STRUCTURAL CALCULATIONS

PROJECT:

SUGARHOUSE PARK - BIG FIELD
SUGARHOUSE PARK
SALT LAKE CITY, UT

PROFESSIONAL ENGINEER
2/28/2025
No. 10272167-2202
RYAN R.
HAZELWOOD
STATE OF UTAH

PROJECT NUMBER:

25-039

CLIENT:

PLAYSPACE DESIGNS

DATE:

2/27/2025

BY:

SAMUEL SPANN, PE

APPROVED BY:

RYAN HAZELWOOD, PE

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Project #:	25-039
Project:	SUGARHOUSE PARK - BIG FIELD
By:	SAMUEL SPANN, PE

Date 2/27/2025

PROJECT INFORMATION

ADDRESS:	SUGARHOUSE PARK
	SALT LAKE CITY, UT
COUNTY:	SALT LAKE
LAT:	40.72
LONG:	-111.85
ELEVATION:	4413

PROJECT DESCRIPTION

FOUNDATIONS FOR PAVILION

DESIGN CRITERIA

STRUCTURE TYPE:	COMMERCIAL
DESIGN CODE:	2021 IBC
RISK CATEGORY:	II

DESIGN LOADS

ROOF LOADS

DL	6	PSF
LLR	20	PSF

THE LOADS SHOWN ON THIS PAGE ARE ONLY USED TO REVIEW PEMB FRAME LOADS (IF SUPPLIED)

SNOW LOADS

GROUND SNOW LOADS (P_g)	**31**	PSF

WIND LOADS

WIND SPEED	105	MPH
EXPOSURE	C	

SEISMIC LOADS

Ss	1.39	g
S1	0.52	g
SITE CLASS	D (DEFAULT)	

FOUNDATION CRITERIA

SOILS REPORT	
COMPANY	TERRACON
DATE	12/27/2024
NUMBER	61245209

ALLOWABLE BEARING PRESSURE	1400	PSF
1/3 INCREASE FOR TEMPORARY LOADS:	1862	PSF
PASSIVE PRESSURE	315	PSF
ACTIVE PRESSURE	45	PSF
AT REST PRESSURE	60	PSF
COEFFICIENT OF FRICTION (μ)	0.45	

FOOTING ANALYSIS

In accordance with ACI318-14

Tedds calculation version 3.3.08

Summary results

Overall design status PASS

Overall design utilisation 0.791

Description	Unit	Applied	Resisting	FoS	Result
Uplift verification	kips	35.9			Pass
Overturning stability, x	kip_ft	109.07	-170.46	1.56	Pass
Sliding stability, x	kips	8.2	16.1	1.969	Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	1.472	1.86	0.791	Pass
Description	**Unit**	**Required**	**Provided**	**Utilization**	**Result**
Moment, positive, x-direction	kip_ft	81.8	564.1	0.145	Pass
Moment, negative, x-direction	kip_ft	46.2	602.7	0.077	Pass
Moment, positive, y-direction	kip_ft	1.5	544.8	0.003	Pass
Shear, one-way, x-direction	kips	23.9	193.4	0.123	Pass
Shear, one-way, y-direction	kips	0.4	187.0	0.002	Pass
Min.area of reinf, bot., x-direction	in^2	5.335	5.720		Pass
Min.area of reinf, top, x-direction	in^2	5.335	5.720		Pass
Max.reinf.spacing, top, x-direction	in	18.0	9.1		Pass
Min.area of reinf, bot., y-direction	in^2	5.335	5.720		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	9.1		Pass

