EEIS CONSULTING ENGINEERS, INC.
P.O. BOX 92169 • ANCHORAGE, ALASKA 99509 • (907)258-3231

Structural Calculations

for

Project: City Of Fairbanks Public Works Department
Pump Enclosures
EEIS Project #: 223004.0000.001.0100

for
City Of Fairbanks Public Works
800 Cushman Street
Fairbanks, AK 99701

June 23, 2023

The attached calculations have been produced by me or under my direct supervision. EEIS makes no warranty as to the accuracy or completeness of the construction documents other than the work in which EEIS was directly involved.

STATE OF ALASKA
49TH
Richard C. Button
SE - 13698
6/23/2023
REGISTERED PROFESSIONAL ENGINEER

EEIS CONSULTING ENGINEERS, INC.

STRUCTURAL CALCULATIONS

For

City Of Fairbanks Public Works Department
Pump Enclosure

223004.0000.001.0100

INDEX | Page

REFERENCES

HILTI KWIK BOLT TZ CARBON AND STAINLESS STEEL ANCHORS IN CRACKED AND UNCRACKED CONCRETE
ICC-Evaluation Report ESR-4266 (Renewal May 2022)

l-1

Fairbanks Safety Alerts

City of
Fairbanks
ALASKA

Search

BUILDING

Building Design Information

Minimum Design Parameters for Fairbanks, Alaska

- Minimum roof snow load (Pf): 50 PSF
- Minimum ground snow load (Pg): 60 PSF
- Snow exposure coefficient: 1.0
  - (Note: 15% LDF is not allowed for snow loads on wood framed or wood trussed roofs)
- Basic wind speed: 90 MPH

Additional Building Details

Standard Foundation Details

Egress Window Well & Stair Details

Electrical Details

Foam Board Survey

Vapor Barrier Details

L-2

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

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

ATC Hazards by Location

Search Information

Coordinates:	64.82838538684695, -147.76928369941407
Elevation:	440 ft
Timestamp:	2023-04-07T18:39:53.293Z
Hazard Type:	Wind

ASCE 7-16

MRI 10-Year	75 mph
MRI 25-Year	85 mph
MRI 50-Year	90 mph
MRI 100-Year	95 mph
Risk Category I	105 mph
Risk Category II	110 mph
Risk Category III	115 mph
Risk Category IV	120 mph

ASCE 7-10

MRI 10-Year	78 mph
MRI 25-Year	83 mph
MRI 50-Year	90 mph
MRI 100-Year	95 mph
Risk Category I	105 mph
Risk Category II	110 mph
Risk Category III-IV	115 mph

ASCE 7-05

ASCE 7-05 Wind Speed	90 mph

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

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

Disclaimer

Hazard loads are interpolated from data provided in ASCE 7 and rounded up to the nearest whole integer. Per ASCE 7, islands and coastal areas outside the last contour should use the last wind speed contour of the coastal area – in some cases, this website will extrapolate past the last wind speed contour and therefore, provide a wind speed that is slightly higher. NOTE: For queries near wind-borne debris region boundaries, the resulting determination is sensitive to rounding which may affect whether or not it is considered to be within a wind-borne debris region.

Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

While the information presented on this website is believed to be correct, ATC and its sponsors and contributors assume no responsibility or liability for its accuracy. The material presented in the report should not be used or relied upon for any specific application without competent examination and verification of its accuracy, suitability and applicability by engineers or other licensed professionals. ATC does not intend that the use of this information replace the sound judgment of such competent professionals, having experience and knowledge in the field of practice, nor to substitute for the standard of care required of such professionals in interpreting and applying the results of the report provided by this website. Users of the information from this website assume all liability arising from such use. Use of the output of this website does not imply approval by the governing building code bodies responsible for building code approval and interpretation for the building site described by latitude/longitude location in the report.

1-3

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

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

ATC Hazards by Location

Search Information

Coordinates:	64.82838538684695, -147.76928369941407
Elevation:	440 ft
Timestamp:	2023-04-07T18:36:49.157Z
Hazard Type:	Seismic
Reference Document:	ASCE7-16
Risk Category:	II
Site Class:	D-default

Basic Parameters

Name	Value	Description
S_S	0.996	MCE_R ground motion (period=0.2s)
S_1	0.38	MCE_R ground motion (period=1.0s)
S_{MS}	1.196	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	0.797	Numeric seismic design value at 0.2s SA
S_{D1}	* null	Numeric seismic design value at 1.0s SA

* See Section 11.4.8

▾Additional Information

Name	Value	Description
SDC	* null	Seismic design category
F_a	1.2	Site amplification factor at 0.2s
F_v	* null	Site amplification factor at 1.0s
CR_S	0.944	Coefficient of risk (0.2s)
CR_1	0.972	Coefficient of risk (1.0s)
PGA	0.406	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.487	Site modified peak ground acceleration
T_L	6	Long-period transition period (s)
SsRT	0.996	Probabilistic risk-targeted ground motion (0.2s)
SsUH	1.055	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.38	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.391	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.6	Factored deterministic acceleration value (1.0s)
PGAd	0.5	Factored deterministic acceleration value (PGA)

* See Section 11.4.8

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

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

Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.

EEIS CONSULTING ENGINEERS, Inc.

PROJECT	City Of Fairbanks PWD- Pump Enclosure
SHEET NO.	Lat -
CALCULATED BY	Benito
DATE	4/7/2023
PROJECT NO.	223004

CALCULATE DEAD LOADS FOR SEISMIC CALCS

TYPICAL ROOF

4" Standing Seam Insulated Roof Panel	2.65	psf
HSS6X6X3/8 at 4'-0" o.c.	2.29	psf
Misc./Mech/Elec	2.50	psf
	7.44	psf

TYPICAL EXT. WALL

4" Insulated Wall Panel	2.62	psf
HSS6X6X3/8 at 4'-0" o.c.	2.29	psf
Misc. Elect/Mech	2.50	psf
	7.41	psf

L-5

EEIS CONSULTING ENGINEERS, Inc.

PROJECT	City Of Fairbanks PWD- Pump Enclosure
SHEET NO	Lat -
CALCULATED BY	Benito
DATE	4/7/2023
PROJECT NO	223004

SEISMIC BASE SHEAR CALCULATION -- EQUIVALENT LATERAL FORCE Cs

Building Risk Category		**II**	(2018 IBC, ASCE/SEI 7-16 Table 1.5-1)
Ie		**1.0**	(ASCE/SEI Table 1.5-2)
Site Class (Assumed)		**D**	(ASCE/SEI Section 11.4.2)
		Default	(ASCE/SEI Section 11.4-2)
Ss		**0.996g**	(ASCE/SEI Figure 22-1 THRU 22-8)
Fa		**1.2**	(ASCE/SEI Table 11.4-1)
S_{DS}	= 2/3*Ss*Fa =	**0.80**	(ASCE/SEI Sec 11.4.5(EQ 11.4-3))
S_1		**0.380g**	(ASCE/SEI Figure 22-1 THRU 22-8)
Fv		**1.64**	(ASCE/SEI Table 11.4-2)
S_{D1}	= 2/3*S_1*Fv =	**0.415**	(ASCE/SEI Sec 11.4.4(EQ 11.4-4))
Roof Snow Load		**50.0 psf**	(ASCE/SEI Sec 7.3 - Flat Roof Snow Load)
Seismic Design Category		**D**	(ASCE/SEI Sec 11.6)

Response modification factor, Overstrength Factor, Deflection Amplification Factor

R =	**6.5**	Light frame	(ASCE/SEI Table 12.2-1, A16)
Ω =	**3.0**		(ASCE/SEI Table 12.2-1, A16)
Cd =	**4.0**		(ASCE/SEI Table 12.2-1, A16)

Redundancy Factor

ρ = 1.0 (1.0 or 1.3) (ASCE/SEI Sec 12.3.4)

Calculated Seismic response Coefficient Cs: (LRFD)

$Cs = S_{DS}/(R/I_e)$ (ASCE/SEI Eq 12.8-2)

Cs = 0.123

Maximum Seismic response coefficient Cs shall not exceed:

$Cs = S_{D1}/(T(R/I_e))$ For: $T \le T_L$ (ASCE Eq 12.8-3)

Cs = 0.264

$Cs = S_{D1}T_L/(T^2(R/I_e))$ For: $T \ge T_L$ (ASCE Eq 12.8-4)

Cs = 17.443

Cu =	1.4	(ASCE Table 12.8-1) Coeff. Upper limit on calc. Period
T =	Ta Cu =	0.339 shall not exceed
Ta =	$C_t h_n^x$	(ASCE Eq 12.8-7) Approximate fundamental period
h_n =	17.75 ft	height above base
x =	0.75	(ASCE/SEI Table 12.8-2)
Ct =	0.028	(ASCE/SEI Table 12.8-2)
T = Ta =	0.24 s	approximate building period
T_L =	16	(ASCE Figure 22-14 Thru 22-17) Long-period transition

Seismic response coefficient Cs shall not be less than:

Cs = 0.01 (ASCE Eq 12.8-5)

Cs = $0.044 \cdot S_{DS} \cdot I_E$

0.0351

$Cs = 0.5S_1/(R/I_E)$ (ASCE Eq 12.8-6)

0.0292

Cs min = $\max(0.044S_{DS}I_e, 0.01)$ = 0.0350592

Equivalent Lateral Force (ELF)

V= CsW (ASCE/SEI Eq 12.8-1)

V_{LRFD} = Cs * (ρ)*W — Cs = Seismic Response Coefficient with Section 12.8.1.1

= 0.123 *W — W = Effective Seismic Weight of Structure

V_{ASD} = 0.086 *W — $V_{ASD} = 0.7 \cdot V_{LRFD}$

EEIS CONSULTING ENGINEERS, Inc.

PROJECT	City Of Fairbanks PWD- Pump Enclosure
SHEET NO.	Lat -
CALCULATED BY	Benito
DATE	4/7/2023
PROJECT NO	223004

ASCE/SEI 7-16 Snow Load
(Chapter 7)

Locations		Fairbanks		(Table 7.2-1 Ground Snow Loads for Alaska Locations)
Ground Snow Load	p_g =	**60**	psf	
Risk Category of Buildings		**II**		(Table 1.5-1)
Snow Importance Factor	I_s =	**1.00**		(Table 1.5-2)
Surface Roughness		**B**		(Section 26.7.2)
Exposure		**B**		(Section 26.7.3)
Roof Exposure		**Partially Exposed**		
Exposure factor	C_e =	**1.00**		(Table 7.3-1) *check foot notes, parapets make roof partially exposed
Thermal factor	C_t =	**1.00**		(Table 7.3-2)
Surface Type		**2**	**All other surfaces**	
Roof slope angle	1 12	**4.8**	degree	
		Y	Enter "Y" if roof is low slope as defined in the section.	
	Warm	Cold Roof C_t	Cold Roof C_t	
Roof slope factor	Roof C_t<1	=1.1	=1.2	(Section 7.4.1 thru 7.4.4)
	C_s = **1.000**	--	--	

Flat roof snow loads (roofs with slope equal of less than 5 degree)

$p_f = 0.7*C_e*C_t*Is*p_g$ (Eq 7.3-1)

p_f = 42.00 psf

$p_{f\,min}$ for low slope roofs as defined in section 7.3.4

if p_g=<20psf	=> $p_f = I*p_g$	p_f min =	N/A psf
if p_g>20psf	=> $p_f = 20*I$	p_f min =	20 psf

Pf min for flat roofs as defined by Local Amendments and section of the ASCE 7-16, Fairbanks Alaska

$p_{f\,min}$ = 50.0 psf **Fairbanks**

Flat roof snow load taken for further calculations

p_f =	**50.00**	**psf**

Sloped roof snow load

$p_s = C_s*p_f$

p_s =	**N/A**	**psf**

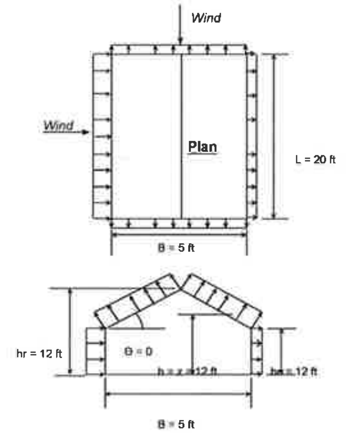
I-7

EEIS CONSULTING ENGINEERS, Inc.

PROJECT	City Of Fairbanks PWD- Pump Enclosure
SHEET NO	Lat -
CALCULATED BY	Benito
DATE	4/7/2023
PROJECT NO	223004

MWFRS WIND LOADS - ALL HEIGHTS METHOD - WALLS AND ROOFS

ASCE/SEI 7-16 CHAPTER 27 PART 1



Input Data:

Design Method	ASD	
Bldg. Classification	II	(Table 1.5-1 Risk Category)
Exposure Category	C	(Sect. 26.7.3)
Is Bldg. in Anchorage?	N	
Outside Of Anchorage?	Y	
Ultimate Wind Speed (3 Sec. Gust)	110 mph	- Use Wind Speed Map on Fig. 26.5-1B - Or Use Specifed Wind Speed per EOR
Eave Height, he	12.00 ft	
Building Length L	20.00 ft	(Parallel to Building Ridge)
Building Width B	5.00 ft	(Normal to Building Ridge)
Roof Type	Flat	
Roof Pitch	0	12
G	0.85	Gust Effect Factor, Sect. 26.11.1
Direct. Factor, Kd	0.85	(Table 26.6-1)
Ground Elevation Factor, Ke	1.00	(Sect. 26.9)
Enclosed? (Y/N)	Y	(Sect. 26.2)
Hurricane Region?	N	

Resulting Parameters and Coefficients:

Roof Angle, θ ,degrees = 0.00°

Ridge Height, hr = 12.0 ft

Mean Roof Height, he = 12.0 ft

Internal Pressure

Positive & Negative Internal Pressure Coefficients, GCpi (Table 26.13-1):

Coefficient GC_{pi} ± 0.18 internal pressure coefficient

Topographic Factor, K_{zt} (Sect. 26.8.1 & Figure 26.8-1)

Building Upwind/Downwind?	Upwind	
Hill Height H =	1.00 ft	
Half Hill Length L_h =	1000.00 ft	
Hill Shape =	2-D Ridges	Parameters for Speed-Up Over Hills and Escarpments
H/L_h =	0.0010	
From top of crest: x =	1.00 ft	
x/L_h =	0.0010	
z/L_h =	0.01	
$K_1/(H/Lh)$ =	1.45	from Figure 26.8-1
γ =	3.00	
μ =	1.50	
K_1 =	0.0015	
K_2 =	1.00	
K_3 =		Calculated below for z per floor
K_{zt} =		Calculated below for z per floor

Velocity Pressure Exposure Coefficients, Kh and Kz (Table 26.10-1)

For $z \leq 15$ ft. then: $Kz = 2.01*(15/zg)^{2/\alpha}$

α =	9.5	Terrain Exposure Constants (Table 26.11-1)
Z_g =	900	
K_h =	0.85	($K_h = K_z$ evaluated at z = h Table 26.10-1)

EEIS CONSULTING ENGINEERS, Inc.

PROJECT	City Of Fairbanks PWD- Pump Enclosure
SHEET NO	Lat -
CALCULATED BY	Benito
DATE	4/7/2023
PROJECT NO	223004

MWFRS WIND LOADS - ALL HEIGHTS METHOD - WALLS AND ROOFS

ASCE/SEI 7-16 CHAPTER 27 PART 1

Basic Velocity Pressure

qh = 26.33 psf (Eq. 26.10-1) - evaluated at z = h (Without Kzt and Kz)

Level	Floor Height	z (ft)	K_3	K_{zt}	K_z	q_z (psf)
Mean Roof Height qz = qh	12.0	12.0	0.96	1.00	0.85	22.4
Fourth Story, mid story	0.00	0.0	0.00	0.00	0.00	0.0
Third story, mid story	0.00	0.0	0.00	0.00	0.00	0.0
Second Story, mid story	0.00	0.0	0.00	0.00	0.00	0.0
First Story, mid story	12.0	12.0	0.96	1.00	0.85	22.4

Design Wind Pressures (Sect. 27.3.1):

$p = qGC_p - qi(\pm GC_{pi})]$ psf (Eq. 27.3-1)

Walls:

Wind Parallel to "L" Dimension, Parallel to Ridge

L/B = 4.00

$C_{p,WW}$ = 0.80 external pressure coeff. (Fig 27.3-1) use with qz

$C_{p,LW}$ = -0.20 external pressure coeff. (Fig 27.3-1) use with qh

Windward (WW) $P_{ww} = q_z*G\ C_p\text{-}q_h*(GC_{pi})$ Eq 27.3-1- for building of all heights

Leeward (LW) $P_{Lw} = q_h*G\ C_p\text{-}q_h*(GC_{pi})$ Eq 27.3-1- for building of all heights

Level	Internal Pressure Case I (+Gcpi)			Internal Suction Pressure Case II (-Gcpi)		
	P_{WW}	P_{LW}	Σ P	P_{WW}	P_{LW}	Σ P
	psf	psf	psf	psf	psf	psf
Mean Roof Height, qh = qz	11.21	-7.84	19.1	19.28	0.22	19.5
Fourth Story, mid story	0.00	0.00	0.0	0.00	0.00	0.0
Third story, mid story	0.00	0.00	0.0	0.00	0.00	0.0
Second Story, mid story	0.00	0.00	0.0	0.00	0.00	0.0
First Story, mid story	11.21	-7.84	19.1	19.28	0.22	19.5

Walls:

Wind Parallel to "B" Dimension, Perpendicular to Ridge

B/L = 0.25

$C_{p,WW}$ = 0.80 external pressure coeff. (Fig 27.4-1) use with qz

$C_{p,LW}$ = -0.50 external pressure coeff. (Fig 27.4-1) use with qh

Windward (WW) $P_{ww} = q_z*G\ C_p\text{-}q_h*(GC_{pi})$ Eq 27.4-1 - for building of all heights

Leeward (LW) $P_{Lw} = q_h*G\ C_p\text{-}q_h*(GC_{pi})$

Level	Internal Pressure Case I (+Gcpi)			Internal Suction Pressure Case II (-Gcpi)		
	P_{WW}	P_{LW}	Σ P	P_{WW}	P_{LW}	Σ P
	psf	psf	psf	psf	psf	psf
Mean Roof Height, qh = qz	11.2	-13.6	24.8	19.3	-5.5	24.8
Fourth Story, mid story	0.0	0.0	0.0	0.0	0.0	0.0
Third story, mid story	0.0	0.0	0.0	0.0	0.0	0.0
Second Story, mid story	0.0	0.0	0.0	0.0	0.0	0.0
First Story, mid story	11.2	-13.6	24.8	19.3	-5.5	24.8

Walls:

Side Walls:

B/L = 0.25

Cp, Side Wall = -0.70 external pressure coeff. (Fig 27.4-1) use with qh

Windward (WW) $P_{ww} = q_z*G\ C_p\text{-}q_h*(GC_{pi})$ Eq 27.4-1- for building of all heights

Leeward (LW) $P_{Lw} = q_h*G\ C_p\text{-}q_h*(GC_{pi})$

Level	Internal Pressure Case I (+Gcpi)	Internal Suction Pressure Case II (-Gcpi)
	P_{SW} (psf)	P_{SW} (psf)
Mean Roof Height, qh = qz	-17.4	-9.3
Fourth Story, mid story	0.0	0.0
Third story, mid story	0.0	0.0
Second Story, mid story	0.0	0.0
First Story, mid story	-17.4	-9.3

EEIS CONSULTING ENGINEERS, Inc.

