STRUCTURAL CALCULATIONS

FOR

MCDONALD'S

LILLINGTON, NC

102 WEST CORNELIUS HARNETT BOULEVARD.

032-0312

i'm lovin' it™

AECOM

ARCHITECTS-ENGINEERS-PLANNERS
CLIFTON, NEW JERSEY

NORTH CAROLINA PROFESSIONAL SEAL 052352 ENGINEER AHMED MOSTAFA

09/06/2024

AUGUST 2024

Structural Project Book Tabs

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1. Basis of Design

- Codes
  2018 NORTH CAROLINA BUILDING CODE W/IBC 2015 AMENDMENTS.

- Building Type
  Building Risk Category: II

  Wood bearing wall system, with wood roof joists, concrete slab on grade floor and shallow concrete foundations.

- Lateral Load Resisting System

  Bearing wall system (light frame wood walls sheathed with shear panels) and steel system not specifically detailed for seismic resistance.

- Geotechnical Recommendations
  Allowable Bearing Pressure
  3000 psf
  Site class: D

  For more information refer Geotechnical report provided by ECS Southeast, LLP, report #09:30120 dated on 12/06/2023.

2. Loads

	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	Loads				2-1	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

SNOW LOADING (LONG DIRECTION)

Snow loading

In accordance with ASCE7-10

Tedds calculation version 1.0.11

Building details

Roof type;	Flat
Width of roof;	b = **85.67** ft

Ground snow load

Ground snow load (Figure 7-1);	p_g = **15.00** lb/ft^2
Density of snow;	$g = \min(0.13 \cdot p_g / 1\text{ft} + 14\text{lb/ft}^3, 30\text{lb/ft}^3)$ = **15.95** lb/ft^3
Terrain typeSect. 26.7;	C
Exposure condition (Table 7-2);	Partially exposed
Exposure factor (Table 7-2);	C_e = **1.00**
Thermal condition (Table 7-3);	All
Thermal factor (Table 7-3);	C_t = **1.00**
Importance category (Table 1.5-1);	II
Importance factor (Table 1.5-2);	I_s = **1.00**
Min snow load for low slope roofs (Sect 7.3.4);	$p_{f\_min} = I_s \cdot p_g$ = **15.00** lb/ft^2
Flat roof snow load (Sect 7.3);	$p_f = 0.7 \cdot C_e \cdot C_t \cdot I_s \cdot p_g$ = **10.50** lb/ft^2

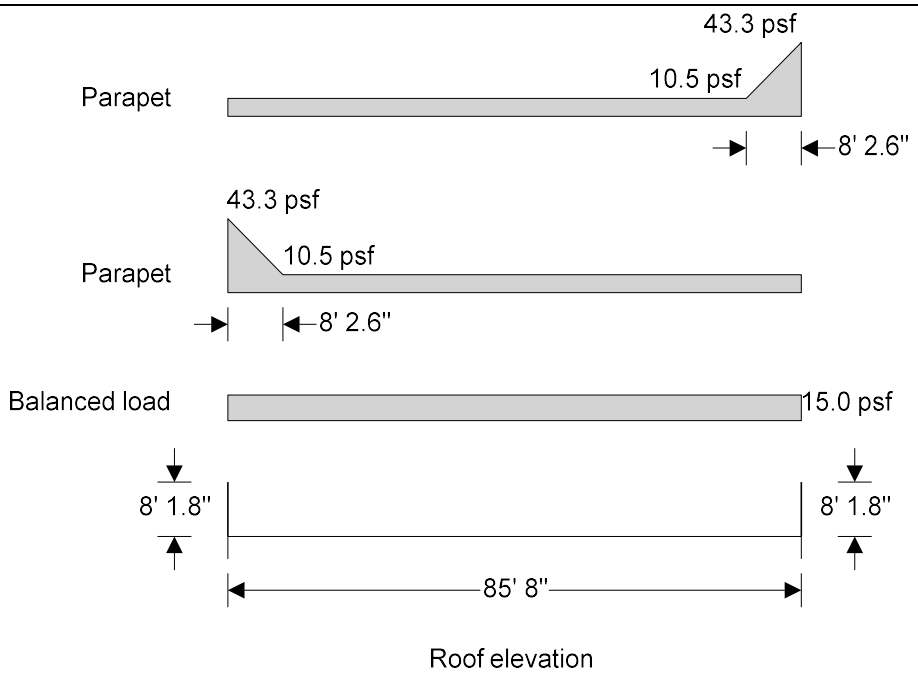
Left parapet

Balanced snow load height;	$h_b = p_f / g$ = **0.66** ft
Height of left parapet;	h_{pptL} = **8.15** ft
Height from balance load to top of left parapet;	$h_{c\_pptL} = h_{pptL} - h_b$ = **7.49** ft
Length of roof - left parapet;	$l_{u\_pptL} = b$ = **85.67** ft
Drift height windward drift - left parpet;	$h_{d\_l\_pptL} = 0.75 \cdot (0.43 \cdot (\max(20\text{ ft}, l_{u\_pptL}) \cdot 1\text{ft}^2)^{1/3} \cdot (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft})$ = **2.05** ft
Drift height - left parapet;	$h_{d\_pptL} = \min(h_{d\_l\_pptL}, h_{pptL} - h_b)$ = **2.05** ft
Drift width;	$W_{d\_pptL} = \min(4 \cdot h_{d\_l\_pptL}, 8 \cdot (h_{pptL} - h_b), b)$ = **8.22** ft
Drift surcharge load - left parapet;	$p_{d\_pptL} = h_{d\_pptL} \cdot g$ = **32.76** lb/ft^2

Right parapet

Height of right parapet;	h_{pptR} = **8.15** ft
Height from balance load to top of right parapet;	$h_{c\_pptR} = h_{pptR} - h_b$ = **7.49** ft
Length of roof - right parapet;	$l_{u\_pptR} = b$ = **85.67** ft
Drift height windward drift - right parpet;	$h_{d\_l\_pptR} = 0.75 \cdot (0.43 \cdot (\max(20\text{ ft}, l_{u\_pptR}) \cdot 1\text{ft}^2)^{1/3} \cdot (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft})$ = **2.05** ft
Drift height - right parapet;	$h_{d\_pptR} = \min(h_{d\_l\_pptR}, h_{pptR} - h_b)$ = **2.05** ft
Drift width;	$W_{d\_pptR} = \min(4 \cdot h_{d\_l\_pptR}, 8 \cdot (h_{pptR} - h_b), b)$ = **8.22** ft
Drift surcharge load - right parapet;	$p_{d\_pptR} = h_{d\_pptR} \cdot g$ = **32.76** lb/ft^2

	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	Loads				2-2	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024



Roof elevation

	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	Loads				2-3	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

SNOW LOAD (SHORT DIRECTION)

SNOW LOADING

In accordance with ASCE7-10

Tedds calculation version 1.0.11

Building details

Roof type	Flat
Width of roof	b = **46.33** ft

Ground snow load

Ground snow load (Figure 7-1)	p_g = **15.00** lb/ft^2
Density of snow	$\gamma = \min(0.13 \times p_g / 1ft + 14lb/ft^3, 30lb/ft^3)$ = **15.95** lb/ft^3
Terrain typeSect. 26.7	C
Exposure condition (Table 7-2)	Partially exposed
Exposure factor (Table 7-2)	C_e = **1.00**
Thermal condition (Table 7-3)	All
Thermal factor (Table 7-3)	C_t = **1.00**
Importance category (Table 1.5-1)	II
Importance factor (Table 1.5-2)	I_s = **1.00**
Min snow load for low slope roofs (Sect 7.3.4)	$p_{f\_min} = I_s \times p_g$ = **15.00** lb/ft^2
Flat roof snow load (Sect 7.3)	$p_f = 0.7 \times C_e \times C_t \times I_s \times p_g$ = **10.50** lb/ft^2

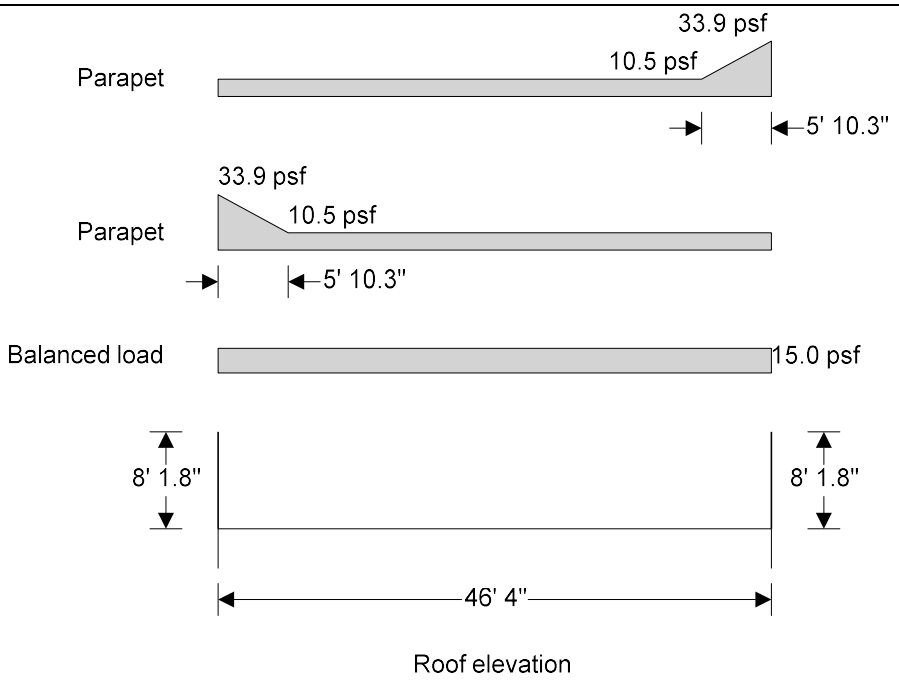
Left parapet

Balanced snow load height	$h_b = p_f / \gamma$ = **0.66** ft
Height of left parapet	h_{pptL} = **8.15** ft
Height from balance load to top of left parapet	$h_{c\_pptL} = h_{pptL} - h_b$ = **7.49** ft
Length of roof - left parapet	$l_{u\_pptL} = b$ = **46.33** ft
Drift height windward drift - left parpet	$h_{d\_l\_pptL} = 0.75 \times (0.43 \times (\max(20\ ft, l_{u\_pptL}) \times 1ft^2)^{1/3} \times (p_g / 1lb/ft^2 + 10)^{1/4} - 1.5ft)$ = **1.47** ft
Drift height - left parapet	$h_{d\_pptL} = \min(h_{d\_l\_pptL}, h_{pptL} - h_b)$ = **1.47** ft
Drift width	$W_{d\_pptL} = \min(4 \times h_{d\_l\_pptL}, 8 \times (h_{pptL} - h_b), b)$ = **5.86** ft
Drift surcharge load - left parapet	$p_{d\_pptL} = h_{d\_pptL} \times \gamma$ = **23.37** lb/ft^2

Right parapet

Height of right parapet	h_{pptR} = **8.15** ft
Height from balance load to top of right parapet	$h_{c\_pptR} = h_{pptR} - h_b$ = **7.49** ft
Length of roof - right parapet	$l_{u\_pptR} = b$ = **46.33** ft
Drift height windward drift - right parpet	$h_{d\_l\_pptR} = 0.75 \times (0.43 \times (\max(20\ ft, l_{u\_pptR}) \times 1ft^2)^{1/3} \times (p_g / 1lb/ft^2 + 10)^{1/4} - 1.5ft)$ = **1.47** ft
Drift height - right parapet	$h_{d\_pptR} = \min(h_{d\_l\_pptR}, h_{pptR} - h_b)$ = **1.47** ft
Drift width	$W_{d\_pptR} = \min(4 \times h_{d\_l\_pptR}, 8 \times (h_{pptR} - h_b), b)$ = **5.86** ft
Drift surcharge load - right parapet	$p_{d\_pptR} = h_{d\_pptR} \times \gamma$ = **23.37** lb/ft^2

	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	Loads				2-4	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024



Roof elevation

	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	Loads				2-5	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

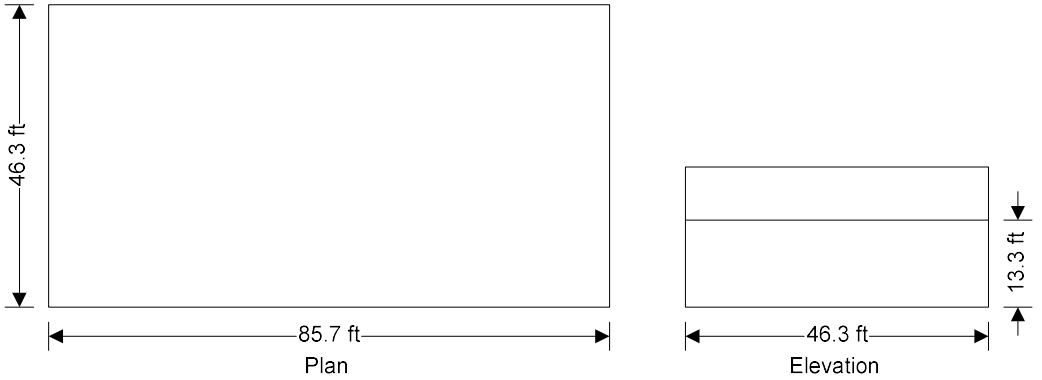
WIND LOADING (SHORT DIRECTION)

WIND LOADING

In accordance with ASCE7-10

Using the directional design method

Tedds calculation version 2.1.14



Building data

Type of roof	Flat
Length of building	b = **85.66** ft
Width of building	d = **46.33** ft
Height to eaves	H = **13.33** ft
Height of parapet	h_p = **8.15** ft
Mean height	h = **13.33** ft

General wind load requirements

Basic wind speed	V = **117.0** mph
Risk category	II
Velocity pressure exponent coef (Table 26.6-1)	K_d = **0.85**
Exposure category (cl 26.7.3)	C
Enclosure classification (cl.26.10)	Enclosed buildings
Internal pressure coef +ve (Table 26.11-1)	GC_{pi_p} = **0.18**
Internal pressure coef –ve (Table 26.11-1)	GC_{pi_n} = **-0.18**
Gust effect factor	G_f = **0.85**
Minimum design wind loading (cl.27.1.5)	p_{min_r} = **8** lb/ft^2

Topography

Topography factor not significant	K_{zt} = 1.0
Velocity pressure equation	$q = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \times 1\text{psf/mph}^2$

Velocity pressures table

z (ft)	K_z (Table 27.3-1)	q_z (psf)
13.33	0.85	25.32

z (ft)	K_z (Table 27.3-1)	q_z (psf)
21.48	0.91	27.16

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.) q_i = **25.32** psf

Parapet pressures and forces

Velocity pressure at top of parapet q_p = **27.16** psf

Combined net pressure coefficient, leeward GC_{pnl} = **-1.0**

Combined net parapet pressure, leeward $p_{pl} = q_p \times GC_{pnl}$ = **-27.16** psf

Combined net pressure coefficient, windward GC_{pnw} = **1.5**

Combined net parapet pressure, windward $p_{pw} = q_p \times GC_{pnw}$ = **40.74** psf

Wind direction 90 deg:

Leeward parapet force $F_{w,wpl\_90} = p_{pl} \times h_p \times d$ = **-10.3** kips

Windward parapet force $F_{w,wpw\_90} = p_{pw} \times h_p \times d$ = **15.4** kips

Pressures and forces

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

Net force $F_w = p \times A_{ref}$

Roof load case 1 - Wind 90, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	13.33	-0.90	25.32	-23.93	308.79	-7.39
B (-ve)	13.33	-0.90	25.32	-23.93	308.79	-7.39
C (-ve)	13.33	-0.50	25.32	-15.32	617.58	-9.46
D (-ve)	13.33	-0.30	25.32	-11.01	2733.47	-30.11

Total vertical net force $F_{w,v}$ = **-54.34** kips

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

Walls load case 1 - Wind 90, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	13.33	0.80	25.32	12.66	617.58	7.82
B	13.33	-0.33	25.32	-11.66	617.58	-7.20
C	13.33	-0.70	25.32	-19.62	1141.85	-22.41
D	13.33	-0.70	25.32	-19.62	1141.85	-22.41

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_90} = d \times (H + h_p)$ = **995.17** ft^2

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

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90}$ = **15.92** kips

Leeward net force $F_l = F_{w,wB} + F_{w,wpl\_90}$ = **-17.5** kips

Windward net force $F_w = F_{w,wA} + F_{w,wpw\_90}$ = **23.2** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min})$ = **40.7** kips

Roof load case 2 - Wind 90, GC_{pi} -0.18, $+c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (+ve)	13.33	-0.18	25.32	0.68	308.79	0.21
B (+ve)	13.33	-0.18	25.32	0.68	308.79	0.21
C (+ve)	13.33	-0.18	25.32	0.68	617.58	0.42
D (+ve)	13.33	-0.18	25.32	0.68	2733.47	1.87

Total vertical net force $F_{w,v}$ = **2.71** kips

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

Walls load case 2 - Wind 90, GC_{pi} -0.18, $+c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	13.33	0.80	25.32	21.77	617.58	13.45
B	13.33	-0.33	25.32	-2.55	617.58	-1.57
C	13.33	-0.70	25.32	-10.51	1141.85	-12.00
D	13.33	-0.70	25.32	-10.51	1141.85	-12.00

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_90} = d \times (H + h_p)$ = **995.17** ft^2

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

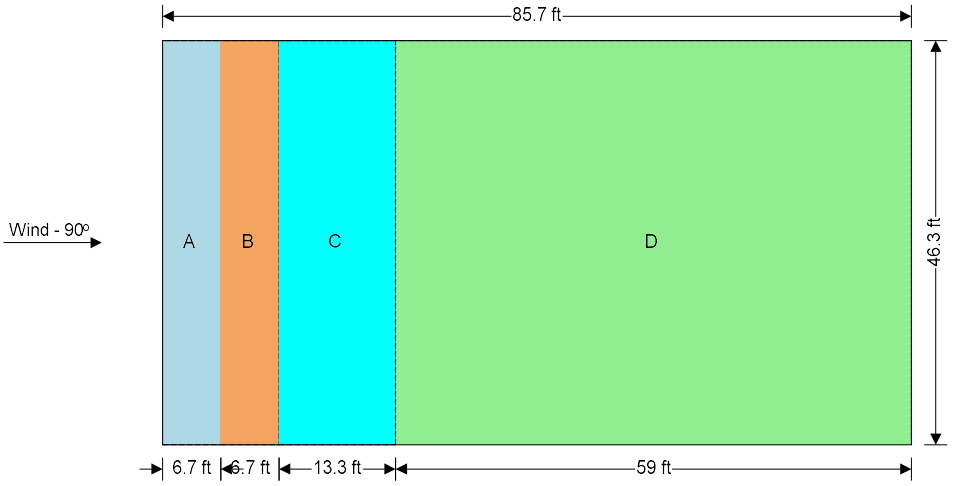
Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90}$ = **15.92** kips

Leeward net force $F_l = F_{w,wB} + F_{w,wpl\_90}$ = **-11.8** kips

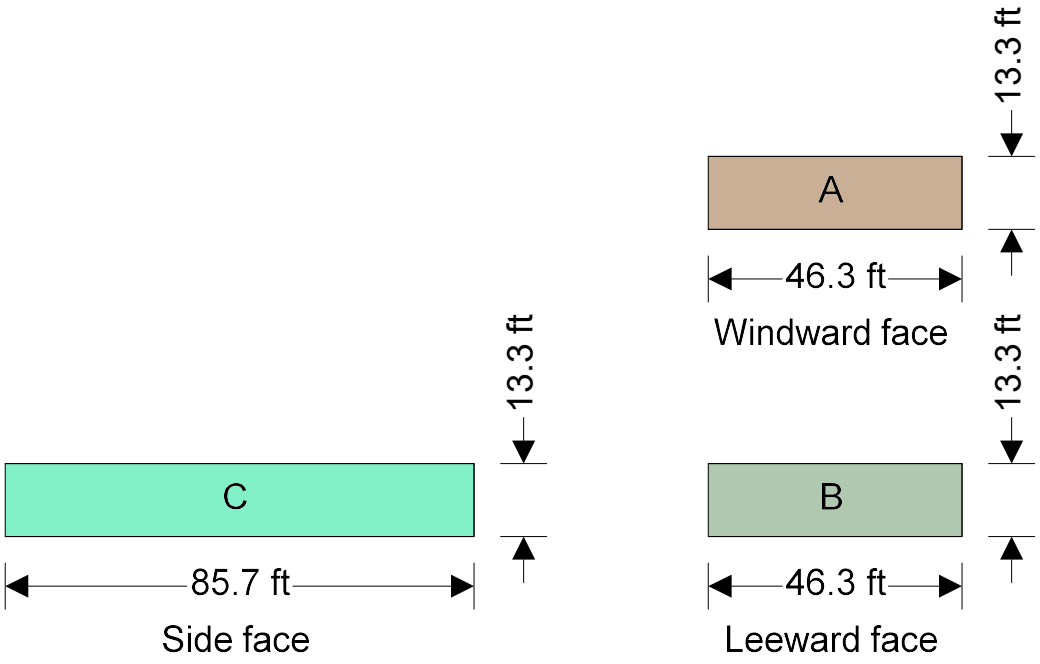
Windward net force $F_w = F_{w,wA} + F_{w,wpw\_90}$ = **28.8** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min})$ = **40.7** kips

AECOM	Project: McDonalds Lillington, NC				Job Ref.: 032-0312	
	Section: Loads				Sheet no./rev.: 2-8	
	Calc. by: SA/PB	Date: 8/7/2024	Chk'd by: ES	Date: 8/7/2024	App'd by: MY	Date: 8/7/2024



Plan view - Flat roof



Shear Diaphragm force from wind $F_{DIA} = F_{w,total} - ((F_{w,wA} - F_{w,wB}) / 2) =$ **33.19** kips (Factored)

	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	Loads				2-9	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

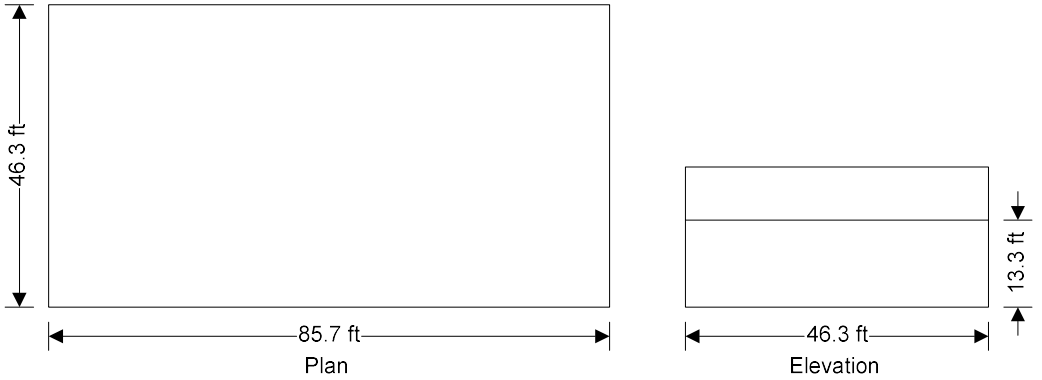
WIND LOADING (LONG DIRECTION)

WIND LOADING

In accordance with ASCE7-10

Using the directional design method

Tedds calculation version 2.1.14



Building data

Type of roof	Flat
Length of building	b = **85.67** ft
Width of building	d = **46.33** ft
Height to eaves	H = **13.33** ft
Height of parapet	h_p = **8.15** ft
Mean height	h = **13.33** ft

General wind load requirements

Basic wind speed	V = **117.0** mph
Risk category	II
Velocity pressure exponent coef (Table 26.6-1)	K_d = **0.85**
Exposure category (cl 26.7.3)	C
Enclosure classification (cl.26.10)	Enclosed buildings
Internal pressure coef +ve (Table 26.11-1)	GC_{pi_p} = **0.18**
Internal pressure coef –ve (Table 26.11-1)	GC_{pi_n} = **-0.18**
Gust effect factor	G_f = **0.85**
Minimum design wind loading (cl.27.1.5)	p_{min_r} = **8** lb/ft^2

Topography

Topography factor not significant	K_{zt} = 1.0
Velocity pressure equation	$q = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \times 1\text{psf/mph}^2$

Velocity pressures table

z (ft)	K_z (Table 27.3-1)	q_z (psf)
13.33	0.85	25.32

z (ft)	K_z (Table 27.3-1)	q_z (psf)
21.48	0.91	27.16

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.) q_i = **25.32** psf

Parapet pressures and forces

Velocity pressure at top of parapet q_p = **27.16** psf

Combined net pressure coefficient, leeward GC_{pnl} = **-1.0**

Combined net parapet pressure, leeward $p_{pl} = q_p \times GC_{pnl}$ = **-27.16** psf

Combined net pressure coefficient, windward GC_{pnw} = **1.5**

Combined net parapet pressure, windward $p_{pw} = q_p \times GC_{pnw}$ = **40.74** psf

Wind direction 0 deg:

Leeward parapet force $F_{w,wpl\_0} = p_{pl} \times h_p \times b$ = **-19** kips

Windward parapet force $F_{w,wpw\_0} = p_{pw} \times h_p \times b$ = **28.4** kips

Pressures and forces

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

Net force $F_w = p \times A_{ref}$

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	13.33	-0.90	25.32	-23.93	570.96	-13.66
B (-ve)	13.33	-0.90	25.32	-23.93	570.96	-13.66
C (-ve)	13.33	-0.50	25.32	-15.32	1141.93	-17.49
D (-ve)	13.33	-0.30	25.32	-11.01	1685.05	-18.56

Total vertical net force $F_{w,v}$ = **-63.37** kips

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

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	13.33	0.80	25.32	12.66	1141.93	14.46
B	13.33	-0.50	25.32	-15.32	1141.93	-17.49
C	13.33	-0.70	25.32	-19.62	617.58	-12.12
D	13.33	-0.70	25.32	-19.62	617.58	-12.12

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_0} = b \times (H + h_p)$ = **1840.11** ft^2

Projected vertical area of roof $A_{vert\_r\_0}$ = **0.00** ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0}$ = **29.44** kips

Leeward net force $F_l = F_{w,wB} + F_{w,wpl\_0}$ = **-36.5** kips

Windward net force $F_w = F_{w,wA} + F_{w,wpw\_0}$ = **42.9** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min})$ = **79.4** kips

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (+ve)	13.33	-0.18	25.32	0.68	570.96	0.39
B (+ve)	13.33	-0.18	25.32	0.68	570.96	0.39
C (+ve)	13.33	-0.18	25.32	0.68	1141.93	0.78
D (+ve)	13.33	-0.18	25.32	0.68	1685.05	1.15

Total vertical net force $F_{w,v}$ = **2.71** kips

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

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	13.33	0.80	25.32	21.77	1141.93	24.86
B	13.33	-0.50	25.32	-6.20	1141.93	-7.08
C	13.33	-0.70	25.32	-10.51	617.58	-6.49
D	13.33	-0.70	25.32	-10.51	617.58	-6.49

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_0} = b \times (H + h_p)$ = **1840.11** ft^2

Projected vertical area of roof $A_{vert\_r\_0}$ = **0.00** ft^2

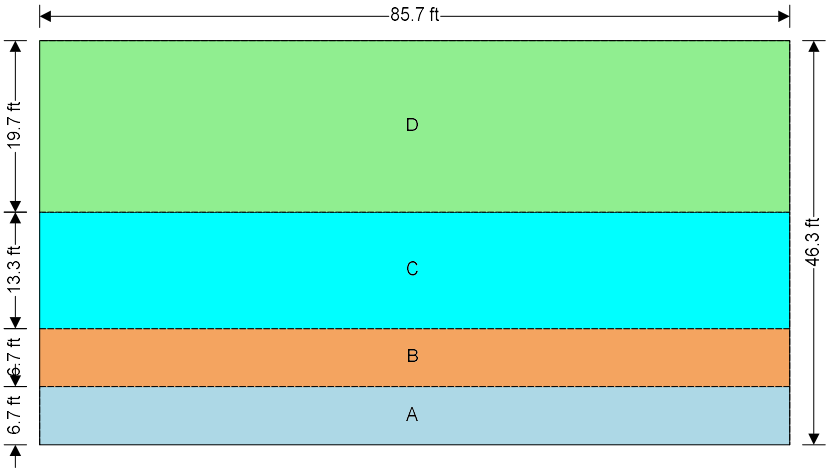
Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0}$ = **29.44** kips

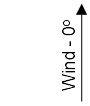
Leeward net force $F_l = F_{w,wB} + F_{w,wpl\_0}$ = **-26.0** kips

Windward net force $F_w = F_{w,wA} + F_{w,wpw\_0}$ = **53.3** kips

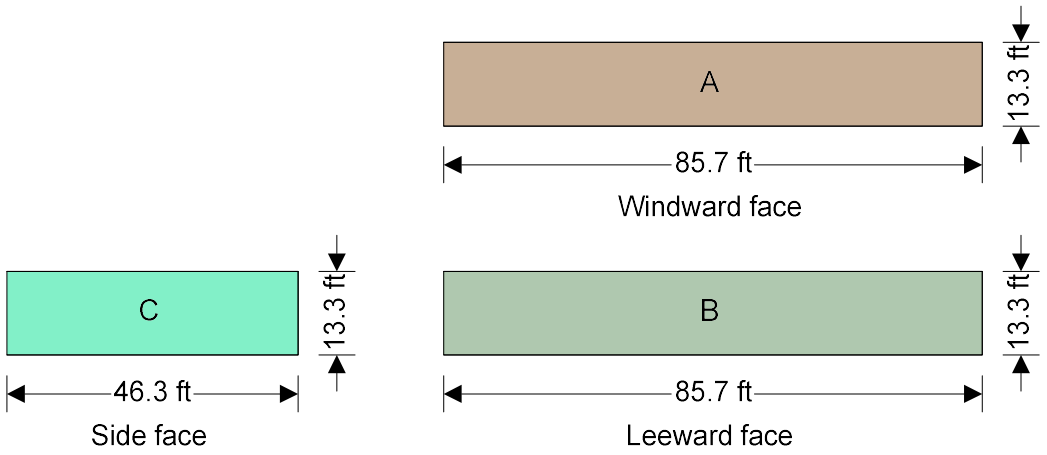
Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min})$ = **79.4** kips

	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	Loads				2-12	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024





Plan view - Flat roof



Shear Diaphragm force from wind $F_{DIA} = F_{w,total} - ((F_{w,wA} - F_{w,wB}) / 2) =$ **63.43** kips (Factored)

SEISMIC LOADING (ASCE 7-10) (CASE 1)

The restaurant has multiple lateral systems, steel ordinary concentrically braced frame system in longitudinal direction (R=3.25) and light frame wood walls sheathed with wood structural panels rated for shear resistance in transverse direction (R=6.5). Conservatively calculated the seismic load resisted by steel system not specifically detailed for seismic resistance since it has smallest R value and which would result in largest Cs value and therefore largest seismic load.

Equivalent wall weight = 40 psf

Total building weight = 20 psf * 85.66 ft * 46.33 ft + 40 psf * 21.48 ft * (85.66 ft + 46.33 ft) *2 = 306.2 kips

Diaphragm tributary weight at roof level = 20 psf * 85.66 ft * 46.33 ft + 40 psf * (21.48 ft - 13.33ft/2) * (85.66 ft + 46.33 ft) * 2 = 235.8 kips

SEISMIC FORCES

In accordance with ASCE 7-10

Tedds calculation version 3.1.04

Site parameters

Site class	E
Mapped acceleration parameters (Section 11.4.1)	
at short period	S_S = **0.181**
at 1 sec period	S_1 = **0.085**
Site coefficientat short period (Table 11.4-1)	F_a = **2.500**
at 1 sec period (Table 11.4-2)	F_v = **3.500**

Spectral response acceleration parameters

at short period (Eq. 11.4-1)	$S_{MS} = F_a \times S_S$ = **0.453**
at 1 sec period (Eq. 11.4-2)	$S_{M1} = F_v \times S_1$ = **0.298**

Design spectral acceleration parameters (Sect 11.4.4)

at short period (Eq. 11.4-3)	$S_{DS} = 2 / 3 \times S_{MS}$ = **0.302**
at 1 sec period (Eq. 11.4-4)	$S_{D1} = 2 / 3 \times S_{M1}$ = **0.198**

Seismic design category

Risk category (Table 1.5-1)	II

Seismic design category based on short period response acceleration (Table 11.6-1)

B

Seismic design category based on 1 sec period response acceleration (Table 11.6-2)

C

Seismic design category C

Approximate fundamental period

Height above base to highest level of building h_n = **13.33** ft

From Table 12.8-2:

Structure type All other systems

Building period parameter C_t	C_t = **0.02**
Building period parameter x	x = **0.75**
Approximate fundamental period (Eq 12.8-7)	$T_a = C_t \times (h_n)^x \times 1sec / (1ft)^x$= **0.140** sec
Building fundamental period (Sect 12.8.2)	$T = T_a$ = **0.140** sec
Long-period transition period	T_L = **8** sec

Seismic response coefficient

Seismic force-resisting system (Table 12.2-1)	B_BUILDING_FRAME_SYSTEMS 3. Ordinary steel concentrically braced frames
Response modification factor (Table 12.2-1)	R = **3.25**
Seismic importance factor (Table 1.5-2)	I_e = **1.000**
Seismic response coefficient (Sect 12.8.1.1)	
Calculated (Eq 12.8-2)	$C_{s\_calc} = S_{DS} / (R / I_e)$ = **0.0928**
Maximum (Eq 12.8-3)	$C_{s\_max} = S_{D1} / ((T / 1\ sec) \times (R / I_e))$ = **0.4374**
Minimum (Eq 12.8-5)	$C_{s\_min} = \max(0.044 \times S_{DS} \times I_e, 0.01)$ = **0.0133**
Seismic response coefficient	C_s = **0.0928**

Seismic base shear (Sect 12.8.1)

Effective seismic weight of the structure	W = **306.2** kips
Seismic response coefficient	C_s = **0.0928**
Seismic base shear (Eq 12.8-1)	$V = C_s \times W$ = **28.4** kips

Vertical distribution of seismic forces (Sect 12.8.3)

Vertical distribution factor (Eq 12.8-12)	$C_{vx} = w_x \times h_x^k / \Sigma(w_i \times h_i^k)$
Lateral force induced at level i (Eq 12.8-11)	$F_x = C_{vx} \times V$

Minimum diaphragm forces (Section 12.10.1.1)

Calculated min. diaphragm force (Eq 12.10-1)	$F_{px} = \Sigma F_i \times w_{px} / \Sigma w_i$,(i=x to n)
	$F_{pxmin} = 0.2 \times S_{DS} \times I_e \times w_{px}$ = 14.24 **kips**
	$F_{pxmax} = 0.4 \times S_{DS} \times I_e \times w_{px}$ = 28.49 **kips**

Vertical force distribution table

Level	Height from base to Level i (ft), h_x	Portion of effective seismic weight assigned to Level i (kips), w_x	Distribution exponent related to building period, k	Vertical distribution factor, C_{vx}	Lateral force induced at Level i (kips), F_x	Weight tributary to the diaphragm at Level i (kips), w_{px}	Minimum diaphragm force at Level i (kips), F_{px}
1	13.3;	306.2;	1.00;	1.000;	28.4	235.8	21.9

Lateral force at level i falls out of the range of seismic diaphragm design force. Fx = **28.49** kips for seismic design.

SEISMIC LOADING (ASCE 7-10) (CASE 2)

The seismic load in transverse direction is resisted by light frame wood walls sheathed with wood structural panels rated for shear resistance (R=6.5).

Equivalent wall weight = 40 psf

Total building weight = 20 psf * 85.66 ft * 46.33 ft + 40 psf * 21.48 ft * (85.66 ft + 46.33 ft) *2 = 306.2 kips

Diaphragm tributary weight at roof level = 20 psf * 85.66 ft * 46.33 ft + 40 psf * (21.48 ft - 13.33ft/2) * (85.66 ft + 46.33 ft) * 2 = 235.8 kips

SEISMIC FORCES

In accordance with ASCE 7-10

Tedds calculation version 3.1.04

Site parameters

Site class	E
Mapped acceleration parameters (Section 11.4.1)	
at short period	S_S = **0.181**
at 1 sec period	S_1 = **0.085**
Site coefficientat short period (Table 11.4-1)	F_a = **2.500**
at 1 sec period (Table 11.4-2)	F_v = **3.500**

Spectral response acceleration parameters

at short period (Eq. 11.4-1)	$S_{MS} = F_a \times S_S$ = **0.453**
at 1 sec period (Eq. 11.4-2)	$S_{M1} = F_v \times S_1$ = **0.298**

Design spectral acceleration parameters (Sect 11.4.4)

at short period (Eq. 11.4-3)	$S_{DS} = 2 / 3 \times S_{MS}$ = **0.302**
at 1 sec period (Eq. 11.4-4)	$S_{D1} = 2 / 3 \times S_{M1}$ = **0.198**

Seismic design category

Risk category (Table 1.5-1)	II

Seismic design category based on short period response acceleration (Table 11.6-1)

B

Seismic design category based on 1 sec period response acceleration (Table 11.6-2)

C

Seismic design category C

Approximate fundamental period

Height above base to highest level of building	h_n = **13.33** ft

From Table 12.8-2:

Structure type	All other systems
Building period parameter C_t	C_t = **0.02**
Building period parameter x	x = **0.75**

Approximate fundamental period (Eq 12.8-7)	$T_a = C_t \times (h_n)^x \times 1sec / (1ft)^x =$ **0.140** sec
Building fundamental period (Sect 12.8.2)	$T = T_a =$ **0.140** sec
Long-period transition period	$T_L =$ **8** sec

Seismic response coefficient

Seismic force-resisting system (Table 12.2-1)	A. Bearing_Wall_Systems 15. Light-frame (wood) walls sheathed with wood structural panels
Response modification factor (Table 12.2-1)	R = **6.5**
Seismic importance factor (Table 1.5-2)	$I_e =$ **1.000**
Seismic response coefficient (Sect 12.8.1.1)	
Calculated (Eq 12.8-2)	$C_{s\_calc} = S_{DS} / (R / I_e) =$ **0.0464**
Maximum (Eq 12.8-3)	$C_{s\_max} = S_{D1} / ((T / 1\ sec) \times (R / I_e)) =$ **0.2187**
Minimum (Eq 12.8-5)	$C_{s\_min} = \max(0.044 \times S_{DS} \times I_e, 0.01) =$ **0.0133**
Seismic response coefficient	$C_s =$ **0.0464**

Seismic base shear (Sect 12.8.1)

Effective seismic weight of the structure	W = **306.2** kips
Seismic response coefficient	$C_s =$ **0.0464**
Seismic base shear (Eq 12.8-1)	$V = C_s \times W =$ **14.2** kips

Vertical distribution of seismic forces (Sect 12.8.3)

Vertical distribution factor (Eq 12.8-12)	$C_{vx} = w_x \times h_x^k / \Sigma(w_i \times h_i^k)$
Lateral force induced at level i (Eq 12.8-11)	$F_x = C_{vx} \times V$

Minimum diaphragm forces (Section 12.10.1.1)

Calculated min. diaphragm force (Eq 12.10-1)	$F_{px} = \Sigma F_i \times w_{px} / \Sigma w_i, (i=x \text{ to } n)$
	$F_{pxmin} = 0.2 \times S_{DS} \times I_e \times w_{px} =$ 14.24 **kips**
	$F_{pxmax} = 0.4 \times S_{DS} \times I_e \times w_{px} =$ 28.29 **kips**

Vertical force distribution table

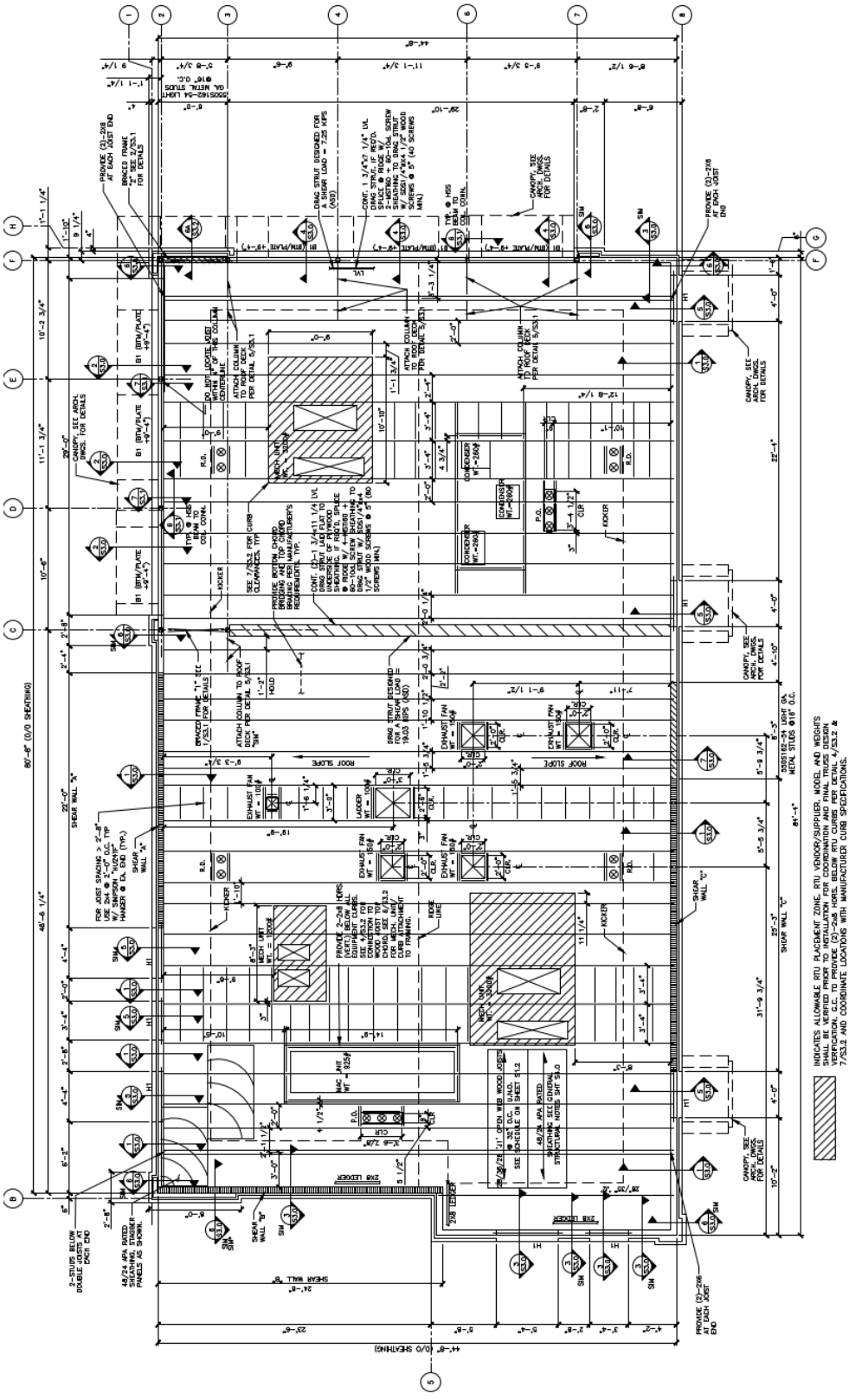
Level	Height from base to Level i (ft), h_x	Portion of effective seismic weight assigned to Level i (kips), w_x	Distribution exponent related to building period, k	Vertical distribution factor, C_{vx}	Lateral force induced at Level i (kips), F_x	Weight tributary to the diaphragm at Level i (kips), w_{px}	Minimum diaphragm force at Level i (kips), F_{px}
1	13.3;	306.2;	1.00;	1.000;	14.2	235.8	14.2

Fx = **14.24** kips for seismic design.

(Wind load is the governing lateral force)

3. Framing

	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	Framing Layout				3-1	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024



	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	H1 header check				3-2	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

H1 DESIGN - LOADS

1. Canopy Load

Canopy projection S= 4.33 ft
Snow load SL= 15 psf
Dead Load: DL=3 psf

$W_{\_snow}$ 15*4.33= 64.95 lb/ft
$W_{\_dead}$ =3*4.33=12.99 lb/ft

Torsion

$W_{\_snow}$ = 15 psf * 1 ft = 15 lb/ft
$M_{\_snow}$ = 1/2 * 15 lb/ft * (4.33ft)^2 = 140.62 lb-ft/ft
$T_{\_snow}$ = 140.62 lb-ft/ft / 4.375 = 32.14 lb/ft
$W_{\_dead}$ = 3 psf * 1 ft = 3 lb/ft
$M_{\_dead}$ = 1/2 * 3 lb/ft * (4.33ft)^2 = 28.12 lb-ft/ft
$T_{\_dead}$ = 28.12 lb-ft/ft / 4.375 = 6.43 lb/ft

2. Wall and EIFS above header

Wall height: h_w=12ft
Wall weight: w= 20 psf (note that brick veneer load is not taken by the header)

Also consider the additional snow loads at overhang portion = 50 plf

$W_{\_dead}$= 20*12+50=290 plf

3. Roof load

Roof Snow Load SL=15 psf
Dead Load: DL=20 psf

$W_{\_dead}$ =20*46.33 ft /2 = 463.3 lb/ft
$W_{\_snow\ drift}$ =15*46.33 ft /2 + 32.80*8.22 ft /2= 482.28 lb/ft

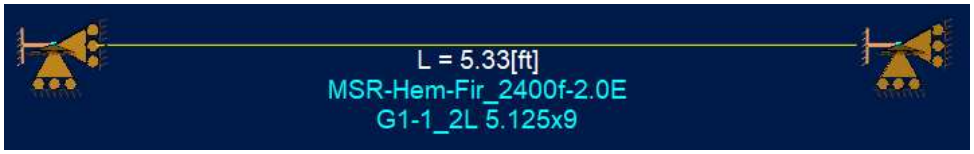
$W_{H1\_dead\_total}$ = 12.99+290+463.3 = 766.29 plf
W_{H1_snow} = 64.95+482.28 = 547.23 plf

4. Wind load

Conservatively using 5'-4" for header span; Maximum Length of header = 5.33 ft
13.33/2*5.33= 35.52 sq.ft. From ASCE C&C wind design table, for 35.52 sq.ft of tributary area wind load is 32 psf.
Wind load on header H1 = 32 psf*13.33 ft/2 = 213.28 plf

Note: Beam depth of 9.5" not available in RAM elements, depth of 9" used conservatively for design.

Beam material of 2.0E-2600FB not available in RAM elements, material of 2.0E-2400FB used conservatively for design.



RAM® Elements CONNECT Edition

AECOM

Current Date: 8/6/2024 10:22 AM
Units system: English
File name: C:\Job\McDonald\Working\Ram Element\Lillington\Header H1 with torsion.retx

Load data

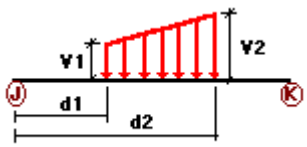
GLOSSARY

Comb : Indicates if load condition is a load combination

Load Conditions

Condition	Description	Comb.	Category
DL	Dead Load	No	DL
SL	Snow Load	No	SNOW
WL	Wind Load	No	WIND
D1	DL	Yes	
D2	DL+SL	Yes	
D3	DL+0.75SL	Yes	
D4	DL+0.6WL	Yes	
D5	DL+0.75SL+0.45WL	Yes	
D6	DL+0.75SL	Yes	
D7	0.6DL+0.6WL	Yes	
S1	DL	Yes	
S2	DL+SL	Yes	
S3	DL+0.75SL	Yes	
S4	DL+0.6WL	Yes	
S5	DL+0.75SL+0.45WL	Yes	
S6	DL+0.75SL	Yes	
S7	0.6DL+0.6WL	Yes	

Distributed force on members



Condition	Member	Dir1	Val1 [Kip/ft]	Val2 [Kip/ft]	Dist1 [ft]	%	Dist2 [ft]	%
DL	1	Y	-0.766	-0.766	0.00	Yes	100.00	Yes
		Z	-0.006	-0.006	0.00	Yes	100.00	Yes
SL	1	Y	-0.547	-0.547	0.00	No	100.00	Yes
		Z	-0.032	-0.032	0.00	Yes	100.00	Yes
WL	1	Z	-0.2133	-0.2133	0.00	Yes	100.00	Yes

Self weight multipliers for load conditions

Condition	Description	Self weight multiplier Comb.	MultX	MultY	MultZ
DL	Dead Load	No	0.00	-1.00	0.00
SL	Snow Load	No	0.00	0.00	0.00
WL	Wind Load	No	0.00	0.00	0.00
D1	DL	Yes	0.00	0.00	0.00
D2	DL+SL	Yes	0.00	0.00	0.00
D3	DL+0.75SL	Yes	0.00	0.00	0.00
D4	DL+0.6WL	Yes	0.00	0.00	0.00
D5	DL+0.75SL+0.45WL	Yes	0.00	0.00	0.00
D6	DL+0.75SL	Yes	0.00	0.00	0.00
D7	0.6DL+0.6WL	Yes	0.00	0.00	0.00
S1	DL	Yes	0.00	0.00	0.00
S2	DL+SL	Yes	0.00	0.00	0.00
S3	DL+0.75SL	Yes	0.00	0.00	0.00
S4	DL+0.6WL	Yes	0.00	0.00	0.00
S5	DL+0.75SL+0.45WL	Yes	0.00	0.00	0.00
S6	DL+0.75SL	Yes	0.00	0.00	0.00
S7	0.6DL+0.6WL	Yes	0.00	0.00	0.00

Wood Design

Design code: ANSI/AF&PA NDS-2005 ASD

Report: Concise

Member : 1
Design status : OK

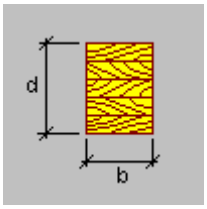
PROPERTIES

Section information

	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	H1 header check				3-5	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

Section name: G1-1_2L 5.125x9 **(US)**

Dimensions



b	=	5.125	[in]	Width
d	=	9.000	[in]	Height

Properties

Section properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	46.125	
Moment of Inertia (principal axes) (I')	[in4]	311.344	100.958
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	69.188	39.398

Material : MSR-Hem-Fir_2400f-2.0E

Properties	Value
Type:	Lumber
Species:	Hem-Fir
Grade:	MSR
Coefficient of variation:	0.11

DESIGN CRITERIA

Description	Unit	Value
Temperature:	--	T<=100F
Moisture conditions:	--	Dry
Wood:	--	Unincised
Repetitive member:	--	No
Type:	--	Beam
End notches at top:	--	Top
Notch length:	[in]	0.00
Notch depth:	[in]	0.00

Description	Unit	Major axis	Minor axis
Physical length	[ft]	5.33	
Effective length for bending (Le)	[ft]	0.00	
Unbraced length for bending (Lu)	[ft]	5.33	
Unbraced compression length (Lx, Ly)	[ft]	5.33	5.33
Effective length factor (K)	--	1.00	1.00
Lateral bracing	--	No	No
Bearing length (Lb)	[in]	0.50	
Length between inflection points (Li)	[ft]	5.33	

SERVICE CONDITIONS

Verification	Unit	Value	Ctrl EQ	Reference

Deflection in compression and/or bending	--	0.00	S2 at 50.00%

DESIGN CHECKS

DESIGN FOR TENSION

✔

Ratio	:	0.00			
Capacity	:	1.73 [Kip/in2]	Reference	:	(Sec. 3.8)
Demand	:	0.00 [Kip/in2]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Axial design value for tension (Ft):	[Kip/in2]	1.93	
Tension axial force (P+):	[Kip]	0.00	

DESIGN FOR COMPRESSION

✔

Ratio	:	0.00			
Capacity	:	1.60 [Kip/in2]	Reference	:	(Sec. 3.6.3)
Demand	:	0.00 [Kip/in2]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Axial design value for compression (Fc):	[Kip/in2]	1.93	
Compression axial force (P-):	[Kip]	0.00	
Modulus of elasticity for stability (Emin):	[Kip/in2]	1020.00	
Adjusted modulus of elasticity for stability (Emin'):	[Kip/in2]	1020.00	
Critical buckling design value (FcE1):	[Kip/in2]	16.60	(Sec. 3.9.2)
Critical buckling design value (FcE2):	[Kip/in2]	5.38	(Sec. 3.9.2)

DESIGN FOR FLEXURE

✔

Bending about major axis, M33

Ratio	:	0.30			
Capacity	:	2.74 [Kip/in2]	Reference	:	(Sec. 3.3)
Demand	:	0.82 [Kip/in2]	Ctrl Eq.	:	D2 at 50.00%

Intermediate results	Unit	Value	Reference
Bending design value (Fb):	[Kip/in2]	2.40	
Bending moment (Mxx):	[Kip*ft]	4.70	
Slenderness Ratio (RB):	--	6.71	(Eq. 3.3-5)
Critical buckling design value (FbE):	[Kip/in2]	27.22	(Sec. 3.3.3.8)

Bending about minor axis, M22

Ratio	:	0.04			
Capacity	:	3.84 [Kip/in2]	Reference	:	(Sec. 3.3)
Demand	:	0.14 [Kip/in2]	Ctrl Eq.	:	D4 at 50.00%

Intermediate results	Unit	Value	Reference
Bending design value (Fbyy):	[Kip/in2]	2.40	
Bending moment (Myy):	[Kip*ft]	-0.48	

DESIGN FOR SHEAR ✔

Shear parallel to minor axis, V2

Ratio	:	0.59			
Capacity	:	0.20 [Kip/in2]	Reference	:	(Sec. 3.4)
Demand	:	0.11 [Kip/in2]	Ctrl Eq.	:	D2 at 100.00%

Intermediate results	Unit	Value	Reference
Shear design value (Fv):	[Kip/in2]	0.17	
Shear Force (Vy):	[Kip]	-3.53	
Notch factor (CN):	--	1.00	(Sec. 3.4.3)

Shear parallel to major axis, V3

Ratio	:	0.04			
Capacity	:	0.27 [Kip/in2]	Reference	:	(Sec. 3.4.2)
Demand	:	0.01 [Kip/in2]	Ctrl Eq.	:	D4 at 0.00%

Intermediate results	Unit	Value	Reference
Shear design value (Fv):	[Kip/in2]	0.17	
Shear Force (Vy):	[Kip]	0.36	

DESIGN FOR TORSION ✔

Ratio	:	0.00			
Capacity	:	0.10 [Kip/in2]	Reference	:	(AITC-TCM)
Demand	:	0.00 [Kip/in2]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Torsion design value (Fvt):	[Kip/in2]	0.11	
Torsion moment (Mtor):	[Kip*ft]	0.00	

DESIGN FOR BEARING (informative)

Intermediate results	Unit	Value	Reference
Maximum reaction (Rmax):	[Kip]	4.44	(Sec. 3.10.3)
Load angle (θ):	--	0.00	
Axial design value for compression (Fc*):	[Kip/in2]	1.73	
Comp. design value perpendicular to grain (Fcp):	[Kip/in2]	0.60	

INTERACTION ✔

Combined axial and bending interaction value

Ratio : 0.31

Ctrl Eq. : D2 at 50.00%
Reference : (Eq. 3.9-3)

CRITICAL STRENGTH RATIO ✔

Ratio : 0.59
Ctrl Eq. : D2 at 100.00% Reference : (Sec. 3.4)

Analysis result

Maximum relative deflections

Remark.- Magnitude of deflections in absolute value.

CONDITION **D1=DL**

Member	Defl. (2) [in]	@(%)	Defl. (3) [in]	@(%)
1	0.02377 (L/2691)	50.00000	0.00055 (< L/10000)	50.00000

CONDITION **D2=DL+SL**

Member	Defl. (2) [in]	@(%)	Defl. (3) [in]	@(%)
1	0.04051 (L/1579)	50.00000	0.00347 (< L/10000)	50.00000

CONDITION **D3=DL+0.75SL**

Member	Defl. (2) [in]	@(%)	Defl. (3) [in]	@(%)
1	0.03632 (L/1761)	50.00000	0.00274 (< L/10000)	50.00000

CONDITION **D4=DL+0.6WL**

Member	Defl. (2) [in]	@(%)	Defl. (3) [in]	@(%)
1	0.02377 (L/2691)	50.00000	0.01224 (L/5225)	50.00000

CONDITION **D5=DL+0.75SL+0.45WL**

Member	Defl. (2) [in]	@(%)	Defl. (3) [in]	@(%)
1	0.03632 (L/1761)	50.00000	0.01151 (L/5556)	50.00000

CONDITION **D6=DL+0.75SL**

Member	Defl. (2) [in]	@(%)	Defl. (3) [in]	@(%)

1	0.03632	(L/1761)	50.00000	0.00274 (< L/10000)	50.00000

CONDITION **D7=0.6DL+0.6WL**

Member	Defl. (2) [in]	@(%)	Defl. (3) [in]	@(%)
1	0.01426 (L/4485)	50.00000	0.01202 (L/5320)	50.00000

CONDITION **S1=DL**

Member	Defl. (2) [in]	@(%)	Defl. (3) [in]	@(%)
1	0.02377 (L/2691)	50.00000	0.00055 (< L/10000)	50.00000

CONDITION **S2=DL+SL**

Member	Defl. (2) [in]	@(%)	Defl. (3) [in]	@(%)
1	0.04051 (L/1579)	50.00000	0.00347 (< L/10000)	50.00000

CONDITION **S3=DL+0.75SL**

Member	Defl. (2) [in]	@(%)	Defl. (3) [in]	@(%)
1	0.03632 (L/1761)	50.00000	0.00274 (< L/10000)	50.00000

CONDITION **S4=DL+0.6WL**

Member	Defl. (2) [in]	@(%)	Defl. (3) [in]	@(%)
1	0.02377 (L/2691)	50.00000	0.01224 (L/5225)	50.00000

CONDITION **S5=DL+0.75SL+0.45WL**

Member	Defl. (2) [in]	@(%)	Defl. (3) [in]	@(%)
1	0.03632 (L/1761)	50.00000	0.01151 (L/5556)	50.00000

CONDITION **S6=DL+0.75SL**

Member	Defl. (2) [in]	@(%)	Defl. (3) [in]	@(%)
1	0.03632 (L/1761)	50.00000	0.00274 (< L/10000)	50.00000

CONDITION **S7=0.6DL+0.6WL**

Member	Defl. (2) [in]	@(%)	Defl. (3) [in]	@(%)
1	0.01426 (L/4485)	50.00000	0.01202 (L/5320)	50.00000

Deflections smaller than L/240 in each direction, serviceability is not an issue for this header.

