laughlin ricks architecture

134 3rd Ave E
Twin Falls, ID 83301
208.736.8050

Addendum No. 2

PROJECT: Kimberly AG Building Addition
Date: October 29, 2024

To the General Contractor, Subcontractors and Suppliers:

The following items contain additions, deletions, or modifications to the Plans and Specifications. This Addendum forms a part of the Contract Documents and shall be bound inside the cover of the Project Manual.

General Contractor shall be responsible for contacting their sub-contractors as this addendum may affect them.

Bidders shall acknowledge receipt of this Addendum on the Contractor Bid Proposal.

GENERAL NOTES:

1. Responses to DOPL review.
2. Structural Calculations provided to DOPL per comments

Architectural Drawings:
Sheet A0-0 REVISED per DOPL comments
Sheet A0-1 REVISED per DOPL comments
Sheet A2-0 REVISED per DOPL comments

Summary of Attachments to Addendum No. 2

Sheet A0-0
Sheet A0-1
Sheet A2-0
Structural Calculations

END OF ADDENDUM No. 2

Laughlin Ricks Architecture, LLC

AN ADDITION FOR:

KIMBERLY SCHOOL DISTRICT

3682 N 3450 E, Kimberly, ID 83341

SHEET INDEX

Sheet Number	Sheet Name
A0-0	TITLE SHEET
A0-1	ACCESSIBILITY DIAGRAMS
A0-2	DEMOLITION SITE PLAN
A1-1	NEW SITE PLAN
A1-2	SITE DETAILS
1 of 1	SURVEY
C1.1	CIVIL NOTES AND LEGEND
C2.1	SEWER SERVICE AND STORM DRAIN PLAN
C3.1	STORM DRAIN DETAILS
C4.1	CIVIL SITE PLAN
C5.1	SITE GRADING PLAN
C6.1	SITE GRADING DETAILS
A1-3	DEMO FLOOR PLAN
A2-0	ROTATED OVERALL FLOOR PLAN
A2-1	ENLARGED FLOOR PLAN
A3-0	EXTERIOR ELEVATIONS
A4-0	NEW ROOF PLAN
A5-0	BLDG SECTIONS
A7-0	NEW CEILING PLAN
A8-0	INTERIOR ELEVATIONS
A9-0	DOOR, WINDOW, & SIGNAGE TYPES
A10-0	DOOR & WINDOW DETAILS

SHEET INDEX

Sheet Number	Sheet Name
A10-1	DETAILS
S001	STRUCTURAL NOTES
S002	STRUCTURAL NOTES
S010	SCHEDULES
S011	SCHEDULES
S012	SCHEDULES
S013	SCHEDULES
S101	FOOTING & FOUNDATION PLAN
S102	ROOF FRAMING PLAN
S201	TYPICAL DETAILS
S210	FOUNDATION DETAILS
S220	ROOF FRAMING DETAILS
P1.1	WATER & GAS PLUMBING FLOOR PLAN
P1.2	WASTE & VENT PIPING FLOOR PLAN
P2.1	PLUMBING DETAILS & SCHEDULES
M1.1	MECHANICAL FLOOR PLAN
M2.1	MECHANICAL DETAILS & SCHEDULES
E0.0	ELECTRICAL SYMBOLS & DETAILS
E0.1	OVERALL & EXISTING ELECTRICAL PLANS
E1.0	LIGHTING PLAN
E2.0	POWER & SYSTEMS PLAN
E3.0	POWER RISER, SCHEDULES & DETAILS

2

GENERAL NOTES:

1. ALL WORK SHALL MEET CURRENT ADOPTED STATE, LOCAL CODES, ORDINANCES, & 2018 IBC and 2018 IEBC 301.3.2 WORK AREA COMPLIANCE METHOD, SECTION 606 ADDITIONS, CHAPTER 11 ADDITIONS.
2. ALL MECHANICAL, ELECTRICAL, & PLUMBING WORK SHALL MEET ALL CURRENT APPLICABLE STATE & LOCAL CODES.
3. ALL UTILITIES SHALL BE PROPERLY IDENTIFIED & LOCATED BEFORE WORK BEGINS ON PROJECT.
4. CONTRACTOR SHALL VERIFY ALL CONDITIONS & DIMENSIONS AT THE JOB SITE & NOTIFY THE ARCHITECT OF ANY DIMENSIONAL ERRORS, OMISSIONS, OR DISCREPANCIES BEFORE BEGINING OR FABRICATING ANY WORK.
5. DO NOT SCALE DRAWINGS.
6. ALL DOOR HANDLES SHALL BE LEVER TYPE, ALL DOOR HARDWARE SHALL BE A.D.A COMPLIANT AS PER CURRENT ANSI 117.1
7. AT MAIN ENTRANCE DOOR SHALL HAVE SINGLE ACTION LOCKING DEVICE &/ OR SIGNED "THIS DOOR TO REMAIN UNLOCKED WHEN BUILDING IS OCCUPIED."

FIRE DEPARTMENT NOTES:

1. IT SHALL BE THE RESPONSIBILITY OF THE GENERAL CONTRACTOR TO INSURE THAT ALL DEFERRED SUBMITTALS REQUIRED BY THE FIRE DEPARTMENT **HAVE BEEN APPROVED BY THE STATE PRIOR TO THE INSTALLATION OF A FIRE ALARM AND/OR FIRE SPRINKLER SYSTEM.** IT SHALL ALSO BE THE RESPONSIBILITY OF THE GENERAL CONTRACTOR TO VERIFY THAT ALL APPROPRIATE TESTING AND/OR INSPECTIONS HAVE BEEN PERFORMED BEFORE COVERING OR CALLING FOR A FINAL INSPECTION.
2. FIRE SPRINKLER UNDERGROUND PIPING
   THE UNDERGROUND FIRE SPRINKLER LINE MUST MEET NFPA 24 AND THE CITY OF KIMBERLY STANDARDS. THE INSPECTION AND TESTING OF THE UNDERGROUND FIRE SPRINKLER LINE SHALL BE OVERSEEN BY THE FIRE MARSHALL.
3. SPRINKLER SYSTEM(S)
   SPRINKLER SYSTEM PLANS SHALL BE SENT TO THE STATE FIRE MARSHAL OFFICE AND DESIGNED IN ACCORDANCE WITH CURRENT NFPA 13 STANDARDS.
   IDAHO STATE FIRE MARSHAL
   700 WEST STATE STREET, 3RD FLOOR
   BOISE, IDAHO 83720
   PLANS SHALL MEET CURRENT IFC, NFPA 13R AND IDAHO STATE PLUMBING CODES, AND BE APPROVED PRIOR TO INSTALLATION.
4. FDC VISUAL ALARM A VISUAL ALARM DEVICE (EXTERIOR HORN/STROBE) SHALL BE PROVIDED IN THE AREA OF THE FDC.
5. APPROVED SIGNS SHALL BE INSTALLED ON THE FIRE RISER ROOM DOOR AND ON THE FIRE DEPARTMENT CONNECTION.

2

PLAN ANALYSIS
Based on 2018 Edition of I.E.B.C
301.3.2 Work area compliance method, section 606 Additions, Chapter 11 Additions

Architect of Record: Laughlin Ricks Architecture, L.L.C.

Engineer: ______

Job Address: ______

Legal Description: ______

Occupancy Classification: E — Occupant Load Per Area: ______

Occupancy Use: AG SHOP & CLASSROOMS

2

Allowable Stories Per Code: 2 — Provided: 1 (IBC Table 505.4) — Total: 250 OCCUPANTS, SEE ROTATED OVERALL FLOOR PLAN A2-0

Floor Area: Basement: ______ 1st: 12,775 SF (E) 3,948 SF (ADDITION) — Exits Required: Basement: ______ 1st: 2

Mezzanine: ______ 3rd: ______ Total: 16,723 SF — 2nd: ______ 3rd: ______ 4th: ______

Total Required Exits Per Occupant Load: 2, 10 PROVIDED (IBC Table 1006.3.2)

Actual furthest travel distance to exit: 96'-0" 2 (IBC Table 1017.2 & 1006.2.1)

Penetrations? Show Approved Listed Products on Plans: N/A

Type of Construction: VB — Allowable Building Height: 60'

Seismic Design Category: C — Allowable Area Calc's: 38,000

Automatic Sprinkler System: Yes: x No: ______ — ______ (IBC Table 506.2)

Maximum Floor Area Allowed: 38,000 — Exit Signs: Yes: x No: ______

Special Inspections Required? Yes: ______ No: x — Emergency Lights: Yes: x No: ______

Firewalls Required? (Specify Type & Rating) Yes: ______ No: x — Fire Extinguishers Shown: Yes: x No: ______ (IFC Section 906)

Occupancy Separation Use? Yes: ______ No: x — Fire Hydrant Locations Shown: Yes: x No: ______

Areas of Refuge Required? (IBC Section 1009.2,3,4) Yes: ______ No: x — Vestibule Required: Yes: (E) No: ______

Area Separation Required? Yes: ______ No: x — Classified Areas? (Show on plans & Show Areas) Yes: ______ No: x

Fire Resistance Ratings of BLDG Elements : 0 (IBC Table 601)
(Specify Rating)

Minimum Roof Class: C (IBC Table 1505.1) — Exterior Wall Openings: N/A (IBC 705.8)

Fire Doors: N/A (IBC Table 716.1.2) — Fire Alarm System: YES (IBC 907.2)

Fire Flow and Duration: ______ — Corridor Width: 72" (IBC Table 1020.2)

Rated Structural Frame: (Roof Supports Only) Yes: ______ No: x — Rated Corridors: (IBC Section 1020.1) Yes: ______ No: x

Rated Bearing Walls-Exterior: Yes: ______ No: x — Rated Bearing Walls-Interior: Yes: ______ No: x

Rated Nonbearing Walls-Exterior: (>30' Fire Separation) Yes: ______ No: x — Rated Bearing Walls-Interior: (Roof Supports Only) Yes: ______ No: x

Rated Nonbearing Walls-Exterior: (10'-30' Fire Separation) Yes: ______ No: x — Rated Nonbearing Walls-Interior: Yes: ______ No: x

Rated Floor Construction: Yes: ______ No: x — Rated Roof Construction: Yes: ______ No: x

Lighting Layout and COM Check? Yes: x No: ______

Comments: ______

1 PLAN ANALYSIS
1/4" = 1'-0"

FIRE SPRINKLER SYSTEM SHALL BE MODIFIED AS REQUIRED.

FIRE ALARM & DETECTION SYSTEM SHALL BE MODIFIED AS REQUIRED.

ABBREVIATIONS

AC	ACOUSTICAL CEILING	DIA	DIAMETER	GYP BD	GYPSUM BOARD	PL	PLATE, PLASTIC LAMINATE	T	THREAD
ADJ	ADJUSTABLE - ADJACENT	DIM	DIMENSION	HB	HOSE BIB	P-LAM	PLASTIC LAMINATE	TBB	TILE BACKER BOARD
AFF	ABOVE FINISH FLOOR	DF	DRINKING FOUNTAIN	HC	HANDICAPPED	PLWD	PLYWOOD	T&G	TONGUE AND GROOVE
AL	ALUMINUM	DP	DEEP	HDR	HEADER	PNL	PANEL	TO	TO OF
ALT	ALTERNATE	DR	DOOR	HM	HOLLOW METAL	PORC. TILE	PORCELAIN TILE	TOW	TOP OF WALL
ANOD	ANODIZED	DS	DOWNSPOUT	HORIZ	HORIZONTAL	PR	PAIR	TPD	TOILET PAPER DISPENSER
AP	ACOUSTICAL WALL PANEL	DWG	DRAWING	HT	HEIGHT	PSF	POUNDS PER SQUARE FOOT	TSCD	TOILET SEAT COVER DISPENSER
APPROX	APPROXIMATE	E	EAST	HVAC	HEATING/VENTILATING/	PSI	POUNDS PER SQUARE INCH	TT	TIRE TREAD
ARCH	ARCHITECT (-URAL)	(E)	EXISTING		AIR CONDITIONING	PT	PAINT, PRESSURE TREATED	TYP	TYPICAL
AW	ACOUSTICAL WALL	EA	EACH	ILO	IN LIEU OF	PTD	PAPER TOWEL DISPENSER	UNO	UNLESS NOTED OTHERWISE
AWF	ACOUSTICAL WALL FABRIC	EJ	EXPANSION JOINT	INSUL	INSULATION	QT	QUARTZ TILE	U/S	UNDERSIDE
BLDG	BUILDING	EL	ELEVATION	INT	INTERIOR	R	RISER, RADIUS	VB	VAPOR BARRIER
BM	BEAM	ELEC	ECLECTRIC (-AL)	JNT	JOINT	RB	RESILIENT BASE	VCT	VINYL COMPOSITION TILE
BOD	BOTTOM OF DECK	EP	ENAMEL PAINT	KD	KNOCK DOWN	RD	ROOF DRAIN	VERT	VERTICAL
BOT	BOTTOM	EQ	EQUAL	LAV	LAVATORY	RO	ROUGH OPENING	VGF	VINYL GYM FLOORING
BTWN	BETWEEN	EW	EACH WAY	MCFP	MULTI-COLORED FINISH	RR	RESTROOM	VIF	VINYL INDUSTRIAL FLOORING
CB	CATCH BASIN	EXG	EXISTING		PAINT SYSTEM	RSF	RUBBER SHEET FLOORING	VR	VAPOR RETARDER
CBT	CABINET	EXP	EXPANSION	MDO	MEDIUM DENSITY	S	SOUTH	VT	VINYL TILE
CG	CORNER GUARD	EXT	EXTERIOR		OVERLAY PLYWOOD	SC	SOLID CORE	VWF	VINYL WALL FABRIC
CJ	CONTROL JOINT	FA	FIRE ALARM	MECH	MECHANIC (-AL)	SCU	STRUCTURAL CLAY UNIT	W	WEST
CL	CENTERLINE	FD	FLOOR DRAIN	MFR	MANUFACTURE (-R)	SD	SOAP DISPENSER	W/C	WATER CLOSET
CLG	CEILING	FE	FIRE EXTINGUISHER	MIN	MINIMUM	SDSV	STATIC DISIPATIVE SHEET VINYL	WD	WOOD
CLR	CLEAR (-ANCE)	FEC	FIRE EXTINGUISHER CABINET	MISC	MISCELLANEOUS	SF	SPECIALTY FINISH	W/D	WASHER & DRYER
CMT	CERAMIC MOSAIC TILE	FF	FACTORY FINISH, FINISH FLOOR	MRGB	MOISTURE RESISTANT	SFGL	SAFETY GLASS	WDO	WINDOW
CMU	CONCRETE MASONRY UNIT	FIN	FINISH (-ED)		GYPSUM BOARD	SHTG	SHEATHING	WF	WALL FABRIC
CO	CLEAN OUT	FLR	FLOOR (-ING)	MTL	METAL	SIM	SIMILAR	WFV	WOOD FACE VENEER
COL	COLUMN	FND	FOUNDATION	N	NORTH	SL	SLOPE	WG	WIRE GUARD
CONC	CONCRETE	FOC	FACE OF CONCRETE	(N)	NEW	SND	SANITARY NAPKIN DISPENSER	WGL	WIRED GLASS
CONT	CONTINUOUS, CONTINUE	FRP	FIBERGLASS REINFORCED	NA, N/A	NOT APPLICABLE	SP	SPACE (-S)	WM	WIRE MESH
CORR	CORRIDOR		PLASTIC PANEL	NIC	NOT IN CONTRACT	SPEC	SPECIFICATION	W/O	WITHOUT
CP	CARPET	FRVR	FLAME RESISTANT VAPOR BARRIER	NDU	SANITARY NAPKIN	SQ	SQUARE	WOC	WALK-OFF CARPET
CS	CONCRETE SLAB, SEALED	FT	FOOT, FEET		DISPOSAL UNIT	S/S	STAINLESS STEEL	WP	WATERPROOFING
CT	CERAMIC TILE	FTG	FOOTING	NOM	NOMINAL	ST	STAIN	WPS	WALL PROTECTION SYSTEM
CTJ	CONTROL JOINT	FWC	FABRIC WALL COVERING	NTS	NOT TO SCALE	STL	STEEL	WR	WATER RESISTANT
CTR	COUNTER (-TOP)	GA	GAUGE	OC	ON CENTER	STR	STRUCTURE (-AL)	WRGB	WATER RESISTANT GYPSUM
DBL	DOUBLE	GALV	GALVANIZED	OD	OUTSIDE DIAMETER	STRG	STORAGE		WALLBOARD
DET	DETAIL	GH	GARMENT HOOK	OPP	OPPOSITE	SV	SHEET VINYL FLOORING	WWF	WELDED WIRE FABRIC
		GMM	GLASS MESH MORTAR BOARD	PCMU	PRE-FACED CMU			W/	WITH

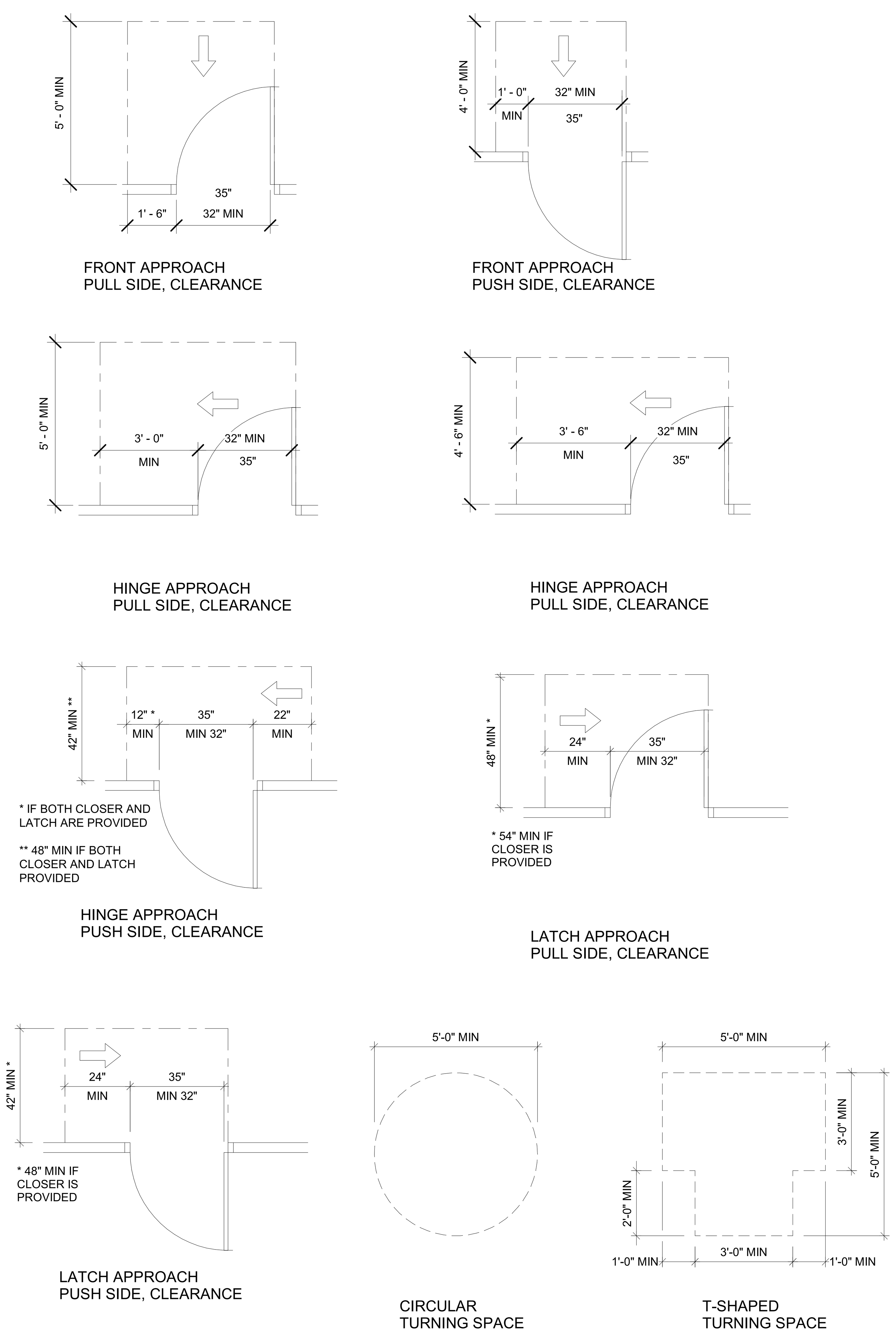
LICENSED ARCHITECT AR-985708
R. Colby Ricks
R. COLBY RICKS
STATE OF IDAHO
10/8/2024

DATE	
10/29/24	ADD #2 PER DOPL COMMENTS

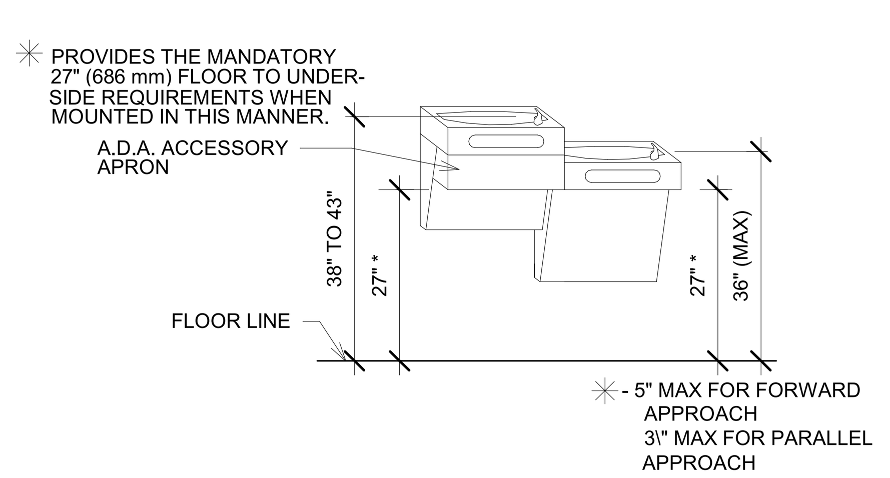
AN ADDITION FOR:
KIMBERLY SCHOOL DISTRICT
3682 N 3450 E, Kimberly, ID 83341
TITLE SHEET

Laughlin Ricks Architecture
architecture/planning
134 3RD Ave East, * Twin Falls, Idaho 83301
(208) 736-8050

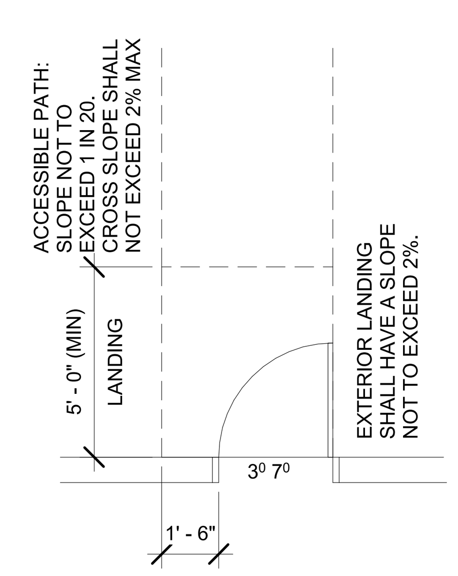
DATE: 10/8/2024
NM Drawn — RCR Checked
#23067 PROJECT #



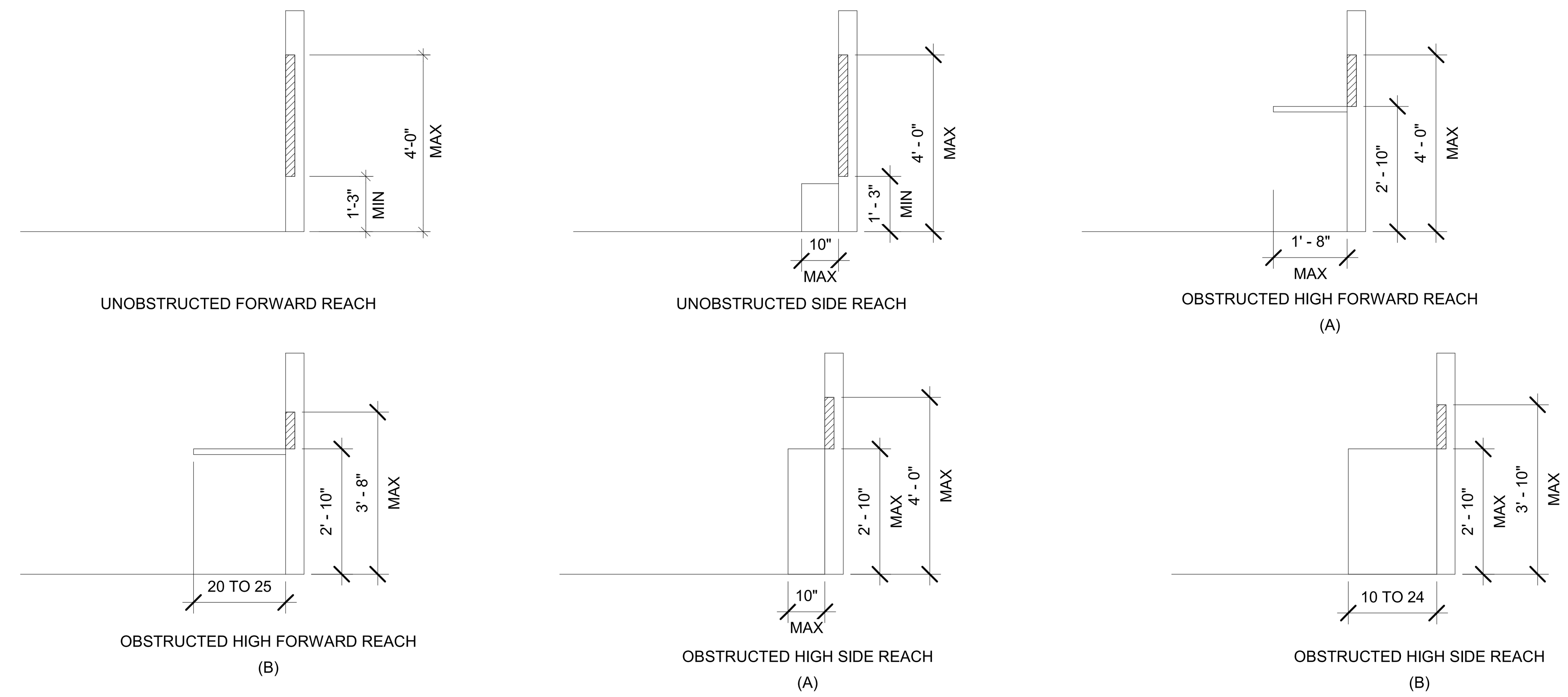
1 DOOR CLEARANCE REQUIREMENTS
3/8" = 1'-0"



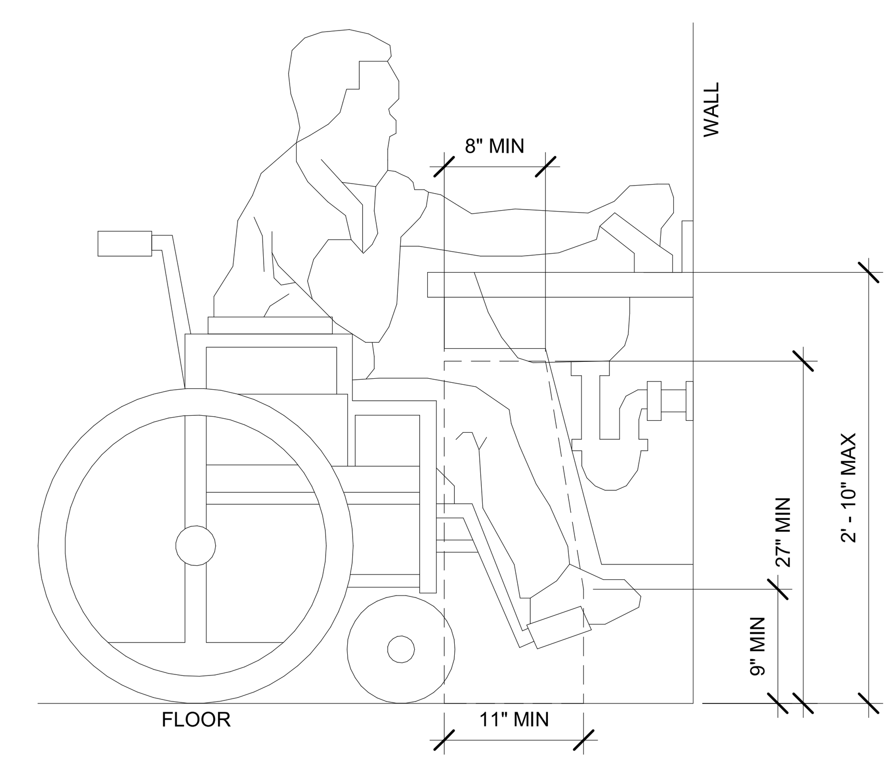
2 DRINKING FOUNTAIN DETAIL
1/4" = 1'-0"



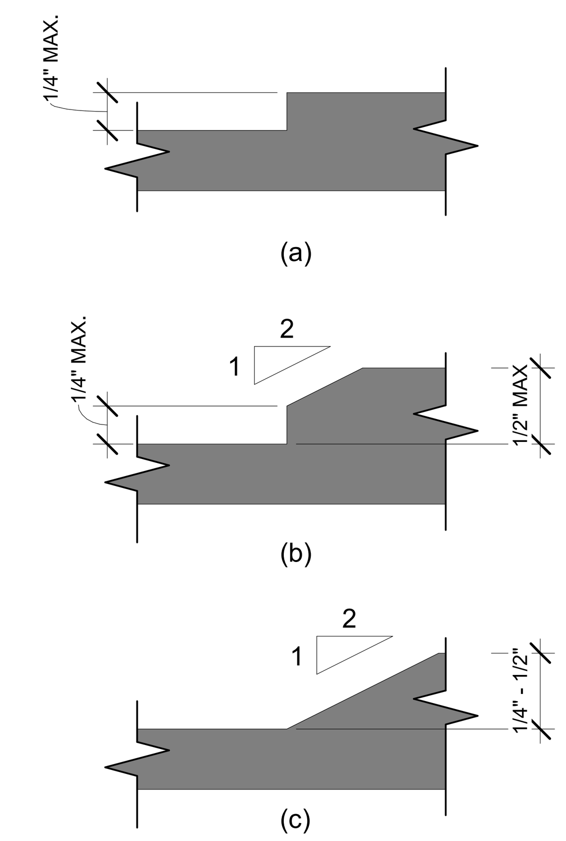
3 ACCESSIBLE ENTRANCE
1/4" = 1'-0"



4 OPERABLE PARTS & REACH RANGES
3/8" = 1'-0"



5 LAVATORY CLEARANCE
1" = 1'-0"



6 VERTICAL CHANGES IN LEVEL
12" = 1'-0"

LICENSED ARCHITECT AR-985708
R. COLBY RICKS
STATE OF IDAHO
10/8/2024

DATE	
10/29/24	ADD #2 PER DOPL COMMENTS

AN ADDITION FOR:
KIMBERLY SCHOOL DISTRICT
3682 N 3450 E, Kimberly, ID 83341
ACCESSIBILITY DIAGRAMS

Laughlin Ricks Architecture
architecture/planning
134 3RD Ave East, * Twin Falls, Idaho 83301
(208) 736-8050

DATE: 10/8/2024

NM	RCR
Drawn	Checked

#23067
PROJECT #

A0-1

PLUMBING FIXTURE COUNT
250 TOTAL OCCUPANTS
1 PER 50 REQUIRED
250/50 = 5 FIXTURES REQUIRED, 5 PROVIDED

LAVATORIES
MALE/FEMALE 1 PER 50 REQUIRED
250/50 = 5 FIXTURES REQUIRED, 9 PROVIDED

DRINKING FOUNTAINS
1 PER 100 REQUIRED
250/100 = 3 FIXTURES REQUIRED, 6 PROVIDED

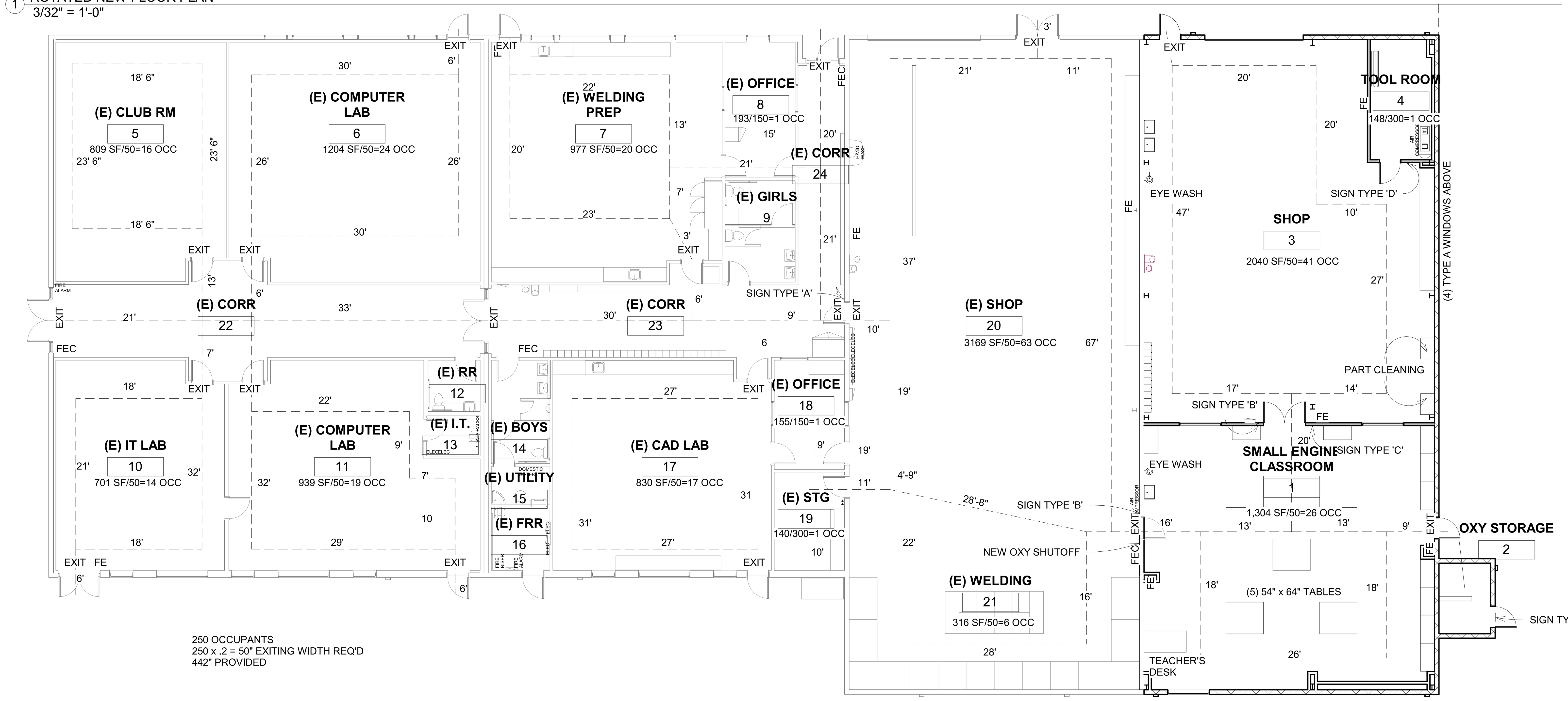
SERVICE SINKS
1 FIXTURE REQUIRED, 1 PROVIDED

ACCESSORY STORAGE AREA DIVIDE BY 300 GROSS

BUSINESS AREAS DIVIDE BY 150 GROSS

VOCATIONAL SHOP AND VOCATIONAL CLASSROOM FACILITY
EDUCATIONAL SHOPS AND OTHER VOCATIONAL ROOM AREAS DIVIDE BY 50 NET

1 ROTATED NEW FLOOR PLAN
3/32" = 1'-0"



250 OCCUPANTS
250 x .2 = 50" EXITING WIDTH REQ'D
442" PROVIDED

2 EXITING
3/32" = 1'-0"

LICENSED ARCHITECT
AR-985708
R. COLBY RICKS
STATE OF IDAHO
10/8/2024

DATE	
10/29/24	ADD #2 PER DOPL COMMENTS

AN ADDITION FOR:
KIMBERLY SCHOOL DISTRICT
3682 N 3450 E, Kimberly, ID 83341
ROTATED OVERALL FLOOR PLAN

Laughlin Ricks Architecture
architecture/planning
134 3RD Ave East, * Twin Falls, Idaho 83301
(208) 736-8050

DATE: 10/8/2024
NM Drawn
RCR Checked
#23067 PROJECT #

A2-0

ARW ENGINEERS
structural consultants

Structural Calculations

For

Kimberly AG Shop Addition

24091

October 8, 2024

PROFESSIONAL ENGINEER
LICENSED
10/8/24
16183
STATE OF IDAHO
ROBERT M. MOYLE

STRUCTURAL CALCULATIONS

FOR

Kimberly AG Shop Addition

Client: Laughlin Ricks Architecture

Project Number: 24091

DESIGN CRITERIA

GOVERNING CODE: IBC 2018

GENERAL: Risk Category = III

SEISMIC: Seismic Design Category = B

I_E = 1.25 R = 5

S_{DS} = 0.207 S_{D1} = 0.132

WIND: Basic Wind Speed = 109 mph

Exposure Classification = C

DESIGN LOADS

ROOF: DL = 20 PSF SL = 23.1 PSF

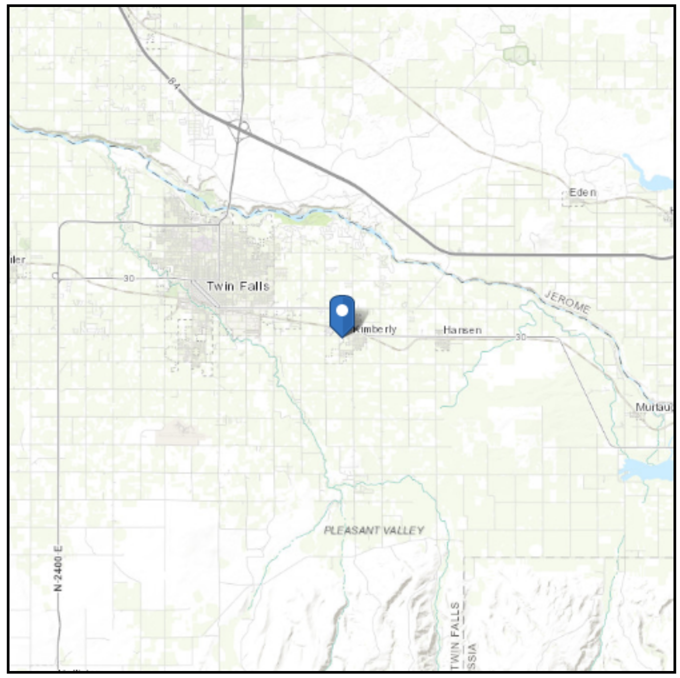
ROOF FRAMING

ASCE
AMERICAN SOCIETY OF CIVIL ENGINEERS

ASCE Hazards Report

Address:
885 Center St W
Kimberly, Idaho
83341

Standard:	ASCE/SEI 7-16	**Latitude:**	42.532247
Risk Category:	II	**Longitude:**	-114.372886
Soil Class:	D - Default (see Section 11.4.3)	**Elevation:**	3919.9542000874803 ft (NAVD 88)



Wind

Results:

Wind Speed	103 Vmph
10-year MRI	72 Vmph
25-year MRI	78 Vmph
50-year MRI	83 Vmph
100-year MRI	88 Vmph

Data Source:	ASCE/SEI 7-16, Fig. 26.5-1B and Figs. CC.2-1–CC.2-4, and Section 26.5.2
Date Accessed:	Tue Sep 24 2024

Value provided is 3-second gust wind speeds at 33 ft above ground for Exposure C Category, based on linear interpolation between contours. Wind speeds are interpolated in accordance with the 7-16 Standard. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (annual exceedance probability = 0.00143, MRI = 700 years).

Site is not in a hurricane-prone region as defined in ASCE/SEI 7-16 Section 26.2.

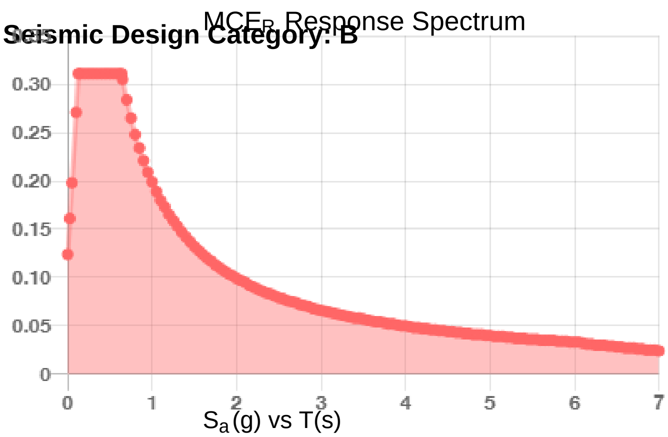
Seismic

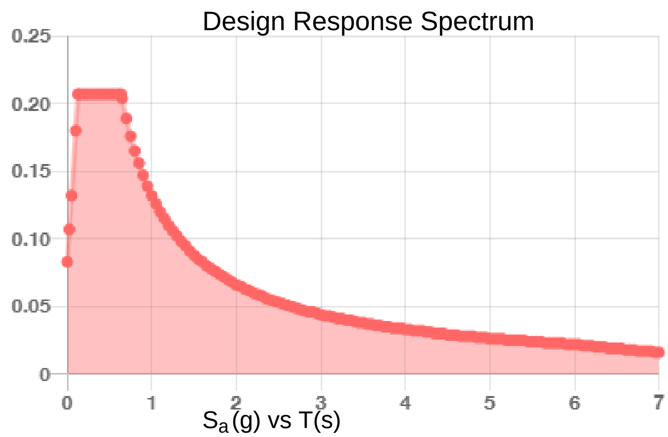
Site Soil Class: D - Default (see Section 11.4.3)

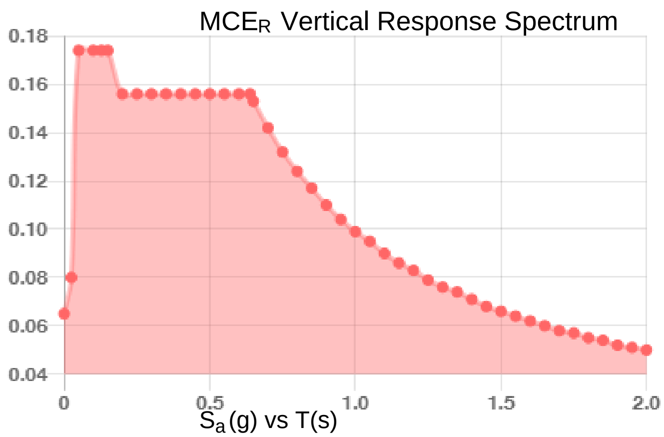
Results:

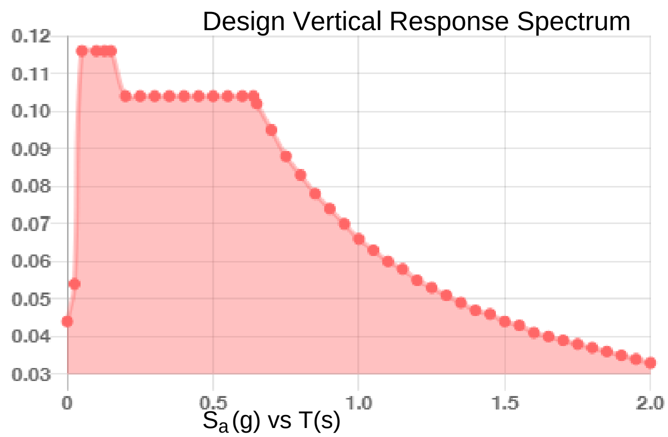
S_S :	0.194	S_{D1} :	0.132
S_1 :	0.083	T_L :	6
F_a :	1.6	PGA :	0.086
F_v :	2.4	PGA_M :	0.137
S_{MS} :	0.311	F_{PGA} :	1.6
S_{M1} :	0.199	I_e :	1
S_{DS} :	0.207	C_v :	0.7

Seismic Design Category: B









Data Accessed: **Tue Sep 24 2024**

Date Source:

USGS Seismic Design Maps based on ASCE/SEI 7-16 and ASCE/SEI 7-16 Table 1.5-2. Additional data for site-specific ground motion procedures in accordance with ASCE/SEI 7-16 Ch. 21 are available from USGS.

Snow

Results:

Ground Snow Load, p_g : 14 lb/ft^2

Mapped Elevation: 3920.0 ft

Data Source:

Date Accessed: Tue Sep 24 2024

Statutory requirements of the Authority Having Jurisdiction are not included.

Snow load values are mapped to a 0.5 mile resolution. This resolution can create a mismatch between the mapped elevation and the site-specific elevation in topographically complex areas. Engineers should consult the local authority having jurisdiction in locations where the reported ‘elevation’ and ‘mapped elevation’ differ significantly from each other.

Rain

Results:

15-minute Precipitation Intensity: 3.34 in./h

60-minute Precipitation Intensity: 1.21 in./h

Data Source: NOAA National Weather Service, Precipitation Frequency Data Server, Atlas 14 (https://www.nws.noaa.gov/oh/hdsc/)

Date Accessed: Tue Sep 24 2024

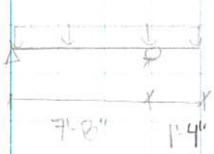
The ASCE Hazard Tool is provided for your convenience, for informational purposes only, and is provided “as is” and without warranties of any kind. The location data included herein has been obtained from information developed, produced, and maintained by third party providers; or has been extrapolated from maps incorporated in the ASCE standard. While ASCE has made every effort to use data obtained from reliable sources or methodologies, ASCE does not make any representations or warranties as to the accuracy, completeness, reliability, currency, or quality of any data provided herein. Any third-party links provided by this Tool should not be construed as an endorsement, affiliation, relationship, or sponsorship of such third-party content by or from ASCE.

ASCE does not intend, nor should anyone interpret, the results provided by this Tool to replace the sound judgment of a competent professional, having knowledge and experience in the appropriate field(s) of practice, nor to substitute for the standard of care required of such professionals in interpreting and applying the contents of this Tool or the ASCE standard.

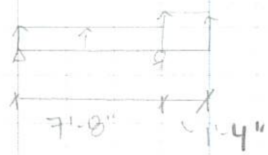
In using this Tool, you expressly assume all risks associated with your use. Under no circumstances shall ASCE or its officers, directors, employees, members, affiliates, or agents be liable to you or any other person for any direct, indirect, special, incidental, or consequential damages arising from or related to your use of, or reliance on, the Tool or any information obtained therein. To the fullest extent permitted by law, you agree to release and hold harmless ASCE from any and all liability of any nature arising out of or resulting from any use of data provided by the ASCE Hazard Tool.

OXY Storage Room:

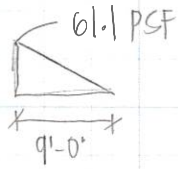
* JOIST DESIGN:



OK



DL = 15 PSF
CL = 23.1 PSF,
DRIFT:



Total = 15 + 23.1 = 38.1 PSF
+ DRIFT

W = 68.3 PSF (zone 3)
SOFFIT: 64 PSF

W = 0.6D − 0.6W
= 0.6(8 PSF) − 0.6(68.3 PSF)
= −36.2 PSF

* WIND UPLIFT:

SOFFIT:

$q_h = 0.00256(0.85)(1.0)(0.85)(0.87)(109^2)$
$= 19.1$ PSF

$p = 19.1$ PSF $(-3.2 - 0.18) = -64$ PSF

→ 600S162-54 @ 16" O.C.

