ARW ENGINEERS
structural consultants

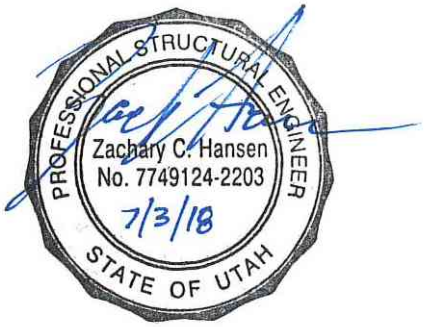
Structural Calculations

For

Kimberly Clark Parts Storage Building

Project Number: 18179

July 3, 2018



Prepared by
ARW Engineers
1594 West Park Circle
Ogden, Utah 84404

ENGINEERS

Uplift Capacity of Exterior Concrete Footings

Version Date: January 27, 2010 Author: SLE Reviewed By: ZCH

JOB TITLE: Kimberly Clark Parts Storage
DESCRIPTION: Footing Uplift Schedule

JOB #: 18179
DESIGNED BY: ZCH

Slab-on Grade Parameters: (See User Note #1)
Slab Thickness = 6 inches
Slab Percentage = 50%

Foundation Wall Parameters: (See User Note #2)
Wall Thickness = 8 inches
Foundation Depth = 24 inches

Strip Footing Parameters: (Strip footing below foundation wall)
Footing Width = 24 inches
Footing Thickness = 12 inches
inches

Additional Wall and Footing Length = 4 feet
(See User Note #3)

Soil Weight Parameters: (See User Note #4)
Soil Weight = 115 pcf
Soil Depth = 18 inches

Concrete Weight 145 pcf

User Notes:

1- If a concrete slab-on-grade is present, the program will calculate and include a portion of the slab weight above the spot footing. The "Slab Percentage" cell is used to determine what percentage of the footing area is covered by the slab.

2- If a foundation wall is present, the program will calculate and include the weight of the wall that is directly above the spot footing.

3- Due to "beam action" from the foundation wall, a portion of the foundation wall and strip footing beyond the dimensions of the spot footing may be used. The distance to be included beyond each side of the spot footing is entered in the "Additional Wall and Footing Length" cell.

4- The program will calculate the weight of soil directly above the spot footing. Do not include the thickness of the slab in the soil depth.

NOTE: FOOTING THICKNESSES NEED TO BE ADJUSTED TO MATCH THE FOOTING SCHEDULE ON THE DRAWINGS

	Width (ft)	Area (sq. ft)	Thickness (in)	Volume (cu. ft)	Spot Ftg. Weight (lbs.)	Slab Weight (lbs.)	Foundation Wall Weight (lbs.)	Strip Ftg. Weight (lbs.)	Soil Weight (lbs.)	Total Weight (kips)
F3.0	3	9	12	9	1305	326	2127	2320	1553	***7.63***
F3.5	3.5	12.25	12	12	1776	444	2223	2320	2113	***8.88***
F4.0	4	16	12	16	2320	580	2320	2320	2760	***10.30***
F4.5	4.5	20.25	12	20	2936	734	2417	2320	3493	***11.90***
F5.0	5	25	12	25	3625	906	2513	2320	4313	***13.68***
F5.5	5.5	30.25	14	35	5117	1097	2610	2320	5218	***16.36***
F6.0	6	36	16	48	6960	1305	2707	2320	6210	***19.50***
F6.5	6.5	42.25	16	56	8168	1532	2803	2320	7288	***22.11***
F7.0	7	49	18	74	10658	1776	2900	2320	8453	***26.11***
F7.5	7.5	56.25	18	84	12234	2039	2997	2320	9703	***29.29***
F8.0	8	64	20	107	15467	2320	3093	2320	11040	***34.24***
F8.5	8.5	72.25	20	120	17460	2619	3190	2320	12463	***38.05***
F9.0	9	81	22	149	21533	2936	3287	2320	13973	***44.05***
F9.5	9.5	90.25	24	181	26173	3272	3383	2320	15568	***50.72***
F10.0	10	100	24	200	29000	3625	3480	2320	17250	***55.68***
F10.5	10.5	110.25	26	239	34637	3997	3577	2320	19018	***63.55***
F11.0	11	121	26	262	38014	4386	3673	2320	20873	***69.27***
F11.5	11.5	132.25	28	309	44745	4794	3770	2320	22813	***78.44***
F12.0	12	144	28	336	48720	5220	3867	2320	24840	***84.97***
F12.5	12.5	156.25	0	0	0	5664	3963	2320	26953	***38.90***
F13.0	13	169	0	0	0	6126	4060	2320	29153	***41.66***
F13.5	13.5	182.25	0	0	0	6607	4157	2320	31438	***44.52***
F14.0	14	196	0	0	0	7105	4253	2320	33810	***47.49***
F14.5	14.5	210.25	0	0	0	7622	4350	2320	36268	***50.56***
F15.0	15	225	0	0	0	8156	4447	2320	38813	***53.74***
F15.5	15.5	240.25	0	0	0	8709	4543	2320	41443	***57.02***
F16.0	16	256	0	0	0	9280	4640	2320	44160	***60.40***
F16.5	16.5	272.25	0	0	0	9869	4737	2320	46963	***63.89***
F17.0	17	289	0	0	0	10476	4833	2320	49853	***67.48***
F17.5	17.5	306.25	0	0	0	11102	4930	2320	52828	***71.18***
F18.0	18	324	0	0	0	11745	5027	2320	55890	***74.98***

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FOOTING SUMMARY (PER IBC 2015 & ACI 318-14)

6/22/18 2:40 PM

JOB TITLE: Kimberly Clark Parts Storage
DESCRIPTION: Footing Schedule

JOB #: 18179
DESIGNER: ZCH
Version : September 1, 2017

COLUMN SUMMARY:

Material
Steel or Wood

Size
8 in

Base Plate Dimension
14 x 14 in^2

SOIL SUMMARY:

Net Soil Bearing Pressure
2500 psf

CONCRETE SUMMARY:

Concrete Strength
3000 psi

WIDTH (ft)	LENGTH (ft)	THICK (in)	$A_{s\ req'd}$ (in^2)	$A_{s\ actual}$ (in^2)	Number of Bars Ea. Way	Bar No.	Max Total Load (kips)
2	2	12	0.52	0.62	(2)	# 5	10.0
2.5	2.5	12	0.65	0.93	(3)	# 5	15.6
3	3	12	0.78	0.93	(3)	# 5	22.5
3.5	3.5	12	0.91	0.93	(3)	# 5	30.6
4	4	12	1.04	1.24	(4)	# 5	40.0
4.5	4.5	12	1.17	1.24	(4)	# 5	50.6
5	5	12	1.37	1.55	(5)	# 5	62.5
5.5	5.5	14	1.66	2.20	(5)	# 6	75.6
6	6	16	2.07	2.64	(6)	# 6	90.0
6.5	6.5	16	2.27	2.64	(6)	# 6	105.6
7	7	18	2.72	3.08	(7)	# 6	122.5
7.5	7.5	18	3.14	3.08	(7)	# 6	140.6
8	8	20	3.46	3.52	(8)	# 6	160.0
8.5	8.5	20	4.15	4.80	(8)	# 7	180.6
9	9	22	4.45	5.40	(9)	# 7	202.5
9.5	9.5	24	4.92	5.40	(9)	# 7	225.6
10	10	24	5.63	6.00	(10)	# 7	250.0
10.5	10.5	26	6.00	6.00	(10)	# 7	275.6
11	11	26	6.96	6.60	(11)	# 7	302.5
11.5	11.5	28	7.36	8.69	(11)	# 8	330.6
12	12	28	8.42	9.48	(12)	# 8	360.0

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25-Jun-18
8:30 AM

IBC 2015 Cast-in-place Anchor Bolt Design referencing ACI 318-14 Chapter 17

Version Date: December 5, 2017 Author: Troy M. Dye Reviewed By: Scott Porter

JOB TITLE: Kimberly Clark Parts Storage JOB #: 18179
DESCRIPTION: Grid 2 thru 10 Anchors DESIGNER: ZCH

Pier and Anchor Bolt Geometry

	Min Edge or Spacing		Bolt Quantities
Top Pier Edge	6		
AB-f5	0		
AB-f4	0		
AB-f3	0		
AB-f2	0		
AB-f1	2		2
Column centerline	0	<<	
AB-b1	2		2
AB-b2	0		
AB-b3	0		
AB-b4	0		
AB-b5	0		
Bottom Pier Edge	10		
			4 # Bolts in Group
Left Pier Edge	6		
AB-l1	2		
Column centerline	0	<<	
AB-r1	2		
Right Pier Edge	6		