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	Header H1 Brick Lintel Check			3-6	
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	SA/PM	8/7/2024	ES	8/7/2024	MY	8/7/2024

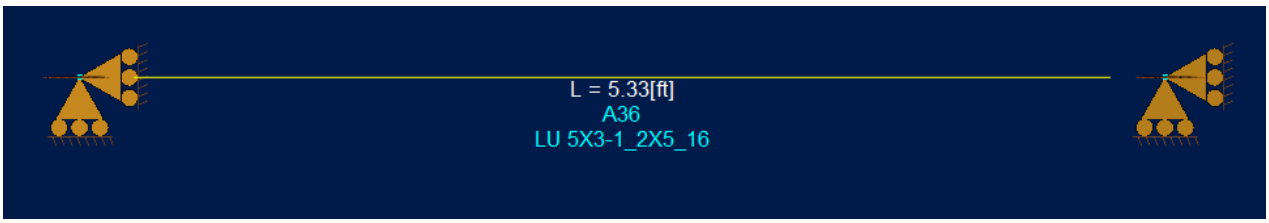
HEADER H1 BRICK LINTEL CHECK (5'-4" SPAN W/ 12.67FT OF BRICK VENEER ABOVE)

Loads:

DL:

40 psf brick veneer x 12.67 ft wall height = 506.8 plf

Total dead Load = 506.8 plf



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AECOM

Current Date: 7/31/2024 6:27 PM
Units system: English
File name: C:\Job\McDonald\Working\Ram Element\Lillington\H1 brick lintel check.retx

Load data

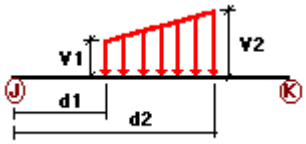
GLOSSARY

Comb : Indicates if load condition is a load combination

Load Conditions

Condition	Description	Comb.	Category
DL	Dead Load	No	DL
SL	Snow Load	No	SNOW
LC1	1.4DL	Yes	
D1	1.4DL	Yes	
D2	1.2DL+0.5SL	Yes	
D3	1.2DL+1.6SL	Yes	
D4	1.2DL+0.2SL	Yes	
S1	DL	Yes	
S2	DL+SL	Yes	
S3	DL+0.75SL	Yes	
S4	DL+0.75SL	Yes	

Distributed force on members



Condition	Member	Dir1	Val1 [Kip/ft]	Val2 [Kip/ft]	Dist1 [ft]	%	Dist2 [ft]	%
DL	1	Y	-0.506	-0.506	0.00	Yes	100.00	Yes

Self weight multipliers for load conditions

Condition	Description	Self weight multiplier Comb.	MultX	MultY	MultZ
DL	Dead Load	No	0.00	-1.00	0.00
SL	Snow Load	No	0.00	0.00	0.00
LC1	1.4DL	Yes	0.00	0.00	0.00
D1	1.4DL	Yes	0.00	0.00	0.00
D2	1.2DL+0.5SL	Yes	0.00	0.00	0.00
D3	1.2DL+1.6SL	Yes	0.00	0.00	0.00
D4	1.2DL+0.2SL	Yes	0.00	0.00	0.00
S1	DL	Yes	0.00	0.00	0.00
S2	DL+SL	Yes	0.00	0.00	0.00
S3	DL+0.75SL	Yes	0.00	0.00	0.00
S4	DL+0.75SL	Yes	0.00	0.00	0.00

Steel Code Check

Report: Concise

Members: Hot-rolled
Design code: AISC 360-2010 LRFD

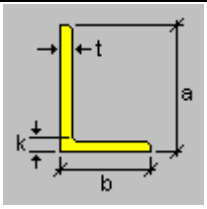
Member : 1
Design status : OK

Section information

Section name: LU 5X3-1_2X5_16 **(US)**

Dimensions

	Project				Job Ref.	
AECOM	McDonald's Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	Header H1 Brick Lintel Check				3-8	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PM	8/7/2024	ES	8/7/2024	MY	8/7/2024



a	=	5.000	[in]	Height
b	=	3.500	[in]	Width
k	=	0.750	[in]	Distance k
t	=	0.313	[in]	Thickness

Properties

Section properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	2.560	
Moment of Inertia (local axes) (I)	[in4]	6.580	2.690
Moment of Inertia (principal axes) (I')	[in4]	7.799	1.471
Bending constant for moments (principal axis) (J')	[in]	1.194	2.741
Radius of gyration (local axes) (r)	[in]	1.603	1.025
Radius of gyration (principal axes) (r')	[in]	1.745	0.758
Saint-Venant torsion constant. (J)	[in4]	0.088	
Section warping constant. (Cw)	[in6]	0.128	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	-0.687	-1.423
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	1.918	1.010
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	4.191	3.234
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	2.279	0.814
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	3.010	1.074
Plastic section modulus (local axis) (Z)	[in3]	3.450	1.770
Plastic section modulus (principal axis) (Z')	[in3]	3.903	1.635
Polar radius of gyration. (ro)	[in]	2.470	
Area for shear (Aw)	[in2]	1.090	1.560
Torsional constant. (C)	[in3]	0.267	

Material : A36

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	36.00
Tensile strength (Fu):	[Kip/in2]	58.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11507.94

DESIGN CRITERIA

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	5.33

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
5.33	5.33

Laterally unbraced length

Length [ft]	Effective length factor

Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
5.33	5.33	5.33	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway
Single angle connected through width	No
Planar element	No
Consider eccentricity	No
Shear load point of application	Gravity center

DESIGN CHECKS

AXIAL TENSION DESIGN ✔

Axial tension

Ratio	:	0.00			
Capacity	:	82.94 [Kip]	Reference	:	Eq. Sec. D2
Demand	:	0.00 [Kip]	Ctrl Eq.	:	LC1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	82.94	Eq. Sec. D2

AXIAL COMPRESSION DESIGN ✔

Compression in the major axis 33

Ratio	:	0.00			
Capacity	:	0.00 [Kip]			
Demand	:	0.00 [Kip]	Ctrl Eq.	:	--

Intermediate results	Unit	Value	Reference
Section classification			

Compression in the minor axis 22

Ratio	:	0.00			
Capacity	:	42.34 [Kip]	Reference	:	Sec. E1
Demand	:	0.00 [Kip]	Ctrl Eq.	:	LC1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	42.34	Sec. E1

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	SA/PM	8/7/2024	ES	8/7/2024	MY	8/7/2024

FLEXURAL DESIGN ✔

Bending about major axis, M33

Ratio	:	0.30			
Capacity	:	7.58 [Kip*ft]	Reference	:	Sec. F1
Demand	:	2.30 [Kip*ft]	Ctrl Eq.	:	LC1 at 50.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(ϕM_n):	[Kip*ft]	9.23	Sec. F1
Factored lateral-torsional buckling strength(ϕM_n):	[Kip*ft]	7.58	Sec. F1
Factored compression flange local buckling strength(ϕM_n):	[Kip*ft]	8.98	Sec. F1

Bending about minor axis, M22

Ratio	:	0.34			
Capacity	:	3.30 [Kip*ft]	Reference	:	Sec. F1
Demand	:	-1.12 [Kip*ft]	Ctrl Eq.	:	LC1 at 50.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(ϕM_n):	[Kip*ft]	3.30	Sec. F1
Factored compression flange local buckling strength(ϕM_n):	[Kip*ft]	5.58	Sec. F1

DESIGN FOR SHEAR ✔

Shear in major axis 33

Ratio	:	0.04			
Capacity	:	21.19 [Kip]	Reference	:	Sec. G1
Demand	:	-0.84 [Kip]	Ctrl Eq.	:	LC1 at 100.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	21.19	Sec. G1

Shear in minor axis 22

Ratio	:	0.06			
Capacity	:	30.33 [Kip]	Reference	:	Sec. G1
Demand	:	1.73 [Kip]	Ctrl Eq.	:	LC1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	30.33	Sec. G1

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AECOM	McDonald's Lillington, NC			032-0312	
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	Header H1 Brick Lintel Check			3-11	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PM	8/7/2024	ES	8/7/2024	MY	8/7/2024

COMBINED ACTIONS DESIGN ✔

Combined flexure and axial compression

Ratio : 0.64
Ctrl Eq. : LC1 at 50.00% Reference : Eq. H2-1

Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.64	Eq. H2-1

Combined flexure and axial tension

Ratio : 0.64
Ctrl Eq. : LC1 at 50.00% Reference : Eq. H2-1

Intermediate results	Unit	Value	Reference

Combined flexure and axial compression about local axis

Ratio : N/A
Ctrl Eq. : -- Reference :

Combined flexure and axial tension about local axis

Ratio : N/A
Ctrl Eq. : -- Reference :

Combined torsion and shear stresses

Ratio : 0.10
Ctrl Eq. : LC1 at 100.00% Reference : Eq. H3-8

Intermediate results	Unit	Value	Reference
Available shear stress for shear yielding(ϕF_{nv}):	[Kip/in2]	19.44	Eq. H3-8

Analysis result

Maximum relative deflections

Remark.- Magnitude of deflections in absolute value.

CONDITION **D2=1.2DL+0.5SL**

Member	Defl. (2) [in]		@(%)	Defl. (3) [in]		@(%)
1	0.09183	(L/696)	50.00000	0.08419	(L/760)	50.00000

CONDITION **D3=1.2DL+1.6SL**

Member	Defl. (2) [in]	@(%)	Defl. (3) [in]	@(%)
1	0.09183 (L/696)	50.00000	0.08419 (L/760)	50.00000

CONDITION **D4=1.2DL+0.2SL**

Member	Defl. (2) [in]	@(%)	Defl. (3) [in]	@(%)
1	0.09183 (L/696)	50.00000	0.08419 (L/760)	50.00000

CONDITION **S1=DL**

Member	Defl. (2) [in]	@(%)	Defl. (3) [in]	@(%)
1	0.07653 (L/836)	50.00000	0.07016 (L/912)	50.00000

CONDITION **S2=DL+SL**

Member	Defl. (2) [in]	@(%)	Defl. (3) [in]	@(%)
1	0.07653 (L/836)	50.00000	0.07016 (L/912)	50.00000

CONDITION **S3=DL+0.75SL**

Member	Defl. (2) [in]	@(%)	Defl. (3) [in]	@(%)
1	0.07653 (L/836)	50.00000	0.07016 (L/912)	50.00000

CONDITION **S4=DL+0.75SL**

Member	Defl. (2) [in]	@(%)	Defl. (3) [in]	@(%)
1	0.07653 (L/836)	50.00000	0.07016 (L/912)	50.00000

Deflection < L/600 or 0.3 (OK)

Lintel End Bearing Check

Reference: Technical Notes on Brick Construction 31B Revised, Brick Industry Association, May 1987

	Project				Job Ref.	
AECOM	McDonald's Lillington, NC				032-0312	
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	Header H1 Brick Lintel Check				3-13	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
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TABLE 3
End Reaction[1] and Required Length of Bearing[2] for Structural Angle Lintels

2 1/2" Leg Horizontal				
f'_m	Length of Bearing			
psi	3	4	5	6
400	3000	4000	5000	6000
350	2625	3500	4375	5250
300	2250	3000	3750	4500
275	2063	2750	3438	4125
250	1875	2500	3125	3750
225	1688	2250	2813	3375
215	1613	2150	2688	3225
200	1500	2000	2500	3000
175	1313	1750	2188	2625
160	1200	1600	2000	2400
155	1163	1550	1938	2325
150	1125	1500	1875	2250
140	1050	1400	1750	2100
125	938	1250	1563	1875
115	863	1150	1438	1725
110	825	1100	1375	1650
100	750	1000	1250	1500
85	638	850	1063	1275
75	563	750	938	1125
70	525	700	875	1050

3 1/2" Leg Horizontal				
f'_m	Length of Bearing			
psi	3	4	5	6
400	4200	5600	7000	8400
350	3675	4900	6125	7350
300	3150	4200	5250	6300
275	2888	3850	4813	5775
250	2625	3500	4375	5250
225	2363	3150	3938	4725
215	2258	3010	3763	4515
200	2100	2800	3500	4200
175	1838	2450	3063	3675
160	1680	2240	2800	3360
155	1628	2170	2713	3255
150	1575	2100	2625	3150
140	1470	1960	2450	2940
125	1313	1750	2188	2625
115	1208	1610	2013	2415
110	1155	1540	1925	2310
100	1050	1400	1750	2100
85	893	1190	1488	1785
75	788	1050	1313	1575
70	735	980	1225	1470

[1] End Reaction in lbs.
[2] Length of Bearing in inches.

The reaction of lintel at each end is 0.506 k/ft * 5.33 ft * 0.5 = 1.348 kips (conservatively assume fm is 70 psi)
With 6" end bearing and 3-1/2" horizontal leg, the bearing capacity is 1.47 kips (ASD) > 1.348 kips (OK)

The technical design notes can be found on the link below:
https://www.gobrick.com/docs/default-source/read-research-documents/technicalnotes/31b-structural-steel-lintels.pdf?sfvrsn=0

AECOM	Project				Job Ref.	
	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	Gravity beam B1 Check				3-13	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

GRAVITY BEAM B1 CHECK

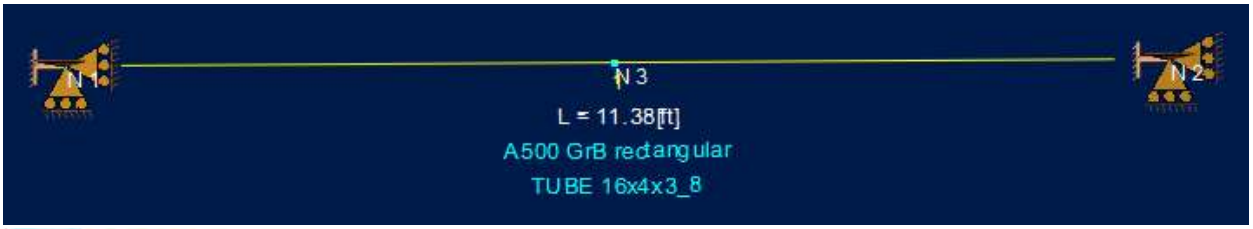
1. Loads from canopy
   DL=3 psf * 3 ft = 9 plf
   SL=15 psf * 3 ft = 45 plf

   Torsion

   W_{snow} = 15 psf * 1 ft = 15 lb/ft
   M_{snow} = 1/2 * 15 lb/ft * (3ft)^2 = 67.5 lb-ft/ft
   T_{snow} = 67.5 lb-ft/ft / 3.74 = 18.05 lb/ft
   W_{dead} = 3 psf * 1 ft = 3 lb/ft
   M_{dead} = 1/2 * 3 lb/ft * (3ft)^2 = 13.5 lb-ft/ft
   T_{dead} = 13.5 lb-ft/ft / 3.74 = 3.61 lb/ft

2. Load from roof
   DL=1/2 * 20 psf * 46.33 ft = 463.3 plf
   SL=1/2 * 15 psf * 46.33 ft + 1/2 * 32.8 psf * 8.216 ft = 482.217 plf
3. Load from exterior wall (note an eccentricity of 12” should be included in calculation)
   DL=40 psf * 11.33 ft = 453.2 plf
4. Load from wind (Taking C&C tributary area 75.8 ft^2 (11.375ft.*13.33ft./2))-
   WL=29.3 psf * (13.33 ft/2) = 0.195 k/ft

   Total dead load acting on the beam B1=0.009k + 0.463k + 0.453=0.926 k/ft
   Total dead load torsion on the beam B1=0.453k/ft * 12in/(12in/ft)=0.453k-ft/ft (RAM Element doesn’t support applying continuous torsion on the beam, conservatively apply concentrated torsion of 0.453k-ft/ft * 13.33 ft.= 6.04 k-ft at mid span)
   Total snow load acting on the beam B1=0.482k/ft + 0.045k/ft = 0.527 k/ft
   Total wind load acting on the beam B1=0.195 k/ft (acting on minor axis)



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AECOM

Current Date: 8/6/2024 10:33 AM
Units system: English
File name: C:\Job\McDonald\Working\Ram Element\Lillington\Gravity beam B1 check.retx

Load data

AECOM	Project: McDonalds Lillington, NC				Job Ref.: 032-0312	
	Section: Gravity beam B1 Check				Sheet no./rev.: 3-14	
	Calc. by: SA/PB	Date: 8/7/2024	Chk'd by: ES	Date: 8/7/2024	App'd by: MY	Date: 8/7/2024

GLOSSARY

Comb : Indicates if load condition is a load combination

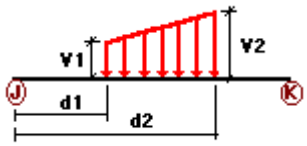
Load Conditions

Condition	Description	Comb.	Category
DL	Dead Load	No	DL
SL	Snow Load	No	SNOW
WL	Wind Load	No	WIND
D1	1.4DL	Yes	
D2	1.2DL+0.5SL	Yes	
D3	1.2DL+1.6SL	Yes	
D4	1.2DL+0.5WL	Yes	
D5	1.2DL+1.6SL+0.5WL	Yes	
D6	1.2DL+WL	Yes	
D7	1.2DL+WL+0.5SL	Yes	
D8	1.2DL+0.2SL	Yes	
D9	0.9DL+WL	Yes	
S1	DL	Yes	
S2	DL+SL	Yes	
S3	DL+0.75SL	Yes	
S4	DL+0.6WL	Yes	
S5	DL+0.75SL	Yes	
S6	0.6DL+0.6WL	Yes	

Load on nodes

Condition	Node	FX [Kip]	FY [Kip]	FZ [Kip]	MX [Kip*ft]	MY [Kip*ft]	MZ [Kip*ft]
DL	3	0.00	0.00	0.00	6.04	0.00	0.00

Distributed force on members



Condition	Member	Dir1	Val1 [Kip/ft]	Val2 [Kip/ft]	Dist1 [ft]	%	Dist2 [ft]	%
DL	1	Y	-0.926	-0.926	0.00	Yes	100.00	Yes
		Z	-0.004	-0.004	0.00	Yes	100.00	Yes
SL	1	Y	-0.527	-0.527	0.00	Yes	100.00	Yes
		Z	-0.018	-0.018	0.00	Yes	100.00	Yes
WL	1	Z	-0.195	-0.195	0.00	Yes	100.00	Yes

Self weight multipliers for load conditions

Condition	Description	Self weight multiplier Comb.	MultX	MultY	MultZ
DL	Dead Load	No	0.00	-1.00	0.00
SL	Snow Load	No	0.00	0.00	0.00
WL	Wind Load	No	0.00	0.00	0.00
D1	1.4DL	Yes	0.00	0.00	0.00
D2	1.2DL+0.5SL	Yes	0.00	0.00	0.00
D3	1.2DL+1.6SL	Yes	0.00	0.00	0.00
D4	1.2DL+0.5WL	Yes	0.00	0.00	0.00
D5	1.2DL+1.6SL+0.5WL	Yes	0.00	0.00	0.00
D6	1.2DL+WL	Yes	0.00	0.00	0.00
D7	1.2DL+WL+0.5SL	Yes	0.00	0.00	0.00
D8	1.2DL+0.2SL	Yes	0.00	0.00	0.00
D9	0.9DL+WL	Yes	0.00	0.00	0.00
S1	DL	Yes	0.00	0.00	0.00
S2	DL+SL	Yes	0.00	0.00	0.00
S3	DL+0.75SL	Yes	0.00	0.00	0.00
S4	DL+0.6WL	Yes	0.00	0.00	0.00
S5	DL+0.75SL	Yes	0.00	0.00	0.00
S6	0.6DL+0.6WL	Yes	0.00	0.00	0.00

Steel Code Check

Report: Concise

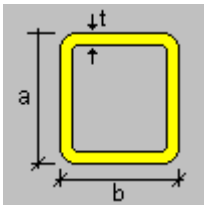
Members: Hot-rolled
Design code: AISC 360-2010 LRFD

Member : 1 (Gravity Beam B1)
Design status : OK

Section information

Section name: TUBE 16x4x3_8 **(US)**

Dimensions



a	=	16.000	[in]	Height
b	=	4.000	[in]	Width

t = 0.375 [in] Thickness

Properties

Section properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	14.093	
Moment of Inertia (local axes) (I)	[in4]	382.781	40.347
Moment of Inertia (principal axes) (I')	[in4]	382.781	40.347
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	5.212	1.692
Radius of gyration (principal axes) (r')	[in]	5.212	1.692
Saint-Venant torsion constant. (J)	[in4]	126.792	
Section warping constant. (Cw)	[in6]	373.035	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	47.848	20.174
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	47.848	20.174
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	47.848	20.174
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	47.848	20.174
Plastic section modulus (local axis) (Z)	[in3]	64.288	23.042
Plastic section modulus (principal axis) (Z')	[in3]	64.288	23.042
Polar radius of gyration. (ro)	[in]	5.479	
Area for shear (Aw)	[in2]	2.719	11.719
Torsional constant. (C)	[in3]	42.272	

Material : A500 GrB rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	46.00
Tensile strength (Fu):	[Kip/in2]	58.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

DESIGN CRITERIA

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	11.38

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
11.38	11.38

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
11.38	11.38	11.38	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None

Major axis frame type : Sway
Minor axis frame type : Sway

DESIGN CHECKS

AXIAL TENSION DESIGN ✔

Axial tension

Ratio : 0.00
Capacity : 583.46 [Kip]
Demand : 0.00 [Kip]
Reference : Eq. Sec. D2
Ctrl Eq. : D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	583.46	Eq. Sec. D2

AXIAL COMPRESSION DESIGN ✔

Compression in the major axis 33

Ratio : 0.00
Capacity : 524.66 [Kip]
Demand : 0.00 [Kip]
Reference : Sec. E1
Ctrl Eq. : D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	524.66	Sec. E1

Compression in the minor axis 22

Ratio : 0.00
Capacity : 363.21 [Kip]
Demand : 0.00 [Kip]
Reference : Sec. E1
Ctrl Eq. : D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	363.21	Sec. E1

FLEXURAL DESIGN ✔

Bending about major axis, M33

Ratio : 0.15
Capacity : 221.79 [Kip*ft]
Demand : 32.54 [Kip*ft]
Reference : Sec. F1
Ctrl Eq. : D3 at 50.00%

Intermediate results	Unit	Value	Reference
Section classification			

Factored yielding strength(ϕM_n):	[Kip*ft]	221.79	Sec. F1

Bending about minor axis, M22

Ratio : 0.05
Capacity : 64.71 [Kip*ft] — Reference : Sec. F1
Demand : -3.38 [Kip*ft] — Ctrl Eq. : D7 at 50.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(ϕM_n):	[Kip*ft]	79.49	Sec. F1
Factored compression flange local buckling strength(ϕM_n):	[Kip*ft]	64.71	Sec. F1

DESIGN FOR SHEAR ✔

Shear in major axis 33

Ratio : 0.02
Capacity : 67.53 [Kip] — Reference : Sec. G1
Demand : 1.19 [Kip] — Ctrl Eq. : D7 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	67.53	Sec. G1

Shear in minor axis 22

Ratio : 0.04
Capacity : 291.09 [Kip] — Reference : Sec. G1
Demand : 11.44 [Kip] — Ctrl Eq. : D3 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	291.09	Sec. G1

TORSION DESIGN ✔

Torsion

Ratio : 0.05
Capacity : 87.50 [Kip*ft] — Reference : Eq. H3-1
Demand : 4.24 [Kip*ft] — Ctrl Eq. : D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	87.50	Eq. H3-1

COMBINED ACTIONS DESIGN ✔

Combined flexure and axial compression

Ratio : 0.18
Ctrl Eq. : D5 at 50.00% Reference : Eq. H1-1b

Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.18	Eq. H1-1b

Combined flexure and axial tension

Ratio : 0.18
Ctrl Eq. : D5 at 50.00% Reference : Eq. H1-1b

Intermediate results	Unit	Value	Reference

Combined flexure and axial compression about local axis

Ratio : N/A
Ctrl Eq. : -- Reference :

Combined flexure and axial tension about local axis

Ratio : N/A
Ctrl Eq. : -- Reference :

Combined torsion, flexure, shear and axial compression

Ratio : N/A
Ctrl Eq. : -- Reference :

Combined torsion, flexure, shear and axial tension

Ratio : N/A
Ctrl Eq. : -- Reference :

Analysis result

Maximum relative deflections

Remark.- Magnitude of deflections in absolute value.

CONDITION **D1=1.4DL**

Member	Defl. (2) [in]		@(%)	Defl. (3) [in]		@(%)
1	0.04828	(L/2827)	50.00000	0.00181	(< L/10000)	50.00000

AECOM	Project: McDonalds Lillington, NC				Job Ref.: 032-0312	
	Section: Gravity beam B1 Check				Sheet no./rev.: 3-20	
	Calc. by: SA/PB	Date: 8/7/2024	Chk'd by: ES	Date: 8/7/2024	App'd by: MY	Date: 8/7/2024

CONDITION **D2=1.2DL+0.5SL**

Member	Defl. (2) [in]		@(%)	Defl. (3) [in]		@(%)
1	0.05071	(L/2692)	50.00000	0.00446	(< L/10000)	50.00000

CONDITION **D3=1.2DL+1.6SL**

Member	Defl. (2) [in]		@(%)	Defl. (3) [in]		@(%)
1	0.07124	(L/1916)	50.00000	0.01087	(< L/10000)	50.00000

CONDITION **D4=1.2DL+0.5WL**

Member	Defl. (2) [in]		@(%)	Defl. (3) [in]		@(%)
1	0.04138	(L/3299)	50.00000	0.03309	(L/4126)	50.00000

CONDITION **D5=1.2DL+1.6SL+0.5WL**

Member	Defl. (2) [in]		@(%)	Defl. (3) [in]		@(%)
1	0.07124	(L/1916)	50.00000	0.04240	(L/3219)	50.00000

CONDITION **D6=1.2DL+WL**

Member	Defl. (2) [in]		@(%)	Defl. (3) [in]		@(%)
1	0.04138	(L/3299)	50.00000	0.06462	(L/2112)	50.00000

CONDITION **D7=1.2DL+WL+0.5SL**

Member	Defl. (2) [in]		@(%)	Defl. (3) [in]		@(%)
1	0.05071	(L/2692)	50.00000	0.06753	(L/2021)	50.00000

CONDITION **D8=1.2DL+0.2SL**

Member	Defl. (2) [in]		@(%)	Defl. (3) [in]		@(%)
1	0.04511	(L/3026)	50.00000	0.00272	(< L/10000)	50.00000

CONDITION **D9=0.9DL+WL**

Member	Defl. (2) [in]		@(%)	Defl. (3) [in]		@(%)
1	0.03103	(L/4398)	50.00000	0.06423	(L/2125)	50.00000

CONDITION **S1=DL**

Member	Defl. (2) [in]		@(%)	Defl. (3) [in]		@(%)
1	0.03448	(L/3958)	50.00000	0.00129	(< L/10000)	50.00000

CONDITION **S2=DL+SL**

Member	**Defl. (2) [in]**		**@(%)**	**Defl. (3) [in]**		**@(%)**
1	0.05315	(L/2568)	50.00000	0.00712	(< L/10000)	50.00000

CONDITION **S3=DL+0.75SL**

Member	**Defl. (2) [in]**		**@(%)**	**Defl. (3) [in]**		**@(%)**
1	0.04848	(L/2816)	50.00000	0.00566	(< L/10000)	50.00000

CONDITION **S4=DL+0.6WL**

Member	**Defl. (2) [in]**		**@(%)**	**Defl. (3) [in]**		**@(%)**
1	0.03448	(L/3958)	50.00000	0.03913	(L/3488)	50.00000

CONDITION **S5=DL+0.75SL**

Member	**Defl. (2) [in]**		**@(%)**	**Defl. (3) [in]**		**@(%)**
1	0.04848	(L/2816)	50.00000	0.00566	(< L/10000)	50.00000

CONDITION **S6=0.6DL+0.6WL**

Member	**Defl. (2) [in]**		**@(%)**	**Defl. (3) [in]**		**@(%)**
1	0.02069	(L/6597)	50.00000	0.03862	(L/3535)	50.00000

Deflections smaller than the smaller of 0.3” and L/600 in each direction, serviceability is not an issue for this beam

	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	Gravity Column C1 Check				3-18	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

GRAVITY COLUMN C1 CHECK

Take gravity column at gridline E/2 as the target column to analyze as it has the largest combined span of beams framed into this column.

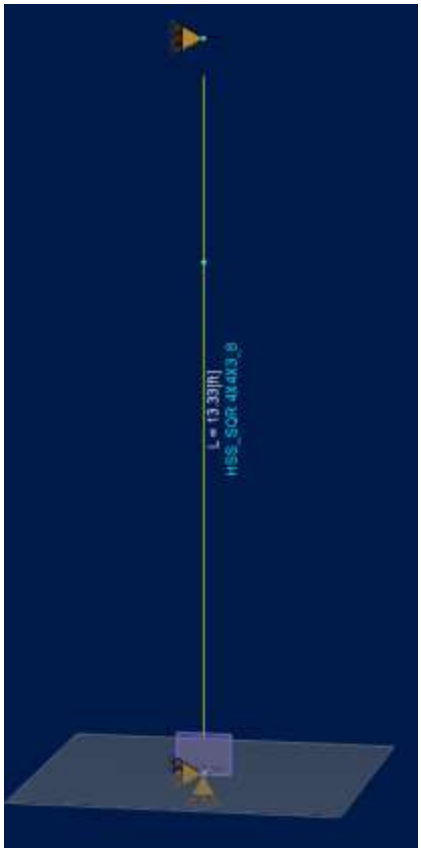
1. Load from Beam B1
   Total span length is: 11.375 ft + 10.00 ft = 21.375 ft
   The tributary span used for column C1 check is ½ * 21.375 ft = 10.688 ft

   Total axial dead load = 0.926 k/ft * 10.688 ft = 9.891 k
   Total axial snow load = 0.527 k/ft * 10.688 ft = 5.635 k

   Torsion $_{\text{dead load}}$ from canopy = 0.0135 kip-ft
   Torsion $_{\text{snow load}}$ from canopy = 0.0675 kip-ft

2. Load from roof
   None (Already included in section 1)

3. Load from wind (Use tributary area for wind C &C = 10.70 ft * 13.33 ft= 143 ft^2 for calculation)
   Use 27.1 psf * 10.70 ft (tributary width) = 0.290 k/ft



RAM® Elements CONNECT Edition

AECOM

Current Date: 8/6/2024 10:40 AM
Units system: English
File name: C:\Job\McDonald\Working\Ram Element\Lillington\Gravity column C1 design with torsion.retx

Load data

GLOSSARY

Comb : Indicates if load condition is a load combination

Load Conditions

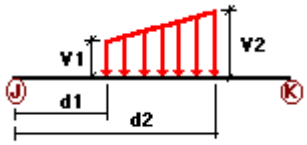
Condition	Description	Comb.	Category
DL	Dead Load	No	DL
SL	Roof Snow Load	No	SNOW
WL	Wind Load	No	WIND
D1	1.4DL	Yes	
D2	1.2DL+0.5SL	Yes	
D3	1.2DL+1.6SL	Yes	
D4	1.2DL+0.5WL	Yes	
D5	1.2DL+1.6SL+0.5WL	Yes	
D6	1.2DL+WL	Yes	
D7	1.2DL+WL+0.5SL	Yes	
D8	1.2DL+0.2SL	Yes	
D9	0.9DL+WL	Yes	
S1	DL	Yes	
S2	DL+SL	Yes	
S3	DL+0.75SL	Yes	
S4	DL+0.6WL	Yes	
S5	DL+0.75SL	Yes	
S6	0.6DL+0.6WL	Yes	

Load on nodes

Condition	Node	FX [Kip]	FY [Kip]	FZ [Kip]	MX [Kip*ft]	MY [Kip*ft]	MZ [Kip*ft]
DL	3	0.00	-9.891	0.00	0.00	0.00	0.00
	4	0.00	0.00	0.00	0.00	0.00	-0.0135
SL	3	0.00	-5.635	0.00	0.00	0.00	0.00
	4	0.00	0.00	0.00	0.00	0.00	-0.0675

Distributed force on members

AECOM	Project: McDonalds Lillington, NC				Job Ref.: 032-0312	
	Section: Gravity Column C1 Check				Sheet no./rev.: 3-20	
	Calc. by: SA/PB	Date: 8/7/2024	Chk'd by: ES	Date: 8/7/2024	App'd by: MY	Date: 8/7/2024



Condition	Member	Dir1	Val1 [Kip/ft]	Val2 [Kip/ft]	Dist1 [ft]	%	Dist2 [ft]	%
WL	1	X	-0.29	-0.29	0.00	Yes	100.00	Yes

Self weight multipliers for load conditions

Condition	Description	Self weight multiplier Comb.	MultX	MultY	MultZ
DL	Dead Load	No	0.00	-1.00	0.00
SL	Roof Snow Load	No	0.00	0.00	0.00
WL	Wind Load	No	0.00	0.00	0.00
D1	1.4DL	Yes	0.00	0.00	0.00
D2	1.2DL+0.5SL	Yes	0.00	0.00	0.00
D3	1.2DL+1.6SL	Yes	0.00	0.00	0.00
D4	1.2DL+0.5WL	Yes	0.00	0.00	0.00
D5	1.2DL+1.6SL+0.5WL	Yes	0.00	0.00	0.00
D6	1.2DL+WL	Yes	0.00	0.00	0.00
D7	1.2DL+WL+0.5SL	Yes	0.00	0.00	0.00
D8	1.2DL+0.2SL	Yes	0.00	0.00	0.00
D9	0.9DL+WL	Yes	0.00	0.00	0.00
S1	DL	Yes	0.00	0.00	0.00
S2	DL+SL	Yes	0.00	0.00	0.00
S3	DL+0.75SL	Yes	0.00	0.00	0.00
S4	DL+0.6WL	Yes	0.00	0.00	0.00
S5	DL+0.75SL	Yes	0.00	0.00	0.00
S6	0.6DL+0.6WL	Yes	0.00	0.00	0.00

Steel Code Check

Report: Concise

Members: Hot-rolled
Design code: AISC 360-2010 LRFD

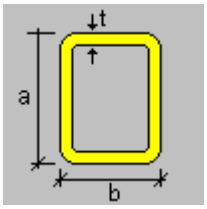
Member : 1 (Gravity column C1)
Design status : OK

Section information

	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	Gravity Column C1 Check				3-21	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

Section name: HSS_SQR 4X4X3_8 **(US)**

Dimensions



a	=	4.000	[in]	Height
b	=	4.000	[in]	Width
T	=	0.349	[in]	Thickness

Properties

Section properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	4.780	
Moment of Inertia (local axes) (I)	[in4]	10.300	10.300
Moment of Inertia (principal axes) (I')	[in4]	10.300	10.300
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	1.468	1.468
Radius of gyration (principal axes) (r')	[in]	1.468	1.468
Saint-Venant torsion constant. (J)	[in4]	17.500	
Section warping constant. (Cw)	[in6]	0.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	5.100	5.100
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	5.100	5.100
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	5.100	5.100
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	5.100	5.100
Plastic section modulus (local axis) (Z)	[in3]	6.400	6.400
Plastic section modulus (principal axis) (Z')	[in3]	6.400	6.400
Polar radius of gyration. (ro)	[in]	2.067	
Area for shear (Aw)	[in2]	2.061	2.061
Torsional constant. (C)	[in3]	9.136	

Material : A500 GrB rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	46.00
Tensile strength (Fu):	[Kip/in2]	58.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

DESIGN CRITERIA

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	13.33

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
13.33	13.33

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
13.33	13.33	13.33	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

DESIGN CHECKS

AXIAL TENSION DESIGN ✔

Axial tension

Ratio : 0.00
Capacity : 197.89 [Kip] — Reference : Eq. Sec. D2
Demand : 0.00 [Kip] — Ctrl Eq. : D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	197.89	Eq. Sec. D2

AXIAL COMPRESSION DESIGN ✔

Compression in the major axis 33

Ratio : 0.24
Capacity : 89.03 [Kip] — Reference : Sec. E1
Demand : 21.14 [Kip] — Ctrl Eq. : D3 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	89.03	Sec. E1

Compression in the minor axis 22

Ratio : 0.24
Capacity : 89.03 [Kip] — Reference : Sec. E1
Demand : 21.14 [Kip] — Ctrl Eq. : D3 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			

Factored flexural buckling strength(ϕP_{n22}):	[Kip]	89.03	Sec. E1

FLEXURAL DESIGN ✔

Bending about major axis, M33

Ratio	:	0.29			
Capacity	:	22.08 [Kip*ft]	Reference	:	Sec. F1
Demand	:	-6.47 [Kip*ft]	Ctrl Eq.	:	D7 at 50.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(ϕM_n):	[Kip*ft]	22.08	Sec. F1

Bending about minor axis, M22

Ratio	:	0.00			
Capacity	:	22.08 [Kip*ft]	Reference	:	Sec. F1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(ϕM_n):	[Kip*ft]	22.08	Sec. F1

DESIGN FOR SHEAR ✔

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	51.20 [Kip]	Reference	:	Sec. G1
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	51.20	Sec. G1

Shear in minor axis 22

Ratio	:	0.04			
Capacity	:	51.20 [Kip]	Reference	:	Sec. G1
Demand	:	-1.94 [Kip]	Ctrl Eq.	:	D7 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	51.20	Sec. G1

TORSION DESIGN ✔

Torsion

Ratio : 0.00
Capacity : 18.91 [Kip*ft]
Demand : 0.00 [Kip*ft]
Reference : Eq. H3-1
Ctrl Eq. : D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	18.91	Eq. H3-1

COMBINED ACTIONS DESIGN ✔

Combined flexure and axial compression

Ratio : 0.38
Ctrl Eq. : D7 at 50.00%
Reference : Eq. H1-1b

Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.38	Eq. H1-1b

Combined flexure and axial tension

Ratio : 0.29
Ctrl Eq. : D7 at 50.00%
Reference : Eq. H1-1b

Intermediate results	Unit	Value	Reference

Combined flexure and axial compression about local axis

Ratio : N/A
Ctrl Eq. : --
Reference :

Combined flexure and axial tension about local axis

Ratio : N/A
Ctrl Eq. : --
Reference :

Combined torsion, flexure, shear and axial compression

Ratio : N/A
Ctrl Eq. : --
Reference :

Combined torsion, flexure, shear and axial tension

Ratio : N/A
Ctrl Eq. : --
Reference :

	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	Gravity Column C1 Check				3-25	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

Analysis result

Maximum relative deflections

Remark.- Magnitude of deflections in absolute value.

CONDITION **S1=DL**

Member	Defl. (2) [in]	@(%)	Defl. (3) [in]	@(%)
1	0.00055 (< L/10000)	50.00000	0.00000 (< L/10000)	0.00000

CONDITION **S2=DL+SL**

Member	Defl. (2) [in]	@(%)	Defl. (3) [in]	@(%)
1	0.00333 (< L/10000)	50.00000	0.00000 (< L/10000)	0.00000

CONDITION **S3=DL+0.75SL**

Member	Defl. (2) [in]	@(%)	Defl. (3) [in]	@(%)
1	0.00264 (< L/10000)	50.00000	0.00000 (< L/10000)	0.00000

CONDITION **S4=DL+0.6WL**

Member	Defl. (2) [in]	@(%)	Defl. (3) [in]	@(%)
1	0.41587 (L/385)	50.00000	0.00000 (< L/10000)	0.00000

CONDITION **S5=DL+0.75SL**

Member	Defl. (2) [in]	@(%)	Defl. (3) [in]	@(%)
1	0.00264 (< L/10000)	50.00000	0.00000 (< L/10000)	0.00000

CONDITION **S6=0.6DL+0.6WL**

Member	Defl. (2) [in]	@(%)	Defl. (3) [in]	@(%)
1	0.41565 (L/385)	50.00000	0.00000 (< L/10000)	0.00000

Deflection smaller than L/240, serviceability is not an issue for this column

GRAVITY COLUMN BASE PLATE DESIGN

See load input from section of "gravity column C1 check"

RAM Elements CONNECT Edition

AECOM

Current Date: 7/31/2024 7:31 PM
Units system: English
File name: C:\Job\McDonald\Working\Ram Element\Lillington\Gravity column C1 design with torsion.retx

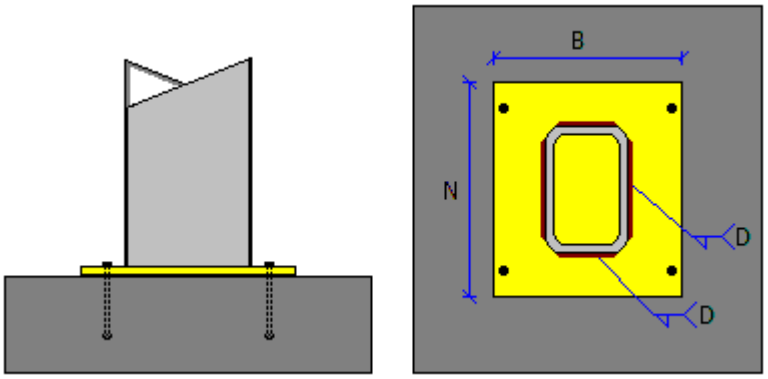
Steel Connections Data

Connection: 1 - Pinned BP

Family: Column - Base (CB)
Type: Base plate
Description: Smart Pinned Base Plate 1

General information

Connector



Members

Column

Column type	:	Prismatic member
Section	:	HSS_SQR 4X4X3_8
Material	:	A500 GrB rectangular
Longitudinal offset	:	0 in

Base plate

Base plate

Plate shape	:	Rectangular
Connection type	:	Unstiffened
Position on the support	:	Center
N: Longitudinal dimension	:	12 in
B: Transversal dimension	:	12 in
Thickness	:	0.75 in
Material	:	A36
Column weld	:	E70XX
D: Column weld size (1/16 in)	:	5
Override A2/A1 ratio	:	No
Include shear lug	:	No

Support

With pedestal	:	Yes
Longitudinal dimension (pedestal)	:	16 in
Transversal dimension (pedestal)	:	16 in
Thickness	:	16 in
Material	:	C 3-60
Include grouting	:	No

Anchor

Anchor position	:	Longitudinal position
Rows number per side	:	1
Anchors per row	:	2
Longitudinal edge distance on the plate	:	2 in
Transverse edge distance on the plate	:	2 in
Anchor type	:	Headed
Head type	:	Square
Include lock nut	:	No
Anchor	:	3/4"
Effective embedment depth	:	9 in
Total length	:	10.74 in
Material	:	F1554 Gr36
Fy	:	36 kip/in2
Fu	:	58 kip/in2
Cracked concrete	:	Yes
Brittle steel	:	No
Anchors welded to base plate	:	No

Anchor reinforcement

Type of reinforcement	:	Supplementary
Tension reinforcement	:	No
Shear reinforcement	:	No

Steel Connections
Results

Connection: 1 - Pinned BP

Family: Column - Base (CB)
Type: Base plate
Description: Smart Pinned Base Plate 1

Design code: AISC 360-10 LRFD, ACI 318-11

Demands

Description	Pu [kip]	Mu22 [kip*ft]	Mu33 [kip*ft]	Vu2 [kip]	Vu3 [kip]	Load type
DL	-10.11	0.00	0.00	0.00	0.00	Design
SL	-5.64	0.00	0.00	0.01	0.00	Design
WL	0.00	0.00	0.00	1.93	0.00	Design
D1	-14.15	0.00	0.00	0.00	0.00	Design
D2	-14.95	0.00	0.00	0.00	0.00	Design
D3	-21.14	0.00	0.00	0.01	0.00	Design
D4	-12.13	0.00	0.00	0.97	0.00	Design
D5	-21.14	0.00	0.00	0.98	0.00	Design
D6	-12.13	0.00	0.00	1.93	0.00	Design
D7	-14.95	0.00	0.00	1.94	0.00	Design
D8	-13.25	0.00	0.00	0.00	0.00	Design
D9	-9.10	0.00	0.00	1.93	0.00	Design

Design calculations

Design for major axis

Base plate (AISC 360-10 LRFD)

Geometric Considerations

Dimensions	Unit	Value	Min.	Max.	Sta.	References
Base plate						
Distance from anchor to edge	[in]	1.62	0.25	--	✔	
Weld size	[1/16in]	5	3	--	✔	table J2.4

Design Check

Verification	Unit	Capacity	Demand	Ctrl EQ	Ratio	References
Pedestal						
Axial bearing	[Kip/in2]	2.21	0.15	D3	0.07	DG1 3.1.1
Base plate						
Flexural yielding (bearing interface)	[Kip*ft/ft]	4.56	1.23	D3	0.27	DG1 Sec 3.1.2
Flexural yielding (tension interface)	[Kip*ft/ft]	4.56	0.00	DL	0.00	DG1 Eq. 3.3.13
Column						
Weld capacity	[Kip/ft]	125.29	0.00	DL	0.00	Sec. J2.4
Elastic method weld shear and axial capacity	[Kip/ft]	125.29	3.93	D7	0.03	Sec. J2.4
Ratio	**0.27**					

Anchors

Geometric Considerations

Dimensions	Unit	Value	Min.	Max.	Sta.	References
Anchors						

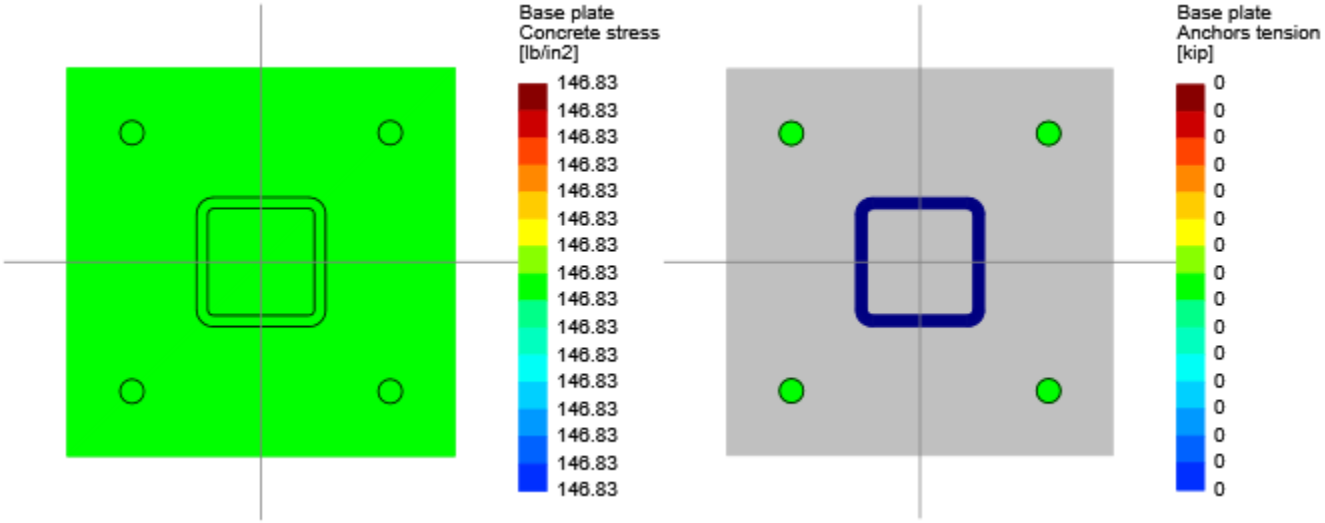
Anchor spacing	[in]	8.00	3.00	--	✔	Sec. D.8.1
Concrete cover	[in]	3.62	2.00	--	✔	Sec. 7.7.1
Effective length	[in]	9.49	--	15.51	✔	

Design Check

Verification	Unit	Capacity	Demand	Ctrl EQ	Ratio	References
Anchor tension	[Kip]	14.55	0.00	DL	0.00	Eq. D-2
Breakout of anchor in tension	[Kip]	5.55	0.00	DL	0.00	Sec. D.3.3.4.4
Pullout of anchor in tension	[Kip]	10.38	0.00	DL	0.00	Sec. D.3.3.4.4
Anchor shear	[Kip]	7.57	0.48	D7	0.06	Eq. D-29
Breakout of anchor in shear	[Kip]	2.07	0.48	D7	0.23	Table D.4.1.1, Sec. D.4.3
Breakout of group of anchors in shear	[Kip]	4.66	1.94	D7	0.42	Table D.4.1.1, Sec. D.4.3
Pryout of anchor in shear	[Kip]	14.81	0.48	D7	0.03	Table D.4.1.1, Sec. D.4.3
Pryout of group of anchors in shear	[Kip]	32.06	1.94	D7	0.06	Table D.4.1.1, Sec. D.4.3
Ratio	**0.42**					
Global critical strength ratio	**0.42**					

Major axis

Maximum compression (D3)



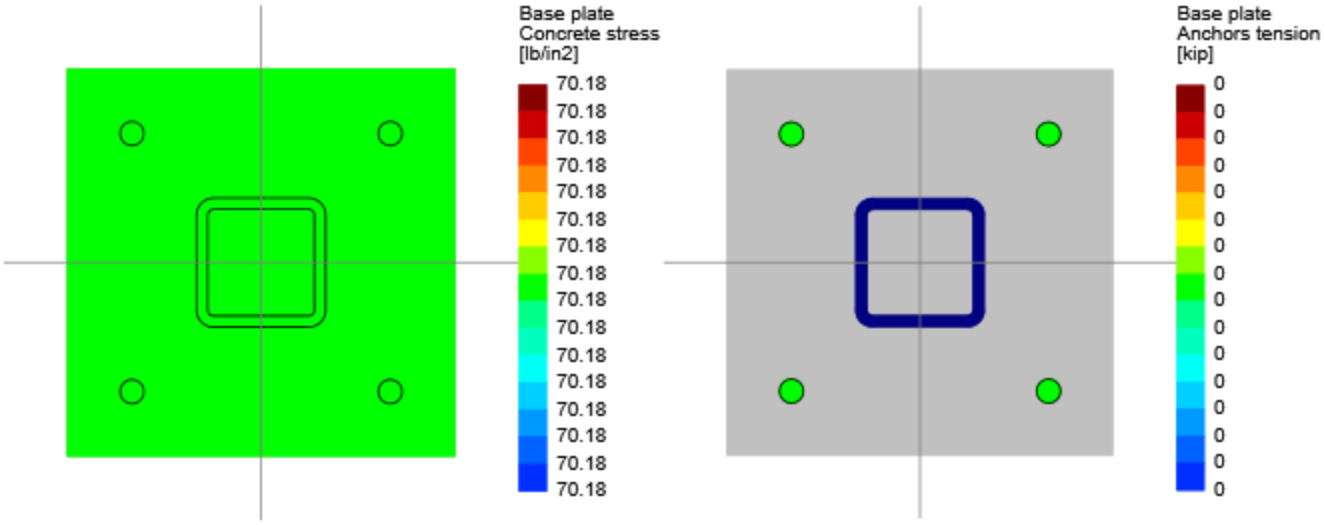
Maximum bearing pressure	146.83	[lb/in2]
Minimum bearing pressure	146.83	[lb/in2]
Maximum anchor tension	0.00	[kip]
Minimum anchor tension	0.00	[kip]
Neutral axis angle	0.00	[deg]
Neutral axis location	1.2E31	[in]

Bearing length	12.00	[in]

Anchors tensions

Anchor	Transverse [in]	Longitudinal [in]	Shear [kip]	Tension [kip]
1	-4.00	-4.00	0.00	0.00
2	-4.00	4.00	0.00	0.00
3	4.00	4.00	0.00	0.00
4	4.00	-4.00	0.00	0.00

Maximum tension (DL)



Maximum bearing pressure	70.18	[lb/in2]
Minimum bearing pressure	70.18	[lb/in2]
Maximum anchor tension	0.00	[kip]
Minimum anchor tension	0.00	[kip]
Neutral axis angle	0.00	[deg]
Neutral axis location	1.2E31	[in]
Bearing length	12.00	[in]

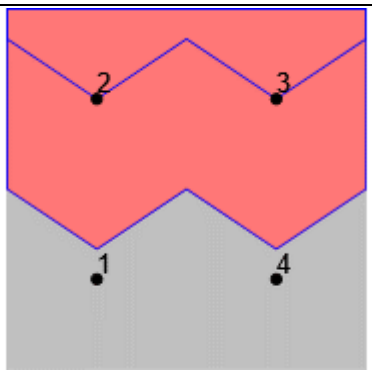
Anchors tensions

Anchor	Transverse [in]	Longitudinal [in]	Shear [kip]	Tension [kip]
1	-4.00	-4.00	0.00	0.00
2	-4.00	4.00	0.00	0.00
3	4.00	4.00	0.00	0.00
4	4.00	-4.00	0.00	0.00

Major axis

Results for shear breakout (D7)

	Project				Job Ref.	
AECOM	McDonald's Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	Base Plate Design for Gravity Columns				3-31	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

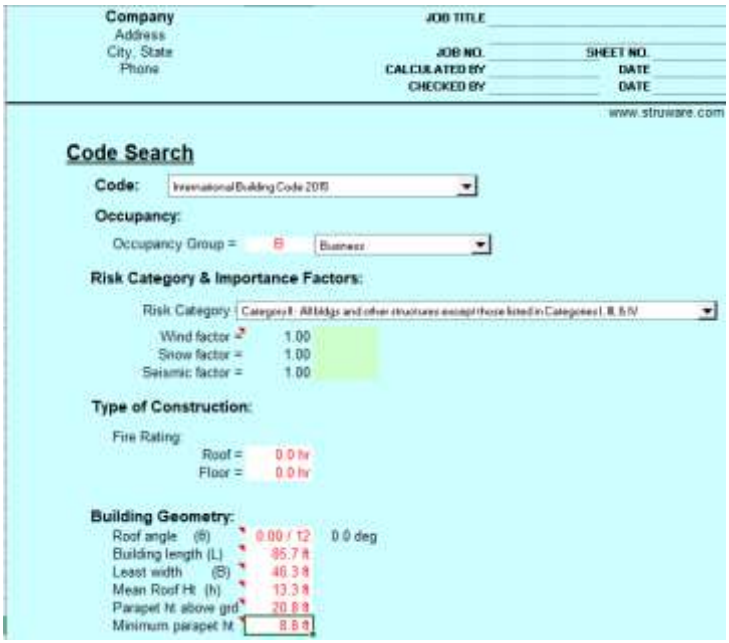


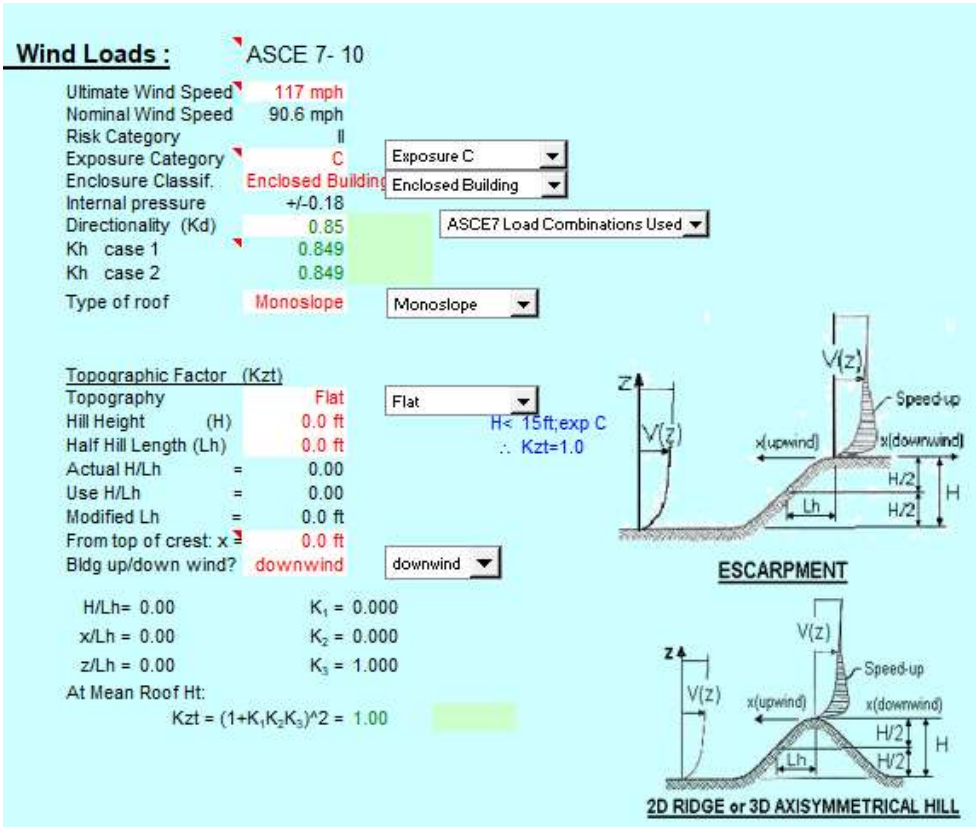
Group	Area [in2]	Shear [kip]	Anchors
1	256.00	1.94	1, 2, 3, 4
2	96.00	0.97	2, 3

	Project			Job Ref.	
AECOM	McDonalds Lillington, NC			032-0312	
	Section			Sheet no./rev.	
	Kicker Check			3-31	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

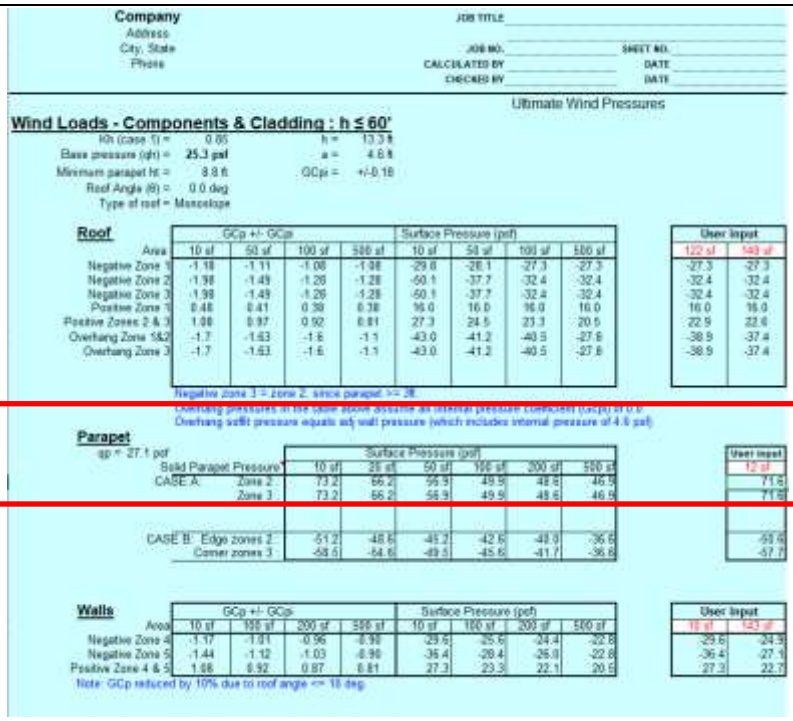
KICKER CHECK

1) Load from wind (use tributary area for parapet wind C&C : 8.795 ft [parapet height]) * 1.33 ft (stud spacing) = 11.69 sq.ft for calculation.