* connection design:

* ATTACHMENT TO Face OF CMU. R = 412# (ASD) (F2)

M = 412# (3") = 1236 #·in (M1)

CL600-68 CAP: $\frac{412\#}{2510\#} + \frac{1236\#\cdot in}{2435\#\cdot in} = 0.67$ ∴ OK

Anchordesign: −see next page−

ARW ENGINEERS structural consultants

8-Oct-24
6:04 AM

ASCE 7-16 SNOW DRIFT ANALYSIS

Version Date: May 17, 2021 Author: TMD

JOB TITLE: Kimberly AG Building
DRIFT LOCATION: At Oxy storage room
JOB #: 24091

DATA USED IN ANALYSIS

Ground Snow Load	P_g :	30	psf
Exposure Factor	C_e :	1.0	
Thermal Factor	C_t :	1.0	
Importance Factor	I_s :	1.1	
		r	
Roof Dead Load		15	psf
Joist Span		9	ft
Distance from P_m to P_z		0	ft
Joist Spacing		24	ft

Horizontal Dimension of High Roof	L_u :	85	ft
Horizontal Dimension of Low Roof	L_l :	9	ft
Vertical Distance Between Upper and Lower Roof	h_r :	10	ft

Type of Roof Step : Normal

RESULTS OF ANALYSIS

Roof Snow Load	P_f :	23.1	psf
Depth of Uniform Load	h_b :	1.29	ft
Density of Snow	D :	17.90	pcf
	$h_c = (h_r - h_b)$:	8.71	ft
	h_c/h_b :	6.75	**> 0.2 Drifting Needs to be Considered**

Height of Snow Drift

h_d :	3.41	ft	Leeward	**Controls Width Drift Calc**
h_d :	0.59	ft	Windward	

Length of Snow Drift	w :	9.0	ft
Length of Drift on Joist	w_j :	**9.0**	ft
Height of Snow Drift	h_d :	**3.41**	ft
	P_d :	**61.1**	psf

Equation used (4hd)

Intensity of Drift at	P_z :	**61.1**	psf
Intensity of Drift at	P_e :	**0.0**	psf

DRIFT CAUSING TRIANGULAR LOAD ON JOISTS

	Joist Reactions (kips) based on 24 foot spacing		**Joist Reactions (kips) based on 1 foot spacing**	
	Left End	Right End	Left End	Right End
DL	1.62	1.62	0.07	0.07
SL	2.49	2.49	0.10	0.10
DRIFT	4.40	2.20	0.18	0.09
TOTAL	8.51	6.31	0.35	0.26

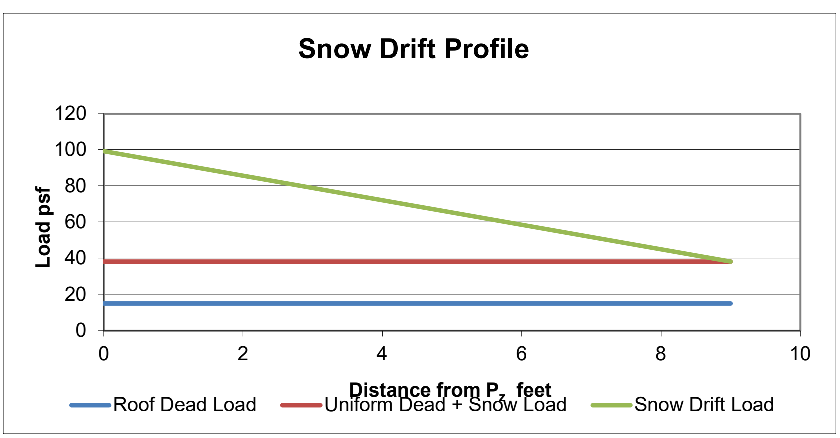
Maximum Moment 16.77 k-ft
occurring @ 4.14 ft from Left End of Joist

Equivalent Uniform Joist Load (plf)

DL	360	plf
SL	1297	plf
TOTAL	1657	plf

Drift Information

Maximum weight of drift :	6599	lbs
center of gravity from Left End :	3.00	feet



Definitions

P_m : Maximum Magnitude of Snow Drift
P_z : Maximum Magnitude of Snow Drift at 0 feet away from Pm
P_e : Magnitude of Drift at End of Joist

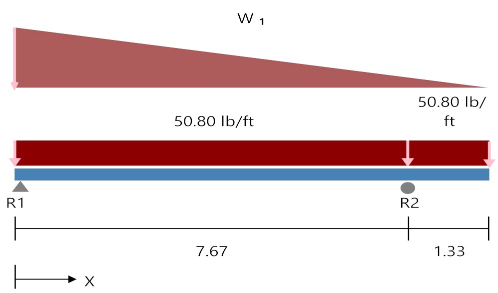
Equation of Snow Drift Line

y = -6.78918732570663 * X + 61.1026859

Distance X is in feet from P_m

Project Name: 24091 - Kimberly AG Shop
Model: Roof Framing - gravity
Code: 2012 NASPEC [AISI S100-2012]

Date: 10/07/2024
Simpson Strong-Tie® CFS Designer™ 5.0.0.1



Section: 600S162-54 (50 ksi) @ 16" o.c. Single C Stud (punched)
Maxo = 2527.1 ft-lb **Va =** 2822.9 lb **I =** 2.86 in^4

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Bridging Connectors - Design Method =AISI S100

Span	Axial KyLy, KtLt	Flexual, Distortional	Connector	Stress Ratio
Span	NA	None, 92.0"	N/A	-
Right Cant.	NA	None, 16.0"	N/A	-

Web Crippling

Support	Load (lb)	Bearing (in)	Pa (lb)	M (ft-lbs)	Max Int.	Stiffener?
R1	412.1	1.00	598.9	0.0	0.36	NO
R2	411.8	1.00	1295.2	48.7	0.19	NO

"*" after support means punched near support

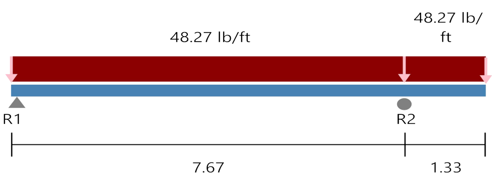
Sloped/Partial Loads

	X-Start (ft)	W-Start (lb/ft)	X-End(ft)	W-End (lb/ft)
W1	0.00	81.50	9.00	0.00

	Code Check	Required	Allowed	Interaction	Notes
Span	Max. Axial, lbs	0.0(t)	-	0%	KΦ=0.00 lb-in/in Max KL/r = N/A
	Max. Shear, lbs	412.1	1947.4	21%	Shear (Punched)
	Max. Moment (MaFy, Ma-dist), ft-lbs	696.8	2158.3	32%	Ma-dist (control),KΦ=0.00 lb-in/in
	Moment Stability, ft-lbs	696.8	1035.3	67%	
	Shear/Moment	0.28	1.00	28%	Shear 0.3, Moment 696.8
	Axial/Moment	0.67	1.00	67%	Axial 0.0(c), Moment 696.8
	Deflection Span, in	0.086	--meets L/1066--		
Right Cant.	Max. Axial, lbs	0.0(t)	-	0%	KΦ=0.00 lb-in/in Max KL/r = N/A
	Max. Shear, lbs	75.8	1947.4	4%	Shear (Punched)
	Max. Moment (MaFy, Ma-dist), ft-lbs	48.7	2158.3	2%	Ma-dist (control),KΦ=0.00 lb-in/in
	Moment Stability, ft-lbs	30.6	2313.4	1%	
	Shear/Moment	0.04	1.00	4%	Shear 75.8, Moment 48.7
	Axial/Moment	0.02	1.00	2%	Axial 0.0(c), Moment 48.7
	Deflection Cant., in	0.045	--meets L/711--		2 x Cantilever

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie Connector	Connector Interaction	Anchor Interaction
R1	0.0	412.1	By Others & Anchorage Designed by Engineer	NA	NA
R2	0.0	411.9	By Others & Anchorage Designed by Engineer	NA	NA

* Reference catalog for connector and anchor requirements notes as well as screw placement requirements



Section: 600S162-54 (50 ksi) @ 16" o.c. Single C Stud (punched)
Maxo = 2527.1 ft-lb **Va =** 2822.9 lb **I =** 2.86 in^4

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Bridging Connectors - Design Method =AISI S100

Span	Axial KyLy, KtLt	Flexual, Distortional	Connector	Stress Ratio
Span	NA	None, 92.0"	N/A	-
Right Cant.	NA	None, 16.0"	N/A	-

Web Crippling

Support	Load (lb)	Bearing (in)	Pa (lb)	M (ft-lbs)	Max Int.	Stiffener?
R1	179.4	1.00	598.9	0.0	0.16	NO
R2	255.0	1.00	1295.2	42.9	0.12	NO

"*" after support means punched near support

	Code Check	Required	Allowed	Interaction	Notes
Span	Max. Axial, lbs	0.0(t)	-	0%	KΦ=0.00 lb-in/in Max KL/r = N/A
	Max. Shear, lbs	190.6	1947.4	10%	Shear (Punched)
	Max. Moment (MaFy, Ma-dist), ft-lbs	333.5	2158.3	15%	Ma-dist (control),KΦ=0.00 lb-in/in
	Moment Stability, ft-lbs	333.5	1035.5	32%	
	Shear/Moment	0.13	1.00	13%	Shear 0.0, Moment 333.5
	Axial/Moment	0.32	1.00	32%	Axial 0.0(c), Moment 333.5
	Deflection Span, in	0.041	--meets L/2230--		
Right Cant.	Max. Axial, lbs	0.0(t)	-	0%	KΦ=0.00 lb-in/in Max KL/r = N/A
	Max. Shear, lbs	64.3	1947.4	3%	Shear (Punched)
	Max. Moment (MaFy, Ma-dist), ft-lbs	42.9	2158.3	2%	Ma-dist (control),KΦ=0.00 lb-in/in
	Moment Stability, ft-lbs	27.4	2313.4	1%	
	Shear/Moment	0.04	1.00	4%	Shear 64.3, Moment 42.9
	Axial/Moment	0.02	1.00	2%	Axial 0.0(c), Moment 42.9
	Deflection Cant., in	0.021	--meets L/1498--		2 x Cantilever

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie Connector	Connector Interaction	Anchor Interaction
R1	0.0	179.4	By Others & Anchorage Designed by Engineer	NA	NA
R2	0.0	255.0	By Others & Anchorage Designed by Engineer	NA	NA

* Reference catalog for connector and anchor requirement notes as well as screw placement requirements

OXY storage room (CONT'D):

* connection (cont'd):

TRY JOISTS @ 16" O.C.:

Reaction = 393#

$$T = \frac{3(393\#(3"))}{5.5"} \simeq 643\#$$

→ 1/2" ϕ screw anchor ok w/ 4 1/2" min embedment
4" clear edge distance
→ see printout.

www.hilti.com

Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Kimberly AG shop - joist into CMU face	Date:	10/8/2024
Fastening point:			

Specifier's comments:

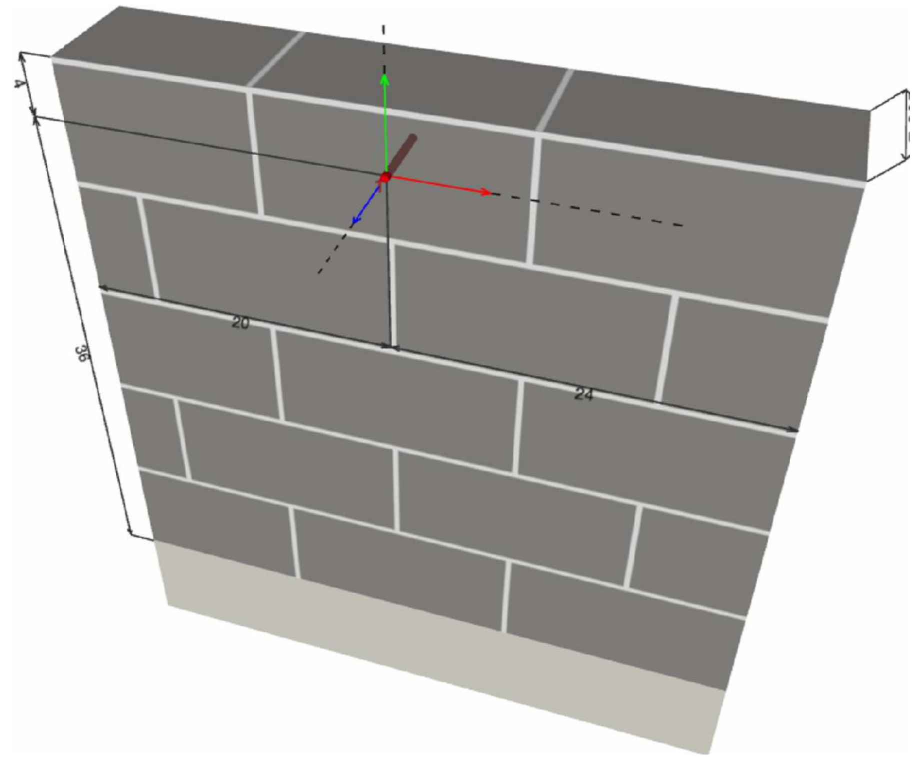
1 Input data

Anchor type and diameter:	**HY 270 + threaded rod 5.8 1/2**
Item number:	385424 HAS 5.8 1/2"x6-1/2" (element) / 2194247 HIT-HY 270 (adhesive)
Specification text:	Hilti HIT-V 5.8 threaded rod with HIT-HY 270 injection mortar with 4.5 in embedment hef, 1/2, Steel galvanized, Hammer drilled installation per instruction for use
Effective embedment depth:	h_{ef} = 4.500 in.
Material:	5.8
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ASD Masonry
Stand-off installation:	
Profile:	
Base material:	Grout-filled CMU, L x W x H: 16.000 in. x 8.000 in. x 8.000 in.; Joints: vertical: 0.375 in.; horizontal: 0.375 in. Base material temperature: 68 °F
Installation:	Face installation
Seismic loads	no

www.hilti.com

Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Kimberly AG shop - joist into CMU face	Date:	10/8/2024
Fastening point:			

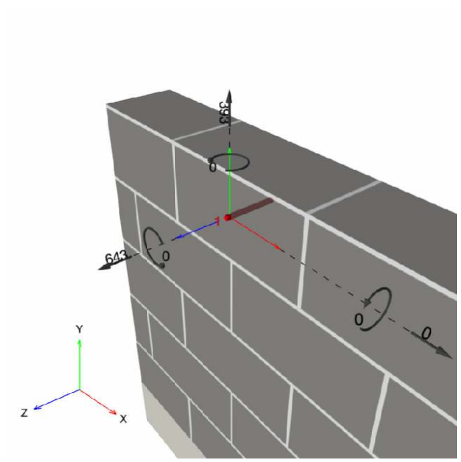
Geometry [in.]



www.hilti.com

Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Kimberly AG shop - joist into CMU face	Date:	10/8/2024
Fastening point:			

Geometry [in.] & Loading [lb, in.lb]



1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 643; V_x = 0; V_y = 393; M_x = 0; M_y = 0; M_z = 0;	no	59

2 Load case/Resulting anchor forces

Load case: Service loads

Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	643	393	0	393

www.hilti.com

Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Kimberly AG shop - joist into CMU face	Date:	10/8/2024
Fastening point:			

3 Tension load (Most utilized anchor 1)

	Load P_s [lb]	Capacity P_t [lb]	Utilization $\beta_P = P_s/P_t$ [%]	Status
Steel strength	643	4,700	14	OK
Bond strength	643	1,547	42	OK

3.1 Steel strength

$P_{t,s}$ = Value   refer to Hilti Technical Data
$P_{t,s} \geq P_s$

Results

$P_{t,s}$ [lb]	P_s [lb]
4,700	643

3.2 Bond strength

$P_{t,b,Base}$ = Value   refer to Hilti Technical Data
$P_{t,b} = P_{t,b,Base} \cdot f_{red,E} \cdot f_{red,s} \cdot f_{red,Temp} \cdot f_{red,Bedjoint}$
$P_{t,b} \geq P_s$

Variables

c_{min} [in.]	c_{cr} [in.]	s_{min} [in.]	s_{cr} [in.]	Temperature [°F]
4.000	20.000	4.000	18.000	68

Results

$P_{t,b}$ [lb]	$P_{t,b,Base}$ [lb]	P_s [lb]	$f_{red,E}$	$f_{red,S}$	$f_{red,Temp}$	$f_{red,Bedjoint}$
1,547	2,035	643	0.760	1.000	1.000	1.000

Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Kimberly AG shop - joist into CMU face	Date:	10/8/2024
Fastening point:			

4 Shear load (Most utilized anchor 1)

	Load V_s [lb]	Capacity V_t [lb]	Utilization $\beta_V = V_s/V_t$ [%]	Status
Steel strength	393	2,420	17	OK
Bond strength para and perp, (Dir. y+)[1]	-	-	54	OK

[1]Shear utilization may result from parallel and perpendicular shear (see details)

4.1 Steel strength

$V_{t,s}$ = Value refer to Hilti Technical Data
$V_{t,s} \geq V_s$

Results

$V_{t,s}$ [lb]	V_s [lb]
2,420	393

4.2 Bond strength parallel

$V_{t,b,Base||}$ = Value refer to Hilti Technical Data
$V_{t,b,||} = V_{t,b,Base||} \cdot f_{red,E,||} \cdot f_{red,s,||} \cdot f_{red,Temp}$
$V_{t,b,||} \geq V_{s,||}$

Variables

c_{min} [in.]	c_{cr} [in.]	s_{min} [in.]	s_{cr} [in.]	Temperature [°F]
4.000	12.000	4.000	18.000	68

Results

$V_{t,b,||}$ [lb]	$V_{t,b,Base||}$ [lb]	$V_{s,||}$ [lb]	$f_{red,E,||}$	$f_{red,S,||}$	$f_{red,Temp}$	Utilization $\beta_{V,||}$ [%]
0	1,495	0	0.000	0.000	1.000	0

4.3 Bond strength perpendicular

$V_{t,b,Base,\perp}$ = Value refer to Hilti Technical Data
$V_{t,b,\perp} = V_{t,b,Base,\perp} \cdot f_{red,E,\perp} \cdot f_{red,s,\perp} \cdot f_{red,Temp}$
$V_{t,b,\perp} \geq V_{s,\perp}$

Variables

c_{min} [in.]	c_{cr} [in.]	s_{min} [in.]	s_{cr} [in.]	Temperature [°F]
4.000	12.000	4.000	18.000	68

Results

$V_{t,b,\perp}$ [lb]	$V_{t,b,Base,\perp}$ [lb]	$V_{s,\perp}$ [lb]	$f_{red,E,\perp}$	$f_{red,S,\perp}$	$f_{red,Temp}$	Utilization $\beta_{V,\perp}$ [%]
733	1,495	393	0.490	1.000	1.000	54

Company:		Page:	6
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Kimberly AG shop - joist into CMU face	Date:	10/8/2024
Fastening point:			

4.4 Shear interaction

$\beta_{V\|\|} = \frac{V_{s\|\|}}{V_{t\|\|}}$	$\beta_{V,\perp} = \frac{V_{s,\perp}}{V_{t,\perp}}$	δ	Utilization β_V [%]	Status
0.000	0.536	1.667	54	OK

$\beta_V = \beta_{V||}^{\delta} + \beta_{V,\perp}^{\delta} <= 1.0$

5 Combined tension and shear loads (Most utilized anchor 1)

$\beta_P = \frac{P_s}{P_t}$	$\beta_{V\|\|} = \frac{V_{s\|\|}}{V_{t\|\|}}$	$\beta_{V,\perp} = \frac{V_{s,\perp}}{V_{t,\perp}}$	α	Utilization $\beta_{P,V}$ [%]	Status
0.232	0.000	0.354	1.667	59	OK

$\beta_{P,V} = \beta_P^{\alpha} + \beta_{V||}^{\alpha} + \beta_{V,\perp}^{\alpha} <= 1.0$

6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- The min. sizes of the bricks, the masonry compressive strength, the type / strength of the mortar and the grout (in case of fully grouted CMU walls) has to fulfill the requirements given in the relevant ESR-approval or in the PTG.
- Only the local load transfer from the anchor(s) to the wall is considered, a further load transfer in the wall is not covered by PROFIS!
- Wall is assumed as being perfectly aligned vertically – checking required(!): Noncompliance can lead to significantly different distribution of forces and higher tension loads than those calculated by PROFIS. Masonry wall must not have any damages (neither visible nor not visible)! While installation, the positioning of the anchors needs to be maintained as in the design phase i.e. either relative to the brick or relative to the mortar joints.
- The effect of the joints on the compressive stress distribution on the plate / bricks was not taken into consideration.
- If no significant resistance is felt over the entire depth of the hole when drilling (e.g. in unfilled butt joints), the anchor should not be set at this position or the area should be assessed and reinforced. Hilti recommends the anchoring in masonry always with sieve sleeve. Anchors can only be installed without sieve sleeves in solid bricks when it is guaranteed that it has not any hole or void.
- The accessories and installation remarks listed on this report are for the information of the user only. In any case, the instructions for use provided with the product have to be followed to ensure a proper installation.
- The compliance with current standards (e.g. 2018, 2015, 2012, 2009 and 2006 IBC) is the responsibility of the user.
- Drilling method (hammer, rotary) to be in accordance with the approval!
- Masonry needs to be built in a regular way in accordance with state-of the art guidelines!

Fastening meets the design criteria!

Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Kimberly AG shop - joist into CMU face	Date:	10/8/2024
Fastening point:			

7 Installation data

Profile: -	Anchor type and diameter: HY 270 + threaded rod 5.8 1/2 Item number: 385424 HAS 5.8 1/2"x6-1/2" (element) / 2194247 HIT-HY 270 (adhesive)
Hole diameter in the fixture: -	Maximum installation torque: 90 in.lb
Plate thickness (input): -	Hole diameter in the base material: 0.562 in.
	Hole depth in the base material: 4.500 in.
Drilling method: Drilled in hammer mode	Minimum thickness of the base material: 7.625 in.

Hilti HIT-V 5.8 threaded rod with HIT-HY 270 injection mortar with 4.5 in embedment hef, 1/2, Steel galvanized, Hammer drilled installation per instruction for use

Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	0.000	20.000	24.000	36.000	4.000

Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Kimberly AG shop - joist into CMU face	Date:	10/8/2024
Fastening point:			

8 Remarks; Your Cooperation Duties

- Any and all information and data contained in the Software concern solely the use of Hilti products and are based on the principles, formulas and security regulations in accordance with Hilti's technical directions and operating, mounting and assembly instructions, etc., that must be strictly complied with by the user. All figures contained therein are average figures, and therefore use-specific tests are to be conducted prior to using the relevant Hilti product. The results of the calculations carried out by means of the Software are based essentially on the data you put in. Therefore, you bear the sole responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear sole responsibility for having the results of the calculation checked and cleared by an expert, particularly with regard to compliance with applicable norms and permits, prior to using them for your specific facility. The Software serves only as an aid to interpret norms and permits without any guarantee as to the absence of errors, the correctness and the relevance of the results or suitability for a specific application.
- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

ARW ENGINEERS
structural consultants

Project No. 24091 | Sheet No. 15
Project Kimberly AG Shop
Prepared By ANB | Date 9/26/24

OXY STORAGE ROOM (CONT'D):

* JOIST ATTACHMENT :

OVER TOP OF CMU: UPLIFT = 255#
GRAVITY = 412#

CL CAP = 1067# SO OK.

1/2" Φ SCREW ANCHOR OK @ TOP OF WALL.

Hilti PROFIS Engineering 3.1.4

www.hilti.com

Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Kimberly AG shop - over top of wall	Date:	10/8/2024
Fastening point:			

Specifier's comments:

1 Input data

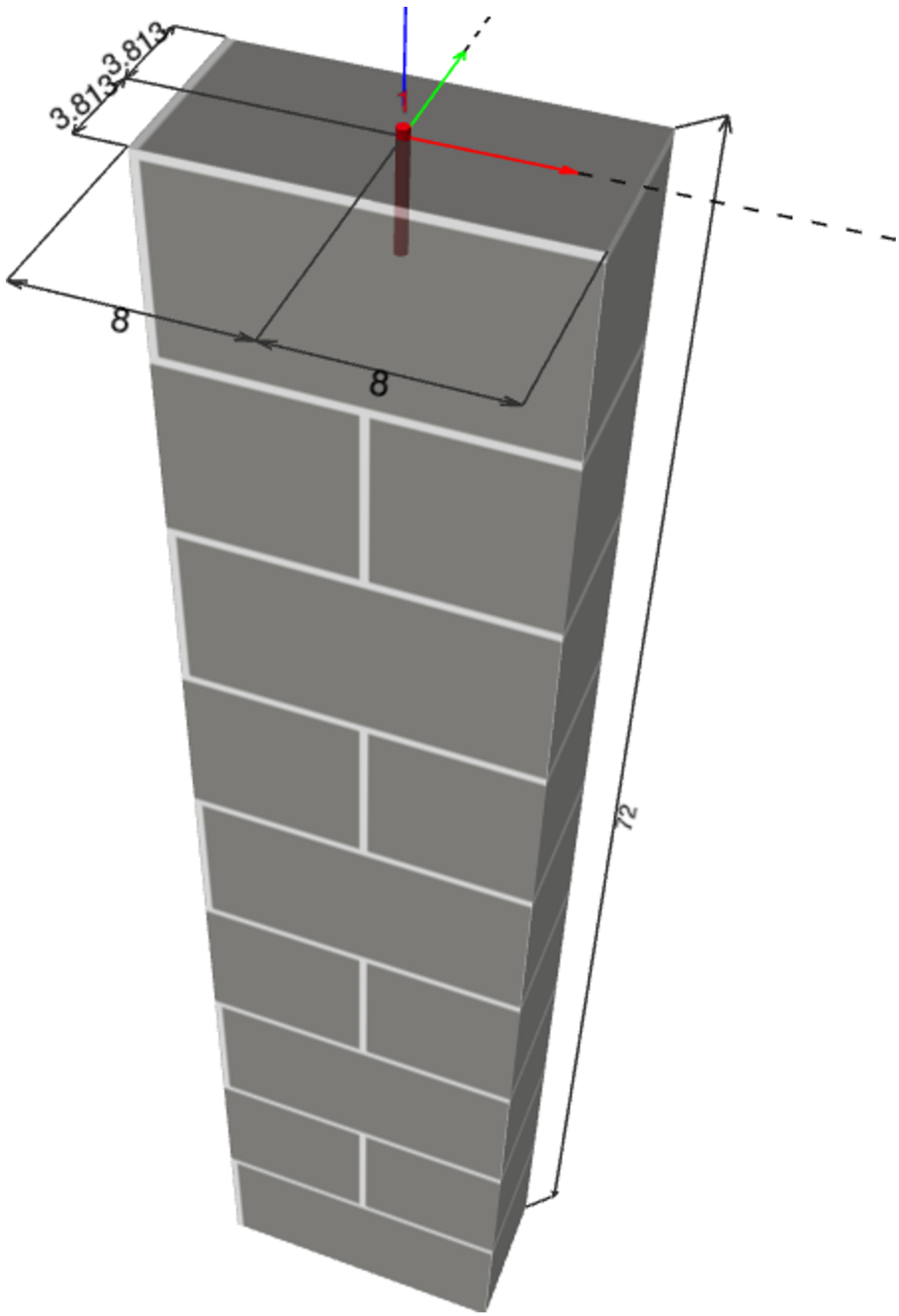
Anchor type and diameter:	**KWIK HUS-EZ (KH-EZ) 1/2 (4 1/4)**
Item number:	418075 KH-EZ 1/2"x4 1/2"
Specification text:	Hilti KH-EZ screw anchor with 4.25 in embedment, 1/2 (4 1/4), Steel galvanized, installation per instruction for use
Effective embedment depth:	h_{ef} = 4.250 in.
Material:	Carbon Steel
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ASD Masonry
Stand-off installation:	
Profile:	
Base material:	Grout-filled CMU, L x W x H: 16.000 in. x 8.000 in. x 8.000 in.; Joints: vertical: 0.375 in.; horizontal: 0.375 in. Base material temperature: 68 °F
Installation:	Top installation
Seismic loads	no

Hilti PROFIS Engineering 3.1.4

www.hilti.com

Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Kimberly AG shop - over top of wall	Date:	10/8/2024
Fastening point:			

Geometry [in.]

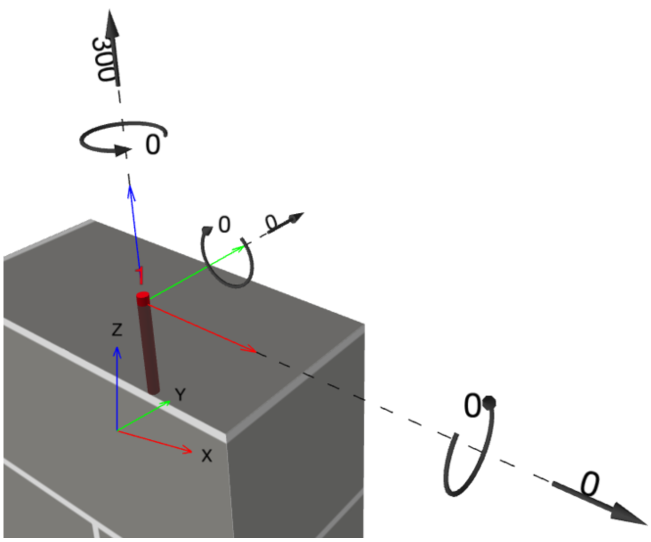


Hilti PROFIS Engineering 3.1.4

www.hilti.com

Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Kimberly AG shop - over top of wall	Date:	10/8/2024
Fastening point:			

Geometry [in.] & Loading [lb, in.lb]



1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 300; V_x = 0; V_y = 0; M_x = 0; M_y = 0; M_z = 0;	no	56

2 Load case/Resulting anchor forces

Load case: Service loads

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	300	0	0	0

Hilti PROFIS Engineering 3.1.4

www.hilti.com

Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Kimberly AG shop - over top of wall	Date:	10/8/2024
Fastening point:			

3 Tension load (Most utilized anchor 1)

	Load P_s [lb]	Capacity P_t [lb]	Utilization $\beta_P = P_s/P_t$ [%]	Status
Overall strength	300	540	56	OK

3.1 Overall strength

$P_{t,Base}$ = Value refer to Hilti Technical Data

$P_t = P_{t,Base} \cdot f_{red,E} \cdot f_{red,s} \cdot f_{red,Temp}$

$P_t \geq P_s$

Variables

c_{min} [in.]	c_{cr} [in.]	s_{min} [in.]	s_{cr} [in.]	Temperature [°F]
1.750	1.750	8.000	8.000	68

Results

P_t [lb]	$P_{t,Base}$ [lb]	P_s [lb]	$f_{red,E}$	$f_{red,S}$	$f_{red,Temp}$
540	540	300	1.000	1.000	1.000

Hilti PROFIS Engineering 3.1.4

www.hilti.com

Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Kimberly AG shop - over top of wall	Date:	10/8/2024
Fastening point:			

4 Shear load (Most utilized anchor 1)

	Load V_s [lb]	Capacity V_t [lb]	Utilization $\beta_V = V_s/V_t$ [%]	Status
Overall strength	N/A	N/A	N/A	N/A

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- The min. sizes of the bricks, the masonry compressive strength, the type / strength of the mortar and the grout (in case of fully grouted CMU walls) has to fulfill the requirements given in the relevant ESR-approval or in the PTG.
- Only the local load transfer from the anchor(s) to the wall is considered, a further load transfer in the wall is not covered by PROFIS!
- Wall is assumed as being perfectly aligned vertically – checking required(!): Noncompliance can lead to significantly different distribution of forces and higher tension loads than those calculated by PROFIS. Masonry wall must not have any damages (neither visible nor not visible)! While installation, the positioning of the anchors needs to be maintained as in the design phase i.e. either relative to the brick or relative to the mortar joints.
- The effect of the joints on the compressive stress distribution on the plate / bricks was not taken into consideration.
- If no significant resistance is felt over the entire depth of the hole when drilling (e.g. in unfilled butt joints), the anchor should not be set at this position or the area should be assessed and reinforced. Hilti recommends the anchoring in masonry always with sieve sleeve. Anchors can only be installed without sieve sleeves in solid bricks when it is guaranteed that it has not any hole or void.
- The accessories and installation remarks listed on this report are for the information of the user only. In any case, the instructions for use provided with the product have to be followed to ensure a proper installation.
- The compliance with current standards (e.g. 2018, 2015, 2012, 2009 and 2006 IBC) is the responsibility of the user.
- Drilling method (hammer, rotary) to be in accordance with the approval!
- Masonry needs to be built in a regular way in accordance with state-of the art guidelines!

Fastening meets the design criteria!

6 Installation data

Anchor type and diameter: KWIK HUS-EZ (KH-EZ) 1/2 (4 1/4)

Profile: -

Item number: 418075 KH-EZ 1/2"x4 1/2"

Hole diameter in the fixture: -

Maximum installation torque: 408 in.lb

Plate thickness (input): -

Hole diameter in the base material: 0.500 in.

Hole depth in the base material: 4.625 in.

Drilling method: Drilled in hammer mode

Minimum thickness of the base material: 7.625 in.

Hilti KH-EZ screw anchor with 4.25 in embedment, 1/2 (4 1/4), Steel galvanized, installation per instruction for use

Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	0.000	8.000	8.000	3.812	3.812

Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Kimberly AG shop - over top of wall	Date:	10/8/2024
Fastening point:			

7 Remarks; Your Cooperation Duties

- Any and all information and data contained in the Software concern solely the use of Hilti products and are based on the principles, formulas and security regulations in accordance with Hilti's technical directions and operating, mounting and assembly instructions, etc., that must be strictly complied with by the user. All figures contained therein are average figures, and therefore use-specific tests are to be conducted prior to using the relevant Hilti product. The results of the calculations carried out by means of the Software are based essentially on the data you put in. Therefore, you bear the sole responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear sole responsibility for having the results of the calculation checked and cleared by an expert, particularly with regard to compliance with applicable norms and permits, prior to using them for your specific facility. The Software serves only as an aid to interpret norms and permits without any guarantee as to the absence of errors, the correctness and the relevance of the results or suitability for a specific application.
- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

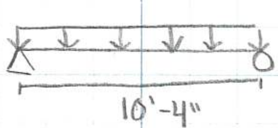
LVL Design

unit weight = 1000 lb   calc weight = 1500 lb

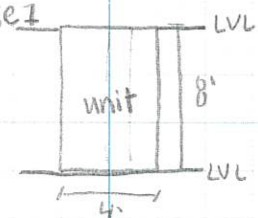
footprint of unit = 4' × 8'

span = 10'-4"

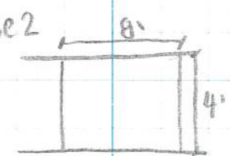
unit load = 1500 lb / (4' × 8') = 46.875 psf



case 1



case 2



case 1

$W = 46.875 \text{ psf} (8' \cdot \frac{2}{3}) = 250 \text{ plf}$

case 2

$W = 46.875 \text{ psf} (4' \cdot \frac{2}{3}) = 125 \text{ plf}$

case 1 will control design

applied uniform load across entire beam to be conservative

analyzed w/ Forte Web

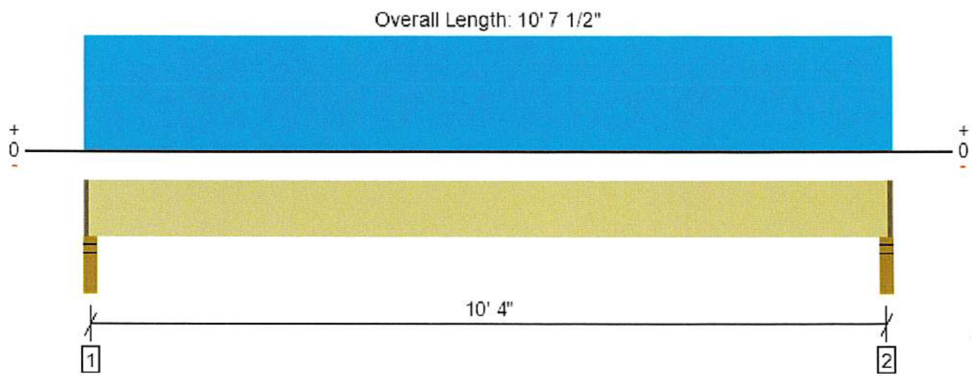
9 1/2" | 3/4" 2.0E Microlam LVL 2 ply

MEMBER REPORT

PASSED

Level, Unit Support LVL

2 piece(s) 1 3/4" x 9 1/2" 2.0E Microllam® LVL



Drawing is Conceptual. All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1457 @ 2"	4922 (2.25")	Passed (30%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	1098 @ 1' 1"	5686	Passed (19%)	0.90	1.0 D (All Spans)
Moment (Ft-lbs)	3438 @ 5' 3 3/4"	10597	Passed (32%)	0.90	1.0 D (All Spans)
Live Load Defl. (in)	0.011 @ 5' 3 3/4"	0.515	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.154 @ 5' 3 3/4"	0.686	Passed (L/802)	--	1.0 D + 1.0 S (All Spans)

- Deflection criteria: LL (L/240) and TL (L/180).
- Allowed moment does not reflect the adjustment for the beam stability factor.

Member Length : 10' 5"
System : Roof
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD
Member Pitch : 0/12

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Factored	
1 - Stud wall - DF	3.50"	2.25"	1.50"	1379	106	1485	1 1/4" Rim Board
2 - Stud wall - DF	3.50"	2.25"	1.50"	1379	106	1485	1 1/4" Rim Board

- Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	10' 5" o/c	
Bottom Edge (Lu)	10' 5" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	1 1/4" to 10' 6 1/4"	N/A	9.7	--	
1 - Uniform (PLF)	0 to 10' 7 1/2" (Top)	N/A	250.0	20.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
Olivia Murphy ARW Engineers (801) 782-6008 oliviam@arwengineers.com	

Weyerhaeuser

5/28/2024 7:41:51 PM UTC
ForteWEB v3.7, Engine: V8.4.0.40, Data: V8.1.5.0
File Name: 24091- Kimberly

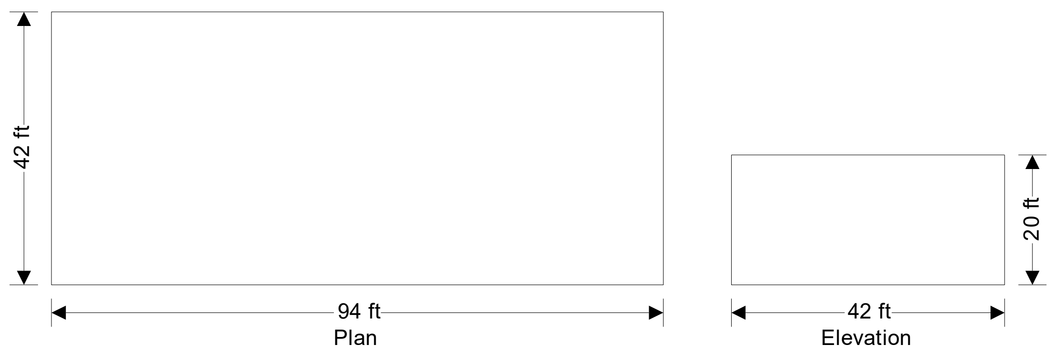
LATERAL ANALYSIS

WIND LOADING

In accordance with ASCE7-16

Using the directional design method

Tedds calculation version 2.1.14



Building data

Type of roof	Flat
Length of building	b = **94.00** ft
Width of building	d = **42.00** ft
Height to eaves	H = **20.00** ft
Mean height	h = **20.00** ft

General wind load requirements

Basic wind speed	V = **109.0** mph
Risk category	III
Velocity pressure exponent coef (Table 26.6-1)	K_d = **0.85**
Ground elevation above sea level	z_{gl} = **3900** ft
Ground elevation factor	$K_e = \exp(-0.0000362 \times z_{gl}/1\text{ft}) = \mathbf{0.87}$
Exposure category (cl 26.7.3)	C
Enclosure classification (cl.26.12)	Enclosed buildings
Internal pressure coef +ve (Table 26.13-1)	GC_{pi_p} = **0.18**
Internal pressure coef –ve (Table 26.13-1)	GC_{pi_n} = **-0.18**
Gust effect factor	G_f = **0.85**
Minimum design wind loading (cl.27.1.5)	p_{min_r} = **8** lb/ft^2

Topography

Topography factor not significant	K_{zt} = 1.0
Velocity pressure equation	$q = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \times 1\text{psf/mph}^2$

Velocity pressures table

z (ft)	K_z (Table 26.10-1)	q_z (psf)
15.00	0.85	19.08
20.00	0.90	20.20

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.)	q_i = **20.20** psf

Pressures and forces

Net pressure $p = q \times G_f \times C_{pe} - q_i \times GC_{pi}$

Net force $F_w = p \times A_{ref}$

Roof load case 1 - Wind 0, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	20.00	-0.90	20.20	-19.09	940.00	-17.95
B (-ve)	20.00	-0.90	20.20	-19.09	940.00	-17.95
C (-ve)	20.00	-0.50	20.20	-12.22	1880.00	-22.98
D (-ve)	20.00	-0.30	20.20	-8.79	188.00	-1.65

Total vertical net force $F_{w,v}$ = **-60.53** kips

Total horizontal net force $F_{w,h}$ = **0.00** kips

Walls load case 1 - Wind 0, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	19.08	9.34	1410.00	13.17
A_2	20.00	0.80	20.20	10.10	470.00	4.75
B	20.00	-0.50	20.20	-12.22	1880.00	-22.98
C	20.00	-0.70	20.20	-15.66	840.00	-13.15
D	20.00	-0.70	20.20	-15.66	840.00	-13.15

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_0} = b \times H =$ **1880.00** ft^2

Projected vertical area of roof $A_{vert\_r\_0} =$ **0.00** ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0} =$ **30.08** kips

Leeward net force $F_l = F_{w,wB} =$ **-23.0** kips

Windward net force $F_w = F_{w,wA\_1} + F_{w,wA\_2} =$ **17.9** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **40.9** kips

Roof load case 2 - Wind 0, GC_{pi} -0.18, $-0c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (+ve)	20.00	-0.18	20.20	0.55	940.00	0.51
B (+ve)	20.00	-0.18	20.20	0.55	940.00	0.51
C (+ve)	20.00	-0.18	20.20	0.55	1880.00	1.03
D (+ve)	20.00	-0.18	20.20	0.55	188.00	0.10

Total vertical net force $F_{w,v}$ = **2.15** kips

Total horizontal net force $F_{w,h}$ = **0.00** kips

Walls load case 2 - Wind 0, GC_{pi} -0.18, -0c_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	19.08	16.61	1410.00	23.42
A_2	20.00	0.80	20.20	17.38	470.00	8.17
B	20.00	-0.50	20.20	-4.95	1880.00	-9.31
C	20.00	-0.70	20.20	-8.38	840.00	-7.04
D	20.00	-0.70	20.20	-8.38	840.00	-7.04

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_0} = b \times H =$ **1880.00** ft^2

Projected vertical area of roof $A_{vert\_r\_0} =$ **0.00** ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0} =$ **30.08** kips

Leeward net force $F_l = F_{w,wB} =$ **-9.3** kips

Windward net force $F_w = F_{w,wA\_1} + F_{w,wA\_2} =$ **31.6** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **40.9** kips

Roof load case 3 - Wind 90, GC_{pi} 0.18, -c_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	20.00	-0.90	20.20	-19.09	420.00	-8.02
B (-ve)	20.00	-0.90	20.20	-19.09	420.00	-8.02
C (-ve)	20.00	-0.50	20.20	-12.22	840.00	-10.27
D (-ve)	20.00	-0.30	20.20	-8.79	2268.00	-19.93

Total vertical net force $F_{w,v} =$ **-46.24** kips

Total horizontal net force $F_{w,h} =$ **0.00** kips

Walls load case 3 - Wind 90, GC_{pi} 0.18, -c_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	19.08	9.34	630.00	5.88
A_2	20.00	0.80	20.20	10.10	210.00	2.12
B	20.00	-0.29	20.20	-8.58	840.00	-7.21
C	20.00	-0.70	20.20	-15.66	1880.00	-29.44
D	20.00	-0.70	20.20	-15.66	1880.00	-29.44

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_90} = d \times H =$ **840.00** ft^2

Projected vertical area of roof $A_{vert\_r\_90} =$ **0.00** ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} =$ **13.44** kips

Leeward net force $F_l = F_{w,wB} =$ **-7.2** kips

Windward net force $F_w = F_{w,wA\_1} + F_{w,wA\_2} =$ **8.0** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **15.2** kips

Roof load case 4 - Wind 90, GC_{pi} -0.18, $+c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (+ve)	20.00	-0.18	20.20	0.55	420.00	0.23
B (+ve)	20.00	-0.18	20.20	0.55	420.00	0.23
C (+ve)	20.00	-0.18	20.20	0.55	840.00	0.46
D (+ve)	20.00	-0.18	20.20	0.55	2268.00	1.24

Total vertical net force $F_{w,v} =$ **2.15** kips

Total horizontal net force $F_{w,h} =$ **0.00** kips

Walls load case 4 - Wind 90, GC_{pi} -0.18, $+c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	19.08	16.61	630.00	10.47
A_2	20.00	0.80	20.20	17.38	210.00	3.65
B	20.00	-0.29	20.20	-1.31	840.00	-1.10
C	20.00	-0.70	20.20	-8.38	1880.00	-15.76
D	20.00	-0.70	20.20	-8.38	1880.00	-15.76

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_90} = d \times H =$ **840.00** ft^2

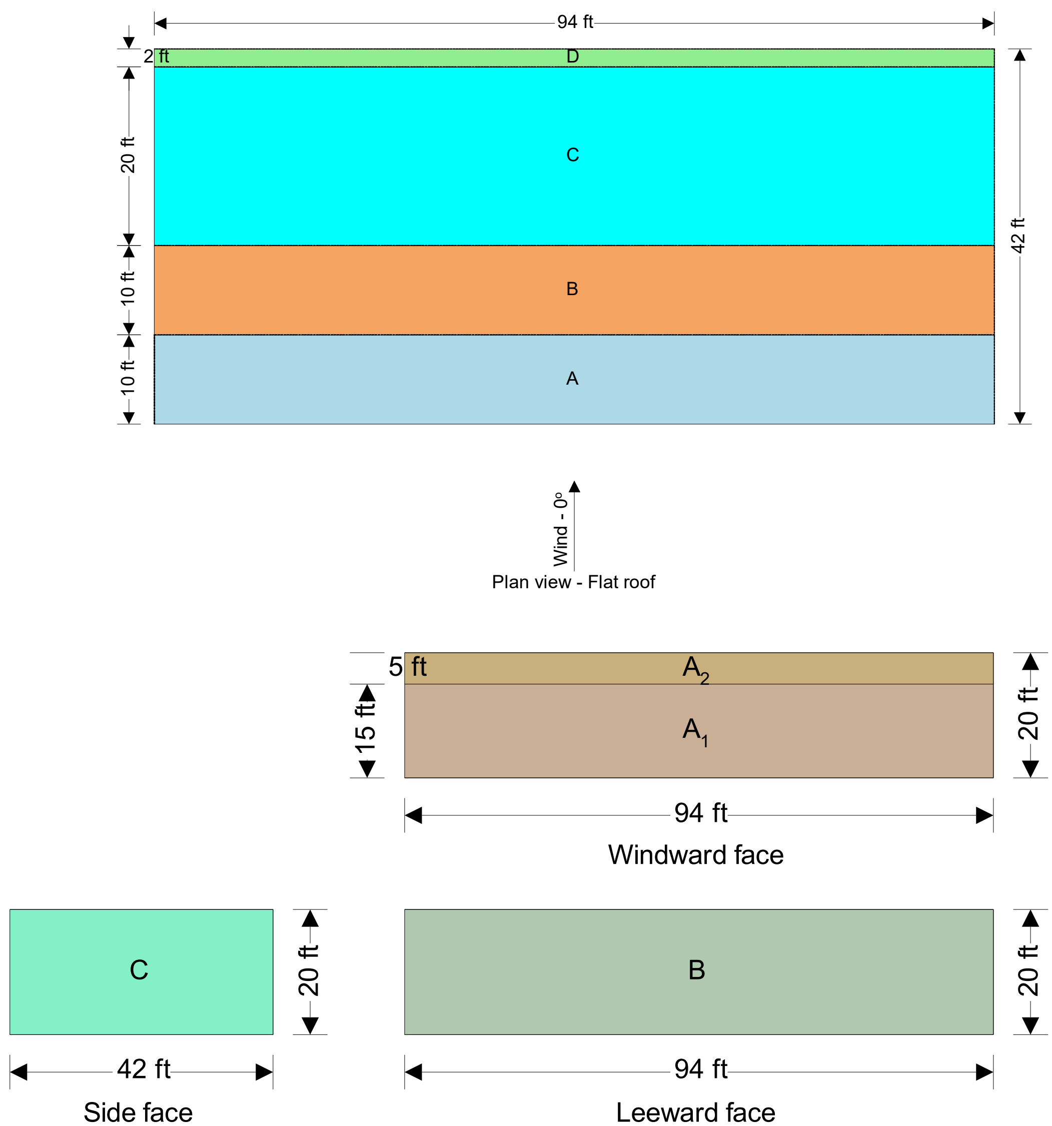
Projected vertical area of roof $A_{vert\_r\_90} =$ **0.00** ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} =$ **13.44** kips

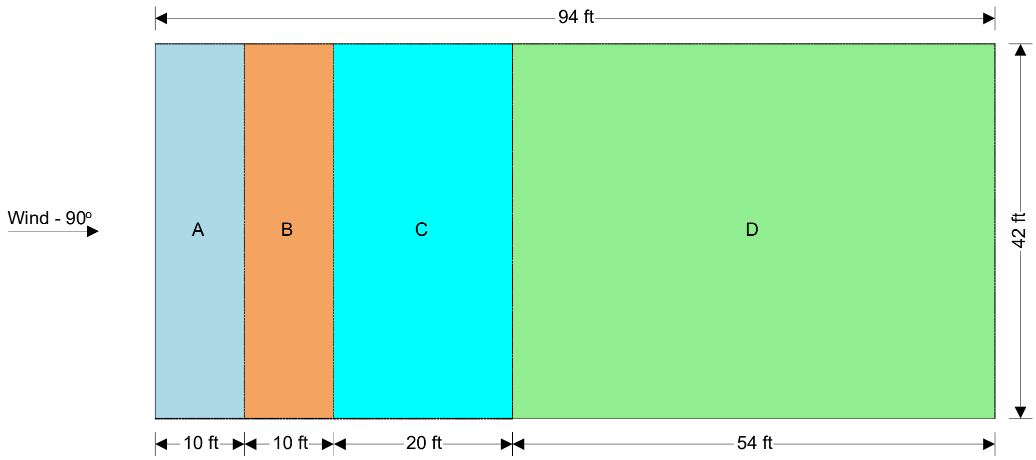
Leeward net force $F_l = F_{w,wB} =$ **-1.1** kips

Windward net force $F_w = F_{w,wA\_1} + F_{w,wA\_2} =$ **14.1** kips

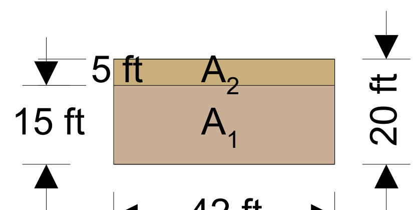
Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **15.2** kips



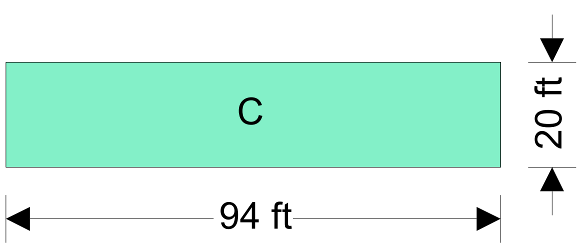
94 ft
2 ft
D
C
B
A
20 ft
10 ft
10 ft
42 ft
Wind - 0°
Plan view - Flat roof
5 ft
A2
A1
15 ft
20 ft
94 ft
Windward face
C
20 ft
42 ft
Side face
B
20 ft
94 ft
Leeward face



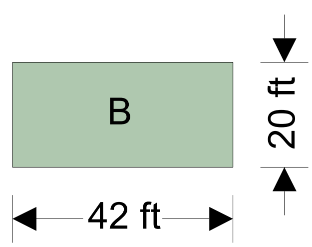
Plan view - Flat roof



Windward face



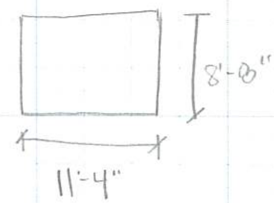
Side face



Leeward face

Lateral Analysis:

* SEISMIC:



ROOF DL = 15 PSF (9'-4")(11'-4") = 1.6 K
Wall DL = 84 PSF (10'-0"/2)(8'-8" + 11'-4")(2) = 16.8 K

TOTAL = 18.4 K

$C_s = 0.207/(5/1.25) = 0.052$

V = 0.052(18.4 K) = 0.96 K

* WIND:

DIRECTION 0 (8'-8"):

Windward = 9.34 PSF (10'-0"/2)(8'-8") = 0.4 K

Leeward = -12.2 PSF (10'-0"/2)(8'-8") = 0.53 K

Total = 0.93 K

DIRECTION 90 (11'-4"):

Windward = 17.38 PSF (10'-0"/2)(11'-4") = 1.0 K

* ASD:

SEISMIC: 0.7(0.96 K) = 0.67 K * GOVERNS*

WIND = 0.6(1.0 K) = 0.6 K

↳ see spreadsheets for shear wall design

Lateral Analysis (CONT'D):

* DECK DESIGN:

$V_D = 0.7K / 8'-8" = 81$ PLF

↳ metal roof deck, B deck 22 GA.

Attachment #10 screws 36/4

#10 @ 18" o.c. sidelap

#10 @ 12" o.c. spacing ∥ to chords.

CAP = 350 PLF ∴ OK.

* CHORD DESIGN:

$M = WL^2/8$ $W = 0.7K / 11'-4" = 62$ PLF

$M = 62 \text{ PLF} (11'-4')^2 / 8 = 995$ #·FT

$T = C = M/d = 995 \text{ \#·FT} / 8'-0" = 125$ #

↳ (2) #5 BARS @ TOP OF WALL OK.

22 ga HSB®-36-SS Grade 50 Roof Deck

Seismic Diaphragm Shear

For Both Ends Lapped Deck

#10 Screw Connections to Supports
36 / 4 Perpendicular Connection Pattern to Supports
#10 Screw Sidelap Connections

A572 GR50 Support Member or Equivalent
0.054 ≤ Support Thickness (in.) ≤ 0.175
3 in. Minimum Deck End Bearing Length

ASD Allowable Seismic Diaphragm Shear Strength Sn/Ω (plf) Generic 3 Span Condition

Sidelap Connection Spacing (in.)	Span 2'-0"	3'-0"	4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"
4	433	433	433	433	433	433	433	433	433
6	433	433	433	433	428	423	420	417	415
8	433	432	400	398	379	381	368	371	361
12	407	365	340	323	311	302	295	290	286
18	407	319	301	290	252	251	250	226	228
24	350	319	257	254	218	222	197	203	185
36	350	261	257	213	180	191	166	147	159

Average Connection Spacing to Supports at Parallel Chords & Collectors (in.)

Sidelap Connection Spacing (in.)	Span 2'-0"	3'-0"	4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"
4	9	9	9	9	9	9	9	9	9
6	12	12	12	12	12	12	12	14	13
8	12	12	16	15	14	14	14	15	15
12	12	18	16	20	18	17	19	18	20
18	12	18	16	20	24	21	24	27	24
24	24	18	24	20	24	21	24	27	24
36	24	18	24	20	24	21	24	27	24

Seismic or Wind Diaphragm Shear Stiffness, G' (kip/in.) Generic 3 Span Condition

Sidelap Connection Spacing (in.)	Span 2'-0"	3'-0"	4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"
4	181	176	174	173	172	171	170	170	169
6	166	160	156	154	152	151	150	149	149
8	156	152	143	143	138	139	135	136	134
12	143	132	126	122	119	117	115	113	112
18	143	119	115	112	101	101	101	95	95
24	126	119	101	101	91	92	85	87	81
36	126	102	101	89	79	82	75	69	73

22 ga HSB®-36-SS Grade 50 Roof Deck

Diaphragm Shear & Wind Uplift Interaction

For Both Ends Lapped Deck
with MWFRS Allowable Net Wind Uplift (ASD) of 36 psf

#10 Screw Connections to Supports
36 / 4 Perpendicular Connection Pattern to Supports
#10 Screw Sidelap Connections

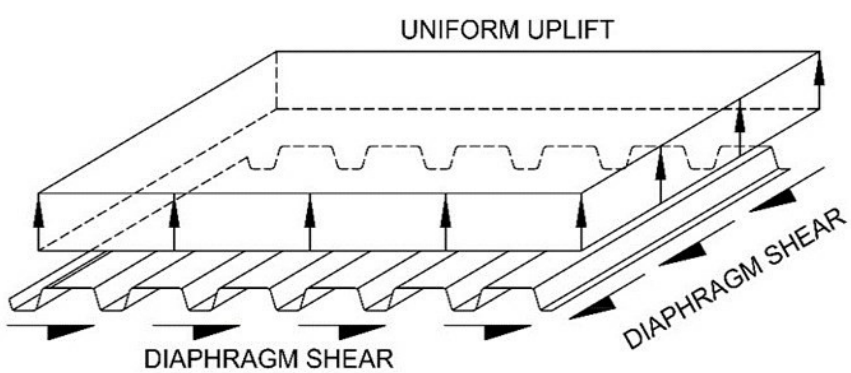
A572 GR50 Support Member or Equivalent
0.054 ≤ Support Thickness (in.) ≤ 0.175
3 in. Minimum Deck End Bearing Length

ASD Allowable Combined Wind Uplift & Diaphragm Shear Strength Sn/Ω (plf) Generic 3 Span Condition

Sidelap Connection Spacing (in.)	Span 2'-0"	3'-0"	4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"
4	331	272	212	153	93	34	-	-	-
6	331	272	212	153	93	34	-	-	-
8	331	272	212	153	93	34	-	-	-
12	331	272	212	153	93	34	-	-	-
18	331	245	208	153	93	34	-	-	-
24	291	245	178	152	93	34	-	-	-
36	291	194	178	129	88	34	-	-	-

Average Connection Spacing to Supports at Parallel Chords & Collectors (in.)