PROJECT	City Of Fairbanks PWD- Pump Enclosure
SHEET NO	Lat -
CALCULATED BY	Benito
DATE	4/7/2023
PROJECT NO	223004

MWFRS WIND LOADS - ALL HEIGHTS METHOD - WALLS AND ROOFS

ASCE/SEI 7-16 CHAPTER 27 PART 1

Roofs:

Wind Direction Normal to Ridge for θ ≥ 10°

Roof Angle, θ = 0.00° < 10 degree Roof Slope Does not Apply.

Windward C_p max. = N/A external pressure coeff. (Fig 27.4-1); use with qh, +GCPi

Windward C_p min. = N/A external pressure coeff. (Fig 27.4-1); use with qh, -GCPi

Leeward C_p = N/A external pressure coeff. (Fig 27.4-1); use with qh

P up = qz*G Cp - Gcpi

h/L	Level	Internal Pressure	Internal Pressure Case I (+Gcpi)		Internal Suction Pressure Case II (-Gcpi)	
			P_{WW}	P_{LW}	P_{WW}	P_{LW}
		psf	psf	psf	psf	psf
0.60	Roof	3.51	N/A	N/A	N/A	N/A

Roofs:

Wind Direction Normal to Ridge for θ < 10°, Windward Side Only, and Wind Parallel to Ridge for all (0).

Roof Angle θ = 0.00°

Windward C_p = -0.9 external pressure coeff. (Fig 27.4-1)

Windward C_p = -0.18 external pressure coeff. (Fig 27.4-1)

Leeward C_p = -0.30 external pressure coeff. (Fig 27.4-1)

$P_{up} = q_z$*G Cp - Gcpi

Windward Edge For h/L ≤ 0.5

h/L	Level	From ft	To ft	Windward Cp (Max.)	Pup (Max.) psf	Cp (Min.)	Pup (Min.) psf
0.60	0 to h/2	0	6.00	-0.9	-21.2	-0.18	-7.5
For h/L ≤ 0.5	h/2 to h	6.00	12.00	-0.9	-21.2	-0.18	-7.5
Intrplt w/next table	h to 2h	12.00	24.00	-0.5	-13.6	-0.18	-7.5
	> 2h	24.00	>	-0.3	-9.7	-0.18	-7.5

Windward Edge for h/L ≥ 1.0

h/L	Level	From ft	To ft	Windward Cp (Max.)	Pup (Max.) psf	Cp (Min.)	Pup (Min.) psf
0.60	0 to h/2	0	6.00	-1.3	-28.8	-0.18	-7.5
Level Does Not Apply	h/2 to h	6.00	12.00	-0.7	-17.4	-0.18	-7.5

EEIS CONSULTING ENGINEERS, Inc.

PROJECT	City of Fairbanks-PWD-Pump enclosure
SHEET NO.	
CALCULATED BY	Sai
DATE	6/23/2023
PROJECT NO.	223004.0000

SEISMIC LOAD DISTRIBUTION AND LATERAL SUMMARY

Element ID	Area (sq. ft.)	Unit Weight (ksf)	Weight (kips)
Ar	100	0.023	2.3
Aew	600	0.010	6.0
Aiw	0	0.008	0.0

Diaphragm ID	Weight	
R/1	5.3	kips
Total Weight	5.3	kips
Base Shear	0.7	kips

LATERAL SUMMARY						
Level	WIND-F/R		WIND--S/S		SEISMIC	
	Total Load (kips)	Trib (ft)	Total Load (kips)	Trib (ft)	Total Load (kips)	Trib (ft)
BASE	6.0	20.0	1.2	5.0	0.7	5.0

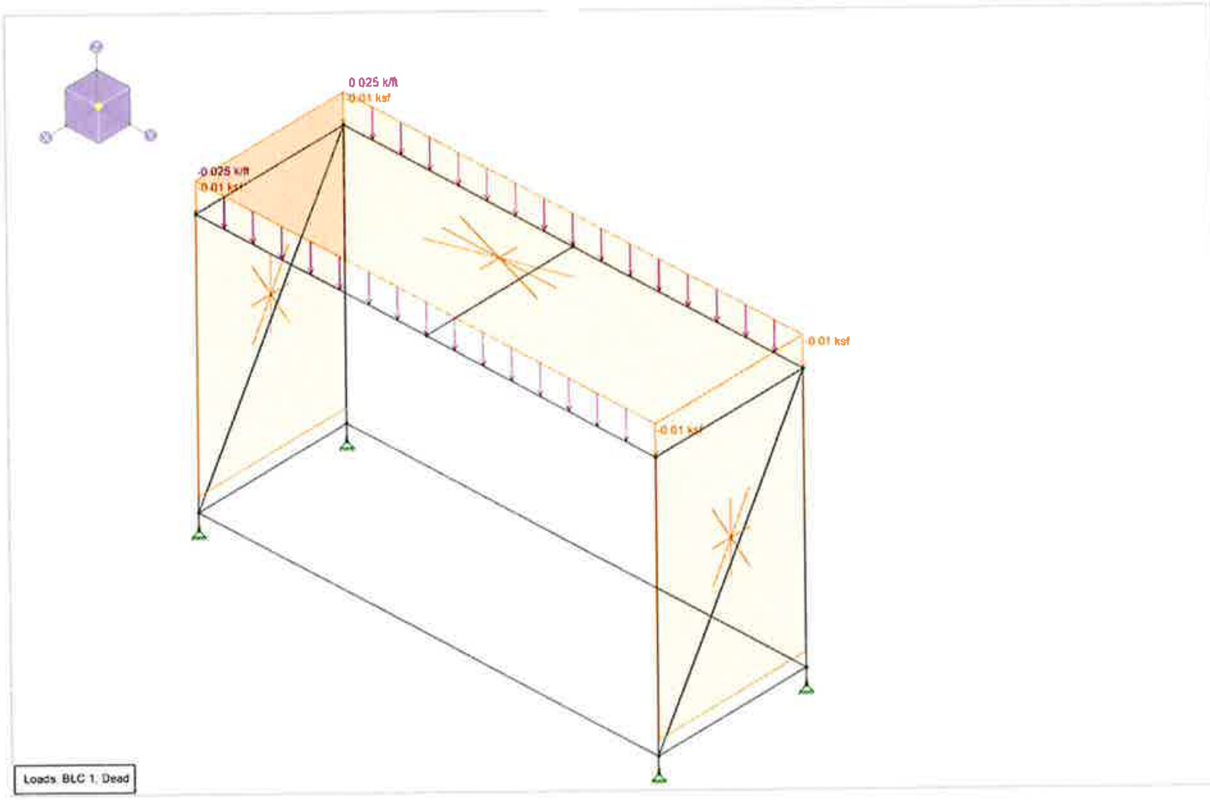
EEIS CONSULTING ENGINEERS, Inc.

PROJECT 22300 4
SHEET NO.
CALCULATED BY SAI
DATE
PROJECT NO.

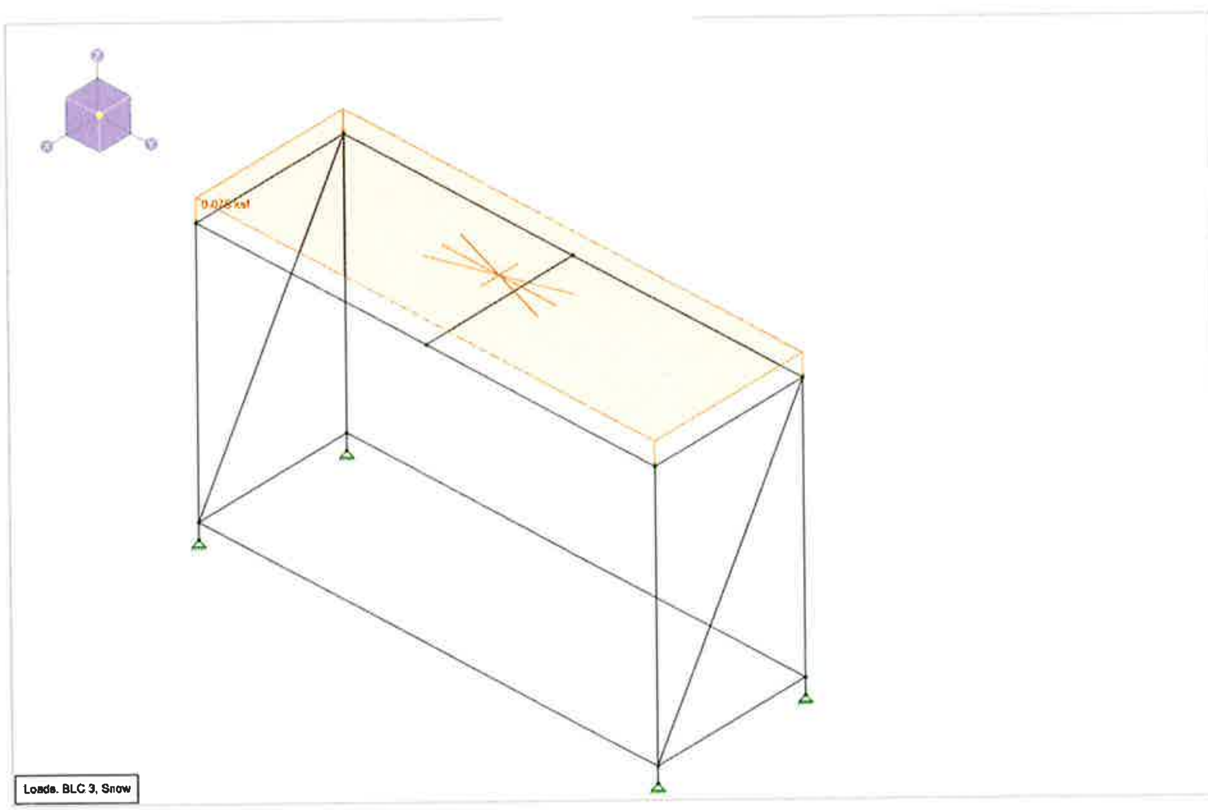
WIND LOAD DISTRIBUTION AND LATERAL SUMMARY

Height	12	ft
Width	5	ft
Length	20	ft
W f/r	24.8	psf
W s/s	19.5	psf
P f/r	6.0	k
P s/s	1.2	k