Pier Dimensions
16.0 " wide
20.0 " long

Bolt Group Dimensions
4.0 " wide
4.0 " long

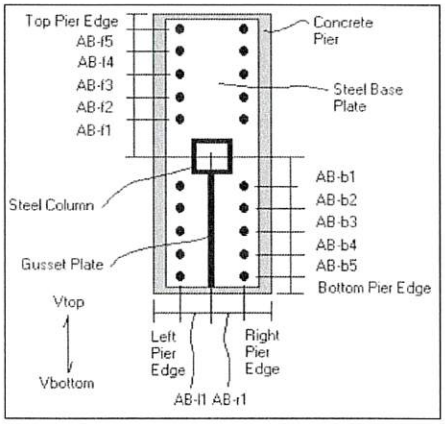

Figure 1. Generic Anchor Bolt Locations

Design Loads

Shear V_u : 16 kips *Use ACI 318 Section 5.3 load combinations*

Tension P_u: 9.3 kips

Is the seismic tension component more than 20% of the total factored tension force? YES

Increase Shear Vu and Tension Pu above by Omega

Pier confined for tension force? YES

Pier confined for shear force? NO

Anchors Torqued: NO
Bolt Grade: F1554 Gr. 55
Bolt Diameter: 0.75 inches
Embedment Length: 12 inches
Concrete f'_c: 4500 psi

Rebar f_y: 60000 psi
Anchor F_u : 75 ksi
Bolt Area A_{bolt} : 0.33 in^2
Seismic Design Category Factor: 0.75 *Section 17.2.3.4.4*

Pier Ht / Ftg Thick: 36 inches **OK**
Bolt type: H

Omega Level Force 17.2.3.4.3d & 17.2.3.5.3c

Bearing Area: 0.79 in^2
Steel ϕ_t: 0.75 *Section 17.3.3*
Steel ϕ_s: 0.65 *Section 17.3.3*
Concrete ϕ: 0.75 *Section 17.3.3*
λ : 1 *Section 19.2.4*
λ_a : 1 *Section 17.2.6*

Eccentricity for Tension
0 in
Eccentricity for Shear
0

Bolt Diameter (in^2)	*Threads per* (inch)	*Tensile Stress Area* (in^2)
0.5	13	0.14
0.625	11	0.23
0.75	10	0.33
0.875	9	0.46
1	8	0.61
1.125	7	0.76
1.25	7	0.97
1.375	6	1.15
1.5	6	1.41
1.75	5	1.90
2	4.5	2.50
2.25	4.5	3.25
2.5	4	4.00
2.75	4	4.93
3	4	5.97

Tension Calculations

A_N : 320 in^2
A_{NO} : 400 in^2

c_{a2}
Left Edge Dist. : 6 inches
Right Edge Dist. : 6 inches

c_{a1}
Top Edge Dist. : 6 inches
Bottom Edge Dist. : 10 inches

Concrete Tension Capacity (Section 17.4.2) - Breakout Strength

$\psi_{ec,N}$: 1.00 $\psi_{ed,N}$: 0.80 $\psi_{c,N}$: 1

N_b = 25.3 kips N_{cb} = 16.2 kips

Concrete Tension Capacity (Section 17.4.3) - Pullout Strength

$\psi_{c,P}$: 1 N_{pn} = 28.4 kips

N_p = 28.4 kips Capacity of Group: 113.8 kips

Concrete Tension Capacity (Section 17.4.4) - Side-Face Blowout Strength

Use A_N : 320 in^2

N_{sb} = N/A kips N_{sbg} = N/A kips

ϕP_c : 64.0 kips P_c : **113.8** kips Ultimate Concrete Strength Based on Pullout Only

Steel Tension Capacity (Section 17.4.1)

P_{ss}' : 25.1 kips / AB P_{ss} : **100.3** kips Ultimate Steel Tension Capacity

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8:30 AM

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IBC 2015 Cast-in-place Anchor Bolt Design referencing ACI 318-14 Chapter 17

Version Date: December 5, 2017
JOB TITLE: Kimberly Clark Parts Storage
DESCRIPTION: Grid 2 thru 10 Anchors

JOB #: 18179
DESIGNER: ZCH

Shear Calculations

Bolts Resisting Shear

B — Specify which system resists shear
Bolts resist tension and shear

Steel Shear Capacity (Section 17.5.1)

V_{sa}: 15.05 kips / AB — $\mathbf{V_{sa}}$: **60.2** kips — Ultimate Steel Shear Capacity based on Assumed Breakout Surface

n: 4 — Number of anchors resisting shear based on assumed concrete breakout surface

Grout height below base plate, h_2: 0 inches

Concrete Shear Breakout Capacity (Section 17.5.2)

$\psi_{ec,V}$: 1.00 $\psi_{ed,V}$: 0.82 $\psi_{c,V}$: 1.2 $\psi_{h,V}$: 1.00

Top Edge Bolts			Bottom Edge Bolts		
Edge Distance:	6.0	inches	Edge Distance :	10.0	inches
A_{Vc} =	144.0	in^2	A_{Vc} =	240.0	in^2
A_{Vco} =	162.0	in^2	A_{Vco} =	450.0	in^2
V_b =	9.1	kips	V_b =	19.5	kips
V_{cb} =	7.9	kips	V_{cb} =	10.2	kips

Concrete Shear Pryout Capacity (Section 17.5.3)

V_{cp} =	86.4	kips	N_b =	67.5	kips
Group Capacity $\mathbf{V_n}$:	**31.7**	kips	Group Capacity $\mathbf{V_n}$:	**40.9**	kips

Ultimate Concrete Strength

Combined Tension and Shear

Shear Force Acting Towards Top Pier Edge, V_{top}			Shear Force Acting Towards Bottom Pier Edge, V_{bottom}		
			0.145	< 1 - OK	Concrete Tension ($P_u/\phi P_n$)
0.897	< 1 - OK	Concrete Shear ($V_u/\phi V_n$)	0.695	< 1 - OK	Concrete Shear ($V_u/\phi V_n$)
			0.695	< 1 - OK	Concrete Combined (Section D.7)
			0.124	< 1 - OK	Steel Tension ($P_u/\phi P_{ss}$)
0.409	< 1 - OK	Steel Shear ($V_u/\phi V_{ss}$)	0.409	< 1 - OK	Steel Shear ($V_u/\phi V_{ss}$)
			0.409	< 1 - OK	Steel Combined (Section D.7)

Anchor Bolts O.K.
Concrete O.K.
Checks Shear Only

Anchor Bolts O.K.
Concrete O.K.
Checks Shear and Tension

FORCES INCLUDE OMEGA PER 17.2.3.4.3d and 17.2.3.5.3c

DESIGN SUMMARY

Anchor Bolts — **(4) 0.75" diameter headed anchor bolts w/ 12"minimum embedment**
Designed for combined tension and shear

Tension Confined Pier — **16" wide x 20" long w/ min (2) #5 vertical bars**
Designed to transfer anchor bolt tension into reinforcement

Shear Confined Pier — **N/A**
Designed to transfer anchor bolt shear into reinforcement

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IBC 2015 Cast-in-place Anchor Bolt Design referencing ACI 318-14 Chapter 17

Version Date: December 5, 2017

JOB TITLE: Kimberly Clark Parts Storage

DESCRIPTION: Grid 2 thru 10 Anchors

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JOB #: 18179

DESIGNER: ZCH

Anchor Reinforcing

Reinforcing Data

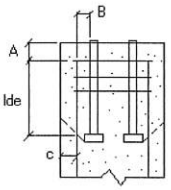
Ψt :	1
Ψe :	1
Ψs :	1
γ :	0.8

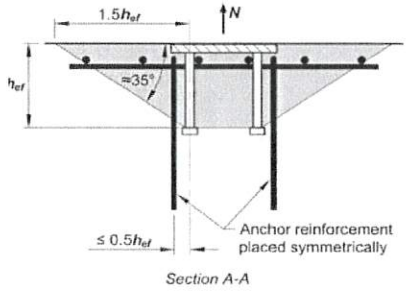
Assumes that vertical reinforcement layout is symmetrical around anchor bolt pattern

NOTE: The calculation for the concrete anchor capacities are based on 1) Tension pullout, 2) Shear pryout. The breakout strength in tension and shear and side face blowout are omitted because the vertical reinforcement is used to confine this failure cone.