	Project			Job Ref.	
AECOM	McDonalds Lillington, NC			032-0312	
	Section			Sheet no./rev.	
	Kicker Check			3-32	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024



Company
Address
City, State
Phone

JOB TITLE
JOB NO. SHEET NO.
CALCULATED BY DATE
CHECKED BY DATE

Ultimate Wind Pressures

Wind Loads - Components & Cladding : h ≤ 60'

Kh (case 1) = 0.85 h = 13.3 ft
Base pressure (qh) = 25.3 psf a = 4.6 ft
Minimum parapet ht = 3.8 ft GCp = +/-0.18
Roof Angle (θ) = 0.0 deg
Type of roof = Monoslope

Roof

Area	GCp +/- GCp				Surface Pressure (psf)				User Input	
	10 sf	50 sf	100 sf	500 sf	10 sf	50 sf	100 sf	500 sf	122 sf	149 sf
Negative Zone 1	-1.18	-1.11	-1.08	-1.08	-29.8	-28.1	-27.3	-27.3	-27.3	-27.3
Negative Zone 2	-1.98	-1.49	-1.28	-1.28	-50.1	-37.7	-32.4	-32.4	-32.4	-32.4
Negative Zone 3	-1.98	-1.49	-1.28	-1.28	-50.1	-37.7	-32.4	-32.4	-32.4	-32.4
Positive Zone 1	0.48	0.41	0.38	0.38	16.0	16.0	16.0	16.0	16.0	16.0
Positive Zones 2 & 3	1.08	0.97	0.92	0.81	27.3	24.5	23.3	20.5	22.9	22.6
Overhang Zone 1&2	-1.7	-1.63	-1.6	-1.1	-43.0	-41.2	-40.5	-27.8	-38.9	-37.4
Overhang Zone 3	-1.7	-1.63	-1.6	-1.1	-43.0	-41.2	-40.5	-27.8	-38.9	-37.4

Negative zone 3 = zone 2, since parapet >= 3ft
Overhang pressures in the table above assume an internal pressure coefficient (GCpi) of 0.0
Overhang soffit pressure equals adj wall pressure (which includes internal pressure of 4.6 psf)

Parapet

qp = 27.1 psf

		Surface Pressure (psf)						User Input
		10 sf	20 sf	50 sf	100 sf	200 sf	500 sf	12 sf
Solid Parapet Pressure								
CASE A:	Zone 2	73.2	66.2	56.9	49.5	48.6	46.5	71.6
	Zone 3	73.2	66.2	56.9	49.5	48.6	46.5	71.6
CASE B:	Edge zones 2	-51.2	-48.6	-45.2	-42.6	-41.9	-36.6	-50.6
	Corner zones 3	-58.6	-54.6	-49.5	-45.6	-41.7	-36.6	-57.7

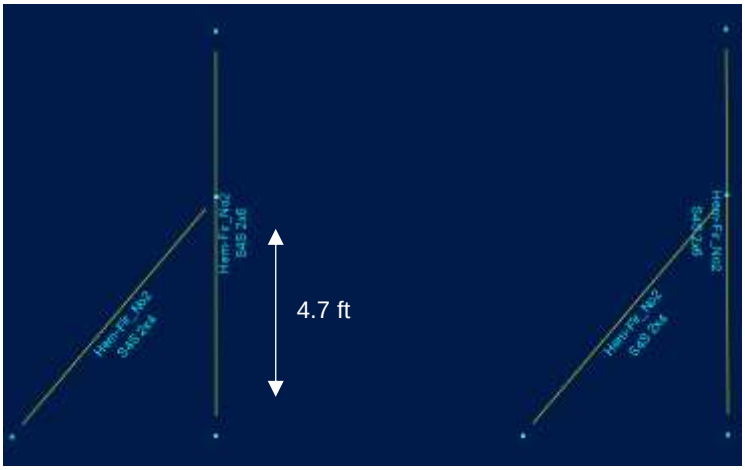
Walls

Area	GCp +/- GCp				Surface Pressure (psf)				User Input	
	10 sf	100 sf	200 sf	500 sf	10 sf	100 sf	200 sf	500 sf	10 sf	143 sf
Negative Zone 4	-1.17	-1.01	-0.96	-0.90	-29.6	-25.6	-24.4	-22.8	-29.6	-24.3
Negative Zone 5	-1.44	-1.12	-1.03	-0.90	-36.4	-28.4	-26.0	-22.8	-36.4	-27.1
Positive Zone 4 & 5	1.08	0.92	0.87	0.81	27.3	23.3	22.1	20.5	27.3	22.7

Note: GCp reduced by 10% due to roof angle <= 10 deg.

From ASCE C&C Wind Design table, for 11.69 sq.ft of tributary area wind load is 71.6 psf. Use 71.6 psf * 1.33 ft = 0.096 k/ft

2) Load from snow = (15 psf + 32.8 psf) * 1.33 ft = 0.064 k/ft



Wind blowing from left to right Wind blowing from right to left

Note- 2X8 brace has used parallel to wooden joist. Conservatively using 2X4 brace for design

RAM® Elements CONNECT Edition

AECOM

Current Date: 8/2/2024 8:41 PM
Units system: English
File name: C:\Job\McDonald\Working\Ram Element\Lillington\WL on kicker joists Case 2_Kicker moved .retx

Load data

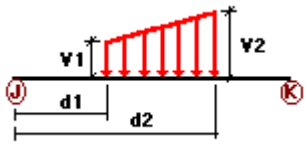
GLOSSARY

Comb : Indicates if load condition is a load combination

Load Conditions

Condition	Description	Comb.	Category
DL	Dead Load	No	DL
WL	Wind Load	No	WIND
SL	Snow Load	No	SNOW
D1	DL	Yes	
D2	DL+SL	Yes	
D3	DL+0.75SL	Yes	
D4	DL+0.6WL	Yes	
D5	DL+0.45WL+0.75SL	Yes	
D6	0.6DL+0.6WL	Yes	
D7	DL+0.75SL	Yes	
S1	DL	Yes	
S2	DL+SL	Yes	
S3	DL+0.75SL	Yes	
S4	DL+0.6WL	Yes	
S5	DL+0.45WL+0.75SL	Yes	
S6	DL+0.75SL	Yes	
S7	0.6DL+0.6WL	Yes	

Distributed force on members



Condition	Member	Dir1	Val1 [Kip/ft]	Val2 [Kip/ft]	Dist1 [ft]	%	Dist2 [ft]	%
WL	3	X	0.096	0.096	0.00	Yes	100.00	Yes
	5	X	-0.096	-0.096	0.00	Yes	100.00	Yes
SL	2	y	-0.064	-0.064	0.00	Yes	100.00	Yes

4	y	-0.064	-0.064	0.00	Yes	100.00	Yes

Self weight multipliers for load conditions

Condition	Description	Self weight multiplier Comb.	MultX	MultY	MultZ
DL	Dead Load	No	0.00	-1.00	0.00
WL	Wind Load	No	0.00	0.00	0.00
SL	Snow Load	No	0.00	0.00	0.00
D1	DL	Yes	0.00	0.00	0.00
D2	DL+SL	Yes	0.00	0.00	0.00
D3	DL+0.75SL	Yes	0.00	0.00	0.00
D4	DL+0.6WL	Yes	0.00	0.00	0.00
D5	DL+0.45WL+0.75SL	Yes	0.00	0.00	0.00
D6	0.6DL+0.6WL	Yes	0.00	0.00	0.00
D7	DL+0.75SL	Yes	0.00	0.00	0.00
S1	DL	Yes	0.00	0.00	0.00
S2	DL+SL	Yes	0.00	0.00	0.00
S3	DL+0.75SL	Yes	0.00	0.00	0.00
S4	DL+0.6WL	Yes	0.00	0.00	0.00
S5	DL+0.45WL+0.75SL	Yes	0.00	0.00	0.00
S6	DL+0.75SL	Yes	0.00	0.00	0.00
S7	0.6DL+0.6WL	Yes	0.00	0.00	0.00

Glossary

Comb : Indicates if load condition is a load combination

Wood Design

Design code: ANSI/AF&PA NDS-2005 ASD

Report: Concise

Member : 2
Design status : OK

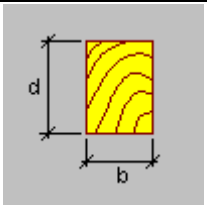
PROPERTIES

Section information

Section name: S4S 2x4 **(US)**

Dimensions

	Project			Job Ref.	
AECOM	McDonalds Lillington, NC			032-0312	
	Section			Sheet no./rev.	
	Kicker Check			3-35	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024



b	=	1.500	[in]	Width
d	=	3.500	[in]	Height

Properties

Section properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	5.250	
Moment of Inertia (principal axes) (I')	[in4]	5.359	0.984
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	3.062	1.312

Material : Hem-Fir_No2

Properties	Value
Type:	Lumber
Species:	Hem-Fir
Grade:	No.2
Coefficient of variation:	0.25

DESIGN CRITERIA

Description	Unit	Value
Temperature:	--	T<=100F
Moisture conditions:	--	Dry
Wood:	--	Unincised
Repetitive member:	--	No
Type:	--	Column
End notches at top:	--	Top
Notch length:	[in]	0.00
Notch depth:	[in]	0.00

Description	Unit	Major axis	Minor axis
Physical length	[ft]	6.17	
Effective length for bending (Le)	[ft]	0.00	
Unbraced length for bending (Lu)	[ft]	6.17	
Unbraced compression length (Lx, Ly)	[ft]	6.17	6.17
Effective length factor (K)	--	1.00	1.00
Lateral bracing	--	No	No
Bearing length (Lb)	[in]	0.50	
Length between inflection points (Li)	[ft]	6.17	

SERVICE CONDITIONS

Verification	Unit	Value	Ctrl EQ	Reference
Deflection in compression and/or bending	--	-0.01	S2 at 45.00%	

	Project			Job Ref.	
AECOM	McDonalds Lillington, NC			032-0312	
	Section			Sheet no./rev.	
	Kicker Check			3-36	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

DESIGN CHECKS

DESIGN FOR TENSION

✔

Ratio	:	0.09			
Capacity	:	1.26 [Kip/in2]	Reference	:	(Sec. 3.8)
Demand	:	0.11 [Kip/in2]	Ctrl Eq.	:	D4 at 100.00%

Intermediate results	Unit	Value	Reference
Axial design value for tension (Ft):	[Kip/in2]	0.53	
Tension axial force (P+):	[Kip]	0.60	

DESIGN FOR COMPRESSION

✔

Ratio	:	0.12			
Capacity	:	0.19 [Kip/in2]	Reference	:	(Sec. 3.6.3)
Demand	:	-0.02 [Kip/in2]	Ctrl Eq.	:	D2 at 0.00%

Intermediate results	Unit	Value	Reference
Axial design value for compression (Fc):	[Kip/in2]	1.30	
Compression axial force (P-):	[Kip]	-0.12	
Modulus of elasticity for stability (Emin):	[Kip/in2]	470.00	
Adjusted modulus of elasticity for stability (Emin'):	[Kip/in2]	574.60	
Critical buckling design value (FcE1):	[Kip/in2]	1.05	(Sec. 3.9.2)
Critical buckling design value (FcE2):	[Kip/in2]	0.19	(Sec. 3.9.2)

DESIGN FOR FLEXURE

✔

Bending about major axis, M33

Ratio	:	0.30			
Capacity	:	1.41 [Kip/in2]	Reference	:	(Sec. 3.3)
Demand	:	0.43 [Kip/in2]	Ctrl Eq.	:	D2 at 100.00%

Intermediate results	Unit	Value	Reference
Bending design value (Fb):	[Kip/in2]	0.85	
Bending moment (Mxx):	[Kip*ft]	-0.11	
Slenderness Ratio (RB):	--	14.56	(Eq. 3.3-5)
Critical buckling design value (FbE):	[Kip/in2]	3.25	(Sec. 3.3.3.8)

Bending about minor axis, M22

Ratio	:	0.00			
Capacity	:	1.26 [Kip/in2]	Reference	:	(Sec. 3.3)
Demand	:	0.00 [Kip/in2]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Bending design value (Fbyy):	[Kip/in2]	0.85	

Bending moment (Myy):	[Kip*ft]	0.00

DESIGN FOR SHEAR ✔

Shear parallel to minor axis, V2

Ratio	:	0.17			
Capacity	:	0.17 [Kip/in2]	Reference	:	(Sec. 3.4)
Demand	:	0.03 [Kip/in2]	Ctrl Eq.	:	D2 at 100.00%

Intermediate results	Unit	Value	Reference
Shear design value (Fv):	[Kip/in2]	0.15	
Shear Force (Vy):	[Kip]	-0.10	
Notch factor (CN):	--	1.00	(Sec. 3.4.3)

Shear parallel to major axis, V3

Ratio	:	0.00			
Capacity	:	0.14 [Kip/in2]	Reference	:	(Sec. 3.4.2)
Demand	:	0.00 [Kip/in2]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Shear design value (Fv):	[Kip/in2]	0.15	
Shear Force (Vy):	[Kip]	0.00	

DESIGN FOR TORSION ✔

Ratio	:	0.00			
Capacity	:	0.09 [Kip/in2]	Reference	:	(AITC-TCM)
Demand	:	0.00 [Kip/in2]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Torsion design value (Fvt):	[Kip/in2]	0.10	
Torsion moment (Mtor):	[Kip*ft]	0.00	

DESIGN FOR BEARING (informative)

Intermediate results	Unit	Value	Reference
Maximum reaction (Rmax):	[Kip]	1.01	(Sec. 3.10.3)
Load angle (θ):	--	0.00	
Axial design value for compression (Fc*):	[Kip/in2]	1.35	
Comp. design value perpendicular to grain (Fcp):	[Kip/in2]	0.41	

INTERACTION ✔

Combined axial and bending interaction value

Ratio : 0.31

Ctrl Eq. : D2 at 100.00%
Reference : (Eq. 3.9-1)

CRITICAL STRENGTH RATIO

Ratio : 0.31
Ctrl Eq. : D2 at 100.00% Reference : (Eq. 3.9-1)

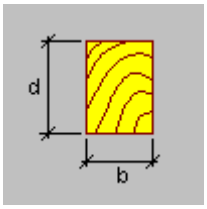
Member : 3
Design status : OK

PROPERTIES

Section information

Section name: S4S 2x6 **(US)**

Dimensions



b	=	1.500	[in]	Width
d	=	5.500	[in]	Height

Properties

Section properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	8.250	
Moment of Inertia (principal axes) (I')	[in4]	20.797	1.547
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	7.562	2.063

Material : Hem-Fir_No2

Properties	Value
Type:	Lumber
Species:	Hem-Fir
Grade:	No.2
Coefficient of variation:	0.25

DESIGN CRITERIA

	Project			Job Ref.	
AECOM	McDonalds Lillington, NC			032-0312	
	Section			Sheet no./rev.	
	Kicker Check			3-39	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

Description	Unit	Value
Temperature:	--	T<=100F
Moisture conditions:	--	Dry
Wood:	--	Unincised
Repetitive member:	--	No
Type:	--	Column
End notches at top:	--	Top
Notch length:	[in]	0.00
Notch depth:	[in]	0.00

Description	Unit	Major axis	Minor axis
Physical length	[ft]	7.96	
Effective length for bending (Le)	[ft]	0.00	
Unbraced length for bending (Lu)	[ft]	7.96	
Unbraced compression length (Lx, Ly)	[ft]	4.10	4.10
Effective length factor (K)	--	1.00	1.00
Lateral bracing	--	No	No
Bearing length (Lb)	[in]	0.50	
Length between inflection points (Li)	[ft]	7.96	

SERVICE CONDITIONS

Verification	Unit	Value	Ctrl EQ	Reference
Deflection in compression and/or bending	--	-0.01	S7 at 100.00%	

DESIGN CHECKS

DESIGN FOR TENSION

✔

Ratio : 0.00
Capacity : 0.61 [Kip/in2]
Demand : 0.00 [Kip/in2]
Reference : (Sec. 3.8)
Ctrl Eq. : D1 at 0.00%

Intermediate results	Unit	Value	Reference
Axial design value for tension (Ft):	[Kip/in2]	0.53	
Tension axial force (P+):	[Kip]	0.00	

DESIGN FOR COMPRESSION

✔

Ratio : 0.17
Capacity : 0.35 [Kip/in2]
Demand : -0.06 [Kip/in2]
Reference : (Sec. 3.6.3)
Ctrl Eq. : D4 at 0.00%

Intermediate results	Unit	Value	Reference
Axial design value for compression (Fc):	[Kip/in2]	1.30	
Compression axial force (P-):	[Kip]	-0.47	
Modulus of elasticity for stability (Emin):	[Kip/in2]	470.00	

Adjusted modulus of elasticity for stability (Emin'):	[Kip/in2]	470.00	
Critical buckling design value (FcE1):	[Kip/in2]	4.83	(Sec. 3.9.2)
Critical buckling design value (FcE2):	[Kip/in2]	0.36	(Sec. 3.9.2)

DESIGN FOR FLEXURE ✔

Bending about major axis, M33

Ratio	:	0.40			
Capacity	:	1.19 [Kip/in2]	Reference	:	(Sec. 3.3)
Demand	:	0.48 [Kip/in2]	Ctrl Eq.	:	D4 at 59.38%

Intermediate results	Unit	Value	Reference
Bending design value (Fb):	[Kip/in2]	0.85	
Bending moment (Mxx):	[Kip*ft]	-0.30	
Slenderness Ratio (RB):	--	20.73	(Eq. 3.3-5)
Critical buckling design value (FbE):	[Kip/in2]	1.31	(Sec. 3.3.3.8)

Bending about minor axis, M22

Ratio	:	0.00			
Capacity	:	1.14 [Kip/in2]	Reference	:	(Sec. 3.3)
Demand	:	0.00 [Kip/in2]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Bending design value (Fbyy):	[Kip/in2]	0.85	
Bending moment (Myy):	[Kip*ft]	0.00	

DESIGN FOR SHEAR ✔

Shear parallel to minor axis, V2

Ratio	:	0.14			
Capacity	:	0.24 [Kip/in2]	Reference	:	(Sec. 3.4)
Demand	:	0.03 [Kip/in2]	Ctrl Eq.	:	D4 at 59.38%

Intermediate results	Unit	Value	Reference
Shear design value (Fv):	[Kip/in2]	0.15	
Shear Force (Vy):	[Kip]	0.19	
Notch factor (CN):	--	1.00	(Sec. 3.4.3)

Shear parallel to major axis, V3

Ratio	:	0.00			
Capacity	:	0.14 [Kip/in2]	Reference	:	(Sec. 3.4.2)
Demand	:	0.00 [Kip/in2]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Shear design value (Fv):	[Kip/in2]	0.15	

Shear Force (Vy):	[Kip]	0.00

DESIGN FOR TORSION ✔

Ratio	:	0.00			
Capacity	:	0.09 [Kip/in2]	Reference	:	(AITC-TCM)
Demand	:	0.00 [Kip/in2]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Torsion design value (Fvt):	[Kip/in2]	0.10	
Torsion moment (Mtor):	[Kip*ft]	0.00	

DESIGN FOR BEARING (informative)

Intermediate results	Unit	Value	Reference
Maximum reaction (Rmax):	[Kip]	0.97	(Sec. 3.10.3)
Load angle (θ):	--	0.00	
Axial design value for compression (Fc*):	[Kip/in2]	1.29	
Comp. design value perpendicular to grain (Fcp):	[Kip/in2]	0.41	

INTERACTION ✔

Combined axial and bending interaction value

Ratio	:	0.40			
			Ctrl Eq.	:	D4 at 59.38%
			Reference	:	(Eq. 3.9-3)

CRITICAL STRENGTH RATIO ✔

Ratio	:	0.40			
Ctrl Eq.	:	D4 at 59.38%	Reference	:	(Eq. 3.9-3)

Member : 4
Design status : OK

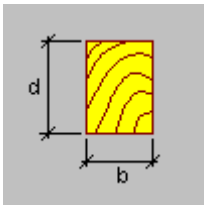
PROPERTIES

Section information

Section name: S4S 2x4 **(US)**

	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	Kicker Check				3-42	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

Dimensions



b	=	1.500	[in]	Width
d	=	3.500	[in]	Height

Properties

Section properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	5.250	
Moment of Inertia (principal axes) (I')	[in4]	5.359	0.984
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	3.062	1.312

Material : Hem-Fir_No2

Properties	Value
Type:	Lumber
Species:	Hem-Fir
Grade:	No.2
Coefficient of variation:	0.25

DESIGN CRITERIA

Description	Unit	Value
Temperature:	--	T<=100F
Moisture conditions:	--	Dry
Wood:	--	Unincised
Repetitive member:	--	No
Type:	--	Column
End notches at top:	--	Top
Notch length:	[in]	0.00
Notch depth:	[in]	0.00

Description	Unit	Major axis	Minor axis
Physical length	[ft]	6.17	
Effective length for bending (Le)	[ft]	0.00	
Unbraced length for bending (Lu)	[ft]	6.17	
Unbraced compression length (Lx, Ly)	[ft]	6.17	6.17
Effective length factor (K)	--	1.00	1.00
Lateral bracing	--	No	No
Bearing length (Lb)	[in]	0.50	
Length between inflection points (Li)	[ft]	6.17	

SERVICE CONDITIONS

Verification	Unit	Value	Ctrl EQ	Reference
Deflection in compression and/or bending	--	-0.01	S2 at 45.00%	

DESIGN CHECKS

DESIGN FOR TENSION

Ratio : 0.02
Capacity : 0.91 [Kip/in2] Reference : (Sec. 3.8)
Demand : 0.02 [Kip/in2] Ctrl Eq. : D2 at 100.00%

Intermediate results	Unit	Value	Reference
Axial design value for tension (Ft):	[Kip/in2]	0.53	
Tension axial force (P+):	[Kip]	0.08	

DESIGN FOR COMPRESSION

Ratio : 0.60
Capacity : 0.19 [Kip/in2] Reference : (Sec. 3.6.3)
Demand : -0.11 [Kip/in2] Ctrl Eq. : D4 at 0.00%

Intermediate results	Unit	Value	Reference
Axial design value for compression (Fc):	[Kip/in2]	1.30	
Compression axial force (P-):	[Kip]	-0.60	
Modulus of elasticity for stability (Emin):	[Kip/in2]	470.00	
Adjusted modulus of elasticity for stability (Emin'):	[Kip/in2]	574.60	
Critical buckling design value (FcE1):	[Kip/in2]	1.05	(Sec. 3.9.2)
Critical buckling design value (FcE2):	[Kip/in2]	0.19	(Sec. 3.9.2)

DESIGN FOR FLEXURE

Bending about major axis, M33

Ratio : 0.30
Capacity : 1.41 [Kip/in2] Reference : (Sec. 3.3)
Demand : 0.43 [Kip/in2] Ctrl Eq. : D2 at 100.00%

Intermediate results	Unit	Value	Reference
Bending design value (Fb):	[Kip/in2]	0.85	
Bending moment (Mxx):	[Kip*ft]	-0.11	
Slenderness Ratio (RB):	--	14.56	(Eq. 3.3-5)
Critical buckling design value (FbE):	[Kip/in2]	3.25	(Sec. 3.3.3.8)

Bending about minor axis, M22

Ratio : 0.00
Capacity : 1.26 [Kip/in2] Reference : (Sec. 3.3)
Demand : 0.00 [Kip/in2] Ctrl Eq. : D1 at 0.00%

Intermediate results	Unit	Value	Reference

Bending design value (Fbyy):	[Kip/in2]	0.85
Bending moment (Myy):	[Kip*ft]	0.00

DESIGN FOR SHEAR ✔

Shear parallel to minor axis, V2

Ratio	:	0.17			
Capacity	:	0.17 [Kip/in2]	Reference	:	(Sec. 3.4)
Demand	:	0.03 [Kip/in2]	Ctrl Eq.	:	D2 at 100.00%

Intermediate results	Unit	Value	Reference
Shear design value (Fv):	[Kip/in2]	0.15	
Shear Force (Vy):	[Kip]	-0.10	
Notch factor (CN):	--	1.00	(Sec. 3.4.3)

Shear parallel to major axis, V3

Ratio	:	0.00			
Capacity	:	0.14 [Kip/in2]	Reference	:	(Sec. 3.4.2)
Demand	:	0.00 [Kip/in2]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Shear design value (Fv):	[Kip/in2]	0.15	
Shear Force (Vy):	[Kip]	0.00	

DESIGN FOR TORSION ✔

Ratio	:	0.00			
Capacity	:	0.09 [Kip/in2]	Reference	:	(AITC-TCM)
Demand	:	0.00 [Kip/in2]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Torsion design value (Fvt):	[Kip/in2]	0.10	
Torsion moment (Mtor):	[Kip*ft]	0.00	

DESIGN FOR BEARING (informative)

Intermediate results	Unit	Value	Reference
Maximum reaction (Rmax):	[Kip]	1.01	(Sec. 3.10.3)
Load angle (θ):	--	0.00	
Axial design value for compression (Fc*):	[Kip/in2]	1.35	
Comp. design value perpendicular to grain (Fcp):	[Kip/in2]	0.41	

INTERACTION ✔

Combined axial and bending interaction value

Ratio : 0.41

Ctrl Eq. : D6 at 100.00%
Reference : (Eq. 3.9-3)

CRITICAL STRENGTH RATIO ✔

Ratio : 0.60
Ctrl Eq. : D4 at 0.00% Reference : (Sec. 3.6.3)

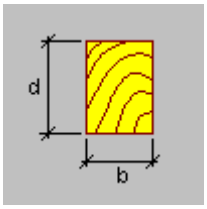
Member : 5
Design status : OK

PROPERTIES

Section information

Section name: S4S 2x6 **(US)**

Dimensions



b	=	1.500	[in]	Width
d	=	5.500	[in]	Height

Properties

Section properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	8.250	
Moment of Inertia (principal axes) (I')	[in4]	20.797	1.547
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	7.562	2.063

Material : Hem-Fir_No2

Properties	Value
Type:	Lumber
Species:	Hem-Fir
Grade:	No.2
Coefficient of variation:	0.25

	Project			Job Ref.	
AECOM	McDonalds Lillington, NC			032-0312	
	Section			Sheet no./rev.	
	Kicker Check			3-46	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

DESIGN CRITERIA

Description	Unit	Value
Temperature:	--	T<=100F
Moisture conditions:	--	Dry
Wood:	--	Unincised
Repetitive member:	--	No
Type:	--	Column
End notches at top:	--	Top
Notch length:	[in]	0.00
Notch depth:	[in]	0.00

Description	Unit	Major axis	Minor axis
Physical length	[ft]	7.96	
Effective length for bending (Le)	[ft]	0.00	
Unbraced length for bending (Lu)	[ft]	7.96	
Unbraced compression length (Lx, Ly)	[ft]	4.10	4.10
Effective length factor (K)	--	1.00	1.00
Lateral bracing	--	No	No
Bearing length (Lb)	[in]	0.50	
Length between inflection points (Li)	[ft]	7.96	

SERVICE CONDITIONS

Verification	Unit	Value	Ctrl EQ	Reference
Deflection in compression and/or bending	--	0.01	S4 at 100.00%	

DESIGN CHECKS

DESIGN FOR TENSION

✔

Ratio : 0.05
Capacity : 1.09 [Kip/in2] Reference : (Sec. 3.8)
Demand : 0.05 [Kip/in2] Ctrl Eq. : D6 at 56.25%

Intermediate results	Unit	Value	Reference
Axial design value for tension (Ft):	[Kip/in2]	0.53	
Tension axial force (P+):	[Kip]	0.45	

DESIGN FOR COMPRESSION

✔

Ratio : 0.05
Capacity : 0.34 [Kip/in2] Reference : (Sec. 3.6.3)
Demand : -0.02 [Kip/in2] Ctrl Eq. : D2 at 0.00%

Intermediate results	Unit	Value	Reference
Axial design value for compression (Fc):	[Kip/in2]	1.30	

Compression axial force (P-):	[Kip]	-0.14	
Modulus of elasticity for stability (Emin):	[Kip/in2]	470.00	
Adjusted modulus of elasticity for stability (Emin'):	[Kip/in2]	470.00	
Critical buckling design value (FcE1):	[Kip/in2]	4.83	(Sec. 3.9.2)
Critical buckling design value (FcE2):	[Kip/in2]	0.36	(Sec. 3.9.2)

DESIGN FOR FLEXURE

Bending about major axis, M33

Ratio : 0.40
Capacity : 1.19 [Kip/in2] Reference : (Sec. 3.3)
Demand : 0.48 [Kip/in2] Ctrl Eq. : D4 at 59.38%

Intermediate results	Unit	Value	Reference
Bending design value (Fb):	[Kip/in2]	0.85	
Bending moment (Mxx):	[Kip*ft]	0.30	
Slenderness Ratio (RB):	--	20.73	(Eq. 3.3-5)
Critical buckling design value (FbE):	[Kip/in2]	1.31	(Sec. 3.3.3.8)

Bending about minor axis, M22

Ratio : 0.00
Capacity : 1.14 [Kip/in2] Reference : (Sec. 3.3)
Demand : 0.00 [Kip/in2] Ctrl Eq. : D1 at 0.00%

Intermediate results	Unit	Value	Reference
Bending design value (Fbyy):	[Kip/in2]	0.85	
Bending moment (Myy):	[Kip*ft]	0.00	

DESIGN FOR SHEAR

Shear parallel to minor axis, V2

Ratio : 0.14
Capacity : 0.24 [Kip/in2] Reference : (Sec. 3.4)
Demand : 0.03 [Kip/in2] Ctrl Eq. : D4 at 59.38%

Intermediate results	Unit	Value	Reference
Shear design value (Fv):	[Kip/in2]	0.15	
Shear Force (Vy):	[Kip]	-0.19	
Notch factor (CN):	--	1.00	(Sec. 3.4.3)

Shear parallel to major axis, V3

Ratio : 0.00
Capacity : 0.14 [Kip/in2] Reference : (Sec. 3.4.2)
Demand : 0.00 [Kip/in2] Ctrl Eq. : D1 at 0.00%

Intermediate results	Unit	Value	Reference

Shear design value (Fv):	[Kip/in2]	0.15
Shear Force (Vy):	[Kip]	0.00

DESIGN FOR TORSION ✔

Ratio	:	0.00			
Capacity	:	0.09 [Kip/in2]	Reference	:	(AITC-TCM)
Demand	:	0.00 [Kip/in2]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Torsion design value (Fvt):	[Kip/in2]	0.10	
Torsion moment (Mtor):	[Kip*ft]	0.00	

DESIGN FOR BEARING (informative)

Intermediate results	Unit	Value	Reference
Maximum reaction (Rmax):	[Kip]	0.97	(Sec. 3.10.3)
Load angle (θ):	--	0.00	
Axial design value for compression (Fc*):	[Kip/in2]	1.29	
Comp. design value perpendicular to grain (Fcp):	[Kip/in2]	0.41	

INTERACTION ✔

Combined axial and bending interaction value

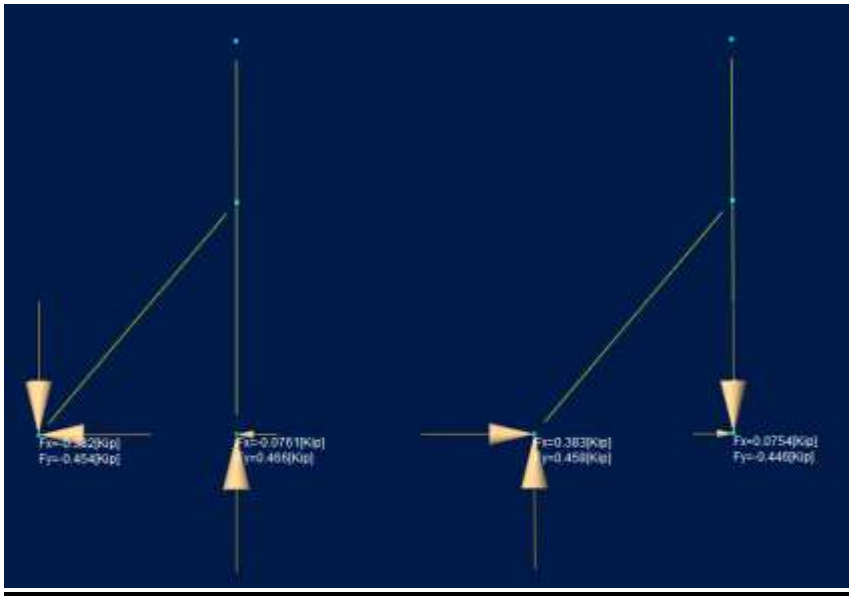
Ratio	:	0.40			
			Ctrl Eq.	:	D4 at 59.38%
			Reference	:	(Eq. 3.9-3)

CRITICAL STRENGTH RATIO ✔

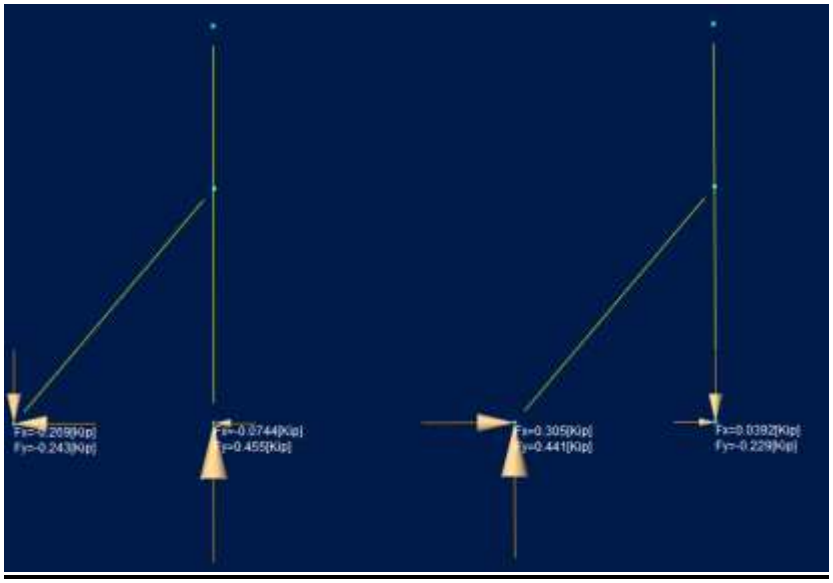
Ratio	:	0.40			
Ctrl Eq.	:	D4 at 59.38%	Reference	:	(Eq. 3.9-3)

	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	Kicker Check				3-49	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

Reaction Summary (0.6D+0.6WL)



Reaction Summary (D+0.45WL+0.75SL)



For design value : The uplift force is 0.458 kip/1.33 ft. (kicker spacing) = 0.350 kip/ft.
Conservatively using 0.400 kip/ft = **400 plf**

AECOM	Project: McDonalds Lillington, NC				Job Ref.: 032-0312	
	Section: Kicker Check				Sheet no./rev.: 3-50	
	Calc. by: SA/PB	Date: 8/7/2024	Chk'd by: ES	Date: 8/7/2024	App'd by: MY	Date: 8/7/2024

CONNECTION CHECK

a) Connection check between kicker to wood plate (rear side):

The uplift force at parapet is 0.458 kip/connection = 458 lb < 615 lb (Simpson H2.5A Tie) **OK**

The shear force at parapet is 0.383 kip/connection = 383 lb < 370 + 110 = 480 lb (Simpson A34 + H2.5A) **OK**

Model No.	Type of Connection	Fasteners (in.)	Direction of Load	DF/SP Allowable Loads			SPF/HF Allowable Loads		
				Floor (100)	Roof (125)	(160)	Floor (100)	Roof (125)	(160)
SS A34	1	(8) 0.131 x 1½	F_1	395	480	545	340	415	480
			F_2[6]	395	430	430	340	370	370
		(8) #9 x 1½" SD	F_1	640	640	640	550	550	550
			F_2	495	495	495	425	425	425
			Uplift	240	240	240	170	170	170

H/TSP

Strong-Tie

Seismic and Hurricane Ties (cont.)

These products are available with additional corrosion protection. For more information, see p. 14. SS For stainless-steel fasteners, see p. 21. SD Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 348–352 for more information.

Model No.	Ga.	Fasteners (in.)			DF/SP Allowable Loads			Uplift with 0.131" x 1½" Nails (160)	SPF/HF Allowable Loads			Uplift with 0.131" x 1½" Nails (160)	Code Ref.
		To Rafters/Truss	To Plates	To Studs	Uplift (160)	Lateral (160) F_1	Lateral (160) F_2		Uplift (160)	Lateral (160) F_1	Lateral (160) F_2		
H1	18	(6) 0.131 x 1½	(4) 0.131 x 2½	—	480	510	190	455	425	440	165	370	IBC, FL, LA
H1.81Z	18	(6) 0.131 x 1½	(4) 0.131 x 2½	—	540	440	170	460	465	380	130	395	—
H2A	18	(5) 0.131 x 1½	(2) 0.131 x 1½	(5) 0.131 x 1½	525	130	55	—	495	130	55	—	IBC, FL, LA
SS H2ASS	18	(5) 0.131 x 1½	(2) 0.131 x 1½	(5) 0.131 x 1½	400	130	55	400	345	130	55	345	—
H2.5A	18	(5) 0.131 x 2½	(5) 0.131 x 2½	—	700	110	110	625	615	110	110	540	IBC, FL, LA
SS H2.5ASS	18	(5) 0.131 x 2½	(5) 0.131 x 2½	—	440	75	70	365	380	75	70	310	—
H2.5T	18	(5) 0.131 x 2½	(5) 0.131 x 2½	—	590	135	145	480	565	135	145	475	IBC, FL, LA
H3	18	(4) 0.131 x 2½	(4) 0.131 x 2½	—	400	210	170	400	365	180	145	290	

b) Connection check between horizontal & vertical wood plate (rear side):

The uplift force at parapet is 0.458 kips/connection = 458 lb < 1360 lb **OK** (W /#14 X 5" long lag screws)

The shear force at parapet is 0.383 kips/connection = 383 lb < 452lb **OK** (W /#14 X 5" long lag screws)

Combined shear & withdrawl = 597 lb < 772 lb **OK** (W /#14 X 5" long lag screws)

AECOM	Project: McDonalds Lillington, NC				Job Ref.: 032-0312	
	Section: Kicker Check				Sheet no./rev.: 3-51	
	Calc. by: SA/PB	Date: 8/7/2024	Chk'd by: ES	Date: 8/7/2024	App'd by: MY	Date: 8/7/2024

URS

Job No.
Sheet No. of
Calculated By Date : #####
Checked By Date :

Screw Connection Design (Mansard Roof Overturning Check)

CONNECTION @ HORIZONTAL & VERTICAL W/ PLATE

Tension (Withdrawal):

Tension Loa = **458** lbs

Use 0.242" DIAM X 5" LONG (#14) WOOD SCREWS

Unit fastener capacity	=	680	lbs
Spacing of the fasteners	=	6	in
Total Quantity of	=	2	

	Capacity			Load		
W'=	1360	lbs	>	458	lbs	**OK**

Shear:

Shear = **383** lbs

Use 0.242" DIAM X 5" LONG (#14) WOOD SCREWS

Unit fastener capacity	=	226	lbs
Spacing of the fasteners	=	6	in
Total Quantity of	=	2	

	Capacity			Load		
W'=	452	lbs	>	383	lbs	**OK**

Combined Shear And Withdrawl

$$Z'_\alpha = \frac{(W'p)Z'}{(W'p)\cos^2(\alpha)+z'\sin^2(\alpha)}$$ NDS 2012

α	52	Degrees	Angle Between Wood And Direction Of Applied Load (exist Truss)
	0.91	Rad	
Z'α	772	Lbs	
Z'α	**772**	**>=**	**597** **OK**

	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	Kicker Check				3-52	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

c) Connection check vertical wood plate & LUS hanger (rear side):

The uplift force at parapet is 0.458 kip /1.33 ft. (parapet spacing) * 3.33 ft (max. joist spacing) / 2 = 0.58 kips

The shear force at parapet is 0.383 kip /1.33 ft. (parapet spacing) * 3.33 ft (max. joist spacing) / 2 = 0.48 kips

Force load per conection = sqrt (580^2 + 480^2) = 752 lb/connection < 950 lb **OK**

Model No.	Fasteners		DF/SP Allowable Loads (lbs)				SPF/HF Allowable Loads (lbs)			
	Joist	Face	Uplift (160)	Floor (100)	Snow (115)	Roof (125)	Uplift (160)	Floor (100)	Snow (115)	Roof (125)
LUS24	(2) SDS212	(4) SDS112	495	670	1000	1085	425	605	695	755
LUS26	(4) SDS212	(4) SDS112	1160	1055	1210	1315	1015	780	875	950
LUS28	(4) SDS212	(6) SDS112	1335	1395	1570	1570	1130	985	1135	1230
LUS210	(4) SDS212	(6) SDS112	1240	1735	1995	2170	1065	1210	1390	1510

d) Connection check between vertical stud to wood plate (front side)

The uplift force is 0.466 kip/connection = 466 lbs < 615 lbs (Simpson H2.5A Tie) **OK**

The shear force is 0.0761 kip/connection = 76 lbs < 370 lbs (Simpson A34) **OK**

H/TSP

Strong-Tie

Seismic and Hurricane Ties (cont.)

These products are available with additional corrosion protection. For more information, see p. 14. SS For stainless-steel fasteners, see p. 21. SD Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 348–352 for more information.

Model No.	Ga.	Fasteners (in.)			DF/SP Allowable Loads			Uplift with 0.131" x 1½" Nails (160)	SPF/HF Allowable Loads			Uplift with 0.131" x 1½" Nails (160)	Code Ref.
		To Rafters/Truss	To Plates	To Studs	Uplift (160)	Lateral (160) F_1	Lateral (160) F_2		Uplift (160)	Lateral (160) F_1	Lateral (160) F_2		
H1	18	(6) 0.131 x 1½	(4) 0.131 x 2½	—	480	510	190	455	425	440	165	370	IBC, FL, LA
H1.81Z	18	(6) 0.131 x 1½	(4) 0.131 x 2½	—	540	440	170	480	465	380	130	395	—
H2A	18	(5) 0.131 x 1½	(2) 0.131 x 1½	(5) 0.131 x 1½	525	130	55	—	495	130	55	—	IBC, FL, LA
H2A-SD	18	(5) 0.131 x 1½	(2) 0.131 x 1½	(5) 0.131 x 1½	400	130	55	400	345	130	55	345	—
H2.5A	18	(5) 0.131 x 2½	(5) 0.131 x 2½	—	700	110	110	625	615	110	110	540	IBC, FL, LA
H2.5A-SS	18	(5) 0.131 x 2½	(5) 0.131 x 2½	—	440	75	70	365	380	75	70	310	—
H2.5T	18	(5) 0.131 x 2½	(5) 0.131 x 2½	—	590	135	145	480	565	135	145	475	IBC, FL, LA
H3	18	(4) 0.131 x 2½	(4) 0.131 x 2½	—	480	210	170	400	365	180	145	290	

Model No.	Type of Connection	Fasteners (in.)	Direction of Load	DF/SP Allowable Loads			SPF/HF Allowable Loads		
				Floor (100)	Roof (125)	(160)	Floor (100)	Roof (125)	(160)
		(8) 0.131 x 1½	F_1	395	480	545	340	415	480
			F_2[6]	395	430	430	340	370	370
SS A34	1	(8) #9 x 1½" SD	F_1	640	640	640	550	550	550
			F_2	495	495	495	425	425	425
			Uplift	240	240	240	170	170	170

e) Connection check between lag screws to joist top chord (front side):

The uplift force is 0.466 kip/connection = 466 lbs * 40" / 16" = 1165 lbs < 1472 lbs

OK (W /(4)-1/4" X 3" lag screws)

	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	Kicker Check				3-53	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

The shear force is 0.076 kip/connection = 76 lbs * 40” /16” = 190 lbs < 908 lbs

OK (W /(4)-1/4” X 3” lag screws)

Combined shear & withdrawl = 1180 lbs < 1406 lbs **OK**

Note- Simpson Hurricane (H)8 has been used parallel to wood joist for kicker connection. Conservatively H2.5 design values are used in the calculation before.

Model No.	Ga.	Fasteners (in.)			DF/SP Allowable Loads			Uplift with 0.131" x 1½" Nails (160)	SPF/HF Allowable Loads			Uplift with 0.131" x 1½" Nails (160)	Code Ref.
		To Rafters/Truss	To Plates	To Studs	Uplift (160)	Lateral (160) F_1	Lateral (160) F_2		Uplift (160)	Lateral (160) F_1	Lateral (160) F_2		
H1A	18	(4) 0.131 x 1½	(4) 0.131 x 1½	—	545	420	265	—	470	360	205	—	IBC®, FL, LA
H1.81Z	18	(6) 0.131 x 1½	(4) 0.131 x 2½	—	540	440	170	460	465	380	130	395	—
H2A	18	(5) 0.131 x 1½	(2) 0.131 x 1½	(5) 0.131 x 1½	525	130	55	—	495	130	55	—	IBC, FL, LA
H2ASS	18	(5) 0.131 x 1½	(2) 0.131 x 1½	(5) 0.131 x 1½	400	130	55	400	345	130	55	345	—
H2.5A	18	(5) 0.131 x 2½	(5) 0.131 x 2½	—	700	110	110	625	615	110	110	540	IBC, FL, LA
H2.5ASS	18	(5) 0.131 x 2½	(5) 0.131 x 2½	—	440	75	70	365	380	75	70	310	—
H2.5T	18	(5) 0.131 x 2½	(5) 0.131 x 2½	—	590	135	145	480	565	135	145	475	IBC, FL, LA
H3	18	(4) 0.131 x 2½	(4) 0.131 x 2½	—	400	210	170	400	365	180	145	290	
H3SS	18	(4) 0.131 x 2½	(4) 0.131 x 2½	—	280	145	120	275	225	100	85	210	—
H6 (to Plates)	16	—	(8) 0.131 x 2½	(8) 0.131 x 2½	930	—	—	—	800	—	—	—	IBC, FL, LA
H6 (to Rim)	16	(8) 0.131 x 2½	—	(8) 0.131 x 2½	1,230	—	—	—	1,065	—	—	—	
H7Z	16	(4) 0.131 x 2½	(2) 0.131 x 1½	(8) 0.131 x 2½	830	410	—	—	715	355	—	—	
H8	18	(5) 0.148 x 1½	(5) 0.148 x 1½	—	780	95	90	630	710	95	90	510	

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URS

Job No. 015-1200 New Market, MD
Sheet No. of
Calculated By TL Date : ########
Checked By Date :

Screw Connection Design (Nonsaed Roof Overturning Check)

CONNECTION @ JOIST TOP CHORD (FRONT SIDE)

Tension (Withdrawal):

Tension Loa = **1330** lbs

Use (4)-1/4" x 3" LAG SCREWS

Unit fastener capacity	=	368	lbs
Spacing of the fasteners	=	40	in
Total Quantity of Fasteners	=	4	

	Capacity			Load		
W'=	1472	lbs	>	1330	lbs	**OK**

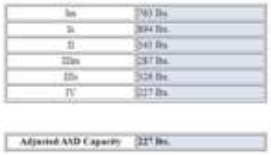
Shear:

Shear = **127.5** lbs

Use (4)-1/4" x 3" LAG SCREWS

Unit fastener capacity	=	227	lbs
Spacing of the fasteners	=	40	in
Total Quantity of Fatseners	=	4	

	Capacity			Load		
W'=	908	lbs	>	127.5	lbs	**OK**



Combined Shear And Withdrawl

$$Z'\alpha = \frac{(W'p)Z'}{(W'p)\cos 2(\alpha)+z'\sin^2(\alpha)}$$ NDS 2012

α	74	Degrees	Angle Between Wood And Direction Of Applied Load (exist Truss)
	1.29	Rad	
Z'α	1406	Lbs	
Z'α	**1406**	**>=**	**1336** **OK**

	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	Roof Diaphragm Check				3-46	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

SHEATHING LATERAL CAPACITY CHECK

1. Wind Load Normal to Short Direction (Case 3)

Wind load (short direction) = 0.6 * 0.5 * 33.19 kips / 85.66 ft = 116.24 plf

For Hem-Fir species per NDS 2018 Table 12.3.3A, the specific gravity G = 0.43, then Factor = (1 – (0.5-G)) = 0.93

From Table 4.2C, Case 3, the allowable unit shear capacity (ASD) = 600 plf / 2 * G = 300 * 0.93 = 279 plf > 116.4 plf **OK**

2. Wind Load Normal to Long Direction (Case 1)

Wind load (long direction) = 0.6 * 0.31 * 63.43 kips / 46.33 ft = 254.65 plf

From Table 4.2C, Case 1, the allowable unit shear capacity (ASD) = 800 plf / 2 * G = 400 * 0.93 = 372 plf > 254.65 plf **OK**

3. Seismic Load Normal to Short Direction (Case 3)

Seismic load (short direction) = 0.6 * 0.5 *14.24 kips / 85.66 ft = 49.87 plf

From Table 4.2C, Case 3, the allowable unit shear capacity (ASD) = 430 plf / 2 * G = 215 * 0.93 = 200 plf > 49.87 plf **OK**

4. Seismic Load Normal to Long Direction (Case 1)

Seismic load (long direction) = 0.6 * 0.31 * 28.49 kips / 46.33 ft = 114.38 plf

From Table 4.2C, Case 1, the allowable unit shear capacity (ASD) = 570 plf / 2 * G = 285 * 0.93 = 265 plf > 114.38 plf **OK**

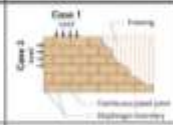
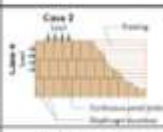
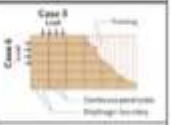
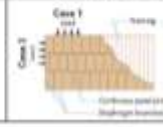
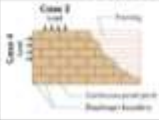
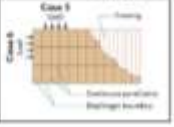
AECOM	Project: McDonalds Lillington, NC				Job Ref.: 032-0312	
	Section: Roof Diaphragm Check				Sheet no./rev.: 3-47	
	Calc. by: SA/PB	Date: 8/7/2024	Chk'd by: ES	Date: 8/7/2024	App'd by: MY	Date: 8/7/2024

Table 4.2C Nominal Unit Shear Capacities for Wood-Frame Diaphragms

Unblocked Wood Structural Panel Diaphragms[1,2,3,4,5]

Sheathing Grade	Common Nail Size	Minimum Fastener Penetration in Framing (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal Width of Nailed Face at Supported Edges and Boundaries (in.)	A SEISMIC: 6 in. Nail Spacing at diaphragm boundaries and supported panel edges — Case 1: v_s (plf)	Case 1: G_a (kips/in.) OSB	Case 1: G_a (kips/in.) PLY	Cases 2,3,4,5,6: v_s (plf)	Cases 2,3,4,5,6: G_a (kips/in.) OSB	Cases 2,3,4,5,6: G_a (kips/in.) PLY	B WIND: 6 in. Nail Spacing at diaphragm boundaries and supported panel edges — Case 1: v_w (plf)	Cases 2,3,4,5,6: v_w (plf)
Structural I	6d	1-1/4	5/16	2	330	9.0	7.0	250	6.0	4.5	460	350
				3	370	7.0	6.0	280	4.5	4.0	520	390
	8d	1-3/8	3/8	2	480	8.5	7.0	360	6.0	4.5	670	505
				3	530	7.5	6.0	400	5.0	4.0	740	560
	10d	1-1/2	15/32	2	570	14	10	430	9.5	7.0	800	600
				3	640	12	9.0	480	8.0	6.0	895	670
Sheathing and Single-Floor	6d	1-1/4	5/16	2	300	9.0	6.5	220	6.0	4.0	420	310
				3	340	7.0	5.5	250	5.0	3.5	475	350
			3/8	2	330	7.5	5.5	250	5.0	4.0	460	350
				3	370	6.0	4.5	280	4.0	3.0	520	390
	8d	1-3/8	3/8	2	430	9.0	6.5	320	6.0	4.5	600	450
				3	480	7.5	5.5	360	5.0	3.5	670	505
			7/16	2	460	8.5	6.0	340	5.5	4.0	645	475
				3	510	7.0	5.5	380	4.5	3.5	715	530
			15/32	2	480	7.5	5.5	360	5.0	4.0	670	505
				3	530	6.5	5.0	400	4.0	3.5	740	560
	10d	1-1/2	15/32	2	510	15	9.0	380	10	6.0	715	530
				3	580	12	8.0	430	8.0	5.5	810	600
			19/32	2	570	13	8.5	430	8.5	5.5	800	600
				3	640	10	7.5	480	7.0	5.0	895	670

1. Nominal unit shear capacities shall be adjusted in accordance with 4.2.3 to determine ASD allowable unit shear capacity and LRFD factored unit resistance. For general construction requirements see 4.2.6. For specific requirements, see 4.2.7.1 for wood structural panel diaphragms. See Appendix A for common nail dimensions.
2. For species and grades of framing other than Douglas-Fir-Larch or Southern Pine, reduced nominal unit shear capacities shall be determined by multiplying the tabulated nominal unit shear capacity by the Specific Gravity Adjustment Factor = [1-(0.5-G)], where G = Specific Gravity of the framing lumber from the *NDS* (Table 12.3.3A). The Specific Gravity Adjustment Factor shall not be greater than 1.
3. Apparent shear stiffness values, G_a, are based on nail slip in framing with moisture content less than or equal to 19% at time of fabrication and panel stiffness values for diaphragms constructed with either OSB or 3-ply plywood panels. When 4-ply or 5-ply plywood panels or composite panels are used, G_a values shall be permitted to be multiplied by 1.2.
4. Where moisture content of the framing is greater than 19% at time of fabrication, G_a values shall be multiplied by 0.5.
5. Diaphragm resistance depends on the direction of continuous panel joints with respect to the loading direction and direction of framing members, and is independent of the panel orientation.

