canyons
STRUCTURAL

PROFESSIONAL ENGINEER
06/24/2024
ALEXSYS A. FLOCKEN
No. 10141589
STATE OF UTAH

STRUCTURAL CALCULATIONS

Snowbasin Resort

Legend Well House

April 2024

3925 East Snowbasin Road

Huntsville, UT

CLIENT

Talisman Civil Consultants

1245 Brickyard Rd #200
Salt Lake City, UT 84106
p. 801.486.6848
www.canyonsstructural.com

Engineering Criteria

PROJECT INFORMATION:

Project Name	Snowbasin Resort – Legend Well House
Project Location:	3925 East Snowbasin Rd Huntsville, UT
Project Designed:	Talisman Civil Consultants

DESIGN CRITERIA:

Governing Code:	IBC 2021 (ASCE 7-16)
Type of Construction:	Ordinary Reinforced Concrete Shear wall
Wind Speed & Exposure:	103 Mph 3sec gust - C exposure
Seismic Site Class:	D (determined by geotechnical engineer)
Seismic Design Category:	D (seismic controls over wind everywhere)
Snow (ground):	386 psf
Snow (roof):	270 psf, warm roof with snow retention 324 psf cold, roof with snow retention elevation = 8643 ft; Ce=1.0; I=1.0; Ct=1.0 & Ct=1.2

CONSTRUCTION MATERIALS:

Concrete at 28 days	Footings and walls are designed for 4000 psi
Reinforcing Grade:	60 ksi

SOILS CRITERIA:

Bearing Pressure	3000 psf (per GSH soils report #3572-003-23, dated 12/07/23)
Min. Frost Depth	4’-0”
Min. Footing Width	24”

Utah Ground Snow Load Map

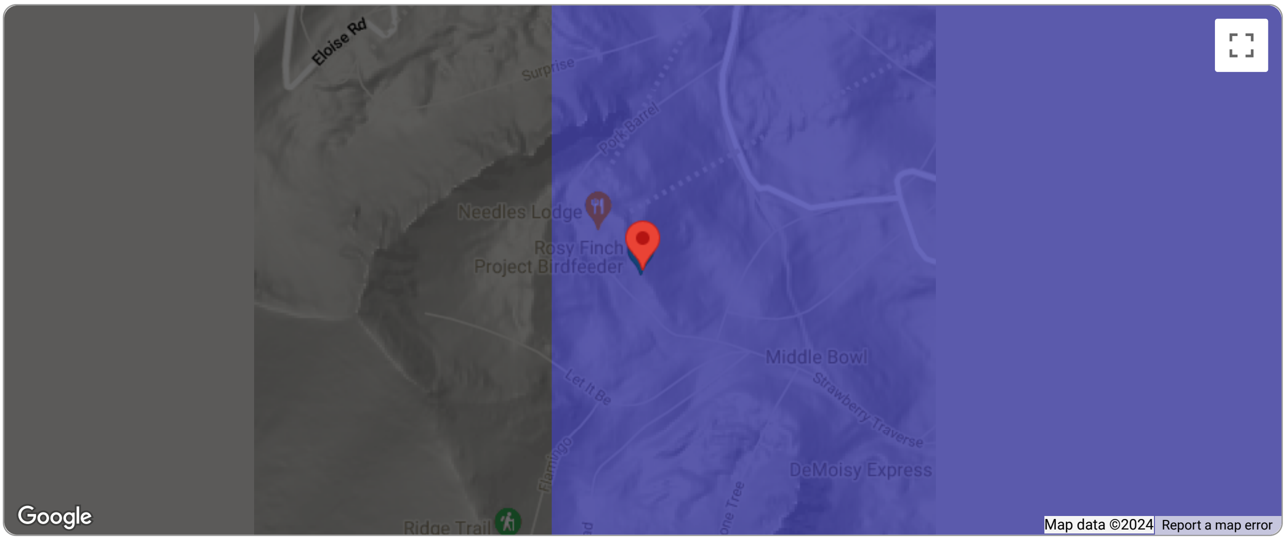
Snowbasin Legend Well house

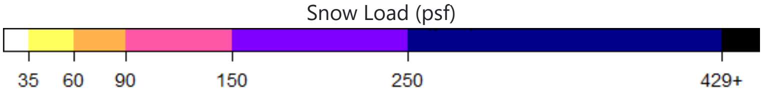
Latitude: 41.193
Longitude: -111.873
Elevation: 8,643 ft

Ground Snow Load:
386 psf / 18.54 kPa

Warning: Region not covered by study. Value given may be inaccurate. Site-specific study recommended.

*This document is not legally binding. The user is urged to verify ground snow load values with the local authority having jurisdiction.





These ground snow load values represent 50-year ground snow load estimated value at a 2% probability of exceedance for the location given. The grid used in the map is 3350ft by 3350ft. Elevations for these grid cells were estimated by aggregating data from 100ft by 100ft USGS digital elevation models and may not coincide with the actual site elevation. These predictions are calculated using the process outlined in The Utah Snow Load Study.[1]

Final predictions given are bounded at a lower limit for a minimum ground snow load of 21 psf to meet ASCE 7. Estimated values for snow loads at elevations significantly higher than all nearby stations lead to unreasonably high snow load estimates, therefore, the predictions in the map are not allowed to extend beyond the highest 50-year station ground snow load of 429 psf. Elevations over 9,000 ft are also considered less accurate due to the limited number of stations at these elevations. The results shown in this report have included a warning if the results have reached or exceeded the upper limit.

While great efforts have been made to ensure these predictions are as accurate as possible, designers must use expert judgement to ensure that such predictions are appropriate for their particular project. The SEAU and the authors cannot accept responsibility for prediction errors or any consequences resulting therefrom.

1 Bean, Brennan; Maguire, Marc; and Sun, Yan, "The Utah Snow Load Study" (2018). Civil and Environmental Engineering Faculty Publications. Paper 3589.

GRAVITY LOADS - roof w/retention, warm

Roof Snow Load:

Ground Snow Load $P_g := 386 \cdot psf$

$rise := 0 \cdot in$ $run := 12 \cdot in$

Roof temp (ASCE 7-05, t7-3) $Ct := 1.0$

Ct 1.0 if warm
1.1 if cold vented
1.2 if cold unvented

Roofing is "metal" or "other" $surface := other$

Snow Exposure Factor $C_e := 1.0$

Importance Factor $I := 1.0$

Roof Snow (ASCE-7) $P_f := 0.7 C_e \cdot Ct \cdot I \cdot P_g$

$$C_s := \left| \begin{array}{l} \text{if } surface = other \\ \quad \left| \begin{array}{l} a \leftarrow \left| \begin{array}{ll} (ang1 \cdot deg) & \text{if } pitch < ang1 \cdot deg \\ (70 \cdot deg) & \text{if } pitch > 70 \cdot deg \\ pitch & \text{otherwise} \end{array} \right. \\ \dfrac{70 \cdot deg - a}{ang2 \cdot deg} \\ break \end{array} \right. \\ a \leftarrow if(pitch > 70 \cdot deg, 70 \cdot deg, pitch) \\ \dfrac{65 \cdot deg - a}{ang3 \cdot deg} \quad \text{if } surface = metal \end{array} \right.$$