Sidelap Connection Spacing (in.)	Span 2'-0"	3'-0"	4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"
4	9	9	9	9	9	9	-	-	-
6	12	12	12	12	12	12	-	-	-
8	12	12	12	12	12	12	-	-	-
12	12	12	12	12	12	12	-	-	-
18	12	18	12	12	12	12	-	-	-
24	12	18	16	12	12	12	-	-	-
36	12	18	16	15	14	12	-	-	-

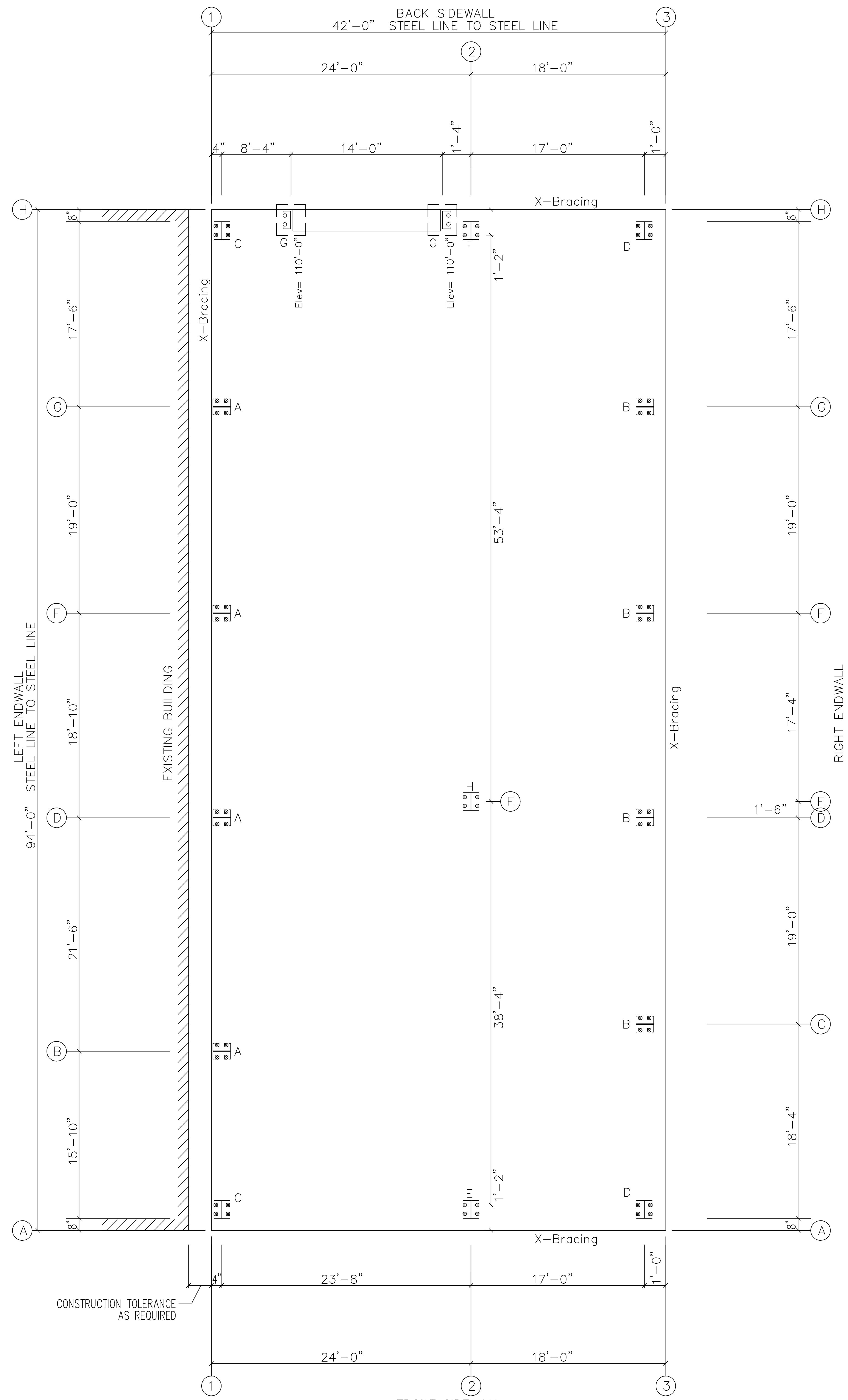


V1.0.4 of calculator based on AISI S100-16 (2020) w/S2-20 & AISI S310-16 as modified by IAPMO ER-2018 or ER-0423

Date: 9/26/2024

FOOTINGS/FOUNDATIONS

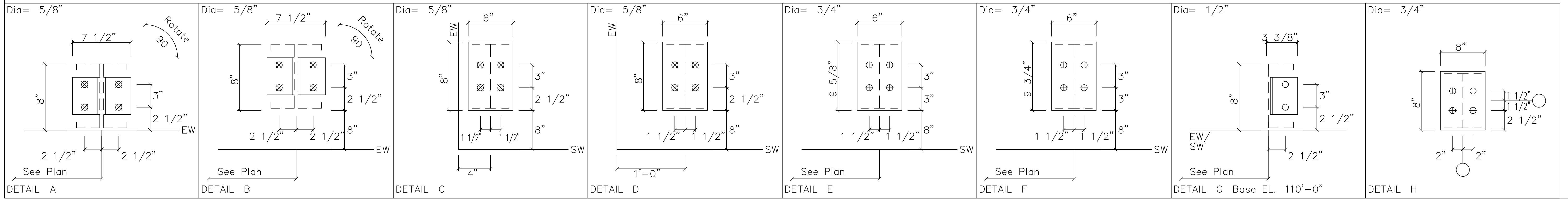
FOR INFORMATION ONLY



ANCHOR BOLT PLAN

NOTE: All Base Plates @ 100'-0" (U.N.)

NOT FOR CONSTRUCTION



CO BUILDING SYSTEMS INC.

MBMA MEMBER

IAS ACCREDITED Metal Building Systems AC 472

ANCHOR BOLT PLAN & DETAILS

KIMBERLY SCHOOL DISTRICT
885 CENTER ST W
KIMBERLY, ID 83341

EDUCATIONAL

REVISIONS

REV	DATE:	DESCRIPTION
1		
2		
3		
4		
5		
6		

DESIGN: CP
DRAFT: CP
CHECK:
DATE: 9/19/24

JOB # 24082490

SHEET | OF | REV 0

FOR INFORMATION ONLY

ENDWALL COLUMN: BASIC COLUMN REACTIONS (k)

Frm Line	Col Line	Dead Vert	Collat Vert	Live Vert	Snow Vert	Wind_Left1 Horz	Wind_Left1 Vert	Wind_Right1 Horz	Wind_Right1 Vert	Wind_Left2 Horz	Wind_Left2 Vert	Wind_Right2 Horz	Wind_Right2 Vert	Wind Press Horz
1	H	0.6	0.5	2.0	2.1	-4.0	-6.5	0.0	4.6	-4.2	-5.9	0.0	5.3	-1.7
1	G	1.2	1.3	5.0	5.2	0.0	-1.4	4.9	-9.0	0.0	0.6	4.7	-7.0	0.0
1	F	1.1	1.1	4.5	4.7	0.0	-2.6	0.0	-2.7	0.0	-1.8	0.0	-2.1	0.0
1	D	1.2	1.3	5.0	5.3	0.0	-3.1	0.0	-4.5	0.0	-1.4	0.0	-2.9	0.0
1	B	1.1	1.3	5.1	5.3	0.0	-3.3	0.0	-5.4	0.0	-1.6	0.0	-3.6	-1.1
1	A	0.5	0.4	1.8	1.9	0.0	-1.5	0.0	-1.6	0.0	-0.7	0.0	-0.8	-1.4

Frm Line	Col Line	Wind Suct Horz	Wind_Long1 Horz	Wind_Long1 Vert	Wind_Long2 Horz	Wind_Long2 Vert	Seis_Left Horz	Seis_Left Vert	Seis_Right Horz	Seis_Right Vert	Seis Long Vert	-MIN_SNOW-- Horz	-MIN_SNOW-- Vert
1	H	1.9	0.0	-1.3	-0.9	-2.6	-0.9	-1.0	0.0	1.2	0.0	0.0	2.0
1	G	0.0	0.7	-6.1	0.0	-2.1	0.0	0.9	0.9	-1.1	0.0	0.0	5.0
1	F	0.0	0.0	-2.8	0.0	-2.2	0.0	0.2	0.0	-0.2	0.0	0.0	4.5
1	D	0.0	0.0	-2.9	0.0	-5.2	0.0	0.0	0.0	0.0	0.0	0.0	5.0
1	B	1.2	0.0	-3.2	0.0	-5.3	0.0	0.0	0.0	0.0	0.0	0.0	5.1
1	A	1.6	0.0	-1.2	0.0	-1.9	0.0	0.0	0.0	0.0	0.0	0.0	1.8

Frm Line	Col Line	E1UNB_SL_L- Horz	E1UNB_SL_L- Vert	E1UNB_SL_R- Horz	E1UNB_SL_R- Vert
1	H	0.0	1.9	0.0	0.7
1	G	0.0	7.1	0.0	1.1
1	F	0.0	6.2	0.0	4.5
1	D	0.0	1.0	0.0	6.9
1	B	0.0	1.7	0.0	5.1
1	A	0.0	0.5	0.0	1.9

Frm Line	Col Line	Dead Vert	Collat Vert	Live Vert	Snow Vert	Wind_Left1 Horz	Wind_Left1 Vert	Wind_Right1 Horz	Wind_Right1 Vert	Wind_Left2 Horz	Wind_Left2 Vert	Wind_Right2 Horz	Wind_Right2 Vert
3	A	0.5	0.4	1.7	1.8	0.0	-1.7	0.0	-1.3	0.0	-1.0	0.0	-0.7
3	C	1.0	1.0	4.0	4.2	0.0	-4.4	0.0	-2.7	0.0	-3.0	0.0	-1.3
3	D	1.0	0.9	3.5	3.7	-2.7	-6.7	0.0	1.0	-2.6	-5.4	0.0	2.4
3	F	1.0	0.9	3.6	3.8	0.0	1.7	2.3	-6.1	0.0	2.2	2.5	-5.5
3	G	1.0	1.0	3.9	4.1	0.0	-2.7	0.0	-4.3	0.0	-1.3	0.0	-2.9
3	H	0.5	0.4	1.6	1.7	0.0	-1.4	0.0	-1.6	0.0	-0.7	0.0	-1.0

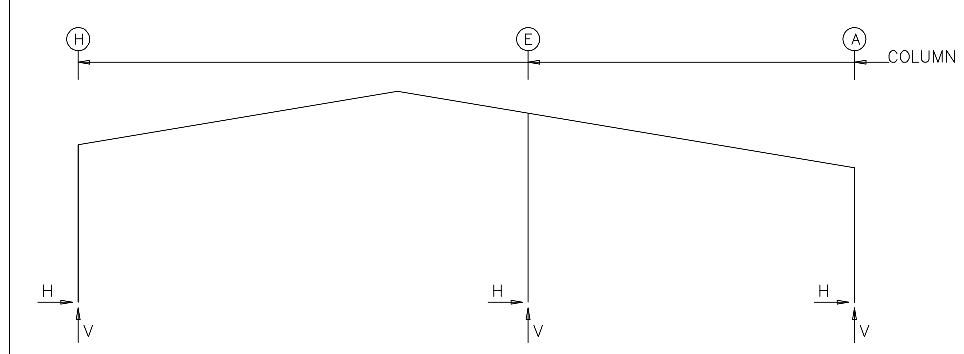
Frm Line	Col Line	Wind_Press Horz	Wind_Press Vert	Wind_Suct Horz	Wind_Suct Vert	Wind_Long1 Horz	Wind_Long1 Vert	Wind_Long2 Horz	Wind_Long2 Vert	Seis_Left Horz	Seis_Left Vert	Seis_Right Horz	Seis_Right Vert	Seis_Long Horz	Seis_Long Vert
3	A	-9.0	-6.2	1.2	6.2	0.0	-1.8	0.0	-1.1	0.0	0.0	0.0	0.0	-1.5	-1.1
3	C	-2.7	0.0	3.0	0.0	0.0	-4.3	0.0	-2.6	0.0	0.0	0.0	0.0	0.0	0.0
3	D	-3.2	0.0	3.6	0.0	0.0	-2.7	-0.5	-2.7	-0.7	-0.9	0.0	1.0	0.0	0.0
3	F	-3.6	0.0	4.0	0.0	0.8	-3.3	0.0	-1.7	0.0	0.9	0.7	-1.0	0.0	0.0
3	G	-3.0	0.0	3.4	0.0	0.0	-2.4	0.0	-4.4	0.0	0.0	0.0	0.0	0.0	0.0
3	H	-9.8	-8.0	1.5	8.0	0.0	-1.1	0.0	-1.7	0.0	0.0	0.0	0.0	-1.5	-1.4

Frm Line	Col Line	-MIN_SNOW-- Horz	-MIN_SNOW-- Vert	E2UNB_SL_L- Horz	E2UNB_SL_L- Vert	E2UNB_SL_R- Horz	E2UNB_SL_R- Vert
3	A	0.0	1.7	0.0	1.8	0.0	0.5
3	C	0.0	4.0	0.0	4.0	0.0	1.4
3	D	0.0	3.5	0.0	5.0	0.0	0.7
3	F	0.0	3.6	0.0	3.7	0.0	4.9
3	G	0.0	3.9	0.0	0.8	0.0	5.6
3	H	0.0	1.6	0.0	0.6	0.0	1.5

ENDWALL COLUMN: MAXIMUM REACTIONS, ANCHOR BOLTS, & BASE PLATES

Frm Line	Col Line	Column_Reactions(k) Load Id	Hmax H	V Vmax	Load Id	Hmin H	V Vmin	Bolt(in) Qty	Bolt(in) Dia	Base_Plate(in) Width	Base_Plate(in) Length	Base_Plate(in) Thick	Grout (in)
1	H	10 12	1.2 0.9	-3.6 5.1	11 10	-1.0 1.2	-1.2 -3.6	4	0.625	6.000	8.000	0.250	0.0
1	G	2 13	0.0 0.0	-4.7 9.5	2	0.0	-4.7	4	0.625	7.500	6.000	0.250	0.0
1	F	5 13	0.0 0.0	-1.0 8.5	5	0.0	-1.0	4	0.625	7.500	6.000	0.250	0.0
1	D	6 14	0.0 0.0	-2.4 9.3	6	0.0	-2.4	4	0.625	7.500	6.000	0.250	0.0
1	B	15 1	0.7 0.0	-2.6 7.7	11 15	-0.7 0.7	-2.5 -2.6	4	0.625	7.500	6.000	0.250	0.0
1	A	16 14	1.0 0.0	-0.8 2.9	11 16	-0.9 1.0	-0.8 -0.8	4	0.625	6.000	8.000	0.250	0.0
3	A	17 12	0.7 0.6	3.0 4.7	18	-5.4	-4.5	4	0.625	6.000	8.000	0.375	0.0
3	C	10 1	1.8 0.0	-2.1 6.2	18 10	-1.6 1.8	-2.0 -2.1	4	0.625	7.500	6.000	0.250	0.0
3	D	10 19	2.1 0.0	-3.5 6.9	11 10	-1.9 2.1	-1.1 -3.5	4	0.625	7.500	6.000	0.250	0.0
3	F	15 20	2.4 0.0	-3.1 6.8	18 15	-2.2 2.4	-1.4 -3.1	4	0.625	7.500	6.000	0.250	0.0
3	G	16 20	2.0 0.0	-2.1 7.5	11 16	-1.8 2.0	-2.1 -2.1	4	0.625	7.500	6.000	0.250	0.0
3	H	16 21	0.9 0.7	4.1 5.5	11	-5.9	-5.5	4	0.625	6.000	8.000	0.375	0.0

FRAME LINES: 2



RIGID FRAME: MAXIMUM REACTIONS, ANCHOR BOLTS, & BASE PLATES

Frm Line	Col Line	Column_Reactions(k) Load Id	Hmax H	V Vmax	Load Id	Hmin H	V Vmin	Bolt(in) Qty	Bolt(in) Dia	Base_Plate(in) Width	Base_Plate(in) Length	Base_Plate(in) Thick	Grout (in)
2	H	8	7.7	26.9	3 5	-3.5 0.2	-2.3 -9.9	4	0.750	6.000	9.750	0.500	0.0
2	A	4 1	4.0 -7.0	-2.3 16.8	8 6	-7.7 -0.9	9.4 -7.4	4	0.750	6.000	9.625	0.375	0.0
2	E	7 9	0.0 0.0	-7.1 39.2	7	0.0	-7.1	4	0.750	8.000	8.000	0.375	0.0

RIGID FRAME: BASIC COLUMN REACTIONS (k)

Frame Line	Column Line	-----Dead---- Horz	Dead Vert	--Collateral- Horz	Collateral Vert	-----Live----- Horz	Live Vert	-----Snow----- Horz	Snow Vert	--Wind_Left1- Horz	Wind_Left1 Vert	-Wind_Right1- Horz	Wind_Right1 Vert
2	H	1.0	3.6	1.2	3.8	4.6	15.1	4.9	15.9	-6.6	-12.7	2.1	-7.9
2	A	-1.0	2.6	-1.2	2.8	-4.6	10.9	-4.9	11.5	-2.3	-4.9	7.1	-10.7
2	E	0.0	5.5	0.0	6.3	0.0	25.0	0.0	26.2	0.0	-15.5	0.0	-14.8

Frame Line	Column Line	--Wind_Left2- Horz	Wind_Left2 Vert	-Wind_Right2- Horz	Wind_Right2 Vert	--Wind_Long1- Horz	Wind_Long1 Vert	--Wind_Long2- Horz	Wind_Long2 Vert	-Seismic_Left Horz	Seismic_Left Vert	Seismic_Right Horz	Seismic_Right Vert
2	H	-6.8	-7.4	1.5	-2.5	-0.7	-20.0	-0.9	-16.4	-0.5	-0.1	0.5	0.1
2	A	-2.0	-0.8	7.7	-6.4	1.8	-13.0	-0.5	-14.8	-1.0	0.5	1.0	-0.5
2	E	0.0	-8.2	0.0	-7.7	0.0	-12.8	0.0	-17.3	0.0	-0.3	0.0	0.3

Frame Line	Column Line	-Seismic_Long Horz	Seismic_Long Vert	-----Temp---- Horz	Temp Vert	--MIN_SNOW-- Horz	MIN_SNOW Vert	F1UNB_SL_L- Horz	F1UNB_SL_L- Vert	F1UNB_SL_R- Horz	F1UNB_SL_R- Vert
2	H	0.0	-1.4	1.1	0.6	4.6	15.1	5.6	19.5	2.7	6.1
2	A	0.0	-1.1	-1.1	0.8	-4.6	10.9	-5.6	4.1	-2.7	10.7
2	E	0.0	0.0	0.0	-1.5	0.0	25.0	0.0	16.9	0.0	27.4

NOTES FOR REACTIONS

Building reactions are based on the following building data:

Width (ft)	=	94.0
Length (ft)	=	42.0
Eave Height (ft)	=	19.0/ 16.2
Roof Slope (rise/12)	=	2.0/2.0
Dead Load (psf)	=	3.0
Collateral Load (psf)	=	5.0
Live Load (psf)	=	20.0
Snow Load (psf)	=	21.0
Wind Speed (mph)	=	105.0
Wind Code	=	IBC 21
Exposure	=	C
Closure	=	Enclosed
Importance Wind	=	1.00
Importance Seismic	=	1.00
Seismic Design Category	=	B
Seismic Coeff (Fa*Ss)	=	0.31
Temperature Change	=	100

ID	Description
1	Dead+Collateral+Snow
2	0.6Dead+0.6Wind_Right1
3	0.6Dead+0.6Wind_Left2
4	0.6Dead+0.6Wind_Right2
5	0.6Dead+0.6Wind_Long1L
6	0.6Dead+0.6Wind_Long2L
7	0.6Dead+0.6Wind_Long2R
8	Dead+Collateral+F1UNB_SL_L
9	Dead+Collateral
10	0.6Dead+0.6Wind_Left1+0.6Wind_Suction
11	0.6Dead+0.6Wind_Pressure+0.6Wind_Long2L
12	Dead+Collateral+0.75Snow+0.45Wind_Right2+0.45Wind_Suction
13	Dead+Collateral+E1UNB_SL_L
14	Dead+Collateral+E1UNB_SL_R
15	0.6Dead+0.6Wind_Right1+0.6Wind_Suction
16	0.6Dead+0.6Wind_Suction+0.6Wind_Long2L
17	0.6Dead+0.6Wind_Suction+0.6Wind_Long1L
18	0.6Dead+0.6Wind_Pressure+0.6Wind_Long1L
19	Dead+Collateral+E2UNB_SL_L
20	Dead+Collateral+E2UNB_SL_R
21	Dead+Collateral+0.75Snow+0.45Wind_Left2+0.45Wind_Suction

ANCHOR BOLT SUMMARY

	Qty	Locate	Dia (in)	Type	Proj (in)
○	4	Jamb	1/2"	GR36	1.50
⊠	48	Endwall	5/8"	GR36	2.00
⊕	12	Frame	3/4"	GR36	2.50

BUILDING BRACING REACTIONS

Wall Loc	Wall Line	Col Line	± Reactions(k) Wind Horz	Wind Vert	Seismic Horz	Seismic Vert	Panel_Shear (lb/ft) Wind	Panel_Shear Seis
L_EW	1	H,G	4.9	5.7	0.9	1.1		
F_SW	A	2,3	7.9	6.2	1.5	1.2		
R_EW	3	D,F	2.7	3.5	0.7	0.9		
B_SW	H	3,2	8.5	8.0	1.5	1.4		

Reactions for seismic represent shear force, Eh

NOT FOR CONSTRUCTION

CO BUILDING SYSTEMS INC.

MBMA METAL BUILDING MANUFACTURERS ASSOCIATION MEMBER

IAS ACCREDITED Metal Building Systems AC 472

ANCHOR BOLT REACTIONS

KIMBERLY SCHOOL DISTRICT
885 CENTER ST W
KIMBERLY, ID 83341

EDUCATIONAL

REVISIONS

REV	DATE:	DESCRIPTION
1		
2		
3		
4		
5		
6		

DESIGN: CP
DRAFT: CP
CHECK:
DATE: 9/19/24

JOB # 24082490

SHEET | OF | REV 0

IBC 2018 FACTORED REACTIONS FOR PRE-MANUFACTURED METAL BUILDINGS

Version Date: January 23, 2019

JOB TITLE: Kimberly AG Shop

JOB #: 24091

ARW ENGINEERS structural consultants

Tuesday, October 8, 2024

Grid 1

	Grid H			Grid G			Grid F			Grid D			Grid B			Grid A		
	X	Y	Z	X	Y	Z	X	Y	Z	X	Y	Z	X	Y	Z	X	Y	Z
Dead		0.6			1.2			1.1			1.2			1.1			0.5	
Collateral, C		0.5			1.3			1.1			1.3			1.3			0.4	
Floor Live																		
Roof Live		2.0			5.0			4.5			5.0			5.1			1.8	
Crane, C_r																		
Snow		2.1			7.1			6.2			6.9			5.3			1.9	
Wind	4.9	5.7	1.9	4.9	5.7										1.2			1.6
Wind (Uplift)	4.9	6.5	1.7	4.9	9.0			2.8			5.2			5.4	1.1		1.9	1.4
Seismic	0.9	1.2		0.9	1.1			0.2										
Seismic (Uplift)	0.9	1.1		0.9	1.1			0.2										
Seismic Design Factors																		
Seismic Coefficient Sds	0.207	0.207	0.207	0.207	0.207	0.207	0.207	0.207	0.207	0.207	0.207	0.207	0.207	0.207	0.207	0.207	0.207	0.207
Amplification Factor - Ωo	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
Redundancy Factor - ρ	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3
ASD Combinations	ASD Factored Reactions																	
D		0.6			1.2			1.1			1.2			1.1			0.5	
D+L+C_r		0.6			1.2			1.1			1.2			1.1			0.5	
D+C+L_r		3.1			7.5			6.7			7.5			7.5			2.7	
D+C+S		3.2			9.6			8.4			9.4			7.7			2.8	
D+C+0.75*(L+C_r)+0.75*L_r		2.6			6.3			5.6			6.3			6.2			2.3	
D+C+0.75*(L+C_r)+0.75*S		2.7			7.8			6.9			7.7			6.4			2.3	
D+0.6*W	2.9	4.0	1.1	2.9	4.6			1.1			1.2			1.1	0.7		0.5	1.0
D-0.6*W	-2.9	-3.3	-1.0	-2.9	-4.2			-0.6			-1.9			-2.1	-0.7		-0.6	-0.8
(D+0.14*Sds*D)+0.7*(E*ρ)	0.8	1.7		0.8	2.2			1.3			1.2			1.1			0.5	
(D-0.14*Sds*D)-0.7*(E*ρ)	-0.8	-0.4		-0.8	0.2			0.9			1.2			1.1			0.5	
D+C+0.75*0.6*W+0.75*(L+C_r)+0.75*L_r	2.2	5.2	0.9	2.2	8.8			5.6			6.3			6.2	0.5		2.3	0.7
D+C-0.75*0.6*W+0.75*(L+C_r)+0.75*L_r	-2.2	-0.3	-0.8	-2.2	2.2			4.3			3.9			3.8	-0.5		1.4	-0.6
D+C+0.75*0.6*W+0.75*(L+C_r)+0.75*S	2.2	5.2	0.9	2.2	10.4			6.9			7.7			6.4	0.5		2.3	0.7
D+C-0.75*0.6*W+0.75*(L+C_r)+0.75*S	-2.2	-0.3	-0.8	-2.2	3.8			5.6			5.3			3.9	-0.5		1.5	-0.6
(D+0.105*Sds*D)+0.525*(E*ρ)+C+0.75*(L+C_r)+0.75*L_r	0.6	3.4		0.6	7.0			5.7			6.3			6.2			2.3	
(D-0.105*Sds*D)-0.525*(E*ρ)+C+0.75*(L+C_r)+0.75*L_r	-0.6	1.8		-0.6	5.5			5.4			6.2			6.2			2.2	
(D+0.105*Sds*D)+C+0.525*(E*ρ)+0.75*(L+C_r)+0.75*S	0.6	3.5		0.6	8.6			7.0			7.7			6.4			2.3	
(D-0.105*Sds*D)+C-0.525*(E*ρ)+0.75*(L+C_r)+0.75*S	-0.6	1.9		-0.6	7.0			6.7			7.6			6.4			2.3	
0.6D+0.6*W	2.9	3.8	1.1	2.9	4.1			0.7			0.7			0.7	0.7		0.3	1.0
0.6D-0.6*W	-2.9	-3.5	-1.0	-2.9	-4.7			-1.0			-2.4			-2.6	-0.7		-0.8	-0.8
0.6*(D+0.14*Sds*D)+0.7*(E*ρ)	0.8	1.5		0.8	1.7			0.9			0.7			0.7			0.3	
0.6*(D-0.14*Sds*D)-0.7*(E*ρ)	-0.8	-0.7		-0.8	-0.3			0.5			0.7			0.6			0.3	
LRFD Combinations	LRFD Factored Reactions																	
1.4D		0.8			1.7			1.5			1.7			1.5			0.7	
1.2D+1.6(L+C_r)+0.5S		2.3			6.3			5.5			6.2			5.3			2.0	
1.2D+1.6(L+C_r)+0.5L_r+C		2.2			5.2			4.7			5.2			5.2			1.9	
1.2D+1.6*(L_r+C)+L+C_r		4.7			11.5			10.3			11.5			11.6			4.1	
1.2D+1.6(L_r+C)+0.5W	2.5	7.6	1.0	2.5	14.4			10.3			11.5			11.6	0.6		4.1	0.8
1.2D+1.6(L_r+C)-0.5W	-2.5	1.5	-0.9	-2.5	7.0			8.9			8.9			8.9	-0.6		3.2	-0.7
1.2D+1.6(C+S)+(L+C_r)		4.9			14.9			13.0			14.6			11.9			4.3	
1.2D+1.6(C+S)+0.5W	2.5	7.7	1.0	2.5	17.7			13.0			14.6			11.9	0.6		4.3	0.8
1.2D+1.6(C+S)-0.5W	-2.5	1.6	-0.9	-2.5	10.4			11.6			12.0			9.2	-0.6		3.3	-0.7
1.2D+1.0W+(L+C_r)+0.5L_r+C	4.9	7.9	1.9	4.9	10.9			4.7			5.2			5.2	1.2		1.9	1.6
1.2D-1.0W+(L+C_r)+0.5L_r+C	-4.9	-4.3	-1.7	-4.9	-3.8			1.9			0.0			-0.2	-1.1		0.0	-1.4
1.2D+1.0W+(L+C_r)+0.5S+C	4.9	8.0	1.9	4.9	12.0			5.5			6.2			5.3	1.2		1.9	1.6
1.2D-1.0W+(L+C_r)+0.5S+C	-4.9	-4.2	-1.7	-4.9	-2.7			2.7			1.0			-0.1	-1.1		0.1	-1.4
1.2*(D+0.2*Sds*D)+(E*ρ)+(L+C_r)+0.2S+C	1.2	3.2		1.2	5.6			4.0			4.2			3.7			1.4	
1.2*(D-0.2*Sds*D)-(E*ρ)+(L+C_r)+0.2S+C	-1.2	-0.3		-1.2	1.4			2.2			2.8			2.3			1.0	
1.2*(D+0.2*Sds*D)+(E*Ωo)+(L+C_r)+0.2S+C	2.3	4.7		2.3	7.0			4.2			4.2			3.7			1.4	
1.2*(D-0.2*Sds*D)-(E*Ωo)+(L+C_r)+0.2S+C	-2.3	-1.6		-2.3	0.1			2.0			2.8			2.3			1.0	
0.9D+1.0W	4.9	6.2	1.9	4.9	6.8			1.0			1.1			1.0	1.2		0.5	1.6
0.9D-1.0W	-4.9	-6.0	-1.7	-4.9	-7.9			-1.8			-4.1			-4.4	-1.1		-1.5	-1.4
0.9*(D+0.2*Sds*D)+(E*ρ)	1.2	2.1		1.2	2.6			1.3			1.1			1.0			0.5	
0.9*(D-0.2*Sds*D)-(E*ρ)	-1.2	-0.9		-1.2	-0.4			0.7			1.0			0.9			0.4	
0.9*(D+0.2*Sds*D)+(E*Ωo)	2.3	3.6		2.3	3.9			1.5			1.1			1.0			0.5	
0.9*(D-0.2*Sds*D)-(E*Ωo)	-2.3	-2.2		-2.3	-1.7			0.4			1.0			0.9			0.4	
Bolt Pattern Callout																		
Worst Case Shear (ULT)	4.9		1.9	4.9		0.0	0.0		0.0	0.0		0.0	0.0		1.2	0.0		1.6
Worst Case Uplift (ULT)		-6.0			-7.9			-1.8			-4.1			-4.4			-1.5	
Worst Case Shear (ASD)	2.9		1.1	2.9		0.0	0.0		0.0	0.0		0.0	0.0		0.7	0.0		1.0
Passive Pressure (pcf)		250			251			252			253			254			255	
Friction Coefficient		0.3			0.3			0.3			0.3			0.3			0.3	
Sliding Footing Thickness (in)		12.0			12.0			12.0			12.0			12.0			12.0	
Soil Depth above Footing (in)		24			24			24			24			24			24	
Sliding Footing Width (ft)		3			3			3			3			3			3	
Friction Resistance (Kip)		0.5			0.6			0.6			0.6			0.6			0.5	
Passive Pressure Resistance (Kip)			3.4			3.4			3.4			3.4			3.4			3.4
Total Lateral Resistance (Kip)			**3.9**			**4.0**			**4.0**			**4.0**			**4.0**			**3.9**
							Consider Hairpins			**Consider Hairpins**			**Consider Hairpins**			**Consider Hairpins**		
Worst Case Uplift Force (ASD Kip)		-3.5			-4.7			-1.0			-2.4			-2.6			-0.8	
Footing Size (Up) (ft)		**3.1**			**3.6**			**1.7**			**2.5**			**2.6**			**1.5**	
Worst Case Down Force (ASD Kip)		5.2			10.4			8.4			9.4			7.7			2.8	
Allowable Bearing Pressure		1500			1500			1500			1500			1500			1500	
Footing Size (Down)		**2.0**			**2.8**			**2.5**			**2.6**			**2.4**			**1.4**	

Grid 2

	Grid H			Grid A			Grid E		
	X	Y	Z	X	Y	Z	X	Y	Z
Dead		3.6	1.0		2.6	1.0		5.5	
Collateral, C		3.8	1.2		2.8	1.2		6.3	
Floor Live									
Roof Live		15.1	4.6		10.9	4.6		25.0	
Crane, C_r									
Snow		19.5			11.5			27.4	
Wind	8.5	8.0		7.9	6.2				
Wind (Uplift)	8.5	20.0		7.9	14.8			17.3	
Seismic	1.5	1.4		1.5	1.2			0.3	
Seismic (Uplift)	1.5	1.4		1.5	1.2			0.3	
Seismic Design Factors									
Seismic Coefficient Sds	0.207	0.207	0.207	0.207	0.207	0.207	0.207	0.207	0.207
Amplification Factor - Ωo	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
Redundancy Factor - ρ	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3
ASD Combinations	ASD Factored Reactions								
D		3.6	1.0		2.6	1.0		5.5	
D+L+C_r		3.6	1.0		2.6	1.0		5.5	
D+C+L_r		22.5	6.8		16.3	6.8		36.8	
D+C+S		26.9	2.2		16.9	2.2		39.2	
D+C+0.75*(L+C_r)+0.75*L_r		18.7	5.7		13.6	5.7		30.6	
D+C+0.75*(L+C_r)+0.75*S		22.0	2.2		14.0	2.2		32.4	
D+0.6*W	5.1	8.4	1.0	4.7	6.3	1.0		5.5	
D-0.6*W	-5.1	-8.4	1.0	-4.7	-6.3	1.0		-4.9	
(D+0.14*Sds*D)+0.7*(E*ρ)	1.4	5.0	1.0	1.4	3.8	1.0		5.9	
(D-0.14*Sds*D)-0.7*(E*ρ)	-1.4	2.2	1.0	-1.4	1.4	1.0		5.1	
D+C+0.75*0.6*W+0.75*(L+C_r)+0.75*L_r	3.8	22.3	5.7	3.6	16.4	5.7		30.6	
D+C-0.75*0.6*W+0.75*(L+C_r)+0.75*L_r	-3.8	9.7	5.7	-3.6	6.9	5.7		22.8	
D+C+0.75*0.6*W+0.75*(L+C_r)+0.75*S	3.8	25.6	2.2	3.6	16.8	2.2		32.4	
D+C-0.75*0.6*W+0.75*(L+C_r)+0.75*S	-3.8	13.0	2.2	-3.6	7.4	2.2		24.6	
(D+0.105*Sds*D)+0.525*(E*ρ)+C+0.75*(L+C_r)+0.75*L_r	1.0	19.8	5.7	1.0	14.5	5.7		30.9	
(D-0.105*Sds*D)-0.525*(E*ρ)+C+0.75*(L+C_r)+0.75*L_r	-1.0	17.7	5.6	-1.0	12.7	5.6		30.2	
(D+0.105*Sds*D)+C+0.525*(E*ρ)+0.75*(L+C_r)+0.75*S	1.0	23.1	2.2	1.0	14.9	2.2		32.7	
(D-0.105*Sds*D)+C-0.525*(E*ρ)+0.75*(L+C_r)+0.75*S	-1.0	21.0	2.2	-1.0	13.1	2.2		32.0	
0.6D+0.6*W	5.1	7.0	0.6	4.7	5.3	0.6		3.3	
0.6D-0.6*W	-5.1	-9.8	0.6	-4.7	-7.3	0.6		-7.1	
0.6*(D+0.14*Sds*D)+0.7*(E*ρ)	1.4	3.5	0.6	1.4	2.7	0.6		3.7	
0.6*(D-0.14*Sds*D)-0.7*(E*ρ)	-1.4	0.8	0.6	-1.4	0.4	0.6		2.9	
LRFD Combinations	LRFD Factored Reactions								
1.4D		5.0	1.4		3.6	1.4		7.7	
1.2D+1.6(L+C_r)+0.5S		17.9	2.4		11.7	2.4		26.6	
1.2D+1.6(L+C_r)+0.5L_r+C		15.7	4.7		11.4	4.7		25.4	
1.2D+1.6*(L_r+C)+L+C_r		34.6	10.5		25.0	10.5		56.7	
1.2D+1.6(L_r+C)+0.5W	4.3	38.6	10.5	4.0	28.1	10.5		56.7	
1.2D+1.6(L_r+C)-0.5W	-4.3	24.6	10.5	-4.0	17.6	10.5		48.0	
1.2D+1.6(C+S)+(L+C_r)		41.6	3.1		26.0	3.1		60.5	
1.2D+1.6(C+S)+0.5W	4.3	45.6	3.1	4.0	29.1	3.1		60.5	
1.2D+1.6(C+S)-0.5W	-4.3	31.6	3.1	-4.0	18.6	3.1		51.9	
1.2D+1.0W+(L+C_r)+0.5L_r+C	8.5	23.7	4.7	7.9	17.6	4.7		25.4	
1.2D-1.0W+(L+C_r)+0.5L_r+C	-8.5	-4.3	4.7	-7.9	-3.4	4.7		8.1	
1.2D+1.0W+(L+C_r)+0.5S+C	8.5	25.9	2.4	7.9	17.9	2.4		26.6	
1.2D-1.0W+(L+C_r)+0.5S+C	-8.5	-2.1	2.4	-7.9	-3.1	2.4		9.3	
1.2*(D+0.2*Sds*D)+(E*ρ)+(L+C_r)+0.2S+C	2.0	14.0	2.4	2.0	9.9	2.4		19.0	
1.2*(D-0.2*Sds*D)-(E*ρ)+(L+C_r)+0.2S+C	-2.0	6.2	1.2	-2.0	3.7	1.2		11.4	
1.2*(D+0.2*Sds*D)+(E*Ωo)+(L+C_r)+0.2S+C	3.8	15.7	2.4	3.8	11.3	2.4		19.4	
1.2*(D-0.2*Sds*D)-(E*Ωo)+(L+C_r)+0.2S+C	-3.8	4.5	1.2	-3.8	2.3	1.2		11.1	
0.9D+1.0W	8.5	11.2	0.9	7.9	8.5	0.9		5.0	
0.9D-1.0W	-8.5	-16.8	0.9	-7.9	-12.5	0.9		-12.4	
0.9*(D+0.2*Sds*D)+(E*ρ)	2.0	5.2	0.9	2.0	4.0	0.9		5.5	
0.9*(D-0.2*Sds*D)-(E*ρ)	-2.0	1.3	0.9	-2.0	0.7	0.9		4.4	
0.9*(D+0.2*Sds*D)+(E*Ωo)	3.8	6.9	0.9	3.8	5.4	0.9		5.9	
0.9*(D-0.2*Sds*D)-(E*Ωo)	-3.8	-0.4	0.9	-3.8	-0.8	0.9		4.0	
Footing Results for Gravity, Uplift, and Shear									
Bolt Pattern Callout									
Worst Case Shear (ULT)	8.5		10.5	7.9		10.5	0.0		0.0
Worst Case Uplift (ULT)		-16.8			-12.5			-12.4	
Worst Case Shear (ASD)	5.1		6.8	4.7		6.8	0.0		0.0
Passive Pressure (pcf)		256			257			258	
Friction Coefficient		0.3			0.3			0.3	
Sliding Footing Thickness (in)		12.0			12.0			12.0	
Soil Depth above Footing (in)		24			24			24	
Sliding Footing Width (ft)		6			5			6.5	
Friction Resistance (Kip)		2.3			1.6			2.9	
Passive Pressure Resistance (Kip)			6.9			5.8			7.5
Total Lateral Resistance (Kip)			**9.2**			**7.4**			**10.4**
	Consider Hairpins			**Consider Hairpins**			**Consider Hairpins**		
Worst Case Uplift Force (ASD Kip)		-9.8			-7.3			-7.1	
Footing Size (Up) (ft)		**5.2**			**4.4**			**4.4**	
Worst Case Down Force (ASD Kip)		26.9			16.9			39.2	
Allowable Bearing Pressure		1500			1500			1500	
Footing Size (Down)		**4.4**			**3.5**			**5.4**	

Grid 3

	Grid A			Grid C			Grid D			Grid F			Grid G			Grid H		
	X	Y	Z	X	Y	Z	X	Y	Z	X	Y	Z	X	Y	Z	X	Y	Z
Dead		0.5			1.0			1.0			1.0			1.0			0.5	
Collateral, C		0.4			1.0			0.9			0.9			1.0			0.4	
Floor Live																		
Roof Live		1.7			4.0			3.5			3.6			3.9			1.6	
Crane, C_r																		
Snow		1.8	1.2		4.2	3.0		5.0			4.9			5.6			1.7	
Wind	7.9	6.2	9.0			2.7	2.7	3.5		2.5	2.2	4.0			3.4	8.5	8.0	1.5
Wind (Uplift)	7.9	6.2			4.4		2.7	6.7			6.1	3.6	4.4	4.3	3.0	8.5	8.0	9.8
Seismic	1.5	1.2			4.0		0.7	5.0			4.9			5.6		1.5	1.5	
Seismic (Uplift)	1.5	1.2					0.7	0.9			1.0					1.5	1.4	
Seismic Design Factors																		
Seismic Coefficient Sds	0.207	0.207	0.207	0.207	0.207	0.207	0.207	0.207	0.207	0.207	0.207	0.207	0.207	0.207	0.207	0.207	0.207	0.207
Amplification Factor - Ωo	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
Redundancy Factor - ρ	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3
ASD Combinations	ASD Factored Reactions																	
D		0.5			1.0			1.0			1.0			1.0			0.5	
D+L+C_r		0.5			1.0			1.0			1.0			1.0			0.5	
D+C+L_r		2.6			6.0			5.4			5.5			5.9			2.5	
D+C+S		2.7	1.2		6.2	3.0		6.9			6.8			7.6			2.6	
D+C+0.75*(L+C_r)+0.75*L_r		2.2			5.0			4.5			4.6			4.9			2.1	
D+C+0.75*(L+C_r)+0.75*S		2.3	0.9		5.2	2.3		5.7			5.6			6.2			2.2	
D+0.6*W	4.7	4.2	5.4		1.0	1.6	1.6	3.1		1.5	2.3	2.4		1.0	2.0	5.1	5.3	0.9
D-0.6*W	-4.7	-3.2			-1.6		-1.6	-3.0			-2.7	-2.2	-2.6	-1.6	-1.8	-5.1	-4.3	-5.9
(D+0.14*Sds*D)+0.7*(E*ρ)	1.4	1.6			4.7		0.6	5.6			5.5			6.1		1.4	1.9	
(D-0.14*Sds*D)-0.7*(E*ρ)	-1.4	-0.6			1.0		-0.6	0.2			0.1			1.0		-1.4	-0.8	
D+C+0.75*0.6*W+0.75*(L+C_r)+0.75*L_r	3.6	5.0	4.1		5.0	1.2	1.2	6.1		1.1	5.6	1.8		4.9	1.5	3.8	5.7	0.7
D+C-0.75*0.6*W+0.75*(L+C_r)+0.75*L_r	-3.6	-0.6			3.0		-1.2	1.5			1.9	-1.6	-2.0	3.0	-1.4	-3.8	-1.5	-4.4
D+C+0.75*0.6*W+0.75*(L+C_r)+0.75*S	3.6	5.0	5.0		5.2	3.5	1.2	7.2		1.1	6.6	1.8		6.2	1.5	3.8	5.8	0.7
D+C-0.75*0.6*W+0.75*(L+C_r)+0.75*S	-3.6	-0.5	0.9		3.2	2.3	-1.2	2.6			2.8	-1.6	-2.0	4.3	-1.4	-3.8	-1.4	-4.4
(D+0.105*Sds*D)+0.525*(E*ρ)+C+0.75*(L+C_r)+0.75*L_r	1.0	3.0			7.8		0.5	8.0			8.0			8.8		1.0	3.1	
(D-0.105*Sds*D)-0.525*(E*ρ)+C+0.75*(L+C_r)+0.75*L_r	-1.0	1.3			5.0		-0.5	3.9			3.9			4.9		-1.0	1.1	
(D+0.105*Sds*D)+C+0.525*(E*ρ)+0.75*(L+C_r)+0.75*S	1.0	3.1	0.9		7.9	2.3	0.5	9.1			8.9			10.0		1.0	3.2	
(D-0.105*Sds*D)+C-0.525*(E*ρ)+0.75*(L+C_r)+0.75*S	-1.0	1.4	0.9		5.1	2.3	-0.5	5.0			4.9			6.2		-1.0	1.2	
0.6D+0.6*W	4.7	4.0	5.4		0.6	1.6	1.6	2.7		1.5	1.9	2.4		0.6	2.0	5.1	5.1	0.9
0.6D-0.6*W	-4.7	-3.4			-2.0		-1.6	-3.4			-3.1	-2.2	-2.6	-2.0	-1.8	-5.1	-4.5	-5.9
0.6*(D+0.14*Sds*D)+0.7*(E*ρ)	1.4	1.4			4.3		0.6	5.2			5.1			5.7		1.4	1.7	
0.6*(D-0.14*Sds*D)-0.7*(E*ρ)	-1.4	-0.8			0.6		-0.6	-0.2			-0.3			0.6		-1.4	-1.0	
LRFD Combinations	LRFD Factored Reactions																	
1.4D		0.7			1.4			1.4			1.4			1.4			0.7	
1.2D+1.6(L+C_r)+0.5S		1.9	0.6		4.3	1.5		4.6			4.6			5.0			1.9	
1.2D+1.6(L+C_r)+0.5L_r+C		1.9			4.2			3.9			3.9			4.2			1.8	
1.2D+1.6*(L_r+C)+L+C_r		4.0			9.2			8.2			8.4			9.0			3.8	
1.2D+1.6(L_r+C)+0.5W	4.0	7.1	4.5		9.2	1.4	1.4	10.0		1.3	9.5	2.0		9.0	1.7	4.3	7.8	0.8
1.2D+1.6(L_r+C)-0.5W	-4.0	0.9			7.0		-1.4	4.9			5.4	-1.8	-2.2	6.9	-1.5	-4.3	-0.2	-4.9
1.2D+1.6(C+S)+(L+C_r)		4.1	1.9		9.5	4.8		10.6			10.5			11.8			4.0	
1.2D+1.6(C+S)+0.5W	4.0	7.2	6.4		9.5	6.2	1.4	12.4		1.3	11.6	2.0		11.8	1.7	4.3	8.0	0.8
1.2D+1.6(C+S)-0.5W	-4.0	1.0	1.9		7.3	4.8	-1.4	7.3			7.4	-1.8	-2.2	9.6	-1.5	-4.3	0.0	-4.9
1.2D+1.0W+(L+C_r)+0.5L_r+C	7.9	8.1	9.0		4.2	2.7	2.7	7.4		2.5	6.1	4.0		4.2	3.4	8.5	9.8	1.5
1.2D-1.0W+(L+C_r)+0.5L_r+C	-7.9	-4.4			-0.2		-2.7	-2.9			-2.2	-3.6	-4.4	-0.2	-3.0	-8.5	-6.2	-9.8
1.2D+1.0W+(L+C_r)+0.5S+C	7.9	8.1	9.6		4.3	4.2	2.7	8.1		2.5	6.8	4.0		5.0	3.4	8.5	9.9	1.5
1.2D-1.0W+(L+C_r)+0.5S+C	-7.9	-4.3	0.6		-0.1	1.5	-2.7	-2.1			-1.6	-3.6	-4.4	0.7	-3.0	-8.5	-6.2	-9.8
1.2*(D+0.2*Sds*D)+(E*ρ)+(L+C_r)+0.2S+C	2.0	2.9	0.2		8.3	0.6	0.9	9.6			9.5			10.6		2.0	3.3	
1.2*(D-0.2*Sds*D)-(E*ρ)+(L+C_r)+0.2S+C	-2.0	-0.6	0.2		2.0	0.6	-0.9	1.0			0.8			2.3		-2.0	-0.9	
1.2*(D+0.2*Sds*D)+(E*Ωo)+(L+C_r)+0.2S+C	3.8	4.4	0.2		13.1	0.6	1.8	15.6			15.4			17.4		3.8	5.1	
1.2*(D-0.2*Sds*D)-(E*Ωo)+(L+C_r)+0.2S+C	-3.8	-2.1	0.2		2.0	0.6	-1.8	-0.1			-0.4			2.3		-3.8	-2.6	
0.9D+1.0W	7.9	6.7	9.0		0.9	2.7	2.7	4.4		2.5	3.1	4.0		0.9	3.4	8.5	8.5	1.5
0.9D-1.0W	-7.9	-5.8			-3.5		-2.7	-5.8			-5.2	-3.6	-4.4	-3.4	-3.0	-8.5	-7.6	-9.8
0.9*(D+0.2*Sds*D)+(E*ρ)	2.0	2.0			6.1		0.9	7.4			7.3			8.2		2.0	2.4	
0.9*(D-0.2*Sds*D)-(E*ρ)	-2.0	-1.1			0.9		-0.9	-0.3			-0.4			0.9		-2.0	-1.4	
0.9*(D+0.2*Sds*D)+(E*Ωo)	3.8	3.5			10.9		1.8	13.4			13.2			14.9		3.8	4.2	
0.9*(D-0.2*Sds*D)-(E*Ωo)	-3.8	-2.6			0.9		-1.8	-1.4			-1.6			0.9		-3.8	-3.1	
Bolt Pattern Callout																		
Worst Case Shear (ULT)	7.9		9.6	0.0		6.2	2.7		0.0	2.5		4.0	4.4		3.4	8.5		9.8
Worst Case Uplift (ULT)		-5.8			-3.5			-5.8			-5.2			-3.4			-7.6	
Worst Case Shear (ASD)	4.7		5.4	0.0		3.5	1.6		0.0	1.5		2.4	2.6		2.0	5.1		5.9
Passive Pressure (pcf)		259			260			261			262			263			264	
Friction Coefficient		0.3			0.3			0.3			0.3			0.3			0.3	
Sliding Footing Thickness (in)		12.0			12.0			12.0			12.0			12.0			12.0	
Soil Depth above Footing (in)		24			24			24			24			24			24	
Sliding Footing Width (ft)		4			4			3			3			3			4	
Friction Resistance (Kip)		0.8			0.9			0.6			0.6			0.6			0.8	
Passive Pressure Resistance (Kip)			4.7			4.7			3.5			3.5			3.6			4.8
Total Lateral Resistance (Kip)			**5.5**			**5.6**			**4.1**			**4.1**			**4.1**			**5.6**
Worst Case Uplift Force (ASD Kip)		-3.4			-2.0			-3.4			-3.1			-2.0			-4.5	
Footing Size (Up) (ft)		**3.0**			**2.3**			**3.0**			**2.9**			**2.3**			**3.5**	
Worst Case Down Force (ASD Kip)		5.0			7.9			9.1			8.9			10.0			5.8	
Allowable Bearing Pressure		1500			1500			1500			1500			1500			1500	
Footing Size (Down)		**4.5**			**4.0**			**3.0**			**3.0**			**3.0**			**4.0**	

See following sheets for footing design.
Each footing was designed individually
using reactions indicated on this page.

