kpff

February 13, 2024

Ms. Serena Lim
Rowell Brokaw Architects
1203 Willamette St., #210
Eugene, OR 97401

Re: Oregon State University
Azalea House 2nd Floor Remodel

Dear Serena,

Attached please find calculation sheets 1 through 96, dated February 13, 2024, which verify the structural adequacy of the OSU Azalea House Remodel Project as shown on drawings S-001 through S-602, dated February 13, 2024. Design is based on the requirements of the 2022 Oregon Structural Specialty Code, which is based on the 2021 International Building Code.

If you have any questions or need further information, please call me.

Sincerely,

Michael Arellano, PE

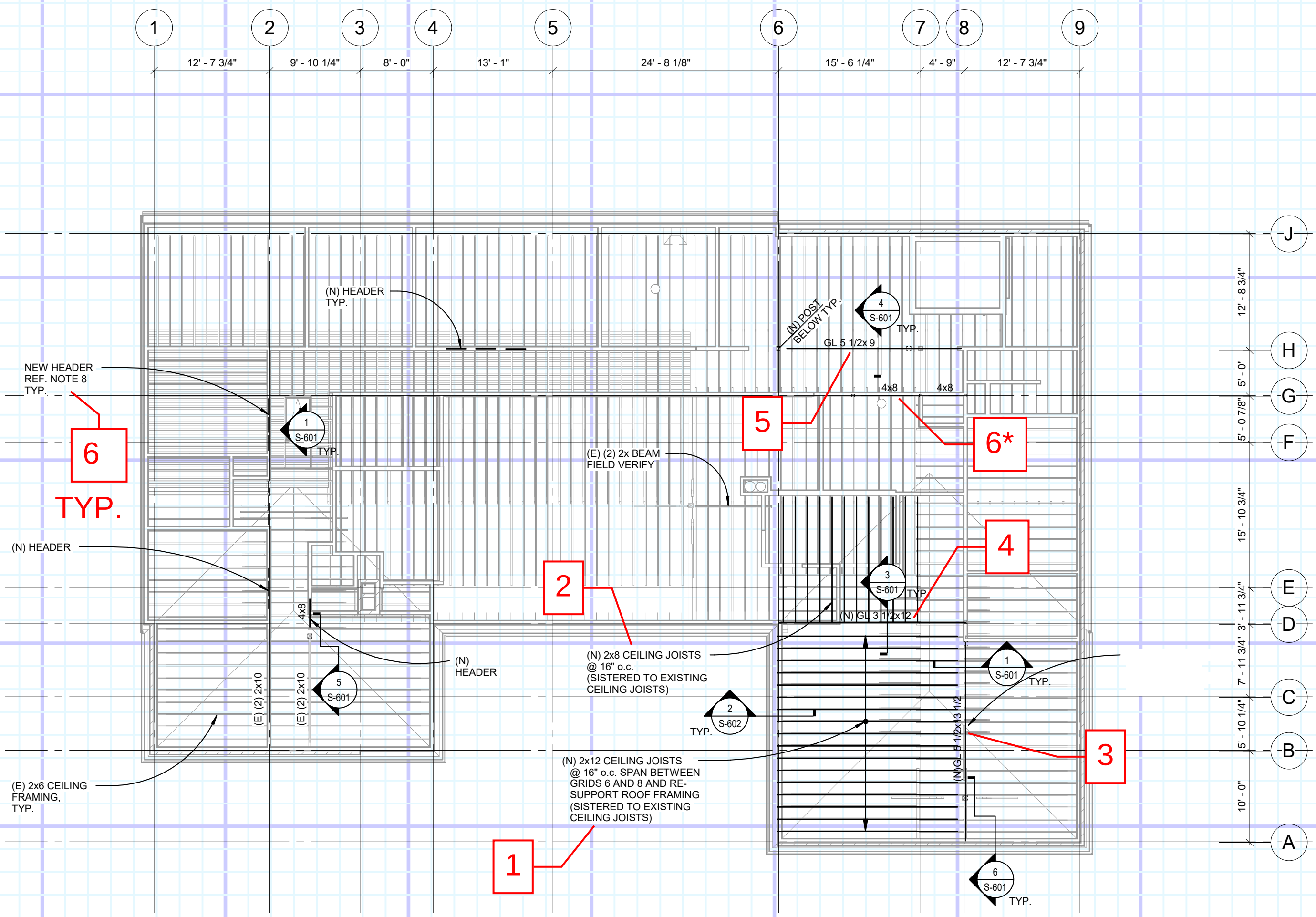
Project No. 10022300346

STRUCTURAL
REGISTERED PROFESSIONAL
ENGINEER
15,773
OREGON
JAN. 21, 1992
JERALD L. ABDIE
EXPIRES 12/31/25

kpff Portland, Oregon	Project OSU Azalea House	By MAA	Sheet No.
	Location Corvallis, OR	Date 02/09/24	
	Client Rowell Brokaw	Revised	Job No.
		Date	223346

CEILING LOADING

DL = 10 PSF
LL = 25 PSF



* Indicates critical element

Ceiling Framing plan
NTS

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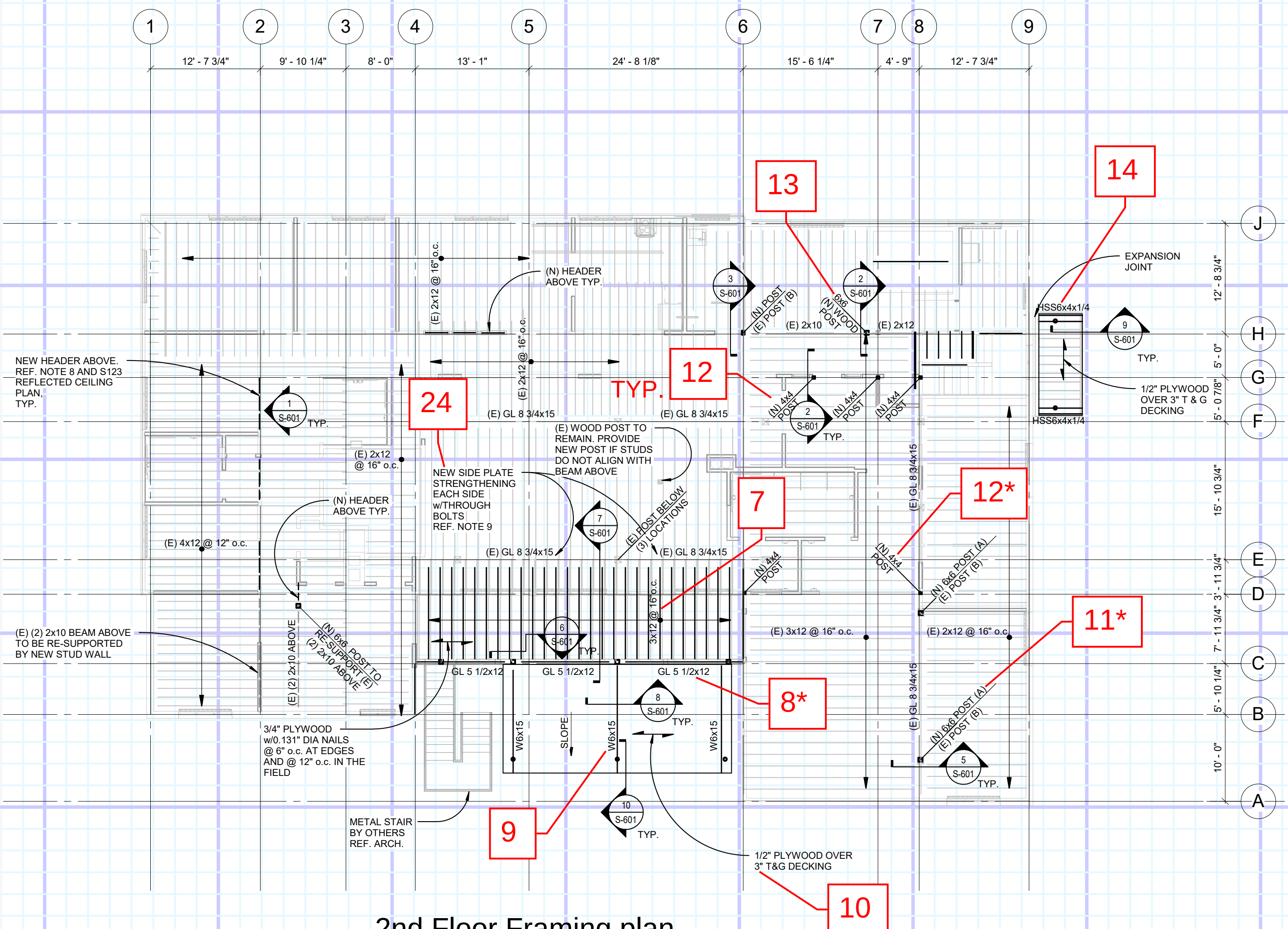
2nd FLOOR LOADING

DL = 15 PSF
EXISTING LL = 50 PSF +15 PSF Partitions (per DCI Renovation Dwgs, dated Feb 2, 2015)

OUTDOOR DECK
DL = 30 PSF
LL = 100 PSF

CANOPIES
DL = 15 PSF
LL = 25 PSF



2nd Floor Framing plan
NTS

* Indicates critical element

kpff
Portland, Oregon

Project	OSU Azalea House	By	MAA	Sheet No.
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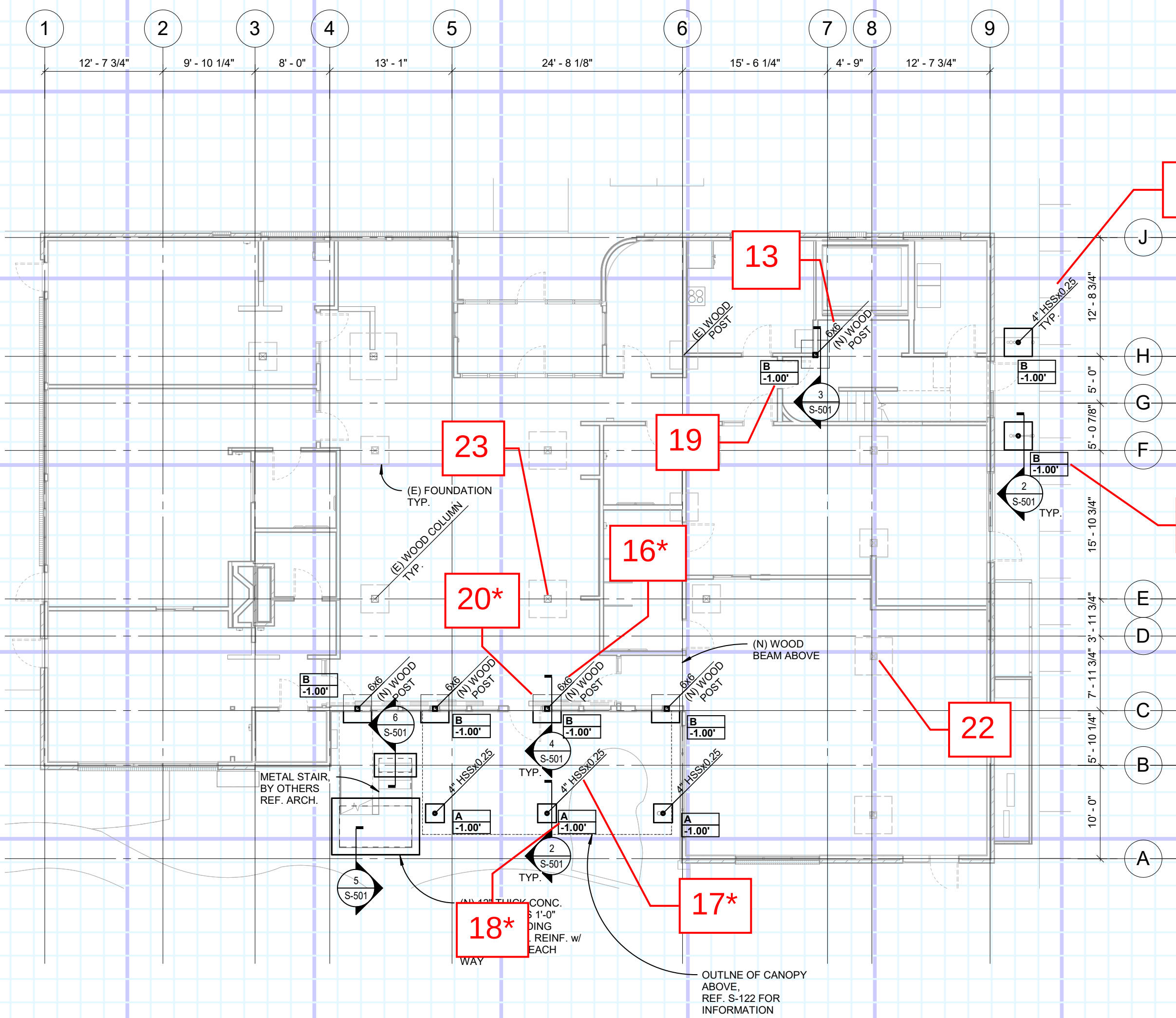
Client	Rowell Brokaw	Revised		Job No.
		Date		223346

FOUNDATIONS

Allowable Bearing Pressure = 2500 PSF Existing Foundations Per DCI Remodel Drawings (Reference Attached)

1500 PSF New Foundations per OSSC 1806.3



Foundation plan
NTS

* Indicates critical element

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CEILING FRAMING

1) Typical 2x12 Ceiling Joist

-Span 20'-0" WDL = 10 psf
WLL = 25 psf

2x12 DF No.2 @ 16" o.c. (See attached)

2) Typical 2x8 Ceiling Joist

-Span 13'-6" WDL = 10 psf
WLL = 25 psf

2x8 DF No.2 @ 16" o.c. (See attached)

3) Ceiling Beam grid 8

-Span 17' + 4.5' Cantilever WDL = 16' x 10 psf = 160 plf
WLL = 16' x 25 psf = 400 plf

GL5-1/2x13-1/2 24FV8 (see attached)

4) Ceiling Beam grid D

-Span 19.5' WDL = 7' x 10 psf = 70 plf
WLL = 7' x 25 psf = 175 plf

GL3-1/2x12 24FV4 (see attached)

5) Ceiling Beam grid H

-Span 14.5' WDL = 9' x 10 psf = 90 plf
WLL = 9' x 25 psf = 225 plf

GL5-1/2x9 24FV4 (see attached)

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CEILING FRAMING

6) Typical Headers

-Span 4' WDL = 10' x 10 psf = 100 plf
WLL = 10' x 25 psf = 250 plf

(2) 2x6 DF No.2 (See attached)

-Span 6' WDL = 10' x 10 psf = 100 plf
WLL = 10' x 25 psf = 250 plf

4x6 DF No.2 (See attached)

-Span 8' WDL = 10' x 100 psf = 100 plf
WLL = 10' x 250 psf = 250 plf

4x8 DF No.2 (See attached)

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2nd FL FRAMING

7) Typical Deck Joist

-Span 12'-0" WDL = 30 psf applied from 0' to 8'
WLL = 100 psf applied from 0' to 8'

3x12 DF No.2 @ 16" o.c. (See attached)

8) Critical Deck Beam Grid C

-Span 12'-75" +1.5' cantilever WDL = (8' x 30 psf) x 8'/12' = 160 plf
WLL = (8' x 100 psf) x 8'/12' = 533 plf

GL5-1/2x12 24FV8 (see attached)

9) Critical Canopy Beam

-Span 11'-0" +1.5' cantilever WDL = 12' x 15pf = 180 plf
WLL = 12' x 25 psf = 300 plf

W6x15 (see attached)

10) 3" T&G Decking

-Span 12'-0" WTotal = 40 PSF Allowable WLL = 95 psf OK//

(see attached)

11) Critical Grid 8 Column

PDL = 2408 lbs
PLL = 5438 lbs

6x6 DF No. 1 (see attached)

12) Critical Grid D and G Post

PDL = 780 lbs
PLL = 1714 lbs

4x4 DF No. 2 (see attached)

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2nd FL FRAMING

13) Critical Grid H Post

PDL = 18'/2 x 20'/2 x 10 psf = 900 lbs
PLL = 18'/2 x 20'/2 x 25 psf = 2250 lbs

6x6 DF No. 1 (see attached)

14) Typical Entry Canopy Beam

WDL = 12'/2 x 15 psf = 90 plf
WLL = 12'/2 x 25 psf = 150 plf

HSS6x4x1/4 (see attached)

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ALLOWABLE SOIL BEARING PRESSURE FOR EXISTING FOUNDATIONS PER DCI RENOVATION DRAWINGS DATED 2/9/2015

SOILS AND FOUNDATIONS

REFERENCE STANDARDS: Conform to OSSC Chapter 18 "Soils and Foundations."

GEOTECHNICAL REPORT: Recommendations contained in a memorandum by Foundation Engineering, Inc. dated December 9th, 2014 were used for design.

CONTRACTOR'S RESPONSIBILITIES: Contractor shall be responsible to review the Geotechnical Report and shall follow the recommendations specified therein including, but not limited to, subgrade preparations, pile installation procedures, ground water management and steep slope Best Management Practices."

GEOTECHNICAL SUBGRADE INSPECTION: The Geotechnical Engineer shall inspect all sub-grades and prepared soil bearing surfaces, prior to placement of foundation reinforcing steel and concrete. Geotechnical Engineers shall provide a letter to the owner stating that soils are adequate to support the "Allowable Foundation Bearing Pressure(s)" shown below.

DESIGN SOIL VALUES:

Safety Factor per Soils Report 1.5

Allowable Foundation Bearing Pressure .... 2500 PSF – Native

FOUNDATIONS and FOOTINGS: Foundations shall bear on either on competent native soil or compacted structural fill as per the geotechnical report. Exterior perimeter footings shall bear not less than 18 inches below finish grade, unless otherwise specified by the geotechnical engineer and/or the building official.

REGISTERED PROFESSIONAL STRUCTURAL ENGINEER 74858PE OREGON MARCH 23, 2011 SHIRLEY CHALUPA

EXPIRES: 12-31-15

DCI ENGINEERS
400 SW 6TH AVENUE • SUITE 605
PORTLAND, OREGON 97204
PHONE: (503) 242-2448 • FAX: (503) 242-2449
WEBSITE: www.dci-engineers.com
CIVIL / STRUCTURAL

Hennebery Eddy Architects Inc.
921 SW WASHINGTON STREET SUITE 250
PORTLAND OREGON 97205
503 227 4860 TEL
503 227 4920 FAX
www.henneberyeddy.com

CONSTRUCTION SET

OSU STUDENT COMMUNITY CENTER

DCI Project no. 14031-0087
HEA Project no. 14011
Date: FEBRUARY 9, 2015

Revisions:

Drawn by: IK
Checked by: SC
Sheet:

STRUCTURAL GENERAL NOTES LEGEND & ABBREVIATIONS

S100

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COLUMNS AND FOOTINGS

15) Entry Canopy Column

PDL = 12'/2 x 5' x 15 psf = 450 lbs
PLL = 12'/2 x 5' x 25 psf = 750 lbs

Eccentricity = 6"

MDL = 450 x 0.5' = 225 lb-ft
MLL = 750 x 0.5' = 375 lb-ft

Seismic Loading V = 0.703 (R=1 cantilever column) x 450 lbs = 320 lbs

ME = 320 lbs x 10' = 3200 lbs-ft

4" Diameter HSS x0.25" (see attached)

16) Critical Grid C Column

- Deck Loads

PDL = 12' x 160 plf = 1920 lbs
PLL = 12' x 533 plf = 6396 lbs

- Canopy Loads

PDL = 11'/2 x 24'/2 x 15 psf = 990 lbs
PLL = 11'/2 x 24'/2 x 25 psf = 1650 lbs

6x6 DF No. 1 (see attached)

17) Critical Back Canopy Column

PDL = 11'/2 x 24'/2 x 15 psf = 990 lbs
PLL = 11'/2 x 24'/2 x 25 psf = 1650 lbs

4" Diameter HSS x0.25" (see attached)

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COLUMNS AND FOOTINGS

18) Critical Back Canopy Footing

PDL = 11'/2 x 24'/2 x 15 psf = 990 lbs
PLL = 11'/2 x 24'/2 x 25 psf = 1650 lbs

Type A Footing 2'x2'x1' w/(3) #5 bars each way (see attached)

19) Grid H Footing

PDL = 900 lbs
PLL = 2250 lbs

Type B Footing 3'x3'x1' w/(4) #5 bars each way (see attached)

20) Grid C Footing

PDL = 1920 lbs
PLL = 6396 lbs

Type B Footing 3'x3'x1' w/(4) #5 bars each way (see attached)

21) Entry Canopy Footing

PDL = 450 lbs
PLL = 750 lbs

Eccentricity = 6"

MDL = 225 lb-ft
MLL = 375 lb-ft

Seismic Loading ME = 3200 lbs-ft

Type B Footing 3'x3'x1' w/(4) #5 bars each way (see attached)

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COLUMNS AND FOOTINGS

22) Check Existing Column and Footing Grid 8

-Ceiling Loads

PDL = 28'/2 x 32'/2 x 10 psf = 2240 lbs
PLL = 28'/2 x 32'/2 x 25 psf = 5600 lbs

2nd Floor Loads

PDL = 28'/2 x 32'/2 x 15 psf = 3360 lbs
PLL = 28'/2 x 32'/2 x 65 psf = 14560 lbs

Existing Column GL8-3/4x9 L2 OK// (see attached)

Existing Footing 4'x4'x1' w/(5) #5 bars each way OK// (see attached)

23) Check Existing Column and Footing Grid E

-Ceiling Loads

PDL = 28'/2 x 36'/2 x 10 psf = 2520 lbs
PLL = 28'/2 x 36'/2 x 25 psf = 6300 lbs

2nd Floor Loads

PDL = 28'/2 x 36'/2 x 15 psf = 3780 lbs
PLL = 28'/2 x 36'/2 x 65 psf = 16380 lbs

Deck Loads

PDL = 18' x 80 plf = 1440 lbs
PLL = 18' x 267 plf = 4806 lbs

Existing Column GL8-3/4x9 L2 OK// (see attached)

Existing Footing 4'x4'x1' w/(5) #5 bars each way OK// (see attached)

GL BEAM REINFORCING GRID E

24) Check Pl reinforcing for loads imposed by new deck loading

Use Full Deck DL and LL (conservatively) rather than difference from original low roof framing and snow load.

-Deck Loads

PDL = 30psf x 8' x (4'/12') = 80 plf
PLL = 100 psf x 8' x (4'/12') = 267 plf

Existing GL Beam span = 18'

Mmax = 347 plf x 18'^2 / 8 = 14,054 lb-ft or 168.6 k-in

Plate Reinforcing 1/4" thick x 15" each side.

Splates = 2x b x d^2 / 6 = 2 x .25 x 15^2 /6 = 18.75 in^3

- Check plate stress Fb = 168.6 / 18.75 = 8.99 ksi

Fallowable = 0.6x 36 = 24 ksi   OK//

	COMPANY	PROJECT
WoodWorks® SOFTWARE FOR WOOD DESIGN	KPFF Consulting Engineers Feb. 12, 2024 10:34	1 - Typical Ceiling Joist.wwb

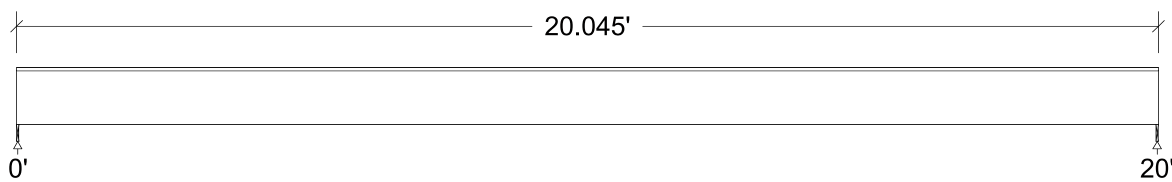
Design Check Calculation Sheet

WoodWorks Sizer 2019 (Update 1)

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	Location [ft] End	Magnitude Start	Magnitude End	Unit
DL	Dead	Full Area				10.00(16.0")		psf
LL	Snow	Full Area				25.00(16.0")		psf
Self-weight	Dead	Full UDL				4.0		plf

Maximum Reactions (lbs), Bearing Capacities (lbs) and Bearing Lengths (in) :



Unfactored:			
Dead	174		174
Snow	334		334
Factored:			
Total	508		508
Bearing:			
Capacity			
Joist	508		508
Support	635		635
Des ratio			
Joist	1.00		1.00
Support	0.80		0.80
Load comb	#2		#2
Length	0.54		0.54
Min req'd	0.54		0.54
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	1.25		1.25
Fcp sup	625		625

1 - Typical Ceiling Joist

Lumber-soft, D.Fir-L, No.2, 2x12 (1-1/2"x11-1/4")

Supports: All - Timber-soft Beam, D.Fir-L No.2

Floor joist spaced at 16.0" c/c; Total length: 20.05'; Clear span: 19.955'; Volume = 2.3 cu.ft.