Pier Reinforcement to Resist Tension Breakout (17.4.2.9)

Vert. Pier Reinforcing Size :	5	bar
Distance from A.B. to Rebar (B) :	4	in
Cover above vert. reinf (A) :	2	in
c :	3	in
Rebar Area :	0.31	in^2
Rebar Diameter :	0.625	in
l_{db} :	17.44	in
l_{de} :	6.00	in
0.75*F_y of Rebar @ l_{de} - f_s :	15.48	ksi
A_{st} :	0.60	in^2
Total # of vertical bars required :	2	Quantity of reinforcement placed symmetrically around anchor bolts
Embedment of standard hook :	6.00	in





Section A-A

Pier Reinforcement to Resist Shear Breakout (17.5.2.9)

Hairpin/stirrup reinforcing size :	7	bar
Rebar area :	1.2	in^2
A_{st} :	0.36	in^2
Total # of hairpins/stirrups required :	1	Quantity of hairpins or stirrups wrapped around anchor bolts

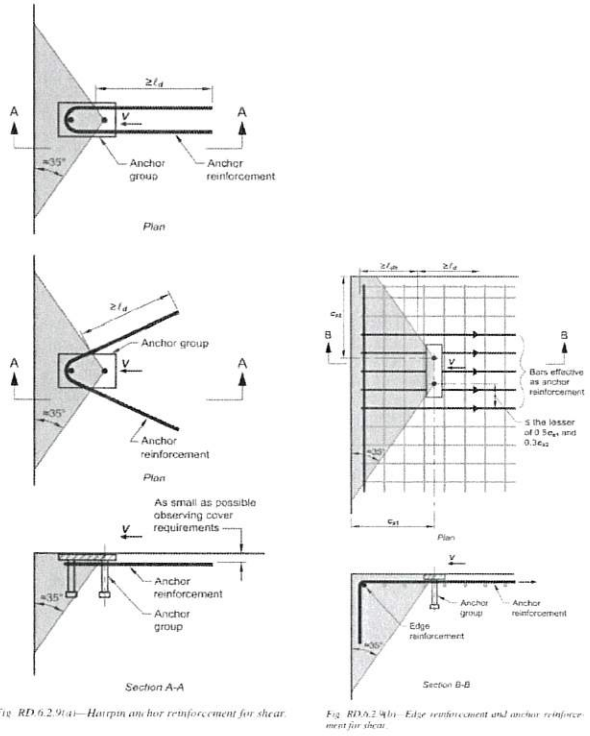


Fig. RD.6.2.9(a)—Hairpin anchor reinforcement for shear.

Fig. RD.6.2.9(b)—Edge reinforcement and anchor reinforcement for shear.

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25-Jun-18
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IBC 2015 Cast-in-place Anchor Bolt Design referencing ACI 318-14 Chapter 17

Version Date: December 5, 2017 Author: Troy M. Dye Reviewed By: Scott Porter

JOB TITLE: Kimberly Clark Parts Storage JOB #: 18179
DESCRIPTION: Grid 2 thru 10 Anchors DESIGNER: ZCH

Pier and Anchor Bolt Geometry

	Min Edge or Spacing		Bolt Quantities
Top Pier Edge	12		
AB-f5	0		
AB-f4	0		
AB-f3	0		
AB-f2	0		
AB-f1	2		2
Column centerline	0	<<	
AB-b1	2		2
AB-b2	0		
AB-b3	0		
AB-b4	0		
AB-b5	0		
Bottom Pier Edge	8		4 # Bolts in Group
Left Pier Edge	6		Pier Dimensions
AB-l1	2		16.0 " wide
Column centerline	0	<<	24.0 " long
AB-r1	2		Bolt Group Dimensions
Right Pier Edge	6		4.0 " wide
			4.0 " long

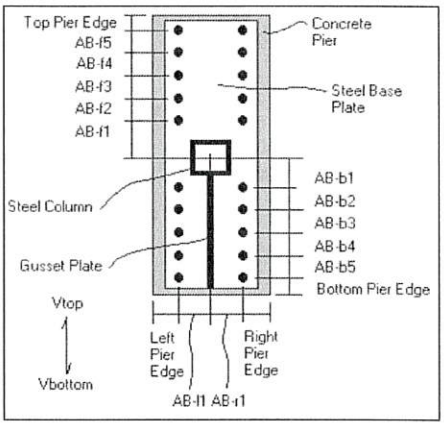


Figure 1. Generic Anchor Bolt Locations

Design Loads

Shear V_u : 55.5 kips *Use ACI 318 Section 5.3 load combinations*

Tension P_u: 22.2 kips

Is the seismic tension component more than 20% of the total factored tension force? YES

Increase Shear Vu and Tension Pu above by Omega

Pier confined for tension force? YES

Pier confined for shear force? YES

Anchors Torqued: NO
Bolt Grade: F1554 Gr. 55
Bolt Diameter: 1 inches
Embedment Length: 16 inches
Concrete f'_c: 4500 psi

Rebar f_y: 60000 psi
Anchor F_u : 75 ksi
Bolt Area A_{bolt} : 0.61 in^2
Seismic Design Category Factor: 0.75 *Section 17.2.3.4.4*

Pier Ht / Ftg Thick: 36 inches **OK**
Bolt type: H

Omega Level Force 17.2.3.4.3d & 17.2.3.5.3c

Bearing Area: 0.98 in^2
Steel ϕ_t: 0.75 *Section 17.3.3*
Steel ϕ_s: 0.65 *Section 17.3.3*
Concrete ϕ: 0.75 *Section 17.3.3*
λ : 1 *Section 19.2.4*
λ_a : 1 *Section 17.2.6*

Eccentricity for Tension
0 in
Eccentricity for Shear
0

Bolt Diameter (in^2)	Threads per (inch)	Tensile Stress Area (in^2)
0.5	13	0.14
0.625	11	0.23
0.75	10	0.33
0.875	9	0.46
1	8	0.61
1.125	7	0.76
1.25	7	0.97
1.375	6	1.15
1.5	6	1.41
1.75	5	1.90
2	4.5	2.50
2.25	4.5	3.25
2.5	4	4.00
2.75	4	4.93
3	4	5.97

Tension Calculations

A_N : 384 in^2
A_{NO} : 576 in^2

	c_{a2}		c_{a1}	
Left Edge Dist. :	6	inches	Top Edge Dist. :	12 inches
Right Edge Dist. :	6	inches	Bottom Edge Dist. :	8 inches

Concrete Tension Capacity (Section 17.4.2) - Breakout Strength

$\psi_{ec,N}$: 1.00 $\psi_{ed,N}$: 0.78 $\psi_{c,N}$: 1

N_b = 34.3 kips N_{cb} = 17.7 kips

Concrete Tension Capacity (Section 17.4.3) - Pullout Strength

$\psi_{c,P}$: 1 N_{pn} = 35.3 kips

N_p = 35.3 kips Capacity of Group: 141.1 kips

Concrete Tension Capacity (Section 17.4.4) - Side-Face Blowout Strength

Use A_N : 384 in^2

N_{sb} = 63.8 kips N_{sbg} = 70.8 kips

ϕP_c : 39.8 kips P_c : **70.8** kips Ultimate Concrete Strength Based on Pullout Only

Steel Tension Capacity (Section 17.4.1)

P_{ss}' : 45.4 kips / AB P_{ss} : **181.7** kips Ultimate Steel Tension Capacity

ENGINEERS

IBC 2015 Cast-in-place Anchor Bolt Design referencing ACI 318-14 Chapter 17

25-Jun-18
11:52 AM

Version Date: December 5, 2017
JOB TITLE: Kimberly Clark Parts Storage
DESCRIPTION: Grid 2 thru 10 Anchors

JOB #: 18179
DESIGNER: ZCH

Shear Calculations

Bolts Resisting Shear

B — Specify which system resists shear
Bolts resist tension and shear

Steel Shear Capacity (Section 17.5.1)

V_{sa} : 21.81 kips / AB

V_{sa} : **87.2** kips — Ultimate Steel Shear Capacity based on Assumed Breakout Surface

n : 4 Number of anchors resisting shear based on assumed concrete breakout surface

Grout height below base plate, h_2 : 1.5 inches

Concrete Shear Breakout Capacity (Section 17.5.2)