	Cases 1&3: Continuous Panel Joints Perpendicular to Framing	**Cases 2&4**: Continuous Panel Joints Parallel to Framing	**Cases 5&6**: Continuous Panel Joints Perpendicular and Parallel to Framing
Long Panel Direction Perpendicular to Supports			
Long Panel Direction Parallel to Supports[a]			

[a] Panel span rating for out-of-plane loads may be lower than the span rating with the long panel direction perpendicular to supports (See Section 3.2.2 and Section 3.2.3)

AECOM	Project: McDonalds Lillington, NC				Job Ref.: 032-0312	
	Section: Roof Diaphragm Check				Sheet no./rev.: 3-48	
	Calc. by: SA/PB	Date: 8/7/2024	Chk'd by: ES	Date: 8/7/2024	App'd by: MY	Date: 8/7/2024

SHEATHING GRAVITY CAPACITY CHECK

1. Dead load

W_{dead} = 20 psf

2. Roof Live load

W_{live} = 20 psf

3. Snow load

Snow load = 15 psf + 32.80 psf = 47.80 psf

4. Uplift/downward wind pressure = 32.4 psf (per C&C WL using joist tributary area of 2.67 ft * 46.33 ft = 124 ft^2)

Load Combinations ASD

Combo 1 = D + S = 20 psf + 47.80 psf = 67.80 psf

Combo 2 = D ± 0.6W = 20 psf + 0.6 * 32.4 psf = 39.44 psf

Combo 3 = D + 0.75L + 0.75(0.6*W) + 0.75S = 20 psf + 0.75 * 20 + 0.75*0.6* 32.4 psf + 0.75*47.80 psf= 85.43 psf

Combo 4 = 0.6D + 0.6W = 0.6* 20 psf + 0.6* 32.4 psf = 31.44 psf

Maximum typical joist clear span = 2'-8" = 32"

From SDPWS Special Design Provisions for Wind and Seismic Code, for 48/24 sheathing and clear span 32", the Maximum Allowable Uniform Load = 220 psf / 1.6 = 137.5 psf, then 85.43 psf < 137.5 psf, **OK**

Table 3.2.2 Nominal Uniform Load Capacities (psf) for Roof Sheathing Resisting Out-of-Plane Wind Loads[1,2,6]

Sheathing Type[5]	Span Rating or Grade	Minimum Thickness (in.)	Strength Axis[7] Applied Perpendicular to Supports — Rafter/Truss Spacing (in.) — Nominal Uniform Loads (psf)						Strength Axis[7] Applied Parallel to Supports — Rafter/Truss Spacing (in.) — Nominal Uniform Loads (psf)		
			12	16	19.2	24	32	48	12	16	24
Wood Structural Panels (Sheathing Grades, C-C, C-D, C-C Plugged, OSB)	24/0	3/8	425	240	165	105	-	-	90	50	30[3]
	24/16	7/16	540	305	210	135	-	-	110	60	35[3]
	32/16	15/32	625	355	245	155	90	-	155	90	45[3]
	40/20	19/32	955	595	415	265	150	-	255	145	75[3]
	48/24	23/32	1160[3]	840[3]	615[3]	395[3]	220[3]	100[3]	455[3]	255[3]	115[3]
Wood Structural Panels (Single Floor Grades, Underlayment, C-C Plugged)	16 o.c.	19/32	705	395	275	175	100	-	170	95	50[3]
	20 o.c.	19/32	815	455	320	205	115	-	235	135	70[3]
	24 o.c.	23/32	1160[3]	670[3]	465[3]	300[3]	170[3]	-	440[3]	250[3]	110[3]
	32 o.c.	7/8	1395[4]	1000[4]	695[4]	445[4]	250[4]	110[4]	1160[4]	655[4]	290[4]
	48 o.c.	1-1/8	1790[4]	1295[4]	1060[4]	805[4]	455[4]	200[4]	1790[4]	1145[4]	510[4]

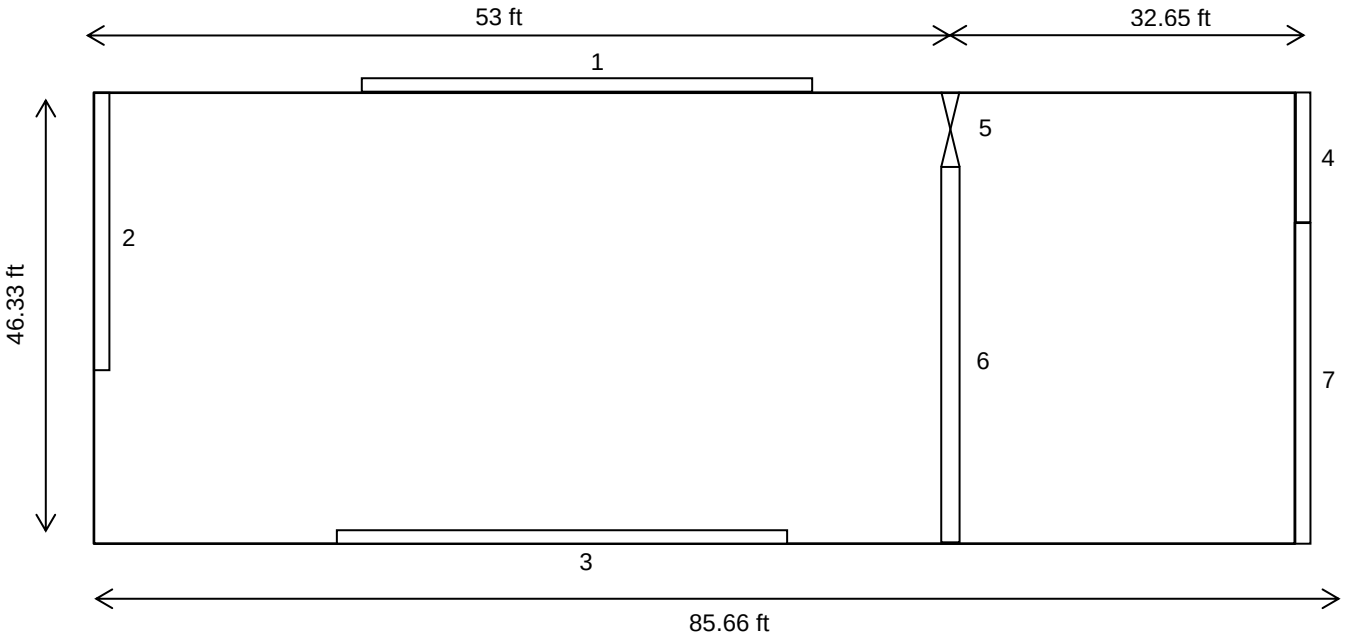
1. Nominal capacities shall be adjusted in accordance with Section 3.2.3 to determine ASD uniform load capacity and LRFD uniform resistances.
2. Unless otherwise noted, tabulated values are based on the lesser of nominal values for either OSB or plywood with 3 or more plies.
3. Tabulated values are based on the lesser of nominal values for either OSB or plywood with 4 or more plies.
4. Tabulated values are based on the lesser of nominal values for either OSB or plywood with 5 or more plies.
5. Wood structural panels shall conform to the requirements for its type in DOC PS 1 or PS 2.
6. Tabulated values are for maximum bending loads from wind. Loads are limited by bending or shear stress assuming a 2-span continuous condition. Where panels are continuous over 3 or more spans, the tabulated values shall be permitted to be increased in accordance with the *ASD/LRFD Manual for Engineered Wood Construction*.
7. Strength axis is defined as the axis parallel to the face and back orientation of the flakes or the grain (veneer), which is generally the long panel direction, unless otherwise marked.

4. Lateral Load System

	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	Lateral System Diagram				4-1	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

LATERAL SYSTEM DIAGRAM FOR BUILDING

Diagram Number	Lateral Member (Main Building)	Percent of Shear Load
1	Shear Wall A	46.5 in X direction
2	Shear Wall B	30.94 in Y direction
3	Shear Wall C	53.5 in X direction
4	Braced Frame 2	19.06 in Y direction
5	Braced Frame 1	50 in Y direction
6	Drag Strut for BF1	---
7	Drag Strut for BF2	---



*not drawn to scale

	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	Braced Frame 1, Base Plate & Anchor Bolts Design				4-2	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

BRACED FRAME-BF1

Shear Force from wind V_{wy}= 63.43 kips

Shear From Seismic V_{Sy}= 28.49 kips

Lateral Loads on BF1 From Wind $V_w = 0.50 \times V_{wy}$ = 31.72 kips

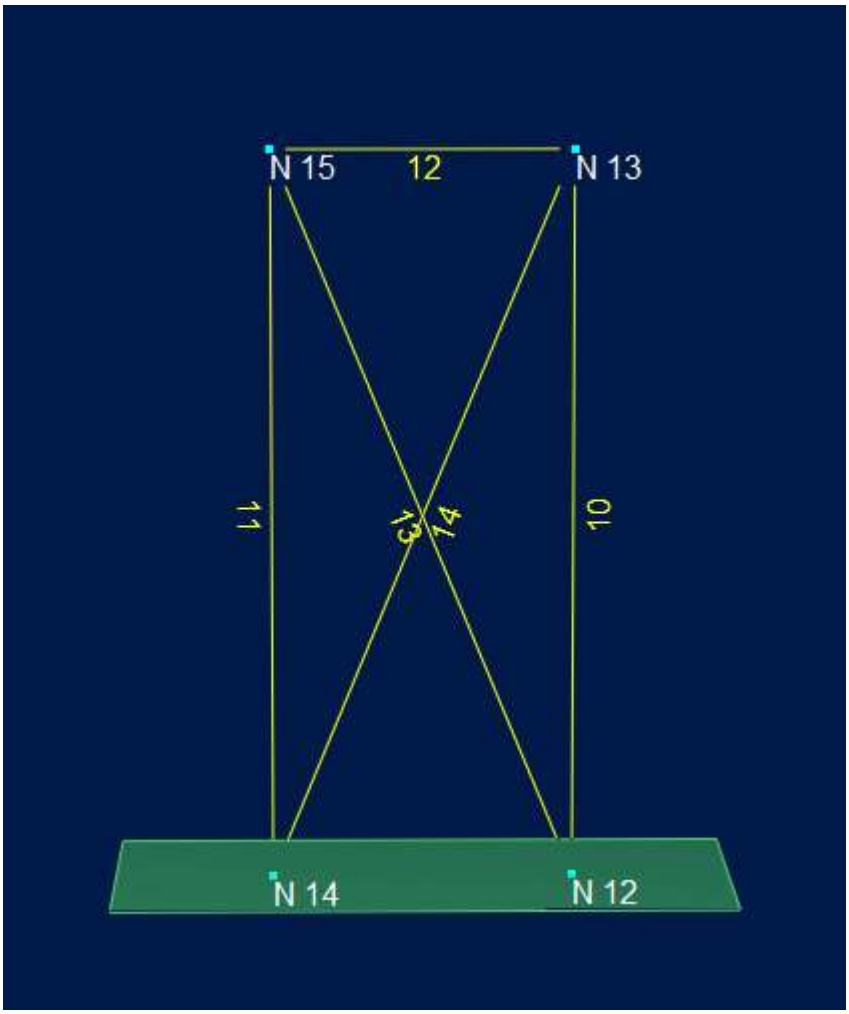
Lateral Loads on BF1 From Seismic $V_s = 0.50 \times V_{Sy}$ = 14.25 kips

ρ=1.0 (Per ASCE 12.3.4, redundancy factor ρ=1.0)

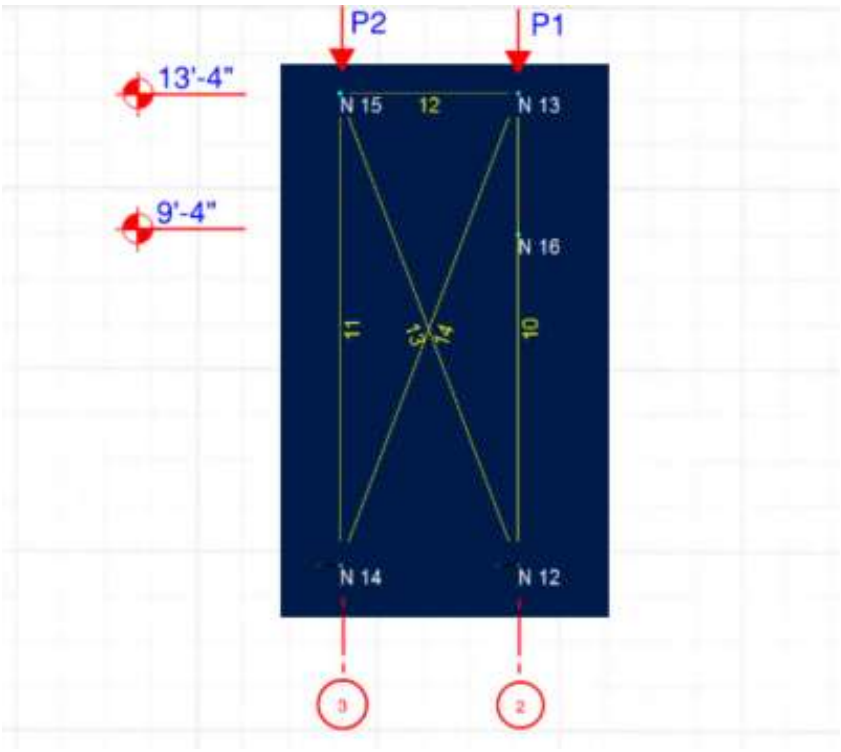
Ω=1.0

Ev (vertical seismic force) needs to be included since Sds=0.302 > 0.125 per ASCE (Eq.12.4-4)

Gravity load (roof dead load, and roof snow loads are included in the “Load Data” section of the RAM Element design report)



	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	Braced Frame 1, Base Plate & Anchor Bolts Design				4-3	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024



DL on col B/3 at 13'4"	=	20 psf * 2.67 ft (tributary width) * 46.33 ft * 0.5 = 1.24 kips
SL on col B/3 at 13'4"	=	15 psf * 2.67 ft (tributary width) * 46.33 ft *0.5 = 0.93 kips
DL on col B/2 at 9'4"	=	0.926 k/ft * 10.5 ft * 0.5 = 4.86 kips
SL on col B/2 at 9'4"	=	0.527 k/ft * 10.5 ft * 0.5 = 2.77 kips

For convenience apply all the DL & SL at 13'4"
VERTICAL SEISMIC LOAD (12.4.2.2 OF ASCE 7-10)
Evh = 0.2 * Sds * DL = 0.2 * 0.302 * DL = 0.06 * DL
(Apply vertical sesmic load where DL applies)

RAM® Elements CONNECT Edition

AECOM

Current Date: 8/6/2024 11:40 AM
Units system: English
File name: C:\Job\McDonald\Working\Ram Element\Lillington\BF1 main model and footing .retx

Load data

GLOSSARY

Comb : Indicates if load condition is a load combination

Load Conditions

Condition	Description	Comb.	Category
DL	Dead Load	No	DL
SL	Snow Load	No	SNOW
WL1	Wind Load 1	No	WIND
WL2	Wind Load 2	No	WIND
EQ1	Horizontal Seismic Load 1	No	EQ
EQ2	Horizontal Seismic Load 2	No	EQ
D1	1.4DL	Yes	
D2	1.2DL+0.5SL	Yes	
D3	1.2DL+1.6SL	Yes	
D4	1.2DL+0.5WL1	Yes	
D5	1.2DL+0.5WL2	Yes	
D6	1.2DL+1.6SL+0.5WL1	Yes	
D7	1.2DL+1.6SL+0.5WL2	Yes	
D8	1.2DL+WL1	Yes	
D9	1.2DL+WL2	Yes	
D10	1.2DL+0.5SL+WL1	Yes	
D11	1.2DL+0.5SL+WL2	Yes	
D12	1.2DL+0.2SL	Yes	
D13	1.2DL+EQ1	Yes	
D14	1.25DL+EQ2	Yes	
D15	1.25DL+0.2SL+EQ1	Yes	
D16	1.25DL+0.2SL+EQ2	Yes	
D17	0.9DL+WL1	Yes	
D18	0.9DL+WL2	Yes	
D19	0.85DL+EQ1	Yes	
D20	0.85DL+EQ2	Yes	
S1	DL	Yes	
S2	DL+SL	Yes	
S3	DL+0.75SL	Yes	
S4	DL+0.6WL1	Yes	
S5	DL+0.6WL2	Yes	
S6	1.04DL+0.7EQ1	Yes	
S7	1.04DL+0.7EQ2	Yes	
S8	1.04DL+0.525EQ1	Yes	
S9	1.04DL+0.525EQ2	Yes	
S10	DL+0.75SL	Yes	
S11	1.03DL+0.75SL+0.525EQ1	Yes	
S12	1.03DL+0.75SL+0.525EQ2	Yes	
S13	0.6DL+0.6WL1	Yes	

S14	0.6DL+0.6WL2	Yes
S15	0.57DL+0.7EQ1	Yes
S16	0.57DL+0.7EQ2	Yes

Load on nodes

Condition	Node	FX [Kip]	FY [Kip]	FZ [Kip]	MX [Kip*ft]	MY [Kip*ft]	MZ [Kip*ft]
DL	13	0.00	-4.86	0.00	0.00	0.00	0.00
	15	0.00	-1.24	0.00	0.00	0.00	0.00
SL	13	0.00	-2.77	0.00	0.00	0.00	0.00
	15	0.00	-0.93	0.00	0.00	0.00	0.00
WL1	15	0.00	0.00	-31.72	0.00	0.00	0.00
WL2	13	0.00	0.00	31.72	0.00	0.00	0.00
EQ1	15	0.00	0.00	-14.25	0.00	0.00	0.00
EQ2	13	0.00	0.00	14.25	0.00	0.00	0.00

Self weight multipliers for load conditions

Condition	Description	Self weight multiplier Comb.	MultX	MultY	MultZ
DL	Dead Load	No	0.00	-1.00	0.00
SL	Snow Load	No	0.00	0.00	0.00
WL1	Wind Load 1	No	0.00	0.00	0.00
WL2	Wind Load 2	No	0.00	0.00	0.00
EQ1	Horizontal Seismic Load 1	No	0.00	0.00	0.00
EQ2	Horizontal Seismic Load 2	No	0.00	0.00	0.00
D1	1.4DL	Yes	0.00	0.00	0.00
D2	1.2DL+0.5SL	Yes	0.00	0.00	0.00
D3	1.2DL+1.6SL	Yes	0.00	0.00	0.00
D4	1.2DL+0.5WL1	Yes	0.00	0.00	0.00
D5	1.2DL+0.5WL2	Yes	0.00	0.00	0.00
D6	1.2DL+1.6SL+0.5WL1	Yes	0.00	0.00	0.00
D7	1.2DL+1.6SL+0.5WL2	Yes	0.00	0.00	0.00
D8	1.2DL+WL1	Yes	0.00	0.00	0.00
D9	1.2DL+WL2	Yes	0.00	0.00	0.00
D10	1.2DL+0.5SL+WL1	Yes	0.00	0.00	0.00
D11	1.2DL+0.5SL+WL2	Yes	0.00	0.00	0.00
D12	1.2DL+0.2SL	Yes	0.00	0.00	0.00
D13	1.2DL+EQ1	Yes	0.00	0.00	0.00
D14	1.25DL+EQ2	Yes	0.00	0.00	0.00
D15	1.25DL+0.2SL+EQ1	Yes	0.00	0.00	0.00
D16	1.25DL+0.2SL+EQ2	Yes	0.00	0.00	0.00
D17	0.9DL+WL1	Yes	0.00	0.00	0.00
D18	0.9DL+WL2	Yes	0.00	0.00	0.00
D19	0.85DL+EQ1	Yes	0.00	0.00	0.00
D20	0.85DL+EQ2	Yes	0.00	0.00	0.00
S1	DL	Yes	0.00	0.00	0.00
S2	DL+SL	Yes	0.00	0.00	0.00
S3	DL+0.75SL	Yes	0.00	0.00	0.00
S4	DL+0.6WL1	Yes	0.00	0.00	0.00
S5	DL+0.6WL2	Yes	0.00	0.00	0.00
S6	1.04DL+0.7EQ1	Yes	0.00	0.00	0.00

S7	1.04DL+0.7EQ2	Yes	0.00	0.00	0.00
S8	1.04DL+0.525EQ1	Yes	0.00	0.00	0.00
S9	1.04DL+0.525EQ2	Yes	0.00	0.00	0.00
S10	DL+0.75SL	Yes	0.00	0.00	0.00
S11	1.03DL+0.75SL+0.525EQ1	Yes	0.00	0.00	0.00
S12	1.03DL+0.75SL+0.525EQ2	Yes	0.00	0.00	0.00
S13	0.6DL+0.6WL1	Yes	0.00	0.00	0.00
S14	0.6DL+0.6WL2	Yes	0.00	0.00	0.00
S15	0.57DL+0.7EQ1	Yes	0.00	0.00	0.00
S16	0.57DL+0.7EQ2	Yes	0.00	0.00	0.00

Steel Code Check

Report: Concise

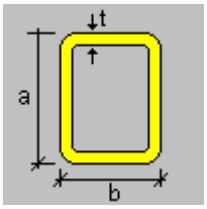
Members: Hot-rolled
Design code: AISC 360-2010 LRFD

Member : 10 (BF-1 COL C2)
Design status : OK

Section information

Section name: HSS_SQR 4X4X3_8 **(US)**

Dimensions



a	=	4.000	[in]	Height
b	=	4.000	[in]	Width
T	=	0.349	[in]	Thickness

Properties

Section properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	4.780	
Moment of Inertia (local axes) (I)	[in4]	10.300	10.300
Moment of Inertia (principal axes) (I')	[in4]	10.300	10.300
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	1.468	1.468
Radius of gyration (principal axes) (r')	[in]	1.468	1.468
Saint-Venant torsion constant. (J)	[in4]	17.500	
Section warping constant. (Cw)	[in6]	0.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	5.100	5.100
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	5.100	5.100
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	5.100	5.100

Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	5.100	5.100
Plastic section modulus (local axis) (Z)	[in3]	6.400	6.400
Plastic section modulus (principal axis) (Z')	[in3]	6.400	6.400
Polar radius of gyration. (ro)	[in]	2.067	
Area for shear (Aw)	[in2]	2.061	2.061
Torsional constant. (C)	[in3]	9.136	

Material : A500 GrB rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	46.00
Tensile strength (Fu):	[Kip/in2]	58.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

DESIGN CRITERIA

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	13.90

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
13.90	13.90

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
13.90	13.90	13.90	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

DESIGN CHECKS

AXIAL TENSION DESIGN ✔

Axial tension

Ratio	:	0.18			
Capacity	:	197.89 [Kip]	Reference	:	Eq. Sec. D2
Demand	:	35.65 [Kip]	Ctrl Eq.	:	D18 at 100.00%

Intermediate results	Unit	Value	Reference

Factored axial tension capacity(ϕP_n):	[Kip]	197.89	Eq. Sec. D2

AXIAL COMPRESSION DESIGN ✔

Compression in the major axis 33

Ratio	: 0.52			
Capacity	: 83.03 [Kip]	Reference	: Sec. E1	
Demand	: 43.23 [Kip]	Ctrl Eq.	: D10 at 0.00%	

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	83.03	Sec. E1

Compression in the minor axis 22

Ratio	: 0.52			
Capacity	: 83.03 [Kip]	Reference	: Sec. E1	
Demand	: 43.23 [Kip]	Ctrl Eq.	: D10 at 0.00%	

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	83.03	Sec. E1

FLEXURAL DESIGN ✔

Bending about major axis, M33

Ratio	: 0.00			
Capacity	: 22.08 [Kip*ft]	Reference	: Sec. F1	
Demand	: 0.00 [Kip*ft]	Ctrl Eq.	: D1 at 0.00%	

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(ϕM_n):	[Kip*ft]	22.08	Sec. F1

Bending about minor axis, M22

Ratio	: 0.00			
Capacity	: 22.08 [Kip*ft]	Reference	: Sec. F1	
Demand	: 0.00 [Kip*ft]	Ctrl Eq.	: D1 at 0.00%	

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(ϕM_n):	[Kip*ft]	22.08	Sec. F1

DESIGN FOR SHEAR ✔

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	51.20 [Kip]	Reference	:	Sec. G1
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	51.20	Sec. G1

Shear in minor axis 22

Ratio	:	0.00			
Capacity	:	51.20 [Kip]	Reference	:	Sec. G1
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	51.20	Sec. G1

TORSION DESIGN ✔

Torsion

Ratio	:	0.00			
Capacity	:	18.91 [Kip*ft]	Reference	:	Eq. H3-1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	18.91	Eq. H3-1

COMBINED ACTIONS DESIGN ✔

Combined flexure and axial compression

Ratio	:	0.52			
Ctrl Eq.	:	D10 at 0.00%	Reference	:	Eq. H1-1a

Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.52	Eq. H1-1a

Combined flexure and axial tension

Ratio	:	0.09			
Ctrl Eq.	:	D18 at 100.00%	Reference	:	Eq. H1-1b

Intermediate results	Unit	Value	Reference

Combined flexure and axial compression about local axis

Ratio : N/A
Ctrl Eq. : -- Reference :

Combined flexure and axial tension about local axis

Ratio : N/A
Ctrl Eq. : -- Reference :

Combined torsion, flexure, shear and axial compression

Ratio : N/A
Ctrl Eq. : -- Reference :

Combined torsion, flexure, shear and axial tension

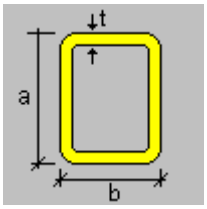
Ratio : N/A
Ctrl Eq. : -- Reference :

Member : 11 (BF-1 COL C2)
Design status : OK

Section information

Section name: HSS_SQR 4X4X3_8 **(US)**

Dimensions



a	=	4.000	[in]	Height
b	=	4.000	[in]	Width
T	=	0.349	[in]	Thickness

Properties

Section properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	4.780	
Moment of Inertia (local axes) (I)	[in4]	10.300	10.300
Moment of Inertia (principal axes) (I')	[in4]	10.300	10.300
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000

Radius of gyration (local axes) (r)	[in]	1.468	1.468
Radius of gyration (principal axes) (r')	[in]	1.468	1.468
Saint-Venant torsion constant. (J)	[in4]	17.500	
Section warping constant. (Cw)	[in6]	0.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	5.100	5.100
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	5.100	5.100
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	5.100	5.100
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	5.100	5.100
Plastic section modulus (local axis) (Z)	[in3]	6.400	6.400
Plastic section modulus (principal axis) (Z')	[in3]	6.400	6.400
Polar radius of gyration. (ro)	[in]	2.067	
Area for shear (Aw)	[in2]	2.061	2.061
Torsional constant. (C)	[in3]	9.136	

Material : A500 GrB rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	46.00
Tensile strength (Fu):	[Kip/in2]	58.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

DESIGN CRITERIA

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	13.90

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
13.90	13.90

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
13.90	13.90	13.90	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

DESIGN CHECKS

AXIAL TENSION DESIGN ✔

Axial tension

Ratio : 0.20
Capacity : 197.89 [Kip]
Demand : 38.91 [Kip]
Reference : Eq. Sec. D2
Ctrl Eq. : D8 at 100.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	197.89	Eq. Sec. D2

AXIAL COMPRESSION DESIGN ✔

Compression in the major axis 33

Ratio : 0.46
Capacity : 83.03 [Kip]
Demand : 37.99 [Kip]
Reference : Sec. E1
Ctrl Eq. : D11 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	83.03	Sec. E1

Compression in the minor axis 22

Ratio : 0.46
Capacity : 83.03 [Kip]
Demand : 37.99 [Kip]
Reference : Sec. E1
Ctrl Eq. : D11 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	83.03	Sec. E1

FLEXURAL DESIGN ✔

Bending about major axis, M33

Ratio : 0.00
Capacity : 22.08 [Kip*ft]
Demand : 0.00 [Kip*ft]
Reference : Sec. F1
Ctrl Eq. : D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(ϕM_n):	[Kip*ft]	22.08	Sec. F1

Bending about minor axis, M22

Ratio : 0.00
Capacity : 22.08 [Kip*ft]
Demand : 0.00 [Kip*ft]
Reference : Sec. F1
Ctrl Eq. : D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(ϕM_n):	[Kip*ft]	22.08	Sec. F1

DESIGN FOR SHEAR ✔

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	51.20 [Kip]	Reference	:	Sec. G1
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	51.20	Sec. G1

Shear in minor axis 22

Ratio	:	0.00			
Capacity	:	51.20 [Kip]	Reference	:	Sec. G1
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	51.20	Sec. G1

TORSION DESIGN ✔

Torsion

Ratio	:	0.00			
Capacity	:	18.91 [Kip*ft]	Reference	:	Eq. H3-1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	18.91	Eq. H3-1

COMBINED ACTIONS DESIGN ✔

Combined flexure and axial compression

Ratio	:	0.46			
Ctrl Eq.	:	D11 at 0.00%	Reference	:	Eq. H1-1a

Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.46	Eq. H1-1a

Combined flexure and axial tension

Ratio : 0.10
Ctrl Eq. : D8 at 100.00% Reference : Eq. H1-1b

Intermediate results	Unit	Value	Reference

Combined flexure and axial compression about local axis

Ratio : N/A
Ctrl Eq. : -- Reference :

Combined flexure and axial tension about local axis

Ratio : N/A
Ctrl Eq. : -- Reference :

Combined torsion, flexure, shear and axial compression

Ratio : N/A
Ctrl Eq. : -- Reference :

Combined torsion, flexure, shear and axial tension

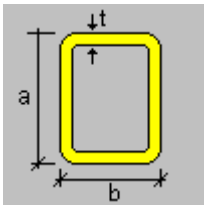
Ratio : N/A
Ctrl Eq. : -- Reference :

Member : 12 (BF-1 TOP BEAM)
Design status : OK

Section information

Section name: HSS_SQR 4X4X3_8 **(US)**

Dimensions



a	=	4.000	[in]	Height
b	=	4.000	[in]	Width
T	=	0.349	[in]	Thickness

Properties

Section properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	4.780	
Moment of Inertia (local axes) (I)	[in4]	10.300	10.300
Moment of Inertia (principal axes) (I')	[in4]	10.300	10.300
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	1.468	1.468
Radius of gyration (principal axes) (r')	[in]	1.468	1.468
Saint-Venant torsion constant. (J)	[in4]	17.500	
Section warping constant. (Cw)	[in6]	0.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	5.100	5.100
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	5.100	5.100
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	5.100	5.100
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	5.100	5.100
Plastic section modulus (local axis) (Z)	[in3]	6.400	6.400
Plastic section modulus (principal axis) (Z')	[in3]	6.400	6.400
Polar radius of gyration. (ro)	[in]	2.067	
Area for shear (Aw)	[in2]	2.061	2.061
Torsional constant. (C)	[in3]	9.136	

Material : A500 GrB rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	46.00
Tensile strength (Fu):	[Kip/in2]	58.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

DESIGN CRITERIA

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	5.73

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
5.73	5.73

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
5.73	5.73	5.73	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway

Minor axis frame type : Sway

DESIGN CHECKS

AXIAL TENSION DESIGN ✔

Axial tension

Ratio : 0.01
Capacity : 197.89 [Kip]
Demand : 1.21 [Kip]
Reference : Eq. Sec. D2
Ctrl Eq. : D3 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	197.89	Eq. Sec. D2

AXIAL COMPRESSION DESIGN ✔

Compression in the major axis 33

Ratio : 0.09
Capacity : 170.74 [Kip]
Demand : 15.10 [Kip]
Reference : Sec. E1
Ctrl Eq. : D17 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	170.74	Sec. E1

Compression in the minor axis 22

Ratio : 0.09
Capacity : 170.74 [Kip]
Demand : 15.10 [Kip]
Reference : Sec. E1
Ctrl Eq. : D17 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	170.74	Sec. E1

FLEXURAL DESIGN ✔

Bending about major axis, M33

Ratio : 0.00
Capacity : 22.08 [Kip*ft]
Demand : 0.09 [Kip*ft]
Reference : Sec. F1
Ctrl Eq. : D1 at 50.00%

Intermediate results	Unit	Value	Reference
Section classification			

Factored yielding strength(ϕM_n):	[Kip*ft]	22.08	Sec. F1

Bending about minor axis, M22

Ratio	:	0.00		
Capacity	:	22.08 [Kip*ft]	Reference	: Sec. F1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(ϕM_n):	[Kip*ft]	22.08	Sec. F1

DESIGN FOR SHEAR ✔

Shear in major axis 33

Ratio	:	0.00		
Capacity	:	51.20 [Kip]	Reference	: Sec. G1
Demand	:	0.00 [Kip]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	51.20	Sec. G1

Shear in minor axis 22

Ratio	:	0.00		
Capacity	:	51.20 [Kip]	Reference	: Sec. G1
Demand	:	-0.06 [Kip]	Ctrl Eq.	: D1 at 100.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	51.20	Sec. G1

TORSION DESIGN ✔

Torsion

Ratio	:	0.00		
Capacity	:	18.91 [Kip*ft]	Reference	: Eq. H3-1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	18.91	Eq. H3-1

COMBINED ACTIONS DESIGN ✔

Combined flexure and axial compression

Ratio : 0.05
Ctrl Eq. : D8 at 50.00% Reference : Eq. H1-1b

Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.05	Eq. H1-1b

Combined flexure and axial tension

Ratio : 0.01
Ctrl Eq. : D3 at 50.00% Reference : Eq. H1-1b

Intermediate results	Unit	Value	Reference

Combined flexure and axial compression about local axis

Ratio : N/A
Ctrl Eq. : -- Reference :

Combined flexure and axial tension about local axis

Ratio : N/A
Ctrl Eq. : -- Reference :

Combined torsion, flexure, shear and axial compression

Ratio : N/A
Ctrl Eq. : -- Reference :

Combined torsion, flexure, shear and axial tension

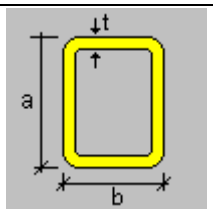
Ratio : N/A
Ctrl Eq. : -- Reference :

Member : 13 (BF-1 DIAGONALS)
Design status : OK

Section information

Section name: HSS_SQR 4X4X3_8 **(US)**

Dimensions



a	=	4.000	[in]	Height
b	=	4.000	[in]	Width
T	=	0.349	[in]	Thickness

Properties

Section properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	4.780	
Moment of Inertia (local axes) (I)	[in4]	10.300	10.300
Moment of Inertia (principal axes) (I')	[in4]	10.300	10.300
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	1.468	1.468
Radius of gyration (principal axes) (r')	[in]	1.468	1.468
Saint-Venant torsion constant. (J)	[in4]	17.500	
Section warping constant. (Cw)	[in6]	0.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	5.100	5.100
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	5.100	5.100
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	5.100	5.100
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	5.100	5.100
Plastic section modulus (local axis) (Z)	[in3]	6.400	6.400
Plastic section modulus (principal axis) (Z')	[in3]	6.400	6.400
Polar radius of gyration. (ro)	[in]	2.067	
Area for shear (Aw)	[in2]	2.061	2.061
Torsional constant. (C)	[in3]	9.136	

Material : A500 GrB rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	46.00
Tensile strength (Fu):	[Kip/in2]	58.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

DESIGN CRITERIA

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	15.03

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
15.03	15.03

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)

15.03	15.03	15.03	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

DESIGN CHECKS

AXIAL TENSION DESIGN

✔

Axial tension

Ratio : 0.20
Capacity : 197.89 [Kip]
Demand : 39.73 [Kip]
Reference : Eq. Sec. D2
Ctrl Eq. : D18 at 50.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	197.89	Eq. Sec. D2

AXIAL COMPRESSION DESIGN

✔

Compression in the major axis 33

Ratio : 0.63
Capacity : 71.49 [Kip]
Demand : 44.90 [Kip]
Reference : Sec. E1
Ctrl Eq. : D10 at 100.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	71.49	Sec. E1

Compression in the minor axis 22

Ratio : 0.63
Capacity : 71.49 [Kip]
Demand : 44.90 [Kip]
Reference : Sec. E1
Ctrl Eq. : D10 at 100.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	71.49	Sec. E1

FLEXURAL DESIGN

✔

Bending about major axis, M33

Ratio : 0.03
Capacity : 22.08 [Kip*ft] Reference : Sec. F1
Demand : -0.56 [Kip*ft] Ctrl Eq. : D9 at 50.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(ϕM_n):	[Kip*ft]	22.08	Sec. F1

Bending about minor axis, M22

Ratio : 0.00
Capacity : 22.08 [Kip*ft] Reference : Sec. F1
Demand : 0.00 [Kip*ft] Ctrl Eq. : D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(ϕM_n):	[Kip*ft]	22.08	Sec. F1

DESIGN FOR SHEAR ✔

Shear in major axis 33

Ratio : 0.00
Capacity : 51.20 [Kip] Reference : Sec. G1
Demand : 0.00 [Kip] Ctrl Eq. : D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	51.20	Sec. G1

Shear in minor axis 22

Ratio : 0.00
Capacity : 51.20 [Kip] Reference : Sec. G1
Demand : 0.10 [Kip] Ctrl Eq. : D10 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	51.20	Sec. G1

TORSION DESIGN ✔

Torsion

Ratio : 0.00
Capacity : 18.91 [Kip*ft] Reference : Eq. H3-1
Demand : 0.00 [Kip*ft] Ctrl Eq. : D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	18.91	Eq. H3-1

COMBINED ACTIONS DESIGN ✔

Combined flexure and axial compression

Ratio : 0.65
Ctrl Eq. : D10 at 50.00% Reference : Eq. H1-1a

Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.65	Eq. H1-1a

Combined flexure and axial tension

Ratio : 0.22
Ctrl Eq. : D18 at 50.00% Reference : Eq. H1-1a

Intermediate results	Unit	Value	Reference

Combined flexure and axial compression about local axis

Ratio : N/A
Ctrl Eq. : -- Reference :

Combined flexure and axial tension about local axis

Ratio : N/A
Ctrl Eq. : -- Reference :

Combined torsion, flexure, shear and axial compression

Ratio : N/A
Ctrl Eq. : -- Reference :

Combined torsion, flexure, shear and axial tension

Ratio : N/A
Ctrl Eq. : -- Reference :

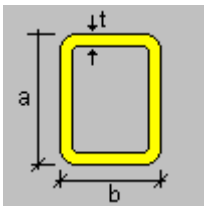
Member : 14 (BF-1 DIAGONALS)
Design status : OK

	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	Braced Frame 1, Base Plate & Anchor Bolts Design				4-23	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

Section information

Section name: HSS_SQR 4X4X3_8 **(US)**

Dimensions



a	=	4.000	[in]	Height
b	=	4.000	[in]	Width
T	=	0.349	[in]	Thickness

Properties

Section properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	4.780	
Moment of Inertia (local axes) (I)	[in4]	10.300	10.300
Moment of Inertia (principal axes) (I')	[in4]	10.300	10.300
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	1.468	1.468
Radius of gyration (principal axes) (r')	[in]	1.468	1.468
Saint-Venant torsion constant. (J)	[in4]	17.500	
Section warping constant. (Cw)	[in6]	0.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	5.100	5.100
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	5.100	5.100
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	5.100	5.100
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	5.100	5.100
Plastic section modulus (local axis) (Z)	[in3]	6.400	6.400
Plastic section modulus (principal axis) (Z')	[in3]	6.400	6.400
Polar radius of gyration. (ro)	[in]	2.067	
Area for shear (Aw)	[in2]	2.061	2.061
Torsional constant. (C)	[in3]	9.136	

Material : A500 GrB rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	46.00
Tensile strength (Fu):	[Kip/in2]	58.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

DESIGN CRITERIA

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	15.03

Distance between member lateral bracing points

Length (Lb) [ft]

Top	Bottom
15.03	15.03

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
15.03	15.03	15.03	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

DESIGN CHECKS

AXIAL TENSION DESIGN ✔

Axial tension

Ratio : 0.20
Capacity : 197.89 [Kip]
Demand : 39.75 [Kip]
Reference : Eq. Sec. D2
Ctrl Eq. : D17 at 50.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	197.89	Eq. Sec. D2

AXIAL COMPRESSION DESIGN ✔

Compression in the major axis 33

Ratio : 0.63
Capacity : 71.49 [Kip]
Demand : 44.88 [Kip]
Reference : Sec. E1
Ctrl Eq. : D11 at 100.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	71.49	Sec. E1

Compression in the minor axis 22

Ratio : 0.63
Capacity : 71.49 [Kip]
Demand : 44.88 [Kip]
Reference : Sec. E1
Ctrl Eq. : D11 at 100.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	71.49	Sec. E1

FLEXURAL DESIGN ✔

Bending about major axis, M33

Ratio	:	0.03			
Capacity	:	22.08 [Kip*ft]	Reference	:	Sec. F1
Demand	:	-0.63 [Kip*ft]	Ctrl Eq.	:	D10 at 50.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(ϕM_n):	[Kip*ft]	22.08	Sec. F1

Bending about minor axis, M22

Ratio	:	0.00			
Capacity	:	22.08 [Kip*ft]	Reference	:	Sec. F1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(ϕM_n):	[Kip*ft]	22.08	Sec. F1

DESIGN FOR SHEAR ✔

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	51.20 [Kip]	Reference	:	Sec. G1
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	51.20	Sec. G1

Shear in minor axis 22

Ratio	:	0.00			
Capacity	:	51.20 [Kip]	Reference	:	Sec. G1
Demand	:	0.11 [Kip]	Ctrl Eq.	:	D10 at 50.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	51.20	Sec. G1

TORSION DESIGN ✔

Torsion

Ratio : 0.00
Capacity : 18.91 [Kip*ft]
Demand : 0.00 [Kip*ft]
Reference : Eq. H3-1
Ctrl Eq. : D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	18.91	Eq. H3-1

COMBINED ACTIONS DESIGN ✔

Combined flexure and axial compression

Ratio : 0.64
Ctrl Eq. : D11 at 50.00%
Reference : Eq. H1-1a

Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.64	Eq. H1-1a

Combined flexure and axial tension

Ratio : 0.23
Ctrl Eq. : D17 at 50.00%
Reference : Eq. H1-1a

Intermediate results	Unit	Value	Reference

Combined flexure and axial compression about local axis

Ratio : N/A
Ctrl Eq. : --
Reference :

Combined flexure and axial tension about local axis

Ratio : N/A
Ctrl Eq. : --
Reference :

Combined torsion, flexure, shear and axial compression

Ratio : N/A
Ctrl Eq. : --
Reference :

Combined torsion, flexure, shear and axial tension

Ratio : N/A
Ctrl Eq. : --
Reference :

	Project			Job Ref.	
AECOM	McDonalds Lillington, NC			032-0312	
	Section			Sheet no./rev.	
	Braced Frame 1, Base Plate & Anchor Bolts Design			4-27	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

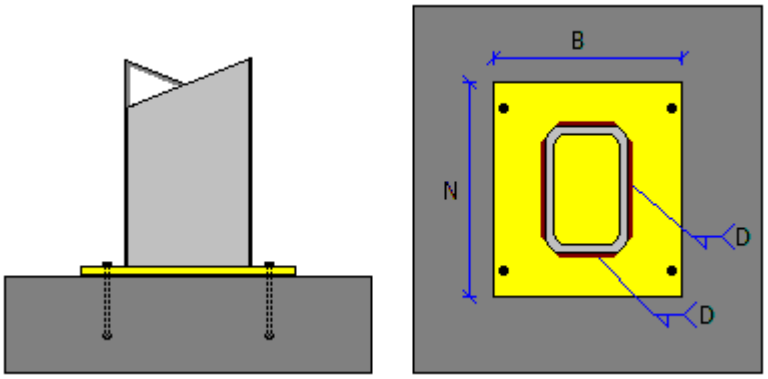
Steel Connections
Data

Connection: 2 - Pinned BP - HSS Member

Family: Column - Base (CB)
Type: Base plate
Description: Smart Pinned Base Plate 2

General information

Connector



Members

Column

Column type	:	Prismatic member
Section	:	HSS_SQR 4X4X3_8
Material	:	A500 GrB rectangular
Longitudinal offset	:	0 in

Base plate

Base plate

Plate shape	:	Rectangular
Connection type	:	Unstiffened
Position on the support	:	Center
N: Longitudinal dimension	:	12 in
B: Transversal dimension	:	12 in
Thickness	:	1.5 in
Material	:	A36
Column weld	:	E70XX
D: Column weld size (1/16 in)	:	5
Override A2/A1 ratio	:	No
Include shear lug	:	No

Support

With pedestal	:	No

Longitudinal dimension	:	84 in
Transversal dimension	:	84 in
Thickness	:	36 in
Material	:	C 3-60
Include grouting	:	No
Anchor		
Anchor position	:	Longitudinal position
Rows number per side	:	1
Anchors per row	:	2
Longitudinal edge distance on the plate	:	2 in
Transverse edge distance on the plate	:	2 in
Anchor type	:	Headed
Head type	:	Hexagonal
Include lock nut	:	No
Anchor	:	1 1/4"
Effective embedment depth	:	18 in
Total length	:	21.15 in
Material	:	F1554 Gr36
Fy	:	36 kip/in2
Fu	:	58 kip/in2
Cracked concrete	:	Yes
Brittle steel	:	No
Anchors welded to base plate	:	No
Anchor reinforcement		
Type of reinforcement	:	Primary
Tension reinforcement	:	No
Shear reinforcement	:	No

Steel Connections
Results

Connection: 2 - Pinned BP - HSS Member

Family: Column - Base (CB)
Type: Base plate
Description: Smart Pinned Base Plate 2

Design code: AISC 360-10 LRFD, ACI 318-11

Demands

Description	Pu [kip]	Mu22 [kip*ft]	Mu33 [kip*ft]	Vu2 [kip]	Vu3 [kip]	Load type
DL	-5.44	0.00	0.00	0.00	0.60	Design
SL	-2.81	0.00	0.00	0.00	0.32	Design
WL1	-76.95	0.00	0.00	0.00	15.48	Design
WL2	76.95	0.00	0.00	0.00	-15.03	Design
EQ1	-34.57	0.00	0.00	0.00	6.95	Design
EQ2	34.57	0.00	0.00	0.00	-6.75	Design
D1	-7.61	0.00	0.00	0.00	0.85	Design
D2	-7.93	0.00	0.00	0.00	0.89	Design
D3	-11.02	0.00	0.00	0.00	1.24	Design
D4	-45.00	0.00	0.00	0.00	8.47	Design

D5	31.95	0.00	0.00	0.00	-6.79	Design
D6	-49.49	0.00	0.00	0.00	8.98	Design
D7	27.45	0.00	0.00	0.00	-6.28	Design
D8	-83.47	0.00	0.00	0.00	16.20	Design
D9	70.42	0.00	0.00	0.00	-14.31	Design
D10	-84.88	0.00	0.00	0.00	16.36	Design
D11	69.02	0.00	0.00	0.00	-14.15	Design
D12	-7.09	0.00	0.00	0.00	0.79	Design
D13	-41.09	0.00	0.00	0.00	7.68	Design
D14	28.05	0.00	0.00	0.00	-6.03	SeismicLoadEffects
D15	-41.76	0.00	0.00	0.00	7.76	Design
D16	27.37	0.00	0.00	0.00	-5.95	SeismicLoadEffects
D17	-81.84	0.00	0.00	0.00	16.02	Design
D18	72.06	0.00	0.00	0.00	-14.49	Design
D19	-39.35	0.00	0.00	0.00	7.49	Design
D20	29.78	0.00	0.00	0.00	-6.22	SeismicLoadEffects

Design calculations

Design for major axis

Base plate (AISC 360-10 LRFD)

Geometric Considerations

Dimensions	Unit	Value	Min.	Max.	Sta.	References
Base plate						
Distance from anchor to edge	[in]	1.37	0.25	--	✔	
Weld size	[1/16in]	5	3	--	✔	table J2.4

Design Check

Verification	Unit	Capacity	Demand	Ctrl EQ	Ratio	References
Concrete base						
Axial bearing	[Kip/in2]	3.32	0.59	D10	0.18	DG1 3.1.1
Base plate						
Flexural yielding (bearing interface)	[Kip*ft/ft]	18.23	4.95	D10	0.27	DG1 Sec 3.1.2
Flexural yielding (tension interface)	[Kip*ft/ft]	18.23	9.85	WL2	0.54	DG1 Eq. 3.3.13
Column						
Weld capacity	[Kip/ft]	125.29	46.17	WL2	0.37	HSS Manual p. 7-10
Elastic method weld shear and axial capacity	[Kip/ft]	125.29	78.17	WL2	0.62	Sec. J2.4
Ratio	**0.62**					

Anchors

Geometric Considerations

Dimensions	Unit	Value	Min.	Max.	Sta.	References
Anchors						

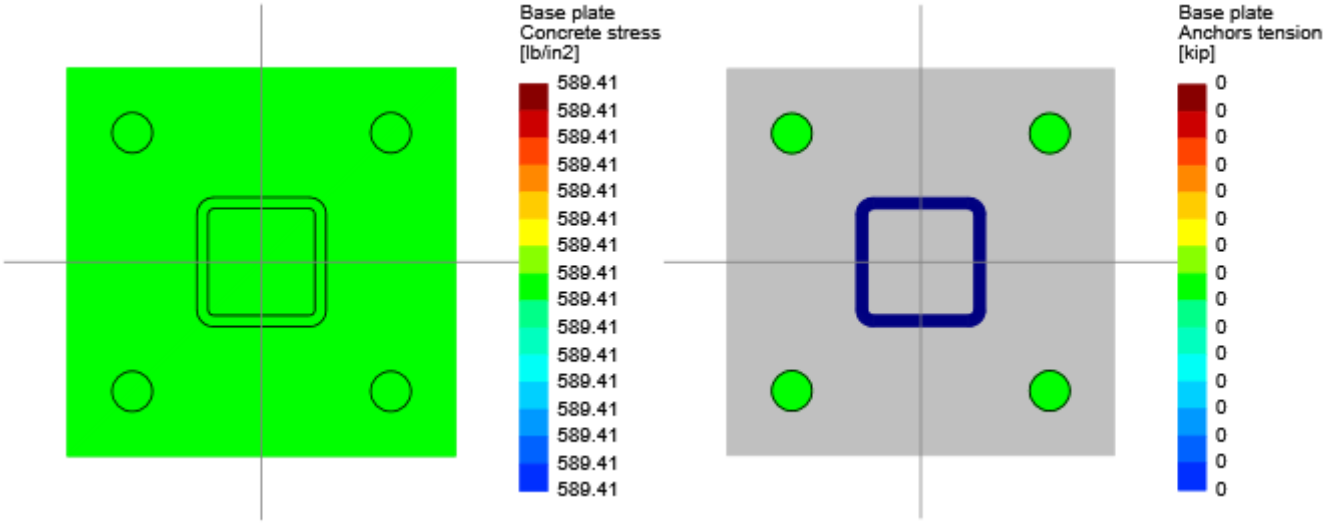
Anchor spacing	[in]	8.00	5.00	--	✓	Sec. D.8.1
Concrete cover	[in]	37.37	3.00	--	✓	Sec. 7.7.1
Effective length	[in]	18.81	--	35.19	✓	

Design Check

Verification	Unit	Capacity	Demand	Ctrl EQ	Ratio	References
Anchor tension	[Kip]	42.16	19.24	WL2	0.46	Eq. D-2
Breakout of anchor in tension	[Kip]	75.84	19.24	WL2	0.25	Sec. D.3.3.4.4
Breakout of group of anchors in tension	[Kip]	99.98	76.95	WL2	0.77	Sec. D.3.3.4.4
Pullout of anchor in tension	[Kip]	30.54	19.24	WL2	0.63	Sec. D.3.3.4.4
Anchor shear	[Kip]	21.92	0.00	DL	0.00	Eq. D-29
Breakout of anchor in shear	[Kip]	44.86	0.00	DL	0.00	Table D.4.1.1, Sec. D.4.3
Pryout of anchor in shear	[Kip]	151.68	0.00	DL	0.00	Table D.4.1.1, Sec. D.4.3
Ratio	**0.77**					
Global critical strength ratio	**0.77**					

Major axis

Maximum compression (D10)

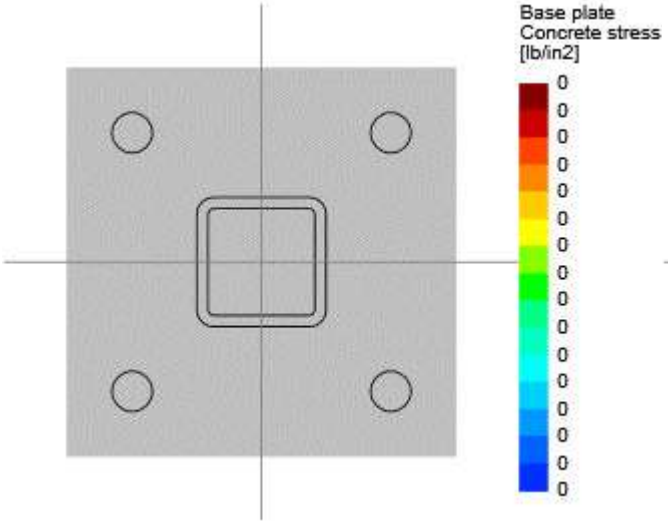


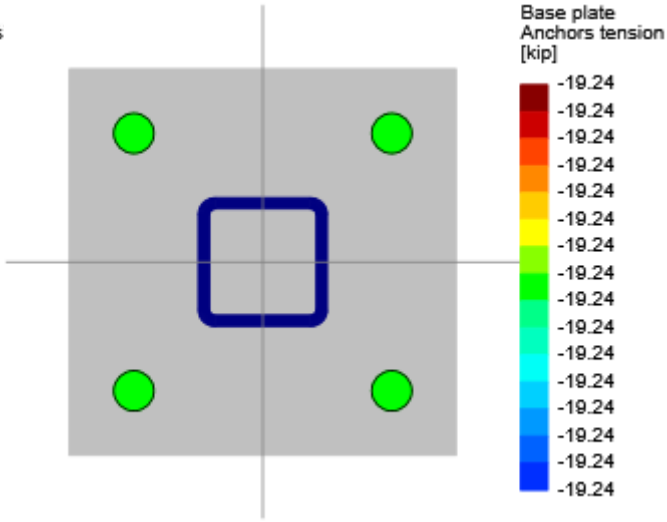
Maximum bearing pressure	589.41	[lb/in2]
Minimum bearing pressure	589.41	[lb/in2]
Maximum anchor tension	0.00	[kip]
Minimum anchor tension	0.00	[kip]
Neutral axis angle	0.00	[deg]
Neutral axis location	1.2E31	[in]
Bearing length	12.00	[in]

Anchors tensions

Anchor	Transverse [in]	Longitudinal [in]	Shear [kip]	Tension [kip]
1	-4.00	-4.00	0.00	0.00
2	-4.00	4.00	0.00	0.00
3	4.00	4.00	0.00	0.00
4	4.00	-4.00	0.00	0.00

Maximum tension (WL2)





Maximum bearing pressure	0.00	[lb/in2]
Minimum bearing pressure	0.00	[lb/in2]
Maximum anchor tension	19.24	[kip]
Minimum anchor tension	19.24	[kip]
Neutral axis angle	0.00	[deg]
Neutral axis location	-1.2E31	[in]
Bearing length	0.00	[in]

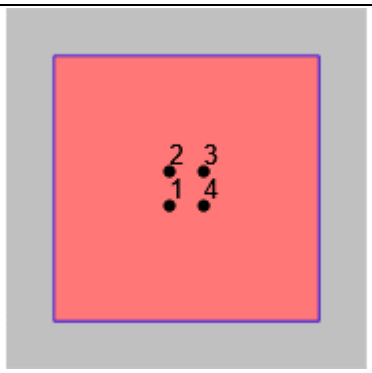
Anchors tensions

Anchor	Transverse [in]	Longitudinal [in]	Shear [kip]	Tension [kip]
1	-4.00	-4.00	0.00	19.24
2	-4.00	4.00	0.00	19.24
3	4.00	4.00	0.00	19.24
4	4.00	-4.00	0.00	19.24

Major axis

Results for tensile breakout (WL2)

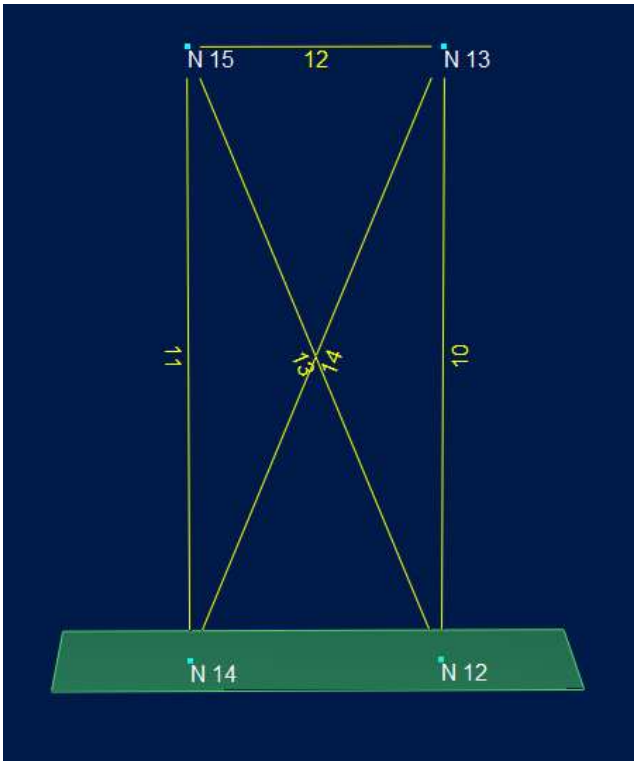
	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	Braced Frame 1, Base Plate & Anchor Bolts Design				4-32	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024



Group	Area [in2]	Tension [kip]	Anchors
1	3844.00	76.95	1, 2, 3, 4

	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	BF1 connection check				4-34	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA\PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

BF1 CONNECTION CHECK



Per RAM-Element design report, the maximum compression/tension force on HSS beam (member 12) is 15.21 kips,

ϕRn = 1.392 * D * L = 1.392 * 4 * (4" * 4) = 89 kips > 15.21 kips (OK)

Per RAM-Element design report, the maximum compression/tension force on HSS diagonals (member 13 & member 14) is 44.57 kips,

ϕRn = 1.392 * D * L = 1.392 * 4 * (4" * 4) = 89 kips > 44.57 kips (OK)

***See detail 1/S3.1 for additional information on BF-1**

	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.-	
	Braced Frame 2, Base Plate & Anchor Bolts Design				4-35	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

BRACED FRAME-BF2

Shear Force from wind V_{wy}= 63.43 kips

Shear From Seismic V_{Sy}= 28.49 kips;

Lateral Loads on BF2 From Wind $V_w = 0.1906 \times V_{wy} = 12.09$ kips

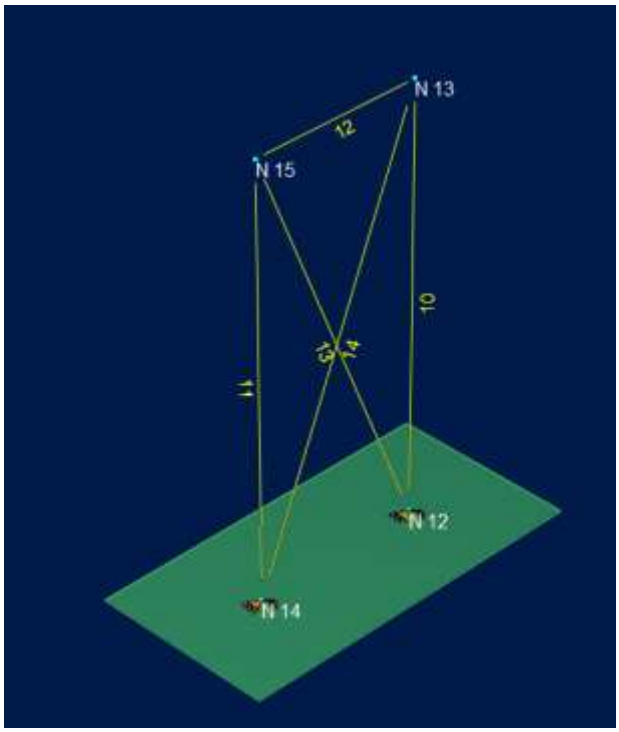
Lateral Loads on BF2 From Seismic $V_s = 0.1906 \times V_{Sy} = 5.43$ kips

