STRUCTURAL ENGINEERS

Project Paragon Container Bar Project No. 24-385

Calc. By NS Checked By TJ Date 11/6/2024

STRUCTURAL CALCULATIONS FOR :

Paragon Container Bar
Lee's Summit, MO

STATE OF MISSOURI
TRAVIS A. JENNINGS
NUMBER
PE-2023011045
11/08/2024
PROFESSIONAL ENGINEER

BSE Structural Engineers LLC
9911 Pflumm Road
Lenexa, Kansas 66215
Phone 913.492.7400
www.BSEstructural.com

From: Travis Jennings - BSE
To: Whom It May Concern
RE: Paragon Container Bar – Overturning Assessment

Date: 11/8/2024
Project No: 24-385

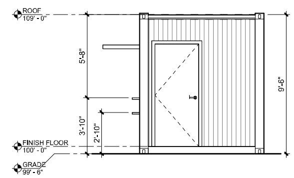
BSE was asked by Finkle + Williams to check the overturning potential of this new container structure against design lateral forces as determined based on the governing International Building Code (IBC) and the American Society of Civil Engineers Minimum Design Loads for Buildings and Other Structures (ASCE 7). We also provide our analysis and design of the container connection points to the existing paving slab design by others.

PLAN NOTE LEGEND

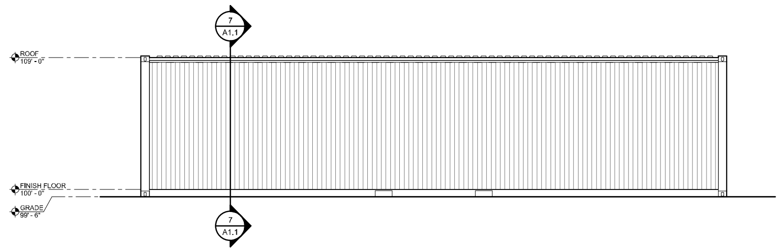
- A TEXT NOTE
- B TEXT NOTE

CONSTRUCTION GENERAL NOTES

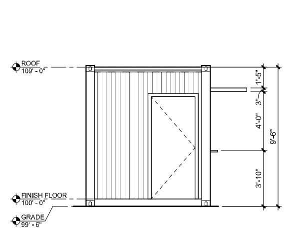
1. TBD



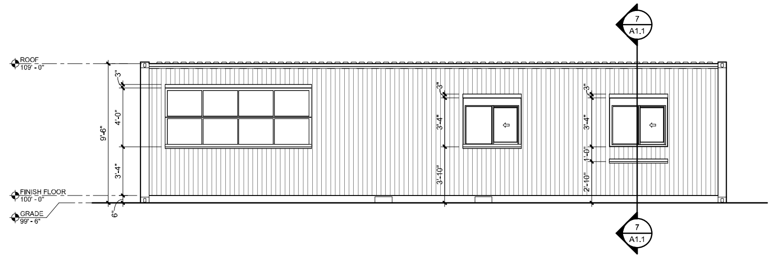
6 RIGHT ELEVATION
A1.1 SCALE : 1/4" = 1'-0"



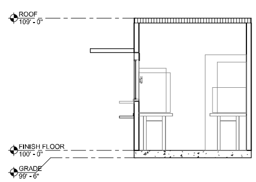
4 REAR ELEVATION
A1.1 SCALE : 1/4" = 1'-0"



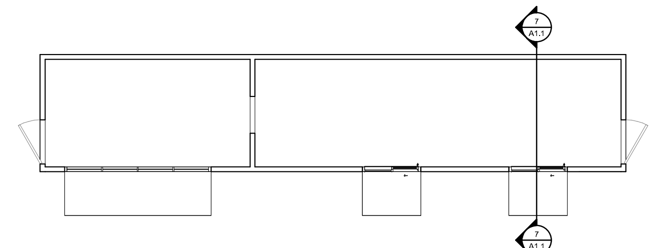
5 LEFT ELEVATION
A1.1 SCALE : 1/4" = 1'-0"



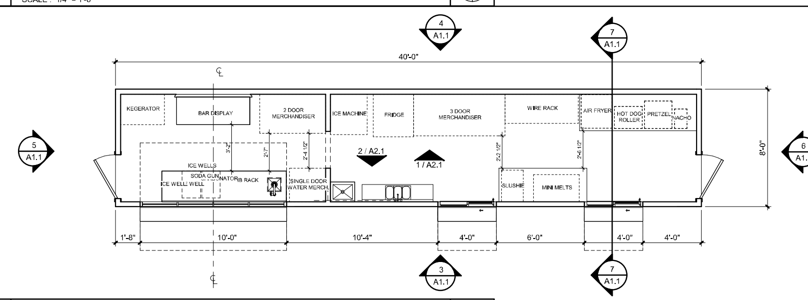
3 FRONT ELEVATION
A1.1 SCALE : 1/4" = 1'-0"