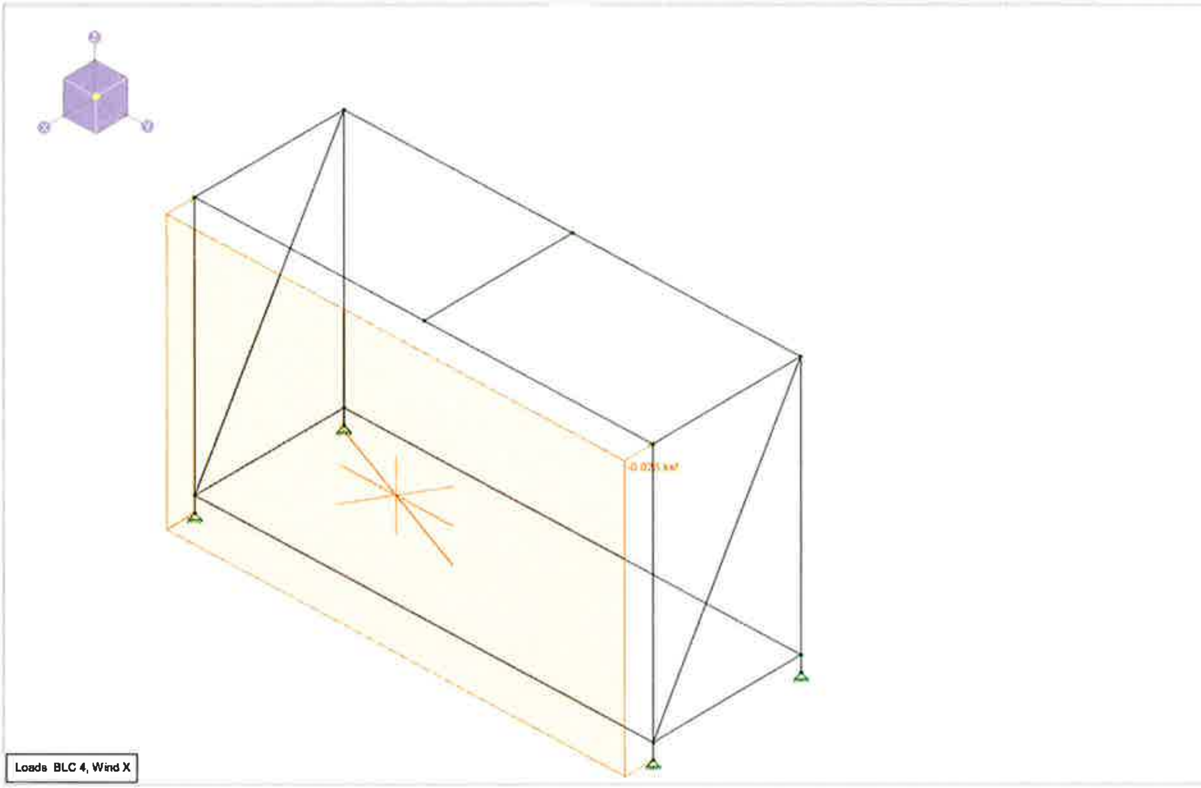
DEAD LOAD



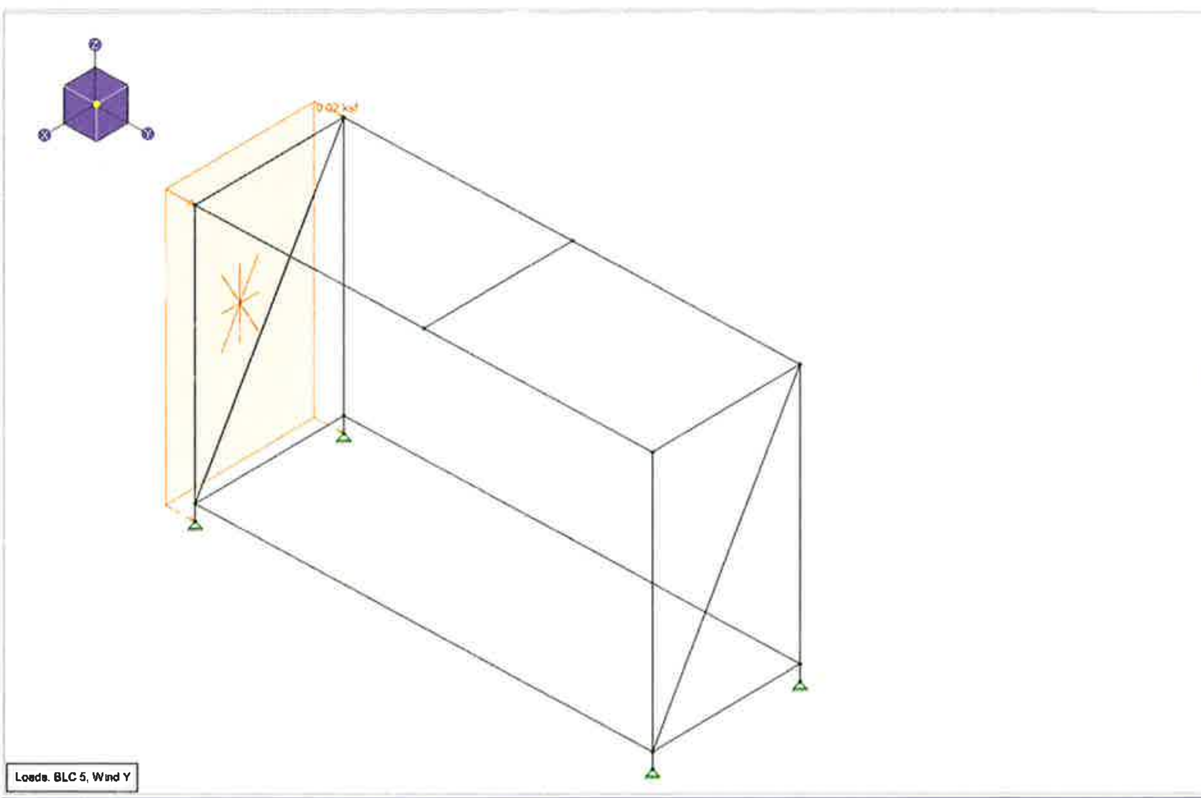
SNOW LOAD



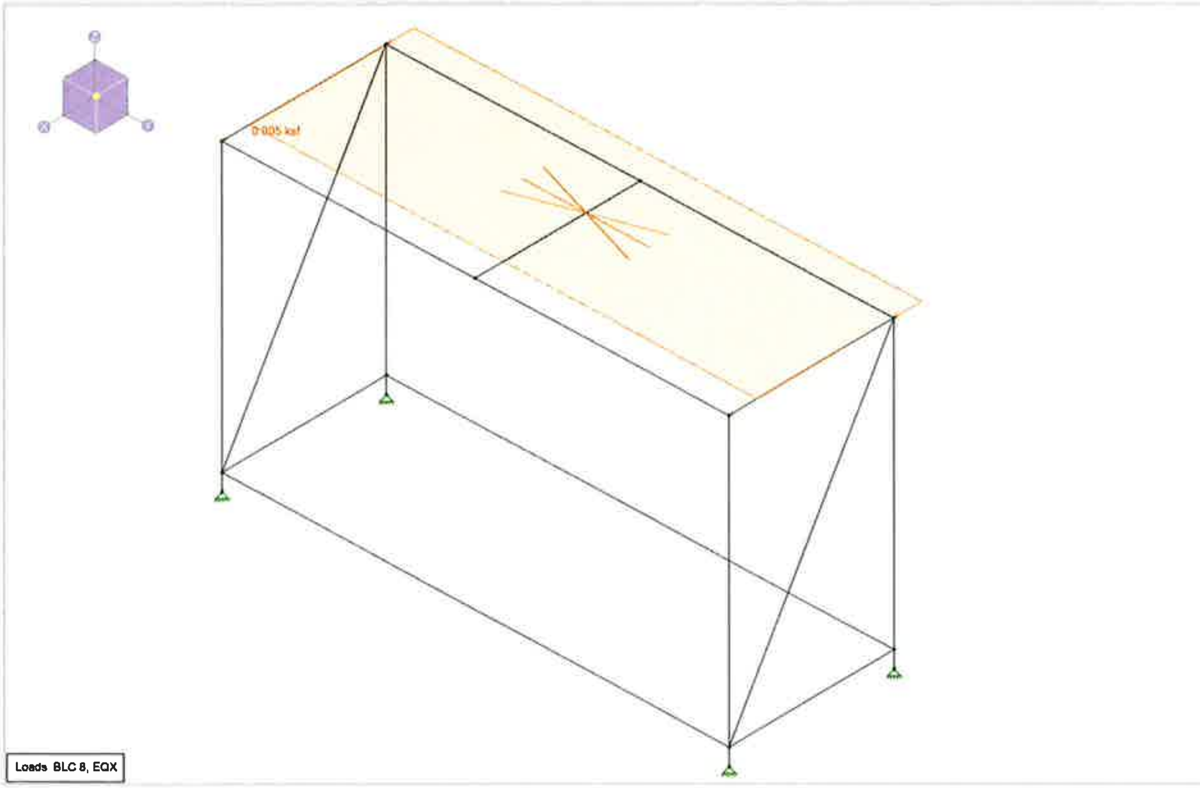
WIND-X LOAD



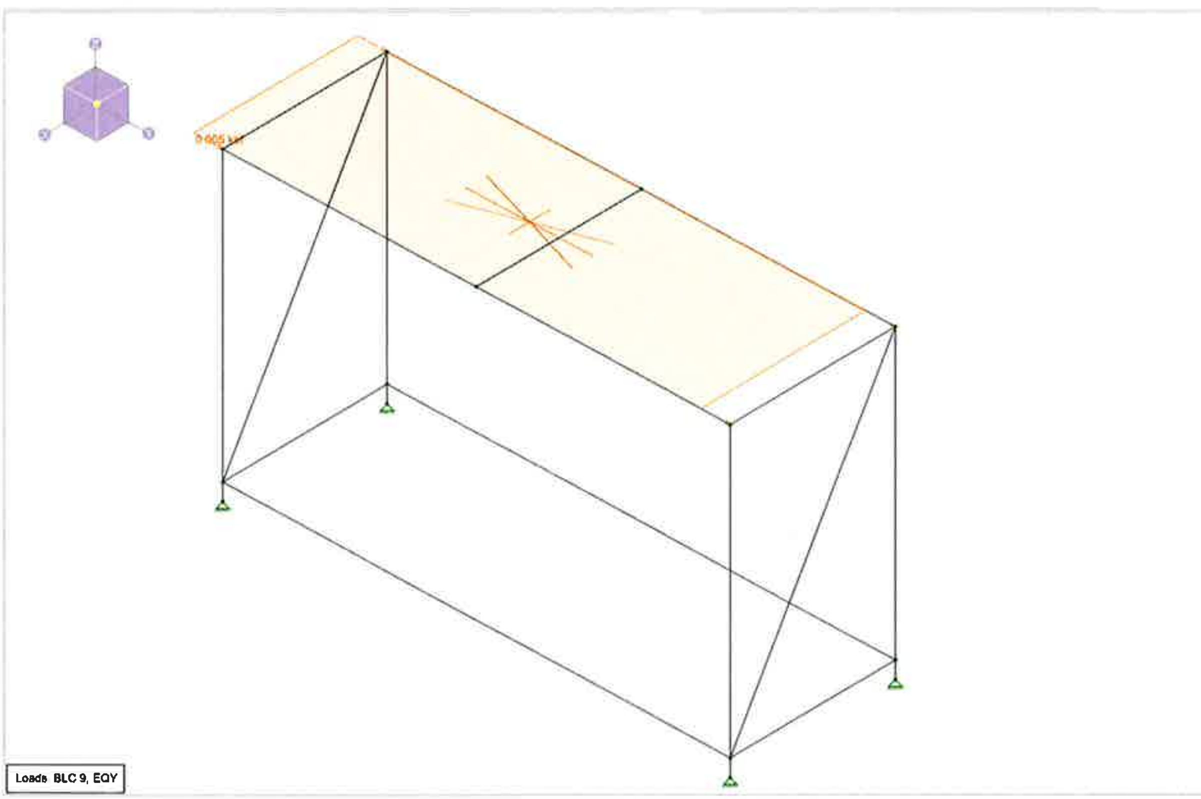
WIND-Y LOAD



SEISMIC – X LOAD



SEISMIC-Y LOAD



MEMBERS

	Label	Shape	Type	Design List	Material	Design Rule	Area [in²]	Iyy [in⁴]	Izz [in⁴]	J [in⁴]
1	Beams	HSS6X4X6	Beam	Tube	A36 Gr.36	Typical	6.18	14.9	28.3	32.8
2	Column	HSS6X4X6	Column	Tube	A36 Gr.36	Typical	6.18	14.9	28.3	32.8
3	Diagonal	L3X3X4	HBrace	Single Angle	A36 Gr.36	Typical	1.44	1.23	1.23	0.031
4	Frame Rafter	L3X3X4	Beam	Single Angle	A36 Gr.36	Typical	1.44	1.23	1.23	0.031

LOAD CASES

Basic Load Cases

	BLC Description	Category
1	Dead	DL
2	Live	LL
3	Snow	SL
4	Wind X	WLX
5	Wind Y	WLY
6	Pick	OL3
7	Drift	SL
8	EQX	EL
9	EQY	EL

LOAD COMBINATIONS

Load Combinations

Combinations | Design

LC Generator | RSA Scaling Factor

	Description	Solve	P-Delta	SRSS	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	Total pick...	☑	Y		1	1	6	1	L2	1	WLX	0.6				
2	Self weight	☑	Y		12	1										
3	Imposed...	☑	Y		1	1										
4	Added we...	☑	Y		6	1										
5	D+L	☑	Y		DL	1	LL	1	LLS	1						
6	D+S	☑	Y		DL	1	SL	1	SLN	1						
7	D+0.75L+...	☑	Y		DL	1	LL	0.75	LLS	0.75	SL	0.75	SLN	0.75		
8	IBC 16-12...	☑	Y		DL	1	WLX	0.6								
9	IBC 16-12...	☑	Y		DL	1	WLY	0.6								
10	IBC 16-13...	☑	Y		DL	1	WLX	0.45	LL	0.75	LLS	0.75				
11	IBC 16-13...	☑	Y		DL	1	WLY	0.45	LL	0.75	LLS	0.75				
12	IBC 16-13...	☑	Y		DL	1	WLX	0.45	LL	0.75	LLS	0.75	SL	0.75	SLN	0.75
13	IBC 16-13...	☑	Y		DL	1	WLY	0.45	LL	0.75	LLS	0.75	SL	0.75	SLN	0.75
14	IBC 16-15...	☑	Y		DL	0.6	WLX	0.6								
15	IBC 16-15...	☑	Y		DL	0.6	WLY	0.6								
16	Deflection...	☑	Y		DL	1										
17	Deflection...	☑	Y		LL	1										
18	Deflection...	☑	Y		DL	1	LL	1								
19	IBC 16-8	☑	Y		DL	1										
20	IBC 16-9	☑	Y		DL	1	LL	1	LLS	1						
21	IBC 16-10...	☑	Y		DL	1	SL	1	SLN	1						
22	IBC 16-11...	☑	Y		DL	1	LL	0.75	LLS	0.75	SL	0.75	SLN	0.75		
23	DEAD ON...	☑	Y		12	1	1	1	6							
24	LIVE ONLY	☑	Y		LL	1										
25	SNOW O...	☑	Y		SL	1										
26	WIND ON...	☑	Y		4	1										
27	IBC 16-5	☑	Y		DL	0.9	EL	1	SL	0.6						
28	IBC 16-7	☑	Y		DL	0.9	EL	1								

M-5

BEAM

Detail Report - M209

Load Combination: Envelope | << | Member Label: M209 | >> | Add to Full Report | Options | Print

Detail Report: M209 | **Unity Check: 0.168 (LC 21)** | **Load Combination: Envelope**

Input Data:

Shape:	HSS6X4X6	I Node:	N88
Member Type:	Beam	J Node:	N84
Length (in):	177	I Release:	Fixed
Material Type:	Hot Rolled Steel	J Release:	Fixed
Design Rule:	Typical	I Offset (in):	N/A
Number of Internal Sections:	97	J Offset (in):	N/A

Material Properties:

Material:	A36 Gr.36	Therm. Coeff. ($1e^{5}$°F^{-1}):	0.65	R_y:	1.5
E (ksi):	29000	Density (k/ft^3):	0.49	F_u (ksi):	58
G (ksi):	11154	F_y (ksi):	36	R_t:	1.2
Nu:	0.3				

Shape Properties:

d (in):	6	I_{yy} (in^4):	14.9	Area (in^2):	6.18
b_f (in):	4	I_{zz} (in^4):	28.3	J (in^4):	32.8
t (in):	0.349				

Design Properties:

$L_{b\,y-y}$ (in):	N/A	K_{y-y}:	1	Max Defl Ratio:	L/1111
$L_{b\,z-z}$ (in):	N/A	K_{z-z}:	1	Max Defl Location:	88.5
$L_{comp\,top}$ (in):	Lbyy	y sway:	No	Span:	1
$L_{comp\,bot}$ (in):	N/A	z sway:	No		
L_{torque} (in):	N/A	Function:	Lateral		
		Seismic DR:	None		

AISC 15th (360-16): ASD Code Check

Limit State	Gov. LC	Required	Available	Unity Check	Result
Applied Loading - Bending/Axial	21	-	-	-	-
Applied Loading - Shear + Torsion	21	-	-	-	-
Axial Tension Analysis	21	0.000 k	133.222 k	-	-
Axial Compression Analysis	21	0.417 k	67.218 k	-	-
Flexural Analysis (Strong Axis)	21	3.501 k-ft	21.377 k-ft	-	-
Flexural Analysis (Weak Axis)		0.01 k-ft	16.06 k-ft	-	-
Shear Analysis (Major Axis y)	21	1.579 k	44.716 k	0.035	Pass
Shear Analysis (Minor Axis z)	21	0.076 k	26.66 k	0.003	Pass
Bending & Axial Interaction Check (UC Bending Max)	21	-	-	0.168	Pass
Torsional Analysis	21	0.042 k-ft	15.345 k-ft	0.003	Pass

Close

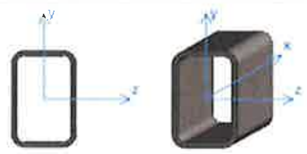
M-6

COLUMN

Detail Report - M143

Load Combination: Envelope | << | Member Label: M143 | >> | Add to Full Report | Options | Print

Detail Report: M143 | Unity Check: 0.158 (LC 21) | Load Combination: Envelope



Input Data:

Shape:	HSS6X4X6	I Node:	N88
Member Type:	Column	J Node:	N85
Length (in):	108	I Release:	Fixed
Material Type:	Hot Rolled Steel	J Release:	Fixed
Design Rule:	Typical	I Offset (in):	N/A
Number of Internal Sections:	97	J Offset (in):	N/A

Material Properties:

Material:	A36 Gr.36	Therm. Coeff. ($1e^{5}$°F^{-1}):	0.65	R_y:	1.5
E (ksi):	29000	Density (k/ft^3):	0.49	F_u (ksi):	58
G (ksi):	11154	F_y (ksi):	36	R_t:	1.2
Nu:	0.3				

Shape Properties:

d (in):	6	I_{yy} (in^4):	14.9	Area (in^2):	6.18
b_f (in):	4	I_{zz} (in^4):	28.3	J (in^4):	32.8
t (in):	0.349				

Design Properties:

$L_{b\,y\text{-}y}$ (in):	N/A	$K_{y\text{-}y}$:	1	Max Defl Ratio:	L/1125
$L_{b\,z\text{-}z}$ (in):	N/A	$K_{z\text{-}z}$:	1	Max Defl Location:	0
$L_{comp\ top}$ (in):	Lbyy	y sway:	No	Span:	N/A
$L_{comp\ bot}$ (in):	N/A	z sway:	No		
L_{torque} (in):	N/A	Function:	Lateral		
		Seismic DR:	None		

AISC 15th (360-16): ASD Code Check

Limit State	Gov. LC	Required	Available	Unity Check	Result
Applied Loading - Bending/Axial	21	-	-	-	-
Applied Loading - Shear + Torsion	26	-	-	-	-
Axial Tension Analysis	21	0.000 k	133.222 k	-	-
Axial Compression Analysis	21	1.792 k	103.268 k	-	-
Flexural Analysis (Strong Axis)	21	3.031 k-ft	21.377 k-ft	-	-
Flexural Analysis (Weak Axis)		0.119 k-ft	16.06 k-ft	-	-
Shear Analysis (Major Axis y)	26	2.913 k	44.716 k	0.065	Pass
Shear Analysis (Minor Axis z)	26	1.985 k	26.66 k	0.074	Pass
Bending & Axial Interaction Check (UC Bending Max)	21	-	-	0.158	Pass
Torsional Analysis	21	0.005 k-ft	15.345 k-ft	0.000	Pass

Close

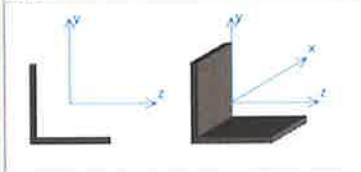
M-7

DIAGONALS AND RAFTERS

Detail Report - M141

Load Combination: Envelope   << Member Label: M141 >>   Add to Full Report   Options   Print

Detail Report: M141   Unity Check: 0.173 (LC 26)   Load Combination: Envelope



Input Data:

Shape:	L3X3X4	I Node:	N90
Member Type:	HBrace	J Node:	N104
Length (in):	118.338	I Release:	Fixed
Material Type:	Hot Rolled Steel	J Release:	Fixed
Design Rule:	Typical	I Offset (in):	N/A
Number of Internal Sections:	97	J Offset (in):	N/A

Material Properties:

Material:	A36 Gr.36	Therm. Coeff. ($1e^{5}$ °F^{-1}):	0.65	R_y:	1.5
E (ksi):	29000	Density (k/ft^3):	0.49	F_u (ksi):	58
G (ksi):	11154	F_y (ksi):	36	R_t:	1.2
Nu:	0.3				

Shape Properties:

d (in):	3	Area (in^2):	1.44	J (in^4):	0.031
b_f (in):	3	I_{yy} (in^4):	1.23	r_z (in):	0.585
t (in):	0.25	I_{zz} (in^4):	1.23	k* (in):	0.625

Design Properties:

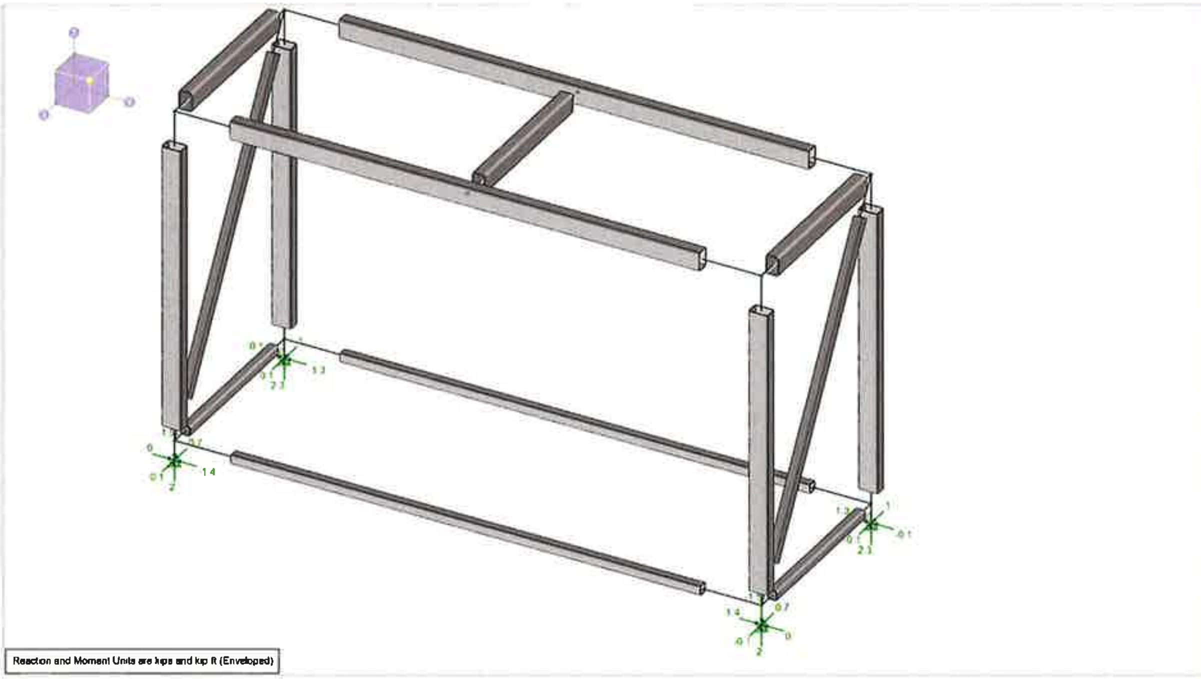
$L_{b\,y\text{-}y}$ (in):	N/A	$K_{y\text{-}y}$:	1	Max Defl Ratio:	L/3154
$L_{b\,z\text{-}z}$ (in):	N/A	$K_{z\text{-}z}$:	1	Max Defl Location:	0
$L_{comp\ top}$ (in):	Lbyy	y sway:	No	Span:	N/A
$L_{comp\ bot}$ (in):	N/A	z sway:	No		
L_{torque} (in):	N/A	Function:	Lateral		
		Seismic DR:	None		

AISC 15th (360-16): ASD Code Check

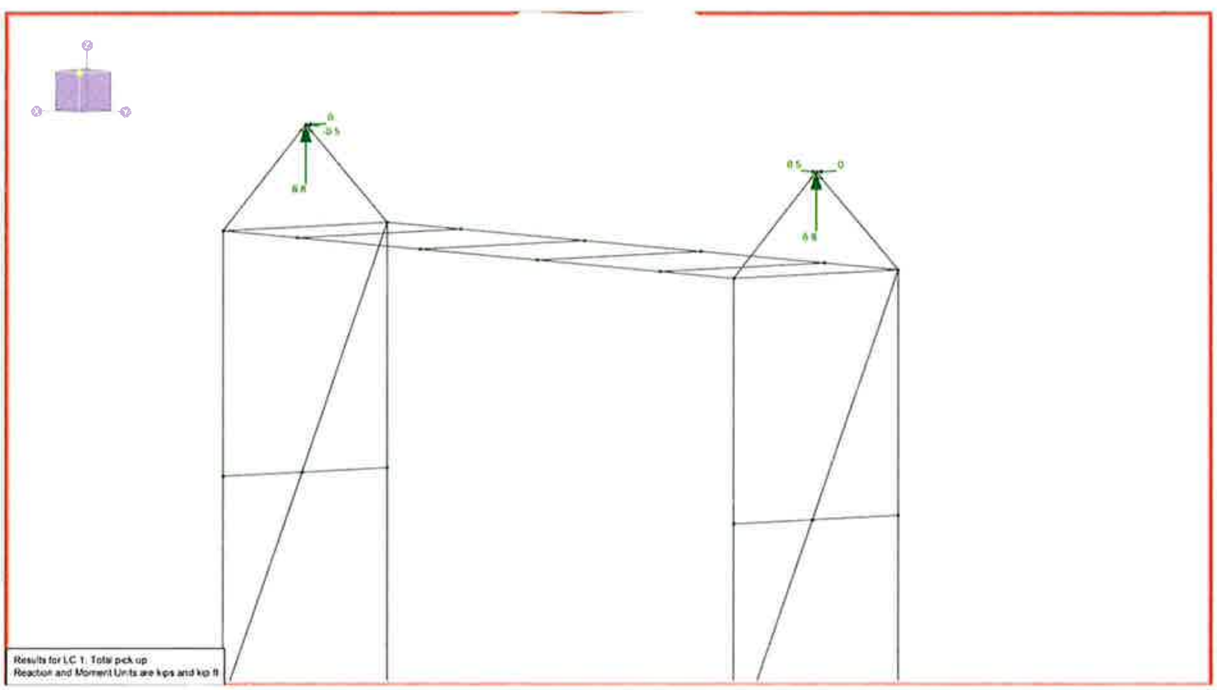
Limit State	Gov. LC	Required	Available	Unity Check	Result
Applied Loading - Bending/Axial	26	-	-	-	-
Applied Loading - Shear + Torsion	26	-	-	-	-
Axial Tension Analysis	26	1.686 k	31.042 k	-	-
Axial Compression Analysis	26	0.000 k	5.289 k	-	-
Flexural Analysis (Strong Axis)	26	0.134 k-ft	1.992 k-ft	-	-
Flexural Analysis (Weak Axis)		0.057 k-ft	1.123 k-ft	-	-
Shear Analysis (Major Axis y)	26	0.073 k	9.701 k	0.007	Pass
Shear Analysis (Minor Axis z)	26	0.061 k	9.701 k	0.006	Pass
Bending & Axial Interaction Check (UC Bending Max)	26	-	-	0.173	Pass

