PCS Structural *Solutions*

PRCTI20250548

Seattle | 1011 Western Avenue, Suite 810 | Seattle, WA 98104 | 206.292.5076
Tacoma | 1250 Pacific Avenue, Suite 701 | Tacoma, WA 98402 | 253.383.2797
Portland | 101 SW Main Street, Suite 1602 | Portland, OR 97204 | 503.232.3746

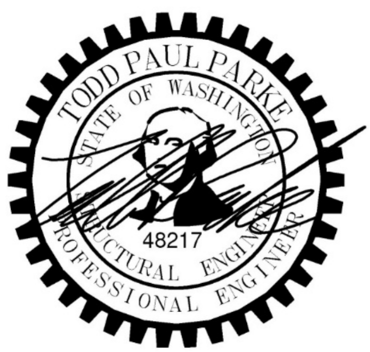
www.pcs-structural.com

STRUCTURAL CALCULATIONS

FOR

GOOD SAMARITAN HOSPITAL KITCHEN
401 15TH AVE SE
PUYALLUP, WA 98372

PREPARED BY
PCS STRUCTURAL *SOLUTIONS*



APRIL 18, 2025
24-707

Project: Good Samaritan Hospital Nutrition Department Expansion | Job Number: 24-707

Sheet: ____ of ____ | Name: KDK

Originating Office: Tacoma | Date: 4/9/2025

DESIGN CRITERIA CHECKLIST

CODE: IBC 2021, ASCE 7-16 LOCATION: PUYALLUP, WA

RISK CATEGORY: IV (Per ASCE 7-16 Table 1.5-1 & IBC Table 1604.5)

VERTICAL DESIGN CRITERIA

	DEAD	LIVE	PARTITION	CONCENTRATED
ROOF:	20 PSF	25 PSF		
PATIENT ROOM	75 PSF	40 PSF	+ 20 PSF	1000 #
OPERATING ROOM	75 PSF	60 PSF		1000 #
CORRIDORS/STAIRS:	75 PSF	80 PSF		

WIND DESIGN CRITERIA

BASIC WIND SPEED (V) = 108 MPH (Per ASCE 7-16 Sec. 26.5.1, Fig. 26.5-1A; 1B; 1C & 1D, or as required by Bld'g Dept.)

EXPOSURE CATEGORY: B (Per ACSE 7-16 Section 26.7.3)

DIRECTIONALITY FACTOR (K_d): 0.85 (Per ASCE 7-16 Table 26.6-1)

GUST EFFECT FACTOR (G): 0.85 (Per ASCE 7-16 Section 26.11)

TOPOGRAPHIC FEATURE: None (See ASCE 7-16 Figure 26.8-1)

HILL HEIGHT (H): 100 FT (See ASCE 7-16 Figure 26.8-1)

UPWIND DISTANCE TO HALF HILL (Lh): 100 FT (See ASCE 7-16 Figure 26.8-1)

DISTANCE FROM CREST TO SITE (x): 100 FT UPWIND (See ASCE 7-16 Figure 26.8-1)

MEAN ROOF HEIGHT: 50 FT (See ASCE 7-16 Section 26.2 - Definitions)

ELEVATION: 135 FT (See ASCE 7-16 Section 26.9)

ENCLOSURE CLASSIFICATION: Enclosed (See ASCE 7-16 Secion 26.2 & Table 26.13-1)

ROOF TYPE: Monoslope (See ASCE 7-16 Figure 27.3-1)

ROOF SLOPE (__:12): 0.50:12 (Enter vertical rise in 12 horizontal units) θ (degrees): 2.39

SEISMIC DESIGN CRITERIA

SITE CLASS: D (Per IBC Section 1613.2.2, Assumed as "D" or per Geotech.)

IMPORTANCE FACTOR (I_E): 1.5 (Per ASCE 7-16 Table 1.5-2)

No changes will be made to existing lateral system.

INFORMATION BELOW FROM "ASCE HAZARD TOOL"

LATITUDE: 47.179 S_S = 1.267 F_a = 1.200

LONGITUDE: -122.290 S_1 = 0.436 F_v = 1.900

DEFLECTION CRITERIA

FLOOR (LIVE): L/ 480 ROOF (LIVE): L/ 360

FLOOR (TOTAL): L/ 360 ROOF (TOTAL): L/ 240

WALLS: L/ 360 SPECIAL: L/

SOIL DESIGN CRITERIA

REPORT: NO

SEE SOILS REPORT FOR ACTIVE, PASSIVE PRESSURES AND FRICTION COEFFICIENT

BEARING: 1500 PSF

ACTIVE: 35 PCF

PASSIVE: 200 PCF

COEFFICIENT OF FRICTION: 0.35

MINIMUM FOOTING DIMENSIONS:

CONTINUOUS: 1'-4"

SPREAD: 1'-6"

FROST DEPTH: 1'-6"

PILE TYPE: NONE

VERTICAL CAPACITY : N/A LATERAL CAPACITY: N/A

UPLIFT CAPACITY: N/A SIZE: N/A

Project:	Good Samaritan Hospital Nutrition Department Expansion	Job Number:	24-707
Sheet:	of	Name:	KDK
Originating Office:	Tacoma	Date:	03/04/25

MATERIALS

CONCRETE

Footings/Piles:	3000 PSI	Columns:	4000 PSI
Slabs/Walls:	4000 PSI	Beams:	4000 PSI
-	-	-	-

REINFORCING

Steel Grade =	60	f_y =	60 KSI

STRUCTURAL STEEL

W-Flange Beams	ASTM A992	f_y =	50 KSI
Shapes & Plates	ASTM A36	f_y =	36 KSI
Pipes	ASTM A53, Grade B	f_y =	35 KSI
HSS Rect.	ASTM A500, Grade C	f_y =	50 KSI
HSS Round	ASTM A500, Grade C	f_y =	46 KSI

PCS Structural Solutions

DESIGN CRITERIA - WIND

BASIC WIND SPEED (V):	108 MPH	MEAN ROOF HEIGHT:	50 FT
RISK CATEGORY:	IV	GROUND ELEVATION FACTOR (Ke):	1.00
EXPOSURE CATEGORY:	B	ENCLOSURE CLASSIFICATION:	Enclosed
DIRECTIONALITY FACTOR (K_d):	0.85	ROOF TYPE:	Monoslope
GUST EFFECT FACTOR (G):	0.85	ROOF SLOPE (__:12):	0.5:12
		θ (degrees):	2.39

ROOF PRESSURES (Figure 27.3-1)

		External Pressures (q_h*(GC_p)):			Internal Pressures (±q_i*(GC_{pi}))
Wind Direction:	h/L:	Windward (Positive)	Windward (Negative)	Leeward	All Roofs
Normal to Ridge for θ ≥ 10°	≤0.25	N/A	N/A	N/A	3.7
	0.50	N/A	N/A	N/A	
	≥1.0	N/A	N/A	N/A	
Normal to Ridge for θ < 10° and Parallel to Ridge for All θ	h/L:	Horizontal Distance from Windward Edge	External Pressures (q*(GC_p)): Positive Pressure	Negative Pressure	Internal Pressures (±q_i*(GC_{pi})) All Roofs
	≤0.5	0 to h	-3.1	-15.7	3.7
		h to 2h		-8.7	
		>2h		-5.2	
			-3.1	-22.6	
		>h/2		-12.2	

ASCE 7-16 CHAPTER 27: WIND LOADS ON BUILDINGS: MWFRS (DIRECTIONAL PROCEDURE)
PART 1: ENCLOSED AND PARTIALLY ENCLOSED BUILDINGS OF ALL HEIGHTS
HORIZONTAL WALL PRESSURES (Figure 27.3-1)

Windward External Pressures (q_z*(GC_p)):			Leeward & Sidewall External Pressures (q_h*(GC_p)):			Internal Pressures (±q_i*(GC_{pi}))
Height Above Ground Level, z	K_{zt}	Windward wall	L/B:	Leeward wall	Sidewall	All walls
15	1.00	9.8	0-1	-8.7	-12.2	3.7
20	1.00	10.6	2	-5.2		
25	1.00	11.3	≥4	-3.5		
30	1.00	12.0				
40	1.00	13.1				
50	1.00	13.9				
60	1.00	14.6				
70	1.00	15.3				
80	1.00	16.0				
90	1.00	16.5				
100	1.00	17.0				
120	1.00	17.9				
140	1.00	18.7				
160	1.00	19.4				
180	1.00	20.1				
200	1.00	20.6				
250	1.00	22.0				
300	1.00	23.2				
350	1.00	24.2				
400	1.00	25.2				
450	1.00	26.1				
500	1.00	26.8				

NOTES:

1) Minimum Design Wind Loads (Per ASCE 7-16 27.1.5): The wind load used for design of the MWFRS shall not be less than 16 PSF multiplied by the wall area of the building, and 8 PSF multiplied by the roof area of the building projected on a vertical plane normal to the assumed wind direction. Wall and roof loads shall be applied simultaneously.

2) q_i has conservatively been taken equal to q_h

K_{ht} = 1.00

q_h = 20.5 PSF

DESIGN CRITERIA - WIND

BASIC WIND SPEED (V):	108 MPH	MEAN ROOF HEIGHT:	50 FT
RISK CATEGORY:	IV	GROUND ELEVATION FACTOR (Ke):	1.00
EXPOSURE CATEGORY:	B	ENCLOSURE CLASSIFICATION:	Enclosed
DIRECTIONALITY FACTOR (K_d):	0.85	ROOF TYPE:	Monoslope
GUST EFFECT FACTOR (G):	0.85	ROOF SLOPE (__:12):	0.5:12
		θ (degrees):	2.39

ASCE 7-16 CHAPTER 30: WIND LOADS: COMPONENTS AND CLADDING
PART 1: LOW-RISE BUILDINGS (h≤60 ft)

ROOF SURFACES

Effective Wind Area	POSITIVE PRESSURES	NEGATIVE PRESSURES					
		ZONE					
	ALL ZONES	1'	1	2	3	N/A	N/A
10 SF	16.0	-22.1	-38.5	-50.7	-69.1	N/A	N/A
20 SF	16.0	-22.1	-35.9	-47.5	-62.6	N/A	N/A
50 SF	16.0	-22.1	-32.6	-43.2	-54.0	N/A	N/A
100 SF	16.0	-22.1	-30.0	-39.9	-47.5	N/A	N/A

WALL SURFACES & ROOF OVERHANGS

Effective Wind Area	WALL ZONES				ROOF OVERHANG ZONES					
	POSITIVE PRESSURES		NEGATIVE PRESSURES		NEGATIVE PRESSURES					
	4	5	4	5	1'	1	2	3	N/A	N/A
10 SF	24.1	24.1	-26.2	-32.3	-34.8	-34.8	-47.1	-65.5	N/A	N/A
20 SF	23.1	23.1	-25.1	-30.1	-34.2	-34.2	-42.7	-57.9	N/A	N/A
50 SF	21.6	21.6	-23.7	-27.3	-33.3	-33.3	-37.0	-47.8	N/A	N/A
100 SF	20.5	20.5	-22.6	-25.1	-32.7	-32.7	-32.6	-40.2	N/A	N/A
500 SF	18.0	18.0	-20.0	-20.0	-31.3	-31.3	-22.5	-22.5	N/A	N/A

NOTES:

1) ASCE 7-16 30.2.2: Minimum Design Wind Loads: The design wind pressure for C&C of buildings shall not be less than a net pressure of 16 PSF acting in either direction normal to the surface.

2) q_i has conservatively been taken equal to q_h

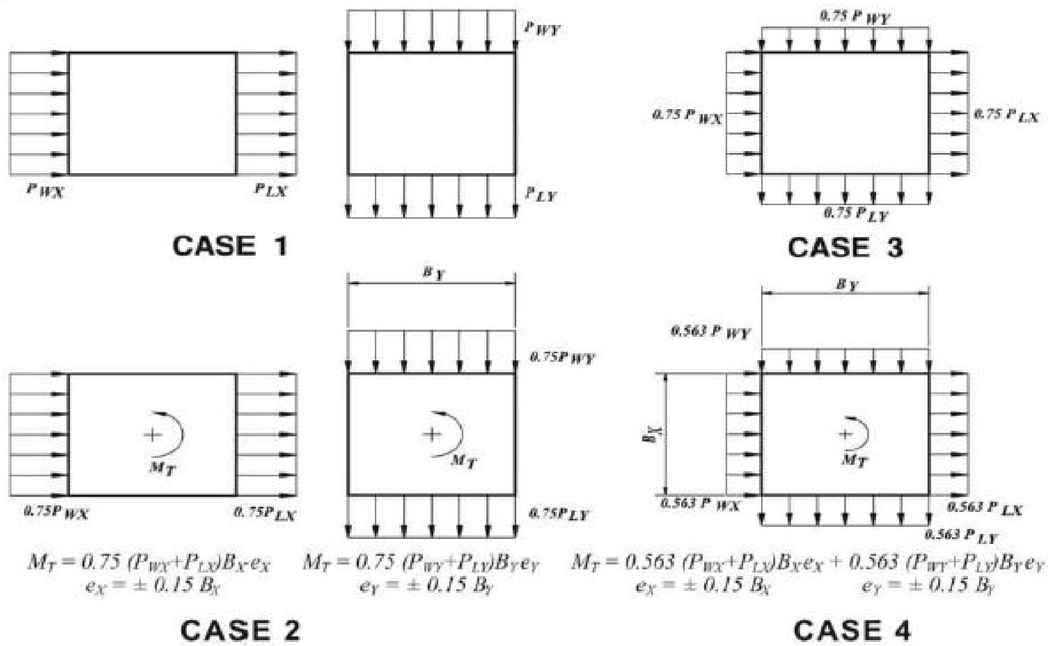
K_{ht} = 1.00

q_h = 20.5 PSF

DESIGN CRITERIA - WIND

FIGURE 27.3-8: Main Wind Force Resisting System, Part 1 (All Heights): Design Wind Load Cases per ASCE 7-16

Diagrams



Notation

P_{WX}, P_{WY} = Windward face design pressure acting in the x, y principal axis, respectively.
P_{LX}, P_{LY} = Leeward face design pressure acting in the x, y principal axis, respectively.
$e(e_X, e_Y)$ = Eccentricity for the x, y principal axis of the structure, respectively.
M_T = Torsional moment per unit height acting about a vertical axis of the building.

Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.
Case 2. Three-quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.
Case 3. Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.
Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notes

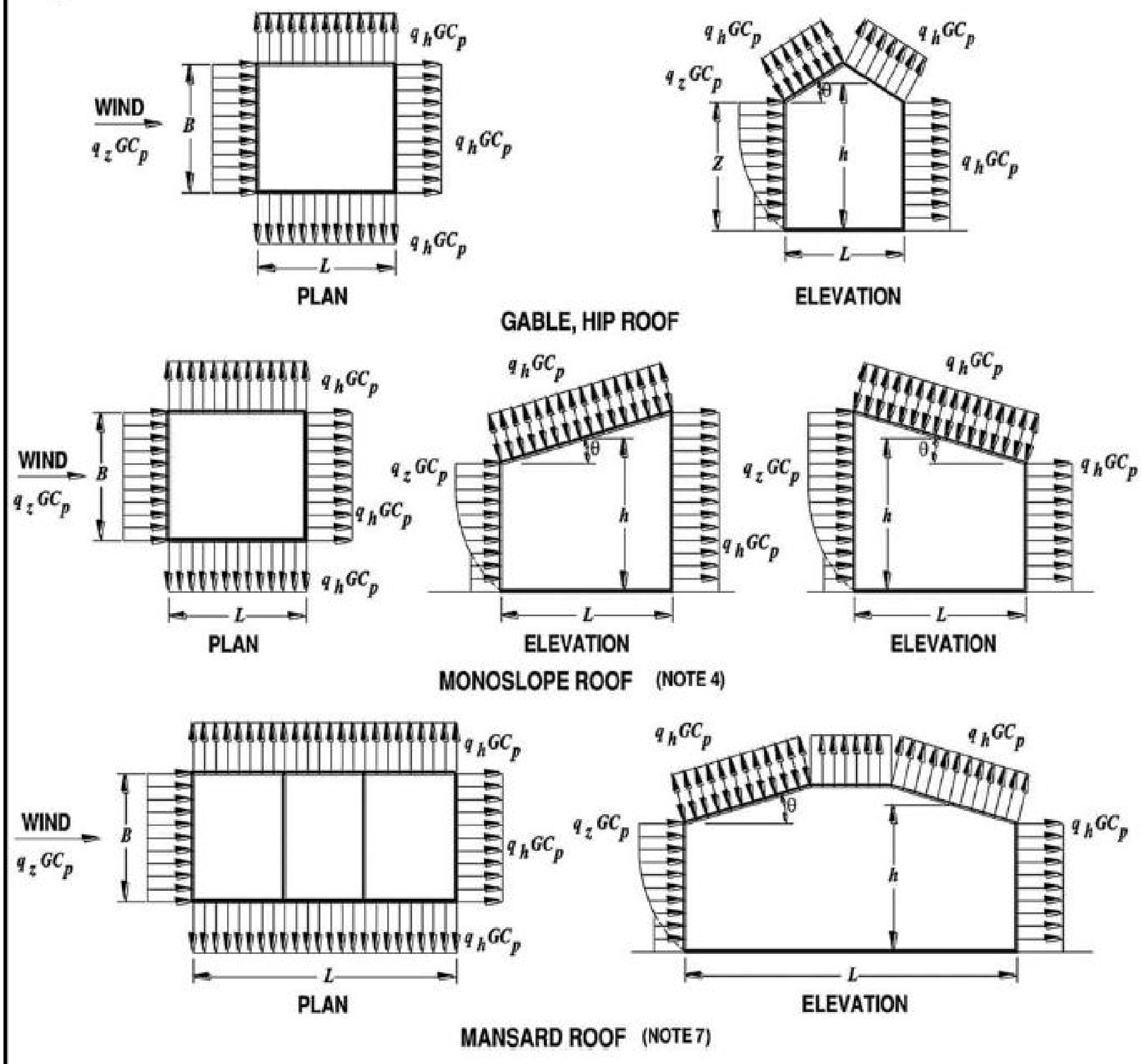
1. Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of Sections 27.3.1 and 27.3.2 as applicable for buildings of all heights.
2. Diagrams show plan views of buildings.

FIGURE 27.3-8 Main Wind Force Resisting System, Part 1 (All Heights): Design Wind Load Cases

DESIGN CRITERIA - WIND

FIGURE 27.3-1 Main Wind Force Resisting System, Part 1 (All Heights): External Pressure Coefficients, Cp, for Enclosed and Partially Enclosed Buildings - Walls and Roofs per ASCE 7-16

Diagrams



Notation

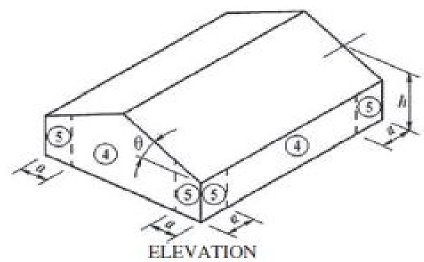
B = Horizontal dimension of building, in ft (m), measured normal to wind direction.
L = Horizontal dimension of building, in ft (m), measured parallel to wind direction.
h = Mean roof height, in ft (m), except that eave height shall be used for $\theta \leq 10$ degrees.
z = Height above ground, in ft (m).
G = Gust-effect factor.
q_z, q_h = Velocity pressure, in lb/ft^2 (N/m^2), evaluated at respective height.
θ = Angle of plane of roof from horizontal, in degrees.

FIGURE 27.3-1 Main Wind Force Resisting System, Part 1 (All Heights): External Pressure Coefficients, C_p, for Enclosed and Partially Enclosed Buildings—Walls and Roofs

DESIGN CRITERIA - WIND

FIGURE 30.3-1: Components and Cladding [h ≤ 60 ft]: External Pressure Coefficients, (GCp), for Enclosed and Partially Enclosed Buildings - Walls

Diagram



Notation

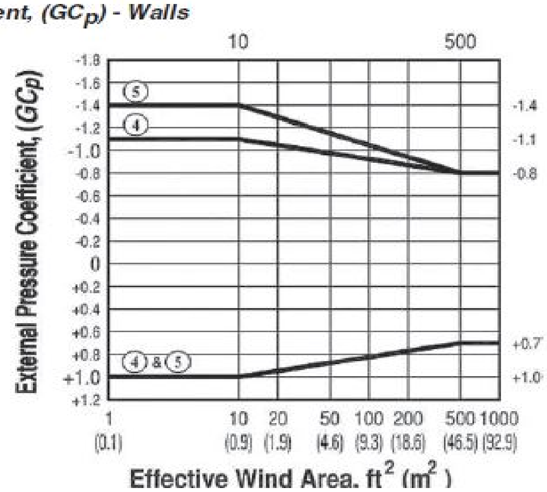
a = 10% of least horizontal dimension or $0.4h$, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft (0.9 m).

Exception: For buildings with θ = 0° to 7° and a least horizontal dimension greater than 300 ft (90 m), dimension a shall be limited to a maximum of $0.8h$.

h = Mean roof height, in ft (m), except that eave height shall be used for θ ≤ 10°.

θ = Angle of plane of roof from horizontal, in degrees.

External Pressure Coefficient, (GC_p) - Walls



Notes

1. Vertical scale denotes (GCp) to be used with qh.
2. Horizontal scale denotes effective wind area, in ft² (m²).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of (GCp) for walls shall be reduced by 10% when θ ≤ 10°.

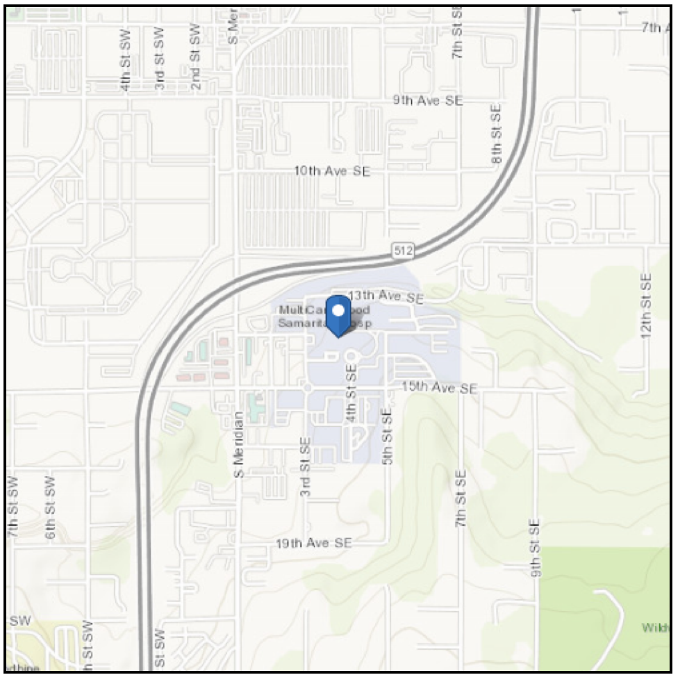
FIGURE 30.3-1 Components and Cladding [h ≤ 60 ft (h ≤ 18.3 m)]: External Pressure Coefficients, (GC_p), for Enclosed and Partially Enclosed Buildings—Walls

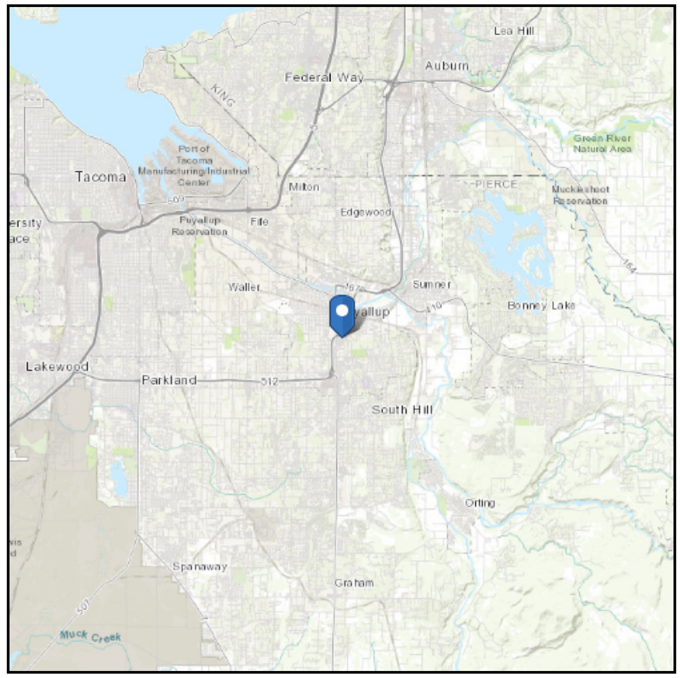
ASCE Hazards Report

Address:
MultiCare Good Samaritan Hospital - 401 15th Ave SE
Puyallup,

Standard: ASCE/SEI 7-16
Risk Category: IV
Soil Class: D - Default (see Section 11.4.3)

Latitude: 47.178606
Longitude: -122.289924
Elevation: 135.1922121017412 ft (NAVD 88)





Wind

Results:

Wind Speed	108 Vmph
10-year MRI	67 Vmph
25-year MRI	73 Vmph
50-year MRI	78 Vmph
100-year MRI	83 Vmph

Data Source:	ASCE/SEI 7-16, Fig. 26.5-1D and Figs. CC.2-1–CC.2-4, and Section 26.5.2
Date Accessed:	Tue Feb 04 2025

Value provided is 3-second gust wind speeds at 33 ft above ground for Exposure C Category, based on linear interpolation between contours. Wind speeds are interpolated in accordance with the 7-16 Standard. Wind speeds correspond to approximately a 1.6% probability of exceedance in 50 years (annual exceedance probability = 0.00033, MRI = 3,000 years).