ρ=1.0 (Per ASCE 12.3.4, redundancy factor ρ=1.0)

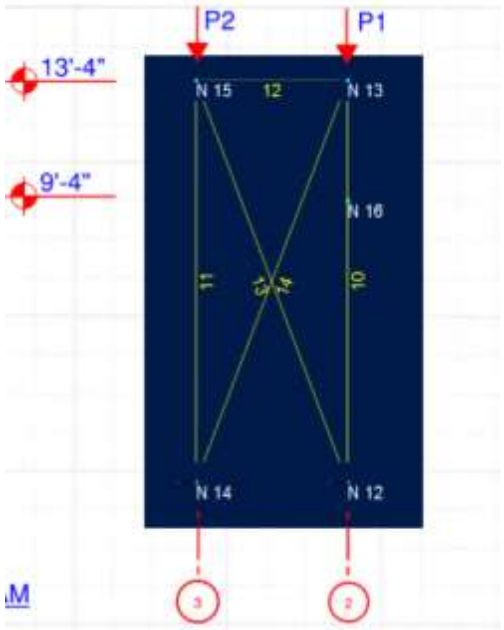
Ω=1.0

Ev (vertical seismic force) needs to be included since Sds=0.302 > 0.125 per ASCE (Eq.12.4-4)

Gravity load (roof dead load, and roof snow loads are included in the “Load Data” section of the RAM Element design report)



	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.-	
	Braced Frame 2, Base Plate & Anchor Bolts Design				4-36	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024



DL on roof beam,UDL	=	20 psf * 3 ft * 0.5 = 0.03 k/ft
SL on roof beam,UDL	=	(15 psf +32.8 psf) * 3 ft * 0.5 = 0.072 k/ft
DL on roof beam,Point load on col F/2 at 13'4"	=	0.03 k/ft * 5.73 ft * 0.5 = 0.086 kips
SL on roof beam, Point load on col F/2 at 13'4"	=	0.072 k/ft * 5.73 ft * 0.5 = 0.205 kips
DL on roof beam,Point load on col F/3 at 13'4"	=	0.03 k/ft * 5.73 ft * 0.5 = 0.086 kips
SL on roof beam, Point load on col F/3 at 13'4"	=	0.072 k/ft * 5.73 ft * 0.5 = 0.205 kips
DL on col F/2 at 13'4"	=	0.926 k/ft * (11.375+5.73) ft * 0.5 = 7.92 kips
SL on col F/2 at 13'4"	=	0.527 k/ft * (11.375+5.73) ft * 0.5 = 4.51 kips
DL on col F/3 at 9'4"	=	20 psf * 3 ft (tributary width) * (10.65+5.73) ft * 0.5 = 0.49 kips
SL on col F/3 at 9'4"	=	15 psf * 3 ft (tributary width) * (11.65+5.73) ft *0.5 = 0.37 kips
For convenience apply all the DL & SL at 13'4"		
VERTICAL SEISMIC LOAD (12.4.2.2 OF ASCE 7-10)		
Evh = 0.2 * Sds * DL = 0.2 * 0.302 * DL = 0.06 * DL		
(Apply vertical sesmic load where DL applies)		

RAM® Elements CONNECT Edition

AECOM

Current Date: 8/6/2024 11:59 AM
Units system: English
File name: C:\Job\McDonald\Working\Ram Element\Lillington\BF2 main model and footing.retx

Load data

GLOSSARY

Comb : Indicates if load condition is a load combination

Load Conditions

Condition	Description	Comb.	Category
DL	Dead Load	No	DL
SL	Snow Load	No	SNOW
WL1	Wind Load 1	No	WIND
WL2	Wind Load 2	No	WIND
EQ1	Horizontal Seismic Load 1	No	EQ
EQ2	Horizontal Seismic Load 2	No	EQ
D1	1.4DL	Yes	
D2	1.2DL+0.5SL	Yes	
D3	1.2DL+1.6SL	Yes	
D4	1.2DL+0.5WL1	Yes	
D5	1.2DL+0.5WL2	Yes	
D6	1.2DL+1.6SL+0.5WL1	Yes	
D7	1.2DL+1.6SL+0.5WL2	Yes	
D8	1.2DL+WL1	Yes	
D9	1.2DL+WL2	Yes	
D10	1.2DL+0.5SL+WL1	Yes	
D11	1.2DL+0.5SL+WL2	Yes	
D12	1.2DL+0.2SL	Yes	
D13	1.2DL+EQ1	Yes	
D14	1.25DL+EQ2	Yes	
D15	1.25DL+0.2SL+EQ1	Yes	
D16	1.25DL+0.2SL+EQ2	Yes	
D17	0.9DL+WL1	Yes	
D18	0.9DL+WL2	Yes	
D19	0.85DL+EQ1	Yes	
D20	0.85DL+EQ2	Yes	
S1	DL	Yes	
S2	DL+SL	Yes	
S3	DL+0.75SL	Yes	
S4	DL+0.6WL1	Yes	
S5	DL+0.6WL2	Yes	
S6	1.04DL+0.7EQ1	Yes	
S7	1.04DL+0.7EQ2	Yes	
S8	1.04DL+0.525EQ1	Yes	
S9	1.04DL+0.525EQ2	Yes	
S10	DL+0.75SL	Yes	
S11	1.03DL+0.75SL+0.525EQ1	Yes	
S12	1.03DL+0.75SL+0.525EQ2	Yes	
S13	0.6DL+0.6WL1	Yes	

S14	0.6DL+0.6WL2	Yes
S15	0.57DL+0.7EQ1	Yes
S16	0.57DL+0.7EQ2	Yes

Load on nodes

Condition	Node	FX [Kip]	FY [Kip]	FZ [Kip]	MX [Kip*ft]	MY [Kip*ft]	MZ [Kip*ft]
DL	13	0.00	-8.00	0.00	0.00	0.00	0.00
	15	0.00	-0.58	0.00	0.00	0.00	0.00
SL	13	0.00	-4.71	0.00	0.00	0.00	0.00
	15	0.00	-0.57	0.00	0.00	0.00	0.00
WL1	15	0.00	0.00	-12.09	0.00	0.00	0.00
WL2	13	0.00	0.00	12.09	0.00	0.00	0.00
EQ1	15	0.00	0.00	-5.43	0.00	0.00	0.00
EQ2	13	0.00	0.00	5.43	0.00	0.00	0.00

Self weight multipliers for load conditions

Condition	Description	Self weight multiplier Comb.	MultX	MultY	MultZ
DL	Dead Load	No	0.00	-1.00	0.00
SL	Snow Load	No	0.00	0.00	0.00
WL1	Wind Load 1	No	0.00	0.00	0.00
WL2	Wind Load 2	No	0.00	0.00	0.00
EQ1	Horizontal Seismic Load 1	No	0.00	0.00	0.00
EQ2	Horizontal Seismic Load 2	No	0.00	0.00	0.00
D1	1.4DL	Yes	0.00	0.00	0.00
D2	1.2DL+0.5SL	Yes	0.00	0.00	0.00
D3	1.2DL+1.6SL	Yes	0.00	0.00	0.00
D4	1.2DL+0.5WL1	Yes	0.00	0.00	0.00
D5	1.2DL+0.5WL2	Yes	0.00	0.00	0.00
D6	1.2DL+1.6SL+0.5WL1	Yes	0.00	0.00	0.00
D7	1.2DL+1.6SL+0.5WL2	Yes	0.00	0.00	0.00
D8	1.2DL+WL1	Yes	0.00	0.00	0.00
D9	1.2DL+WL2	Yes	0.00	0.00	0.00
D10	1.2DL+0.5SL+WL1	Yes	0.00	0.00	0.00
D11	1.2DL+0.5SL+WL2	Yes	0.00	0.00	0.00
D12	1.2DL+0.2SL	Yes	0.00	0.00	0.00
D13	1.2DL+EQ1	Yes	0.00	0.00	0.00
D14	1.25DL+EQ2	Yes	0.00	0.00	0.00
D15	1.25DL+0.2SL+EQ1	Yes	0.00	0.00	0.00
D16	1.25DL+0.2SL+EQ2	Yes	0.00	0.00	0.00
D17	0.9DL+WL1	Yes	0.00	0.00	0.00
D18	0.9DL+WL2	Yes	0.00	0.00	0.00
D19	0.85DL+EQ1	Yes	0.00	0.00	0.00
D20	0.85DL+EQ2	Yes	0.00	0.00	0.00
S1	DL	Yes	0.00	0.00	0.00
S2	DL+SL	Yes	0.00	0.00	0.00
S3	DL+0.75SL	Yes	0.00	0.00	0.00
S4	DL+0.6WL1	Yes	0.00	0.00	0.00
S5	DL+0.6WL2	Yes	0.00	0.00	0.00
S6	1.04DL+0.7EQ1	Yes	0.00	0.00	0.00

S7	1.04DL+0.7EQ2	Yes	0.00	0.00	0.00
S8	1.04DL+0.525EQ1	Yes	0.00	0.00	0.00
S9	1.04DL+0.525EQ2	Yes	0.00	0.00	0.00
S10	DL+0.75SL	Yes	0.00	0.00	0.00
S11	1.03DL+0.75SL+0.525EQ1	Yes	0.00	0.00	0.00
S12	1.03DL+0.75SL+0.525EQ2	Yes	0.00	0.00	0.00
S13	0.6DL+0.6WL1	Yes	0.00	0.00	0.00
S14	0.6DL+0.6WL2	Yes	0.00	0.00	0.00
S15	0.57DL+0.7EQ1	Yes	0.00	0.00	0.00
S16	0.57DL+0.7EQ2	Yes	0.00	0.00	0.00

Steel Code Check

Report: Concise

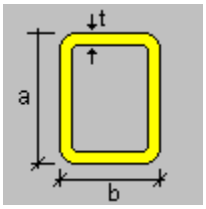
Members: Hot-rolled
Design code: AISC 360-2010 LRFD

Member : 10 (BF-2 COL C2)
Design status : OK

Section information

Section name: HSS_SQR 4X4X3_8 **(US)**

Dimensions



a	=	4.000	[in]	Height
b	=	4.000	[in]	Width
T	=	0.349	[in]	Thickness

Properties

Section properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	4.780	
Moment of Inertia (local axes) (I)	[in4]	10.300	10.300
Moment of Inertia (principal axes) (I')	[in4]	10.300	10.300
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	1.468	1.468
Radius of gyration (principal axes) (r')	[in]	1.468	1.468
Saint-Venant torsion constant. (J)	[in4]	17.500	
Section warping constant. (Cw)	[in6]	0.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	5.100	5.100
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	5.100	5.100
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	5.100	5.100
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	5.100	5.100

Plastic section modulus (local axis) (Z)	[in3]	6.400	6.400
Plastic section modulus (principal axis) (Z')	[in3]	6.400	6.400
Polar radius of gyration. (ro)	[in]	2.067	
Area for shear (Aw)	[in2]	2.061	2.061
Torsional constant. (C)	[in3]	9.136	

Material : A500 GrB rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	46.00
Tensile strength (Fu):	[Kip/in2]	58.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

DESIGN CRITERIA

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	13.90

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
13.90	13.90

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
13.90	13.90	13.90	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

DESIGN CHECKS

AXIAL TENSION DESIGN ✔

Axial tension

Ratio	:	0.05			
Capacity	:	197.89 [Kip]	Reference	:	Eq. Sec. D2
Demand	:	9.16 [Kip]	Ctrl Eq.	:	D18 at 100.00%

Intermediate results	Unit	Value	Reference

Factored axial tension capacity(ϕP_n):	[Kip]	197.89	Eq. Sec. D2

AXIAL COMPRESSION DESIGN ✔

Compression in the major axis 33

Ratio	:	0.29			
Capacity	:	83.03 [Kip]	Reference	:	Sec. E1
Demand	:	23.92 [Kip]	Ctrl Eq.	:	D10 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	83.03	Sec. E1

Compression in the minor axis 22

Ratio	:	0.29			
Capacity	:	83.03 [Kip]	Reference	:	Sec. E1
Demand	:	23.92 [Kip]	Ctrl Eq.	:	D10 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	83.03	Sec. E1

FLEXURAL DESIGN ✔

Bending about major axis, M33

Ratio	:	0.00			
Capacity	:	22.08 [Kip*ft]	Reference	:	Sec. F1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(ϕM_n):	[Kip*ft]	22.08	Sec. F1

Bending about minor axis, M22

Ratio	:	0.00			
Capacity	:	22.08 [Kip*ft]	Reference	:	Sec. F1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(ϕM_n):	[Kip*ft]	22.08	Sec. F1

DESIGN FOR SHEAR ✔

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	51.20 [Kip]	Reference	:	Sec. G1
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	51.20	Sec. G1

Shear in minor axis 22

Ratio	:	0.00			
Capacity	:	51.20 [Kip]	Reference	:	Sec. G1
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	51.20	Sec. G1

TORSION DESIGN ✔

Torsion

Ratio	:	0.00			
Capacity	:	18.91 [Kip*ft]	Reference	:	Eq. H3-1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	18.91	Eq. H3-1

COMBINED ACTIONS DESIGN ✔

Combined flexure and axial compression

Ratio	:	0.29			
Ctrl Eq.	:	D10 at 0.00%	Reference	:	Eq. H1-1a

Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.29	Eq. H1-1a

Combined flexure and axial tension

Ratio	:	0.02			
Ctrl Eq.	:	D18 at 100.00%	Reference	:	Eq. H1-1b

Intermediate results	Unit	Value	Reference

Combined flexure and axial compression about local axis

Ratio : N/A
Ctrl Eq. : -- Reference :

Combined flexure and axial tension about local axis

Ratio : N/A
Ctrl Eq. : -- Reference :

Combined torsion, flexure, shear and axial compression

Ratio : N/A
Ctrl Eq. : -- Reference :

Combined torsion, flexure, shear and axial tension

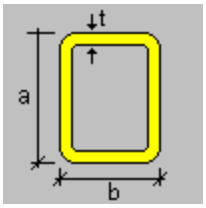
Ratio : N/A
Ctrl Eq. : -- Reference :

Member : 11 (BF-2 COL C2)
Design status : OK

Section information

Section name: HSS_SQR 4X4X3_8 **(US)**

Dimensions



a	=	4.000	[in]	Height
b	=	4.000	[in]	Width
T	=	0.349	[in]	Thickness

Properties

Section properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	4.780	
Moment of Inertia (local axes) (I)	[in4]	10.300	10.300
Moment of Inertia (principal axes) (I')	[in4]	10.300	10.300
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000

Radius of gyration (local axes) (r)	[in]	1.468	1.468
Radius of gyration (principal axes) (r')	[in]	1.468	1.468
Saint-Venant torsion constant. (J)	[in4]	17.500	
Section warping constant. (Cw)	[in6]	0.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	5.100	5.100
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	5.100	5.100
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	5.100	5.100
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	5.100	5.100
Plastic section modulus (local axis) (Z)	[in3]	6.400	6.400
Plastic section modulus (principal axis) (Z')	[in3]	6.400	6.400
Polar radius of gyration. (ro)	[in]	2.067	
Area for shear (Aw)	[in2]	2.061	2.061
Torsional constant. (C)	[in3]	9.136	

Material : A500 GrB rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	46.00
Tensile strength (Fu):	[Kip/in2]	58.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

DESIGN CRITERIA

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	13.90

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
13.90	13.90

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
13.90	13.90	13.90	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

DESIGN CHECKS

AXIAL TENSION DESIGN ✔

Axial tension

Ratio : 0.08
Capacity : 197.89 [Kip]
Demand : 16.45 [Kip]
Reference : Eq. Sec. D2
Ctrl Eq. : D10 at 100.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	197.89	Eq. Sec. D2

AXIAL COMPRESSION DESIGN ✔

Compression in the major axis 33

Ratio : 0.16
Capacity : 83.03 [Kip]
Demand : 13.58 [Kip]
Reference : Sec. E1
Ctrl Eq. : D18 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	83.03	Sec. E1

Compression in the minor axis 22

Ratio : 0.16
Capacity : 83.03 [Kip]
Demand : 13.58 [Kip]
Reference : Sec. E1
Ctrl Eq. : D18 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	83.03	Sec. E1

FLEXURAL DESIGN ✔

Bending about major axis, M33

Ratio : 0.00
Capacity : 22.08 [Kip*ft]
Demand : 0.00 [Kip*ft]
Reference : Sec. F1
Ctrl Eq. : D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(ϕM_n):	[Kip*ft]	22.08	Sec. F1

Bending about minor axis, M22

Ratio : 0.00
Capacity : 22.08 [Kip*ft]
Demand : 0.00 [Kip*ft]
Reference : Sec. F1
Ctrl Eq. : D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(ϕM_n):	[Kip*ft]	22.08	Sec. F1

DESIGN FOR SHEAR ✔

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	51.20 [Kip]	Reference	:	Sec. G1
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	51.20	Sec. G1

Shear in minor axis 22

Ratio	:	0.00			
Capacity	:	51.20 [Kip]	Reference	:	Sec. G1
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	51.20	Sec. G1

TORSION DESIGN ✔

Torsion

Ratio	:	0.00			
Capacity	:	18.91 [Kip*ft]	Reference	:	Eq. H3-1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	18.91	Eq. H3-1

COMBINED ACTIONS DESIGN ✔

Combined flexure and axial compression

Ratio	:	0.08			
Ctrl Eq.	:	D18 at 0.00%	Reference	:	Eq. H1-1b

Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.08	Eq. H1-1b

Combined flexure and axial tension

Ratio : 0.04
Ctrl Eq. : D10 at 100.00% Reference : Eq. H1-1b

Intermediate results	Unit	Value	Reference

Combined flexure and axial compression about local axis

Ratio : N/A
Ctrl Eq. : -- Reference :

Combined flexure and axial tension about local axis

Ratio : N/A
Ctrl Eq. : -- Reference :

Combined torsion, flexure, shear and axial compression

Ratio : N/A
Ctrl Eq. : -- Reference :

Combined torsion, flexure, shear and axial tension

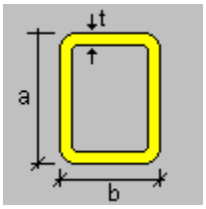
Ratio : N/A
Ctrl Eq. : -- Reference :

Member : 12 (BF-2 TOP BEAM)
Design status : OK

Section information

Section name: HSS_RECT 10X4X3_8 **(US)**

Dimensions



a	=	10.000	[in]	Height
b	=	4.000	[in]	Width
T	=	0.349	[in]	Thickness

Properties

Section properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	8.970	
Moment of Inertia (local axes) (I)	[in4]	104.000	24.300
Moment of Inertia (principal axes) (I')	[in4]	104.000	24.300
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	3.405	1.646
Radius of gyration (principal axes) (r')	[in]	3.405	1.646
Saint-Venant torsion constant. (J)	[in4]	66.500	
Section warping constant. (Cw)	[in6]	48.521	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	20.800	12.100
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	20.800	12.100
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	20.800	12.100
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	20.800	12.100
Plastic section modulus (local axis) (Z)	[in3]	27.000	14.000
Plastic section modulus (principal axis) (Z')	[in3]	27.000	14.000
Polar radius of gyration. (ro)	[in]	3.784	
Area for shear (Aw)	[in2]	2.061	6.249
Torsional constant. (C)	[in3]	24.426	

Material : A500 GrB rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	46.00
Tensile strength (Fu):	[Kip/in2]	58.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

DESIGN CRITERIA

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	5.73

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
5.73	5.73

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
5.73	5.73	5.73	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway

Minor axis frame type : Sway

DESIGN CHECKS

AXIAL TENSION DESIGN ✔

Axial tension

Ratio : 0.00
Capacity : 371.36 [Kip]
Demand : 1.72 [Kip]
Reference : Eq. Sec. D2
Ctrl Eq. : D3 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	371.36	Eq. Sec. D2

AXIAL COMPRESSION DESIGN ✔

Compression in the major axis 33

Ratio : 0.01
Capacity : 361.31 [Kip]
Demand : 5.27 [Kip]
Reference : Sec. E1
Ctrl Eq. : D18 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	361.31	Sec. E1

Compression in the minor axis 22

Ratio : 0.02
Capacity : 330.22 [Kip]
Demand : 5.27 [Kip]
Reference : Sec. E1
Ctrl Eq. : D18 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	330.22	Sec. E1

FLEXURAL DESIGN ✔

Bending about major axis, M33

Ratio : 0.00
Capacity : 93.15 [Kip*ft]
Demand : 0.17 [Kip*ft]
Reference : Sec. F1
Ctrl Eq. : D1 at 50.00%

Intermediate results	Unit	Value	Reference
Section classification			

Factored yielding strength(ϕM_n):	[Kip*ft]	93.15	Sec. F1

Bending about minor axis, M22

Ratio	:	0.00		
Capacity	:	48.30 [Kip*ft]	Reference	: Sec. F1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(ϕM_n):	[Kip*ft]	48.30	Sec. F1

DESIGN FOR SHEAR ✔

Shear in major axis 33

Ratio	:	0.00		
Capacity	:	51.20 [Kip]	Reference	: Sec. G1
Demand	:	0.00 [Kip]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	51.20	Sec. G1

Shear in minor axis 22

Ratio	:	0.00		
Capacity	:	155.23 [Kip]	Reference	: Sec. G1
Demand	:	0.12 [Kip]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	155.23	Sec. G1

TORSION DESIGN ✔

Torsion

Ratio	:	0.00		
Capacity	:	50.56 [Kip*ft]	Reference	: Eq. H3-1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	: D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	50.56	Eq. H3-1

COMBINED ACTIONS DESIGN ✔

Combined flexure and axial compression

Ratio : 0.01
Ctrl Eq. : D9 at 50.00% Reference : Eq. H1-1b

Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.01	Eq. H1-1b

Combined flexure and axial tension

Ratio : 0.00
Ctrl Eq. : D3 at 50.00% Reference : Eq. H1-1b

Intermediate results	Unit	Value	Reference

Combined flexure and axial compression about local axis

Ratio : N/A
Ctrl Eq. : -- Reference :

Combined flexure and axial tension about local axis

Ratio : N/A
Ctrl Eq. : -- Reference :

Combined torsion, flexure, shear and axial compression

Ratio : N/A
Ctrl Eq. : -- Reference :

Combined torsion, flexure, shear and axial tension

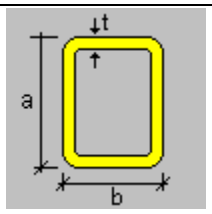
Ratio : N/A
Ctrl Eq. : -- Reference :

Member : 13 (BF-2 DIAGONALS)
Design status : OK

Section information

Section name: HSS_SQR 4X4X3_8 **(US)**

Dimensions



a	=	4.000	[in]	Height
b	=	4.000	[in]	Width
T	=	0.349	[in]	Thickness

Properties

Section properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	4.780	
Moment of Inertia (local axes) (I)	[in4]	10.300	10.300
Moment of Inertia (principal axes) (I')	[in4]	10.300	10.300
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	1.468	1.468
Radius of gyration (principal axes) (r')	[in]	1.468	1.468
Saint-Venant torsion constant. (J)	[in4]	17.500	
Section warping constant. (Cw)	[in6]	0.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	5.100	5.100
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	5.100	5.100
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	5.100	5.100
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	5.100	5.100
Plastic section modulus (local axis) (Z)	[in3]	6.400	6.400
Plastic section modulus (principal axis) (Z')	[in3]	6.400	6.400
Polar radius of gyration. (ro)	[in]	2.067	
Area for shear (Aw)	[in2]	2.061	2.061
Torsional constant. (C)	[in3]	9.136	

Material : A500 GrB rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	46.00
Tensile strength (Fu):	[Kip/in2]	58.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

DESIGN CRITERIA

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	15.03

Distance between member lateral bracing points

Length (Lb) [ft]	
Top	Bottom
15.03	15.03

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)

15.03	15.03	15.03	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

DESIGN CHECKS

AXIAL TENSION DESIGN ✔

Axial tension

Ratio	:	0.07			
Capacity	:	197.89 [Kip]	Reference	:	Eq. Sec. D2
Demand	:	13.83 [Kip]	Ctrl Eq.	:	D18 at 0.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	197.89	Eq. Sec. D2

AXIAL COMPRESSION DESIGN ✔

Compression in the major axis 33

Ratio	:	0.27			
Capacity	:	71.49 [Kip]	Reference	:	Sec. E1
Demand	:	19.48 [Kip]	Ctrl Eq.	:	D10 at 100.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	71.49	Sec. E1

Compression in the minor axis 22

Ratio	:	0.27			
Capacity	:	71.49 [Kip]	Reference	:	Sec. E1
Demand	:	19.48 [Kip]	Ctrl Eq.	:	D10 at 100.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	71.49	Sec. E1

FLEXURAL DESIGN ✔

Bending about major axis, M33

Ratio : 0.01
Capacity : 22.08 [Kip*ft] Reference : Sec. F1
Demand : 0.24 [Kip*ft] Ctrl Eq. : D10 at 50.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(ϕM_n):	[Kip*ft]	22.08	Sec. F1

Bending about minor axis, M22

Ratio : 0.00
Capacity : 22.08 [Kip*ft] Reference : Sec. F1
Demand : 0.00 [Kip*ft] Ctrl Eq. : D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(ϕM_n):	[Kip*ft]	22.08	Sec. F1

DESIGN FOR SHEAR ✔

Shear in major axis 33

Ratio : 0.00
Capacity : 51.20 [Kip] Reference : Sec. G1
Demand : 0.00 [Kip] Ctrl Eq. : D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	51.20	Sec. G1

Shear in minor axis 22

Ratio : 0.00
Capacity : 51.20 [Kip] Reference : Sec. G1
Demand : -0.06 [Kip] Ctrl Eq. : D10 at 100.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	51.20	Sec. G1

TORSION DESIGN ✔

Torsion

Ratio : 0.00
Capacity : 18.91 [Kip*ft] Reference : Eq. H3-1
Demand : 0.00 [Kip*ft] Ctrl Eq. : D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	18.91	Eq. H3-1

COMBINED ACTIONS DESIGN ✔

Combined flexure and axial compression

Ratio : 0.28
Ctrl Eq. : D10 at 50.00% Reference : Eq. H1-1a

Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.28	Eq. H1-1a

Combined flexure and axial tension

Ratio : 0.04
Ctrl Eq. : D18 at 50.00% Reference : Eq. H1-1b

Intermediate results	Unit	Value	Reference

Combined flexure and axial compression about local axis

Ratio : N/A
Ctrl Eq. : -- Reference :

Combined flexure and axial tension about local axis

Ratio : N/A
Ctrl Eq. : -- Reference :

Combined torsion, flexure, shear and axial compression

Ratio : N/A
Ctrl Eq. : -- Reference :

Combined torsion, flexure, shear and axial tension

Ratio : N/A
Ctrl Eq. : -- Reference :

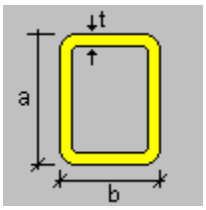
Member : 14 (BF-2 DIAGONALS)
Design status : OK

	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.-	
	Braced Frame 2, Base Plate & Anchor Bolts Design				4-56	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

Section information

Section name: HSS_SQR 4X4X3_8 **(US)**

Dimensions



a	=	4.000	[in]	Height
b	=	4.000	[in]	Width
T	=	0.349	[in]	Thickness

Properties

Section properties	Unit	Major axis	Minor axis
Gross area of the section. (Ag)	[in2]	4.780	
Moment of Inertia (local axes) (I)	[in4]	10.300	10.300
Moment of Inertia (principal axes) (I')	[in4]	10.300	10.300
Bending constant for moments (principal axis) (J')	[in]	0.000	0.000
Radius of gyration (local axes) (r)	[in]	1.468	1.468
Radius of gyration (principal axes) (r')	[in]	1.468	1.468
Saint-Venant torsion constant. (J)	[in4]	17.500	
Section warping constant. (Cw)	[in6]	0.000	
Distance from centroid to shear center (principal axis) (xo,yo)	[in]	0.000	0.000
Top elastic section modulus of the section (local axis) (Ssup)	[in3]	5.100	5.100
Bottom elastic section modulus of the section (local axis) (Sinf)	[in3]	5.100	5.100
Top elastic section modulus of the section (principal axis) (S'sup)	[in3]	5.100	5.100
Bottom elastic section modulus of the section (principal axis) (S'inf)	[in3]	5.100	5.100
Plastic section modulus (local axis) (Z)	[in3]	6.400	6.400
Plastic section modulus (principal axis) (Z')	[in3]	6.400	6.400
Polar radius of gyration. (ro)	[in]	2.067	
Area for shear (Aw)	[in2]	2.061	2.061
Torsional constant. (C)	[in3]	9.136	

Material : A500 GrB rectangular

Properties	Unit	Value
Yield stress (Fy):	[Kip/in2]	46.00
Tensile strength (Fu):	[Kip/in2]	58.00
Elasticity Modulus (E):	[Kip/in2]	29000.00
Shear modulus for steel (G):	[Kip/in2]	11153.85

DESIGN CRITERIA

Description	Unit	Value
Length for tension slenderness ratio (L)	[ft]	15.03

Distance between member lateral bracing points

Length (Lb) [ft]

Top	Bottom
15.03	15.03

Laterally unbraced length

Length [ft]			Effective length factor		
Major axis(L33)	Minor axis(L22)	Torsional axis(Lt)	Major axis(K33)	Minor axis(K22)	Torsional axis(Kt)
15.03	15.03	15.03	1.0	1.0	1.0

Additional assumptions

Continuous lateral torsional restraint	No
Tension field action	No
Continuous flexural torsional restraint	No
Effective length factor value type	None
Major axis frame type	Sway
Minor axis frame type	Sway

DESIGN CHECKS

AXIAL TENSION DESIGN ✔

Axial tension

Ratio : 0.07
Capacity : 197.89 [Kip]
Demand : 13.84 [Kip]
Reference : Eq. Sec. D2
Ctrl Eq. : D17 at 50.00%

Intermediate results	Unit	Value	Reference
Factored axial tension capacity(ϕP_n):	[Kip]	197.89	Eq. Sec. D2

AXIAL COMPRESSION DESIGN ✔

Compression in the major axis 33

Ratio : 0.27
Capacity : 71.49 [Kip]
Demand : 19.43 [Kip]
Reference : Sec. E1
Ctrl Eq. : D11 at 100.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n33}):	[Kip]	71.49	Sec. E1

Compression in the minor axis 22

Ratio : 0.27
Capacity : 71.49 [Kip]
Demand : 19.43 [Kip]
Reference : Sec. E1
Ctrl Eq. : D11 at 100.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored flexural buckling strength(ϕP_{n22}):	[Kip]	71.49	Sec. E1

FLEXURAL DESIGN ✔

Bending about major axis, M33

Ratio	:	0.02			
Capacity	:	22.08 [Kip*ft]	Reference	:	Sec. F1
Demand	:	-0.33 [Kip*ft]	Ctrl Eq.	:	D10 at 50.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(ϕM_n):	[Kip*ft]	22.08	Sec. F1

Bending about minor axis, M22

Ratio	:	0.00			
Capacity	:	22.08 [Kip*ft]	Reference	:	Sec. F1
Demand	:	0.00 [Kip*ft]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Section classification			
Factored yielding strength(ϕM_n):	[Kip*ft]	22.08	Sec. F1

DESIGN FOR SHEAR ✔

Shear in major axis 33

Ratio	:	0.00			
Capacity	:	51.20 [Kip]	Reference	:	Sec. G1
Demand	:	0.00 [Kip]	Ctrl Eq.	:	D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	51.20	Sec. G1

Shear in minor axis 22

Ratio	:	0.00			
Capacity	:	51.20 [Kip]	Reference	:	Sec. G1
Demand	:	0.07 [Kip]	Ctrl Eq.	:	D10 at 50.00%

Intermediate results	Unit	Value	Reference
Factored shear capacity(ϕV_n):	[Kip]	51.20	Sec. G1

	Project			Job Ref.	
AECOM	McDonalds Lillington, NC			032-0312	
	Section			Sheet no./rev.-	
	Braced Frame 2, Base Plate & Anchor Bolts Design			4-59	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

TORSION DESIGN ✔

Torsion

Ratio : 0.00
Capacity : 18.91 [Kip*ft]
Demand : 0.00 [Kip*ft]
Reference : Eq. H3-1
Ctrl Eq. : D1 at 0.00%

Intermediate results	Unit	Value	Reference
Factored torsion capacity(ϕT_n):	[Kip*ft]	18.91	Eq. H3-1

COMBINED ACTIONS DESIGN ✔

Combined flexure and axial compression

Ratio : 0.27
Ctrl Eq. : D11 at 68.75%
Reference : Eq. H1-1a

Intermediate results	Unit	Value	Reference
Interaction of flexure and axial force:	--	0.27	Eq. H1-1a

Combined flexure and axial tension

Ratio : 0.05
Ctrl Eq. : D17 at 50.00%
Reference : Eq. H1-1b

Intermediate results	Unit	Value	Reference

Combined flexure and axial compression about local axis

Ratio : N/A
Ctrl Eq. : --
Reference :

Combined flexure and axial tension about local axis

Ratio : N/A
Ctrl Eq. : --
Reference :

Combined torsion, flexure, shear and axial compression

Ratio : N/A
Ctrl Eq. : --
Reference :

Combined torsion, flexure, shear and axial tension

Ratio : N/A
Ctrl Eq. : --
Reference :

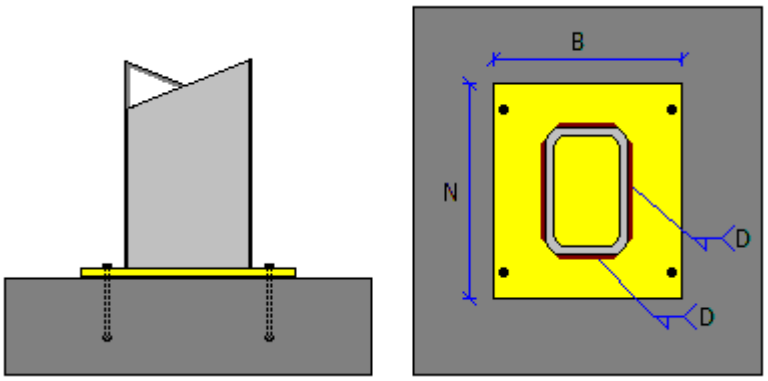
Steel Connections Data

Connection: 2 - Pinned BP - HSS Member

Family: Column - Base (CB)
Type: Base plate
Description: Smart Pinned Base Plate 2

General information

Connector



Members

Column

Column type	:	Prismatic member
Section	:	HSS_SQR 4X4X3_8
Material	:	A500 GrB rectangular
Longitudinal offset	:	0 in

Base plate

Base plate

Plate shape	:	Rectangular
Connection type	:	Unstiffened
Position on the support	:	Center
N: Longitudinal dimension	:	12 in
B: Transversal dimension	:	12 in
Thickness	:	1.5 in
Material	:	A36
Column weld	:	E70XX
D: Column weld size (1/16 in)	:	5
Override A2/A1 ratio	:	No
Include shear lug	:	No

Support

With pedestal	:	No
Longitudinal dimension	:	60 in
Transversal dimension	:	60 in
Thickness	:	36 in
Material	:	C 3-60
Include grouting	:	No
Anchor		
Anchor position	:	Longitudinal position
Rows number per side	:	1
Anchors per row	:	2
Longitudinal edge distance on the plate	:	2 in
Transverse edge distance on the plate	:	2 in
Anchor type	:	Headed
Head type	:	Hexagonal
Include lock nut	:	No
Anchor	:	1 1/4"
Effective embedment depth	:	18 in
Total length	:	21.15 in
Material	:	F1554 Gr36
Fy	:	36 kip/in2
Fu	:	58 kip/in2
Cracked concrete	:	Yes
Brittle steel	:	No
Anchors welded to base plate	:	No
Anchor reinforcement		
Type of reinforcement	:	Primary
Tension reinforcement	:	No
Shear reinforcement	:	No

Steel Connections
Results

Connection: 2 - Pinned BP - HSS Member

Family: Column - Base (CB)
Type: Base plate
Description: Smart Pinned Base Plate 2

Design code: AISC 360-10 LRFD, ACI 318-11

Demands

Description	Pu [kip]	Mu22 [kip*ft]	Mu33 [kip*ft]	Vu2 [kip]	Vu3 [kip]	Load type
DL	-8.55	0.00	0.00	0.00	0.84	Design
SL	-4.71	0.00	0.00	0.00	0.47	Design
WL1	-29.33	0.00	0.00	0.00	6.09	Design
WL2	29.33	0.00	0.00	0.00	-6.00	Design
EQ1	-13.17	0.00	0.00	0.00	2.74	Design
EQ2	13.17	0.00	0.00	0.00	-2.69	Design
D1	-11.97	0.00	0.00	0.00	1.18	Design
D2	-12.62	0.00	0.00	0.00	1.25	Design

D3	-17.80	0.00	0.00	0.00	1.76	Design
D4	-24.93	0.00	0.00	0.00	4.06	Design
D5	4.40	0.00	0.00	0.00	-1.98	Design
D6	-32.46	0.00	0.00	0.00	4.81	Design
D7	-3.14	0.00	0.00	0.00	-1.24	Design
D8	-39.59	0.00	0.00	0.00	7.11	Design
D9	19.06	0.00	0.00	0.00	-4.98	Design
D10	-41.95	0.00	0.00	0.00	7.34	Design
D11	16.71	0.00	0.00	0.00	-4.75	Design
D12	-11.21	0.00	0.00	0.00	1.11	Design
D13	-23.44	0.00	0.00	0.00	3.75	Design
D14	2.48	0.00	0.00	0.00	-1.64	SeismicLoadEffects
D15	-24.81	0.00	0.00	0.00	3.89	Design
D16	1.54	0.00	0.00	0.00	-1.54	SeismicLoadEffects
D17	-37.03	0.00	0.00	0.00	6.85	Design
D18	21.63	0.00	0.00	0.00	-5.24	Design
D19	-20.44	0.00	0.00	0.00	3.45	Design
D20	5.90	0.00	0.00	0.00	-1.98	SeismicLoadEffects

Design calculations

Design for major axis

Base plate (AISC 360-10 LRFD)

Geometric Considerations

Dimensions	Unit	Value	Min.	Max.	Sta.	References
Base plate						
Distance from anchor to edge	[in]	1.37	0.25	--	✔	
Weld size	[1/16in]	5	3	--	✔	table J2.4

Design Check

Verification	Unit	Capacity	Demand	Ctrl EQ	Ratio	References
Concrete base						
Axial bearing	[Kip/in2]	3.32	0.29	D10	0.09	DG1 3.1.1
Base plate						
Flexural yielding (bearing interface)	[Kip*ft/ft]	18.23	2.45	D10	0.13	DG1 Sec 3.1.2
Flexural yielding (tension interface)	[Kip*ft/ft]	18.23	3.76	WL2	0.21	DG1 Eq. 3.3.13
Column						
Weld capacity	[Kip/ft]	125.29	17.60	WL2	0.14	HSS Manual p. 7-10
Elastic method weld shear and axial capacity	[Kip/ft]	125.29	29.80	WL2	0.24	Sec. J2.4
Ratio	**0.24**					

Anchors

Geometric Considerations

Dimensions	Unit	Value	Min.	Max.	Sta.	References

	Project			Job Ref.	
AECOM	McDonalds Lillington, NC			032-0312	
	Section			Sheet no./rev.-	
	Braced Frame 2, Base Plate & Anchor Bolts Design			4-63	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

Anchors

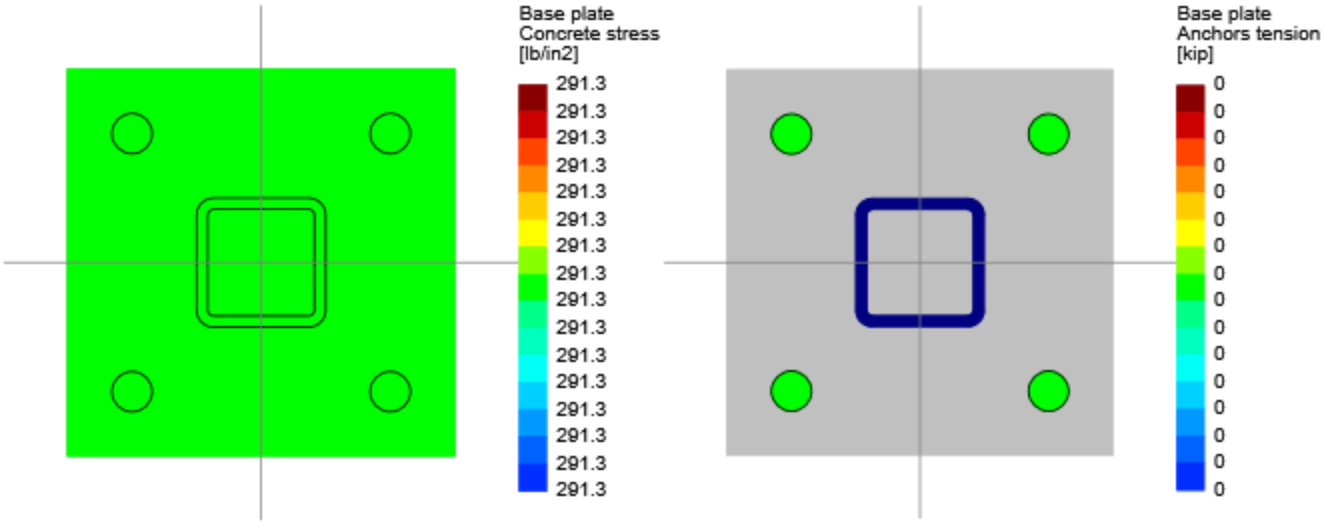
Anchor spacing	[in]	8.00	5.00	--	✔	Sec. D.8.1
Concrete cover	[in]	25.37	3.00	--	✔	Sec. 7.7.1
Effective length	[in]	24.81	--	35.19	✔	

Design Check

Verification	Unit	Capacity	Demand	Ctrl EQ	Ratio	References
Anchor tension	[Kip]	42.16	7.33	WL2	0.17	Eq. D-2
Breakout of anchor in tension	[Kip]	80.58	7.33	WL2	0.09	Sec. D.3.3.4.4
Breakout of group of anchors in tension	[Kip]	94.82	29.33	WL2	0.31	Sec. D.3.3.4.4
Pullout of anchor in tension	[Kip]	30.54	7.33	WL2	0.24	Sec. D.3.3.4.4
Anchor shear	[Kip]	21.92	0.00	DL	0.00	Eq. D-29
Breakout of anchor in shear	[Kip]	30.99	0.00	DL	0.00	Table D.4.1.1, Sec. D.4.3
Pryout of anchor in shear	[Kip]	161.17	0.00	DL	0.00	Table D.4.1.1, Sec. D.4.3
Ratio	**0.31**					
Global critical strength ratio	**0.31**					

Major axis

Maximum compression (D10)

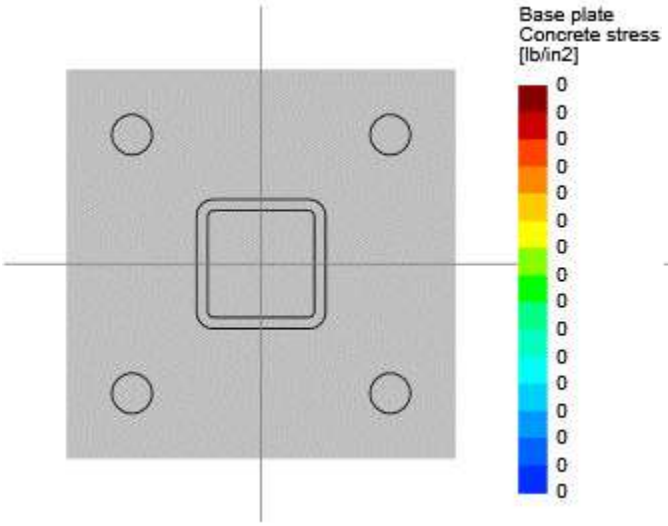


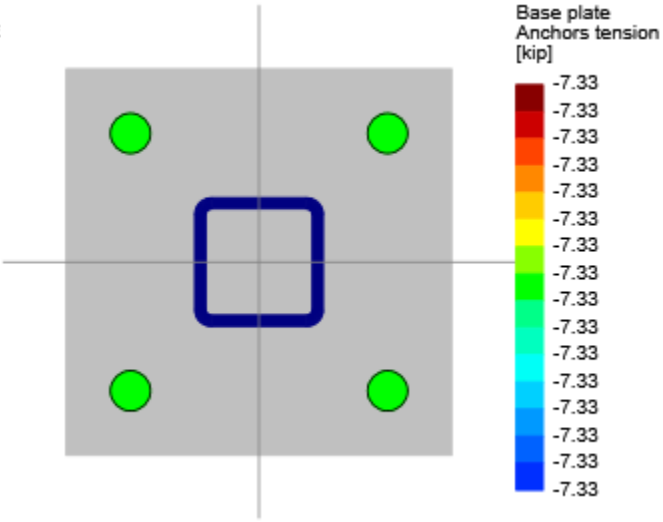
Maximum bearing pressure	291.30	[lb/in2]
Minimum bearing pressure	291.30	[lb/in2]
Maximum anchor tension	0.00	[kip]
Minimum anchor tension	0.00	[kip]
Neutral axis angle	0.00	[deg]
Neutral axis location	1.2E31	[in]
Bearing length	12.00	[in]

Anchors tensions

Anchor	Transverse [in]	Longitudinal [in]	Shear [kip]	Tension [kip]
1	-4.00	-4.00	0.00	0.00
2	-4.00	4.00	0.00	0.00
3	4.00	4.00	0.00	0.00
4	4.00	-4.00	0.00	0.00

Maximum tension (WL2)





Maximum bearing pressure	0.00	[lb/in2]
Minimum bearing pressure	0.00	[lb/in2]
Maximum anchor tension	7.33	[kip]
Minimum anchor tension	7.33	[kip]
Neutral axis angle	0.00	[deg]
Neutral axis location	-1.2E31	[in]
Bearing length	0.00	[in]

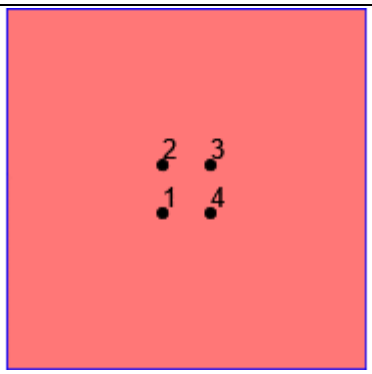
Anchors tensions

Anchor	Transverse [in]	Longitudinal [in]	Shear [kip]	Tension [kip]
1	-4.00	-4.00	0.00	7.33
2	-4.00	4.00	0.00	7.33
3	4.00	4.00	0.00	7.33
4	4.00	-4.00	0.00	7.33

Major axis

Results for tensile breakout (WL2)

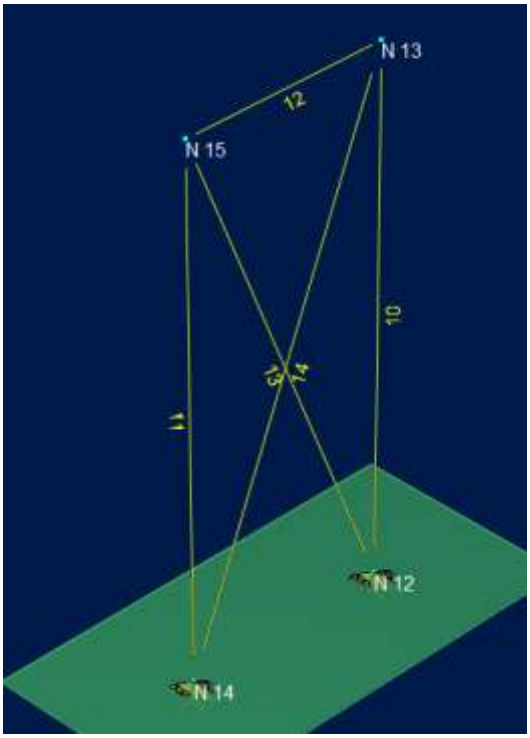
	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.-	
	Braced Frame 2, Base Plate & Anchor Bolts Design				4-65	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024



Group	Area [in2]	Tension [kip]	Anchors
1	3600.00	29.33	1, 2, 3, 4

	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	BF2 connection check				4-68	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

BF2 CONNECTION CHECK



Per RAM-Element design report, the maximum compression/tension force on HSS beam (member 12) is 5.28 kips,

ϕRn = 1.392 * D * L = 1.392 * 3 * (4" * 2+ 10" *2) = 117 kips > 5.28 kips (OK)

Per RAM-Element design report, the maximum compression/tension force on HSS diagonals (member 13 & member 14) is 19.43 kips,

ϕRn = 1.392 * D * L = 1.392 * 4 * (4" * 4) = 89 kips > 19.43 kips (OK)

***See detail 2/S3.1 for additional information on BF-2**

	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	Drag Strut Connection for BF-1 & BF-2				4-69	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

DRAG STRUT CONNECTION FOR BF-1 (NDS)

Shear Force from wind V_{wy}= 63.43 kips

Allowable Shear Force on BF-1 Vasd= 0.5 * 63.43 * 0.6 = 19.029 kips (controls)

Shear Force from earthquake V_{wy}= 28.49 kips

Allowable Shear Force on BF-1 Vasd= 28.49 * 0.5 * 0.7 = 9.97 kips

ρ=1.0 (Per ASCE 12.3.4, redundancy factor ρ=1.0)

Overstrength factor already considered, see “Loads” page.

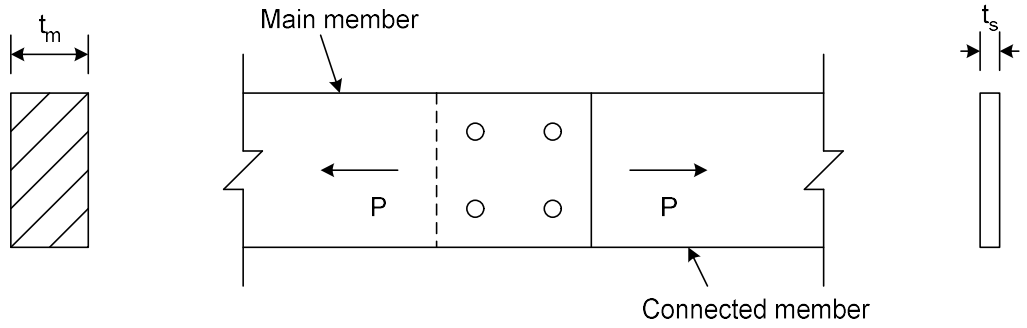
LAG SCREW TIMBER TO STEEL CONNECTION DESIGN

In accordance with NDS 2018

Tedds calculation version 1.2.04

Design results summary

	Unit	Required	Provided	Utilization	Result
Connection capacity	lbs	19029	24710	0.770	PASS



Main timber member details

Species of main member	**Hem-Fir**
Size of main member (Table 1B)	**4 x 12**
Number of main member	N_m = **1**
Thickness of main member	t_m = **3.500** in
Angle of load to grain of main member	θ_m = **0**°

Connected steel member details

Number of connected steel member	N_s = **1**
Connected steel member thickness	t_s = **0.250** in
Number of interfaces	$N_{int} = (N_m + N_s) - 1$ = **1**

Lag screw details

Lag screw diameter (Table L2)	**5/8"**
Lag screw root diameter of threaded portion	D_r = **0.471** in
Length of lag screw	L = **4.000** in
Washer thickness	W = **0.000** in
Number of rows of lag screw	R = **2**

Number of columns of lag screw — C = **11**

Total number of lag screw — $N_{total} = R \times C =$ **22**

Applied load

Applied load to the connection — P = **19029** lb

Dowel bearing length (main) (12.3.5)

Size of tapered tip — E = **0.406** in

Actual penetration in main member — $P_{act} = L - W - t_s =$ **3.750** in

Dowel bearing length in main member — $l_m =$ **3.344** in

Dowel bearing length (connected) (12.3.5)

Dowel bearing length in connected member — $l_s = t_s =$ **0.250** in

Bending yield strength (lag screw) (Table 12J to 12K footnote no. 2)

Bending yield strength of lag screw — $F_{yb} =$ **45000** psi

Dowel bearing strength (main member) (Table 12.3.3 footnote no. 2)

Dowel bearing strength parallel to grain — $F_{e\_par} = 11200 \times G_m \times 1$ psi = **4816** psi

Dowel bearing strength perpendicular to grain — $F_{e\_perp} = 6100 \times G_m^{1.45} \times 1$ psi $/ \sqrt{(D / 1 \text{ in})} =$ **2269** psi

Dowel bearing strength for small dia. fasteners — $F_e = 16600 \times G_m^{1.84} \times 1$ psi = **3513** psi

Dowel bearing strength at an angle of load to grain — $F_{e\theta m} = (F_{e\_par} \times F_{e\_perp}) / ((F_{e\_par} \times (\sin(\theta_m))^2) + (F_{e\_perp} \times (\cos(\theta_m))^2))$

$F_{e\theta m} =$ **4816** psi

Dowel bearing strength of main member — $F_{em} =$ **4816** psi

Dowel bearing strength (connected steel member) (Table 12K footnote no. 2)

Dowel bearing strength of connected member — $F_{es} =$ **87000** psi

Preliminary yield limit equation coefficients (Table 12.3.1A notes)

Dowel bearing strength ratio — $R_e = F_{em} / F_{es} =$ **0.055**

Dowel bearing length ratio — $R_t = l_m / l_s =$ **13.375**

Preliminary yield limit equation coefficient k_1 — $k_1 = ((\sqrt{(R_e+(2\times R_e^2\times(1+R_t+R_t^2))+(R_t^2\times R_e^3)))}-(R_e\times(1+R_t)))/(1+R_e)$

$k_1 =$ **0.314**

Preliminary yield limit equation coefficient k_2 — $k_2 = -1+\sqrt{((2\times(1+R_e))+((2\times F_{yb}\times(1+(2\times R_e))\times D_r^2))/(3\times F_{em}\times l_m^2))}$

$k_2 =$ **0.499**

Preliminary yield limit equation coefficient k_3 — $k_3 = -1+\sqrt{(((2\times(1+R_e))/R_e)+((2\times F_{yb}\times(2+R_e)\times D_r^2))/(3\times F_{em}\times l_s^2))}$

$k_3 =$ **8.142**

Angle of load to grain coefficient k_θ — $k_\theta = 1 + (0.25 \times \max(\theta_m, \theta_s) / 90) =$ **1.000**

Reduction term (Table 12.3.1B) — $K_D =$ **3.00**

Yield limit equations (single shear)

Mode l_m (eq. 12.3-1) — $Z_{lm} = (D_r \times l_m \times F_{em}) / (4 \times k_\theta) =$ **1896** lb

Mode l_s (eq. 12.3-2) — $Z_{ls} = (D_r \times l_s \times F_{es}) / (4 \times k_\theta) =$ **2561** lb

Mode II (eq. 12.3-3) — $Z_{II} = (k_1 \times D_r \times l_s \times F_{es}) / (3.6 \times k_\theta) =$ **893** lb

Mode III_m (eq. 12.3-4) — $Z_{IIIm} = (k_2 \times D_r \times l_m \times F_{em}) / ((1 + (2 \times R_e)) \times 3.2 \times k_\theta) =$ **1066** lb

Mode III_s (eq. 12.3-5) — $Z_{IIIs} = (k_3 \times D_r \times l_s \times F_{em}) / ((2 + R_e) \times 3.2 \times k_\theta) =$ **702** lb

Mode IV (eq. 12.3-6) — $Z_{IV} = (D_r^2 \times (\sqrt{((2 \times F_{em} \times F_{yb}) / (3 \times (1 + R_e)))})) / (3.2 \times k_\theta) =$ **811** lb

$Z = \min(Z_{lm}, Z_{ls}, Z_{II}, Z_{IIIm}, Z_{IIIs}, Z_{IV}) =$ **702** lb

AECOM	Project: McDonalds Lillington, NC				Job Ref.: 032-0312	
	Section: Drag Strut Connection for BF-1 & BF-2				Sheet no./rev.: 4-71	
	Calc. by: SA/PB	Date: 8/7/2024	Chk'd by: ES	Date: 8/7/2024	App'd by: MY	Date: 8/7/2024

Nominal capacity of single fastener	Z = **702** lb
Slenderness (Table 12.5.1C footnote no.1)	
Slenderness	l / D = **5.350**
Spacing requirements (parallel to grain loading)	
End distance (Table 12.5.1A)	
End distance (full strength)	a_{p_full} = **4.375** in
End distance (minimum)	a_{p_min} = **2.188** in
End distance (actual)	a_p = **5.000** in
Edge distance (Table 12.5.1C)	
Edge distance	e = (1.5 × D) = 0.938 in
Center to center spacing (Table 12.5.1B)	
Center to center spacing (full strength)	$s_{\_full}$ = 4 × D = **2.500** in
Center to center spacing (minimum)	$s_{\_min}$ = 3 × D = **1.875** in
Center to center spacing (actual)	s = **5.000** in
Row spacing (Table 12.5.1D)	
Row spacing	s_{row} = 1.5 × D = **0.938** in
Geometry factor C_Δ (12.5.1)	
End distance (actual)	a_p = **5.000** in
End distance (full strength)	a_{p_full} = **4.375** in
Geometry factor for end distance	$C_{\Delta 1} = a_p / a_{p\_full}$ = **1.14**
Center to center spacing (actual)	s = **5.000** in
Center to center spacing (full strength)	$s_{\_full}$ = **2.500** in
Geometry factor for spacing	$C_{\Delta 2} = s / s_{\_full}$ = **2.00**
Geometry factor	$C_\Delta = \min(1, C_{\Delta 1}, C_{\Delta 2})$ = **1.00**
Adjustment factor	
Load duration factor (Table 2.3.2)	C_D = **1.60**
Wet service factor (Table 11.3.3)	C_M = **1.0**
Temperature factor (Table 11.3.4)	C_t = **1.0**
Group action factor (eq. 11.3-1)	C_g = **1.0**
Geometry factor (12.5.1)	C_Δ = **1.00**
End grain factor (12.5.2)	C_{eg} = **1.0**
Diaphragm factor (12.5.3)	C_{di} = **1.0**
Toe nail factor (12.5.4)	C_{tn} = **1.0**
Total capacity of connection	
Capacity of connection	$Z' = Z \times N_{total} \times N_{int} \times C_D \times C_M \times C_\Delta$= **24710** lb
	P / Z' = **0.770**
Design result	
	PASS - Connection capacity exceeds applied load

	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	Drag Strut Connection for BF-1 & BF-2				4-72	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

Splice Connection Strength

Strap strength — S_{strap} = **5080** lb

MSTI60 + 60-10d (fasteners have 1.6 load duration factor)

Number of straps required — n_{strap} = ceiling(P/S_{strap},1) = 4

Wood screw strength — S_{screw} = 165 lb × 1.6 = **264.000** lb

SDS ¼" diam x 4 ½" wood screw into
½" wood side member(sheathing)
with 1.6 load duration factor

Number of screws required — n_{screw} = ceiling(P/S_{screw},1) = 72 (Use 80 screws)

AECOM	Project: McDonalds Lillington, NC				Job Ref.: 032-0312	
	Section: Drag Strut Connection for BF-1 & BF-2				Sheet no./rev.: 4-73	
	Calc. by: SA/PB	Date: 8/7/2024	Chk'd by: ES	Date: 8/7/2024	App'd by: MY	Date: 8/7/2024

HRS/ST/PS/HST/HTP/LSTA/LSTI/MST/MSTA/MSTC/MSTI *Strap Ties*

SIMPSON Strong-Tie

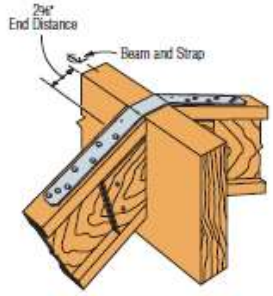
CODES: See page 20 for Code Reference Key Chart.