$ang1 = 30$

$ang2 = 40$

$ang3 = 65$

$P_g = 386\,psf$

$P_f = 270\,psf$

$C_s = 1$

$P_s := if(C_s \cdot P_f \le 30 \cdot psf, 30 \cdot psf, C_s \cdot P_f)$

$P_s = 270.2\,psf$

WARM ROOF Snow is $P_s = 270\,psf$

FLOOR LL is 40 psf

ROOF DL's (concrete Belgard pavers)

Roofing ----------- $R_{rf} := 5 \cdot psf$

Sheathing--------- $S_{rf} := 2.2 \cdot psf$

Insulation--------- $I_{rf} := 0.5 \cdot psf$

Fixtures------------ $E_{rf} := 0.5 \cdot psf$

Ceiling------------- $C_{rf} := 2.2 \cdot psf$

Rafters------------ $SP_{rf} := 2.5 \cdot psf$

Beams-------------- $P_{rf} := 2.0 \cdot psf$

TOTAL ROOF DL

$TL_{rf} := R_{rf} + S_{rf} + I_{rf} + E_{rf} + C_{rf} + SP_{rf} + P_{rf}$

$TL_{rf} = 14.9\,psf$

Use a pavered roof DL of 15 psf

GRAVITY LOADS - roof w/retention, cold

Roof Snow Load:

Ground Snow Load $P_g := 386 \cdot psf$

$rise := 0 \cdot in$ $run := 12 \cdot in$

Roof temp (ASCE 7-05, t7-3) $Ct := 1.2$

Ct 1.0 if warm
1.1 if cold vented
1.2 if cold unvented

Roofing is "metal" or "other" $surface := other$

Snow Exposure Factor $C_e := 1.0$

Importance Factor $I := 1.0$

Roof Snow (ASCE-7) $P_f := 0.7C_e \cdot Ct \cdot I \cdot P_g$

$$C_s := \left| \begin{array}{l} \text{if } surface = other \\ \quad \left| \begin{array}{l} a \leftarrow \left| \begin{array}{ll} (ang1 \cdot deg) & \text{if } pitch < ang1 \cdot deg \\ (70 \cdot deg) & \text{if } pitch > 70 \cdot deg \\ pitch & \text{otherwise} \end{array} \right. \\ \dfrac{70 \cdot deg - a}{ang2 \cdot deg} \\ break \end{array} \right. \\ a \leftarrow if(pitch > 70 \cdot deg, 70 \cdot deg, pitch) \\ \dfrac{65 \cdot deg - a}{ang3 \cdot deg} \quad \text{if } surface = metal \end{array} \right.$$

$ang1 = 45$

$ang2 = 25$

$ang3 = 55$

$P_g = 386\,psf$

$P_f = 324\,psf$

$C_s = 1$

$P_s := if(C_s \cdot P_f \le 30 \cdot psf, 30 \cdot psf, C_s \cdot P_f)$

$P_s = 324.24\,psf$

Cold Unvented Snow is $P_s = 324\,psf$

Strip Footing - FC2.0-(Basement walls)

Design properties

Roof

$roof_{trib} := 7 \cdot ft$

$Roof_{DL} := 120 \cdot psf$

$Roof_{LL} := 324 psf$

Floor

$floor_{trib} := 6 \cdot ft$

$Floor_{DL} := 25 \cdot psf$

$Floor_{LL} := 40 \cdot psf$

$Wall_{DL} := 97 \cdot psf$

wall_height := 10·ft

no_st := 1 Number of stories

no_fl := 0 Number of suspended floors

Foundation

fndn_height := 10·ft

Fndn_thick := 10·in

ftng_width := 24·in

Soil_bearing := 3000·psf

Roof Loads

Roof = 3108 plf

Floor Loads

Floors = 970 plf

Foundation Loads

Fndn = 1208 plf

p = 5286 plf Total pressure applied to soil @ underside of footing

w = 21.1 in Required footing width

ftng_width = 24 in **Footing width provided ADEQUATE**

This is a beta release of the new ATC Hazards by Location website. Please contact us with feedback.

The ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

ATC Hazards by Location

Search Information

Address: 3925 E Snow Basin Rd, Huntsville, UT 84317, USA

Coordinates: 41.1934347, -111.8734869

Elevation: 8710 ft

Timestamp: 2024-01-30T17:03:02.055Z

Hazard Type: Wind

ASCE 7-16

MRI 10-Year	74 mph
MRI 25-Year	80 mph
MRI 50-Year	84 mph
MRI 100-Year	89 mph
Risk Category I	97 mph
Risk Category II	103 mph
Risk Category III	109 mph
Risk Category IV	113 mph

ASCE 7-10

MRI 10-Year	76 mph
MRI 25-Year	84 mph
MRI 50-Year	90 mph
MRI 100-Year	96 mph
Risk Category I	105 mph
Risk Category II	115 mph
Risk Category III-IV	120 mph

ASCE 7-05

ASCE 7-05 Wind Speed	90 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.

Please note that the ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

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.

This is a beta release of the new ATC Hazards by Location website. Please contact us with feedback.

The ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

ATC Hazards by Location

Search Information

Address: 3925 E Snow Basin Rd, Huntsville, UT 84317, USA

Coordinates: 41.1934347, -111.8734869

Elevation: 8710 ft

Timestamp: 2024-01-30T17:04:13.428Z

Hazard Type: Seismic

Reference Document: ASCE7-16

Risk Category: II

Site Class: D-default

Basic Parameters

Name	Value	Description
S_S	1.061	MCE_R ground motion (period=0.2s)
S_1	0.388	MCE_R ground motion (period=1.0s)
S_{MS}	1.273	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	0.849	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	0.872	Coefficient of risk (0.2s)
CR_1	0.88	Coefficient of risk (1.0s)
PGA	0.478	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.574	Site modified peak ground acceleration
T_L	8	Long-period transition period (s)
SsRT	1.061	Probabilistic risk-targeted ground motion (0.2s)
SsUH	1.216	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.908	Factored deterministic acceleration value (0.2s)
S1RT	0.388	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.441	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.84	Factored deterministic acceleration value (1.0s)
PGAd	0.749	Factored deterministic acceleration value (PGA)

* See Section 11.4.8

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.

Seismic Loads: ASCE 7- 16 — Strength Level Forces

Risk Category : II
Importance Factor (I) : 1.00
Site Class :) - code default

Ss (0.2 sec) = 106.10 %g
S1 (1.0 sec) = 38.80 %g

Fa =	1.200	Sms =	1.273	S_{DS} =	0.849	Design Category =	D
Fv =	1.912	Sm1 =	0.742	S_{D1} =	0.495	Design Category =	D

Seismic Design Category = **D**

Redundancy Coefficient ρ = 1.00 Code exception must be met for p to equal 1.0
Number of Stories: 1
Structure Type: Light Frame
Horizontal Struct Irregularities: No plan Irregularity
Vertical Structural Irregularities: No vertical Irregularity
Flexible Diaphragms: Yes
Building System: **Bearing Wall Systems**
Seismic resisting system: **Ordinary reinforced concrete shear walls**
System Structural Height Limit: **System not permitted for this seismic design category**
Actual Structural Height (hn) = 12.0 ft
See ASCE7 Section 12.2.5 for exceptions and other system limitations

DESIGN COEFFICIENTS AND FACTORS

Response Modification Coefficient (R) = 4
Over-Strength Factor (Ωo) = 2
Deflection Amplification Factor (Cd) = 4
S_{DS} = 0.849
S_{D1} = 0.495