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

ASCE
AMERICAN SOCIETY OF CIVIL ENGINEERS

Seismic

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

Results:

S_S :	1.267	S_{D1} :	N/A
S_1 :	0.436	T_L :	6
F_a :	1.2	PGA :	0.5
F_v :	N/A	PGA_M :	0.6
S_{MS} :	1.52	F_{PGA} :	1.2
S_{M1} :	N/A	I_e :	1.5
S_{DS} :	1.013	C_v :	1.353

Ground motion hazard analysis may be required. See ASCE/SEI 7-16 Section 11.4.8.

Data Accessed: Tue Feb 04 2025

Date Source: USGS Seismic Design Maps

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

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

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

Project:	Good Samaritan Hospital Nutrition Department Expansion
Job Number:	24-707
Sheet:	of
Name:	KDK
Originating Office:	Tacoma
Date:	3/4/2025

PCS Structural Solutions

MECHANICAL UNIT ANCHORAGE SPREADSHEET

UNIT NAME:	MUA-1
CODE:	IBC 2021, ASCE 7-16
UNIT EXT. OR INT.?	EXTERNAL
LOCATION:	PUYALLUP, WA

UNIT INFORMATION:

OPERATING WEIGHT (W_p) =	5344 lb	
UNIT HEIGHT (h) =	73 in	6.1 ft
UNIT WIDTH (w) =	87 in	7.2 ft
UNIT LENGTH (l) =	173 in	14.4 ft
C.O.G. (VERTICAL) =	49 in	4.1 ft (2/3)*UNIT HEIGHT (ASSUMED)
TOTAL WEIGHT (W_{tot}) =	5554 lb	
h_{wind} =	3.6 ft	(1/2)*(UNIT HT + CURB HT)
$h_{seismic}$ =	5.2 ft	CENTER OF GRAVITY + CURB HT

CURB INFORMATION:

IS THERE A CURB?	YES	
CURB WEIGHT, $W_{p\text{-}2}$ =	210 lb	
CURB HEIGHT (h) =	14 in	1.2 ft
CURB WIDTH AT BASE (w) =	87 in	7.2 ft
CURB LENGTH AT BASE (l) =	118 in	9.8 ft
DESIGN WIDTH (w_{des}) =	7.2 ft	
DESIGN LENGTH (l_{des}) =	9.8 ft	

BUILDING INFORMATION

ARE BUILDING DIMENSIONS KNOWN?	YES	BUILDING WIDTH (B) =	87.8 ft	B*h =	7750 sq ft
		BUILDING LENGTH (L) =	155.0 ft	B*L =	13601 sq ft
		AVERAGE ROOF HT. (h) =	50.0 ft		

DESIGN CRITERIA - GRAVITY

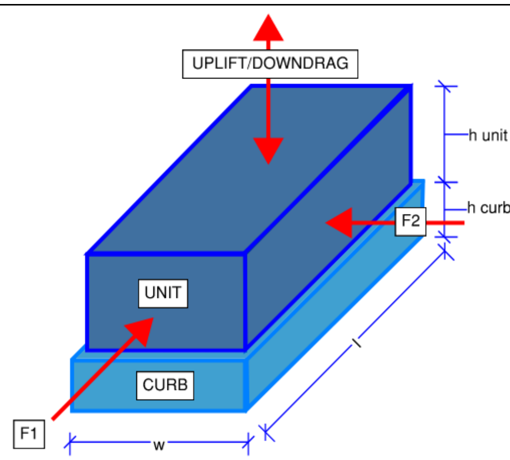
ROOF DEAD (DL) =	20 PSF
ROOF SNOW (SL) =	25 PSF
$P_{roof\ DL}$ =	2081 lb
$P_{roof\ SL}$ =	2602 lb

DESIGN CRITERIA - LATERAL (THE UNIT DOES NOT HAVE A SIGNIFICANT IMPACT TO THE BUILDING'S CAPACITY TO RESIST WIND OR SEISMIC FORCES)

SEISMIC (ASCE 7-16, CHAPTER 13):

COMPONENT TYPE: AIR-SIDE HVACR, FANS, AIR HANDLERS, A/C UNITS, CABINET HEATERS, AIR DISTRIBUTION BOXES, AND OTHER MECH. COMPONENTS CONSTRUCTED OF SHEET METAL FRAMING

S_{DS} =	1.014	
a_p =	2.5	(TABLE 13.6-1)
I_p =	1.5	(§ 13.1.3)
R_p =	6	(TABLE 13.6-1)
OVERSTRENGTH FACTOR (Ω_o) =	2.0	(TABLE 13.6-1)
ATTACHMENT OR BTM. OF CURB HT. (z) =	50.0 ft	
AVERAGE ROOF HT. (h) =	50.0 ft	
$F_p = 0.4*S_{DS}*(I_p/R_p)*a_p*W_p*[1+2*z/h]$ =	4224 lb	(EQN. 13.3-1)
$F_{p\ max} = 1.6*S_{DS}*I_p*W_p$ =	13516 lb	(EQN. 13.3-2)
$F_{p\ min} = 0.3*S_{DS}*I_p*W_p$ =	2534 lb	(EQN. 13.3-3)
HORIZ. EQ. DESIGN FORCE, F_p =	**4224 lb**	
VERT. EQ. DESIGN FORCE, $F_{pv} = \pm 0.2*S_{DS}W_p$ =	**1126 lb**	(§ 13.3.1.2)



WIND (ASCE7-16, CHAPTER 29 - DIRECTIONAL PROCEDURE):

WIND SPEED (V) =	108 MPH	
RISK CATEGORY:	IV	(TABLE 1.5-2)
WIND EXPOSURE:	B	(§ 26.7.3)
TOPO. EFFECT (K_{zt}) =	1.0	(FIG. 26.8-1)
K_h =	0.81	(TABLE 26.10-1)
DIRECTIONALITY FACTOR, K_d =	0.85	(TABLE 26.6-1)
GROUND ELEV. FACTOR, K_e =	1.00	(TABLE 26.9-1)

		F_1	F_2	
	A_f =	52.4 sq ft	99.0 sq ft	
	A_r =	104.1 sq ft		
	$q_h = 0.00256*K_d*K_h*K_{zt}*K_e*V^2$ =	20.6 psf		(EQN. 26.10-1)
(§ 29.4.1)	GC_r =	1.90	1.90	
(§ 29.4.1)	GC_r =	1.50		

		F_1	F_2
(EQN. 29.4-2)	$F_h = q_h*(GC_r)*A_f$ =	**2047 lb**	**3867 lb**
(EQN. 29.4-3)	$F_v = q_h*(GC_r)*A_r$ =	**3209 lb**	

Project: Good Samaritan Hospital Nutrition Department Expansion | Job Number: 24-707

Sheet: of | Name: KDK

Originating Office: Tacoma | Date: 03/04/25

MECHANICAL UNIT ANCHORAGE SPREADSHEET

MOMENTS AND REACTIONS (UNFACTORED)

	F_1	F_2
$M_{OT\ Seismic}$ =	22058 lb-ft	22058 lb-ft
$M_{OT\ Wind}$ =	7421 lb-ft	14018 lb-ft
$M_{R\ SL}$ =	12738 lb-ft	9404 lb-ft
$M_{R\ DL\ Unit}$ =	26163 lb-ft	19316 lb-ft
$M_{R\ DL\ Curb}$ =	1028 lb-ft	759 lb-ft

P_{SL} =	2602 lb
$P_{DL\ Unit}$ =	5344 lb
$P_{DL\ Curb}$ =	210 lb

	VERTICAL FORCE		OVERTURNING T/C		HORIZ. SHEAR	
LOAD CASE	UPLIFT	DOWNWARD	T_1/C_1	T_2/C_2	V_1	V_2
SEISMIC:	1126 lb	1126 lb	2253 lb	3051 lb	4224 lb	4224 lb
WIND:	3209 lb	N/A	758 lb	1939 lb	2047 lb	3867 lb
SNOW:	N/A	2602 lb	N/A	N/A	N/A	N/A
DEAD:	N/A	5554 lb	N/A	N/A	N/A	N/A

COMBINATIONS (ASD)

	OVERTURNING MOMENT	
COMBINATION	OT_1	OT_2
0.6DL-0.6WL	-11862 lb-ft	-3635 lb-ft
0.6DL-0.7EQ	-874 lb-ft	3395 lb-ft
DL+SL	-39929 lb-ft	-29480 lb-ft
DL+0.6WL	-31644 lb-ft	-28486 lb-ft
DL+0.45WL+0.75SL	-40084 lb-ft	-33437 lb-ft
DL+0.7EQ	-42632 lb-ft	-35516 lb-ft
DL+0.525EQ+0.75SL	-48325 lb-ft	-38709 lb-ft

	VERTICAL FORCE	
COMBINATION	UPLIFT	DOWNWARD
$0.6*F_v$	1926 lb	
$\pm0.7*F_{pv}$	788 lb	788 lb
$0.6*F_v$	1926 lb	
$0.45*F_v$	1444 lb	
$\pm0.7*F_{pv}$	788 lb	788 lb
$\pm0.525*F_{pv}$	591 lb	591 lb

	TENSION		COMPRESSION		HORIZ. SHEAR	
RESULTANT COMBINATION	T_1	T_2	C_1	C_2	V_1	V_2
0.6DL-0.6WL	N/A	460 lb	1211 lb	503 lb	-1228 lb	-2320 lb
$(0.6\text{-}0.14S_{DS})$DL-0.7EQ	305 lb	864 lb	484 lb	N/A	-2957 lb	-2957 lb
DL+SL	N/A	N/A	4078 lb	4078 lb	N/A	N/A
DL+0.6WL	N/A	N/A	3232 lb	3940 lb	1228 lb	2320 lb
DL+0.45WL+0.75SL	N/A	N/A	4094 lb	4625 lb	921 lb	1740 lb
$(1+0.14S_{DS})$DL+0.7EQ	N/A	N/A	4748 lb	5307 lb	2957 lb	2957 lb
$(1+0.105S_{DS})$DL+0.525EQ+0.75SL	N/A	N/A	5231 lb	5650 lb	2218 lb	2218 lb

DESIGN LOADS (ASD) AND ATTACHMENT DESIGN

	TENSION		COMPRESSION		HORIZ. SHEAR	
	T_1	T_2	C_1	C_2	V_1	V_2
FORCE:	**305 lb**	**864 lb**	**5231 lb**	**5650 lb**	**2957 lb**	**2957 lb**
# OF SCREWS/SIDE:	**2 SCREWS**	**5 SCREWS**			**4 SCREWS**	**4 SCREWS**

SCREW SIZE	CURB/UNIT THICKNESS
#14	16ga

Project: Good Samaritan Hospital Nutrition Department Expansion Job Number: 24-707

Sheet: of Name: KDK

Originating Office: Tacoma Date: 03/04/25

PCS Structural Solutions

MECHANICAL UNIT ANCHORAGE SPREADSHEET

MOMENTS AND REACTIONS (UNFACTORED, INCLUDING OVERSTRENGTH)

	F_1	F_2
Ω_0*$M_{OT\ Seismic}$ =	44115 lb-ft	44115 lb-ft
$M_{OT\ Wind}$ =	7421 lb-ft	14018 lb-ft
$M_{R\ SL}$ =	12738 lb-ft	9404 lb-ft
$M_{R\ DL\ Unit}$ =	26163 lb-ft	19316 lb-ft
$M_{R\ DL\ Curb}$ =	1028 lb-ft	759 lb-ft

P_{SL} =	2602 lb
$P_{DL\ Unit}$ =	5344 lb
$P_{DL\ Curb}$ =	210 lb

	VERTICAL FORCE		OVERTURNING T/C		HORIZ. SHEAR	
LOAD CASE	UPLIFT	DOWNWARD	T_1/C_1	T_2/C_2	V_1	V_2
Ω_0*SEISMIC:	2253 lb	2253 lb	4505 lb	6102 lb	8448 lb	8448 lb
WIND:	3209 lb	N/A	758 lb	1939 lb	2047 lb	3867 lb
SNOW:	N/A	2602 lb	N/A	N/A	N/A	N/A
DEAD:	N/A	5554 lb	N/A	N/A	N/A	N/A

COMBINATIONS (ASD, INCLUDING OVERSTRENGTH)

	OVERTURNING MOMENT	
COMBINATION	OT_1	OT_2
0.6DL-0.6WL	-11862 lb-ft	-3635 lb-ft
0.6DL-0.7(Ω_0*EQ)	14566 lb-ft	18836 lb-ft
DL+SL	-39929 lb-ft	-29480 lb-ft
DL+0.6WL	-31644 lb-ft	-28486 lb-ft
DL+0.45WL+0.75SL	-40084 lb-ft	-33437 lb-ft
DL+0.7(Ω_0*EQ)	-58072 lb-ft	-50956 lb-ft
DL+0.525(Ω_0*EQ)+0.75SL	-59905 lb-ft	-50289 lb-ft

	VERTICAL FORCE	
COMBINATION	UPLIFT	DOWNWARD
0.6*Fv	1926 lb	
±0.7*Fpv	1577 lb	1577 lb
0.6*Fv	1926 lb	
0.45*Fv	1444 lb	
±0.7*Fpv	1577 lb	1577 lb
±0.525*Fpv	1183 lb	1183 lb

	TENSION		COMPRESSION		HORIZ. SHEAR	
RESULTANT COMBINATION	T_1	T_2	C_1	C_2	V_1	V_2
0.6DL-0.6WL	N/A	460 lb	1211 lb	503 lb	-1228 lb	-2320 lb
(0.6-0.14S_{DS})DL-0.7(Ω_0*EQ)	2276 lb	3394 lb	N/A	N/A	-5913 lb	-5913 lb
DL+SL	N/A	N/A	4078 lb	4078 lb	N/A	N/A
DL+0.6WL	N/A	N/A	3232 lb	3940 lb	1228 lb	2320 lb
DL+0.45WL+0.75SL	N/A	N/A	4094 lb	4625 lb	921 lb	1740 lb
(1+0.14S_{DS})DL+0.7(Ω_0*EQ)	N/A	N/A	6719 lb	7837 lb	5913 lb	5913 lb
(1+0.105S_{DS})DL+0.525(Ω_0*EQ) +0.75SL	N/A	N/A	6709 lb	7548 lb	4435 lb	4435 lb

DESIGN LOADS (ASD, INCLUDING OVERSTRENGTH)

	TENSION		COMPRESSION		HORIZ. SHEAR	
	T_1	T_2	C_1	C_2	V_1	V_2
FORCE:	**2276 lb**	**3394 lb**	**6719 lb**	**7837 lb**	**5913 lb**	**5913 lb**

MECHANICAL UNIT ANCHORAGE SPREADSHEET

UNIT NAME:	A9 Condensing Unit	LOCATION:	PUYALLUP, WA
CODE:	IBC 2021, ASCE 7-16		
UNIT EXT. OR INT.?	EXTERNAL		

UNIT INFORMATION: / CURB INFORMATION:

OPERATING WEIGHT (W_p) =	375 lb		IS THERE A CURB?	NO	
			CURB WEIGHT, W_{p-2} =		
UNIT HEIGHT (h) =	35 in	2.9 ft	CURB HEIGHT (h) =		0.0 ft
UNIT WIDTH (w) =	33 in	2.8 ft	CURB WIDTH AT BASE (w) =		0.0 ft
UNIT LENGTH (l) =	42 in	3.5 ft	CURB LENGTH AT BASE (l) =		0.0 ft
C.O.G. (VERTICAL) =	18 in	1.5 ft	(2/3)*UNIT HEIGHT (ASSUMED)		

TOTAL WEIGHT (W_{tot}) =	375 lb		DESIGN WIDTH (w_{des}) =	2.8 ft
h_{wind} =	1.5 ft	(1/2)*(UNIT HT + CURB HT)	DESIGN LENGTH (l_{des}) =	3.5 ft
$h_{seismic}$ =	1.5 ft	CENTER OF GRAVITY + CURB HT		

BUILDING INFORMATION

ARE BUILDING DIMENSIONS KNOWN?	YES	BUILDING WIDTH (B) =	87.8 ft	B*h =	7750 sq ft
		BUILDING LENGTH (L) =	155.0 ft	B*L =	13601 sq ft
		AVERAGE ROOF HT. (h) =	50.0 ft		

DESIGN CRITERIA - GRAVITY

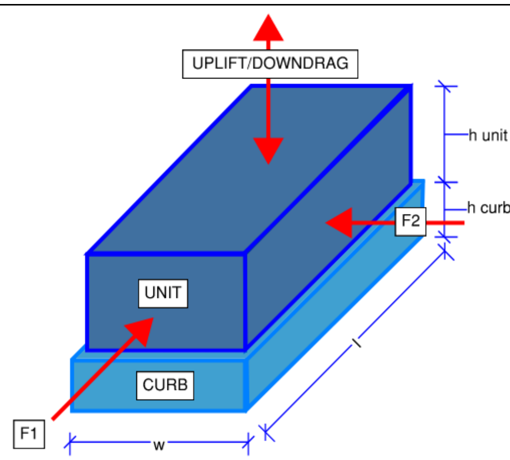
ROOF DEAD (DL) =	20 PSF
ROOF SNOW (SL) =	25 PSF
$P_{roof\,DL}$ =	194 lb
$P_{roof\,SL}$ =	242 lb

DESIGN CRITERIA - LATERAL (THE UNIT DOES NOT HAVE A SIGNIFICANT IMPACT TO THE BUILDING'S CAPACITY TO RESIST WIND OR SEISMIC FORCES)

SEISMIC (ASCE 7-16, CHAPTER 13):

COMPONENT TYPE: AIR-SIDE HVACR, FANS, AIR HANDLERS, A/C UNITS, CABINET HEATERS, AIR DISTRIBUTION BOXES, AND OTHER MECH. COMPONENTS CONSTRUCTED OF SHEET METAL FRAMING

S_{DS} =	1.014	
a_p =	2.5	(TABLE 13.6-1)
I_p =	1.5	(§ 13.1.3)
R_p =	3	(TABLE 13.6-1)
OVERSTRENGTH FACTOR (Ω_o) =	1.5	(TABLE 13.6-1)
ATTACHMENT OR BTM. OF CURB HT. (z) =	50.0 ft	
AVERAGE ROOF HT. (h) =	50.0 ft	
$F_p = 0.4*S_{DS}*(I_p/R_p)*a_p*W_p*[1+2*z/h]$ =	570 lb	(EQN. 13.3-1)
$F_{p\,max} = 1.6*S_{DS}*I_p*W_p$ =	913 lb	(EQN. 13.3-2)
$F_{p\,min} = 0.3*S_{DS}*I_p*W_p$ =	171 lb	(EQN. 13.3-3)
HORIZ. EQ. DESIGN FORCE, F_p =	**570 lb**	
VERT. EQ. DESIGN FORCE, $F_{pv} = \pm 0.2*S_{DS}W_p$ =	**76 lb**	(§ 13.3.1.2)



WIND (ASCE7-16, CHAPTER 29 - DIRECTIONAL PROCEDURE):

WIND SPEED (V) =	108 MPH	
RISK CATEGORY:	IV	(TABLE 1.5-2)
WIND EXPOSURE:	B	(§ 26.7.3)
TOPO. EFFECT (K_{zt}) =	1.0	(FIG. 26.8-1)
K_h =	0.81	(TABLE 26.10-1)
DIRECTIONALITY FACTOR, K_d =	0.85	(TABLE 26.6-1)
GROUND ELEV. FACTOR, K_e =	1.00	(TABLE 26.9-1)

		F_1	F_2	
	A_f =	8.0 sq ft	10.3 sq ft	
	A_r =	9.7 sq ft		
	$q_h = 0.00256*K_d*K_h*K_{zt}*K_e*V^2$ =	20.6 psf		(EQN. 26.10-1)
(§ 29.4.1)	GC_r =	1.90	1.90	
(§ 29.4.1)	GC_r =	1.50		
		F_1	F_2	
(EQN. 29.4-2)	$F_h = q_h*(GC_r)*A_f$ =	**313 lb**	**401 lb**	
(EQN. 29.4-3)	$F_v = q_h*(GC_r)*A_r$ =	**299 lb**		

Project:	Good Samaritan Hospital Nutrition Department Expansion	Job Number:	24-707
Sheet:	of	Name:	KDK
Originating Office:	Tacoma	Date:	03/04/25

PCS Structural Solutions

MECHANICAL UNIT ANCHORAGE SPREADSHEET

MOMENTS AND REACTIONS (UNFACTORED)

	F_1	F_2
$M_{OT\ Seismic}$ =	832 lb-ft	832 lb-ft
$M_{OT\ Wind}$ =	457 lb-ft	585 lb-ft
$M_{R\ SL}$ =	426 lb-ft	333 lb-ft
$M_{R\ DL\ Unit}$ =	660 lb-ft	516 lb-ft
$M_{R\ DL\ Curb}$ =	0 lb-ft	0 lb-ft

P_{SL} =	242 lb
$P_{DL\ Unit}$ =	375 lb
$P_{DL\ Curb}$ =	0 lb

	VERTICAL FORCE		OVERTURNING T/C		HORIZ. SHEAR	
LOAD CASE	UPLIFT	DOWNWARD	T_1/C_1	T_2/C_2	V_1	V_2
SEISMIC:	76 lb	76 lb	236 lb	302 lb	570 lb	570 lb
WIND:	299 lb	N/A	130 lb	213 lb	313 lb	401 lb
SNOW:	N/A	242 lb	N/A	N/A	N/A	N/A
DEAD:	N/A	375 lb	N/A	N/A	N/A	N/A

COMBINATIONS (ASD)

	OVERTURNING MOMENT	
COMBINATION	OT_1	OT_2
0.6DL-0.6WL	-122 lb-ft	42 lb-ft
0.6DL-0.7EQ	186 lb-ft	273 lb-ft
DL+SL	-1086 lb-ft	-848 lb-ft
DL+0.6WL	-934 lb-ft	-867 lb-ft
DL+0.45WL+0.75SL	-1185 lb-ft	-1028 lb-ft
DL+0.7EQ	-1242 lb-ft	-1098 lb-ft
DL+0.525EQ+0.75SL	-1416 lb-ft	-1202 lb-ft

	VERTICAL FORCE	
COMBINATION	UPLIFT	DOWNWARD
$0.6*F_v$	179 lb	
$\pm 0.7*F_{pv}$	53 lb	53 lb
$0.6*F_v$	179 lb	
$0.45*F_v$	134 lb	
$\pm 0.7*F_{pv}$	53 lb	53 lb
$\pm 0.525*F_{pv}$	40 lb	40 lb

	TENSION		COMPRESSION		HORIZ. SHEAR	
RESULTANT COMBINATION	T_1	T_2	C_1	C_2	V_1	V_2
0.6DL-0.6WL	55 lb	105 lb	35 lb	N/A	-188 lb	-241 lb
$(0.6-0.14S_{DS})$DL-0.7EQ	79 lb	126 lb	N/A	N/A	-399 lb	-399 lb
DL+SL	N/A	N/A	309 lb	309 lb	N/A	N/A
DL+0.6WL	N/A	N/A	265 lb	315 lb	188 lb	241 lb
DL+0.45WL+0.75SL	N/A	N/A	337 lb	374 lb	141 lb	181 lb
$(1+0.14S_{DS})$DL+0.7EQ	N/A	N/A	379 lb	426 lb	399 lb	399 lb
$(1+0.105S_{DS})$DL+0.525EQ+0.75SL	N/A	N/A	422 lb	457 lb	299 lb	299 lb

DESIGN LOADS (ASD) AND ATTACHMENT DESIGN

	TENSION		COMPRESSION		HORIZ. SHEAR	
	T_1	T_2	C_1	C_2	V_1	V_2
FORCE:	**79 lb**	**126 lb**	**422 lb**	**457 lb**	**399 lb**	**399 lb**
# OF SCREWS/SIDE:	**1 SCREWS**	**1 SCREWS**			**1 SCREWS**	**1 SCREWS**

SCREW SIZE	CURB/UNIT THICKNESS
#14	16ga

Project: Good Samaritan Hospital Nutrition Department Expansion | Job Number: 24-707

Sheet: of | Name: KDK

Originating Office: Tacoma | Date: 03/04/25

PCS Structural Solutions

MECHANICAL UNIT ANCHORAGE SPREADSHEET

MOMENTS AND REACTIONS (UNFACTORED, INCLUDING OVERSTRENGTH)

	F_1	F_2
Ω_0*$M_{OT\ Seismic}$ =	1248 lb-ft	1248 lb-ft
$M_{OT\ Wind}$ =	457 lb-ft	585 lb-ft
$M_{R\ SL}$ =	426 lb-ft	333 lb-ft
$M_{R\ DL\ Unit}$ =	660 lb-ft	516 lb-ft
$M_{R\ DL\ Curb}$ =	0 lb-ft	0 lb-ft

P_{SL} =	242 lb
$P_{DL\ Unit}$ =	375 lb
$P_{DL\ Curb}$ =	0 lb

	VERTICAL FORCE		OVERTURNING T/C		HORIZ. SHEAR	
LOAD CASE	UPLIFT	DOWNWARD	T_1/C_1	T_2/C_2	V_1	V_2
Ω_0*SEISMIC:	114 lb	114 lb	354 lb	454 lb	856 lb	856 lb
WIND:	299 lb	N/A	130 lb	213 lb	313 lb	401 lb
SNOW:	N/A	242 lb	N/A	N/A	N/A	N/A
DEAD:	N/A	375 lb	N/A	N/A	N/A	N/A

COMBINATIONS (ASD, INCLUDING OVERSTRENGTH)