These products are available with additional corrosion protection. Additional products on this page may also be available with this option, check with Simpson Strong-Tie for details.

These products are approved for installation with the Strong-Drive SD Structural-Connector screw. See page 30 for the correct substitution and SD screw size.

Model No.	Ga	Dimensions W	Dimensions L	Fasteners (Total)	Allowable Tension Loads (DF/SP) (160)	Allowable Tension Loads (SPF/HF) (160)	Code Ref.
LSTA9	20	1¼	9	8-10d	740	635	
LSTA12		1¼	12	10-10d	925	795	
LSTA15		1¼	15	12-10d	1110	950	
LSTA18		1¼	18	14-10d	1235	1110	
LSTA21		1¼	21	16-10d	1235	1235	
LSTA24		1¼	24	18-10d	1235	1235	
ST292		2⅛	9⅛	12-16d	1265	1120	
ST2122		2⅛	12⅛	16-16d	1530	1505	
ST2115		¾	16⅛	10-16d	660	660	
ST2215		2⅛	16⅛	20-16d	1875	1880	
LSTA30	18	1¼	30	22-10d	1640	1640	I4, L3, F2
LSTA36		1¼	36	24-10d	1640	1640	
LSTI49		3¼	49	32-10dx1½	2975	2555	
LSTI73		3¼	73	48-10dx1½	4205	3830	
MSTA9		1¼	9	8-10d	750	645	
MSTA12		1¼	12	10-10d	940	810	
MSTA15		1¼	15	12-10d	1130	970	
MSTA18		1¼	18	14-10d	1315	1130	
MSTA21		1¼	21	16-10d	1505	1290	
MSTA24		1¼	24	18-10d	1640	1455	
MSTA30	16	1¼	30	22-10d	2050	1820	
MSTA36		1¼	36	26-10d	2050	2050	
MSTA49		1¼	49	26-10d	2020	2020	F26, L3
ST6215		2⅛	16⅛	20-16d	2095	1900	I4, IL14, L3, F2
ST6224		2⅛	23⅜	28-16d	2540	2540	I4, L3, F2
ST9		1¼	9	8-16d	885	760	
ST12		1¼	11⅜	10-16d	1105	950	
ST18		1¼	17¾	14-16d	1420	1330	
ST22		1¼	21⅜	18-16d	1420	1420	I4, L3, F2
MSTC28		3	28¼	36-16d sinkers	3455	2980	
MSTC40		3	40¼	52-16d sinkers	4745	4305	
MSTC52		3	52¼	62-16d sinkers	4745	4745	
HTP37Z		3	7	20-10dx1½	1850	1600	170
MSTC66	14	3	65¾	76-16d sinkers	5860	5860	
MSTC78		3	77¾	76-16d sinkers	5860	5860	I4, L3, F2
ST6236		2⅛	33⅜	40-16d	3845	3845	
HRS6	12	1⅜	6	6-10d	605	525	
HRS8		1⅜	8	10-10d	1010	880	F26
HRS12		1⅜	12	14-10d	1415	1230	
MSTI26		2⅛	26	26-10dx1½	2745	2325	
MSTI36		2⅛	36	36-10dx1½	3800	3220	
MSTI48		2⅛	48	48-10dx1½	5065	4290	I4, L3, F2
MSTI60		2⅛	60	60-10dx1½	5080	5080	
MSTI72		2⅛	72	72-10dx1½	5080	5080	
HRS416Z		3¼	16	16-SDS ¼"x1½"	2835	2305	170

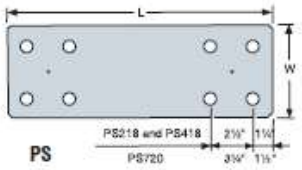
1. Loads include a 60% load duration increase on the fasteners for wind or earthquake loading.
2. 10dx1½" nails may be substituted where 16d sinkers or 10d are specified at 100% of the table loads except where straps are installed over sheathing.
3. 10d commons may be substituted where 16d sinkers are specified at 100% of table loads.
4. 16d sinkers (0.148" dia. x 3¼" long) or 10d commons may be substituted where 16d commons are specified at 0.84 of the table loads.
5. Use half of the nails in each member being connected to achieve the listed loads.
6. Tension loads apply for uplift when installed vertically.
7. **NAILS:** 16d = 0.162" dia. x 3½" long, 16d Sinker = 0.148" dia. x 3¼" long, 10d = 0.148" dia. x 3" long. 10dx1½ = 0.148" dia. x 1½" long. See page 24-25 for other nail sizes and information.



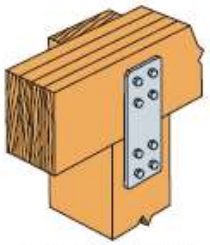
Typical LSTA Installation
(Hanger not shown)
Bend strap one time only

Model No.	Material Thickness Gauge	Dim. W	Dim. L	Bolts Qty	Bolts Dia	Code Ref.
PS218	7 ga	2	18	4	¾	180
PS418		4	18	4	¾	
PS720		6¼	20	8	½	

1. PS strap design loads must be determined by the Designer for each installation. Bolts are installed both perpendicular and parallel-to-grain. Hole diameter in the part may be oversized to accommodate the HDG. Designer must determine if the oversize creates an unacceptable installation.



PS



Typical PS720 Installation

Simpson Strong-Tie® Fastening Systems

STRONG-DRIVE® STRUCTURAL SCREWS

Strong-Drive® SDS Structural Wood Screw (cont.)

Allowable Shear Loads for Steel Side-Plate Applications

Size (in.)	Finish/ Material	Model No.	Thread Length (in.)	DF/SP Allowable Loads[3] Steel Side Plate Shear (100)[1]			SPF/HF Allowable Loads[3] Steel Side Plate Shear (100)[1]		
				16 ga	14 ga & 12 ga	10 ga or Greater	16 ga	14 ga & 12 ga	10 ga or Greater
¼ x 1½	Double-Barrier Coating	SDS25112	1	250	250	250	180	180	180
¼ x 2		SDS25200	1¼	250	290	290	180	210	210
¼ x 2½		SDS25212	1½	250	390	420	180	280	300
¼ x 3		SDS25300	2	250	420	420	180	300	300
¼ x 3½		SDS25312	2¼	250	420	420	180	300	300
¼ x 4½		SDS25412	2¾	250	420	420	180	300	300
¼ x 5		SDS25500	2¾	250	420	420	180	300	300
¼ x 6		SDS25600	3¼	250	420	420	180	300	300
¼ x 8		SDS25800	3¼	250	420	420	180	300	300
¼ x 1½	Type-316 Stainless Steel	SDS25112SS	1	250	250	250	180	180	180
¼ x 2		SDS25200SS	1¼	250	290	290	180	210	210
¼ x 2½		SDS25212SS	1½	250	390	420	180	280	300
¼ x 3		SDS25300SS	2	250	420	420	180	300	300
¼ x 3½		SDS25312SS	2¼	250	420	420	180	300	300

1. Allowable loads for SDS screws are based on ICC-ES Code Report ESR-2236. Screws may be provided with the 4CUT™ or Type-17 point.
2. SDS screws install best with a low speed ⅛" drill with a ⅜" hex head driver.
3. Allowable loads are shown at the wood load duration factor of C_D = 1.00. Loads may be increased for load duration up to a C_D = 1.60.
4. Allowable withdrawal load for DF/SP/SCL is 172 lbs./in. and for SPF/HF withdrawal is 150 lbs./in.. Total withdrawal load is based on actual thread penetration into the main member.
5. LSL wood-to-wood applications that require 4½", 5", 6" and 8" SDS screws are limited to interior-dry use only.
6. Minimum spacing requirements are listed in ICC-ES ESR-2236.

Allowable Shear Loads for Wood Side-Plate Applications

Size (in.)	Model No.	DF/SP Allowable Loads[3,4] Wood Side Plate Thickness (in.)												
		1½	1⅝	1¾	2	2½	2¾	3	3½	3⅝	4	4½	5	5½
¼ x 2	SDS25200	145	•	•	•	•	•	•	•	•	•	•	•	•
¼ x 2½	SDS25212	165	165	170	165	•	•	190[1]	•	•	•	•	•	•
¼ x 3	SDS25300	165	165	170	185	195	205	280[1]	•	•	•	•	•	•
¼ x 3½	SDS25312	165	165	170	185	195	205	340[1]	340[1]	•	•	•	•	•
¼ x 4½	SDS25412	165	165	170	185	195	205	350[1]	340[1]	230	200	•	•	•
¼ x 5	SDS25500	165	165	170	185	195	205	350[1]	340[1]	230	230	200	•	•
¼ x 6	SDS25600	165	165	170	185	195	205	350[1]	340[1]	340[1]	340[1]	340[1]	230	200
¼ x 8	SDS25800	165	165	170	185	195	205	350[1]	340[1]	340[1]	340[1]	340[1]	230	230

Size (in.)	Model No.	SPF/HF Allowable Loads[4] Wood Side Plate Thickness (in.)												
		1½	1⅝	1¾	2	2½	2¾	3	3½	3⅝	4	4½	5	5½
¼ x 2	SDS25200	105	•	•	•	•	•	•	•	•	•	•	•	•
¼ x 2½	SDS25212	130	135	130	120	•	•	135[1]	•	•	•	•	•	•
¼ x 3	SDS25300	130	140	140	150	150	145	200[1]	•	•	•	•	•	•
¼ x 3½	SDS25312	130	140	140	150	155	165	245[1]	245[1]	•	•	•	•	•
¼ x 4½	SDS25412	130	140	140	150	155	165	250[1]	245[1]	190	160	•	•	•
¼ x 5	SDS25500	130	140	140	150	155	165	250[1]	245[1]	190	190	160	•	•
¼ x 6	SDS25600	130	140	140	150	155	165	250[1]	245[1]	245[1]	245[1]	245[1]	190	160
¼ x 8	SDS25800	130	140	140	150	155	165	250[1]	245[1]	245[1]	245[1]	245[1]	195	195

1. Noted loads are based on ICC-ES Code Report ESR-2236 and/or testing per ICC AC233 and assume a minimum main member thickness of the screw length minus the side member thickness. All other allowable loads are based on the 2005 National Design Specification (NDS) and a minimum penetration of 6D = 1.45" into the main member.
2. Values are valid for a connection involving only two members. Where the side and main members have different specific gravities, the lower values shall be used.
3. Allowable loads are also applicable to structural composite lumber *(e.g., LVL, PSL, and LSL)* having an equivalent specific gravity of 0.50 or greater.
4. Allowable loads are shown at the wood load duration factor of C_D = 1.00. Loads may be increased for load duration by the building code up to a C_D = 1.60. The Designer shall apply all adjustment factors required per NDS.
5. Loads are based on installation into the side grain of the wood members with the screw axis perpendicular to the wood fibers.
6. Loads apply to appropriate stainless-steel models.

	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	Drag Strut Connection for BF-1 & BF-2				4-75	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

COMPRESSION CHECK FOR DRAG STRUT MEMBER FOR BF-1 (NDS)

Allowable Shear Force on BF-1 Vasd= 0.5 * 63.43 * 0.6 = 19.029 kips (controls)

WOOD MEMBER DESIGN (NDS 2018)

In accordance with the ANSI/AF&PA NDS 2018 using the ASD method

Tedds calculation version 2.2.20

Design summary

Overall design utilisation	0.704
Overall design status	PASS

Design section s1 results summary	Unit	Capacity	Maximum	Utilization	Result
Compressive stress	lb/in^2	801	564	0.704	PASS

Design section 1

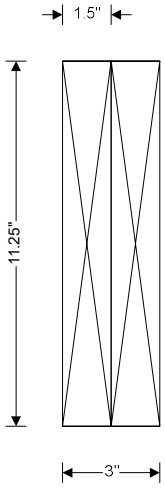
User note: Check column at base

Member details

Service condition	Dry
Load duration - Table 2.3.2	Ten minutes

Sawn lumber section details

Number of sections in member	N = **2**
Nominal breadth of sections	b_{nom} = **2** in
Breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **12** in
Depth of sections	d = **11.25** in
Material	**Hem-Fir, 2" && wider, No.1 grade**



2/2"x12" sawn lumber sections
Cross-sectional area, A, 33.75 in^2
Section modulus, S_x, 63.3 in^3
Section modulus, S_y, 16.9 in^3
Second moment of area, I_x, 356 in^4
Second moment of area, I_y, 25.3 in^4
Radius of gyration, r_x, 3.248 in
Radius of gyration, r_y, 0.866 in
Hem-Fir, 2" & wider, No.1 grade
Bending, F_b, 975 psi
Shear parallel to grain, F_v, 150 psi
Compression parallel to grain, F_c, 1350 psi
Compression perpendicular to grain, F_{c_perp}, 405 psi
Tension parallel to grain, F_t, 625 psi
Modulus of elasticity, E, 1500000 psi
Minimum modulus of elasticity, E_{min}, 550000 psi
Density, ρ, 29.743 lbm/ft^3
Specific gravity, G, 0.43

Span details

Unbraced length - Major axis	L_x = **4** ft
Effective bending length - Major axis	$L_{e,x} = 1.63 \times L_x + 3 \times N \times b$ = **7.27** ft

Column buckling length - Major axis	$L_{b,x} = L_x =$ **4** ft
Unbraced length - Minor axis	$L_y =$ **4** ft
Effective bending length - Minor axis	$L_{e,y} = 1.63 \times L_y + 3 \times d =$ **9.333** ft
Column buckling length - Minor axis	$L_{b,y} = L_y =$ **4** ft

Analysis results

Design axial compression force	P = **19029** lb

Adjustment factors - Table 4.3.1

Load duration factor - Table 2.3.2	$C_D =$ **1.6**
Repetitive member factor - Table 4.3.9	$C_r =$ **1.15**
Column fixing factor for nails - cl.15.3.2	$K_f =$ **0.6**
Reference compression design value	$F_c^* = F_c \times C_D =$ **2160** lb/in^2
Adjusted modulus of elasticity	$E_{min}' = E_{min} =$ **550000** lb/in^2
Critical buckling design value	$F_{cE} = 0.822 \times E_{min}' / (L_{b,y} / (N \times b))^2 =$ **1766** lb/in^2
Column stability factor - eq.3.7-1	

$$C_P = K_f \times ((1 + (F_{cE} / F_c^*)) / 1.6 - \sqrt{((1 + (F_{cE} / F_c^*)) / 1.6)^2 - (F_{cE} / F_c^*) / 0.8)}) = \mathbf{0.371}$$

Compression members - General - cl.3.6

Design axial compression force	P = **19029** lb
Design compression parallel to grain - Table 4.3.1	$F_c' = F_c \times C_D \times C_P =$ **801** lb/in^2
Actual compression parallel to grain	$f_c = P / (N \times b \times d) =$ **564** lb/in^2
	$f_c / F_c' =$ **0.704**
	PASS - Design compression stress exceeds actual compression stress

DRAG STRUT CONNECTION FOR BF-2 (NDS)

Shear Force from wind V_{wy}= 63.43 kips

Allowable Shear Force on BF-2 Vasd= 0.19 * 63.43 * 0.6 = 7.25 kips (controls)

Shear Force from earthquake V_{wy}= 28.49 kips

Allowable Shear Force on BF-2 Vasd= 28.49 * 0.19 * 0.7 = 3.79 kips

ρ=1.0 (Per ASCE 12.3.4, redundancy factor ρ=1.0)

Overstrength factor already considered, see “Loads” page.

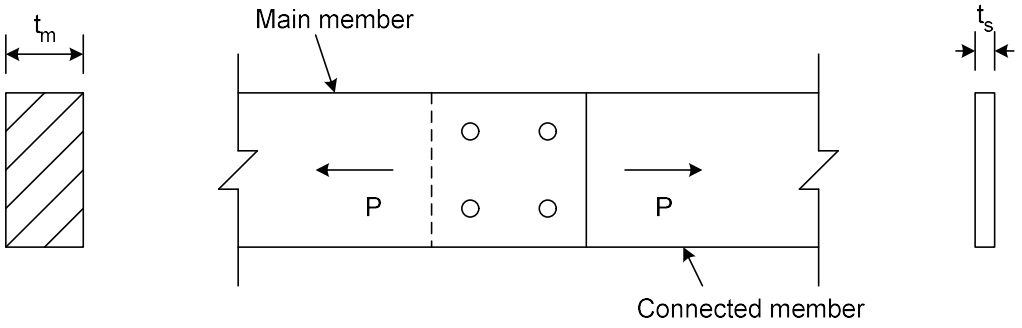
BOLTED TIMBER TO STEEL CONNECTION DESIGN

In accordance with NDS 2018

Tedds calculation version 1.2.04

Design results summary

	Unit	Required	Provided	Utilization	Result
Connection capacity	lbs	7250	12200	0.594	PASS



Main timber member details

Species of main member	**Hem-Fir**
Size of main member (Table 1B)	**2 x 8**
Number of main member	N_m = **1**
Thickness of main member	t_m = **1.500** in
Angle of load to grain of main member	θ_m = **0**°

Connected steel member details

Number of connected steel member	N_s = **1**
Connected steel member thickness	t_s = **0.375** in
Number of interfaces	$N_{int} = (N_m + N_s) - 1$ = **1**

Bolt details

Bolt diameter (Table L1)	**3/4"**
Number of rows of bolts	R = **5**
Number of columns of bolts	C = **2**

Total number of bolts $N_{total} = R \times C =$ **10**

Applied load

Applied load to the connection $P =$ **7250** lb

Dowel bearing length (main) (12.3.5)

Dowel bearing length in main member $l_m = t_m =$ **1.500** in

Dowel bearing length (connected) (12.3.5)

Dowel bearing length in connected member $l_s = t_s =$ **0.375** in

Bending yield strength (bolt) (Table 12A to 12I footnote no. 2)

Bending yield strength of bolt $F_{yb} =$ **45000** psi

Dowel bearing strength (main member) (Table 12.3.3 footnote no. 2)

Dowel bearing strength parallel to grain $F_{e\_par} = 11200 \times G_m \times 1\text{ psi} =$ **4816** psi

Dowel bearing strength perpendicular to grain $F_{e\_perp} = 6100 \times G_m^{1.45} \times 1\text{ psi} / \sqrt{(D / 1\text{ in})} =$ **2072** psi

Dowel bearing strength for small dia. fasteners $F_e = 16600 \times G_m^{1.84} \times 1\text{ psi} =$ **3513** psi

Dowel bearing strength at an angle of load to grain $F_{e\theta m} = (F_{e\_par} \times F_{e\_perp}) / ((F_{e\_par} \times (\sin(\theta_m))^2) + (F_{e\_perp} \times (\cos(\theta_m))^2))$

$F_{e\theta m} =$ **4816** psi

Dowel bearing strength of main member $F_{em} =$ **4816** psi

Dowel bearing strength (connected steel member) (Table 12K footnote no. 2)

Dowel bearing strength of connected member $F_{es} =$ **87000** psi

Preliminary yield limit equation coefficients (Table 12.3.1A notes)

Dowel bearing strength ratio $R_e = F_{em} / F_{es} =$ **0.055**

Dowel bearing length ratio $R_t = l_m / l_s =$ **4.000**

Preliminary yield limit equation coefficient k_1 $k_1 = ((\sqrt{(R_e+(2\times R_e^2\times(1+R_t+R_t^2))+(R_t^2\times R_e^3)))}-(R_e\times(1+R_t)))/(1+R_e)$

$k_1 =$ **0.147**

Preliminary yield limit equation coefficient k_2 $k_2 = -1+\sqrt{((2\times(1+R_e))+((2\times F_{yb}\times(1+(2\times R_e))\times D^2))/(3\times F_{em}\times l_m^2))}$

$k_2 =$ **0.960**

Preliminary yield limit equation coefficient k_3 $k_3 = -1+\sqrt{(((2\times(1+R_e))/R_e)+((2\times F_{yb}\times(2+R_e)\times D^2))/(3\times F_{em}\times l_s^2))}$

$k_3 =$ **8.452**

Angle of load to grain coefficient k_θ $k_\theta = 1 + (0.25 \times \max(\theta_m, \theta_s) / 90) =$ **1.000**

Yield limit equations (single shear)

Mode l_m (eq. 12.3-1) $Z_{lm} = (D \times l_m \times F_{em}) / (4 \times k_\theta) =$ **1355** lb

Mode l_s (eq. 12.3-2) $Z_{ls} = (D \times l_s \times F_{es}) / (4 \times k_\theta) =$ **6117** lb

Mode II (eq. 12.3-3) $Z_{II} = (k_1 \times D \times l_s \times F_{es}) / (3.6 \times k_\theta) =$ **1001** lb

Mode III_m (eq. 12.3-4) $Z_{IIIm} = (k_2 \times D \times l_m \times F_{em}) / ((1 + (2 \times R_e)) \times 3.2 \times k_\theta) =$ **1463** lb

Mode III_s (eq. 12.3-5) $Z_{IIIs} = (k_3 \times D \times l_s \times F_{em}) / ((2 + R_e) \times 3.2 \times k_\theta) =$ **1741** lb

Mode IV (eq. 12.3-6) $Z_{IV} = D^2 \times (\sqrt{((2 \times F_{em} \times F_{yb}) / (3 \times (1 + R_e)))}) / (3.2 \times k_\theta) =$ **2057** lb

$Z = \min(Z_{lm}, Z_{ls}, Z_{II}, Z_{IIIm}, Z_{IIIs}, Z_{IV}) =$ **1001** lb

Nominal capacity of single fastener $Z =$ **1001** lb

Slenderness (Table 12.5.1C footnote no.1)

Slenderness $l / D =$ **2.000**

AECOM	Project				Job Ref.	
	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	Drag Strut Connection for BF-1 & BF-2				4-79	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

Spacing requirements (parallel to grain loading)

End distance (Table 12.5.1A)

End distance (full strength)	a_{p_full} = **5.250** in
End distance (minimum)	a_{p_min} = **2.625** in
End distance (actual)	a_p = **4.000** in

Edge distance (Table 12.5.1C)

Edge distance	e = (1.5 × D) = 1.125 in

Center to center spacing (Table 12.5.1B)

Center to center spacing (full strength)	$s_{\_full}$ = 4 × D = **3.000** in
Center to center spacing (minimum)	$s_{\_min}$ = 3 × D = **2.250** in
Center to center spacing (actual)	s = **5.000** in

Row spacing (Table 12.5.1D)

Row spacing	s_{row} = 1.5 × D = **1.125** in

Geometry factor C_Δ (12.5.1)

End distance (actual)	a_p = **4.000** in
End distance (full strength)	a_{p_full} = **5.250** in
Geometry factor for end distance	$C_{\Delta 1} = a_p / a_{p\_full}$ = **0.76**
Center to center spacing (actual)	s = **5.000** in
Center to center spacing (full strength)	$s_{\_full}$ = **3.000** in
Geometry factor for spacing	$C_{\Delta 2} = s / s_{\_full}$ = **1.67**
Geometry factor	$C_\Delta = \min(1, C_{\Delta 1}, C_{\Delta 2})$ = **0.76**

Adjustment factor

Load duration factor (Table 2.3.2)	C_D = **1.60**
Wet service factor (Table 11.3.3)	C_M = **1.0**
Temperature factor (Table 11.3.4)	C_t = **1.0**
Group action factor (eq. 11.3-1)	C_g = **1.0**
Geometry factor (12.5.1)	C_Δ = **0.76**
End grain factor (12.5.2)	C_{eg} = **1.0**
Diaphragm factor (12.5.3)	C_{di} = **1.0**
Toe nail factor (12.5.4)	C_{tn} = **1.0**

Total capacity of connection

Capacity of connection	$Z' = Z \times N_{total} \times N_{int} \times C_D \times C_M \times C_\Delta$= **12200** lb
	P / Z' = **0.594**

Design result

PASS - Connection capacity exceeds applied load

	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	Drag Strut Connection for BF-1 & BF-2				4-80	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

Splice Connection Strength

Strap strength — S_{strap} = **5080** lb

MSTI60 + 60-10d (fasteners have 1.6 load duration factor)

Number of straps required — n_{strap} = ceiling(P/S_{strap},1) = 2

Wood screw strength — S_{screw} = 165 lb × 1.6 = **264.000** lb

SDS ¼" diam x 4 ½" wood screw into
½" wood side member(sheathing)
with 1.6 load duration factor

Number of screws required — n_{screw} = ceiling(P/S_{screw},1) = 28 (Use 40 screws)

AECOM	Project: McDonalds Lillington, NC				Job Ref.: 032-0312	
	Section: Drag Strut Connection for BF-1 & BF-2				Sheet no./rev.: 4-81	
	Calc. by: SA/PB	Date: 8/7/2024	Chk'd by: ES	Date: 8/7/2024	App'd by: MY	Date: 8/7/2024

HRS/ST/PS/HST/HTP/LSTA/LSTI/MST/MSTA/MSTC/MSTI *Strap Ties*

SIMPSON Strong-Tie

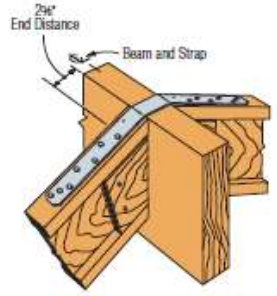
CODES: See page 20 for Code Reference Key Chart.

These products are available with additional corrosion protection. Additional products on this page may also be available with this option, check with Simpson Strong-Tie for details.

These products are approved for installation with the Strong-Drive SD Structural-Connector screw. See page 30 for the correct substitution and SD screw size.

Model No.	Ga	Dimensions W	Dimensions L	Fasteners (Total)	Allowable Tension Loads (DF/SP) (160)	Allowable Tension Loads (SPF/HF) (160)	Code Ref.
LSTA9	20	1¼	9	8-10d	740	635	
LSTA12		1¼	12	10-10d	925	795	
LSTA15		1¼	15	12-10d	1110	950	
LSTA18		1¼	18	14-10d	1235	1110	
LSTA21		1¼	21	16-10d	1235	1235	
LSTA24		1¼	24	18-10d	1235	1235	
ST292		2⅛	9⅛	12-16d	1265	1120	
ST2122		2⅛	12⅛	16-16d	1530	1505	
ST2115		¾	16⅛	10-16d	660	660	
ST2215		2⅛	16⅛	20-16d	1875	1880	
LSTA30	18	1¼	30	22-10d	1640	1640	I4, L3, F2
LSTA36		1¼	36	24-10d	1640	1640	
LSTI49		3¼	49	32-10dx1½	2975	2555	
LSTI73		3¼	73	48-10dx1½	4205	3830	
MSTA9		1¼	9	8-10d	750	645	
MSTA12		1¼	12	10-10d	940	810	
MSTA15		1¼	15	12-10d	1130	970	
MSTA18		1¼	18	14-10d	1315	1130	
MSTA21		1¼	21	16-10d	1505	1290	
MSTA24		1¼	24	18-10d	1640	1455	
MSTA30	16	1¼	30	22-10d	2050	1820	
MSTA36		1¼	36	26-10d	2050	2050	
MSTA49		1¼	49	26-10d	2020	2020	F26, L3
ST6215		2⅛	16⅛	20-16d	2095	1900	I4, IL14, L3, F2
ST6224		2⅛	23⅛	28-16d	2540	2540	I4, L3, F2
ST9		1¼	9	8-16d	885	760	
ST12		1¼	11⅞	10-16d	1105	950	
ST18		1¼	17¾	14-16d	1420	1330	
ST22		1¼	21⅞	18-16d	1420	1420	I4, L3, F2
MSTC28		3	28¼	36-16d sinkers	3455	2980	
MSTC40		3	40¼	52-16d sinkers	4745	4305	
MSTC52		3	52¼	62-16d sinkers	4745	4745	
HTP37Z		3	7	20-10dx1½	1850	1600	170
MSTC66	14	3	65¾	76-16d sinkers	5860	5860	
MSTC78		3	77¾	76-16d sinkers	5860	5860	I4, L3, F2
ST6236		2⅛	33⅞	40-16d	3845	3845	
HRS6	12	1⅜	6	6-10d	605	525	
HRS8		1⅜	8	10-10d	1010	880	F26
HRS12		1⅜	12	14-10d	1415	1230	
MSTI26		2⅛	26	26-10dx1½	2745	2325	
MSTI36		2⅛	36	36-10dx1½	3800	3220	
MSTI48		2⅛	48	48-10dx1½	5065	4290	I4, L3, F2
MSTI60		2⅛	60	60-10dx1½	5080	5080	
MSTI72		2⅛	72	72-10dx1½	5080	5080	
HRS416Z		3¼	16	16-SDS ¼"x1½"	2835	2305	170

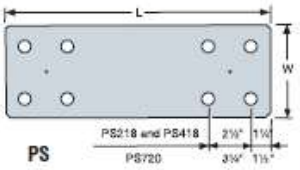
1. Loads include a 60% load duration increase on the fasteners for wind or earthquake loading.
2. 10dx1½" nails may be substituted where 16d sinkers or 10d are specified at 100% of the table loads except where straps are installed over sheathing.
3. 10d commons may be substituted where 16d sinkers are specified at 100% of table loads.
4. 16d sinkers (0.148" dia. x 3¼" long) or 10d commons may be substituted where 16d commons are specified at 0.84 of the table loads.
5. Use half of the nails in each member being connected to achieve the listed loads.
6. Tension loads apply for uplift when installed vertically.
7. **NAILS:** 16d = 0.162" dia. x 3½" long, 16d Sinker = 0.148" dia. x 3¼" long, 10d = 0.148" dia. x 3" long. 10dx1½ = 0.148" dia. x 1½" long. See page 24-25 for other nail sizes and information.



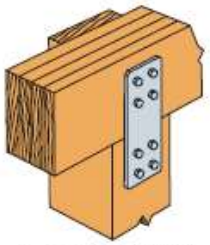
Typical LSTA Installation
(Hanger not shown)
Bend strap one time only

Model No.	Material Thickness Gauge	Dim. W	Dim. L	Bolts Qty	Bolts Dia	Code Ref.
PS218	7 ga	2	18	4	¾	180
PS418		4	18	4	¾	
PS720		6¼	20	8	½	

1. PS strap design loads must be determined by the Designer for each installation. Bolts are installed both perpendicular and parallel-to-grain. Hole diameter in the part may be oversized to accommodate the HDG. Designer must determine if the oversize creates an unacceptable installation.



PS



Typical PS720 Installation

AECOM	Project: McDonalds Lillington, NC				Job Ref.: 032-0312	
	Section: Drag Strut Connection for BF-1 & BF-2				Sheet no./rev.: 4-82	
	Calc. by: SA/PB	Date: 8/7/2024	Chk'd by: ES	Date: 8/7/2024	App'd by: MY	Date: 8/7/2024

Strong-Drive® SDS Structural Wood Screw (cont.)

Allowable Shear Loads for Steel Side-Plate Applications

Size (in.)	Finish/ Material	Model No.	Thread Length (in.)	DF/SP Allowable Loads[3] Steel Side Plate Shear (100)[1]			SPF/HF Allowable Loads[3] Steel Side Plate Shear (100)[1]		
				16 ga	14 ga & 12 ga	10 ga or Greater	16 ga	14 ga & 12 ga	10 ga or Greater
¼ x 1½	Double-Barrier Coating	SDS25112	1	250	250	250	180	180	180
¼ x 2		SDS25200	1¼	250	290	290	180	210	210
¼ x 2½		SDS25212	1½	250	390	420	180	280	300
¼ x 3		SDS25300	2	250	420	420	180	300	300
¼ x 3½		SDS25312	2¼	250	420	420	180	300	300
¼ x 4½		SDS25412	2¾	250	420	420	180	300	300
¼ x 5		SDS25500	2¾	250	420	420	180	300	300
¼ x 6		SDS25600	3¼	250	420	420	180	300	300
¼ x 8		SDS25800	3¼	250	420	420	180	300	300
¼ x 1½	Type-316 Stainless Steel	SDS25112SS	1	250	250	250	180	180	180
¼ x 2		SDS25200SS	1¼	250	290	290	180	210	210
¼ x 2½		SDS25212SS	1½	250	390	420	180	280	300
¼ x 3		SDS25300SS	2	250	420	420	180	300	300
¼ x 3½		SDS25312SS	2¼	250	420	420	180	300	300

1. Allowable loads for SDS screws are based on ICC-ES Code Report ESR-2236. Screws may be provided with the 4CUT™ or Type-17 point.
2. SDS screws install best with a low speed ⅛" drill with a ⅜" hex head driver.
3. Allowable loads are shown at the wood load duration factor of C_D = 1.00. Loads may be increased for load duration up to a C_D = 1.60.
4. Allowable withdrawal load for DF/SP/SCL is 172 lbs./in. and for SPF/HF withdrawal is 150 lbs./in.. Total withdrawal load is based on actual thread penetration into the main member.
5. LSL wood-to-wood applications that require 4½", 5", 6" and 8" SDS screws are limited to interior-dry use only.
6. Minimum spacing requirements are listed in ICC-ES ESR-2236.

Allowable Shear Loads for Wood Side-Plate Applications

Size (in.)	Model No.	DF/SP Allowable Loads[3,4] Wood Side Plate Thickness (in.)												
		1½	1⅝	1¾	2	2½	2¾	3	3½	3⅝	4	4½	5	5½
¼ x 2	SDS25200	145	•	•	•	•	•	•	•	•	•	•	•	•
¼ x 2½	SDS25212	165	165	170	165	•	•	190[1]	•	•	•	•	•	•
¼ x 3	SDS25300	165	165	170	185	195	205	280[1]	•	•	•	•	•	•
¼ x 3½	SDS25312	165	165	170	185	195	205	340[1]	340[1]	•	•	•	•	•
¼ x 4½	SDS25412	165	165	170	185	195	205	350[1]	340[1]	230	200	•	•	•
¼ x 5	SDS25500	165	165	170	185	195	205	350[1]	340[1]	230	230	200	•	•
¼ x 6	SDS25600	165	165	170	185	195	205	350[1]	340[1]	340[1]	340[1]	340[1]	230	200
¼ x 8	SDS25800	165	165	170	185	195	205	350[1]	340[1]	340[1]	340[1]	340[1]	230	230

Size (in.)	Model No.	SPF/HF Allowable Loads[4] Wood Side Plate Thickness (in.)												
		1½	1⅝	1¾	2	2½	2¾	3	3½	3⅝	4	4½	5	5½
¼ x 2	SDS25200	105	•	•	•	•	•	•	•	•	•	•	•	•
¼ x 2½	SDS25212	130	135	130	120	•	•	135[1]	•	•	•	•	•	•
¼ x 3	SDS25300	130	140	140	150	150	145	200[1]	•	•	•	•	•	•
¼ x 3½	SDS25312	130	140	140	150	155	165	245[1]	245[1]	•	•	•	•	•
¼ x 4½	SDS25412	130	140	140	150	155	165	250[1]	245[1]	190	160	•	•	•
¼ x 5	SDS25500	130	140	140	150	155	165	250[1]	245[1]	190	190	160	•	•
¼ x 6	SDS25600	130	140	140	150	155	165	250[1]	245[1]	245[1]	245[1]	245[1]	190	160
¼ x 8	SDS25800	130	140	140	150	155	165	250[1]	245[1]	245[1]	245[1]	245[1]	195	195

1. Noted loads are based on ICC-ES Code Report ESR-2236 and/or testing per ICC AC233 and assume a minimum main member thickness of the screw length minus the side member thickness. All other allowable loads are based on the 2005 National Design Specification (NDS) and a minimum penetration of 6D = 1.45" into the main member.
2. Values are valid for a connection involving only two members. Where the side and main members have different specific gravities, the lower values shall be used.
3. Allowable loads are also applicable to structural composite lumber *(e.g., LVL, PSL, and LSL)* having an equivalent specific gravity of 0.50 or greater.
4. Allowable loads are shown at the wood load duration factor of C_D = 1.00. Loads may be increased for load duration by the building code up to a C_D = 1.60. The Designer shall apply all adjustment factors required per NDS.
5. Loads are based on installation into the side grain of the wood members with the screw axis perpendicular to the wood fibers.
6. Loads apply to appropriate stainless-steel models.

	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	Drag Strut Connection for BF-1 & BF-2				4-83	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

COMPRESSION CHECK FOR DRAG STRUT MEMBER FOR BF-2 (NDS)

Allowable Shear Force on BF-2 Vasd= 0.19 * 63.43 * 0.6 = 7.25 kips (controls)

WOOD MEMBER DESIGN (NDS 2018)

In accordance with the ANSI/AF&PA NDS 2018 using the ASD method

Tedds calculation version 2.2.20

Design summary

Overall design utilisation	0.533
Overall design status	PASS

Design section s1 results summary	Unit	Capacity	Maximum	Utilization	Result
Compressive stress	lb/in^2	1072	571	0.533	PASS

Design section 1

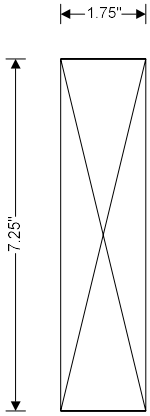
User note: Check column at base

Member details

Service condition	Dry
Load duration - Table 2.3.2	Ten minutes

Composite section details

Number of sawn lumber sections in member	N = **1**
Breadth of sections	b = **1.75** in
Depth of sections	d = **7.25** in
Material	**Microllam, 2.0E LVL grade**



1.75"x7.25" composite section
Cross-sectional area, A, 12.688 in^2
Section modulus, S_x, 15.3 in^3
Section modulus, S_y, 3.7 in^3
Second moment of area, I_x, 55.6 in^4
Second moment of area, I_y, 3.2 in^4
Radius of gyration, r_x, 2.093 in
Radius of gyration, r_y, 0.505 in
Microllam, x, 2.0E LVL grade
Bending about x-x axis, F_{bx}, 2600 psi
Bending about y-y axis, F_{by}, 2690 psi
Shear parallel to grain, bending about x-x axis, F_{vx}, 285 psi
Shear parallel to grain, bending about y-y axis, F_{vy}, 190 psi
Compression parallel to grain, F_c, 2510 psi
Compression perpendicular to grain, F_{c_perpx}, 750 psi
Compression perpendicular to grain, F_{c_perpy}, 680 psi
Tension parallel to grain, F_t, 1895 psi
Modulus of elasticity, E, 2000000 psi
Minimum modulus of elasticity, E_{min}, 1016535 psi
Density, ρ, 34.204 lbm/ft^3

Span details

Unbraced length - Major axis	L_x = **4** ft
Effective bending length - Major axis	$L_{e,x} = 1.63 \times L_x + 3 \times b$ = **6.957** ft
Column buckling length - Major axis	$L_{b,x} = L_x$ = **4** ft
Unbraced length - Minor axis	L_y = **4** ft
Effective bending length - Minor axis	$L_{e,y} = 1.63 \times L_y + 3 \times d$ = **8.333** ft

Column buckling length - Minor axis	$L_{b,y} = L_y =$ **4** ft
Analysis results	
Design axial compression force	P = **7250** lb
Adjustment factors - Table 8.3.1	
Load duration factor - Table 2.3.2	$C_D =$ **1.6**
Reference compression design value	$F_c^* = F_c \times C_D =$ **4016** lb/in^2
Adjusted modulus of elasticity	$E_{min}' = E_{min} =$ **1016535** lb/in^2
Critical buckling design value	$F_{cE} = 0.822 \times E_{min}' / (L_{b,y} / b)^2 =$ **1111** lb/in^2
Column stability factor - eq.3.7-1	
	$C_P = (1 + (F_{cE} / F_c^*)) / 1.8 - \sqrt{(((1 + (F_{cE} / F_c^*)) / 1.8)^2 - (F_{cE} / F_c^*) / 0.9)} =$ **0.267**
Compression members - General - cl.3.6	
Design axial compression force	P = **7250** lb
Design compression parallel to grain - Table 8.3.1	$F_c' = F_c \times C_D \times C_P =$ **1072** lb/in^2
Actual compression parallel to grain	$f_c = P / (b \times d) =$ **571** lb/in^2
	$f_c / F_c' =$ **0.533**
	PASS - Design compression stress exceeds actual compression stress

AECOM	Project				Job Ref.	
	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	Shear Wall A, B, C Design				4-85	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

SHEAR WALL SUMMARY

MARK	SHEATHING	SHEATHING NAIL SIZE	SHEATHING NAIL SPACING @ PANEL EDGES	HOLDDOWN ANDHORS	ANCHOR BOLT SPACING IN
A	1-15/32”	10d	3”	1-HDU8-SDS2.5 7/8” ADHESIVE ANCH.	24”
B	2-15/32” (BOTH INTERIOR AND EXTERIOR SHEATHING)	10d	4”	2-HDU8-SDS2.5 7/8” ADHESIVE ANCH.	24”
C	1-15/32”	10d	3”	1-HDU8-SDS2.5 7/8” ADHESIVE ANCH.	24”

SHEAR WALL A & C

Dead Load; DL= 20 psf * 46.33 ft / 2 = 0.463 kips/ft;

Snow Load; SL= 15 psf * 46.33 ft / 2 = 0.347 kips/ft;

Snow Drift Load; SL= 23.40 psf * 5.858 ft /2 = 0.069 kip/ft ;

Total Snow Load; SL= 0.347 kips/ft + 0.069 kips/ft = 0.416 kips/ft;

Shear from Wind; V_{Wx}= 33.19 kips;

Shear From Seismic; V_{Sx}= 14.24 kips;

Lateral Loads on Shear Wall A From Wind ; V_{Aw}= 0.535∗V_{Wx} = 17.76 kips (Governs)

Lateral Loads on Shear Wall A From Seismic ; V_{As}= 0.535*V_{Sx} = 7.62 kips

ρ=1.0 (Per ASCE 12.3.4, redundancy factor ρ=1.0)

*Self-weight of wall is ignored in calculation because it is included in the TEKLA-TEDDS analysis

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	17186	10654	0.620	PASS
Chord capacity	lb/in^2	507	278	0.685	PASS
Deflection	in	0.267	0.233	0.874	PASS

Panel details

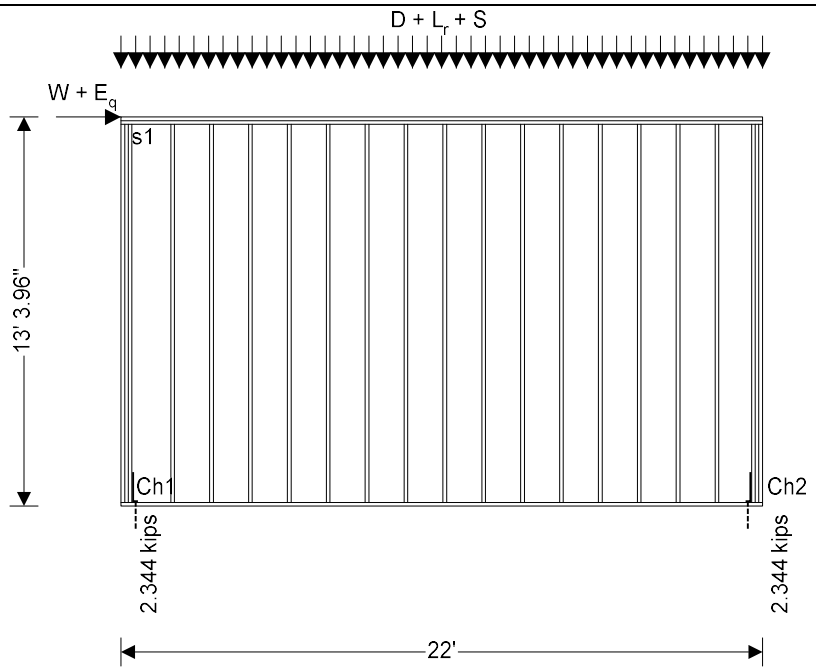
Structural wood panel sheathing on one side

Panel height h = **13.33** ft

Panel length b = **22** ft

Total area of wall A = h × b = **293.26** ft^2

	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	Shear Wall A, B, C Design				4-87	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024



Panel construction

Nominal stud size	2" x 6"
Dressed stud size	1.5" x 5.5"
Cross-sectional area of studs	A_s = **8.25** in^2
Stud spacing	s = **16** in
Nominal end post size	3 x 2" x 6"
Dressed end post size	3 x 1.5" x 5.5"
Cross-sectional area of end posts	A_e = **24.75** in^2
Hole diameter	Dia = **1** in
Net cross-sectional area of end posts	A_{en} = **20.25** in^2
Nominal collector size	2 x 2" x 6"
Dressed collector size	2 x 1.5" x 5.5"
Service condition	Dry
Temperature	100 degF or less
Vertical anchor stiffness	k_a = **30000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification	Hem-Fir, no.1 grade, 2" & wider
Specific gravity	G = **0.43**
Tension parallel to grain	F_t = **625** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **405** lb/in^2
Modulus of elasticity	E = **1500000** lb/in^2
Minimum modulus of elasticity	E_{min} = **550000** lb/in^2

Sheathing details

Sheathing material	15/32" wood panel oriented strandboard sheathing

Fastener type	10d common nails at 3"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design	v_s = 1200 plf × min[1 - (0.5 - G), 1] = **1116** lb/ft
Nominal unit shear capacity for wind design	v_w = 1680 plf × min[1 - (0.5 - G), 1] = **1562.4** lb/ft
Apparent shear wall shear stiffness	G_a = **37** kips/in

Loading details

Dead load acting on top of panel	D = **463** lb/ft
Roof live load acting on top of panel	L_r = **463.3** lb/ft
Snow load acting on top of panel	S = **416** lb/ft
Self weight of panel	S_{wt} = **12** lb/ft^2
In plane wind load acting at head of panel	W = **17756** lbs
Wind load serviceability factor	f_{Wserv} = **0.60**
In plane seismic load acting at head of panel	E_q = **7620** lbs
Design spectral response accel. par., short periods	S_{DS} = **0.302**

From ASCE 7-10 - cl.2.4.1 Basic combinations

Load combination no.1	D + 0.6W
Load combination no.2	D + 0.7E
Load combination no.3	D + 0.75L_f + 0.45W + 0.75(L_r or S or R)
Load combination no.4	D + 0.75L_f + 0.525E + 0.75S
Load combination no.5	0.6D + 0.6W
Load combination no.6	0.6D + 0.7E

Adjustment factors

Load duration factor – Table 2.3.2	C_D = **1.60**
Size factor for tension – Table 4A	C_{Ft} = **1.30**
Size factor for compression – Table 4A	C_{Fc} = **1.10**
Wet service factor for tension – Table 4A	C_{Mt} = **1.00**
Wet service factor for compression – Table 4A	C_{Mc} = **1.00**
Wet service factor for modulus of elasticity – Table 4A	C_{ME} = **1.00**
Temperature factor for tension – Table 2.3.3	C_{tt} = **1.00**
Temperature factor for compression – Table 2.3.3	C_{tc} = **1.00**
Temperature factor for modulus of elasticity – Table 2.3.3	C_{tE} = **1.00**
Incising factor – cl.4.3.8	C_i = **1.00**
Buckling stiffness factor – cl.4.4.2	C_T = **1.00**
Bearing area factor - cl. 3.10.4	C_b = **1.0**
Adjusted modulus of elasticity	$E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **550000** psi
Critical buckling design value	$F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **534** psi
Reference compression design value	$F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2376** psi
For sawn lumber	c = **0.8**
Column stability factor – eqn.3.7-1	$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.21**

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From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio	3.5
Shear wall length	b = **22** ft
Shear wall aspect ratio	h / b = **0.606**

Segmented shear wall capacity

Maximum shear force under wind loading	$V_{w\_max} = 0.6 \times W =$ **10.654** kips
Shear capacity for wind loading	$V_w = v_w \times b / 2 =$ **17.186** kips
	$V_{w\_max} / V_w =$ **0.62**
	PASS - Shear capacity for wind load exceeds maximum shear force
Maximum shear force under seismic loading	$V_{s\_max} = 0.7 \times E_q =$ **5.334** kips
Shear capacity for seismic loading	$V_s = v_s \times b / 2 =$ **12.276** kips
	$V_{s\_max} / V_s =$ **0.435**
	PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio	h / b = **0.606**
Load combination 5	
Shear force for maximum tension	$V = 0.6 \times W =$ **10.654** kips
Axial force for maximum tension	$P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2 =$ **4.112** kips
Maximum tensile force in chord	$T = V \times h / b - P =$ **2.344** kips
Maximum applied tensile stress	$f_t = T / A_{en} =$ **116** lb/in^2
Design tensile stress	$F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1300** lb/in^2
	$f_t / F_t' =$ **0.089**
	PASS - Design tensile stress exceeds maximum applied tensile stress
Load combination 1	
Shear force for maximum compression	$V = 0.6 \times W =$ **10.654** kips
Axial force for maximum compression	$P = ((D + S_{wt} \times h)) \times s / 2 =$ **0.415** kips
Maximum compressive force in chord	$C = V \times h / b + P =$ **6.870** kips
Maximum applied compressive stress	$f_c = C / A_e =$ **278** lb/in^2
Design compressive stress	$F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **507** lb/in^2
	$f_c / F_c' =$ **0.548**
	PASS - Design compressive stress exceeds maximum applied compressive stress
Design bearing compr. stress, bottom plate	$F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b =$ **405** lb/in^2
	$f_c / F_{c\_perp}' =$ **0.685**
	PASS - Design bearing compressive stress exceeds maximum applied bearing compressive stress

Hold down force

Chord 1	$T_1 =$ **2.344** kips
Chord 2	$T_2 =$ **2.344** kips

Wind load deflection

Design shear force	$V_{\delta w} = f_{Wserv} \times W =$ **10.654** kips
Deflection limit	$\Delta_{w\_allow} = h / 600 =$ **0.267** in
Induced unit shear	$v_{\delta w} = V_{\delta w} / b =$ **484.25** lb/ft
Anchor tension force	$T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2) =$ **2.344** kips
Vertical elongation at anchor	$\Delta_a = T_\delta / k_a =$ **0.078** in

Shear wall deflection – Eqn. 4.3-1	$\delta_{sww} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_a) + h \times \Delta_a / b =$ **0.233** in
	$\delta_{sww} / \Delta_{w\_allow} =$ **0.874**
	PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force	$V_{\delta s} = E_q =$ **7.62** kips
Deflection limit	$\Delta_{s\_allow} = 0.020 \times h =$ **3.199** in
Induced unit shear	$v_{\delta s} = V_{\delta s} / b =$ **346.36** lb/ft
Anchor tension force	$T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2) =$ **0.919** kips
Vertical elongation at anchor	$\Delta_a = T_\delta / k_a =$ **0.031** in
Shear wall elastic deflection – Eqn. 4.3-1	$\delta_{swse} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta s} \times h / (G_a) + h \times \Delta_a / b =$ **0.151** in
Deflection ampification factor	$C_{d\delta} =$ **4**
Seismic importance factor	$I_e =$ **1.25**
Amp. seis. deflection – ASCE7 Eqn. 12.8-15	$\delta_{sws} = C_{d\delta} \times \delta_{swse} / I_e =$ **0.484** in
	$\delta_{sws} / \Delta_{s\_allow} =$ **0.151**
	PASS - Shear wall deflection is less than deflection limit

Uplift

Uplift force;	T = **3.92** kips (2.345 kips uplift ASD equals to 3.92 kips uplift LRFD)
1 HDU8-SDS2.5 7/8" Diam Epoxy anchor	R = **5.980** kips (ASD)
Number of anchors on shear wall;	n = 1
	PASS – Anchor hold down capacity exceeds uplift force

Concrete Foundation Anchor Bolt Spacing

Design bolt shear strength in concrete;	$V_{bolt,\ concrete} =$ 1950 lb
Design bolt shear strength in masonry;	$V_{bolt,\ masonry} =$ 585 lb
Maximum shear force under wind loading;	$V_{s\_max} = V =$ **10.66** kips (ASD)
Minimum end distance;	$d_{end} =$ 8 in
Minimum number of bolts required in concrete;	$N_{min,\ con} = \text{ceiling}(V_{s\_max}/(V_{bolt,concrete}),1) =$ **6**
Minimum number of bolts required in masonry;	$N_{min,\ mas} = \text{ceiling}(V_{s\_max}/(V_{bolt,masonry}),1) =$ **19**
Actual number of bolts used in concrete;	$N_{bolt,\ con} = N_{min,\ con} + 1 =$ **7**
Actual number of bolts used in concrete;	$N_{bolt,\ mas} = N_{min,\ mas} + 1 =$ **20**
Anchor bolt spacing in concrete;	$s_{bolt,\ con} = (b-2\times d_{end})/ N_{bolt,\ con} =$ **36** in (use 24 in for design when applicable)
Anchor bolt spacing in masonry;	$s_{bolt,\ mas} = (b-2\times d_{end})/ N_{bolt,\ mas} =$ **13** in (use 8 in for design when applicable)

SIMPSON Strong-Tie

Anchor Designer™ for Concrete Software

Version 3.3.2404.1

Company:	AECOM	Date:	8/1/2024
Engineer:	SA/PB	Page:	1/5
Project:	McD_Lillington,NC		
Address:			
Phone:			
E-mail:			

1.Project information

Project description:
Location:
Fastening description:

Comment:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-19
Units: Imperial units

Anchor Information:
Anchor type: Bonded anchor
Material: F1554 Grade 36
Diameter (inch): 0.875
Effective Embedment depth, h_{ef} (inch): 15.000
Code report: ICC-ES ESR-2508
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 20.00
c_{ac} (inch): 26.65
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 24.00
State: Uncracked
Compressive strength, f'_c (psi): 2500
Reinforcement condition: Supplementary reinforcement not present
Supplemental edge reinforcement: No
Reinforcement provided at corners: Yes
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Hole condition: Dry concrete
Inspection: Continuous
Temperature range, Short/Long: 150/110°F
Reduced installation torque (for AT-3G): Not applicable
Ignore 6do requirement: Not applicable
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 5.50 x 12.00 x 0.25

Recommended Anchor
Anchor Name: SET-XP® (Discontinued) - SET-XP w/ 7/8"Ø F1554 Gr. 36
Code Report: ICC-ES ESR-2508

SIMPSON Strong-Tie

Anchor Designer™ for Concrete Software
Version 3.3.2404.1

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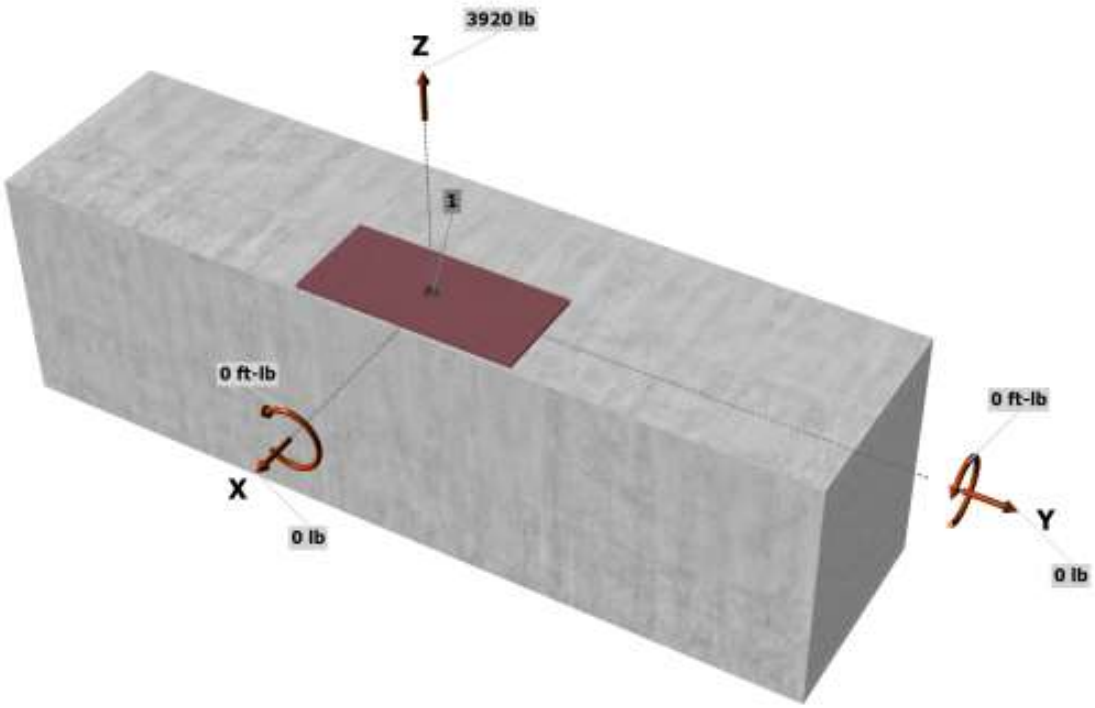
Project:	McD_Lillington,NC		
Address:			
Phone:			
E-mail:			

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: No
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

Strength level loads:

N_{ua} [lb]: 3920
V_{uax} [lb]: 0
V_{uay} [lb]: 0
M_{ux} [ft-lb]: 0
M_{uy} [ft-lb]: 0

<Figure 1>



SIMPSON Strong-Tie Anchor Designer™ for Concrete Software
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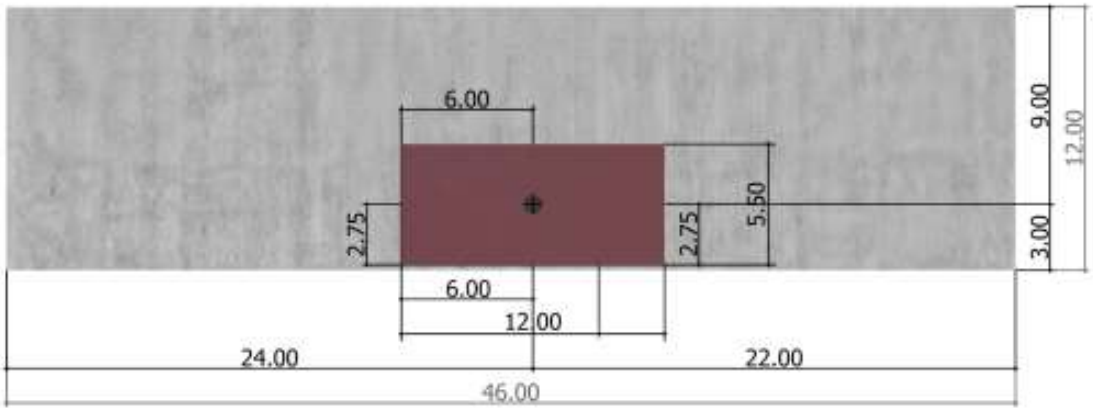
<Figure 2>

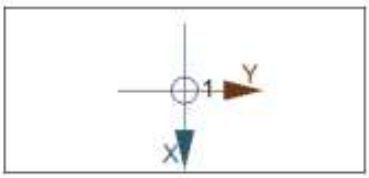


3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	3920.0	0.0	0.0	0.0
Sum	3920.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 3920
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

<Figure 3>



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4. Steel Strength of Anchor in Tension (Sec. 17.6.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
26795	0.75	20096

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.6.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.6.2.2.1)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
24.0	1.00	2500	14.667	67403

$\phi N_{cb} = \phi (A_{Nc} / A_{Nco}) \psi_{ec,N} \psi_{c,N} \psi_{cp,N} N_b$ (Sec. 17.5.1.2 & Eq. 17.6.2.1a)

A_{Nc} (in²)	A_{Nco} (in²)	$c_{a,min}$ (in)	$\psi_{ec,N}$	$\psi_{c,N}$	$\psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cb} (lb)
528.00	1936.00	3.00	0.741	1.00	0.844	67403	0.65	7473

6. Adhesive Strength of Anchor in Tension (Sec. 17.6.5)

$\tau_{k,uncr} = \tau_{k,uncr} f_{shear,seis} K_{init}$

$\tau_{k,uncr}$ (psi)	$f_{shear,seis}$	K_{init}	$\tau_{k,uncr}$ (psi)
885	1.00	1.00	885

$N_{ba} = \lambda_a \tau_{uncr} \pi d_a h_{ef}$ (Eq. 17.6.5.2.1)

λ_a	τ_{uncr} (psi)	d_a (in)	h_{ef} (in)	N_{ba} (lb)
1.00	885	0.88	15.000	36492

$\phi N_a = \phi (A_{Na} / A_{Nao}) \psi_{ed,Na} \psi_{cp,Na} N_{ba}$ (Sec. 17.5.1.2 & Eq. 17.6.5.1a)

A_{Na} (in²)	A_{Nao} (in²)	c_{Na} (in)	$c_{a,min}$ (in)	$\psi_{ed,Na}$	$\psi_{cp,Na}$	N_{ba} (lb)	ϕ	ϕN_a (lb)
170.29	246.39	7.85	3.00	0.815	0.294	36492	0.65	3933

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.8)

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	3920	20096	0.20	Pass
Concrete breakout	3920	7473	0.52	Pass

SIMPSON Strong-Tie Anchor Designer™ for Concrete Software
Version 3.3.2404.1

Company:	AECOM	Date:	8/1/2024
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Address:			
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Adhesive	**3920**	**3933**	**1.00**	**Pass (Governs)**

SET-XP w/ 7/8"Ø F1554 Gr. 36 with hef = 15.000 inch meets the selected design criteria.