7 CROSS SECTION
A1.1 SCALE : 1/4" = 1'-0"



2 RCP
A1.1 SCALE : 1/4" = 1'-0"



1 FLOOR PLAN
A1.1 SCALE : 1/4" = 1'-0"

EXCERPT FROM ARCH DRAWINGS PROVIDED TO BSE ON 11/4/24

CONTAINER STRUCTURE IS NOT BY BSE

PARAGON - CONTAINER BAR

1401 NW River Rd.
Lee's Summit, MO 64081

Project No.: 19050.09
Date: 10/25/2024
Issued For: PERMIT

REVISIONS

No.	Date	Description

REGISTRATION

NOT FOR CONSTRUCTION

PROJECT TEAM

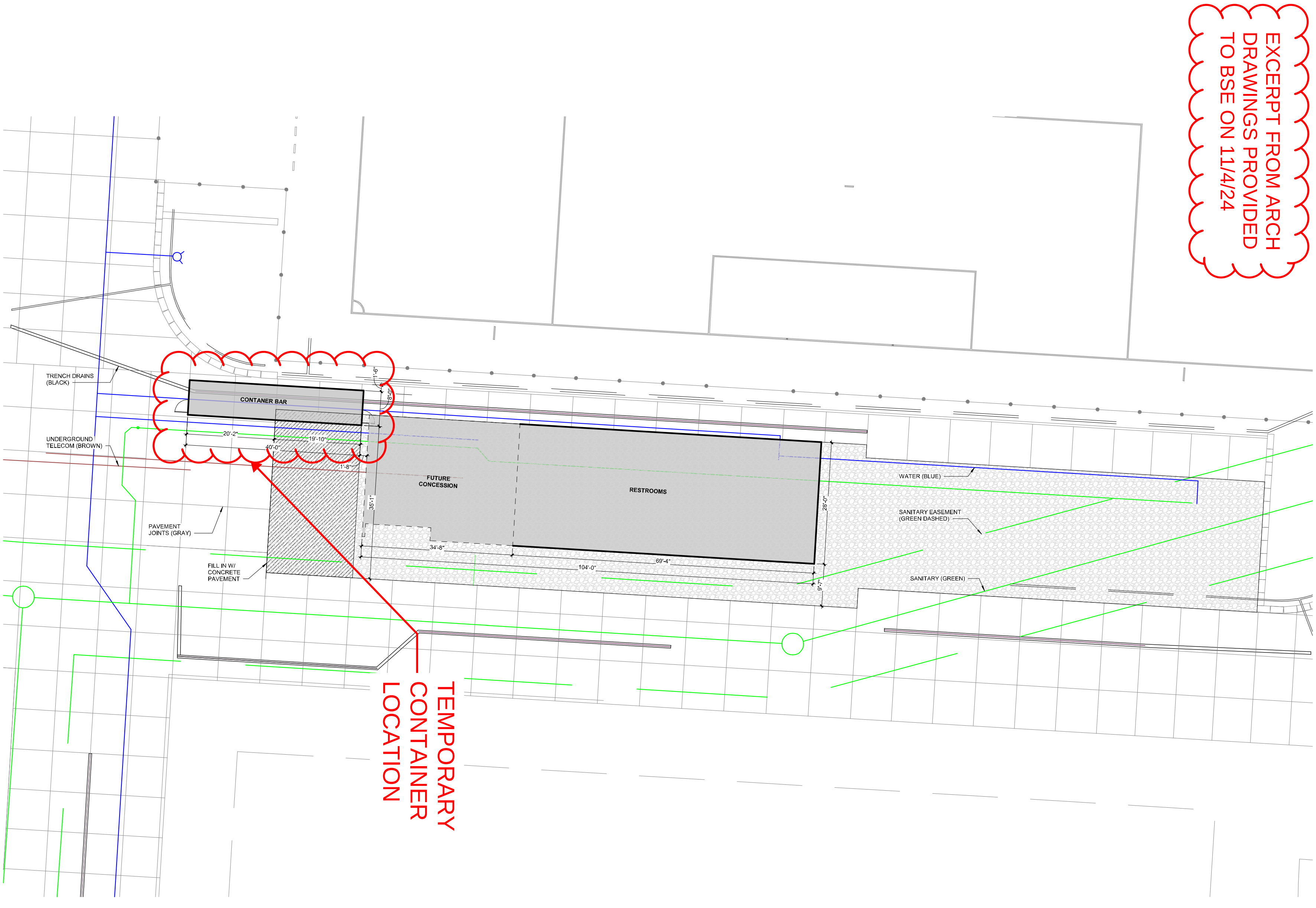
ARCHITECT	FINKLE+WILLIAMS ARCHITECTURE
CIVIL	GBA
LANDSCAPE	N/A
FOUNDATIONS	N/A
STRUCTURAL	N/A
PLUMBING	PKMR
MECHANICAL	PKMR
ELECTRICAL	PKMR
FIRE PROTECTION	N/A
CONTRACTOR	AT&S

FINKLE + WILLIAMS
ARCHITECTURE

8787 RENNER BLVD., SUITE 100
LENEXA, KANSAS 66219
913 .498.1550
www.finklewilliams.com

SHEET TITLE
FLOOR/RCP PLANS & ELEVATIONS

SHEET NUMBER
A1.1



1 SITE PLAN
A0.10 SCALE : 1" = 10'-0"

PARAGON - CONTAINER BAR

1401 NW River Rd.
Lee's Summit, MO 64081

Project No.: 19050.09
Date: 11/04/2024
Issued For: SITE PLAN & UTILITIES

REVISIONS

No.	Date	Description

REGISTRATION

NOT FOR CONSTRUCTION

PROJECT TEAM

ARCHITECT	FINKLE+WILLIAMS ARCHITECTURE
CIVIL	GBA
LANDSCAPE	N/A
FOUNDATIONS	N/A
STRUCTURAL	N/A
PLUMBING	PKMR
MECHANICAL	PKMR
ELECTRICAL	PKMR
FIRE PROTECTION	N/A
CONTRACTOR	AT&S