	OVERTURNING MOMENT	
COMBINATION	OT_1	OT_2
0.6DL-0.6WL	-122 lb-ft	42 lb-ft
0.6DL-0.7(Ω_0*EQ)	477 lb-ft	564 lb-ft
DL+SL	-1086 lb-ft	-848 lb-ft
DL+0.6WL	-934 lb-ft	-867 lb-ft
DL+0.45WL+0.75SL	-1185 lb-ft	-1028 lb-ft
DL+0.7(Ω_0*EQ)	-1534 lb-ft	-1389 lb-ft
DL+0.525(Ω_0*EQ)+0.75SL	-1635 lb-ft	-1420 lb-ft

	VERTICAL FORCE	
COMBINATION	UPLIFT	DOWNWARD
0.6*Fv	179 lb	
±0.7*Fpv	80 lb	80 lb
0.6*Fv	179 lb	
0.45*Fv	134 lb	
±0.7*Fpv	80 lb	80 lb
±0.525*Fpv	60 lb	60 lb

	TENSION		COMPRESSION		HORIZ. SHEAR	
RESULTANT COMBINATION	T_1	T_2	C_1	C_2	V_1	V_2
0.6DL-0.6WL	55 lb	105 lb	35 lb	N/A	-188 lb	-241 lb
(0.6-0.14S_{DS})DL-0.7(Ω_0*EQ)	175 lb	245 lb	N/A	N/A	-599 lb	-599 lb
DL+SL	N/A	N/A	309 lb	309 lb	N/A	N/A
DL+0.6WL	N/A	N/A	265 lb	315 lb	188 lb	241 lb
DL+0.45WL+0.75SL	N/A	N/A	337 lb	374 lb	141 lb	181 lb
(1+0.14S_{DS})DL+0.7(Ω_0*EQ)	N/A	N/A	475 lb	545 lb	599 lb	599 lb
(1+0.105S_{DS})DL+0.525(Ω_0*EQ) +0.75SL	N/A	N/A	494 lb	546 lb	449 lb	449 lb

DESIGN LOADS (ASD, INCLUDING OVERSTRENGTH)

	TENSION		COMPRESSION		HORIZ. SHEAR	
	T_1	T_2	C_1	C_2	V_1	V_2
FORCE:	**175 lb**	**245 lb**	**494 lb**	**546 lb**	**599 lb**	**599 lb**

Project:	Good Samaritan Hospital Nutrition Department Expansion	Job Number:	24-707
Sheet:	of	Name:	KDK
Originating Office:	Tacoma	Date:	3/4/2025

MECHANICAL UNIT ANCHORAGE SPREADSHEET

UNIT NAME:	A17 Condensing Unit	LOCATION:	PUYALLUP, WA
CODE:	IBC 2021, ASCE 7-16		
UNIT EXT. OR INT.?	EXTERNAL		

UNIT INFORMATION: / CURB INFORMATION:

OPERATING WEIGHT (W_p) =	850 lb		IS THERE A CURB?	NO	
			CURB WEIGHT, W_{p-2} =		
UNIT HEIGHT (h) =	36 in	3.0 ft	CURB HEIGHT (h) =		0.0 ft
UNIT WIDTH (w) =	38 in	3.2 ft	CURB WIDTH AT BASE (w) =		0.0 ft
UNIT LENGTH (l) =	47 in	3.9 ft	CURB LENGTH AT BASE (l) =		0.0 ft
C.O.G. (VERTICAL) =	18 in	1.5 ft	(2/3)*UNIT HEIGHT (ASSUMED)		
TOTAL WEIGHT (W_{tot}) =	850 lb		DESIGN WIDTH (w_{des}) =	3.2 ft	
h_{wind} =	1.5 ft	(1/2)*(UNIT HT + CURB HT)	DESIGN LENGTH (l_{des}) =	3.9 ft	
$h_{seismic}$ =	1.5 ft	CENTER OF GRAVITY + CURB HT			

BUILDING INFORMATION

ARE BUILDING DIMENSIONS KNOWN?	YES	BUILDING WIDTH (B) =	87.8 ft	B*h =	7750 sq ft
		BUILDING LENGTH (L) =	155.0 ft	B*L =	13601 sq ft
		AVERAGE ROOF HT. (h) =	50.0 ft		

DESIGN CRITERIA - GRAVITY

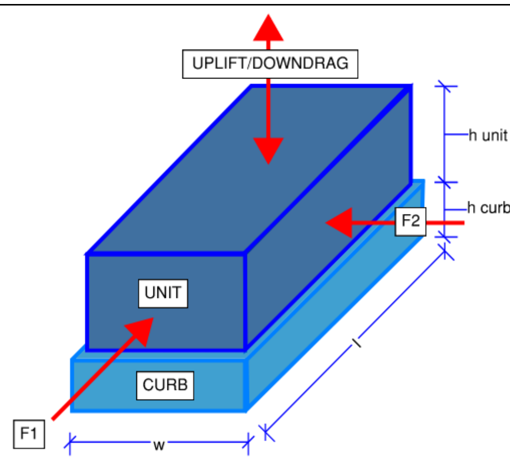
ROOF DEAD (DL) =	20 PSF
ROOF SNOW (SL) =	25 PSF
$P_{roof\ DL}$ =	251 lb
$P_{roof\ SL}$ =	314 lb

DESIGN CRITERIA - LATERAL (THE UNIT DOES NOT HAVE A SIGNIFICANT IMPACT TO THE BUILDING'S CAPACITY TO RESIST WIND OR SEISMIC FORCES)

SEISMIC (ASCE 7-16, CHAPTER 13):

COMPONENT TYPE: AIR-SIDE HVACR, FANS, AIR HANDLERS, A/C UNITS, CABINET HEATERS, AIR DISTRIBUTION BOXES, AND OTHER MECH. COMPONENTS CONSTRUCTED OF SHEET METAL FRAMING

S_{DS} =	1.014	
a_p =	2.5	(TABLE 13.6-1)
I_p =	1.5	(§ 13.1.3)
R_p =	3	(TABLE 13.6-1)
OVERSTRENGTH FACTOR (Ω_o) =	1.5	(TABLE 13.6-1)
ATTACHMENT OR BTM. OF CURB HT. (z) =	50.0 ft	
AVERAGE ROOF HT. (h) =	50.0 ft	
$F_p = 0.4*S_{DS}*(I_p/R_p)*a_p*W_p*[1+2*z/h]$ =	1293 lb	(EQN. 13.3-1)
$F_{p\ max} = 1.6*S_{DS}*I_p*W_p$ =	2069 lb	(EQN. 13.3-2)
$F_{p\ min} = 0.3*S_{DS}*I_p*W_p$ =	388 lb	(EQN. 13.3-3)
HORIZ. EQ. DESIGN FORCE, F_p =	**1293 lb**	
VERT. EQ. DESIGN FORCE, $F_{pv} = \pm 0.2*S_{DS}W_p$ =	**172 lb**	(§ 13.3.1.2)



WIND (ASCE7-16, CHAPTER 29 - DIRECTIONAL PROCEDURE):

WIND SPEED (V) =	108 MPH	
RISK CATEGORY:	IV	(TABLE 1.5-2)
WIND EXPOSURE:	B	(§ 26.7.3)
TOPO. EFFECT (K_{zt}) =	1.0	(FIG. 26.8-1)
K_h =	0.81	(TABLE 26.10-1)
DIRECTIONALITY FACTOR, K_d =	0.85	(TABLE 26.6-1)
GROUND ELEV. FACTOR, K_e =	1.00	(TABLE 26.9-1)

		F_1	F_2	
	A_f =	9.7 sq ft	11.9 sq ft	
	A_r =	12.6 sq ft		
	$q_h = 0.00256*K_d*K_h*K_{zt}*K_e*V^2$ =	20.6 psf		(EQN. 26.10-1)
(§ 29.4.1)	GC_r =	1.90	1.90	
(§ 29.4.1)	GC_r =	1.50		
		F_1	F_2	
(EQN. 29.4-2)	$F_h = q_h*(GC_r)*A_f$ =	**378 lb**	**466 lb**	
(EQN. 29.4-3)	$F_v = q_h*(GC_r)*A_r$ =	**387 lb**		

Project:	Good Samaritan Hospital Nutrition Department Expansion	Job Number:	24-707
Sheet:	of	Name:	KDK
Originating Office:	Tacoma	Date:	03/04/25

PCS Structural Solutions

MECHANICAL UNIT ANCHORAGE SPREADSHEET

MOMENTS AND REACTIONS (UNFACTORED)

	F_1	F_2
$M_{OT\ Seismic}$ =	1939 lb-ft	1939 lb-ft
$M_{OT\ Wind}$ =	573 lb-ft	706 lb-ft
$M_{R\ SL}$ =	617 lb-ft	501 lb-ft
$M_{R\ DL\ Unit}$ =	1671 lb-ft	1357 lb-ft
$M_{R\ DL\ Curb}$ =	0 lb-ft	0 lb-ft

P_{SL} =	314 lb
$P_{DL\ Unit}$ =	850 lb
$P_{DL\ Curb}$ =	0 lb

	VERTICAL FORCE		OVERTURNING T/C		HORIZ. SHEAR	
LOAD CASE	UPLIFT	DOWNWARD	T_1/C_1	T_2/C_2	V_1	V_2
SEISMIC:	172 lb	172 lb	493 lb	607 lb	1293 lb	1293 lb
WIND:	387 lb	N/A	146 lb	221 lb	378 lb	466 lb
SNOW:	N/A	314 lb	N/A	N/A	N/A	N/A
DEAD:	N/A	850 lb	N/A	N/A	N/A	N/A

COMBINATIONS (ASD)

	OVERTURNING MOMENT	
COMBINATION	OT_1	OT_2
0.6DL-0.6WL	-659 lb-ft	-391 lb-ft
0.6DL-0.7EQ	355 lb-ft	543 lb-ft
DL+SL	-2288 lb-ft	-1858 lb-ft
DL+0.6WL	-2015 lb-ft	-1780 lb-ft
DL+0.45WL+0.75SL	-2392 lb-ft	-2050 lb-ft
DL+0.7EQ	-3029 lb-ft	-2714 lb-ft
DL+0.525EQ+0.75SL	-3152 lb-ft	-2751 lb-ft

	VERTICAL FORCE	
COMBINATION	UPLIFT	DOWNWARD
$0.6*F_v$	232 lb	
$\pm 0.7*F_{pv}$	121 lb	121 lb
$0.6*F_v$	232 lb	
$0.45*F_v$	174 lb	
$\pm 0.7*F_{pv}$	121 lb	121 lb
$\pm 0.525*F_{pv}$	90 lb	90 lb

	TENSION		COMPRESSION		HORIZ. SHEAR	
RESULTANT COMBINATION	T_1	T_2	C_1	C_2	V_1	V_2
0.6DL-0.6WL	N/A	N/A	168 lb	122 lb	-227 lb	-279 lb
$(0.6-0.14S_{DS})$DL-0.7EQ	151 lb	231 lb	N/A	N/A	-905 lb	-905 lb
DL+SL	N/A	N/A	582 lb	582 lb	N/A	N/A
DL+0.6WL	N/A	N/A	512 lb	558 lb	227 lb	279 lb
DL+0.45WL+0.75SL	N/A	N/A	608 lb	642 lb	170 lb	210 lb
$(1+0.14S_{DS})$DL+0.7EQ	N/A	N/A	831 lb	911 lb	905 lb	905 lb
$(1+0.105S_{DS})$DL+0.525EQ+0.75SL	N/A	N/A	847 lb	907 lb	679 lb	679 lb

DESIGN LOADS (ASD) AND ATTACHMENT DESIGN

	TENSION		COMPRESSION		HORIZ. SHEAR	
	T_1	T_2	C_1	C_2	V_1	V_2
FORCE:	**151 lb**	**231 lb**	**847 lb**	**911 lb**	**905 lb**	**905 lb**
# OF SCREWS/SIDE:	**1 SCREWS**	**2 SCREWS**			**2 SCREWS**	**2 SCREWS**

SCREW SIZE	CURB/UNIT THICKNESS
#14	16ga

Project: Good Samaritan Hospital Nutrition Department Expansion Job Number: 24-707

Sheet: ______ of ______ Name: KDK

Originating Office: Tacoma Date: 03/04/25

PCS Structural Solutions

MECHANICAL UNIT ANCHORAGE SPREADSHEET

MOMENTS AND REACTIONS (UNFACTORED, INCLUDING OVERSTRENGTH)

	F_1	F_2
Ω_0*$M_{OT\ Seismic}$ =	2909 lb-ft	2909 lb-ft
$M_{OT\ Wind}$ =	573 lb-ft	706 lb-ft
$M_{R\ SL}$ =	617 lb-ft	501 lb-ft
$M_{R\ DL\ Unit}$ =	1671 lb-ft	1357 lb-ft
$M_{R\ DL\ Curb}$ =	0 lb-ft	0 lb-ft

P_{SL} =	314 lb
$P_{DL\ Unit}$ =	850 lb
$P_{DL\ Curb}$ =	0 lb

	VERTICAL FORCE		OVERTURNING T/C		HORIZ. SHEAR	
LOAD CASE	UPLIFT	DOWNWARD	T_1/C_1	T_2/C_2	V_1	V_2
Ω_0*SEISMIC:	259 lb	259 lb	740 lb	911 lb	1939 lb	1939 lb
WIND:	387 lb	N/A	146 lb	221 lb	378 lb	466 lb
SNOW:	N/A	314 lb	N/A	N/A	N/A	N/A
DEAD:	N/A	850 lb	N/A	N/A	N/A	N/A

COMBINATIONS (ASD, INCLUDING OVERSTRENGTH)

	OVERTURNING MOMENT	
COMBINATION	OT_1	OT_2
0.6DL-0.6WL	-659 lb-ft	-391 lb-ft
0.6DL-0.7(Ω_0*EQ)	1034 lb-ft	1222 lb-ft
DL+SL	-2288 lb-ft	-1858 lb-ft
DL+0.6WL	-2015 lb-ft	-1780 lb-ft
DL+0.45WL+0.75SL	-2392 lb-ft	-2050 lb-ft
DL+0.7(Ω_0*EQ)	-3707 lb-ft	-3393 lb-ft
DL+0.525(Ω_0*EQ)+0.75SL	-3661 lb-ft	-3260 lb-ft

	VERTICAL FORCE	
COMBINATION	UPLIFT	DOWNWARD
0.6*Fv	232 lb	
±0.7*Fpv	181 lb	181 lb
0.6*Fv	232 lb	
0.45*Fv	174 lb	
±0.7*Fpv	181 lb	181 lb
±0.525*Fpv	136 lb	136 lb

	TENSION		COMPRESSION		HORIZ. SHEAR	
RESULTANT COMBINATION	T_1	T_2	C_1	C_2	V_1	V_2
0.6DL-0.6WL	N/A	N/A	168 lb	122 lb	-227 lb	-279 lb
(0.6-0.14S_{DS})DL-0.7(Ω_0*EQ)	353 lb	473 lb	N/A	N/A	-1357 lb	-1357 lb
DL+SL	N/A	N/A	582 lb	582 lb	N/A	N/A
DL+0.6WL	N/A	N/A	512 lb	558 lb	227 lb	279 lb
DL+0.45WL+0.75SL	N/A	N/A	608 lb	642 lb	170 lb	210 lb
(1+0.14S_{DS})DL+0.7(Ω_0*EQ)	N/A	N/A	1033 lb	1153 lb	1357 lb	1357 lb
(1+0.105S_{DS})DL+0.525(Ω_0*EQ) +0.75SL	N/A	N/A	999 lb	1089 lb	1018 lb	1018 lb

DESIGN LOADS (ASD, INCLUDING OVERSTRENGTH)

	TENSION		COMPRESSION		HORIZ. SHEAR	
	T_1	T_2	C_1	C_2	V_1	V_2
FORCE:	**353 lb**	**473 lb**	**1033 lb**	**1153 lb**	**1357 lb**	**1357 lb**

Project:	Good Samaritan Hospital Nutrition Department Expansion	Job Number:	24-707
Sheet:	of	Name:	KDK
Originating Office:	Tacoma	Date:	3/4/2025

PCS Structural Solutions

MECHANICAL UNIT ANCHORAGE SPREADSHEET

UNIT NAME:	A23 Condensing Unit	LOCATION:	PUYALLUP, WA
CODE:	IBC 2021, ASCE 7-16		
UNIT EXT. OR INT.?	EXTERNAL		

UNIT INFORMATION: / CURB INFORMATION:

OPERATING WEIGHT (W_p) =	352 lb		IS THERE A CURB?	NO	
			CURB WEIGHT, W_{p-2} =		
UNIT HEIGHT (h) =	35 in	2.9 ft	CURB HEIGHT (h) =		0.0 ft
UNIT WIDTH (w) =	33 in	2.8 ft	CURB WIDTH AT BASE (w) =		0.0 ft
UNIT LENGTH (l) =	42 in	3.5 ft	CURB LENGTH AT BASE (l) =		0.0 ft
C.O.G. (VERTICAL) =	18 in	1.5 ft	(2/3)*UNIT HEIGHT (ASSUMED)		

TOTAL WEIGHT (W_{tot}) =	352 lb		DESIGN WIDTH (w_{des}) =	2.8 ft
h_{wind} =	1.5 ft	(1/2)*(UNIT HT + CURB HT)	DESIGN LENGTH (l_{des}) =	3.5 ft
$h_{seismic}$ =	1.5 ft	CENTER OF GRAVITY + CURB HT		

BUILDING INFORMATION

ARE BUILDING DIMENSIONS KNOWN?	YES	BUILDING WIDTH (B) =	87.8 ft	B*h =	7750 sq ft
		BUILDING LENGTH (L) =	155.0 ft	B*L =	13601 sq ft
		AVERAGE ROOF HT. (h) =	50.0 ft		

DESIGN CRITERIA - GRAVITY

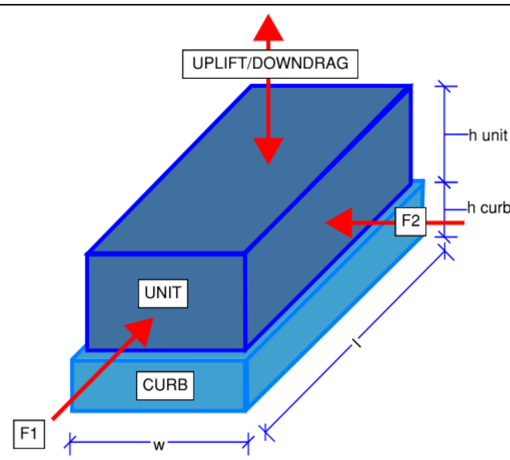
ROOF DEAD (DL) =	20 PSF
ROOF SNOW (SL) =	25 PSF
$P_{roof\ DL}$ =	194 lb
$P_{roof\ SL}$ =	242 lb

DESIGN CRITERIA - LATERAL (THE UNIT DOES NOT HAVE A SIGNIFICANT IMPACT TO THE BUILDING'S CAPACITY TO RESIST WIND OR SEISMIC FORCES)

SEISMIC (ASCE 7-16, CHAPTER 13):

COMPONENT TYPE: AIR-SIDE HVACR, FANS, AIR HANDLERS, A/C UNITS, CABINET HEATERS, AIR DISTRIBUTION BOXES, AND OTHER MECH. COMPONENTS CONSTRUCTED OF SHEET METAL FRAMING

S_{DS} =	1.014	
a_p =	2.5	(TABLE 13.6-1)
I_p =	1.5	(§ 13.1.3)
R_p =	3	(TABLE 13.6-1)
OVERSTRENGTH FACTOR (Ω_o) =	1.5	(TABLE 13.6-1)
ATTACHMENT OR BTM. OF CURB HT. (z) =	50.0 ft	
AVERAGE ROOF HT. (h) =	50.0 ft	
$F_p = 0.4*S_{DS}*(I_p/R_p)*a_p*W_p*[1+2*z/h]$ =	535 lb	(EQN. 13.3-1)
$F_{p\ max} = 1.6*S_{DS}*I_p*W_p$ =	857 lb	(EQN. 13.3-2)
$F_{p\ min} = 0.3*S_{DS}*I_p*W_p$ =	161 lb	(EQN. 13.3-3)
HORIZ. EQ. DESIGN FORCE, F_p =	**535 lb**	
VERT. EQ. DESIGN FORCE, $F_{pv} = \pm 0.2*S_{DS}W_p$ =	**71 lb**	(§ 13.3.1.2)



WIND (ASCE7-16, CHAPTER 29 - DIRECTIONAL PROCEDURE):

WIND SPEED (V) =	108 MPH	
RISK CATEGORY:	IV	(TABLE 1.5-2)
WIND EXPOSURE:	B	(§ 26.7.3)
TOPO. EFFECT (K_{zt}) =	1.0	(FIG. 26.8-1)
K_h =	0.81	(TABLE 26.10-1)
DIRECTIONALITY FACTOR, K_d =	0.85	(TABLE 26.6-1)
GROUND ELEV. FACTOR, K_e =	1.00	(TABLE 26.9-1)

		F_1	F_2	
	A_f =	8.0 sq ft	10.3 sq ft	
	A_r =	9.7 sq ft		
	$q_h = 0.00256*K_d*K_h*K_{zt}*K_e*V^2$ =	20.6 psf		(EQN. 26.10-1)
(§ 29.4.1)	GC_r =	1.90	1.90	
(§ 29.4.1)	GC_r =	1.50		

		F_1	F_2
(EQN. 29.4-2)	$F_h = q_h*(GC_r)*A_f$ =	**313 lb**	**401 lb**
(EQN. 29.4-3)	$F_v = q_h*(GC_r)*A_r$ =	**299 lb**	

PCS Structural Solutions

Project:	Good Samaritan Hospital Nutrition Department Expansion	Job Number:	24-707
Sheet:	of	Name:	KDK
Originating Office:	Tacoma	Date:	03/04/25

MECHANICAL UNIT ANCHORAGE SPREADSHEET

MOMENTS AND REACTIONS (UNFACTORED)

	F_1	F_2		
$M_{OT\ Seismic}$ =	781 lb-ft	781 lb-ft		
$M_{OT\ Wind}$ =	457 lb-ft	585 lb-ft		
$M_{R\ SL}$ =	426 lb-ft	333 lb-ft	P_{SL} =	242 lb
$M_{R\ DL\ Unit}$ =	620 lb-ft	484 lb-ft	$P_{DL\ Unit}$ =	352 lb
$M_{R\ DL\ Curb}$ =	0 lb-ft	0 lb-ft	$P_{DL\ Curb}$ =	0 lb

	VERTICAL FORCE		OVERTURNING T/C		HORIZ. SHEAR	
LOAD CASE	UPLIFT	DOWNWARD	T_1/C_1	T_2/C_2	V_1	V_2
SEISMIC:	71 lb	71 lb	222 lb	284 lb	535 lb	535 lb
WIND:	299 lb	N/A	130 lb	213 lb	313 lb	401 lb
SNOW:	N/A	242 lb	N/A	N/A	N/A	N/A
DEAD:	N/A	352 lb	N/A	N/A	N/A	N/A

COMBINATIONS (ASD)

	OVERTURNING MOMENT	
COMBINATION	OT_1	OT_2
0.6DL-0.6WL	-98 lb-ft	61 lb-ft
0.6DL-0.7EQ	175 lb-ft	256 lb-ft
DL+SL	-1046 lb-ft	-817 lb-ft
DL+0.6WL	-894 lb-ft	-835 lb-ft
DL+0.45WL+0.75SL	-1145 lb-ft	-997 lb-ft
DL+0.7EQ	-1166 lb-ft	-1031 lb-ft
DL+0.525EQ+0.75SL	-1349 lb-ft	-1144 lb-ft

	VERTICAL FORCE	
COMBINATION	UPLIFT	DOWNWARD
$0.6*F_v$	179 lb	
$\pm 0.7*F_{pv}$	50 lb	50 lb
$0.6*F_v$	179 lb	
$0.45*F_v$	134 lb	
$\pm 0.7*F_{pv}$	50 lb	50 lb
$\pm 0.525*F_{pv}$	37 lb	37 lb

	TENSION		COMPRESSION		HORIZ. SHEAR	
RESULTANT COMBINATION	T_1	T_2	C_1	C_2	V_1	V_2
0.6DL-0.6WL	62 lb	112 lb	28 lb	N/A	-188 lb	-241 lb
$(0.6-0.14S_{DS})$DL-0.7EQ	75 lb	118 lb	N/A	N/A	-375 lb	-375 lb
DL+SL	N/A	N/A	297 lb	297 lb	N/A	N/A
DL+0.6WL	N/A	N/A	254 lb	304 lb	188 lb	241 lb
DL+0.45WL+0.75SL	N/A	N/A	325 lb	362 lb	141 lb	181 lb
$(1+0.14S_{DS})$DL+0.7EQ	N/A	N/A	356 lb	400 lb	375 lb	375 lb
$(1+0.105S_{DS})$DL+0.525EQ+0.75SL	N/A	N/A	402 lb	435 lb	281 lb	281 lb

DESIGN LOADS (ASD) AND ATTACHMENT DESIGN

	TENSION		COMPRESSION		HORIZ. SHEAR	
	T_1	T_2	C_1	C_2	V_1	V_2
FORCE:	**75 lb**	**118 lb**	**402 lb**	**435 lb**	**375 lb**	**375 lb**
# OF SCREWS/SIDE:	**1 SCREWS**	**1 SCREWS**			**1 SCREWS**	**1 SCREWS**

SCREW SIZE	CURB/UNIT THICKNESS
#14	16ga

Project:	Good Samaritan Hospital Nutrition Department Expansion	Job Number:	24-707
Sheet:	of	Name:	KDK
Originating Office:	Tacoma	Date:	03/04/25

PCS Structural Solutions

MECHANICAL UNIT ANCHORAGE SPREADSHEET

MOMENTS AND REACTIONS (UNFACTORED, INCLUDING OVERSTRENGTH)