Seismic Load Effect (E) = Eh +/-Ev = $\rho\, Q_E$ +/- $0.2S_{DS}$ D = Qe +/- 0.170D Q_E = horizontal seismic force
Special Seismic Load Effect (Em) = Emh +/- Ev = $\Omega o\, Q_E$ +/- $0.2S_{DS}$ D = 2Qe +/- 0.170D D = dead load

PERMITTED ANALYTICAL PROCEDURES

Simplified Analysis - Use Equivalent Lateral Force Analysis

Equivalent Lateral-Force Analysis - Permitted

Building period coef. (C_T) =	0.020			Cu =	1.40
Approx fundamental period (Ta) =	$C_T h_n$^ =	0.129 sec	x= 0.75	Tmax = CuTa =	0.181
User calculated fundamental period (T) =		sec		Use T =	0.129
Long Period Transition Period (TL) =	ASCE7 map =	8			
Seismic response coef. (Cs) =	S_{DS}I/R =	0.212			
need not exceed Cs =	Sd1 I /RT =	0.959			
but not less than Cs =	0.044SdsI =	0.037			
USE Cs =		0.212			

Design Base Shear V = 0.212W

Model & Seismic Response Analysis - Permitted (see code for procedure)

ALLOWABLE STORY DRIFT

Structure Type: All other structures

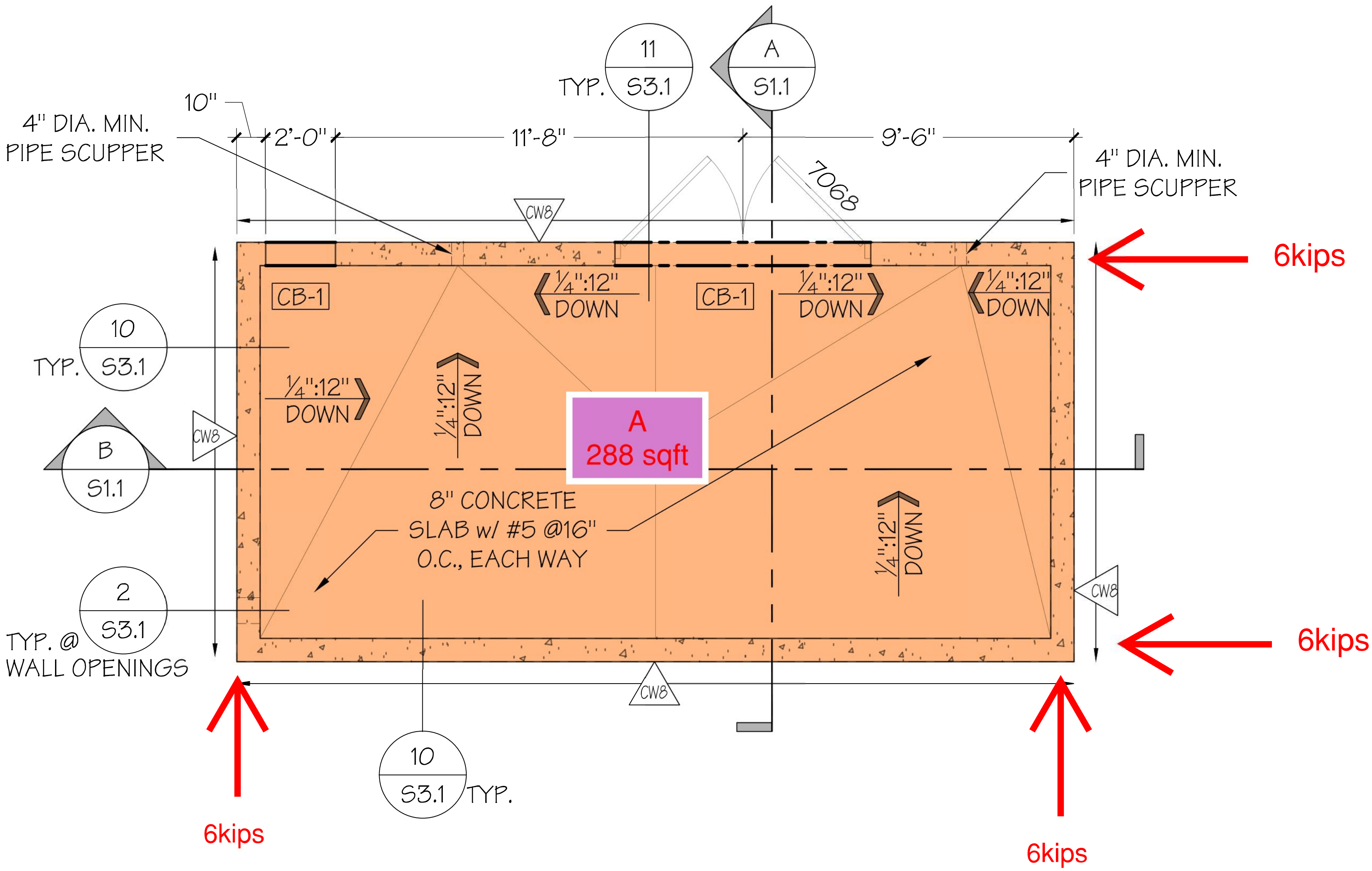
Allowable story drift Δa = 0.020hsx where hsx is the story height below level x

SEISMIC SNOW LOAD warm roof & rooftop decks

Snow Load to include in lateral design (Utah Snow Study Amendment to the Code)

$elevation := 8643$

$Roof_{snowL} := 325 \cdot psf$ $\quad Roof_{LL} := if\left[Roof_{snowL} > 30 \cdot psf, \left[Roof_{snowL} \cdot \left[0.20 + 0.025 \cdot \left(\frac{elevation - 5000}{1000}\right)\right]\right], 0 \cdot psf\right]$ $\quad Roof_{LL} = 95\, psf$



2
S1.1

ROOF FRAMING PLAN

SCALE: 1/4" = 1'-0"

Laitala-Williams

Horizontal Seismic Force Distribution

by Cambria Flowers PE, SE

V= **0.212 *W (Shearwall ASD)**

Location	Area	DL	Seismic SL	Seismic Wt., W_x	Level Force, V_1	# of walls	Eave length	Wall Length, L	Wall Height, H	Wall Wt., W_w	Wall Force, Vw	Total Force, Vs	
A	288 ft^2	120 psf	95 psf	61.9 kips	13.1 kips	4	0.00 ft	24.0 ft	10.0 ft	11.4 kips	2.4 kips	15.5 kips	A

Project:	Legend Well House
Engineer:	SMP
Date:	4/25/2024

Equivalent Lateral Force Procedure per latest version of ASCE 7

Seismic Forces Equivalent Lateral Force Procedure

V =	**0.212W**	Base Shear ASCE 7-16 Equation 12.8-1
Cs=	0.212	Seismic Response Coefficient (input from Code Search Spreadsheet)
T=	0.129	Building Period (input from Code Search Spreadsheet)

Total Seismic loads:

high roof	62 kips
upper level walls	23 kips
main floor	0 kips
mid level walls	0 kips
mid floor	0 kips
lower level walls	0 kips
Total Building wt. =	**85 kips**

Total Base Shear, V:

V, Seismic: 18 kips

V, Wind: Wind will never control for this house, because the seismic weight is so high.