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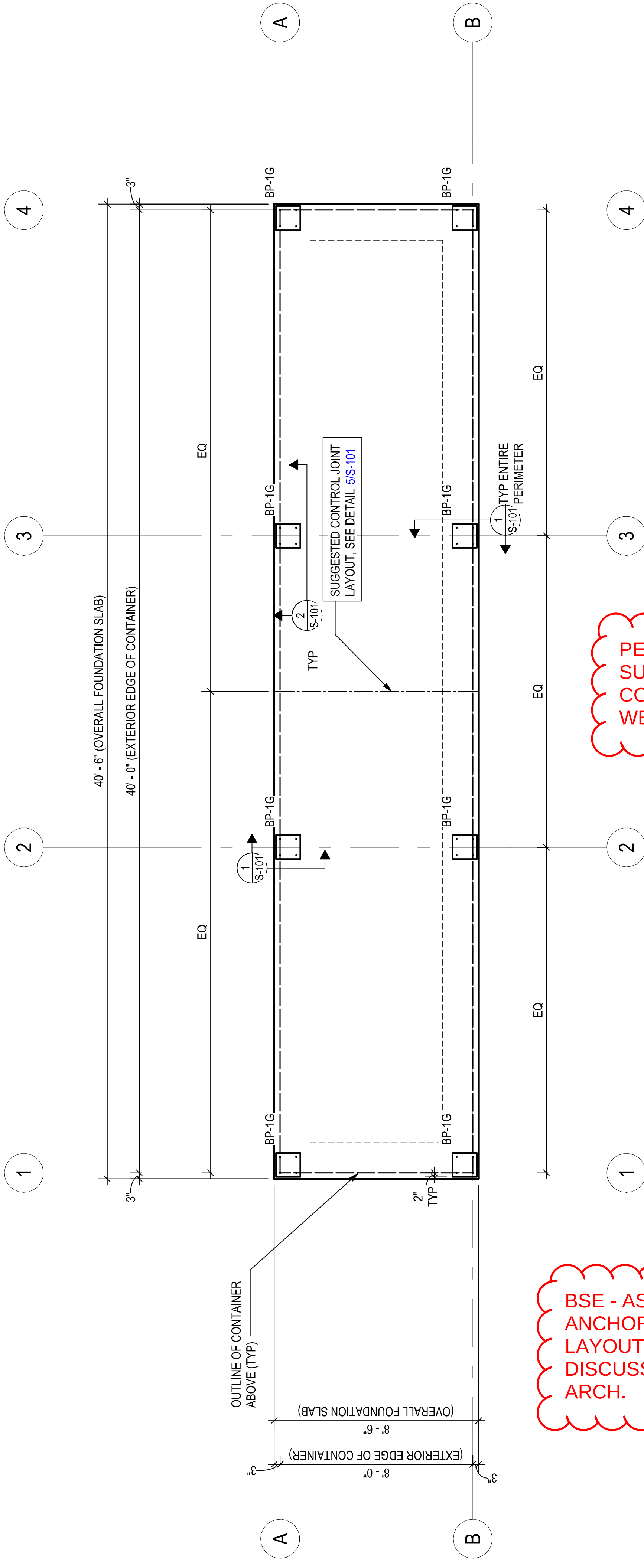
SHEET TITLE
SITE PLAN

SHEET NUMBER
A0.10

DESIGN LOADS



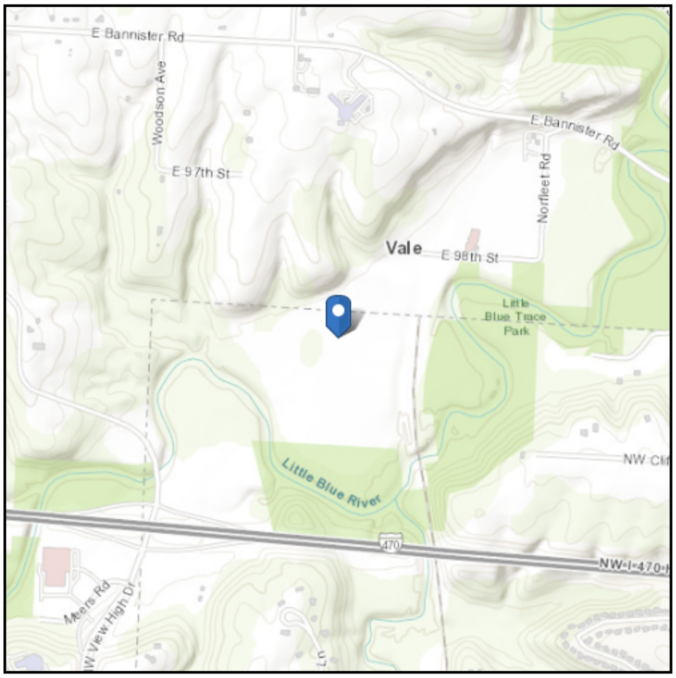
PER CONTAINER SUPPLIER, MINIMUM CONTAINER SELF WEIGHT IS 8300 LBS.

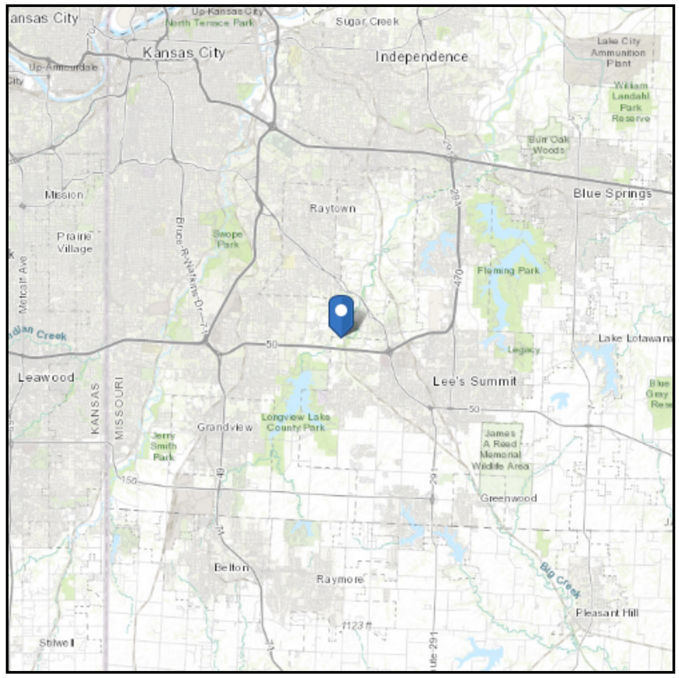
BSE - ASSUMED ANCHORAGE LAYOUT PER DISCUSSION WITH ARCH.

ASCE Hazards Report

Address:
No Address at This Location

Standard:	ASCE/SEI 7-16
Risk Category:	II
Soil Class:	D - Stiff Soil
Latitude:	38.941479
Longitude:	-94.441245
Elevation:	808.513497020725 ft (NAVD 88)





Wind

Results:

Wind Speed	109 Vmph
10-year MRI	76 Vmph
25-year MRI	83 Vmph
50-year MRI	88 Vmph
100-year MRI	94 Vmph

Data Source:	ASCE/SEI 7-16, Fig. 26.5-1B and Figs. CC.2-1–CC.2-4, and Section 26.5.2
Date Accessed:	Fri Nov 08 2024