Lateral support: top = continuous, bottom = at supports; Repetitive factor: applied where permitted (refer to online help);

This section PASSES the design code check.

WARNING: Member length exceeds typical stock length of 18.0 [ft]

Analysis vs. Allowable Stress and Deflection using NDS 2018 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 41	Fv' = 207	psi	fv/Fv' = 0.20
Bending(+)	fb = 961	Fb' = 1190	psi	fb/Fb' = 0.81
Live Defl'n	0.42 = L/569	0.67 = L/360	in	0.63
Total Defl'n	0.64 = L/374	1.00 = L/240	in	0.64

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL	CF	Cfu	Cr	Cfrt	Ci	Cn	LC#
Fv'	180	1.15	1.00	1.00	-	-	-	-	1.00	1.00	1.00	2
Fb'+	900	1.15	1.00	1.00	1.000	1.000	-	1.15	1.00	1.00	-	2
Fcp'	625	-	1.00	1.00	-	-	-	-	1.00	1.00	-	-
E'	1.6 million		1.00	1.00	-	-	-	-	1.00	1.00	-	2
Emin'	0.58 million		1.00	1.00	-	-	-	-	1.00	1.00	-	2

CRITICAL LOAD COMBINATIONS:

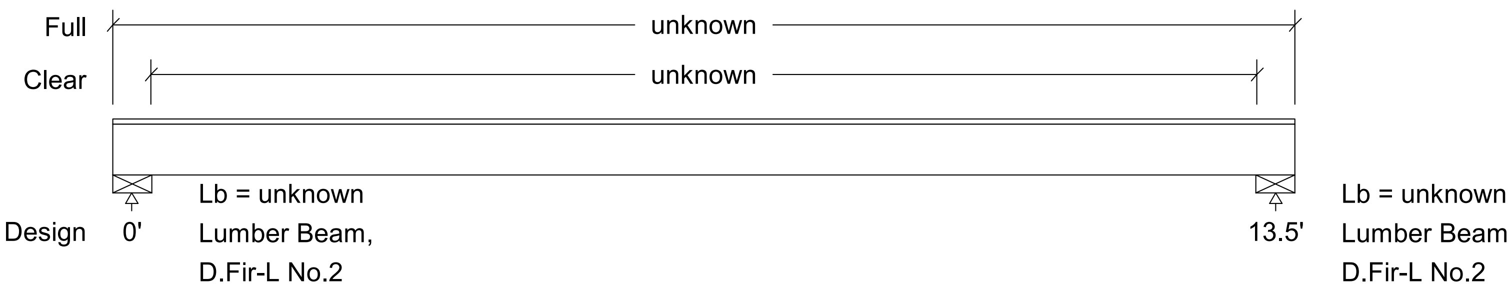
```
 Shear     : LC #2  = D+S
 Bending(+): LC #2  = D+S
 Deflection: LC #2  = D+S  (live)
             LC #2  = D+S  (total)
 Bearing   : Support 1 - LC #2 = D+S
             Support 2 - LC #2 = D+S
 D=dead L=live S=snow W=wind I=impact Lr=roof live Lc=concentrated E=earthquake
 All LC's are listed in the Analysis output
 Load combinations: ASD Basic from ASCE 7-16 2.4 / IBC 2018 1605.3.2
```

CALCULATIONS:

```
 V max = 507, V design = 458 lbs; M(+) = 2534 lbs-ft
 EI = 284.76e06 lb-in^2
 "Live" deflection is due to all non-dead loads (live, wind, snow…)
 Total deflection = 1.0 dead + "live"
```

Design Notes:

1. WoodWorks analysis and design are in accordance with the ICC International Building Code (IBC 2018), the National Design Specification (NDS 2018), and NDS Design Supplement.
2. Please verify that the default deflection limits are appropriate for your application.
3. Sawn lumber bending members shall be laterally supported according to the provisions of NDS Clause 4.4.1.


Full
unknown
Clear
unknown
Lb = unknown
Lumber Beam,
D.Fir-L No.2
Design
0'
13.5'
Lb = unknown
Lumber Beam,
D.Fir-L No.2

WoodWorks® SOFTWARE FOR WOOD DESIGN	**COMPANY** KPFF Consulting Engineers Feb. 12, 2024 10:36	**PROJECT** 3 - Ceiling Beam Grid 8.wwb

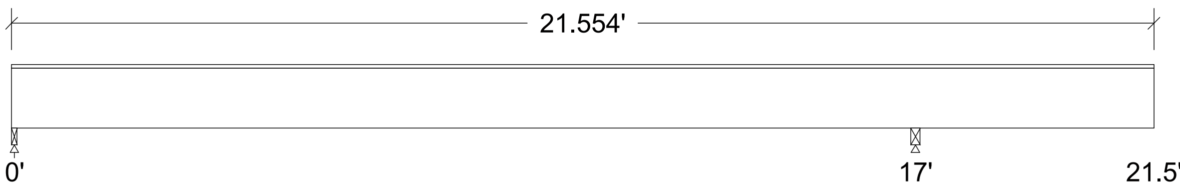
Design Check Calculation Sheet

WoodWorks Sizer 2019 (Update 1)

Loads:

Load	Type	Distribution	Pattern	Location [ft] Start	End	Magnitude Start	End	Unit
DL	Dead	Full UDL	No			160.0		plf
LL	Snow	Full UDL	No			400.0		plf
Self-weight	Dead	Full UDL	No			17.1		plf

Maximum Reactions (lbs), Bearing Capacities (lbs) and Bearing Lengths (in) :



Unfactored:					
Dead	1408		2408		
Snow	3183		5438		
Factored:					
Total	4592		7846		
Bearing:					
Capacity					
Beam	4592		8980		
Support	4716		7846		
Des ratio					
Beam	1.00		0.87		
Support	0.97		1.00		
Load comb	#2		#2		
Length	1.28		2.14		
Min req'd	1.28		2.14**		
Cb	1.00		1.18		
Cb min	1.00		1.18		
Cb support	1.07		1.07		
Fcp sup	625		625		

**Minimum bearing length governed by the required width of the supporting member.

3 - Ceiling GL Beam Grid 8

Glulam-Balanced, West Species, 24F-V8 DF, 5-1/2"x13-1/2"

Supports: All - Timber-soft Beam, D.Fir-L No.2

Total length: 21.55'; Clear span: 16.857', 4.411'; Volume = 11.1 cu.ft.; 9 laminations, 5-1/2" maximum width,

Lateral support: top = continuous, bottom = at supports;

This section PASSES the design code check.

Analysis vs. Allowable Stress and Deflection using NDS 2018 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 92	Fv' = 305	psi	fv/Fv' = 0.30
Bending(+)	fb = 1295	Fb' = 2760	psi	fb/Fb' = 0.47
Bending(-)	fb = 420	Fb' = 2674	psi	fb/Fb' = 0.16
Deflection:				
Interior Live	0.31 = L/662	0.57 = L/360	in	0.54
Total	0.44 = L/459	0.85 = L/240	in	0.52
Cantil. Live	-0.21 = L/259	0.30 = L/180	in	0.69
Total	-0.30 = L/179	0.45 = L/120	in	0.67

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrt	Notes	Cn*Cvr	LC#
Fv'	265	1.15	1.00	1.00	-	-	-	-	1.00	1.00	1.00	2
Fb'+	2400	1.15	1.00	1.00	1.000	1.000	-	-	1.00	1.00	-	2
Fb'-	2400	1.15	1.00	1.00	0.969	1.000	-	-	1.00	1.00	-	2
Fcp'	650	-	1.00	1.00	-	-	-	-	1.00	-	-	-
E'	1.8 million		1.00	1.00	-	-	-	-	1.00	-	-	2
Eminy'	0.85 million		1.00	1.00	-	-	-	-	1.00	-	-	2

CRITICAL LOAD COMBINATIONS:

```
 Shear     : LC #2  = D+S
 Bending(+): LC #2  = D+S
 Bending(-): LC #2  = D+S
 Deflection: LC #2  = D+S  (live)
             LC #2  = D+S  (total)
 Bearing   : Support 1 - LC #2 = D+S
             Support 2 - LC #2 = D+S
 D=dead L=live S=snow W=wind I=impact Lr=roof live Lc=concentrated E=earthquake
 All LC's are listed in the Analysis output
 Load combinations: ASD Basic from ASCE 7-16 2.4 / IBC 2018 1605.3.2
```

CALCULATIONS:

```
 V max = 5249, V design = 4550 lbs; M(+) = 18028 lbs-ft; M(-) = 5843 lbs-ft
 EI = 2029.78e06 lb-in^2
 "Live" deflection is due to all non-dead loads (live, wind, snow…)
 Total deflection = 1.0 dead + "live"
 Lateral stability(-):  Lu = 17.00'  Le = 27.88'  RB = 12.2; Lu based on full span
```

Design Notes:

1. WoodWorks analysis and design are in accordance with the ICC International Building Code (IBC 2018), the National Design Specification (NDS 2018), and NDS Design Supplement.
2. Please verify that the default deflection limits are appropriate for your application.
3. Glulam design values are for materials conforming to ANSI 117-2015 and manufactured in accordance with ANSI A190.1-2012
4. Grades with equal bending capacity in the top and bottom edges of the beam cross-section are recommended for continuous beams.
5. GLULAM: bxd = actual breadth x actual depth.
6. Glulam Beams shall be laterally supported according to the provisions of NDS Clause 3.3.3.
7. GLULAM: bearing length based on smaller of Fcp(tension), Fcp(comp'n).

WoodWorks® SOFTWARE FOR WOOD DESIGN	**COMPANY** KPFF Consulting Engineers Feb. 12, 2024 10:38	**PROJECT** 4 - Ceiling Beam Grid D.wwb

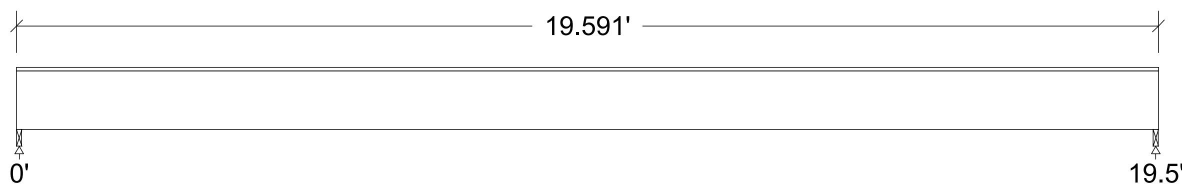
Design Check Calculation Sheet

WoodWorks Sizer 2019 (Update 1)

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	End	Magnitude Start	End	Unit
DL	Dead	Full UDL				70.0		plf
LL	Snow	Full UDL				175.0		plf
Self-weight	Dead	Full UDL				9.7		plf

Maximum Reactions (lbs), Bearing Capacities (lbs) and Bearing Lengths (in) :



	0'		19.5'
Unfactored:			
Dead	780		780
Snow	1714		1714
Factored:			
Total	2494		2494
Bearing:			
Capacity			
Beam	2494		2494
Support	2655		2655
Des ratio			
Beam	1.00		1.00
Support	0.94		0.94
Load comb	#2		#2
Length	1.10		1.10
Min req'd	1.10		1.10
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	1.11		1.11
Fcp sup	625		625

4 - Ceiling GL Beam Grid D

Glulam-Unbalan., West Species, 24F-V4 DF, 3-1/2"x12"

Supports: All - Timber-soft Beam, D.Fir-L No.2

Total length: 19.59'; Clear span: 19.409'; Volume = 5.7 cu.ft.; 8 laminations, 3-1/2" maximum width,

Lateral support: top = continuous, bottom = at supports;

This section PASSES the design code check.

Analysis vs. Allowable Stress and Deflection using NDS 2018 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 79	Fv' = 305	psi	fv/Fv' = 0.26
Bending(+)	fb = 1729	Fb' = 2760	psi	fb/Fb' = 0.63
Live Defl'n	0.63 = L/372	0.65 = L/360	in	0.97
Total Defl'n	0.91 = L/256	0.98 = L/240	in	0.94

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrt	Notes	Cn*Cvr	LC#
Fv'	265	1.15	1.00	1.00	-	-	-	-	1.00	1.00	1.00	2
Fb'+	2400	1.15	1.00	1.00	1.000	1.000	-	-	1.00	1.00	-	2
Fcp'	650	-	1.00	1.00	-	-	-	-	1.00	-	-	-
E'	1.8 million		1.00	1.00	-	-	-	-	1.00	-	-	2
Eminy'	0.85 million		1.00	1.00	-	-	-	-	1.00	-	-	2

CRITICAL LOAD COMBINATIONS:

```
 Shear     : LC #2  = D+S
 Bending(+): LC #2  = D+S
 Deflection: LC #2  = D+S  (live)
             LC #2  = D+S  (total)
 Bearing   : Support 1 - LC #2 = D+S
             Support 2 - LC #2 = D+S
 D=dead L=live S=snow W=wind I=impact Lr=roof live Lc=concentrated E=earthquake
 All LC's are listed in the Analysis output
 Load combinations: ASD Basic from ASCE 7-16 2.4 / IBC 2018 1605.3.2
```

CALCULATIONS:

```
 V max = 2483, V design = 2217 lbs; M(+) = 12105 lbs-ft
 EI = 907.19e06 lb-in^2
 "Live" deflection is due to all non-dead loads (live, wind, snow…)
 Total deflection = 1.0 dead + "live"
```

Design Notes:

1. WoodWorks analysis and design are in accordance with the ICC International Building Code (IBC 2018), the National Design Specification (NDS 2018), and NDS Design Supplement.
2. Please verify that the default deflection limits are appropriate for your application.
3. Glulam design values are for materials conforming to ANSI 117-2015 and manufactured in accordance with ANSI A190.1-2012
4. GLULAM: bxd = actual breadth x actual depth.
5. Glulam Beams shall be laterally supported according to the provisions of NDS Clause 3.3.3.
6. GLULAM: bearing length based on smaller of Fcp(tension), Fcp(comp'n).

WoodWorks® SOFTWARE FOR WOOD DESIGN	**COMPANY** KPFF Consulting Engineers Feb. 12, 2024 10:44	**PROJECT** 5 - Ceiling Beam Grid H.wwb

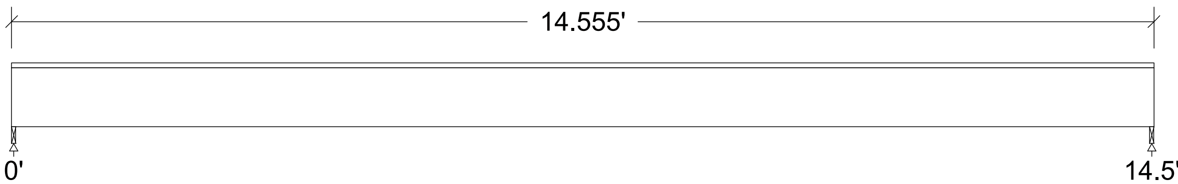
Design Check Calculation Sheet

WoodWorks Sizer 2019 (Update 1)

Loads:

Load	Type	Distribution	Pat- tern	Location [ft] Start	End	Magnitude Start	End	Unit
DL	Dead	Full UDL				90.0		plf
LL	Snow	Full UDL				225.0		plf
Self-weight	Dead	Full UDL				11.4		plf

Maximum Reactions (lbs), Bearing Capacities (lbs) and Bearing Lengths (in) :



	0'		14.5'
Unfactored:			
Dead	738		738
Snow	1637		1637
Factored:			
Total	2375		2375
Bearing:			
Capacity			
Beam	2375		2375
Support	2439		2439
Des ratio			
Beam	1.00		1.00
Support	0.97		0.97
Load comb	#2		#2
Length	0.66		0.66
Min req'd	0.66		0.66
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	1.07		1.07
Fcp sup	625		625

5 - Ceiling GL Beam Grid H

Glulam-Unbalan., West Species, 24F-V4 DF, 5-1/2"x9"

Supports: All - Timber-soft Beam, D.Fir-L No.2

Total length: 14.56'; Clear span: 14.445'; Volume = 5.0 cu.ft.; 6 laminations, 5-1/2" maximum width,

Lateral support: top = continuous, bottom = at supports;

This section PASSES the design code check.

Analysis vs. Allowable Stress and Deflection using NDS 2018 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 64	Fv' = 305	psi	fv/Fv' = 0.21
Bending(+)	fb = 1386	Fb' = 2760	psi	fb/Fb' = 0.50
Live Defl'n	0.37 = L/467	0.48 = L/360	in	0.77
Total Defl'n	0.54 = L/322	0.73 = L/240	in	0.74

Additional Data:

```
FACTORS:  F/E(psi) CD    CM    Ct    CL     CV    Cfu    Cr    Cfrt Notes Cn*Cvr LC#
 Fv'       265    1.15  1.00  1.00   -      -      -      -    1.00  1.00  1.00   2
 Fb'+     2400    1.15  1.00  1.00  1.000  1.000   -      -    1.00  1.00   -     2
 Fcp'      650     -    1.00  1.00   -      -      -      -    1.00   -     -     -
 E'        1.8 million  1.00  1.00   -      -      -      -    1.00   -     -     2
 Eminy'   0.85 million  1.00  1.00   -      -      -      -    1.00   -     -     2
```

CRITICAL LOAD COMBINATIONS:

```
 Shear     : LC #2  = D+S
 Bending(+): LC #2  = D+S
 Deflection: LC #2  = D+S  (live)
             LC #2  = D+S  (total)
 Bearing   : Support 1 - LC #2 = D+S
             Support 2 - LC #2 = D+S
 D=dead L=live S=snow W=wind I=impact Lr=roof live Lc=concentrated E=earthquake
 All LC's are listed in the Analysis output
 Load combinations: ASD Basic from ASCE 7-16 2.4 / IBC 2018 1605.3.2
```

CALCULATIONS:

```
 V max = 2366, V design = 2113 lbs; M(+) = 8578 lbs-ft
 EI = 601.42e06 lb-in^2
 "Live" deflection is due to all non-dead loads (live, wind, snow…)
 Total deflection = 1.0 dead + "live"
```

Design Notes:

1. WoodWorks analysis and design are in accordance with the ICC International Building Code (IBC 2018), the National Design Specification (NDS 2018), and NDS Design Supplement.
2. Please verify that the default deflection limits are appropriate for your application.
3. Glulam design values are for materials conforming to ANSI 117-2015 and manufactured in accordance with ANSI A190.1-2012
4. GLULAM: bxd = actual breadth x actual depth.
5. Glulam Beams shall be laterally supported according to the provisions of NDS Clause 3.3.3.
6. GLULAM: bearing length based on smaller of Fcp(tension), Fcp(comp'n).

WoodWorks® SOFTWARE FOR WOOD DESIGN	**COMPANY** KPFF Consulting Engineers Feb. 12, 2024 10:56	**PROJECT** 6 - Typical Header 4ft Span.wwb

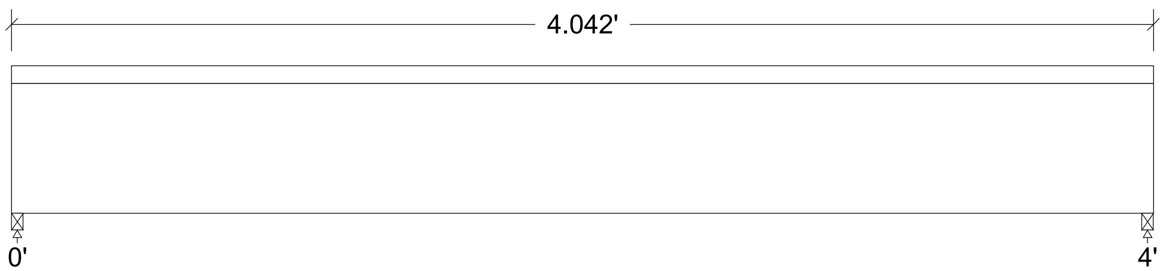
Design Check Calculation Sheet

WoodWorks Sizer 2019 (Update 1)

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	Location [ft] End	Magnitude Start	Magnitude End	Unit
DL	Dead	Full UDL				50.0		plf
LL	Snow	Full UDL				125.0		plf
Self-weight	Dead	Full UDL				2.0		plf

Maximum Reactions (lbs), Bearing Capacities (lbs) and Bearing Lengths (in) :



Unfactored:			
Dead	105		105
Snow	253		253
Factored:			
Total	358		358
Bearing:			
Capacity			
Joist	469		469
Support	586		586
Des ratio			
Joist	0.76		0.76
Support	0.61		0.61
Load comb	#2		#2
Length	0.50*		0.50*
Min req'd	0.50*		0.50*
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	1.25		1.25
Fcp sup	625		625

*Minimum bearing length setting used: 1/2" for end supports

6 - Typical Header 4ft Span

Lumber-soft, D.Fir-L, No.2, 2x6 (1-1/2"x5-1/2")

Supports: All - Timber-soft Beam, D.Fir-L No.2

Floor joist spaced at 12.0" c/c; Total length: 4.04'; Clear span: 3.958'; Volume = 0.2 cu.ft.

Lateral support: top = continuous, bottom = at supports;

This section PASSES the design code check.

Analysis vs. Allowable Stress and Deflection using NDS 2018 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 49	Fv' = 207	psi	fv/Fv' = 0.24
Bending(+)	fb = 562	Fb' = 1345	psi	fb/Fb' = 0.42
Live Defl'n	0.02 = < L/999	0.13 = L/360	in	0.16
Total Defl'n	0.03 = < L/999	0.20 = L/240	in	0.15

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL	CF	Cfu	Cr	Cfrt	Ci	Cn	LC#
Fv'	180	1.15	1.00	1.00	-	-	-	-	1.00	1.00	1.00	2
Fb'+	900	1.15	1.00	1.00	1.000	1.300	-	1.00	1.00	1.00	-	2
Fcp'	625	-	1.00	1.00	-	-	-	-	1.00	1.00	-	-
E'	1.6 million		1.00	1.00	-	-	-	-	1.00	1.00	-	2
Emin'	0.58 million		1.00	1.00	-	-	-	-	1.00	1.00	-	2

CRITICAL LOAD COMBINATIONS:

```
 Shear     : LC #2  = D+S
 Bending(+): LC #2  = D+S
 Deflection: LC #2  = D+S  (live)
             LC #2  = D+S  (total)
 Bearing   : Support 1 - LC #2 = D+S
             Support 2 - LC #2 = D+S
 D=dead L=live S=snow W=wind I=impact Lr=roof live Lc=concentrated E=earthquake
 All LC's are listed in the Analysis output
 Load combinations: ASD Basic from ASCE 7-16 2.4 / IBC 2018 1605.3.2
```

CALCULATIONS:

```
 V max = 354, V design = 269 lbs; M(+) = 354 lbs-ft
 EI = 33.27e06 lb-in^2
 "Live" deflection is due to all non-dead loads (live, wind, snow…)
 Total deflection = 1.0 dead + "live"
```

Design Notes:

1. WoodWorks analysis and design are in accordance with the ICC International Building Code (IBC 2018), the National Design Specification (NDS 2018), and NDS Design Supplement.
2. Please verify that the default deflection limits are appropriate for your application.
3. Sawn lumber bending members shall be laterally supported according to the provisions of NDS Clause 4.4.1.