$\psi_{ec,V}$: 1.00 $\psi_{ed,V}$: 1.00 $\psi_{c,V}$: 1.2 $\psi_{h,V}$: 1.00

Top Edge Bolts			Bottom Edge Bolts		
Edge Distance:	12.0	inches	Edge Distance :	8.0	inches
A_{Vc} =	288.0	in^2	A_{Vc} =	192.0	in^2
A_{Vco} =	648.0	in^2	A_{Vco} =	288.0	in^2
V_b =	29.6	kips	V_b =	16.1	kips
V_{cb} =	15.8	kips	V_{cb} =	12.9	kips

Concrete Shear Pryout Capacity (Section 17.5.3)

V_{cp} =	112.7	kips	N_b =	109.0	kips
Group Capacity V_n:	**112.7**	kips	Group Capacity V_n:	**112.7**	kips

Ultimate Concrete Capacity Based on Pryout Strength Only

Combined Tension and Shear

Shear Force Acting Towards Top Pier Edge, V_{top}			Shear Force Acting Towards Bottom Pier Edge, V_{bottom}		
			0.280	< 1 - OK	Concrete Tension ($P_u/\phi P_n$)
0.876	< 1 - OK	Concrete Shear ($V_u/\phi V_n$)	0.876	< 1 - OK	Concrete Shear ($V_u/\phi V_n$)
			1.155	<1.2 - O.K.	Concrete Combined (Section D.7)
			0.163	< 1 - OK	Steel Tension ($P_u/\phi P_{ss}$)
0.979	< 1 - OK	Steel Shear ($V_u/\phi V_{ss}$)	0.979	< 1 - OK	Steel Shear ($V_u/\phi V_{ss}$)
			0.979	< 1 - OK	Steel Combined (Section D.7)

Anchor Bolts O.K.
Concrete O.K. Checks Shear Only

Anchor Bolts O.K.
Concrete O.K. Checks Shear and Tension

FORCES INCLUDE OMEGA PER 17.2.3.4.3d and 17.2.3.5.3c

DESIGN SUMMARY

Anchor Bolts — **(4) 1" diameter headed anchor bolts w/ 16"minimum embedment**
Designed for combined tension and shear

Tension Confined Pier — **16" wide x 24" long w/ min (3) #5 vertical bars**
Designed to transfer anchor bolt tension into reinforcement

Shear Confined Pier — **16" wide x 24" long w/ min (2) #7 hairpin**
Designed to transfer anchor bolt shear into reinforcement

ENGINEERS

IBC 2015 Cast-in-place Anchor Bolt Design referencing ACI 318-14 Chapter 17

Version Date: December 5, 2017

JOB TITLE: Kimberly Clark Parts Storage

DESCRIPTION: Grid 2 thru 10 Anchors

25-Jun-18

11:52 AM

JOB #: 18179

DESIGNER: ZCH

Anchor Reinforcing

Reinforcing Data

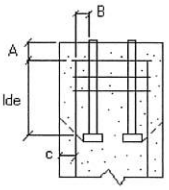
Ψt :	1
Ψe :	1
Ψs :	1
γ :	0.8

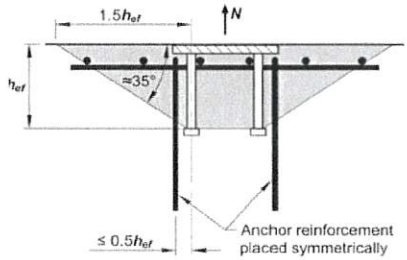
Assumes that vertical reinforcement layout is symmetrical around anchor bolt pattern

NOTE: The calculation for the concrete anchor capacities are based on 1) Tension pullout, 2) Shear pryout. The breakout strength in tension and shear and side face blowout are omitted because the vertical reinforcement is used to confine this failure cone.

Pier Reinforcement to Resist Tension Breakout (17.4.2.9)

Vert. Pier Reinforcing Size :	5	bar
Distance from A.B. to Rebar (B) :	4	in
Cover above vert. reinf (A) :	2	in
c :	3	in
Rebar Area :	0.31	in^2
Rebar Diameter :	0.625	in
l_{db} :	17.44	in
l_{de} :	10.00	in
0.75*F_y of Rebar @ l_{de} - f_s :	25.80	ksi
A_{st} :	0.86	in^2
Total # of vertical bars required :	3	Quantity of reinforcement placed symmetrically around anchor bolts
Embedment of standard hook :	6.00	in





Section A-A

Pier Reinforcement to Resist Shear Breakout (17.5.2.9)

Hairpin/stirrup reinforcing size :	7	bar
Rebar area :	1.2	in^2
A_{st} :	1.23	in^2
Total # of hairpins/stirrups required :	2	Quantity of hairpins or stirrups wrapped around anchor bolts

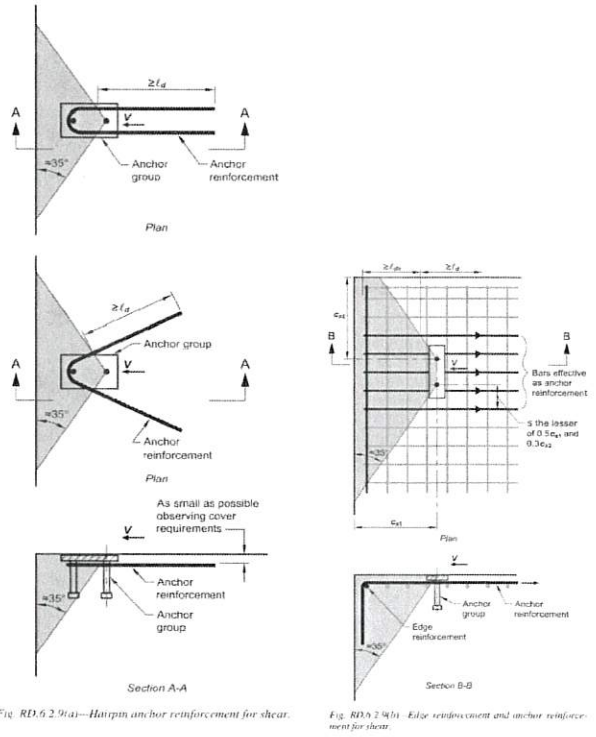


Fig. RD.6.2.9(a)—Hairpin anchor reinforcement for shear.

Fig. RD.6.2.9(b)—Edge reinforcement and anchor reinforcement for shear.

ENGINEERS

IBC 2015 FACTORED REACTIONS FOR PRE-MANUFACTURED METAL BUILDINGS

Version Date: July 19, 2016

JOB TITLE:

JOB #:

Monday, June 25, 2018

	Grid 1 & 11															Grid 2 -10								
	Grid A			Grid B			Grid C			Grid D			Grid E			Grid A & E								
	X	Y	Z	X	Y	Z	X	Y	Z	X	Y	Z	X	Y	Z	X	Y	Z	X	Y	Z	X	Y	Z
Dead		1.0			3.0			3.0			3.0			1.0		5.0	8.0							
Collateral, C		1.0			2.0			2.0			2.0			1.0		4.0	6.0							
Floor Live																								
Roof Live		1.0			5.0			4.0			5.0			1.0		9.0	14.0							
Crane, C,																								
Snow		3.0			13.0			10.0			13.0			3.0		22.0	34.0							
Wind																								
Wind (Uplift)	2.0	3.0		3.0	10.0		5.0	8.0		3.0	12.0		2.0	3.0		14.0	20.0							
Seismic		1.0			1.0		8.0	8.0		8.0	7.0			1.0		20.0	14.0							
Seismic (Uplift)																20.0	14.0							

Seismic Design Factors																								
Seismic Coefficient Sds	0.955	0.955	0.955	0.955	0.955	0.955	0.955	0.955	0.955	0.955	0.955	0.955	0.955	0.955	0.955	0.955	0.955							
Amplification Factor - Ωo	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2							
Redundancy Factor - ρ	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1							