Seismic Controls for all wall designs

Vertical Distribution of Forces: **A**

k= 1.0

Level	Key Plan Loc.	wi	hi	wi*hi^k	wi*hi^k/Σwi*hi^k	Cs	Fx	Vx (kips)	ASD REDUCTION
High Roof	A	73 kips	10.0 ft.	733					
Crawlspace	---	11 kips	5.0 ft.	57	0.9	0.212	16.7 kips	**16.7 kips**	11.9 kips
	---	0 kips	0.0 ft.	0	0.1	0.212	1.3 kips	**18.0 kips**	12.8 kips
	---			0	0.0	0.212	0.0 kips	**0.0 kips**	0.0 kips
	---			0	0.0	0.212	0.0 kips	**0.0 kips**	0.0 kips
Σ	---	**85 kips**	--	**790**			**18.0 kips**		

ONE-WAY CONCRETE SLAB DESIGN

BENDING

$DL := 20 \cdot psf$ superimposed dead load, unfactored (does not include self-weight)

$LL := 325 \cdot psf$ superimposed live load, unfactored

$thk := 8 \cdot in$ thickness of slab

$l_n := 10.67 \cdot ft$ clear span of slab

$b_w := 12 \cdot in$ assumed width of slab

$conc\_wt := 145 \cdot pcf$ weight of concrete

$no := 6$ size of bars

$spacing := 16 \cdot in$ spacing of bars

$cover := 2.0 \cdot in$ min cover (ACI 20.5.1.3.1) is 0.75

$f_y := 60000 \cdot psi$ strength of reinforcing steel

$f_c := 4000 \cdot psi$ compressive strength of concrete

$\phi_b := 0.9$ strength reduction factor for bending (ACI 21.2.1)

$\phi_v := 0.85$ strength reduction factor for shear (ACI 21.2.1)

$w_u := 1.4 \cdot (DL + thk \cdot conc\_wt) + 1.6 \cdot (LL)$ ACI eq'n 5.3.1

$w_u = 683.3\, psf$ total factored load to be resisted

$w := (DL + thk \cdot conc\_wt) + LL$ service loads for deflection calc

$d_b := no \cdot 0.125 \cdot in$ $d_b = 0.75\, in$ diameter of reinforcing bar

$A_s := \frac{\pi \cdot d_b^{\,2}}{4} \cdot \frac{1}{spacing}$ $A_s = 0.33\, \frac{in^2}{ft}$ area of reinforcing steel per foot

$a := \frac{A_s \cdot f_y}{0.85 \cdot f_c}$ $a = 0.49\, in$ depth of stress block

$d := thk - cover - \frac{d_b}{2}$ $d = 5.625\, in$ distance from extreme fiber in compression to centroid of tensile steel

$M_n := \phi_b \cdot A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)$ $M_n = 96.3\, \frac{kip \cdot in}{ft}$ moment capacity of slab per foot of width

$M_u := \frac{1}{16} \cdot \left(w_u \cdot l_n^{\,2}\right)$ $M_u = 58.3\, \frac{kip \cdot in}{ft}$ applied moment, factored, should be less than Mn.. (see ACI 7.5.2)

$\frac{M_u}{M_n} = 0.606$ **< 1.0 .. okay in bending**

SHEAR

Shear strength check

$Vc_1 := \phi_v \cdot 2 \cdot \sqrt{f_c \cdot psi} \cdot d$ $\quad Vc_1 = 7.3 \dfrac{kip}{ft}$ ACI eq'n 22.5.5.1

$\rho_w := \dfrac{A_s}{d}$

$V_u := w_u \cdot \left(\dfrac{l_n}{2} - d \right)$ $\quad V_u = 3.3 \dfrac{kip}{ft}$ critical shear force (at 'd' away from the support

$Mu := w_u \cdot d \cdot \left(\dfrac{l_n}{2} - \dfrac{d}{2} \right)$ $\quad Mu = 19.6 \dfrac{kip \cdot in}{ft}$ bending moment at 'd' away from support

$Vc_2 := \phi_v \cdot \left(1.9 \cdot \sqrt{f_c \cdot psi} + 2500 \cdot psi \cdot \rho_w \cdot \dfrac{V_u \cdot d}{Mu} \right) \cdot d$ ACI eq'n 22.5.5.1

$Vc_2 = 7.6 \dfrac{kip}{ft}$

$V_c := \max\left(\left(Vc_1 \;\; Vc_2\right)\right)$ $\quad V_c = 7.6 \dfrac{kip}{ft}$ shear capacity of concrete

$\dfrac{V_u}{V_c} = 0.439$ **< 1.0 .. okay for shear** (slabs are excluded from requirement of ACI 7.5.3)

CHECK DEFLECTION

$h_{min} := \dfrac{l_n}{20}$ $\quad h_{min} = 6.402\,in$ minimum thickness without deflection calculation ACI 7.3.1.1

Project Title:
Engineer:
Project ID:
Project Descr:

Concrete Beam

Project File: Masonry.ec6

LIC# : KW-06018372, Build:20.23.08.30 | Canyons Structural Inc |

DESCRIPTION: --None--

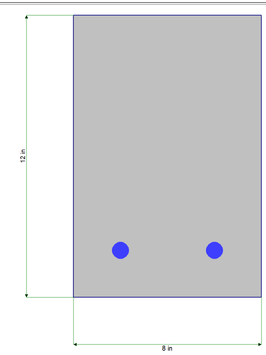
CODE REFERENCES

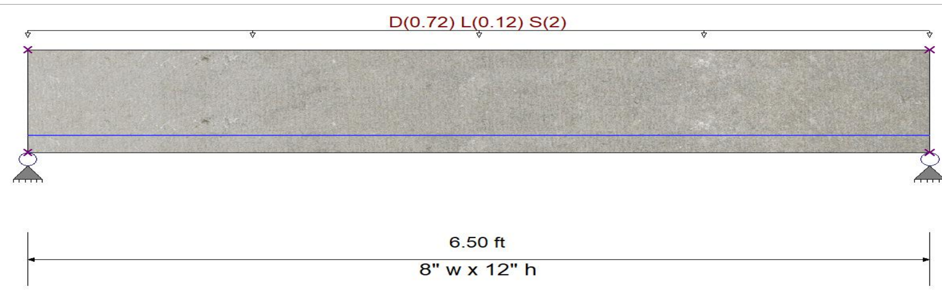
Calculations per ACI 318-19, IBC 2021, ASCE 7-16
Load Combination Set : ASCE 7-16

General Information

f'c	=	2.50 ksi	ϕ Phi Values	Flexure :	0.90
fr = $f'c^{1/2} \cdot 7.50$	=	375.0 psi		Shear :	0.750
ψ Density	=	145.0 pcf	β_1	=	0.850
λ LtWt Factor	=	1.0			
Elastic Modulus	=	2,850.0 ksi	Fy - Stirrups		40.0 ksi
fy - Main Rebar	=	60.0 ksi	E - Stirrups	=	29,000.0 ksi
E - Main Rebar	=	29,000.0 ksi	Stirrup Bar Size #		3
			Number of Resisting Legs Per Stirrup =		2

Seismic Design Category = A





Cross Section & Reinforcing Details

Rectangular Section, Width = 8.0 in, Height = 12.0 in
Span #1 Reinforcing....
2-#6 at 2.0 in from Bottom, from 0.0 to 6.50 ft in this span