Pad footing details

Length of footing L_x = **9.5** ft

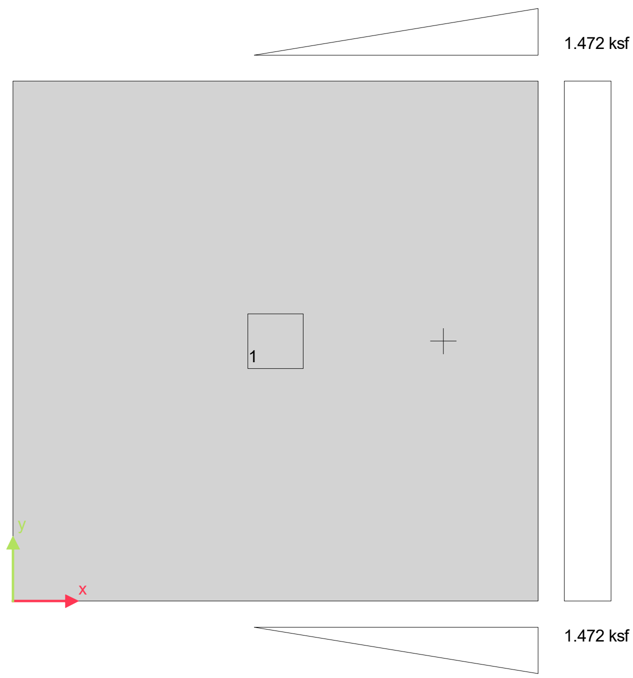
Width of footing L_y = **9.5** ft

Footing area $A = L_x \times L_y$ = **90.25** ft^2

Depth of footing h = **26** in

Depth of soil over footing h_{soil} = **6** in

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



Column no.1 details

Length of column l_{x1} = **12.00** in

Width of column l_{y1} = **12.00** in

position in x-axis x_1 = **57.00** in

position in y-axis y_1 = **57.00** in

Soil properties

Gross allowable bearing pressure q_{allow_Gross} = **1.86** 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} = **24.2** deg

Coefficient of base friction $\tan(\delta_{bb})$ = **0.450**

Footing loads

Self weight $F_{swt} = h \times \gamma_{conc}$ = **325** psf

Soil weight $F_{soil} = h_{soil} \times \gamma_{soil}$ = **60** psf

Column no.1 loads

Dead load in z F_{Dz1} = **1.2** kips

Dead load in x F_{Dx1} = **8.2** kips

Dead load moment in x M_{Dx1} = **91.3** kip_ft

Footing analysis for soil and stability

Load combinations per ASCE 7-16

1.0D (0.791)

Combination 1 results: 1.0D

Forces on footing

Force in x-axis $F_{dx} = \gamma_D \times F_{Dx1} = \mathbf{8.2}$ kips

Force in z-axis $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) = \mathbf{35.9}$ kips

Moments on footing

Moment in x-axis, about x is 0 $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1 + M_{Dx1} + F_{Dx1} \times (h)) = \mathbf{279.5}$ kip_ft

Moment in y-axis, about y is 0 $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1) = \mathbf{170.5}$ kip_ft

Uplift verification

Vertical force $F_{dz} = \mathbf{35.886}$ kips

PASS - Footing is not subject to uplift

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

Overturning moment $M_{OTxL} = \gamma_D \times (M_{Dx1} + F_{Dx1} \times (h)) = \mathbf{109.07}$ kip_ft

Resisting moment $M_{RxL} = -1 \times (\gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2)) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times (x_1 - L_x)) = \mathbf{-170.46}$ kip_ft

Factor of safety $abs(M_{RxL} / M_{OTxL}) = \mathbf{1.563}$

PASS - Overturning moment safety factor exceeds the minimum of 1.50

Stability against sliding

Resistance due to base friction $F_{RFriction} = max(F_{dz}, 0\ kN) \times tan(\delta_{bb}) = \mathbf{16.149}$ kips

Stability against sliding in x direction

Total sliding resistance $F_{Rx} = F_{RFriction} = \mathbf{16.149}$ kips

Factor of safety $abs(F_{Rx} / F_{dx}) = \mathbf{1.97}$

PASS - Sliding factor of safety exceeds the minimum of 1.50

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis $e_{dx} = M_{dx} / F_{dz} - L_x / 2 = \mathbf{36.471}$ 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 $L'_{xd} = min(L_x, 3 \times (L_x / 2 - abs(e_{dx}))) = \mathbf{61.588}$ in