ASD Combinations	ASD Factored Reactions																							
D		1.0			3.0			3.0			3.0			1.0		5.0	8.0							
D+L+C_r		1.0			3.0			3.0			3.0			1.0		5.0	8.0							
D+C+L_r		3.0			10.0			9.0			10.0			3.0		18.0	28.0							
D+C+S		5.0			18.0			15.0			18.0			5.0		31.0	48.0							
D+C+0.75*(L+C_r)+0.75*L_r		2.8			8.8			8.0			8.8			2.8		15.8	24.5							
D+C+0.75*(L+C_r)+0.75*S		4.3			14.8			12.5			14.8			4.3		25.5	39.5							
D+0.6*W		1.0			3.0			3.0			3.0			1.0		5.0	8.0							
D-0.6*W	-1.2	-0.8		-1.8	-3.0		-3.0	-1.8		-1.8	-4.2		-1.2	-0.8		-3.4	-4.0							
(D+0.14*Sds*D)+0.7*(E*ρ)		1.8			4.1		5.6	9.0		5.6	8.3			1.8		19.7	18.9							
(D-0.14*Sds*D)-0.7*(E*ρ)		0.9			2.6			2.6			2.6			0.9		-9.7	-2.9							
D+C+0.75*0.6*W+0.75*(L+C_r)+0.75*L_r		2.8			8.8			8.0			8.8			2.8		15.8	24.5							
D+C-0.75*0.6*W+0.75*(L+C_r)+0.75*L_r	-0.9	1.4		-1.4	4.3		-2.3	4.4		-1.4	3.4		-0.9	1.4		9.5	15.5							
D+C+0.75*0.6*W+0.75*(L+C_r)+0.75*S		4.3			14.8			12.5			14.8			4.3		25.5	39.5							
D+C-0.75*0.6*W+0.75*(L+C_r)+0.75*S	-0.9	2.9		-1.4	10.3		-2.3	8.9		-1.4	9.4		-0.9	2.9		19.2	30.5							
(D+0.105*Sds*D)+0.525*(E*ρ)+C+0.75*(L+C_r)+0.75*L_r		3.4			9.6		4.2	12.5		4.2	12.7			3.4		26.8	32.7							
(D-0.105*Sds*D)-0.525*(E*ρ)+C+0.75*(L+C_r)+0.75*L_r		2.6			8.4			7.7			8.4			2.6		4.7	16.3							
(D+0.105*Sds*D)+C+0.525*(E*ρ)+0.75*(L+C_r)+0.75*S		4.9			15.6		4.2	17.0		4.2	18.7			4.9		36.5	47.7							
(D-0.105*Sds*D)+C-0.525*(E*ρ)+0.75*(L+C_r)+0.75*S		4.1			14.4			12.2			14.4			4.1		14.5	31.3							
0.6D+0.6*W		0.6			1.8			1.8			1.8			0.6		3.0	4.8							
0.6D-0.6*W	-1.2	-1.2		-1.8	-4.2		-3.0	-3.0		-1.8	-5.4		-1.2	-1.2		-5.4	-7.2							
0.6*(D+0.14*Sds*D)+0.7*(E*ρ)		1.4			2.7		5.6	7.6		5.6	6.9			1.4		17.4	15.2							
0.6*(D-0.14*Sds*D)-0.7*(E*ρ)		0.5			1.6			1.6			1.6			0.5		-11.4	-5.6							

LRFD Combinations	LRFD Factored Reactions																							
1.4D		1.4			4.2			4.2			4.2			1.4		7.0	11.2							
1.2D+1.6(L+C_r)+0.5S		3.7			12.1			10.6			12.1			3.7		21.0	32.6							
1.2D+1.6(L+C_r)+0.5L_r+C		2.7			8.1			7.6			8.1			2.7		14.5	22.6							
1.2D+1.6*(L+C)+L+C_r		4.4			14.8			13.2			14.8			4.4		26.8	41.6							
1.2D+1.6(L_r+C)+0.5W		4.4			14.8			13.2			14.8			4.4		26.8	41.6							
1.2D+1.6(L_r+C)-0.5W	-1.0	2.9		-1.5	9.8		-2.5	9.2		-1.5	8.8		-1.0	2.9		19.8	31.6							
1.2D+1.6(C+S)+(L+C_r)		7.6			27.6			22.8			27.6			7.6		47.6	73.6							
1.2D+1.6(C+S)+0.5W		7.6			27.6			22.8			27.6			7.6		47.6	73.6							
1.2D+1.6(C+S)-0.5W	-1.0	6.1		-1.5	22.6		-2.5	18.8		-1.5	21.6		-1.0	6.1		40.6	63.6							
1.2D+1.0W+(L+C_r)+0.5L_r+C		2.7			8.1			7.6			8.1			2.7		14.5	22.6							
1.2D-1.0W+(L+C_r)+0.5L_r+C	-2.0	-0.3		-3.0	-1.9		-5.0	-0.4		-3.0	-3.9		-2.0	-0.3		0.5	2.6							
1.2D+1.0W+(L+C_r)+0.5S+C		3.7			12.1			10.6			12.1			3.7		21.0	32.6							
1.2D-1.0W+(L+C_r)+0.5S+C	-2.0	0.7		-3.0	2.1		-5.0	2.6		-3.0	0.1		-2.0	0.7		7.0	12.6							
1.2*(D+0.2*Sds*D)+(E*ρ)+(L+C_r)+0.2S+C		4.0			9.9		8.0	16.3		8.0	15.9			4.0		35.5	38.2							
1.2*(D-0.2*Sds*D)-(E*ρ)+(L+C_r)+0.2S+C		1.6			5.5			4.9			5.5			1.6		-10.7	0.6							
1.2*(D+0.2*Sds*D)+(E*Ωo)+(L+C_r)+0.2S+C		5.0			10.9		16.0	24.3		16.0	22.9			5.0		55.5	52.2							
1.2*(D-0.2*Sds*D)-(E*Ωo)+(L+C_r)+0.2S+C		1.6			5.5			4.9			5.5			1.6		-30.7	-13.4							
0.9D+1.0W		0.9			2.7			2.7			2.7			0.9		4.5	7.2							
0.9D-1.0W	-2.0	-2.1		-3.0	-7.3		-5.0	-5.3		-3.0	-9.3		-2.0	-2.1		-9.5	-12.8							
0.9*(D+0.2*Sds*D)+(E*ρ)		2.1			4.2		8.0	11.2		8.0	10.2			2.1		25.4	22.6							
0.9*(D-0.2*Sds*D)-(E*ρ)		0.7			2.2			2.2			2.2			0.7		-16.4	-8.2							
0.9*(D+0.2*Sds*D)+(E*Ωo)		3.1			5.2		16.0	19.2		16.0	17.2			3.1		45.4	36.6							
0.9*(D-0.2*Sds*D)-(E*Ωo)		0.7			2.2			2.2			2.2			0.7		-36.4	-22.2							

	Footing Results for Gravity, Uplift, and Shear																							
Bolt Pattern Callout																								
Worst Case Shear (ULT)	2.0		0.0	3.0		0.0	16.0		0.0	16.0		0.0	2.0		0.0	55.5		0.0	0.0		0.0	0.0		0.0
Worst Case Uplift (ULT)		-2.1			-7.3			-5.3			-9.3			-2.1			-22.2			0.0			0.0	
Worst Case Shear (ASD)	1.2		0.0	1.8		0.0	5.6		0.0	5.6		0.0	1.2		0.0	36.5		0.0	0.0		0.0	0.0		0.0
Passive Pressure (pcf)		300			300			300			300			300			300			300			300	
Friction Coefficient		0.3			0.3			0.3			0.3			0.3			0.3			0.3			0.3	
Sliding Footing Thickness (in)		12.0			12.0			12.0			12.0			12.0			12.0			12.0			12.0	
Soil Depth above Footing (in)		24			24			24			24			24			24			24			24	
Sliding Footing Width (ft)		4.3			3			3			3			3			3			3			3	
Friction Resistance (Kip)		0.6			0.9			0.9			0.9			0.6			1.8			#######			#######	
Passive Pressure Resistance (Kip)			4.1			4.1			4.1			4.1			4.1			4.1			4.1			4.1
Total Lateral Resistance (Kip)			4.6			5.0			5.0			5.0			4.6			5.9			######			######
	Consider Hairpins						Consider Hairpins						Consider Hairpins						Consider Hairpins			Consider Hairpins		
Worst Case Uplift Force (ASD Kip)		-1.2			-4.2			-3.0			-5.4			-1.2			-7.2			0.0			0.0	
Footing Size (Up) (ft)		1.8			3.4			2.8			3.8			1.8			4.4			0.0			0.0	
Worst Case Down Force (ASD Kip)		5.0			18.0			17.0			18.7			5.0			48.0			0.0			0.0	
Allowable Bearing Pressure		2500			2500			2500			2500			2500			2500			2500			2500	
Footing Size (Down)		1.5			2.8			2.7			2.8			1.5			4.5			0.0			0.0	
Footing Used		**F3**			**F3**			**F3**			**F3**			**F3**			**F4.5**							

NUCOR BUILDING SYSTEMS GROUP
1050 North Watery Lane Ph: (435) 919-3100
Brigham City, UT 84302 Fax: (435) 919-3101