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

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

ASCE
AMERICAN SOCIETY OF CIVIL ENGINEERS

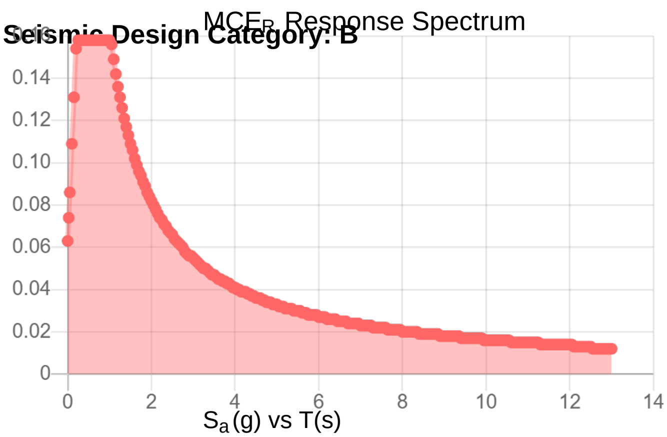
Seismic

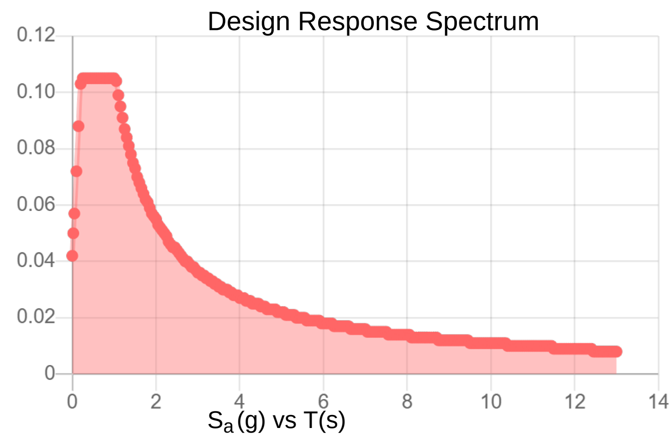
Site Soil Class: D - Stiff Soil

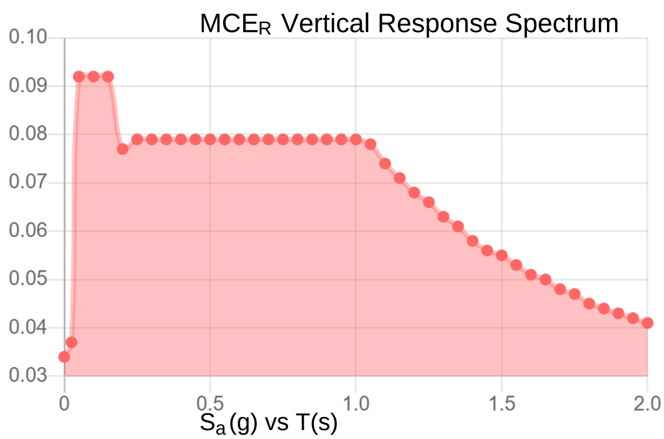
Results:

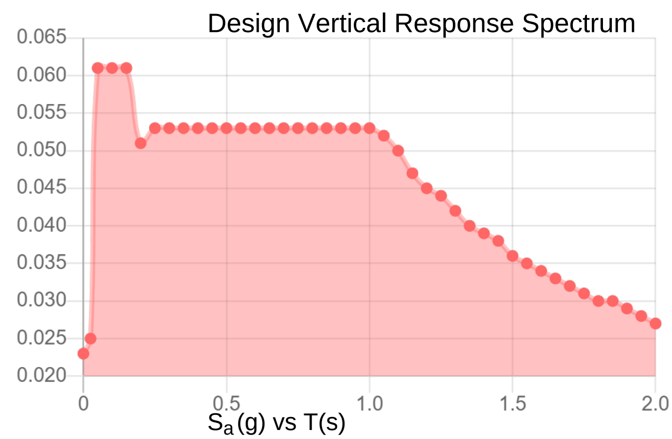
S_S :	0.099	S_{D1} :	0.109
S_1 :	0.068	T_L :	12
F_a :	1.6	PGA :	0.047
F_v :	2.4	PGA_M :	0.075
S_{MS} :	0.158	F_{PGA} :	1.6
S_{M1} :	0.164	I_e :	1
S_{DS} :	0.105	C_v :	0.7

Seismic Design Category: B









Data Accessed: **Fri Nov 08 2024**

Date Source:

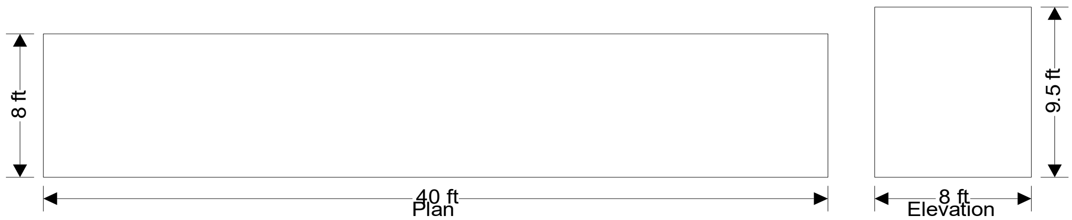
USGS Seismic Design Maps based on ASCE/SEI 7-16 and ASCE/SEI 7-16 Table 1.5-2. Additional data for site-specific ground motion procedures in accordance with ASCE/SEI 7-16 Ch. 21 are available from USGS.

WIND LOADING

In accordance with ASCE7-16

Using the directional design method

Tedds calculation version 2.1.14



Building data

Type of roof — Flat

Length of building — b = **40.00** ft

Width of building — d = **8.00** ft

Height to eaves — H = **9.50** ft

Mean height — h = **9.50** ft

General wind load requirements

Basic wind speed — V = **110.0** mph

Risk category — II

Velocity pressure exponent coef (Table 26.6-1) — K_d = **0.85**

Ground elevation above sea level — z_{gl} = **809** ft

Ground elevation factor — $K_e = \exp(-0.0000362 \times z_{gl}/1\text{ft}) = \mathbf{0.97}$

Exposure category (cl 26.7.3) — B

Enclosure classification (cl.26.12) — Enclosed buildings

Internal pressure coef +ve (Table 26.13-1) — GC_{pi_p} = **0.18**

Internal pressure coef –ve (Table 26.13-1) — GC_{pi_n} = **-0.18**

Gust effect factor — G_f = **0.85**

Minimum design wind loading (cl.27.1.5) — p_{min_r} = **8** lb/ft^2

Topography

Topography factor not significant — K_{zt} = 1.0

Velocity pressure equation — $q = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \times 1\text{psf/mph}^2$