IBC 2018 Cast-in-place Anchor Bolt Design referencing ACI 318-14 Chapter 17

08-Oct-24
6:00 AM

Version Date: December 4, 2020 — Author: Troy M. Dye — Reviewed By: Scott Porter

JOB TITLE: Kimberly AG Shop — JOB #: 24091
DESCRIPTION: Pattern A anchor bolt design — DESIGNER: ANB

Pier and Anchor Bolt Geometry

	Min Edge or Spacing		Bolt Quantities
Top Pier Edge	2.5		
AB-f5	0		
AB-f4	0		
AB-f3	0		
AB-f2	0		
AB-f1	1.5		2
Column centerline	0	<<	
AB-b1	1.5		2
AB-b2	0		
AB-b3	0		
AB-b4	0		
AB-b5	0		
Bottom Pier Edge	5.5		4 # Bolts in Group
Left Pier Edge	4.5		Pier Dimensions: 14.0 " wide
AB-l1	2.5		11.0 " long
Column centerline	0	<<	Bolt Group Dimensions
AB-r1	2.5		5.0 " wide
Right Pier Edge	4.5		3.0 " long

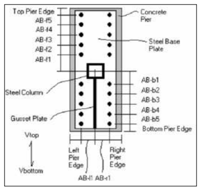


Figure 1. Generic Anchor Bolt Locations

Design Loads

Shear V_u : **4.9** kips — *Use ACI 318 Section 5.3 load combinations*

Tension P_u: **7.9** kips

Is the seismic tension component more than 20% of the total factored tension force? NO
Pier confined for tension force? YES
Pier confined for shear force? NO

Is Anchorage part of the Main Seismic Force Resisting System? Yes
Select the Seismic Design Category: B

Anchors Torqued:	NO		
Bolt Grade:	A36		
Bolt Diameter:	0.625	inches	
Nut Type:	HEX		
Embedment Length:	8	inches	
Concrete f'_c:	4500	psi	
Rebar f_y:	60000	psi	
Anchor F_u :	58	ksi	
Bolt Area A_{bolt} :	0.23	in^2	
Seismic Design Category Factor:	1		*Section 17.2.3.4.4*
Pier Ht / Ftg Thick:	12	inches	OK
Bolt type:	H		

Yielding Mechanism — 17.2.3.4.3b & 17.2.3.5.3a

Bearing Area:	0.69	in^2	
Steel ϕ_t:	0.75		*Section 17.3.3*
Steel ϕ_s:	0.65		*Section 17.3.3*
Concrete ϕ:	0.75		*Section 17.3.3*
λ :	1		*Section 19.2.4*
λ_a :	1		*Section 17.2.6*

Eccentricity for Tension: 0 in
Eccentricity for Shear: 0

Bolt Diameter (in.)	*Threads per (inch)*	*Tensile Stress Area (in^2)*
0.5	13	0.14
0.625	11	0.23
0.75	10	0.33
0.875	9	0.46
1	8	0.61
1.125	7	0.76
1.25	7	0.97
1.375	6	1.15
1.5	6	1.41
1.75	5	1.90
2	4.5	2.50
2.25	4.5	3.25
2.5	4	4.00
2.75	4	4.93
3	4	5.97

Tension Calculations

A_N : 154 in^2 — Left Edge Dist. : c_{a2} 4.5 inches — Top Edge Dist. : c_{a1} 2.5 inches
A_{NO} : 121 in^2 — Right Edge Dist. : 4.5 inches — Bottom Edge Dist. : 5.5 inches

Concrete Tension Capacity (Section 17.4.2) - Breakout Strength

$\psi_{ec,N}$: 1.00 — $\psi_{ed,N}$: 0.76 — $\psi_{c,N}$: 1
N_b = 11.3 kips — N_{cb} = 11.0 kips

Concrete Tension Capacity (Section 17.4.3) - Pullout Strength

$\psi_{c,P}$: 1 — N_{pn} = 24.8 kips
N_p = 24.8 kips — Capacity of Group: 99.4 kips

Concrete Tension Capacity (Section 17.4.4) - Side-Face Blowout Strength

Use A_N : 154 in^2
N_{sb} = 22.3 kips — N_{sbg} = 29.7 kips
ϕP_c : 22.3 kips — P_c : **29.7** kips — Ultimate Concrete Strength Based on Pullout Only

Steel Tension Capacity (Section 17.4.1)

P_{ss}' : 13.1 kips / AB — P_{ss} : **52.4** kips — Ultimate Steel Tension Capacity

IBC 2018 Cast-in-place Anchor Bolt Design referencing ACI 318-14 Chapter 17

08-Oct-24
6:00 AM

Version Date: December 4, 2020

JOB TITLE: Kimberly AG Shop — JOB #: 24091
DESCRIPTION: Pattern A anchor bolt design — DESIGNER: ANB

Shear Calculations

B — Specify which system resists shear

Bolts Resisting Shear — Bolts resist tension and shear

Steel Shear Capacity (Section 17.5.1)

V_{sa} : 6.29 kips / AB — V_{sa} : **12.6** kips — Ultimate Steel Shear Capacity based on Assumed Breakout Surface
n : 2 — Number of anchors resisting shear based on assumed concrete breakout surface
Grout height below base plate, h_2 : 1.5 inches

Concrete Shear Breakout Capacity (Section 17.5.2)

$\psi_{ec,V}$: 1.00 — $\psi_{ed,V}$: 0.86 — $\psi_{c,V}$: 1.2 — $\psi_{h,V}$: 1.00

Top Edge Bolts			Bottom Edge Bolts		
Edge Distance:	2.5	inches	Edge Distance :	5.5	inches
A_{Vc} =	46.9	in^2	A_{Vc} =	115.5	in^2
A_{Vco} =	28.1	in^2	A_{Vco} =	136.1	in^2
V_b =	2.2	kips	V_b =	7.3	kips
V_{cb} =	3.8	kips	V_{cb} =	6.4	kips

Concrete Shear Pryout Capacity (Section 17.5.3)

V_{cp} = 70.7 kips — N_b = 36.4 kips
Group Capacity V_n: **15.4** kips — Group Capacity V_n: **25.5** kips — Ultimate Concrete Strength

Combined Tension and Shear

Shear Force Acting Towards Top Pier Edge, V_{top}

0.425	< 1 - OK	Concrete Shear ($V_u/\phi V_n$)
0.599	< 1 - OK	Steel Shear ($V_u/\phi V_{ss}$)

Anchor Bolts O.K.
Concrete O.K. — Checks Shear Only

Shear Force Acting Towards Bottom Pier Edge, V_{bottom}

0.106	< 1 - OK	Concrete Tension ($P_u/\phi P_n$)
0.256	< 1 - OK	Concrete Shear ($V_u/\phi V_n$)
0.256	< 1 - OK	Concrete Combined (Section D.7)
0.201	< 1 - OK	Steel Tension ($P_u/\phi P_{ss}$)
0.599	< 1 - OK	Steel Shear ($V_u/\phi V_{ss}$)
0.800	<1.2 - O.K.	Steel Combined (Section D.7)

Anchor Bolts O.K.
Concrete O.K. — Checks Shear and Tension

ACI 318 SECTION 17.2.3.4.3 DOES NOT APPLY

DESIGN SUMMARY

Anchor Bolts — **(4) 0.625" diameter headed anchor bolts w/ 8"minimum embedment**
Designed for combined tension and shear

Tension Confined Pier — **14" wide x 11" long w/ min (3) #5 vertical bars**
Designed to transfer anchor bolt tension into reinforcement

Shear Confined Pier — **N/A**
Designed to transfer anchor bolt shear into reinforcement

IBC 2018 Cast-in-place Anchor Bolt Design referencing ACI 318-14 Chapter 17

08-Oct-24
6:00 AM

Version Date: December 4, 2020

JOB TITLE: Kimberly AG Shop — JOB #: 24091
DESCRIPTION: Pattern A anchor bolt design — DESIGNER: ANB

Anchor Reinforcing

Reinforcing Data

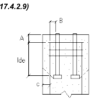
λ :	1
ψ_e :	1
ψ_s :	0.8
ψ_t :	1
ψ_c :	1
ψ_r :	1

Assumes that vertical reinforcement layout is symmetrical around anchor bolt pattern

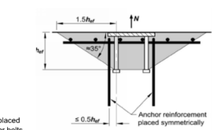
NOTE: The calculation for the concrete anchor capacities are based on 1) Tension pullout, 2) Shear pryout. The breakout strength in tension and shear and side face blowout are omitted because the vertical reinforcement is used to confine this failure cone.

Pier Reinforcement to Resist Tension Breakout (17.4.2.9)

Vert. Pier Reinforcing Size :	5	bar
Distance from A.B. to Rebar (B) :	4	in
Cover above vert. reinf (A) :	2	in
c :	1.5	in
Rebar Area :	0.31	in^2
Rebar Diameter :	0.625	in
l_{db} :	13.98	in
l_{de} :	3.33	in
0.75^*F_y of Rebar @ l_{de} - f_s :	10.73	ksi
A_{st} :	0.74	in^2
Total # of vertical bars required :	3	
Embedment of standard hook :	6.00	in



Quantity of reinforcement placed symmetrically around anchor bolts



Section A-A

Pier Reinforcement to Resist Shear Breakout (17.5.2.9)

Hairpin/stirrup reinforcing size :	4	bar
Rebar area :	0.4	in^2
A_{st} :	0.11	in^2
Total # of hairpins/stirrups required :	1	Quantity of hairpins or stirrups wrapp around anchor bolts

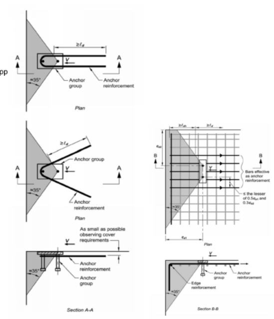


Fig. RD.6.2.9(a)—Hairpin anchor reinforcement for shear.

Fig. RD.6.2.9(b)—Edge reinforcement and anchor reinforcement for shear.

IBC 2018 Cast-in-place Anchor Bolt Design referencing ACI 318-14 Chapter 17

08-Oct-24
6:00 AM

Version Date: December 4, 2020 | Author: Troy M. Dye | Reviewed By: Scott Porter

JOB TITLE: Kimberly AG Shop | JOB #: 24091
DESCRIPTION: Pattern B anchor bolt design | DESIGNER: ANB

Pier and Anchor Bolt Geometry

	Min Edge or Spacing		Bolt Quantities
Top Pier Edge	5.5		
AB-f5	0		
AB-f4	0		
AB-f3	0		
AB-f2	0		
AB-f1	1.5		2
Column centerline	0	<<	
AB-b1	1.5		2
AB-b2	0		
AB-b3	0		
AB-b4	0		
AB-b5	0		
Bottom Pier Edge	5.5		4 # Bolts in Group
Left Pier Edge	4.5		
AB-l1	2.5		
Column centerline	0	<<	
AB-r1	2.5		
Right Pier Edge	4.5		

Pier Dimensions
14.0 " wide
14.0 " long

Bolt Group Dimensions
5.0 " wide
3.0 " long

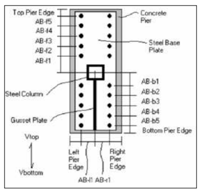


Figure 1. Generic Anchor Bolt Locations

Design Loads

Shear V_u : 4.4 kips

Tension P_u: 5.8 kips

Use ACI 318 Section 5.3 load combinations

Is the seismic tension component more than 20% of the total factored tension force?	NO
Pier confined for tension force?	YES
Pier confined for shear force?	NO
Is Anchorage part of the Main Seismic Force Resisting System?	Yes
Select the Seismic Design Category:	B

Anchors Torqued:	NO	
Bolt Grade:	A36	
Bolt Diameter:	0.625	inches
Nut Type:	HEX	
Embedment Length:	8	inches
Concrete f'_c:	4500	psi
Rebar f_y:	60000	psi
Anchor F_u :	58	ksi
Bolt Area A_{bolt} :	0.23	in^2
Seismic Design Category Factor:	1	*Section 17.2.3.4.4*
Pier Ht / Ftg Thick:	12	inches **OK**
Bolt type:	H	

Yielding Mechanism | 17.2.3.4.3b & 17.2.3.5.3a

Bearing Area:	0.69	in^2
Steel ϕ_t:	0.75	*Section 17.3.3*
Steel ϕ_s:	0.65	*Section 17.3.3*
Concrete ϕ:	0.75	*Section 17.3.3*
λ :	1	*Section 19.2.4*
λ_a :	1	*Section 17.2.6*

Eccentricity for Tension
0 in
Eccentricity for Shear
0

Bolt Diameter (in.)	*Threads per (inch)*	*Tensile Stress Area (in^2)*
0.5	13	0.14
0.625	11	0.23
0.75	10	0.33
0.875	9	0.46
1	8	0.61
1.125	7	0.76
1.25	7	0.97
1.375	6	1.15
1.5	6	1.41
1.75	5	1.90
2	4.5	2.50
2.25	4.5	3.25
2.5	4	4.00
2.75	4	4.93
3	4	5.97

Tension Calculations

A_N : 196 in^2
A_{NO} : 121 in^2

	c_{a2}			c_{a1}	
Left Edge Dist. :	4.5	inches	Top Edge Dist. :	5.5	inches
Right Edge Dist. :	4.5	inches	Bottom Edge Dist. :	5.5	inches

Concrete Tension Capacity (Section 17.4.2) - Breakout Strength

$\Psi_{ec,N}$: 1.00 $\Psi_{ed,N}$: 0.81 $\Psi_{c,N}$: 1

N_b = 11.3 kips N_{cb} = 14.9 kips

Concrete Tension Capacity (Section 17.4.3) - Pullout Strength

$\Psi_{c,P}$: 1 N_{pn} = 24.8 kips

N_p = 24.8 kips Capacity of Group: 99.4 kips

Concrete Tension Capacity (Section 17.4.4) - Side-Face Blowout Strength

Use A_N : 196 in^2

N_{sb} = N/A kips N_{sbg} = N/A kips

ϕP_c : 74.5 kips P_c : 99.4 kips — Ultimate Concrete Strength Based on Pullout Only

Steel Tension Capacity (Section 17.4.1)

P_{ss}' : 13.1 kips / AB P_{ss} : 52.4 kips — Ultimate Steel Tension Capacity

---

IBC 2018 Cast-in-place Anchor Bolt Design referencing ACI 318-14 Chapter 17

08-Oct-24
6:00 AM

Version Date: December 4, 2020

JOB TITLE: Kimberly AG Shop | JOB #: 24091
DESCRIPTION: Pattern B anchor bolt design | DESIGNER: ANB

Shear Calculations

B — Specify which system resists shear

Bolts Resisting Shear — Bolts resist tension and shear

Steel Shear Capacity (Section 17.5.1)

V_{sa} : 6.29 kips / AB V_{sa} : 12.6 kips — Ultimate Steel Shear Capacity based on Assumed Breakout Surface

n : 2 Number of anchors resisting shear based on assumed concrete breakout surface

Grout height below base plate, h_2 : 1.5 inches

Concrete Shear Breakout Capacity (Section 17.5.2)

$\Psi_{ec,V}$: 1.00 $\Psi_{ed,V}$: 0.86 $\Psi_{c,V}$: 1.2 $\Psi_{h,V}$: 1.00

Top Edge Bolts			Bottom Edge Bolts		
Edge Distance:	5.5	inches	Edge Distance :	5.5	inches
A_{Vc} =	115.5	in^2	A_{Vc} =	115.5	in^2
A_{Vco} =	136.1	in^2	A_{Vco} =	136.1	in^2
V_b =	7.3	kips	V_b =	7.3	kips
V_{cb} =	6.4	kips	V_{cb} =	6.4	kips

Concrete Shear Pryout Capacity (Section 17.5.3)

V_{cp} =	95.9	kips	N_b =	36.4	kips
Group Capacity V_n:	25.5	kips	Group Capacity V_n:	25.5	kips

Ultimate Concrete Strength

Combined Tension and Shear

Shear Force Acting Towards Top Pier Edge, V_{top}

0.230	< 1 - OK	Concrete Shear ($V_u/\phi V_n$)
0.538	< 1 - OK	Steel Shear ($V_u/\phi V_{ss}$)

Anchor Bolts O.K.
Concrete O.K. Checks Shear Only

Shear Force Acting Towards Bottom Pier Edge, V_{bottom}

0.078	< 1 - OK	Concrete Tension ($P_u/\phi P_n$)
0.230	< 1 - OK	Concrete Shear ($V_u/\phi V_n$)
0.230	< 1 - OK	Concrete Combined (Section D.7)
0.147	< 1 - OK	Steel Tension ($P_u/\phi P_{ss}$)
0.538	< 1 - OK	Steel Shear ($V_u/\phi V_{ss}$)
0.538	< 1 - OK	Steel Combined (Section D.7)

Anchor Bolts O.K.
Concrete O.K. Checks Shear and Tension

ACI 318 SECTION 17.2.3.4.3 DOES NOT APPLY

DESIGN SUMMARY

Anchor Bolts — **(4) 0.625" diameter headed anchor bolts w/ 8"minimum embedment**
Designed for combined tension and shear

Tension Confined Pier — **14" wide x 14" long w/ min (2) #5 vertical bars**
Designed to transfer anchor bolt tension into reinforcement

Shear Confined Pier — **N/A**
Designed to transfer anchor bolt shear into reinforcement

IBC 2018 Cast-in-place Anchor Bolt Design referencing ACI 318-14 Chapter 17

08-Oct-24
6:00 AM

Version Date: December 4, 2020

JOB TITLE: Kimberly AG Shop | JOB #: 24091
DESCRIPTION: Pattern B anchor bolt design | DESIGNER: ANB

Anchor Reinforcing

Reinforcing Data

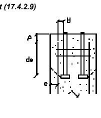
λ :	1
ψ_e :	1
ψ_s :	0.8
ψ_t :	1
ψ_c :	1
ψ_r :	1

Assumes that vertical reinforcement layout is symmetrical around anchor bolt pattern

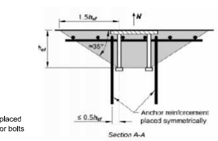
NOTE: The calculation for the concrete anchor capacities are based on 1) Tension pullout, 2) Shear pryout. The breakout strength in tension and shear and side face blowout are omitted because the vertical reinforcement is used to confine this failure cone.

Pier Reinforcement to Resist Tension Breakout (17.4.2.9)

Vert. Pier Reinforcing Size :	5	bar
Distance from A.B. to Rebar (B) :	4	in
Cover above vert. reinf (A) :	2	in
c :	1.5	in
Rebar Area :	0.31	in^2
Rebar Diameter :	0.625	in
l_{db} :	13.98	in
l_{de} :	3.33	in
$0.75 \cdot F_y$ of Rebar @ l_{de} - f_s :	10.73	ksi
A_{st} :	0.54	in^2
Total # of vertical bars required :	2	
Embedment of standard hook :	6.00	in

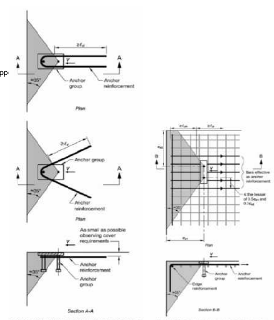


Quantity of reinforcement placed symmetrically around anchor bolts



Pier Reinforcement to Resist Shear Breakout (17.5.2.9)

Hairpin/stirrup reinforcing size :	4	bar
Rebar area :	0.4	in^2
A_{st} :	0.10	in^2
Total # of hairpins/stirrups required :	1	Quantity of hairpins or stirrups wrapp around anchor bolts



IBC 2018 Cast-in-place Anchor Bolt Design referencing ACI 318-14 Chapter 17

08-Oct-24
6:00 AM

Version Date: December 4, 2020 Author: Troy M. Dye Reviewed By: Scott Porter

JOB TITLE: Kimberly AG Shop JOB #: 24091
DESCRIPTION: Pattern C anchor bolt design DESIGNER: ANB

Pier and Anchor Bolt Geometry

	Min Edge or Spacing	Bolt Quantities
Top Pier Edge	5.5	
AB-f5	0	
AB-f4	0	
AB-f3	0	
AB-f2	0	
AB-f1	1.5	2
Column centerline	0 <<	
AB-b1	1.5	2
AB-b2	0	
AB-b3	0	
AB-b4	0	
AB-b5	0	
Bottom Pier Edge	5.5	4 # Bolts in Group

Pier Dimensions
10.0 " wide
14.0 " long

Left Pier Edge	2.5
AB-l1	1.5
Column centerline	0 <<
AB-r1	1.5
Right Pier Edge	4.5

Bolt Group Dimensions
3.0 " wide
3.0 " long

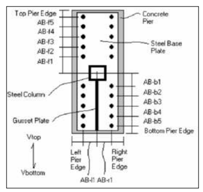


Figure 1. Generic Anchor Bolt Locations

Design Loads

Shear V_u : **4.9** kips

Tension P_u: **6** kips

Use ACI 318 Section 5.3 load combinations

Is the seismic tension component more than 20% of the total factored tension force? NO

Pier confined for tension force? YES

Pier confined for shear force? NO

Is Anchorage part of the Main Seismic Force Resisting System? Yes

Select the Seismic Design Category: B

Anchors Torqued: NO
Bolt Grade: A36
Bolt Diameter: 0.625 inches
Nut Type: HEX
Embedment Length: 8 inches
Concrete f'_c: 4500 psi
Rebar f_y: 60000 psi
Anchor F_u : 58 ksi
Bolt Area A_{bolt} : 0.23 in²
Seismic Design Category Factor: 1 *Section 17.2.3.4.4*

Pier Ht / Ftg Thick: 12 inches **OK**
Bolt type: H

Yielding Mechanism 17.2.3.4.3b & 17.2.3.5.3a
Bearing Area: 0.69 in²
Steel ϕ_t: 0.75 *Section 17.3.3*
Steel ϕ_s: 0.65 *Section 17.3.3*
Concrete ϕ: 0.75 *Section 17.3.3*
λ : 1 *Section 19.2.4*
λ_a : 1 *Section 17.2.6*

Eccentricity for Tension
0 in
Eccentricity for Shear
0

Bolt Diameter (in.)	*Threads per* (inch)	*Tensile Stress Area* (in²)
0.5	13	0.14
0.625	11	0.23
0.75	10	0.33
0.875	9	0.46
1	8	0.61
1.125	7	0.76
1.25	7	0.97
1.375	6	1.15
1.5	6	1.41
1.75	5	1.90
2	4.5	2.50
2.25	4.5	3.25
2.5	4	4.00
2.75	4	4.93
3	4	5.97

Tension Calculations

A_N : 140 in²
A_{NO} : 121 in²

	c_{a2}			c_{a1}	
Left Edge Dist. :	2.5	inches	Top Edge Dist. :	5.5	inches
Right Edge Dist. :	4.5	inches	Bottom Edge Dist. :	5.5	inches

Concrete Tension Capacity (Section 17.4.2) - Breakout Strength
$\psi_{ec,N}$: 1.00 $\psi_{ed,N}$: 0.76 $\psi_{c,N}$: 1
N_b = 11.3 kips N_{cb} = 10.0 kips

Concrete Tension Capacity (Section 17.4.3) - Pullout Strength
$\psi_{c,P}$: 1 N_{pn} = 24.8 kips
N_p = 24.8 kips Capacity of Group: 99.4 kips

Concrete Tension Capacity (Section 17.4.4) - Side-Face Blowout Strength
Use A_N : 140 in²
N_{sb} = 22.3 kips N_{sbg} = 26.7 kips
ϕP_c : 20.1 kips **P_c :** **26.7** kips Ultimate Concrete Strength Based on Pullout Only

Steel Tension Capacity (Section 17.4.1)
P_{ss}' : 13.1 kips / AB **P_{ss} :** **52.4** kips Ultimate Steel Tension Capacity

IBC 2018 Cast-in-place Anchor Bolt Design referencing ACI 318-14 Chapter 17

08-Oct-24
6:00 AM

Version Date: December 4, 2020
JOB TITLE: Kimberly AG Shop JOB #: 24091
DESCRIPTION: Pattern C anchor bolt design DESIGNER: ANB

Shear Calculations

B Specify which system resists shear

Bolts Resisting Shear Bolts resist tension and shear

Steel Shear Capacity (Section 17.5.1)
V_{sa} : 6.29 kips / AB **V_{sa} :** **12.6** kips Ultimate Steel Shear Capacity based on Assumed Breakout Surface
n : 2 Number of anchors resisting shear based on assumed concrete breakout surface
Grout height below base plate, h_2 : 1.5 inches

Concrete Shear Breakout Capacity (Section 17.5.2)
$\psi_{ec,V}$: 1.00 $\psi_{ed,V}$: 0.86 $\psi_{c,V}$: 1.2 $\psi_{h,V}$: 1.00

Top Edge Bolts			Bottom Edge Bolts		
Edge Distance:	5.5	inches	Edge Distance :	5.5	inches
A_{Vc} =	82.5	in²	A_{Vc} =	82.5	in²
A_{Vco} =	136.1	in²	A_{Vco} =	136.1	in²
V_b =	7.3	kips	V_b =	7.3	kips
V_{cb} =	4.6	kips	V_{cb} =	4.6	kips

Concrete Shear Pryout Capacity (Section 17.5.3)
V_{cp} = 64.3 kips N_b = 36.4 kips
Group Capacity **V_n:** **18.2** kips Group Capacity **V_n:** **18.2** kips Ultimate Concrete Strength

Combined Tension and Shear

Shear Force Acting Towards Top Pier Edge, V_{top}

0.358	< 1 - OK	Concrete Shear ($V_u/\phi V_n$)
0.599	< 1 - OK	Steel Shear ($V_u/\phi V_{ss}$)

Anchor Bolts O.K.
Concrete O.K. Checks Shear Only

Shear Force Acting Towards Bottom Pier Edge, V_{bottom}

0.081	< 1 - OK	Concrete Tension ($P_u/\phi P_n$)
0.358	< 1 - OK	Concrete Shear ($V_u/\phi V_n$)
0.358	< 1 - OK	Concrete Combined (Section D.7)
0.153	< 1 - OK	Steel Tension ($P_u/\phi P_{ss}$)
0.599	< 1 - OK	Steel Shear ($V_u/\phi V_{ss}$)
0.599	< 1 - OK	Steel Combined (Section D.7)

Anchor Bolts O.K.
Concrete O.K. Checks Shear and Tension
ACI 318 SECTION 17.2.3.4.3 DOES NOT APPLY

DESIGN SUMMARY

Anchor Bolts **(4) 0.625" diameter headed anchor bolts w/ 8"minimum embedment**
Designed for combined tension and shear
Tension Confined Pier **10" wide x 14" long w/ min (2) #5 vertical bars**
Designed to transfer anchor bolt tension into reinforcement
Shear Confined Pier **N/A**
Designed to transfer anchor bolt shear into reinforcement

IBC 2018 Cast-in-place Anchor Bolt Design referencing ACI 318-14 Chapter 17

08-Oct-24
6:00 AM

Version Date: December 4, 2020
JOB TITLE: Kimberly AG Shop JOB #: 24091
DESCRIPTION: Pattern C anchor bolt design DESIGNER: ANB

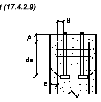
Anchor Reinforcing

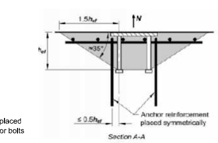
Reinforcing Data
λ : 1
ψ_e : 1
ϕ_t : 0.8
ψ_s : 1
ψ_c : 1
ψ_t : 1

Assumes that vertical reinforcement layout is symmetrical around anchor bolt pattern

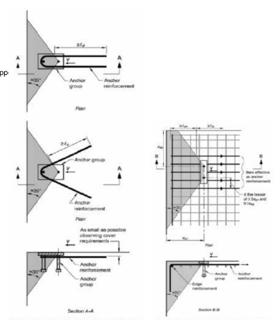
NOTE: The calculation for the concrete anchor capacities are based on 1) Tension pullout, 2) Shear pryout. The breakout strength in tension and shear and side face blowout are omitted because the vertical reinforcement is used to confine this failure cone.

Pier Reinforcement to Resist Tension Breakout (17.4.2.9)
Vert. Pier Reinforcing Size : 5 bar
Distance from A.B. to Rebar (B) : 4 in
Cover above vert. reinf (A) : 2 in
c : 1.5 in
Rebar Area : 0.31 in²
Rebar Diameter : 0.625 in
l_{dh} : 13.98 in
l_{dh} : 3.33 in
0.75*F_y of Rebar @ l_{dh} * l_s : 10.73 ksi
A_s : 0.56 in²
Total # of vertical bars required 2 Quantity of reinforcement placed symmetrically around anchor bolts
Embedment of standard hook : 6.00 in





Pier Reinforcement to Resist Shear Breakout (17.5.2.9)
Hairpin/stirrup reinforcing size : 4 bar
Rebar area : 0.4 in²
A_s : 0.11 in²
Total # of hairpins/stirrups required 1 Quantity of hairpins or stirrups wrapp around anchor bolts



08-Oct-24
6:00 AM

IBC 2018 Cast-in-place Anchor Bolt Design referencing ACI 318-14 Chapter 17

Version Date: December 4, 2020 | Author: Troy M. Dye | Reviewed By: Scott Porter

JOB TITLE: Kimberly AG Shop | JOB #: 24091
DESCRIPTION: Pattern D anchor bolt design | DESIGNER: ANB

Pier and Anchor Bolt Geometry

	Min Edge or Spacing		Bolt Quantities
Top Pier Edge	5.5		
AB-f5	0		
AB-f4	0		
AB-f3	0		
AB-f2	0		
AB-f1	1.5		2
Column centerline	0	<<	
AB-b1	1.5		2
AB-b2	0		
AB-b3	0		
AB-b4	0		
AB-b5	0		
Bottom Pier Edge	5.5		4 # Bolts in Group
Left Pier Edge	4.5		
AB-l1	1.5		
Column centerline	0	<<	
AB-r1	1.5		
Right Pier Edge	4.5		

Pier Dimensions: 12.0 " wide; 14.0 " long
Bolt Group Dimensions: 3.0 " wide; 3.0 " long

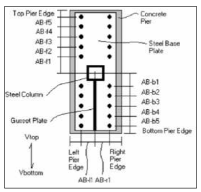


Figure 1. Generic Anchor Bolt Locations

Design Loads

Shear V_u: **8.5** kips
Tension P_u: **7.6** kips

Use ACI 318 Section 5.3 load combinations

Is the seismic tension component more than 20% of the total factored tension force? NO
Pier confined for tension force? YES
Pier confined for shear force? NO
Is Anchorage part of the Main Seismic Force Resisting System? Yes
Select the Seismic Design Category: B

Anchors Torqued:	NO	
Bolt Grade:	F1554 Gr. 55	
Bolt Diameter:	0.625	inches
Nut Type:	HEX	
Embedment Length:	8	inches
Concrete f'_c:	4500	psi
Rebar f_y:	60000	psi
Anchor F_u:	75	ksi
Bolt Area A_{bolt}:	0.23	in^2
Seismic Design Category Factor:	1	*Section 17.2.3.4.4*
Pier Ht / Ftg Thick:	12	inches **OK**
Bolt type:	H	

Yielding Mechanism — 17.2.3.4.3b & 17.2.3.5.3a

Bearing Area:	0.69	in^2
Steel ϕ_t:	0.75	*Section 17.3.3*
Steel ϕ_s:	0.65	*Section 17.3.3*
Concrete ϕ:	0.75	*Section 17.3.3*
λ:	1	*Section 19.2.4*
λ_a:	1	*Section 17.2.6*

Eccentricity for Tension: 0 in
Eccentricity for Shear: 0

Bolt Diameter (in.)	*Threads per (inch)*	*Tensile Stress Area (in^2)*
0.5	13	0.14
0.625	11	0.23
0.75	10	0.33
0.875	9	0.46
1	8	0.61
1.125	7	0.76
1.25	7	0.97
1.375	6	1.15
1.5	6	1.41
1.75	5	1.90
2	4.5	2.50
2.25	4.5	3.25
2.5	4	4.00
2.75	4	4.93
3	4	5.97

Tension Calculations

A_N: 168 in^2
A_{NO}: 121 in^2

	c_{a2}			c_{a1}	
Left Edge Dist. :	4.5	inches	Top Edge Dist. :	5.5	inches
Right Edge Dist. :	4.5	inches	Bottom Edge Dist. :	5.5	inches

Concrete Tension Capacity (Section 17.4.2) - Breakout Strength
$\psi_{ec,N}$: 1.00 $\psi_{ed,N}$: 0.81 $\psi_{c,N}$: 1
N_b = 11.3 kips N_{cb} = 12.8 kips

Concrete Tension Capacity (Section 17.4.3) - Pullout Strength
$\psi_{c,P}$: 1 N_{pn} = 24.8 kips
N_p = 24.8 kips Capacity of Group: 99.4 kips

Concrete Tension Capacity (Section 17.4.4) - Side-Face Blowout Strength
Use A_N: 168 in^2
N_{sb} = N/A kips N_{sbg} = N/A kips
ϕP_c: 74.5 kips **P_c: 99.4** kips — Ultimate Concrete Strength Based on Pullout Only

Steel Tension Capacity (Section 17.4.1)
P_{ss}': 17.0 kips / AB **P_{ss}: 67.8** kips — Ultimate Steel Tension Capacity

---

08-Oct-24
6:00 AM

IBC 2018 Cast-in-place Anchor Bolt Design referencing ACI 318-14 Chapter 17

Version Date: December 4, 2020
JOB TITLE: Kimberly AG Shop | JOB #: 24091
DESCRIPTION: Pattern D anchor bolt design | DESIGNER: ANB

Shear Calculations

B — Specify which system resists shear
Bolts Resisting Shear — Bolts resist tension and shear

Steel Shear Capacity (Section 17.5.1)
V_{sa}: 8.14 kips / AB **V_{sa}: 16.3** kips — Ultimate Steel Shear Capacity based on Assumed Breakout Surface
n : 2 Number of anchors resisting shear based on assumed concrete breakout surface
Grout height below base plate, h_2: 1.5 inches

Concrete Shear Breakout Capacity (Section 17.5.2)
$\psi_{ec,V}$: 1.00 $\psi_{ed,V}$: 0.86 $\psi_{c,V}$: 1.2 $\psi_{h,V}$: 1.00

Top Edge Bolts			Bottom Edge Bolts		
Edge Distance:	5.5	inches	Edge Distance :	5.5	inches
A_{Vc} =	99.0	in^2	A_{Vc} =	99.0	in^2
A_{Vco} =	136.1	in^2	A_{Vco} =	136.1	in^2
V_b =	7.3	kips	V_b =	7.3	kips
V_{cb} =	5.5	kips	V_{cb} =	5.5	kips

Concrete Shear Pryout Capacity (Section 17.5.3)
V_{cp} = 82.2 kips N_b = 36.4 kips
Group Capacity **V_n: 21.9** kips Group Capacity **V_n: 21.9** kips — Ultimate Concrete Strength

Combined Tension and Shear

Shear Force Acting Towards Top Pier Edge, V_{top}

0.518	< 1 - OK	Concrete Shear ($V_u/\phi V_n$)
0.804	< 1 - OK	Steel Shear ($V_u/\phi V_{ss}$)

Anchor Bolts O.K.
Concrete O.K. Checks Shear Only

Shear Force Acting Towards Bottom Pier Edge, V_{bottom}

0.102	< 1 - OK	Concrete Tension ($P_u/\phi P_n$)
0.518	< 1 - OK	Concrete Shear ($V_u/\phi V_n$)
0.518	< 1 - OK	Concrete Combined (Section D.7)
0.149	< 1 - OK	Steel Tension ($P_u/\phi P_{ss}$)
0.804	< 1 - OK	Steel Shear ($V_u/\phi V_{ss}$)
0.804	< 1 - OK	Steel Combined (Section D.7)

Anchor Bolts O.K.
Concrete O.K. Checks Shear and Tension
ACI 318 SECTION 17.2.3.4.3 DOES NOT APPLY

DESIGN SUMMARY

Anchor Bolts — **(4) 0.625" diameter headed anchor bolts w/ 8"minimum embedment**
Designed for combined tension and shear
Tension Confined Pier — **12" wide x 14" long w/ min (3) #5 vertical bars**
Designed to transfer anchor bolt tension into reinforcement
Shear Confined Pier — **N/A**
Designed to transfer anchor bolt shear into reinforcement

08-Oct-24
6:00 AM

IBC 2018 Cast-in-place Anchor Bolt Design referencing ACI 318-14 Chapter 17

Version Date: December 4, 2020
JOB TITLE: Kimberly AG Shop | JOB #: 24091
DESCRIPTION: Pattern D anchor bolt design | DESIGNER: ANB

Anchor Reinforcing

Reinforcing Data

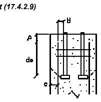
λ:	1
ψ_e:	1
ψ_s:	0.8
ψ_t:	1
ψ_c:	1
ψ_r:	1

Assumes that vertical reinforcement layout is symmetrical around anchor bolt pattern

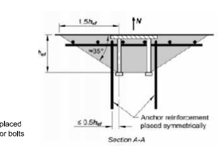
NOTE: The calculation for the concrete anchor capacities are based on 1) Tension pullout, 2) Shear pryout. The breakout strength in tension and shear and side face blowout are omitted because the vertical reinforcement is used to confine this failure cone.

Pier Reinforcement to Resist Tension Breakout (17.4.2.9)

Vert. Pier Reinforcing Size :	5	bar
Distance from A.B. to Rebar (B) :	4	in
Cover above vert. reinf (A) :	2	in
c :	1.5	in
Rebar Area :	0.31	in^2
Rebar Diameter :	0.625	in
l_{db}:	13.98	in
l_{de}:	3.33	in
$0.75{*}F_y$ of Rebar @ l_{de} - f_s:	10.73	ksi
A_{st}:	0.71	in^2
Total # of vertical bars required :	3	
Embedment of standard hook :	6.00	in

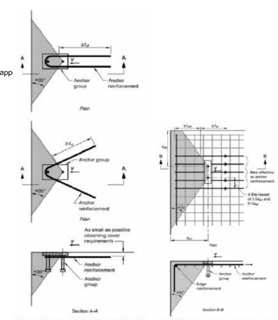


Quantity of reinforcement placed symmetrically around anchor bolts



Pier Reinforcement to Resist Shear Breakout (17.5.2.9)

Hairpin/stirrup reinforcing size :	4	bar
Rebar area :	0.4	in^2
A_{st}:	0.19	in^2
Total # of hairpins/stirrups required :	1	Quantity of hairpins or stirrups wrapp around anchor bolts



08-Oct-24

IBC 2018 Cast-in-place Anchor Bolt Design referencing ACI 318-14 Chapter 17

6:00 AM

Version Date: December 4, 2020 | Author: Troy M. Dye | Reviewed By: Scott Porter

JOB TITLE: Kimberly AG Shop | JOB #: 24091
DESCRIPTION: Pattern E & F anchor bolt design | DESIGNER: ANB

Pier and Anchor Bolt Geometry

	Min Edge or Spacing		Bolt Quantities
Top Pier Edge	6.5		
AB-f5	0		
AB-f4	0		
AB-f3	0		
AB-f2	0		
AB-f1	1.5		2
Column centerline	0	<<	
AB-b1	1.5		2
AB-b2	0		
AB-b3	0		
AB-b4	0		
AB-b5	0		
Bottom Pier Edge	6.5		4 # Bolts in Group

Left Pier Edge	4.5		Pier Dimensions
AB-l1	1.5		12.0 " wide
Column centerline	0	<<	16.0 " long
AB-r1	1.5		Bolt Group Dimensions: 3.0 " wide
Right Pier Edge	4.5		3.0 " long

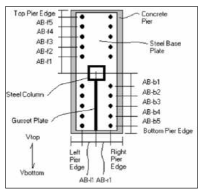


Figure 1. Generic Anchor Bolt Locations

Design Loads

Shear V_u : **8.5** kips — *Use ACI 318 Section 5.3 load combinations*

Tension P_u: **16.8** kips

Is Anchorage part of the Main Seismic Force Resisting System? Yes

Is the seismic tension component more than 20% of the total factored tension force? NO

Select the Seismic Design Category: B

Pier confined for tension force? YES

Pier confined for shear force? NO

Anchors Torqued:	NO				
Bolt Grade:	F1554 Gr. 55		Yielding Mechanism		17.2.3.4.3b & 17.2.3.5.3a
Bolt Diameter:	0.625	inches			
Nut Type:	HEX		Bearing Area:	0.69	in^2
Embedment Length:	8	inches	Steel ϕ_t:	0.75	*Section 17.3.3*
Concrete f'_c:	4500	psi	Steel ϕ_s:	0.65	*Section 17.3.3*
Rebar f_y:	60000	psi	Concrete ϕ:	0.75	*Section 17.3.3*
Anchor F_u :	75	ksi	λ :	1	*Section 19.2.4*
Bolt Area A_{bolt} :	0.23	in^2	λ_a :	1	*Section 17.2.6*
Seismic Design Category Factor:	1	*Section 17.2.3.4.4*			
			Eccentricity for Tension	0	in
Pier Ht / Ftg Thick:	12	inches OK	Eccentricity for Shear	0	
Bolt type:	H				

Bolt Diameter (in.)	*Threads per (inch)*	*Tensile Stress Area (in^2)*
0.5	13	0.14
0.625	11	0.23
0.75	10	0.33
0.875	9	0.46
1	8	0.61
1.125	7	0.76
1.25	7	0.97
1.375	6	1.15
1.5	6	1.41
1.75	5	1.90
2	4.5	2.50
2.25	4.5	3.25
2.5	4	4.00
2.75	4	4.93
3	4	5.97

Tension Calculations

A_N : 192 in^2 | Left Edge Dist. (c_{a2}) : 4.5 inches | Top Edge Dist. (c_{a1}) : 6.5 inches

A_{NO} : 169 in^2 | Right Edge Dist. : 4.5 inches | Bottom Edge Dist. : 6.5 inches

Concrete Tension Capacity (Section 17.4.2) - Breakout Strength

$\psi_{ec,N}$: 1.00 | $\psi_{ed,N}$: 0.81 | $\psi_{c,N}$: 1

N_b = 14.5 kips | N_{cb} = 13.4 kips

Concrete Tension Capacity (Section 17.4.3) - Pullout Strength

$\psi_{c,P}$: 1 | N_{pn} = 24.8 kips

N_p = 24.8 kips | Capacity of Group: 99.4 kips

Concrete Tension Capacity (Section 17.4.4) - Side-Face Blowout Strength

Use A_N : 192 in^2

N_{sb} = N/A kips | N_{sbg} = N/A kips

ϕP_c : 74.5 kips | **P_c** : **99.4** kips — Ultimate Concrete Strength Based on Pullout Only

Steel Tension Capacity (Section 17.4.1)

P_{ss}' : 17.0 kips / AB | **P_{ss}** : **67.8** kips — Ultimate Steel Tension Capacity

---

08-Oct-24

IBC 2018 Cast-in-place Anchor Bolt Design referencing ACI 318-14 Chapter 17

6:00 AM

Version Date: December 4, 2020

JOB TITLE: Kimberly AG Shop | JOB #: 24091
DESCRIPTION: Pattern E & F anchor bolt design | DESIGNER: ANB

Shear Calculations

B — Specify which system resists shear

Bolts Resisting Shear — Bolts resist tension and shear

Steel Shear Capacity (Section 17.5.1)

V_{sa} : 8.14 kips / AB | **V_{sa}** : **16.3** kips — Ultimate Steel Shear Capacity based on Assumed Breakout Surface

n : 2 Number of anchors resisting shear based on assumed concrete breakout surface

Grout height below base plate, h_2 : 1.5 inches

Concrete Shear Breakout Capacity (Section 17.5.2)

$\psi_{ec,V}$: 1.00 | $\psi_{ed,V}$: 0.84 | $\psi_{c,V}$: 1.2 | $\psi_{h,V}$: 1.00

Top Edge Bolts			Bottom Edge Bolts		
Edge Distance:	6.5	inches	Edge Distance :	6.5	inches
A_{Vc} =	117.0	in^2	A_{Vc} =	117.0	in^2
A_{Vco} =	190.1	in^2	A_{Vco} =	190.1	in^2
V_b =	9.3	kips	V_b =	9.3	kips
V_{cb} =	5.8	kips	V_{cb} =	5.8	kips

Concrete Shear Pryout Capacity (Section 17.5.3)

V_{cp} = 67.3 kips | N_b = 36.4 kips

Group Capacity **V_n**: **23.1** kips | Group Capacity **V_n**: **23.1** kips — Ultimate Concrete Strength

Combined Tension and Shear

Shear Force Acting Towards Top Pier Edge, V_{top}			Shear Force Acting Towards Bottom Pier Edge, V_{bottom}		
			0.225	< 1 - OK	Concrete Tension ($P_u/\phi P_n$)
0.491	< 1 - OK	Concrete Shear ($V_u/\phi V_n$)	0.491	< 1 - OK	Concrete Shear ($V_u/\phi V_n$)
			0.716	<1.2 - O.K.	Concrete Combined (Section D.7)
			0.330	< 1 - OK	Steel Tension ($P_u/\phi P_{ss}$)
0.804	< 1 - OK	Steel Shear ($V_u/\phi V_{ss}$)	0.804	< 1 - OK	Steel Shear ($V_u/\phi V_{ss}$)
			1.134	<1.2 - O.K.	Steel Combined (Section D.7)

Anchor Bolts O.K.
Concrete O.K. — Checks Shear Only

Anchor Bolts O.K.
Concrete O.K. — Checks Shear and Tension

ACI 318 SECTION 17.2.3.4.3 DOES NOT APPLY

DESIGN SUMMARY

Anchor Bolts — **(4) 0.625" diameter headed anchor bolts w/ 8"minimum embedment**
Designed for combined tension and shear

Tension Confined Pier — **12" wide x 16" long w/ min (6) #5 vertical bars**
Designed to transfer anchor bolt tension into reinforcement

Shear Confined Pier — **N/A**
Designed to transfer anchor bolt shear into reinforcement

08-Oct-24

IBC 2018 Cast-in-place Anchor Bolt Design referencing ACI 318-14 Chapter 17

6:00 AM

Version Date: December 4, 2020

JOB TITLE: Kimberly AG Shop | JOB #: 24091
DESCRIPTION: Pattern E & F anchor bolt design | DESIGNER: ANB

Anchor Reinforcing

Reinforcing Data

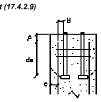
λ :	1
ψ_e :	1
ψ_s :	0.8
ψ_t :	1
ψ_c :	1
ψ_r :	1

Assumes that vertical reinforcement layout is symmetrical around anchor bolt pattern

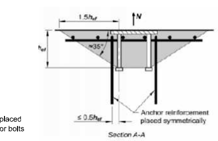
NOTE: The calculation for the concrete anchor capacities are based on 1) Tension pullout, 2) Shear pryout. The breakout strength in tension and shear and side face blowout are omitted because the vertical reinforcement is used to confine this failure cone.

Pier Reinforcement to Resist Tension Breakout (17.4.2.9)

Vert. Pier Reinforcing Size :	5	bar
Distance from A.B. to Rebar (B) :	4	in
Cover above vert. reinf (A) :	2	in
c :	1.5	in
Rebar Area :	0.31	in^2
Rebar Diameter :	0.625	in
l_{db} :	13.98	in
l_{de} :	3.33	in
0.75^*F_y of Rebar @ l_{de} - f_s :	10.73	ksi
A_{st} :	1.57	in^2
Total # of vertical bars required :	6	
Embedment of standard hook :	6.00	in

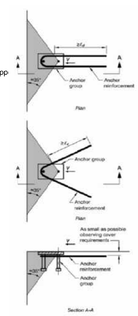


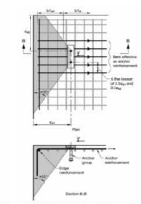
Quantity of reinforcement placed symmetrically around anchor bolts



Pier Reinforcement to Resist Shear Breakout (17.5.2.9)

Hairpin/stirrup reinforcing size :	4	bar
Rebar area :	0.4	in^2
A_{st} :	0.19	in^2
Total # of hairpins/stirrups required :	1	Quantity of hairpins or stirrups wrapp around anchor bolts





NEW FOOTINGS @ (E) BUILDING:

* FOOTING @ 1/C:

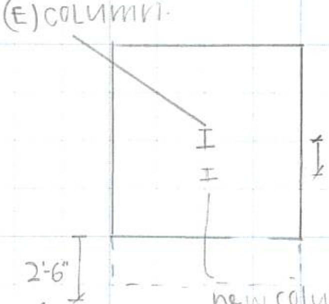
(E) FOOTING IS 8'-9" SQUARE, 12" THICK.

MAX REACTION = (15 PSF + 21 PSF)(16'-8"/2 + 24'-0"/2)(42'-8"/2) = 15.6 K

DL = 6.5 K

SL = 9.1 K

(E) COLUMN



– d = 4' + 12" + 4" = 20"

2'-6"

new column

2'-6" IS Adequate

FROM SPREADSHEET: UPLIFT = 2.6 K (ASD)

MAX DOWN : 7.7 K (ASD)

NEW REQ'D FTG UPLIFT SIZE : F4.5 *

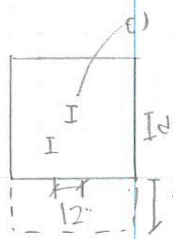
DOWN = F3

↳ see Todds printout.

* FOOTING @ 1/D & E:

(E) FOOTING IS 5'-6" SQUARE, 12" THICK

MAX (E) Reaction = (15 PSF + 21 PSF)(40'-2"/2)(42'-8"/2) = 15.5 K



d = 4" + 12" + 4" = 20"

12"

? → 3'-0" IS OK.

FROM SPDST new column UPLIFT = 2.4K → F4

DOWN = 9.4K → F3

↳ see Todds printout.