Page: R1 of 6
Date: 6/14/2018

GENERAL INFORMATION FOR COLUMN BASE REACTIONS

○ FOR REVIEW ◉ FOR CONSTRUCTION

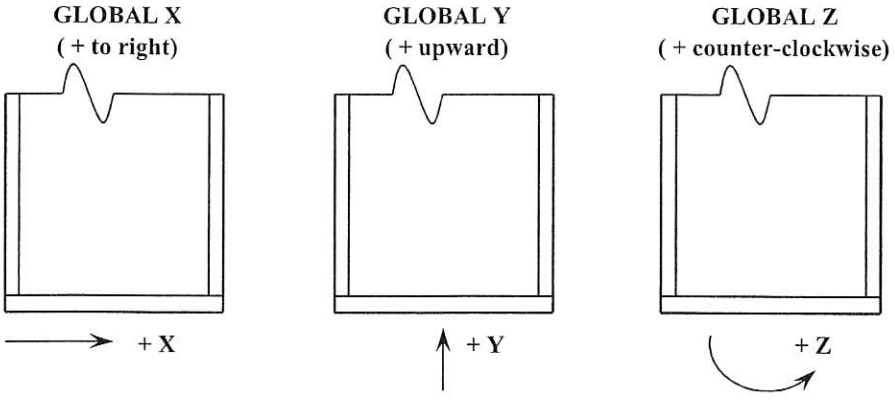
Project Name: Kimberly Clark - Opt C
NBS Project Number: U18G0576A
Customer: Commercial Service Unlimited
NBG Engineer: Louis Lo

Column base reactions are included in this packet for a building designed by Nucor Buildings Group. These reactions result from frame analysis done by a NBG Engineer for this specific job. They reflect all loading to which the building may be subject, per the appropriate building code and loading information provided to Nucor Buildings Group at the date of design. Reaction packets marked "FOR REVIEW" are subject to change and are usually provided at the request of the customer, although the NBG Engineer believes he/she is working with undefined, incomplete or assumed information.

Reactions are provided by load case in order to aid the foundation engineer in determining the appropriate load factors and combinations to be used with either Working Stress or Ultimate Strength design methods. Wind load cases are given for each primary wind direction.

For ASCE7-10 based building codes, the unfactored load case reactions due to wind are generated using the ultimate design wind speed (Vult).

Sign conventions for computer generated frame reactions are as follows and should be taken in the sense of the frame sketch given on the reactions sheets.



Anchor bolt diameter, grade, location and projection is provided on the Anchor Bolt Plan. Anchor bolt embedment lengths and types are not provided by Nucor Buildings Group. This Information is closely related to the complete foundation design which should be done by a Registered Professional Engineer familiar with the local site conditions and construction practices.

U18G0576A LL R2 of 6

Version 2018.4.3 (4/8/18)

NUCOR BUILDING SYSTEMS GROUP

1050 North Watery Lane Ph: (435) 919-3100
Brigham City, UT 84302 Fax: (435) 919-3101

ERECTION DRAWING COVER SHEET LOADS

Information to be verified on Erection Drawing Cover Sheet:

Project Name: Kimberly Clark - Opt C
NBS Project Number: U18G0576A
Customer: Commercial Service Unlimited
NBG Engineer: Louis Lo
Date: 6/14/2018

BUILDING LOADS

DESIGN CODE: **IBC 2015**
ROOF LIVE LOAD: **20.00** PSF
REDUCIBLE AS PER CODE.
GROUND SNOW LOAD: **43.00** PSF
SNOW IMPORTANCE FACTOR, I_s: **1.00**

RISK CATEGORY: **II - Standard Buildings**
SNOW EXP. FACTOR, Ce: **1.00**

WIND: **115 mph (Vult)** **89 mph (Vasd)**
WIND EXPOSURE: B

***C & C PRESSURES 22 psf / -29 psf
UL 90? **No**

SEISMIC INFORMATION: **Ss: 1.433 S1: 0.596**
Design Sds / Sd1: **0.955/0.596**
Seismic Imp. Factor, Ie: **1.00**

Site Class: **D**
Seismic Design Category: **D**

Analysis Procedure: Equiv. Lat. Force Procedure
Basic SFRS: **Ord. Steel Mom. Frames & Ord. Steel Conc.-Br. Frames**

NOTES:

1) COLLATERAL DEAD LOADS, UNLESS OTHERWISE NOTED, ARE ASSUMED TO BE UNIFORMLY DISTRIBUTED. WHEN SUSPENDED SPRINKLER SYSTEMS, LIGHTING, HVAC EQUIPMENT, CEILINGS, ETC., ARE SUSPENDED FROM ROOF MEMBERS, CONSULT THE M.B.S. IF THESE CONCENTRATED LOADS EXCEED 500 POUNDS (USING THE WEB MOUNT DETAIL), OR 200 POUNDS (USING THE FLANGE MOUNT DETAIL), OR IF INDIVIDUAL MEMBERS ARE LOADED SIGNIFICANTLY MORE THAN OTHERS.

2) THE DESIGN OF STRUCTURAL MEMBERS SUPPORTING GRAVITY LOADS IS CONTROLLED BY THE MORE CRITICAL EFFECT OF ROOF LIVE LOAD OR ROOF SNOW LOAD, AS DETERMINED BY THE APPLICABLE CODE.

3) **PM IS BASED ON THE MINIMUM ROOF SNOW LOAD CALCULATED PER BUILDING CODE OR THE CONTRACT-SPECIFIED ROOF SNOW LOAD, WHICHEVER IS GREATER. THIS VALUE, PM, IS ONLY APPLIED IN COMBINATION WITH DEAD AND COLLATERAL LOADS. ROOF SNOW IN OTHER LOADING CONDITIONS IS DETERMINED PER THE SPECIFIED BUILDING CODE.

BUILDING-SPECIFIC LOADING INFORMATION:

	Roof Dead	Collateral Dead		Snow Coefficient		Snow Load (psf)		Wind		Seismic		
Bldg	(psf)*	Pri (psf)	Sec (psf)	Ct	Cs	Ps (psf)	**Pm (psf)	Enclosure	GCpi	R	Cs	V (kips)
A	4.3	5.0	5.0	1.0	1.00	30.10	20.00	Enclosed	± 0.18	3.25	0.294	112.1

**Primary Structural Not Included*

****Design wind pressures to be used for wall exterior component and cladding materials not provided by Nucor Building Systems.*

NUCOR BUILDING SYSTEMS GROUP
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ERECTION DRAWING COVER SHEET NOTES

Special notes to be placed on Erection Drawing Cover Sheet:

CVNOTE21:

Windows and doors that are provided by others are assumed to meet the wind loading requirements of the structure and the openings for these must be impact-resistant or protected by an impact-resistant covering as specified in the building code when a high wind event is anticipated.

CVNOTE27:

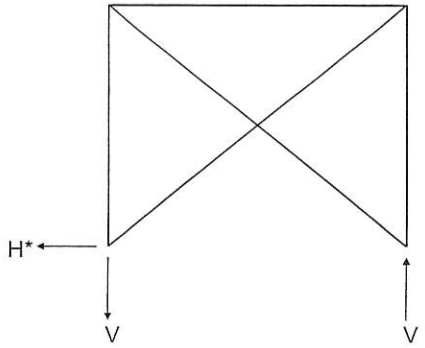
For buildings with an Occupancy Category I or II, IBC allows for single story buildings to have no limit for the seismic story drift. Please note that any interior walls, partitions, ceilings, and exterior walls should be detailed (by others) to accommodate this story drift.