Close

Reaction loads



Pickup reactions

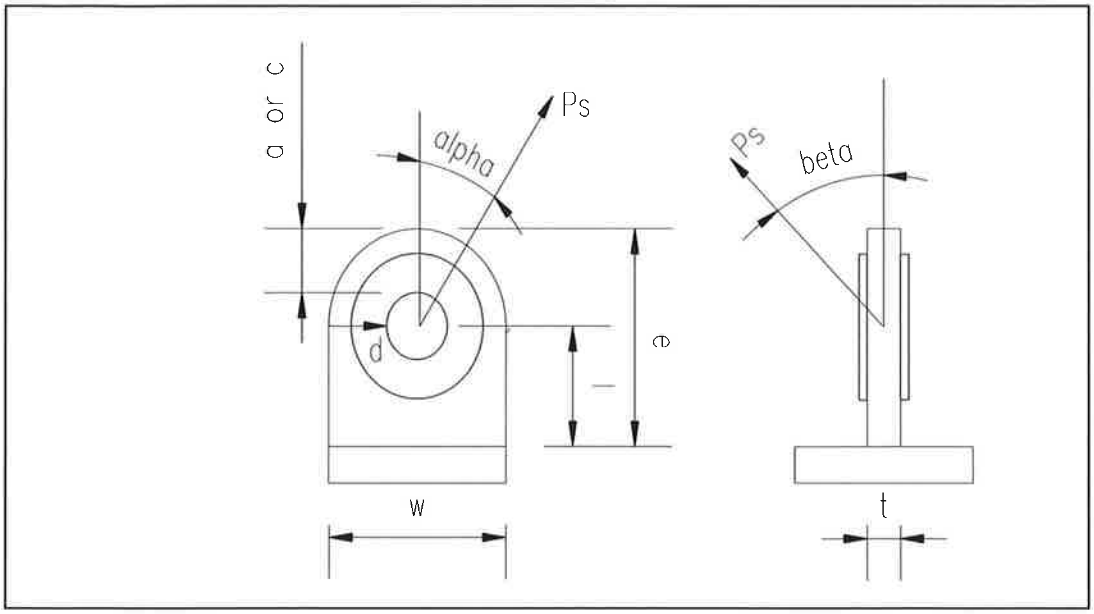


EEIS CONSULTING ENGINEERS, INC.

Project: 221001_OSA
Module Lifting Lugs

By: Sai
Date: 6/23/2023

HOIST LIFTING PLATE DESIGN SPREADSHEET



ALLOWABLE LOAD SUMMARY	
$P_{a\_(D5-1)}$ =	24
$P_{a\_(D5-2)}$ =	18
$P_{a\_(J7-1)}$ =	30
$P_{a\_(D2-1)}$ =	45
$P_{a\_(J2-5)}$ =	15
Min =	15
RATING	**15 [Kips]**

Dimensions

	a =	**1.500**	[in]
Lug Width	w =	**4.500**	[in]
	c =	**1.500**	[in]
Hole Diameter	d =	**1.500**	[in]
Lug Height	e =	**15.000**	[in]
Fillet Weld Leg Thickness	t_w =	**1/4**	[in]
Fillet Weld Total Length	l_w =	**3.000**	[in]
	l =	**12.000**	[in]
Lug Thickness	t =	**0.500**	[in]
Cheek Plate Thickness	t_c =	**0.500**	[in]
Shackle Pin Diameter	d_s =	**0.750**	[in]

Load

Load	P_s	**11.00**	[Kips]
Factored Load			
	$\mathbf{P_s} = P_s * l / \cos(alpha)$ =	**44.0**	[Kips]
In-Plane Angle	alpha	**60**	[deg]
Out-Plane Angle	beta	**0**	[deg]

Material Strength

Ultimate Tensile Stress	F_u	**65**	[ksi]
Yield Stress	F_y	**50**	[ksi]
Electrode Classificatior	F_{EXX}	**70**	[ksi]

Desired Safety Factors

[1]Impact Factor	I	**2.00**	
[2]Yield Stress Factor of Safety, Ω_y	Ω_y	**3.33**	
[2]Ultimate Stress Factor of Safety, Ω_u	Ω_u	**4.00**	= 3* 65ksi / 50 ksi

1) Impact factor of 2.0 is per Eni Spec.

2) Stess factors of safety are per BPXA new structural spec interoffice email review, from previous project, accepted industry use.

A -1

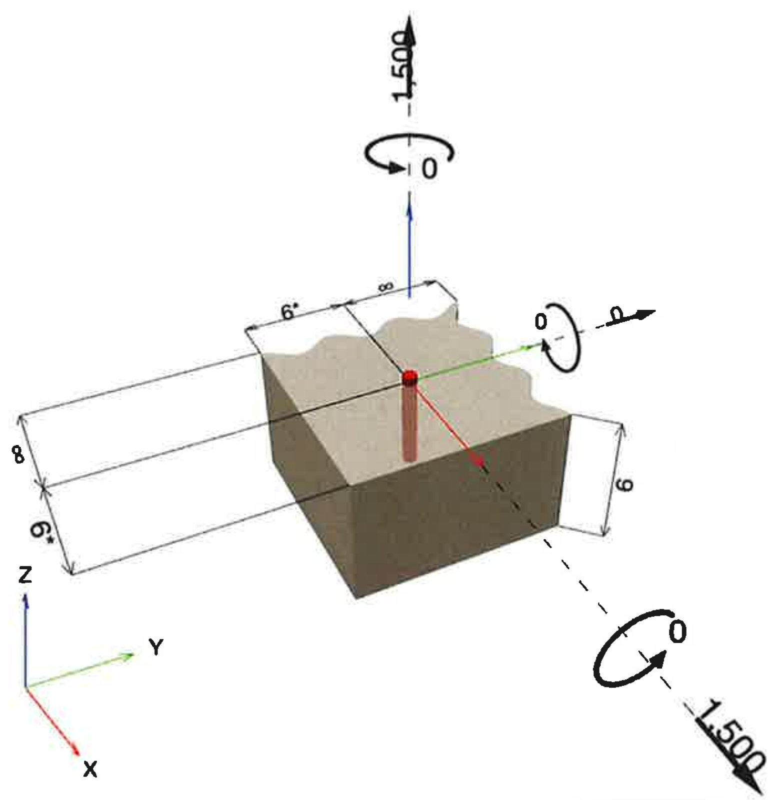
Hilti PROFIS Engineering 3.0.85

Specifier's comments:

1 Input data

Anchor type and diameter:	**Kwik Bolt TZ2 - CS 5/8 (4) hnom3**
Item number:	2210272 KB-TZ2 5/8x5 1/2
Effective embedment depth:	$h_{ef,act}$ = 4.000 in., h_{nom} = 4.500 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-4266
Issued I Valid:	12/17/2021 | 12/1/2023
Proof:	Design Method ACI 318-14 / Mech
Stand-off installation:	
Profile:	
Base material:	cracked concrete, 2500, f_c' = 2,500 psi; h = 6.000 in.
Installation:	**automatic cleaned drilled hole, Installation condition: Dry**
Reinforcement:	tension: condition B, shear: condition B; no supplemental splitting reinforcement present edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	Tension load: yes (17.2.3.4.3 (d)) Shear load: yes (17.2.3.5.3 (c))

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



1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	AC-1 Restraint Loading	N = 1,500; V_x = 1,500; V_y = 0; M_x = 0; M_y = 0; M_z = 0;	yes	69

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	1,500	1,500	1,500	0

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 0 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	1,500	13,157	12	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	1,500	3,315	46	OK

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

3.1 Steel Strength

N_{sa} = ESR value refer to ICC-ES ESR-4266
ϕ $N_{sa} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.16	106,704

Calculations

N_{sa} [lb]
17,542

Results

N_{sa} [lb]	ϕ_{steel}	$\phi_{nonductile}$	ϕ N_{sa} [lb]	N_{ua} [lb]
17,542	0.750	1.000	13,157	1,500

3.2 Concrete Breakout Failure

$N_{cb} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$	ACI 318-14 Eq. (17.4.2.1a)
$\phi\ N_{cb} \geq N_{ua}$	ACI 318-14 Table 17.3.1.1
A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)	
$A_{Nc0} = 9\ h_{ef}^2$	ACI 318-14 Eq. (17.4.2.1c)
$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$	ACI 318-14 Eq. (17.4.2.5b)
$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$	ACI 318-14 Eq. (17.4.2.7b)
$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$	ACI 318-14 Eq. (17.4.2.2a)

Variables

h_{ef} [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
4.000	6.000	1.000	8.750	17	1.000	2,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
144.00	144.00	1.000	1.000	6,800

Results

N_{cb} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ N_{cb}$ [lb]	N_{ua} [lb]
6,800	0.650	0.750	1.000	3,315	1,500

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	1,500	6,668	23	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	1,500	9,520	16	OK
Concrete edge failure in direction x+**	1,500	2,527	60	OK

* highest loaded anchor   **anchor group (relevant anchors)

4.1 Steel Strength

$V_{sa,eq}$ = ESR value   refer to ICC-ES ESR-4266
ϕ $V_{steel} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]	$\alpha_{V,seis}$
0.16	106,704	1.000

Calculations

$V_{sa,eq}$ [lb]
10,259

Results

$V_{sa,eq}$ [lb]	ϕ_{steel}	$\phi_{nonductile}$	ϕ $V_{sa,eq}$ [lb]	V_{ua} [lb]
10,259	0.650	1.000	6,668	1,500

4.2 Pryout Strength

$V_{cp} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b\right]$ — ACI 318-14 Eq. (17.5.3.1a)

$\phi\ V_{cp} \geq V_{ua}$ — ACI 318-14 Table 17.3.1.1

A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ — ACI 318-14 Eq. (17.4.2.1c)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ — ACI 318-14 Eq. (17.4.2.5b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-14 Eq. (17.4.2.7b)

$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ — ACI 318-14 Eq. (17.4.2.2a)

Variables

k_{cp}	h_{ef} [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
2	4.000	6.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
8.750	17	1.000	2,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
144.00	144.00	1.000	1.000	6,800

Results

V_{cp} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cp}$ [lb]	V_{ua} [lb]
13,600	0.700	1.000	1.000	9,520	1,500

4.3 Concrete edge failure in direction x+

$V_{cb} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ed,V}\ \psi_{c,V}\ \psi_{h,V}\ \psi_{parallel,V}\ V_b$ ACI 318-14 Eq. (17.5.2.1a)

$\phi\ V_{cb} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

A_{Vc} see ACI 318-14, Section 17.5.2.1, Fig. R 17.5.2.1(b)

$A_{Vc0} = 4.5\ c_{a1}^2$ ACI 318-14 Eq. (17.5.2.1c)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ ACI 318-14 Eq. (17.5.2.6b)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ ACI 318-14 Eq. (17.5.2.8)

$V_b = \left(7\left(\frac{l_e}{d_a}\right)^{0.2} \sqrt{d_a}\right) \lambda_a \sqrt{f'_c}\ c_{a1}^{1.5}$ ACI 318-14 Eq. (17.5.2.2a)

Variables

c_{a1} [in.]	c_{a2} [in.]	$\psi_{c,V}$	h_a [in.]	l_e [in.]
6.000	6.000	1.000	6.000	4.000

λ_a	d_a [in.]	f'_c [psi]	$\psi_{parallel,V}$
1.000	0.625	2,500	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
90.00	162.00	0.900	1.225	5,895

Results

V_{cb} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cb}$ [lb]	V_{ua} [lb]
3,610	0.700	1.000	1.000	2,527	1,500

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.452	0.594	5/3	69	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- 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/
- An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-14, Chapter 17, Section 17.2.3.4.3 (a) that requires the governing design strength of an anchor or group of anchors be limited by ductile steel failure. If this is NOT the case, the connection design (tension) shall satisfy the provisions of Section 17.2.3.4.3 (b), Section 17.2.3.4.3 (c), or Section 17.2.3.4.3 (d). The connection design (shear) shall satisfy the provisions of Section 17.2.3.5.3 (a), Section 17.2.3.5.3 (b), or Section 17.2.3.5.3 (c).
- Section 17.2.3.4.3 (b) / Section 17.2.3.5.3 (a) require the attachment the anchors are connecting to the structure be designed to undergo ductile yielding at a load level corresponding to anchor forces no greater than the controlling design strength. Section 17.2.3.4.3 (c) / Section 17.2.3.5.3 (b) waive the ductility requirements and require the anchors to be designed for the maximum tension / shear that can be transmitted to the anchors by a non-yielding attachment. Section 17.2.3.4.3 (d) / Section 17.2.3.5.3 (c) waive the ductility requirements and require the design strength of the anchors to equal or exceed the maximum tension / shear obtained from design load combinations that include E, with E increased by ω_0.
- Hilti post-installed anchors shall be installed in accordance with the Hilti Manufacturer's Printed Installation Instructions (MPII). Reference ACI 318-14, Section 17.8.1.

Fastening meets the design criteria!

7 Installation data

Anchor type and diameter: Kwik Bolt TZ2 - CS 5/8 (4) hnom3
Item number: 2210272 KB-TZ2 5/8x5 1/2
Maximum installation torque: 481 in.lb
Hole diameter in the base material: 0.625 in.
Hole depth in the base material: 4.750 in.
Minimum thickness of the base material: 6.000 in.

Profile: -
Hole diameter in the fixture: -
Plate thickness (input): -

Drilling method: SafeSet - automatic cleaning
Cleaning: Automatically performed while drilling
Setting: Machine torqueing with SIW 6AT-A22 and SI-AT-A22 module

Hilti KB-TZ2 stud anchor with 4.5 in embedment, 5/8 (4) hnom3, Carbon steel, installation per ESR-4266

7.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit for SAFEset - automatic cleaning (TE-CD / TE-YD) • Vacuum cleaner	• No accessory required	• 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	0.000	-	6.000	6.000	-

8 Remarks; Your Cooperation Duties

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

ICC-ES Evaluation Report

ESR-4266

Issued December 2020
Revised March 2021
This report is subject to renewal December 2021.

www.icc-es.org | (800) 423-6587 | (562) 699-0543 *A Subsidiary of the International Code Council®*

DIVISION: 03 00 00—CONCRETE
Section: 03 16 00—Concrete Anchors

DIVISION: 05 00 00—METALS
Section: 05 05 19—Post-Installed Concrete Anchors

REPORT HOLDER:

HILTI, INC.

EVALUATION SUBJECT:

HILTI KWIK BOLT TZ2 CARBON AND STAINLESS STEEL ANCHORS IN CRACKED AND UNCRACKED CONCRETE

1.0 EVALUATION SCOPE

Compliance with the following codes:

- 2021, 2018, 2015, and 2012 *International Building Code®* (IBC)
- 2021, 2018, 2015, and 2012 *International Residential Code®* (IRC)

For evaluation for compliance with the *National Building Code of Canada®* (NBCC), see listing report ELC-4266.

For evaluation for compliance with codes adopted by the Los Angeles Department of Building and Safety (LADBS), see ESR-4266 LABC and LARC Supplement.

Property evaluated:

Structural

2.0 USES

The Hilti Kwik Bolt TZ2 anchor (KB-TZ2) is used as anchorage to resist static, wind, and seismic (Seismic Design Categories A through F) tension and shear loads in cracked and uncracked normal-weight concrete and lightweight concrete having a specified compressive strength, f'_c, of 2,500 psi to 8,500 psi (17.2 MPa to 58.6 MPa).

The $^1/_4$-inch-, $^3/_8$-inch- and $^1/_2$-inch diameter (6.4 mm, 9.5 mm and 12.7 mm) carbon steel KB-TZ2 anchors may be installed in the topside of cracked and uncracked normal-weight or sand-lightweight concrete over metal deck having a minimum member thickness, $h_{min,deck}$, as noted in Table 9 of this evaluation report and a specified compressive strength, f'_c, of 3,000 psi to 8,500 psi (20.7 MPa to 58.6 MPa)

The $^1/_4$-inch-, $^3/_8$-inch-, $^1/_2$-inch-, $^5/_8$-inch- and $^3/_4$-inch diameter (6.4 mm, 9.5 mm, 12.7 mm, 15.9 mm and 19.1 mm) carbon steel KB-TZ2 anchors may be installed in the soffit of cracked and uncracked normal-weight or sand-lightweight concrete over metal deck having a minimum specified compressive strength, f'_c, of 3,000 psi (20.7 MPa).

The anchoring system complies with anchors as described in Section 1901.3 of the 2021, 2018 and 2015 IBC, and Section 1909 of the 2012 IBC. The anchoring system is an alternative to cast-in-place anchors described in Section 1908 of the 2012 IBC. The anchors may also be used where an engineered design is submitted in accordance with Section R301.1.3 of the IRC.

3.0 DESCRIPTION

3.1 KB-TZ2:

KB-TZ2 anchors are torque-controlled, mechanical expansion anchors. KB-TZ2 anchors consist of a stud (anchor body), wedge (expansion elements), nut, and washer. The anchor (carbon steel version) is illustrated in Figure 1. The stud is manufactured from carbon steel or AISI Type 304 or Type 316 stainless steel materials. Carbon steel KB-TZ2 anchors have a minimum 5 μm (0.0002 inch) zinc-nickel plating. The expansion elements for the carbon steel KB-TZ2 anchors are fabricated from carbon steel. The expansion elements for the stainless steel KB-TZ2 anchors are fabricated from stainless steel. The hex nut for carbon steel conforms to ASTM A563-04, Grade A, and the hex nut for stainless steel conforms to ASTM F594.

The anchor body is comprised of a high-strength rod threaded at one end and a tapered mandrel at the other end. The tapered mandrel is enclosed by a three-section expansion element. The expansion element movement is restrained by the mandrel taper and by a collar. The anchor is installed in a predrilled hole with a hammer. When torque is applied to the nut of the installed anchor, the mandrel is drawn into the expansion element, which is in turn expanded against the wall of the drilled hole.

3.2 Concrete:

Normal-weight and lightweight concrete must conform to Sections 1903 and 1905 of the IBC.