	COMPANY	PROJECT
WoodWorks® SOFTWARE FOR WOOD DESIGN	KPFF Consulting Engineers Feb. 12, 2024 10:57	6 - Typical Header 6ft Span.wwb

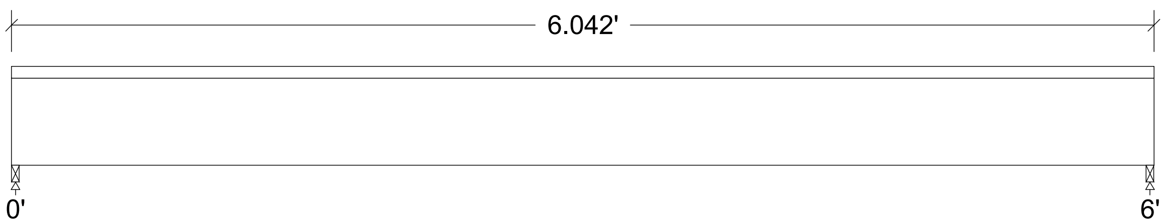
Design Check Calculation Sheet

WoodWorks Sizer 2019 (Update 1)

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	Location [ft] End	Magnitude Start	Magnitude End	Unit
DL	Dead	Full UDL				100.0		plf
LL	Snow	Full UDL				250.0		plf
Self-weight	Dead	Full UDL				4.6		plf

Maximum Reactions (lbs), Bearing Capacities (lbs) and Bearing Lengths (in) :



	0'		6'
Unfactored:			
Dead	316		316
Snow	755		755
Factored:			
Total	1071		1071
Bearing:			
Capacity			
Joist	1094		1094
Support	1211		1211
Des ratio			
Joist	0.98		0.98
Support	0.88		0.88
Load comb	#2		#2
Length	0.50*		0.50*
Min req'd	0.50*		0.50*
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	1.11		1.11
Fcp sup	625		625

*Minimum bearing length setting used: 1/2" for end supports

6 - Typical Header 6ft Span

Lumber-soft, D.Fir-L, No.2, 4x6 (3-1/2"x5-1/2")

Supports: All - Timber-soft Beam, D.Fir-L No.2

Floor joist spaced at 12.0" c/c; Total length: 6.04'; Clear span: 5.958'; Volume = 0.8 cu.ft.

Lateral support: top = continuous, bottom = at supports;

This section PASSES the design code check.

Analysis vs. Allowable Stress and Deflection using NDS 2018 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 70	Fv' = 207	psi	fv/Fv' = 0.34
Bending(+)	fb = 1085	Fb' = 1345	psi	fb/Fb' = 0.81
Live Defl'n	0.09 = L/766	0.20 = L/360	in	0.47
Total Defl'n	0.13 = L/540	0.30 = L/240	in	0.44

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL	CF	Cfu	Cr	Cfrt	Ci	Cn	LC#
Fv'	180	1.15	1.00	1.00	-	-	-	-	1.00	1.00	1.00	2
Fb'+	900	1.15	1.00	1.00	1.000	1.300	-	1.00	1.00	1.00	-	2
Fcp'	625	-	1.00	1.00	-	-	-	-	1.00	1.00	-	-
E'	1.6 million		1.00	1.00	-	-	-	-	1.00	1.00	-	2

CRITICAL LOAD COMBINATIONS:

```
 Shear     : LC #2  = D+S
 Bending(+): LC #2  = D+S
 Deflection: LC #2  = D+S  (live)
             LC #2  = D+S  (total)
 Bearing   : Support 1 - LC #2 = D+S
             Support 2 - LC #2 = D+S
 D=dead L=live S=snow W=wind I=impact Lr=roof live Lc=concentrated E=earthquake
 All LC's are listed in the Analysis output
 Load combinations: ASD Basic from ASCE 7-16 2.4 / IBC 2018 1605.3.2
```

CALCULATIONS:

```
 V max = 1064, V design = 894 lbs; M(+) = 1596 lbs-ft
 EI = 77.64e06 lb-in^2
 "Live" deflection is due to all non-dead loads (live, wind, snow…)
 Total deflection = 1.0 dead + "live"
```

Design Notes:

1. WoodWorks analysis and design are in accordance with the ICC International Building Code (IBC 2018), the National Design Specification (NDS 2018), and NDS Design Supplement.
2. Please verify that the default deflection limits are appropriate for your application.
3. Sawn lumber bending members shall be laterally supported according to the provisions of NDS Clause 4.4.1.

	COMPANY	PROJECT
WoodWorks® SOFTWARE FOR WOOD DESIGN	KPFF Consulting Engineers Feb. 12, 2024 10:57	6 - Typical Header 8ft Span.wwb

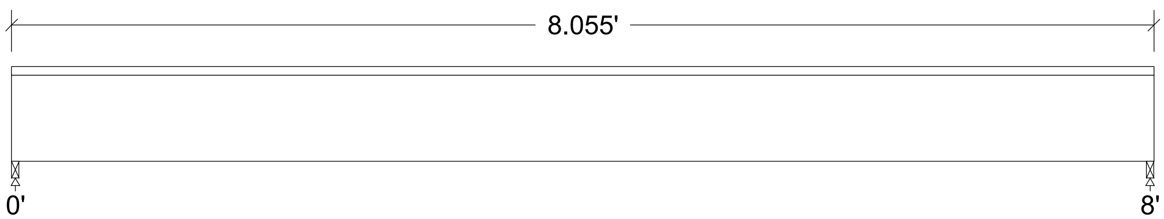
Design Check Calculation Sheet

WoodWorks Sizer 2019 (Update 1)

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	Location [ft] End	Magnitude Start	Magnitude End	Unit
DL	Dead	Full UDL				100.0		plf
LL	Snow	Full UDL				250.0		plf
Self-weight	Dead	Full UDL				6.0		plf

Maximum Reactions (lbs), Bearing Capacities (lbs) and Bearing Lengths (in) :



	0'		8'
Unfactored:			
Dead	427		427
Snow	1007		1007
Factored:			
Total	1434		1434
Bearing:			
Capacity			
Joist	1434		1434
Support	1587		1587
Des ratio			
Joist	1.00		1.00
Support	0.90		0.90
Load comb	#2		#2
Length	0.66		0.66
Min req'd	0.66		0.66
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	1.11		1.11
Fcp sup	625		625

6 - Typical Header 8ft Span

Lumber-soft, D.Fir-L, No.2, 4x8 (3-1/2"x7-1/4")

Supports: All - Timber-soft Beam, D.Fir-L No.2

Floor joist spaced at 12.0" c/c; Total length: 8.05'; Clear span: 7.945'; Volume = 1.4 cu.ft.

Lateral support: top = continuous, bottom = at supports;

This section PASSES the design code check.

Analysis vs. Allowable Stress and Deflection using NDS 2018 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 71	Fv' = 207	psi	fv/Fv' = 0.34
Bending(+)	fb = 1115	Fb' = 1345	psi	fb/Fb' = 0.83
Live Defl'n	0.13 = L/740	0.27 = L/360	in	0.49
Total Defl'n	0.18 = L/520	0.40 = L/240	in	0.46

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL	CF	Cfu	Cr	Cfrt	Ci	Cn	LC#
Fv'	180	1.15	1.00	1.00	-	-	-	-	1.00	1.00	1.00	2
Fb'+	900	1.15	1.00	1.00	1.000	1.300	-	1.00	1.00	1.00	-	2
Fcp'	625	-	1.00	1.00	-	-	-	-	1.00	1.00	-	-
E'	1.6 million		1.00	1.00	-	-	-	-	1.00	1.00	-	2
Emin'	0.58 million		1.00	1.00	-	-	-	-	1.00	1.00	-	2

CRITICAL LOAD COMBINATIONS:

```
 Shear     : LC #2  = D+S
 Bending(+): LC #2  = D+S
 Deflection: LC #2  = D+S  (live)
             LC #2  = D+S  (total)
 Bearing   : Support 1 - LC #2 = D+S
             Support 2 - LC #2 = D+S
 D=dead L=live S=snow W=wind I=impact Lr=roof live Lc=concentrated E=earthquake
 All LC's are listed in the Analysis output
 Load combinations: ASD Basic from ASCE 7-16 2.4 / IBC 2018 1605.3.2
```

CALCULATIONS:

```
 V max = 1424, V design = 1199 lbs; M(+) = 2848 lbs-ft
 EI = 177.83e06 lb-in^2
 "Live" deflection is due to all non-dead loads (live, wind, snow…)
 Total deflection = 1.0 dead + "live"
```

Design Notes:

1. WoodWorks analysis and design are in accordance with the ICC International Building Code (IBC 2018), the National Design Specification (NDS 2018), and NDS Design Supplement.
2. Please verify that the default deflection limits are appropriate for your application.
3. Sawn lumber bending members shall be laterally supported according to the provisions of NDS Clause 4.4.1.

	COMPANY	PROJECT
WoodWorks® SOFTWARE FOR WOOD DESIGN	KPFF Consulting Engineers Feb. 12, 2024 11:32	7 - 2nd Fl Deck Joist.wwb

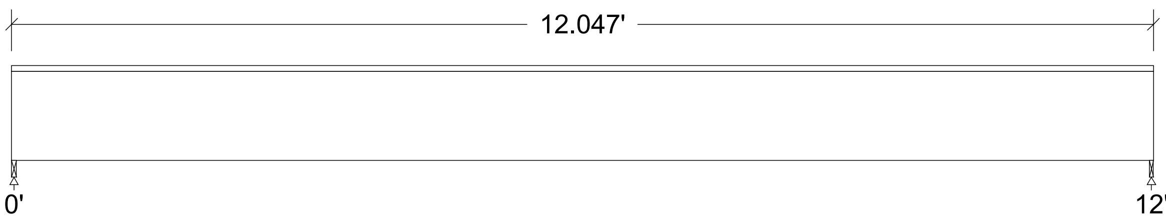
Design Check Calculation Sheet

WoodWorks Sizer 2019 (Update 1)

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	Location [ft] End	Magnitude Start	Magnitude End	Unit
DL	Dead	Partial Area		0.03	8.03	30.00(16.0")		psf
LL	Live	Partial Area		0.03	8.03	100.00(16.0")		psf
Self-weight	Dead	Full UDL				6.7		plf

Maximum Reactions (lbs), Bearing Capacities (lbs) and Bearing Lengths (in) :



Unfactored:			
Dead	253		147
Live	711		356
Factored:			
Total	965		502
Bearing:			
Capacity			
Joist	965		781
Support	1109		898
Des ratio			
Joist	1.00		0.64
Support	0.87		0.56
Load comb	#2		#2
Length	0.62		0.50*
Min req'd	0.62		0.50*
Cb	1.00		1.00
Cb min	1.00		1.00
Cb support	1.15		1.15
Fcp sup	625		625

*Minimum bearing length setting used: 1/2" for end supports

7 - 2nd Fl Deck Joist

Lumber-soft, D.Fir-L, No.2, 3x12 (2-1/2"x11-1/4")

Supports: All - Timber-soft Beam, D.Fir-L No.2

Floor joist spaced at 16.0" c/c; Total length: 12.05'; Clear span: 11.953'; Volume = 2.4 cu.ft.

Lateral support: top = continuous, bottom = at supports; Repetitive factor: applied where permitted (refer to online help);

This section PASSES the design code check.

Analysis vs. Allowable Stress and Deflection using NDS 2018 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 43	Fv' = 180	psi	fv/Fv' = 0.24
Bending(+)	fb = 588	Fb' = 1035	psi	fb/Fb' = 0.57
Live Defl'n	0.10 = < L/999	0.40 = L/360	in	0.25
Total Defl'n	0.13 = < L/999	0.60 = L/240	in	0.22

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL	CF	Cfu	Cr	Cfrt	Ci	Cn	LC#
Fv'	180	1.00	1.00	1.00	-	-	-	-	1.00	1.00	1.00	2
Fb'+	900	1.00	1.00	1.00	1.000	1.000	-	1.15	1.00	1.00	-	2
Fcp'	625	-	1.00	1.00	-	-	-	-	1.00	1.00	-	-
E'	1.6 million		1.00	1.00	-	-	-	-	1.00	1.00	-	2
Emin'	0.58 million		1.00	1.00	-	-	-	-	1.00	1.00	-	2

CRITICAL LOAD COMBINATIONS:

```
 Shear     : LC #2  = D+L
 Bending(+): LC #2  = D+L
 Deflection: LC #2  = D+L  (live)
             LC #2  = D+L  (total)
 Bearing   : Support 1 - LC #2 = D+L
             Support 2 - LC #2 = D+L
 D=dead L=live S=snow W=wind I=impact Lr=roof live Lc=concentrated E=earthquake
 All LC's are listed in the Analysis output
 Load combinations: ASD Basic from ASCE 7-16 2.4 / IBC 2018 1605.3.2
```

CALCULATIONS:

```
 V max = 965, V design = 798 lbs; M(+) = 2584 lbs-ft
 EI = 474.60e06 lb-in^2
 "Live" deflection is due to all non-dead loads (live, wind, snow…)
 Total deflection = 1.0 dead + "live"
```

Design Notes:

1. WoodWorks analysis and design are in accordance with the ICC International Building Code (IBC 2018), the National Design Specification (NDS 2018), and NDS Design Supplement.
2. Please verify that the default deflection limits are appropriate for your application.
3. Sawn lumber bending members shall be laterally supported according to the provisions of NDS Clause 4.4.1.

	COMPANY	PROJECT
WoodWorks® SOFTWARE FOR WOOD DESIGN	KPFF Consulting Engineers Feb. 12, 2024 11:45	8 - 2nd Fl Beam Grid C.wwb

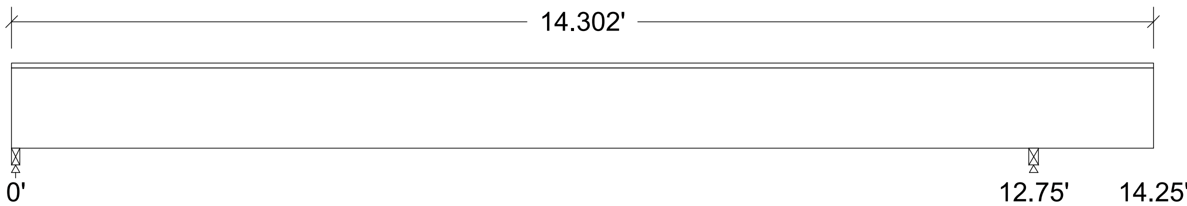
Design Check Calculation Sheet

WoodWorks Sizer 2019 (Update 1)

Loads:

Load	Type	Distribution	Pat-tern	Location [ft] Start	End	Magnitude Start	End	Unit
DL	Dead	Full UDL	No			160.0		plf
LL	Live	Full UDL	No			533.0		plf
Self-weight	Dead	Full UDL	No			15.2		plf

Maximum Reactions (lbs), Bearing Capacities (lbs) and Bearing Lengths (in) :



Unfactored:				
Dead	1110		1395	
Live	3379		4244	
Factored:				
Total	4489		5640	
Bearing:				
Capacity				
Beam	4489		6831	
Support	4610		5640	
Des ratio				
Beam	1.00		0.83	
Support	0.97		1.00	
Load comb	#2		#2	
Length	1.26		1.54	
Min req'd	1.26		1.54**	
Cb	1.00		1.24	
Cb min	1.00		1.24	
Cb support	1.07		1.07	
Fcp sup	625		625	

**Minimum bearing length governed by the required width of the supporting member.

8 - 2nd Fl GL Beam Grid C

Glulam-Balanced, West Species, 24F-V8 DF, 5-1/2"x12"

Supports: All - Timber-soft Beam, D.Fir-L No.2

Total length: 14.3'; Clear span: 12.634', 1.436'; Volume = 6.6 cu.ft.; 8 laminations, 5-1/2" maximum width,

Lateral support: top = continuous, bottom = at supports;

This section PASSES the design code check.

Analysis vs. Allowable Stress and Deflection using NDS 2018 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Shear	fv = 87	Fv' = 265	psi	fv/Fv' = 0.33
Bending(+)	fb = 1272	Fb' = 2400	psi	fb/Fb' = 0.53
Bending(-)	fb = 72	Fb' = 2364	psi	fb/Fb' = 0.03
Deflection:				
Interior Live	0.21 = L/711	0.43 = L/360	in	0.51
Total	0.29 = L/535	0.64 = L/240	in	0.45
Cantil. Live	-0.08 = L/228	0.10 = L/180	in	0.79
Total	-0.10 = L/172	0.15 = L/120	in	0.70

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL	CV	Cfu	Cr	Cfrt	Notes	Cn*Cvr	LC#
Fv'	265	1.00	1.00	1.00	-	-	-	-	1.00	1.00	1.00	2
Fb'+	2400	1.00	1.00	1.00	1.000	1.000	-	-	1.00	1.00	-	2
Fb'-	2400	1.00	1.00	1.00	0.985	1.000	-	-	1.00	1.00	-	2
Fcp'	650	-	1.00	1.00	-	-	-	-	1.00	-	-	-
E'	1.8 million		1.00	1.00	-	-	-	-	1.00	-	-	2
Eminy'	0.85 million		1.00	1.00	-	-	-	-	1.00	-	-	2

CRITICAL LOAD COMBINATIONS:

```
 Shear     : LC #2  = D+L
 Bending(+): LC #2  = D+L
 Bending(-): LC #2  = D+L
 Deflection: LC #2  = D+L  (live)
             LC #2  = D+L  (total)
 Bearing   : Support 1 - LC #2 = D+L
             Support 2 - LC #2 = D+L
 D=dead L=live S=snow W=wind I=impact Lr=roof live Lc=concentrated E=earthquake
 All LC's are listed in the Analysis output
 Load combinations: ASD Basic from ASCE 7-16 2.4 / IBC 2018 1605.3.2
```

CALCULATIONS:

```
 V max = 4577, V design = 3824 lbs; M(+) = 13995 lbs-ft; M(-) = 797 lbs-ft
 EI = 1425.58e06 lb-in^2
 "Live" deflection is due to all non-dead loads (live, wind, snow…)
 Total deflection = 1.0 dead + "live"
 Lateral stability(-):  Lu = 12.75'  Le = 21.38'  RB = 10.1; Lu based on full span
```

Design Notes:

1. WoodWorks analysis and design are in accordance with the ICC International Building Code (IBC 2018), the National Design Specification (NDS 2018), and NDS Design Supplement.
2. Please verify that the default deflection limits are appropriate for your application.
3. Glulam design values are for materials conforming to ANSI 117-2015 and manufactured in accordance with ANSI A190.1-2012
4. Grades with equal bending capacity in the top and bottom edges of the beam cross-section are recommended for continuous beams.
5. GLULAM: bxd = actual breadth x actual depth.
6. Glulam Beams shall be laterally supported according to the provisions of NDS Clause 3.3.3.
7. GLULAM: bearing length based on smaller of Fcp(tension), Fcp(comp'n).

kpff Portland, Oregon	Project OSU Azalea House	By MAA	Sheet No. 1
	Location Corvallis, OR	Date 2/12/2024	
	Client Rowell Brokaw	Revised	Job No. 223346
		Date	

9 - TYPICAL CANOPY BEAM

In accordance with AISC360 15th Edition published 2016 using the LRFD method

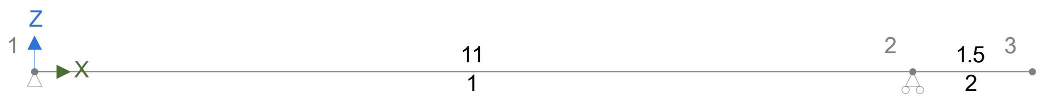
Tedds calculation version 4.4.08

ANALYSIS

Tedds calculation version 1.0.36

Geometry

Geometry (ft) - Steel (AISC) - W 6x15

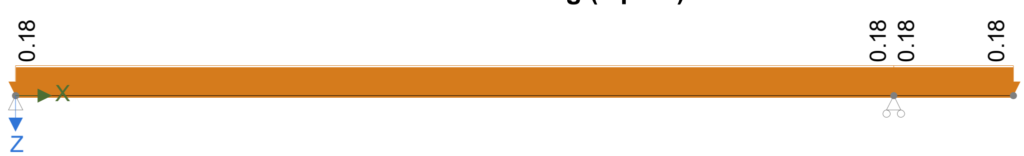


Span	Length (ft)	Section	Start Support	End Support
1	11	W 6x15	Pinned	Roller Pin X
2	1.5	W 6x15	Roller Pin X	Free
W 6x15: Area 4 in^2, Inertia Major 29 in^4, Inertia Minor 9 in^4, Shear area parallel to Minor 1 in^2, Shear area parallel to Major 3 in^2				
Steel (AISC): Density 490 lbm/ft^3, Youngs 29000 ksi, Shear 11200 ksi, Thermal 0.000012 °C^{-1}				

Loading

Self weight included

Dead - Loading (kips/ft)



Live - Loading (kips/ft)

Load combination factors

Load combination	Self Weight	Dead	Live
1.2D + 1.6L (Strength)	1.20	1.20	1.60
1.0D + 1.0L (Service)	1.00	1.00	1.00

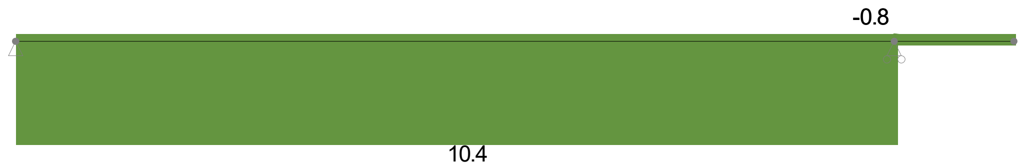
Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Dead	UDL	GlobalZ	0.18 kips/ft
Beam	Live	UDL	GlobalZ	0.3 kips/ft

Results

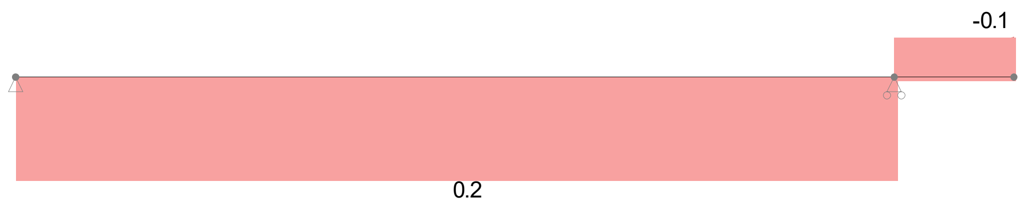
Forces

Strength combinations - Moment envelope (kip_ft)



Strength combinations - Shear envelope (kips)

Service combinations - Deflection envelope (in)



Resistance factors

Shear	ϕ_v = **1.00**
Flexure	ϕ_b = **0.90**
Tensile yielding	$\phi_{t,y}$ = **0.90**
Tensile rupture	$\phi_{t,r}$ = **0.75**
Compression	ϕ_c = **0.90**

Beam - Span 1 design

Section details

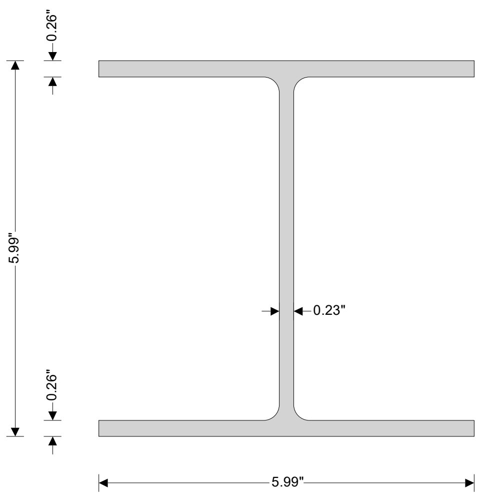
Section type W 6x15 (AISC 15th Edn (v15.0))

ASTM steel designation A992

Steel yield stress F_y = **50** ksi

Steel tensile stress F_u = **65** ksi

Modulus of elasticity E = **29000** ksi



W 6x15 (AISC 15th Edn (v15.0))
Section depth, d, 5.99 in
Section breadth, b_f, 5.99 in
Weight of section, Weight, 15 lbf/ft
Flange thickness, t_f, 0.26 in
Web thickness, t_w, 0.23 in
Area of section, A, 4.4 in²
Radius of gyration about x-axis, r_x, 2.56 in
Radius of gyration about y-axis, r_y, 1.45 in
Elastic section modulus about x-axis, S_x, 9.72 in³
Elastic section modulus about y-axis, S_y, 3.11 in³
Plastic section modulus about x-axis, Z_x, 10.8 in³
Plastic section modulus about y-axis, Z_y, 4.75 in³
Second moment of area about x-axis, I_x, 29.1 in⁴
Second moment of area about y-axis, I_y, 9.32 in⁴

Lateral restraint

Top flange has full lateral restraint

Bottom flange has lateral restraint at supports only

Classification of sections for local buckling - Section B4

Classification of flanges in flexure - Table B4.1b (case 10)

Width to thickness ratio $b_f / (2 \times t_f)$ = **11.52**

Limiting ratio for compact section $\lambda_{pff} = 0.38 \times \sqrt{[E / F_y]}$ = **9.15**

Limiting ratio for non-compact section $\lambda_{rff} = 1.0 \times \sqrt{[E / F_y]}$ = **24.08** Noncompact

Classification of web in flexure - Table B4.1b (case 15)

Width to thickness ratio $(d - 2 \times k) / t_w$ = **21.61**

Limiting ratio for compact section $\lambda_{pwf} = 3.76 \times \sqrt{[E / F_y]}$ = **90.55**

Limiting ratio for non-compact section $\lambda_{rwf} = 5.70 \times \sqrt{[E / F_y]}$ = **137.27** Compact