Beam self weight calculated and added to loads
Load for Span Number 1
Uniform Load : D = 0.720, L = 0.120, S = 2.0 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.679** : 1	
Section used for this span	**Typical Section**	
Mu : Applied	22.709	k-ft
Mn * Phi : Allowable	33.450	k-ft
Location of maximum on span	3.256	ft
Span # where maximum occurs	Span # 1	

Maximum Deflection

Max Downward Transient Deflection	0.050 in	Ratio =	1548	>=360.0	L Only
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360.0	S Only
Max Downward Total Deflection	0.076 in	Ratio =	1030	>=180.0	Span: 1 : +D+S
Max Upward Total Deflection	0.000 in	Ratio =	0	<180.0	Span: 1 : +D+S

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	9.154	9.154
Max Upward from Load Combinations	9.154	9.154
Max Upward from Load Cases	6.500	6.500
D Only	2.654	2.654
+D+L	3.044	3.044
+D+S	9.154	9.154
+D+0.750L	2.947	2.947

Concrete Beam

Project File: Masonry.ec6

DESCRIPTION: --None--

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
+D+0.750L+0.750S	7.822	7.822
+0.60D	1.592	1.592
L Only	0.390	0.390
S Only	6.500	6.500

Shear Stirrup Requirements

Between 0.00 to 2.55 ft, Phi*lambda*sqrt(f'c)*bw*d < Vu <= Phi*Vc, Req'd Vs = Min per 9.6.3.1, use #3 stirrups spaced at 5.000 in
Between 2.56 to 3.94 ft, Vu <= Phi*lambda*sqrt(f'c)*bw*d, Req'd Vs = Not Reqd per 9.3.6.1, Stirrups are not required.
Between 3.95 to 6.49 ft, Phi*Vc < Vu, Req'd Vs = 7.975, use #3 stirrups spaced at 5.000 in

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu Actual	(k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd
+1.20D+L+1.60S	1	0.00	10.00	13.97	13.97	0.00	1.00	6.00	Phi*Vc < Vu	7.975	19.2	5.0
+1.20D+L+1.60S	1	0.07	10.00	13.67	13.67	0.98	1.00	6.00	Phi*Vc < Vu	7.670	19.2	5.0
+1.20D+L+1.60S	1	0.14	10.00	13.36	13.36	1.94	1.00	6.00	Phi*Vc < Vu	7.364	19.2	5.0
+1.20D+L+1.60S	1	0.21	10.00	13.06	13.06	2.88	1.00	6.00	Phi*Vc < Vu	7.059	19.2	5.0
+1.20D+L+1.60S	1	0.28	10.00	12.75	12.75	3.80	1.00	6.00	Phi*Vc < Vu	6.753	19.2	5.0
+1.20D+L+1.60S	1	0.36	10.00	12.45	12.45	4.69	1.00	6.00	Phi*Vc < Vu	6.448	19.2	5.0
+1.20D+L+1.60S	1	0.43	10.00	12.14	12.14	5.57	1.00	6.00	Phi*Vc < Vu	6.142	19.2	5.0
+1.20D+L+1.60S	1	0.50	10.00	11.84	11.84	6.42	1.00	6.00	Phi*Vc < Vu	5.837	19.2	5.0
+1.20D+L+1.60S	1	0.57	10.00	11.53	11.53	7.25	1.00	6.00	Phi*Vc < Vu	5.531	19.2	5.0
+1.20D+L+1.60S	1	0.64	10.00	11.23	11.23	8.06	1.00	6.00	Phi*Vc < Vu	5.226	19.2	5.0
+1.20D+L+1.60S	1	0.71	10.00	10.92	10.92	8.84	1.00	6.00	Phi*Vc < Vu	4.920	19.2	5.0
+1.20D+L+1.60S	1	0.78	10.00	10.61	10.61	9.61	0.92	6.00	Phi*Vc < Vu	4.615	19.2	5.0
+1.20D+L+1.60S	1	0.85	10.00	10.31	10.31	10.35	0.83	6.00	Phi*Vc < Vu	4.309	19.2	5.0
+1.20D+L+1.60S	1	0.92	10.00	10.00	10.00	11.07	0.75	6.00	Phi*Vc < Vu	4.004	19.2	5.0
+1.20D+L+1.60S	1	0.99	10.00	9.70	9.70	11.77	0.69	6.00	Phi*Vc < Vu	3.699	19.2	5.0
+1.20D+L+1.60S	1	1.07	10.00	9.39	9.39	12.45	0.63	6.00	Phi*Vc < Vu	3.393	19.2	5.0
+1.20D+L+1.60S	1	1.14	10.00	9.09	9.09	13.11	0.58	6.00	Phi*Vc < Vu	3.088	19.2	5.0
+1.20D+L+1.60S	1	1.21	10.00	8.78	8.78	13.74	0.53	6.00	Phi*Vc < Vu	2.782	19.2	5.0
+1.20D+L+1.60S	1	1.28	10.00	8.48	8.48	14.35	0.49	6.00	Phi*Vc < Vu	2.477	19.2	5.0
+1.20D+L+1.60S	1	1.35	10.00	8.17	8.17	14.95	0.46	6.00	Phi*Vc < Vu	2.171	19.2	5.0
+1.20D+L+1.60S	1	1.42	10.00	7.87	7.87	15.52	0.42	6.00	Phi*Vc < Vu	1.866	19.2	5.0
+1.20D+L+1.60S	1	1.49	10.00	7.56	7.56	16.06	0.39	6.00	Phi*Vc < Vu	1.560	19.2	5.0
+1.20D+L+1.60S	1	1.56	10.00	7.25	7.25	16.59	0.36	6.00	Phi*Vc < Vu	1.255	19.2	5.0
+1.20D+L+1.60S	1	1.63	10.00	6.95	6.95	17.09	0.34	6.00	Phi*Vc < Vu	0.9493	19.2	5.0
+1.20D+L+1.60S	1	1.70	10.00	6.64	6.64	17.58	0.31	6.00	Phi*Vc < Vu	0.6439	19.2	5.0
+1.20D+L+1.60S	1	1.78	10.00	6.34	6.34	18.04	0.29	6.00	Phi*Vc < Vu	0.3384	19.2	5.0
+1.20D+L+1.60S	1	1.85	10.00	6.03	6.03	18.48	0.27	6.00	Phi*Vc < Vu	0.03292	19.2	5.0
+1.20D+L+1.60S	1	1.92	10.00	5.73	5.73	18.89	0.25	6.00	Phi*lambda*sqrt	in per 9.6.	19.2	5.0
+1.20D+L+1.60S	1	1.99	10.00	5.42	5.42	19.29	0.23	6.00	Phi*lambda*sqrt	in per 9.6.	19.2	5.0
+1.20D+L+1.60S	1	2.06	10.00	5.12	5.12	19.67	0.22	6.00	Phi*lambda*sqrt	in per 9.6.	19.2	5.0
+1.20D+L+1.60S	1	2.13	10.00	4.81	4.81	20.02	0.20	6.00	Phi*lambda*sqrt	in per 9.6.	19.2	5.0
+1.20D+L+1.60S	1	2.20	10.00	4.51	4.51	20.35	0.18	6.00	Phi*lambda*sqrt	in per 9.6.	19.2	5.0
+1.20D+L+1.60S	1	2.27	10.00	4.20	4.20	20.66	0.17	6.00	Phi*lambda*sqrt	in per 9.6.	19.2	5.0
+1.20D+L+1.60S	1	2.34	10.00	3.89	3.89	20.95	0.15	6.00	Phi*lambda*sqrt	in per 9.6.	19.2	5.0
+1.20D+L+1.60S	1	2.42	10.00	3.59	3.59	21.21	0.14	6.00	Phi*lambda*sqrt	in per 9.6.	19.2	5.0
+1.20D+L+1.60S	1	2.49	10.00	3.28	3.28	21.46	0.13	6.00	Phi*lambda*sqrt	in per 9.6.	19.2	5.0
+1.20D+L+1.60S	1	2.56	10.00	2.98	2.98	21.68	0.11	5.34	Vu <= Phi*lambd	ot Reqd pe	5.3	0.0
+1.20D+L+1.60S	1	2.63	10.00	2.67	2.67	21.88	0.10	5.34	Vu <= Phi*lambd	ot Reqd pe	5.3	0.0
+1.20D+L+1.60S	1	2.70	10.00	2.37	2.37	22.06	0.09	5.34	Vu <= Phi*lambd	ot Reqd pe	5.3	0.0
+1.20D+L+1.60S	1	2.77	10.00	2.06	2.06	22.22	0.08	5.34	Vu <= Phi*lambd	ot Reqd pe	5.3	0.0
+1.20D+L+1.60S	1	2.84	10.00	1.76	1.76	22.35	0.07	5.34	Vu <= Phi*lambd	ot Reqd pe	5.3	0.0
+1.20D+L+1.60S	1	2.91	10.00	1.45	1.45	22.46	0.05	5.34	Vu <= Phi*lambd	ot Reqd pe	5.3	0.0
+1.20D+L+1.60S	1	2.98	10.00	1.15	1.15	22.56	0.04	5.34	Vu <= Phi*lambd	ot Reqd pe	5.3	0.0
+1.20D+L+1.60S	1	3.05	10.00	0.84	0.84	22.63	0.03	5.34	Vu <= Phi*lambd	ot Reqd pe	5.3	0.0
+1.20D+L+1.60S	1	3.13	10.00	0.53	0.53	22.68	0.02	5.34	Vu <= Phi*lambd	ot Reqd pe	5.3	0.0
+1.20D+L+1.60S	1	3.20	10.00	0.23	0.23	22.70	0.01	5.34	Vu <= Phi*lambd	ot Reqd pe	5.3	0.0
+1.20D+L+1.60S	1	3.27	10.00	-0.08	0.08	22.71	0.00	5.34	Vu <= Phi*lambd	ot Reqd pe	5.3	0.0

