor/roof framing (plf)		30.00	150.00
wall weight (plf)		90.00	110.00
hold down force (kips) with 0.6DL		0.05	2.53
hold down type per schedule on GN		**NO HD**	**4**

NS Shear Wall Design

shear wall location:	WEST	roof diaphragm	2nd flr diaphragm
shear force (kips)		1.98	3.27
floor height (ft)		9.00	11.00
wall length without opening (ft)		16.00	17.00
wall length with opening (ft)		8.00	17.00
wall segment length (ft)		16.00	8.50
shear flow (plf)		247.84	192.50
shear wall type per schedule on GN		SW6	SW6
dead loads from floor/roof framing (plf)		187.50	30.00
wall weight (plf)		90.00	110.00
hold down force (kips) with 0.6DL		0.90	1.76
hold down type per schedule on GN		A	2

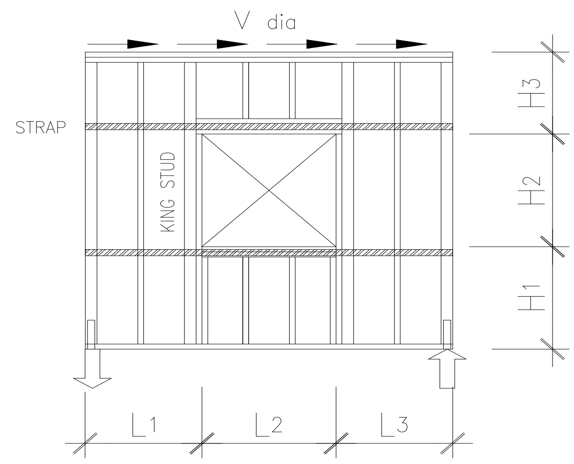
shear wall location:	MIDDLE	roof diaphragm	2nd flr diaphragm
shear force (kips)		3.61	5.95
floor height (ft)		9.00	11.00
wall length without opening (ft)		12.00	12.00
wall length with opening (ft)		12.00	12.00
wall segment length (ft)		12.00	12.00
shear flow (plf)		300.42	495.83
shear wall type per schedule on GN		SW6	CONC/DRAG STRUT
dead loads from floor/roof framing (plf)		142.50	60.00
wall weight (plf)		90.00	110.00
hold down force (kips) with 0.6DL		1.87	6.71
hold down type per schedule on GN		2	NA

shear wall location:	EAST	roof diaphragm	2nd flr diaphragm
shear force (kips)		1.62	2.68
floor height (ft)		9.00	11.00
wall length without opening (ft)		11.00	30.00
wall length with opening (ft)		11.00	30.00
wall segment length (ft)		11.00	30.00
shear flow (plf)		147.48	89.25
shear wall type per schedule on GN		SW6	CONC
dead loads from floor/roof framing (plf)		135.00	30.00
wall weight (plf)		90.00	110.00
hold down force (kips) with 0.6DL		0.58	0.31
hold down type per schedule on GN		2	NA

Wood Shear Wall with an Opening Based on NDS

window width = total wall pier length = 340/2=170 plf

INPUT DATA

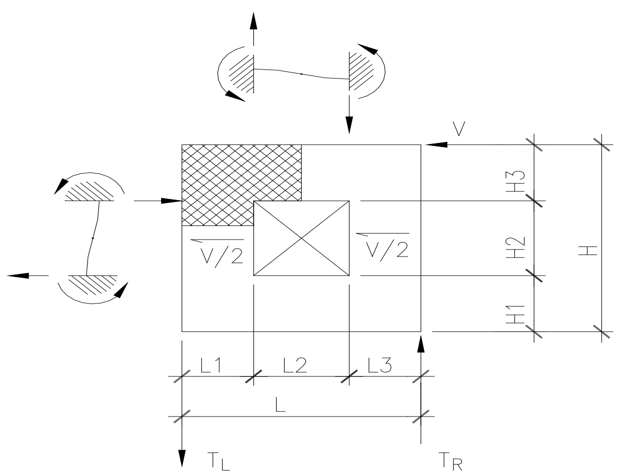


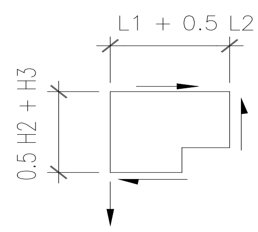
LATERAL FORCE ON DIAPHRAGM:		$v_{dia,\ WIND}$ =	170	plf,for wind			
(SERVICE LOADS)		$v_{dia,\ SEISMIC}$ =	170	plf,for seismic			
DIMENSIONS:	L_1 =	3	ft , L_2 =	6	ft , L_3 =	3	ft
	H_1 =	2	ft , H_2 =	5	ft , H_3 =	2	ft
KING STUD SECTION		1	pcs, b =	2	in , h =	6	in
EDGE STUD SECTION		2	pcs, b =	2	in , h =	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