Section is noncompact in flexure

Check design at start of span

Design of members for shear - Chapter G

Required shear strength $V_{r,x}$ = **3.9** kips

Web area $A_w = d \times t_w$ = **1.378** in^2

Web plate buckling coefficient k_v = **5.34**

$(d - 2 \times k) / t_w <= 2.24 \times \sqrt{(E / F_y)}$

Web shear coefficient - eq G2-2 C_{v1} = **1.000**

Nominal shear strength - eq G2-1 $V_{n,x} = 0.6 \times F_y \times A_w \times C_{v1}$ = **41.3** kips

Resistance factor $\phi_v = 1.00$

Design shear strength $V_{c,x} = \phi_v \times V_{n,x}$ = **41.3** kips

$V_{r,x} / V_{c,x}$ = **0.093**

PASS - Design shear strength exceeds required shear strength

Check design 5ft 4.773in along span

Design of members for flexure - Chapter F

Required flexural strength $M_{r,x}$ = **10.4** kips_ft

Compression flange local buckling - Section F3.2

$\lambda = b_f / (2 \times t_f)$ = **11.519**

Nominal flexural strength for compression flange local buckling - eq F3-1

$M_{n,flb,x} = M_{p,x} - (M_{p,x} - 0.7 \times F_y \times S_x) \times (\lambda - \lambda_{pff}) / (\lambda_{rff} - \lambda_{pff})$ = **42.4** kips_ft

Design flexural strength - F1

Nominal flexural strength $M_{n,x} = M_{n,flb,x}$ = **42.4** kips_ft

Design flexural strength $M_{c,x} = \phi_b \times M_{n,x}$ = **38.1** kips_ft

$M_{r,x} / M_{c,x}$ = **0.273**

PASS - Design flexural strength exceeds required flexural strength

Check design at end of span

Design of members for shear - Chapter G

Required shear strength $V_{r,x}$ = **4** kips

Web area $A_w = d \times t_w$ = **1.378** in^2

Web plate buckling coefficient k_v = **5.34**

$(d - 2 \times k) / t_w <= 2.24 \times \sqrt{(E / F_y)}$

Web shear coefficient - eq G2-2 C_{v1} = **1.000**

Nominal shear strength - eq G2-1 $V_{n,x} = 0.6 \times F_y \times A_w \times C_{v1}$ = **41.3** kips

Resistance factor $\phi_v = 1.00$

Design shear strength $V_{c,x} = \phi_v \times V_{n,x}$ = **41.3** kips

$V_{r,x} / V_{c,x}$ = **0.097**

PASS - Design shear strength exceeds required shear strength

Design of members for flexure - Chapter F

Required flexural strength $M_{r,x}$ = **0.8** kips_ft

Plastic moment - eq F2-1 $M_{p,x} = F_y \times Z_x$ = **45** kips_ft

Lateral-torsional buckling - Section F3.1

Unbraced length L_b = **11** ft

Limiting unbraced length for yielding - eq F2-5 $L_p = 1.76 \times r_y \times \sqrt{(E / F_y)}$ = **5.122** ft

Distance between flange centroids h_o = **5.73** in

c = **1**

r_{ts} = **1.66** in

Limiting unbraced length for inelastic LTB - eq F2-6 $L_r = 1.95 \times r_{ts} \times E / (0.7 \times F_y) \times \sqrt{((J \times c / (S_x \times h_o)) + \sqrt{((J \times c / (S_x \times h_o))^2 + 6.76 \times (0.7 \times F_y / E)^2)})}$ = **16.482** ft

Moment at quarter point of segment M_A = **7.9** kips_ft

Moment at center-line of segment M_B = **10.4** kips_ft

Moment at three quarter point of segment M_C = **7.5** kips_ft

Maximum moment in segment M_{max} = **10.4** kips_ft

LTB modification factor - eq F1-1 $C_b = 12.5 \times M_{max} / (2.5 \times M_{max} + 3 \times M_A + 4 \times M_B + 3 \times M_C)$ = **1.143**

Nominal flexural strength for lateral-torsional buckling - eq F2-2

$M_{n,ltb,x} = \min(C_b \times (M_{p,x} - (M_{p,x} - 0.7 \times F_y \times S_x) \times (L_b - L_p) / (L_r - L_p)), M_{p,x})$ = **41.6** kips_ft

Compression flange local buckling - Section F3.2

$\lambda = b_f / (2 \times t_f)$ = **11.519**

Nominal flexural strength for compression flange local buckling - eq F3-1

$M_{n,flb,x} = M_{p,x} - (M_{p,x} - 0.7 \times F_y \times S_x) \times (\lambda - \lambda_{pff}) / (\lambda_{rff} - \lambda_{pff})$ = **42.4** kips_ft

Design flexural strength - F1

Nominal flexural strength $M_{n,x} = \min(M_{n,ltb,x}, M_{n,flb,x})$ = **41.6** kips_ft

Design flexural strength $M_{c,x} = \phi_b \times M_{n,x}$ = **37.4** kips_ft

$M_{r,x} / M_{c,x}$ = **0.021**

PASS - Design flexural strength exceeds required flexural strength

Consider Combination 2 - 1.0D + 1.0L (Service)

Check design 5ft 5.586in along span

Design of members for x-x axis deflection

Maximum deflection δ_x = **0.19** in

Allowable deflection $\delta_{x,Allowable} = L_{m1\_s1} / 360$ = **0.367** in

$\delta_x / \delta_{x,Allowable}$ = **0.519**

PASS - Allowable deflection exceeds design deflection

TABLE 3
BENDING STRESS AND MODULUS OF ELASTICITY
VALUES FOR HEAVY TIMBER DECKING SPECIES[a]

Species	Select Quality		Commercial Quality		Agency[d]
	Bending Stress[b] psi	Modulus of Elasticity[c] psi	Bending Stress[b] psi	Modulus of Elasticity[c] psi	
Cedar, Northern White	1100	800,000	950	700,000	1
Cedars, Western	1450	1,100,000	1200	1,000,000	3,4
Cedars, Western (North)	1400	1,100,000	1200	1,000,000	2
Coast Species	1450	1,500,000	1200	1,400,000	2
Douglas Fir-Larch	2000	1,800,000	1650	1,700,000	3,4
Douglas Fir-Larch (North)	2000	1,800,000	1650	1,700,000	2
Douglas Fir (South)	1900	1,400,000	1600	1,300,000	3
Fir, Balsam	1650	1,500,000	1400	1,300,000	1
Hem-Fir	1600	1,500,000	1350	1,400,000	3,4
Hem-Fir (North)	1500	1,500,000	1300	1,400,000	2
Hemlock, Eastern-Tamarack	1700	1,300,000	1450	1,100,000	1
Hemlock, Eastern-Tamarack (North)	1700	1,300,000	1450	1,100,000	2
Hemlock, Western	1750	1,600,000	1450	1,400,000	4
Hemlock, Western (North)	1750	1,600,000	1450	1,400,000	2
Northern Species	1050	1,100,000	875	1,000,000	2
Pine, Eastern White	1300	1,200,000	1100	1,100,000	1
Pine, Eastern White (North)	1050	1,200,000	875	1,100,000	2
Pine, Northern	1550	1,400,000	1300	1,300,000	1
Pine, Ponderosa	1450	1,300,000	1250	1,100,000	2
Pine, Red	1350	1,300,000	1100	1,200,000	2
Pine, Southern	1650	1,600,000	1650	1,600,000	5
Pine, Western White	1300	1,400,000	1050	1,300,000	2
Redwood, California	1700	1,100,000	1350	1,000,000	6
SPF, South	1350	1,400,000	1100	1,200,000	1,3
Spruce, Coast Sitka	1450	1,700,000	1200	1,500,000	2
Spruce, Eastern	1300	1,500,000	1100	1,400,000	1
Spruce-Pine-Fir	1400	1,500,000	1150	1,300,000	2
Spruce, Sitka	1500	1,500,000	1250	1,300,000	4
Western Woods	1300	1,200,000	1100	1,100,000	3

[a] The design values in bending (F_b), except for Redwood, are based on decking 4 in. thick. For other thicknesses, multiply by the size factor, C_F, as follows:

Thickness	C_F
2 in.	1.10
3 in.	1.04

Design values for visually graded decking are those recommended by the regional lumber rules writing agencies. These values are ased on decking that is used where the moisture content in-service will not exceed 19%. When the moisture content in-service exceeds 19% for an extended period of time, the tabular design values shall be multiplied by the wet service factor, C_M, as follows:

C_M

F_b	$F_{c\perp}$	E
0.85*	0.67	0.9

* When (F_b) (C_F) < 1150 psi, C_M = 1.0 for bending.

[b] Repetitive member use values.

[c] The tabulated values for modulus of elasticity are the average for the species grouping. For information concerning coefficient of variation of modulus of elasticity, see the appropriate grading rules for the species.

[d] Stresses listed are as assigned by the following grading rules agencies: NELMA (1), NLGA (Canadian) (2), WWPA (3), WCLIB (4), SPIB (5), and RIS (6).

[e] If specified as "close grain", California Redwood select decking is assigned a bending stress value of 1850 psi and a modulus of elasticity value of 1,400,000 psi when used at 19% M.C.

TABLE 6

THREE AND FOUR INCH NOMINAL THICKNESS
ALLOWABLE ROOF LOAD LIMITED BY BENDING
SIMPLE SPAN AND CONTROLLED RANDOM LAYUPS (3 or more spans)

Bending Stress	Allowable Uniformly Distributed Total Roof Load [a, c, e, f, g], psf																									
	3 inch Nominal Thickness [b] Span, ft													4 inch Nominal Thickness [d] Span, ft												
psi	8	9	10	11	12	13	14	15	16	17	18	19	20	8	9	10	11	12	13	14	15	16	17	18	19	20
875	114	90	73	60	51	43	37	32	28	25	22	20	18	223	176	143	118	99	84	73	64	56	49	44	40	36
950	124	98	79	65	55	47	40	35	31	27	24	22	20	242	192	155	128	108	92	79	69	61	54	48	43	39
1000	130	103	83	69	58	49	42	37	32	29	26	23	21	255	202	163	135	113	97	83	72	64	56	50	45	41
1050	137	108	88	72	61	52	45	39	34	30	27	24	22	268	212	172	142	119	101	88	76	67	59	53	48	43
1100	143	113	92	76	64	54	47	41	36	32	28	25	23	281	222	180	148	125	106	92	80	70	62	55	50	45
1150	150	118	96	79	66	57	49	42	37	33	30	26	24	293	232	188	155	130	111	96	83	73	65	58	52	47
1200	156	123	100	83	69	59	51	44	39	35	31	28	25	306	242	196	162	136	116	100	87	76	68	60	54	49
1250	163	129	104	86	72	62	53	46	41	36	32	29	26	319	252	204	169	142	121	104	91	80	71	63	56	51
1300	169	134	108	90	75	64	55	48	42	37	33	30	27	332	262	212	175	147	126	108	94	83	73	66	59	53
1350	176	139	112	93	78	66	57	50	44	39	35	31	28	344	272	220	182	153	130	112	98	86	76	68	61	55
1400	182	144	117	96	81	69	60	52	46	40	36	32	29	357	282	229	189	159	135	117	102	89	79	70	63	57
1450	189	149	121	100	84	71	62	54	47	42	37	33	30	370	292	237	196	164	140	121	105	92	82	73	66	59
1500	195	154	125	103	87	74	64	56	49	43	38	35	31	383	302	245	202	170	145	125	109	96	85	76	68	61
1550	202	159	129	107	90	76	66	57	50	45	40	36	32	396	312	253	209	176	150	129	112	99	88	78	70	63
1600	208	165	133	110	92	79	68	59	52	46	41	37	33	408	323	261	216	181	155	133	116	102	90	81	72	65
1650	215	170	138	114	$\boxed{95}$	81	70	61	54	48	42	38	34	421	333	270	223	187	159	138	120	105	93	83	75	67
1700	221	175	142	117	98	84	72	63	55	49	44	39	35	434	343	278	229	193	164	142	123	108	96	86	77	69
1750	228	180	146	120	101	86	74	65	57	50	45	40	36	447	353	286	236	198	169	146	127	112	99	88	79	71
1900	247	195	158	131	110	94	81	70	62	55	49	44	40	485	383	310	256	216	184	158	138	121	107	96	86	78
2000	260	206	167	138	116	99	85	74	65	58	51	46	42	510	403	327	270	227	193	167	145	128	113	101	90	82

[a] These load values may also be used for cantilevered pieces intermixed, combination simple span and two-span continuous, and two-span continuous layups.

[b] 2-1/2 in. net thickness. To determine allowable loads for 2-5/8 in. net thickness, multiply tabulated loads by 1.10.

[c] All spans to the right of the double line require special ordering of additional long lengths to assure that at least 20% of the decking is equal to the span length or longer.

[d] 3-1/2 in. net thickness.

[e] Duration of load, $C_D = 1.0$ used in this table. For other durations of load, adjust by the appropriate factor.

[f] No increase for size effect has been applied ($C_F = 1.00$). F_b values have been previously adjusted.

[g] Dry conditions of use.

	COMPANY	PROJECT
WoodWorks® SOFTWARE FOR WOOD DESIGN	KPFF Consulting Engineers Feb. 13, 2024 10:27	11 - Critical Grid 8 Column.wwc

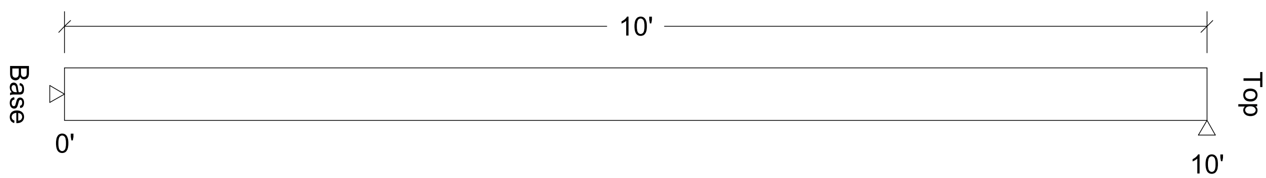
Design Check Calculation Sheet

WoodWorks Sizer 2019 (Update 1)

Loads:

Load	Type	Distribution	Location [ft] Start End	Magnitude Start End	Unit
DL	Dead	Axial	(Ecc. = 0.00")	2408	lbs
LL	Snow	Axial	(Ecc. = 0.00")	5438	lbs
Self-weight	Dead	Axial		72	lbs

Reactions (lbs):



11 - Critical grid 8 Column

Timber-soft, D.Fir-L, No.1, 6x6 (5-1/2"x5-1/2")

Support: Non-wood

Total length: 10.0'; Volume = 2.1 cu.ft.; Post or timber

Pinned base; Ke x Lb: 1.0 x 10.0 = 10.0 ft; Ke x Ld: 1.0 x 10.0 = 10.0 ft;

This section PASSES the design code check.

Analysis vs. Allowable Stress and Deflection using NDS 2018 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Axial	fc = 262	Fc' = 738	psi	fc/Fc' = 0.35
Axial Bearing	fc = 262	Fc* = 1150	psi	fc/Fc* = 0.23

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL/CP	CF	Cfu	Cr	Cfrt	Ci	LC#
Fc'	1000	1.15	1.00	1.00	0.641	1.000	-	-	1.00	1.00	2
Fc*	1000	1.15	1.00	1.00	-	1.000	-	-	1.00	1.00	2

CRITICAL LOAD COMBINATIONS:

```
Axial      : LC #2   = D+S
D=dead L=live S=snow W=wind I=impact Lr=roof live Lc=concentrated E=earthquake
All LC's are listed in the Analysis output
Load combinations: ASD Basic from ASCE 7-16 2.4 / IBC 2018 1605.3.2
```

Design Notes:

1. WoodWorks analysis and design are in accordance with the ICC International Building Code (IBC 2018), the National Design Specification (NDS 2018), and NDS Design Supplement.
2. Please verify that the default deflection limits are appropriate for your application.

	COMPANY	PROJECT
WoodWorks® SOFTWARE FOR WOOD DESIGN	KPFF Consulting Engineers Feb. 13, 2024 10:28	12 - Critical Grid D and G Post.wwc

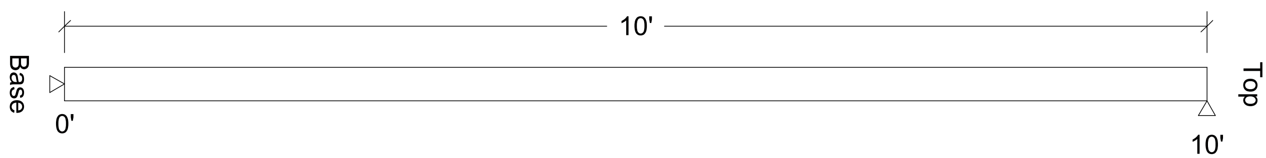
Design Check Calculation Sheet

WoodWorks Sizer 2019 (Update 1)

Loads:

Load	Type	Distribution	Location [ft] Start End	Magnitude Start End	Unit
DL	Dead	Axial	(Ecc. = 0.00")	780	lbs
LL	Snow	Axial	(Ecc. = 0.00")	1714	lbs
Self-weight	Dead	Axial		29	lbs

Reactions (lbs):



12 - Critical grid D and G Post

Lumber Post, D.Fir-L, No.2, 4x4 (3-1/2"x3-1/2")

Support: Non-wood

Total length: 10.0'; Volume = 0.9 cu.ft.

Pinned base; Ke x Lb: 1.0 x 10.0 = 10.0 ft; Ke x Ld: 1.0 x 10.0 = 10.0 ft;

This section PASSES the design code check.

Analysis vs. Allowable Stress and Deflection using NDS 2018 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Axial	fc = 206	Fc' = 384	psi	fc/Fc' = 0.54
Axial Bearing	fc = 206	Fc* = 1785	psi	fc/Fc* = 0.12

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL/CP	CF	Cfu	Cr	Cfrt	Ci	LC#
Fc'	1350	1.15	1.00	1.00	0.215	1.150	-	-	1.00	1.00	2
Fc*	1350	1.15	1.00	1.00	-	1.150	-	-	1.00	1.00	2

CRITICAL LOAD COMBINATIONS:

```
 Axial      : LC #2   = D+S
 D=dead L=live S=snow W=wind I=impact Lr=roof live Lc=concentrated E=earthquake
 All LC's are listed in the Analysis output
 Load combinations: ASD Basic from ASCE 7-16 2.4 / IBC 2018 1605.3.2
```

Design Notes:

1. WoodWorks analysis and design are in accordance with the ICC International Building Code (IBC 2018), the National Design Specification (NDS 2018), and NDS Design Supplement.
2. Please verify that the default deflection limits are appropriate for your application.

	COMPANY	PROJECT
WoodWorks® SOFTWARE FOR WOOD DESIGN	KPFF Consulting Engineers Feb. 13, 2024 10:29	13 - Critical Grid H Post.wwc

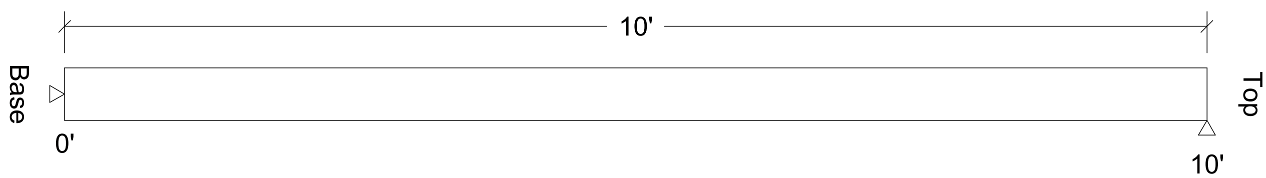
Design Check Calculation Sheet

WoodWorks Sizer 2019 (Update 1)

Loads:

Load	Type	Distribution	Location [ft] Start End	Magnitude Start End	Unit
DL	Dead	Axial	(Ecc. = 0.00")	900	lbs
LL	Snow	Axial	(Ecc. = 0.00")	2250	lbs
Self-weight	Dead	Axial		72	lbs

Reactions (lbs):



13 - Critical grid H Post

Timber-soft, D.Fir-L, No.1, 6x6 (5-1/2"x5-1/2")

Support: Non-wood

Total length: 10.0'; Volume = 2.1 cu.ft.; Post or timber

Pinned base; Ke x Lb: 1.0 x 10.0 = 10.0 ft; Ke x Ld: 1.0 x 10.0 = 10.0 ft;

This section PASSES the design code check.

Analysis vs. Allowable Stress and Deflection using NDS 2018 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Axial	fc = 107	Fc' = 738	psi	fc/Fc' = 0.14
Axial Bearing	fc = 107	Fc* = 1150	psi	fc/Fc* = 0.09

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL/CP	CF	Cfu	Cr	Cfrt	Ci	LC#
Fc'	1000	1.15	1.00	1.00	0.641	1.000	-	-	1.00	1.00	2
Fc*	1000	1.15	1.00	1.00	-	1.000	-	-	1.00	1.00	2

CRITICAL LOAD COMBINATIONS:

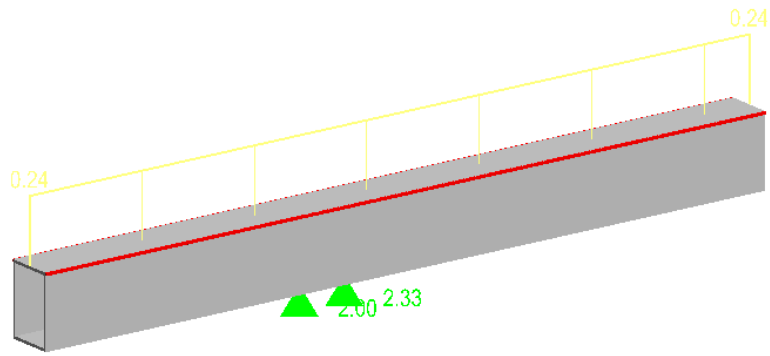
```
Axial      : LC #2   = D+S
D=dead L=live S=snow W=wind I=impact Lr=roof live Lc=concentrated E=earthquake
All LC's are listed in the Analysis output
Load combinations: ASD Basic from ASCE 7-16 2.4 / IBC 2018 1605.3.2
```

Design Notes:

1. WoodWorks analysis and design are in accordance with the ICC International Building Code (IBC 2018), the National Design Specification (NDS 2018), and NDS Design Supplement.
2. Please verify that the default deflection limits are appropriate for your application.

RAM SBeam
OSU Azalea House
Entry Canopy Beam

Tuesday, February 13, 2024, 10:29



Gravity Beam Design

RAM SBeam v5.01
OSU Azalea House
Entry Canopy Beam

02/13/24 10:30:02

STEEL CODE: AISC 360-05 ASD

SPAN INFORMATION (ft): I-End (0.00,0.00) J-End (5.33,0.00)

Beam Size (User Selected)	=	HSS6X4X1/4	Fy =	46.0 ksi
Total Beam Length (ft)	=	5.33		
Cantilever on left (ft)	=	2.00		
Cantilever on right (ft)	=	3.00		

Mp (kip-ft) = 32.70
Top flange braced by decking.