↳ TOP & BOTTOM REINFORCEMENT REQUIRED

FOOTING ANALYSIS

In accordance with ACI318-14

Tedds calculation version 3.3.08

Summary results

Overall design status PASS

Overall design utilisation 0.428

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	55.6			Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	0.684	1.6	0.428	Pass
Description	**Unit**	**Required**	**Provided**	**Utilization**	**Result**
Moment, positive, x-direction	kip_ft	30.2	146.8	0.206	Pass
Moment, positive, y-direction	kip_ft	32.1	129.1	0.248	Pass
Shear, one-way, x-direction	kips	12.5	87.3	0.144	Pass
Shear, one-way, y-direction	kips	12.3	67.9	0.182	Pass
Shear, two-way, Col 1	psi	38.894	150.000	0.259	Pass
Min.area of reinf, bot., x-direction	in^2	2.916	4.400		Pass
Max.reinf.spacing, bot, x-direction	in	18.0	14.2		Pass
Min.area of reinf, bot., y-direction	in^2	2.268	3.520		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	14.0		Pass

Pad footing details

Length of footing L_x = **8.75** ft

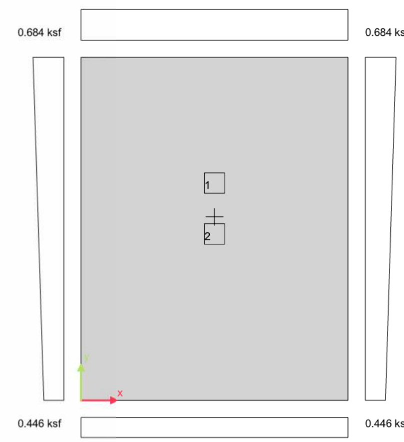
Width of footing L_y = **11.25** ft

Footing area $A = L_x \times L_y$ = **98.438** ft^2

Depth of footing h = **12** in

Depth of soil over footing h_{soil} = **18** in

Density of concrete γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column l_{x1} = **8.00** in

Width of column l_{y1} = **8.00** in

position in x-axis x_1 = **52.50** in

position in y-axis y_1 = **85.50** in

Column no.2 details

Length of column l_{x2} = **8.00** in

Width of column l_{y2} = **8.00** in

position in x-axis x_2 = **52.50** in

position in y-axis y_2 = **65.50** in

Soil properties

Net allowable bearing pressure q_{allow_Net} = **1.5** ksf using a soil factor of safety, FS_{soil}, of 3

Density of soil γ_{soil} = **120.0** lb/ft^3

Angle of internal friction ϕ_b = **30.0** deg

Design base friction angle δ_{bb} = **30.0** deg

Coefficient of base friction $\tan(\delta_{bb})$ = **0.577**

Footing loads

Self weight $F_{swt} = h \times \gamma_{conc}$ = **150** psf

Soil weight $F_{soil} = h_{soil} \times \gamma_{soil}$ = **180** psf

Column no.1 loads

Dead load in z F_{Dz1} = **15.6** kips

Column no.2 loads

Dead load in z F_{Dz2} = **7.7** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.428)

Combination 1 results: 1.0D

Forces on footing

Force in z-axis: $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_D \times (F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil})$ = **55.6** kips

Moments on footing

Moment in x-axis, about x is 0: $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_D \times (((F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil})) \times x_2)$ = **243.4** kip_ft

Moment in y-axis, about y is 0: $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_D \times (((F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil})) \times y_2)$ = **334.9** kip_ft

Uplift verification

Vertical force: F_{dz} = **55.624** kips

PASS - Footing is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis: $e_{dx} = M_{dx} / F_{dz} - L_x / 2$ = **0** in

Eccentricity of base reaction in y-axis: $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **4.748** in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **0.446** ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **0.684** ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **0.446** ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **0.684** ksf

Minimum base pressure: $q_{min} = \min(q_1,q_2,q_3,q_4)$ = **0.446** ksf

Maximum base pressure: $q_{max} = \max(q_1,q_2,q_3,q_4)$ = **0.684** ksf

Allowable bearing capacity

Allowable bearing capacity: $q_{allow} = q_{allow\_Net} + ((h + h_{soil}) \times \gamma_{soil}) / FS_{soil}$ = **1.6** ksf

q_{max} / q_{allow} = **0.428**

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN

In accordance with ACI318-14

Tedds calculation version 3.3.08

Material details

Compressive strength of concrete	f'_c = **2500** psi
Yield strength of reinforcement	f_y = **60000** psi
Compression-controlled strain limit (21.2.2)	ε_{ty} = **0.00200**
Cover to top of footing	c_{nom_t} = **3** in
Cover to side of footing	c_{nom_s} = **3** in
Cover to bottom of footing	c_{nom_b} = **3** in
Concrete type	Normal weight

Concrete modification factor	λ = **1.00**
Column type	Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-16

1.4D (0.259)

Combination 1 results: 1.4D

Forces on footing

Ultimate force in z-axis: $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_D \times (F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil})$ = **77.9** kips

Moments on footing

Ultimate moment in x-axis, about x is 0: $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_D \times (((F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil})) \times x_2)$ = **340.7** kip_ft

Ultimate moment in y-axis, about y is 0: $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_D \times (((F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil})) \times y_2)$ = **468.9** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis: $e_{ux} = M_{ux} / F_{uz} - L_x / 2$ = **0** in

Eccentricity of base reaction in y-axis: $e_{uy} = M_{uy} / F_{uz} - L_y / 2$ = **4.748** in

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **0.624** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **0.958** ksf

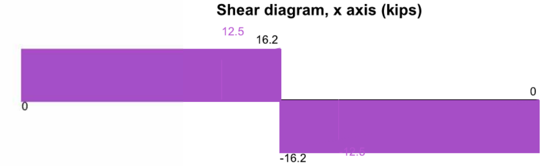
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **0.624** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **0.958** ksf

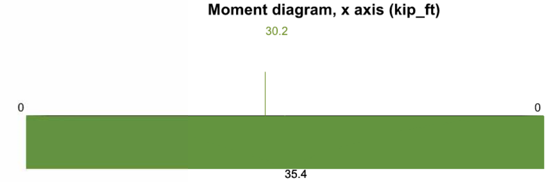
Minimum ultimate base pressure: $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4})$ = **0.624** ksf

Maximum ultimate base pressure: $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4})$ = **0.958** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment	$M_{u.x.max}$ = **30.244** kip_ft
Tension reinforcement provided	10 No.6 bottom bars (14.2 in c/c)
Area of tension reinforcement provided	$A_{sx.bot.prov}$ = **4.4** in^2
Minimum area of reinforcement (8.6.1.1)	$A_{s.min} = 0.0018 \times L_y \times h$ = **2.916** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2)	$s_{max} = \min(2 \times h, 18\text{ in})$ = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement	$d = h - c_{nom\_b} - \phi_{y.bot} - \phi_{x.bot} / 2$ = **7.875** in
Depth of compression block	$a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y)$ = **0.920** in
Neutral axis factor	β_1 = **0.85**
Depth to neutral axis	$c = a / \beta_1$ = **1.083** in
Strain in tensile reinforcement	$\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.01882**
Minimum tensile strain(8.3.3.1)	$\varepsilon_{min} = 0.004$ = **0.00400**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity	$M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2)$ = **163.127** kip_ft
Flexural strength reduction factor	$\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.005 - \varepsilon_{ty}), 0.65), 0.9)$ = **0.900**
Design moment capacity	$\phi M_n = \phi_f \times M_n$ = **146.814** kip_ft
	$M_{u.x.max} / \phi M_n$ = **0.206**

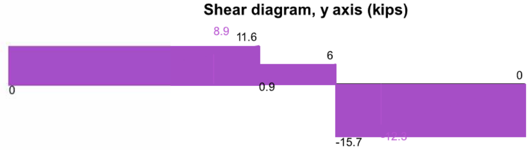
PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

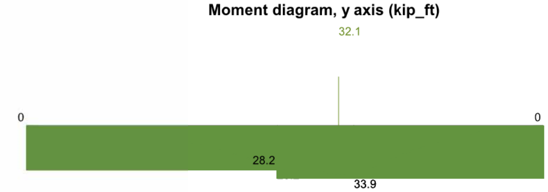
Ultimate shear force	$V_{u.x}$ = **12.535** kips
Depth to reinforcement	$d_v = h - c_{nom\_b} - \phi_{y.bot} - \phi_{x.bot} / 2$ = **7.875** in
Shear strength reduction factor	ϕ_v = **0.75**
Nominal shear capacity (Eq. 22.5.5.1)	$V_n = 2 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} \times L_y \times d_v$ = **106.313** kips
Design shear capacity	$\phi V_n = \phi_v \times V_n$ = **79.734** kips
	$V_{u.x} / \phi V_n$ = **0.157**

PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment	$M_{u.y.max}$ = **32.06** kip_ft
Tension reinforcement provided	8 No.6 bottom bars (14.0 in c/c)
Area of tension reinforcement provided	$A_{sy.bot.prov}$ = **3.52** in^2
Minimum area of reinforcement (8.6.1.1)	$A_{s.min} = 0.0018 \times L_x \times h$ = **2.268** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2)	$s_{max} = \min(2 \times h, 18\text{ in})$ = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement	$d = h - c_{nom\_b} - \phi_{y.bot} / 2$ = **8.625** in
Depth of compression block	$a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.947** in
Neutral axis factor	β_1 = **0.85**
Depth to neutral axis	$c = a / \beta_1$ = **1.114** in
Strain in tensile reinforcement	$\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.02024**
Minimum tensile strain(8.3.3.1)	$\varepsilon_{min} = 0.004$ = **0.00400**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity	$M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2)$ = **143.47** kip_ft
Flexural strength reduction factor	$\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.005 - \varepsilon_{ty}), 0.65), 0.9)$ = **0.900**
Design moment capacity	$\phi M_n = \phi_f \times M_n$ = **129.123** kip_ft
	$M_{u.y.max} / \phi M_n$ = **0.248**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force	$V_{u.y}$ = **12.333** kips
Depth to reinforcement	$d_v = h - c_{nom\_b} - \phi_{y.bot} / 2$ = **8.625** in
Shear strength reduction factor	ϕ_v = **0.75**
Nominal shear capacity (Eq. 22.5.5.1)	$V_n = 2 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} \times L_x \times d_v$ = **90.563** kips
Design shear capacity	$\phi V_n = \phi_v \times V_n$ = **67.922** kips
	$V_{u.y} / \phi V_n$ = **0.182**

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement	d_{v2} = **8.25** in
Shear perimeter length (22.6.4)	l_{xp} = **16.250** in
Shear perimeter width (22.6.4)	l_{yp} = **16.250** in
Shear perimeter (22.6.4)	$b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2})$ = **65.000** in

Shear area	$A_p = l_{x,perim} \times l_{y,perim}$ = **264.062** in^2
Surcharge loaded area	$A_{sur} = A_p - l_{x1} \times l_{y1}$ = **200.062** in^2
Ultimate bearing pressure at center of shear area	$q_{up.avg}$ = **0.876** ksf
Ultimate shear load	$F_{up} = \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p$ = **20.857** kips
Ultimate shear stress from vertical load	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}),0\text{ psi})$ = **38.894** psi
Column geometry factor (Table 22.6.5.2)	$\beta = l_{y1} / l_{x1}$ = **1.00**
Column location factor (22.6.5.3)	α_s =**40**
Concrete shear strength (22.6.5.2)	$v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})}$ = **300.000** psi
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})}$ = **353.846** psi
	$v_{cpc} = 4 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})}$ = **200.000** psi
	$v_{cp} = \min(v_{cpa},v_{cpb},v_{cpc})$ = **200.000** psi
Shear strength reduction factor	ϕ_v = **0.75**
Nominal shear stress capacity (Eq. 22.6.1.2)	$v_n = v_{cp}$ = **200.000** psi
Design shear stress capacity (8.5.1.1(d))	$\phi v_n = \phi_v \times v_n$ = **150.000** psi
	$v_{ug} / \phi v_n$ = **0.259**

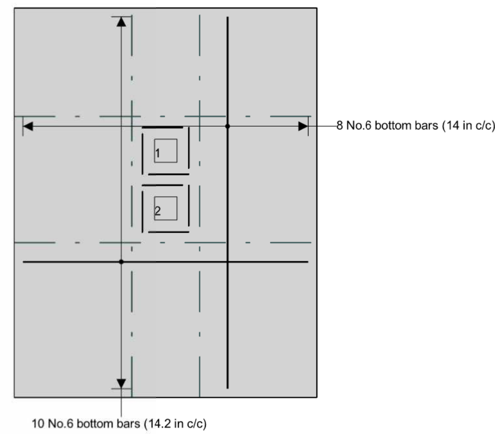
PASS - Design shear stress capacity exceeds ultimate shear stress load

Two-way shear design at column 2

Depth to reinforcement	d_{v2} = **8.25** in
Shear perimeter length (22.6.4)	l_{xp} = **16.250** in
Shear perimeter width (22.6.4)	l_{yp} = **16.250** in
Shear perimeter (22.6.4)	$b_o = 2 \times (l_{x2} + d_{v2}) + 2 \times (l_{y2} + d_{v2})$ = **65.000** in
Shear area	$A_p = l_{x,perim} \times l_{y,perim}$ = **264.062** in^2
Surcharge loaded area	$A_{sur} = A_p - l_{x2} \times l_{y2}$ = **200.062** in^2
Ultimate bearing pressure at center of shear area	$q_{up.avg}$ = **0.826** ksf
Ultimate shear load	$F_{up} = \gamma_D \times (F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil}) + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p$ = **9.888** kips
Ultimate shear stress from vertical load	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}),0\text{ psi})$ = **18.439** psi
Column geometry factor (Table 22.6.5.2)	$\beta = l_{y2} / l_{x2}$ = **1.00**
Column location factor (22.6.5.3)	α_s =**40**
Concrete shear strength (22.6.5.2)	$v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})}$ = **300.000** psi
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})}$ = **353.846** psi
	$v_{cpc} = 4 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})}$ = **200.000** psi
	$v_{cp} = \min(v_{cpa},v_{cpb},v_{cpc})$ = **200.000** psi
Shear strength reduction factor	ϕ_v = **0.75**
Nominal shear stress capacity (Eq. 22.6.1.2)	$v_n = v_{cp}$ = **200.000** psi
Design shear stress capacity (8.5.1.1(d))	$\phi v_n = \phi_v \times v_n$ = **150.000** psi
	$v_{ug} / \phi v_n$ = **0.123**

PASS - Design shear stress capacity exceeds ultimate shear stress load

Note: Two-way shear is not calculated for columns that may share a shear perimeter with another column. This may need to be calculated conditionally by the engineer.



FOOTING ANALYSIS

In accordance with ACI318-14

Tedds calculation version 3.3.08

Summary results

Overall design status PASS

Overall design utilisation 0.894

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	39.2			Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	1.431	1.6	0.894	Pass
Description	**Unit**	**Required**	**Provided**	**Utilization**	**Result**
Moment, positive, x-direction	kip_ft	19.0	112.8	0.168	Pass
Moment, positive, y-direction	kip_ft	11.0	87.9	0.125	Pass
Moment, negative, y-direction	kip_ft	0.1	96.8	0.001	Pass
Shear, one-way, x-direction	kips	9.8	53.2	0.184	Pass
Shear, one-way, y-direction	kips	6.7	50.5	0.133	Pass
Shear, two-way, Col 1	psi	35.622	150.000	0.237	Pass
Min.area of reinf, bot., x-direction	in²	1.944	3.080		Pass
Max.reinf.spacing, bot, x-direction	in	18.0	13.8		Pass
Min.area of reinf, bot., y-direction	in²	1.685	2.640		Pass
Min.area of reinf, top, y-direction	in²	1.685	2.640		Pass
Max.reinf.spacing, top, y-direction	in	18.0	14.2		Pass

Pad footing details

Length of footing L_x = **6.5** ft

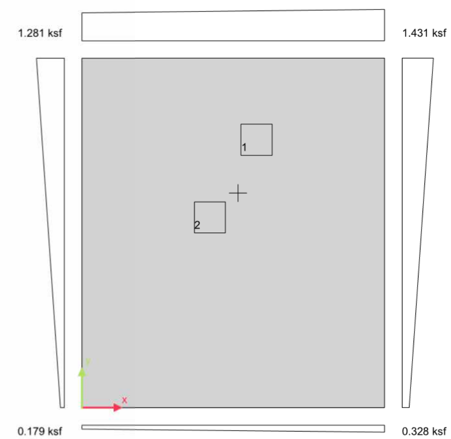
Width of footing L_y = **7.5** ft

Footing area $A = L_x \times L_y$ = **48.75** ft^2

Depth of footing h = **12** in

Depth of soil over footing h_{soil} = **18** in

Density of concrete γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column l_{x1} = **8.00** in

Width of column l_{y1} = **8.00** in

position in x-axis x_1 = **45.00** in

position in y-axis y_1 = **69.00** in

Column no.2 details

Length of column l_{x2} = **8.00** in

Width of column l_{y2} = **8.00** in

position in x-axis x_2 = **33.00** in

position in y-axis y_2 = **49.00** in

Soil properties

Net allowable bearing pressure q_{allow_Net} = **1.5** ksf using a soil factor of safety, FS_{soil}, of 3

Density of soil γ_{soil} = **120.0** lb/ft^3

Angle of internal friction ϕ_b = **30.0** deg

Design base friction angle δ_{bb} = **30.0** deg

Coefficient of base friction $\tan(\delta_{bb})$ = **0.577**

Footing loads

Self weight $F_{swt} = h \times \gamma_{conc}$ = **150** psf

Soil weight $F_{soil} = h_{soil} \times \gamma_{soil}$ = **180** psf

Column no.1 loads

Dead load in z F_{Dz1} = **15.6** kips

Column no.2 loads

Dead load in z F_{Dz2} = **7.7** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.894)

Combination 1 results: 1.0D

Forces on footing

Force in z-axis — $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_D \times (F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil})$ = **39.2** kips

Moments on footing

Moment in x-axis, about x is 0 — $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_D \times (((F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil})) \times x_2)$ = **131.4** kip_ft

Moment in y-axis, about y is 0 — $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_D \times (((F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil})) \times y_2)$ = **180.7** kip_ft

Uplift verification

Vertical force — F_{dz} = **39.228** kips

PASS - Footing is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis — $e_{dx} = M_{dx} / F_{dz} - L_x / 2$ = **1.208** in

Eccentricity of base reaction in y-axis — $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **10.272** in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **0.179** ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.281** ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **0.328** ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.431** ksf

Minimum base pressure — $q_{min} = \min(q_1,q_2,q_3,q_4)$ = **0.179** ksf

Maximum base pressure — $q_{max} = \max(q_1,q_2,q_3,q_4)$ = **1.431** ksf

Allowable bearing capacity

Allowable bearing capacity — $q_{allow} = q_{allow\_Net} + ((h + h_{soil}) \times \gamma_{soil}) / FS_{soil}$ = **1.6** ksf

q_{max} / q_{allow} = **0.894**

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN

In accordance with ACI318-14

Tedds calculation version 3.3.08

Material details

Compressive strength of concrete	f'_c = **2500** psi
Yield strength of reinforcement	f_y = **60000** psi
Compression-controlled strain limit (21.2.2)	ε_{ty} = **0.00200**
Cover to top of footing	c_{nom_t} = **3** in
Cover to side of footing	c_{nom_s} = **3** in
Cover to bottom of footing	c_{nom_b} = **3** in
Concrete type	Normal weight

Concrete modification factor	λ = **1.00**
Column type	Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-16

1.4D (0.237)

Combination 1 results: 1.4D

Forces on footing

Ultimate force in z-axis — $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_D \times (F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil})$ = **54.9** kips

Moments on footing

Ultimate moment in x-axis, about x is 0 — $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_D \times (((F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil})) \times x_2)$ = **184.0** kip_ft

Ultimate moment in y-axis, about y is 0 — $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_D \times (((F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil})) \times y_2)$ = **253.0** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis — $e_{ux} = M_{ux} / F_{uz} - L_x / 2$ = **1.208** in

Eccentricity of base reaction in y-axis — $e_{uy} = M_{uy} / F_{uz} - L_y / 2$ = **10.272** in

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **0.25** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **1.793** ksf

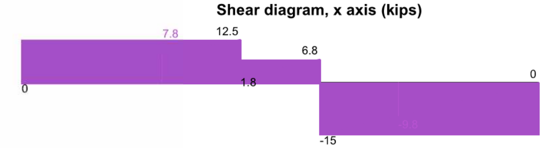
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **0.46** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **2.003** ksf

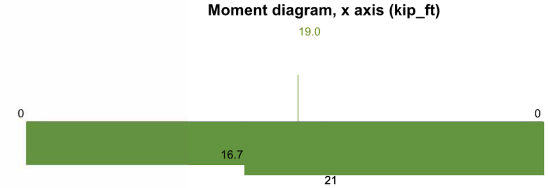
Minimum ultimate base pressure — $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4})$ = **0.25** ksf

Maximum ultimate base pressure — $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4})$ = **2.003** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment: $M_{u.x.max}$ = **19.003** kip_ft

Tension reinforcement provided: 7 No.6 bottom bars (13.8 in c/c)

Area of tension reinforcement provided: $A_{sx.bot.prov}$ = **3.08** in^2

Minimum area of reinforcement (8.6.1.1): $A_{s.min} = 0.0018 \times L_y \times h$ = **1.944** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2): $s_{max} = \min(2 \times h, 18 \text{ in})$ = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement: $d = h - c_{nom\_b} - \phi_{x.bot} / 2$ = **8.625** in

Depth of compression block: $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y)$ = **0.966** in

Neutral axis factor: β_1 = **0.85**

Depth to neutral axis: $c = a / \beta_1$ = **1.137** in

Strain in tensile reinforcement: $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.01976**

Minimum tensile strain(8.3.3.1): ε_{min} = 0.004 = **0.00400**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity: $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2)$ = **125.385** kip_ft

Flexural strength reduction factor: $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.005 - \varepsilon_{ty}), 0.65), 0.9)$ = **0.900**

Design moment capacity: $\phi M_n = \phi_f \times M_n$ = **112.846** kip_ft

$M_{u.x.max} / \phi M_n$ = **0.168**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force: $V_{u.x}$ = **9.782** kips

Depth to reinforcement: $d_v = h - c_{nom\_b} - \phi_{y.bot} - \phi_{x.bot} / 2$ = **7.875** in

Shear strength reduction factor: ϕ_v = **0.75**

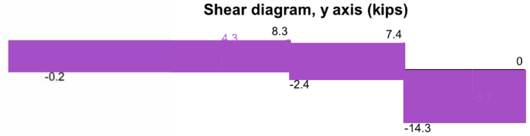
Nominal shear capacity (Eq. 22.5.5.1): $V_n = 2 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_y \times d_v$ = **70.875** kips

Design shear capacity: $\phi V_n = \phi_v \times V_n$ = **53.156** kips

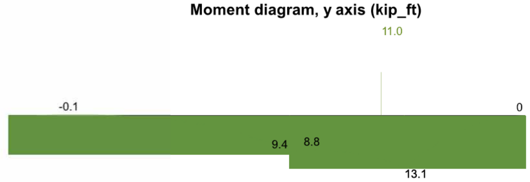
$V_{u.x} / \phi V_n$ = **0.184**

PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment: $M_{u.y.max}$ = **11** kip_ft

Tension reinforcement provided: 6 No.6 bottom bars (14.2 in c/c)

Area of tension reinforcement provided: $A_{sy.bot.prov}$ = **2.64** in^2

Minimum area of reinforcement (8.6.1.1): $A_{s.min} = 0.0018 \times L_x \times h$ = **1.685** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2): $s_{max} = \min(2 \times h, 18 \text{ in})$ = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement: $d = h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **7.875** in

Depth of compression block: $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.956** in

Neutral axis factor: β_1 = **0.85**

Depth to neutral axis: $c = a / \beta_1$ = **1.124** in

Strain in tensile reinforcement: $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.01801**

Minimum tensile strain(8.3.3.1): ε_{min} = 0.004 = **0.00400**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity: $M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2)$ = **97.643** kip_ft

Flexural strength reduction factor: $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.005 - \varepsilon_{ty}), 0.65), 0.9)$ = **0.900**

Design moment capacity: $\phi M_n = \phi_f \times M_n$ = **87.878** kip_ft

$M_{u.y.max} / \phi M_n$ = **0.125**

PASS - Design moment capacity exceeds ultimate moment load

Moment design, y direction, negative moment

Ultimate bending moment: $M_{u.y.min}$ = **-0.125** kip_ft

Tension reinforcement provided: 6 No.6 top bars (14.2 in c/c)

Area of tension reinforcement provided: $A_{sy.top.prov}$ = **2.64** in^2

Minimum area of reinforcement (8.6.1.1): $A_{s.min} = 0.0018 \times L_x \times h$ = **1.685** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2): $s_{max} = \min(2 \times h, 18 \text{ in})$ = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement: $d = h - c_{nom\_t} - \phi_{y.top} / 2$ = **8.625** in

Depth of compression block: $a = A_{sy.top.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.956** in

Neutral axis factor: β_1 = **0.85**

Depth to neutral axis: $c = a / \beta_1$ = **1.124** in

Strain in tensile reinforcement: $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.02001**

Minimum tensile strain(8.3.3.1): ε_{min} = 0.004 = **0.00400**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity: $M_n = A_{sy.top.prov} \times f_y \times (d - a / 2)$ = **107.543** kip_ft

Flexural strength reduction factor: $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.005 - \varepsilon_{ty}), 0.65), 0.9)$ = **0.900**

Design moment capacity: $\phi M_n = \phi_f \times M_n$ = **96.788** kip_ft

$\text{abs}(M_{u.y.min}) / \phi M_n$ = **0.001**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force: $V_{u.y}$ = **6.711** kips

Tekla Tedds	Project Kimberly AG Shop				Job Ref. 24091	
	Section Footing @ 1/D&E				Sheet no./rev. 7	
	Calc. by ANB	Date 10/7/2024	Chk'd by	Date	App'd by	Date

Depth to reinforcement	$d_v = \min(h - c_{nom\_b} - \phi_{y.bot} / 2, h - c_{nom\_t} - \phi_{y.top} / 2) = \mathbf{8.625}$ in
Shear strength reduction factor	$\phi_v = \mathbf{0.75}$
Nominal shear capacity (Eq. 22.5.5.1)	$V_n = 2 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_x \times d_v = \mathbf{67.275}$ kips
Design shear capacity	$\phi V_n = \phi_v \times V_n = \mathbf{50.456}$ kips
	$V_{u.y} / \phi V_n = \mathbf{0.133}$

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement	$d_{v2} = \mathbf{8.25}$ in
Shear perimeter length (22.6.4)	$l_{xp} = \mathbf{16.250}$ in
Shear perimeter width (22.6.4)	$l_{yp} = \mathbf{16.250}$ in
Shear perimeter (22.6.4)	$b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2}) = \mathbf{65.000}$ in
Shear area	$A_p = l_{x,perim} \times l_{y,perim} = \mathbf{264.062}$ in^2
Surcharge loaded area	$A_{sur} = A_p - l_{x1} \times l_{y1} = \mathbf{200.062}$ in^2
Ultimate bearing pressure at center of shear area	$q_{up.avg} = \mathbf{1.833}$ ksf
Ultimate shear load	$F_{up} = \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p = \mathbf{19.102}$ kips
Ultimate shear stress from vertical load	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0 \text{ psi}) = \mathbf{35.622}$ psi
Column geometry factor (Table 22.6.5.2)	$\beta = l_{y1} / l_{x1} = \mathbf{1.00}$
Column location factor (22.6.5.3)	$\alpha_s = \mathbf{40}$
Concrete shear strength (22.6.5.2)	$v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} = \mathbf{300.000}$ psi
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} = \mathbf{353.846}$ psi
	$v_{cpc} = 4 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} = \mathbf{200.000}$ psi
	$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) = \mathbf{200.000}$ psi
Shear strength reduction factor	$\phi_v = \mathbf{0.75}$
Nominal shear stress capacity (Eq. 22.6.1.2)	$v_n = v_{cp} = \mathbf{200.000}$ psi
Design shear stress capacity (8.5.1.1(d))	$\phi v_n = \phi_v \times v_n = \mathbf{150.000}$ psi
	$v_{ug} / \phi v_n = \mathbf{0.237}$

PASS - Design shear stress capacity exceeds ultimate shear stress load

Two-way shear design at column 2

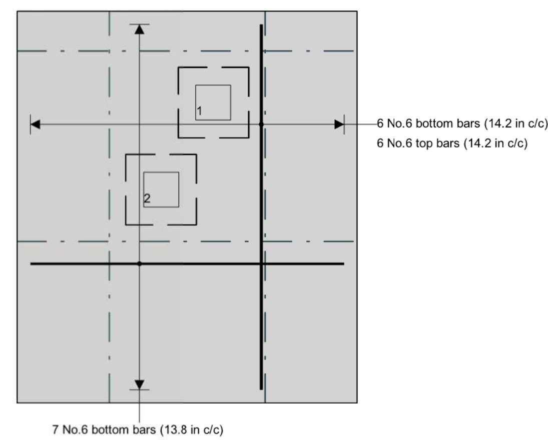
Depth to reinforcement	$d_{v2} = \mathbf{8.25}$ in
Shear perimeter length (22.6.4)	$l_{xp} = \mathbf{16.250}$ in
Shear perimeter width (22.6.4)	$l_{yp} = \mathbf{16.250}$ in
Shear perimeter (22.6.4)	$b_o = 2 \times (l_{x2} + d_{v2}) + 2 \times (l_{y2} + d_{v2}) = \mathbf{65.000}$ in
Shear area	$A_p = l_{x,perim} \times l_{y,perim} = \mathbf{264.063}$ in^2
Surcharge loaded area	$A_{sur} = A_p - l_{x2} \times l_{y2} = \mathbf{200.063}$ in^2
Ultimate bearing pressure at center of shear area	$q_{up.avg} = \mathbf{1.458}$ ksf
Ultimate shear load	$F_{up} = \gamma_D \times (F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil}) + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p = \mathbf{8.730}$ kips
Ultimate shear stress from vertical load	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0 \text{ psi}) = \mathbf{16.280}$ psi
Column geometry factor (Table 22.6.5.2)	$\beta = l_{y2} / l_{x2} = \mathbf{1.00}$
Column location factor (22.6.5.3)	$\alpha_s = \mathbf{40}$
Concrete shear strength (22.6.5.2)	$v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} = \mathbf{300.000}$ psi

Tekla Tedds	Project Kimberly AG Shop				Job Ref. 24091	
	Section Footing @ 1/D&E				Sheet no./rev. 8	
	Calc. by ANB	Date 10/7/2024	Chk'd by	Date	App'd by	Date

	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} = \mathbf{353.846}$ psi
	$v_{cpc} = 4 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} = \mathbf{200.000}$ psi
	$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) = \mathbf{200.000}$ psi
Shear strength reduction factor	$\phi_v = \mathbf{0.75}$
Nominal shear stress capacity (Eq. 22.6.1.2)	$v_n = v_{cp} = \mathbf{200.000}$ psi
Design shear stress capacity (8.5.1.1(d))	$\phi v_n = \phi_v \times v_n = \mathbf{150.000}$ psi
	$v_{ug} / \phi v_n = \mathbf{0.109}$

PASS - Design shear stress capacity exceeds ultimate shear stress load

Note: Two-way shear is not calculated for columns that may share a shear perimeter with another column. This may need to be calculated conditionally by the engineer.



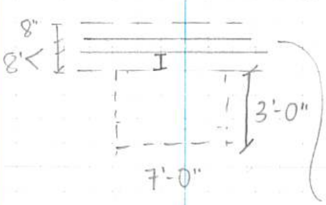
ARW ENGINEERS
structural consultants

Project No. 24091 | Sheet No. 54
Project Kimberly AG Shop
Prepared By ANB | Date 10/1/24

NEW FOOTINGS @ (E) BUILDING (CONT'D):

* FOOTING @ 1/F & 1/G:

(E) FOOTING IS 2'-0" STRIP FTG, 12" THICK



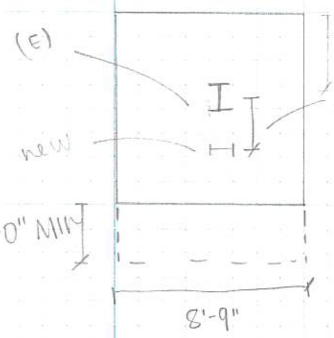
↳ NEW COLUMN REACTION: UPLIFT = 4.7K F5.5
DOWN: 10.4K F3

LINE LOAD: DL = 55 PSF (10') = 550 PLF

↳ NEW 3'-0" x 7'-0" FTG REQ'D.
see Tedds printout.

* FOOTING @ 1/H:

(E) FOOTING IS 8'-9" FTG, 12" THICK

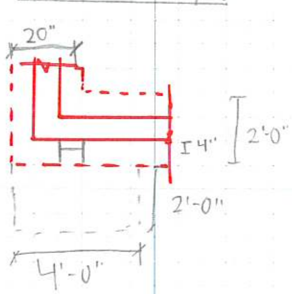


↳ (E) column reaction:
Max = (15 + 21 PSF)(42'-8"/2)(24'-0"/2) = 9.3K

New column reaction: UPLIFT: 3.5K (F5)
DOWN: 5.2K (F3)

↳ 3'-0" add'l
FTG OK - see Tedds printout.

* FOOTING @ 1/A:



(E) STRIP FTG REACTION = 550 PLF

NEW COLUMN REACTION: UPLIFT: 0.8K (F3)
DOWN: 2.8K (F3)

↳ 2'-0" x 4'-0" new FTG OK

see Tedds printout.

FOOTING ANALYSIS

In accordance with ACI318-14

Tedds calculation version 3.3.08

Summary results

Overall design status PASS

Overall design utilisation 0.728

Description	**Unit**	**Applied**	**Resisting**	**FoS**	**Result**
Uplift verification	kips	24.9			Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	1.165	1.6	0.728	Pass
Description	**Unit**	**Required**	**Provided**	**Utilization**	**Result**
Moment, positive, x-direction	kip_ft	1.8	53.7	0.034	Pass
Moment, negative, x-direction	kip_ft	0.4	58.1	0.007	Pass
Moment, positive, y-direction	kip_ft	5.1	81.3	0.063	Pass
Shear, one-way, x-direction	kips	1.3	39.1	0.034	Pass
Shear, one-way, y-direction	kips	3.2	50.8	0.063	Pass
Shear, two-way, Col 1	psi	22.428	150.000	0.150	Pass
Min.area of reinf, bot., x-direction	in^2	1.296	1.550		Pass
Min.area of reinf, top, x-direction	in^2	1.296	1.550		Pass
Max.reinf.spacing, top, x-direction	in	18.0	13.3		Pass
Min.area of reinf, bot., y-direction	in^2	1.814	2.170		Pass
Max.reinf.spacing, top, y-direction	in	18.0	12.8		Pass

Pad footing details

Length of footing L_x = **7** ft

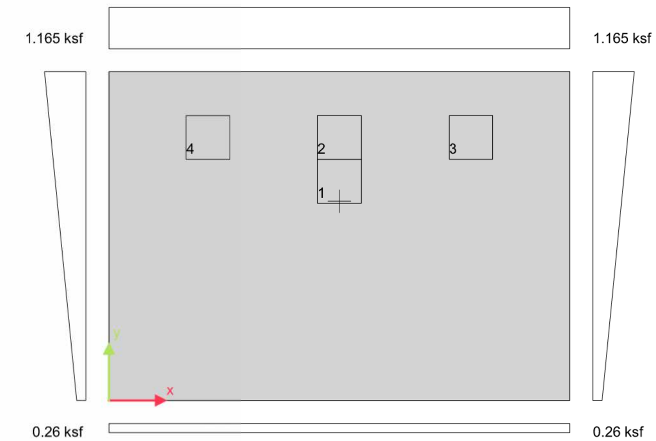
Width of footing L_y = **5** ft

Footing area $A = L_x \times L_y$ = **35** ft^2

Depth of footing h = **12** in

Depth of soil over footing h_{soil} = **18** in

Density of concrete γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column l_{x1} = **8.00** in

Width of column l_{y1} = **8.00** in

position in x-axis x_1 = **42.00** in

position in y-axis y_1 = **40.00** in

Column no.2 details

Length of column l_{x2} = **8.00** in

Width of column l_{y2} = **8.00** in

position in x-axis x_2 = **42.00** in

position in y-axis y_2 = **48.00** in

Column no.3 details

Length of column l_{x3} = **8.00** in

Width of column l_{y3} = **8.00** in

position in x-axis x_3 = **66.00** in

position in y-axis y_3 = **48.00** in

Column no.4 details

Length of column l_{x4} = **8.00** in

Width of column l_{y4} = **8.00** in

position in x-axis x_4 = **18.00** in

position in y-axis y_4 = **48.00** in

Soil properties

Net allowable bearing pressure q_{allow_Net} = **1.5** ksf using a soil factor of safety, FS_{soil}, of 3

Density of soil γ_{soil} = **120.0** lb/ft^3

Angle of internal friction	ϕ_b = **30.0** deg
Design base friction angle	δ_{bb} = **30.0** deg
Coefficient of base friction	$\tan(\delta_{bb})$ = **0.577**

Footing loads

Self weight	$F_{swt} = h \times \gamma_{conc}$ = **150** psf
Soil weight	$F_{soil} = h_{soil} \times \gamma_{soil}$ = **180** psf

Column no.1 loads

Dead load in z	F_{Dz1} = **10.4** kips

Column no.2 loads

Dead load in z	F_{Dz2} = **1.1** kips

Column no.3 loads

Dead load in z	F_{Dz3} = **1.1** kips

Column no.4 loads

Dead load in z	F_{Dz4} = **1.1** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.728)

Combination 1 results: 1.0D

Forces on footing

Force in z-axis

$F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_D \times (F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil}) + \gamma_D \times (F_{Dz3} - l_{x3} \times l_{y3} \times h_{soil} \times \gamma_{soil}) + \gamma_D \times (F_{Dz4} - l_{x4} \times l_{y4} \times h_{soil} \times \gamma_{soil})$ = **24.9** kips

Moments on footing

Moment in x-axis, about x is 0

$M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_D \times (((F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil})) \times x_2) + \gamma_D \times (((F_{Dz3} - l_{x3} \times l_{y3} \times h_{soil} \times \gamma_{soil})) \times x_3) + \gamma_D \times (((F_{Dz4} - l_{x4} \times l_{y4} \times h_{soil} \times \gamma_{soil})) \times x_4)$ = **87.3** kip_ft

Moment in y-axis, about y is 0

$M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_D \times (((F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil})) \times y_2) + \gamma_D \times (((F_{Dz3} - l_{x3} \times l_{y3} \times h_{soil} \times \gamma_{soil})) \times y_3) + \gamma_D \times (((F_{Dz4} - l_{x4} \times l_{y4} \times h_{soil} \times \gamma_{soil})) \times y_4)$ = **75.5** kip_ft

Uplift verification

Vertical force	F_{dz} = **24.93** kips

PASS - Footing is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis	$e_{dx} = M_{dx} / F_{dz} - L_x / 2$ = **0** in
Eccentricity of base reaction in y-axis	$e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **6.349** in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **0.26** ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.165** ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **0.26** ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.165** ksf

Minimum base pressure	$q_{min} = \min(q_1,q_2,q_3,q_4)$ = **0.26** ksf
Maximum base pressure	$q_{max} = \max(q_1,q_2,q_3,q_4)$ = **1.165** ksf

Allowable bearing capacity

Allowable bearing capacity	$q_{allow} = q_{allow\_Net} + ((h + h_{soil}) \times \gamma_{soil}) / FS_{soil}$ = **1.6** ksf
	q_{max} / q_{allow} = **0.728**

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN

In accordance with ACI318-14

Tedds calculation version 3.3.08

Material details

Compressive strength of concrete	f'_c = **2500** psi
Yield strength of reinforcement	f_y = **60000** psi
Compression-controlled strain limit (21.2.2)	ε_{ty} = **0.00200**
Cover to top of footing	c_{nom_t} = **3** in
Cover to side of footing	c_{nom_s} = **3** in
Cover to bottom of footing	c_{nom_b} = **3** in
Concrete type	Normal weight
Concrete modification factor	λ = **1.00**
Column type	Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-16

1.4D (0.150)

Combination 1 results: 1.4D

Forces on footing

Ultimate force in z-axis

$F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_D \times (F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil}) + \gamma_D \times (F_{Dz3} - l_{x3} \times l_{y3} \times h_{soil} \times \gamma_{soil}) + \gamma_D \times (F_{Dz4} - l_{x4} \times l_{y4} \times h_{soil} \times \gamma_{soil})$ = **34.9** kips

Moments on footing

Ultimate moment in x-axis, about x is 0

$M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_D \times (((F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil})) \times x_2) + \gamma_D \times (((F_{Dz3} - l_{x3} \times l_{y3} \times h_{soil} \times \gamma_{soil})) \times x_3) + \gamma_D \times (((F_{Dz4} - l_{x4} \times l_{y4} \times h_{soil} \times \gamma_{soil})) \times x_4)$ = **122.2** kip_ft

Ultimate moment in y-axis, about y is 0

$M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_D \times (((F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil})) \times y_2) + \gamma_D \times (((F_{Dz3} - l_{x3} \times l_{y3} \times h_{soil} \times \gamma_{soil})) \times y_3) + \gamma_D \times (((F_{Dz4} - l_{x4} \times l_{y4} \times h_{soil} \times \gamma_{soil})) \times y_4)$ = **105.7** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis	$e_{ux} = M_{ux} / F_{uz} - L_x / 2$ = **0** in
Eccentricity of base reaction in y-axis	$e_{uy} = M_{uy} / F_{uz} - L_y / 2$ = **6.349** in

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **0.364** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **1.63** ksf

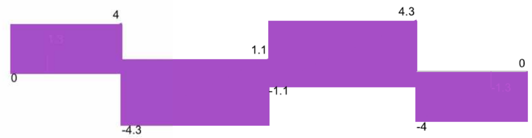
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **0.364** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **1.63** ksf

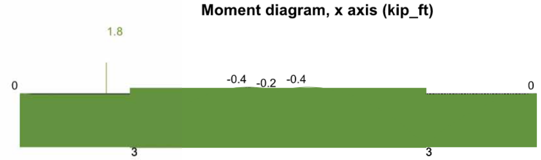
Minimum ultimate base pressure — $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4}) =$ **0.364** ksf

Maximum ultimate base pressure — $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4}) =$ **1.63** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment — $M_{u.x.max} =$ **1.822** kip_ft

Tension reinforcement provided — 5 No.5 bottom bars (13.3 in c/c)

Area of tension reinforcement provided — $A_{sx.bot.prov} =$ **1.55** in^2

Minimum area of reinforcement (8.6.1.1) — $A_{s.min} = 0.0018 \times L_y \times h =$ **1.296** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2) — $s_{max} = \min(2 \times h, 18\text{ in}) =$ **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement — $d = h - c_{nom\_b} - \phi_{y.bot} - \phi_{x.bot} / 2 =$ **8.062** in

Depth of compression block — $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y) =$ **0.729** in

Neutral axis factor — $\beta_1 =$ **0.85**

Depth to neutral axis — $c = a / \beta_1 =$ **0.858** in

Strain in tensile reinforcement — $\varepsilon_t = 0.003 \times d / c - 0.003 =$ **0.02519**

Minimum tensile strain(8.3.3.1) — $\varepsilon_{min} = 0.004 =$ **0.00400**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity — $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2) =$ **59.658** kip_ft

Flexural strength reduction factor — $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.005 - \varepsilon_{ty}), 0.65), 0.9) =$ **0.900**

Design moment capacity — $\phi M_n = \phi_f \times M_n =$ **53.692** kip_ft

$M_{u.x.max} / \phi M_n =$ **0.034**

PASS - Design moment capacity exceeds ultimate moment load

Moment design, x direction, negative moment

Ultimate bending moment — $M_{u.x.min} =$ **-0.388** kip_ft

Tension reinforcement provided — 5 No.5 top bars (13.3 in c/c)

Area of tension reinforcement provided — $A_{sx.top.prov} =$ **1.55** in^2

Minimum area of reinforcement (8.6.1.1) — $A_{s.min} = 0.0018 \times L_y \times h =$ **1.296** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2) — $s_{max} = \min(2 \times h, 18\text{ in}) =$ **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement — $d = h - c_{nom\_t} - \phi_{x.top} / 2 =$ **8.688** in

Depth of compression block — $a = A_{sx.top.prov} \times f_y / (0.85 \times f'_c \times L_y) =$ **0.729** in

Neutral axis factor — $\beta_1 =$ **0.85**

Depth to neutral axis — $c = a / \beta_1 =$ **0.858** in

Strain in tensile reinforcement — $\varepsilon_t = 0.003 \times d / c - 0.003 =$ **0.02737**

Minimum tensile strain(8.3.3.1) — $\varepsilon_{min} = 0.004 =$ **0.00400**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity — $M_n = A_{sx.top.prov} \times f_y \times (d - a / 2) =$ **64.502** kip_ft

Flexural strength reduction factor — $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.005 - \varepsilon_{ty}), 0.65), 0.9) =$ **0.900**

Design moment capacity — $\phi M_n = \phi_f \times M_n =$ **58.051** kip_ft

$\text{abs}(M_{u.x.min}) / \phi M_n =$ **0.007**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force — $V_{u.x} =$ **1.325** kips

Depth to reinforcement — $d_v = \min(h - c_{nom\_b} - \phi_{x.bot} / 2, h - c_{nom\_t} - \phi_{x.top} / 2) =$ **8.688** in

Shear strength reduction factor — $\phi_v =$ **0.75**

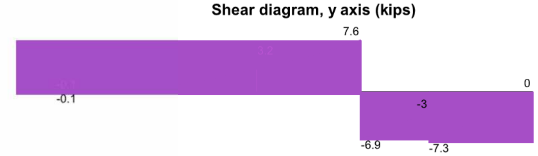
Nominal shear capacity (Eq. 22.5.5.1) — $V_n = 2 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} \times L_y \times d_v =$ **52.125** kips

Design shear capacity — $\phi V_n = \phi_v \times V_n =$ **39.094** kips

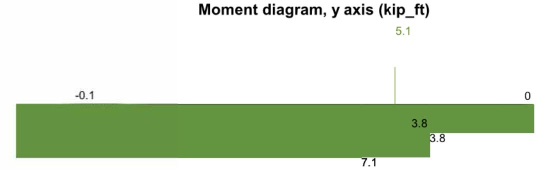
$V_{u.x} / \phi V_n =$ **0.034**

PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment — $M_{u.y.max} =$ **5.141** kip_ft

Tension reinforcement provided	7 No.5 bottom bars (12.8 in c/c)
Area of tension reinforcement provided	$A_{sy.bot.prov}$ = **2.17** in^2
Minimum area of reinforcement (8.6.1.1)	$A_{s.min} = 0.0018 \times L_x \times h$ = **1.814** in^2
	PASS - Area of reinforcement provided exceeds minimum
Maximum spacing of reinforcement (8.7.2.2)	$s_{max} = \min(2 \times h, 18\text{ in})$ = **18** in
	PASS - Maximum permissible reinforcement spacing exceeds actual spacing
Depth to tension reinforcement	$d = h - c_{nom\_b} - \phi_{y.bot} / 2$ = **8.688** in
Depth of compression block	$a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.729** in
Neutral axis factor	β_1 = **0.85**
Depth to neutral axis	$c = a / \beta_1$ = **0.858** in
Strain in tensile reinforcement	$\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.02737**
Minimum tensile strain(8.3.3.1)	$\varepsilon_{min} = 0.004$ = **0.00400**
	PASS - Tensile strain exceeds minimum required
Nominal moment capacity	$M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2)$ = **90.302** kip_ft
Flexural strength reduction factor	$\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.005 - \varepsilon_{ty}), 0.65), 0.9)$ = **0.900**
Design moment capacity	$\phi M_n = \phi_f \times M_n$ = **81.272** kip_ft
	$M_{u.y.max} / \phi M_n$ = **0.063**
	PASS - Design moment capacity exceeds ultimate moment load

Moment design, y direction, negative moment

Ultimate bending moment	$M_{u.y.min}$ = **-0.068** kip_ft
Tension reinforcement provided	7 No.5 top bars (12.8 in c/c)
Area of tension reinforcement provided	$A_{sy.top.prov}$ = **2.17** in^2
Minimum area of reinforcement (8.6.1.1)	$A_{s.min} = 0.0018 \times L_x \times h$ = **1.814** in^2
	PASS - Area of reinforcement provided exceeds minimum
Maximum spacing of reinforcement (8.7.2.2)	$s_{max} = \min(2 \times h, 18\text{ in})$ = **18** in
	PASS - Maximum permissible reinforcement spacing exceeds actual spacing
Depth to tension reinforcement	$d = h - c_{nom\_t} - \phi_{x.top} - \phi_{y.top} / 2$ = **8.062** in
Depth of compression block	$a = A_{sy.top.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.729** in
Neutral axis factor	β_1 = **0.85**
Depth to neutral axis	$c = a / \beta_1$ = **0.858** in
Strain in tensile reinforcement	$\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.02519**
Minimum tensile strain(8.3.3.1)	$\varepsilon_{min} = 0.004$ = **0.00400**
	PASS - Tensile strain exceeds minimum required
Nominal moment capacity	$M_n = A_{sy.top.prov} \times f_y \times (d - a / 2)$ = **83.521** kip_ft
Flexural strength reduction factor	$\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.005 - \varepsilon_{ty}), 0.65), 0.9)$ = **0.900**
Design moment capacity	$\phi M_n = \phi_f \times M_n$ = **75.169** kip_ft
	$\text{abs}(M_{u.y.min}) / \phi M_n$ = **0.001**
	PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force	$V_{u.y}$ = **3.21** kips
Depth to reinforcement	$d_v = \min(h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2, h - c_{nom\_t} - \phi_{y.top} / 2)$ = **8.062** in
Shear strength reduction factor	ϕ_v = **0.75**
Nominal shear capacity (Eq. 22.5.5.1)	$V_n = 2 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} \times L_x \times d_v$ = **67.725** kips

Design shear capacity	$\phi V_n = \phi_v \times V_n$ = **50.794** kips
	$V_{u.y} / \phi V_n$ = **0.063**
	PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement	d_{v2} = **8.375** in
Shear perimeter length (22.6.4)	l_{xp} = **16.375** in
Shear perimeter width (22.6.4)	l_{yp} = **16.375** in
Shear perimeter (22.6.4)	$b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2})$ = **65.500** in
Shear area	$A_p = l_{x,perim} \times l_{y,perim}$ = **268.141** in^2
Surcharge loaded area	$A_{sur} = A_p - l_{x1} \times l_{y1}$ = **204.141** in^2
Ultimate bearing pressure at center of shear area	$q_{up.avg}$ = **1.554** ksf
Ultimate shear load	$F_{up} = \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p$ = **12.303** kips
Ultimate shear stress from vertical load	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0\text{ psi})$ = **22.428** psi
Column geometry factor (Table 22.6.5.2)	$\beta = l_{y1} / l_{x1}$ = **1.00**
Column location factor (22.6.5.3)	α_s =**40**
Concrete shear strength (22.6.5.2)	$v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})}$ = **300.000** psi
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})}$ = **355.725** psi
	$v_{cpc} = 4 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})}$ = **200.000** psi
	$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc})$ = **200.000** psi
Shear strength reduction factor	ϕ_v = **0.75**
Nominal shear stress capacity (Eq. 22.6.1.2)	$v_n = v_{cp}$ = **200.000** psi
Design shear stress capacity (8.5.1.1(d))	$\phi v_n = \phi_v \times v_n$ = **150.000** psi
	$v_{ug} / \phi v_n$ = **0.150**
	PASS - Design shear stress capacity exceeds ultimate shear stress load

Two-way shear design at column 2

Depth to reinforcement	d_{v2} = **8.375** in
Shear perimeter length (22.6.4)	l_{xp} = **50.188** in
Shear perimeter width (22.6.4)	l_{yp} = **20.188** in
Shear perimeter (22.6.4)	$b_o = l_{x,perim} + l_{y,perim}$ = **70.375** in
Shear area	$A_p = l_{x,perim} \times l_{y,perim}$ = **1013.160** in^2
Surcharge loaded area	$A_{sur} = A_p - l_{x2} \times l_{y2}$ = **949.160** in^2
Ultimate bearing pressure at center of shear area	$q_{up.avg}$ = **1.843** ksf
Ultimate shear load	$F_{up} = \gamma_D \times (F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil}) + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p$ = **-8.403** kips
Ultimate shear stress from vertical load	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0\text{ psi})$ = **0.000** psi
Column geometry factor (Table 22.6.5.2)	$\beta = l_{y2} / l_{x2}$ = **1.00**
Column location factor (22.6.5.3)	α_s =**20**
Concrete shear strength (22.6.5.2)	$v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})}$ = **300.000** psi
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})}$ = **219.005** psi
	$v_{cpc} = 4 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})}$ = **200.000** psi
	$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc})$ = **200.000** psi
Shear strength reduction factor	ϕ_v = **0.75**

Nominal shear stress capacity (Eq. 22.6.1.2)	$v_n = v_{cp} =$ **200.000** psi
Design shear stress capacity (8.5.1.1(d))	$\phi v_n = \phi_v \times v_n =$ **150.000** psi
	$v_{ug} / \phi v_n =$ **0.000**

PASS - Design shear stress capacity exceeds ultimate shear stress load

Two-way shear design at column 3

Depth to reinforcement	$d_{v2} =$ **8.375** in
Shear perimeter length (22.6.4)	$l_{xp} =$ **26.187** in
Shear perimeter width (22.6.4)	$l_{yp} =$ **20.188** in
Shear perimeter (22.6.4)	$b_o = l_{x,perim} + l_{y,perim} =$ **46.375** in
Shear area	$A_p = l_{x,perim} \times l_{y,perim} =$ **528.660** in^2
Surcharge loaded area	$A_{sur} = A_p - l_{x3} \times l_{y3} =$ **464.660** in^2
Ultimate bearing pressure at center of shear area	$q_{up.avg} =$ **1.843** ksf
Ultimate shear load	$F_{up} = \gamma_D \times (F_{Dz3} - l_{x3} \times l_{y3} \times h_{soil} \times \gamma_{soil}) + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p =$ **-3.755** kips
Ultimate shear stress from vertical load	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0\text{ psi}) =$ **0.000** psi
Column geometry factor (Table 22.6.5.2)	$\beta = l_{y3} / l_{x3} =$ **1.00**
Column location factor (22.6.5.3)	$\alpha_s =$ **20**
Concrete shear strength (22.6.5.2)	$v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} =$ **300.000** psi
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} =$ **280.593** psi
	$v_{cpc} = 4 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} =$ **200.000** psi
	$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) =$ **200.000** psi
Shear strength reduction factor	$\phi_v =$ **0.75**
Nominal shear stress capacity (Eq. 22.6.1.2)	$v_n = v_{cp} =$ **200.000** psi
Design shear stress capacity (8.5.1.1(d))	$\phi v_n = \phi_v \times v_n =$ **150.000** psi
	$v_{ug} / \phi v_n =$ **0.000**

PASS - Design shear stress capacity exceeds ultimate shear stress load

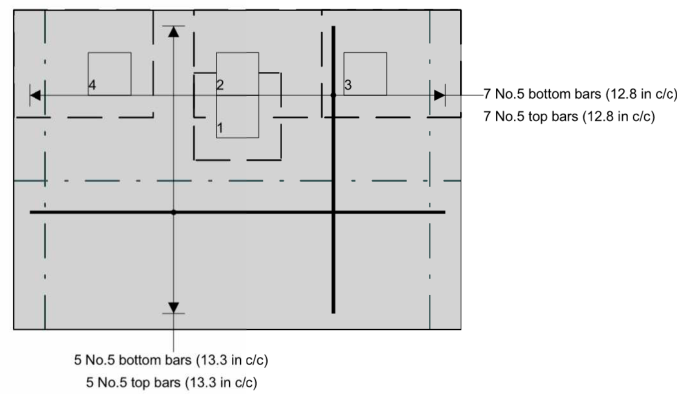
Two-way shear design at column 4

Depth to reinforcement	$d_{v2} =$ **8.375** in
Shear perimeter length (22.6.4)	$l_{xp} =$ **26.188** in
Shear perimeter width (22.6.4)	$l_{yp} =$ **20.188** in
Shear perimeter (22.6.4)	$b_o = l_{x,perim} + l_{y,perim} =$ **46.375** in
Shear area	$A_p = l_{x,perim} \times l_{y,perim} =$ **528.660** in^2
Surcharge loaded area	$A_{sur} = A_p - l_{x4} \times l_{y4} =$ **464.660** in^2
Ultimate bearing pressure at center of shear area	$q_{up.avg} =$ **1.843** ksf
Ultimate shear load	$F_{up} = \gamma_D \times (F_{Dz4} - l_{x4} \times l_{y4} \times h_{soil} \times \gamma_{soil}) + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p =$ **-3.755** kips
Ultimate shear stress from vertical load	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0\text{ psi}) =$ **0.000** psi
Column geometry factor (Table 22.6.5.2)	$\beta = l_{y4} / l_{x4} =$ **1.00**
Column location factor (22.6.5.3)	$\alpha_s =$ **20**
Concrete shear strength (22.6.5.2)	$v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} =$ **300.000** psi
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} =$ **280.593** psi
	$v_{cpc} = 4 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} =$ **200.000** psi
	$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) =$ **200.000** psi

Shear strength reduction factor	$\phi_v =$ **0.75**
Nominal shear stress capacity (Eq. 22.6.1.2)	$v_n = v_{cp} =$ **200.000** psi
Design shear stress capacity (8.5.1.1(d))	$\phi v_n = \phi_v \times v_n =$ **150.000** psi
	$v_{ug} / \phi v_n =$ **0.000**

PASS - Design shear stress capacity exceeds ultimate shear stress load

Note: Two-way shear is not calculated for columns that may share a shear perimeter with another column. This may need to be calculated conditionally by the engineer.