Velocity pressures table

z (ft)	K_z (Table 26.10-1)	q_z (psf)
9.50	0.57	15.93

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.) — q_i = **15.93** psf

Pressures and forces

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

Net force — $F_w = p \times A_{ref}$

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	9.50	-1.22	15.93	-19.41	190.00	-3.69

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft²)	Net force F_w (kips)
B (-ve)	9.50	-0.70	15.93	-12.35	130.00	-1.60

Total vertical net force $F_{w,v}$ = **-5.29** kips

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

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft²)	Net force F_w (kips)
A	9.50	0.80	15.93	7.97	380.00	3.03
B	9.50	-0.50	15.93	-9.64	380.00	-3.66
C	9.50	-0.70	15.93	-12.35	76.00	-0.94
D	9.50	-0.70	15.93	-12.35	76.00	-0.94

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_0} = b \times H =$ **380.00** ft²

Projected vertical area of roof $A_{vert\_r\_0} =$ **0.00** ft²

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0} =$ **6.08** kips

Leeward net force $F_l = F_{w,wB} =$ **-3.7** kips

Windward net force $F_w = F_{w,wA} =$ **3.0** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **6.7** kips

Roof load case 2 - Wind 0, GC_{pi} -0.18, $-1c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft²)	Net force F_w (kips)
A (+ve)	9.50	-0.18	15.93	0.43	190.00	0.08
B (+ve)	9.50	-0.18	15.93	0.43	130.00	0.06

Total vertical net force $F_{w,v}$ = **0.14** kips

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

DESIGN COMBO

Walls load case 2 - Wind 0, GC_{pi} -0.18, $-1c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft²)	Net force F_w (kips)
A	9.50	0.80	15.93	13.70	380.00	5.21
B	9.50	-0.50	15.93	-3.90	380.00	-1.48
C	9.50	-0.70	15.93	-6.61	76.00	-0.50
D	9.50	-0.70	15.93	-6.61	76.00	-0.50

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_0} = b \times H =$ **380.00** ft²

Projected vertical area of roof $A_{vert\_r\_0} =$ **0.00** ft²

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0} =$ **6.08** kips

Leeward net force $F_l = F_{w,wB} =$ **-1.5** kips

Windward net force $F_w = F_{w,wA} =$ **5.2** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **6.7** kips

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	9.50	-0.90	15.93	-15.05	38.00	-0.57
B (-ve)	9.50	-0.90	15.93	-15.05	38.00	-0.57
C (-ve)	9.50	-0.50	15.93	-9.64	76.00	-0.73
D (-ve)	9.50	-0.30	15.93	-6.93	168.00	-1.16

Total vertical net force $F_{w,v}$ = **-3.04** kips

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

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	9.50	0.80	15.93	7.97	76.00	0.61
B	9.50	-0.20	15.93	-5.58	76.00	-0.42
C	9.50	-0.70	15.93	-12.35	380.00	-4.69
D	9.50	-0.70	15.93	-12.35	380.00	-4.69

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_90} = d \times H =$ **76.00** ft^2

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

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} =$ **1.22** kips

Leeward net force $F_l = F_{w,wB} =$ **-0.4** kips

Windward net force $F_w = F_{w,wA} =$ **0.6** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **1.2** kips

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (+ve)	9.50	-0.18	15.93	0.43	38.00	0.02
B (+ve)	9.50	-0.18	15.93	0.43	38.00	0.02
C (+ve)	9.50	-0.18	15.93	0.43	76.00	0.03
D (+ve)	9.50	-0.18	15.93	0.43	168.00	0.07

Total vertical net force $F_{w,v}$ = **0.14** kips

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

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	9.50	0.80	15.93	13.70	76.00	1.04
B	9.50	-0.20	15.93	0.16	76.00	0.01
C	9.50	-0.70	15.93	-6.61	380.00	-2.51
D	9.50	-0.70	15.93	-6.61	380.00	-2.51

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_90} = d \times H =$ **76.00** ft^2

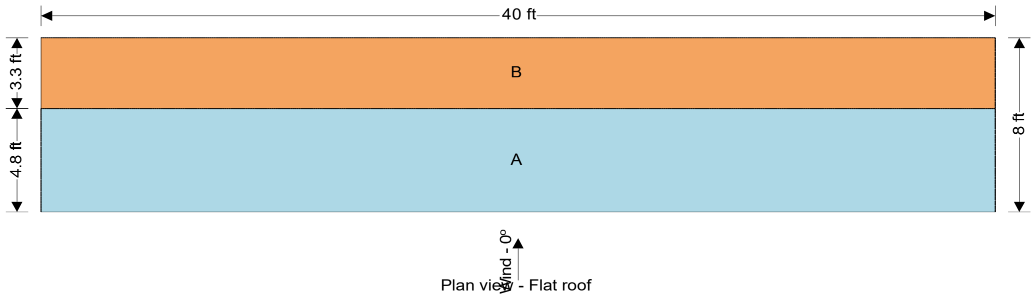
Projected vertical area of roof $A_{vert\_r\_90}$ = **0.00** ft^2

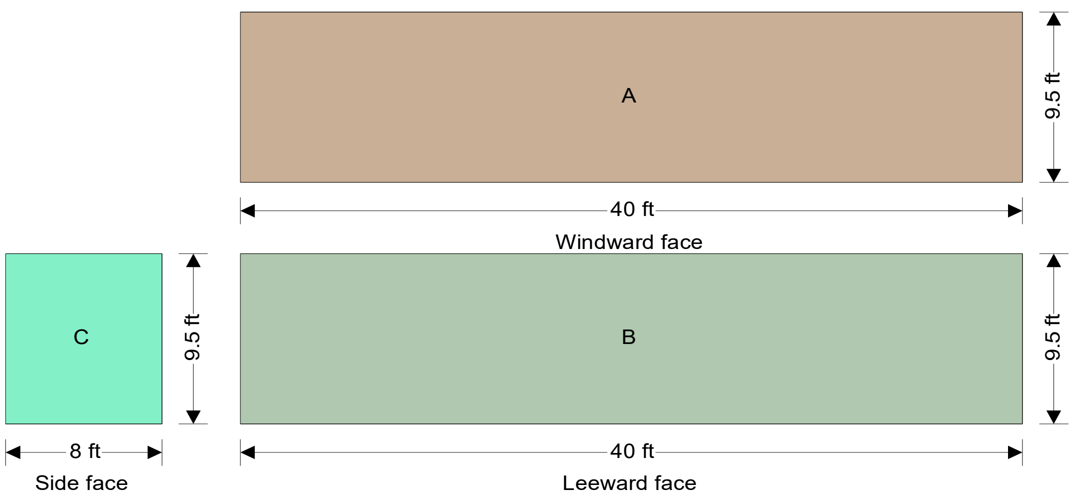
Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90}$ = **1.22** kips