LINE LOADS (k/ft):

Load	Dist (ft)	DL	LL
1	0.000	0.015	0.000
	2.000	0.015	0.000
2	0.000	0.090	0.150
	2.000	0.090	0.150
3	2.000	0.015	0.000
	2.333	0.015	0.000
4	2.000	0.090	0.150
	2.333	0.090	0.150
5	2.333	0.015	0.000
	5.333	0.015	0.000
6	2.333	0.090	0.150
	5.333	0.090	0.150

SHEAR: Max Va (DL+LL) = 2.85 kips Vn/1.67 = 46.21 kips

MOMENTS:

Span	Cond	LoadCombo	Ma kip-ft	@ ft	Lb ft	Cb	Ω	Mn / Ω kip-ft
Left	Max -	DL+LL	-0.5	2.0	2.0	1.00	1.67	19.58
Center	Max -	DL+LL	-1.1	2.3	0.3	1.29	1.67	19.58
Right	Max -	DL+LL	-1.1	2.3	3.0	1.00	1.67	19.58
Controlling		DL+LL	-1.1	2.3	3.0	1.00	1.67	19.58

REACTIONS (kips):

	Left	**Right**
DL reaction	-0.56	1.12
Max +LL reaction	1.23	2.50
Max -LL reaction	-2.03	-0.90
Max +total reaction	0.67	3.61
Max -total reaction	-2.59	0.22

DEFLECTIONS:

Left cantilever:

Dead load (in)	=	-0.001		
Pos Live load (in)	=	-0.001	L/D =	38136
Pos Total load (in)	=	-0.002	L/D =	22475

Gravity Beam Design

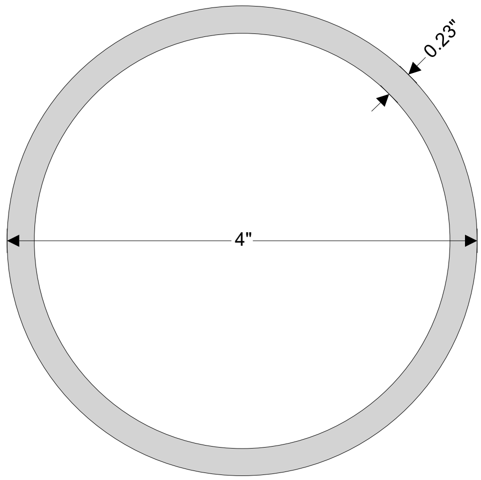
Center span:							
Dead load (in)	at	2.18 ft	=			0.000
Live load (in)	at	2.18 ft	=			0.000
Net Total load (in)	at	2.18 ft	=			0.000
Right cantilever:						
Dead load (in)	=	-0.004		L/D	=	20196
Pos Live load (in)	=	-0.005		L/D	=	14082
Pos Total load (in)	=	-0.009		L/D	=	8297

kpff	Project OSU Azalea House	By MAA	Sheet No. 1
	Location Corvallis, OR	Date 2/12/2024	
	Client Rowell Brokaw	Revised	Job No. 223346
Portland, Oregon		Date	

15 - ENTRY CANOPY COLUMN

Steel column design in accordance with AISC360-16 and the LRFD method

Tedds calculation version 1.0.10



Column and loading details

Column details

Column section **HSS 4x0.250**

Design loading

Required axial strength P_r = **2** kips (Compression)

Maximum moment about x axis M_x = **0.9** kips_ft

Maximum moment about y axis M_y = **3.2** kips_ft

Maximum shear force parallel to y axis V_{ry} = **0.0** kips

Maximum shear force parallel to x axis V_{rx} = **0.3** kips

Material details

Steel grade **A500 Gr. B**

Yield strength F_y = **42** ksi

Ultimate strength F_u = **58** ksi

Modulus of elasticity E = **29000** ksi

Shear modulus of elasticity G = **11200** ksi

Unbraced lengths

For buckling about x axis L_x = **120** in

For buckling about y axis L_y = **120** in

For torsional buckling L_z = **120** in

Effective length factors

For buckling about x axis K_x = **1.00**

For buckling about y axis K_y = **1.00**

For torsional buckling K_z = **1.00**

Effective unbraced lengths

For buckling about x axis $L_{cx} = L_x \times K_x$ = **120** in

For buckling about y axis $L_{cy} = L_y \times K_y$ = **120** in

For torsional buckling $L_{cz} = L_z \times K_z$ = **120** in

Section classification

Section classification for local buckling (cl. B4)

Width to thickness ratio $\lambda = D_o / t$ = **17.167**

Compression

Limit for nonslender section $\lambda_{r\_c} = 0.11 \times E / F_y$ = **75.952**

The section is nonslender in compression

Flexure

Limit for compact section $\lambda_{p\_f} = 0.07 \times E / F_y$ = **48.333**

Limit for noncompact section $\lambda_{r\_f} = 0.31 \times E / F_y$ = **214.048**

The section is compact in flexure

Slenderness

Member slenderness

Slenderness ratio about x axis $SR_x = L_{cx} / r_x$ = **90.2**

Slenderness ratio about y axis $SR_y = L_{cy} / r_y$ = **90.2**

Second order effects

Second order effects for bending about y axis (cl. C2.1b)

Second order effects are already included or do not need to be considered therefore:-

P-δ amplifier $B_{1x} = B_{1y}$ = **1.0**

Required flexural strength (x axis) $M_{rx} = B_{1x} \times M_x$ = **0.9** kips_ft

Required flexural strength (y axis) $M_{ry} = B_{1y} \times M_y$ = **3.2** kips_ft

Design of members for shear parallel to x axis - Chapter G

Required shear strength V_{rx} = **0.320** kips

Nominal shear strength - eq G5-1 $V_{nx} = 0.6 \times F_y \times A / 2$ = **34.776** kips

Resistance factor for shear ϕ_v = **0.90**

Design shear strength $V_{cx} = \phi_v \times V_{nx}$ = **31.298** kips

Compressive strength

Flexural buckling about x axis (cl. E3)

Elastic critical buckling stress $F_{ex} = \pi^2 \times E / (SR_x)^2$ = **35.2** ksi

Flexural buckling stress $F_{crx} = (0.658^{F_y / F_{ex}}) \times F_y =$ **25.5** ksi

Nominal compressive strength for flexural buckling $P_{nx} = F_{crx} \times A =$ **70.3** kips

Flexural buckling about y axis (cl. E3)

Elastic critical buckling stress $F_{ey} = \pi^2 \times E / (SR_y)^2 =$ **35.2** ksi

Flexural buckling stress $F_{cry} = (0.658^{F_y / F_{ey}}) \times F_y =$ **25.5** ksi

Nominal compressive strength for flexural buckling $P_{ny} = F_{cry} \times A =$ **70.3** kips

Design compressive strength (cl.E1)

Resistance factor for compression $\phi_c =$ **0.90**

Design compressive strength $P_c = \phi_c \times \min(P_{nx}, P_{ny}) =$ **63.3** kips

PASS - The design compressive strength exceeds the required compressive strength

Flexural strength about the major axis

Yielding (cl. F8.1)

Nominal flexural strength $M_{nx\_yld} = M_{ny\_yld} = F_y \times Z =$ **11.6** kips_ft

Design flexural strength (cl. F1)

Resistance factor for flexure $\phi_b =$ **0.90**

Design flexural strength $M_{cx} = M_{cy} = \phi_b \times M_{nx\_yld} =$ **10.4** kips_ft

PASS - The design flexural strength about the x axis exceeds the required flexural strength

PASS - The design flexural strength about the y axis exceeds the required flexural strength

Combined forces

Member utilization (cl. H1.1)

Equation H1-1b $UR = abs(P_r) / (2 \times P_c) + (M_{rx} / M_{cx} + M_{ry} / M_{cy}) =$ **0.407**

PASS - The member is adequate for the combined forces

	COMPANY	PROJECT
WoodWorks® SOFTWARE FOR WOOD DESIGN	KPFF Consulting Engineers Feb. 13, 2024 10:34	16 - Critical Grid C Column.wwc

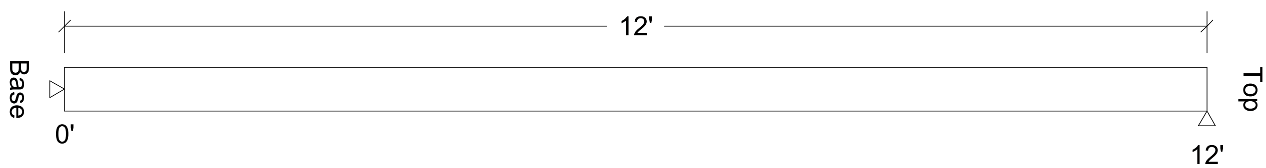
Design Check Calculation Sheet

WoodWorks Sizer 2019 (Update 1)

Loads:

Load	Type	Distribution	Location [ft] Start End	Magnitude Start End	Unit
DL	Dead	Axial	(Ecc. = 0.00")	2910	lbs
LL	Snow	Axial	(Ecc. = 0.00")	8046	lbs
Self-weight	Dead	Axial		86	lbs

Reactions (lbs):



16 - Critical grid C Column

Timber-soft, D.Fir-L, No.1, 6x6 (5-1/2"x5-1/2")

Support: Non-wood

Total length: 12.0'; Volume = 2.5 cu.ft.; Post or timber

Pinned base; Ke x Lb: 1.0 x 12.0 = 12.0 ft; Ke x Ld: 1.0 x 12.0 = 12.0 ft;

This section PASSES the design code check.

Analysis vs. Allowable Stress and Deflection using NDS 2018 :

Criterion	Analysis Value	Design Value	Unit	Analysis/Design
Axial	fc = 365	Fc' = 578	psi	fc/Fc' = 0.63
Axial Bearing	fc = 365	Fc* = 1150	psi	fc/Fc* = 0.32

Additional Data:

FACTORS:	F/E(psi)	CD	CM	Ct	CL/CP	CF	Cfu	Cr	Cfrt	Ci	LC#
Fc'	1000	1.15	1.00	1.00	0.503	1.000	-	-	1.00	1.00	2
Fc*	1000	1.15	1.00	1.00	-	1.000	-	-	1.00	1.00	2

CRITICAL LOAD COMBINATIONS:

```
Axial      : LC #2  = D+S
D=dead L=live S=snow W=wind I=impact Lr=roof live Lc=concentrated E=earthquake
All LC's are listed in the Analysis output
Load combinations: ASD Basic from ASCE 7-16 2.4 / IBC 2018 1605.3.2
```

Design Notes:

1. WoodWorks analysis and design are in accordance with the ICC International Building Code (IBC 2018), the National Design Specification (NDS 2018), and NDS Design Supplement.
2. Please verify that the default deflection limits are appropriate for your application.

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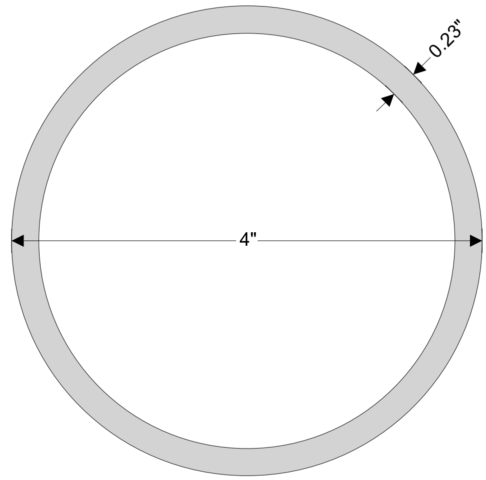
	Client Rowell Brokaw	Revised	Job No. 223346
		Date	

17 - DECK CANOPY COLUMN

Steel column design in accordance with AISC360-16 and the LRFD method

Tedds calculation version 1.0.10



Column and loading details

Column details

Column section	**HSS 4x0.250**

Design loading

Required axial strength	P_r = **4** kips (Compression)
Maximum moment about x axis	M_x = **0.0** kips_ft
Maximum moment about y axis	M_y = **0.0** kips_ft
Maximum shear force parallel to y axis	V_{ry} = **0.0** kips
Maximum shear force parallel to x axis	V_{rx} = **0.0** kips

Material details

Steel grade	**A500 Gr. B**
Yield strength	F_y = **42** ksi
Ultimate strength	F_u = **58** ksi
Modulus of elasticity	E = **29000** ksi
Shear modulus of elasticity	G = **11200** ksi

Unbraced lengths

For buckling about x axis	L_x = **120** in
For buckling about y axis	L_y = **120** in
For torsional buckling	L_z = **120** in

Effective length factors

For buckling about x axis	K_x = **1.00**

For buckling about y axis K_y = **1.00**

For torsional buckling K_z = **1.00**

Effective unbraced lengths

For buckling about x axis $L_{cx} = L_x \times K_x$ = **120** in

For buckling about y axis $L_{cy} = L_y \times K_y$ = **120** in

For torsional buckling $L_{cz} = L_z \times K_z$ = **120** in

Section classification

Section classification for local buckling (cl. B4)

Width to thickness ratio $\lambda = D_o / t$ = **17.167**

Compression

Limit for nonslender section $\lambda_{r\_c} = 0.11 \times E / F_y$ = **75.952**

The section is nonslender in compression

Slenderness

Member slenderness

Slenderness ratio about x axis $SR_x = L_{cx} / r_x$ = **90.2**

Slenderness ratio about y axis $SR_y = L_{cy} / r_y$ = **90.2**

Compressive strength

Flexural buckling about x axis (cl. E3)

Elastic critical buckling stress $F_{ex} = \pi^2 \times E / (SR_x)^2$ = **35.2** ksi

Flexural buckling stress $F_{crx} = (0.658^{F_y / F_{ex}}) \times F_y$ = **25.5** ksi

Nominal compressive strength for flexural buckling $P_{nx} = F_{crx} \times A$ = **70.3** kips

Flexural buckling about y axis (cl. E3)

Elastic critical buckling stress $F_{ey} = \pi^2 \times E / (SR_y)^2$ = **35.2** ksi

Flexural buckling stress $F_{cry} = (0.658^{F_y / F_{ey}}) \times F_y$ = **25.5** ksi

Nominal compressive strength for flexural buckling $P_{ny} = F_{cry} \times A$ = **70.3** kips

Design compressive strength (cl.E1)

Resistance factor for compression ϕ_c = **0.90**

Design compressive strength $P_c = \phi_c \times \min(P_{nx}, P_{ny})$ = **63.3** kips

PASS - The design compressive strength exceeds the required compressive strength

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	Client Rowell Brokaw	Revised	Job No. 223346
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18 - TYPICAL HSS FOOTING

Footing analysis in accordance with ACI318-19

Tedds calculation version 3.3.02

Summary results

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	3.7			Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	0.93	1.5	0.620	Pass
Description	**Unit**	**Provided**	**Required**	**Utilization**	**Result**
Moment, positive, x-direction	kip_ft	0.3	32.3	0.010	Pass
Moment, positive, y-direction	kip_ft	0.3	32.3	0.010	Pass
Shear, two-way, Col 1	psi	2.678	189.737	0.014	Pass
Min.area of reinf, bot., x-direction	in^2	0.518	0.930		Pass
Max.reinf.spacing, bot, x-direction	in	18.0	8.6		Pass
Min.area of reinf, bot., y-direction	in^2	0.518	0.930		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	8.6		Pass

Pad footing details

Length of footing L_x = **2** ft

Width of footing L_y = **2** ft

Footing area $A = L_x \times L_y$ = **4** ft^2

Depth of footing h = **12** in

Depth of soil over footing h_{soil} = **12** in

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

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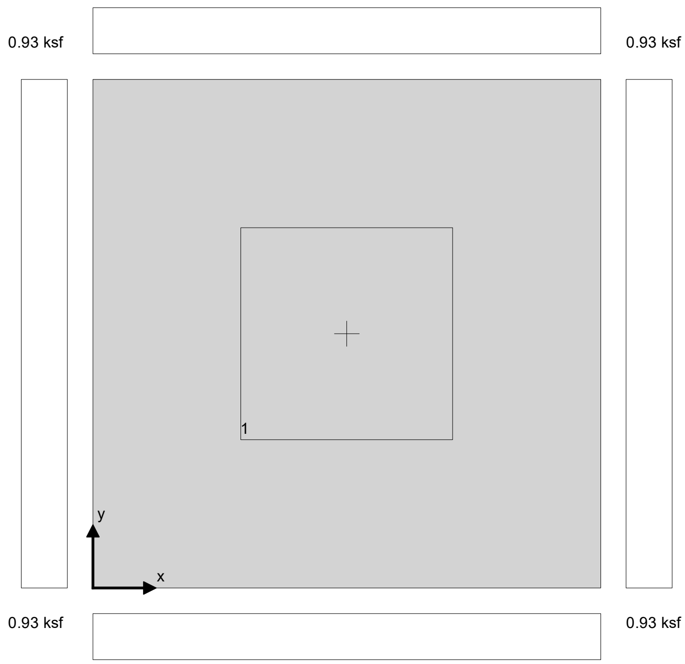

Column no.1 details

Length of column	l_{x1} = **10.00** in
Width of column	l_{y1} = **10.00** in
position in x-axis	x_1 = **12.00** in
position in y-axis	y_1 = **12.00** in

Soil properties

Gross allowable bearing pressure	q_{allow_Gross} = **1.5** ksf
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	$F_{soil} = h_{soil} \times \gamma_{soil}$ = **120** psf

Column no.1 loads

Dead load in z	F_{Dz1} = **1.0** kips
Live load in z	F_{Lz1} = **1.7** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.345)

1.0D + 1.0L (0.620)

Combination 2 results: 1.0D + 1.0L

Forces on footing

Force in z-axis $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1}$ = **3.7** kips

Moments on footing

Moment in x-axis, about x is 0 $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1) + \gamma_L \times (F_{Lz1} \times x_1)$ = **3.7** kip_ft

Moment in y-axis, about y is 0 $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_L \times (F_{Lz1} \times y_1)$ = **3.7** kip_ft

Uplift verification

Vertical force F_{dz} = **3.72** 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 $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **0** in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **0.93** ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **0.93** ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **0.93** ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **0.93** ksf

Minimum base pressure $q_{min} = \min(q_1,q_2,q_3,q_4)$ = **0.93** ksf

Maximum base pressure $q_{max} = \max(q_1,q_2,q_3,q_4)$ = **0.93** ksf

Allowable bearing capacity

Allowable bearing capacity $q_{allow} = q_{allow\_Gross}$ = **1.5** ksf

q_{max} / q_{allow} = **0.620**

PASS - Allowable bearing capacity exceeds design base pressure

18 - TYPICAL HSS FOOTING

Footing design in accordance with ACI318-19

Tedds calculation version 3.3.02

Material details

Compressive strength of concrete f'_c = **4000** 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

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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 (0.004)

1.2D + 1.6L + 0.5Lr (0.014)

Combination 2 results: 1.2D + 1.6L + 0.5Lr

Forces on footing

Ultimate force in z-axis — $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1} = \mathbf{5.1}$ kips

Moments on footing

Ultimate moment in x-axis, about x is 0 — $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) = \mathbf{5.1}$ kip_ft

Ultimate moment in y-axis, about y is 0 — $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) = \mathbf{5.1}$ kip_ft

Eccentricity of base reaction

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

Eccentricity of base reaction in y-axis — $e_{uy} = M_{uy} / F_{uz} - L_y / 2 = \mathbf{0}$ in

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{1.281}$ ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{1.281}$ ksf

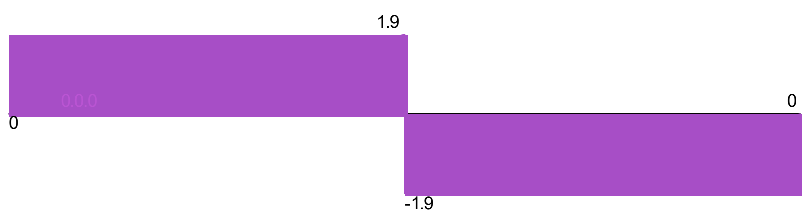
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{1.281}$ ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) = \mathbf{1.281}$ ksf

Minimum ultimate base pressure — $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4}) = \mathbf{1.281}$ ksf

Maximum ultimate base pressure — $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4}) = \mathbf{1.281}$ ksf

Shear diagram, x axis (kips)



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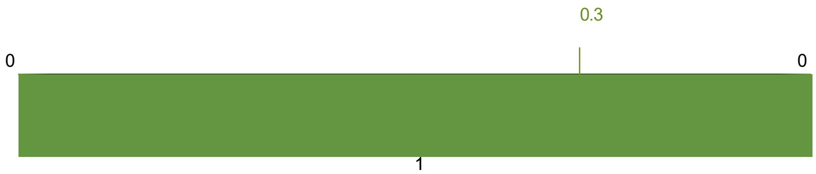
Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment	$M_{u.x.max}$ = **0.326** kip_ft
Tension reinforcement provided	3 No.5 bottom bars (8.6 in c/c)
Area of tension reinforcement provided	$A_{sx.bot.prov}$ = **0.93** in^2
Minimum area of reinforcement (8.6.1.1)	$A_{s.min} = 0.0018 \times L_y \times h$ = **0.518** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2)	s_{max} = min(2 × h, 18 in) = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement	$d = h - c_{nom\_b} - \phi_{x.bot} / 2$ = **8.688** in
Depth of compression block	$a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y)$ = **0.684** in
Neutral axis factor	β_1 = **0.85**
Depth to neutral axis	$c = a / \beta_1$ = **0.804** in
Strain in tensile reinforcement	$\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.02940**
Minimum tensile strain(8.3.3.1)	$\varepsilon_{min} = \varepsilon_{ty} + 0.003$ = **0.00500**

PASS - Tensile strain exceeds minimum required

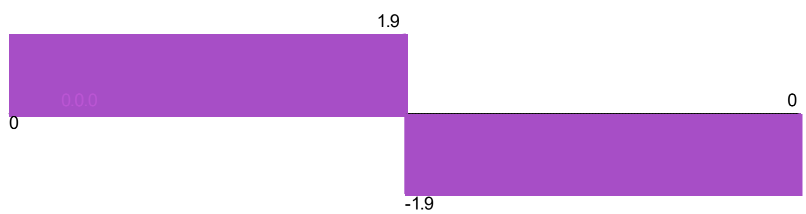
Nominal moment capacity	$M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2)$ = **38.807** kip_ft
Flexural strength reduction factor	$\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9)$ = **0.900**
Design moment capacity	$\phi M_n = \phi_f \times M_n$ = **34.926** kip_ft
	$M_{u.x.max} / \phi M_n$ = **0.009**

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 $M_{u.y.max}$ = **0.326** kip_ft

Tension reinforcement provided 3 No.5 bottom bars (8.6 in c/c)

Area of tension reinforcement provided $A_{sy.bot.prov}$ = **0.93** in^2

Minimum area of reinforcement (8.6.1.1) $A_{s.min} = 0.0018 \times L_x \times h$ = **0.518** 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 $d = h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **8.062** in

Depth of compression block $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.684** in

Neutral axis factor β_1 = **0.85**

Depth to neutral axis $c = a / \beta_1$ = **0.804** in

Strain in tensile reinforcement $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.02707**

Minimum tensile strain(8.3.3.1) $\varepsilon_{min} = \varepsilon_{ty} + 0.003$ = **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity $M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2)$ = **35.901** kip_ft

Flexural strength reduction factor $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9)$ = **0.900**

Design moment capacity $\phi M_n = \phi_f \times M_n$ = **32.311** kip_ft

$M_{u.y.max} / \phi M_n$ = **0.010**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

One-way shear design does not apply. Shear failure plane fall outside extents of foundation.