	F_1	F_2
$\Omega_0 * M_{OT\ Seismic}$ =	1171 lb-ft	1171 lb-ft
$M_{OT\ Wind}$ =	457 lb-ft	585 lb-ft
$M_{R\ SL}$ =	426 lb-ft	333 lb-ft
$M_{R\ DL\ Unit}$ =	620 lb-ft	484 lb-ft
$M_{R\ DL\ Curb}$ =	0 lb-ft	0 lb-ft

P_{SL} =	242 lb
$P_{DL\ Unit}$ =	352 lb
$P_{DL\ Curb}$ =	0 lb

	VERTICAL FORCE		OVERTURNING T/C		HORIZ. SHEAR	
LOAD CASE	UPLIFT	DOWNWARD	T_1/C_1	T_2/C_2	V_1	V_2
Ω_0*SEISMIC:	107 lb	107 lb	333 lb	426 lb	803 lb	803 lb
WIND:	299 lb	N/A	130 lb	213 lb	313 lb	401 lb
SNOW:	N/A	242 lb	N/A	N/A	N/A	N/A
DEAD:	N/A	352 lb	N/A	N/A	N/A	N/A

COMBINATIONS (ASD, INCLUDING OVERSTRENGTH)

	OVERTURNING MOMENT	
COMBINATION	OT_1	OT_2
0.6DL-0.6WL	-98 lb-ft	61 lb-ft
0.6DL-0.7(Ω_0*EQ)	448 lb-ft	529 lb-ft
DL+SL	-1046 lb-ft	-817 lb-ft
DL+0.6WL	-894 lb-ft	-835 lb-ft
DL+0.45WL+0.75SL	-1145 lb-ft	-997 lb-ft
DL+0.7(Ω_0*EQ)	-1439 lb-ft	-1304 lb-ft
DL+0.525(Ω_0*EQ)+0.75SL	-1554 lb-ft	-1348 lb-ft

	VERTICAL FORCE	
COMBINATION	UPLIFT	DOWNWARD
0.6*Fv	179 lb	
±0.7*Fpv	75 lb	75 lb
0.6*Fv	179 lb	
0.45*Fv	134 lb	
±0.7*Fpv	75 lb	75 lb
±0.525*Fpv	56 lb	56 lb

	TENSION		COMPRESSION		HORIZ. SHEAR	
RESULTANT COMBINATION	T_1	T_2	C_1	C_2	V_1	V_2
0.6DL-0.6WL	62 lb	112 lb	28 lb	N/A	-188 lb	-241 lb
(0.6-0.14S_{DS})DL-0.7(Ω_0*EQ)	165 lb	230 lb	N/A	N/A	-562 lb	-562 lb
DL+SL	N/A	N/A	297 lb	297 lb	N/A	N/A
DL+0.6WL	N/A	N/A	254 lb	304 lb	188 lb	241 lb
DL+0.45WL+0.75SL	N/A	N/A	325 lb	362 lb	141 lb	181 lb
(1+0.14S_{DS})DL+0.7(Ω_0*EQ)	N/A	N/A	446 lb	512 lb	562 lb	562 lb
(1+0.105S_{DS})DL+0.525(Ω_0*EQ) +0.75SL	N/A	N/A	470 lb	518 lb	422 lb	422 lb

DESIGN LOADS (ASD, INCLUDING OVERSTRENGTH)

	TENSION		COMPRESSION		HORIZ. SHEAR	
	T_1	T_2	C_1	C_2	V_1	V_2
FORCE:	**165 lb**	**230 lb**	**470 lb**	**518 lb**	**562 lb**	**562 lb**

Structural Solutions

Project:	Good Samaritan Hospital Nutrition Department Expansion	Job Number:	24-707
Sheet:	of	Name:	KDK
Originating Office:	Tacoma	Date:	3/4/2025

MECHANICAL UNIT ANCHORAGE SPREADSHEET

UNIT NAME:	A26 Condensing Unit	LOCATION:	PUYALLUP, WA
CODE:	IBC 2021, ASCE 7-16		
UNIT EXT. OR INT.?	EXTERNAL		

UNIT INFORMATION: / CURB INFORMATION:

OPERATING WEIGHT (W_p) =	205 lb		IS THERE A CURB?	NO	
			CURB WEIGHT, W_{p-2} =		
UNIT HEIGHT (h) =	19 in	1.6 ft	CURB HEIGHT (h) =		0.0 ft
UNIT WIDTH (w) =	28 in	2.4 ft	CURB WIDTH AT BASE (w) =		0.0 ft
UNIT LENGTH (l) =	38 in	3.2 ft	CURB LENGTH AT BASE (l) =		0.0 ft
C.O.G. (VERTICAL) =	9 in	0.7 ft	(2/3)*UNIT HEIGHT (ASSUMED)		

TOTAL WEIGHT (W_{tot}) =	205 lb		DESIGN WIDTH (w_{des}) =	2.4 ft
h_{wind} =	0.8 ft	(1/2)*(UNIT HT + CURB HT)	DESIGN LENGTH (l_{des}) =	3.2 ft
$h_{seismic}$ =	0.7 ft	CENTER OF GRAVITY + CURB HT		

BUILDING INFORMATION

ARE BUILDING DIMENSIONS KNOWN?	YES	BUILDING WIDTH (B) =	87.8 ft	B*h =	7750 sq ft
		BUILDING LENGTH (L) =	155.0 ft	B*L =	13601 sq ft
		AVERAGE ROOF HT. (h) =	50.0 ft		

DESIGN CRITERIA - GRAVITY

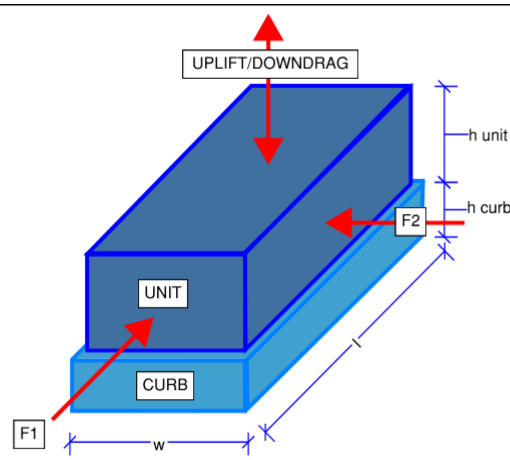
ROOF DEAD (DL) =	20 PSF
ROOF SNOW (SL) =	25 PSF
$P_{roof\ DL}$ =	150 lb
$P_{roof\ SL}$ =	188 lb

DESIGN CRITERIA - LATERAL (THE UNIT DOES NOT HAVE A SIGNIFICANT IMPACT TO THE BUILDING'S CAPACITY TO RESIST WIND OR SEISMIC FORCES)

SEISMIC (ASCE 7-16, CHAPTER 13):

COMPONENT TYPE: AIR-SIDE HVACR, FANS, AIR HANDLERS, A/C UNITS, CABINET HEATERS, AIR DISTRIBUTION BOXES, AND OTHER MECH. COMPONENTS CONSTRUCTED OF SHEET METAL FRAMING

S_{DS} =	1.014	
a_p =	2.5	(TABLE 13.6-1)
I_p =	1.5	(§ 13.1.3)
R_p =	3	(TABLE 13.6-1)
OVERSTRENGTH FACTOR (Ω_o) =	1.5	(TABLE 13.6-1)
ATTACHMENT OR BTM. OF CURB HT. (z) =	50.0 ft	
AVERAGE ROOF HT. (h) =	50.0 ft	
$F_p = 0.4*S_{DS}*(I_p/R_p)*a_p*W_p*[1+2*z/h]$ =	312 lb	(EQN. 13.3-1)
$F_{p\ max} = 1.6*S_{DS}*I_p*W_p$ =	499 lb	(EQN. 13.3-2)
$F_{p\ min} = 0.3*S_{DS}*I_p*W_p$ =	94 lb	(EQN. 13.3-3)
HORIZ. EQ. DESIGN FORCE, F_p =	**312 lb**	
VERT. EQ. DESIGN FORCE, $F_{pv} = \pm 0.2*S_{DS}W_p$ =	**42 lb**	(§ 13.3.1.2)



WIND (ASCE7-16, CHAPTER 29 - DIRECTIONAL PROCEDURE):

WIND SPEED (V) =	108 MPH	
RISK CATEGORY:	IV	(TABLE 1.5-2)
WIND EXPOSURE:	B	(§ 26.7.3)
TOPO. EFFECT (K_{zt}) =	1.0	(FIG. 26.8-1)
K_h =	0.81	(TABLE 26.10-1)
DIRECTIONALITY FACTOR, K_d =	0.85	(TABLE 26.6-1)
GROUND ELEV. FACTOR, K_e =	1.00	(TABLE 26.9-1)

		F_1	F_2	
	A_f =	3.7 sq ft	5.0 sq ft	
	A_r =	7.5 sq ft		
	$q_h = 0.00256*K_d*K_h*K_{zt}*K_e*V^2$ =	20.6 psf		(EQN. 26.10-1)
(§ 29.4.1)	GC_r =	1.90	1.90	
(§ 29.4.1)	GC_r =	1.50		

		F_1	F_2
(EQN. 29.4-2)	$F_h = q_h*(GC_r)*A_f$ =	**144 lb**	**195 lb**
(EQN. 29.4-3)	$F_v = q_h*(GC_r)*A_r$ =	**231 lb**	

MECHANICAL UNIT ANCHORAGE SPREADSHEET

MOMENTS AND REACTIONS (UNFACTORED)

	F_1	F_2
$M_{OT\ Seismic}$ =	221 lb-ft	221 lb-ft
$M_{OT\ Wind}$ =	112 lb-ft	152 lb-ft
$M_{R\ SL}$ =	299 lb-ft	221 lb-ft
$M_{R\ DL\ Unit}$ =	327 lb-ft	241 lb-ft
$M_{R\ DL\ Curb}$ =	0 lb-ft	0 lb-ft

P_{SL} =	188 lb
$P_{DL\ Unit}$ =	205 lb
$P_{DL\ Curb}$ =	0 lb

	VERTICAL FORCE		OVERTURNING T/C		HORIZ. SHEAR	
LOAD CASE	UPLIFT	DOWNWARD	T_1/C_1	T_2/C_2	V_1	V_2
SEISMIC:	42 lb	42 lb	69 lb	94 lb	312 lb	312 lb
WIND:	231 lb	N/A	35 lb	65 lb	144 lb	195 lb
SNOW:	N/A	188 lb	N/A	N/A	N/A	N/A
DEAD:	N/A	205 lb	N/A	N/A	N/A	N/A

COMBINATIONS (ASD)

	OVERTURNING MOMENT	
COMBINATION	OT_1	OT_2
0.6DL-0.6WL	-129 lb-ft	-54 lb-ft
0.6DL-0.7EQ	-41 lb-ft	10 lb-ft
DL+SL	-626 lb-ft	-462 lb-ft
DL+0.6WL	-394 lb-ft	-332 lb-ft
DL+0.45WL+0.75SL	-601 lb-ft	-475 lb-ft
DL+0.7EQ	-481 lb-ft	-396 lb-ft
DL+0.525EQ+0.75SL	-667 lb-ft	-523 lb-ft

	VERTICAL FORCE	
COMBINATION	UPLIFT	DOWNWARD
$0.6*F_v$	139 lb	
$\pm 0.7*F_{pv}$	29 lb	29 lb
$0.6*F_v$	139 lb	
$0.45*F_v$	104 lb	
$\pm 0.7*F_{pv}$	29 lb	29 lb
$\pm 0.525*F_{pv}$	22 lb	22 lb

	TENSION		COMPRESSION		HORIZ. SHEAR	
RESULTANT COMBINATION	T_1	T_2	C_1	C_2	V_1	V_2
0.6DL-0.6WL	29 lb	47 lb	40 lb	23 lb	-86 lb	-117 lb
$(0.6-0.14S_{DS})$DL-0.7EQ	2 lb	19 lb	28 lb	10 lb	-218 lb	-218 lb
DL+SL	N/A	N/A	196 lb	196 lb	N/A	N/A
DL+0.6WL	N/A	N/A	124 lb	141 lb	86 lb	117 lb
DL+0.45WL+0.75SL	N/A	N/A	189 lb	202 lb	65 lb	88 lb
$(1+0.14S_{DS})$DL+0.7EQ	N/A	N/A	166 lb	183 lb	218 lb	218 lb
$(1+0.105S_{DS})$DL+0.525EQ+0.75SL	N/A	N/A	220 lb	233 lb	164 lb	164 lb

DESIGN LOADS (ASD) AND ATTACHMENT DESIGN

	TENSION		COMPRESSION		HORIZ. SHEAR	
	T_1	T_2	C_1	C_2	V_1	V_2
FORCE:	**29 lb**	**47 lb**	**220 lb**	**233 lb**	**218 lb**	**218 lb**
# OF SCREWS/SIDE:	**1 SCREWS**	**1 SCREWS**			**1 SCREWS**	**1 SCREWS**

SCREW SIZE	CURB/UNIT THICKNESS
#14	16ga

Project:	Good Samaritan Hospital Nutrition Department Expansion	Job Number:	24-707
Sheet:	of	Name:	KDK
Originating Office:	Tacoma	Date:	03/04/25

PCS Structural Solutions

MECHANICAL UNIT ANCHORAGE SPREADSHEET

MOMENTS AND REACTIONS (UNFACTORED, INCLUDING OVERSTRENGTH)

	F_1	F_2		
Ω_0*$M_{OT\ Seismic}$ =	331 lb-ft	331 lb-ft		
$M_{OT\ Wind}$ =	112 lb-ft	152 lb-ft		
$M_{R\ SL}$ =	299 lb-ft	221 lb-ft	P_{SL} =	188 lb
$M_{R\ DL\ Unit}$ =	327 lb-ft	241 lb-ft	$P_{DL\ Unit}$ =	205 lb
$M_{R\ DL\ Curb}$ =	0 lb-ft	0 lb-ft	$P_{DL\ Curb}$ =	0 lb

	VERTICAL FORCE		OVERTURNING T/C		HORIZ. SHEAR	
LOAD CASE	UPLIFT	DOWNWARD	T_1/C_1	T_2/C_2	V_1	V_2
Ω_0*SEISMIC:	62 lb	62 lb	104 lb	141 lb	468 lb	468 lb
WIND:	231 lb	N/A	35 lb	65 lb	144 lb	195 lb
SNOW:	N/A	188 lb	N/A	N/A	N/A	N/A
DEAD:	N/A	205 lb	N/A	N/A	N/A	N/A

COMBINATIONS (ASD, INCLUDING OVERSTRENGTH)

	OVERTURNING MOMENT			VERTICAL FORCE	
COMBINATION	OT_1	OT_2	COMBINATION	UPLIFT	DOWNWARD
0.6DL-0.6WL	-129 lb-ft	-54 lb-ft	0.6*Fv	139 lb	
0.6DL-0.7(Ω_0*EQ)	36 lb-ft	87 lb-ft	±0.7*Fpv	44 lb	44 lb
DL+SL	-626 lb-ft	-462 lb-ft			
DL+0.6WL	-394 lb-ft	-332 lb-ft	0.6*Fv	139 lb	
DL+0.45WL+0.75SL	-601 lb-ft	-475 lb-ft	0.45*Fv	104 lb	
DL+0.7(Ω_0*EQ)	-559 lb-ft	-473 lb-ft	±0.7*Fpv	44 lb	44 lb
DL+0.525(Ω_0*EQ)+0.75SL	-725 lb-ft	-581 lb-ft	±0.525*Fpv	33 lb	33 lb

	TENSION		COMPRESSION		HORIZ. SHEAR	
RESULTANT COMBINATION	T_1	T_2	C_1	C_2	V_1	V_2
0.6DL-0.6WL	29 lb	47 lb	40 lb	23 lb	-86 lb	-117 lb
(0.6-0.14S_{DS})DL-0.7(Ω_0*EQ)	33 lb	59 lb	11 lb	N/A	-327 lb	-327 lb
DL+SL	N/A	N/A	196 lb	196 lb	N/A	N/A
DL+0.6WL	N/A	N/A	124 lb	141 lb	86 lb	117 lb
DL+0.45WL+0.75SL	N/A	N/A	189 lb	202 lb	65 lb	88 lb
(1+0.14S_{DS})DL+0.7(Ω_0*EQ)	N/A	N/A	197 lb	223 lb	327 lb	327 lb
(1+0.105S_{DS})DL+0.525(Ω_0*EQ) +0.75SL	N/A	N/A	244 lb	263 lb	246 lb	246 lb

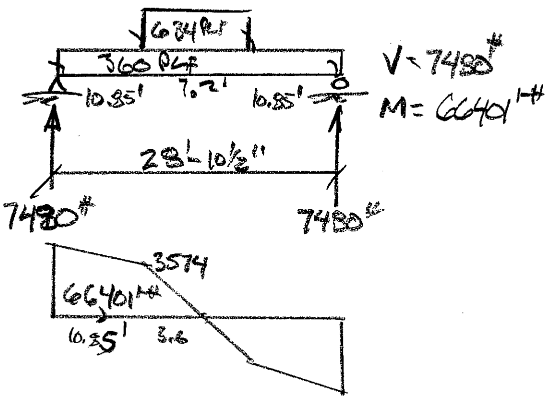
DESIGN LOADS (ASD, INCLUDING OVERSTRENGTH)

	TENSION		COMPRESSION		HORIZ. SHEAR	
	T_1	T_2	C_1	C_2	V_1	V_2
FORCE:	**33 lb**	**59 lb**	**244 lb**	**263 lb**	**327 lb**	**327 lb**

PCS Structural Solutions

CHECK EXISTING FRAMING FOR NEW MECHANICAL UNITS

W14x22 AT NEW MUA-1 UNIT:



$V = 7480^{\#}$
$M = 66401^{\prime\text{-}\#}$

EXIST DEAD LOAD:

ROOFING: 10^{PSF}
MISC = 5^{PSF}
DECK = 2^{PSF}
BEAMS = 3^{PSF}
20^{PSF}

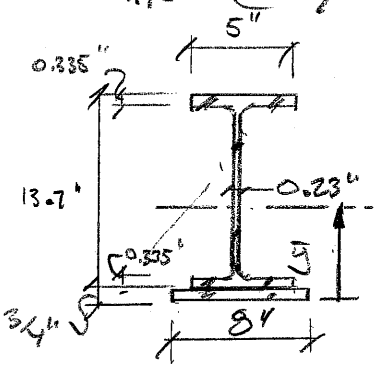
SNOW = 25^{PSF}

$W_1 = (20+25^{PSF})(8') = 360^{PLF}$

MUA = $5600^{\#}/(7.2')(9.8') = 79.3^{PSF}$

$W_2 = (79.3^{PSF})(8') = 634^{PLF}$

$M_n/\Omega = (36^{ksi}/1.67)(33.2\,in^3)(\tfrac{1}{12}) = \underline{59.6^{\prime\text{-}k}} < 66.4^{\prime\text{-}k}$ ∴ PROVIDE COVER PLATE



	A	$\bar{y}$	$A\bar{y}$	I_o	Ad^2
W14x22	6.49	7.6	49.32	199	78.15
PL3/4x8	6	.375	2.25	0.281	84.6
	12.49		51.57	199.3	162.7

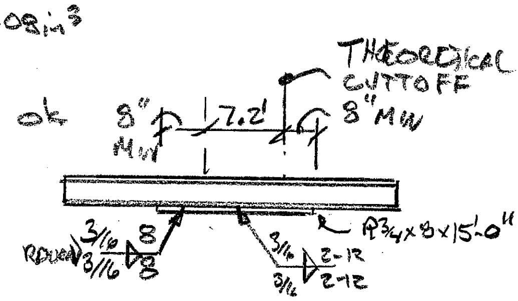
$\bar{y} = 51.57/12.49 = 4.13'$

$I = 199.3 + 162.7 = 362.0\ in^4$

$Z = (6.49)(3.47) + 6(3.76) = 45.08\ in^3$

$Q = (6)(3.76) = 22.56\ in^3$

$M_n/\Omega = (36^{ksi}/1.67)(45.08)(\tfrac{1}{12}) = \underline{80.98^{\prime\text{-}k}} > 66.4^{\prime\text{-}k}$ ok



THEORETICAL CUTTOFF ≅ 10.85' ⇒ $M = 59.9^{\prime\text{-}k}$

$MQ/I = (59.9^{\prime\text{-}k})(12^{"/'})(22.56\,in^3)/362\,in^4 = 44.8^{k}$

$L_{REQ'D} = 44.8^{k}/(0.928)(2)(3) = 8.05"$

$f_v = VQ/I = (7480^{\#})(22.56\,in^3)/362\,in^4 = 466^{lb/in}$

$D_{REQ'D} = 0.466^{k/in} / .6(70^{ksi}/2)(\tfrac{\sqrt{2}}{2})(2) = 0.0157"$

$\% = 0.0157/3/16 = 0.084$ ∴ 2-12 (16.6%) MIN

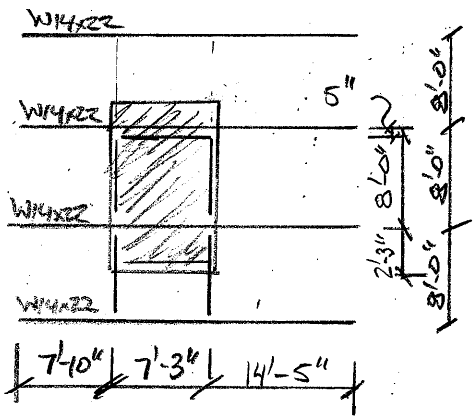
∴ EXIST W14x22 WITH COVER PL3/4x8 IS ADEQUATE TO SUPPORT NEW MAKE UP AIR UNIT.

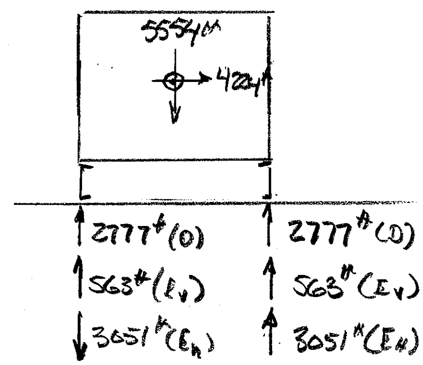
PCS Structural Solutions

Project: GSH | Job No: 24707
Subject: | Sheet | Name: KOK
Originating Office: ☐ Seattle ☒ Tacoma ☐ Portland | Date: 3-2-25

CHECK EXISTING FRAMING FOR LOAD FROM MAU-2

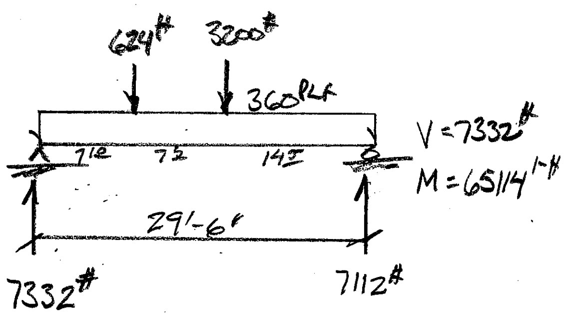
PLAN LAYOUT:



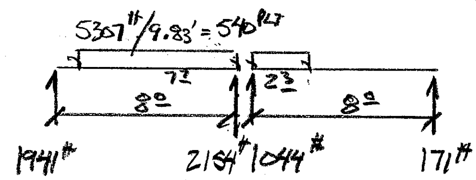


P_{MAX} = 2777 + .7(3051 + 563) = 5307#

P_{MIN} = 2777 + .7(563 − 3051) = 1035#

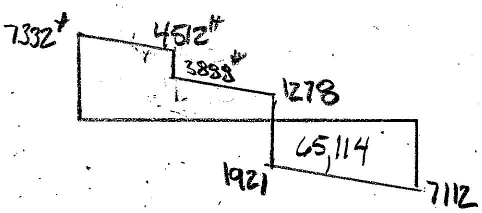


W = (20 + 25)(8') = 360 PLF

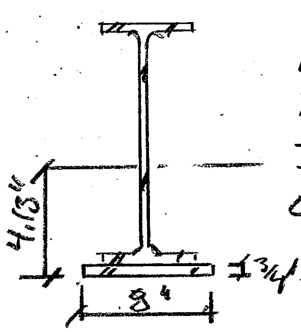


P_1 = 2154 + 1044 = 3200#

P_2 = (3200)(1035/5307) = 624#



M_n/Ω = (36/1.67)(45.98)(1/12) = 82.4 k-ft > 65.1 k-ft ok



A = 12.49 in²

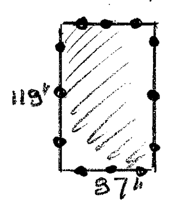
Z = 45.08 in³

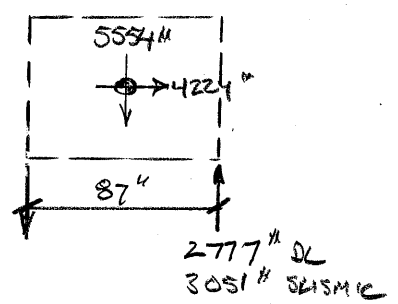
I = 362.0 in⁴

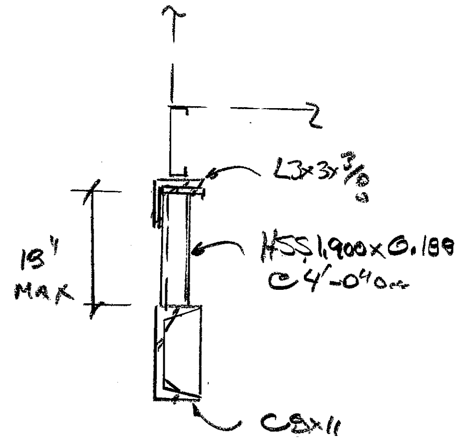
Q = 22.56 in³

MECHANICAL UNIT SUPPORT FRAME

MAU-2:

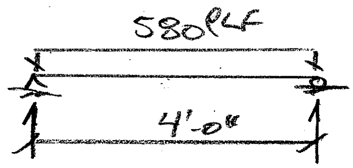






$$W_{D+.7E} = \frac{\left[.7\left(\frac{1126}{2}\right) + .7(3051) + 2777\right]}{118"/12"} = 580^{PLF}$$

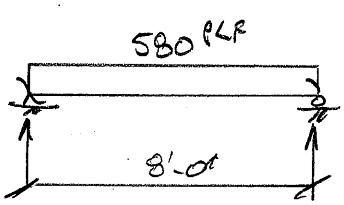
L3x3x3/8



V = 1160#
M = 1160 ft-#

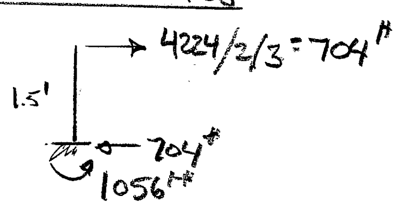
$M_n/\Omega = (36^{ksi}/1.67)(.825 in^3)(\frac{1}{12}) = 1.48^{k\text{-}ft} > 1.16^{k\text{-}ft}$ ok

C8x11.5



V = 2320#
$M = 4640^{ft\text{-}\#} < M_n/\Omega = (36^{ksi}/1.67)(9.63 in^3)(\frac{1}{12}) = 17.3^{k\text{-}ft}$ ok

HSS1.900x0.188



$M_n/\Omega = (50/1.67)(.520)(\frac{1}{12}) = 1.30^{k\text{-}ft} > 1.06^{k\text{-}ft}$ ok

PCS Structural Solutions

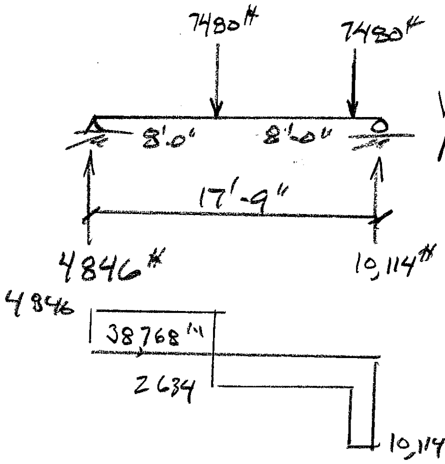
Project: GSH NUTRITION DEPARTMENT EXPANSION Job No: 24707

Subject: ______ Sheet ______ Name: KOK

Originating Office: ☐ Seattle ☒ Tacoma ☐ Portland Date: 2-10-25

CHECK EXISTING FRAMING FOR NEW MECHANICAL UNITS

W14x22 GIRDER:



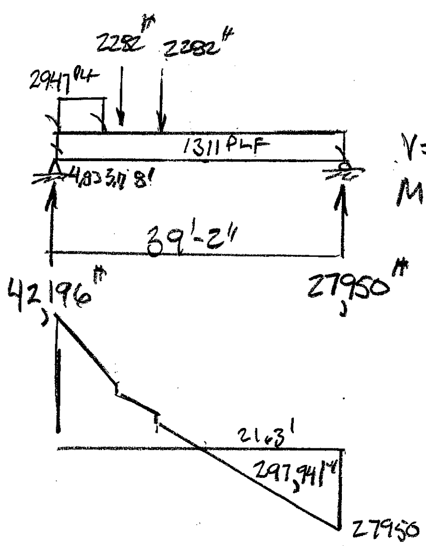
V = 10,114#

M = 38,768'#

$M_n/\Omega = (36/1.67)(33.2 in^3)(1/12) = 59.6^{'k} > 38.8^{'k}$ ok

∴ EXISTING W14x22 GIRDER IS ADEQUATE TO SUPPORT NEW MAKEUP AIR UNIT.

W24x76 GIRDER



V = 27950#

M = 297,941'k

REACTIONS FROM MUA-1: R = (634)(7.2/2) = 2282#

LOW ROOF: (20+25)(58.25/2) = 1311 PLF

HIGH ROOF: (20+25)(28.75/2) = 647 PLF

FLOOR: (60+100)(28.75/2) = 2300 PLF

$V_n/\Omega = (.6)(36)(.44)(23.9) = 227^k > 27.95^k$ ok

$M_n/\Omega = (36/1.67)(200)(1/12) = 359^{'k} > 297.9^{'k}$ ok

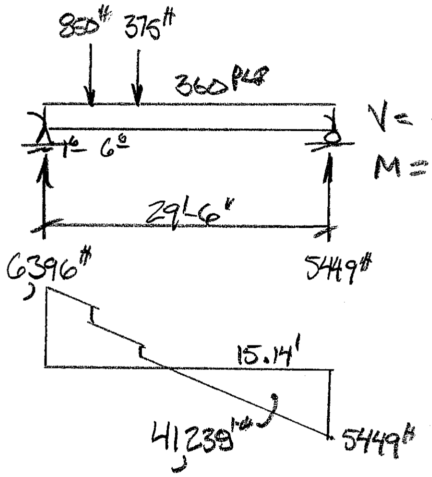
∴ EXISTING W24x76 GIRDER IS ADEQUATE TO SUPPORT NEW MAKE UP AIR UNIT.

PCS Structural Solutions

Project: GSH NUTRITION DEPT EXPANSION Job No: 24707
Subject: Sheet Name: KDK
Originating Office: ☐ Seattle ☒ Tacoma ☐ Portland Date: 2-10-25

CHECK EXIST FRAMING FOR NEW MECHANICAL UNITS

W14x22



V = 6396#
M = 41,238 lb-ft

EXIST. DEAD LOAD:

ROOFING: 10 PSF
MISC: 5 PSF
DECK: 2 PSF
BEAMS: 3 PSF
20 PSF

SNOW: 25 PSF

W = (20 + 25 PSF)(8') = 360 PLF

$V_n/\Omega = (.6)(36)(13.7")(0.23")/1.5 = 45.4^k \geq 6.4^k$ ok

$M_n/\Omega = (36^{ksi})(33.2\ in^3)(1/12)/1.67 = 59.6^k \geq 41.2^k$ ok

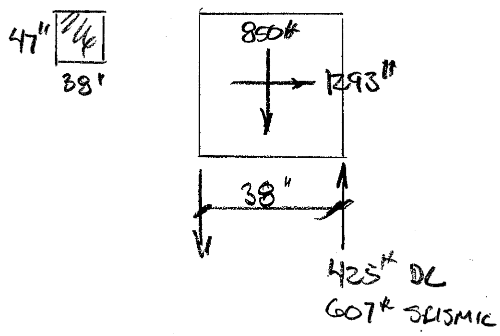
∴ EXISTING W14x22 BEAMS ARE ADEQUATE FOR NEW CONDENSING UNITS.

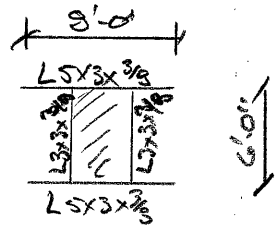
Structural Solutions

Project: GSH | Job No: 24707
Subject: | Sheet | Name: KOC
Originating Office: ☐ Seattle ☒ Tacoma ☐ Portland | Date: 3.2-25

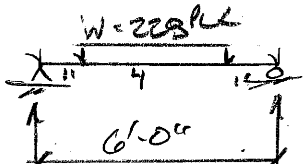
MECHANICAL UNIT SUPPORT FRAME

CONDENSING UNIT A17:





L3x3x3/8

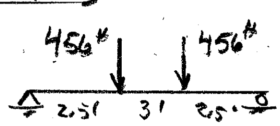


V = 456#

M = 912 1#' < M_n/Ω = (36/1.67)(.825)(1/2) = 1.48 1K

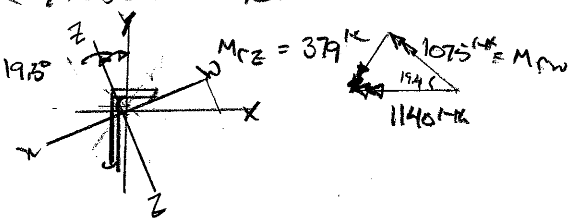
W = [.7(172/2) + .7(607) + 425]/4' = 228 PLF

L5x3x3/8



V = 456#

M = 1140 1#



MAJOR AXIS (LTB)

$$M_e = \frac{4.9 E I_z C_b}{L^2}\sqrt{\beta_w + .052\left(\frac{Lz}{r_z}\right)^2} + \beta_w = \frac{4.9(29000)(1.2)(1.0)}{(96)^2}\sqrt{(2.99)^2 + .052\left(\frac{96 \cdot .375}{.646}\right)^2} + 2.99 = 243.7^{in^k}$$

$$M_y = S_{w\,LONG\,TIP} F_y = (2.44)(36) = 87.8^{in^k}$$

$$M_e > M_y \therefore M_n = \left(1.92 - 1.17\sqrt{\frac{M_y}{M_e}}\right)M_y = \left(1.92 - 1.17\sqrt{\frac{87}{243.7}}\right)87 = 1.22(87) = 107.2^{in^k}$$

MINOR AXIS (YIELD)

$$M_{nz} = 1.5 M_y = 1.5 F_y S_{z\,SHORT\,TIP} = 1.5(36)(.7) = 37.8^{in^k}$$

INTERACTION:

$$\frac{1.075 \cdot 12}{107.2/1.67} + \frac{.379 \cdot 12}{37.8/1.67} = 0.402 < 1.0 \quad ok$$

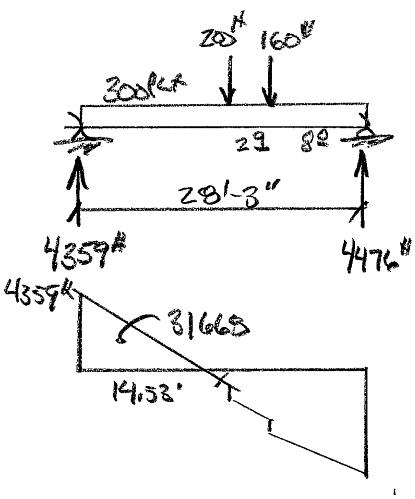
CHECK EXISTING FRAMING FOR NEW MECHANICAL UNITS

EXIST 16K7

EXIST DEAD LOAD:

3 5/16" NWC ON 24 GA VERCOR:	35 PSF
16K7 @ 4' O.C.	2 PSF
ROOF	10 PSF
MISC.	3 PSF
	50 PSF
SNOW	= 25 PSF

EF-4 = 200#
DEF-2 = 140#



V_{MAX} = 4476#
M_{MAX} = 31668 '-#

NOTE:
LOADING IS CONSERVATIVE & ASSUMES NO LOAD SHARING WITH ADJACENT JOIST.

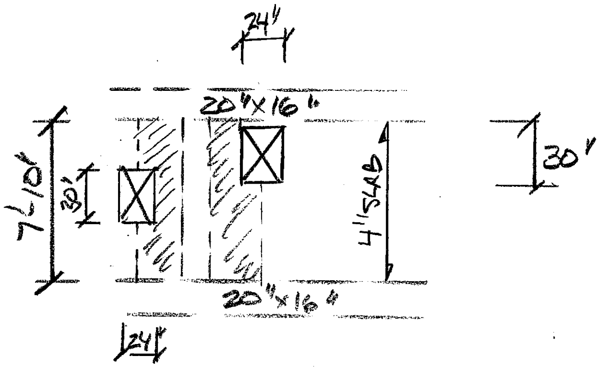
16K7

W_{TL} = 340 PLF ← PER SJI CATALOGUE
V_{ALL} = (340 PLF)(28/2) = 4760# > 4476# ok
M_{ALL} = (340 PLF)(28')²/8 = 33320 '# > 31668 '# ok

∴ EXIST 16K7, AND BY INSPECTION 20K7, IS ADEQUATE TO SUPPORT NEW EXHAUST FANS.

CHECK EXIST SLAB FOR NEW MECHANICAL PENETRATIONS

LEVEL 2 - WORST CASE OPENING 30"x24"



EXIST DEAD LOAD:

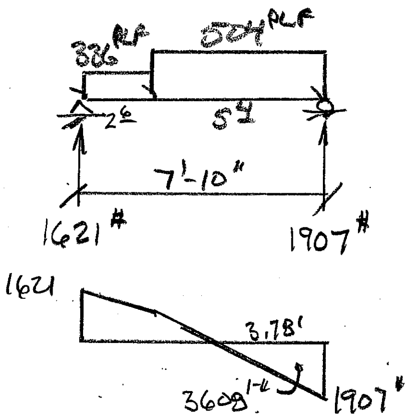
4" SLAB: 50 PSF
SDL: 10 PSF
60 PSF

LIVE: 60 PSF

$f'_c = 4^{ksi}$
$f_y = 40^{ksi}$

ASSUME 24" EFFECTIVE WIDTH:

$W_{u1} = [1.2(60) + 1.6(60)](2') = 336^{PLF}$
$W_{u2} = (168^{PSF})(3') = 504^{PLF}$



$V_{u\,MAX} = 1907^{\#}$
$M_{u\,MAX} = 3608^{1\text{-}\#}$

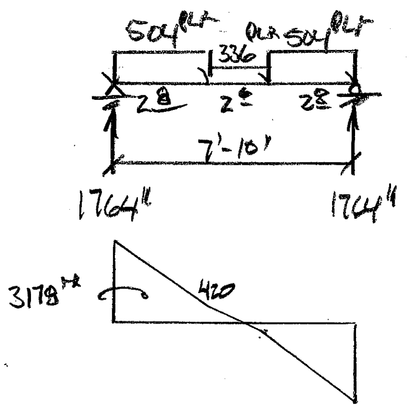
CHECK STRENGTH:

SHEAR:

$\phi V_c = .75(2)\sqrt{4000}(24)(3") = 6830^{\#} > 1907^{\#}$ ok

FLEXURE:

$A_s = .2in^2/16" = .299 in^2/'$
$a = (40)(.299)/.85(4)(12) = .293"$
$\phi M_n = (.9)(40)(.299)(2)(3 - .293/2)(1/12) = 5.12^{1\text{-}k} > 3.61^{1\text{-}k}$ ok



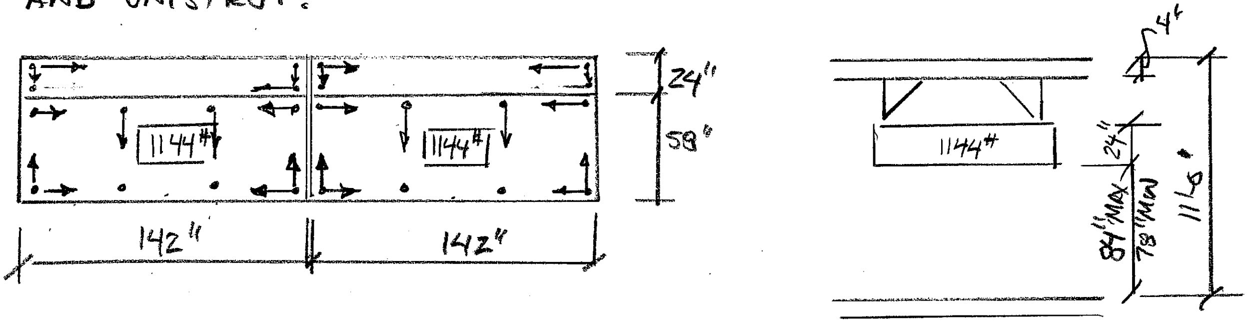
$V_{u\,MAX} = 1764^{\#}$
$M_{u\,MAX} = 3178^{1\text{-}\#}$

PCS Structural Solutions

Project: GSH NUTRITION DEPT EXPANSION — Job No: 24707
Subject: — Sheet — Name: KOK
Originating Office: ☐ Seattle ☒ Tacoma ☐ Portland — Date: 2-10-24

BRACE KITCHEN HOOD

BRACE HOOD UP TO EXISTING STRUCTURE USING 1/2"ϕ ALL-THREAD RODS AND UNISTRUT.



SEISMIC FORCE

$$F_p = \frac{0.4 a_p S_{DS} W_p}{R_p / I_p}\left(1 + 2\frac{z}{h}\right) = \frac{0.4(2.5)(1.014)W_p(1+2(.31))}{6/1.5} = \underline{0.411\ W_p}$$

$S_{DS} = 1.014$

$a_p = 2.5$

$R_p = 6$

$I_p = 1.5$

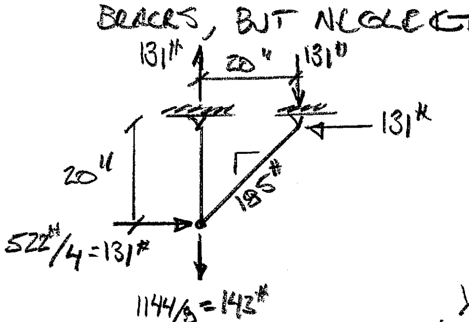
$W_p = 1144^{\#}$

$z/h = 11/35.5 = 0.31$

$$F_{p\,MIN} = 0.3 S_{DS} I_p W_p = 0.3(1.014)(1.5)W_p = 0.456 W_p = 0.456(1144^{\#}) = \underline{522^{\#}}$$

BRACE FORCE

USE (4) BRACES EFFECTIVE PER HOOD (UPPER FAN UNIT HAS ADDITIONAL BRACES, BUT NEGLECT.)



1/2"ϕ ATR: $P_{ALL} = 1350^{\#} > 143 + 131(.7) = 235^{\#}$ ok

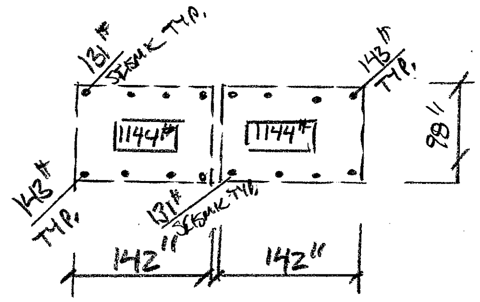
P1000 (KL=30"): $P_{ALL} = 3190^{\#} > 185^{\#}(.7) = 130^{\#}$ ok

CONNECTIONS:

1/2"ϕ KBTZ: (INSTALLED IN "B" DECK)
$P_{ALL} = 410^{\#} > 143^{\#} + .7(131) = 235^{\#}$ ok
$V_{ALL} = 1125^{\#} > .7(131) = 92^{\#}$ ok

1/2"ϕ HDI-P: (INSTALLED IN "B" DECK)
$P_{ALL} = 685^{\#} > 235^{\#}$ ok
$V_{ALL} = 1335^{\#} > 92^{\#}$ ok.

CHECK EXIST. FRAMING FOR NEW KITCHEN HOOD LOADS



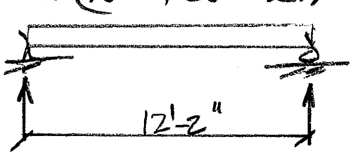
$P_{MAX} = 2(.7(131^{\#}) + 1.14(143)) = 510^{\#}$ ← WORST CASE WHERE HOODS ARE JOINED

EXIST. 6" SLAB (1951 ROOF)

THE 6" SLAB WAS PART OF THE 1951 ROOF STRUCTURE. IN 1987, A 7½" SLAB WAS BUILT ON TOP OF THE SLAB & WAS DETAILED TO NOT ADD LOAD TO THE SLAB. CHECK ORIGINAL DESIGN (SELF WEIGHT + ROOF SNOW LOAD) AGAINST PROPOSED (SELF WEIGHT + HOOD REACTION).

ORIGINAL DESIGN

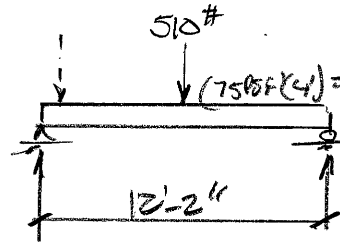
$W = (75PSF + 25PSF)(4') = 400PLF$



$V = 2433^{\#}$

$M = 7402'^{\#}$

PROPOSED



$V_{MAX} = 2,335^{\#} < 2433^{\#}$ OK

$M_{MAX} = 7,103'^{\#} < 7402'^{\#}$ OK

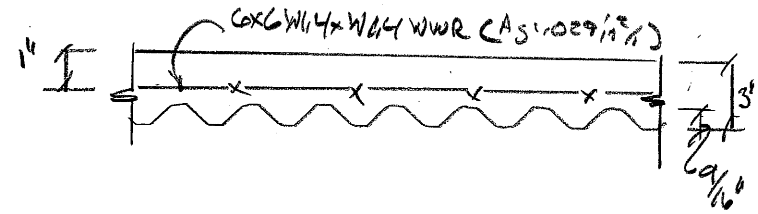
∴ EXISTING 6" SLAB IS ADEQUATE TO SUPPORT NEW KITCHEN HOOD LOADS.

CHECK EXISTING FRAMING FOR NEW KITCHEN HOOD LOADS

CHECK EXIST COMPOSITE SLAB ON METAL DECK

26 GA 9/16"x32" W/ 3" TOTAL SLAB, 3 SPAN CONDITION, 3'-0" MAX SPAN.

$W_{ALL} \sim 143^{PSF}$ < $V_{all} = 215^{PLF}$, $M_{all} = 161^{\text{1-#/FT}}$



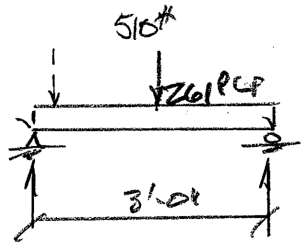
EXIST DEAD LOAD:

3" NWC ON 26 GA FORM DECK: 30 PSF
FLOORING 2 PSF
MISC. 5 PSF
37 PSF
LIVE 50 PSF

CHECK SLAB FOR (2) HANGERS WITH SEISMIC:

$P = 2(131 \cdot 0.7 + 143 \cdot 1.14) = 510^{\#}$

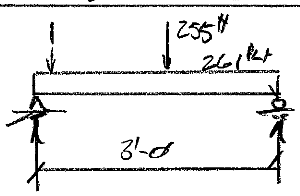
$W = (37 + 50^{PSF})(3') = 261\ PLF$



$V = 902^{\#} < V_a = (215)(3) = 645^{\#}$ NG.

$M = 676^{\text{1-#}} < M_a = (161)(3) = 483^{\text{1-#}}$ NG

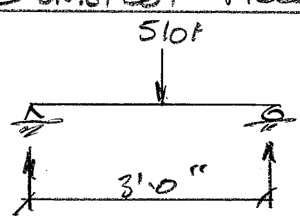
CHECK SLAB FOR (1) HANGER WITH SEISMIC:



$V = 646^{\#} \cong V_a = 645^{\#}$ ok

$M = 485^{\text{1-#}} \cong M_a = 483^{\text{1-#}}$ ok

CHECK UNISTRUT P1000 FOR (2) HANGERS:



$M = 382.5^{\text{1-\#}} = 4590^{\text{in-lb}} < 5070^{\text{in-lb}}$ ok

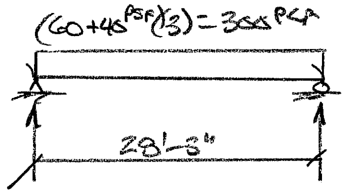
CHECK EXIST. FRAMING FOR NEW KITCHEN HOOD LOADS

EXIST. 18K7 JOISTS:

EXIST DEAD LOAD:

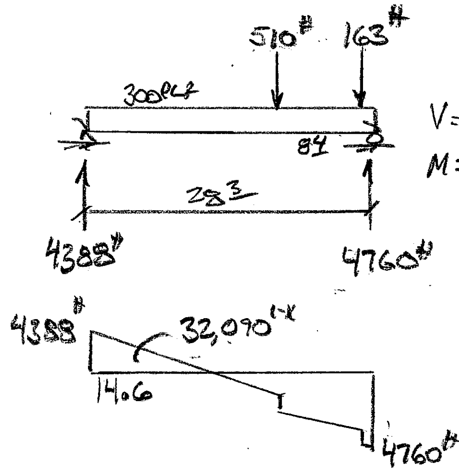
3" NWC ON 26 GA FORM DECK:	37 PSF
18K7 @ 3'-0" o.c.	3 PSF
PARTITION	20 PSF
	60 PSF
LIVE	= 40 PSF

ORIGINAL DESIGN



V = 4238#
M = 29,927 '-#

PROPOSED



V = 4760# (12% OVER)
M = 32,090 '-# (7% OVER

18K7 CAPACITY

W_{TL} = 385 PLF ◄ PER SJI LOAD TABLES

V_{MAX} = (385 PLF)(28.25/2) = 5438# > 4760# ok

M_{MAX} = (385 PLF)(28.25')²/8 = 38,407 '-# > 32,090 '-# ok.

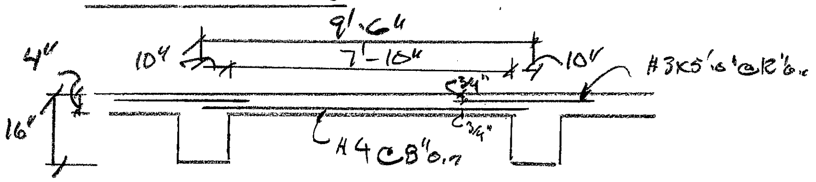
∴ EXISTING 18K7 JOISTS ARE ADEQUATE TO SUPPORT NEW KITCHEN HOODS.