3.3 Steel Deck Panels:

Steel deck panels must be in accordance with the configuration in Figure 5A, Figure 5B, Figure 5C and Figure 5D and have a minimum base steel thickness of 0.035 inch (0.899 mm, 20 gauge). Steel must comply with ASTM A653/A653M SS Grade 55 and have a minimum yield strength of 55,000 psi (379 MPa).

4.0 DESIGN AND INSTALLATION

4.1 Strength Design:

4.1.1 General: Design strength of anchors complying with the 2021 IBC, as well as Section R301.1.3 of the 2021 IRC must be determined in accordance with ACI 318-19 Chapter 17 and this report.

Design strength of anchors complying with the 2018 and 2015 IBC, as well as Section R301.1.3 of the 2018 and 2015 IRC must be determined in accordance with ACI 318-14 Chapter 17 and this report.

Design strength of anchors complying with the 2012 IBC as well as Section R301.1.3 of the 2012 IRC, must be determined in accordance with ACI 318-11 Appendix D and this report.

Design parameters provided in Table 4, Table 5, Table 6 and Table 7 of this report are based on the 2021 IBC (ACI 318-19), 2018 and 2015 IBC (ACI 318-14) and the 2012 IBC (ACI 318-11) unless noted otherwise in Sections 4.1.1 through 4.1.12. The strength design of anchors must comply with ACI 318-19 17.5.1.2, ACI 318-14 17.3.1 or ACI 318-11 D.4.1, as applicable, except as required in ACI 318-19 17.10, ACI 318-14 17.2.3 or ACI 318-11 D.3.3, as applicable.

Strength reduction factors, ϕ, as given in ACI 318-19 17.5.3, ACI 318-14 17.3.3 or ACI 318-11 D.4.3, as applicable, and noted in Table 4, Table 5, Table 6, and Table 7 of this report, must be used for load combinations calculated in accordance with Section 1605.1 of the 2021 IBC or Section 1605.2 of the 2018, 2015 and 2012 IBC and Section 5.3 of ACI 318 (-19 and -14) or Section 9.2 of ACI 318-11, as applicable. Strength reduction factors, ϕ, as given in ACI 318-11 D.4.4 must be used for load combinations calculated in accordance with ACI 318-11 Appendix C. The value of f'_c used in the calculations must be limited to a maximum of 8,000 psi (55.2 MPa), in accordance with ACI 318-19 17.3.1, ACI 318-14 17.2.7 or ACI 318-11 D.3.7, as applicable.

4.1.2 Requirements for Static Steel Strength in Tension: The nominal static steel strength, N_{sa}, of a single anchor in tension must be calculated in accordance with ACI 318-19 17.6.1.2, ACI 318-14 17.4.1.2 or ACI 318-11 D.5.1.2, as applicable. The resulting N_{sa} values are provided in Table 4 and Table 5 of this report. Strength reduction factors ϕ corresponding to ductile steel elements may be used.

4.1.3 Requirements for Static Concrete Breakout Strength in Tension: The nominal concrete breakout strength of a single anchor or group of anchors in tension, N_{cb} or N_{cbg}, respectively, must be calculated in accordance with ACI 318-19 17.6.2, ACI 318-14 17.4.2 or ACI 318-11 D.5.2, as applicable, with modifications as described in this section. The basic concrete breakout strength in tension, N_b, must be calculated in accordance with ACI 318-19 17.6.2.2, ACI 318-14 17.4.2.2 or ACI 318-11 D.5.2.2, as applicable, using the values of h_{ef} and k_{cr} as given in Table 4 and Table 5. The nominal concrete breakout strength in tension in regions where analysis indicates no cracking in accordance with ACI 318-19 17.6.2.5.1, ACI 318-14 17.4.2.6 or ACI 318-11 D.5.2.6, as applicable, must be calculated with k_{uncr} as given in Table 4 and Table 5 and with $\Psi_{c,N}$ = 1.0.

For carbon steel KB-TZ2 anchors installed in the soffit of sand-lightweight or normal-weight concrete on steel deck floor and roof assemblies, as shown in Figure 5A, Figure 5B and Figure 5C, calculation of the concrete breakout strength is not required.

4.1.4 Requirements for Static Pullout Strength in Tension: The nominal pullout strength of a single anchor in accordance with ACI 318-19 17.6.3.1 and 17.6.3.2.1, ACI 318-14 17.4.3.1 and 17.4.3.2 or ACI 318-11 D.5.3.1 and D.5.3.2, respectively, as applicable, in cracked and uncracked concrete, $N_{p,cr}$ and n_{cr}, $N_{p,uncr}$ and n_{uncr}, respectively, are given in Table 4 and Table 5. For all design cases $\Psi_{c,P}$ = 1.0. In accordance with ACI 318-19 17.6.3, ACI 318-14 17.4.3 or ACI 318-11 D.5.3, as applicable, the nominal pullout strength in cracked concrete may be calculated in accordance with the following equation:

$$N_{p,f'_c} = N_{p,cr}\left(\frac{f'_c}{2{,}500}\right)^{n_{cr}} \quad \text{(lb, psi)} \qquad \text{(Eq-1)}$$

$$N_{p,f'_c} = N_{p,cr}\left(\frac{f'_c}{17.2}\right)^{n_{cr}} \quad \text{(N, MPa)}$$

In regions where analysis indicates no cracking in accordance with ACI 318-19 17.6.3.3, ACI 318-14 17.4.3.6 or ACI 318-11 D.5.3.6, as applicable, the nominal pullout strength in tension may be calculated in accordance with the following equation:

$$N_{p,f'_c} = N_{p,uncr}\left(\frac{f'_c}{2{,}500}\right)^{n_{uncr}} \quad \text{(lb, psi)} \qquad \text{(Eq-2)}$$

$$N_{p,f'_c} = N_{p,uncr}\left(\frac{f'_c}{17.2}\right)^{n_{uncr}} \quad \text{(N, MPa)}$$

Where values for $N_{p,cr}$ or $N_{p,uncr}$ are not provided in Table 4 or Table 5, the pullout strength in tension need not be evaluated.

The nominal pullout strength in cracked concrete of the carbon steel KB-TZ2 installed in the soffit of sand-lightweight or normal-weight concrete on steel deck floor and roof assemblies, as shown in Figure 5A, Figure 5B and Figure 5C, is given in Table 8. In accordance with ACI 318-19 17.6.3.2.1, ACI 318-14 17.4.3.2 or ACI 318-11 D.5.3.2, as applicable, the nominal pullout strength in cracked concrete must be calculated in accordance with Eq-1, whereby the value of $N_{p,deck,cr}$ must be substituted for $N_{p,cr}$ and the value of 3,000 psi (20.7 MPa) must be substituted for the value of 2,500 psi (17.2 MPa) in the denominator. In regions where analysis indicates no cracking in accordance with ACI 318-19 17.6.3.3, ACI 318-14 17.4.3.6 or ACI 318-11 D.5.3.6, as applicable, the nominal strength in uncracked concrete must be calculated according to Eq-2, whereby the value of $N_{p,deck,uncr}$ must be substituted for $N_{p,uncr}$ and the value of 3,000 psi (20.7 MPa) must be substituted for the value of 2,500 psi (17.2 MPa) in the denominator. The use of stainless steel KB-TZ2 anchors installed in the soffit of concrete on steel deck assemblies is beyond the scope of this report.

4.1.5 Requirements for Static Steel Strength in Shear: The nominal steel strength in shear, V_{sa}, of a single anchor in accordance with ACI 318-19 17.7.1.2, ACI 318-14 17.5.1.2 or ACI 318-11 D.6.1.2, as applicable, is given in Table 6 and Table 7 of this report and must be used in lieu of the values derived by calculation from ACI 318-19 Eq. 17.7.1.2b, ACI 318-14 Eq. 17.5.1.2b or ACI 318-11 Eq. D-29, as applicable. The shear strength $V_{sa,deck}$ of the carbon-steel KB-TZ2 as governed by steel failure of the KB-TZ2 installed in the soffit of sand-lightweight or normal-weight concrete on steel deck floor and roof assemblies, as shown in Figure 5A, Figure 5B and Figure 5C, is given in Table 8.

4.1.6 Requirements for Static Concrete Breakout Strength in Shear: The nominal concrete breakout strength of a single anchor or group of anchors in shear, V_{cb} or V_{cbg}, respectively, must be calculated in accordance with ACI 318-19 17.7.2, ACI 318-14 17.5.2 or ACI 318-11 D.6.2, as applicable, with modifications as described in this section. The basic concrete breakout strength, V_b, must be calculated in accordance with ACI 318-19 17.7.2.2.1, ACI 318-14 17.5.2.2 or ACI 318-11 D.6.2.2, as applicable, based on the values provided in Table 6 and Table 7. The value of ℓ_e used in ACI 318-19 Eq. 17.7.2.2.1a, ACI 318-14 Eq. 17.5.2.2a or ACI 318-11 Eq. D-33 must be taken as no greater than the lesser of h_{ef} or $8d_a$. Anchors installed in light-weight concrete must use the reduction factors provided in ACI 318-19 17.2.4, ACI 318-14 17.2.6 or ACI 318-11 D.3.6, as applicable.

For carbon steel KB-TZ2 anchors installed in the soffit of sand-lightweight or normal-weight concrete on steel deck floor and roof assemblies, as shown in Figure 5A, Figure 5B and Figure 5C, calculation of the concrete breakout strength in shear is not required.

4.1.7 Requirements for Static Concrete Pryout Strength in Shear: The nominal concrete pryout strength of a single anchor or group of anchors, V_{cp} or V_{cpg}, respectively, must be calculated in accordance with ACI 318-19 17.7.3, ACI 318-14 17.5.3 or ACI 318-11 D.6.3, as applicable, modified by using the value of k_{cp} provided in Table 6 and Table 7 of this report and the value of N_{cb} or N_{cbg} as calculated in Section 4.1.3 of this report.

For carbon steel KB-TZ2 anchors installed in the soffit of sand-lightweight or normal-weight concrete over profile steel deck floor and roof assemblies, as shown in Figure 5A, Figure 5B, and Figure 5C, calculation of the concrete pry-out strength in accordance with ACI 318-19 17.7.3, ACI 318-14 17.5.3 or ACI 318-11 D.6.3 is not required.

4.1.8 Requirements for Seismic Design:

4.1.8.1 General: For load combinations including seismic, the design must be performed in accordance with ACI 318-19 17.10, ACI 318-14 17.2.3 or ACI 318-11 D.3.3, as applicable. Modifications to ACI 318-14 17.2.3 shall be applied under Section 1905.1.8 of the 2018 and 2015 IBC. For the 2012 IBC, Section 1905.1.9 shall be omitted.

The anchors comply with ACI 318 (-19 and -14) 2.3 or ACI 318-11 D.1, as applicable, as ductile steel elements and must be designed in accordance with ACI 318-19 17.10.5, 17.10.6, 17.10.7 or 17.10.4 ACI 318-14 17.2.3.4, 17.2.3.5, 17.2.3.6 and 17.2.3.7; or ACI 318-11 D.3.3.4, D.3.3.5, D.3.3.6 and D.3.3.7, as applicable. Strength reduction factors, ϕ, are given in Table 4, Table 5, Table 6, and Table 7 of this report. The anchors may be installed in structures assigned to Seismic Design Categories A through F of the IBC.

4.1.8.2 Seismic Tension: The nominal steel strength and nominal concrete breakout strength for anchors in tension must be calculated in accordance with ACI 318-19 17.6.1 and 17.6.2, ACI 318-14 17.4.1 and 17.4.2 or ACI 318-11 D.5.1 and D.5.2, as applicable, as described in Sections 4.1.2 and 4.1.3 of this report. In accordance with ACI 318-19 17.6.3.2.1, ACI 318-14 17.4.3.2 or ACI 318-11 D.5.3.2, as applicable, the appropriate pullout strength in tension for seismic loads, $N_{p,eq}$, described in Table 4 and Table 5 or $N_{p,deck,cr}$ described in Table 8 must be used in lieu of N_p, as applicable. The value of $N_{p,eq}$ or $N_{p,deck,cr}$ may be adjusted by calculation for concrete strength in accordance with Eq-1 and Section 4.1.4 whereby the value of $N_{p,deck,cr}$ must be substituted for $N_{p,cr}$ and the value of 3,000 psi (20.7 MPa) must be substituted for the value of 2,500 psi (17.2 MPa) in the denominator. If no values for $N_{p,eq}$ or $N_{p,deck,eq}$ are given in Table 4, Table 5, or Table 8, the static design strength values govern.

4.1.8.3 Seismic Shear: The nominal concrete breakout strength and pryout strength in shear must be calculated in accordance with ACI 318-19 17.7.2 and 17.7.3, ACI 318-14 17.5.2 and 17.5.3 or ACI 318-11 D.6.2 and D.6.3, respectively, as applicable, as described in Sections 4.1.6 and 4.1.7 of this report. In accordance with ACI 318-19 17.7.1.2, ACI 318-14 17.5.1.2 or ACI 318-11 D.6.1.2, as applicable, the appropriate value for nominal steel strength for seismic loads, $V_{sa,eq}$ described in Table 6 and Table 7 or $V_{sa,deck,eq}$ described in Table 8 must be used in lieu of V_{sa}, as applicable.

4.1.9 Requirements for Interaction of Tensile and Shear Forces: For anchors or groups of anchors that are subject to the effects of combined tension and shear forces, the design must be performed in accordance with ACI 318-19 17.8, ACI 318-14 17.6 or ACI 318-11 D.7, as applicable.

4.1.10 Requirements for Minimum Member Thickness, Minimum Anchor Spacing and Minimum Edge Distance: In lieu of ACI 318-19 17.9.2, ACI 318-14 17.7.1 and 17.7.3 or ACI 318-11 D.8.1 and D.8.3, respectively, as applicable, values of s_{min} and c_{min} as given in Table 3 of this report must be used. In lieu of ACI 318-19 17.9.4, ACI 318-14 17.7.5 or ACI 318-11 D.8.5, as applicable, minimum member thicknesses h_{min} as given in Tables 3 and 4 of this report must be used. Additional combinations for minimum edge distance, c_{min}, and spacing, s_{min}, may be derived by linear interpolation between the given boundary values as described in Figure 4.

For carbon steel KB-TZ2 anchors installed in the topside of sand-lightweight or normal-weight concrete over profile steel deck floor and roof assemblies, the anchors must be installed in accordance with Table 9 and Figure 5D.

For carbon steel KB-TZ2 anchors installed in the soffit of sand-lightweight or normal-weight concrete over profile steel deck floor and roof assemblies, the anchors must be installed in accordance with Figure 5A, Figure 5B and Figure 5C and shall have an axial spacing along the flute equal to the greater of $3h_{ef}$ or 1.5 times the flute width.

4.1.11 Requirements for Critical Edge Distance: In applications where $c < c_{ac}$ and supplemental reinforcement to control splitting of the concrete is not present, the concrete breakout strength in tension for uncracked concrete, calculated in accordance with ACI 318-19 17.6.2, ACI 318-14 17.4.2 or ACI 318-11 D.5.2, as applicable, must be further multiplied by the factor $\Psi_{cp,N}$ as given by Eq-3:

$$\Psi_{cp,N} = \frac{c}{c_{ac}} \qquad \text{(Eq-3)}$$

whereby the factor $\Psi_{cp,N}$ need not be taken as less than $\frac{1.5 h_{ef}}{c_{ac}}$. For all other cases, $\Psi_{cp,N}$ = 1.0. In lieu of using ACI 318-19 17.9.5, ACI 318-14 17.7.6 or ACI 318-11 D.8.6, as applicable, values of c_{ac} must comply with Table 4 or Table 5.

4.1.12 Lightweight Concrete: For the use of anchors in lightweight concrete, the modification factor λ_a equal to 0.8λ is applied to all values of $\sqrt{f'_c}$ affecting N_n and V_n.

For ACI 318-19 (2021 IBC), ACI 318-14 (2018 and 2015 IBC) and ACI 318-11 (2012 IBC), λ shall be determined in accordance with the corresponding version of ACI 318.

For anchors installed in the soffit of sand-lightweight concrete-filled steel deck and floor and roof assemblies, further reduction of the pullout values provided in this report is not required.

4.2 Allowable Stress Design (ASD):

4.2.1 General: Design values for use with allowable stress design (working stress design) load combinations calculated in accordance with Section 1605.1 of the 2021 IBC or Section 1605.3 of the 2018, 2015 and 2012 IBC, must be established as follows:

$$T_{allowable,ASD} = \frac{\phi N_n}{\alpha}$$

$$V_{allowable,ASD} = \frac{\phi V_n}{\alpha}$$

where:

$T_{allowable,ASD}$ = Allowable tension load (lbf or kN).

$V_{allowable,ASD}$ = Allowable shear load (lbf or kN).

ϕN_n = Lowest design strength of an anchor or anchor group in tension as determined in accordance with ACI 318 (-19 and -14) Chapter 17 and 2021, 2018 and 2015 IBC Section 1905.1.8, ACI 318-11 Appendix D, and Section 4.1 of this report, as applicable (lbf or N). For 2012 IBC, Section 1905.1.9 shall be omitted.

ϕV_n = Lowest design strength of an anchor or anchor group in shear as determined in accordance with ACI 318 (-19 and -14) Chapter 17 and 2021, 2018 and 2015 IBC Section 1905.1.8, ACI 318-11 Appendix D, and Section 4.1 of this report, as applicable (lbf or N). For 2012 IBC, Section 1905.1.9 shall be omitted.

α = Conversion factor calculated as a weighted average of the load factors for the controlling load combination. In addition, α must include all applicable factors to account for nonductile failure modes and required over-strength.

The requirements for member thickness, edge distance and spacing, described in this report, must apply.

4.2.2 Interaction of Tensile and Shear Forces: The interaction must be calculated and consistent with ACI 318-19 17.8, ACI 318-14 17.6 or ACI 318-11 D.7, as applicable, as follows:

For shear loads $V_{applied} \leq 0.2V_{allowable,ASD}$, the full allowable load in tension is permitted.

For tension loads $T_{applied} \leq 0.2T_{allowable,ASD}$, the full allowable load in shear is permitted.

For all other cases:

$$\frac{T_{applied}}{T_{allowable,ASD}} + \frac{V_{applied}}{V_{allowable,ASD}} \leq 1.2 \qquad \text{(Eq-4)}$$

4.3 Installation:

Installation parameters are provided in Table 1 and Figure 2, Figure 5A, Figure 5B, Figure 5C and Figure 5D. Anchor locations must comply with this report and plans and specifications approved by the code official. The Hilti KB-TZ2 must be installed in accordance with manufacturer's published instructions and this report. In case of conflict, this report governs. Anchors must be installed in holes drilled into the concrete using carbide-tipped masonry drill bits complying with ANSI B212.15-1994, using the Hilti SafeSet System™ with Hilti TE-YD or TE-CD Hollow Drill Bits complying with ANSI B212.15-1994 with a Hilti vacuum in accordance with Figure 6 and Figure 7, or using Hilti SPX-T core bits in accordance with Figure 7. The Hollow Drill Bits are not permitted for use with the 1/4-inch- and 3/8-inch diameter KB-TZ2 anchors. The Hilti SPX-T core bits are not permitted for use with the 1/4-inch-diameter KB-TZ2 anchors. The minimum drilled hole depth, h_o, is given in Table 1. If dust and debris is removed from the drilled hole with the Hilti TE-YD or TE-CD Hollow Drill Bits, the DRS attachment system, or compressed air or a manual pump, h_{nom} is achieved at the specified value of h_o noted in Table 1. The anchor must be hammered into the predrilled hole until h_{nom} is achieved. The nut must be tightened against the washer until the torque values specified in Table 1 are achieved, or the anchors may be installed using the Hilti AT Tool in accordance with Figure 7. The Hilti AT Tool is not permitted for use with the 1/4-inch- and 3/4-inch- diameter KB-TZ2 anchors. For installation in the soffit of concrete on steel deck assemblies, the hole diameter in the steel deck must not exceed the diameter of the hole in the concrete by more than 1/8 inch (3.2 mm). For member thickness and edge distance restrictions for installations into the soffit of concrete on steel deck assemblies, see Figure 5A, Figure 5B, and Figure 5C.