Pad base pressures

$q_1 = \mathbf{0}$ ksf

$q_2 = \mathbf{0}$ ksf

$q_3 = \mathbf{1.472}$ ksf

$q_4 = \mathbf{1.472}$ ksf

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

Maximum base pressure $q_{max} = max(q_1, q_2, q_3, q_4) = \mathbf{1.472}$ ksf

Allowable bearing capacity

Allowable bearing capacity $q_{allow} = q_{allow\_Gross} = \mathbf{1.86}$ ksf

$q_{max} / q_{allow} = \mathbf{0.791}$

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN

In accordance with ACI318-14

Tedds calculation version 3.3.08

Material details

Compressive strength of concrete — f'_c = **2500** psi

Yield strength of reinforcement — f_y = **60000** psi

Compression-controlled strain limit (21.2.2) — ε_{ty} = **0.00200**

Cover to top of footing — c_{nom_t} = **1.5** in

Cover to side of footing — c_{nom_s} = **2** 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 (0.145)

Combination 1 results: 1.4D

Forces on footing

Ultimate force in x-axis — $F_{ux} = \gamma_D \times F_{Dx1}$ = **11.5** kips

Ultimate force in z-axis — $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})$ = **50.2** kips

Moments on footing

Ultimate moment in x-axis, about x is 0 — $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times x_1 + M_{Dx1} + F_{Dx1} \times (h))$ = **391.3** kip_ft

Ultimate moment in y-axis, about y is 0 — $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (((F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil})) \times y_1)$ = **238.6** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis — $e_{ux} = M_{ux} / F_{uz} - L_x / 2$ = **36.471** in

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

Length of bearing in x-axis — $L'_{xu} = \min(L_x, 3 \times (L_x / 2 - \text{abs}(e_{ux})))$ = **61.588** in

Pad base pressures

q_{u1} = **0** ksf

q_{u2} = **0** ksf

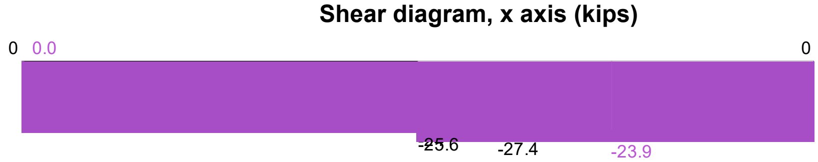
q_{u3} = **2.061** ksf

q_{u4} = **2.061** ksf

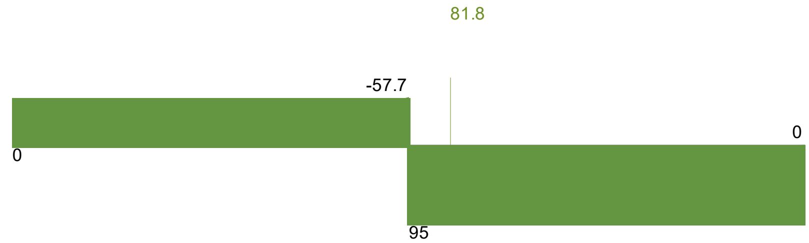
Minimum ultimate base pressure — $q_{umin} = \min(q_{u1}, q_{u2}, q_{u3}, q_{u4})$ = **0** ksf

Maximum ultimate base pressure — $q_{umax} = \max(q_{u1}, q_{u2}, q_{u3}, q_{u4})$ = **2.061** ksf

Shear diagram, x axis (kips)



Moment diagram, x axis (kip_ft)



Moment design, x direction, positive moment

Ultimate bending moment $M_{u.x.max}$ = **81.765** kip_ft

Tension reinforcement provided 13 No.6 bottom bars (9.1 in c/c)

Area of tension reinforcement provided $A_{sx.bot.prov}$ = **5.72** in^2

Minimum area of reinforcement (8.6.1.1) $A_{s.min} = 0.0018 \times L_y \times h$ = **5.335** 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 $d = h - c_{nom\_b} - \phi_{x.bot} / 2$ = **22.625** in

Depth of compression block $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y)$ = **1.417** in

Neutral axis factor β_1 = **0.85**

Depth to neutral axis $c = a / \beta_1$ = **1.667** in

Strain in tensile reinforcement $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.03772**

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

PASS - Tensile strain exceeds minimum required

Nominal moment capacity $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2)$ = **626.816** kip_ft

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

Design moment capacity $\phi M_n = \phi_f \times M_n$ = **564.134** kip_ft

$M_{u.x.max} / \phi M_n$ = **0.145**

PASS - Design moment capacity exceeds ultimate moment load

Moment design, x direction, negative moment

Ultimate bending moment $M_{u.x.min}$ = **-46.237** kip_ft

Tension reinforcement provided 13 No.6 top bars (9.1 in c/c)