FOOTING ANALYSIS

In accordance with ACI318-14

Tedds calculation version 3.3.08

Summary results

Overall design status PASS

Overall design utilisation 0.320

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	49.0			Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	0.512	1.6	0.320	Pass
Description	**Unit**	**Required**	**Provided**	**Utilization**	**Result**
Moment, positive, x-direction	kip_ft	19.7	147.2	0.134	Pass
Moment, positive, y-direction	kip_ft	18.8	129.1	0.146	Pass
Shear, one-way, x-direction	kips	8.1	91.2	0.089	Pass
Shear, one-way, y-direction	kips	7.1	62.0	0.115	Pass
Shear, two-way, Col 1	psi	24.906	150.000	0.166	Pass
Min.area of reinf, bot., x-direction	in²	3.046	4.400		Pass
Max.reinf.spacing, bot, x-direction	in	18.0	14.9		Pass
Min.area of reinf, bot., y-direction	in²	2.268	3.520		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	14.0		Pass

Pad footing details

Length of footing L_x = **8.75** ft

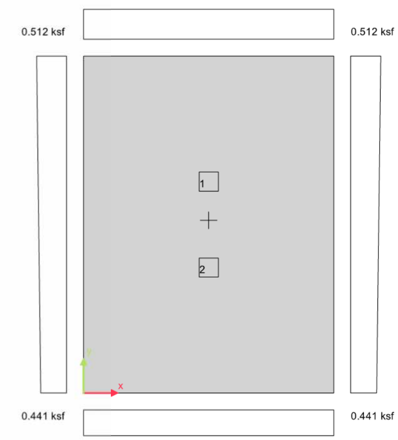
Width of footing L_y = **11.75** ft

Footing area $A = L_x \times L_y$ = **102.813** ft^2

Depth of footing h = **12** in

Depth of soil over footing h_{soil} = **18** in

Density of concrete γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column l_{x1} = **8.00** in

Width of column l_{y1} = **8.00** in

position in x-axis x_1 = **52.50** in

position in y-axis y_1 = **88.50** in

Column no.2 details

Length of column l_{x2} = **8.00** in

Width of column l_{y2} = **8.00** in

position in x-axis x_2 = **52.50** in

position in y-axis y_2 = **52.50** in

Soil properties

Net allowable bearing pressure q_{allow_Net} = **1.5** ksf using a soil factor of safety, FS_{soil}, of 3

Density of soil γ_{soil} = **120.0** lb/ft^3

Angle of internal friction ϕ_b = **30.0** deg

Design base friction angle δ_{bb} = **30.0** deg

Coefficient of base friction $\tan(\delta_{bb})$ = **0.577**

Footing loads

Self weight $F_{swt} = h \times \gamma_{conc}$ = **150** psf

Soil weight $F_{soil} = h_{soil} \times \gamma_{soil}$ = **180** psf

Column no.1 loads

Dead load in z F_{Dz1} = **10.0** kips

Column no.2 loads

Dead load in z F_{Dz2} = **5.2** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.320)

Combination 1 results: 1.0D

Forces on footing

Force in z-axis — $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_D \times (F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil})$ = **49.0** kips

Moments on footing

Moment in x-axis, about x is 0 — $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_D \times (((F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil})) \times x_2)$ = **214.2** kip_ft

Moment in y-axis, about y is 0 — $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_D \times (((F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil})) \times y_2)$ = **294.9** kip_ft

Uplift verification

Vertical force — F_{dz} = **48.968** kips

PASS - Footing is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis — $e_{dx} = M_{dx} / F_{dz} - L_x / 2$ = **0** in

Eccentricity of base reaction in y-axis — $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **1.764** in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **0.441** ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **0.512** ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **0.441** ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **0.512** ksf

Minimum base pressure — $q_{min} = \min(q_1,q_2,q_3,q_4)$ = **0.441** ksf

Maximum base pressure — $q_{max} = \max(q_1,q_2,q_3,q_4)$ = **0.512** ksf

Allowable bearing capacity

Allowable bearing capacity — $q_{allow} = q_{allow\_Net} + ((h + h_{soil}) \times \gamma_{soil}) / FS_{soil}$ = **1.6** ksf

q_{max} / q_{allow} = **0.320**

PASS - Allowable bearing capacity exceeds design base pressure

__FOOTING DESIGN__

In accordance with ACI318-14

Tedds calculation version 3.3.08

Material details

Compressive strength of concrete	f'_c = **2500** psi
Yield strength of reinforcement	f_y = **60000** psi
Compression-controlled strain limit (21.2.2)	ε_{ty} = **0.00200**
Cover to top of footing	c_{nom_t} = **3** in
Cover to side of footing	c_{nom_s} = **3** in
Cover to bottom of footing	c_{nom_b} = **3** in
Concrete type	Normal weight

Concrete modification factor	λ = **1.00**
Column type	Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-16

1.4D (0.166)

Combination 1 results: 1.4D

Forces on footing

Ultimate force in z-axis — $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_D \times (F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil})$ = **68.6** kips

Moments on footing

Ultimate moment in x-axis, about x is 0 — $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_D \times (((F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil})) \times x_2)$ = **299.9** kip_ft

Ultimate moment in y-axis, about y is 0 — $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_D \times (((F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil})) \times y_2)$ = **412.8** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis — $e_{ux} = M_{ux} / F_{uz} - L_x / 2$ = **0** in

Eccentricity of base reaction in y-axis — $e_{uy} = M_{uy} / F_{uz} - L_y / 2$ = **1.764** in

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **0.617** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **0.717** ksf

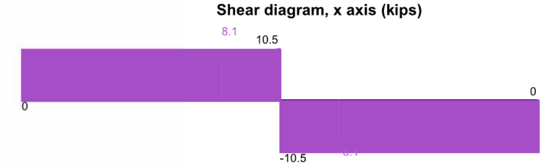
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **0.617** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **0.717** ksf

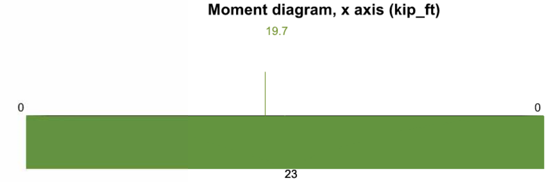
Minimum ultimate base pressure — $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4})$ = **0.617** ksf

Maximum ultimate base pressure — $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4})$ = **0.717** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Tekla Tedds	Project				Job Ref.	
	Kimberly AG Shop				24091	
	Section				Sheet no./rev.	
	Footing @ 1/H				5	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	ANB	10/2/2024				

Moment design, x direction, positive moment

Ultimate bending moment — $M_{u.x.max}$ = **19.658** kip_ft
Tension reinforcement provided — 10 No.6 bottom bars (14.9 in c/c)
Area of tension reinforcement provided — $A_{sx.bot.prov}$ = **4.4** in^2
Minimum area of reinforcement (8.6.1.1) — $A_{s.min} = 0.0018 \times L_y \times h$ = **3.046** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2) — $s_{max} = \min(2 \times h, 18\text{ in})$ = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement — $d = h - c_{nom\_b} - \phi_{y.bot} - \phi_{x.bot} / 2$ = **7.875** in
Depth of compression block — $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y)$ = **0.881** in
Neutral axis factor — β_1 = **0.85**
Depth to neutral axis — $c = a / \beta_1$ = **1.037** in
Strain in tensile reinforcement — $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.01979**
Minimum tensile strain(8.3.3.1) — $\varepsilon_{min} = 0.004$ = **0.00400**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity — $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2)$ = **163.558** kip_ft
Flexural strength reduction factor — $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.005 - \varepsilon_{ty}), 0.65), 0.9)$ = **0.900**
Design moment capacity — $\phi M_n = \phi_f \times M_n$ = **147.202** kip_ft
$M_{u.x.max} / \phi M_n$ = **0.134**

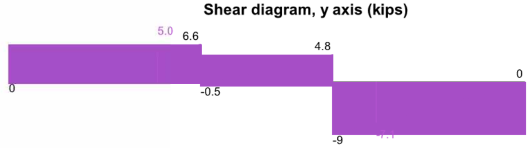
PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force — $V_{u.x}$ = **8.147** kips
Depth to reinforcement — $d_v = h - c_{nom\_b} - \phi_{x.bot} / 2$ = **8.625** in
Shear strength reduction factor — ϕ_v = **0.75**
Nominal shear capacity (Eq. 22.5.5.1) — $V_n = 2 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} \times L_y \times d_v$ = **121.613** kips
Design shear capacity — $\phi V_n = \phi_v \times V_n$ = **91.209** kips
$V_{u.x} / \phi V_n$ = **0.089**

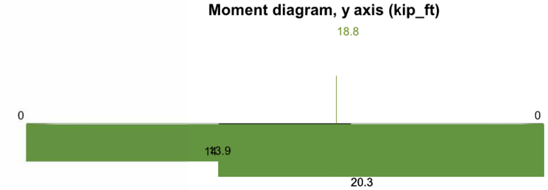
PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Tekla Tedds	Project				Job Ref.	
	Kimberly AG Shop				24091	
	Section				Sheet no./rev.	
	Footing @ 1/H				6	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	ANB	10/2/2024				

Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment — $M_{u.y.max}$ = **18.793** kip_ft
Tension reinforcement provided — 8 No.6 bottom bars (14.0 in c/c)
Area of tension reinforcement provided — $A_{sy.bot.prov}$ = **3.52** in^2
Minimum area of reinforcement (8.6.1.1) — $A_{s.min} = 0.0018 \times L_x \times h$ = **2.268** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2) — $s_{max} = \min(2 \times h, 18\text{ in})$ = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement — $d = h - c_{nom\_b} - \phi_{y.bot} / 2$ = **8.625** in
Depth of compression block — $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.947** in
Neutral axis factor — β_1 = **0.85**
Depth to neutral axis — $c = a / \beta_1$ = **1.114** in
Strain in tensile reinforcement — $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.02024**
Minimum tensile strain(8.3.3.1) — $\varepsilon_{min} = 0.004$ = **0.00400**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity — $M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2)$ = **143.47** kip_ft
Flexural strength reduction factor — $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.005 - \varepsilon_{ty}), 0.65), 0.9)$ = **0.900**
Design moment capacity — $\phi M_n = \phi_f \times M_n$ = **129.123** kip_ft
$M_{u.y.max} / \phi M_n$ = **0.146**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force — $V_{u.y}$ = **7.122** kips
Depth to reinforcement — $d_v = h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **7.875** in
Shear strength reduction factor — ϕ_v = **0.75**
Nominal shear capacity (Eq. 22.5.5.1) — $V_n = 2 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} \times L_x \times d_v$ = **82.688** kips
Design shear capacity — $\phi V_n = \phi_v \times V_n$ = **62.016** kips
$V_{u.y} / \phi V_n$ = **0.115**

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement — d_{v2} = **8.25** in
Shear perimeter length (22.6.4) — l_{xp} = **16.250** in
Shear perimeter width (22.6.4) — l_{yp} = **16.250** in
Shear perimeter (22.6.4) — $b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2})$ = **65.000** in
Shear area — $A_p = l_{x.perim} \times l_{y.perim}$ = **264.062** in^2

Surcharge loaded area	$A_{sur} = A_p - l_{x1} \times l_{y1} = \mathbf{200.062}\ in^2$
Ultimate bearing pressure at center of shear area	$q_{up.avg} = \mathbf{0.691}$ ksf
Ultimate shear load	$F_{up} = \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p = \mathbf{13.356}$ kips
Ultimate shear stress from vertical load	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}),0\ psi) = \mathbf{24.906}$ psi
Column geometry factor (Table 22.6.5.2)	$\beta = l_{y1} / l_{x1} = \mathbf{1.00}$
Column location factor (22.6.5.3)	$\alpha_s = \mathbf{40}$
Concrete shear strength (22.6.5.2)	$v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{(f'_c \times 1\ psi)} = \mathbf{300.000}$ psi
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{(f'_c \times 1\ psi)} = \mathbf{353.846}$ psi
	$v_{cpc} = 4 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} = \mathbf{200.000}$ psi
	$v_{cp} = \min(v_{cpa},v_{cpb},v_{cpc}) = \mathbf{200.000}$ psi
Shear strength reduction factor	$\phi_v = \mathbf{0.75}$
Nominal shear stress capacity (Eq. 22.6.1.2)	$v_n = v_{cp} = \mathbf{200.000}$ psi
Design shear stress capacity (8.5.1.1(d))	$\phi v_n = \phi_v \times v_n = \mathbf{150.000}$ psi
	$v_{ug} / \phi v_n = \mathbf{0.166}$

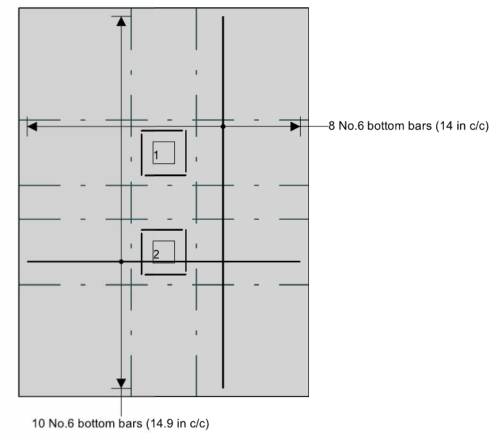
PASS - Design shear stress capacity exceeds ultimate shear stress load

Two-way shear design at column 2

Depth to reinforcement	$d_{v2} = \mathbf{8.25}$ in
Shear perimeter length (22.6.4)	$l_{xp} = \mathbf{16.250}$ in
Shear perimeter width (22.6.4)	$l_{yp} = \mathbf{16.250}$ in
Shear perimeter (22.6.4)	$b_o = 2 \times (l_{x2} + d_{v2}) + 2 \times (l_{y2} + d_{v2}) = \mathbf{65.000}$ in
Shear area	$A_p = l_{x,perim} \times l_{y,perim} = \mathbf{264.062}\ in^2$
Surcharge loaded area	$A_{sur} = A_p - l_{x2} \times l_{y2} = \mathbf{200.062}\ in^2$
Ultimate bearing pressure at center of shear area	$q_{up.avg} = \mathbf{0.666}$ ksf
Ultimate shear load	$F_{up} = \gamma_D \times (F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil}) + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p = \mathbf{6.683}$ kips
Ultimate shear stress from vertical load	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}),0\ psi) = \mathbf{12.462}$ psi
Column geometry factor (Table 22.6.5.2)	$\beta = l_{y2} / l_{x2} = \mathbf{1.00}$
Column location factor (22.6.5.3)	$\alpha_s = \mathbf{40}$
Concrete shear strength (22.6.5.2)	$v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{(f'_c \times 1\ psi)} = \mathbf{300.000}$ psi
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{(f'_c \times 1\ psi)} = \mathbf{353.846}$ psi
	$v_{cpc} = 4 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} = \mathbf{200.000}$ psi
	$v_{cp} = \min(v_{cpa},v_{cpb},v_{cpc}) = \mathbf{200.000}$ psi
Shear strength reduction factor	$\phi_v = \mathbf{0.75}$
Nominal shear stress capacity (Eq. 22.6.1.2)	$v_n = v_{cp} = \mathbf{200.000}$ psi
Design shear stress capacity (8.5.1.1(d))	$\phi v_n = \phi_v \times v_n = \mathbf{150.000}$ psi
	$v_{ug} / \phi v_n = \mathbf{0.083}$

PASS - Design shear stress capacity exceeds ultimate shear stress load

Note: Two-way shear is not calculated for columns that may share a shear perimeter with another column. This may need to be calculated conditionally by the engineer.



FOOTING ANALYSIS

In accordance with ACI318-14

Tedds calculation version 3.3.08

Summary results

Overall design status PASS

Overall design utilisation 0.676

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	11.3			Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	1.082	1.6	0.676	Pass
Description	**Unit**	**Required**	**Provided**	**Utilization**	**Result**
Moment, positive, x-direction	kip_ft	0.1	43.0	0.002	Pass
Moment, negative, x-direction	kip_ft	0.4	46.4	0.008	Pass
Moment, positive, y-direction	kip_ft	1.9	46.4	0.041	Pass
Shear, one-way, y-direction	kips	0.9	29.0	0.031	Pass
Shear, two-way, Col 1	psi	3.946	150.000	0.026	Pass
Min.area of reinf, bot., x-direction	in²	1.037	1.240		Pass
Min.area of reinf, top, x-direction	in²	1.037	1.240		Pass
Max.reinf.spacing, top, x-direction	in	18.0	13.7		Pass
Min.area of reinf, bot., y-direction	in²	1.037	1.240		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	13.7		Pass

Pad footing details

Length of footing L_x = **4** ft

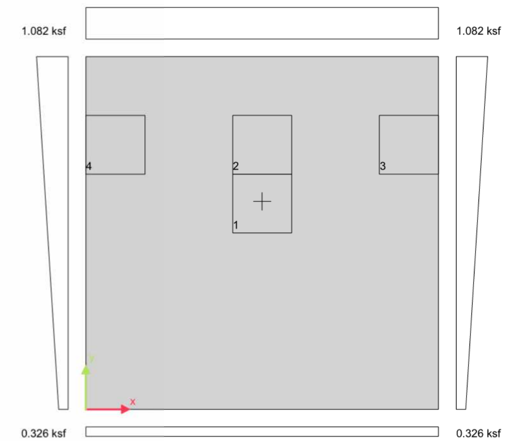
Width of footing L_y = **4** ft

Footing area $A = L_x \times L_y$ = **16** ft^2

Depth of footing h = **12** in

Depth of soil over footing h_{soil} = **18** in

Density of concrete γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column l_{x1} = **8.00** in

Width of column l_{y1} = **8.00** in

position in x-axis x_1 = **24.00** in

position in y-axis y_1 = **28.00** in

Column no.2 details

Length of column l_{x2} = **8.00** in

Width of column l_{y2} = **8.00** in

position in x-axis x_2 = **24.00** in

position in y-axis y_2 = **36.00** in

Column no.3 details

Length of column l_{x3} = **8.00** in

Width of column l_{y3} = **8.00** in

position in x-axis x_3 = **44.00** in

position in y-axis y_3 = **36.00** in

Column no.4 details

Length of column l_{x4} = **8.00** in

Width of column l_{y4} = **8.00** in

position in x-axis x_4 = **4.00** in

position in y-axis y_4 = **36.00** in

Soil properties

Net allowable bearing pressure q_{allow_Net} = **1.5** ksf using a soil factor of safety, FS_{soil}, of 3

Density of soil γ_{soil} = **120.0** lb/ft^3

Angle of internal friction ϕ_b = **30.0** deg

Design base friction angle δ_{bb} = **30.0** deg

Coefficient of base friction $\tan(\delta_{bb})$ = **0.577**

Footing loads

Self weight $F_{swt} = h \times \gamma_{conc}$ = **150** psf

Soil weight $F_{soil} = h_{soil} \times \gamma_{soil}$ = **180** psf

Column no.1 loads

Dead load in z F_{Dz1} = **3.0** kips

Column no.2 loads

Dead load in z F_{Dz2} = **1.1** kips

Column no.3 loads

Dead load in z F_{Dz3} = **1.1** kips

Column no.4 loads

Dead load in z F_{Dz4} = **1.1** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.676)

Combination 1 results: 1.0D

Forces on footing

Force in z-axis $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_D \times (F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil}) + \gamma_D \times (F_{Dz3} - l_{x3} \times l_{y3} \times h_{soil} \times \gamma_{soil}) + \gamma_D \times (F_{Dz4} - l_{x4} \times l_{y4} \times h_{soil} \times \gamma_{soil})$ = **11.3** kips

Moments on footing

Moment in x-axis, about x is 0 $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_D \times (((F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil})) \times x_2) + \gamma_D \times (((F_{Dz3} - l_{x3} \times l_{y3} \times h_{soil} \times \gamma_{soil})) \times x_3) + \gamma_D \times (((F_{Dz4} - l_{x4} \times l_{y4} \times h_{soil} \times \gamma_{soil})) \times x_4)$ = **22.5** kip_ft

Moment in y-axis, about y is 0 $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_D \times (((F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil})) \times y_2) + \gamma_D \times (((F_{Dz3} - l_{x3} \times l_{y3} \times h_{soil} \times \gamma_{soil})) \times y_3) + \gamma_D \times (((F_{Dz4} - l_{x4} \times l_{y4} \times h_{soil} \times \gamma_{soil})) \times y_4)$ = **26.6** kip_ft

Uplift verification

Vertical force F_{dz} = **11.26** kips

PASS - Footing is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{dx} = M_{dx} / F_{dz} - L_x / 2$ = **0** in

Eccentricity of base reaction in y-axis $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **4.298** in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **0.326** ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.082** ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **0.326** ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.082** ksf

Minimum base pressure $q_{min} = \min(q_1,q_2,q_3,q_4)$ = **0.326** ksf

Maximum base pressure $q_{max} = \max(q_1,q_2,q_3,q_4)$ = **1.082** ksf

Allowable bearing capacity

Allowable bearing capacity $q_{allow} = q_{allow\_Net} + ((h + h_{soil}) \times \gamma_{soil}) / FS_{soil}$ = **1.6** ksf

q_{max} / q_{allow} = **0.676**

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN

In accordance with ACI318-14

Tedds calculation version 3.3.08

Material details

Compressive strength of concrete f'_c = **2500** psi

Yield strength of reinforcement f_y = **60000** psi

Compression-controlled strain limit (21.2.2) ε_{ty} = **0.00200**

Cover to top of footing c_{nom_t} = **3** in

Cover to side of footing c_{nom_s} = **3** in

Cover to bottom of footing c_{nom_b} = **3** in

Concrete type Normal weight

Concrete modification factor λ = **1.00**

Column type Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-16

1.4D (0.041)

Combination 1 results: 1.4D

Forces on footing

Ultimate force in z-axis $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_D \times (F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil}) + \gamma_D \times (F_{Dz3} - l_{x3} \times l_{y3} \times h_{soil} \times \gamma_{soil}) + \gamma_D \times (F_{Dz4} - l_{x4} \times l_{y4} \times h_{soil} \times \gamma_{soil})$ = **15.8** kips

Moments on footing

Ultimate moment in x-axis, about x is 0 $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1) + \gamma_D \times (((F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil})) \times x_2) + \gamma_D \times (((F_{Dz3} - l_{x3} \times l_{y3} \times h_{soil} \times \gamma_{soil})) \times x_3) + \gamma_D \times (((F_{Dz4} - l_{x4} \times l_{y4} \times h_{soil} \times \gamma_{soil})) \times x_4)$ = **31.5** kip_ft

Ultimate moment in y-axis, about y is 0 $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) + \gamma_D \times (((F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil})) \times y_2) + \gamma_D \times (((F_{Dz3} - l_{x3} \times l_{y3} \times h_{soil} \times \gamma_{soil})) \times y_3) + \gamma_D \times (((F_{Dz4} - l_{x4} \times l_{y4} \times h_{soil} \times \gamma_{soil})) \times y_4)$ = **37.2** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{ux} = M_{ux} / F_{uz} - L_x / 2$ = **0** in

Eccentricity of base reaction in y-axis $e_{uy} = M_{uy} / F_{uz} - L_y / 2$ = **4.298** in

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **0.456** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **1.515** ksf

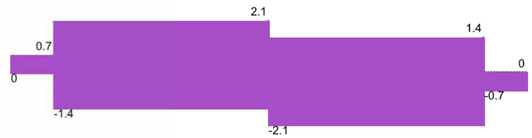
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{0.456}$ ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{1.515}$ ksf

Minimum ultimate base pressure — $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4}) = \mathbf{0.456}$ ksf

Maximum ultimate base pressure — $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4}) = \mathbf{1.515}$ ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)

Moment design, x direction, positive moment

Ultimate bending moment — $M_{u.x.max} = \mathbf{0.093}$ kip_ft

Tension reinforcement provided — 4 No.5 bottom bars (13.7 in c/c)

Area of tension reinforcement provided — $A_{sx.bot.prov} = \mathbf{1.24}$ in^2

Minimum area of reinforcement (8.6.1.1) — $A_{s.min} = 0.0018 \times L_y \times h = \mathbf{1.037}$ in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2) — $s_{max} = \min(2 \times h, 18\text{ in}) = \mathbf{18}$ in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement — $d = h - c_{nom\_b} - \phi_{y.bot} - \phi_{x.bot} / 2 = \mathbf{8.062}$ in

Depth of compression block — $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y) = \mathbf{0.729}$ in

Neutral axis factor — $\beta_1 = \mathbf{0.85}$

Depth to neutral axis — $c = a / \beta_1 = \mathbf{0.858}$ in

Strain in tensile reinforcement — $\varepsilon_t = 0.003 \times d / c - 0.003 = \mathbf{0.02519}$

Minimum tensile strain(8.3.3.1) — $\varepsilon_{min} = 0.004 = \mathbf{0.00400}$

PASS - Tensile strain exceeds minimum required

Nominal moment capacity — $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2) = \mathbf{47.726}$ kip_ft

Flexural strength reduction factor — $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.005 - \varepsilon_{ty}), 0.65), 0.9) = \mathbf{0.900}$

Design moment capacity — $\phi M_n = \phi_f \times M_n = \mathbf{42.954}$ kip_ft

$M_{u.x.max} / \phi M_n = \mathbf{0.002}$

PASS - Design moment capacity exceeds ultimate moment load

Moment design, x direction, negative moment

Ultimate bending moment — $M_{u.x.min} = \mathbf{-0.361}$ kip_ft

Tension reinforcement provided — 4 No.5 top bars (13.7 in c/c)

Area of tension reinforcement provided — $A_{sx.top.prov} = \mathbf{1.24}$ in^2

Minimum area of reinforcement (8.6.1.1) — $A_{s.min} = 0.0018 \times L_y \times h = \mathbf{1.037}$ in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2) — $s_{max} = \min(2 \times h, 18\text{ in}) = \mathbf{18}$ in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement — $d = h - c_{nom\_t} - \phi_{x.top} / 2 = \mathbf{8.688}$ in

Depth of compression block — $a = A_{sx.top.prov} \times f_y / (0.85 \times f'_c \times L_y) = \mathbf{0.729}$ in

Neutral axis factor — $\beta_1 = \mathbf{0.85}$

Depth to neutral axis — $c = a / \beta_1 = \mathbf{0.858}$ in

Strain in tensile reinforcement — $\varepsilon_t = 0.003 \times d / c - 0.003 = \mathbf{0.02737}$

Minimum tensile strain(8.3.3.1) — $\varepsilon_{min} = 0.004 = \mathbf{0.00400}$

PASS - Tensile strain exceeds minimum required

Nominal moment capacity — $M_n = A_{sx.top.prov} \times f_y \times (d - a / 2) = \mathbf{51.601}$ kip_ft

Flexural strength reduction factor — $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.005 - \varepsilon_{ty}), 0.65), 0.9) = \mathbf{0.900}$

Design moment capacity — $\phi M_n = \phi_f \times M_n = \mathbf{46.441}$ kip_ft

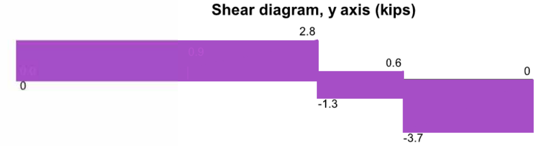
$\text{abs}(M_{u.x.min}) / \phi M_n = \mathbf{0.008}$

PASS - Design moment capacity exceeds ultimate moment load

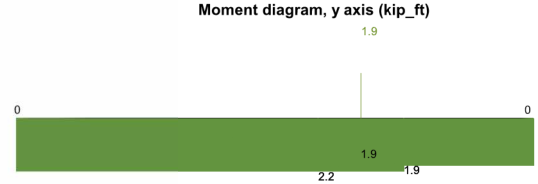
One-way shear design, x direction

One-way shear design does not apply. Shear failure plane fall outside extents of foundation.

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment — $M_{u.y.max} = \mathbf{1.896}$ kip_ft

Tension reinforcement provided — 4 No.5 bottom bars (13.7 in c/c)

Area of tension reinforcement provided — $A_{sy.bot.prov} = \mathbf{1.24}$ in^2

Minimum area of reinforcement (8.6.1.1) — $A_{s.min} = 0.0018 \times L_x \times h = \mathbf{1.037}$ in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2) — $s_{max} = \min(2 \times h, 18\text{ in}) = \mathbf{18}$ in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement — $d = h - c_{nom\_b} - \phi_{y.bot} / 2 = \mathbf{8.688}$ in

Depth of compression block	$a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x) = \mathbf{0.729}$ in
Neutral axis factor	$\beta_1 = \mathbf{0.85}$
Depth to neutral axis	$c = a / \beta_1 = \mathbf{0.858}$ in
Strain in tensile reinforcement	$\varepsilon_t = 0.003 \times d / c - 0.003 = \mathbf{0.02737}$
Minimum tensile strain(8.3.3.1)	$\varepsilon_{min} = 0.004 = \mathbf{0.00400}$
	PASS - Tensile strain exceeds minimum required
Nominal moment capacity	$M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2) = \mathbf{51.601}$ kip_ft
Flexural strength reduction factor	$\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.005 - \varepsilon_{ty}), 0.65), 0.9) = \mathbf{0.900}$
Design moment capacity	$\phi M_n = \phi_f \times M_n = \mathbf{46.441}$ kip_ft
	$M_{u.y.max} / \phi M_n = \mathbf{0.041}$
	PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force	$V_{u.y} = \mathbf{0.902}$ kips
Depth to reinforcement	$d_v = \min(h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2, h - c_{nom\_t} - \phi_{y.top} / 2) = \mathbf{8.062}$ in
Shear strength reduction factor	$\phi_v = \mathbf{0.75}$
Nominal shear capacity (Eq. 22.5.5.1)	$V_n = 2 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_x \times d_v = \mathbf{38.7}$ kips
Design shear capacity	$\phi V_n = \phi_v \times V_n = \mathbf{29.025}$ kips
	$V_{u.y} / \phi V_n = \mathbf{0.031}$
	PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement	$d_{v2} = \mathbf{8.375}$ in
Shear perimeter length (22.6.4)	$l_{xp} = \mathbf{16.375}$ in
Shear perimeter width (22.6.4)	$l_{yp} = \mathbf{16.375}$ in
Shear perimeter (22.6.4)	$b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2}) = \mathbf{65.500}$ in
Shear area	$A_p = l_{x,perim} \times l_{y,perim} = \mathbf{268.141}$ in^2
Surcharge loaded area	$A_{sur} = A_p - l_{x1} \times l_{y1} = \mathbf{204.141}$ in^2
Ultimate bearing pressure at center of shear area	$q_{up.avg} = \mathbf{1.435}$ ksf
Ultimate shear load	$F_{up} = \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p = \mathbf{2.165}$ kips
Ultimate shear stress from vertical load	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0 \text{ psi}) = \mathbf{3.946}$ psi
Column geometry factor (Table 22.6.5.2)	$\beta = l_{y1} / l_{x1} = \mathbf{1.00}$
Column location factor (22.6.5.3)	$\alpha_s = \mathbf{40}$
Concrete shear strength (22.6.5.2)	$v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} = \mathbf{300.000}$ psi
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} = \mathbf{355.725}$ psi
	$v_{cpc} = 4 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} = \mathbf{200.000}$ psi
	$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) = \mathbf{200.000}$ psi
Shear strength reduction factor	$\phi_v = \mathbf{0.75}$
Nominal shear stress capacity (Eq. 22.6.1.2)	$v_n = v_{cp} = \mathbf{200.000}$ psi
Design shear stress capacity (8.5.1.1(d))	$\phi v_n = \phi_v \times v_n = \mathbf{150.000}$ psi
	$v_{ug} / \phi v_n = \mathbf{0.026}$
	PASS - Design shear stress capacity exceeds ultimate shear stress load

Two-way shear design at column 2

Depth to reinforcement	$d_{v2} = \mathbf{8.375}$ in
Shear perimeter length (22.6.4)	$l_{xp} = \mathbf{32.188}$ in
Shear perimeter width (22.6.4)	$l_{yp} = \mathbf{20.188}$ in
Shear perimeter (22.6.4)	$b_o = l_{x,perim} + l_{y,perim} = \mathbf{52.375}$ in
Shear area	$A_p = l_{x,perim} \times l_{y,perim} = \mathbf{649.785}$ in^2
Surcharge loaded area	$A_{sur} = A_p - l_{x2} \times l_{y2} = \mathbf{585.785}$ in^2
Ultimate bearing pressure at center of shear area	$q_{up.avg} = \mathbf{1.737}$ ksf
Ultimate shear load	$F_{up} = \gamma_D \times (F_{Dz2} - l_{x2} \times l_{y2} \times h_{soil} \times \gamma_{soil}) + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p = \mathbf{-4.439}$ kips
Ultimate shear stress from vertical load	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0 \text{ psi}) = \mathbf{0.000}$ psi
Column geometry factor (Table 22.6.5.2)	$\beta = l_{y2} / l_{x2} = \mathbf{1.00}$
Column location factor (22.6.5.3)	$\alpha_s = \mathbf{20}$
Concrete shear strength (22.6.5.2)	$v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} = \mathbf{300.000}$ psi
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} = \mathbf{259.905}$ psi
	$v_{cpc} = 4 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} = \mathbf{200.000}$ psi
	$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) = \mathbf{200.000}$ psi
Shear strength reduction factor	$\phi_v = \mathbf{0.75}$
Nominal shear stress capacity (Eq. 22.6.1.2)	$v_n = v_{cp} = \mathbf{200.000}$ psi
Design shear stress capacity (8.5.1.1(d))	$\phi v_n = \phi_v \times v_n = \mathbf{150.000}$ psi
	$v_{ug} / \phi v_n = \mathbf{0.000}$
	PASS - Design shear stress capacity exceeds ultimate shear stress load

Two-way shear design at column 3

Depth to reinforcement	$d_{v2} = \mathbf{8.375}$ in
Shear perimeter length (22.6.4)	$l_{xp} = \mathbf{12.188}$ in
Shear perimeter width (22.6.4)	$l_{yp} = \mathbf{20.188}$ in
Shear perimeter (22.6.4)	$b_o = l_{x,perim} + l_{y,perim} = \mathbf{32.375}$ in
Shear area	$A_p = l_{x,perim} \times l_{y,perim} = \mathbf{246.035}$ in^2
Surcharge loaded area	$A_{sur} = A_p - l_{x3} \times l_{y3} = \mathbf{182.035}$ in^2
Ultimate bearing pressure at center of shear area	$q_{up.avg} = \mathbf{1.737}$ ksf
Ultimate shear load	$F_{up} = \gamma_D \times (F_{Dz3} - l_{x3} \times l_{y3} \times h_{soil} \times \gamma_{soil}) + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p = \mathbf{-0.863}$ kips
Ultimate shear stress from vertical load	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0 \text{ psi}) = \mathbf{0.000}$ psi
Column geometry factor (Table 22.6.5.2)	$\beta = l_{y3} / l_{x3} = \mathbf{1.00}$
Column location factor (22.6.5.3)	$\alpha_s = \mathbf{20}$
Concrete shear strength (22.6.5.2)	$v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} = \mathbf{300.000}$ psi
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} = \mathbf{358.687}$ psi
	$v_{cpc} = 4 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} = \mathbf{200.000}$ psi
	$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) = \mathbf{200.000}$ psi
Shear strength reduction factor	$\phi_v = \mathbf{0.75}$
Nominal shear stress capacity (Eq. 22.6.1.2)	$v_n = v_{cp} = \mathbf{200.000}$ psi
Design shear stress capacity (8.5.1.1(d))	$\phi v_n = \phi_v \times v_n = \mathbf{150.000}$ psi
	$v_{ug} / \phi v_n = \mathbf{0.000}$

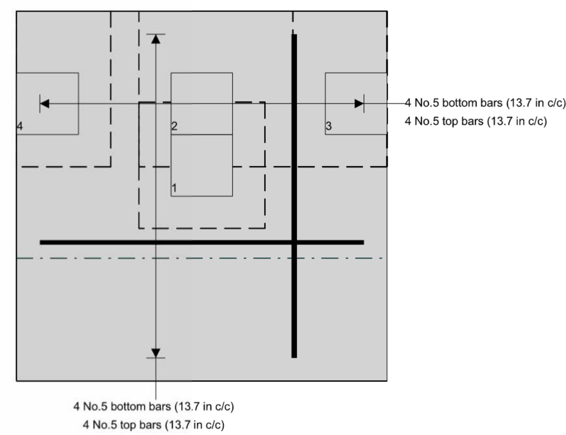
PASS - Design shear stress capacity exceeds ultimate shear stress load

Two-way shear design at column 4

Depth to reinforcement	d_{v2} = **8.375** in
Shear perimeter length (22.6.4)	l_{xp} = **12.188** in
Shear perimeter width (22.6.4)	l_{yp} = **20.188** in
Shear perimeter (22.6.4)	$b_o = l_{x,perim} + l_{y,perim}$ = **32.375** in
Shear area	$A_p = l_{x,perim} \times l_{y,perim}$ = **246.035** in^2
Surcharge loaded area	$A_{sur} = A_p - l_{x4} \times l_{y4}$ = **182.035** in^2
Ultimate bearing pressure at center of shear area	$q_{up.avg}$ = **1.737** ksf
Ultimate shear load	$F_{up} = \gamma_D \times (F_{Dz4} - l_{x4} \times l_{y4} \times h_{soil} \times \gamma_{soil}) + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p$ = **-0.863** kips
Ultimate shear stress from vertical load	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}),0\text{ psi})$ = **0.000** psi
Column geometry factor (Table 22.6.5.2)	$\beta = l_{y4} / l_{x4}$ = **1.00**
Column location factor (22.6.5.3)	α_s =**20**
Concrete shear strength (22.6.5.2)	$v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})}$ = **300.000** psi
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})}$ = **358.687** psi
	$v_{cpc} = 4 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})}$ = **200.000** psi
	$v_{cp} = \min(v_{cpa},v_{cpb},v_{cpc})$ = **200.000** psi
Shear strength reduction factor	ϕ_v = **0.75**
Nominal shear stress capacity (Eq. 22.6.1.2)	$v_n = v_{cp}$ = **200.000** psi
Design shear stress capacity (8.5.1.1(d))	$\phi v_n = \phi_v \times v_n$ = **150.000** psi
	$v_{ug} / \phi v_n$ = **0.000**

PASS - Design shear stress capacity exceeds ultimate shear stress load

Note: Two-way shear is not calculated for columns that may share a shear perimeter with another column. This may need to be calculated conditionally by the engineer.



FOOTING DESIGN:

* GRID 2/H:

Max up: -9.8K → F6 *
down: 26.9K → F4.5

* GRID 2/E:

Max up: -7.1K → F6.5 * (TOF @ 99'-0" & no strip)
Down: 39.2K → F5.5

* GRID 2/A:

Max up: -7.3K → F5
Down: 16.9K → F3.5

* GRID 3/A:

Max up: -3.4K → F3
Down: 5K → F3

* GRID 3/C:

Max up: -2.0K → F3
Down: 7.9K → F3
HORIZ: 3.5K → F4

FOOTING DESIGN (CONT'D):

* GRID 3/D:

MAX UP: -3.4K → F3
DOWN: 9.1K → F3
} STRIP FTG OK

* GRID 3/F:

MAX UP: -3.1K → F3
DOWN: 8.9K → F3
} STRIP FTG OK.

* GRID 3/G:

MAX UP: -2.0K → F3
DOWN: 10K → F3
} STRIP FTG OK

* GRID 3/H:

MAX UP: -4.5K → F3.5 → USE F4
DOWN: 5.8K → F3

ARW ENGINEERS structural consultants

ARW ENGINEERS
FOOTING SUMMARY (PER IBC 2018 & ACI 318-14)

10/8/24 6:00 AM

JOB TITLE: Kimberly AG Shop
DESCRIPTION: Footing Schedule

JOB #: 24091
DESIGNER: ANB
Version : January 23, 2019

COLUMN SUMMARY:

Material
Steel or Wood

Size
8 in

Base Plate Dimension
14 x 14 in^2

SOIL SUMMARY:

Net Soil Bearing Pressure
1500 psf

CONCRETE SUMMARY:

Concrete Strength
2500 psi

WIDTH (ft)	LENGTH (ft)	THICK (in)	$A_{s\ req'd}$ (in^2)	$A_{s\ actual}$ (in^2)	Number of Bars Ea. Way	Bar No.	Max Total Load (kips)
2	2	12	0.52	0.62	(2)	# 5	6.0
2.5	2.5	12	0.65	0.93	(3)	# 5	9.4
3	3	12	0.78	0.93	(3)	# 5	13.5
3.5	3.5	12	0.91	0.93	(3)	# 5	18.4
4	4	12	1.04	1.24	(4)	# 5	24.0
4.5	4.5	12	1.17	1.24	(4)	# 5	30.4
5	5	12	1.30	1.55	(5)	# 5	37.5
5.5	5.5	12	1.43	1.55	(5)	# 5	45.4
6	6	12	1.56	1.86	(6)	# 5	54.0
6.5	6.5	14	1.97	2.17	(7)	# 5	63.4
7	7	14	2.12	2.17	(7)	# 5	73.5
7.5	7.5	14	2.60	3.08	(7)	# 6	84.4
8	8	16	2.76	3.52	(8)	# 6	96.0
8.5	8.5	16	3.28	3.52	(8)	# 6	108.4
9	9	18	3.50	3.96	(9)	# 6	121.5
9.5	9.5	18	4.05	4.40	(10)	# 6	135.4
10	10	20	4.32	4.40	(10)	# 6	150.0
10.5	10.5	20	4.91	6.00	(10)	# 7	165.4
11	11	22	5.23	6.60	(11)	# 7	181.5
11.5	11.5	22	5.85	6.60	(11)	# 7	198.4
12	12	22	6.69	7.20	(12)	# 7	216.0

ARW ENGINEERS structural consultants

October 8, 2024
6:00 AM

Uplift Capacity of Exterior Concrete Footings

Version Date: January 27, 2010 Author: SLE Reviewed By: ZCH

JOB TITLE: Kimberly AG shop
DESCRIPTION: Footing Uplift Schedule @ existing wall line

JOB #: 24091
DESIGNED BY: ANB

Weight Factor for Design: 0.6

Slab-on Grade Parameters:
(See User Note #1)
Slab Thickness = 4 inches
Slab Percentage = 100%

Foundation Wall Parameters:
(See User Note #2)
Wall Thickness = 0 inches
Foundation Depth = 0 inches

Strip Footing Parameters:
(Strip footing below foundation wall)
Footing Width = 24 inches
Footing Thickness = 12 inches
inches

Additional Wall and Footing Length = 0 feet
(See User Note #3)

Soil Weight Parameters:
(See User Note #4)
Soil Weight = 115 pcf
Soil Depth = 8 inches

Concrete Weight 145 pcf

User Notes:

1- If a concrete slab-on-grade is present, the program will calculate and include a portion of the slab weight above the spot footing. The "Slab Percentage" cell is used to determine what percentage of the footing area is covered by the slab.

2- If a foundation wall is present, the program will calculate and include the weight of the wall that is directly above the spot footing.

3- Due to "beam action" from the foundation wall, a portion of the foundation wall and strip footing beyond the dimensions of the spot footing may be used. The distance to be included beyond each side of the spot footing is entered in the "Additional Wall and Footing Length" cell.

4- The program will calculate the weight of soil directly above the spot footing. Do not include the thickness of the slab in the soil depth.

NOTE: FOOTING THICKNESSES NEED TO BE ADJUSTED TO MATCH THE FOOTING SCHEDULE ON THE DRAWINGS

	Width (ft)	Area (sq. ft)	Thickness (in)	Volume (cu. ft)	Spot Ftg. Weight (lbs.)	Slab Weight (lbs.)	Foundation Wall Weight (lbs.)	Strip Ftg. Weight (lbs.)	Soil Weight (lbs.)	Total Weight (kips)	0.6x Wt. (kips)
F3.0	3	9	12	9	1305	435	0	0	690	***2.43***	***1.46***
F3.5	3.5	12.25	12	12	1776	592	0	0	939	***3.31***	***1.98***
F4.0	4	16	12	16	2320	773	0	0	1227	***4.32***	***2.59***
F4.5	4.5	20.25	12	20	2936	979	0	0	1553	***5.47***	***3.28***
F5.0	5	25	12	25	3625	1208	0	0	1917	***6.75***	***4.05***
F5.5	5.5	30.25	12	30	4386	1462	0	0	2319	***8.17***	***4.90***
F6.0	6	36	12	36	5220	1740	0	0	2760	***9.72***	***5.83***
F6.5	6.5	42.25	14	49	7147	2042	0	0	3239	***12.43***	***7.46***
F7.0	7	49	14	57	8289	2368	0	0	3757	***14.41***	***8.65***
F7.5	7.5	56.25	14	66	9516	2719	0	0	4313	***16.55***	***9.93***
F8.0	8	64	16	85	12373	3093	0	0	4907	***20.37***	***12.22***
F8.5	8.5	72.25	16	96	13968	3492	0	0	5539	***23.00***	***13.80***
F9.0	9	81	18	122	17618	3915	0	0	6210	***27.74***	***16.65***
F9.5	9.5	90.25	18	135	19629	4362	0	0	6919	***30.91***	***18.55***
F10.0	10	100	20	167	24167	4833	0	0	7667	***36.67***	***22.00***
F10.5	10.5	110.25	20	184	26644	5329	0	0	8453	***40.43***	***24.26***
F11.0	11	121	22	222	32166	5848	0	0	9277	***47.29***	***28.37***
F11.5	11.5	132.25	22	242	35156	6392	0	0	10139	***51.69***	***31.01***
F12.0	12	144	22	264	38280	6960	0	0	11040	***56.28***	***33.77***
F12.5	12.5	156.25	24	313	45313	7552	0	0	11979	***64.84***	***38.91***
F13.0	13	169	24	338	49010	8168	0	0	12957	***70.14***	***42.08***
F13.5	13.5	182.25	24	365	52853	8809	0	0	13973	***75.63***	***45.38***
F14.0	14	196	28	457	66313	9473	0	0	15027	***90.81***	***54.49***
F14.5	14.5	210.25	28	491	71135	10162	0	0	16119	***97.42***	***58.45***
F15.0	15	225	28	525	76125	10875	0	0	17250	***104.25***	***62.55***
F15.5	15.5	240.25	28	561	81285	11612	0	0	18419	***111.32***	***66.79***
F16.0	16	256	28	597	86613	12373	0	0	19627	***118.61***	***71.17***
F16.5	16.5	272.25	32	726	105270	13159	0	0	20873	***139.30***	***83.58***
F17.0	17	289	32	771	111747	13968	0	0	22157	***147.87***	***88.72***
F17.5	17.5	306.25	32	817	118417	14802	0	0	23479	***156.70***	***94.02***
F18.0	18	324	36	972	140940	15660	0	0	24840	***181.44***	***108.86***
F18.5	18.5	342.25	36	1027	148879	16542	0	0	26239	***191.66***	***115.00***

October 8, 2024

6:00 AM

Uplift Capacity of Exterior Concrete Footings

Version Date: January 27, 2010 Author: SLE Reviewed By: ZCH

JOB TITLE: Kimberly AG shop

DESCRIPTION: Footing Uplift Schedule - @ grids 2 & 3

JOB #: 24091

DESIGNED BY: ANB

Weight Factor for Design: 0.6

Slab-on Grade Parameters: Slab Thickness = 6 inches
(See User Note #1) Slab Percentage = 100%

Foundation Wall Parameters: Wall Thickness = 8 inches
(See User Note #2) Foundation Depth = 24 inches

Strip Footing Parameters: Footing Width = 36 inches
(Strip footing below foundation wall) Footing Thickness = 12 inches
inches

Additional Wall and Footing Length = 2 feet
(See User Note #3)

Soil Weight Parameters: Soil Weight = 115 pcf
(See User Note #4) Soil Depth = 18 inches

Concrete Weight 145 pcf

User Notes:

1- If a concrete slab-on-grade is present, the program will calculate and include a portion of the slab weight above the spot footing. The "Slab Percentage" cell is used to determine what percentage of the footing area is covered by the slab.

2- If a foundation wall is present, the program will calculate and include the weight of the wall that is directly above the spot footing.

3- Due to "beam action" from the foundation wall, a portion of the foundation wall and strip footing beyond the dimensions of the spot footing may be used. The distance to be included beyond each side of the spot footing is entered in the "Additional Wall and Footing Length" cell.

4- The program will calculate the weight of soil directly above the spot footing. Do not include the thickness of the slab in the soil depth.

NOTE: FOOTING THICKNESSES NEED TO BE ADJUSTED TO MATCH THE FOOTING SCHEDULE ON THE DRAWINGS

	Width (ft)	Area (sq. ft)	Thickness (in)	Volume (cu. ft)	Spot Ftg. Weight (lbs.)	Slab Weight (lbs.)	Foundation Wall Weight (lbs.)	Strip Ftg. Weight (lbs.)	Soil Weight (lbs.)	Total Weight (kips)	0.6x Wt. (kips)
F3.0	3	9	12	9	1305	653	1353	1740	1553	***6.60***	***3.96***
F3.5	3.5	12.25	12	12	1776	888	1450	1740	2113	***7.97***	***4.78***
F4.0	4	16	12	16	2320	1160	1547	1740	2760	***9.53***	***5.72***
F4.5	4.5	20.25	12	20	2936	1468	1643	1740	3493	***11.28***	***6.77***
F5.0	5	25	12	25	3625	1813	1740	1740	4313	***13.23***	***7.94***
F5.5	5.5	30.25	12	30	4386	2193	1837	1740	5218	***15.37***	***9.22***
F6.0	6	36	12	36	5220	2610	1933	1740	6210	***17.71***	***10.63***
F6.5	6.5	42.25	14	49	7147	3063	2030	1740	7288	***21.27***	***12.76***
F7.0	7	49	14	57	8289	3553	2127	1740	8453	***24.16***	***14.50***
F7.5	7.5	56.25	14	66	9516	4078	2223	1740	9703	***27.26***	***16.36***
F8.0	8	64	16	85	12373	4640	2320	1740	11040	***32.11***	***19.27***
F8.5	8.5	72.25	16	96	13968	5238	2417	1740	12463	***35.83***	***21.50***
F9.0	9	81	18	122	17618	5873	2513	1740	13973	***41.72***	***25.03***
F9.5	9.5	90.25	18	135	19629	6543	2610	1740	15568	***46.09***	***27.65***
F10.0	10	100	20	167	24167	7250	2707	1740	17250	***53.11***	***31.87***
F10.5	10.5	110.25	20	184	26644	7993	2803	1740	19018	***58.20***	***34.92***
F11.0	11	121	22	222	32166	8773	2900	1740	20873	***66.45***	***39.87***
F11.5	11.5	132.25	22	242	35156	9588	2997	1740	22813	***72.29***	***43.38***
F12.0	12	144	22	264	38280	10440	3093	1740	24840	***78.39***	***47.04***
F12.5	12.5	156.25	24	313	45313	11328	3190	1740	26953	***88.52***	***53.11***
F13.0	13	169	24	338	49010	12253	3287	1740	29153	***95.44***	***57.27***
F13.5	13.5	182.25	24	365	52853	13213	3383	1740	31438	***102.63***	***61.58***
F14.0	14	196	28	457	66313	14210	3480	1740	33810	***119.55***	***71.73***
F14.5	14.5	210.25	28	491	71135	15243	3577	1740	36268	***127.96***	***76.78***
F15.0	15	225	28	525	76125	16313	3673	1740	38813	***136.66***	***82.00***
F15.5	15.5	240.25	28	561	81285	17418	3770	1740	41443	***145.66***	***87.39***
F16.0	16	256	28	597	86613	18560	3867	1740	44160	***154.94***	***92.96***
F16.5	16.5	272.25	32	726	105270	19738	3963	1740	46963	***177.67***	***106.60***
F17.0	17	289	32	771	111747	20953	4060	1740	49853	***188.35***	***113.01***
F17.5	17.5	306.25	32	817	118417	22203	4157	1740	52828	***199.34***	***119.61***
F18.0	18	324	36	972	140940	23490	4253	1740	55890	***226.31***	***135.79***
F18.5	18.5	342.25	36	1027	148879	24813	4350	1740	59038	***238.82***	***143.29***

October 8, 2024
6:00 AM

ARW ENGINEERS structural consultants

Uplift Capacity of Interior Concrete Footings

Version Date: January 27, 2010 Author: SLE Reviewed By: ZCH

JOB TITLE: Kimberly AG shop
DESCRIPTION: Interior Footing Uplift Capacity

JOB #: 24091
DESIGNED BY: ANB

Weight Factor for Design: 0.6

Slab-on Grade Parameters:
(See User Note #1)
Slab Thickness = 6 inches
Slab Percentage = 100%

Soil Weight Parameters:
(See User Note #2)
Soil Weight = 115 pcf
Soil Depth = 4 inches

Concrete Weight 145 pcf

User Notes:

1- If a concrete slab-on-grade is present, the program will calculate and include a portion of the slab weight above the spot footing. The "Slab Percentage" cell is used to determine what percentage of the footing area is covered by the slab.