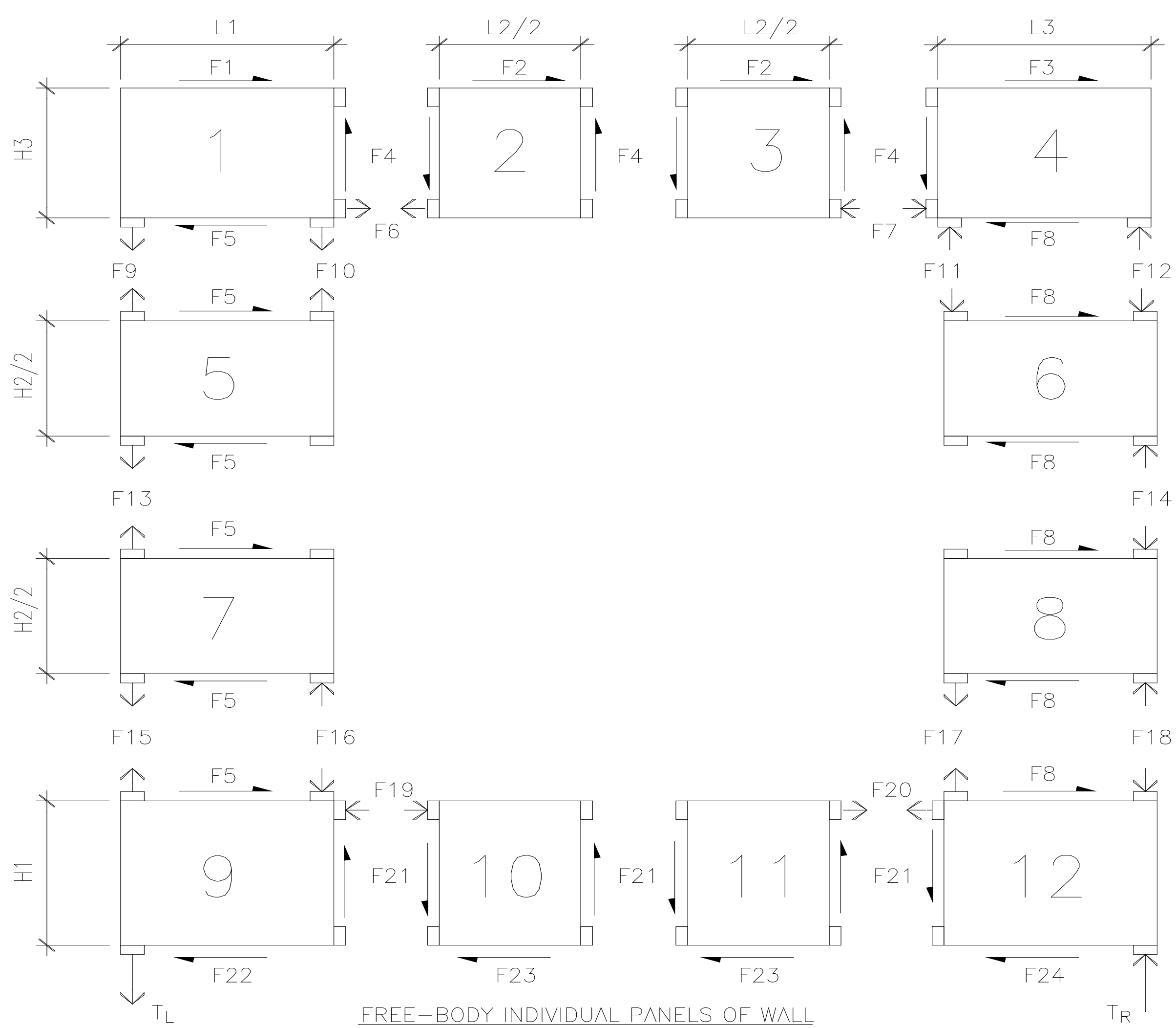
BLOCKED 15/32 SHEATHING WITH 10d COMMON NAILS
@ 4 in O.C. BOUNDARY & ALL EDGES / 12 in O.C. FIELD,
SILL PLATE ATTACHMENT 16d AT 6" O.C.

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, L_w = 6 ft, E = 1.7E+06 psi
A = 16.50 in², 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} = 0.85 kips

F_c = 1500 psi, C_D = 1.60, C_P = 0.43, A = 8.25 in²

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 in²

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.