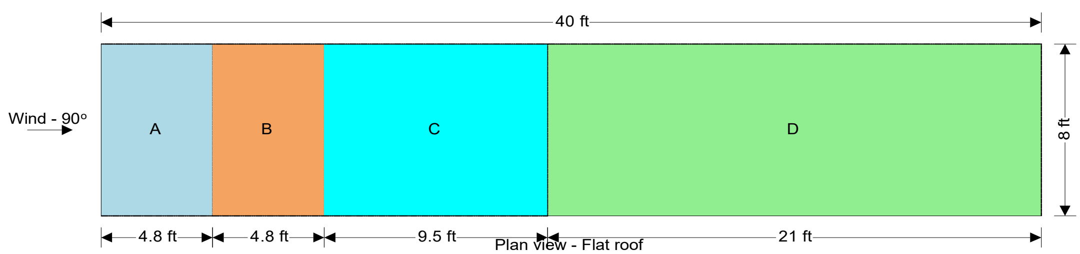
Leeward net force $F_l = F_{w,wB}$ = **0.0** kips

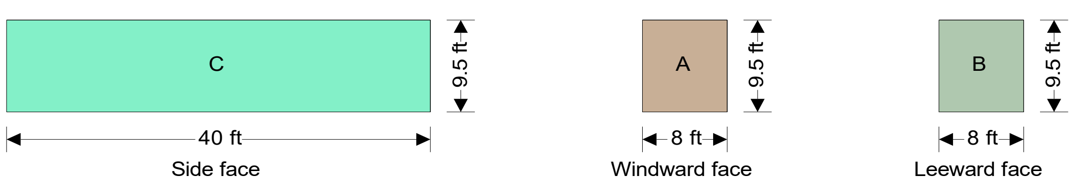
Windward net force $F_w = F_{w,wA}$ = **1.0** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min})$ = **1.2** kips









Seismic Loads on Nonstructural Components: ASCE 7-10, Chapter 13

Horizontal Load:

S_{DS} =	0.1050 g	(Short period spectral response acceleration factor)
a_p =	1.0	(ASCE 7-10, Table 13.5-1 or 13.6-1)
R_p =	2.5	(ASCE 7-10, Table 13.5-1 or 13.6-1)
W_p =	8,300 lbs	(Component operating weight)
I_p =	1.5	(Component importance factor per ASCE 7-10, Section 13.1.3)
z =	9.5 ft.	(Height to point of attachment with respect to base, ASCE 7-10, Section 13.3.1)
h =	9.5 ft.	(Average roof height)
F_p =	627 lbs	(ASCE 7-10, EQ.13.3-1)
$F_{p,max}$ =	2,092 lbs	(ASCE 7-10, EQ.13.3-2)
$F_{p,min}$ =	392 lbs	(ASCE 7-10, EQ.13.3-3)
F_{ph} =	**627 lbs**	(Force is strength level and can be applied in any horizontal direction)

Vertical Load:

		(ASCE 7-10, Section 13.3.1)
F_{pv} =	**174 lbs**	(Force is strength level and can be upward or downward)

BSE - (13.7+3.9 PSF)*(40 FT)*(9.5 FT) = 6688 lbs. > 627 lbs.

THEREFORE, WIND GOVERNS LATERAL DESIGN

OVERTURNING CHECK

Overturning Check

LOAD

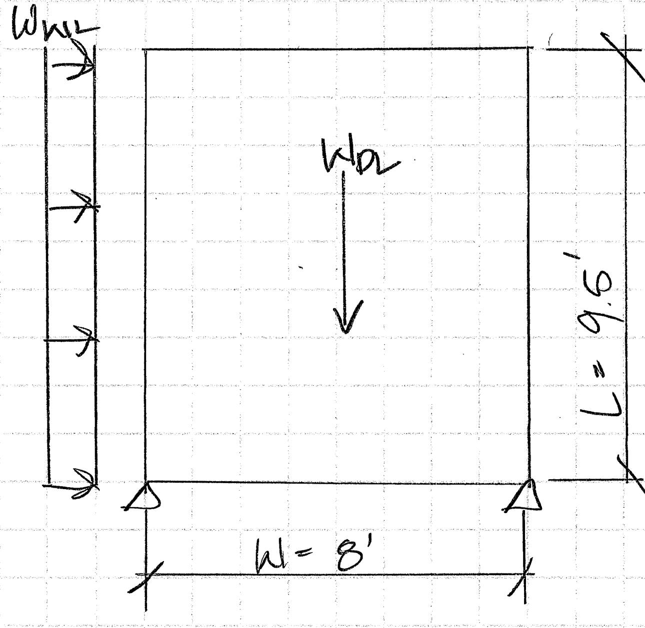
DL = 8300#

TRIB Between anchors
= 13'-4" (13.33')