PCS Structural Solutions

Project: GSH NUTRITION DEPT EXPANSION   Job No: 24707
Subject: ______   Sheet ______   Name: KDA
Originating Office: ☐ Seattle ☒ Tacoma ☐ Portland   Date: 2-10-25

CHECK EXIST. FRAMING FOR NEW KITCHEN HOOD LOADS

EXIST 4" SLAB



SLAB CAPACITY

$f_y = 40^{ksi}$

$f'_c = 4^{ksi}$

$\phi M_n^+ = (.9)(40)(.279)(3 - .293/2)(\frac{1}{12}) = 2.56^{1-k}/_{FT}$

$\phi M_n^- = (.9)(40)(.11)(3.06 - .108/2)(\frac{1}{12}) = 0.99^{1-k}/_{FT}$

$\phi V_c = (.75)(2)\frac{\sqrt{4000}}{1000}(12'')(3'') = 3.42^{k}/_{FT}$

EXIST. DEAD LOAD

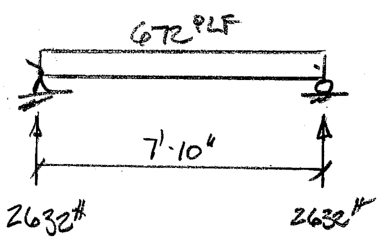
4" SLAB:	50 PSF
FLOORING	5 PSF
MISC	5 PSF
	60 PSF
LIVE =	60 PSF

$W_u = 1.2(60) + 1.6(60) = (168^{PSF})(4') = 672^{PLF}$

$P_{u_1} = 2[131^{\#} + 1.4(143)] = 662^{\#}$

$P_{u_2} = 1.4(143) = 200^{\#}$

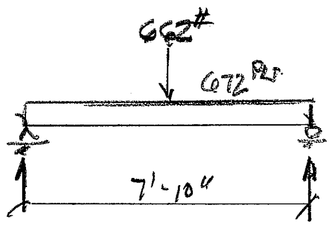
ORIGINAL DESIGN



$V_u = 2632^{\#}$

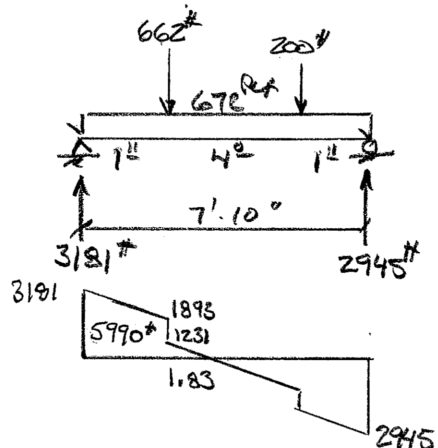
$M_u = 5154^{1-\#}$

PROPOSED



$V_u = 2962^{\#}$

$M_u = 6450^{1-\#}$



V_u =

M_u =

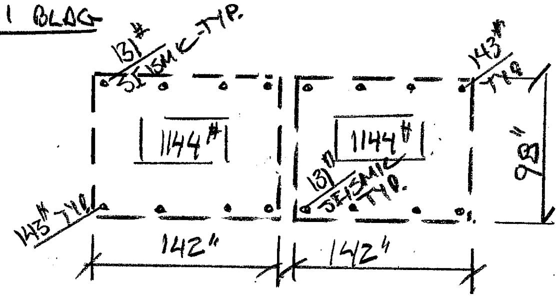
$V_{u_{MAX}} = 3.18^{k} < (3.42^{k}/_{1})(4') = 13.7^{k}$ ok

$M_{u_{MAX}} = 5.99^{1-k} < (2.56^{1-k}/_{1})(4') = 10.24^{1-k}$ ok

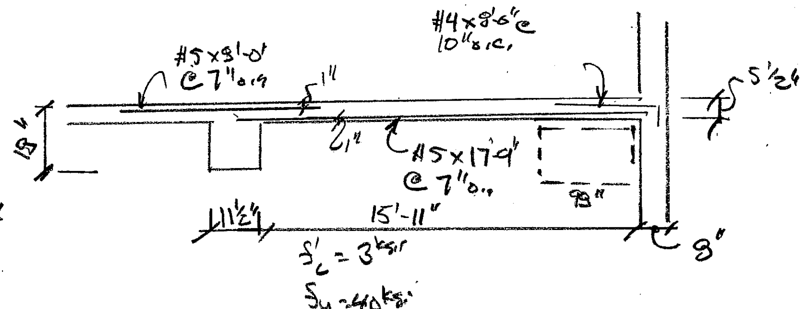
∴ EXISTING 4" REINFORCED CONCRETE SLAB IS ADEQUATE TO SUPPORT NEW KITCHEN HOOD.

CHECK EXISTING FRAMING FOR NEW KITCHEN HOOD LOADS

1951 BLDG



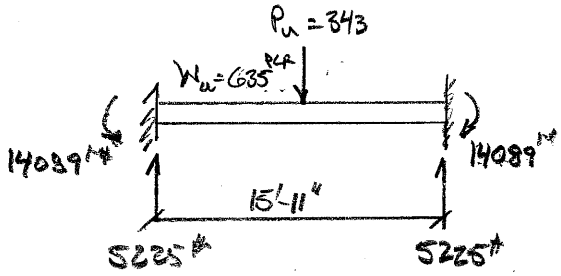
KITCHEN HOODS



EXIST'G STRUCTURE

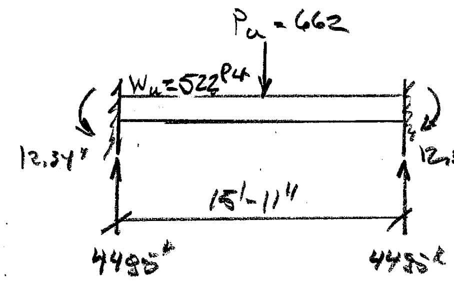
DL = 69 psf
SDL = 10 psf
L = 40 psf

$1.2D + 1.6L$
$P_u = 1.2(143)(2)$
$W_u = [1.2(79) + 1.6(40)](4') = 635$ plf



$M_u^- = 14.09^{k}$
$M_u^+ = 7.43^{k}$
$V_u = 5.23^{k}$

$1.4D + .5L + 1.0E$
$P_u = 1.4(143) + 131](2) = 662$
$W_u = [1.4(79) + .5(40)](4) = 522$



$M_u^- = 12.34^{1-k}$
$M_u^+ = 6.83^{1-k}$
$V_u = 4.49^{k}$

CHECK SHEAR:

$\phi V_c = (.75)(2)\frac{\sqrt{3000}}{1000}(48")(4.1875") = 16.5^k > 5.23^k$ ok

CHECK FLEXURE:

$A_s^- = .2 in^2/.83' = 0.241 in^2/ft \Rightarrow a = (.241)(40)/.85(3)(12) = 0.315"$
$\phi M_n = (.9)(40)(.241)(4)(4.25 - .315/2)(\frac{1}{12}) = 11.84^{1-k} < 14.09^{1-k}$

CHECK CENTER SPAN ASSUMING SIMPLE (HINGE FORMS @ ENDS)

$M_{u_{MAX}}^+ = 635(15.917)^2/8 + 343(15.917)/4 = 21475^{1-\#}$
$A_s = .31/.583 = 0.532 in^2/ft \Rightarrow a = 40(.532)/.85(3)(12) = 0.695"$
$\phi M_n = (.9)(40)(.532)(4)(4.1875 - .695/2)(\frac{1}{12}) = 24.51^{1-k} > 21.48^{1-k}$ ok.

∴ EXISTING STRUCTURE AT CENTER WING (1951 BLDG) IS ADEQUATE TO SUPPORT NEW KITCHEN HOODS.

ICC-ES Evaluation Report

ESR-4266

Issued December 2020

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:

- 2018, 2015, and 2012 *International Building Code®* (IBC)
- 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-, $^1/_2$-inch-, $^5/_8$-inch- and $^3/_4$-inch diameter (6.4 mm, 9.5 mm, 12.7 mm and 15.9 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 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, and Figure 5C 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 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 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 318-14 17.3.1 or ACI 318-11 D.4.1, as applicable, except as required in ACI 318-14 17.2.3 or ACI 318-11 D.3.3, as applicable.

Strength reduction factors, ϕ, as given in 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 1605.2 of the IBC and Section 5.3 of ACI 318-14 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 318-14 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 318-14 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 318-14 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 318-14 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 318-14 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 318-14 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 318-14 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 318-14 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 318-14 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 318-14 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 318-14 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 318-14 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 318-14 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 318-14 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 318-14 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 318-14 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 318-14 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 318-14 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 318-14 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 318-14 2.3 or ACI 318-11 D.1, as applicable, as ductile steel elements and must be designed in accordance with 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 318-14 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 318-14 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 318-14 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 318-14 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 318-14 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 318-14 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 318-14 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 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 318-14 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 318-14 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 318-14 (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 1605.3 of the 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 318-14 Chapter 17 and 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 318-14 Chapter 17 and 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 318-14 17.6 or ACI 318-11 D.7, as applicable, as follows:

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

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

For all other cases:

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

4.3 Installation:

Installation parameters are provided in Table 1 and Figure 2, Figure 5A, Figure 5B, and Figure 5C. 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 or 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. The Hollow Drill Bits are not permitted for use with the $^1/_4$-inch- and $^3/_8$-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. 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 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 Table 2, and Figure 5A, Figure 5B, Figure 5C.

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 April 2018), which incorporates requirements in 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.

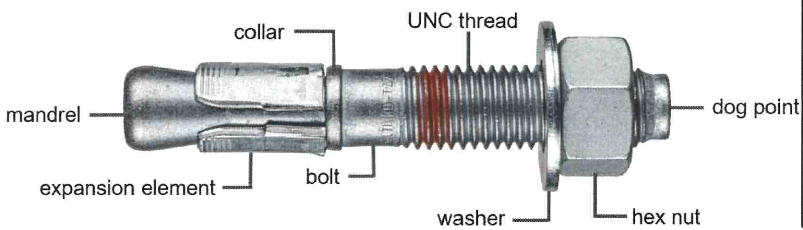


FIGURE 1—HILTI CARBON STEEL KWIK BOLT TZ (KB-TZ2)

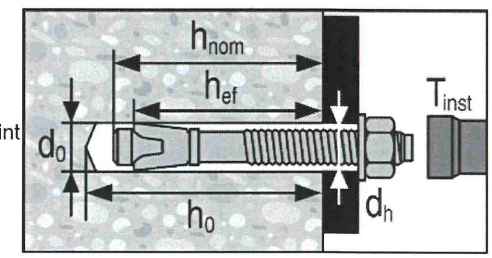


FIGURE 2—HILTI KB-TZ2 INSTALLED

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.

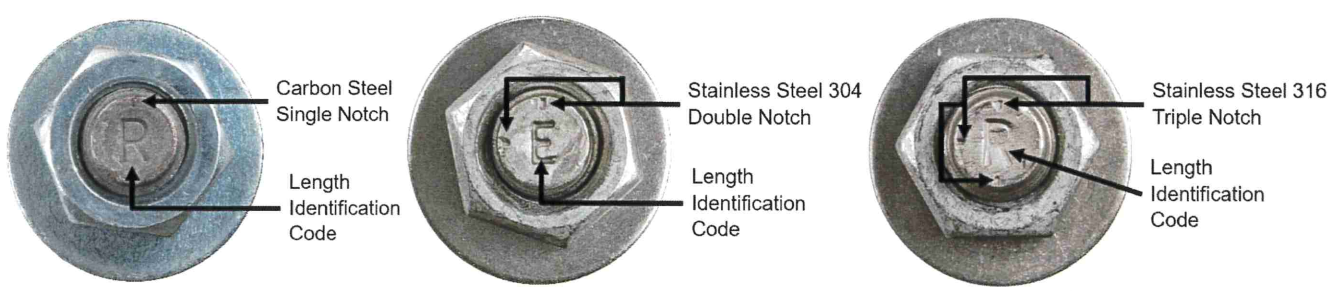


FIGURE 3—BOLT HEAD WITH LENGTH IDENTIFICATION CODE AND KB-TZ2 HEAD NOTCH EMBOSSMENT

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

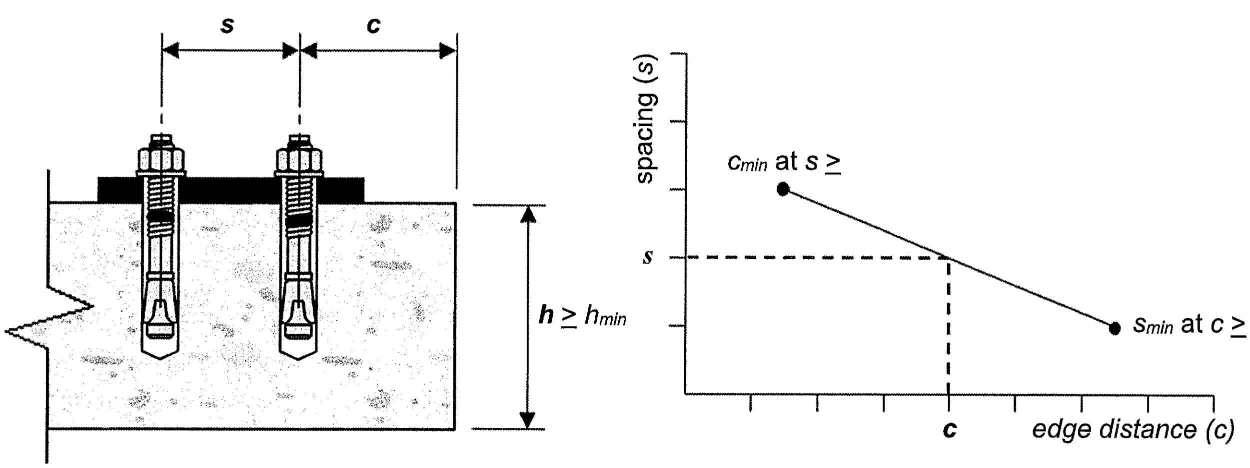


FIGURE 4—INTERPOLATION OF MINIMUM EDGE DISTANCE AND ANCHOR SPACING

TABLE 4 – HILTI CARBON STEEL KB-TZ2 DESIGN INFORMATION, 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[3]	$\Phi_{c,N}$, $\Phi_{p,N}$	-	0.45	0.65			0.65				0.65			0.65		
Effectiveness factor for uncracked concrete	k_{uncr}	-	24	24			27		24		24			24	27	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 318-14 2.3 or ACI 318-11 D.1.
[3] For use with the load combinations of ACI 318-14 Section 5.3, ACI 318-11 Section 9.2 or IBC Section 1605.2. Condition B applies where supplementary reinforcement in conformance with 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 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.

TABLE 5 – HILTI STAINLESS STEEL KB-TZ2 DESIGN INFORMATION, 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/in^2 (N/mm^2)	100,900 (696)	96,300 (664)			96,300 (664)			91,600 (632)			84,100 (580)		
Min. specified ult. strength	f_{uta}	lb/in^2 (N/mm^2)	122,400 (844)	120,100 (828)			120,400 (830)			114,600 (790)			100,500 (693)		
Effective tensile stress area	$A_{se,N}$	In^2 (mm^2)	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 [3]	$\Phi_{c,N}$, $\Phi_{p,N}$	-	0.45	0.65			0.65			0.65			0.65		
Effectiveness factor for uncracked concrete	k_{uncr}	-	24	24			24			24			24	27	24
Effectiveness factor for cracked concrete	k_{cr}	-	17	21		17	17	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)	5-1/2 (140)	4 (102)	6 (152)	6 (152)	8 (203)	10 (254)	7 (178)	9 (229)	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	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	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 318-14 2.3 or ACI 318-11 D.1.
[3] For use with the load combinations of ACI 318-14 Section 5.3, ACI 318-11 Section 9.2 or IBC Section 1605.2. Condition B applies where supplementary reinforcement in conformance with 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 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

TABLE 6 – HILTI CARBON STEEL KB-TZ2 DESIGN INFORMATION, 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 [3]	$\Phi_{c,V}$, $\Phi_{p,V}$	-	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 318-14 2.3 or ACI 318-11 D.1.
[3] For use with the load combinations of ACI 318-14 Section 5.3, ACI 318-11 Section 9.2 or IBC Section 1605.2. Condition B applies where supplementary reinforcement in conformance with 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 for concrete breakout failure may be used.

TABLE 7 – HILTI STAINLESS STEEL KB-TZ2 DESIGN INFORMATION, 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 [3]	$\Phi_{c,V}$, $\Phi_{p,V}$	-	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 318-14 2.3 or ACI 318-11 D.1.
[3] For use with the load combinations of ACI 318-14 Section 5.3, ACI 318-11 Section 9.2 or IBC Section 1605.2. Condition B applies where supplementary reinforcement in conformance with 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 for concrete breakout failure may be used.

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[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
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	3,285	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 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	2,620	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 318-14 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.

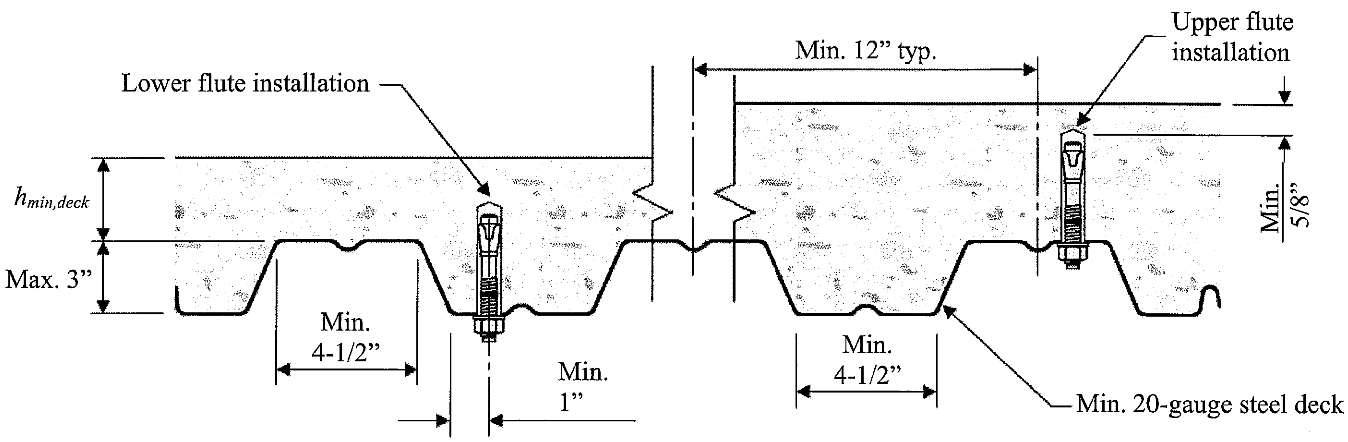


FIGURE 5A—KB-TZ2 IN THE SOFFIT OF CONCRETE FILLED PROFILE STEEL DECK ASSEMBLIES – W DECK

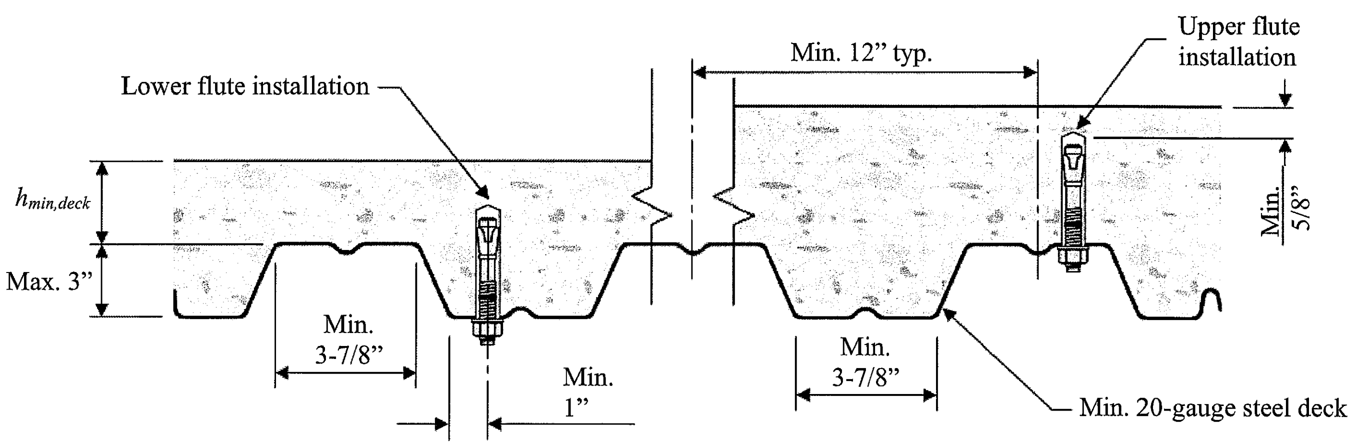


FIGURE 5B—KB-TZ2 IN THE SOFFIT OF CONCRETE FILLED PROFILE STEEL DECK ASSEMBLIES – W DECK

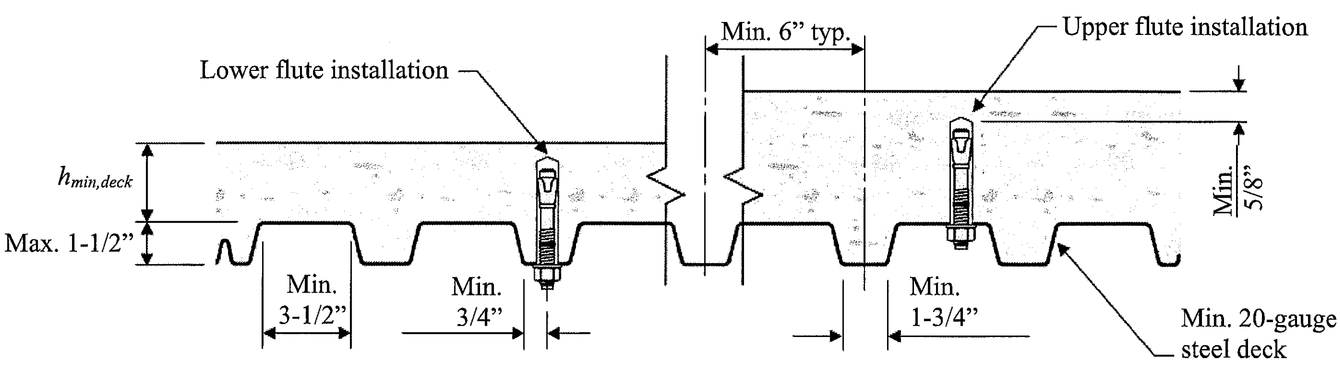
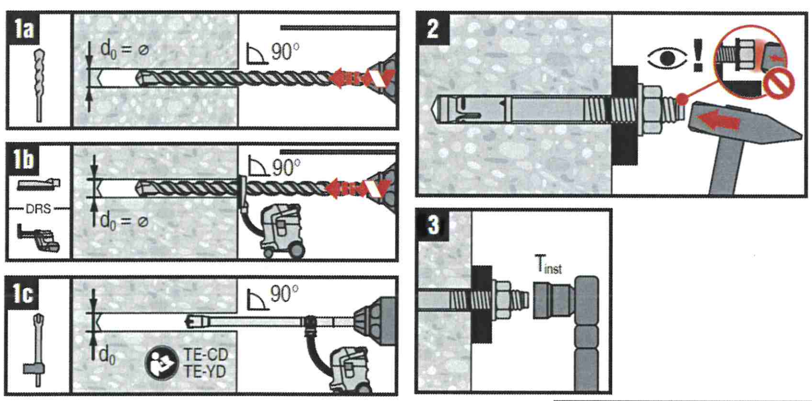


FIGURE 5C—KB-TZ2 IN THE SOFFIT OF CONCRETE FILLED PROFILE STEEL DECK ASSEMBLIES – B DECK

Hilti SafeSet™ System with Hollow Drill Bit	**Hilti Dust Removal Systems**
 Hilti TE-CD or TE-YD Hollow Carbide Drill Bit, with	 Hilti Rotary Hammer Drill with DRS (Dust Removal System) Module, or
 Hilti Vacuum (per section 4.3)	 Hilti TE DRS-D Dust Removal System with Hilti Vacuum

FIGURE 6—HILTI SYSTEM COMPONENTS



Symbol	Setting Information	Units	Anchor Diameter [inch]				
			$^1/_4$	$^3/_8$	$^1/_2$	$^5/_8$	$^3/_4$
HDB	Hollow Drill Bit	–	–	–	✔	✔	✔
DRS	Dust Removal Systems	–	✔	✔	✔	✔	✔

FIGURE 7—INSTALLATION INSTRUCTIONS

ICC-ES Evaluation Report

ESR-4266 LABC and LARC Supplement

Issued December 2020

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.

ICC-ES Evaluation Report

ESR-4266 FBC Supplement

Issued December 2020

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 and 2017 *Florida Building Code—Building*
- 2020 and 2017 *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 and 2015 *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 condition:

a) Design and installation must meet the requirements of Section 2122.7 of the *Florida Building Code—Building.*

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.

ICC-ES Evaluation Report

ESR-4236

Reissued July 2023

Revised May 2024

This report is subject to renewal July 2025.

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 HDI-P TZ AND HDI-TZ ANCHORS IN CRACKED AND UNCRACKED CONCRETE

1.0 EVALUATION SCOPE

Compliance with the following codes:

- 2024, 2021, 2018, and 2015 *International Building Code*® (IBC)
- 2024, 2021, 2018, and 2015 *International Residential Code*® (IRC)

For evaluation for compliance with codes adopted by the Los Angeles Department of Building and Safety (LADBS), see ESR-4236 LABC and LARC Supplement.