12. Warnings

- When cracked concrete is selected, concrete compressive strength used in concrete breakout strength in tension, adhesive strength in tension and concrete pryout strength in shear for SET-XP adhesive anchor is limited to 2,500 psi per ICC-ES ESR-2508 Section 5.3.

- Designer must exercise own judgement to determine if this design is suitable.

- Refer to manufacturer's product literature for hole cleaning and installation instructions.

SHEAR WALL B- LOADS

Dead Load; DL= 20 psf * 3 ft / 2 = 0.03 kips/ft;

Snow Load; SL= (15 psf + 32.8 psf) * 3 ft / 2 = 0.072 kips/ft;

Shear from Wind; V_{Wx}= 63.43 kips;

Shear From Seismic; V_{Sx}= 28.49 kips;

Lateral Loads on Shear Wall B From Wind ; V_{Bw}= 0.3094 * V_{Wy} = 19.63 kips

Lateral Loads on Shear Wall B From Seismic ; V_{Bs}= 0.3094 * V_{Sy} = 8.82 kips

ρ=1.0 (Per ASCE 12.3.4, redundancy factor ρ=1.0)

*Self-weight of wall is ignored in calculation because it is included in the TEKLA-TEDDS analysis

WOOD SHEAR WALL DESIGN (NDS)

In accordance with NDS2018 and SDPWS2015 allowable stress design and the segmented shear wall method

Tedds calculation version 1.2.10

Design summary

Description	Unit	Provided	Required	Utilization	Result
Shear capacity	lbs	29393	11778	0.401	PASS
Chord capacity	lb/in^2	507	264	0.652	PASS
Deflection	in	0.267	0.208	0.779	PASS

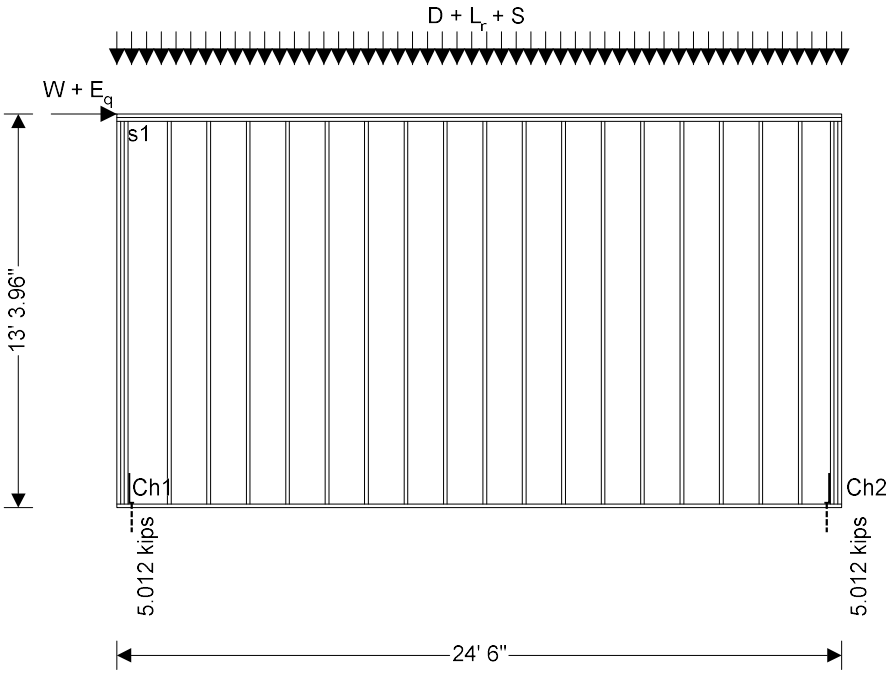
Panel details

Structural wood panel sheathing on both sides

Panel height h = **13.33** ft

Panel length b = **24.5** ft

Total area of wall A = h × b = **326.585** ft^2



AECOM	Project: McDonalds Lillington, NC				Job Ref.: 032-0312	
	Section: Shear Wall A, B, C Design				Sheet no./rev.: 4-97	
	Calc. by: SA/PB	Date: 8/7/2024	Chk'd by: ES	Date: 8/7/2024	App'd by: MY	Date: 8/7/2024

Panel construction

Nominal stud size	2" x 6"
Dressed stud size	1.5" x 5.5"
Cross-sectional area of studs	A_s = **8.25** in^2
Stud spacing	s = **16** in
Nominal end post size	3 x 2" x 6"
Dressed end post size	3 x 1.5" x 5.5"
Cross-sectional area of end posts	A_e = **24.75** in^2
Hole diameter	Dia = **1** in
Net cross-sectional area of end posts	A_{en} = **20.25** in^2
Nominal collector size	2 x 2" x 6"
Dressed collector size	2 x 1.5" x 5.5"
Service condition	Dry
Temperature	100 degF or less
Vertical anchor stiffness	k_a = **30000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification	Hem-Fir, no.1 grade, 2" & wider
Specific gravity	G = **0.43**
Tension parallel to grain	F_t = **625** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **405** lb/in^2
Modulus of elasticity	E = **1500000** lb/in^2
Minimum modulus of elasticity	E_{min} = **550000** lb/in^2

Sheathing details

Sheathing material	15/32" wood panel oriented strandboard sheathing
Fastener type	10d common nails at 4"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design	v_s = 920 plf × min[1 - (0.5 - G), 1] = **855.6** lb/ft
Nominal unit shear capacity for wind design	v_w = 1290 plf × min[1 - (0.5 - G), 1] = **1199.7** lb/ft
Apparent shear wall shear stiffness	G_a = **30** kips/in

Combined unit shear capacities

Combined nominal unit shear capacity for seismic design	$v_{sc} = 2 \times v_s$ = **1711.2** lb/ft
Combined nominal unit shear capacity for wind design	$v_{wc} = 2 \times v_w$ = **2399.4** lb/ft
Combined apparent shear wall shear stiffness	$G_{ac} = G_{a1} + G_{a2}$ = **60** kips/in

Loading details

Dead load acting on top of panel	D = **30** lb/ft
Roof live load acting on top of panel	L_r = **30** lb/ft
Snow load acting on top of panel	S = **72** lb/ft
Self weight of panel	S_{wt} = **12** lb/ft^2
In plane wind load acting at head of panel	W = **19630** lbs
Wind load serviceability factor	f_{Wserv} = **0.60**

In plane seismic load acting at head of panel	E_q = **8820** lbs
Design spectral response accel. par., short periods	S_{DS} = **0.302**

From ASCE 7-10 - cl.2.4.1 Basic combinations

Load combination no.1	D + 0.6W
Load combination no.2	D + 0.7E
Load combination no.3	D + 0.75L_f + 0.45W + 0.75(L_r or S or R)
Load combination no.4	D + 0.75L_f + 0.525E + 0.75S
Load combination no.5	0.6D + 0.6W
Load combination no.6	0.6D + 0.7E

Adjustment factors

Load duration factor – Table 2.3.2	C_D = **1.60**
Size factor for tension – Table 4A	C_{Ft} = **1.30**
Size factor for compression – Table 4A	C_{Fc} = **1.10**
Wet service factor for tension – Table 4A	C_{Mt} = **1.00**
Wet service factor for compression – Table 4A	C_{Mc} = **1.00**
Wet service factor for modulus of elasticity – Table 4A	C_{ME} = **1.00**
Temperature factor for tension – Table 2.3.3	C_{tt} = **1.00**
Temperature factor for compression – Table 2.3.3	C_{tc} = **1.00**
Temperature factor for modulus of elasticity – Table 2.3.3	C_{tE} = **1.00**
Incising factor – cl.4.3.8	C_i = **1.00**
Buckling stiffness factor – cl.4.4.2	C_T = **1.00**
Bearing area factor - cl. 3.10.4	C_b = **1.0**
Adjusted modulus of elasticity	$E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **550000** psi
Critical buckling design value	$F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **534** psi
Reference compression design value	$F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2376** psi
For sawn lumber	c = **0.8**
Column stability factor – eqn.3.7-1	$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.21**

From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios

Maximum shear wall aspect ratio	3.5
Shear wall length	b = **24.5** ft
Shear wall aspect ratio	h / b = **0.544**

Segmented shear wall capacity

Maximum shear force under wind loading	$V_{w\_max} = 0.6 \times W$ = **11.778** kips
Shear capacity for wind loading	$V_w = v_{wc} \times b / 2$ = **29.393** kips
	V_{w_max} / V_w = **0.401**
	PASS - Shear capacity for wind load exceeds maximum shear force
Maximum shear force under seismic loading	$V_{s\_max} = 0.7 \times E_q$ = **6.174** kips
Shear capacity for seismic loading	$V_s = v_{sc} \times b / 2$ = **20.962** kips
	V_{s_max} / V_s = **0.295**
	PASS - Shear capacity for seismic load exceeds maximum shear force

Chord capacity for chords 1 and 2

Shear wall aspect ratio	h / b = **0.544**
Load combination 5	
Shear force for maximum tension	$V = 0.6 \times W$ = **11.778** kips
Axial force for maximum tension	$P = (0.6 \times (D + S_{wt} \times h)) \times b_1 / 2$ = **1.396** kips
Maximum tensile force in chord	$T = V \times h / b - P$ = **5.012** kips
Maximum applied tensile stress	$f_t = T / A_{en}$ = **248** lb/in^2
Design tensile stress	$F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1300** lb/in^2
	f_t / F_t' = **0.190**
	PASS - Design tensile stress exceeds maximum applied tensile stress
Load combination 1	
Shear force for maximum compression	$V = 0.6 \times W$ = **11.778** kips
Axial force for maximum compression	$P = ((D + S_{wt} \times h)) \times s / 2$ = **0.127** kips
Maximum compressive force in chord	$C = V \times h / b + P$ = **6.535** kips
Maximum applied compressive stress	$f_c = C / A_e$ = **264** lb/in^2
Design compressive stress	$F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **507** lb/in^2
	f_c / F_c' = **0.521**
	PASS - Design compressive stress exceeds maximum applied compressive stress
Design bearing compr. stress, bottom plate	$F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **405** lb/in^2
	f_c / F_{c_perp}' = **0.652**
	PASS - Design bearing compressive stress exceeds maximum applied bearing compressive stress

Hold down force

Chord 1	T_1 = **5.012** kips
Chord 2	T_2 = **5.012** kips

Wind load deflection

Design shear force	$V_{\delta w} = f_{Wserv} \times W$ = **11.778** kips
Deflection limit	Δ_{w_allow}= h / 600 = **0.267** in
Induced unit shear	$v_{\delta w} = V_{\delta w} / b$ = **480.73** lb/ft
Anchor tension force	$T_\delta = \max(0 \text{ kips}, v_{\delta w} \times h - 0.6 \times (D + S_{wt} \times h) \times b / 2)$ = **5.012** kips
Vertical elongation at anchor	$\Delta_a = T_\delta / k_a$ = **0.167** in
Shear wall deflection – Eqn. 4.3-1	$\delta_{sww} = 2 \times v_{\delta w} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta w} \times h / (G_{ac}) + h \times \Delta_a / b$ = **0.208** in
	$\delta_{sww} / \Delta_{w\_allow}$ = **0.779**
	PASS - Shear wall deflection is less than deflection limit

Seismic deflection

Design shear force	$V_{\delta s} = E_q$ = **8.82** kips
Deflection limit	Δ_{s_allow}= 0.020 × h = **3.199** in
Induced unit shear	$v_{\delta s} = V_{\delta s} / b$ = **360** lb/ft
Anchor tension force	$T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times b / 2)$ = **3.543** kips
Vertical elongation at anchor	$\Delta_a = T_\delta / k_a$ = **0.118** in
Shear wall elastic deflection – Eqn. 4.3-1	$\delta_{swse} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta s} \times h / (G_{ac}) + h \times \Delta_a / b$ = **0.152** in
Deflection ampification factor	$C_{d\delta}$ = **4**
Seismic importance factor	I_e = **1.25**
Amp. seis. deflection – ASCE7 Eqn. 12.8-15	$\delta_{sws} = C_{d\delta} \times \delta_{swse} / I_e$ = **0.486** in

δ_{sws} / Δ_{s_allow} = **0.152**

PASS - Shear wall deflection is less than deflection limit

Uplift

Uplift force; T = **8.37** kips (5.012 kips uplift ASD equals to 8.37 kips uplift LRFD)

1 HDU8-SDS2.5 7/8" Diam Epoxy anchor R = **5.980** kips (ASD)

Number of anchors on shear wall; n = **2**

PASS – HDU hold down capacity exceeds uplift force

Anchor Bolt Spacing

Design bolt shear strength in concrete; $V_{bolt,\ concrete}$ = 1950 lb

Design bolt shear strength in masonry; $V_{bolt,\ masonry}$ = 585 lb

Maximum shear force under wind loading; V_{s_max} = V = **11.78** kips (ASD)

Minimum end distance; d_{end} = 8 in

Minimum number of bolts required in concrete; $N_{min,\ con}$ = ceiling(V_{s_max}/($V_{bolt,concrete}$),1)= **6**

Minimum number of bolts required in masonry; $N_{min,\ mas}$ = ceiling(V_{s_max}/($V_{bolt,masonry}$),1)= **20**

Actual number of bolts used in concrete; $N_{bolt,\ con}$ = $N_{min,\ con}$ + 1 = **7**

Actual number of bolts used in masonry; $N_{bolt,\ mas}$ = $N_{min,\ mas}$ + 1 = **21**

Anchor bolt spacing in concrete; $s_{bolt,\ con}$ = (b-2×d_{end})/ $N_{bolt,\ con}$ = **40** in (use 24 in for design when applicable)

Anchor bolt spacing in masonry; $s_{bolt,\ mas}$ = (b-2×d_{end})/ $N_{bolt,\ mas}$ = **14** in (use 8 in for design when applicable)

SIMPSON StrongTie

Anchor Designer™ for Concrete Software

Version 3.3.2404.1

Company:	AECOM	Date:	8/1/2024
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Project:	McD_Lillington,NC		
Address:			
Phone:			
E-mail:			

1.Project information

Project description:
Location:
Fastening description:

Comment:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-19
Units: Imperial units

Anchor Information:
Anchor type: Bonded anchor
Material: F1554 Grade 36
Diameter (inch): 0.875
Effective Embedment depth, h_{ef} (inch): 15.000
Code report: ICC-ES ESR-2508
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 20.00
c_{ac} (inch): 26.65
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 24.00
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,v}$: 1.0
Reinforcement condition: Supplementary reinforcement not present
Supplemental edge reinforcement: Not applicable
Reinforcement provided at corners: Yes
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Hole condition: Dry concrete
Inspection: Continuous
Temperature range, Short/Long: 150/110°F
Reduced installation torque (for AT-3G): Not applicable
Ignore 6do requirement: Not applicable
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 5.00 x 12.00 x 0.25

Recommended Anchor
Anchor Name: SET-XP® (Discontinued) - SET-XP w/ 7/8"Ø F1554 Gr. 36
Code Report: ICC-ES ESR-2508

SIMPSON Strong-Tie Anchor Designer™ for Concrete Software
Version 3.3.2404.1

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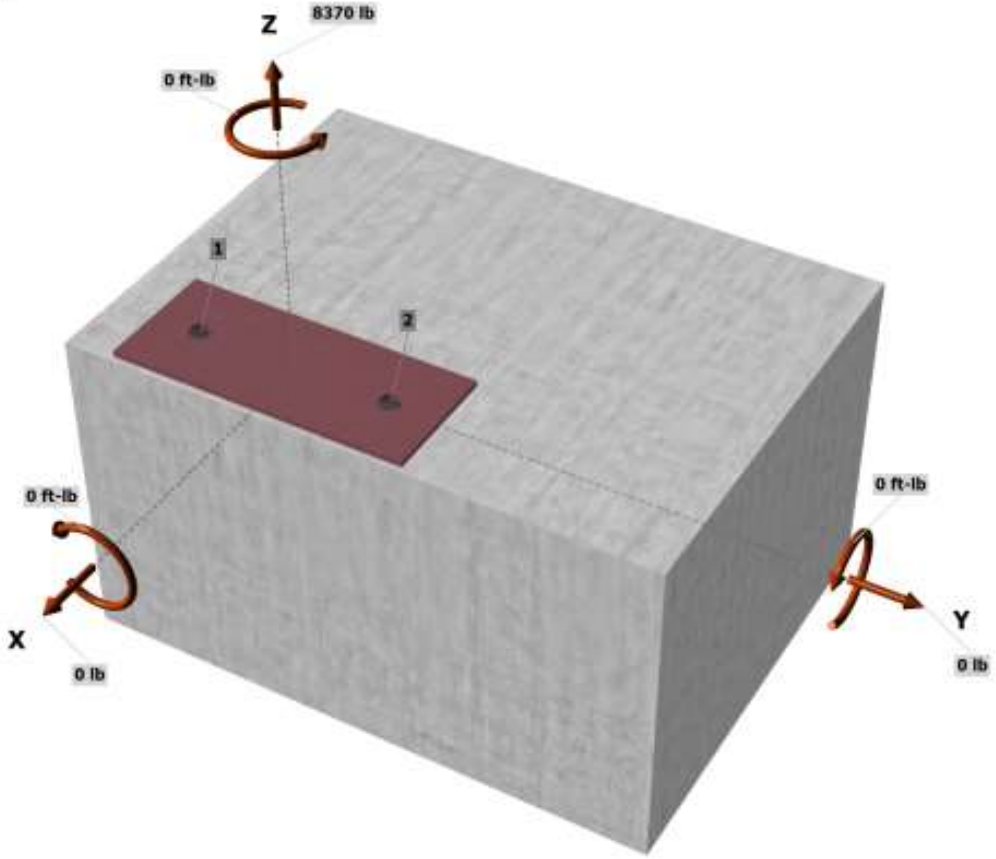
Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: No
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

Strength level loads:

N_{ua} [lb]: 8370
V_{uax} [lb]: 0
V_{uay} [lb]: 0
M_{ux} [ft-lb]: 0
M_{uy} [ft-lb]: 0
M_{uz} [ft-lb]: 0

<Figure 1>



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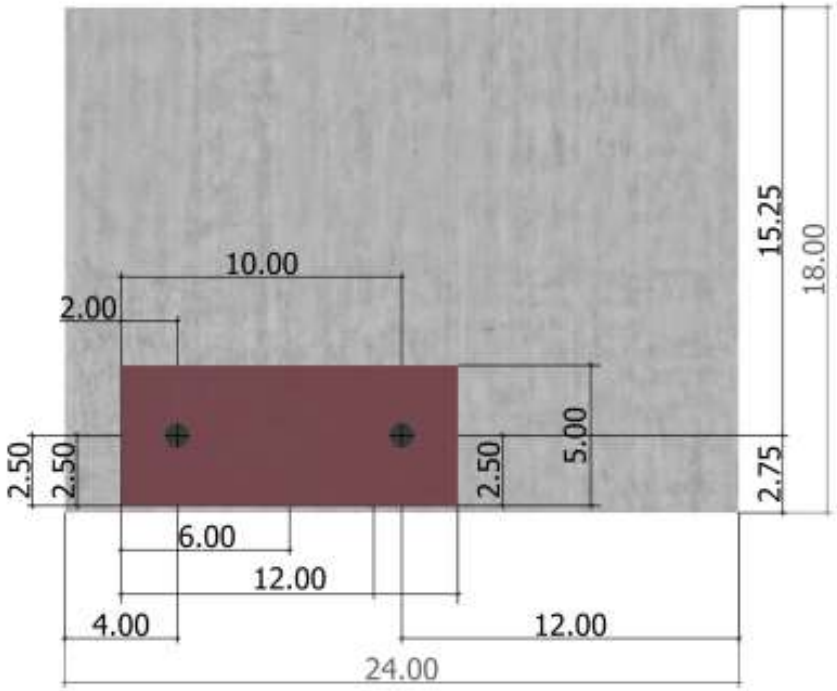
<Figure 2>

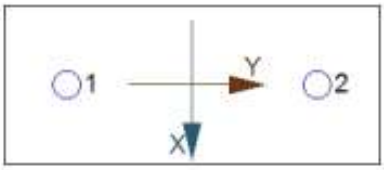


3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	4185.0	0.0	0.0	0.0
2	4185.0	0.0	0.0	0.0
Sum	8370.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 8370
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

<Figure 3>



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4. Steel Strength of Anchor in Tension (Sec. 17.6.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
26795	0.75	20096

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.6.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.6.2.2.1)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
17.0	1.00	2500	10.167	27554

$\phi N_{cbg} = \phi (A_{Nc} / A_{Nco}) \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ (Sec. 17.5.1.2 & Eq. 17.6.2.1a)

A_{Nc} (in²)	A_{Nco} (in²)	$c_{a,min}$ (in)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cbg} (lb)
432.00	930.25	2.75	1.000	0.754	1.00	1.000	27554	0.65	6272

6. Adhesive Strength of Anchor in Tension (Sec. 17.6.5)

$\tau_{k,cr} = \tau_{k,cr} f_{ctm,inch/mm} K_{cr,sus}$

$\tau_{k,cr}$ (psi)	$f_{ctm,inch/mm}$	$K_{cr,sus}$	$\tau_{k,cr}$ (psi)
355	1.00	1.00	355

$N_{ba} = \lambda_a \tau_{cr} \pi d_a h_{ef}$ (Eq. 17.6.5.2.1)

λ_a	τ_{cr} (psi)	d_a (in)	h_{ef} (in)	N_{ba} (lb)
1.00	355	0.88	15.000	14638

$\phi N_{ag} = \phi (A_{Na} / A_{Nao}) \Psi_{ec,Na} \Psi_{ed,Na} \Psi_{cp,Na} N_{ba}$ (Sec. 17.5.1.2 & Eq. 17.6.5.1b)

A_{Na} (in²)	A_{Nao} (in²)	c_{Na} (in)	$c_{a,min}$ (in)	$\Psi_{ec,Na}$	$\Psi_{ed,Na}$	$\Psi_{cp,Na}$	N_{ba}(lb)	ϕ	ϕN_{ag} (lb)
210.36	246.39	7.85	2.75	1.000	0.805	1.000	14638	0.65	6540

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.8)

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	4185	20096	0.21	Pass
Concrete breakout	**8370**	**6272**	**1.33**	**Fail (Governs)**

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Adhesive	8370	6540	1.28	Fail

FAIL! Selected anchor type and embedment do not meet the selected design criteria.

12 Warnings

- When cracked concrete is selected, concrete compressive strength used in concrete breakout strength in tension, adhesive strength in tension and concrete pryout strength in shear for SET-XP adhesive anchor is limited to 2,500 psi per ICC-ES ESR-2508 Section 5.3.

- Designer must exercise own judgement to determine if this design is suitable.

- Refer to manufacturer's product literature for hole cleaning and installation instructions.

Concrete breakout

Factored tensile load; T = **8.37** kips

Concrete breakout reinforcement at anchors location per ACI318-14 Section 17.4.2.9;

Use #5 dowels at anchors location:

Anchor reinforcement design strength; = 0.75 * 60 ksi * As = 0.75 * 60 ksi * 2 (0.31 in2) = **27.9** kips > **8.37** kips

PASS – Anchor reinforcement strength exceeds tensile load

COMPRESSION CHECK FOR DRAG STRUT MEMBER FOR SWB (NDS)

Allowable Shear Force on SWB Vasd= 0.3094 * 63.43 * 0.6 = 11.78 kips (controls)

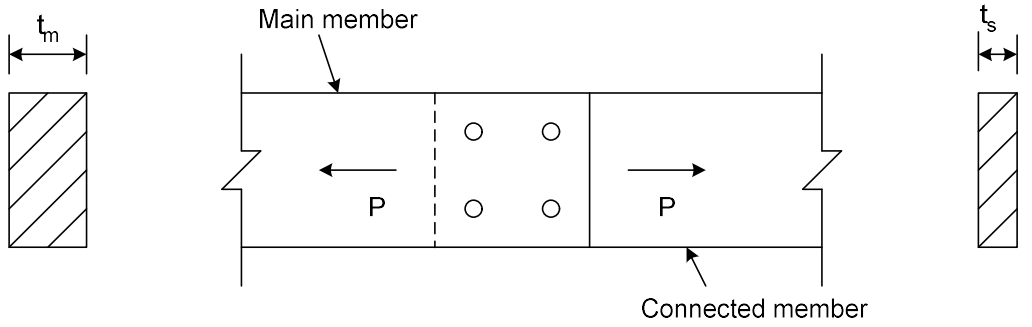
LAG SCREW TIMBER TO TIMBER CONNECTION DESIGN

In accordance with NDS 2018

Tedds calculation version 1.2.04

Design results summary

	Unit	Required	Provided	Utilization	Result
Connection capacity	lbs	11780	12747	0.924	PASS



Main timber member details

Species of main member	**Hem-Fir**
Size of main member (Table 1B)	**2 x 8**
Number of main member	N_m = **1**
Thickness of main member	t_m = **1.500** in
Angle of load to grain of main member	θ_m = **0**°

Connected timber member details

Type of connected member	**Plywood structural I and marine grade**
Number of connected member	N_s = **1**
Thickness of connected member	t_s = **1.500** in
Number of interfaces	$N_{int} = (N_m + N_s) - 1$ = **1**

Lag screw details

Lag screw diameter (Table L2)	**1/4"**

Lag screw root diameter of threaded portion	D_r = **0.173** in
Length of lag screw	L = **4.000** in
Washer thickness	W = **0.000** in
Number of rows of lag screw	R = **54**
Number of columns of lag screw	C = **1**
Total number of lag screw	N_{total} = R × C = **54**

Applied load

Applied load to the connection	P = **11780** lb

Dowel bearing length (main) (12.3.5)

Size of tapered tip	E = **0.156** in
Actual penetration in main member	P_{act} = L - W - t_s = **2.500** in
Dowel bearing length in main member	l_m = **1.500** in

Dowel bearing length (connected) (12.3.5)

Dowel bearing length in connected member	l_s = t_s = **1.500** in

Bending yield strength (lag screw) (Table 12J to 12K footnote no. 2)

Bending yield strength of lag screw	F_{yb} = **70000** psi

Dowel bearing strength (main member) (Table 12.3.3 footnote no. 2)

Dowel bearing strength parallel to grain	$F_{e\_par} = 11200 \times G_m \times 1\ psi$ = **4816** psi
Dowel bearing strength perpendicular to grain	$F_{e\_perp} = 6100 \times G_m^{1.45} \times 1\ psi / \sqrt{(D / 1\ in)}$ = **3588** psi
Dowel bearing strength for small dia. fasteners	$F_e = 16600 \times G_m^{1.84} \times 1\ psi$ = **3513** psi
Dowel bearing strength at an angle of load to grain	$F_{e\theta m} = (F_{e\_par} \times F_{e\_perp}) / ((F_{e\_par} \times (\sin(\theta_m))^2) + (F_{e\_perp} \times (\cos(\theta_m))^2))$
	$F_{e\theta m}$ = **4816** psi
Dowel bearing strength of main member	F_{em} = **4816** psi

Dowel bearing strength (connected timber member) (Table 12.3.3 footnote no. 2)

Dowel bearing strength parallel to grain	$F_{e\_par} = 11200 \times G_s \times 1\ psi$= **5600** psi
Dowel bearing strength perpendicular to grain	$F_{e\_perp} = 6100 \times G_s^{1.45} \times 1\ psi / \sqrt{(D / 1\ in)}$ = **4465** psi
Dowel bearing strength for small dia. fasteners	$F_e = 16600 \times G_s^{1.84} \times 1\ psi$ = **4637** psi
Dowel bearing strength at an angle of load to grain	$F_{e\theta s} = (F_{e\_par} \times F_{e\_perp}) / ((F_{e\_par} \times (\sin(\theta_s))^2) + (F_{e\_perp} \times (\cos(\theta_s))^2))$
	$F_{e\theta s}$ = **5600** psi
Dowel bearing strength of connected member	F_{es} = **5600** psi

Preliminary yield limit equation coefficients (Table 12.3.1A notes)

Dowel bearing strength ratio	$R_e = F_{em} / F_{es}$ = **0.860**
Dowel bearing length ratio	$R_t = l_m / l_s$ = **1.000**
Preliminary yield limit equation coefficient k_1	$k_1 = ((\sqrt{(R_e+(2\times R_e^2\times(1+R_t+R_t^2))+(R_t^2\times R_e^3)))}-(R_e\times(1+R_t)))/(1+R_e)$
	k_1 = **0.385**
Preliminary yield limit equation coefficient k_2	$k_2 = -1+\sqrt{((2\times(1+R_e))+((2\times F_{yb}\times(1+(2\times R_e))\times D_r^2))/(3\times F_{em}\times l_m^2))}$
	k_2 = **1.018**
Preliminary yield limit equation coefficient k_3	$k_3 = -1+\sqrt{(((2\times(1+R_e))/R_e)+((2\times F_{yb}\times(2+R_e)\times D_r^2))/(3\times F_{em}\times l_s^2))}$
	k_3= **1.167**
Angle of load to grain coefficient k_θ	$k_\theta = 1 + (0.25 \times \max(\theta_m, \theta_s) / 90)$ = **1.000**
Reduction term (Table 12.3.1B)	K_D = **2.23**

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	Section: Shear Wall A, B, C Design				Sheet no./rev.: 4-108	
	Calc. by: SA/PB	Date: 8/7/2024	Chk'd by: ES	Date: 8/7/2024	App'd by: MY	Date: 8/7/2024

Yield limit equations (single shear)

Mode I_m (eq. 12.3-1)	$Z_{Im} = (D_r \times l_m \times F_{em}) / (K_D \times k_\theta)$ = **560** lb
Mode I_s (eq. 12.3-2)	$Z_{Is} = (D_r \times l_s \times F_{es}) / (K_D \times k_\theta)$ = **652** lb
Mode II (eq. 12.3-3)	$Z_{II} = (k_1 \times D_r \times l_s \times F_{es}) / (K_D \times k_\theta)$ = **251** lb
Mode III_m (eq. 12.3-4)	$Z_{IIIm} = (k_2 \times D_r \times l_m \times F_{em}) / ((1 + (2 \times R_e)) \times K_D \times k_\theta)$ = **210** lb
Mode III_s (eq. 12.3-5)	$Z_{IIIs} = (k_3 \times D_r \times l_s \times F_{em}) / ((2 + R_e) \times K_D \times k_\theta)$ = **229** lb
Mode IV (eq. 12.3-6)	$Z_{IV} = (D_r^2 \times (\sqrt{((2 \times F_{em} \times F_{yb}) / (3 \times (1 + R_e)))})) / (K_D \times k_\theta)$ = **148** lb
	$Z = \min(Z_{Im}, Z_{Is}, Z_{II}, Z_{IIIm}, Z_{IIIs}, Z_{IV})$ = **148** lb
Nominal capacity of single fastener	Z = **148** lb

Slenderness (Table 12.5.1C footnote no.1)

Slenderness	l / D = **6.000**

Spacing requirements (parallel to grain loading)

End distance (Table 12.5.1A)

End distance (full strength)	a_{p_full} = **1.750** in
End distance (minimum)	a_{p_min} = **0.875** in
End distance (actual)	a_p = **4.000** in

Edge distance (Table 12.5.1C)

Edge distance	e = (1.5 × D) = 0.375 in

Center to center spacing (Table 12.5.1B)

Center to center spacing (full strength)	$s_{\_full}$ = 4 × D = **1.000** in
Center to center spacing (minimum)	$s_{\_min}$ = 3 × D = **0.750** in
Center to center spacing (actual)	s = **12.000** in

Row spacing (Table 12.5.1D)

Row spacing	s_{row} = 1.5 × D = **0.375** in

Geometry factor C_Δ (12.5.1)

End distance (actual)	a_p = **4.000** in
End distance (full strength)	a_{p_full} = **1.750** in
Geometry factor for end distance	$C_{\Delta1} = a_p / a_{p\_full}$ = **2.29**
Center to center spacing (actual)	s = **12.000** in
Center to center spacing (full strength)	$s_{\_full}$ = **1.000** in
Geometry factor for spacing	$C_{\Delta2} = s / s_{\_full}$ = **12.00**
Geometry factor	$C_\Delta = \min(1, C_{\Delta1}, C_{\Delta2})$ = **1.00**

Adjustment factor

Load duration factor (Table 2.3.2)	C_D = **1.60**
Wet service factor (Table 11.3.3)	C_M = **1.0**
Temperature factor (Table 11.3.4)	C_t = **1.0**
Group action factor (eq. 11.3-1)	C_g = **1.0**
Geometry factor (12.5.1)	C_Δ = **1.00**
End grain factor (12.5.2)	C_{eg} = **1.0**
Diaphragm factor (12.5.3)	C_{di} = **1.0**
Toe nail factor (12.5.4)	C_{tn} = **1.0**

Total capacity of connection

Capacity of connection $Z' = Z \times N_{total} \times N_{int} \times C_D \times C_M \times C_\Delta =$ **12747** lb

$P / Z' =$ **0.924**

Design result

PASS - Connection capacity exceeds applied load

Splice Connection Strength

Strap strength $S_{strap} =$ **5080** lb

MSTI48 + 60-10d (fasteners have 1.6 load duration factor)

Number of straps required $n_{strap} = \text{ceiling}(P/S_{strap},1) = 3$

Wood screw strength $S_{screw} = 165 \text{ lb} \times 1.6 =$ **264.000** lb

SDS ¼" diam x 4 ½" wood screw into
½" wood side member(sheathing)
with 1.6 load duration factor

Number of screws required $n_{screw} = \text{ceiling}(P/S_{screw},1) = 45$ (Use 54 screws)

AECOM	Project: McDonalds Lillington, NC				Job Ref.: 032-0312	
	Section: Shear Wall A, B, C Design				Sheet no./rev.: 4-110	
	Calc. by: SA/PB	Date: 8/7/2024	Chk'd by: ES	Date: 8/7/2024	App'd by: MY	Date: 8/7/2024

DIAPHRAGM LOAD TRANSFER TO SHEAR WALL

1. Lateral force from deck to vertical blocking @ SW "A & C"
   Wind load = 17.76 k * 0.6 = 10.65 k (ASD)
   Building Length = 85.66 ft (common nails are distributed along full building length)
   Wind load per ft = 10.65 k / 85.66 ft = 125 lb/ft
   For 10d common nails @ 6" O.C., the allowable load is = 2 * 137 lb = 274 lb/ft > 125 lb/ft **OK**

Design Method	Allowable Stress Design (ASD)
Connection Type	Lateral loading
Fastener Type	Nail
Loading Scenario	Single Shear
	Submit Initial Values

Main Member Type	Hem-Fir
Main Member Thickness	3.5 in.
Side Member Type	Hem-Fir
Side Member Thickness	0.75 in.
Nail Type	Common Wire
Nail Size	10d (D = 0.148 in.; L = 3 in.)
Load Duration Factor	C_D = 1.6
Wet Service Factor	C_M = 1.0
End Grain Factor	C_eg = 1.0
Temperature Factor	C_t = 1.0
Diaphragm Factor	C_di = 1.0

Calculate Connection Capacity

Connection Yield Mode Descriptions | Limits of Use | Diaphragm Factor Help | Load Duration Factor Help | Technical Help | Show Printable View

Connection Yield Modes

Im	848 lbs.
Is	283 lbs.
II	283 lbs.
IIIm	298 lbs.
IIIs	137 lbs.
IV	163 lbs.

Adjusted ASD Capacity	137 lbs.

2. Lateral force from blocking to bevel plate @ SW "A & C"
   SW Length = 22 ft
   Wind load per ft = 10.65 k / 22 ft = 484 lb/ft
   For 10d common nails @ 4" O.C., the allowable load is = 3* 154 lb = 464 lb/ft
   Simpson "RBC" @ 32" O.C., the allowable load is = 380 lb / 2.67 = 142 lb/ft

   The total connection allowable load is = 464 + 142 = 606 lb/ft > 484 lb/ft **OK**

Design Method	Allowable Stress Design (ASD)
Connection Type	Lateral loading
Fastener Type	Nail
Loading Scenario	Single Shear
	Submit Initial Values

Main Member Type	Hem-Fir
Main Member Thickness	1.75 in.
Side Member Type	Hem-Fir
Side Member Thickness	1.75 in.
Nail Type	Common Wire
Nail Size	10d (D = 0.148 in.; L = 3 in.)
Load Duration Factor	C_D = 1.6
Wet Service Factor	C_M = 1.0
End Grain Factor	C_eg = 1.0
Temperature Factor	C_t = 1.0
Diaphragm Factor	C_di = 1.0

Calculate Connection Capacity

Connection Yield Mode Descriptions | Limits of Use | Diaphragm Factor Help | Load Duration Factor Help | Technical Help | Show Printable View

Connection Yield Modes

Im	359 lbs.
Is	659 lbs.
II	226 lbs.
IIIm	154 lbs.
IIIs	240 lbs.
IV	163 lbs.

Adjusted ASD Capacity	154 lbs.

Model No.	Type of Connection	Bending Angle	Fasteners (in.)		DF/SP Allowable Loads	SPF/HF Allowable Loads
			To Wall	To Blocking	Lateral (160)	Lateral (160)
RBC RBCP	1	45° to 90°	(6) 0.148 x 1½	(6) 0.148 x 1½	445	380
	2	< 30°	(6) 0.148 x 1½	(6) 0.148 x 1½	435	375
		30° to 45°	(6) 0.148 x 1½	(6) 0.148 x 1½	465	400
	3	0° to 45°	(3) ¼ x 2¼ Titen Turbo™ [4]	(6) 0.148 x 1½	350	350

3. Lateral force from 2x8 blocking to 3x6 wood blocking @ SW “B”
   SW Length = 24.5 ft
   Wind load per ft = 11.78 k / 24.5 ft = 481 lb/ft
   For 3/8” dia x 6” lag screws @ 6” O.C., the allowable load is = 2 * 298 lb = 596 lb/ft > 481 lb/ft **OK**

	Project			Job Ref.	
AECOM	McDonalds Lillington, NC			032-0312	
	Section			Sheet no./rev.	
	Shear Wall A, B, C Design			4-112	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

Design Method	Allowable Stress Design (ASD)
Connection Type	Lateral loading
Fastener Type	Lag Screw
Loading Scenario	Single Shear
	Submit Initial Values

Main Member Type	Hem-Fir
Main Member Thickness	5.5 in.
Main Member: Angle of Load to Grain	0
Side Member Type	Hem-Fir
Side Member Thickness	1.5 in.
Side Member: Angle of Load to Grain	0
Washer Thickness	1/8 in.
Nominal Diameter	3/8 in.
Length	6 in.
Load Duration Factor	C_D = 1.6
Wet Service Factor	C_M = 1.0
End Grain Factor	C_eg = 1.0
Temperature Factor	C_t = 1.0

Calculate Connection Capacity

Connection Yield Mode Descriptions | Limits of Use | Diaphragm Factor Help | Load Duration Factor Help | Technical Help | Show Printable View

Connection Yield Modes

Im	2226 lbs.
Is	763 lbs.
II	825 lbs.
IIIm	943 lbs.
IIIs	363 lbs.
IV	298 lbs.

Adjusted ASD Capacity	298 lbs.

5. Walls

	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	2X6 @ 16" O.C. Bearing Wall Design				5-1	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

TYP. 2X6 WOOD STUD CHECK

1. Roof Load

Roof dead load = 0.5 * 20 psf * 1.33 ft * 46.33 ft = 616.19 lb

Roof snow load = 0.5 * 15 psf * 1.33 ft * 46.33 ft + 32.80 psf * 8.216 ft * 0.5 * 1.33 ft = 641.35 lb

Total compression load on gravity stud = 616.19 + 641.35 = 1257.54 lb

2. Wind Load

Wind load (ASCE 7-10) = (35.4 * 0.6) psf * 1.33 ft = 28.25 lb/ft

Moment=1/8 * 28.25 lb/ft * (13.33ft)^2 = 627.45 ft-lb

WOOD MEMBER DESIGN (NDS 2018)

In accordance with the ANSI/AF&PA NDS 2018 using the ASD method

Tedds calculation version 2.2.20

Design summary

Overall design utilisation	0.712
Overall design status	PASS

Design section s1 results summary	Unit	Capacity	Maximum	Utilization	Result
Bending stress	lb/in^2	2240	996	0.444	PASS
Shear stress	lb/in^2	240	34	0.143	PASS
Bearing stress	lb/in^2	405	23	0.056	PASS
Compressive stress	lb/in^2	507	152	0.301	PASS
Bending and axial force				0.712	PASS

Design section 1

User note: Check column at base

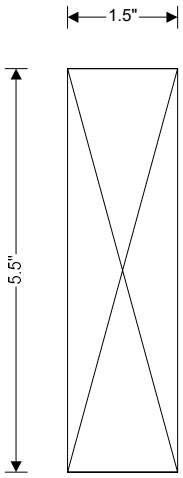
Member details

Service condition	Dry
Load duration - Table 2.3.2	Ten minutes

Sawn lumber section details

Number of sections in member	N = **1**
Nominal breadth of sections	b_{nom} = **2** in
Breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **6** in
Depth of sections	d = **5.5** in
Material	**Hem-Fir, 2" && wider, No.1 && Btr grade**

	Project				Job Ref.	
AECOM	McDonalds Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	2X6 @ 16” O.C. Bearing Wall Design				5-2	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024



2"x6" sawn lumber section
Cross-sectional area, A, 8.25 in^2
Section modulus, S_x, 7.6 in^3
Section modulus, S_y, 2.1 in^3
Second moment of area, I_x, 20.8 in^4
Second moment of area, I_y, 1.5 in^4
Radius of gyration, r_x, 1.588 in
Radius of gyration, r_y, 0.433 in
Hem-Fir, 2" & wider, No.1 & Btr grade
Bending, F_b, 1100 psi
Shear parallel to grain, F_v, 150 psi
Compression parallel to grain, F_c, 1350 psi
Compression perpendicular to grain, F_{c_perp}, 405 psi
Tension parallel to grain, F_t, 725 psi
Modulus of elasticity, E, 1500000 psi
Minimum modulus of elasticity, E_{min}, 550000 psi
Density, ρ, 29.743 lbm/ft^3
Specific gravity, G, 0.43

Span details

Unbraced length - Major axis	L_x = **13.33** ft
Effective bending length - Major axis	$L_{e,x} = 1.63 \times L_x + 3 \times b$ = **22.103** ft
Column buckling length - Major axis	$L_{b,x} = L_x$ = **13.33** ft
Unbraced length - Minor axis	L_y = **1** ft
Effective bending length - Minor axis	$L_{e,y} = 1.63 \times L_y + 3 \times d$ = **3.005** ft
Column buckling length - Minor axis	$L_{b,y} = L_y$ = **1** ft
Bearing length	L_b = **5.5** in

Analysis results

Design bending moment - Major axis	M_x = **627** lb_ft
Design shear force - Major axis	V_x = **188** lb
Design perpendicular compression - Major axis	R_x = **188** lb
Design axial compression force	P = **1258** lb

Adjustment factors - Table 4.3.1

Load duration factor - Table 2.3.2	C_D = **1.6**
Size factor for bending - Table 4A	C_{Fb} = **1.3**
Size factor for compression - Table 4A	C_{Fc} = **1.1**
Depth-to-breadth ratio	d_{nom} / b_{nom} = **3.00**
Slenderness ratio - eq.3.3-5	$R_B = \sqrt{(L_{e,y} \times d / b^2)}$ = **9.389**
Reference bending design value	$F_b^* = F_b \times C_D \times C_{Fb}$ = **2288** lb/in^2
Adjusted modulus of elasticity	$E_{min}' = E_{min}$ = **550000** lb/in^2
Critical buckling design value - cl.3.3.3.8	$F_{bE} = 1.2 \times E_{min}' / R_B^2$ = **7488** lb/in^2
Beam stability factor - eq.3.3-6	

$$C_L = [1 + (F_{bE} / F_b^*)] / 1.9 - \sqrt{[([1 + (F_{bE} / F_b^*)] / 1.9)^2 - (F_{bE} / F_b^*) / 0.95]} = \mathbf{0.979}$$

Reference compression design value	$F_c^* = F_c \times C_D \times C_{Fc}$ = **2376** lb/in^2
Adjusted modulus of elasticity	$E_{min}' = E_{min}$ = **550000** lb/in^2
Critical buckling design value	$F_{cE} = 0.822 \times E_{min}' / (L_{b,x} / d)^2$ = **534** lb/in^2
Column stability factor - eq.3.7-1	

$$C_P = (1 + (F_{cE} / F_c^*)) / 1.6 - \sqrt{(((1 + (F_{cE} / F_c^*)) / 1.6)^2 - (F_{cE} / F_c^*) / 0.8)} = \mathbf{0.213}$$

AECOM	Project: McDonalds Lillington, NC				Job Ref.: 032-0312	
	Section: 2X6 @ 16" O.C. Bearing Wall Design				Sheet no./rev.: 5-3	
	Calc. by: SA/PB	Date: 8/7/2024	Chk'd by: ES	Date: 8/7/2024	App'd by: MY	Date: 8/7/2024

Compression members - General - cl.3.6

Design axial compression force — P = **1258** lb

Design compression parallel to grain - Table 4.3.1 — $F_c' = F_c \times C_D \times C_{Fc} \times C_P$ = **507** lb/in^2

Actual compression parallel to grain — $f_c = P / (b \times d)$ = **152** lb/in^2

f_c / F_c' = **0.301**

PASS - Design compression stress exceeds actual compression stress

Bending members - Flexure - cl.3.3

Design bending moment — M_x = **627** lb_ft

Design bending stress - Table 4.3.1 — $F_{b,x}' = F_b \times C_D \times C_L \times C_{Fb}$ = **2240** lb/in^2

Actual bending stress - eq.3.3-2 — $f_{b,x} = M_x / S_x$ = **996** lb/in^2

$f_{b,x} / F_{b,x}'$ = **0.444**

PASS - Design bending stress exceeds actual bending stress

Bending members - Shear - cl.3.4

Design shear force — V_x = **188** lb

Design shear stress - Table 4.3.1 — $F_{v,x}' = F_v \times C_D$ = **240** lb/in^2

Actual shear stress - eq.3.4-2 — $f_{v,x} = 3 \times V_x / (2 \times b \times d)$ = **34** lb/in^2

$f_{v,x} / F_{v,x}'$ = **0.143**

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression — R_x = **188** lb

Design bearing stress - Table 4.3.1 — $F_{c\_perp,x}' = F_{c\_perp}$ = **405** lb/in^2

Actual bearing stress — $f_{c\_perp,x} = R_x / (b \times L_b)$ = **23** lb/in^2

$f_{c\_perp,x} / F_{c\_perp,x}'$ = **0.056**

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Combined bending and axial loading - cl.3.9

Critical buckling design value in x-axis — $F_{cE1} = 0.822 \times E_{min}' / (L_{b,x} / d)^2$ = **534** lb/in^2

Critical buckling design value in y-axis — $F_{cE2} = 0.822 \times E_{min}' / (L_{b,y} / b)^2$ = **7064** lb/in^2

Bending and compression check - eqs.3.9-3 and 3.9-4

$\max((f_c / F_c')^2 + f_{b,x} / (F_{b,x}' \times (1 - (f_c / F_{cE1}))), (f_c / F_{cE2}))$ = **0.712** < 1.0

PASS - Combined bending and compressive stresses are within permissible limits

(4)-2X6 WOOD STUD CHECK ((2)-2X6 FOR EACH JOIST OF DOUBLE JOISTS)

1. Roof Load

Roof dead load = 0.5 * 20 psf * 1.33 ft * 46.33 ft = 616.19 lb

Roof snow load = 0.5 * 15 psf * 1.33 ft * 46.33 ft + 32.80 psf * 8.216 ft * 0.5 * 1.33 ft = 641.35 lb

Wind uplift load from kicker check = 400 plf * 46.33 ft * 0.5 = 9266 lb

Total compression load on gravity stud = 616.19 + 641.35 + 12972 = 10523.54 lb

2. Wind Load

Wind load (ASCE 7-10) = (35.4 * 0.6) psf * 1.33 ft = 28.25 lb/ft

Moment=1/8 * 28.25 lb/ft * (13.33ft)^2 = 627.45 ft-lb

WOOD MEMBER DESIGN (NDS 2018)

In accordance with the ANSI/AF&PA NDS 2018 using the ASD method

Tedds calculation version 2.2.20

Design summary

Overall design utilisation	0.66
Overall design status	PASS

Design section s1 results summary	Unit	Capacity	Maximum	Utilization	Result
Bending stress	lb/in^2	2332	249	0.107	PASS
Shear stress	lb/in^2	240	9	0.036	PASS
Bearing stress	lb/in^2	405	6	0.014	PASS
Compressive stress	lb/in^2	507	319	0.629	PASS
Bending and axial force				0.660	PASS

Design section 1

User note: Check column at base

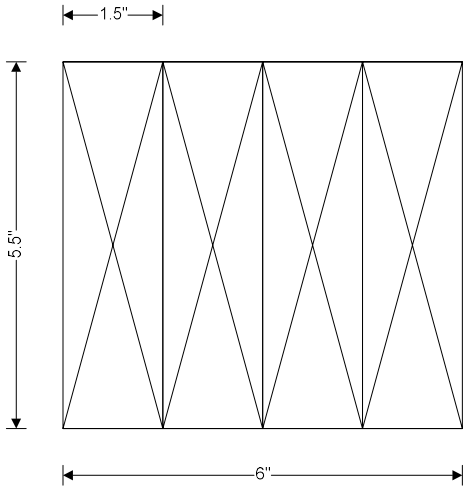
Member details

Service condition	Dry
Load duration - Table 2.3.2	Ten minutes

Sawn lumber section details

Number of sections in member	N = **4**
Nominal breadth of sections	b_{nom} = **2** in
Breadth of sections	b = **1.5** in
Nominal depth of sections	d_{nom} = **6** in
Depth of sections	d = **5.5** in
Material	**Hem-Fir, 2" && wider, No.1 grade**

	Project				Job Ref.	
AECOM	McDonald's Lillington, NC				032-0312	
	Section				Sheet no./rev.	
	(2)-2x6 @ 12" O.C. Bearing Wall Design				5-5	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024



4/2"x6" sawn lumber sections
Cross-sectional area, A, 33 in^2
Section modulus, S_x, 30.3 in^3
Section modulus, S_y, 33 in^3
Second moment of area, I_x, 83.2 in^4
Second moment of area, I_y, 99 in^4
Radius of gyration, r_x, 1.588 in
Radius of gyration, r_y, 1.732 in
Hem-Fir, 2" & wider, No.1 grade
Bending, F_b, 975 psi
Shear parallel to grain, F_v, 150 psi
Compression parallel to grain, F_c, 1350 psi
Compression perpendicular to grain, F_{c_perp}, 405 ps
Tension parallel to grain, F_t, 625 psi
Modulus of elasticity, E, 1500000 psi
Minimum modulus of elasticity, E_{min}, 550000 psi
Density, ρ, 29.743 lbm/ft^3
Specific gravity, G, 0.43

Span details

Unbraced length - Major axis	L_x = **13.33** ft
Effective bending length - Major axis	$L_{e,x} = 1.63 \times L_x + 3 \times N \times b$ = **23.228** ft
Column buckling length - Major axis	$L_{b,x} = L_x$ = **13.33** ft
Unbraced length - Minor axis	L_y = **1** ft
Effective bending length - Minor axis	$L_{e,y} = 1.63 \times L_y + 3 \times d$ = **3.005** ft
Column buckling length - Minor axis	$L_{b,y} = L_y$ = **1** ft
Bearing length	L_b = **5.5** in

Analysis results

Design bending moment - Major axis	M_x = **627** lb_ft
Design shear force - Major axis	V_x = **188** lb
Design perpendicular compression - Major axis	R_x = **188** lb
Design axial compression force	P = **10524** lb

Adjustment factors - Table 4.3.1

Load duration factor - Table 2.3.2	C_D = **1.6**
Size factor for bending - Table 4A	C_{Fb} = **1.3**
Size factor for compression - Table 4A	C_{Fc} = **1.1**
Repetitive member factor - Table 4.3.9	C_r = **1.15**
Reference compression design value	$F_c^* = F_c \times C_D \times C_{Fc}$ = **2376** lb/in^2
Adjusted modulus of elasticity	$E_{min}' = E_{min}$ = **550000** lb/in^2
Critical buckling design value	$F_{cE} = 0.822 \times E_{min}' / (L_{b,x} / d)^2$ = **534** lb/in^2
Column stability factor - eq.3.7-1	

$$C_P = (1 + (F_{cE} / F_c^*)) / 1.6 - \sqrt{(((1 + (F_{cE} / F_c^*)) / 1.6)^2 - (F_{cE} / F_c^*) / 0.8)} = \mathbf{0.213}$$

Compression members - General - cl.3.6

Design axial compression force	P = **10524** lb
Design compression parallel to grain - Table 4.3.1	$F_c' = F_c \times C_D \times C_{Fc} \times C_P$ = **507** lb/in^2
Actual compression parallel to grain	$f_c = P / (N \times b \times d)$ = **319** lb/in^2
	f_c / F_c' = **0.629**

PASS - Design compression stress exceeds actual compression stress

Bending members - Flexure - cl.3.3

Design bending moment — M_x = **627** lb_ft

Design bending stress - Table 4.3.1 — $F_{b,x}' = F_b \times C_D \times C_{Fb} \times C_r$ = **2332** lb/in^2

Actual bending stress - eq.3.3-2 — $f_{b,x} = M_x / S_x$ = **249** lb/in^2

$f_{b,x} / F_{b,x}'$ = **0.107**

PASS - Design bending stress exceeds actual bending stress

Bending members - Shear - cl.3.4

Design shear force — V_x = **188** lb

Design shear stress - Table 4.3.1 — $F_{v,x}' = F_v \times C_D$ = **240** lb/in^2

Actual shear stress - eq.3.4-2 — $f_{v,x} = 3 \times V_x / (2 \times N \times b \times d)$ = **9** lb/in^2

$f_{v,x} / F_{v,x}'$ = **0.036**

PASS - Design shear stress exceeds actual shear stress

Design for bearing - cl.3.10

Design perpendicular compression — R_x = **188** lb

Design bearing stress - Table 4.3.1 — $F_{c\_perp,x}' = F_{c\_perp}$ = **405** lb/in^2

Actual bearing stress — $f_{c\_perp,x} = R_x / (N \times b \times L_b)$ = **6** lb/in^2

$f_{c\_perp,x} / F_{c\_perp,x}'$ = **0.014**

PASS - Design bearing stress exceeds actual bearing stress perpendicular to grain

Combined bending and axial loading - cl.3.9

Critical buckling design value in x-axis — $F_{cE1} = 0.822 \times E_{min}' / (L_{b,x} / d)^2$ = **534** lb/in^2

Critical buckling design value in y-axis — $F_{cE2} = 0.822 \times E_{min}' / (L_{b,y} / (N \times b))^2$ = **113025** lb/in^2

Bending and compression check - eqs.3.9-3 and 3.9-4

$\max((f_c / F_c')^2 + f_{b,x} / (F_{b,x}' \times (1 - (f_c / F_{cE1}))), (f_c / F_{cE2}))$ = **0.660** < 1.0

PASS - Combined bending and compressive stresses are within permissible limits

6. Foundations

	Project				Job Ref.	
AECOM	McDonald's Mebane, NC				032-0312	
	Section				Sheet no./rev.	
	Gravity & Lateral Footing Design				6-1	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

FOOTING DESIGN NOTES

The following notes apply to all footing designs in this section:

1. Minimum area of tension reinforcement failures are O.K. because minimum area reinforcement requirement *INCLUDES* compression reinforcement. Calculation only considers tension reinforcement in minimum area reinforcement requirement.
2. Allowable bearing pressure = net bearing pressure * soil weight above bottom of footing.