2- The program will calculate the weight of soil directly above the spot footing. Do not include the thickness of the slab in the soil depth.

NOTE: FOOTING THICKNESSES NEED TO BE ADJUSTED ON THE "EXTERIOR" FOOTING SHEET TO MATCH THE FOOTING SCHEDULE ON THE DRAWINGS

	Width (ft)	Area (sq. ft)	Thickness (in)	Volume (cu. ft)	Spot Ftg. Weight (lbs.)	Slab Weight (lbs.)	Soil Weight (lbs.)	Total Weight (kips)	0.6x Wt. (kips)
F3.0	3	9	12	9	1305	652.5	345	***2.30***	***1.38***
F3.5	3.5	12.25	12	12	1776.25	888.125	470	***3.13***	***1.88***
F4.0	4	16	12	16	2320	1160	613	***4.09***	***2.46***
F4.5	4.5	20.25	12	20	2936.25	1468.125	776	***5.18***	***3.11***
F5.0	5	25	12	25	3625	1812.5	958	***6.40***	***3.84***
F5.5	5.5	30.25	12	30	4386.25	2193.125	1160	***7.74***	***4.64***
F6.0	6	36	12	36	5220	2610	1380	***9.21***	***5.53***
F6.5	6.5	42.25	14	49	7147.291667	3063.125	1620	***11.83***	***7.10***
F7.0	7	49	14	57	8289.166667	3552.5	1878	***13.72***	***8.23***
F7.5	7.5	56.25	14	66	9515.625	4078.125	2156	***15.75***	***9.45***
F8.0	8	64	16	85	12373.33333	4640	2453	***19.47***	***11.68***
F8.5	8.5	72.25	16	96	13968.33333	5238.125	2770	***21.98***	***13.19***
F9.0	9	81	18	122	17617.5	5872.5	3105	***26.60***	***15.96***
F9.5	9.5	90.25	18	135	19629.375	6543.125	3460	***29.63***	***17.78***
F10.0	10	100	20	167	24166.66667	7250	3833	***35.25***	***21.15***
F10.5	10.5	110.25	20	184	26643.75	7993.125	4226	***38.86***	***23.32***
F11.0	11	121	22	222	32165.83333	8772.5	4638	***45.58***	***27.35***
F11.5	11.5	132.25	22	242	35156.45833	9588.125	5070	***49.81***	***29.89***
F12.0	12	144	22	264	38280	10440	5520	***54.24***	***32.54***
F12.5	12.5	156.25	24	313	45312.5	11328.125	5990	***62.63***	***37.58***
F13.0	13	169	24	338	49010	12252.5	6478	***67.74***	***40.64***
F13.5	13.5	182.25	24	365	52852.5	13213.125	6986	***73.05***	***43.83***
F14.0	14	196	28	457	66313.33333	14210	7513	***88.04***	***52.82***
F14.5	14.5	210.25	28	491	71134.58333	15243.125	8060	***94.44***	***56.66***
F15.0	15	225	28	525	76125	16312.5	8625	***101.06***	***60.64***
F15.5	15.5	240.25	28	561	81284.58333	17418.125	9210	***107.91***	***64.75***
F16.0	16	256	28	597	86613.33333	18560	9813	***114.99***	***68.99***
F16.5	16.5	272.25	32	726	105270	19738.125	10436	***135.44***	***81.27***
F17.0	17	289	32	771	111746.6667	20952.5	11078	***143.78***	***86.27***
F17.5	17.5	306.25	32	817	118416.6667	22203.125	11740	***152.36***	***91.42***
F18.0	18	324	36	972	140940	23490	12420	***176.85***	***106.11***
F18.5	18.5	342.25	36	1027	148878.75	24813.125	13120	***186.81***	***112.09***

Cantilevered Retaining Wall

Project File: 24091 - kimberly ag shop.ec6

LIC# : KW-06015770, Build:20.23.10.02 ARW ENGINEERS (c) ENERCALC INC 1983-2023

DESCRIPTION: Cantilevered wall design/footing

Code References

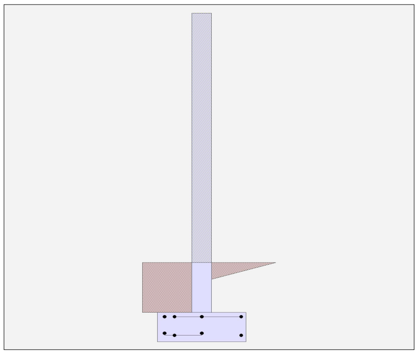
Calculations per IBC 2021 1807.3, ASCE 7-16

Criteria

Retained Height	=	2.00 ft
Wall height above soil	=	10.00 ft
Slope Behind Wall	=	0.00
Height of Soil over Toe	=	24.00 in
Water table above bottom of footing	=	0.0 ft

Soil Data

Allow Soil Bearing	=	2,000.0 psf
Equivalent Fluid Pressure Method		
Active Heel Pressure	=	35.0 psf/ft
	=	
Passive Pressure	=	250.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe	=	110.00 pcf
Footing||Soil Friction	=	0.400
Soil height to ignore for passive pressure	=	12.00 in



Surcharge Loads

Surcharge Over Heel	=	100.0 psf
NOT Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	100.0 psf
NOT Used for Sliding & Overturning		

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
Load Type	=	Wind (W) (Service Level)
Wind on Exposed Stem (Strength Level)	=	27.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Spread Footing
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Cantilevered Retaining Wall

Project File: 24091 - kimberly ag shop.ec6

LIC# : KW-06015770, Build:20.23.10.02 ARW ENGINEERS (c) ENERCALC INC 1983-2023

DESCRIPTION: Cantilevered wall design/footing

Design Summary

Wall Stability Ratios

Overturning	=	1.82	OK
Sliding	=	4.42	OK
Global Stability	=	4.55	
Total Bearing Load	=	2,018 lbs	
...resultant ecc.	=	9.92 in	

Eccentricity outside middle third

Soil Pressure @ Toe	=	1,997 psf	OK
Soil Pressure @ Heel	=	0 psf	OK
Allowable	=	2,000 psf	

Soil Pressure Less Than Allowable

ACI Factored @ Toe	=	3,119 psf	
ACI Factored @ Heel	=	0 psf	
Footing Shear @ Toe	=	5.0 psi	OK
Footing Shear @ Heel	=	5.2 psi	OK
Allowable	=	82.2 psi	

Sliding Calcs

Lateral Sliding Force	=	438.2 lbs	
less 100% Passive Force	-	1,128.5 lbs	
less 100% Friction Force	= -	807.3 lbs	
Added Force Req'd	=	0.0 lbs	OK
....for 1.5 Stability	=	0.0 lbs	OK

Vertical component of active lateral soil pressure IS NOT considered in the calculation of soil bearing

Load Factors

Building Code	
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

Stem Construction

		2nd	Bottom		
		Stem OK	Stem OK		
Design Height Above Ftg	ft =	2.00	0.00		
Wall Material Above "Ht"	=	Masonry	Concrete		
Design Method	=	ASD	SD	SD	SD
Thickness	=	8.00	8.00		
Rebar Size	=	# 5	# 5		
Rebar Spacing	=	32.00	18.00		
Rebar Placed at	=	Center	Center		
Design Data					
fb/FB + fa/Fa	=	**0.747**	**0.906**		
Total Force @ Section					
Service Level	lbs =	162.0			
Strength Level	lbs =		645.8		
Moment....Actual					
Service Level	ft-# =	810.0			
Strength Level	ft-# =		3,200.5		
Moment.....Allowable	ft-# =	1,084.1	3,531.0		
Shear.....Actual					
Service Level	psi =	1.8			
Strength Level	psi =		13.5		
Shear.....Allowable	psi =	52.9	53.5		
Anet (Masonry)	in2 =	91.50			
Wall Weight	psf =	78.0	100.0		
Rebar Depth 'd'	in =	3.81	4.00		
Masonry Data					
f'm	psi =	2,000			
Fs	psi =	32,000			
Solid Grouting	=	Yes			
Modular Ratio 'n'	=	16.11			
Equiv. Solid Thick.	in =	7.63			
Masonry Block Type	=				
Masonry Design Method	=	ASD			
Concrete Data					
f'c	psi =		3,000.0		
Fy	psi =		60,000.0		

Cantilevered Retaining Wall

Project File: 24091 - kimberly ag shop.ec6

DESCRIPTION: Cantilevered wall design/footing

Concrete Stem Rebar Area Details

Bottom Stem	Vertical Reinforcing	Horizontal Reinforcing	
As (based on applied moment) :	0.1932 in2/ft		
0.0018bh : 0.0018(12)(8) :	0.1728 in2/ft	Horizontal Reinforcing Options :	
	============	One layer of :	Two layers of :
Required Area :	0.1932 in2/ft	#4@ 13.89 in	#4@ 27.78 in
Provided Area :	0.2067 in2/ft	#5@ 21.53 in	#5@ 43.06 in
Maximum Area :	0.6503 in2/ft	#6@ 30.56 in	#6@ 61.11 in

Footing Data

Toe Width	=	1.17 ft
Heel Width	=	1.83
Total Footing Width	=	3.00
Footing Thickness	=	14.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft

f'c = 3,000 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top 2.00 @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel		
Factored Pressure	=	3,119	0	psf	
Mu' : Upward	=	1,714	2	ft-#	
Mu' : Downward	=	431	431	ft-#	
Mu: Design	=	1,283 OK	430	ft-#	OK
phiMn	=	14,485	15,880	ft-#	
Actual 1-Way Shear	=	5.03	5.16	psi	
Allow 1-Way Shear	=	82.16	82.16	psi	
Toe Reinforcing	=	# 5 @ 12.00 in			
Heel Reinforcing	=	# 5 @ 12.00 in			
Key Reinforcing	=	None Spec'd			

Footing Torsion, Tu = 0.00 ft-lbs
Footing Allow. Torsion, phi Tu = 0.00 ft-lbs

If torsion exceeds allowable, provide supplemental design for footing torsion.

Other Acceptable Sizes & Spacings

Toe: #4@ 7.93 in, #5@ 12.30 in, #6@ 17.46 in, #7@ 23.80 in, #8@ 31.34 in, #9@ 39.68 in, #10@ 50.39 in

Heel: #4@ 7.93 in, #5@ 12.30 in, #6@ 17.46 in, #7@ 23.80 in, #8@ 31.34 in, #9@ 39.68 in, #10@ 50.39 in

Key: No key defined

Min footing T&S reinf Area	0.91	in2
Min footing T&S reinf Area per foot	0.30	in2 /ft

If one layer of horizontal bars:	If two layers of horizontal bars:
#4@ 7.94 in	#4@ 15.87 in
#5@ 12.30 in	#5@ 24.60 in
#6@ 17.46 in	#6@ 34.92 in

Cantilevered Retaining Wall

DESCRIPTION: Cantilevered wall design/footing

Summary of Overturning & Resisting Forces & Moments

.....OVERTURNING.....

Item		Force lbs	Distance ft	Moment ft-#
HL Act Pres (ab water tbl)		175.5	1.06	185.2
HL Act Pres (be water tbl)				
Hydrostatic Force				
Buoyant Force	=			
Surcharge over Heel	=	100.8	1.58	159.5
Surcharge Over Toe	=			
Adjacent Footing Load	=			
Added Lateral Load	=			
Load @ Stem Above Soil	=	162.0	8.17	1,323.0
	=			
Total	=	438.2	**O.T.M.** =	1,667.8

Resisting/Overturning Ratio = **1.82**
Vertical Loads used for Soil Pressure = 2,018.3 lbs

.....RESISTING.....

Item		Force lbs	Distance ft	Moment ft-#
Soil Over HL (ab. water tbl)		256.7	2.42	620.3
Soil Over HL (bel. water tbl)			2.42	620.3
Water Table				
Sloped Soil Over Heel	=			
Surcharge Over Heel	=			
Adjacent Footing Load	=			
Axial Dead Load on Stem	=			
* Axial Live Load on Stem	=			
Soil Over Toe	=	256.7	0.58	149.7
Surcharge Over Toe	=			
Stem Weight(s)	=	980.0	1.50	1,470.0
Earth @ Stem Transitions	=			
Footing Weight	=	525.0	1.50	787.5
Key Weight	=			
Vert. Component	=			
Total =		2,018.3 lbs	**R.M.=**	3,027.5

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

Vertical component of active lateral soil pressure IS NOT considered in the calculation of Sliding Resistance.

Vertical component of active lateral soil pressure IS NOT considered in the calculation of Overturning Resistance.

Tilt

Horizontal Deflection at Top of Wall due to settlement of soil

(Deflection due to wall bending not considered)

Soil Spring Reaction Modulus	250.0	pci
Horizontal Defl @ Top of Wall (approximate only)	0.222	in

The above calculation is not valid if the heel soil bearing pressure exceeds that of the toe, because the wall would then tend to rotate into the retained soil.

Cantilevered Retaining Wall

Project File: 24091 - kimberly ag shop.ec6

DESCRIPTION: Cantilevered wall design/footing

Rebar Lap & Embedment Lengths Information

Stem Design Segment: 2nd

Stem Design Height: 2.00 ft above top of footing

Calculated Rebar Stress, fs = 23908.62 psi

Lap Splice length for #5 bar specified in this stem design segment (25.4.2.4a) =	29.89 in
Development length for #5 bar specified in this stem design segment =	29.89 in

---

Stem Design Segment: Bottom

Stem Design Height: 0.00 ft above top of footing

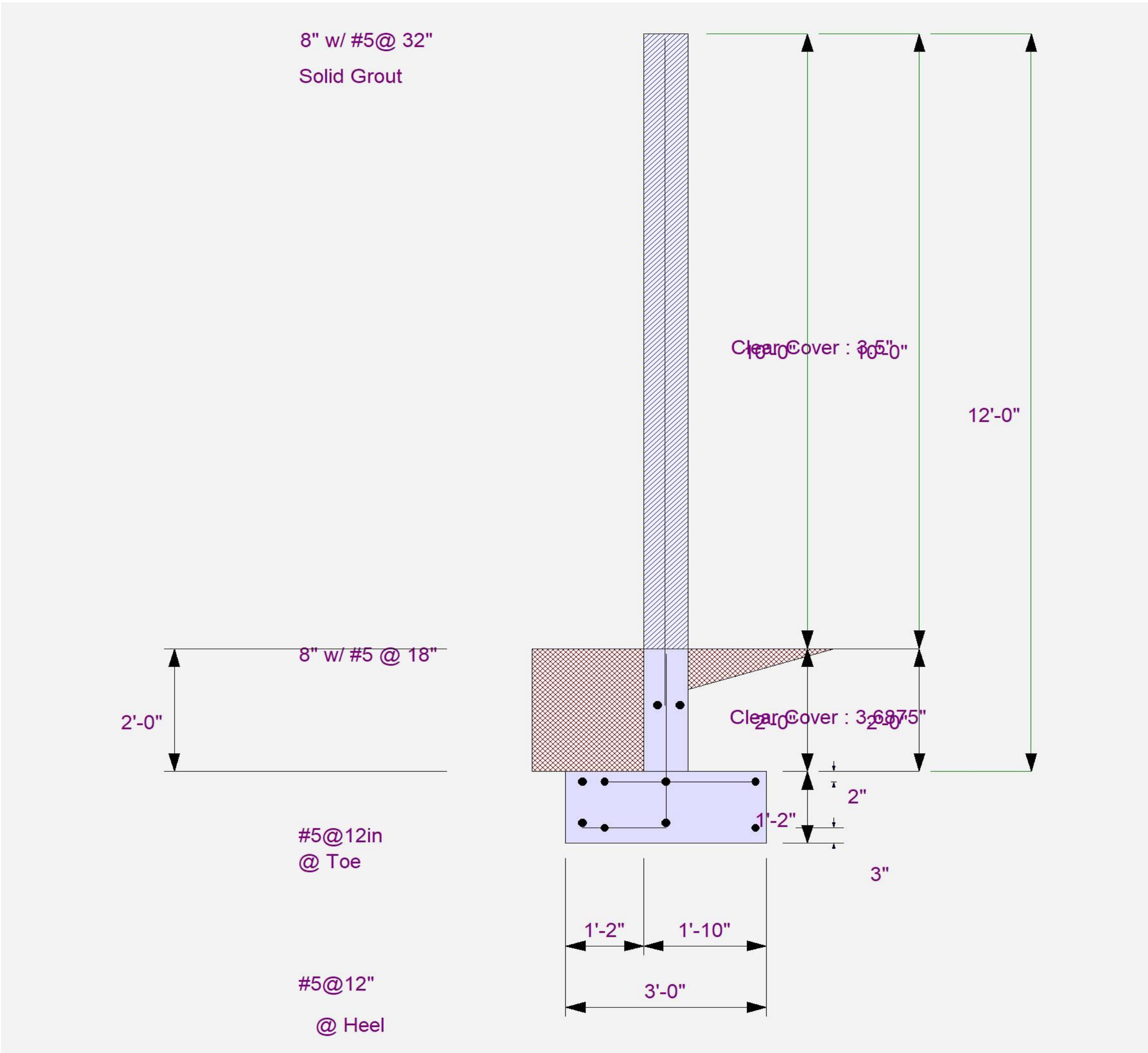
Lap Splice length for #5 bar specified in this stem design segment (25.4.2.4a) =	21.36 in
Development length for #5 bar specified in this stem design segment =	16.43 in
Hooked embedment length into footing for #5 bar specified in this stem design segment =	7.87 in
As Provided =	0.2067 in2/ft
As Required =	0.1932 in2/ft

Cantilevered Retaining Wall

Project File: 24091 - kimberly ag shop.ec6

LIC# : KW-06015770, Build:20.23.10.02 ARW ENGINEERS (c) ENERCALC INC 1983-2023

DESCRIPTION: Cantilevered wall design/footing

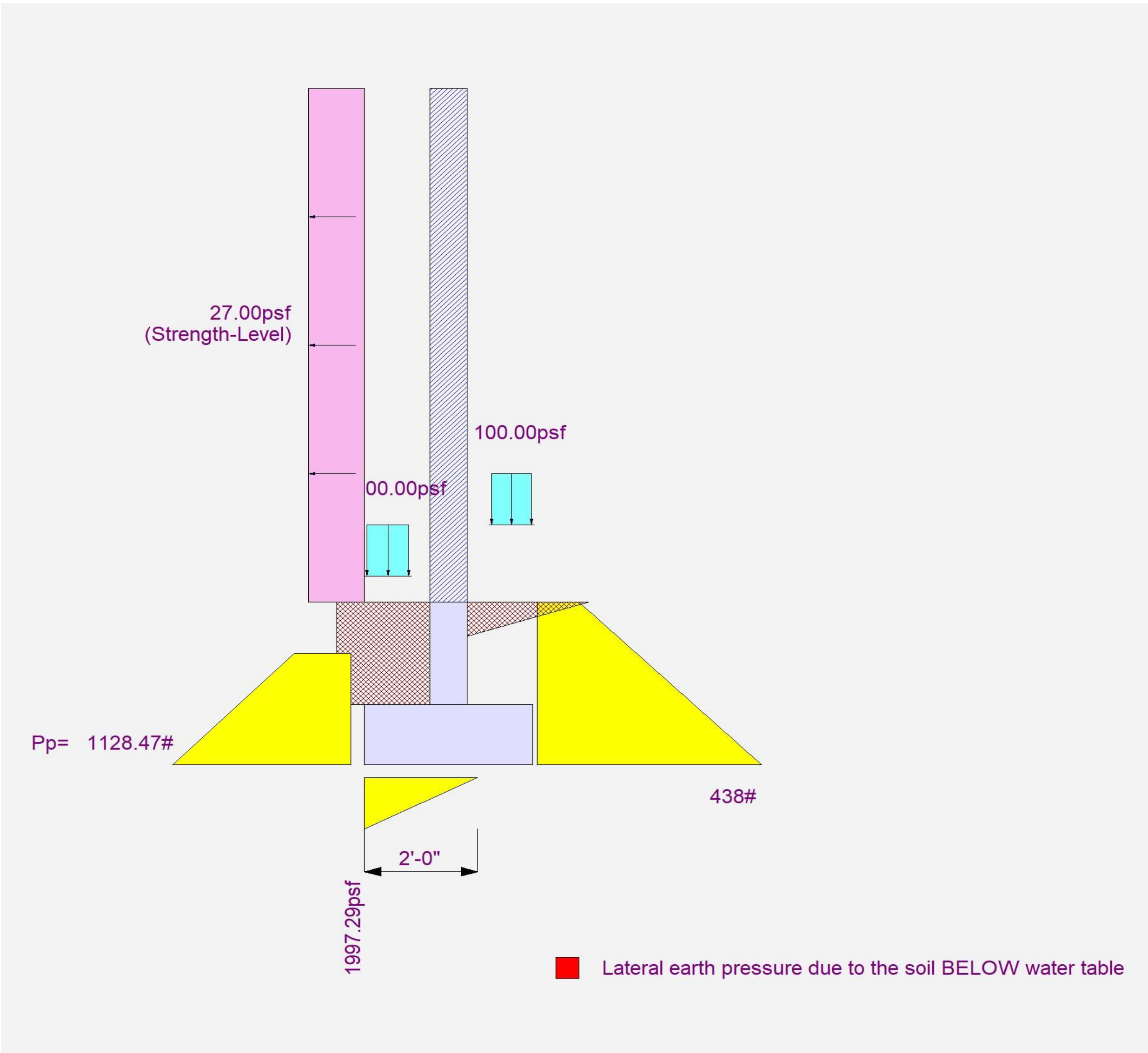


Cantilevered Retaining Wall

Project File: 24091 - kimberly ag shop.ec6

LIC# : KW-06015770, Build:20.23.10.02 ARW ENGINEERS

DESCRIPTION: Cantilevered wall design/footing



Cantilevered Retaining Wall

Project File: 24091 - kimberly ag shop.ec6

LIC# : KW-06015770, Build:20.23.10.02 ARW ENGINEERS (c) ENERCALC INC 1983-2023

DESCRIPTION: Cantilevered wall design/footing - interior wall

Code References

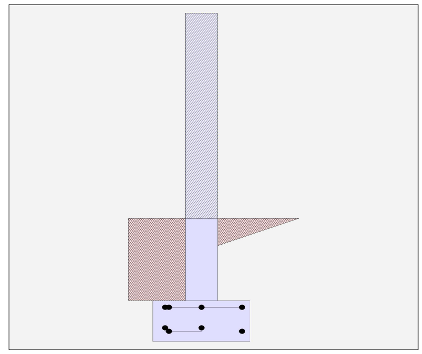
Calculations per IBC 2021 1807.3, ASCE 7-16

Criteria

Retained Height	=	2.00 ft
Wall height above soil	=	5.00 ft
Slope Behind Wall	=	0.00
Height of Soil over Toe	=	24.00 in
Water table above bottom of footing	=	0.0 ft

Soil Data

Allow Soil Bearing	=	2,000.0 psf
Equivalent Fluid Pressure Method		
Active Heel Pressure	=	35.0 psf/ft
	=	
Passive Pressure	=	250.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe	=	110.00 pcf
Footing||Soil Friction	=	0.400
Soil height to ignore for passive pressure	=	12.00 in



Surcharge Loads

Surcharge Over Heel = 100.0 psf
NOT Used To Resist Sliding & Overturning
Surcharge Over Toe = 100.0 psf
NOT Used for Sliding & Overturning

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Lateral Load Applied to Stem

Lateral Load	=	6.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
Load Type	=	Seismic (E) (Strength Level)
Wind on Exposed Stem (Service Level)	=	0.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Spread Footing
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Cantilevered Retaining Wall

Project File: 24091 - kimberly ag shop.ec6

LIC# : KW-06015770, Build:20.23.10.02 ARW ENGINEERS (c) ENERCALC INC 1983-2023

DESCRIPTION: Cantilevered wall design/footing - interior wall

Design Summary

Wall Stability Ratios

Overturning	=	3.94	OK
Sliding	=	5.82	OK
Global Stability	=	5.27	

Total Bearing Load	=	1,183 lbs	
...resultant ecc.	=	3.05 in	

Eccentricity within middle third

Soil Pressure @ Toe	=	1,043 psf	OK
Soil Pressure @ Heel	=	141 psf	OK
Allowable	=	2,000 psf	

Soil Pressure Less Than Allowable

ACI Factored @ Toe	=	1,624 psf	
ACI Factored @ Heel	=	219 psf	
Footing Shear @ Toe	=	0.3 psi	OK
Footing Shear @ Heel	=	0.9 psi	OK
Allowable	=	82.2 psi	

Sliding Calcs

Lateral Sliding Force	=	253.0 lbs	
less 100% Passive Force	-	1,000.0 lbs	
less 100% Friction Force	= -	473.3 lbs	
Added Force Req'd	=	0.0 lbs	OK
....for 1.5 Stability	=	0.0 lbs	OK

Vertical component of active lateral soil pressure IS NOT considered in the calculation of soil bearing

Load Factors

Building Code	
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

Stem Construction

		2nd	Bottom		
		Stem OK	Stem OK		
Design Height Above Ftg	ft =	2.00	0.00		
Wall Material Above "Ht"	=	Masonry	Concrete		
Design Method	=	ASD	SD	SD	SD
Thickness	=	8.00	8.00		
Rebar Size	=	# 5	# 5		
Rebar Spacing	=	32.00	18.00		
Rebar Placed at	=	Center	Center		
Design Data					
fb/FB + fa/Fa	=	**0.000**	**0.049**		
Total Force @ Section					
Service Level	lbs =				
Strength Level	lbs =		213.8		
Moment....Actual					
Service Level	ft-# =				
Strength Level	ft-# =		176.5		
Moment.....Allowable	ft-# =	1,084.1	3,531.0		
Shear.....Actual					
Service Level	psi =				
Strength Level	psi =		4.5		
Shear.....Allowable	psi =	90.7	53.5		
Anet (Masonry)	in2 =	91.50			
Wall Weight	psf =	78.0	100.0		
Rebar Depth 'd'	in =	3.81	4.00		
Masonry Data					
f'm	psi =	2,000			
Fs	psi =	32,000			
Solid Grouting	=	Yes			
Modular Ratio 'n'	=	16.11			
Equiv. Solid Thick.	in =	7.63			
Masonry Block Type	=				
Masonry Design Method	=	ASD			
Concrete Data					
f'c	psi =		3,000.0		
Fy	psi =		60,000.0		

Cantilevered Retaining Wall

Project File: 24091 - kimberly ag shop.ec6

DESCRIPTION: Cantilevered wall design/footing - interior wall

Concrete Stem Rebar Area Details

Bottom Stem	Vertical Reinforcing	Horizontal Reinforcing	
As (based on applied moment) :	0.0107 in2/ft		
0.0018bh : 0.0018(12)(8) :	0.1728 in2/ft	Horizontal Reinforcing Options :	
	============	One layer of :	Two layers of :
Required Area :	0.1728 in2/ft	#4@ 13.89 in	#4@ 27.78 in
Provided Area :	0.2067 in2/ft	#5@ 21.53 in	#5@ 43.06 in
Maximum Area :	0.6503 in2/ft	#6@ 30.56 in	#6@ 61.11 in

Footing Data

Toe Width	=	0.67 ft
Heel Width	=	1.33
Total Footing Width	=	2.00
Footing Thickness	=	12.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft

f'c = 3,000 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top 2.00 @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel	
Factored Pressure	=	1,624	219 psf	
Mu' : Upward	=	326	83 ft-#	
Mu' : Downward	=	134	134 ft-#	
Mu: Design	=	192 OK	51 ft-#	OK
phiMn	=	11,695	13,090 ft-#	
Actual 1-Way Shear	=	0.28	0.88 psi	
Allow 1-Way Shear	=	82.16	82.16 psi	
Toe Reinforcing	=	# 5 @ 12.00 in		
Heel Reinforcing	=	# 5 @ 12.00 in		
Key Reinforcing	=	None Spec'd		

Footing Torsion, Tu = 0.00 ft-lbs
Footing Allow. Torsion, phi Tu = 0.00 ft-lbs

If torsion exceeds allowable, provide supplemental design for footing torsion.

Other Acceptable Sizes & Spacings

Toe: #4@ 9.25 in, #5@ 14.35 in, #6@ 20.37 in, #7@ 27.77 in, #8@ 36.57 in, #9@ 46.29 in, #10@ 58.79 in

Heel: #4@ 9.25 in, #5@ 14.35 in, #6@ 20.37 in, #7@ 27.77 in, #8@ 36.57 in, #9@ 46.29 in, #10@ 58.79 in

Key: No key defined

Min footing T&S reinf Area 0.52 in2
Min footing T&S reinf Area per foot 0.26 in2 /ft

If one layer of horizontal bars:	If two layers of horizontal bars:
#4@ 9.26 in	#4@ 18.52 in
#5@ 14.35 in	#5@ 28.70 in
#6@ 20.37 in	#6@ 40.74 in

Cantilevered Retaining Wall

Project File: 24091 - kimberly ag shop.ec6

LIC# : KW-06015770, Build:20.23.10.02 ARW ENGINEERS (c) ENERCALC INC 1983-2023

DESCRIPTION: Cantilevered wall design/footing - interior wall

Summary of Overturning & Resisting Forces & Moments

		.....OVERTURNING.....		
Item		**Force** lbs	**Distance** ft	**Moment** ft-#
HL Act Pres (ab water tbl)		157.5	1.00	157.5
HL Act Pres (be water tbl)				
Hydrostatic Force				
Buoyant Force	=			
Surcharge over Heel	=	95.5	1.50	143.2
Surcharge Over Toe	=			
Adjacent Footing Load	=			
Added Lateral Load	=		1.00	
Load @ Stem Above Soil	=			
	=			
Total	=	253.0	**O.T.M.** =	300.7

Resisting/Overturning Ratio = **3.94**
Vertical Loads used for Soil Pressure = 1,183.3 lbs

		.....RESISTING.....		
		Force lbs	**Distance** ft	**Moment** ft-#
Soil Over HL (ab. water tbl)		146.7	1.67	244.4
Soil Over HL (bel. water tbl)			1.67	244.4
Water Table				
Sloped Soil Over Heel	=			
Surcharge Over Heel	=			
Adjacent Footing Load	=			
Axial Dead Load on Stem	=			
* Axial Live Load on Stem	=			
Soil Over Toe	=	146.7	0.33	48.9
Surcharge Over Toe	=			
Stem Weight(s)	=	590.0	1.00	590.0
Earth @ Stem Transitions	=			
Footing Weight	=	300.0	1.00	300.0
Key Weight	=			
Vert. Component	=			
Total =		1,183.3 lbs	**R.M.=**	1,183.3

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

Vertical component of active lateral soil pressure IS NOT considered in the calculation of Sliding Resistance.

Vertical component of active lateral soil pressure IS NOT considered in the calculation of Overturning Resistance.

Tilt

Horizontal Deflection at Top of Wall due to settlement of soil

(Deflection due to wall bending not considered)

Soil Spring Reaction Modulus	250.0	pci
Horizontal Defl @ Top of Wall (approximate only)	0.101	in

The above calculation is not valid if the heel soil bearing pressure exceeds that of the toe, because the wall would then tend to rotate into the retained soil.

Cantilevered Retaining Wall

Project File: 24091 - kimberly ag shop.ec6

LIC# : KW-06015770, Build:20.23.10.02 ARW ENGINEERS

DESCRIPTION: Cantilevered wall design/footing - interior wall

Rebar Lap & Embedment Lengths Information

Stem Design Segment: 2nd
Stem Design Height: 2.00 ft above top of footing
Calculated Rebar Stress, fs = 0.00 psi

Lap Splice length for #5 bar specified in this stem design segment (25.4.2.4a) =	25.00 in
Development length for #5 bar specified in this stem design segment =	12.00 in

---

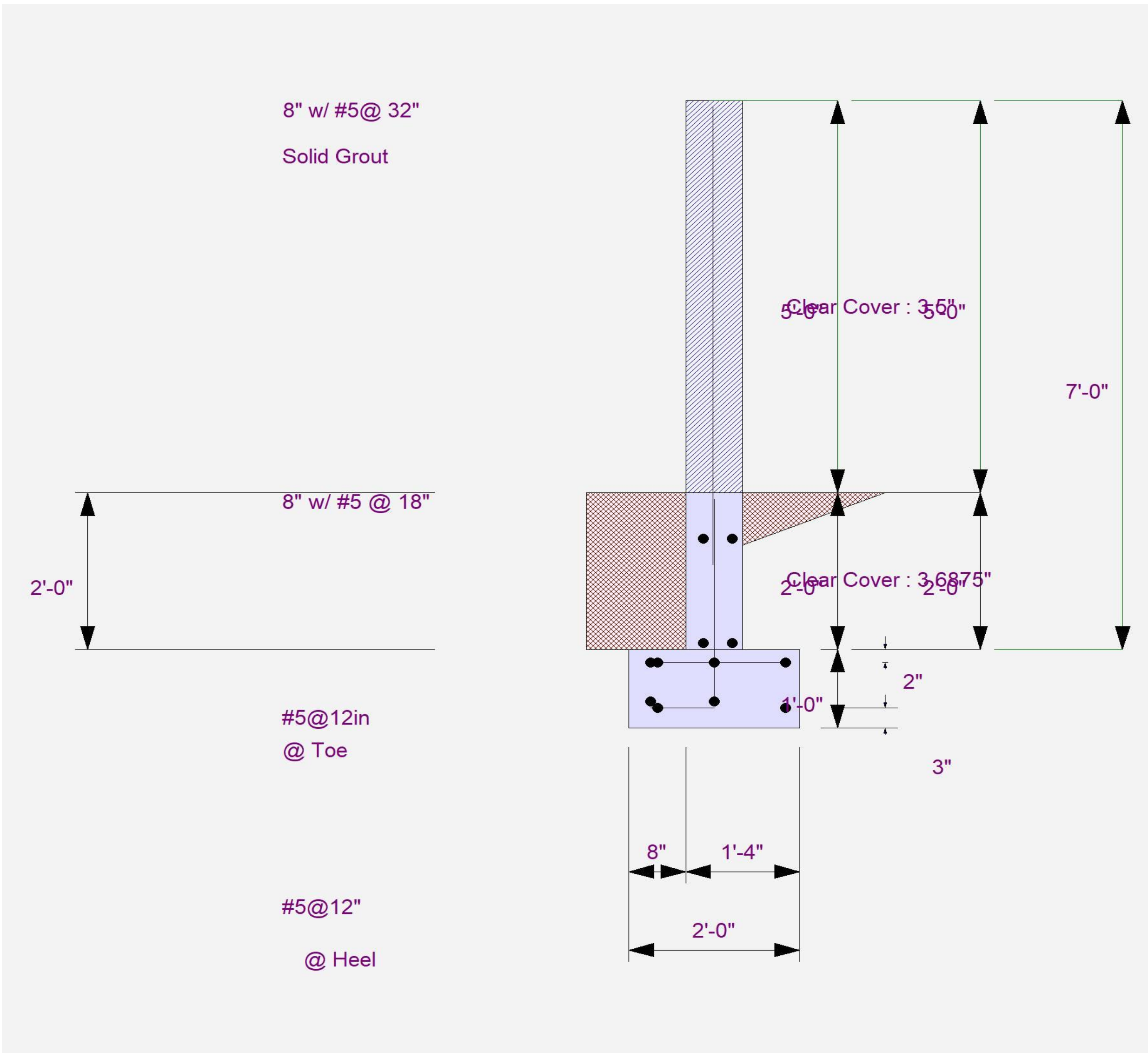
Stem Design Segment: Bottom
Stem Design Height: 0.00 ft above top of footing

Lap Splice length for #5 bar specified in this stem design segment (25.4.2.4a) =	21.36 in
Development length for #5 bar specified in this stem design segment =	16.43 in
Hooked embedment length into footing for #5 bar specified in this stem design segment =	7.87 in
As Provided =	0.2067 in2/ft
As Required =	0.1728 in2/ft

Cantilevered Retaining Wall

Project File: 24091 - kimberly ag shop.ec6

DESCRIPTION: Cantilevered wall design/footing - interior wall

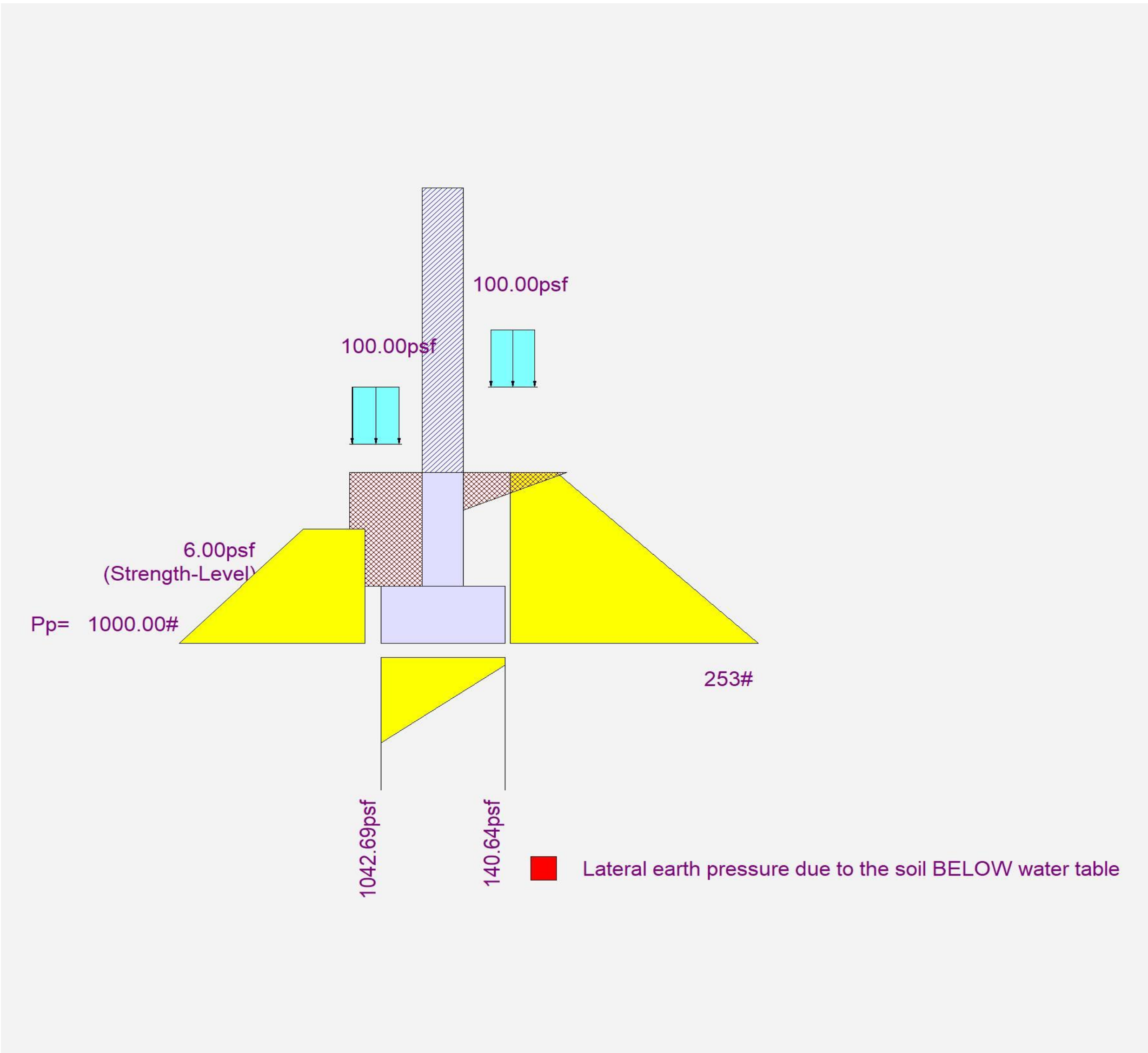


Cantilevered Retaining Wall

Project File: 24091 - kimberly ag shop.ec6

LIC# : KW-06015770, Build:20.23.10.02 ARW ENGINEERS (c) ENERCALC INC 1983-2023

DESCRIPTION: Cantilevered wall design/footing - interior wall



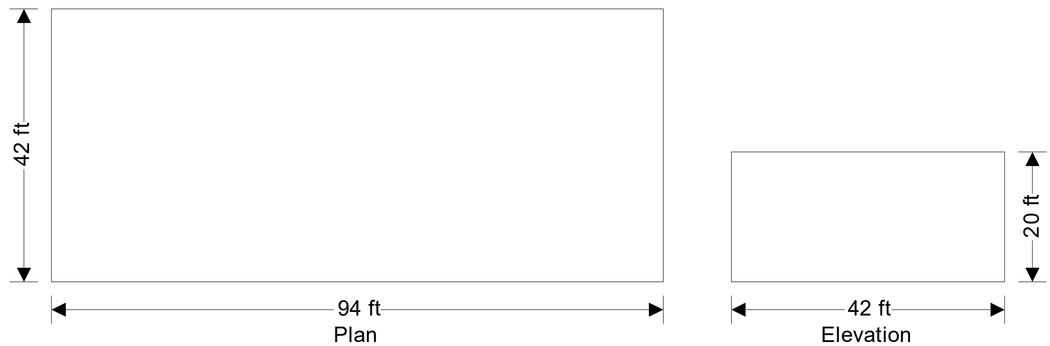
WALLS

WIND LOADING

In accordance with ASCE7-16

Using the components and cladding design method

Tedds calculation version 2.1.14



Building data

Type of roof	Flat
Length of building	b = **94.00** ft
Width of building	d = **42.00** ft
Height to eaves	H = **20.00** ft
Mean height	h = **20.00** ft
End zone width	a = max(min(0.1×min(b, d), 0.4×h), 0.04×min(b, d), 3ft) = **4.20** ft

General wind load requirements

Basic wind speed	V = **109.0** mph
Risk category	III
Velocity pressure exponent coef (Table 26.6-1)	K_d = **0.85**
Ground elevation above sea level	z_{gl} = **3900** ft
Ground elevation factor	$K_e = \exp(-0.0000362 \times z_{gl}/1\text{ft})$ = **0.87**
Exposure category (cl 26.7.3)	C
Enclosure classification (cl.26.12)	Enclosed buildings
Internal pressure coef +ve (Table 26.13-1)	GC_{pi_p} = **0.18**
Internal pressure coef –ve (Table 26.13-1)	GC_{pi_n} = **-0.18**
Gust effect factor	G_f = **0.85**

Topography

Topography factor not significant	K_{zt} = 1.0

Velocity pressure

Velocity pressure coefficient (Table 26.10-1)	K_z = **0.90**
Velocity pressure	$q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1\text{psf/mph}^2$ = **20.2** psf

Peak velocity pressure for internal pressure

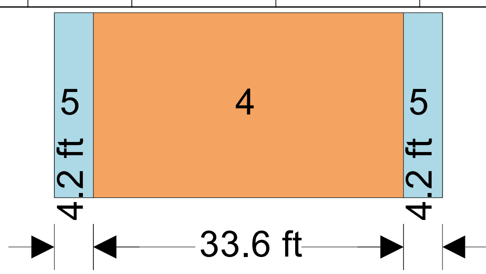
Peak velocity pressure – internal (as roof press.)	q_i = **20.20** psf

Equations used in tables

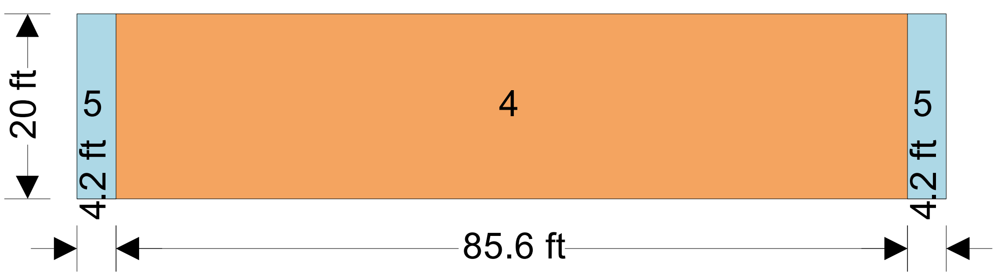
Net pressure	$p = q_h \times [GC_p - GC_{pi}]$

Components and cladding pressures - Wall (Table 30.3-1)

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
<=10 sf	4	-	-	10.0	0.90	-0.99	21.8	-23.6
50 sf	4	-	-	50.0	0.79	-0.88	19.6	-21.4
200 sf	4	-	-	200.0	0.69	-0.78	17.6	-19.5
>500 sf	4	-	-	500.1	0.63	-0.72	16.4	-18.2
<=10 sf	5	-	-	10.0	0.90	-1.26	21.8	-29.1
50 sf	5	-	-	50.0	0.79	-1.04	19.6	-24.6
200 sf	5	-	-	200.0	0.69	-0.85	17.6	-20.7
>500 sf	5	-	-	500.1	0.63	-0.72	16.4	-18.2



Elevation of gable wall



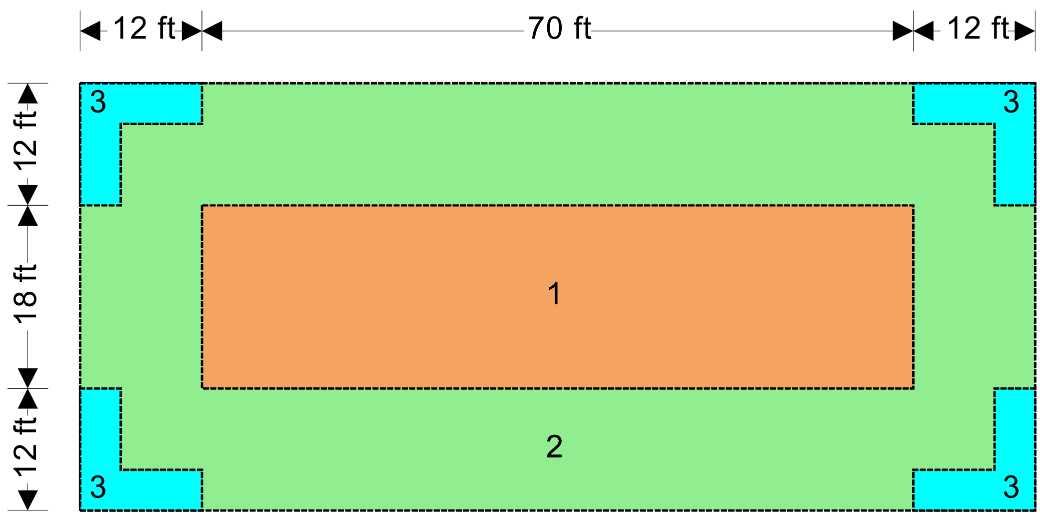
Elevation of side wall

Components and cladding pressures - Roof (Figure 30.3-2A)

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
<=10 sf	1	-	-	10.0	0.30	-1.70	9.7 #	-38.0
100 sf	1	-	-	100.0	0.20	-1.29	7.7 #	-29.7
200 sf	1	-	-	200.0	0.20	-1.16	7.7 #	-27.2
>500 sf	1	-	-	500.1	0.20	-1.00	7.7 #	-23.8
<=10 sf	2	-	-	10.0	0.30	-2.30	9.7 #	-50.1
100 sf	2	-	-	100.0	0.20	-1.77	7.7 #	-39.4
200 sf	2	-	-	200.0	0.20	-1.61	7.7 #	-36.2
>500 sf	2	-	-	500.1	0.20	-1.40	7.7 #	-31.9

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
<=10 sf	3	-	-	10.0	0.30	-3.20	9.7 #	-68.3
100 sf	3	-	-	100.0	0.20	-2.14	7.7 #	-46.9
200 sf	3	-	-	200.0	0.20	-1.82	7.7 #	-40.4
>500 sf	3	-	-	500.1	0.20	-1.40	7.7 #	-31.9

The final net design wind pressure, including all permitted reductions, used in the design shall not be less than 16psf acting in either direction



Plan on roof

IBC2018/IBC2021 SLENDER MASONRY WALL ANALYSIS 8-Oct-24

Version Date: September 8, 2020 Author: Zach Hansen Reviewed By: Troy Dye 5:47 AM

JOB TITLE: Kimberly AG Shop JOB #: 24091

WALL LOCATION: 10'-0" wall Wall Design DESIGNED BY: ANB

USING TMS 402-16

TOTAL WALL HEIGHT : (w/o parapet included)	10	feet	**APPLIED VERTICAL LOADS:**		
UNSUPPORTED HEIGHT : (effective height)	10	feet	DEAD :	203	plf
PARAPET HEIGHT :	0	feet	ROOF LIVE :	0	plf
WALL THICKNESS (Actual) :	7.625	inches	SNOW :	239	plf
WALL WEIGHT :	63	psf	LIVE :	0	plf
{6000 max} f'm : (Special Inspection Required)	2000	psi	ECCENTRICITY :	5	inches
REBAR F_y :	60	ksi	f_1 :	0.5	
TWO WYTHE WALL :	N	(Y/N)	f_2 :	0.2	
DIMENSION d : {comp. face to rebar cent.}	3.81	inches	**APPLIED LATERAL LOADS:**		
WALL SPAN DIRECTION: (Vert. or Horiz.)	V	(V/H)	WIND W:	29.1	psf
MORTAR TYPE :	S	(M, S, or N)	SEISMIC E:	5.2	psf
SOLID GROUTED?	N	(Y/N)	(Two Wythe Walls Must Be Solid Grouted)		

A_s Minimum :	0.064	in^2/ft	(TMS 402-16/ACI 530-16/ASCE 5-16 7.3.2.6(c))
Steel Strain, ε_t :	0.0158		(TMS 402-16/ACI 530-16/ASCE 5-16 9.3.3.2)
Max Reinf. Area :	0.43	in^2/ft	(TMS 402-16/ACI 530-16/ASCE 5-16 9.3.3.2.1)
Factored Axial Load Stress :	22.01	psi	**Fa < 0.05f'm - O.K.**
φ :	0.9		(TMS 402-16/AC 530-16/ASCE 5-16 9.1.4.4)
E_m :	900	f'm	
Masonry Modulus of Elasticity E_m :	1800	ksi	Modular Ratio n: 16.1
ACI 530-08 Allowable D (0.007 h)	0.84	in	
User Allowable D L/ 140	0.86	in	
Allowable D :	**0.84**	in	

SUMMARY

Vertical Bars Try #	**5**	bars	One Layer			
Bar Spacing @	**32**	" o.c.				
Trial A_s :	**0.12**	sq in/ft	**OK**			
vert ratio As =	0.0013		no.	(#)	spa.	ratio
horiz. ratio req'd =	0.0007	=>	2	4	48	0.0011

Strength				Service			
STRENGTH LOAD CASE (16-2) => 1.2D+1.6L+0.5(Lr or S)				**SERVICE LOAD CASES (16-9), (16-10), & (16-11) => D L & D+(Lr or S) & D+0.75L+0.75(Lr or S)**			
M_u =	0.91	k-in/ft					
φM_n =	25.32	k-in/ft	**...OK**	Δ_s :	0.00	in	**...OK**
STRENGTH LOAD CASE (16-3) => 1.2D+1.6(Lr or S)+ 0.5W				**SERVICE LOAD CASE (16-12) => D+(0.6W or 0.7E)**			
M_u =	3.76	k-in/ft					
φM_n =	26.21	k-in/ft	**...OK**	Δ_s :	0.01	in	**...OK**
STRENGTH LOAD CASE (16-4) => 1.2D+1.0W+f1L+0.5(Lr or S)				**SERVICE LOAD CASE (16-13) => D+0.45W+0.75L+0.75(Lr or S)**			
M_u =	5.28	k-in/ft					
φM_n =	25.32	k-in/ft	**...OK**	Δ_s :	0.01	in	**...OK**
STRENGTH LOAD CASE (16-5) => 1.2D+1.0E+f1L+f2S				**SERVICE LOAD CASE (16-14) => D+0.525E+0.75L+0.75S**			
M_u =	1.51	k-in/ft					
φM_n =	25.07	k-in/ft	**...OK**	Δ_s :	0.00	in	**...OK**
STRENGTH LOAD CASES (16-6) & (16-7) => 0.9D+(W or E)				**SERVICE LOAD CASES (16-15) & (16-16) => 0.6D+0.6W & 0.6D+0.7E**			
M_u =	4.83	k-in/ft					
φM_n =	24.38	k-in/ft	**...OK**	Δ_s :	0.01	in	**...OK**