Property evaluated:

Structural

2.0 USES

The Hilti HDI-P TZ and HDI-TZ anchors are used as anchorage to resist static, wind, and seismic (Sesimic 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 ½-inch (6.4 mm, 9.5 mm, and 12.7 mm) HDI-P TZ and $^3/_8$-inch (9.5 mm) HDI-TZ anchors are limited to installation in the formed concrete surface. Use of these anchors are limited to supporting non-structural components.

The $^1/_4$-inch, $^3/_8$-inch, and ½-inch (6.4 mm, 9.5 mm, and 12.7 mm) HDI-P TZ and $^3/_8$-inch and $^1/_2$-inch (9.5 mm and 12.7 mm) HDI-TZ 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 ½-inch and $^5/_8$-inch diameter (12.7 mm and 15.9 mm) HDI-TZ anchors may be installed in top 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 6 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 ¼-inch and $^3/_8$-inch (6.4 mm and 9.5 mm) HDI-P TZ anchors may be installed in the underside of cracked and uncracked hollow-core concrete slabs having a minimum specified compressive strength, f'_c, of 6,000 psi (41.4 MPa). Use of anchors is limited to supporting non-structural components.

The anchor is an alternative to cast-in-place anchors described in Section 1901.3 of the 2024, 2021, 2018 and 2015 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 HDI-P TZ:

HDI-P TZ anchors are internally-threaded, displacement-controlled, mechanical expansion anchors. HDI-P TZ anchors consist of an internally-threaded anchor body with an expansion cone, a wedge (expansion element), and an internal setting plug which expands the anchor and activates the wedge when engaged with the HDI-P TZ setting tool. The HDI-P TZ is illustrated in Figures 1 and 5. The anchor components are manufactured from carbon steel and have a minimum 5 μm (0.0002 inch) zinc plating conforming to DIN EN ISO 4042 A2K.

The anchor is installed in a predrilled hole using a carbide-tipped hammer drill bit meeting the requirements of ANSI B212.15 or with a Hilti HDI-P TZ stop drill bit. The HDI-P TZ is inserted into the predrilled hole and the setting plug is engaged with the manual HDI-P TZ setting tool and a hammer, or the automatic HDI-P TZ setting tool and a hammer drill. See Figure 5 for the proper drilling and setting tools.

3.2 HDI-TZ:

HDI-TZ anchors are internally-threaded, displacement-controlled, mechanical expansion anchors. HDI-TZ anchors consist of an internally-threaded anchor body with an expansion cone, a wedge (expansion element), and an internal setting plug, which expands the anchor and activates the wedge when engaged with the HDI-TZ setting tool. The HDI-TZ is illustrated in Figures 2 and 5. The anchor components are manufactured from carbon steel and have a minimum 5 μm (0.0002 inch) zinc plating conforming to DIN EN ISO 4042 A2K.

The anchor is installed in a predrilled hole using a carbide-tipped hammer drill bit meeting the requirements of ANSI B212.15 or with a Hilti HDI-TZ stop drill bit. The HDI-TZ is inserted into the predrilled hole and the setting plug is engaged with the manual HDI-TZ setting tool and a hammer, or the automatic HDI-TZ setting tool and a hammer drill. See Figure 5 for the proper drilling and setting tools.

3.3 Steel Insert Elements:

A threaded steel insert element must be threaded into the Hilti HDI-P TZ or HDI-TZ anchor after the anchor is set in the concrete. The properties of the insert element must comply with ASTM A36 minimum, or equivalent. See Tables 3 and 4.

3.4 Concrete:

Normal-weight and lightweight concrete must conform to Sections 1903 and 1905 of the IBC. The minimum concrete compressive strength at the time of anchor installation is noted in Section 5.5 of this report.

3.5 Steel Deck Panels:

Steel deck panels must be in accordance with the configuration in Figures 4A, 4B, and 4C 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 33 and have a minimum yield strength of 33,000 psi (345 MPa).

3.6 Hollow Core Concrete Panels:

Hollow core concrete panels shall have a minimum thickness of $1^3/_8$ inches (35 mm) between the horizontal surface and the hollow core as indicated in Figure 3.

4.0 DESIGN AND INSTALLATION

4.1 Strength Design:

4.1.1 General: Design strength of anchors complying with the 2024 and 2021 IBC, as well as Section R301.1.3 of the 2024 and 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 parameters provided in Tables 2, 3, and 4, of this report are based on the 2024 and 2021 IBC (ACI 318-19), and the 2018 and 2015 IBC (ACI 318-14) 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 or ACI 318-14 17.3.1, as applicable, except as required in ACI 318-19 17.10 or ACI 318-14 17.2.3, as applicable.

Strength reduction factors, ϕ, as given in Tables 2 and 4 of this report must be used in lieu of ACI 318-19 17.5.3 or ACI 318-14 17.3.3, as applicable, for load combinations calculated in accordance with Section 1605.1 of the 2024 or 2021 IBC or Section 1605.2 of the 2018, and 2015 IBC and Section 5.3 of ACI 318 (-19 and -14), as applicable. 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 or ACI 318-14 17.2.7, 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 or ACI 318-14 17.4.1.2, as applicable for the threaded steel element, $N_{sa,rod}$, as noted in Table 4 of this report. The lesser of $\phi N_{sa,rod}$ in Table 4 or ϕN_{sa} provided in Table 2 for the HDI-P TZ and HDI-TZ anchors shall be used as the steel strength in tension.

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 or ACI 318-14 17.4.2, 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 or ACI 318-14 17.4.2.2, as applicable, using the values of h_{ef} and k_{cr} as given in Table 2 of this report. 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 or ACI 318-14 17.4.2.6, as applicable, must be calculated with k_{uncr} as given in Table 2 of this report and with $\Psi_{c,N}$ = 1.0.

For HDI-P TZ and HDI-TZ anchors installed in the soffit of sand-lightweight or normal-weight concrete on steel deck floor and roof assemblies, as shown in Figures 4A and 4B, 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, or ACI 318-14 17.4.3.1 and 17.4.3.2, respectively, as applicable, in cracked and uncracked concrete, $N_{p,cr}$ and $N_{p,uncr}$, respectively, is given in Table 2. For all design cases $\Psi_{c,P}$ = 1.0. In accordance with ACI 318-19 17.6.3 or ACI 318-14 17.4.3, as applicable, the nominal pullout strength in cracked concrete may be calculated in accordance with the following equation where the specified concrete compressive strength, f'_c, exceeds 2,500 psi (17.2 MPa):

$$N_{p,f'_c} = N_{p,cr}\left(\frac{f'_c}{2{,}500}\right)^{0.35} \quad \text{(lb, psi)} \qquad \text{(Eq-1)}$$

$$N_{p,f'_c} = N_{p,cr}\left(\frac{f'_c}{17.2}\right)^{0.35} \quad \text{(N, MPa)}$$

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

$$N_{p,f'_c} = N_{p,uncr}\left(\frac{f'_c}{2{,}500}\right)^{0.35} \quad \text{(lb, psi)} \qquad \text{(Eq-2)}$$

$$N_{p,f'_c} = N_{p,uncr}\left(\frac{f'_c}{17.2}\right)^{0.35} \quad \text{(N, MPa)}$$

Where values for $N_{p,cr}$ and $N_{p,uncr}$ are not provided in Table 2, the pullout strength in tension need not be evaluated.

The nominal pullout strength in cracked concrete of the HDI-P TZ and HDI-TZ anchors installed in the soffit of sand-lightweight or normal-weight concrete on steel deck floor and roof assemblies, as shown in Figures 4A and 4B is given in Table 5. In accordance with ACI 318-19 17.6.3.2.1 or ACI 318-14 17.4.3.2, 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 or ACI 318-14 17.4.3.6, 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.

4.1.5 Requirements for Static Steel Strength in Shear: The nominal steel strength in shear, V_{sa}, of a single anchor must be taken as the threaded steel element strength, $V_{sa,rod}$, as noted in Table 4 of this report. The lesser of $\phi V_{sa,rod}$ in Table 4 or ϕV_{sa} provided in Table 2 for the HDI-P TZ and HDI-TZ anchors shall be used as the steel strength in shear, and must be used in lieu of the values derived by calculation from ACI 318-19 17.7.1.2b or ACI 318-14 Eq. 17.5.1.2b, as applicable. The shear strength,

$V_{sa,deck}$, of the HDI-P TZ and HDI-TZ anchors as governed by steel failure of the HDI-P TZ or HDI-TZ anchors installed in the soffit of sand-lightweight or normal-weight concrete on steel deck floor and roof assemblies, as shown in Figures 4A and 4B, is given in Table 5.

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 or ACI 318-14 17.5.2, 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 or ACI 318-14 17.5.2.2, as applicable, based on the values of ℓ_e and d_a provided in Table 2 of this report.

For HDI-P TZ and HDI-TZ anchors installed in the soffit of sand-lightweight or normal-weight concrete on steel deck floor and roof assemblies, as shown in Figures 4A and 4B, calculation of the concrete breakout strength in shear is not required.

For anchors installed in hollow-core concrete panels, the nominal concrete breakout strength of a single anchor or group of anchors in shear, V_{cb} or V_{cbg}, must be calculated in accordance with ACI 318-19 17.7.2 or ACI 318-14 17.5.2, as applicable, using the actual member cover thickness for anchors in the hollow-core concrete slabs as given in Table 1 and Figure 3 of this report, as applicable.

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 or ACI 318-14 17.5.3, as applicable, using the value of k_{cp} provided in Table 2 of this report and the value of N_{cb} or N_{cbg} as calculated in Section 4.1.3 of this report.

For HDI-P TZ and HDI-TZ anchors installed in the soffit of sand-lightweight or normal-weight concrete over profile steel deck floor and roof assesmblies, as shown in Figures 4A and 4B, calculation of the concrete pryout strength in accordance with ACI 318-19 17.7.3 or ACI 318-14 17.5.3, as applicable, 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 or ACI 318-14 17.2.3, as applicable. Modifications to ACI 318-19 17.10 or ACI 318-14 17.2.3 shall be applied under Section 1905.1.8 of the 2024, 2021, 2018 and 2015 IBC.

The anchors comply with ACI 318 (-19 and -14) 2.3, as applicable, as brittle 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; or ACI 318-14 17.2.3.4, 17.2.3.5, 17.2.3.6 or 17.2.3.7, as applicable. Strength reduction factors, ϕ, are given in Table 2 of this report. The Hilti HDI-P TZ and HDI-TZ anchors may be installed in regions designated as IBC Seismic Design Categories A through F.

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, or ACI 318-14 17.4.1 and 17.4.2, 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 or ACI 318-14 17.4.3.2, as applicable, the appropriate pullout strength in tension for seismic loads, $N_{p,eq}$, described in Table 2 or $N_{p,deck,eq}$ described in Table 5 must be used in lieu of N_p, as applicable. The value of $N_{p,eq}$ or $N_{p,deck,eq}$ may be adjusted by calculation for concrete strength in accordance with Eq-1 and Section 4.1.4 of this report whereby the value of $N_{p,deck,eq}$ 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 $N_{p,eq}$ are given in Table 2, the pullout strength need not be calculated and does not 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, or ACI 318-14 17.5.2 and 17.5.3, 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 or ACI 318-14 17.5.1.2, as applicable, the appropriate value for nominal steel strength for seismic loads, $V_{sa,eq}$, described Table 2 or $V_{sa,deck,eq}$ described in Table 5, 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 or ACI 318-14 17.6, 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 or ACI 318-14 17.7.1 and 17.7.3, respectively, as applicable, values of s_{min} and c_{min} as given in Table 1 of this report must be used. In lieu of ACI 318-19 17.9.4 or ACI 318-14 17.7.5, as applicable, minimum member thicknesses, h_{min}, as given in Table 1 of this report must be used.

For HDI-TZ 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 6 and Figure 4C.

For HDI-P TZ and HDI-TZ 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 Figures 4A and 4B 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 or ACI 318-14 17.4.2, as applicable, must be further multiplied by the factor $\Psi_{cp,N}$ as given by Eq-3:

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

whereby the factor $\Psi_{cp,N}$ need not be taken as less than $\frac{1.5h_{ef}}{c_{ac}}$. For all other cases, $\Psi_{cp,N} = 1.0$. In lieu of using ACI 318-19 17.9.5 or ACI 318-14 17.7.6, as applicable, values of c_{ac} in Table 2 must be used.

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 (2024 or 2021 IBC), and ACI 318-14 (2018 and 2015 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 floor and roof assemblies, further reduction of the pullout values provided in this report is not required.

4.1.13 Hollow Core Concrete Panels: Installations in hollow core concrete panels shall be in accordance with the requirements in normal weight concrete provided installations are in accordance with Table 1 and Figure 3.

4.2 Allowable Stress Design (ASD):

4.2.1 General: Design values for use with allowable stress design load combinations calculated in accordance with Section 1605.1 of the 2024 and 2021 IBC or Section 1605.3 of the 2018, and 2015 IBC, must be established as follows:

$$T_{allowable,ASD} = \frac{\phi N_n}{\alpha} \quad \text{(Eq-4)}$$

$$V_{allowable,ASD} = \frac{\phi V_n}{\alpha} \quad \text{(Eq-5)}$$

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, 2024 IBC Section 1905.7, 2021, 2018 and 2015 IBC Section 1905.1.8, and Section 4.1 of this report, as applicable (lbf or N).

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, 2024 IBC Section 1905.7, 2021, 2018 and 2015 IBC Section 1905.1.8, and Section 4.1 of this report, as applicable (lbf or N).

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 Table 1, must apply.

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

For shear loads $V \leq 0.2V_{allowable,ASD}$, the full allowable load in tension $T_{allowable,ASD}$, must be permitted.

For tension loads $T \leq 0.2T_{allowable,ASD}$, the full allowable load in shear $V_{allowable,ASD}$, must be permitted.

For all other cases:

$$\frac{T_{applied}}{T_{allowable,ASD}} + \frac{V_{applied}}{V_{allowable,ASD}} \leq 1.2 \quad \text{(Eq-6)}$$

4.3 Installation:

Installation parameters are provided in Table 1 and Figures 1, 2, 3 and 6. Anchor locations must comply with this report and plans and specifications approved by the code official. The Hilti HDI-P TZ and HDI-TZ anchors 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 or with a Hilti HDI-P TZ or HDI-TZ stop drill bit. The minimum drilled hole depth, h_0, is given in Table 1. The HDI-P TZ or HDI-TZ is inserted into the predrilled hole and the setting plug is engaged into the anchor body using the manual HDI-P TZ or HDI-TZ setting tool and a hammer, or the automatic HDI-P TZ or HDI-TZ setting tool and a hammer drill. The setting plug must be driven until the shoulder of the HDI-P TZ or HDI-TZ setting tool is flush with the surface of the HDI-P TZ or HDI-TZ body. The minimum thread engagement of a threaded rod or bolt insert element assembly into the HDI-P TZ or HDI-TZ anchor must be the minimum thread engagement length as listed in Table 1 of this report.

4.4 Special Inspection:

Periodic special inspection is required in accordance with Section 1705.1.1 and Table 1705.3 of the 2024, 2021, 2018, and 2015 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, 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 HDI-P TZ or HDI-TZ anchors described in this report comply with or are suitable alternatives to what is specified in 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 are 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 The $^1/_4$-inch and $^3/_8$-inch (6.4 mm and 9.5 mm) HDI-P TZ anchors are limited to installation in the formed surface of 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), 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), and cracked and uncracked hollow-core concrete panels with the configuration and dimensions as indicated in Figure 3 having a minimum specified compressive strength, f'_c, of 6,000 psi (41.4 MPa).

5.4 The ½-inch (12.7 mm) HDI-P TZ and $^3/_8$-inch (9.5 mm) HDI-TZ anchors are limited to installation in the formed surface (underside) of 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).

5.5 The ½-inch and $^5/_8$-inch diameter (12.7 mm and 15.9 mm) HDI-TZ anchors are 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).

5.6 The values of f'_c used for calculation purposes must not exceed 8,000 psi (55.2 MPa).

5.7 The concrete shall have attained its minimum design strength prior to installation of the anchors.

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

5.9 Allowable design values are established in accordance with Section 4.2 of this report.

5.10 Anchor spacing and edge distance as well as minimum member thickness must comply with Table 1 and Figures 1, 2, 3, 4A, and 4B of this report.

5.11 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.

5.12 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.

5.13 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.

5.14 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.

5.15 Where not otherwise prohibited in the code, 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 are used to support nonstructural elements.

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

5.17 Use of $^1/_4$-inch, $^3/_8$-inch, and ½-inch (6.4 mm, 9.5 mm, and 12.7 mm) HDI-P TZ and $^3/_8$-inch (12.7 mm) HDI-TZ anchors are limited to supporting non-structural components.

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

5.19 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 April 2024), which incorporates requirements in ACI 355.2 (-19 and -07) for use in cracked and uncracked concrete.

6.2 Reports of tension and shear tests of anchors in hollow-core concrete panels in accordance with ASTM E488 and applicable sections of ACI 355.2 (-19 and -07) which are referenced under the ICC-ES Acceptance Criteria for Mechanical Anchors in Concrete Elements (AC193) in Section 6.1 of this report.

6.3 Quality-control documentation.

7.0 IDENTIFICATION

7.1 The ICC-ES mark of conformity, electronic labeling, or the evaluation report number (ICC-ES ESR-4236) along with the name, registered trademark, or registered logo of the report holder must be included in the product label

7.2 In addition, the anchors are identified by packaging labeled with the company name (Hilti, Inc.) and contact information, anchor name, and anchor size.

7.3 The report holder's contact information is as follows:

HILTI, INC.
7250 DALLAS PARKWAY, SUITE 1000
PLANO, TEXAS 75024
(800) 879-8000
www.hilti.com

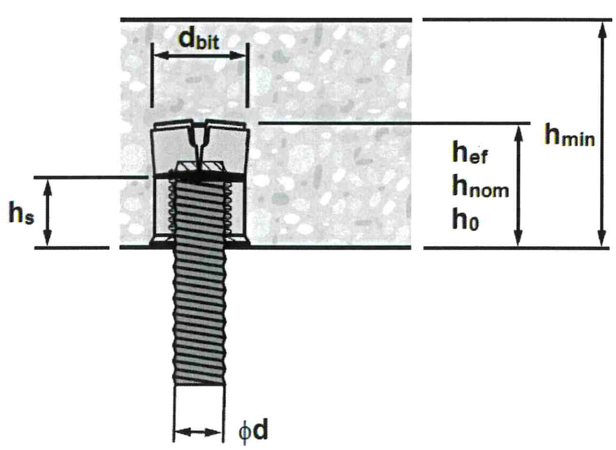


FIGURE 1—HILTI HDI-P TZ INSTALLATION PARAMETERS IN CONCRETE

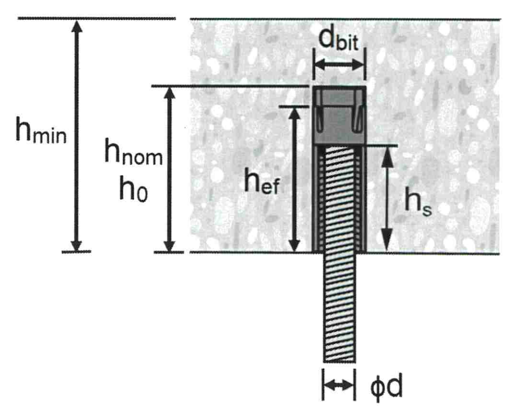


FIGURE 2—HILTI HDI-TZ INSTALLATION PARAMETERS IN CONCRETE

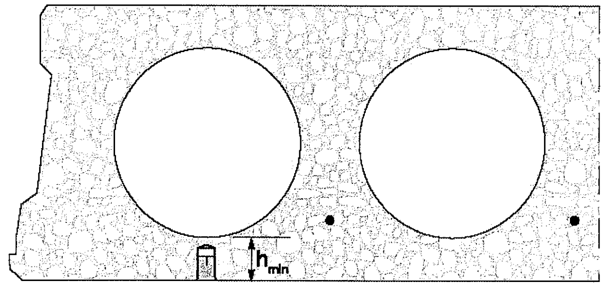


FIGURE 3 – HILTI HDI-P TZ INSTALLATION PARAMETERS IN HOLLOW CORE CONCRETE PANELS

TABLE 1—HILTI HDI-P TZ AND HDI-TZ SETTING INFORMATION

Setting information		Symbol	Units	Nominal anchor size / internal thread diameter (in)							
				HDI-P TZ					HDI-TZ		
				1/4		3/8		1/2	3/8	1/2	5/8
Internal thread diameter		d	in.	1/4		3/8		1/2	3/8	1/2	5/8
Nominal bit diameter		d_{bit}	in.	9/16		9/16		5/8	9/16	5/8	27/32
Effective embedment		h_{ef}	in.	3/4		3/4		1	1.42	1.65	3
			(mm)	(19)		(19)		(25)	(36)	(42)	(76)
Nominal embedment		h_{nom}	in.	3/4		3/4		1	1 9/16	2	3 1/4
			(mm)	(19)		(19)		(25)	(40)	(51)	(83)
Hole depth in base material		h_0	in.	3/4		3/4		1	1 9/16	2	3 1/4
			(mm)	(19)		(19)		(25)	(40)	(51)	(83)
Thread engagement length		h_s	in.	3/16		3/8		1/2	3/8 - 5/8	1/2 - 7/8	5/8 - 1 3/8
			(mm)	(5)		(10)		(13)	(10 – 16)	(13 – 22)	(16 – 35)
Maximum installation torque for threaded element		T_{max}	ft-lb	4.2		5.0		10.4	5.0	10.4	20.8
			(Nm)	(6)		(7)		(14)	(7)	(14)	(28)
Concrete	Minimum base material thickness	h_{min}	in.	2 ½	4	2 ½	4	4	3 1/4	4	6
			(mm)	(64)	(102)	(64)	(102)	(102)	(83)	(102)	(152)
	Minimum edge distance	c_{min}	in.	6	2 ½	6	2 ½	2 ½	3	6	8
			(mm)	(152)	(64)	(152)	(64)	(64)	(76)	(152)	(203)
	Minimum anchor spacing	s_{min}	in.	8	3	8	3	3	6	7	9
			(mm)	(203)	(76)	(203)	(76)	(76)	(152)	(178)	(229)
Hollowcore Concrete Planks	Minimum base material thickness	h_{min}	in.	1 3/8		1 3/8		N/A	N/A	N/A	N/A
			(mm)	(35)		(35)					
	Minimum edge distance	c_{min}	in.	6		6		N/A	N/A	N/A	N/A
			(mm)	(152)		(152)					
	Minimum anchor spacing	s_{min}	in.	8		8		N/A	N/A	N/A	N/A
			(mm)	(203)		(203)					

For **SI:** 1 inch = 25.4 mm, 1 ft-lb = 1.356 Nm

TABLE 2—HDI-P TZ AND HDI-TZ DESIGN INFORMATION

Design information	Symbol	Units	Nominal anchor size / internal thread diameter (in)					
			HDI-P TZ			HDI-TZ		
			1/4	3/8	1/2	3/8	1/2	5/8
Anchor O.D.	d_a	in. (mm)	0.561 (14.2)	0.561 (14.2)	0.625 (15.9)	0.561 (14.2)	0.625 (15.9)	0.844 (21.4)
Effective embedment	h_{ef}	in. (mm)	3/4 (19)	3/4 (19)	1 (25)	1.42 (36)	1.65 (42)	3 (76)
Tension - Steel Failure Mode								
Strength reduction factor for steel in tension [1,2]	$\phi_{sa,N}$	-	0.65					
Min. specified yield strength	f_{ya}	psi (N/mm^2)	70,400 (485)	70,400 (485)	70,400 (484)	79,600 (549)	70,400 (485)	58,000 (400)
Min. specified ult. strength	f_{uta}	psi (N/mm^2)	88,000 (607)	88,000 (607)	88,000 (607)	99,500 (686)	88,000 (607)	72,500 (500)
Effective-cross sectional steel area in tension	$A_{se,N}$	in^2 (mm^2)	0.071 (45.8)	0.071 (45.8)	0.072 (46.5)	0.058 (37.4)	0.068 (43.9)	0.169 (109.0)
Nominal steel strength in tension	N_{sa}	lb (kN)	2000 (8.9)	6,250 (27.8)	6,335 (28.2)	5,770 (25.7)	5,985 (26.6)	12,255 (54.5)
Tension - Concrete Failure Modes								
Anchor category	-	-	1					
Strength reduction factor for concrete failure in tension [2]	$\phi_{c,N}$	-	0.40				0.65	
Effectiveness factor for uncracked concrete	k_{uncr}	in-lb (SI)	24 (10.0)				27 (11.3)	24 (10.0)
Effectiveness factor for cracked concrete	k_{cr}	in-lb (SI)	17 (7.1)		21 (8.8)		24 (10.0)	21 (8.8)
Modification factor for anchor resistance, tension, uncracked conc. [3]	$\psi_{c,N}$	-	1.0					
Critical edge distance	c_{ac}	in. (mm)	6 1/2 (165)	6 1/2 (165)	4 (102)	5 1/2 (140)	6 1/2 (165)	12 (305)
Pullout strength in uncracked concrete [4]	$N_{p,uncr}$	lb (kN)	N/A					
Pullout strength in cracked concrete [4]	$N_{p,cr}$	lb (kN)	470 (2.1)	470 (2.1)	910 (4.0)	N/A		
Pullout strength in cracked concrete, seismic [4]	$N_{p,eq}$	lb (kN)	465 (2.1)	465 (2.1)	820 (3.6)	N/A		
Shear - Steel Failure Mode								
Strength reduction factor for steel in shear [1,2]	$\phi_{sa,V}$	-	0.60					
Nominal steel strength in shear	V_{sa}	lb (kN)	975 (4.3)	975 (4.3)	3,800 (16.9)	3,465 (15.4)	3,590 (16.0)	7,350 (32.7)
Nominal steel strength in shear, seismic	$V_{sa,eq}$	lb (kN)	975 (4.3)	975 (4.3)	2,385 (10.6)	2,355 (10.5)	2,600 (11.6)	5,265 (23.4)
Shear - Concrete Failure Modes								
Strength reduction factor for concrete breakout failure in shear [2]	$\phi_{c,V}$	-	0.45				0.70	
Effectiveness factor for pryout	k_{cp}	-	1.0					2.0
Tension - Axial Stiffness								
Mean axial stiffness [5] — Uncracked concrete	β_{uncr}	lbf/in.	164,365	164,365	95,620	65,420	111,055	101,960
Mean axial stiffness [5] — Cracked concrete	β_{cr}	lbf/in.	48,895	48,895	35,050	66,485	40,450	84,940

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.45 N, 1 psi = 0.006895 N/mm^2.