Area of tension reinforcement provided $A_{sx.top.prov}$ = **5.72** in^2

Minimum area of reinforcement (8.6.1.1) $A_{s.min} = 0.0018 \times L_y \times h$ = **5.335** 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 $d = h - c_{nom\_t} - \phi_{x.top} / 2$ = **24.125** in

Depth of compression block $a = A_{sx.top.prov} \times f_y / (0.85 \times f'_c \times L_y)$ = **1.417** in

Neutral axis factor β_1 = **0.85**

Depth to neutral axis $c = a / \beta_1$ = **1.667** in

Strain in tensile reinforcement $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.04042**

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

PASS - Tensile strain exceeds minimum required

Nominal moment capacity — $M_n = A_{sx.top.prov} \times f_y \times (d - a / 2) = \mathbf{669.716}$ kip_ft

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

Design moment capacity — $\phi M_n = \phi_f \times M_n = \mathbf{602.744}$ kip_ft

$abs(M_{u.x.min}) / \phi M_n = \mathbf{0.077}$

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force — $V_{u.x} = \mathbf{23.854}$ kips

Depth to reinforcement — $d_v = \min(h - c_{nom\_b} - \phi_{x.bot} / 2, h - c_{nom\_t} - \phi_{x.top} / 2) = \mathbf{22.625}$ in

Shear strength reduction factor — $\phi_v = \mathbf{0.75}$

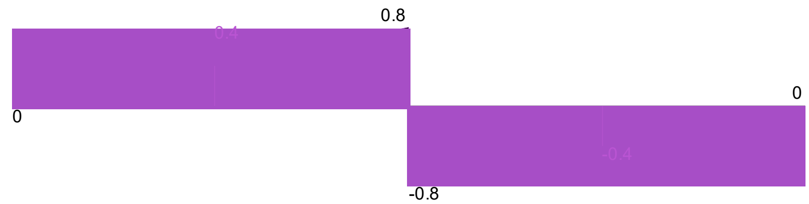
Nominal shear capacity (Eq. 22.5.5.1) — $V_n = 2 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times L_y \times d_v = \mathbf{257.925}$ kips

Design shear capacity — $\phi V_n = \phi_v \times V_n = \mathbf{193.444}$ kips

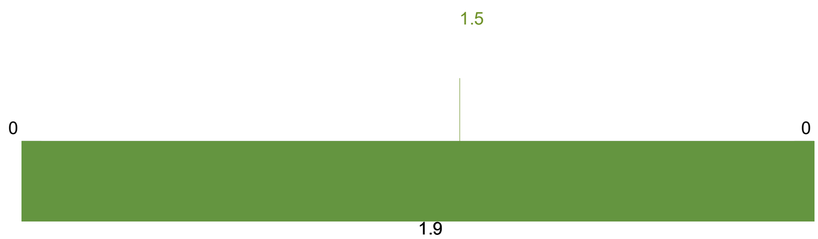
$V_{u.x} / \phi V_n = \mathbf{0.123}$

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 — $M_{u.y.max} = \mathbf{1.517}$ kip_ft

Tension reinforcement provided — 13 No.6 bottom bars (9.1 in c/c)

Area of tension reinforcement provided — $A_{sy.bot.prov} = \mathbf{5.72}$ in^2

Minimum area of reinforcement (8.6.1.1) — $A_{s.min} = 0.0018 \times L_x \times h = \mathbf{5.335}$ in^2

PASS - Area of reinforcement provided exceeds minimum

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

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement — $d = h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2 = \mathbf{21.875}$ in

Depth of compression block — $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x) = \mathbf{1.417}$ in

Neutral axis factor — $\beta_1 = \mathbf{0.85}$

Depth to neutral axis — $c = a / \beta_1 = \mathbf{1.667}$ in

Strain in tensile reinforcement — $\varepsilon_t = 0.003 \times d / c - 0.003 = \mathbf{0.03637}$

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

PASS - Tensile strain exceeds minimum required

Nominal moment capacity $M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2)$ = **605.366** kip_ft

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

Design moment capacity $\phi M_n = \phi_f \times M_n$ = **544.829** kip_ft

$M_{u.y.max} / \phi M_n$ = **0.003**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force $V_{u.y}$ = **0.408** kips