Concrete Beam

Project File: Masonry.ec6

LIC# : KW-06018372, Build:20.23.08.30 Canyons Structural Inc (c) ENERCALC INC 1983-2023

DESCRIPTION: --None--

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu Actual	(k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd
+1.20D+L+1.60S	1	3.34	10.00	-0.38	0.38	22.69	0.01	5.34	Vu <= Phi*lambd	t Reqd pe	5.3	0.0
+1.20D+L+1.60S	1	3.41	10.00	-0.69	0.69	22.65	0.03	5.34	Vu <= Phi*lambd	t Reqd pe	5.3	0.0
+1.20D+L+1.60S	1	3.48	10.00	-0.99	0.99	22.59	0.04	5.34	Vu <= Phi*lambd	t Reqd pe	5.3	0.0
+1.20D+L+1.60S	1	3.55	10.00	-1.30	1.30	22.51	0.05	5.34	Vu <= Phi*lambd	t Reqd pe	5.3	0.0
+1.20D+L+1.60S	1	3.62	10.00	-1.60	1.60	22.41	0.06	5.34	Vu <= Phi*lambd	t Reqd pe	5.3	0.0
+1.20D+L+1.60S	1	3.69	10.00	-1.91	1.91	22.29	0.07	5.34	Vu <= Phi*lambd	t Reqd pe	5.3	0.0
+1.20D+L+1.60S	1	3.77	10.00	-2.21	2.21	22.14	0.08	5.34	Vu <= Phi*lambd	t Reqd pe	5.3	0.0
+1.20D+L+1.60S	1	3.84	10.00	-2.52	2.52	21.97	0.10	5.34	Vu <= Phi*lambd	t Reqd pe	5.3	0.0
+1.20D+L+1.60S	1	3.91	10.00	-2.83	2.83	21.78	0.11	5.34	Vu <= Phi*lambd	t Reqd pe	5.3	0.0
+1.20D+L+1.60S	1	3.98	10.00	-3.13	3.13	21.57	0.12	6.00	Phi*lambda*sqrt	in per 9.6.	19.2	5.0
+1.20D+L+1.60S	1	4.05	10.00	-3.44	3.44	21.34	0.13	6.00	Phi*lambda*sqrt	in per 9.6.	19.2	5.0
+1.20D+L+1.60S	1	4.12	10.00	-3.74	3.74	21.08	0.15	6.00	Phi*lambda*sqrt	in per 9.6.	19.2	5.0
+1.20D+L+1.60S	1	4.19	10.00	-4.05	4.05	20.80	0.16	6.00	Phi*lambda*sqrt	in per 9.6.	19.2	5.0
+1.20D+L+1.60S	1	4.26	10.00	-4.35	4.35	20.51	0.18	6.00	Phi*lambda*sqrt	in per 9.6.	19.2	5.0
+1.20D+L+1.60S	1	4.33	10.00	-4.66	4.66	20.19	0.19	6.00	Phi*lambda*sqrt	in per 9.6.	19.2	5.0
+1.20D+L+1.60S	1	4.40	10.00	-4.96	4.96	19.84	0.21	6.00	Phi*lambda*sqrt	in per 9.6.	19.2	5.0
+1.20D+L+1.60S	1	4.48	10.00	-5.27	5.27	19.48	0.23	6.00	Phi*lambda*sqrt	in per 9.6.	19.2	5.0
+1.20D+L+1.60S	1	4.55	10.00	-5.57	5.57	19.10	0.24	6.00	Phi*lambda*sqrt	in per 9.6.	19.2	5.0
+1.20D+L+1.60S	1	4.62	10.00	-5.88	5.88	18.69	0.26	6.00	Phi*lambda*sqrt	in per 9.6.	19.2	5.0
+1.20D+L+1.60S	1	4.69	10.00	-6.19	6.19	18.26	0.28	6.00	Phi*Vc < Vu	0.1857	19.2	5.0
+1.20D+L+1.60S	1	4.76	10.00	-6.49	6.49	17.81	0.30	6.00	Phi*Vc < Vu	0.4911	19.2	5.0
+1.20D+L+1.60S	1	4.83	10.00	-6.80	6.80	17.34	0.33	6.00	Phi*Vc < Vu	0.7966	19.2	5.0
+1.20D+L+1.60S	1	4.90	10.00	-7.10	7.10	16.84	0.35	6.00	Phi*Vc < Vu	1.102	19.2	5.0
+1.20D+L+1.60S	1	4.97	10.00	-7.41	7.41	16.33	0.38	6.00	Phi*Vc < Vu	1.408	19.2	5.0
+1.20D+L+1.60S	1	5.04	10.00	-7.71	7.71	15.79	0.41	6.00	Phi*Vc < Vu	1.713	19.2	5.0
+1.20D+L+1.60S	1	5.11	10.00	-8.02	8.02	15.23	0.44	6.00	Phi*Vc < Vu	2.018	19.2	5.0
+1.20D+L+1.60S	1	5.19	10.00	-8.32	8.32	14.65	0.47	6.00	Phi*Vc < Vu	2.324	19.2	5.0
+1.20D+L+1.60S	1	5.26	10.00	-8.63	8.63	14.05	0.51	6.00	Phi*Vc < Vu	2.629	19.2	5.0
+1.20D+L+1.60S	1	5.33	10.00	-8.93	8.93	13.43	0.55	6.00	Phi*Vc < Vu	2.935	19.2	5.0
+1.20D+L+1.60S	1	5.40	10.00	-9.24	9.24	12.78	0.60	6.00	Phi*Vc < Vu	3.240	19.2	5.0
+1.20D+L+1.60S	1	5.47	10.00	-9.55	9.55	12.11	0.66	6.00	Phi*Vc < Vu	3.546	19.2	5.0
+1.20D+L+1.60S	1	5.54	10.00	-9.85	9.85	11.42	0.72	6.00	Phi*Vc < Vu	3.851	19.2	5.0
+1.20D+L+1.60S	1	5.61	10.00	-10.16	10.16	10.71	0.79	6.00	Phi*Vc < Vu	4.157	19.2	5.0
+1.20D+L+1.60S	1	5.68	10.00	-10.46	10.46	9.98	0.87	6.00	Phi*Vc < Vu	4.462	19.2	5.0
+1.20D+L+1.60S	1	5.75	10.00	-10.77	10.77	9.23	0.97	6.00	Phi*Vc < Vu	4.768	19.2	5.0
+1.20D+L+1.60S	1	5.83	10.00	-11.07	11.07	8.45	1.00	6.00	Phi*Vc < Vu	5.073	19.2	5.0
+1.20D+L+1.60S	1	5.90	10.00	-11.38	11.38	7.65	1.00	6.00	Phi*Vc < Vu	5.379	19.2	5.0
+1.20D+L+1.60S	1	5.97	10.00	-11.68	11.68	6.84	1.00	6.00	Phi*Vc < Vu	5.684	19.2	5.0
+1.20D+L+1.60S	1	6.04	10.00	-11.99	11.99	5.99	1.00	6.00	Phi*Vc < Vu	5.989	19.2	5.0
+1.20D+L+1.60S	1	6.11	10.00	-12.29	12.29	5.13	1.00	6.00	Phi*Vc < Vu	6.295	19.2	5.0
+1.20D+L+1.60S	1	6.18	10.00	-12.60	12.60	4.25	1.00	6.00	Phi*Vc < Vu	6.60	19.2	5.0
+1.20D+L+1.60S	1	6.25	10.00	-12.91	12.91	3.34	1.00	6.00	Phi*Vc < Vu	6.906	19.2	5.0
+1.20D+L+1.60S	1	6.32	10.00	-13.21	13.21	2.41	1.00	6.00	Phi*Vc < Vu	7.211	19.2	5.0
+1.20D+L+1.60S	1	6.39	10.00	-13.52	13.52	1.46	1.00	6.00	Phi*Vc < Vu	7.517	19.2	5.0
+1.20D+L+1.60S	1	6.46	10.00	-13.82	13.82	0.49	1.00	6.00	Phi*Vc < Vu	7.822	19.2	5.0