$W_{DL} = \left(\frac{8300\#}{40}\right)(13.33)$

$= 2765.98\#$

$W_{WL} = (13.7 + 3.9)(9.5')(13.33')$
$= 2229.28\#$



SINCE DL IS SELF WT, DL + 0.6WL CONTROLS

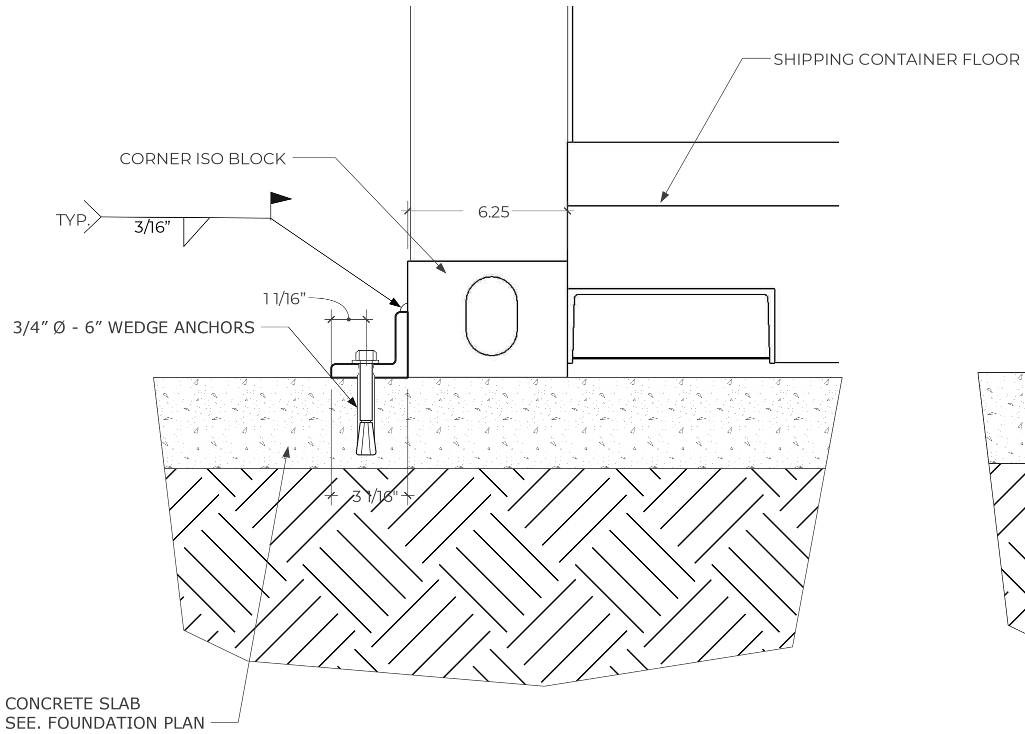
$M_{RES} = W_{DL}(8'/2) = 11063.92\#'$

$M_{OVR} = (0.6)(W_{WL})(9.5/2) = 6353.45\#'$

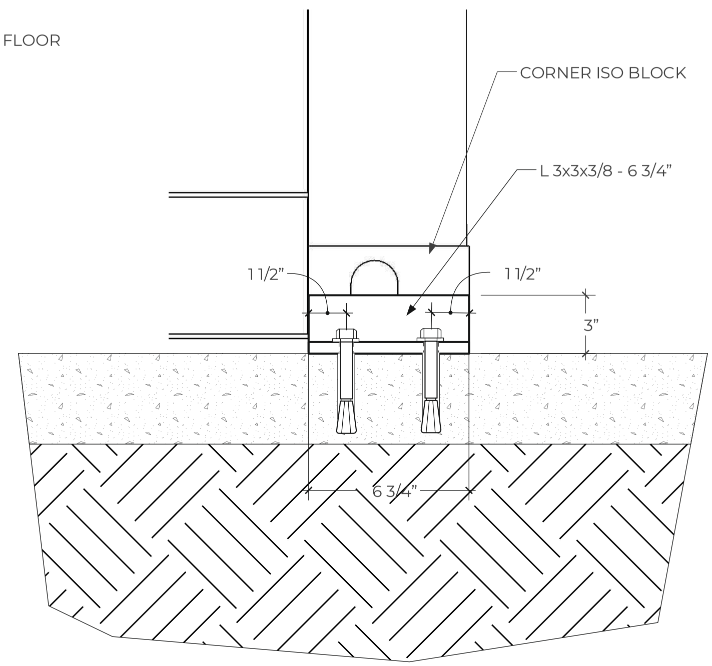
$F.S. = \frac{M_{RES}}{M_{OVR}} = 1.74 > 1.5$ ∴ Overturning not applicable

Anchors provided are for positive connection to the paving.

ANCHORAGE CHECK



3 **SIDE VIEW**
SIDE ANCHOR TO CONCRETE SLAB- OPTION



3A **FRONT VIEW**
SIDE ANCHOR TO CONCRETE SLAB- OPTION

ASSUMED CONNECTION TO EACH ANCHOR POINT PER THE LAYOUT PROVIDED TO BSE

Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - Nov 6, 2024	Date:	11/6/2024
Fastening point:			

Specifier's comments:

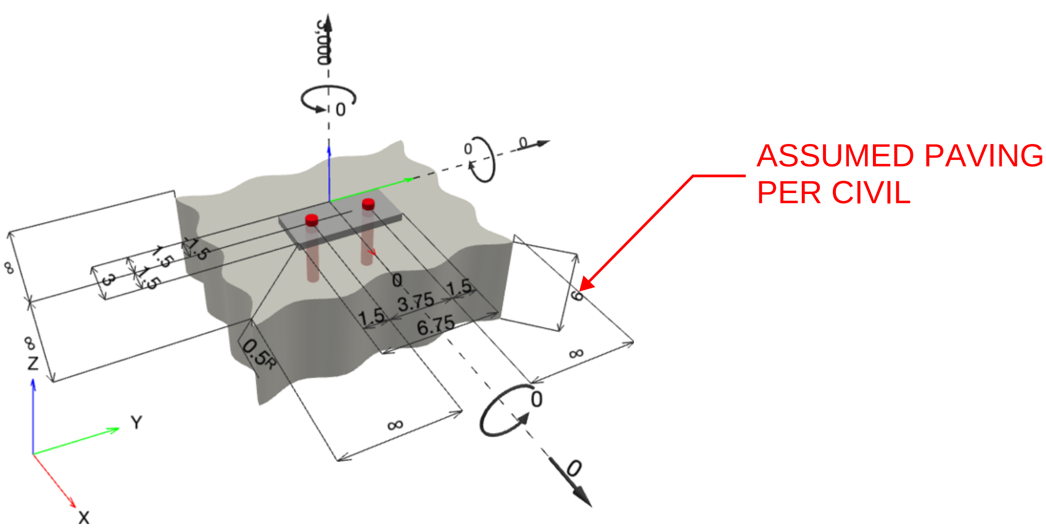
1 Input data

Anchor type and diameter:	**Kwik Bolt TZ2 - CS 3/4 (3 3/4) hnom2**
Item number:	2210312 KB-TZ2 3/4x6 1/4
Specification text:	Hilti KB-TZ2 stud anchor with 4.5 in embedment, 3/4 (3 3/4) hnom2, Carbon steel, installation per ESR-4266
Effective embedment depth:	$h_{ef,act}$ = 3.750 in., h_{nom} = 4.500 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-4266
Issued I Valid:	12/1/2023 | 12/1/2025
Proof:	Design Method ACI 318-19 / Mech
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate^R :	l_x x l_y x t = 3.000 in. x 6.750 in. x 0.500 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked concrete, 2500, f_c' = 2,500 psi; h = 6.000 in.
Installation:	**Hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: not present, shear: not present; no supplemental splitting reinforcement present
	edge reinforcement: none or < No. 4 bar

Conservative assumption

R - The anchor calculation is based on a rigid anchor plate assumption.

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



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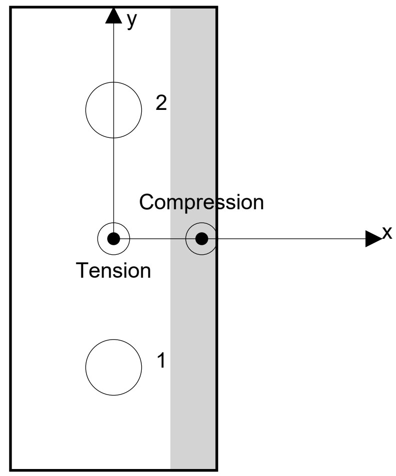
Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - Nov 6, 2024	Date:	11/6/2024
Fastening point:			

1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	$N = 3{,}000$; $V_x = 0$; $V_y = 0$; $M_x = 0$; $M_y = 0$; $M_z = 0$;	no	99

2 Load case/Resulting anchor forces



Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	3,256	0	0	0
2	3,256	0	0	0

Max. concrete compressive strain:	0.36 [‰]
Max. concrete compressive stress:	1,587 [psi]
Resulting tension force in (x/y)=(0.000/0.000):	6,512 [lb]
Resulting compression force in (x/y)=(1.281/0.000):	3,512 [lb]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	3,256	19,009	18	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	6,512	6,608	99	OK

* highest loaded anchor **anchor group (anchors in tension)

3.1 Steel Strength

N_{sa} = ESR value — refer to ICC-ES ESR-4266

$\phi\ N_{sa} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.24	105,904

Calculations

N_{sa} [lb]
25,345

Results

N_{sa} [lb]	$\phi_{\ steel}$	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
25,345	0.750	19,009	3,256

3.2 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{\ ec,N}\ \psi_{\ ed,N}\ \psi_{\ c,N}\ \psi_{\ cp,N}\ N_b$ — ACI 318-19 Eq. (17.6.2.1b)

$\phi\ N_{cbg} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{\ ec,N} = \left(\frac{1}{1 + \frac{2\ e_N^{'}}{3\ h_{ef}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{\ ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{\ cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c\ \lambda_{\ a}\ \sqrt{f_c^{'}}\ h_{ef}^{1.5}$ — ACI 318-19 Eq. (17.6.2.2.1)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{\ c,N}$
3.750	0.000	0.000	∞	1.000

c_{ac} [in.]	k_c	$\lambda_{\ a}$	$f_c^{'}$ [psi]
10.000	21	1.000	2,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{\ ec1,N}$	$\psi_{\ ec2,N}$	$\psi_{\ ed,N}$	$\psi_{\ cp,N}$	N_b [lb]
168.75	126.56	1.000	1.000	1.000	1.000	7,625

Results

N_{cbg} [lb]	$\phi_{\ concrete}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
10,167	0.650	6,608	6,512

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength*	N/A	N/A	N/A	N/A
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)
When the input edge distance is set to "infinity", edge breakout verification is not performed in that direction

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- Hilti post-installed anchors shall be installed in accordance with the Hilti Manufacturer's Printed Installation Instructions (MPII). Reference ACI 318-19, Section 26.7.

Fastening meets the design criteria!

6 Installation data

Anchor type and diameter: Kwik Bolt TZ2 - CS 3/4 (3 3/4) hnom2

Item number: 2210312 KB-TZ2 3/4x6 1/4

Maximum installation torque: 1,324 in.lb

Hole diameter in the base material: 0.750 in.

Hole depth in the base material: 4.750 in.

Minimum thickness of the base material: 6.000 in.

Profile: no profile

Hole diameter in the fixture: d_f = 0.812 in.

Plate thickness (input): 0.500 in.

Recommended plate thickness: not calculated

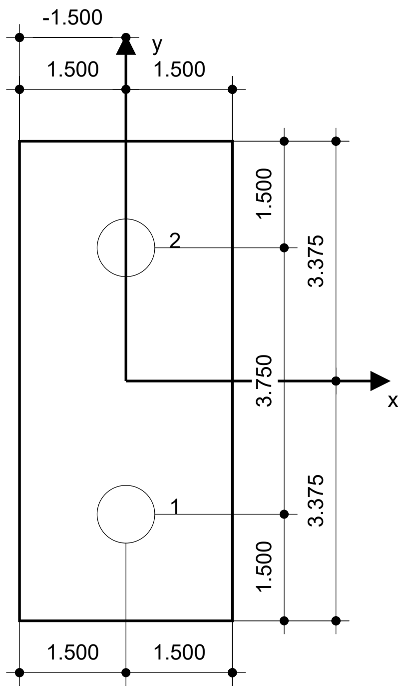
Drilling method: Hammer drilled

Cleaning: Manual cleaning of the drilled hole according to instructions for use is required.

Hilti KB-TZ2 stud anchor with 4.5 in embedment, 3/4 (3 3/4) hnom2, Carbon steel, installation per ESR-4266

6.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Manual blow-out pump	• Torque controlled cordless impact tool • Torque wrench • Hammer



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	-1.875	-	-	-	-
2	0.000	1.875	-	-	-	-