NUCOR BUILDINGS GROUP

JOB NAME: KIMBLEY CLARK - OPT C JOB NUMBER: U18G0576A ENGINEER: LL

LONGITUDINAL X-BRACING REACTIONS

(These reactions must be combined with the appropriate longitudinal frame reactions)



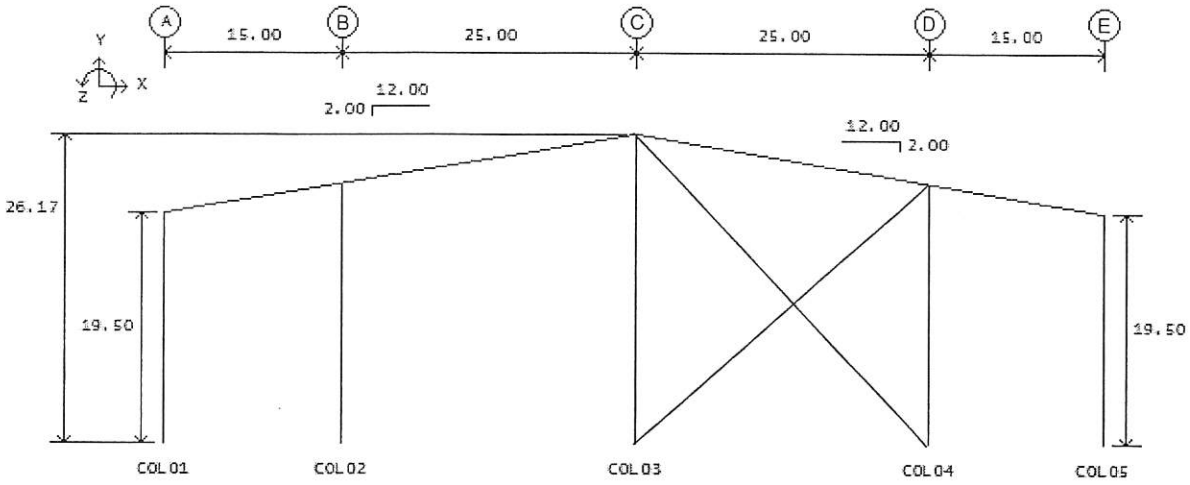
Line:	Bay:	H (KIPS)	V (KIPS)	CASE
A & E	2-3, 4-5, 7-8	3	2	WIND
A & E	2-3, 4-5, 7-8	20	14	SEISMIC

* - Horizontal bracing reactions are orthogonal to horizontal frame reactions.

U18G0576A LL R4 of 6

NUCOR BUILDINGS GROUP Job #: U18G0576A Page: _____
Frame : FRAME LINE 1 & 11 By: BG\louis.lo Date: 06-14-18
Job Name: KIMBERLY CLARK - OPT C File: E01

*** DESIGN SUMMARY - FRAME REACTIONS BY LOAD CASE ***

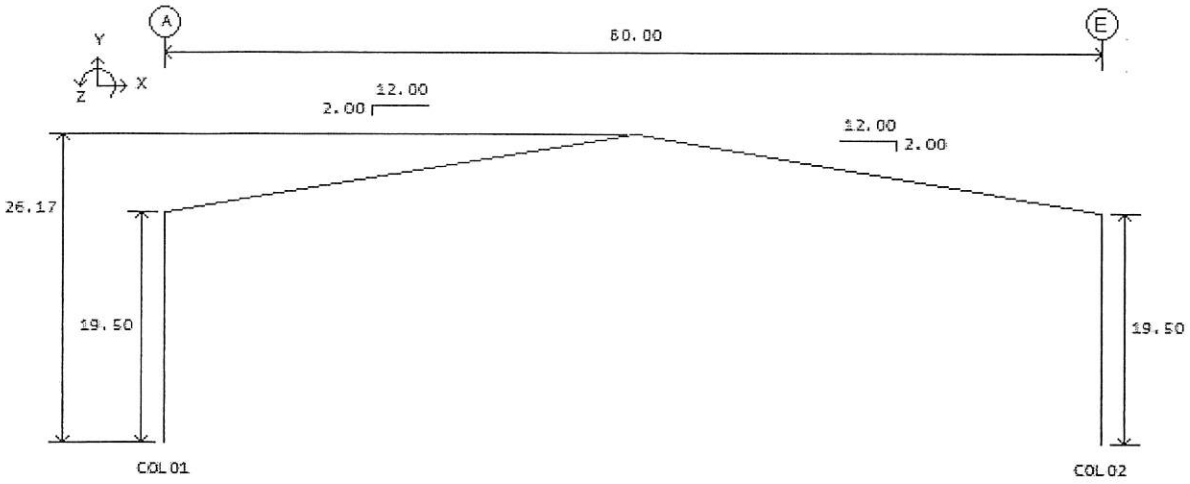


Member	X (kips)	Y (kips)	Z (kip-ft)	Member	X (kips)	Y (kips)	Z (kip-ft)
LOAD CASE 1 - DEAD				LOAD CASE 8 - WIND CASE 2 TO LEFT			
COL01	0	1	0	COL01	0	-2	0
COL02	0	3	0	COL02	0	-6	0
COL03	0	3	0	COL03	0	-4	0
COL04	0	3	0	COL04	3	-12	0
COL05	0	1	0	COL05	0	-2	0
LOAD CASE 2 - COLLATERAL				LOAD CASE 9 - LONG. WIND 1 TO BACK			
COL01	0	1	0	COL01	-1	-3	0
COL02	0	2	0	COL02	-3	-10	0
COL03	0	2	0	COL03	-4	-8	0
COL04	0	2	0	COL04	-3	-10	0
COL05	0	1	0	COL05	-1	-3	0
LOAD CASE 3 - ROOF LIVE				LOAD CASE 10 - LONG. WIND 1 TO FRONT			
COL01	0	1	0	COL01	2	-3	0
COL02	0	5	0	COL02	3	-10	0
COL03	0	4	0	COL03	5	-8	0
COL04	0	5	0	COL04	3	-10	0
COL05	0	1	0	COL05	2	-3	0
LOAD CASE 4 - SNOW				LOAD CASE 11 - SEISMIC TO RIGHT			
COL01	0	3	0	COL01	0	1	0
COL02	0	11	0	COL02	0	1	0
COL03	0	10	0	COL03	-8	-8	0
COL04	0	11	0	COL04	0	7	0
COL05	0	3	0	COL05	0	-1	0
LOAD CASE 5 - WIND CASE 1 TO RIGHT				LOAD CASE 12 - SEISMIC TO LEFT			
COL01	0	-1	0	COL01	0	-1	0
COL02	0	-7	0	COL02	0	-1	0
COL03	-3	-7	0	COL03	0	8	0
COL04	0	-1	0	COL04	8	-7	0
COL05	0	-1	0	COL05	0	1	0
LOAD CASE 6 - WIND CASE 1 TO LEFT				LOAD CASE 13 - ALTERNATE SNOW 1			
COL01	0	-1	0	COL01	0	2	0
COL02	0	-3	0	COL02	0	13	0
COL03	0	-2	0	COL03	0	9	0
COL04	3	-9	0	COL04	0	4	0
COL05	0	-1	0	COL05	0	1	0
LOAD CASE 7 - WIND CASE 2 TO RIGHT				LOAD CASE 14 - ALTERNATE SNOW 2			
COL01	0	-2	0	COL01	0	1	0
COL02	0	-9	0	COL02	0	4	0
COL03	-3	-9	0	COL03	0	9	0
COL04	0	-3	0	COL04	0	13	0
COL05	0	-2	0	COL05	0	2	0

U18G0576A LL R5 of 6

NUCOR BUILDINGS GROUP Job #: U18G0576A Page: _____
Frame : FRAME LINE 2 - 10 By: BG\louis.lo Date: 06-14-18
Job Name: KIMBERLY CLARK - OPT C File: F01

*** DESIGN SUMMARY - FRAME REACTIONS BY LOAD CASE ***



Member	X (kips)	Y (kips)	Z (kip-ft)	Member	X (kips)	Y (kips)	Z (kip-ft)
LOAD CASE 1 - DEAD				LOAD CASE 9 - LONG. WIND 1 TO BACK			
COL01	5	8	0	COL01	-4	-10	0
COL02	-5	8	0	COL02	5	-7	0
LOAD CASE 2 - COLLATERAL				LOAD CASE 10 - LONG. WIND 1 TO FRONT			
COL01	4	6	0	COL01	-5	-7	0
COL02	-4	6	0	COL02	4	-10	0
LOAD CASE 3 - ROOF LIVE				LOAD CASE 11 - LONG. WIND 2 TO BACK			
COL01	9	14	0	COL01	-8	-18	0
COL02	-9	14	0	COL02	9	-15	0
LOAD CASE 4 - SNOW				LOAD CASE 12 - LONG. WIND 2 TO FRONT			
COL01	22	34	0	COL01	-9	-15	0
COL02	-22	34	0	COL02	8	-18	0
LOAD CASE 5 - WIND CASE 1 TO RIGHT				LOAD CASE 13 - SEISMIC TO RIGHT			
COL01	-11	-12	0	COL01	-6	-4	0
COL02	2	-6	0	COL02	-6	4	0
LOAD CASE 6 - WIND CASE 1 TO LEFT				LOAD CASE 14 - SEISMIC TO LEFT			
COL01	-2	-6	0	COL01	6	4	0
COL02	11	-12	0	COL02	6	-4	0
LOAD CASE 7 - WIND CASE 2 TO RIGHT				LOAD CASE 15 - ALTERNATE SNOW 1			
COL01	-14	-20	0	COL01	19	33	0
COL02	6	-14	0	COL02	-18	19	0
LOAD CASE 8 - WIND CASE 2 TO LEFT				LOAD CASE 16 - ALTERNATE SNOW 2			
COL01	-6	-14	0	COL01	18	19	0
COL02	14	-20	0	COL02	-19	33	0