Two-way shear design at column 1

Depth to reinforcement d_{v2} = **8.375** in

Shear perimeter length (22.6.4) l_{xp} = **18.375** in

Shear perimeter width (22.6.4) l_{yp} = **18.375** in

Shear perimeter (22.6.4) $b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2})$ = **73.500** in

Shear area $A_p = l_{x,perim} \times l_{y,perim}$ = **337.641** in^2

Surcharge loaded area $A_{sur} = A_p - l_{x1} \times l_{y1}$ = **237.641** in^2

Ultimate bearing pressure at center of shear area $q_{up.avg}$ = **1.281** ksf

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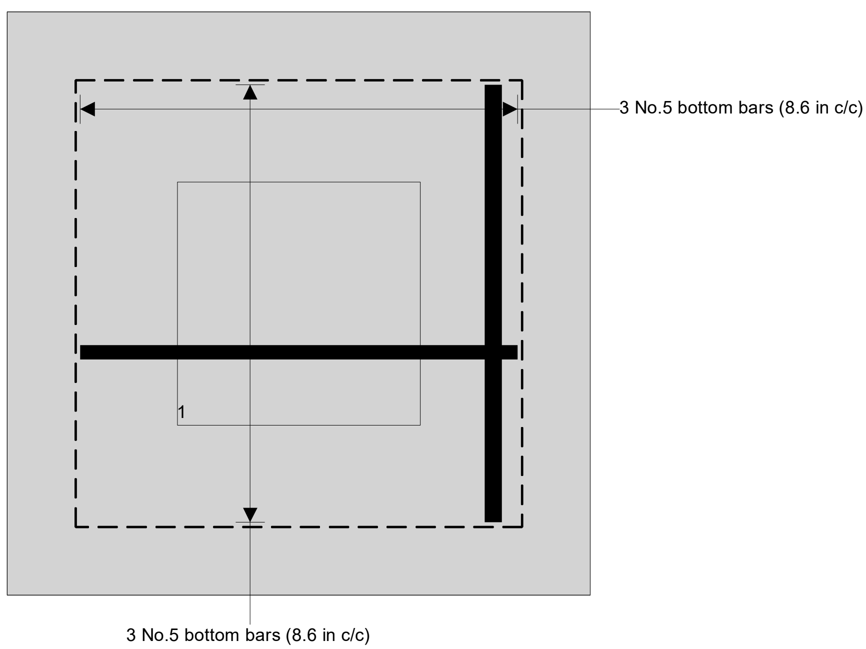
Ultimate shear load	$F_{up} = \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p$ = **1.484** kips
Ultimate shear stress from vertical load	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0\text{ psi})$ = **2.411** 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**
Size effect factor (22.5.5.1.3)	λ_s = **1**
Concrete shear strength (22.6.5.2)	$v_{cpa} = (2 + 4 / \beta) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})}$ = **379.473** psi
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})}$ = **414.753** psi
	$v_{cpc} = 4 \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})}$ = **252.982** psi
	$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc})$ = **252.982** psi
Shear strength reduction factor	ϕ_v = **0.75**
Nominal shear stress capacity (Eq. 22.6.1.2)	$v_n = v_{cp}$ = **252.982** psi
Design shear stress capacity (8.5.1.1(d))	$\phi v_n = \phi_v \times v_n$ = **189.737** psi
	$v_{ug} / \phi v_n$ = **0.013**

PASS - Design shear stress capacity exceeds ultimate shear stress load



19 - GRID H FOOTING

Footing analysis in accordance with ACI318-19

Tedds calculation version 3.3.02

Summary results

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	3.7			Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	0.93	1.5	0.620	Pass
Description	**Unit**	**Provided**	**Required**	**Utilization**	**Result**
Moment, positive, x-direction	kip_ft	0.3	34.9	0.009	Pass
Moment, positive, y-direction	kip_ft	0.3	32.3	0.010	Pass
Shear, two-way, Col 1	psi	2.411	189.737	0.013	Pass
Min.area of reinf, bot., x-direction	in^2	0.518	0.930		Pass
Max.reinf.spacing, bot, x-direction	in	18.0	8.6		Pass
Min.area of reinf, bot., y-direction	in^2	0.518	0.930		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	8.6		Pass

Pad footing details

Length of footing	L_x = **2** ft
Width of footing	L_y = **2** ft
Footing area	$A = L_x \times L_y$ = **4** ft^2
Depth of footing	h = **12** in
Depth of soil over footing	h_{soil} = **12** in
Density of concrete	γ_{conc} = **150.0** lb/ft^3

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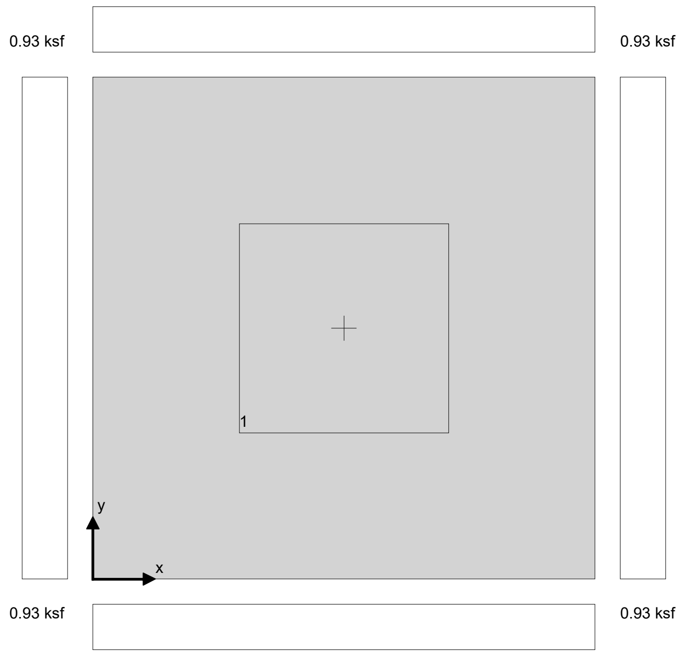


Column no.1 details

Length of column l_{x1} = **10.00** in

Width of column l_{y1} = **10.00** in

position in x-axis x_1 = **12.00** in

position in y-axis y_1 = **12.00** in

Soil properties

Gross allowable bearing pressure q_{allow_Gross} = **1.5** ksf

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 $F_{soil} = h_{soil} \times \gamma_{soil}$ = **120** psf

Column no.1 loads

Dead load in z F_{Dz1} = **1.0** kips

Live load in z F_{Lz1} = **1.7** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.345)

1.0D + 1.0L (0.620)

Combination 2 results: 1.0D + 1.0L

Forces on footing

Force in z-axis $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1} = \mathbf{3.7}$ kips

Moments on footing

Moment in x-axis, about x is 0 $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) = \mathbf{3.7}$ kip_ft

Moment in y-axis, about y is 0 $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) = \mathbf{3.7}$ kip_ft

Uplift verification

Vertical force $F_{dz} = \mathbf{3.72}$ kips

PASS - Footing is not subject to uplift

Bearing resistance

Eccentricity of base reaction

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

Eccentricity of base reaction in y-axis $e_{dy} = M_{dy} / F_{dz} - L_y / 2 = \mathbf{0}$ in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{0.93}$ ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{0.93}$ ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{0.93}$ ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{0.93}$ ksf

Minimum base pressure $q_{min} = \min(q_1,q_2,q_3,q_4) = \mathbf{0.93}$ ksf

Maximum base pressure $q_{max} = \max(q_1,q_2,q_3,q_4) = \mathbf{0.93}$ ksf

Allowable bearing capacity

Allowable bearing capacity $q_{allow} = q_{allow\_Gross} = \mathbf{1.5}$ ksf

$q_{max} / q_{allow} = \mathbf{0.620}$

PASS - Allowable bearing capacity exceeds design base pressure

19 - GRID H FOOTING

Footing design in accordance with ACI318-19

Tedds calculation version 3.3.02

Material details

Compressive strength of concrete $f'_c = \mathbf{4000}$ psi

Yield strength of reinforcement $f_y = \mathbf{60000}$ psi

Compression-controlled strain limit (21.2.2) $\varepsilon_{ty} = \mathbf{0.00200}$

Cover to top of footing $c_{nom\_t} = \mathbf{3}$ in

Cover to side of footing $c_{nom\_s} = \mathbf{3}$ 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 (0.004)

1.2D + 1.6L + 0.5Lr (0.013)

Combination 2 results: 1.2D + 1.6L + 0.5Lr

Forces on footing

Ultimate force in z-axis — $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1}$ = **5.1** kips

Moments on footing

Ultimate moment in x-axis, about x is 0 — $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1) + \gamma_L \times (F_{Lz1} \times x_1)$ = **5.1** kip_ft

Ultimate moment in y-axis, about y is 0 — $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_L \times (F_{Lz1} \times y_1)$ = **5.1** 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 — $e_{uy} = M_{uy} / F_{uz} - L_y / 2$ = **0** in

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **1.281** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **1.281** ksf

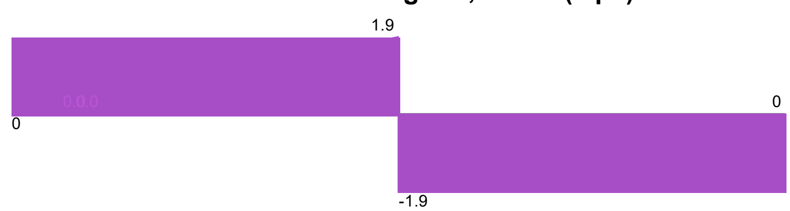
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **1.281** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **1.281** ksf

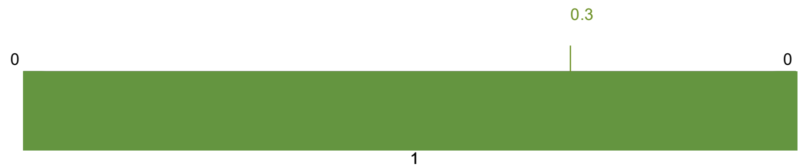
Minimum ultimate base pressure — $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4})$ = **1.281** ksf

Maximum ultimate base pressure — $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4})$ = **1.281** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment: $M_{u.x.max}$ = **0.326** kip_ft

Tension reinforcement provided: 3 No.5 bottom bars (8.6 in c/c)

Area of tension reinforcement provided: $A_{sx.bot.prov}$ = **0.93** in^2

Minimum area of reinforcement (8.6.1.1): $A_{s.min} = 0.0018 \times L_y \times h$ = **0.518** in^2

PASS - Area of reinforcement provided exceeds minimum

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

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement: $d = h - c_{nom\_b} - \phi_{x.bot} / 2$ = **8.688** in

Depth of compression block: $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y)$ = **0.684** in

Neutral axis factor: β_1 = **0.85**

Depth to neutral axis: $c = a / \beta_1$ = **0.804** in

Strain in tensile reinforcement: $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.02940**

Minimum tensile strain(8.3.3.1): $\varepsilon_{min} = \varepsilon_{ty} + 0.003$ = **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity: $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2)$ = **38.807** kip_ft

Flexural strength reduction factor: $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9)$ = **0.900**

Design moment capacity: $\phi M_n = \phi_f \times M_n$ = **34.926** kip_ft

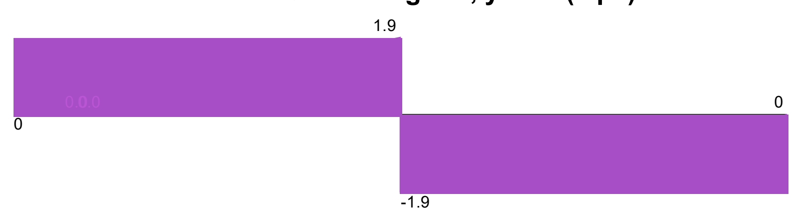
$M_{u.x.max} / \phi M_n$ = **0.009**

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)



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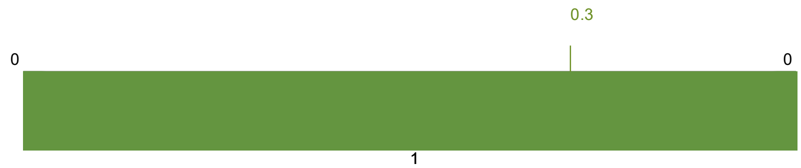
Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment — $M_{u.y.max}$ = **0.326** kip_ft

Tension reinforcement provided — 3 No.5 bottom bars (8.6 in c/c)

Area of tension reinforcement provided — $A_{sy.bot.prov}$ = **0.93** in^2

Minimum area of reinforcement (8.6.1.1) — $A_{s.min} = 0.0018 \times L_x \times h$ = **0.518** in^2

PASS - Area of reinforcement provided exceeds minimum

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

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

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

Depth of compression block — $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.684** in

Neutral axis factor — β_1 = **0.85**

Depth to neutral axis — $c = a / \beta_1$ = **0.804** in

Strain in tensile reinforcement — $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.02707**

Minimum tensile strain(8.3.3.1) — $\varepsilon_{min} = \varepsilon_{ty} + 0.003$ = **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity — $M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2)$ = **35.901** kip_ft

Flexural strength reduction factor — $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9)$ = **0.900**

Design moment capacity — $\phi M_n = \phi_f \times M_n$ = **32.311** kip_ft

$M_{u.y.max} / \phi M_n$ = **0.010**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

One-way shear design does not apply. Shear failure plane fall outside extents of foundation.

Two-way shear design at column 1

Depth to reinforcement — d_{v2} = **8.375** in

Shear perimeter length (22.6.4) — l_{xp} = **18.375** in

Shear perimeter width (22.6.4) — l_{yp} = **18.375** in

Shear perimeter (22.6.4) — $b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2})$ = **73.500** in

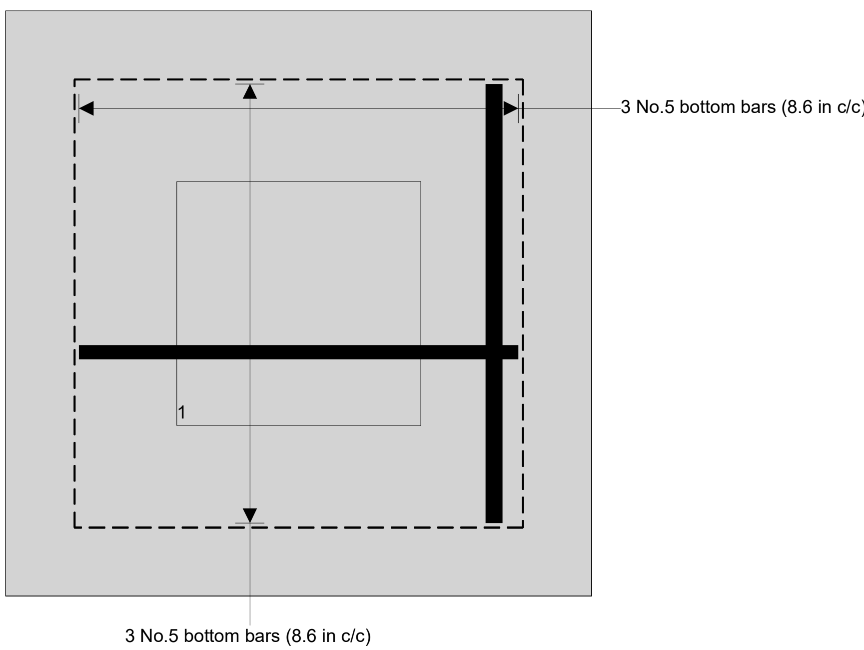
Shear area — $A_p = l_{x,perim} \times l_{y,perim}$ = **337.641** in^2

Surcharge loaded area — $A_{sur} = A_p - l_{x1} \times l_{y1}$ = **237.641** in^2

Ultimate bearing pressure at center of shear area — $q_{up.avg}$ = **1.281** ksf

Ultimate shear load	$F_{up} = \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p =$ **1.484** kips
Ultimate shear stress from vertical load	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}),0\text{ psi}) =$ **2.411** psi
Column geometry factor (Table 22.6.5.2)	$\beta = l_{y1} / l_{x1} =$ **1.00**
Column location factor (22.6.5.3)	$\alpha_s =$**40**
Size effect factor (22.5.5.1.3)	$\lambda_s =$ **1**
Concrete shear strength (22.6.5.2)	$v_{cpa} = (2 + 4 / \beta) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} =$ **379.473** psi
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} =$ **414.753** psi
	$v_{cpc} = 4 \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} =$ **252.982** psi
	$v_{cp} = \min(v_{cpa},v_{cpb},v_{cpc}) =$ **252.982** psi
Shear strength reduction factor	$\phi_v =$ **0.75**
Nominal shear stress capacity (Eq. 22.6.1.2)	$v_n = v_{cp} =$ **252.982** psi
Design shear stress capacity (8.5.1.1(d))	$\phi v_n = \phi_v \times v_n =$ **189.737** psi
	$v_{ug} / \phi v_n =$ **0.013**

PASS - Design shear stress capacity exceeds ultimate shear stress load



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20 - GRID C FOOTING

Footing analysis in accordance with ACI318-19

Tedds calculation version 3.3.02

Summary results

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	10.8			Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	1.194	1.5	0.796	Pass
Description	**Unit**	**Provided**	**Required**	**Utilization**	**Result**
Moment, positive, x-direction	kip_ft	2.5	46.8	0.052	Pass
Moment, positive, y-direction	kip_ft	2.5	43.3	0.057	Pass
Shear, one-way, x-direction	kips	1.7	18.8	0.092	Pass
Shear, one-way, y-direction	kips	1.7	17.9	0.096	Pass
Shear, two-way, Col 1	psi	14.907	189.737	0.079	Pass
Min.area of reinf, bot., x-direction	in^2	0.778	1.240		Pass
Max.reinf.spacing, bot, x-direction	in	18.0	9.7		Pass
Min.area of reinf, bot., y-direction	in^2	0.778	1.240		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	9.7		Pass

Pad footing details

Length of footing L_x = **3** ft

Width of footing L_y = **3** ft

Footing area $A = L_x \times L_y$ = **9** ft^2

Depth of footing h = **12** in

Depth of soil over footing h_{soil} = **12** in

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

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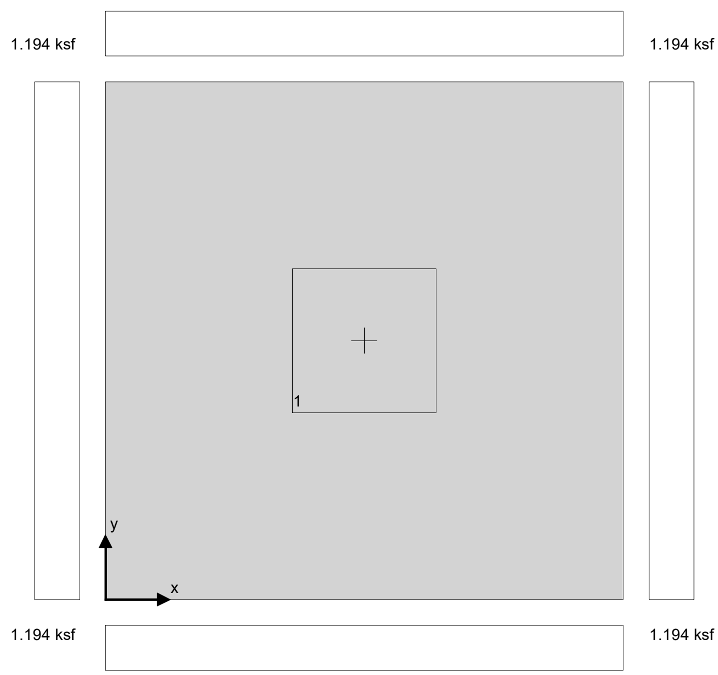


Column no.1 details

Length of column — l_{x1} = **10.00** in

Width of column — l_{y1} = **10.00** in

position in x-axis — x_1 = **18.00** in

position in y-axis — y_1 = **18.00** in

Soil properties

Gross allowable bearing pressure — q_{allow_Gross} = **1.5** ksf

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 — $F_{soil} = h_{soil} \times \gamma_{soil}$ = **120** psf

Column no.1 loads

Dead load in z — F_{Dz1} = **1.9** kips

Live load in z — F_{Lz1} = **6.4** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.322)

1.0D + 1.0L (0.796)

Combination 2 results: 1.0D + 1.0L

Forces on footing

Force in z-axis — $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1}$ = **10.8** kips

Moments on footing

Moment in x-axis, about x is 0 — $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1) + \gamma_L \times (F_{Lz1} \times x_1)$ = **16.1** kip_ft

Moment in y-axis, about y is 0 — $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_L \times (F_{Lz1} \times y_1)$ = **16.1** kip_ft

Uplift verification

Vertical force — F_{dz} = **10.75** 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 — $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **0** in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.194** ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.194** ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.194** ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **1.194** ksf

Minimum base pressure — $q_{min} = \min(q_1,q_2,q_3,q_4)$ = **1.194** ksf

Maximum base pressure — $q_{max} = \max(q_1,q_2,q_3,q_4)$ = **1.194** ksf

Allowable bearing capacity

Allowable bearing capacity — $q_{allow} = q_{allow\_Gross}$ = **1.5** ksf

q_{max} / q_{allow} = **0.796**

PASS - Allowable bearing capacity exceeds design base pressure

20 - GRID C FOOTING

Footing design in accordance with ACI318-19

Tedds calculation version 3.3.02

Material details

Compressive strength of concrete — f'_c = **4000** 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 (0.021)

1.2D + 1.6L + 0.5Lr (0.096)

Combination 2 results: 1.2D + 1.6L + 0.5Lr

Forces on footing

Ultimate force in z-axis — $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1}$ = **15.5** kips

Moments on footing

Ultimate moment in x-axis, about x is 0 — $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1) + \gamma_L \times (F_{Lz1} \times x_1)$ = **23.2** kip_ft

Ultimate moment in y-axis, about y is 0 — $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_L \times (F_{Lz1} \times y_1)$ = **23.2** 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 — $e_{uy} = M_{uy} / F_{uz} - L_y / 2$ = **0** in

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **1.718** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **1.718** ksf

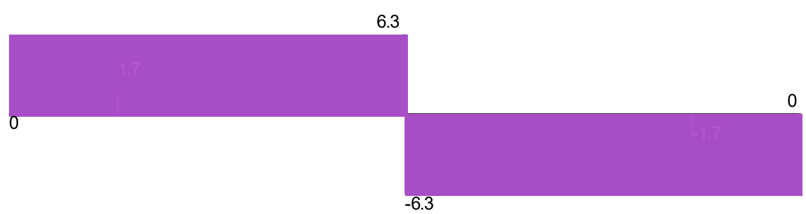
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **1.718** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **1.718** ksf

Minimum ultimate base pressure — $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4})$ = **1.718** ksf

Maximum ultimate base pressure — $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4})$ = **1.718** ksf

Shear diagram, x axis (kips)



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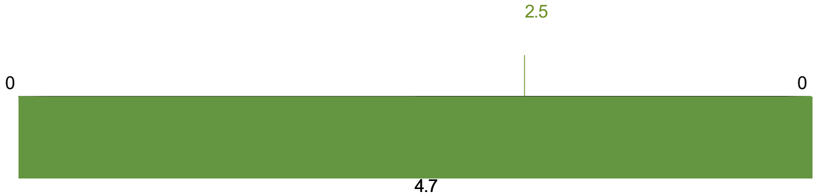
Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment	$M_{u.x.max}$ = **2.454** kip_ft
Tension reinforcement provided	4 No.5 bottom bars (9.7 in c/c)
Area of tension reinforcement provided	$A_{sx.bot.prov}$ = **1.24** in^2
Minimum area of reinforcement (8.6.1.1)	$A_{s.min} = 0.0018 \times L_y \times h$ = **0.778** in^2
	PASS - Area of reinforcement provided exceeds minimum
Maximum spacing of reinforcement (8.7.2.2)	s_{max} = min(2 × h, 18 in) = **18** in
	PASS - Maximum permissible reinforcement spacing exceeds actual spacing
Depth to tension reinforcement	$d = h - c_{nom\_b} - \phi_{x.bot} / 2$ = **8.688** in
Depth of compression block	$a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y)$ = **0.608** in
Neutral axis factor	β_1 = **0.85**
Depth to neutral axis	$c = a / \beta_1$ = **0.715** in
Strain in tensile reinforcement	$\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.03345**
Minimum tensile strain(8.3.3.1)	$\varepsilon_{min} = \varepsilon_{ty} + 0.003$ = **0.00500**
	PASS - Tensile strain exceeds minimum required
Nominal moment capacity	$M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2)$ = **51.978** kip_ft
Flexural strength reduction factor	$\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9)$ = **0.900**
Design moment capacity	$\phi M_n = \phi_f \times M_n$ = **46.78** kip_ft
	$M_{u.x.max} / \phi M_n$ = **0.052**
	PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force	$V_{u.x}$ = **1.72** kips
Depth to reinforcement	$d_v = h - c_{nom\_b} - \phi_{x.bot} / 2$ = **8.688** in
Size effect factor (22.5.5.1.3)	λ_s = **1**
Ratio of longitudinal reinforcement	$\rho_w = A_{sx.bot.prov} / (L_y \times d_v)$ = **0.00396**
Shear strength reduction factor	ϕ_v = **0.75**
Nominal shear capacity (Eq. 22.5.5.1)	$V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_y \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_y \times d_v)$ = **25.045** kips
Design shear capacity	$\phi V_n = \phi_v \times V_n$ = **18.784** kips
	$V_{u.x} / \phi V_n$ = **0.092**

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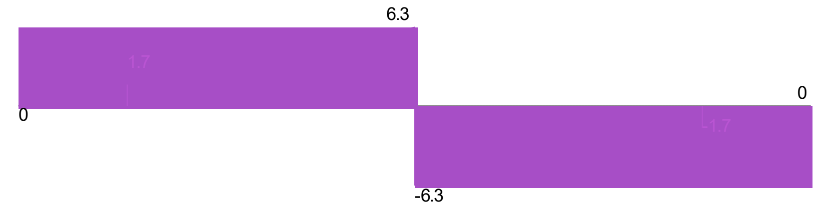
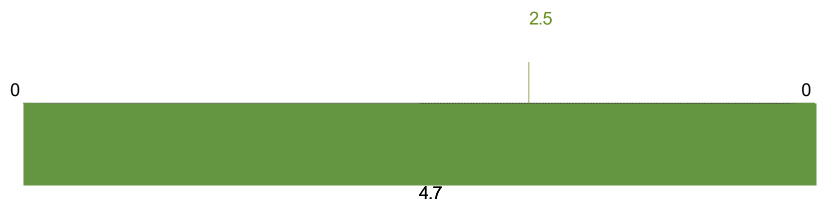
PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment $M_{u.y.max}$ = **2.454** kip_ft

Tension reinforcement provided 4 No.5 bottom bars (9.7 in c/c)

Area of tension reinforcement provided $A_{sy.bot.prov}$ = **1.24** in^2

Minimum area of reinforcement (8.6.1.1) $A_{s.min} = 0.0018 \times L_x \times h$ = **0.778** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2) s_{max} = min(2 × h, 18 in) = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

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

Depth of compression block $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.608** in