4.4 Special Inspection:

Periodic special inspection is required in accordance with Section 1705.1.1 and Table 1705.3 of the 2021, 2018, 2015 and 2012 IBC, as applicable. The special inspector must make periodic inspections during anchor installation to verify anchor type, anchor dimensions, concrete type, concrete compressive strength, anchor spacing, edge distances, concrete member thickness, tightening torque, hole dimensions, anchor embedment and adherence to the manufacturer's printed installation instructions. The special inspector must be present as often as required in accordance with the "statement of special inspection." Under the IBC, additional requirements as set forth in Sections 1705, 1706 and 1707 must be observed, where applicable.

5.0 CONDITIONS OF USE

The Hilti KB-TZ2 anchors described in this report comply with the codes listed in Section 1.0 of this report, subject to the following conditions:

5.1 Anchor sizes, dimensions, minimum embedment depths and other installation parameters as set forth in this report.

5.2 The anchors must be installed in accordance with the manufacturer's published instructions and this report. In case of conflict, this report governs.

5.3 Anchors must be limited to use in cracked and uncracked normal-weight concrete and lightweight concrete having a specified compressive strength, f'_c, of 2,500 psi to 8,500 psi (17.2 MPa to 58.6 MPa), and cracked and uncracked normal-weight or sand-lightweight concrete over metal deck having a specified compressive strength, f'_c, of 3,000 psi to 8,500 psi (20.7 MPa to 58.6 MPa).

5.4 The values of f'_c used for calculation purposes must not exceed 8,000 psi (55.1 MPa).

5.5 The concrete shall have attained its minimum design strength prior to installation of the anchors and must have a minimum age of 21 days.

5.6 Strength design values must be established in accordance with Section 4.1 of this report.

5.7 Allowable design values are established in accordance with Section 4.2.

5.8 Anchor spacing and edge distance as well as minimum member thickness must comply with Tables 3 and 9, and Figure 5A, Figure 5B, Figure 5C and Figure 5D.

5.9 Prior to installation, calculations and details demonstrating compliance with this report must be submitted to the code official. The calculations and details must be prepared by a registered design professional where required by the statutes of the jurisdiction in which the project is to be constructed.

5.10 Since an ICC-ES acceptance criteria for evaluating data to determine the performance of expansion anchors subjected to fatigue or shock loading is unavailable at this time, the use of these anchors under such conditions is beyond the scope of this report.

5.11 Anchors may be installed in regions of concrete where cracking has occurred or where analysis indicates cracking may occur ($f_t > f_r$), subject to the conditions of this report.

5.12 Anchors may be used to resist short-term loading due to wind or seismic forces in locations designated as Seismic Design Categories A through F of the IBC, subject to the conditions of this report.

5.13 Where not otherwise prohibited in the code, KB-TZ2 anchors are permitted for use with fire-resistance-rated construction provided that at least one of the following conditions is fulfilled:

- Anchors are used to resist wind or seismic forces only.
- Anchors that support a fire-resistance-rated envelope or a fire-resistance-rated membrane are protected by approved fire-resistance-rated materials, or have been evaluated for resistance to fire exposure in accordance with recognized standards.
- Anchors are used to support nonstructural elements.

5.14 Use of zinc-coated carbon steel anchors is limited to dry, interior locations.

5.15 Use of anchors made of stainless steel as specified in this report are permitted for exterior exposure and damp environments.

5.16 Use of anchors made of stainless steel as specified in this report are permitted for contact with preservative-treated and fire-retardant-treated wood.

5.17 Anchors are manufactured by Hilti AG under an approved quality-control program with inspections by ICC-ES.

5.18 Special inspection must be provided in accordance with Section 4.4.

6.0 EVIDENCE SUBMITTED

6.1 Data in accordance with the ICC-ES Acceptance Criteria for Mechanical Anchors in Concrete Elements (AC193), dated October 2017, (editorially revised December 2020), which incorporates requirements in ACI 355.2-19 and ACI 355.2-07 for use in cracked and uncracked concrete.

6.2 Quality-control documentation.

7.0 IDENTIFICATION

7.1 The anchors are identified by packaging labeled with the manufacturer's name (Hilti, Inc.) and contact information, anchor name, anchor size, and evaluation report number (ESR-4266). The anchors have the letters KB-TZ2 embossed on the anchor stud and a notch or notches embossed into the anchor head. The letters and notches are visible after installation for verification as depicted in Figure 3 of this report. The number of notches indicate material type. The letter system indicating length embossed on the head of the anchor is described in Table 2.

7.2 The report holder's contact information is the following:

HILTI, INC.
7250 DALLAS PARKWAY, SUITE 1000
PLANO, TEXAS 75024
(918) 872-8000
www.hilti.com

TABLE 1—SETTING INFORMATION

Setting information	Sym.	Units	Nominal anchor diameter (in.)													
			1/4	3/8			1/2				5/8			3/4		
Nominal bit diameter	d_o	In.	1/4	3/8			1/2				5/8			3/4		
Effective min. embedment	h_{ef}	In.	1-1/2	1-1/2	2	2-1/2	1-1/2 [1]	2	2-1/2	3-1/4	2-3/4	3-1/4	4	3-1/4	3-3/4	4-3/4
		(mm)	(38)	(38)	(51)	(64)	(38)	(51)	(64)	(83)	(70)	(83)	(102)	(83)	(95)	(121)
Nominal embedment	h_{nom}	in.	1-3/4	1-7/8	2-1/2	3	2 [1]	2-1/2	3	3-3/4	3-1/4	3-3/4	4-1/2	4	4-1/2	5-1/2
		(mm)	(44)	(48)	(64)	(76)	(51)	(64)	(76)	(95)	(83)	(95)	(114)	(102)	(114)	(140)
Min. hole depth	h_o	In.	2	2	2-3/4	3-1/4	2-1/4 [1]	2-3/4	3-1/4	4-1/4	3-3/4	4-1/4	4-3/4	4-1/4	4-3/4	5-3/4
		(mm)	(51)	(51)	(70)	(83)	(57)	(70)	(83)	(108)	(95)	(108)	(121)	(108)	(121)	(146)
Installation torque Carbon steel[1]	T_{inst}	ft-lb	4	30			50				40			110		
		(Nm)	(5)	(41)			(68)				(54)			(149)		
Installation torque Stainless steel[1]	T_{inst}	ft-lb	6	30			40				60			125		
		(Nm)	(8)	(41)			(54)				(81)			(169)		
Fixture hole diameter	d_h	In.	5/16	7/16			9/16				11/16			13/16		
		(mm)	(7.9)	(11.1)			(14.3)				(17.5)			(20.6)		

[1] Design information for h_{ef} = 1-1/2 is only applicable to carbon steel (CS) KB-TZ2 bolts.

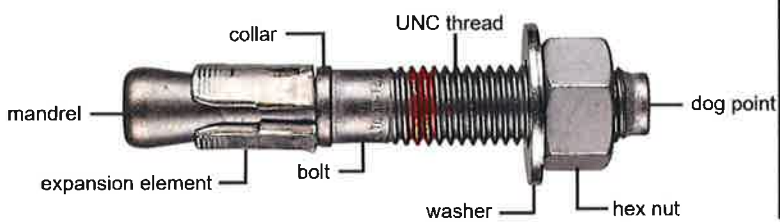


FIGURE 1—HILTI CARBON STEEL KWIK BOLT TZ (KB-TZ2)

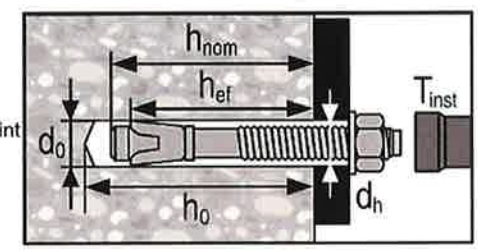


FIGURE 2—HILTI KB-TZ2 INSTALLED

TABLE 2—LENGTH IDENTIFICATION SYSTEM (CARBON STEEL AND STAINLESS STEEL ANCHORS)

Length ID marking on bolt head		A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T	U	V	W
Length of anchor, ℓ_{anch} (inches)	From	1½	2	2½	3	3½	4	4½	5	5½	6	6½	7	7½	8	8½	9	9½	10	11	12	13	14	15
	Up to but not including	2	2½	3	3½	4	4½	5	5½	6	6½	7	7½	8	8½	9	9½	10	11	12	13	14	15	16

For **SI:** 1 inch = 25.4 mm.

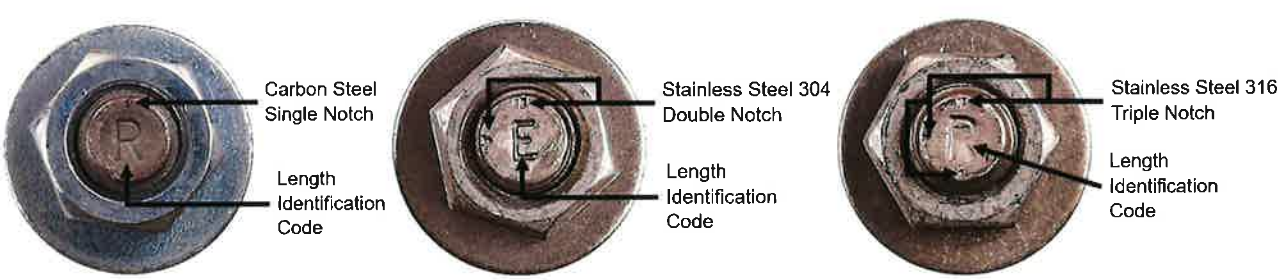


FIGURE 3—BOLT HEAD WITH LENGTH IDENTIFICATION CODE AND KB-TZ2 HEAD NOTCH EMBOSSMENT

TABLE 3—MINIMUM EDGE DISTANCE, SPACING AND CONCRETE THICKNESS FOR KB-TZ2

Setting information	Symbol	Units	Nominal anchor dia. (in.)													
			1/4	3/8			1/2				5/8			3/4		
Effective min. embedment	h_{ef}	in.	1-1/2	1-1/2	2	2-1/2	1-1/2	2	2-1/2	3-1/4	2-3/4	3-1/4	4	3-1/4	3-3/4	4-3/4
		(mm)	(38)	(38)	(51)	(64)	(38)	(51)	(64)	(83)	(70)	(83)	(102)	(83)	(95)	(121)
Min. member thickness	h_{min}	in.	3-1/4	3-1/4	4	5	3-1/2	4	5	5-1/2	5	5-1/2	6	5-1/2	6	8
		(mm)	(83)	(83)	(102)	(127)	(89)	(102)	(127)	(140)	(127)	(140)	(152)	(140)	(152)	(203)
Carbon Steel																
Min. edge distance	c_{min}	in.	1-1/2	5	2-1/2	2-1/2	8	2-3/4	2-3/4	2-1/4	4-1/2	3-1/2	2-3/4	5	4	3-1/2
		(mm)	(38)	(127)	(64)	(64)	(203)	(70)	(70)	(57)	(114)	(89)	(70)	(127)	(102)	(89)
	for $s \geq$	in.	1-1/2	8	6	5	12	5-1/2	9-3/4	5-1/4	6-1/2	5-1/2	7-1/4	10	5-3/4	5-1/2
		(mm)	(38)	(203)	(152)	(127)	(305)	(140)	(248)	(133)	(165)	(140)	(184)	(254)	(146)	(140)
Min. anchor spacing	s_{min}	in.	1-1/2	5	2-1/4	2	12	3-1/2	3	2	4-1/2	2-3/4	2-1/4	4-1/2	3-3/4	3-3/4
		(mm)	(38)	(127)	(57)	(51)	(305)	(89)	(76)	(51)	(114)	(70)	(57)	(114)	(95)	(95)
	for $c \geq$	In.	1-1/2	8	3-1/2	4	8	10	8	4-3/4	5-1/2	7	4-1/4	6	7-1/2	4-3/4
		(mm)	(38)	(203)	(89)	(102)	(203)	(254)	(203)	(121)	(140)	(178)	(108)	(152)	(191)	(121)
Stainless Steel																
Min. edge distance	c_{min}	in.	1-1/2	5	2-1/2	2-1/2		2-3/4	2-1/2	2-1/4	4	3-1/4	2-1/4	5	4	3-3/4
		(mm)	(38)	(127)	(64)	(64)		(70)	(64)	(57)	(102)	(83)	(57)	(127)	(102)	(95)
	for $s \geq$	in.	1-1/2	8	5	5		5-1/2	4-1/2	5-1/4	7	5-1/2	7	11	7-1/2	5-3/4
		(mm)	(38)	(203)	(127)	(127)		(140)	(114)	(133)	(178)	(140)	(178)	(279)	(191)	(146)
Min. anchor spacing	s_{min}	in.	1-1/2	5	2-1/4	2-1/4		2-3/4	2-1/2	2	5-1/2	2-3/4	3	5	4	4
		(mm)	(38)	(127)	(57)	(57)		(70)	(64)	(51)	(140)	(70)	(76)	(127)	(102)	(102)
	for $c \geq$	In.	1-1/2	8	4	3-1/2		4-1/8	5	4-3/4	5-1/2	4	4-1/4	8	6	5-1/4
		(mm)	(38)	(203)	(102)	(89)		(105)	(127)	(121)	(140)	(102)	(108)	(203)	(152)	(133)

For **SI**: 1 inch = 25.4 mm

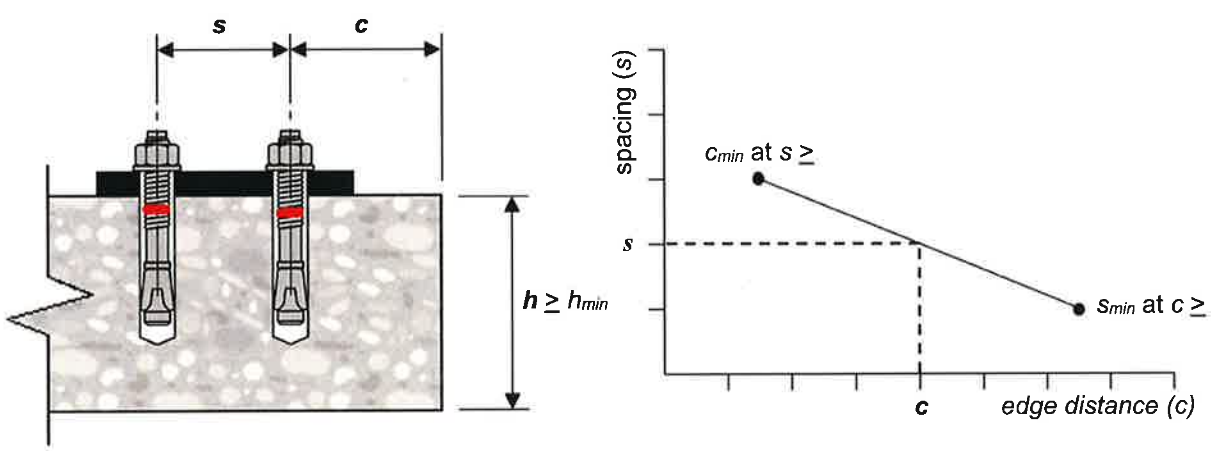


FIGURE 4—INTERPOLATION OF MINIMUM EDGE DISTANCE AND ANCHOR SPACING

TABLE 4—HILTI CARBON STEEL KB-TZ2 DESIGN INFORMATION FOR HAMMER AND CORE DRILLED INSTALLATIONS, TENSION

Design parameter	Symbol	Units	Nominal anchor diameter (in)													
			1/4	3/8			1/2				5/8			3/4		
Effective min. embedment[1]	h_{ef}	in. (mm)	1-1/2 (38)	1-1/2 (38)	2 (51)	2-1/2 (64)	1-1/2 (38)	2 (51)	2-1/2 (64)	3-1/4 (83)	2-3/4 (70)	3-1/4 (83)	4 (102)	3-1/4 (83)	3-3/4 (95)	4-3/4 (121)
Tension, steel failure modes																
Strength reduction factor for steel in tension[2]	$\Phi_{sa,N}$	-	0.75	0.75			0.75				0.75			0.75		
Min. specified yield strength	f_y	lb/in² (N/mm²)	100,900 (696)	100,900 (696)			96,300 (664)				87,000 (600)			84,700 (584)		
Min. specified ult. strength	f_{uta}	lb/in² (N/mm²)	122,400 (844)	126,200 (870)			114,000 (786)				106,700 (736)			105,900 (730)		
Effective tensile stress area	$A_{se,N}$	in² (mm²)	0.024 (15.4)	0.051 (33.2)			0.099 (63.6)				0.164 (106.0)			0.239 (154.4)		
Steel strength in tension	N_{sa}	lb (kN)	2,920 (13.0)	6,490 (28.9)			11,240 (50.0)				17,535 (78.0)			25,335 (112.7)		
Tension, concrete failure modes																
Anchor category	-	-	3	1			1				1			1		
Strength reduction factor for concrete and pullout failure in tension, (Condition B – supplementary reinforcement not present)[3,7]	$\Phi_{c,N}$, $\Phi_{p,N}$	-	0.45	0.65			0.65				0.65			0.65		
Effectiveness factor for uncracked concrete	k_{uncr}	-	24	24			27		24		24			27	27[6]	24
Effectiveness factor for cracked concrete[6]	k_{cr}	-	17	21		17	24	21		17	21		17	21		
Modification factor for anchor resistance, tension, uncracked concrete[4]	$\Psi_{c,N}$	-	1.0	1.0			1.0				1.0			1.0		
Critical edge distance	c_{ac}	in. (mm)	4 (102)	5 (127)	4-3/8 (111)	5-1/2 (140)	8 (203)	5-1/2 (140)	6-3/4 (171)	10 (254)	10 (254)	11-1/2 (292)	8-3/4 (222)	12 (305)	10 (254)	9 (229)
Pullout strength uncracked conc.[5]	$N_{p,uncr}$	lb (kN)	2,100 (9.3)	N/A	N/A	4,180 (18.6)	N/A	N/A	N/A	N/A	5,380 (23.9)	N/A	8,995 (40.0)	N/A	N/A	N/A
Pullout strength cracked conc. [5]	$N_{p,cr}$	lb (kN)	625 (2.8)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	8,835 (39.3)
Pullout strength seismic[5]	$N_{p,eq}$	lb (kN)	625 (2.8)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	8,700 (38.7)
Normalization factor, uncracked concrete	n_{uncr}	-	0.20	0.22	0.24	0.35	0.50	0.42	0.29	0.35	0.50	0.48	0.50	0.35	0.31	0.39
Normalization factor, cracked concrete, seismic	n_{cr}	-	0.39	0.50	0.46	0.28	0.47	0.50	0.48	0.40	0.50	0.47	0.50	0.36	0.42	0.29
Tension, axial stiffness																
Axial stiffness in service load range	β_{uncr}	lb/in.	322,360	131,570			158,585				290,360			412,335		
	β_{cr}	lb/in.	31,035	91,335			113,515				167,365			62,180		

For SI: 1 inch = 25.4 mm, 1 lbf = 4.45 N, 1 psi = 0.006895 MPa. For pound-inch units: 1 mm = 0.03937 inches.