[1] The HDI-P TZ and HDI-TZ anchors are considered a brittle steel element as defined by ACI 318 (-19 and -14) 2.3, as applicable.

[2] The strength reduction factor applies when the load combinations from the IBC or ACI 318 are used and the requirements of ACI 318-19 17.5.3 or ACI 318-14 17.3.3, as applicable, are met. For concrete failure modes with h_{ef} < 1.5-inch (40mm), no increase for Condition A (supplementary reinforcement present) is permitted.

[3] 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.

[4] 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 $(f'_c / 2,500)^{0.35}$ for psi or $(f'_c / 17.2)^{0.35}$ for MPa. N/A (not applicable) denotes that pullout strength does not need to be considered for design.

[5] Mean values shown. Actual stiffness varies considerably depending on concrete strength, loading, and geometry of application.

TABLE 3—SPECIFICATIONS AND PHYSICAL PROPERTIES OF COMMON CARBON STEEL THREADED ROD ELEMENTS

Threaded rod specification	Units	Min. specified ultimate strength f_{uta}	Min. specified yield strength, 0.2 percent offset, f_{ya}	f_{uta} / f_{ya}	Elongation, min. percent	Reduction of area, min. percent	Specification for nuts [2]
Carbon steel: ASTM A36 / A36M [1]	psi (MPa)	58,000 (400)	36,000 (248)	1.61	23	40	ASTM A194 or ASTM A563

For **SI:** 1 inch = 25.4 mm, 1 psi = 0.006895 N/mm².

[1] *Standard Specification for Carbon Structural Steel.*
[2] Nuts of other grades and styles having specified proof load stresses greater than the specified grade and style are also suitable.

TABLE 4—STEEL DESIGN INFORMATION FOR THREADED ELEMENTS USED WITH HDI-P TZ AND HDI-TZ ANCHORS [1,2,3]

	Design Information	Symbol	Units	Nominal anchor size / internal thread diameter (in)			
				1/4	3/8	1/2	5/8
	Nominal rod diameter	d_{rod}	in. (mm)	0.250 (6.4)	0.375 (9.5)	0.500 (12.7)	0.625 (15.9)
	Rod effective cross-sectional area	$A_{se,rod}$	in² (mm²)	0.0318 (21)	0.0775 (50)	0.1419 (92)	0.2260 (146)
ASTM A36 Steel Material	Strength reduction factor for steel in tension[4]	$\phi_{sa,rod,N}$	-	0.75			
	Nominal steel strength in tension	$N_{sa,rod}$	lb (kN)	1,845 (8.2)	4,495 (20.0)	8,230 (36.6)	13,110 (58.3)
	Nominal steel strength in tension, seismic	$N_{sa,rod,eq}$	lb (kN)	1,845 (8.2)	4,495 (20.0)	8,230 (36.6)	13,110 (58.3)
	Strength reduction factor for steel in shear[4]	$\phi_{sa,rod,V}$	-	0.65			
	Nominal steel strength in shear	$V_{sa,rod}$	lb (kN)	1,105 (4.9)	2,695 (12.0)	4,940 (22.0)	7,865 (35.0)
	Nominal steel strength in shear, seismic	$V_{sa,rod,eq}$	lb (kN)	775 (3.4)	1,885 (8.4)	3,460 (15.4)	5,505 (24.5)

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.45 N, 1 psi = 0.006895 N/mm².

[1] Values provided for steel element material types, or equivalent, based on minimum specified strengths and calculated in accordance with ACI 318-19 Eq. (17.6.1.2) and Eq. (17.7.1.2b); or ACI 318-14 Eq. (17.4.1.2) and Eq. (17.5.1.2b), as applicable. $V_{sa,eq,rod}$ must be taken as $0.7V_{sa,rod}$.
[2] ϕN_{sa} shall be the lower of $\phi N_{sa,rod}$ or ϕN_{sa} for static steel strength in tension; for seismic loading, $\phi N_{sa,eq}$ shall be the lower of $\phi N_{sa,rod,eq}$ or $\phi N_{sa,eq}$.
[3] ϕV_{sa} shall be the lower of $\phi V_{sa,rod}$ or ϕV_{sa} for static steel strength in shear; for seismic loading, $\phi V_{sa,eq}$ shall be the lower of $\phi V_{sa,rod,eq}$ or $\phi V_{sa,eq}$.
[4] The strength reduction factor applies when the load combinations from the IBC or ACI 318 are used and the requirements of ACI 318-19 17.5.3 or ACI 318-14 17.3.3, as applicable, are met.

TABLE 5—HDI-P TZ AND HDI-TZ TENSION AND SHEAR DESIGN DATA FOR INSTALLATION IN THE SOFFIT OF 3,000 PSI, LIGHTWEIGHT CONCRETE-FILLED PROFILE STEEL DECK ASSEMBLIES[2,3]

Design Information	Symbol	Units	Nominal Anchor Size / Internal Thread Dia. (in)				
			HDI-P TZ			HDI-TZ	
			1/4	3/8	1/2	3/8	1/2
Effective Embedment[1]	h_{ef}	in.	3/4	3/4	1	1.42	1.65
		(mm)	(19)	(19)	(25)	(36)	(42)
Hole Depth in Base Material	h_o	in.	3/4	3/4	1	1 9/16	2
		(mm)	(19)	(19)	(25)	(40)	(51)
Loads According to Figure 4A							
Minimum Concrete Thickness Over Upper Flute - Lower Flute Installation[4]	$h_{min,deck,lower}$	in.	2	2	2	2	2
		(mm)	(51)	(51)	(51)	(51)	(51)
Minimum Concrete Thickness Over Upper Flute - Upper Flute Installation[4]	$h_{min,deck,upper}$	in.	2	2	2	2 1/2	3 1/4
		(mm)	(51)	(51)	(51)	(64)	(83)
Pullout Strength Uncracked Concrete[5,6]	$N_{p,deck,uncr}$	lb	825	825	1235	1330	1910
		(kN)	(3.7)	(3.7)	(5.5)	(5.9)	(8.5)
Pullout Strength Cracked Concrete[5,6]	$N_{p,deck,cr}$	lb	400	400	935	1165	1695
		(kN)	(1.8)	(1.8)	(4.2)	(5.2)	(7.5)
Pullout Strength Seismic[5,7]	$N_{p,deck,eq}$	lb	395	395	845	1165	1695
		(kN)	(1.8)	(1.8)	(3.8)	(5.2)	(7.5)
Steel Strength in Shear[8]	$V_{sa,deck}$	lb	2995	2995	3425	3210	3590
		(kN)	(13.3)	(13.3)	(15.2)	(14.3)	(16.0)
Steel Strength in Shear, Seismic[7]	$V_{sa,deck,eq}$	lb	2995	2995	2150	2180	2600
		(kN)	(13.3)	(13.3)	(9.6)	(9.7)	(11.6)
Loads According to Figure 4B							
Minimum Concrete Thickness Over Upper Flute - Lower Flute Installation[4]	$h_{min,deck,lower}$	in.	2	2	2	2	2
		(mm)	(51)	(51)	(51)	(51)	(51)
Minimum Concrete Thickness Over Upper Flute - Upper Flute Installation[4]	$h_{min,deck,upper}$	in.	2	2	2	2 1/2	3 1/4
		(mm)	(51)	(51)	(51)	(64)	(83)
Pullout Strength Uncracked Concrete[5,6]	$N_{p,deck,uncr}$	lb	530	530	925	1070	1385
		(kN)	(2.4)	(2.4)	(4.1)	(4.8)	(6.2)
Pullout Strength Cracked Concrete[5,6]	$N_{p,deck,cr}$	lb	255	255	700	940	1235
		(kN)	(1.1)	(1.1)	(3.1)	(4.2)	(5.5)
Pullout Strength Seismic[5,7]	$N_{p,deck,eq}$	lb	250	250	635	940	1235
		(kN)	(1.1)	(1.1)	(2.8)	(4.2)	(5.5)
Steel Strength in Shear[8]	$V_{sa,deck}$	lb	1775	1775	2130	2370	2435
		(kN)	(7.9)	(7.9)	(9.5)	(10.5)	(10.8)
Steel Strength in Shear, Seismic[7]	$V_{sa,deck,eq}$	lb	1775	1775	1335	1610	1765
		(kN)	(7.9)	(7.9)	(5.9)	(7.2)	(7.9)

[1] Installations must comply with Section 4.1.10, Section 4.3, Figure 4A, and Figure 4B of this report.
[2] The values for $\Phi_{p,N}$ in tension can be found in Table 2 of this report. The values for $\Phi_{sa,V}$ in shear can be found in Table 2 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 or ACI 318-14 17.4.2, 17.5.2, and 17.5.3, 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 4A and 4B.
[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 $(f_c / 3{,}000)^{0.35}$ for psi or $(f_c / 20.7)^{0.35}$ 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.

TABLE 6— HDI-TZ SETTING INFORMATION FOR INSTALLATION ON THE TOP OF CONCRETE-FILLED PROFILE STEEL DECK ASSEMBLIES ACCORDING TO FIGURE 4C

Design Information	Symbol	Units	Nominal anchor size / internal thread dia. (in)	
			1/2	5/8
Effective Embedment Depth	h_{ef}	in. (mm)	1.65 (42)	3 (76)
Nominal Embedment Depth	h_{nom}	in. (mm)	2 (51)	3 1/4 (83)
Minimum Hole Depth	h_o	in. (mm)	2 (51)	3 1/4 (83)
Minimum Concrete Thickness[4]	$h_{min,deck}$	in. (mm)	2 1/2 (64)	3 1/4 (83)
Critical Edge Distance	$c_{ac,deck,top}$	in. (mm)	6.50 (165)	16 (406)
Minimum Edge Distance	$c_{min,deck,top}$	in. (mm)	2 (51)	2 (51)
Minimum Spacing	$s_{min,deck,top}$	in. (mm)	4 (102)	4 (102)

[1] Installations must comply with Section 4.1.10, Section 4.3, and Figure 4C of this report.
[2] Design capacity shall be based on calculations according to values in Table 2 of this report.
[3] Applicable for $h_{min,deck} < h_{min,Table\ 1}$. For $h_{min,deck} > h_{min,Table\ 1}$, use setting information in Table 1 and critical edge distances in Table 2 of this report.
[4] Minimum concrete thickness refers to concrete thickness above the upper flute. See Figure 4C.

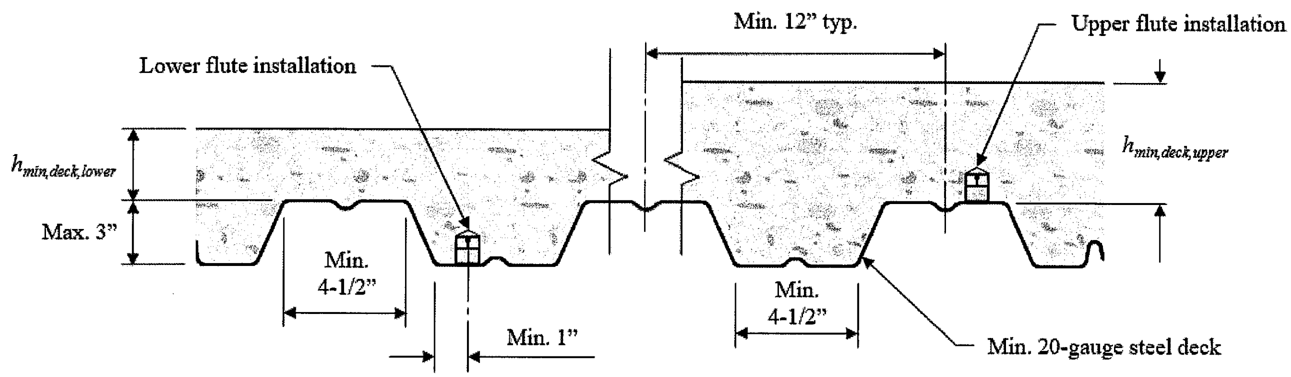


FIGURE 4A – HDI-P TZ AND HDI-TZ IN THE SOFFIT OF CONCRETE FILLED PROFILE STEEL DECK ASSEMBLIES – W DECK

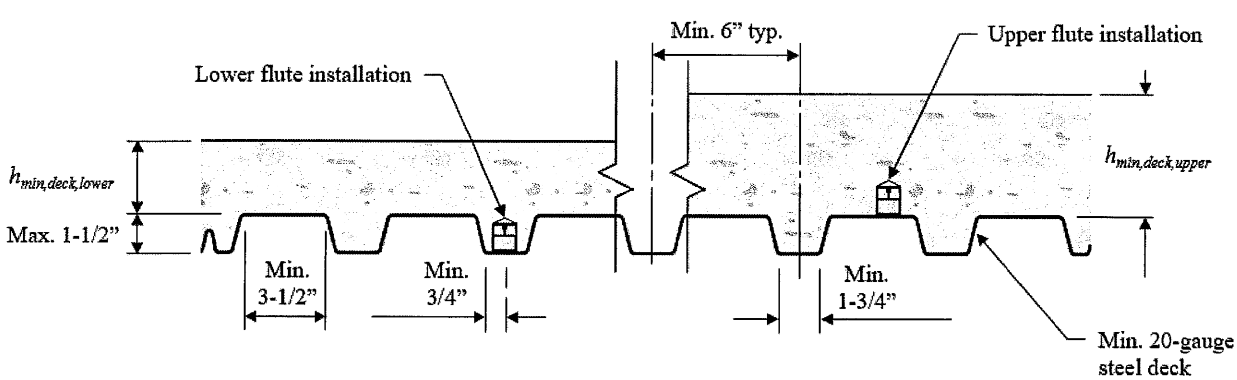


FIGURE 4B – HDI-P TZ AND HDI-TZ IN THE SOFFIT OF CONCRETE FILLED PROFILE STEEL DECK ASSEMBLIES – B DECK

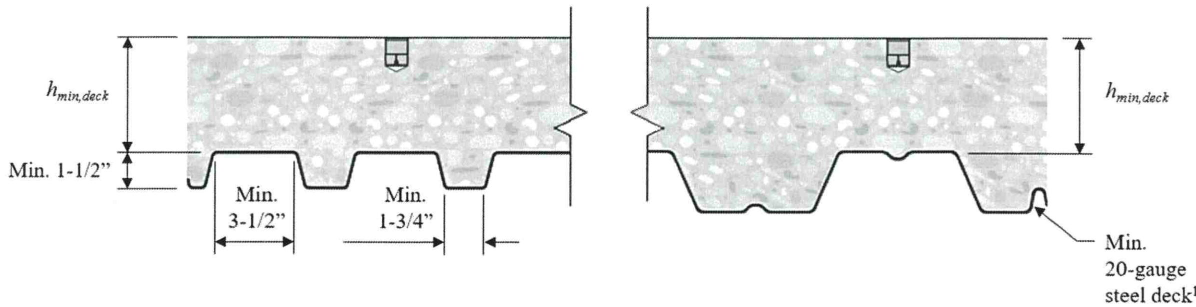


FIGURE 4C – HDI-TZ IN THE TOP OF CONCRETE FILLED PROFILE STEEL DECK ASSEMBLIES

[1] 1-1/2 inches (38mm) B-deck as a minimum profile size. Other deck profiles meeting the B-deck minimum dimensions are also permitted.

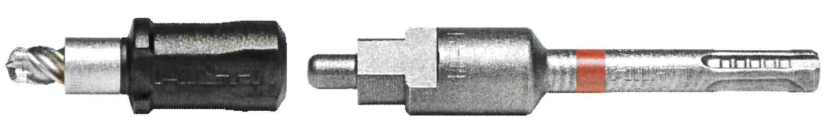
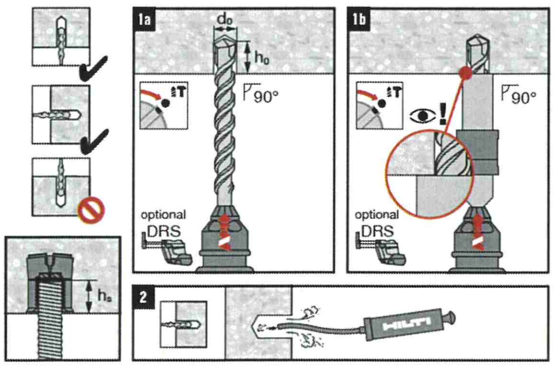
Hilti HDI-P TZ and HDI-TZ anchor	
Optional Dust Removal (DRS) module for drilling for use with Hilti hammer drills	
Hilti HDI-P TZ and HDI-TZ stop drill bit with automatic setting tool combination for use with hammer drill	
Hilti HDI-P TZ and HDI-TZ manual setting tool for use with hammer	

FIGURE 5—HILTI HDI-P TZ AND HDI-TZ ANCHOR, DRILLING, AND SETTING TOOLS

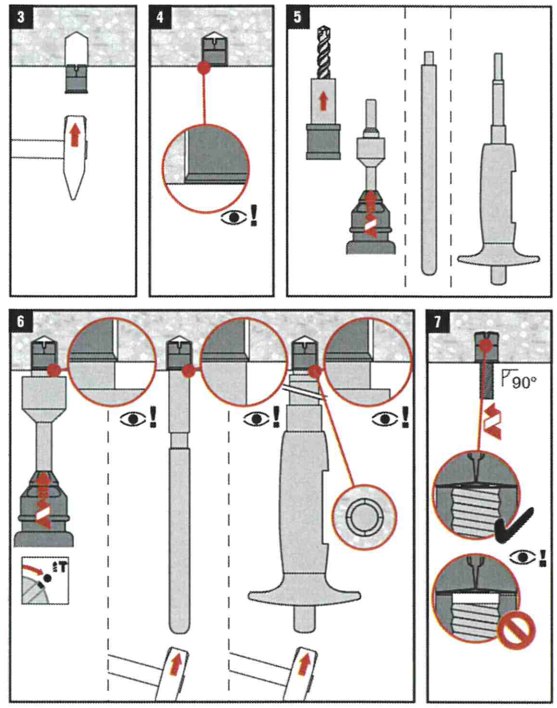
HILTI / HDI-P TZ

2400468-01.2024

Specification	¼"	⅜"	½"
Hole Diameter d_0	9/16"	9/16"	⅝"
Rod size	¼"	⅜"	½"
Thread engagement h_s	~3/16" (4.5 mm)	~⅜" (10 mm)	~½" (13 mm)
Drilling deph h_0	¾"	¾"	1"
Max. Installation torque T_{inst}	4.2 ft-lb (5 Nm)	5 ft-lb (7 Nm)	10.4 ft-lb (14 Nm)
Hand Setting tools	HST-P TZ ¼" HSD-G-P TZ ¼"	HST-P TZ ⅜" HSD-G P TZ ⅜"	HST-P TZ ½" HSD-G P TZ ½"
2-in-1 Setting tools	Setting tool HDI-P TZ ¼"	Setting tool HDI-P TZ ⅜"	Setting tool HDI-P TZ ½"



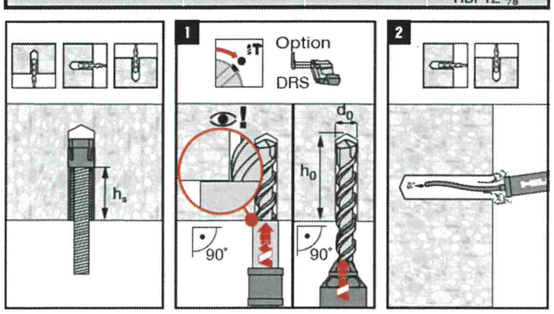
HILTI / HDI-P TZ



HILTI / HDI-TZ

2368343-01.2024

Specification	⅜"	½"	⅝"
Hole Diameter d_0	9/16"	⅝"	27/32"
Rod size	⅜"	½"	⅝"
Fixture hole diameter d_{fix}	7/16"	9/16"	11/16"
Thread engagement h_s	~⅜ – ⅝" (10 – 16 mm)	~½ – ⅞" (13 – 22 mm)	~⅝ – 1⅜" (16 – 35 mm)
Drilling deph h_0	1 9/16"	2"	3¼"
Max. Installation torque T_{inst}	5 ft-lb (7 Nm)	10.4 ft-lb (14 Nm)	20.8 ft-lb (28 Nm)
Hand Setting tools	HST TZ ⅜" HSD-G TZ ⅜"	HST TZ ½" HSD-G TZ ½"	HST TZ ⅝" HSD-G TZ ⅝"
2-in-1 Setting tools	Setting tool HDI TZ ⅜"	Setting tool HDI TZ ½"	Stop drillbit HDI-TZ ⅝" Setting tool HDI-TZ ⅝"



HILTI / HDI-TZ

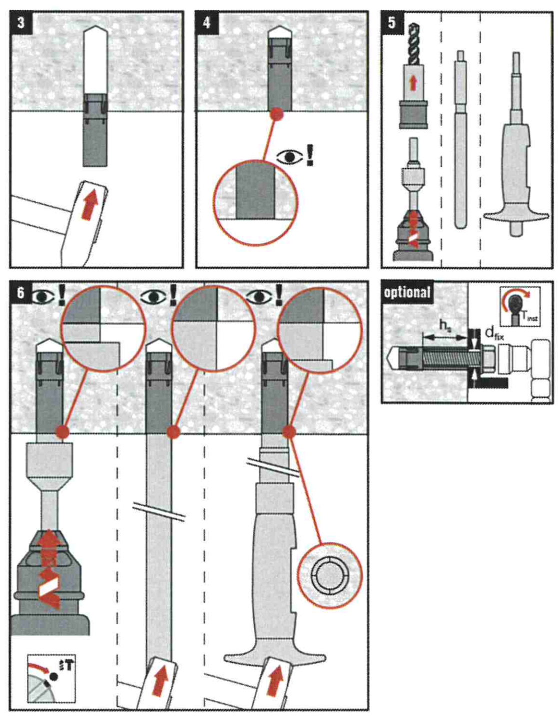


FIGURE 6—INSTALLATION INSTRUCTIONS

ICC-ES Evaluation Report

ESR-4236 LABC and LARC Supplement

Reissued July 2023
Revised May 2024
This report is subject to renewal July 2025.

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 HDI-P TZ AND HDI-TZ ANCHORS IN CRACKED AND UNCRACKED CONCRETE

1.0 REPORT PURPOSE AND SCOPE

Purpose:

The purpose of this evaluation report supplement is to indicate that Hilti HDI-P TZ and HDI-TZ anchors in cracked and uncracked concrete, described in ICC-ES evaluation report ESR-4236, 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:

- 2023 *City of Los Angeles Building Code* (LABC)
- 2023 *City of Los Angeles Residential Code* (LARC)

2.0 CONCLUSIONS

The Hilti HDI-P TZ and HDI-TZ anchors, described in Sections 2.0 through 7.0 of the evaluation report ESR-4236, comply with LABC Chapter 19, and the LARC, and are subjected to the conditions of use described in this supplement.

3.0 CONDITIONS OF USE

The Hilti HDI-P TZ and HDI-TZ anchors described in this evaluation report supplement must comply with all of the following conditions:

- All applicable sections in the evaluation report ESR-4236.
- The design, installation, conditions of use and identification of the anchors are in accordance with the 2021 *International Building Code®* (IBC) provisions noted in the evaluation report ESR-4236.
- The design, installation and inspection are in accordance with additional requirements of LABC Chapters 16 and 17, and City of Los Angeles Information Bulletin P/BC 2020-092, 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 the concrete. The connection between the anchors and the connected members shall be checked for capacity (which may govern).

This supplement expires concurrently with the evaluation report, reissued July 2023 and revised May 2024.

ICC-ES Evaluation Report

ESR-4236 FBC Supplement

Reissued July 2023

Revised May 2024

This report is subject to renewal July 2025.

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 HDI-P TZ AND HDI-TZ 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 Hilti HDI-P TZ and HDI-TZ anchors in cracked and uncracked concrete, described in ICC-ES evaluation report ESR-4236, have also been evaluated for compliance with the codes noted below.

Applicable code editions:

- 2023 *Florida Building Code—Building*
- 2023 *Florida Building Code—Residential*

2.0 CONCLUSIONS

The Hilti HDI-P TZ and HDI-TZ anchors in cracked and uncracked concrete, described in Sections 2.0 through 7.0 of ICC-ES evaluation report ESR-4236, comply with the *Florida Building Code—Building* and the *Florida Building Code—Residential*. The design requirements must be determined in accordance with the *Florida Building Code—Building* or the *Florida Building Code—Residential*, as applicable. The installation requirements noted in ICC-ES evaluation report ESR-4236 for the 2021 *International Building Code®* meet the requirements of the *Florida Building Code—Building* or the *Florida Building Code—Residential*, as applicable.

Use of the Hilti HDI-P TZ and HDI-TZ anchors in cracked and uncracked concrete have also been found to be in compliance with the High-Veloctiy Hurricane Zone provisions of the *Florida Building Code—Building* and the *Florida Building Code—Residential*, with the following condition:

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

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, reissued July 2023 and revised May 2024.