Neutral axis factor β_1 = **0.85**

Depth to neutral axis $c = a / \beta_1$ = **0.715** in

Strain in tensile reinforcement $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.03082**

Minimum tensile strain(8.3.3.1) $\varepsilon_{min} = \varepsilon_{ty} + 0.003$ = **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity $M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2)$ = **48.103** kip_ft

Flexural strength reduction factor $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9)$ = **0.900**

Design moment capacity $\phi M_n = \phi_f \times M_n$ = **43.293** kip_ft

$M_{u.y.max} / \phi M_n$ = **0.057**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force $V_{u.y}$ = **1.72** kips

Depth to reinforcement	$d_v = h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2 =$ **8.062** in
Size effect factor (22.5.5.1.3)	λ_s = **1**
Ratio of longitudinal reinforcement	$\rho_w = A_{sy.bot.prov} / (L_x \times d_v) =$ **0.00427**
Shear strength reduction factor	$\phi_v =$ **0.75**
Nominal shear capacity (Eq. 22.5.5.1)	$V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1\text{ psi})} \times L_x \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} \times L_x \times d_v) =$ **23.829** kips
Design shear capacity	$\phi V_n = \phi_v \times V_n =$ **17.872** kips
	$V_{u.y} / \phi V_n =$ **0.096**

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)	$l_{xp} =$ **18.375** in
Shear perimeter width (22.6.4)	$l_{yp} =$ **18.375** in
Shear perimeter (22.6.4)	$b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2}) =$ **73.500** in
Shear area	$A_p = l_{x,perim} \times l_{y,perim} =$ **337.641** in^2
Surcharge loaded area	$A_{sur} = A_p - l_{x1} \times l_{y1} =$ **237.641** in^2
Ultimate bearing pressure at center of shear area	$q_{up.avg} =$ **1.718** ksf
Ultimate shear load	$F_{up} = \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p =$ **9.176** kips
Ultimate shear stress from vertical load	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0\text{ psi}) =$ **14.907** psi
Column geometry factor (Table 22.6.5.2)	$\beta = l_{y1} / l_{x1} =$ **1.00**
Column location factor (22.6.5.3)	$\alpha_s =$ **40**
Size effect factor (22.5.5.1.3)	λ_s = **1**
Concrete shear strength (22.6.5.2)	$v_{cpa} = (2 + 4 / \beta) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} =$ **379.473** psi
	$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} =$ **414.753** psi
	$v_{cpc} = 4 \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} =$ **252.982** psi
	$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) =$ **252.982** psi
Shear strength reduction factor	$\phi_v =$ **0.75**
Nominal shear stress capacity (Eq. 22.6.1.2)	$v_n = v_{cp} =$ **252.982** psi
Design shear stress capacity (8.5.1.1(d))	$\phi v_n = \phi_v \times v_n =$ **189.737** psi
	$v_{ug} / \phi v_n =$ **0.079**

PASS - Design shear stress capacity exceeds ultimate shear stress load

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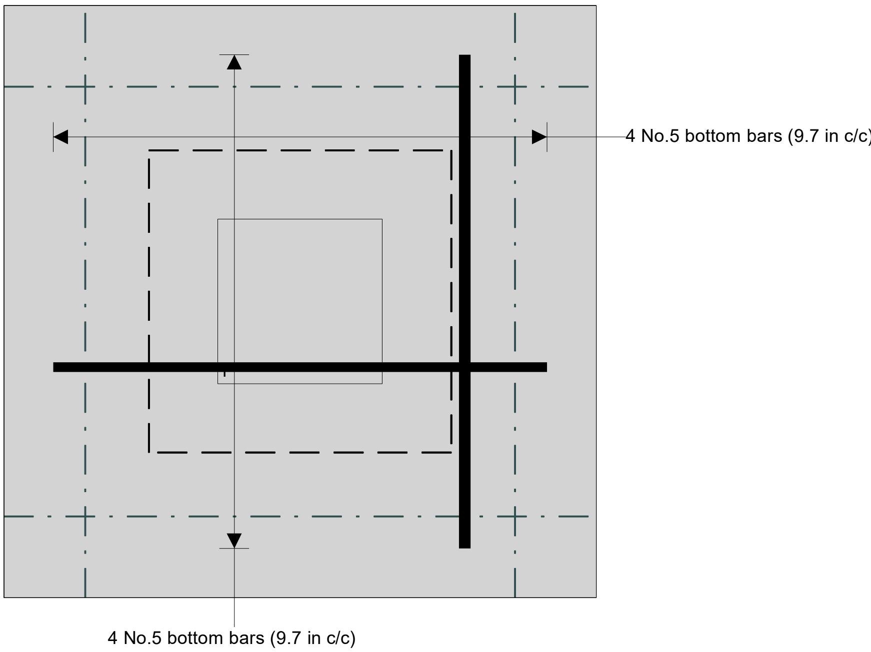


21 - ENTRY CANOPY FOOTING

Footing analysis in accordance with ACI318-19

Tedds calculation version 3.3.02

Summary results

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	3.2			Pass
Overturning stability, x	kip_ft	2.49	-4.76	1.91	Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	0.987	1.5	0.658	Pass
Description	**Unit**	**Provided**	**Required**	**Utilization**	**Result**
Moment, positive, x-direction	kip_ft	0.6	46.8	0.013	Pass
Moment, positive, y-direction	kip_ft	0.3	43.3	0.008	Pass
Shear, one-way, x-direction	kips	0.4	18.8	0.024	Pass
Shear, one-way, y-direction	kips	0.2	17.9	0.013	Pass
Shear, two-way, Col 1	psi	1.928	189.737	0.010	Pass
Min.area of reinf, bot., x-direction	in^2	0.778	1.240		Pass
Max.reinf.spacing, bot, x-direction	in	18.0	9.7		Pass
Min.area of reinf, bot., y-direction	in^2	0.778	1.240		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	9.7		Pass

Pad footing details

Length of footing L_x = **3** ft

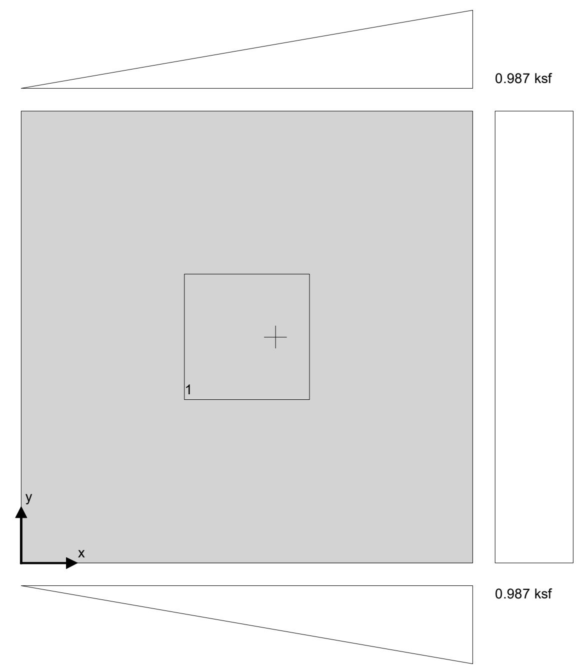
Width of footing L_y = **3** ft

Footing area $A = L_x \times L_y$ = **9** ft^2

Depth of footing h = **12** in

Depth of soil over footing h_{soil} = **12** in

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



Column no.1 details

Length of column	l_{x1} = **10.00** in
Width of column	l_{y1} = **10.00** in
position in x-axis	x_1 = **18.00** in
position in y-axis	y_1 = **18.00** in

Soil properties

Gross allowable bearing pressure	q_{allow_Gross} = **1.5** ksf
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**
Self weight	$F_{swt} = h \times \gamma_{conc}$ = **150** psf
Soil weight	$F_{soil} = h_{soil} \times \gamma_{soil}$ = **120** psf

Column no.1 loads

Dead load in z	F_{Dz1} = **0.5** kips
Live load in z	F_{Lz1} = **0.8** kips
Dead load moment in x	M_{Dx1} = **0.2** kip_ft
Live load moment in x	M_{Lx1} = **0.4** kip_ft
Seismic load moment in x	M_{Ex1} = **3.2** kip_ft

Footing analysis for soil and stability

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Load combinations per ASCE 7-16

1.0D (0.247)

1.0D + 1.0L (0.359)

$(1.0 + 0.14 \times S_{DS})D + 0.7E$ (0.658)

Combination 10 results: $(1.0 + 0.14 \times S_{DS})D + 0.7E$

Forces on footing

Force in z-axis — $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} = \mathbf{3.2}$ kips

Moments on footing

Moment in x-axis, about x is 0 — $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1 + M_{Dx1}) + \gamma_E \times (M_{Ex1}) = \mathbf{7.3}$ kip_ft

Moment in y-axis, about y is 0 — $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) = \mathbf{4.8}$ kip_ft

Uplift verification

Vertical force — $F_{dz} = \mathbf{3.174}$ kips

PASS - Footing is not subject to uplift

Stability against overturning in x direction, moment about x is L_x

Overturning moment — $M_{OTxL} = \gamma_D \times (M_{Dx1}) + \gamma_E \times (M_{Ex1}) = \mathbf{2.49}$ kip_ft

Resisting moment — $M_{RxL} = -1 \times (\gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2)) + \gamma_D \times (F_{Dz1} \times (x_1 - L_x)) = \mathbf{-4.76}$ kip_ft

Factor of safety — $abs(M_{RxL} / M_{OTxL}) = \mathbf{1.910}$

PASS - Overturning moment safety factor exceeds the minimum of 1.50

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis — $e_{dx} = M_{dx} / F_{dz} - L_x / 2 = \mathbf{9.426}$ in

Eccentricity of base reaction in y-axis — $e_{dy} = M_{dy} / F_{dz} - L_y / 2 = \mathbf{0}$ in

Length of bearing in x-axis — $L'_{xd} = \min(L_x, 3 \times (L_x / 2 - abs(e_{dx}))) = \mathbf{25.721}$ in

Pad base pressures

$q_1 = \mathbf{0}$ ksf

$q_2 = \mathbf{0}$ ksf

$q_3 = 2 \times F_{dz} / (3 \times L_y \times (L_x / 2 - e_{dx})) = \mathbf{0.987}$ ksf

$q_4 = 2 \times F_{dz} / (3 \times L_y \times (L_x / 2 - e_{dx})) = \mathbf{0.987}$ ksf

Minimum base pressure — $q_{min} = \min(q_1, q_2, q_3, q_4) = \mathbf{0}$ ksf

Maximum base pressure — $q_{max} = \max(q_1, q_2, q_3, q_4) = \mathbf{0.987}$ ksf

Allowable bearing capacity

Allowable bearing capacity — $q_{allow} = q_{allow\_Gross} = \mathbf{1.5}$ ksf

$q_{max} / q_{allow} = \mathbf{0.658}$

PASS - Allowable bearing capacity exceeds design base pressure

21 - ENTRY CANOPY FOOTING

Footing design in accordance with ACI318-19

Tedds calculation version 3.3.02

Material details

Compressive strength of concrete	f'_c = **4000** 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 (0.005)

1.2D + 1.6L + 0.5Lr (0.013)

Combination 2 results: 1.2D + 1.6L + 0.5Lr

Forces on footing

Ultimate force in z-axis: $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1}$ = **4.7** kips

Moments on footing

Ultimate moment in x-axis, about x is 0: $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1 + M_{Dx1}) + \gamma_L \times (F_{Lz1} \times x_1 + M_{Lx1})$ = **7.9** kip_ft

Ultimate moment in y-axis, about y is 0: $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_L \times (F_{Lz1} \times y_1)$ = **7.0** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis: $e_{ux} = M_{ux} / F_{uz} - L_x / 2$ = **2.278** in

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

Pad base pressures

q_{u1} = **0.321** ksf

q_{u2} = **0.321** ksf

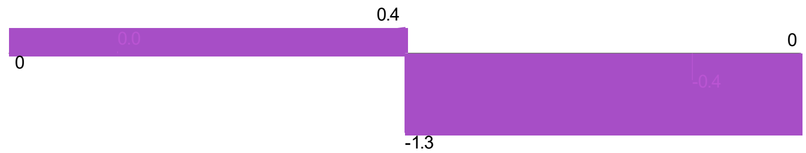
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **0.714** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **0.714** ksf

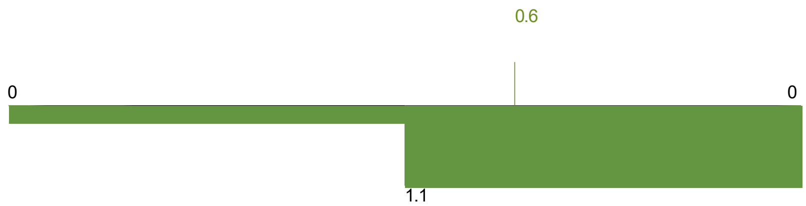
Minimum ultimate base pressure: $q_{umin} = \min(q_{u1}, q_{u2}, q_{u3}, q_{u4})$ = **0.321** ksf

Maximum ultimate base pressure: $q_{umax} = \max(q_{u1}, q_{u2}, q_{u3}, q_{u4})$ = **0.714** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment	$M_{u.x.max}$ = **0.603** kip_ft
Tension reinforcement provided	4 No.5 bottom bars (9.7 in c/c)
Area of tension reinforcement provided	$A_{sx.bot.prov}$ = **1.24** in^2
Minimum area of reinforcement (8.6.1.1)	$A_{s.min} = 0.0018 \times L_y \times h$ = **0.778** in^2
	PASS - Area of reinforcement provided exceeds minimum
Maximum spacing of reinforcement (8.7.2.2)	$s_{max} = \min(2 \times h, 18\ in)$ = **18** in
	PASS - Maximum permissible reinforcement spacing exceeds actual spacing
Depth to tension reinforcement	$d = h - c_{nom\_b} - \phi_{x.bot} / 2$ = **8.688** in
Depth of compression block	$a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y)$ = **0.608** in
Neutral axis factor	β_1 = **0.85**
Depth to neutral axis	$c = a / \beta_1$ = **0.715** in
Strain in tensile reinforcement	$\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.03345**
Minimum tensile strain(8.3.3.1)	$\varepsilon_{min} = \varepsilon_{ty} + 0.003$ = **0.00500**
	PASS - Tensile strain exceeds minimum required
Nominal moment capacity	$M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2)$ = **51.978** kip_ft
Flexural strength reduction factor	$\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9)$ = **0.900**
Design moment capacity	$\phi M_n = \phi_f \times M_n$ = **46.78** kip_ft
	$M_{u.x.max} / \phi M_n$ = **0.013**
	PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force	$V_{u.x}$ = **0.448** kips
Depth to reinforcement	$d_v = h - c_{nom\_b} - \phi_{x.bot} / 2$ = **8.688** in
Size effect factor (22.5.5.1.3)	λ_s = **1**
Ratio of longitudinal reinforcement	$\rho_w = A_{sx.bot.prov} / (L_y \times d_v)$ = **0.00396**

Shear strength reduction factor ϕ_v = **0.75**

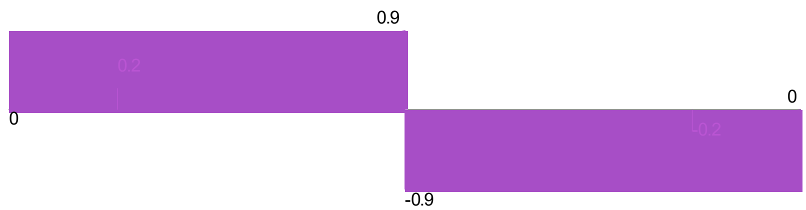
Nominal shear capacity (Eq. 22.5.5.1) $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1\ \text{psi})} \times L_y \times d_v,\ 5 \times \lambda \times \sqrt{(f'_c \times 1\ \text{psi})} \times L_y \times d_v)$ = **25.045** kips

Design shear capacity $\phi V_n = \phi_v \times V_n$ = **18.784** kips

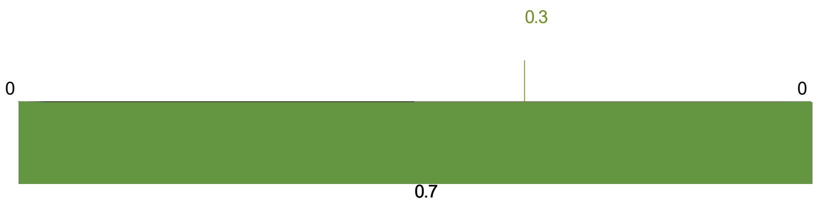
$V_{u.x} / \phi V_n$ = **0.024**

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 $M_{u.y.max}$ = **0.34** kip_ft

Tension reinforcement provided 4 No.5 bottom bars (9.7 in c/c)

Area of tension reinforcement provided $A_{sy.bot.prov}$ = **1.24** in^2

Minimum area of reinforcement (8.6.1.1) $A_{s.min} = 0.0018 \times L_x \times h$ = **0.778** 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 $d = h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **8.062** in

Depth of compression block $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.608** in

Neutral axis factor β_1 = **0.85**

Depth to neutral axis $c = a / \beta_1$ = **0.715** in

Strain in tensile reinforcement $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.03082**

Minimum tensile strain(8.3.3.1) $\varepsilon_{min} = \varepsilon_{ty} + 0.003$ = **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity $M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2)$ = **48.103** kip_ft

Flexural strength reduction factor $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9)$ = **0.900**

Design moment capacity $\phi M_n = \phi_f \times M_n =$ **43.293** kip_ft

$M_{u.y.max} / \phi M_n =$ **0.008**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force $V_{u.y} =$ **0.239** kips

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

Size effect factor (22.5.5.1.3) $\lambda_s =$ **1**

Ratio of longitudinal reinforcement $\rho_w = A_{sy.bot.prov} / (L_x \times d_v) =$ **0.00427**

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

Nominal shear capacity (Eq. 22.5.5.1) $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1\ psi)} \times L_x \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times L_x \times d_v) =$ **23.829** kips

Design shear capacity $\phi V_n = \phi_v \times V_n =$ **17.872** kips

$V_{u.y} / \phi V_n =$ **0.013**

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) $l_{xp} =$ **18.375** in

Shear perimeter width (22.6.4) $l_{yp} =$ **18.375** in

Shear perimeter (22.6.4) $b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2}) =$ **73.500** in

Shear area $A_p = l_{x,perim} \times l_{y,perim} =$ **337.641** in^2

Surcharge loaded area $A_{sur} = A_p - l_{x1} \times l_{y1} =$ **237.641** in^2

Ultimate bearing pressure at center of shear area $q_{up.avg} =$ **0.517** ksf

Ultimate shear load $F_{up} = \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p =$ **1.187** kips

Ultimate shear stress from vertical load $v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0\ psi) =$ **1.928** psi

Column geometry factor (Table 22.6.5.2) $\beta = l_{y1} / l_{x1} =$ **1.00**

Column location factor (22.6.5.3) $\alpha_s =$ **40**

Size effect factor (22.5.5.1.3) $\lambda_s =$ **1**

Concrete shear strength (22.6.5.2) $v_{cpa} = (2 + 4 / \beta) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\ psi)} =$ **379.473** psi

$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\ psi)} =$ **414.753** psi

$v_{cpc} = 4 \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\ psi)} =$ **252.982** psi

$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) =$ **252.982** psi

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

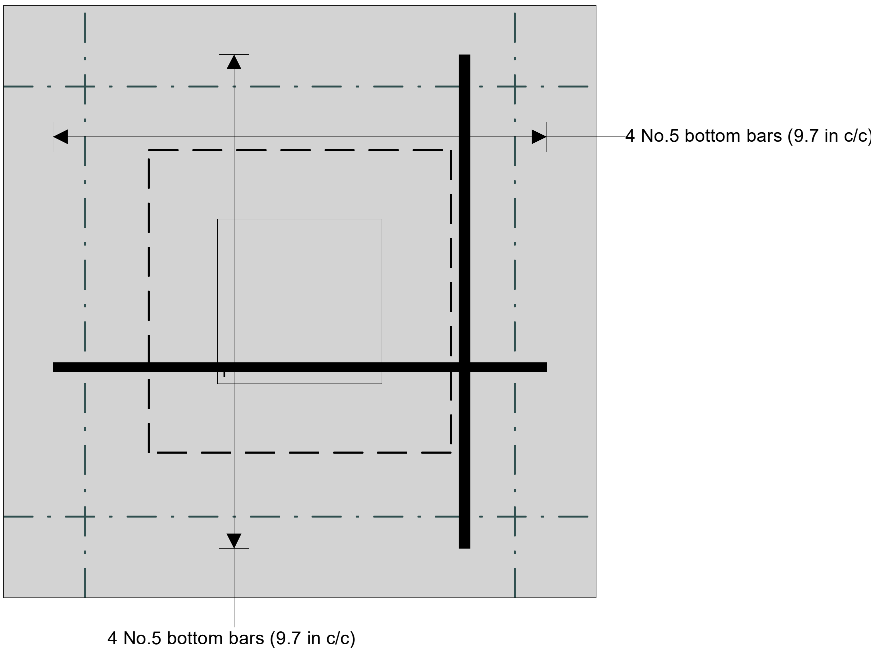
Nominal shear stress capacity (Eq. 22.6.1.2) $v_n = v_{cp} =$ **252.982** psi

Design shear stress capacity (8.5.1.1(d)) $\phi v_n = \phi_v \times v_n =$ **189.737** psi

$v_{ug} / \phi v_n =$ **0.010**

PASS - Design shear stress capacity exceeds ultimate shear stress load

kpff	Project OSU Azalea House	By MAA	Sheet No. 8
	Location Corvallis, OR	Date 2/12/2024	
	Client Rowell Brokaw	Revised	Job No. 223346
Portland, Oregon		Date	



22 - EXISTING GRID 8 FOOTING

Footing analysis in accordance with ACI318-19

Tedds calculation version 3.3.02

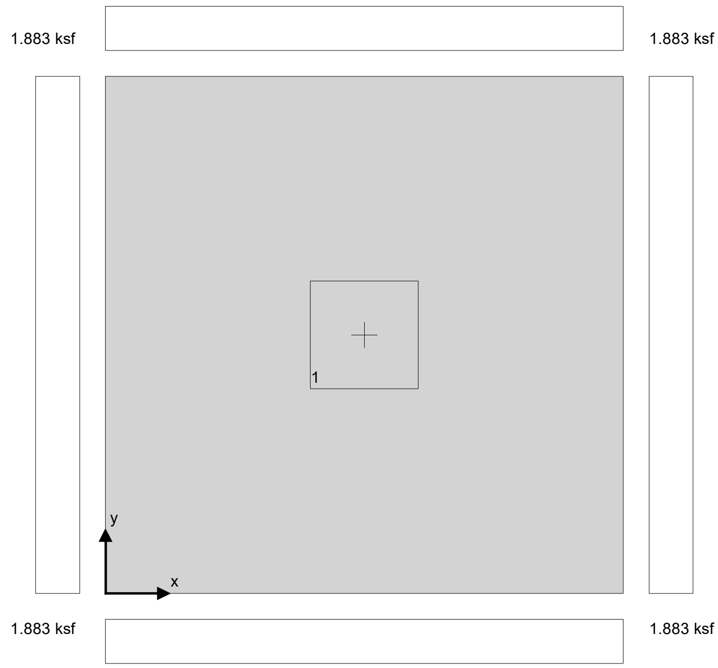
Summary results

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	30.1			Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	1.883	2.5	0.753	Pass
Description	**Unit**	**Provided**	**Required**	**Utilization**	**Result**
Moment, positive, x-direction	kip_ft	12.2	58.6	0.209	Pass
Moment, positive, y-direction	kip_ft	12.2	54.2	0.226	Pass
Shear, one-way, x-direction	kips	8.9	24.5	0.363	Pass
Shear, one-way, y-direction	kips	8.9	23.3	0.381	Pass
Shear, two-way, Col 1	psi	53.965	189.737	0.284	Pass
Min.area of reinf, bot., x-direction	in^2	1.037	1.550		Pass
Max.reinf.spacing, bot, x-direction	in	18.0	10.3		Pass
Min.area of reinf, bot., y-direction	in^2	1.037	1.550		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	10.3		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} = **12** in
Density of concrete	γ_{conc} = **150.0** lb/ft^3

kpff Portland, Oregon	Project OSU Azalea House	By MAA	Sheet No. 2
	Location Corvallis, OR	Date 2/12/2024	
	Client Rowell Brokaw	Revised	Job No. 223346
		Date	



Column no.1 details

Length of column l_{x1} = **10.00** in

Width of column l_{y1} = **10.00** in

position in x-axis x_1 = **24.00** in

position in y-axis y_1 = **24.00** in

Soil properties

Gross allowable bearing pressure q_{allow_Gross} = **2.5** ksf

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 $F_{soil} = h_{soil} \times \gamma_{soil}$ = **120** psf