[1] Figure 2 of this report illustrates the installation parameters.
[2] The KB-TZ2 is considered a ductile steel element in accordance with ACI 318 (-19 and -14) 2.3 or ACI 318-11 D.1.
[3] For use with the load combinations of ACI 318 (-19 and -14) Section 5.3, ACI 318-11 Section 9.2 or 2021 IBC Section 1605.1 or 2018, 2015, 2012 IBC Section 1605.2. Condition B (supplementary reinforcement not present) applies where supplementary reinforcement in conformance with ACI 318-19 Table 17.5.3(b) or (c), ACI 318-14 section 17.3.3 (c) or ACI 318-11 Section 4.3 (c) is not provided, or where pryout strength governs. For cases where the presence of supplementary reinforcement can be verified, the resistance modification factors associated with Condition A (supplementary reinforcement present) for concrete breakout failure may be used.
[4] For all design cases, $\Psi_{c,N}$ = 1.0. The appropriate effectiveness factor for cracked concrete (k_{cr}) or uncracked concrete (k_{uncr}) must be used.
[5] For all design cases, $\Psi_{c,P}$ = 1.0. Tabular value for pullout strength is for a concrete compressive strength of 2,500 psi (17.2 MPa). Pullout strength for concrete compressive strength greater than 2,500 psi (17.2 MPa) may be increased by multiplying the tabular pullout strength by $(f'c / 2{,}500)^n$ for psi, or $(f'c / 17.2)^n$ for MPa, where n is given as n_{uncr} for uncracked concrete and n_{cr} for cracked concrete and seismic. NA (not applicable) denotes that pullout strength does not need to be considered for design.
[6] For core drill installations, k_{uncr} = 24 for 3/4-inch diameter anchors installed at 3-3/4 inches (95 mm) effective embedment.
[7] The supplementary reinforcement classifications "Condition A" and "Condition B" have been replaced by ACI 318-19 Table 17.5.3 (c).

TABLE 5—HILTI STAINLESS STEEL KB-TZ2 DESIGN INFORMATION FOR HAMMER AND CORE DRILLED INSTALLATIONS, TENSION

Design parameter	Symbol	Units	Nominal anchor diameter (in)												
			$^{1}/_{4}$	$^{3}/_{8}$			$^{1}/_{2}$			$^{5}/_{8}$			$^{3}/_{4}$		
Effective min. embedment [1]	h_{ef}	in. (mm)	1-1/2 (38)	1-1/2 (38)	2 (51)	2-1/2 (64)	2 (51)	2-1/2 (64)	3-1/4 (83)	2-3/4 (70)	3-1/4 (83)	4 (102)	3-1/4 (83)	3-3/4 (95)	4-3/4 (121)
Tension, steel failure modes															
Strength reduction factor for steel in tension [2]	$\Phi_{sa,N}$	-	0.75	0.75			0.75			0.75			0.75		
Min. specified yield strength	f_y	lb/in² (N/mm²)	100,900 (696)	96,300 (664)			96,300 (664)			91,600 (632)			84,100 (580)		
Min. specified ult. strength	f_{uta}	lb/in² (N/mm²)	122,400 (844)	120,100 (828)			120,400 (830)			114,600 (790)			100,500 (693)		
Effective tensile stress area	$A_{se,N}$	In² (mm²)	0.024 (15.4)	0.051 (33.2)			0.099 (63.6)			0.164 (106.0)			0.239 (154.4)		
Steel strength in tension	N_{sa}	lb (kN)	2,920 (13.0)	6,180 (27.5)			11,870 (52.8)			18,835 (83.8)			24,045 (107.0)		
Tension, concrete failure modes															
Anchor category	-	-	3	1			1			1			1		
Strength reduction factor for concrete and pullout failure in tension, (Condition B – supplementary reinforcement not present)[3,8]	$\Phi_{c,N}$, $\Phi_{p,N}$	-	0.45	0.65			0.65			0.65			0.65		
Effectiveness factor for uncracked concrete	k_{uncr}	-	24	24			24			24			24	27[6]	24
Effectiveness factor for cracked concrete	k_{cr}	-	17	21		17	17	21	17	21		17	21	21[6]	21
Modification factor for anchor resistance, tension, uncracked concrete [4]	$\Psi_{c,N}$	-	1.0	1.0			1.0			1.0			1.0		
Critical edge distance	c_{ac}	in. (mm)	4 (102)	4-1/2 (114)	5-1/2 (140)	4-1/8 (105)	5-1/2 (140)	6-1/4 (159)	7-1/2 (191)	10 (254)	6-1/2 (165)	8-3/4 (222)	12 (305)	10 (254)	10 (254)
Pullout strength uncracked concrete [5]	$N_{p,uncr}$	lb (kN)	1,570 (7.0)	N/A	N/A	4,185 (18.6)	3,380 (15.0)	4,010 (17.8)	5,500 (24.5)	4,085 (18.2)	6,015 (26.8)	8,050 (35.8)	N/A	N/A	N/A
Pullout strength cracked concrete [5]	$N_{p,cr}$	lb (kN)	670 (3.0)	N/A	N/A	N/A	N/A	N/A	N/A[7]	N/A	N/A	N/A	N/A	N/A	8,795 (39.1)
Pullout strength seismic [5]	$N_{p,eq}$	lb (kN)	670 (3.0)	N/A	N/A	N/A	N/A	N/A	N/A[7]	N/A	N/A	N/A	N/A	N/A	8,795 (39.1)
Normalization factor, uncracked concrete	n_{uncr}	-	0.39	N/A	N/A	0.37	0.46	0.50	0.50	0.50	0.42	0.47	N/A	N/A	N/A
Normalization factor, cracked concrete, seismic	n_{cr}	-	0.50	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	0.50
Tension, axial stiffness															
Axial stiffness in service load range	β_{uncr}	lb/in.	166,490	175,800			137,145			153,925			342,680		
	β_{cr}	lb/in.	33,805	79,860			97,985			69,625			75,715		

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.45 N, 1 psi = 0.006895 MPa For pound-inch units: 1 mm = 0.03937 inches.

[1] Figure 2 of this report illustrates the installation parameters.
[2] The KB-TZ2 is considered a ductile steel element in accordance with ACI 318 (-19 and -14) 2.3 or ACI 318-11 D.1.
[3] For use with the load combinations of ACI 318 (-19 and -14) Section 5.3, ACI 318-11 Section 9.2 or 2021 IBC Section 1605.1 or 2018, 2015, 2012 IBC Section 1605.2. Condition B (supplementary reinforcement not present) applies where supplementary reinforcement in conformance with ACI 318-19 Table 17.5.3(b) or (c), ACI 318-14 section 17.3.3 (c) or ACI 318-11 Section 4.3 (c) is not provided, or where pryout strength governs. For cases where the presence of supplementary reinforcement can be verified, the resistance modification factors associated with Condition A (supplementary reinforcement present) for concrete breakout failure may be used.
[4] For all design cases, $\Psi_{c,N}$ = 1.0. The appropriate effectiveness factor for cracked concrete (k_{cr}) or uncracked concrete (k_{uncr}) must be used.
[5] For all design cases, $\Psi_{c,P}$ = 1.0. Tabular value for pullout strength is for a concrete compressive strength of 2,500 psi (17.2 MPa). Pullout strength for concrete compressive strength greater than 2,500 psi (17.2 MPa) may be increased by multiplying the tabular pullout strength by $(f'c / 2{,}500)^n$ for psi, or $(f'c / 17.2)^n$ for MPa, where n is given as n_{uncr} for uncracked concrete and n_{cr} for cracked concrete.NA (not applicable) denotes that pullout strength does not need to be considered for design.
[6] For core drill installations, k_{uncr} = 24 and k_{cr} = 17 for $^{3}/_{4}$-inch diameter anchors installed at $3^{3}/_{4}$ inches (95 mm) effective embedment.
[7] For core drill installations, $N_{p,cr}$ = 4245 lb (18.9 kN) and $N_{p,eq}$ = 4245lb (18.9 kN) for $^{1}/_{2}$-inch diameter anchors installed at $3^{3}/_{4}$ inches (95 mm) effective embedment.
[8] The supplementary reinforcement classifications "Condition A" and "Condition B" have been replaced by ACI 318-19 Table 17.5.3 (c).

TABLE 6—HILTI CARBON STEEL KB-TZ2 DESIGN INFORMATION FOR HAMMER AND CORE DRILLED INSTALLATIONS, SHEAR

Design parameter	Symbol	Units	Nominal anchor diameter (in)													
			1/4	3/8			1/2				5/8			3/4		
Anchor O.D.	d_a	in.	0.250	0.375			0.500				0.625			0.750		
		(mm)	(6.4)	(9.5)			(12.7)				(15.9)			(19.1)		
Effective min. embedment [1]	h_{ef}	in.	1-1/2	1-1/2	2	2-1/2	1-1/2	2	2-1/2	3-1/4	2-3/4	3-1/4	4	3-1/4	3-3/4	4-3/4
		(mm)	(38)	(38)	(51)	(64)	(38)	(51)	(64)	(83)	(70)	(83)	(102)	(83)	(95)	(121)
Shear, steel failure modes																
Strength reduction factor for steel in shear [2]	$\Phi_{sa,V}$	-	0.65	0.65			0.65				0.65			0.65		
Steel strength in shear	V_{sa}	lb	1,345	3,225	3,385		5,535		6,875		10,255			13,805		
		(kN)	(6.0)	(14.4)	(15.1)		(24.6)		(30.6)		(45.6)			(61.4)		
Steel strength in shear, seismic	$V_{sa,eq}$	lb	1,345	3,225	3,385		5,535		6,875		10,255			13,805		
		(kN)	(6.0)	(14.4)	(15.1)		(24.6)		(30.6)		(45.6)			(61.4)		
Shear, concrete failure modes																
Strength reduction factor for concrete breakout and pryout failure in shear, (Condition B – supplementary reinforcement not present) [3,4]	$\Phi_{c,V}$, $\Phi_{p,V}$	-	0.70	0.70			0.70				0.70			0.70		
Load bearing length of anchor in shear	l_e	in.	1-1/2	1-1/2	2	2-1/2	1-1/2	2	2-1/2	3-1/4	2-3/4	3-1/4	4	3-1/4	3-3/4	4-3/4
		(mm)	(38)	(38)	(51)	(64)	(38)	(51)	(64)	(83)	(70)	(83)	(102)	(83)	(95)	(121)
Coefficient for pryout strength	k_{cp}	-	1	1	1	2	1	1	2	2	2	2	2	2	2	2

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.45 N, 1 psi = 0.006895 MPa For pound-inch units: 1 mm = 0.03937 inches.

[1] Figure 2 of this report illustrates the installation parameters.
[2] The KB-TZ2 is considered a ductile steel element in accordance with ACI 318 (-19 and -14) 2.3 or ACI 318-11 D.1.
[3] For use with the load combinations of ACI 318 (-19 and -14) Section 5.3, ACI 318-11 Section 9.2 or 2021 IBC Section 1605.1 or 2018, 2015, 2012 IBC Section 1605.2. Condition B (supplementary reinforcement not present) applies where supplementary reinforcement in conformance with ACI 318-19 Table 17.5.3(b) or (c), ACI 318-14 section 17.3.3 (c) or ACI 318-11 Section 4.3 (c) is not provided, or where pryout strength governs. For cases where the presence of supplementary reinforcement can be verified, the resistance modification factors associated with Condition A (supplementary reinforcement present) for concrete breakout failure may be used.
[4]The supplementary reinforcement classifications "Condition A" and "Condition B" have been replaced by ACI 318-19 Table 17.5.3 (c).

TABLE 7—HILTI STAINLESS STEEL KB-TZ2 DESIGN INFORMATION FOR HAMMER AND CORE DRILLED INSTALLATIONS, SHEAR

Design parameter	Symbol	Units	Nominal anchor diameter												
			1/4	3/8			1/2			5/8			3/4		
Anchor O.D.	d_a	in.	0.250	0.375			0.500			0.625			0.750		
		(mm)	(6.4)	(9.5)			(12.7)			(15.9)			(19.1)		
Effective min. embedment [1]	h_{ef}	in.	1-1/2	1-1/2	2	2-1/2	2	2-1/2	3-1/4	2-3/4	3-1/4	4	3-1/4	3-3/4	4-3/4
		(mm)	(38)	(38)	(51)	(64)	(51)	(64)	(83)	(70)	(83)	(102)	(83)	(95)	(121)
Shear, steel failure modes															
Strength reduction factor for steel in shear [2]	$\Phi_{sa,V}$	-	0.65	0.65			0.65			0.65			0.65		
Steel strength in shear	V_{sa}	lb	1,460	4,615	4,885		8,345			12,355			16,560		
		(kN)	(6.5)	(20.5)	(21.7)		(37.1)			(55.0)			(73.7)		
Steel strength in shear, seismic	$V_{sa,eq}$	lb	1,110	4,615	4,885		8,345			12,355			13,470		
		(kN)	(4.9)	(20.5)	(21.7)		(37.1)			(55.0)			(59.9)		
Shear, concrete failure modes															
Strength reduction factor for concrete breakout and pryout failure in shear, (Condition B– supplementary reinforcement not present) [3,4]	$\Phi_{c,V}$, $\Phi_{p,V}$	-	0.7	0.7			0.7			0.7			0.7		
Load bearing length of anchor in shear	l_e	in.	1-1/2	1-1/2	2	2-1/2	2	2-1/2	3-1/4	2-3/4	3-1/4	4	3-1/4	3-3/4	4-3/4
		(mm)	(38)	(38)	(51)	(64)	(51)	(64)	(83)	(70)	(83)	(102)	(83)	(95)	(121)
Coefficient for pryout strength	k_{cp}	-	1	1	1	2	1	2	2	2	2	2	2	2	2

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.45 N, 1 psi = 0.006895 MPa For pound-inch units: 1 mm = 0.03937 inches.

[1] Figure 2 of this report illustrates the installation parameters.
[2] The KB-TZ2 is considered a ductile steel element in accordance with ACI 318 (-19 and -14) 2.3 or ACI 318-11 D.1.
[3] For use with the load combinations of ACI 318 (-19 and -14) Section 5.3, ACI 318-11 Section 9.2 or 2021 IBC Section 1605.1 or 2018, 2015, 2012 IBC Section 1605.2. Condition B (supplementary reinforcement not present) applies where supplementary reinforcement in conformance with ACI 318-19 Table 17.5.3(b) or (c), ACI 318-14 section 17.3.3 (c) or ACI 318-11 Section 4.3 (c) is not provided, or where pryout strength governs. For cases where the presence of supplementary reinforcement can be verified, the resistance modification factors associated with Condition A (supplementary reinforcement present) for concrete breakout failure may be used.
[4]The supplementary reinforcement classifications "Condition A" and "Condition B" have been replaced by ACI 318-19 Table 17.5.3 (c).

TABLE 8—HILTI KB-TZ2 CARBON STEEL ANCHORS TENSION AND SHEAR DESIGN DATA FOR INSTALLATION IN THE SOFFIT OF 3000 PSI, LIGHTWEIGHT CONCRETE-FILLED PROFILE STEEL DECK ASSEMBLIES FOR HAMMER AND CORE DRILLED INSTALLATIONS [1,2,3]

Design parameter	Symbol	Units	Anchor Diameter											
			1/4	3/8			1/2				5/8		3/4	
Effective min. embedment [1]	h_{ef}	in.	1-1/2	1-1/2	2	2-1/2	1-1/2	2	2-1/2	3-1/4	2-3/4	4	3-1/4	3-3/4[9]
Minimum hole depth	h_o	in.	2	2	2-3/4	3-1/4	2-1/4	2-3/4	3-1/4	4-1/4	3-3/4	4-3/4	4-1/4	4-3/4
Loads According to Figure 5A														
Minimum concrete thickness over upper flute [4]	$h_{min,deck}$	in.	2-1/2	2-1/2			2-1/2				2-1/2		2-1/2	3-1/4
Pullout strength, uncracked concrete [5,6]	$N_{p,deck,uncr}$	lb	1,725	1,855	2,625	2,995	1,855	2,750	3,745	4,715	4,415	5,815	3,800	4,795
Pullout strength, cracked concrete [5,6]	$N_{p,deck,cr}$	lb	515	1,625	2,295	2,405	1,650	2,135	3,275	3,340	3,930	4,395	3,325	3,730
Pullout strength, seismic [5,7]	$N_{p,deck,eq}$	lb	515	1,625	2,295	2,405	1,650	2,135	3,275	3,340	3,930	4,395	3,325	3,730
Steel strength in shear [8]	$V_{sa,deck}$	lb	1,630	1,355	2,120	2,120	1,790	2,260	3,555	4,345	3,815	6,150	4,085	7,865
Steel strength in shear, seismic [7]	$V_{sa,deck,eq}$	lb	1,630	1,355	2,120	2,120	1,790	2,260	3,555	4,345	3,815	6,150	4,085	7,865
Loads According to Figure 5B														
Minimum concrete thickness over upper flute [4]	$h_{min,deck}$	in.	2-1/2	2-1/2			2-1/2				2-1/2		2-1/2	3-1/4
Pullout strength, uncracked concrete [5,6]	$N_{p,deck,uncr}$	lb	1,725	1,855	2,625	2,995	1,855	2,750	3,745	4,715	4,415	5,815	3,800	4,795
Pullout strength, cracked concrete [5,6]	$N_{p,deck,cr}$	lb	515	1,625	2,295	2,405	1,650	2,135	3,275	3,340	3,930	4,395	3,325	3,730
Pullout strength, seismic [5,7]	$N_{p,deck,eq}$	lb	515	1,625	2,295	2,405	1,650	2,135	3,275	3,340	3,930	4,395	3,325	3,730
Steel strength in shear [8]	$V_{sa,deck}$	lb	1,630	1,355	2,120	2,120	1,790	2,260	3,285	4,235	3,815	4,650	4,085	7,865
Steel strength in shear, seismic [7]	$V_{sa,deck,eq}$	lb	1,630	1,355	2,120	2,120	1,790	2,260	3,285	4,235	3,815	4,650	4,085	7,865
Loads According to Figure 5C														
Minimum concrete thickness over upper flute [4]	$h_{min,deck}$	in.	2-1/4	2-1/4		N/A	2-1/4		N/A	3-1/4	3-1/4	N/A	N/A	N/A
Pullout strength, uncracked concrete [5,6]	$N_{p,deck,uncr}$	lb	1,380	990	2,485	N/A	1,815	1,900	N/A	2,665	2,960	N/A	N/A	N/A
Pullout strength, cracked concrete [5,6]	$N_{p,deck,cr}$	lb	410	870	2,130	N/A	1,480	1,480	N/A	1,890	2,635	N/A	N/A	N/A
Pullout strength, seismic [5,7]	$N_{p,deck,eq}$	lb	410	870	2,130	N/A	1,480	1,480	N/A	1,890	2,635	N/A	N/A	N/A
Steel strength in shear [8]	$V_{sa,deck}$	lb	1,125	2,370	2,505	N/A	2,680	3,175	N/A	3,465	4,085	N/A	N/A	N/A
Steel strength in shear, seismic [7]	$V_{sa,deck,eq}$	lb	1,125	2,370	2,505	N/A	2,680	3,175	N/A	3,465	4,085	N/A	N/A	N/A

[1] Installations must comply with Section 4.1.9 and Section 4.3 and Figure 5A, Figure 5B and Figure 5C of this report.
[2] The values for $\phi_{p,N}$ in tension can be found in Table 4 of this report. The values for $\phi_{sa,V}$ in shear can be found in Table 6 of this report.
[3] Evaluation of concrete breakout capacity in accordance with ACI 318-19 17.6.2, 17.7.2 and 17.7.3, ACI 318-14 17.4.2, 17.5.2 and 17.5.3 or ACI 318-11 D.5.2, D.6.2, and D.6.3, as applicable, is not required for anchors installed in the deck soffit.
[4] Minimum concrete thickness refers to concrete thickness above upper flute. See Figures 5A to 5C.
[5] Characteristic pullout resistance for concrete compressive strengths greater than 3,000 psi (20.7 MPa) may be increased by multiplying the value in the table by $(f'c / 3000)^n$ for psi or $(f'c / 20.7)^n$ for MPa.
[6] The values listed must be used in accordance with Section 4.1.4 of this report.
[7] The values listed must be used in accordance with Sections 4.1.4 and 4.1.8 of this report.
[8] The values listed must be used in accordance with Section 4.1.5 of this report.
[9] For core drill installations, with $^3/_4$-inch diameter anchors installed at $3^3/_4$ inches (95 mm) effective embedment, apply a reduction factor of 0.89 to the design tension strength of anchors installed in uncracked concrete.