Depth to reinforcement $d_v = \min(h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2, h - c_{nom\_t} - \phi_{y.top} / 2)$ = **21.875** in

Shear strength reduction factor ϕ_v = **0.75**

Nominal shear capacity (Eq. 22.5.5.1) $V_n = 2 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_x \times d_v$ = **249.375** kips

Design shear capacity $\phi V_n = \phi_v \times V_n$ = **187.031** kips

$V_{u.y} / \phi V_n$ = **0.002**

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement d_{v2} = **22.25** in

Shear perimeter length (22.6.4) l_{xp} = **74.125** in

Shear perimeter width (22.6.4) l_{yp} = **74.125** in

Shear perimeter (22.6.4) $b_o = l_{x,perim} + l_{y,perim}$ = **148.250** in

Shear area $A_p = l_{x,perim} \times l_{y,perim}$ = **5494.516** in^2

Surcharge loaded area $A_{sur} = A_p - l_{x1} \times l_{y1}$ = **5350.516** in^2

Ultimate bearing pressure at center of shear area $q_{up.avg}$ = **2.575** ksf

Ultimate shear load $F_{up} = \gamma_D \times (F_{Dz1} - l_{x1} \times l_{y1} \times h_{soil} \times \gamma_{soil}) + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p$ = **-76.156** kips

Ultimate shear stress from vertical load $v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0 \text{ psi})$ = **0.000** psi

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

Column location factor (22.6.5.3) α_s =**20**

Concrete shear strength (22.6.5.2) $v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})}$ = **300.000** psi

$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})}$ = **250.084** psi

$v_{cpc} = 4 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})}$ = **200.000** psi

$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc})$ = **200.000** psi

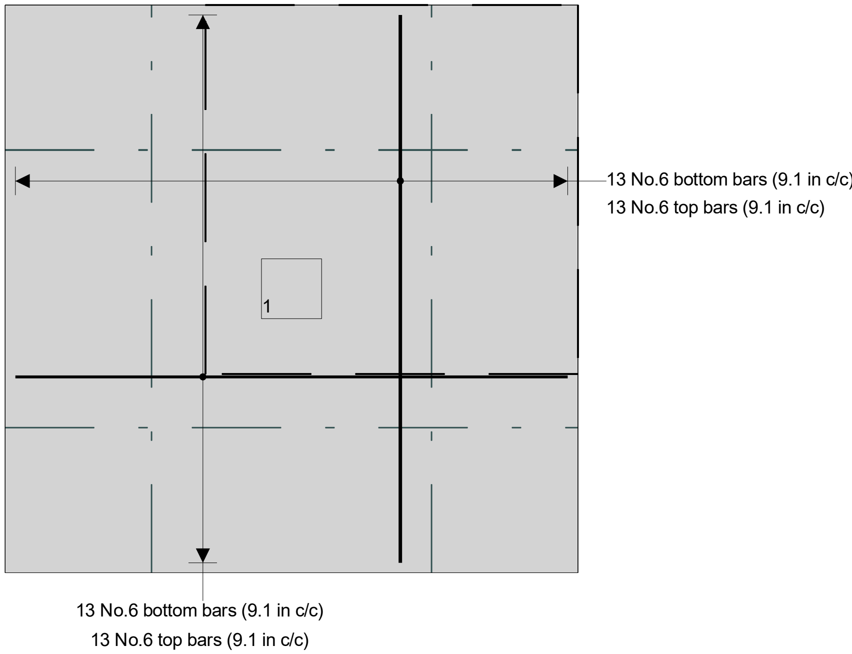
Shear strength reduction factor ϕ_v = **0.75**

Nominal shear stress capacity (Eq. 22.6.1.2) $v_n = v_{cp}$ = **200.000** psi

Design shear stress capacity (8.5.1.1(d)) $\phi v_n = \phi_v \times v_n$ = **150.000** psi

$v_{ug} / \phi v_n$ = **0.000**

PASS - Design shear stress capacity exceeds ultimate shear stress load



1
13 No.6 bottom bars (9.1 in c/c)
13 No.6 top bars (9.1 in c/c)
13 No.6 bottom bars (9.1 in c/c)
13 No.6 top bars (9.1 in c/c)