Maximum Forces & Stresses for Load Combinations

Load Combination Segment	Span #	Location (ft) along Beam	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	6.500	22.71	33.45	0.68
+1.40D					
Span # 1	1	6.500	6.04	33.45	0.18
+1.20D+1.60L					
Span # 1	1	6.500	6.19	33.45	0.19
+1.20D+1.60L+0.50S					
Span # 1	1	6.500	11.47	33.45	0.34
+1.20D+L					
Span # 1	1	6.500	5.81	33.45	0.17
+1.20D					

Concrete Beam

Project File: Masonry.ec6

LIC# : KW-06018372, Build:20.23.08.30 Canyons Structural Inc (c) ENERCALC INC 1983-2023

DESCRIPTION: --None--

Load Combination Segment	Span #	Location (ft) along Beam	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
Span # 1	1	6.500	5.18	33.45	0.15
+1.20D+L+1.60S					
Span # 1	1	6.500	22.71	33.45	0.68
+1.20D+1.60S					
Span # 1	1	6.500	22.08	33.45	0.66
+1.20D+L+0.50S					
Span # 1	1	6.500	11.09	33.45	0.33
+0.90D					
Span # 1	1	6.500	3.88	33.45	0.12
+1.20D+L+0.20S					
Span # 1	1	6.500	7.92	33.45	0.24

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl (in)	Location in Span (ft)	Load Combination	Max. "+" Defl (in)	Location in Span (ft)
+D+S	1	0.0757	3.250		0.0000	0.000

Concrete Shear Wall

Project File: Masonry.ec6

DESCRIPTION: 24ft wall

Code References

Calculations per ACI 318-19, IBC 2021, ASCE 7-16
Load Combinations Used : ASCE 7-16

General Information

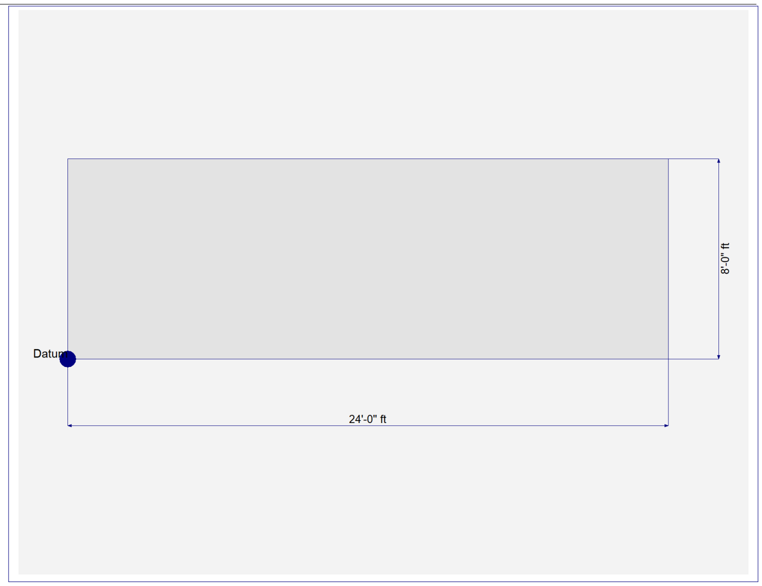
Wall Material CONCRETE
Sds 0.8490

Material Properties

f'c	4.0	ksi	Ec	3,120.0	ksi
fy	60.0	ksi	Ev	1,248.0	ksi
Density	145.0	pcf	Phi - Shear	0.650	

Wall Data

	Bottom
Analysis Height	0.00 ft
Wall Offset	(datum) ft
Wall Length	24.0 ft
Wall Thickness	8.0 in
Structural Depth	12.0 ft



DESIGN SUMMARY

	Bottom Level	
Vu : Story Shear	12.303	+1.370D+0.20S
Mu : Story Moment	48.0	+1.370D+0.20S
Nu : Axial	117.888	+1.20D+1.60S
Uplift @ Left End	0.0	
Uplift @ Right End	0.0	
Phi * 8 * sqrt(f'c)*h*Lw	757.73 k	
Phi * Vc	284.150 k	
Phi * Vs Req'd	0.0 k	
Horizontal As Req'd	0.1920 in^2	
Vertical As Req'd	0.240 in^2	
Bending As Req'd	2.074 in^2	