Column no.1 loads

Dead load in z F_{Dz1} = **5.6** kips

Live load in z F_{Lz1} = **20.2** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.248)

1.0D + 1.0L (0.753)

Combination 2 results: 1.0D + 1.0L

Forces on footing

Force in z-axis — $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1} = \mathbf{30.1}$ kips

Moments on footing

Moment in x-axis, about x is 0 — $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) = \mathbf{60.2}$ kip_ft

Moment in y-axis, about y is 0 — $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) = \mathbf{60.2}$ kip_ft

Uplift verification

Vertical force — $F_{dz} = \mathbf{30.12}$ kips

PASS - Footing is not subject to uplift

Bearing resistance

Eccentricity of base reaction

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

Eccentricity of base reaction in y-axis — $e_{dy} = M_{dy} / F_{dz} - L_y / 2 = \mathbf{0}$ in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{1.882}$ ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{1.882}$ ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{1.882}$ ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = \mathbf{1.882}$ ksf

Minimum base pressure — $q_{min} = \min(q_1,q_2,q_3,q_4) = \mathbf{1.882}$ ksf

Maximum base pressure — $q_{max} = \max(q_1,q_2,q_3,q_4) = \mathbf{1.882}$ ksf

Allowable bearing capacity

Allowable bearing capacity — $q_{allow} = q_{allow\_Gross} = \mathbf{2.5}$ ksf

$q_{max} / q_{allow} = \mathbf{0.753}$

PASS - Allowable bearing capacity exceeds design base pressure

22 - EXISTING GRID 8 FOOTING

Footing design in accordance with ACI318-19

Tedds calculation version 3.3.02

Material details

Compressive strength of concrete — $f'_c = \mathbf{4000}$ psi

Yield strength of reinforcement — $f_y = \mathbf{60000}$ psi

Compression-controlled strain limit (21.2.2) — $\varepsilon_{ty} = \mathbf{0.00200}$

Cover to top of footing — $c_{nom\_t} = \mathbf{3}$ in

Cover to side of footing — $c_{nom\_s} = \mathbf{3}$ 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 (0.077)

1.2D + 1.6L + 0.5Lr (0.381)

Combination 2 results: 1.2D + 1.6L + 0.5Lr

Forces on footing

Ultimate force in z-axis — $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1}$ = **44.2** kips

Moments on footing

Ultimate moment in x-axis, about x is 0 — $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1) + \gamma_L \times (F_{Lz1} \times x_1)$ = **88.4** kip_ft

Ultimate moment in y-axis, about y is 0 — $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_L \times (F_{Lz1} \times y_1)$ = **88.4** 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 — $e_{uy} = M_{uy} / F_{uz} - L_y / 2$ = **0** in

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **2.764** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **2.764** ksf

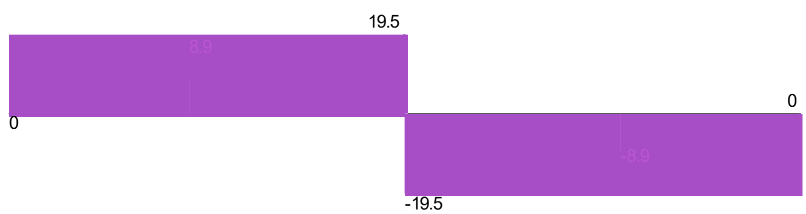
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **2.764** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **2.764** ksf

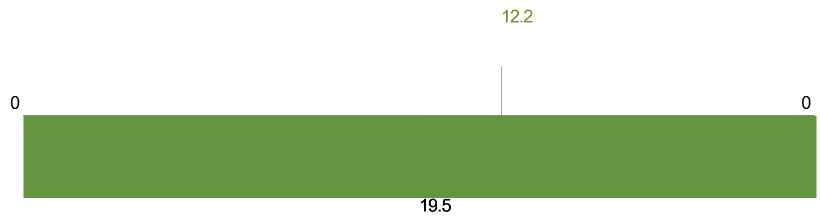
Minimum ultimate base pressure — $q_{umin} = \min(q_{u1}, q_{u2}, q_{u3}, q_{u4})$ = **2.764** ksf

Maximum ultimate base pressure — $q_{umax} = \max(q_{u1}, q_{u2}, q_{u3}, q_{u4})$ = **2.764** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment	$M_{u.x.max}$ = **12.234** kip_ft
Tension reinforcement provided	5 No.5 bottom bars (10.3 in c/c)
Area of tension reinforcement provided	$A_{sx.bot.prov}$ = **1.55** in^2
Minimum area of reinforcement (8.6.1.1)	$A_{s.min} = 0.0018 \times L_y \times h$ = **1.037** in^2

PASS - Area of reinforcement provided exceeds minimum

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

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement	$d = h - c_{nom\_b} - \phi_{x.bot} / 2$ = **8.688** in
Depth of compression block	$a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y)$ = **0.570** in
Neutral axis factor	β_1 = **0.85**
Depth to neutral axis	$c = a / \beta_1$ = **0.670** in
Strain in tensile reinforcement	$\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.03588**
Minimum tensile strain(8.3.3.1)	$\varepsilon_{min} = \varepsilon_{ty} + 0.003$ = **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity	$M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2)$ = **65.12** kip_ft
Flexural strength reduction factor	$\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9)$ = **0.900**
Design moment capacity	$\phi M_n = \phi_f \times M_n$ = **58.608** kip_ft
	$M_{u.x.max} / \phi M_n$ = **0.209**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force	$V_{u.x}$ = **8.896** kips
Depth to reinforcement	$d_v = h - c_{nom\_b} - \phi_{x.bot} / 2$ = **8.688** in
Size effect factor (22.5.5.1.3)	λ_s = **1**
Ratio of longitudinal reinforcement	$\rho_w = A_{sx.bot.prov} / (L_y \times d_v)$ = **0.00372**
Shear strength reduction factor	ϕ_v = **0.75**
Nominal shear capacity (Eq. 22.5.5.1)	$V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1\ psi)} \times L_y \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times L_y \times d_v)$ = **32.683** kips
Design shear capacity	$\phi V_n = \phi_v \times V_n$ = **24.512** kips
	$V_{u.x} / \phi V_n$ = **0.363**

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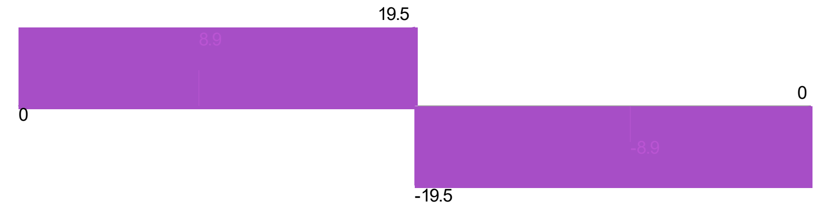
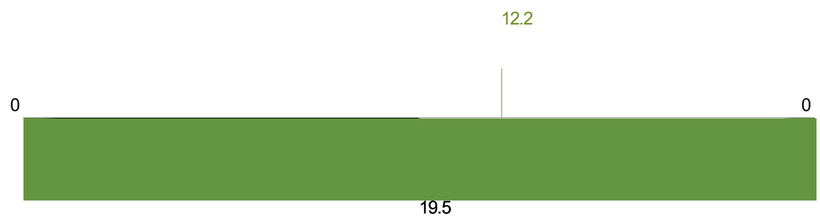
PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment — $M_{u.y.max}$ = **12.234** kip_ft

Tension reinforcement provided — 5 No.5 bottom bars (10.3 in c/c)

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

Minimum area of reinforcement (8.6.1.1) — $A_{s.min} = 0.0018 \times L_x \times h$ = **1.037** in^2

PASS - Area of reinforcement provided exceeds minimum

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

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

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

Depth of compression block — $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.570** in

Neutral axis factor — β_1 = **0.85**

Depth to neutral axis — $c = a / \beta_1$ = **0.670** in

Strain in tensile reinforcement — $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.03308**

Minimum tensile strain(8.3.3.1) — $\varepsilon_{min} = \varepsilon_{ty} + 0.003$ = **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity — $M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2)$ = **60.276** kip_ft

Flexural strength reduction factor — ϕ_f = min(max(0.65 + 0.25 × ($\varepsilon_t - \varepsilon_{ty}$) / (0.003), 0.65), 0.9) = **0.900**

Design moment capacity — $\phi M_n = \phi_f \times M_n$ = **54.249** kip_ft

$M_{u.y.max} / \phi M_n$ = **0.226**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force — $V_{u.y}$ = **8.896** kips

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

Size effect factor (22.5.5.1.3) $\lambda_s = $ **1**

Ratio of longitudinal reinforcement $\rho_w = A_{sy.bot.prov} / (L_x \times d_v) =$ **0.00401**

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

Nominal shear capacity (Eq. 22.5.5.1) $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1\text{ psi})} \times L_x \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} \times L_x \times d_v) =$ **31.096** kips

Design shear capacity $\phi V_n = \phi_v \times V_n =$ **23.322** kips

$V_{u.y} / \phi V_n =$ **0.381**

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) $l_{xp} =$ **18.375** in

Shear perimeter width (22.6.4) $l_{yp} =$ **18.375** in

Shear perimeter (22.6.4) $b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2}) =$ **73.500** in

Shear area $A_p = l_{x,perim} \times l_{y,perim} =$ **337.641** in^2

Surcharge loaded area $A_{sur} = A_p - l_{x1} \times l_{y1} =$ **237.641** in^2

Ultimate bearing pressure at center of shear area $q_{up.avg} =$ **2.764** ksf

Ultimate shear load $F_{up} = \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p =$ **33.219** kips

Ultimate shear stress from vertical load $v_{ug} = \max(F_{up} / (b_o \times d_{v2}),0\text{ psi}) =$ **53.965** psi

Column geometry factor (Table 22.6.5.2) $\beta = l_{y1} / l_{x1} =$ **1.00**

Column location factor (22.6.5.3) $\alpha_s =$ **40**

Size effect factor (22.5.5.1.3) $\lambda_s = $ **1**

Concrete shear strength (22.6.5.2) $v_{cpa} = (2 + 4 / \beta) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} =$ **379.473** psi

$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} =$ **414.753** psi

$v_{cpc} = 4 \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} =$ **252.982** psi

$v_{cp} = \min(v_{cpa},v_{cpb},v_{cpc}) =$ **252.982** psi

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

Nominal shear stress capacity (Eq. 22.6.1.2) $v_n = v_{cp} =$ **252.982** psi

Design shear stress capacity (8.5.1.1(d)) $\phi v_n = \phi_v \times v_n =$ **189.737** psi

$v_{ug} / \phi v_n =$ **0.284**

PASS - Design shear stress capacity exceeds ultimate shear stress load

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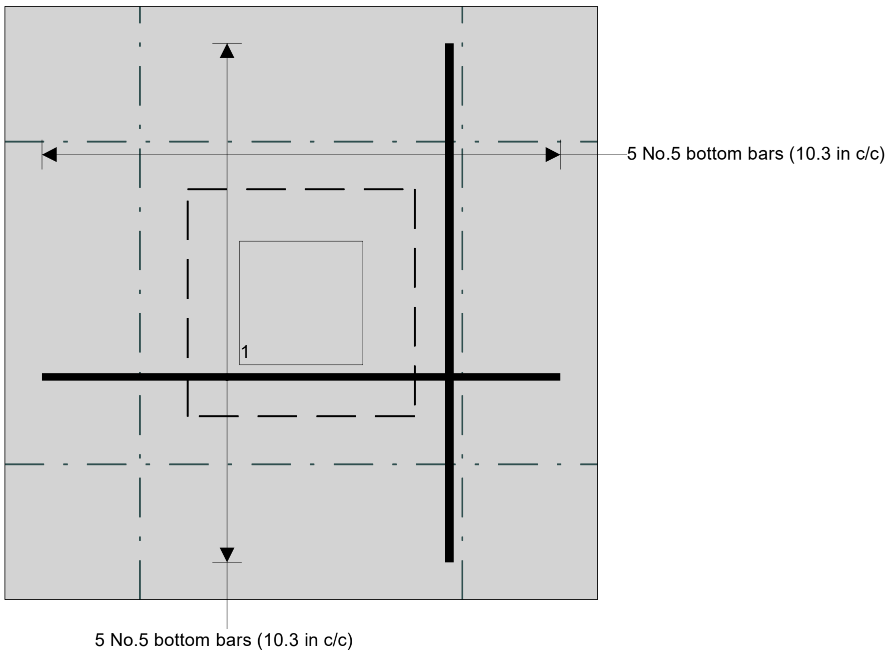

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23 - EXISTING GRID E FOOTING

Footing analysis in accordance with ACI318-19

Tedds calculation version 3.3.02

Summary results

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	39.6			Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	2.473	2.5	0.989	Pass
Description	**Unit**	**Provided**	**Required**	**Utilization**	**Result**
Moment, positive, x-direction	kip_ft	16.7	58.6	0.285	Pass
Moment, positive, y-direction	kip_ft	16.7	54.2	0.308	Pass
Shear, one-way, x-direction	kips	12.1	23.3	0.521	Pass
Shear, one-way, y-direction	kips	12.1	23.3	0.521	Pass
Shear, two-way, Col 1	psi	73.719	189.737	0.389	Pass
Min.area of reinf, bot., x-direction	in^2	1.037	1.550		Pass
Max.reinf.spacing, bot, x-direction	in	18.0	10.3		Pass
Min.area of reinf, bot., y-direction	in^2	1.037	1.550		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	10.3		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} = **12** in

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

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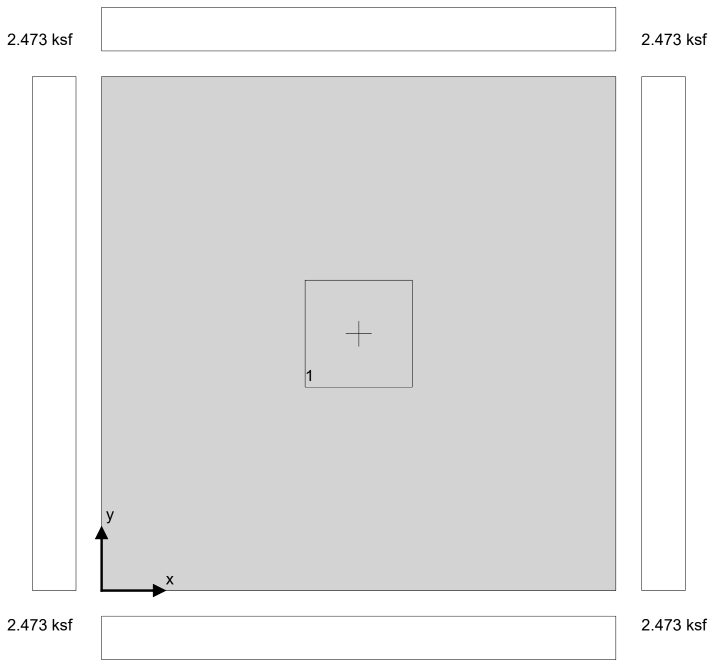

Column no.1 details

Length of column	l_{x1} = **10.00** in
Width of column	l_{y1} = **10.00** in
position in x-axis	x_1 = **24.00** in
position in y-axis	y_1 = **24.00** in

Soil properties

Gross allowable bearing pressure	q_{allow_Gross} = **2.5** ksf
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	$F_{soil} = h_{soil} \times \gamma_{soil}$ = **120** psf

Column no.1 loads

Dead load in z	F_{Dz1} = **7.7** kips
Live load in z	F_{Lz1} = **27.5** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.301)

1.0D + 1.0L (0.989)

Combination 2 results: 1.0D + 1.0L

Forces on footing

Force in z-axis — $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1}$ = **39.6** kips

Moments on footing

Moment in x-axis, about x is 0 — $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1) + \gamma_L \times (F_{Lz1} \times x_1)$ = **79.1** kip_ft

Moment in y-axis, about y is 0 — $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_L \times (F_{Lz1} \times y_1)$ = **79.1** kip_ft

Uplift verification

Vertical force — F_{dz} = **39.56** 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 — $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **0** in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **2.472** ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **2.472** ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **2.472** ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **2.472** ksf

Minimum base pressure — $q_{min} = \min(q_1,q_2,q_3,q_4)$ = **2.472** ksf

Maximum base pressure — $q_{max} = \max(q_1,q_2,q_3,q_4)$ = **2.472** ksf

Allowable bearing capacity

Allowable bearing capacity — $q_{allow} = q_{allow\_Gross}$ = **2.5** ksf

q_{max} / q_{allow} = **0.989**

PASS - Allowable bearing capacity exceeds design base pressure

23 - EXISTING GRID E FOOTING

Footing design in accordance with ACI318-19

Tedds calculation version 3.3.02

Material details

Compressive strength of concrete — f'_c = **4000** 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

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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 (0.101)

1.2D + 1.6L + 0.5Lr (0.521)

Combination 2 results: 1.2D + 1.6L + 0.5Lr

Forces on footing

Ultimate force in z-axis $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1}$ = **58.5** kips

Moments on footing

Ultimate moment in x-axis, about x is 0 $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1) + \gamma_L \times (F_{Lz1} \times x_1)$ = **116.9** kip_ft

Ultimate moment in y-axis, about y is 0 $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_L \times (F_{Lz1} \times y_1)$ = **116.9** 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 $e_{uy} = M_{uy} / F_{uz} - L_y / 2$ = **0** in

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **3.654** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **3.654** ksf

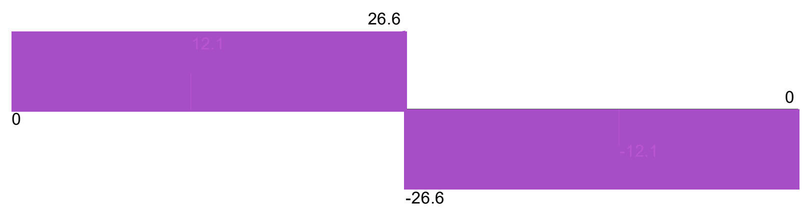
$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **3.654** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **3.654** ksf

Minimum ultimate base pressure $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4})$ = **3.654** ksf

Maximum ultimate base pressure $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4})$ = **3.654** ksf

Shear diagram, x axis (kips)



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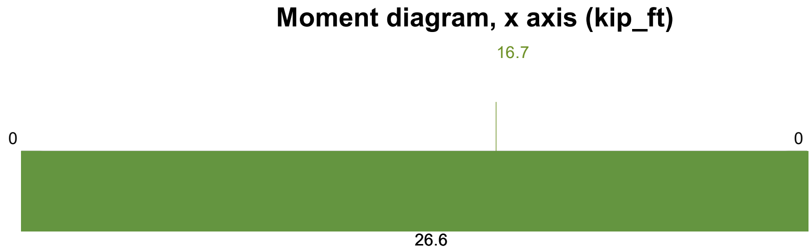

Moment design, x direction, positive moment

Ultimate bending moment — $M_{u.x.max}$ = **16.699** kip_ft

Tension reinforcement provided — 5 No.5 bottom bars (10.3 in c/c)

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

Minimum area of reinforcement (8.6.1.1) — $A_{s.min} = 0.0018 \times L_y \times h$ = **1.037** in^2

PASS - Area of reinforcement provided exceeds minimum

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

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

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

Depth of compression block — $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y)$ = **0.570** in

Neutral axis factor — β_1 = **0.85**

Depth to neutral axis — $c = a / \beta_1$ = **0.670** in

Strain in tensile reinforcement — $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.03588**

Minimum tensile strain(8.3.3.1) — $\varepsilon_{min} = \varepsilon_{ty} + 0.003$ = **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity — $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2)$ = **65.12** kip_ft

Flexural strength reduction factor — $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9)$ = **0.900**

Design moment capacity — $\phi M_n = \phi_f \times M_n$ = **58.608** kip_ft

$M_{u.x.max} / \phi M_n$ = **0.285**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force — $V_{u.x}$ = **12.142** kips

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

Size effect factor (22.5.5.1.3) — λ_s = **1**

Ratio of longitudinal reinforcement — $\rho_w = A_{sx.bot.prov} / (L_y \times d_v)$ = **0.00372**

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

Nominal shear capacity (Eq. 22.5.5.1) — $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_y \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_y \times d_v)$ = **32.683** kips

Design shear capacity — $\phi V_n = \phi_v \times V_n$ = **24.512** kips

$V_{u.x} / \phi V_n$ = **0.495**

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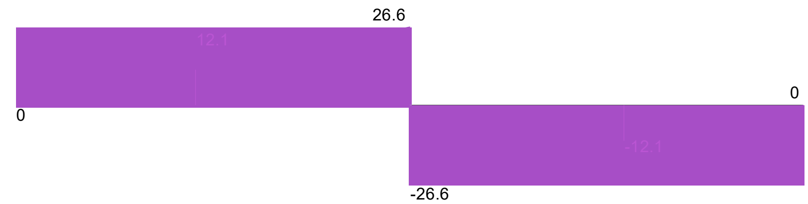
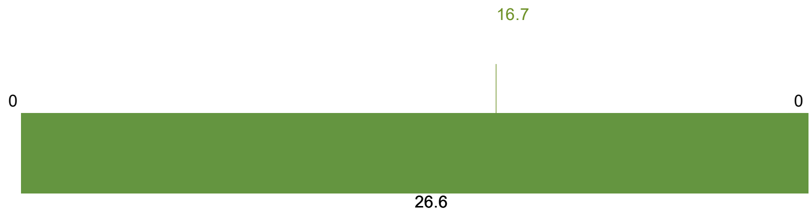
PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)



Moment diagram, y axis (kip_ft)



Moment design, y direction, positive moment

Ultimate bending moment — $M_{u.y.max}$ = **16.699** kip_ft

Tension reinforcement provided — 5 No.5 bottom bars (10.3 in c/c)

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

Minimum area of reinforcement (8.6.1.1) — $A_{s.min} = 0.0018 \times L_x \times h$ = **1.037** in^2

PASS - Area of reinforcement provided exceeds minimum

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

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

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

Depth of compression block — $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.570** in

Neutral axis factor — β_1 = **0.85**

Depth to neutral axis — $c = a / \beta_1$ = **0.670** in

Strain in tensile reinforcement — $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.03308**

Minimum tensile strain(8.3.3.1) — $\varepsilon_{min} = \varepsilon_{ty} + 0.003$ = **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity — $M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2)$ = **60.276** kip_ft

Flexural strength reduction factor — $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9)$ = **0.900**

Design moment capacity — $\phi M_n = \phi_f \times M_n$ = **54.249** kip_ft

$M_{u.y.max} / \phi M_n$ = **0.308**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force — $V_{u.y}$ = **12.142** kips

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

Size effect factor (22.5.5.1.3) — λ_s = **1**

Ratio of longitudinal reinforcement — $\rho_w = A_{sy.bot.prov} / (L_x \times d_v) =$ **0.00401**

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

Nominal shear capacity (Eq. 22.5.5.1) — $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1\ psi)} \times L_x \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times L_x \times d_v) =$ **31.096** kips

Design shear capacity — $\phi V_n = \phi_v \times V_n =$ **23.322** kips

$V_{u.y} / \phi V_n$ = **0.521**

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} = **18.375** in

Shear perimeter width (22.6.4) — l_{yp} = **18.375** in

Shear perimeter (22.6.4) — $b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2}) =$ **73.500** in

Shear area — $A_p = l_{x,perim} \times l_{y,perim} =$ **337.641** in^2

Surcharge loaded area — $A_{sur} = A_p - l_{x1} \times l_{y1} =$ **237.641** in^2

Ultimate bearing pressure at center of shear area — $q_{up.avg}$ = **3.654** ksf

Ultimate shear load — $F_{up} = \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p =$ **45.379** kips

Ultimate shear stress from vertical load — $v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0\ psi) =$ **73.719** 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**

Size effect factor (22.5.5.1.3) — λ_s = **1**

Concrete shear strength (22.6.5.2) — $v_{cpa} = (2 + 4 / \beta) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\ psi)} =$ **379.473** psi

$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\ psi)} =$ **414.753** psi

$v_{cpc} = 4 \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1\ psi)} =$ **252.982** psi

$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) =$ **252.982** psi

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

Nominal shear stress capacity (Eq. 22.6.1.2) — $v_n = v_{cp} =$ **252.982** psi

Design shear stress capacity (8.5.1.1(d)) — $\phi v_n = \phi_v \times v_n =$ **189.737** psi

$v_{ug} / \phi v_n$ = **0.389**

PASS - Design shear stress capacity exceeds ultimate shear stress load

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