This Wall in File: y:\projects 2018\18179 - kimberly clark parts storage building\engineering\calcul

Cantilevered Retaining Wall

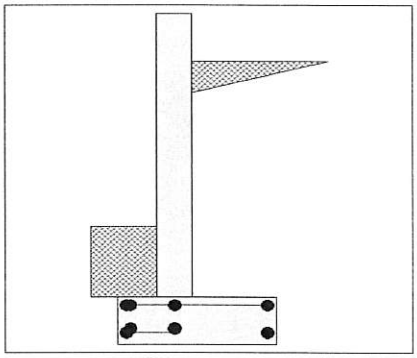
Code: IBC 2015,ACI 318-14,ACI 530-13

Criteria

Retained Height	=	5.00 ft
Wall height above soil	=	1.00 ft
Slope Behind Wall	=	0.00
Height of Soil over Toe	=	18.00 in
Water height over heel	=	0.0 ft

Soil Data

Allow Soil Bearing	=	2,500.0 psf
Equivalent Fluid Pressure Method		
Active Heel Pressure	=	45.0 psf/ft
	=	
Passive Pressure	=	300.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe	=	0.00 pcf
Footing||Soil Friction	=	0.300
Soil height to ignore for passive pressure	=	12.00 in



Surcharge Loads

Surcharge Over Heel	=	100.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	100.0
Used for Sliding & Overturning		

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
Load Type	=	Wind (W) (Service Level)
Wind on Exposed Stem (Service Level)	=	0.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Design Summary

Wall Stability Ratios

Overturning	=	1.66 OK
Sliding	=	2.25 OK
Total Bearing Load	=	2,154 lbs
...resultant ecc.	=	9.96 in
Soil Pressure @ Toe	=	2,143 psf OK
Soil Pressure @ Heel	=	0 psf OK
Allowable	=	2,500 psf
Soil Pressure Less Than Allowable		
ACI Factored @ Toe	=	3,001 psf
ACI Factored @ Heel	=	0 psf
Footing Shear @ Toe	=	0.8 psi OK
Footing Shear @ Heel	=	11.0 psi OK
Allowable	=	75.0 psi

Sliding Calcs

Lateral Sliding Force	=	997.5 lbs
less 100% Passive Force	= -	1,593.3 lbs
less 100% Friction Force	= -	646.3 lbs
Added Force Req'd	=	0.0 lbs OK
....for 1.5 Stability	=	0.0 lbs OK

Vertical component of active lateral soil pressure IS NOT considered in the calculation of soil bearing

Load Factors

Building Code	IBC 2015,ACI
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.000
Seismic, E	1.000

Stem Construction

	Bottom
	Stem OK
Design Height Above Ftg ft =	0.00
Wall Material Above "Ht" =	Concrete
Design Method =	LRFD
Thickness =	8.00
Rebar Size =	# 4
Rebar Spacing =	12.00
Rebar Placed at =	Center
Design Data	
fb/FB + fa/Fa =	**0.684**
Total Force @ Section	
Service Level lbs =	
Strength Level lbs =	1,227.3
Moment....Actual	
Service Level ft-# =	
Strength Level ft-# =	2,318.2
Moment.....Allowable =	3,387.6
Shear.....Actual	
Service Level psi =	
Strength Level psi =	25.6
Shear.....Allowable psi =	75.0
Anet (Masonry) in2 =	
Rebar Depth 'd' in =	4.00
Masonry Data	
f'm psi =	
Fs psi =	
Solid Grouting =	
Modular Ratio 'n' =	
Wall Weight psf =	100.0
Short Term Factor =	
Equiv. Solid Thick. =	
Masonry Block Type =	Medium Weight
Masonry Design Method =	ASD
Concrete Data	
f'c psi =	2,500.0
Fy psi =	60,000.0

Cantilevered Retaining Wall

Code: IBC 2015,ACI 318-14,ACI 530-13

Concrete Stem Rebar Area Details

Bottom Stem	Vertical Reinforcing	Horizontal Reinforcing
As (based on applied moment) :	0.1399 in2/ft	
(4/3) * As :	0.1866 in2/ft	Min Stem T&S Reinf Area 1.152 in2
200bd/fy : 200(12)(4)/60000 :	0.16 in2/ft	Min Stem T&S Reinf Area per ft of stem Height : 0.192 in2/ft
0.0018bh : 0.0018(12)(8) :	0.1728 in2/ft	Horizontal Reinforcing Options :
	==========	One layer of : Two layers of :
Required Area :	0.16 in2/ft	#4@ 12.50 in #4@ 25.00 in
Provided Area :	0.2 in2/ft	#5@ 19.38 in #5@ 38.75 in
Maximum Area :	0.5419 in2/ft	#6@ 27.50 in #6@ 55.00 in

Footing Dimensions & Strengths

Toe Width	=	0.75 ft
Heel Width	=	2.25
Total Footing Width	=	3.00
Footing Thickness	=	12.00 in
Key Width	=	12.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	2.00 ft

f'c = 2,500 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top 2.00 @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel
Factored Pressure	=	3,001	0 psf
Mu' : Upward	=	739	52 ft-#
Mu' : Downward	=	151	1,253 ft-#
Mu: Design	=	588	1,201 ft-#
Actual 1-Way Shear	=	0.82	11.00 psi
Allow 1-Way Shear	=	40.00	40.00 psi

Toe Reinforcing = # 4 @ 16.00 in
Heel Reinforcing = # 4 @ 16.00 in
Key Reinforcing = None Spec'd

Other Acceptable Sizes & Spacings

Toe: Not req'd: Mu < phi*5*lambda*sqrt(f'c)*Sm
Heel: Not req'd: Mu < phi*5*lambda*sqrt(f'c)*Sm
Key: Not req'd: Mu < phi*5*lambda*sqrt(f'c)*Sm

Min footing T&S reinf Area 0.78 in2
Min footing T&S reinf Area per foot 0.26 in2 /ft

If one layer of horizontal bars:	If two layers of horizontal bars:
#4@ 9.26 in	#4@ 18.52 in
#5@ 14.35 in	#5@ 28.70 in
#6@ 20.37 in	#6@ 40.74 in

Summary of Overturning & Resisting Forces & Moments

	.....OVERTURNING..... Force lbs	Distance ft	Moment ft-#		.....RESISTING..... Force lbs	Distance ft	Moment ft-#
Item							
Heel Active Pressure =	810.0	2.00	1,620.0	Soil Over Heel =	870.8	2.21	1,923.1
Surcharge over Heel =	187.5	3.00	562.5	Sloped Soil Over Heel =			
Surcharge Over Toe =				Surcharge Over Heel =	158.3	2.21	349.7
Adjacent Footing Load =				Adjacent Footing Load =			
Added Lateral Load =				Axial Dead Load on Stem =			
Load @ Stem Above Soil =				* Axial Live Load on Stem =			
=				Soil Over Toe =		0.38	
				Surcharge Over Toe =	75.0	0.38	28.1
Total	997.5	**O.T.M.**	2,182.5	Stem Weight(s) =	600.0	1.08	650.0
=		=		Earth @ Stem Transitions =			
				Footing Weight =	450.0	1.50	675.0
				Key Weight =		2.50	
				Vert. Component =			
				Total =	2,154.2 lbs	**R.M.=**	3,625.9

Resisting/Overturning Ratio = 1.66
Vertical Loads used for Soil Pressure = 2,154.2 lbs

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

Vertical component of active lateral soil pressure IS NOT considered in the calculation of Sliding Resistance.

Vertical component of active lateral soil pressure IS NOT considered in the calculation of Overturning Resistance.

Tilt

Horizontal Deflection at Top of Wall due to settlement of soil

(Deflection due to wall bending not considered)

Soil Spring Reaction Modulus	250.0	pci
Horizontal Defl @ Top of Wall (approximate only)	0.119	in

The above calculation is not valid if the heel soil bearing pressure exceeds that of the toe, because the wall would then tend to rotate into the retained soil.