DETAILED ANALYSIS

Strength			Service		
STRENGTH LOAD CASE (16-2) => 1.2D+1.6L+0.5(Lr or S)			**SERVICE LOAD CASES (16-9), (16-10), & (16-11) => D + L, D + (Lr or S), D + 0.75L + 0.75(Lr or S)**		
P_{uf} =	0.36	k/ft	P_{sf} =	0.442	k/ft
P_{uw} =	0.38	k/ft	P_{sw} =	0.32	k/ft
P_u =	0.74	k/ft	P_s =	0.76	k/ft
F_a =	16.26	psi	ω_s =	0	psf
ω_u =	0	psf	M_s =	1.11	k-in/ft
M_u =	0.91	k-in/ft	Δ_s :	0.00	inches
φM_n =	25.32	k-in/ft			
Δ_u :	0.00	inches			
STRENGTH LOAD CASE (16-3) => 1.2D+1.6(Lr or S)+ 0.5W			**SERVICE LOAD CASE (16-12) => D + (0.6W or 0.7E)**		
P_{uf} =	0.63	k/ft	P_{sf} =	0.203	k/ft
P_{uw} =	0.38	k/ft	P_{sw} =	0.32	k/ft
P_u =	1.00	k/ft	P_s =	0.52	k/ft
F_a =	22.01	psi	ω_s =	17.46	psf
ω_u =	14.55	psf	M_s =	3.13	k-in/ft
M_u =	3.76	k-in/ft	Δ_s :	0.01	inches
φM_n =	26.21	k-in/ft			
Δ_u :	0.01	inches			
STRENGTH LOAD CASE (16-4) => 1.2D+1.0W+f_1L+0.5(Lr or S)			**SERVICE LOAD CASE (16-13) => D + 0.45W + 0.75L + 0.75(Lr or S)**		
P_{uf} =	0.36	k/ft	P_{sf} =	0.382	k/ft
P_{uw} =	0.38	k/ft	P_{sw} =	0.32	k/ft
P_u =	0.74	k/ft	P_s =	0.70	k/ft
F_a =	16.26	psi	ω_s =	13.095	psf
ω_u =	29.10	psf	M_s =	2.92	k-in/ft
M_u =	5.28	k-in/ft	Δ_s :	0.01	inches
φM_n =	25.32	k-in/ft			
Δ_u :	0.01	inches			
STRENGTH LOAD CASE (16-5) => 1.2D+1.0E+f_1L+f_2S			**SERVICE LOAD CASE (16-14) => D + 0.525E + 0.75L + 0.75S**		
P_{uf} =	0.29	k/ft	P_{sf} =	0.382	k/ft
P_{uw} =	0.38	k/ft	P_{sw} =	0.32	k/ft
P_u =	0.67	k/ft	P_s =	0.70	k/ft
F_a =	14.69	psi	ω_s =	2.73861	psf
ω_u =	5.22	psf	M_s =	1.37	k-in/ft
M_u =	1.51	k-in/ft	Δ_s :	0.00	inches
φM_n =	25.07	k-in/ft			
Δ_u :	0.00	inches			
STRENGTH LOAD CASES (16-6) & (16-7) => 0.9D+(W or E)			**SERVICE LOAD CASES (16-15) & (16-16) => 0.6D + 0.6W, 0.6D + 0.7E**		
P_{uf} =	0.18	k/ft	P_{sf} =	0.122	k/ft
P_{uw} =	0.28	k/ft	P_{sw} =	0.32	k/ft
P_u =	0.47	k/ft	P_s =	0.44	k/ft
F_a =	10.23	psi	ω_s =	17.46	psf
ω_u =	29.10	psf	M_s =	2.93	k-in/ft
M_u =	4.83	k-in/ft	Δ_s :	0.01	inches
φM_n =	24.38	k-in/ft			
Δ_u :	0.01	inches			

ARW ENGINEERS structural consultants

IBC2018/IBC2021 SLENDER MASONRY WALL ANALYSIS

Version Date: September 8, 2020	Author: Zach Hansen	Reviewed By: Troy Dye	8-Oct-24 5:47 AM
JOB TITLE:	Kimberly AG Shop	JOB #:	24091
WALL LOCATION:	10'-0" wall - jamb design	DESIGNED BY:	ANB
	Jamb Design	Opening Width (ft)= 4	
USING TMS 402-16		Jamb Width (in.)= 24	

TOTAL WALL HEIGHT : (w/o parapet included)	10	feet	**APPLIED VERTICAL LOADS:**		
UNSUPPORTED HEIGHT : (effective height)	10	feet	DEAD :	575	plf
PARAPET HEIGHT :	0	feet	ROOF LIVE :	0	plf
WALL THICKNESS (Actual) :	7.625	inches	SNOW :	477	plf
WALL WEIGHT :	63	psf	LIVE :	0	plf
{6000 max} f'$_m$: (Special Inspection Required)	2000	psi	ECCENTRICITY :	2	inches
REBAR F_y :	60	ksi	f_1 :	0.5	
TWO WYTHE WALL :	N	(Y/N)	f_2 :	0.2	
DIMENSION d : {comp. face to rebar cent.}	3.81	inches	**APPLIED LATERAL LOADS:**		
WALL SPAN DIRECTION: (Vert. or Horiz.)	V	(V/H)	WIND W:	58.2	psf
MORTAR TYPE :	S	(M, S, or N)	SEISMIC E:	10.4	psf
SOLID GROUTED?	N	(Y/N)	(Two Wythe Walls Must Be Solid Grouted)		

A_s Minimum :	0.064	in^2/ft	(TMS 402-16/ACI 530-16/ASCE 5-16 7.3.2.6(c))
Steel Strain, ε_t :	0.0068		(TMS 402-16/ACI 530-16/ASCE 5-16 9.3.3.2)
Max Reinf. Area :	0.42	in^2/ft	(TMS 402-16/ACI 530-16/ASCE 5-16 9.3.3.2.1)
Factored Axial Load Stress :	29.34	psi	**Fa < 0.05f'm - O.K.**
φ :	0.9		(TMS 402-16/AC 530-16/ASCE 5-16 9.1.4.4)
E_m :	900	f'm	
Masonry Modulus of Elasticity E_m :	1800	ksi	Modular Ratio n: 16.1
ACI 530-08 Allowable D (0.007 h)	0.84	in	
User Allowable D L/ 140	0.86	in	
Allowable D :	**0.84**	in	

SUMMARY

Vertical Bars Try #	**5**	bars	One Layer			
Bar Spacing @	**16**	" o.c.				
Trial A_s :	**0.23**	sq in/ft	**OK**			
vert ratio As =	0.0025		no.	(#)	spa.	ratio
horiz. ratio req'd =	0.0007	=>	2	4	48	0.0011

Strength				Service			
STRENGTH LOAD CASE (16-2) => 1.2D+1.6L+0.5(Lr or S)				**SERVICE LOAD CASES (16-9), (16-10), & (16-11) =>** **D L & D+(Lr or S) & D+0.75L+0.75(Lr or S)**			
M_u =	0.93	k-in/ft					
φM_n =	47.25	k-in/ft	**...OK**	Δ_s :	0.00	in	**...OK**
STRENGTH LOAD CASE (16-3) => 1.2D+1.6(Lr or S)+ 0.5W				**SERVICE LOAD CASE (16-12) =>** **D+(0.6W or 0.7E)**			
M_u =	5.84	k-in/ft					
φM_n =	48.82	k-in/ft	**...OK**	Δ_s :	0.01	in	**...OK**
STRENGTH LOAD CASE (16-4) => 1.2D+1.0W+f1L+0.5(Lr or S)				**SERVICE LOAD CASE (16-13) =>** **D+0.45W+0.75L+0.75(Lr or S)**			
M_u =	9.69	k-in/ft					
φM_n =	47.25	k-in/ft	**...OK**	Δ_s :	0.01	in	**...OK**
STRENGTH LOAD CASE (16-5) => 1.2D+1.0E+f1L+f2S				**SERVICE LOAD CASE (16-14) =>** **D+0.525E+0.75L+0.75S**			
M_u =	2.36	k-in/ft					
φM_n =	46.82	k-in/ft	**...OK**	Δ_s :	0.00	in	**...OK**
STRENGTH LOAD CASES (16-6) & (16-7) => 0.9D+(W or E)				**SERVICE LOAD CASES (16-15) & (16-16) =>** **0.6D+0.6W & 0.6D+0.7E**			
M_u =	9.26	k-in/ft					
φM_n =	45.72	k-in/ft	**...OK**	Δ_s :	0.01	in	**...OK**

DETAILED ANALYSIS

Strength			Service		
STRENGTH LOAD CASE (16-2) => 1.2D+1.6L+0.5(Lr or S)			**SERVICE LOAD CASES (16-9), (16-10), & (16-11) =>** **D + L, D + (Lr or S), D + 0.75L + 0.75(Lr or S)**		
P_{uf} =	0.93	k/ft	P_{sf} =	1.052	k/ft
P_{uw} =	0.38	k/ft	P_{sw} =	0.32	k/ft
P_u =	1.31	k/ft	P_s =	1.37	k/ft
F_a =	20.93	psi	ω_s =	0	psf
ω_u =	0	psf	M_s =	1.05	k-in/ft
M_u =	0.93	k-in/ft	Δ_s :	0.00	inches
φM_n =	47.25	k-in/ft			
Δ_u :	0.00	inches			
STRENGTH LOAD CASE (16-3) => 1.2D+1.6(Lr or S)+ 0.5W			**SERVICE LOAD CASE (16-12) =>** **D + (0.6W or 0.7E)**		
P_{uf} =	1.45	k/ft	P_{sf} =	0.575	k/ft
P_{uw} =	0.38	k/ft	P_{sw} =	0.32	k/ft
P_u =	1.83	k/ft	P_s =	0.89	k/ft
F_a =	29.34	psi	ω_s =	34.92	psf
ω_u =	29.10	psf	M_s =	5.82	k-in/ft
M_u =	5.84	k-in/ft	Δ_s :	0.01	inches
φM_n =	48.82	k-in/ft			
Δ_u :	0.01	inches			
STRENGTH LOAD CASE (16-4) => 1.2D+1.0W+f_1L+0.5(Lr or S)			**SERVICE LOAD CASE (16-13) =>** **D + 0.45W + 0.75L + 0.75(Lr or S)**		
P_{uf} =	0.93	k/ft	P_{sf} =	0.932	k/ft
P_{uw} =	0.38	k/ft	P_{sw} =	0.32	k/ft
P_u =	1.31	k/ft	P_s =	1.25	k/ft
F_a =	20.93	psi	ω_s =	26.19	psf
ω_u =	58.20	psf	M_s =	4.87	k-in/ft
M_u =	9.69	k-in/ft	Δ_s :	0.01	inches
φM_n =	47.25	k-in/ft			
Δ_u :	0.02	inches			
STRENGTH LOAD CASE (16-5) => 1.2D+1.0E+f_1L+f_2S			**SERVICE LOAD CASE (16-14) =>** **D + 0.525E + 0.75L + 0.75S**		
P_{uf} =	0.78	k/ft	P_{sf} =	0.932	k/ft
P_{uw} =	0.38	k/ft	P_{sw} =	0.32	k/ft
P_u =	1.16	k/ft	P_s =	1.25	k/ft
F_a =	18.64	psi	ω_s =	5.47722	psf
ω_u =	10.43	psf	M_s =	1.76	k-in/ft
M_u =	2.36	k-in/ft	Δ_s :	0.00	inches
φM_n =	46.82	k-in/ft			
Δ_u :	0.01	inches			
STRENGTH LOAD CASES (16-6) & (16-7) => 0.9D+(W or E)			**SERVICE LOAD CASES (16-15) & (16-16) =>** **0.6D + 0.6W, 0.6D + 0.7E**		
P_{uf} =	0.52	k/ft	P_{sf} =	0.345	k/ft
P_{uw} =	0.28	k/ft	P_{sw} =	0.32	k/ft
P_u =	0.80	k/ft	P_s =	0.66	k/ft
F_a =	12.83	psi	ω_s =	34.92	psf
ω_u =	58.20	psf	M_s =	5.59	k-in/ft
M_u =	9.26	k-in/ft	Δ_s :	0.01	inches
φM_n =	45.72	k-in/ft			
Δ_u :	0.02	inches			

A R W ENGINEERS structural consultants

IBC2018/IBC2021 SLENDER MASONRY WALL ANALYSIS

8-Oct-24
5:47 AM

Version Date: September 8, 2020 | Author: Zach Hansen | Reviewed By: Troy Dye

JOB TITLE:	Kimberly AG Shop	JOB #:	24091
WALL LOCATION:	Cantilevered wall design (compare to enercalc) Wall Design	DESIGNED BY:	ANB

USING TMS 402-16

TOTAL WALL HEIGHT : (w/o parapet included)	21	feet	**APPLIED VERTICAL LOADS:**		
UNSUPPORTED HEIGHT : (effective height)	21	feet	DEAD :	0	plf
PARAPET HEIGHT :	0	feet	ROOF LIVE :	0	plf
WALL THICKNESS (Actual) :	7.625	inches	SNOW :	0	plf
WALL WEIGHT :	63	psf	LIVE :	0	plf
{6000 max} f'_m : (Special Inspection Required)	2000	psi	ECCENTRICITY :	0	inches
REBAR F_y :	60	ksi	f_1 :	0.5	
TWO WYTHE WALL :	N	(Y/N)	f_2 :	0.2	
DIMENSION d : {comp. face to rebar cent.}	3.8125	inches	**APPLIED LATERAL LOADS:**		
WALL SPAN DIRECTION: (Vert. or Horiz.)	V	(V/H)	WIND W:	29.1	psf
MORTAR TYPE :	S	(M, S, or N)	SEISMIC E:	5.2	psf
SOLID GROUTED?	Y	(Y/N)	(Two Wythe Walls Must Be Solid Grouted)		

A_s Minimum :	0.064	in^2/ft	(TMS 402-16/ACI 530-16/ASCE 5-16 7.3.2.6(c))
Steel Strain, ε_t :	0.0163		(TMS 402-16/ACI 530-16/ASCE 5-16 9.3.3.2)
Max Reinf. Area :	0.43	in^2/ft	(TMS 402-16/ACI 530-16/ASCE 5-16 9.3.3.2.1)
Factored Axial Load Stress :	8.70	psi	**Fa < 0.05f'm - O.K.**
φ :	0.9		(TMS 402-16/AC 530-16/ASCE 5-16 9.1.4.4)
E_m :	900	f'm	
Masonry Modulus of Elasticity E_m :	1800	ksi	Modular Ratio n: 16.1
ACI 530-08 Allowable D (0.007 h)	1.76	in	
User Allowable D L/ 140	1.80	in	
Allowable D :	**1.76**	in	

SUMMARY

Vertical Bars Try #	**5**	bars	One Layer			
Bar Spacing @	**32**	" o.c.				
Trial A_s :	**0.12**	sq in/ft	**OK**			
vert ratio As =	0.0013		no.	(#)	spa.	ratio
horiz. ratio req'd =	0.0007	=>	2	4	48	0.0011

STRENGTH LOAD CASE (16-2) => 1.2D+1.6L+0.5(Lr or S)				**SERVICE LOAD CASES (16-9), (16-10), & (16-11) =>**				
M_u =	0.00	k-in/ft		**D L & D+(Lr or S) & D+0.75L+0.75(Lr or S)**				
φM_n =	25.51	k-in/ft	**...OK**	Δ_s :	0.00	in	**...OK**	
STRENGTH LOAD CASE (16-3) => 1.2D+1.6(Lr or S)+ 0.5W				**SERVICE LOAD CASE (16-12) =>**				
M_u =	9.69	k-in/ft		**D+(0.6W or 0.7E)**				
φM_n =	25.51	k-in/ft	**...OK**	Δ_s :	0.10	in	**...OK**	
STRENGTH LOAD CASE (16-4) => 1.2D+1.0W+f1L+0.5(Lr or S)				**SERVICE LOAD CASE (16-13) =>**				
M_u =	19.38	k-in/ft		**D+0.45W+0.75L+0.75(Lr or S)**				
φM_n =	25.51	k-in/ft	**...OK**	Δ_s :	0.07	in	**...OK**	
STRENGTH LOAD CASE (16-5) => 1.2D+1.0E+f1L+f2S				**SERVICE LOAD CASE (16-14) =>**				
M_u =	3.47	k-in/ft		**D+0.525E+0.75L+0.75S**				
φM_n =	25.51	k-in/ft	**...OK**	Δ_s :	0.02	in	**...OK**	
STRENGTH LOAD CASES (16-6) & (16-7) => 0.9D+(W or E)				**SERVICE LOAD CASES (16-15) & (16-16) =>**				
M_u =	19.35	k-in/ft		**0.6D+0.6W & 0.6D+0.7E**				
φM_n =	24.84	k-in/ft	**...OK**	Δ_s :	0.10	in	**...OK**	

DETAILED ANALYSIS

STRENGTH LOAD CASE (16-2) => 1.2D+1.6L+0.5(Lr or S)			**SERVICE LOAD CASES (16-9), (16-10), & (16-11) =>**		
			D + L, D + (Lr or S), D + 0.75L + 0.75(Lr or S)		
P_{uf} =	0.00	k/ft	P_{sf} =	0.000	k/ft
P_{uw} =	0.79	k/ft	P_{sw} =	0.66	k/ft
P_u =	0.79	k/ft	P_s =	0.66	k/ft
F_a =	8.70	psi	ω_s =	0	psf
ω_u =	0	psf	M_s =	0.00	k-in/ft
M_u =	0.00	k-in/ft	Δ_s :	0.00	inches
φM_n =	25.51	k-in/ft			
Δ_u :	0.00	inches			
STRENGTH LOAD CASE (16-3) => 1.2D+1.6(Lr or S)+ 0.5W			**SERVICE LOAD CASE (16-12) =>**		
			D + (0.6W or 0.7E)		
P_{uf} =	0.00	k/ft	P_{sf} =	0.000	k/ft
P_{uw} =	0.79	k/ft	P_{sw} =	0.66	k/ft
P_u =	0.79	k/ft	P_s =	0.66	k/ft
F_a =	8.70	psi	ω_s =	17.46	psf
ω_u =	14.55	psf	M_s =	11.61	k-in/ft
M_u =	9.69	k-in/ft	Δ_s :	0.10	inches
φM_n =	25.51	k-in/ft			
Δ_u :	0.08	inches			
STRENGTH LOAD CASE (16-4) => 1.2D+1.0W+f_1L+0.5(Lr or S)			**SERVICE LOAD CASE (16-13) =>**		
			D + 0.45W + 0.75L + 0.75(Lr or S)		
P_{uf} =	0.00	k/ft	P_{sf} =	0.000	k/ft
P_{uw} =	0.79	k/ft	P_{sw} =	0.66	k/ft
P_u =	0.79	k/ft	P_s =	0.66	k/ft
F_a =	8.70	psi	ω_s =	13.095	psf
ω_u =	29.10	psf	M_s =	8.71	k-in/ft
M_u =	19.38	k-in/ft	Δ_s :	0.07	inches
φM_n =	25.51	k-in/ft			
Δ_u :	0.16	inches			
STRENGTH LOAD CASE (16-5) => 1.2D+1.0E+f_1L+f_2S			**SERVICE LOAD CASE (16-14) =>**		
			D + 0.525E + 0.75L + 0.75S		
P_{uf} =	0.00	k/ft	P_{sf} =	0.000	k/ft
P_{uw} =	0.79	k/ft	P_{sw} =	0.66	k/ft
P_u =	0.79	k/ft	P_s =	0.66	k/ft
F_a =	8.70	psi	ω_s =	2.73861	psf
ω_u =	5.22	psf	M_s =	1.82	k-in/ft
M_u =	3.47	k-in/ft	Δ_s :	0.02	inches
φM_n =	25.51	k-in/ft			
Δ_u :	0.03	inches			
STRENGTH LOAD CASES (16-6) & (16-7) => 0.9D+(W or E)			**SERVICE LOAD CASES (16-15) & (16-16) =>**		
			0.6D + 0.6W, 0.6D + 0.7E		
P_{uf} =	0.00	k/ft	P_{sf} =	0.000	k/ft
P_{uw} =	0.60	k/ft	P_{sw} =	0.66	k/ft
P_u =	0.60	k/ft	P_s =	0.66	k/ft
F_a =	6.53	psi	ω_s =	17.46	psf
ω_u =	29.10	psf	M_s =	11.61	k-in/ft
M_u =	19.35	k-in/ft	Δ_s :	0.10	inches
φM_n =	24.84	k-in/ft			
Δ_u :	0.16	inches			

ARW ENGINEERS
structural consultants

IBC2018/IBC2021 SLENDER MASONRY WALL ANALYSIS

8-Oct-24
5:47 AM

Version Date: September 8, 2020 | Author: Zach Hansen | Reviewed By: Troy Dye

JOB TITLE:	Kimberly AG Shop	JOB #:	24091
WALL LOCATION:	Jamb Design @ cantilevered wall	DESIGNED BY:	ANB
	Jamb Design	Opening Width (ft)= 4	
		Jamb Width (in.)= 24	

USING TMS 402-16

TOTAL WALL HEIGHT : (w/o parapet included)	21	feet	**APPLIED VERTICAL LOADS:**			
UNSUPPORTED HEIGHT : (effective height)	21	feet	DEAD :	914	plf	
PARAPET HEIGHT :	0	feet	ROOF LIVE :	0	plf	
WALL THICKNESS (Actual) :	7.625	inches	SNOW :	0	plf	
WALL WEIGHT :	63	psf	LIVE :	0	plf	
{6000 max} f'_m : (Special Inspection Required)	2000	psi	ECCENTRICITY :	0	inches	
REBAR F_y :	60	ksi	f_1 :	0.5		
TWO WYTHE WALL :	N	(Y/N)	f_2 :	0.2		
DIMENSION d : {comp. face to rebar cent.}	3.8125	inches	**APPLIED LATERAL LOADS:**			
WALL SPAN DIRECTION: (Vert. or Horiz.)	V	(V/H)	WIND W:	58.2	psf	
MORTAR TYPE :	S	(M, S, or N)	SEISMIC E:	12.9	psf	
SOLID GROUTED?	Y	(Y/N)	(Two Wythe Walls Must Be Solid Grouted)			
A_s Minimum :	0.064	in^2/ft	(TMS 402-16/ACI 530-16/ASCE 5-16 7.3.2.6(c))			
Steel Strain, ε_t :	0.0067		(TMS 402-16/ACI 530-16/ASCE 5-16 9.3.3.2)			
Max Reinf. Area :	0.42	in^2/ft	(TMS 402-16/ACI 530-16/ASCE 5-16 9.3.3.2.1)			
Factored Axial Load Stress :	20.72	psi	**Fa < 0.05f'm - O.K.**			
φ :	0.9		(TMS 402-16/AC 530-16/ASCE 5-16 9.1.4.4)			
E_m :	900	f'm				
Masonry Modulus of Elasticity E_m :	1800	ksi	Modular Ratio n:	16.1		
ACI 530-08 Allowable D (0.007 h)	1.76	in				
User Allowable D L/ 140	1.80	in				
Allowable D :	**1.76**	in				

SUMMARY

Vertical Bars Try #	**5**	bars	One Layer			
Bar Spacing @	**16**	" o.c.				
Trial A_s :	**0.23**	sq in/ft	**OK**			
vert ratio As =	0.0025		no.	(#)	spa.	ratio
horiz. ratio req'd =	0.0007	=>	2	4	48	0.0011

Strength				Service			
STRENGTH LOAD CASE (16-2) => 1.2D+1.6L+0.5(Lr or S)				**SERVICE LOAD CASES (16-9), (16-10), & (16-11) =>**			
M_u =	0.00	k-in/ft		**D L & D+(Lr or S) & D+0.75L+0.75(Lr or S)**			
φM_n =	49.03	k-in/ft	**...OK**	Δ_s :	0.00	in	**...OK**
STRENGTH LOAD CASE (16-3) => 1.2D+1.6(Lr or S)+ 0.5W				**SERVICE LOAD CASE (16-12) =>**			
M_u =	19.56	k-in/ft		**D+(0.6W or 0.7E)**			
φM_n =	49.03	k-in/ft	**...OK**	Δ_s :	0.48	in	**...OK**
STRENGTH LOAD CASE (16-4) => 1.2D+1.0W+f1L+0.5(Lr or S)				**SERVICE LOAD CASE (16-13) =>**			
M_u =	42.96	k-in/ft		**D+0.45W+0.75L+0.75(Lr or S)**			
φM_n =	49.03	k-in/ft	**...OK**	Δ_s :	0.15	in	**...OK**
STRENGTH LOAD CASE (16-5) => 1.2D+1.0E+f1L+f2S				**SERVICE LOAD CASE (16-14) =>**			
M_u =	8.68	k-in/ft		**D+0.525E+0.75L+0.75S**			
φM_n =	49.03	k-in/ft	**...OK**	Δ_s :	0.04	in	**...OK**
STRENGTH LOAD CASES (16-6) & (16-7) => 0.9D+(W or E)				**SERVICE LOAD CASES (16-15) & (16-16) =>**			
M_u =	41.72	k-in/ft		**0.6D+0.6W & 0.6D+0.7E**			
φM_n =	47.62	k-in/ft	**...OK**	Δ_s :	0.47	in	**...OK**

DETAILED ANALYSIS

Strength			Service		
STRENGTH LOAD CASE (16-2) => 1.2D+1.6L+0.5(Lr or S)			**SERVICE LOAD CASES (16-9), (16-10), & (16-11) =>**		
			D + L, D + (Lr or S), D + 0.75L + 0.75(Lr or S)		
P_{uf} =	1.10	k/ft	P_{sf} =	0.914	k/ft
P_{uw} =	0.79	k/ft	P_{sw} =	0.66	k/ft
P_u =	1.89	k/ft	P_s =	1.58	k/ft
F_a =	20.72	psi	ω_s =	0	psf
ω_u =	0	psf	M_s =	0.00	k-in/ft
M_u =	0.00	k-in/ft	Δ_s :	0.00	inches
φM_n =	49.03	k-in/ft			
Δ_u :	0.00	inches			
STRENGTH LOAD CASE (16-3) => 1.2D+1.6(Lr or S)+ 0.5W			**SERVICE LOAD CASE (16-12) =>**		
P_{uf} =	1.10	k/ft	**D + (0.6W or 0.7E)**		
P_{uw} =	0.79	k/ft	P_{sf} =	0.914	k/ft
P_u =	1.89	k/ft	P_{sw} =	0.66	k/ft
F_a =	20.72	psi	P_s =	1.58	k/ft
ω_u =	29.10	psf	ω_s =	34.92	psf
M_u =	19.56	k-in/ft	M_s =	23.86	k-in/ft
φM_n =	49.03	k-in/ft	Δ_s :	0.48	inches
Δ_u :	0.16	inches			
STRENGTH LOAD CASE (16-4) => 1.2D+1.0W+f_1L+0.5(Lr or S)			**SERVICE LOAD CASE (16-13) =>**		
P_{uf} =	1.10	k/ft	**D + 0.45W + 0.75L + 0.75(Lr or S)**		
P_{uw} =	0.79	k/ft	P_{sf} =	0.914	k/ft
P_u =	1.89	k/ft	P_{sw} =	0.66	k/ft
F_a =	20.72	psi	P_s =	1.58	k/ft
ω_u =	58.20	psf	ω_s =	26.19	psf
M_u =	42.96	k-in/ft	M_s =	17.55	k-in/ft
φM_n =	49.03	k-in/ft	Δ_s :	0.15	inches
Δ_u :	2.36	inches			
STRENGTH LOAD CASE (16-5) => 1.2D+1.0E+f_1L+f_2S			**SERVICE LOAD CASE (16-14) =>**		
P_{uf} =	1.10	k/ft	**D + 0.525E + 0.75L + 0.75S**		
P_{uw} =	0.79	k/ft	P_{sf} =	0.914	k/ft
P_u =	1.89	k/ft	P_{sw} =	0.66	k/ft
F_a =	20.72	psi	P_s =	1.58	k/ft
ω_u =	12.92	psf	ω_s =	6.78132	psf
M_u =	8.68	k-in/ft	M_s =	4.55	k-in/ft
φM_n =	49.03	k-in/ft	Δ_s :	0.04	inches
Δ_u :	0.07	inches			
STRENGTH LOAD CASES (16-6) & (16-7) => 0.9D+(W or E)			**SERVICE LOAD CASES (16-15) & (16-16) =>**		
P_{uf} =	0.82	k/ft	**0.6D + 0.6W, 0.6D + 0.7E**		
P_{uw} =	0.60	k/ft	P_{sf} =	0.548	k/ft
P_u =	1.42	k/ft	P_{sw} =	0.66	k/ft
F_a =	15.54	psi	P_s =	1.21	k/ft
ω_u =	58.20	psf	ω_s =	34.92	psf
M_u =	41.72	k-in/ft	M_s =	23.66	k-in/ft
φM_n =	47.62	k-in/ft	Δ_s :	0.47	inches
Δ_u :	2.27	inches			

ARW ENGINEERS structural consultants

IBC2018/IBC2021 SLENDER MASONRY WALL ANALYSIS

8-Oct-24
5:47 AM

Version Date: September 8, 2020 | Author: Zach Hansen | Reviewed By: Troy Dye

JOB TITLE:	Kimberly AG Shop	JOB #:	24091
WALL LOCATION:	Jamb Design @ cantilevered wall	DESIGNED BY:	ANB
	Jamb Design	Opening Width (ft)= 6	
		Jamb Width (in.)= 24	

USING TMS 402-16

TOTAL WALL HEIGHT : (w/o parapet included)	21	feet	**APPLIED VERTICAL LOADS:**		
UNSUPPORTED HEIGHT : (effective height)	21	feet	DEAD :	1370	plf
PARAPET HEIGHT :	0	feet	ROOF LIVE :	0	plf
WALL THICKNESS (Actual) :	7.625	inches	SNOW :	0	plf
WALL WEIGHT :	63	psf	LIVE :	0	plf
{6000 max} f'$_m$: (Special Inspection Required)	2000	psi	ECCENTRICITY :	0	inches
REBAR F_y :	60	ksi	f_1 :	0.5	
TWO WYTHE WALL :	N	(Y/N)	f_2 :	0.2	
DIMENSION d : {comp. face to rebar cent.}	5.3125	inches	**APPLIED LATERAL LOADS:**		
WALL SPAN DIRECTION: (Vert. or Horiz.)	V	(V/H)	WIND W:	72.8	psf
MORTAR TYPE :	S	(M, S, or N)	SEISMIC E:	16.1	psf
SOLID GROUTED?	Y	(Y/N)	(Two Wythe Walls Must Be Solid Grouted)		

A_s Minimum :	0.064	in^2/ft	(TMS 402-16/ACI 530-16/ASCE 5-16 7.3.2.6(c))
Steel Strain, ε_t :	0.0099		(TMS 402-16/ACI 530-16/ASCE 5-16 9.3.3.2)
Max Reinf. Area :	0.58	in^2/ft	(TMS 402-16/ACI 530-16/ASCE 5-16 9.3.3.2.1)
Factored Axial Load Stress :	26.73	psi	**Fa < 0.05f'm - O.K.**
φ :	0.9		(TMS 402-16/AC 530-16/ASCE 5-16 9.1.4.4)
E_m :	900	f'm	
Masonry Modulus of Elasticity E_m :	1800	ksi	Modular Ratio n: 16.1
ACI 530-08 Allowable D (0.007 h)	1.76	in	
User Allowable D L/ 140	1.80	in	
Allowable D :	**1.76**	in	

SUMMARY

Vertical Bars Try #	**5**	bars	Each Face			
Bar Spacing @	**16**	" o.c.				
Trial A_s :	**0.47**	sq in/ft	**OK**			
vert ratio As =	0.0051		no.	(#)	spa.	ratio
horiz. ratio req'd =	0.0007	=>	2	4	48	0.0011

STRENGTH				SERVICE			
STRENGTH LOAD CASE (16-2) => 1.2D+1.6L+0.5(Lr or S)				**SERVICE LOAD CASES (16-9), (16-10), & (16-11) =>**			
M_u =	0.00	k-in/ft		**D L & D+(Lr or S) & D+0.75L+0.75(Lr or S)**			
φM_n =	71.58	k-in/ft	**...OK**	Δ_s :	0.00	in	**...OK**
STRENGTH LOAD CASE (16-3) => 1.2D+1.6(Lr or S)+ 0.5W				**SERVICE LOAD CASE (16-12) =>**			
M_u =	24.89	k-in/ft		**D+(0.6W or 0.7E)**			
φM_n =	71.58	k-in/ft	**...OK**	Δ_s :	0.58	in	**...OK**
STRENGTH LOAD CASE (16-4) => 1.2D+1.0W+f1L+0.5(Lr or S)				**SERVICE LOAD CASE (16-13) =>**			
M_u =	51.95	k-in/ft		**D+0.45W+0.75L+0.75(Lr or S)**			
φM_n =	71.58	k-in/ft	**...OK**	Δ_s :	0.21	in	**...OK**
STRENGTH LOAD CASE (16-5) => 1.2D+1.0E+f1L+f2S				**SERVICE LOAD CASE (16-14) =>**			
M_u =	10.90	k-in/ft		**D+0.525E+0.75L+0.75S**			
φM_n =	71.58	k-in/ft	**...OK**	Δ_s :	0.05	in	**...OK**
STRENGTH LOAD CASES (16-6) & (16-7) => 0.9D+(W or E)				**SERVICE LOAD CASES (16-15) & (16-16) =>**			
M_u =	50.95	k-in/ft		**0.6D+0.6W & 0.6D+0.7E**			
φM_n =	69.78	k-in/ft	**...OK**	Δ_s :	0.57	in	**...OK**

DETAILED ANALYSIS

STRENGTH			SERVICE		
STRENGTH LOAD CASE (16-2) => 1.2D+1.6L+0.5(Lr or S)			**SERVICE LOAD CASES (16-9), (16-10), & (16-11) =>**		
			D + L, D + (Lr or S), D + 0.75L + 0.75(Lr or S)		
P_{uf} =	1.64	k/ft	P_{sf} =	1.370	k/ft
P_{uw} =	0.79	k/ft	P_{sw} =	0.66	k/ft
P_u =	2.44	k/ft	P_s =	2.03	k/ft
F_a =	26.73	psi			
ω_u =	0	psf	ω_s =	0	psf
M_u =	0.00	k-in/ft	M_s =	0.00	k-in/ft
φM_n =	71.58	k-in/ft	Δ_s :	0.00	inches
Δ_u :	0.00	inches			
STRENGTH LOAD CASE (16-3) => 1.2D+1.6(Lr or S)+ 0.5W			**SERVICE LOAD CASE (16-12) =>**		
			D + (0.6W or 0.7E)		
P_{uf} =	1.64	k/ft	P_{sf} =	1.370	k/ft
P_{uw} =	0.79	k/ft	P_{sw} =	0.66	k/ft
P_u =	2.44	k/ft	P_s =	2.03	k/ft
F_a =	26.73	psi			
ω_u =	36.38	psf	ω_s =	43.65	psf
M_u =	24.89	k-in/ft	M_s =	30.05	k-in/ft
φM_n =	71.58	k-in/ft	Δ_s :	0.58	inches
Δ_u :	0.34	inches			
STRENGTH LOAD CASE (16-4) => 1.2D+1.0W+f_1L+0.5(Lr or S)			**SERVICE LOAD CASE (16-13) =>**		
			D + 0.45W + 0.75L + 0.75(Lr or S)		
P_{uf} =	1.64	k/ft	P_{sf} =	1.370	k/ft
P_{uw} =	0.79	k/ft	P_{sw} =	0.66	k/ft
P_u =	2.44	k/ft	P_s =	2.03	k/ft
F_a =	26.73	psi			
ω_u =	72.75	psf	ω_s =	32.7375	psf
M_u =	51.95	k-in/ft	M_s =	22.09	k-in/ft
φM_n =	71.58	k-in/ft	Δ_s :	0.21	inches
Δ_u :	1.57	inches			
STRENGTH LOAD CASE (16-5) => 1.2D+1.0E+f_1L+f_2S			**SERVICE LOAD CASE (16-14) =>**		
			D + 0.525E + 0.75L + 0.75S		
P_{uf} =	1.64	k/ft	P_{sf} =	1.370	k/ft
P_{uw} =	0.79	k/ft	P_{sw} =	0.66	k/ft
P_u =	2.44	k/ft	P_s =	2.03	k/ft
F_a =	26.73	psi			
ω_u =	16.15	psf	ω_s =	8.47665	psf
M_u =	10.90	k-in/ft	M_s =	5.70	k-in/ft
φM_n =	71.58	k-in/ft	Δ_s :	0.05	inches
Δ_u :	0.09	inches			
STRENGTH LOAD CASES (16-6) & (16-7) => 0.9D+(W or E)			**SERVICE LOAD CASES (16-15) & (16-16) =>**		
P_{uf} =	1.23	k/ft	**0.6D + 0.6W, 0.6D + 0.7E**		
P_{uw} =	0.60	k/ft	P_{sf} =	0.822	k/ft
P_u =	1.83	k/ft	P_{sw} =	0.66	k/ft
F_a =	20.05	psi	P_s =	1.48	k/ft
ω_u =	72.75	psf	ω_s =	43.65	psf
M_u =	50.95	k-in/ft	M_s =	29.72	k-in/ft
φM_n =	69.78	k-in/ft	Δ_s :	0.57	inches
Δ_u :	1.55	inches			

Masonry Beam Design:

* Beam Design:

Beam @ 3'-4" opening. there is only 12" of CMU above opening

LOAD = 442 PLF.

$M = WL^2/10 = 442\ \text{PLF}\,(3'\text{-}4")^2/10 = 491\ \#\cdot\text{FT}$

$V = WL/2 = 442\ \text{PLF}\,(3'\text{-}4")/2 = 737\ \#$

↳ 8" CMU beam okay - see spreadsheet.

Beam @ cantilevered wall:

OOP Moment: $W = 28\text{PSF}\,(2'\text{-}8' + 7'\text{-}4"/2) = 177\text{PLF}$

$M = wl^2/10 = 177\text{PLF}\,(6'\text{-}0")^2/10 = 638\ \#\cdot\text{FT}$

$V = WL/2 = 177\ \text{PLF}\,(6'\text{-}0")/2 = 531\ \#$

GRAVITY: JUST SW

$W = 78\ \text{PSF}\,(2'\text{-}8") = 208\ \text{PLF}$

$M = WL^2/10 = 208\ \text{PLF}\,(6'\text{-}0")^2/10 = 750\#\cdot\text{FT}$

$V = WL/2 = 208\ \text{PLF}\,(6'\text{-}0")/2 = 624\#$

Int. EQN (Moment): $\frac{0.75\ \text{K}\cdot\text{FT}}{24.21\ \text{K}\cdot\text{FT}} + \frac{0.64\ \text{K}\cdot\text{FT}}{5.72\ \text{K}\cdot\text{FT}} = 0.14$

INT. EQN (Shear): $\frac{0.624\text{K}}{18.6\text{K}} + \frac{0.53\text{K}}{17.2\text{K}} = 0.06$

↳ 24" deep beam w/ (4) #5 BARS.

ENGINEERS
structural consultants

Project Name :	Kimberly AG Shop	08-Oct-24
Project Number :	224091	5:48 AM
Engineer :	ANB	
Notes :	Beam Design (3'-4" opening at steel joists)	

MASONRY BEAM DESIGN AND ANALYSIS

2018 & 2021 International Building Code - TMS 402-16 (Updated 19 March 2019)

Masonry Beam Properties			Steel Reinforcement Properties		
Masonry f 'm =	2000	psi	Steel Fs =	25600	psi
Actual "b" width =	7.625	in	# of Tension Bars =	2	
Grout Depth =	8	in	Area/Tension Bars =	0.2	in2
Lintel Steel "d" =	6	in	Shear Bars Av =	0.2	in2
Beam Length "L" =	3.333	ft	Shear Bar Spacing =	8	in2
End Fixity	Pin		Modular Ratio (n) =	16.11	
M/(Vd)=	1.250		Spec. Inspection =	Y	Y/N
Wall Above Beam	1	ft			

DESIGN SUMMARY

Allowable Moment =	**2.66**	k-ft	np =	0.1409
Allow. Shear Resistance Vn=	**5.46**	kips	k =	0.4083
Capacity of Reinf. Steel Vs =	41.97	kips	j =	0.8639
Capacity of Masonry Vm	3.07	kips	2/kj =	5.6702

Project Name :	Kimberly AG Shop	08-Oct-24
Project Number :	224091	5:48 AM
Engineer :	ANB	
Notes :	Beam Design (6'-0" opening, gravity at cantilevered wall)	

MASONRY BEAM DESIGN AND ANALYSIS

2018 & 2021 International Building Code - TMS 402-16 (Updated 19 March 2019)

Masonry Beam Properties			Steel Reinforcement Properties		
Masonry f 'm =	2000	psi	Steel Fs =	25600	psi
Actual "b" width =	7.625	in	# of Tension Bars =	4	
Grout Depth =	24	in	Area/Tension Bars =	0.31	in2
Lintel Steel "d" =	18	in	Shear Bars Av =	0.2	in2
Beam Length "L" =	6	ft	Shear Bar Spacing =	8	in2
End Fixity	Partial		Modular Ratio (n) =	16.11	
M/(Vd)=	0.800		Spec. Inspection =	Y	Y/N
Wall Above Beam	2.667	ft			

DESIGN SUMMARY

Allowable Moment =	**24.21**	k-ft	np =	0.1456
Allow. Shear Resistance Vn=	**18.55**	kips	k =	0.4133
Capacity of Reinf. Steel Vs =	41.97	kips	j =	0.8622
Capacity of Masonry Vm	10.64	kips	2/kj =	5.6125

Project Name :	Kimberly AG Shop
Project Number :	224091
Engineer :	ANB
Notes :	Beam Design (6'-0" opening, gravity at cantilevered wall)

08-Oct-24
5:48 AM

MASONRY BEAM DESIGN AND ANALYSIS

2018 & 2021 International Building Code - TMS 402-16 (Updated 19 March 2019)

Masonry Beam Properties

Masonry f 'm =	2000	psi
Actual "b" width =	24	in
Grout Depth =	8	in
Lintel Steel "d" =	5.5	in
Beam Length "L" =	6	ft
End Fixity	Partial	
M/(Vd)=	2.618	
Wall Above Beam	2.667	ft

Steel Reinforcement Properties

Steel Fs =	25600	psi
# of Tension Bars =	2	
Area/Tension Bars =	0.31	in2
Shear Bars Av =	0.2	in2
Shear Bar Spacing =	8	in2
Modular Ratio (n) =	16.11	
Spec. Inspection =	Y	Y/N

DESIGN SUMMARY

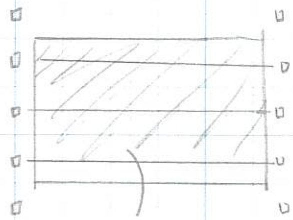
Allowable Moment =	**5.72**	k-ft	np =	0.0757
Allow. Shear Resistance Vn=	**17.17**	kips	k =	0.3207
Capacity of Reinf. Steel Vs =	13.33	kips	j =	0.8931
Capacity of Masonry Vm	9.66	kips	2/kj =	6.9838

ARW ENGINEERS
structural consultants

Project No. 24091 | Sheet No. 102
Project Kimberly AG Shop
Prepared By ANB | Date 9/26/24

(E) Metal stud wall check:

(E) 400S162-43 @ 16" o.c.



LOAD = 1500#

LOAD = 47 PSF

ADD'L LOAD TO STUDS:

LOAD = 47 PSF (16")(4'-0")
= 250#

(E) LOAD TO STUDS = (15 PSF + 23.1 PSF)(16")(30'-8" + 10'-6")/2
= 1045#

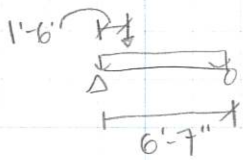
TOTAL = 1045# + 250# = 1295#

SR = 0.98 ∴ OK

(E) LVL CHECK:

(E) LOADING = (18 PSF DL + 30 PSF)(32"/12)(45'-8"/2) = 2.92 K OR 1096 PLF

New load reaction = 375#



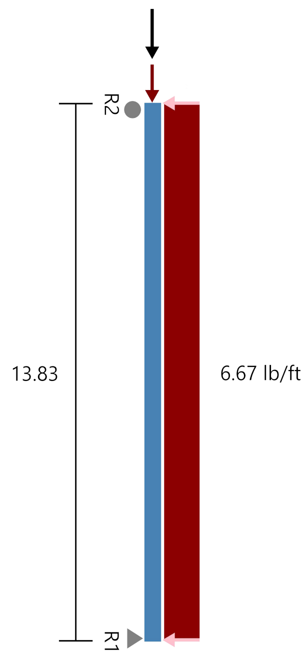
(E) LVL: (2) 1 3/4 x 9 1/4 LVL

→ SR = 0.483

(E) LVL OK.

Project Name: 24091 - Kimberly AG Shop
Model: (E) metal stud check
Code: 2012 NASPEC [AISI S100-2012]

Date: 10/07/2024
Simpson Strong-Tie® CFS Designer™ 5.0.0.1



Section : 400S162-43 (33 ksi) @ 16" o.c. Single C Stud (punched)		
Maxo = 686.0 ft-lb	**Va =** 1739.1 lb	**I =** 0.89 in^4

Loads have not been modified for strength checks
Loads have not been modified for deflection calculations

Bridging Connectors - Design Method =AISI S100

Span	Axial KyLy, KtLt	Flexual, Distortional	Connector	Stress Ratio
Span	60.0", 60.0"	60.0", 166.0"	N/A	-

Web Crippling

Support	Load (lb)	Bearing (in)	Pa (lb)	M (ft-lbs)	Max Int.	Stiffener?
R2	46.1	--Shear Connection w/ clip--				NO
R1	46.1	--Stud/Track Design, Ref Connectors--				NO

Gravity Load

Type	Load (lb)
Uniform	6.67plf
P1y	250.00lb @ 13.83ft
P2y	1045.00lb @ 13.83ft

	Code Check	Required	Allowed	Interaction	Notes
Span	Max. Axial, lbs	1387.2(c)	2139.4(c)	65%	KΦ=0.00 lb-in/in Max KL/r = 105
	Max. Shear, lbs	46.1	809.6	6%	Shear (Punched)
	Max. Moment (MaFy, Ma-dist), ft-lbs	159.5	686.0	23%	MaFy (control),KΦ=0.00 lb-in/in
	Moment Stability, ft-lbs	159.5	616.6	26%	
	Shear/Moment	0.23	1.00	23%	Shear 0.0, Moment 159.5
	Axial/Moment	0.97	1.00	97%	Axial 1342.8(c), Moment 159.3
	Deflection Span, in	0.209	--meets L/795--		

Support	Rx(lb)	Ry(lb)	Simpson Strong-Tie Connector	Connector Interaction	Anchor Interaction
R2	46.1	0.0	SCB45.5(2) & (2) #12-24 SST X or XL to A36 Steel	7.56 %	4.14 %
R1	46.1	1387.2	400T125-33 (33) & (1) .157" SST PDPA/PDPAT-62KP to steel (3/16" to 1/2" thickness)	10.68 %	20.94 %

* Reference catalog for connector and anchor requirement notes as well as screw placement requirements

Wood Beam

Project File: 24091 - kimberly ag shop.ec6

LIC# : KW-06015770, Build:20.23.10.02 ARW ENGINEERS (c) ENERCALC INC 1983-2023

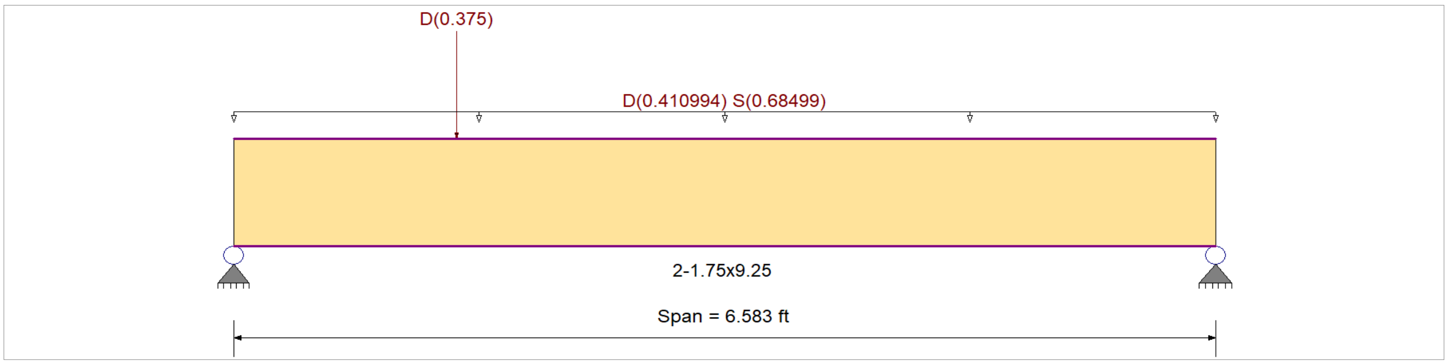
DESCRIPTION: (E) LVL Check

CODE REFERENCES

Calculations per NDS 2018, IBC 2021, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2600 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	2600 psi	Ebend- xx	1900 ksi
		Fc - Prll	2510 psi	Eminbend - xx	965.71 ksi
Wood Species :	iLevel Truss Joist	Fc - Perp	750 psi		
Wood Grade :	MicroLam LVL 1.9 E	Fv	285 psi		
		Ft	1555 psi	Density	42.01 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight NOT internally calculated and added
Point Load : D = 0.3750 k @ 1.50 ft
Uniform Load : D = 0.0180, S = 0.030 ksf, Tributary Width = 22.833 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.483 : 1	Maximum Shear Stress Ratio	=	0.432 : 1
Section used for this span		**2-1.75x9.25**	Section used for this span		**2-1.75x9.25**
fb: Actual	=	1,495.81 psi	fv: Actual	=	141.52 psi
F'b	=	3,097.74 psi	F'v	=	327.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	3.219 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.066 in	Ratio =	1190	>=360	Span: 1 : S Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	0.112 in	Ratio =	706	>=180	Span: 1 : +D+S
Max Upward Total Deflection	0 in	Ratio =	0	<180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 6.559 ft	**1**	0.250	0.240	0.90	1.00	1.00	1.00	1.036	1.00	1.00	1.00	2.52	605.0	2,424.3	1.33	61.5	256.5
Length = 0.02403 ft	**1**	0.003	0.240	0.90	1.00	1.00	1.00	1.036	1.00	1.00	1.00	0.03	8.3	2,424.3	1.12	61.5	256.5
+D+S					1.00	1.00	1.00	1.036	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 6.559 ft	**1**	0.483	0.432	1.15	1.00	1.00	1.00	1.036	1.00	1.00	1.00	6.22	1,495.8	3,097.7	3.05	141.5	327.8
Length = 0.02403 ft	**1**	0.007	0.432	1.15	1.00	1.00	1.00	1.036	1.00	1.00	1.00	0.09	21.3	3,097.7	2.85	141.5	327.8
+D+0.750S					1.00	1.00	1.00	1.036	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 6.559 ft	**1**	0.411	0.371	1.15	1.00	1.00	1.00	1.036	1.00	1.00	1.00	5.29	1,272.9	3,097.7	2.62	121.5	327.8
Length = 0.02403 ft	**1**	0.006	0.371	1.15	1.00	1.00	1.00	1.036	1.00	1.00	1.00	0.07	18.0	3,097.7	2.42	121.5	327.8

Wood Beam

Project File: 24091 - kimberly ag shop.ec6

LIC# : KW-06015770, Build:20.23.10.02 ARW ENGINEERS (c) ENERCALC INC 1983-2023

DESCRIPTION: (E) LVL Check

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
+0.60D					1.00	1.00	1.00	1.036	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 6.559 ft	**1**	0.084	0.081	1.60	1.00	1.00	1.00	1.036	1.00	1.00	1.00	1.51	363.0	4,309.9	0.80	36.9	456.0
Length = 0.02403 ft	**1**	0.001	0.081	1.60	1.00	1.00	1.00	1.036	1.00	1.00	1.00	0.02	5.0	4,309.9	0.67	36.9	456.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.1118	3.292		0.0000	0.000

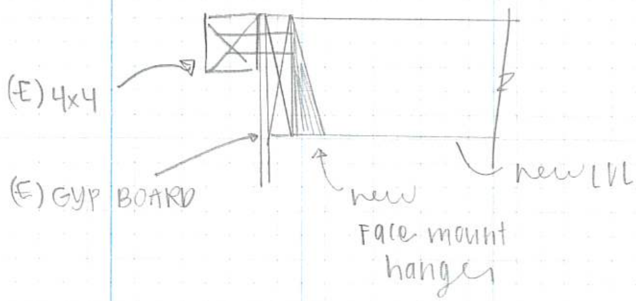
Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	3.897	3.693
Max Upward from Load Combinations	3.897	3.693
Max Upward from Load Cases	2.255	2.255
D Only	1.642	1.438
+D+S	3.897	3.693
+D+0.750S	3.333	3.129
+0.60D	0.985	0.863
S Only	2.255	2.255

New LVL attachment to (E):

* Max Reaction @ each end = 750#/2 = 375#



SDWS screw cap = 265 w/ (1) SDWS screw

use (2) @ 12" o.c. ∴ OK.