TABLE 9—HILTI KB-TZ2 CARBON STEEL ANCHORS SETTING INFORMATION FOR INSTALLATION ON THE TOP OF CONCRETE-FILLED PROFILE STEEL DECK ASSEMBLIES ACCORDING TO FIGURE 5D [1,2,3]

Design Information	Symbol	Units	Nominal anchor diameter (in.)			
			1/4	3/8		1/2
Effective Embedment Depth	h_{ef}	in. (mm)	1-1/2 (38)	1-1/2 (38)	2 (51)	2 (51)
Nominal Embedment Depth	h_{nom}	in. (mm)	1-3/4 (44)	1-7/8 (48)	2-1/2 (64)	2-1/2 (64)
Minimum Hole Depth	h_o	in. (mm)	2 (51)	2 (51)	2-1/2 (64)	2-3/4 (70)
Minimum Concrete Thickness[4]	$h_{min,deck}$	in. (mm)	2-1/2 (64)	2-1/2 (64)	2-1/2 (64)	3-1/4 (83)
Critical Edge Distance	$c_{ac,\,deck,\,top}$	in. (mm)	5 (127)	8 (204)	4-1/2 (114)	6 (153)
Minimum Edge Distance	$c_{min,\,deck,\,top}$	in. (mm)	3 (76)	16 (408)		7-1/2 (191)
Minimum Spacing	$s_{min,\,deck,\,top}$	in. (mm)	3 (76)	8 (204)		9 (229)
Required Installation Torque	T_{inst}	ft-lb (Nm)	4 (5)	30 (41)		50 (68)

[1] Installations must comply with Section 4.1.10 and Section 4.3 and Figure 5D of this report.
[2] Design capacity shall be based on calculations according to values in Tables 4 and 6 of this report.
[3] Applicable for $h_{min,deck}$ < 4 inches (102 mm). For $h_{min,deck}$ ≥ 4 inches (102 mm) , use setting information in Tables 1 and 3 of this report.
[4] Minimum concrete thickness refers to concrete thickness above the upper flute. See Figure 5D.

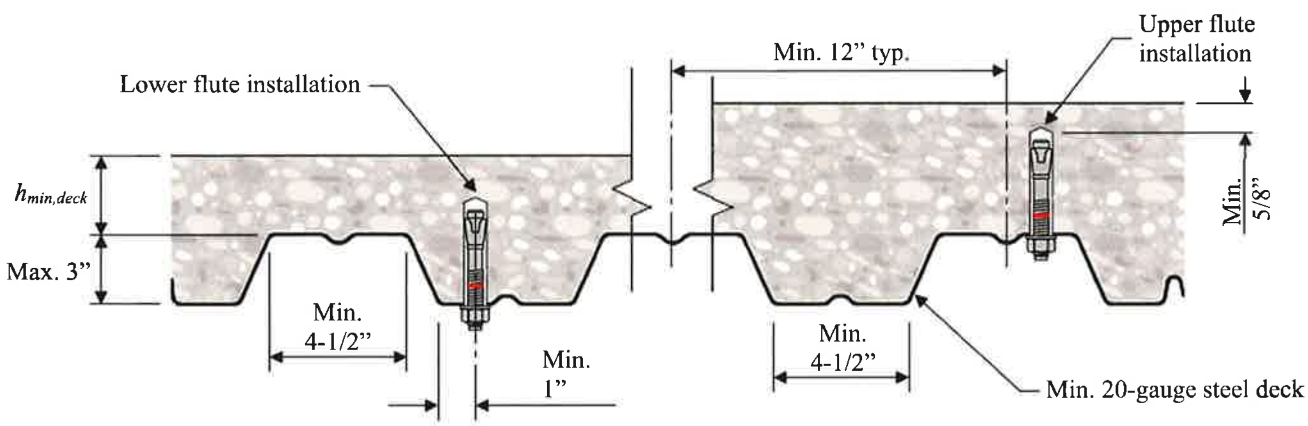


FIGURE 5A—KB-TZ2 IN THE SOFFIT OF CONCRETE FILLED PROFILE STEEL DECK ASSEMBLIES – W DECK

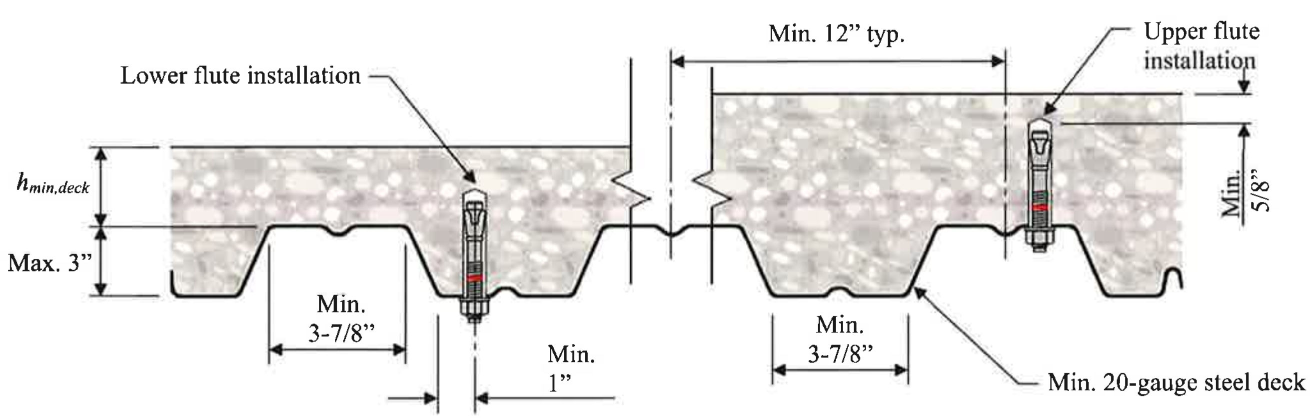


FIGURE 5B—KB-TZ2 IN THE SOFFIT OF CONCRETE FILLED PROFILE STEEL DECK ASSEMBLIES – W DECK

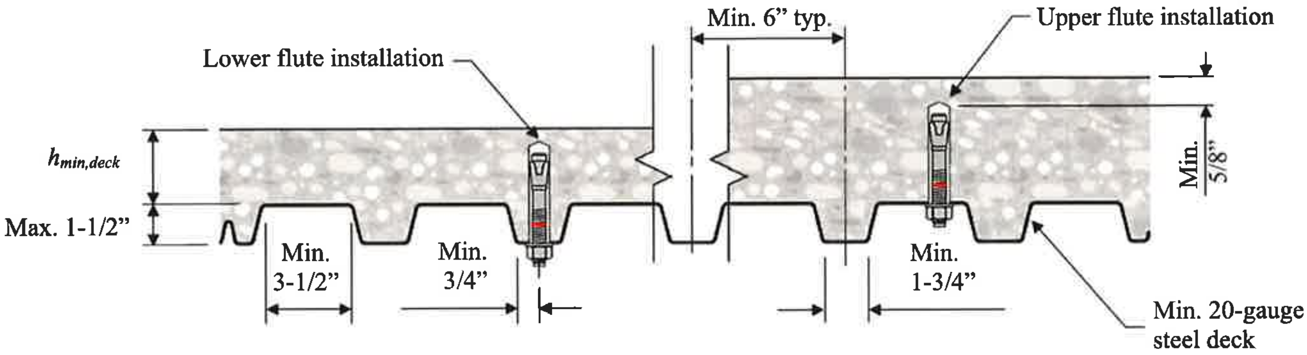


FIGURE 5C—KB-TZ2 IN THE SOFFIT OF CONCRETE FILLED PROFILE STEEL DECK ASSEMBLIES – B DECK

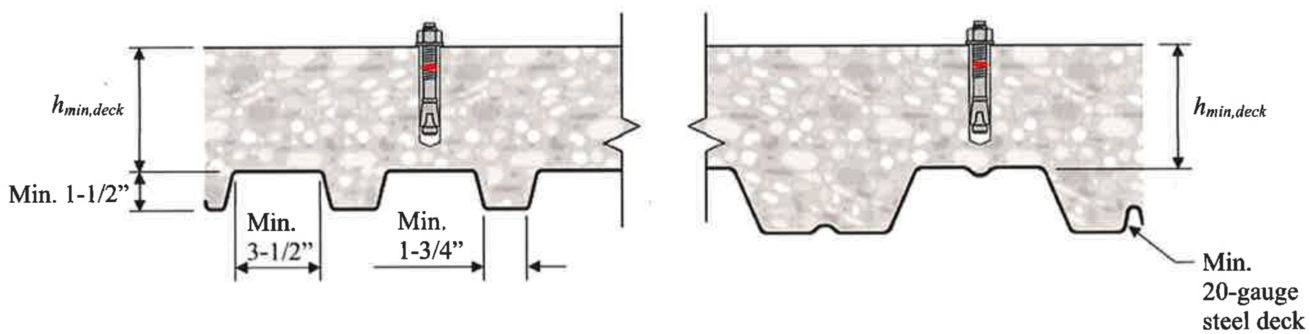
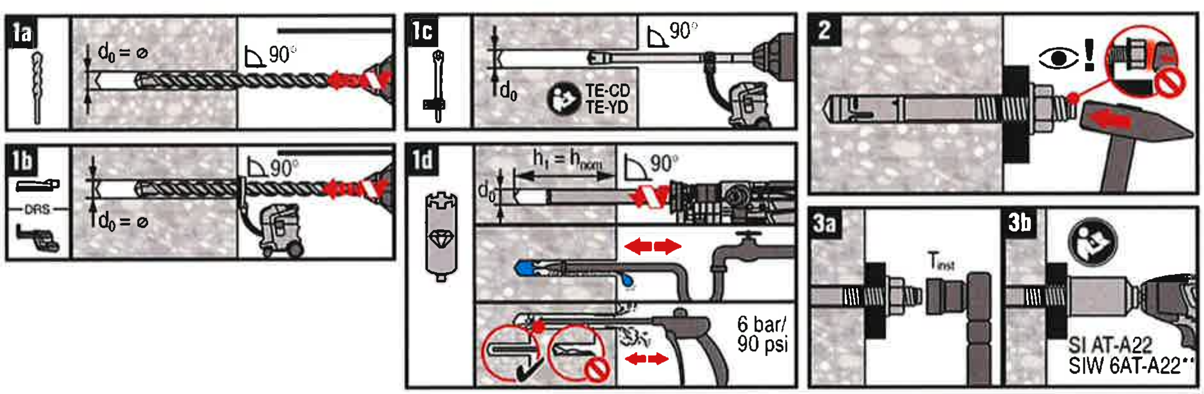


FIGURE 5D—KB-TZ2 IN THE TOP OF CONCRETE FILLED PROFILE STEEL DECK ASSEMBLIES

[1] 1$^1/_2$ inches (38 mm) B-deck as a minimum profile size. Other deck profiles meeting the B-deck minimum dimensions are also permitted.

Hilti SafeSet™ System with Hollow Drill Bit	Hilti Dust Removal Systems	Core Drill Systems
 Hilti TE-CD or TE-YD Hollow Carbide Drill Bit, with	 Hilti Rotary Hammer Drill with DRS (Dust Removal System) Module, or	 Handheld Hilti DD 30 Core Drill, with
 Hilti Vacuum (per section 4.3)	 Hilti TE DRS-D Dust Removal System with Hilti Vacuum	 SPX-T Hilti Core Bits (per Section 4.3)

FIGURE 6—HILTI SYSTEM COMPONENTS



Symbol	Setting Information	Units	Anchor Diameter (inch)				
			¼	⅜	½	⅝	¾
HDB	Hollow Drill Bit	–	–	–	✔	✔	✔
DRS	Dust Removal Systems	–	✔	✔	✔	✔	✔
AT-System	Adaptive Torque System**	–	–	✔	✔	✔	–
	Diamond Core Bit	–	–	✔	✔	✔	✔

FIGURE 7—INSTALLATION INSTRUCTIONS

ICC-ES Evaluation Report

ESR-4266 LABC and LARC Supplement

Issued December 2020
Revised March 2021
This report is subject to renewal December 2021.

www.icc-es.org | (800) 423-6587 | (562) 699-0543 *A Subsidiary of the International Code Council®*

DIVISION: 03 00 00—CONCRETE
Section: 03 16 00—Concrete Anchors

DIVISION: 05 00 00—METALS
Section: 05 05 19—Post-Installed Concrete Anchors

REPORT HOLDER:

HILTI, INC.

EVALUATION SUBJECT:

HILTI KWIK BOLT TZ2 CARBON AND STAINLESS STEEL ANCHORS IN CRACKED AND UNCRACKED CONCRETE

1.0 REPORT PURPOSE AND SCOPE

Purpose:

The purpose of this evaluation report supplement is to indicate that the Kwik Bolt TZ2 (KB-TZ2) carbon and stainless steel anchors in cracked and uncracked concrete, described in ICC-ES evaluation report ESR-4266, have also been evaluated for compliance with the codes noted below as adopted by the Los Angeles Department of Building and Safety (LADBS).

Applicable code editions:

- 2020 *City of Los Angeles Building Code* (LABC)
- 2020 *City of Los Angeles Residential Code* (LARC)

2.0 CONCLUSIONS

The Kwik Bolt TZ2 (KB-TZ2) carbon and stainless steel anchors in cracked and uncracked concrete, described in Sections 2.0 through 7.0 of the evaluation report ESR-4266, comply with LABC Chapter 19, and LARC, and are subject to the conditions of use described in this supplement.

3.0 CONDITIONS OF USE

The Kwik Bolt TZ2 (KB-TZ2) carbon and stainless steel anchors in cracked and uncracked concrete described in this evaluation report supplement must comply with all of the following conditions:

- All applicable sections in the evaluation report ESR-4266.
- The design, installation, conditions of use and labeling of the Kwik Bolt TZ2 (KB-TZ2) anchors are in accordance with the 2018 *International Building Code®* (2018 IBC) provisions noted in the evaluation report ESR-4266.
- The design, installation and inspection are in accordance with additional requirements of LABC Chapters 16 and 17, as applicable.
- Under the LARC, an engineered design in accordance with LARC Section R301.1.3 must be submitted.
- The allowable and strength design values listed in the evaluation report and tables are for the connection of the anchors to concrete. The connection between the anchors and the connected members shall be checked for capacity (which may govern).
- For use in wall anchorage assemblies to flexible diaphragm applications, anchors shall be designed per the requirements of City of Los Angeles Information Bulletin P/BC 2020-071.

This supplement expires concurrently with the evaluation report, issued December 2020 and revised March 2021.

ICC-ES Evaluation Report

ESR-4266 FBC Supplement

Issued December 2020

Revised March 2021

This report is subject to renewal December 2021.

www.icc-es.org | (800) 423-6587 | (562) 699-0543 *A Subsidiary of the International Code Council®*

DIVISION: 03 00 00—CONCRETE
Section: 03 16 00—Concrete Anchors

DIVISION: 05 00 00—METALS
Section: 05 05 19—Post-Installed Concrete Anchors

REPORT HOLDER:

HILTI, INC.

EVALUATION SUBJECT:

HILTI KWIK BOLT TZ2 CARBON AND STAINLESS STEEL ANCHORS IN CRACKED AND UNCRACKED CONCRETE

1.0 REPORT PURPOSE AND SCOPE

Purpose:

The purpose of this evaluation report supplement is to indicate that the Kwik Bolt TZ2 (KB-TZ2) carbon and stainless steel anchors in cracked and uncracked concrete, described in ICC-ES evaluation report ESR-4266, have also been evaluated for compliance with the codes noted below.

Applicable code editions:

- 2020 *Florida Building Code—Building*
- 2020 *Florida Building Code—Residential*

2.0 CONCLUSIONS

The Kwik Bolt TZ2 (KB-TZ2) carbon and stainless steel anchors in cracked and uncracked concrete, described in Sections 2.0 through 7.0 of the evaluation report ESR-4266, comply with the *Florida Building Code—Building* and the *Florida Building Code—Residential*, provided the design requirements are determined in accordance with the *Florida Building Code—Building* or the *Florida Building Code—Residential*, as applicable. The installation requirements noted in the ICC-ES evaluation report ESR-4266 for the 2018 *International Building Code®* meet the requirements of the *Florida Building Code—Building* or the *Florida Building Code—Residential*, as applicable.

Use of the Kwik Bolt TZ2 (KB-TZ2) carbon and stainless steel anchors in cracked and uncracked concrete have also been found to be in compliance with the High-Velocity Hurricane Zone provisions of the *Florida Building Code—Building* and the *Florida Building Code—Residential*, with the following conditions:

a) For anchorage to wood members, the connection subject to uplift, must be designed for no less than 700 pounds (3114 N).

b) For connection to aluminum members, all expansion anchors must be installed no less than 3 inches from the edge of concrete slab and/or footings. All expansion anchors shall develop an ultimate withdrawal resisting force equal to four times the imposed load, with no stress increase for duration of load.

For products falling under Florida Rule 61G20-3, verification that the report holder's quality assurance program is audited by a quality assurance entity approved by the Florida Building Commission for the type of inspections being conducted is the responsibility of an approved validation entity (or the code official, when the report holder does not possess an approval by the Commission).

This supplement expires concurrently with the evaluation report, issued December 2020 and revised March 2021.