Force Summary

Load Combination	Values for Wall section			Resultant	Overturning	Uplift (k)	
Wall Level	Vu (k)	Mu (k)	Pu (k)	Ecc (ft)	Ratio	Left	Right
+1.40D							
Wall Level : 1			50.176				
+1.20D+0.50Lr							
Wall Level : 1			44.448				
+1.20D+0.50S							
Wall Level : 1			66.408				
+1.20D+1.60Lr							
Wall Level : 1			47.616				
+1.20D+1.60S							

Concrete Shear Wall

Project File: Masonry.ec6

LIC# : KW-06018372, Build:20.23.10.02 Canyons Structural Inc

DESCRIPTION: 24ft wall

Force Summary

Load Combination / Wall Level	Values for Wall section Vu (k)	Mu (k)	Pu (k)	Resultant Ecc (ft)	Overturning Ratio	Uplift (k) Left	Right
Wall Level : 1			117.888				
+0.90D							
Wall Level : 1			32.256				
+1.370D+0.20S+E							
Wall Level : 1	12.303	48.000	58.454	0.821	14.613		
+1.370D+0.20S-E							
Wall Level : 1	12.303	48.000	58.454	0.821	14.613		
+0.7302D+E							
Wall Level : 1	12.303	48.000	26.170	1.834	6.543		
+0.7302D-E							
Wall Level : 1	12.303	48.000	26.170	1.834	6.543		

Footing Information

Footing Dimensions

Dist. Left	1.0 ft	f'c	2.50 ksi	Rebar Cover	3.0 in
Wall Length	24.0 ft	Fy	60.0 ksi	Footing Thickness	12.0 in
Dist. Right	1.0 ft			Width	2.0 ft
Total Ftg Length	26.0 ft				

Max Factored Soil Pressures

@ Left Side of Footing 2,013.37 psf
.... governing load comb +1.20D+1.60S

@ Right Side of Footing 2,013.37 psf
.... governing load comb +1.20D+1.60S

Max UNfactored Soil Pressures

@ Left Side of Footing 1,377.81 psf
.... governing load comb +D+S

@ Right Side of Footing 1,377.81 psf
.... governing load comb +D+S

Footing One-Way Shear Check...

vu @ Left End of Footing	5.483 psi
vu @ Right End of Footing	5.483 psi
vn * phi : Allowable	85.0 psi

Overturning Stability...

	@ Left End of Ftg	@ Right End of Ftg
Overturning Moment	54.345 k-ft	54.345 k-ft
Resisting Moment	483.132 k-ft	483.132 k-ft
Stability Ratio	8.890 : 1	8.890 : 1
.... governing load comb	+0.60D+0.70E	+0.60D+0.70E

Footing Bending Design...

	@ Left End	@ Right End
Mu	2.013 k-ft	2.013 k-ft
Ru	13.809 psi	13.809 psi
As % Req'd	0.00180 in^2	0.00180 in^2
As Req'd in Footing Width	0.5184 in^2	0.5184 in^2

Concrete Shear Wall

Project File: Masonry.ec6

DESCRIPTION: 12ft wall

Code References

Calculations per ACI 318-19, IBC 2021, ASCE 7-16
Load Combinations Used : ASCE 7-16

General Information

Wall Material CONCRETE
Sds 0.8490

Material Properties

f'c	4.0	ksi	Ec	3,120.0	ksi
fy	60.0	ksi	Ev	1,248.0	ksi
Density	145.0	pcf	Phi - Shear	0.650	

Wall Data

	Bottom
Analysis Height	0.00 ft
Wall Offset	(datum) ft
Wall Length	12.0 ft
Wall Thickness	8.0 in
Structural Depth	11.5 ft

DESIGN SUMMARY

	Bottom Level	
Vu : Story Shear	18.606	+1.370D+0.20S
Mu : Story Moment	184.320	+1.20D+1.60S
Nu : Axial	41.856	+1.20D+1.60S
Uplift @ Left End	0.6385	+0.7302D+E
Uplift @ Right End	0.6385	+0.7302D+E
Phi * 8 * sqrt(f'c)*h*Lw	378.866 k	
Phi * Vc	142.075 k	
Phi * Vs Req'd	0.0 k	
Horizontal As Req'd	0.1920 in^2	
Vertical As Req'd	0.240 in^2	
Bending As Req'd	1.987 in^2	

Force Summary

Load Combination	Values for Wall section			Resultant	Overturning	Uplift (k)	
Wall Level	Vu (k)	Mu (k)	Pu (k)	Ecc (ft)	Ratio	Left	Right
+1.40D							
Wall Level : 1		40.320	19.712	2.045			
+1.20D+0.50Lr							
Wall Level : 1		37.440	17.376	2.155			
+1.20D+0.50S							
Wall Level : 1		81.360	24.696	3.294			
+1.20D+1.60Lr							
Wall Level : 1		43.776	18.432	2.375			
+1.20D+1.60S							

Concrete Shear Wall

Project File: Masonry.ec6

DESCRIPTION: 12ft wall

Force Summary

Load Combination / Wall Level	Values for Wall section Vu (k)	Mu (k)	Pu (k)	Resultant Ecc (ft)	Overturning Ratio	Uplift (k) Left	Right
Wall Level : 1		184.320	41.856	4.404			
+0.90D							
Wall Level : 1		25.920	12.672	2.045			
+1.370D+0.20S+E							
Wall Level : 1	18.606	106.170	22.407	4.738	1.589		
+1.370D+0.20S-E							
Wall Level : 1	18.606	10.170	22.407	0.454	4.013		
+0.7302D+E							
Wall Level : 1	18.606	69.030	10.281	6.714	0.847	0.638	0.638
+0.7302D-E							
Wall Level : 1	18.606	26.970	10.281	2.623	1.723	0.638	0.638

Footing Information

Footing Dimensions

Dist. Left	1.0 ft	f'c	2.50 ksi	Rebar Cover	3.0 in
Wall Length	12.0 ft	Fy	60.0 ksi	Footing Thickness	12.0 in
Dist. Right	1.0 ft			Width	2.0 ft
Total Ftg Length	14.0 ft				

Max Factored Soil Pressures

@ Left Side of Footing 99999,856 psf
.... governing load comb +1.370D+0.20S+E

@ Right Side of Footing 99999,856 psf
.... governing load comb +1.370D+0.20S+E

Max UNfactored Soil Pressures

@ Left Side of Footing 99999,856 psf
.... governing load comb +D+0.70E

@ Right Side of Footing 99999,856 psf
.... governing load comb +D+0.70E

Footing One-Way Shear Check...

vu @ Left End of Footing	35.263 psi
vu @ Right End of Footing	17.323 psi
vn * phi : Allowable	85.0 psi

Overturning Stability...

	@ Left End of Ftg	@ Right End of Ftg
Overturning Moment	46.073 k-ft	46.073 k-ft
Resisting Moment	132.444 k-ft	97.884 k-ft
Stability Ratio	2.875 : 1	2.125 : 1
.... governing load comb	+0.60D+0.70E	+0.60D+0.70E

Footing Bending Design...

	@ Left End	@ Right End
Mu	5.926 k-ft	6.113 k-ft
Ru	40.644 psi	41.928 psi
As % Req'd	0.00180 in^2	0.00180 in^2
As Req'd in Footing Width	0.5184 in^2	0.5184 in^2