	Project			Job Ref.	
AECOM	McDonald's Mebane, NC			032-0312	
	Section			Sheet no./rev.	
	Gravity & Lateral Footing Design			6-2	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

GRAVITY FOOTING F1: ANALYSIS & DESIGN (ACI-318)

Use the reactions from column C1 at Gridline E/2 as the load on foundation (loads information is available on column C1 design report)

DL= 10.11 k

SL= 5.64 k

RAM® Elements CONNECT Edition

AECOM

Current Date: 8/2/2024 4:00 PM
Units system: English

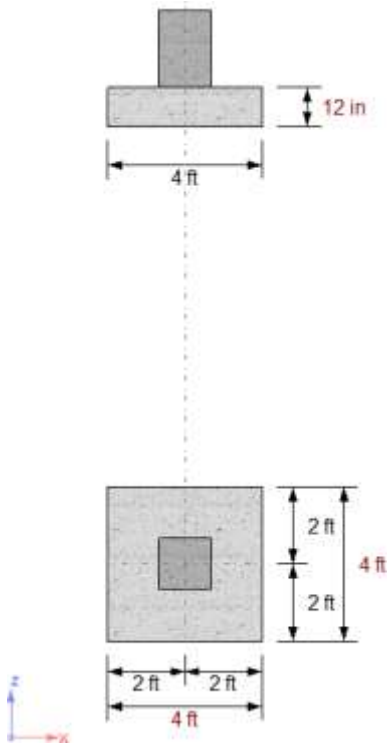
Design Results

Reinforced Concrete Footings

GENERAL INFORMATION:

Global status	:	OK
Design Code	:	ACI 318-2014
Footing type	:	Spread
Column type	:	Pedestal

Geometry



Length	:	4.00 [ft]
Width	:	4.00 [ft]
Thickness	:	1.00 [ft]

Base depth	:	3.00 [ft]
Base area	:	16.00 [ft2]
Footing volume	:	16.00 [ft3]
Pedestal length	:	16.00 [in]
Pedestal width	:	16.00 [in]
Pedestal height	:	24.00 [in]

Pedestal location relative to footing g.c. : Centered

Materials

Concrete, f'c	:	3.00 [Kip/in2]	Steel, fy	:	60.00 [Kip/in2]
Concrete type	:	Normal	Epoxy coated	:	No
Concrete elasticity modulus	:	3122.02 [Kip/in2]	Steel elasticity modulus	:	29000.00 [Kip/in2]
Unit weight	:	0.15 [Kip/ft3]			

Soil

Modulus of subgrade reaction	:	259.20 [Kip/ft3]
Unit weight (wet)	:	0.11 [Kip/ft3]

Footing reinforcement

Free cover	:	3.00 [in]
Maximum Rho/Rho balanced ratio	:	0.75
Bottom reinforcement // to L (xx)	:	5-#5 @ 10.00"
Bottom reinforcement // to B (zz)	:	5-#5 @ 10.00" (Zone 1)

Pedestal reinforcement

Longitudinal	:	8 - #6
Free cover	:	1.00 [in]
Provided area	:	3.52 [in2]
Bars number // to x axis	:	3
Bars number // to z axis	:	3
Transverse	:	#4 @ 12.00"
Legs number // to x axis	:	2
Legs number // to z axis	:	2

Load conditions to be included in design

Service loads:

S1	:	DL
S2	:	DL+SL
S3	:	DL+0.75SL
S4	:	DL+0.6WL
S5	:	DL+0.75SL
S6	:	0.6DL+0.6WL

Design strength loads:

D1	:	1.4DL
D2	:	1.2DL+0.5SL
D3	:	1.2DL+1.6SL
D4	:	1.2DL+0.5WL
D5	:	1.2DL+1.6SL+0.5WL
D6	:	1.2DL+WL
D7	:	1.2DL+WL+0.5SL
D8	:	1.2DL+0.2SL
D9	:	0.9DL+WL

Loads

Condition	Footing	Node	Axial	Mxx	Mzz	Vx	Vz

			[Kip]	[Kip*ft]	[Kip*ft]	[Kip]	[Kip]
DL	1	1	10.11	0.00	0.00	0.00	0.00
SL	1	1	5.64	0.00	0.00	0.01	0.00
WL	1	1	0.00	0.00	0.00	1.87	0.00

RESULTS:

Status : OK

Soil.Foundation interaction

Allowable stress : 3E03 [Lb/ft2]
Min. safety factor for sliding : 1.25
Min. safety factor for overturning : 1.25

Controlling condition : S2 - 1

Condition	qmean	qmax	Δmax	Area in compression		Overturning		FS
Footing	[Lb/ft2]	[Lb/ft2]	[in]	[ft2]	(%)	FSx	FSz	slip
S2 - 1	1.35E03	1.35E03	0.0627	16.00	100	1000.00	1000.00	1000.00

Bending

Factor ϕ : 0.90
Min rebar ratio : 0.00180

Development length

Axis	Pos.	ld [in]	lhd [in]	Dist1 [in]	Dist2 [in]
z	Bot.	18.32	6.41	13.00	13.00
x	Bot.	18.32	6.41	13.00	13.00

Axis	Pos.	Condition Footing	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Asreq [in2]	Asprov [in2]	Asreq/Asprov	Mu/(ϕ*Mn)
zz	Top	D1 - 1	0.00	0.00	0.00	0.00	0.000	0.000
zz	Bot.	D5 - 1	5.43	57.95	1.04	1.55	0.669	0.094
xx	Top	D1 - 1	0.00	0.00	0.00	0.00	0.000	0.000
xx	Bot.	D5 - 1	4.70	53.59	1.04	1.55	0.669	0.088

Shear

Factor ϕ : 0.75
Shear area (plane zz) : 2.90 [ft2]
Shear area (plane xx) : 2.69 [ft2]

Plane	Condition Footing	Vu [Kip]	Vc [Kip]	Vu/(ϕ*Vn)
xy	D5 - 1	3.50	42.39	0.110

yz	D5 - 1	3.77	45.68	0.110	

Punching shear

Factor ϕ : 0.75
Perimeter of critical section (b... : 8.13 [ft]
Punching shear area : 5.67 [ft2]

Column	Condition Footing	Vu [Kip]	Vc [Kip]	Vu/(ϕ*Vn)	
column 1	D3 - 1	15.69	178.90	0.117	

Pedestal design

Reinforcement:

Pedestal	Asreq [in2]	Asprov [in2]	Asreq/Asprov
1	2.56	3.52	0.73

Biaxial bending:

Pedestal	Condition Footing	Muxx [Kip*ft]	Muzz [Kip*ft]	ϕ*Mnxx [Kip*ft]	ϕ*Mnzz [Kip*ft]	Mc/(ϕ*Mn)	
1	D9 - 1	0.00	3.73	0.00	106.19	0.04	

Axial:

Pedestal	Condition Footing	Pu [Kip]	ϕ*Pn [Kip]	Pu/(ϕ*Pn)	
1	D3 - 1	-21.14	-444.61	0.05	

Shear:
Pedestal 1:
S provided : 12.00 [in]
S required : 12.00 [in]

Condition	Dir.	Vu [Kip]	Vc [Kip]	Vs [Kip]	ϕ*Vn [Kip]	Vu/(ϕ*Vn)	
D9	x	1.87	25.20	28.25	40.09	0.05	
D9	z	0.00	25.20	28.25	40.09	0.00	

Notes

* Soil under the footing is considered elastic and homogeneous. A linear soil pressure variation is assumed.

* The required flexural reinforcement considers at least the minimum reinforcement

AECOM	Project				Job Ref.	
	McDonald's Mebane, NC				032-0312	
	Section				Sheet no./rev.	
	Gravity & Lateral Footing Design				6-6	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

* The design bending moment is calculated at the critical sections located at the support faces

* Only rectangular footings with uniform sections and rectangular columns are considered.

* The nominal shear strength is calculated in critical sections located at a distance d from the support face

* The punching shear strength is calculated in a perimetral section located at a distance d/2 from the support faces

* Transverse reinforcement is not considered in footings

* Values shown in red are not in compliance with a provision of the code

*qprom = Mean compression pressure on soil.

*qmax = Maximum compression pressure on soil.

*Δmax = maximum total settlement (considering an elastic soil modeled by the subgrade reaction modulus).

* Mn = Nominal moment strength

* Mu/(ϕ*Mn) = Strength ratio.

* Vn = Nominal shear or punchure force (for footings Vn=Vc).

* Vu/(ϕ*Vn) = Shear or punching shear strength ratio.

FOOTING F2 (FOUNDATION FOR BF1): ANALYSIS & DESIGN (ACI-318)

*See all the loading information on BF1 design report
*Use temperature bars on minor axis of the footing

Foundation Design Results for BF-1

RAM® Elements CONNECT Edition

AECOM

Current Date: 8/6/2024 3:10 PM
Units system: English

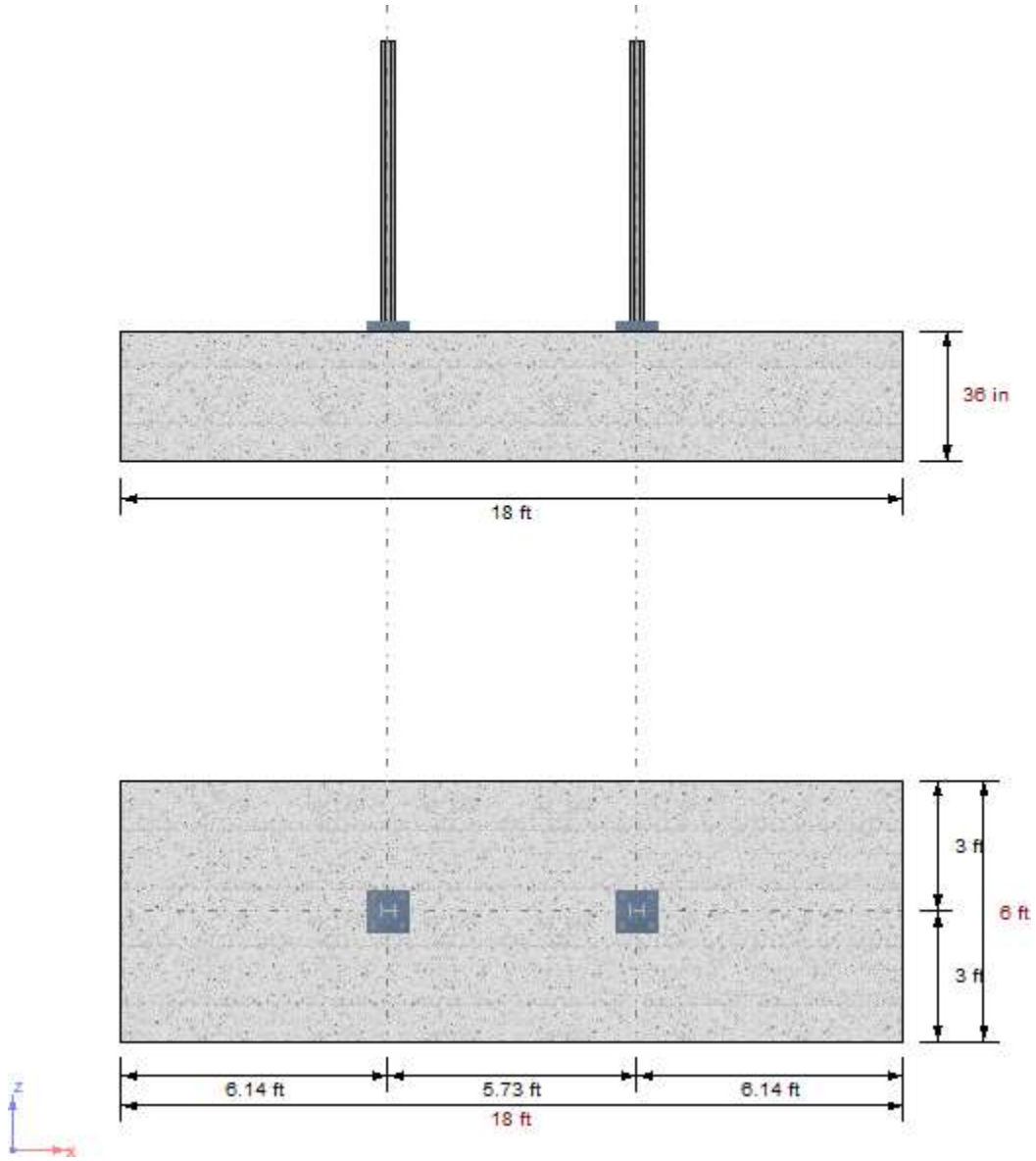
Reinforced Concrete Footings

GENERAL INFORMATION:

Global status	:	OK
Design Code	:	ACI 318-2014
Footing type	:	Combined
Column type	:	Steel

Geometry

	Project				Job Ref.	
AECOM	McDonald's Mebane, NC				032-0312	
	Section				Sheet no./rev.	
	Gravity & Lateral Footing Design				6-8	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024



Length	:	18.00 [ft]
Width	:	6.00 [ft]
Thickness	:	3.00 [ft]
Base depth	:	5.00 [ft]
Base area	:	108.00 [ft2]
Footing volume	:	324.00 [ft3]
Base plate length 1	:	12.00 [in]
Base plate width 1	:	12.00 [in]
Column length 1	:	4.00 [in]
Column width 1	:	4.00 [in]
Base plate length 2	:	12.00 [in]
Base plate width 2	:	12.00 [in]
Column length 2	:	4.00 [in]
Column width 2	:	4.00 [in]

Distance between columns : 5.73 [ft]
Column location relative to footing g.c. : Centered

Materials

Concrete, f'c	:	3.00 [Kip/in2]	Steel, fy	:	60.00 [Kip/in2]
Concrete type	:	Normal	Epoxy coated	:	No
Concrete elasticity modulus	:	3122.02 [Kip/in2]	Steel elasticity modulus	:	29000.00 [Kip/in2]
Unit weight	:	0.15 [Kip/ft3]			

Soil

Modulus of subgrade reaction	:	259.20 [Kip/ft3]
Unit weight (wet)	:	0.11 [Kip/ft3]

Footing reinforcement

Free cover	:	3.00 [in]
Maximum Rho/Rho balanced ratio	:	0.75
Bottom reinforcement // to L (xx)	:	6-#8 @ 13.00"
Top reinforcement // to L (xx)	:	6-#8 @ 13.00"
Bottom reinforcement // to B (zz)	:	4-#8 @ 14.00" (Zone 1)
Bottom reinforcement // to B (zz)	:	5-#8 @ 13.00" (Zone 2)
Bottom reinforcement // to B (zz)	:	2-#8 @ 15.00" (Zone 3)
Bottom reinforcement // to B (zz)	:	5-#8 @ 13.00" (Zone 4)
Bottom reinforcement // to B (zz)	:	4-#8 @ 14.00" (Zone 5)
Top reinforcement // to B (zz)	:	4-#8 @ 14.00"
Top reinforcement // to B (zz)	:	4-#8 @ 17.00"
Top reinforcement // to B (zz)	:	2-#8 @ 15.00"
Top reinforcement // to B (zz)	:	5-#8 @ 13.00"
Top reinforcement // to B (zz)	:	4-#8 @ 14.00"

Load conditions to be included in design

Service loads:

S1	:	DL
S2	:	DL+SL
S3	:	DL+0.75SL
S4	:	DL+0.6WL1
S5	:	DL+0.6WL2
S6	:	1.04DL+0.7EQ1
S7	:	1.04DL+0.7EQ2
S8	:	1.04DL+0.525EQ1
S9	:	1.04DL+0.525EQ2
S10	:	DL+0.75SL
S11	:	1.03DL+0.75SL+0.525EQ1
S12	:	1.03DL+0.75SL+0.525EQ2
S13	:	0.6DL+0.6WL1
S14	:	0.6DL+0.6WL2
S15	:	0.57DL+0.7EQ1
S16	:	0.57DL+0.7EQ2

Design strength loads:

D1	:	1.4DL
D2	:	1.2DL+0.5SL
D3	:	1.2DL+1.6SL
D4	:	1.2DL+0.5WL1
D5	:	1.2DL+0.5WL2
D6	:	1.2DL+1.6SL+0.5WL1
D7	:	1.2DL+1.6SL+0.5WL2
D8	:	1.2DL+WL1
D9	:	1.2DL+WL2
D10	:	1.2DL+0.5SL+WL1
D11	:	1.2DL+0.5SL+WL2
D12	:	1.2DL+0.2SL

D13 : 1.2DL+EQ1
D14 : 1.25DL+EQ2
D15 : 1.25DL+0.2SL+EQ1
D16 : 1.25DL+0.2SL+EQ2
D17 : 0.9DL+WL1
D18 : 0.9DL+WL2
D19 : 0.85DL+EQ1
D20 : 0.85DL+EQ2

Loads

Condition	Footing	Node	Column	Axial [Kip]	Mxx [Kip*ft]	Mzz [Kip*ft]	Vx [Kip]	Vz [Kip]
DL	1	12	1	5.37	0.00	0.00	-0.63	0.00
SL	1	12	1	2.77	0.00	0.00	-0.33	0.00
WL1	1	12	1	76.95	0.00	0.00	-16.10	0.00
WL2	1	12	1	-76.95	0.00	0.00	15.62	0.00
EQ1	1	12	1	34.57	0.00	0.00	-7.23	0.00
EQ2	1	12	1	-34.57	0.00	0.00	7.02	0.00
DL	1	14	2	1.75	0.00	0.00	0.63	0.00
SL	1	14	2	0.93	0.00	0.00	0.33	0.00
WL1	1	14	2	-76.95	0.00	0.00	-15.62	0.00
WL2	1	14	2	76.95	0.00	0.00	16.10	0.00
EQ1	1	14	2	-34.57	0.00	0.00	-7.02	0.00
EQ2	1	14	2	34.57	0.00	0.00	7.23	0.00

RESULTS:

Status : OK

Soil.Foundation interaction

Allowable stress : 3E03 [Lb/ft2]
Min. safety factor for sliding : 1.25
Min. safety factor for overturning : 1.25

Controlling condition : S13 - 1

Condition Footing	qmean [Lb/ft2]	qmax [Lb/ft2]	Δmax [in]	Area in compression [ft2]	Area in compression (%)	Overturning FSx	Overturning FSz	FS slip
S13 - 1	1.25E03	2.5E03	0.116	38.17	35	1000.00	1.31	1.97

Bending

Factor ϕ : 0.90
Min rebar ratio : 0.00180

Development length

Axis	Pos.	ld [in]	lhd [in]	Dist1 [in]	Dist2 [in]
z	Bot.	48.52	13.59	29.00	29.00
x	Bot.	53.91	15.10	66.62	66.62
z	Top	12.00	8.00	31.00	31.00
x	Top	12.00	8.00	68.62	68.62

Axis	Pos.	Condition Footing	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Asreq [in2]	Asprov [in2]	Asreq/Asprov	Mu/(ϕ*Mn)
zz	Top	D10 - 1	-81.12	-676.70	0.56	4.74	0.117	0.120
zz	Bot.	D17 - 1	255.93	676.70	4.25	4.74	0.896	0.378
Zone 1 xx	Top	D1 - 1	0.00	436.65	0.00	3.16	0.000	0.000
Zone 1 xx	Bot.	D1 - 1	0.00	436.65	3.04	3.16	0.961	0.000
Zone 2 xx	Top	D18 - 1	-10.82	438.05	0.08	3.16	0.024	0.025
Zone 2 xx	Bot.	D10 - 1	12.72	544.47	3.39	3.95	0.858	0.023
Zone 3 xx	Top	D1 - 1	0.00	215.30	0.00	1.58	0.000	0.000
Zone 3 xx	Bot.	D1 - 1	0.00	215.30	0.99	1.58	0.626	0.000
Zone 4 xx	Top	D17 - 1	-11.31	544.47	0.08	3.95	0.020	0.021
Zone 4 xx	Bot.	D11 - 1	11.93	544.47	3.39	3.95	0.857	0.022
Zone 5 xx	Top	D1 - 1	0.00	436.65	0.00	3.16	0.000	0.000
Zone 5 xx	Bot.	D1 - 1	0.00	436.65	3.04	3.16	0.961	0.000

Shear

Factor ϕ	:	0.75
Shear area (plane zz)	:	16.25 [ft2]
Shear area (plane xx)	:	47.25 [ft2]

Plane	Condition Footing	Vu [Kip]	Vc [Kip]	Vu/(ϕ*Vn)
xy	D7 - 1	0.10	745.34	0.000
yz	D17 - 1	55.92	256.33	0.291

Punching shear

Factor ϕ	:	0.75
Perimeter of critical section (b...	:	13.33 [ft]
Punching shear area	:	35.56 [ft2]
Perimeter of critical section (b...	:	13.33 [ft]
Punching shear area	:	35.56 [ft2]

Column	Condition Footing	Vu [Kip]	Vc [Kip]	Vu/(ϕ*Vn)
column 1	D17 - 1	88.48	1121.74	0.105
column 2	D17 - 1	-68.67	1121.74	0.082

Notes

* Soil under the footing is considered elastic and homogeneous. A linear soil pressure variation is assumed.

* The required flexural reinforcement considers at least the minimum reinforcement

* The design bending moment is calculated at the critical sections located at the support faces

* Only rectangular footings with uniform sections and rectangular columns are considered.

* The nominal shear strength is calculated in critical sections located at a distance d from the support face

* The punching shear strength is calculated in a perimetral section located at a distance d/2 from the support faces

* Transverse reinforcement is not considered in footings

* Values shown in red are not in compliance with a provision of the code

*qprom = Mean compression pressure on soil.

*qmax = Maximum compression pressure on soil.

*Δmax = maximum total settlement (considering an elastic soil modeled by the subgrade reaction modulus).

* Mn = Nominal moment strength

* Mu/(ϕ*Mn) = Strength ratio.

* Vn = Nominal shear or punchure force (for footings Vn=Vc).

* Vu/(ϕ*Vn) = Shear or punching shear strength ratio.

FOOTING F3 (FOUNDATION FOR BF2): ANALYSIS & DESIGN (ACI-318)

*See all the loading information on BF2 design report
*Use temperature bars on minor axis of the footing

Foundation Design Results for BF-2

RAM Elements CONNECT Edition

AECOM

Current Date: 8/6/2024 3:17 PM
Units system: English

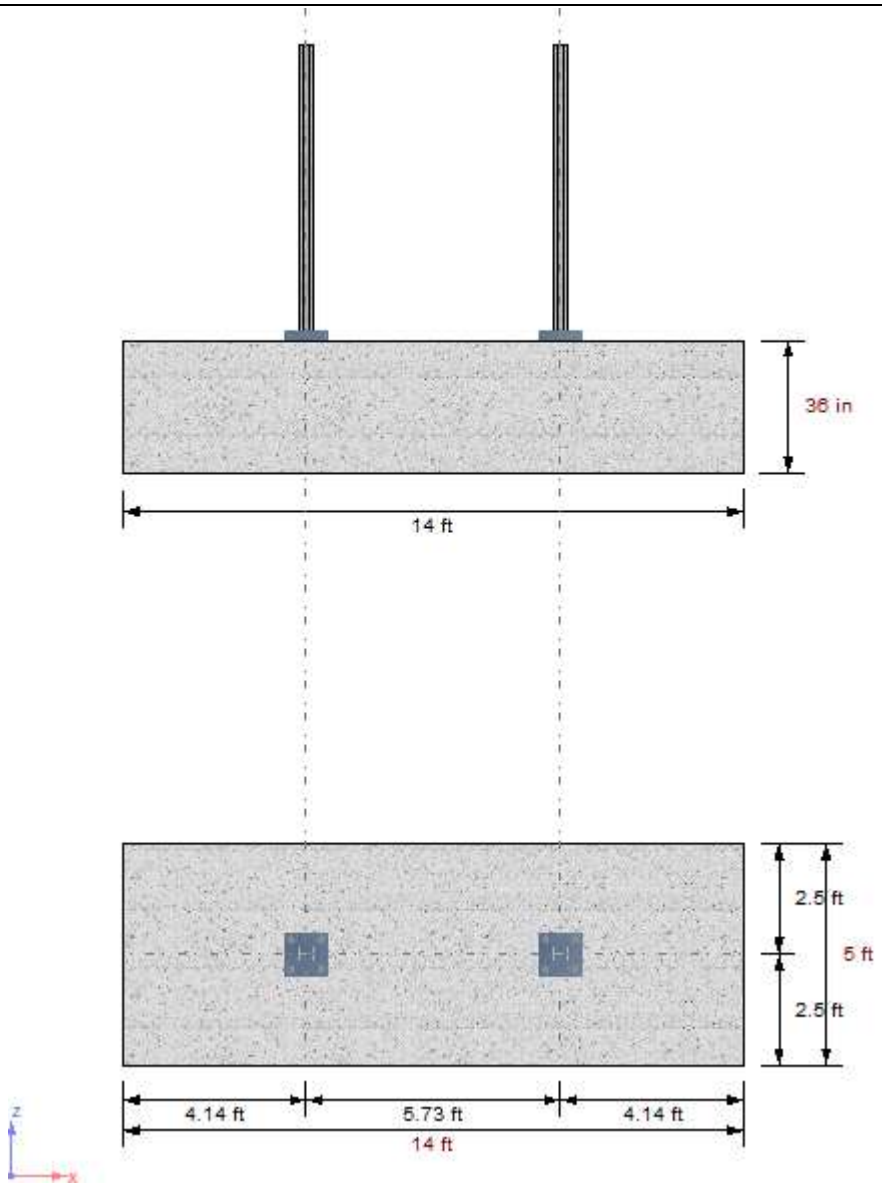
Reinforced Concrete Footings

GENERAL INFORMATION:

Global status	:	OK
Design Code	:	ACI 318-2014
Footing type	:	Combined
Column type	:	Steel

Geometry

	Project				Job Ref.	
AECOM	McDonald's Mebane, NC				032-0312	
	Section				Sheet no./rev.	
	Gravity & Lateral Footing Design				6-14	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024



Length	:	14.00 [ft]
Width	:	5.00 [ft]
Thickness	:	3.00 [ft]
Base depth	:	5.00 [ft]
Base area	:	70.00 [ft2]
Footing volume	:	210.00 [ft3]
Base plate length 1	:	12.00 [in]
Base plate width 1	:	12.00 [in]
Column length 1	:	4.00 [in]
Column width 1	:	4.00 [in]
Base plate length 2	:	12.00 [in]
Base plate width 2	:	12.00 [in]
Column length 2	:	4.00 [in]
Column width 2	:	4.00 [in]

Distance between columns : 5.73 [ft]
Column location relative to footing g.c. : Centered

Materials

Concrete, f'c	:	3.00 [Kip/in2]	Steel, fy	:	60.00 [Kip/in2]
Concrete type	:	Normal	Epoxy coated	:	No
Concrete elasticity modulus	:	3122.02 [Kip/in2]	Steel elasticity modulus	:	29000.00 [Kip/in2]
Unit weight	:	0.15 [Kip/ft3]			

Soil

Modulus of subgrade reaction	:	259.20 [Kip/ft3]
Unit weight (wet)	:	0.11 [Kip/ft3]

Footing reinforcement

Free cover	:	3.00 [in]
Maximum Rho/Rho balanced ratio	:	0.75
Bottom reinforcement // to L (xx)	:	5-#8 @ 12.00"
Top reinforcement // to L (xx)	:	5-#8 @ 12.00"
Bottom reinforcement // to B (zz)	:	3-#8 @ 9.00" (Zone 1)
Bottom reinforcement // to B (zz)	:	6-#8 @ 9.00" (Zone 2)
Bottom reinforcement // to B (zz)	:	2-#8 @ 15.00" (Zone 3)
Bottom reinforcement // to B (zz)	:	6-#8 @ 9.00" (Zone 4)
Bottom reinforcement // to B (zz)	:	3-#8 @ 9.00" (Zone 5)
Top reinforcement // to B (zz)	:	3-#8 @ 9.00"
Top reinforcement // to B (zz)	:	6-#8 @ 9.00"
Top reinforcement // to B (zz)	:	2-#8 @ 15.00"
Top reinforcement // to B (zz)	:	6-#8 @ 9.00"
Top reinforcement // to B (zz)	:	3-#8 @ 9.00"

Load conditions to be included in design

Service loads:

S1	:	DL
S2	:	DL+SL
S3	:	DL+0.75SL
S4	:	DL+0.6WL1
S5	:	DL+0.6WL2
S6	:	1.04DL+0.7EQ1
S7	:	1.04DL+0.7EQ2
S8	:	1.04DL+0.525EQ1
S9	:	1.04DL+0.525EQ2
S10	:	DL+0.75SL
S11	:	1.03DL+0.75SL+0.525EQ1
S12	:	1.03DL+0.75SL+0.525EQ2
S13	:	0.6DL+0.6WL1
S14	:	0.6DL+0.6WL2
S15	:	0.57DL+0.7EQ1
S16	:	0.57DL+0.7EQ2

Design strength loads:

D1	:	1.4DL
D2	:	1.2DL+0.5SL
D3	:	1.2DL+1.6SL
D4	:	1.2DL+0.5WL1
D5	:	1.2DL+0.5WL2
D6	:	1.2DL+1.6SL+0.5WL1
D7	:	1.2DL+1.6SL+0.5WL2
D8	:	1.2DL+WL1
D9	:	1.2DL+WL2
D10	:	1.2DL+0.5SL+WL1
D11	:	1.2DL+0.5SL+WL2
D12	:	1.2DL+0.2SL
D13	:	1.2DL+EQ1

D14 : 1.25DL+EQ2
D15 : 1.25DL+0.2SL+EQ1
D16 : 1.25DL+0.2SL+EQ2
D17 : 0.9DL+WL1
D18 : 0.9DL+WL2
D19 : 0.85DL+EQ1
D20 : 0.85DL+EQ2

Loads

Condition	Footing	Node	Column	Axial [Kip]	Mxx [Kip*ft]	Mzz [Kip*ft]	Vx [Kip]	Vz [Kip]
DL	1	12	1	8.55	0.00	0.00	-0.86	0.00
SL	1	12	1	4.71	0.00	0.00	-0.48	0.00
WL1	1	12	1	29.33	0.00	0.00	-6.09	0.00
WL2	1	12	1	-29.33	0.00	0.00	6.00	0.00
EQ1	1	12	1	13.17	0.00	0.00	-2.74	0.00
EQ2	1	12	1	-13.17	0.00	0.00	2.69	0.00
DL	1	14	2	1.13	0.00	0.00	0.86	0.00
SL	1	14	2	0.57	0.00	0.00	0.48	0.00
WL1	1	14	2	-29.33	0.00	0.00	-6.00	0.00
WL2	1	14	2	29.33	0.00	0.00	6.09	0.00
EQ1	1	14	2	-13.17	0.00	0.00	-2.69	0.00
EQ2	1	14	2	13.17	0.00	0.00	2.74	0.00

RESULTS:

Status : OK

Soil.Foundation interaction

Allowable stress : 3E03 [Lb/ft2]
Min. safety factor for sliding : 1.25
Min. safety factor for overturning : 1.25

Controlling condition : S13 - 1

Condition Footing	qmean [Lb/ft2]	qmax [Lb/ft2]	Δmax [in]	Area in compression [ft2]	Area in compression (%)	Overturning FSx	Overturning FSz	FS slip
S13 - 1	751	1.5E03	0.0696	45.17	65	1000.00	1.75	4.10

Bending

Factor ϕ : 0.90
Min rebar ratio : 0.00180

Development length

Axis	Pos.	ld [in]	lhd [in]	Dist1 [in]	Dist2 [in]
z	Bot.	37.74	10.57	23.00	23.00
x	Bot.	53.91	15.10	42.62	42.62
z	Top	12.00	8.00	25.00	25.00
x	Top	12.00	8.00	44.62	44.62

	Project			Job Ref.	
AECOM	McDonald's Mebane, NC			032-0312	
	Section			Sheet no./rev.	
	Gravity & Lateral Footing Design			6-17	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	SA/PB	8/7/2024	ES	8/7/2024	MY	8/7/2024

Axis	Pos.	Condition Footing	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Asreq [in2]	Asprov [in2]	Asreq/Asprov	Mu/(ϕ*Mn)
zz	Top	D8 - 1	-28.98	-563.92	0.20	3.95	0.050	0.051
zz	Bot.	D17 - 1	56.98	563.92	3.74	3.95	0.947	0.101
Zone 1 xx	Top	D1 - 1	0.00	322.95	0.00	2.37	0.000	0.000
Zone 1 xx	Bot.	D1 - 1	0.00	322.95	1.48	2.37	0.625	0.000
Zone 2 xx	Top	D18 - 1	-2.64	649.66	0.02	4.74	0.004	0.004
Zone 2 xx	Bot.	D10 - 1	5.12	649.66	3.45	4.74	0.727	0.008
Zone 3 xx	Top	D1 - 1	0.00	215.30	0.00	1.58	0.000	0.000
Zone 3 xx	Bot.	D1 - 1	0.00	215.30	0.99	1.58	0.626	0.000
Zone 4 xx	Top	D17 - 1	-3.46	649.66	0.02	4.74	0.005	0.005
Zone 4 xx	Bot.	D11 - 1	3.78	649.66	3.44	4.74	0.726	0.006
Zone 5 xx	Top	D1 - 1	0.00	322.95	0.00	2.37	0.000	0.000
Zone 5 xx	Bot.	D1 - 1	0.00	322.95	1.48	2.37	0.625	0.000

Shear

Factor ϕ	:	0.75
Shear area (plane zz)	:	13.54 [ft2]
Shear area (plane xx)	:	36.75 [ft2]

Plane	Condition Footing	Vu [Kip]	Vc [Kip]	Vu/(ϕ*Vn)
xy	D1 - 1	0.00	579.71	0.000
yz	D17 - 1	9.93	213.61	0.062

Punching shear

Factor ϕ	:	0.75
Perimeter of critical section (b...	:	13.33 [ft]
Punching shear area	:	35.56 [ft2]
Perimeter of critical section (b...	:	13.33 [ft]
Punching shear area	:	35.56 [ft2]

Column	Condition Footing	Vu [Kip]	Vc [Kip]	Vu/(ϕ*Vn)
column 1	D10 - 1	33.08	1121.74	0.039
column 2	D10 - 1	-21.75	1121.74	0.026

Notes

* Soil under the footing is considered elastic and homogeneous. A linear soil pressure variation is assumed.

* The required flexural reinforcement considers at least the minimum reinforcement

* The design bending moment is calculated at the critical sections located at the support faces

* Only rectangular footings with uniform sections and rectangular columns are considered.

* The nominal shear strength is calculated in critical sections located at a distance d from the support face

* The punching shear strength is calculated in a perimetral section located at a distance d/2 from the support faces

* Transverse reinforcement is not considered in footings

* Values shown in red are not in compliance with a provision of the code

*qprom = Mean compression pressure on soil.

*qmax = Maximum compression pressure on soil.

*Δmax = maximum total settlement (considering an elastic soil modeled by the subgrade reaction modulus).

* Mn = Nominal moment strength

* Mu/(ϕ*Mn) = Strength ratio.

* Vn = Nominal shear or punchure force (for footings Vn=Vc).

* Vu/(ϕ*Vn) = Shear or punching shear strength ratio.

WALL FOOTING: BEARING CHECK (ACI-318)

The following loads need to be considered for bearing check:

(1) ASD roof loading: 1258 lb/ 1.33 ft = 945.86 lb/ft
(2) Wall finish weight: 22 psf * 18.75 ft = 413 lb/ft
(3) Foundation wall weight: 150 pcf * 16 in / 12in/ft * 2.5 ft = 500 lb/ft
(4) Wall footing weight: 150 pcf * 24 in / 12 in/ft * 1 ft = 300 lb/ft

Total loading = 945.86+413+500+300 = 2158.86 lb/ft /ft < 6000 psf [(3000 psf * 1ft * 2ft) / ft] allowable soil bearing pressure.

7. Geotechnical Report

ECS Southeast, LLC

Geotechnical Engineering Report

32-0312 McDonald's Lillington, NC, 01.00.14

102 W Cornelius Harnett Blvd.
Lillington, Harnett County, NC

ECS Project No. 33:6900

July 15, 2024

ECS SOUTHEAST, LLC
NC Engineering License No. F-1519
Geotechnical • Construction Materials • Environmental • Facilities

July 15, 2024

Ms. Ana Ferreira-Martinez
McDonald's USA, LLC
110 N Carpenter Street
Chicago, IL 60607

ECS Project No. 33:6900

Reference: Geotechnical Engineering Report
32-0312 McDonald's Lillington, NC, 01.00.14
102 W Cornelius Harnett Blvd.
Lillington, Harnett County, NC

Dear Ms. Ferreira-Martinez:

ECS Southeast, LLC (ECS) has completed the subsurface exploration, laboratory testing, and geotechnical engineering evaluation for the above-referenced project. Our services were performed in general accordance with our agreed to scope of work. This report presents our understanding of the geotechnical aspects of the project along with the results of the field exploration and laboratory testing conducted, and our design and construction recommendations.

It has been our pleasure to be of service to McDonald's USA, LLC during the design phase of this project. We would appreciate the opportunity to remain involved during the continuation of the design phase, and also provide our services during construction phase operations to verify the subsurface conditions identified for this report. Should you have questions concerning the information contained in this report, or if we can be of further assistance to you, please contact us.

Respectfully submitted,

ECS Southeast, LLC

Blake A Hash

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EXECUTIVE SUMMARY

This Executive Summary is intended as a very brief overview of the primary geotechnical conditions that are expected to affect design and construction. Information gleaned from this Executive Summary should not be utilized in lieu of reading the entire geotechnical report.

- Existing fill was encountered in borings B-01 through B-04 to depths ranging between 6 to 8.5 feet below existing grade surface. Trace amounts of organics and debris were observed in some of the fill soil samples. The SPT boring N-values indicate that the existing fill was likely not thoroughly and adequately compacted. The existing fill should be undercut below the planned building pad area and replaced with Engineered Fill.

- Based on the soil test borings, we anticipate undercutting of existing fill and very soft to soft or very loose near-surface natural soils will be necessary across most of the site. If site earthwork is performed during the typically cooler, wetter months of the year, additional undercutting is anticipated due to excessively wet unstable soils.

- We anticipate that some of the soils encountered in the borings within the anticipated excavation depths will be suitable for use as Engineered Fill.

- Groundwater depths measured in the borings at the time of drilling ranged from 12 to 17 feet below the ground surface in borings B-02 through B-04. Groundwater seepage at the remaining test locations was not evident to the depths explored at the time of exploration.

- The planned building should be supported by conventional shallow foundations consisting of column and/or strip footings bearing on Engineered Fill. The footings should be sized using a maximum net allowable soil bearing pressure of 3,000 psf.

- Based on the N-values measured in the borings, a Seismic Site Class "E" designation is appropriate for seismic design of the proposed building.

- ECS should be retained to review the design documents for conformance with our recommendations and should be retained for construction materials testing and special inspections to facilitate proper implementation of our recommendations.

1.0 INTRODUCTION

1.1 GENERAL

The purpose of this study was to provide geotechnical information for the design of the proposed building foundations, floor slab, and parking lot and driveway areas.

The recommendations developed for this report are based on the results of our subsurface exploration and project information provided by McDonald's USA, LLC. This report contains the results of our subsurface exploration and laboratory testing programs, site characterization, engineering evaluation, and recommendations for the design and construction of the planned development.

1.2 SCOPE OF SERVICES

The purposes of this exploration were to explore the soil and groundwater conditions at the site and to develop engineering recommendations to guide design and construction of the proposed project.

We accomplished the purposes of the study by:

- Reviewing the available publications concerning local geology of the site and performing a general site reconnaissance.
- Drilling borings to explore the subsurface soil and groundwater conditions.
- Performing laboratory tests on selected representative soil samples from the borings to evaluate pertinent engineering properties.
- Evaluating the field and laboratory data to develop appropriate engineering recommendations.

1.3 AUTHORIZATION

Our services were provided in accordance with Purchase Order No. 2756366 and our Master Services Agreement between McDonald's and ECS.

2.0 PROJECT INFORMATION

2.1 SITE INFORMATION

The site is located at 102 W Cornelius Harnett Blvd in Lillington, North Carolina, at the approximate location shown in the following figure.

Current Site Condition

2.2 PROPOSED CONSTRUCTION

The project involves the demolition of the existing facility and construction of a new McDonald's building and pavement areas.

The proposed building will be a 1-story, wood or light weight steel-framed structure with a slab-on-grade ground floor. Design foundation loads have not been provided to us. We assume the maximum unfactored foundation loads will be:

- Maximum Column Load = 50 kips
- Maximum Wall Loads = 2 kips per foot
- Maximum Ground Floor Slab Load = 150 pounds per square foot (psf)

The structural engineer should verify these assumptions and notify ECS if the actual unfactored foundation design loads exceed or are significantly less than these assumed values.

3.0 FIELD EXPLORATION AND LABORATORY TESTING

To explore the subsurface conditions at this site, six (6) soil test borings were performed in the proposed development areas to depths below existing grades as shown below.

Boring No.	Proposed Structure/Site Feature	Boring Depth (feet)
B-01, B-03, and B-04	Building	20
B-2	Building/Seismic	50
B-05 and B-06	Pavement	10

The borings were located by an ECS representative using a handheld GPS unit and their approximate locations are shown on the Boring Location Diagram (Figure 2) in Appendix A. Our exploration procedures are explained in greater detail in Appendix B including the insert titled Subsurface Exploration Procedure: Standard Penetration Testing (SPT).

Please note that the ground surface elevations shown on the boring logs and cross sections were not surveyed by a licensed surveyor. These elevations were interpolated using topographic information obtained from Google Earth. They should be considered approximate.

3.1 SUBSURFACE CHARACTERIZATION

3.1.1 Regional Geology

The site is located within the Coastal Plain physiographic province. The Coastal Plain is typically characterized by marine, alluvial, and aeolian sediments that were deposited during periods of fluctuating sea levels and moving shorelines. Basal formations are typical of those laid down in a shallow sloping sea bottom, dense sand, consolidated clay, limestone, chalk, marl, claystone, and sandstone. Overburden soils include marine interbedded gravel, sand, silt, and clay. Many of the clays have been preconsolidated by desiccation from frequent rising and lowering of the sea level and groundwater table. Alluvial gravel, sand, silt, and clay are typically present near rivers and creeks.

The top of the coastal formations on the geologic map are typically on the order of 30 to 100 feet below the ground surface but can be less in the upper Coastal Plain near the fall line between the Piedmont and Coastal Plain. They represent basal, relatively hard formations with consistency over large areas.

According to the *1985 Geologic Map of North Carolina*, the site is underlain by the Middendorf of Cretaceous age (Km). This formation consists of sand, sandstone, and mudstone; gray to pale gray with an orange cast; clay balls and iron concretions common; beds laterally discontinuous, cross-bedding common.

It is important to note that the natural geology within portions of the site has been modified in the past that included the placement of fill materials. The quality of man-made fills can vary significantly, and it is often difficult to assess the engineering properties of existing fills.

3.1.2 Soil Conditions

Data from the soil test borings is included on the logs in Appendix B. The subsurface conditions summarized below and those shown on the boring logs represent an estimate of the subsurface conditions based on interpretation of the boring data using normally accepted geotechnical engineering judgments. We note that the transition between different soil strata is usually less distinct than those shown on the boring logs. Please refer to individual boring logs that are contained in Appendix B.

Stratum	Description	Ranges of SPT (1) N-values (bpf (2))
N/A	**Topsoil** – The surface layer at boring location B-04 consisted of approximately 6 inches of topsoil. Thicknesses are expected to be variable across the project site. Observed topsoil depths do not include root balls which could be significantly deeper and are also not recommended for support of structures and/or pavements.	N/A
N/A	**Pavement** – The surface layer at most of the boring locations consisted of 5 to 9 inches of asphalt pavement. Thicknesses are expected to be variable across the project site.	N/A
I	**FILL –** Soils described as Fill and consisting of very loose to loose Clayey/Silty Sand (SC/SM) were encountered at Borings B-1 through B-04 to depths of approximately 6 to 8.5 feet below the ground surface.	0 to 8
II	**Natural Soils** – Beneath the topsoil, asphalt pavement, or fill soils are natural soils described as very loose to medium dense Clayey Sand (SC) and soft to stiff Sandy Lean Clay/Fat Clay (CL/CH).	2 to 13

Notes: (1) Standard Penetration Testing.
(2) bpf – Blows per foot.

3.2 GROUNDWATER OBSERVATIONS

The water levels recorded in our boring logs are contained on the logs in Appendix B. Groundwater depths measured at the time of drilling ranged from 12 to 17 feet below the ground surface in borings B-02 through B-04. Groundwater seepage was not evident at the remaining test locations at the depths explored.

Variations in the long-term water table may occur as a result of changes in precipitation, evaporation, surface water runoff, construction activities, and other factors.

3.3 LABORATORY TESTING

The samples recovered from the borings were visually classified on the basis of texture and plasticity in accordance with ASTM D2488 Standard Practice for Description and Identification of Soils (Visual-Manual Procedures) and including USCS classification symbols, and ASTM D2487 Standard Practice for Classification for Engineering Purposes (Unified Soil Classification System (USCS)). After classification, the samples were grouped in the major zones noted on the boring logs in Appendix B. The group symbols for

each soil type are indicated in parentheses along with the soil descriptions. The stratification lines between strata on the logs are approximate; in situ, the transitions may be gradual.

Classification and index property tests were performed by ECS on representative soil samples from the test borings to aid in classifying soils according to the Unified Soil Classification System and to quantify and correlate engineering properties. The laboratory testing included moisture content, Atterberg limits, and percent passing the No. 200 sieve. The results of the laboratory testing program are included in Appendix C.

4.0 DESIGN RECOMMENDATIONS

4.1 FOUNDATIONS

Provided full depth removal of existing fill material is performed, and subgrades and Engineered Fills are prepared as recommended in this report, the proposed structure can be supported by shallow foundations including column footings and continuous wall footings. We recommend the foundation design use the following parameters:

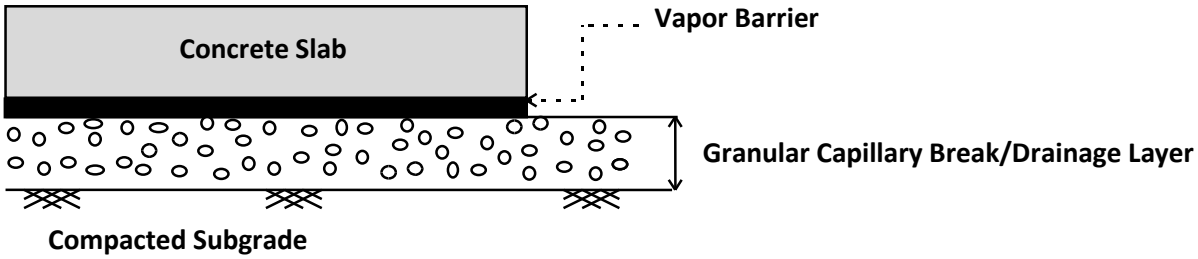
Design Parameter	Column Footing	Wall Footing
Net Allowable Bearing Pressure (1)	3,000 psf (2)	3,000 psf (2)
Acceptable Bearing Soil Material	New Engineered Fill and competent natural soils	
Minimum Width	24 inches	18 inches
Minimum Footing Embedment Depth (Below slab or outside finished grade) (3)	12 inches	12 inches
Estimated Total Settlement (4)	Less than 1 inch	Less than 1 inch
Estimated Differential Settlement (5)	Less than ½-inch	Less than ½-inch

Notes:
(1) Net allowable bearing pressure is the applied pressure in excess of the surrounding overburden soils above the base of the foundation.
(2) Higher bearing capacities would be available where PWR may be present at foundation bearing elevations.
(3) For bearing considerations and frost penetration requirements.
(4) Based on assumed structural loads. If final loads are different, ECS must be contacted to update foundation recommendations and settlement calculations.
(5) Based on maximum column/wall loads and variability in borings. Differential settlement can be re-evaluated once the foundation plans are more complete.

Potential Undercuts: The existing fill materials should be completely removed from beneath the entire footprint of the new building. We anticipate that undercuts up to 8.5 feet will be required to remove the existing fill from the building pad area.

4.2 SLABS ON GRADE

The building floor slab will bear upon new Engineered Fill placed and compacted in accordance with the recommendations provided in this report. The following graphic depicts our soil-supported slab recommendations:



1. Drainage Layer Thickness: 4 inches
2. Drainage Layer Material: A compactable granular fill that will remain stable and support construction traffic. At least 10% to 30% of the material should pass a No. 100 sieve with a maximum aggregate size of ¼ inch. Suitable materials are GRAVEL (ABC, GW, GW-SM), SAND (SP-SM, SW-SM), and SILTY SAND (SM) with less than 30% fines.
3. Subgrade compacted to *98%* maximum dry density per ASTM D698.

Subgrade Modulus: Provided the Engineered Fill and Granular Drainage Layer are constructed in accordance with our recommendations, the slab may be designed assuming a modulus of subgrade reaction, k_1, of 150 pci (lbs./cu. inch).

Vapor Barrier: Before the placement of concrete, a vapor barrier may be placed on top of the granular drainage layer to provide additional protection against moisture vapor penetration through the floor slab. When a vapor barrier is used, special attention should be given to surface curing of the slab to reduce the potential for uneven drying, curling and/or cracking of the slab. Depending on proposed flooring material types, the structural engineer and/or the architect may choose to eliminate the vapor barrier.

Slab Isolation: Soil-supported slabs should be isolated from the foundations and foundation-supported elements of the structure so that differential movement between the foundations and slab will not induce excessive shear and bending stresses in the floor slab. Where the structural configuration restricts the use of a free-floating slab such as in a drop down footing/monolithic slab configuration, the slab should be designed with adequate reinforcement and load transfer devices to reduce overstressing of the slab.

The above should be considered general guidance to assist the Owner/Developer and design team. Project specific designs, plan details or other input from the Structural Engineer of Record should control.

4.3 SEISMIC DESIGN CONSIDERATIONS

Seismic Site Classification: The International Building Code (IBC) requires site classification for seismic design based on the upper 100 feet of a soil profile. Two primary methods are utilized in classifying sites, namely the shear wave velocity (v_s) method and the Standard Penetration Resistance (N-value) method, as indicated in the following table. The N-value method was used for this project.

SEISMIC SITE CLASSIFICATION			
Site Class	**Soil Profile Name**	**Shear Wave Velocity, Vs, (ft./s)**	**N value (bpf)**
A	Hard Rock	Vs > 5,000 fps	N/A
B	Rock	2,500 < Vs ≤ 5,000 fps	N/A
C	Very dense soil and soft rock	1,200 < Vs ≤ 2,500 fps	>50
D	Stiff Soil Profile	600 ≤ Vs ≤ 1,200 fps	15 to 50
E	Soft Soil Profile	Vs < 600 fps	<15

The Site Class for the site was determined by calculating a weighted average SPT N-value for the top 100 feet of the subsurface profile. Based on the conditions encountered in the borings, we recommend that a Site Class "E" be used for design of the proposed building.

4.4 PAVEMENTS

Based on the results of our soil test borings and anticipated site grading, we expect the soils that will be exposed as pavement subgrades, exposed in cuts, and placed as fill will consist mainly of Sandy Lean Clay (CL) and Clayey/Silty Sand (SC, SM) material.

California Bearing Ratio (CBR) testing was not performed as part of this study. For preliminary design purposes, we recommend an assumed design CBR value of 6 be utilized based upon our visual classification of likely pavement subgrade soils. Prior to subbase placement and paving, CBR testing on the actual subgrade soils should be performed to confirm the soil engineering properties.

Since no traffic data was available, we have made assumptions to provide preliminary pavement sections. The pavement sections below are guidelines and may or may not comply with local jurisdictional minimums, but can be adjusted with more complete traffic data and actual CBR lab testing.

PROPOSED MINIMUM PAVEMENT SECTIONS				
MATERIAL	**FLEXIBLE PAVEMENT**		**RIGID PAVEMENT**	
	Light Duty	**Heavy Duty**	**Light Duty**	**Heavy Duty**
Portland Cement Concrete Air-Entrained (f'_c = 4,000 psi)	-	-	5.0 in	6.0 in
Asphalt Surface Course (S9.5B)	2.0 in	1.0 in	-	-
Asphalt Intermediate Course (I19.0C)	-	2.5 in	-	-
Aggregate Base Course (ABC)	6.0 in	8.0 in	4.0 in	6.0 in

In general, heavy-duty sections should be used in areas that will be subjected to trucks, buses, or other similar vehicles including main drive lanes of the development. Light duty sections are appropriate for automobile traffic and parking areas.

Concrete Pavements: Concentrated front-wheel loads are frequently imposed on pavements in trash dumpster and truck loading dock areas. This type of loading typically results in rutting and scuffing of bituminous pavements and ultimately pavement failures and costly repairs. Therefore, we recommend that the pavements in trash pickup and loading dock aprons areas utilize the aforementioned Portland Cement Concrete (PCC) pavement section. It may be prudent to use rigid pavement sections in all areas planned for heavy truck traffic.

The Portland cement concrete pavement section should consist of air-entrained Portland cement concrete having a minimum 28-day compressive strength of 4,000 psi. The rigid pavement section should be provided with construction joints and saw-cut control joints at appropriate intervals per Portland Cement Association (PCA) requirements. The construction joints should be reinforced with dowels to transfer loads across the joints. Wire mesh should be included to control shrinkage cracking of the concrete. The concrete pavement section thickness for plain jointed concrete pavement is with reinforcement dowels only at construction joints.

Construction Traffic: It is important to note that the design sections do not account for construction traffic loading. An incomplete pavement section without the final 1 inch of surface course asphalt can be used for temporary construction traffic, such as concrete trucks and tractor trailer material delivery trucks. Please note, however, that damage to the asphalt already placed is likely to occur in localized areas, and it should be repaired by removal and replacement with new asphalt at or near the end of construction, prior to placement of the surface course.

Alternatively, heavy construction vehicles and traffic should be limited to a temporary pavement section consisting of 12 inches of compacted ABC overlying a high-strength woven geotextile (Tencate Mirafi HP270 or equivalent). The temporary pavement section could then be graded and covered with asphalt to achieve the final design heavy duty pavement section.

Public Streets/Roads: It should also be noted that these design recommendations may not satisfy the local municipality or North Carolina Department of Transportation guidelines. Any roadways constructed for public use and to be dedicated to the local municipality or State for repair and maintenance must be designed in accordance with the local municipality or State requirements.

4.5 SLOPE STABILITY

Our exploration did not include an analysis of slope stability for any temporary or permanent condition. However, within construction areas, we recommend temporary cut slopes without seepage be constructed no steeper than 1.5H:1V.

Permanent fill or cut slopes in the existing site soils without seepage should be designed no steeper than 2H:1V for slopes heights of 20 feet or less. Slopes exceeding 20 feet in height or subject to seepage should be evaluated in more detail. For slopes greater than 25 feet in height, the slopes should include mid-slope benches at about 20 feet (vertical) intervals for stability and to reduce surface erosion due to sheet flow. Benches should be about 8 to 10 feet wide. In building, pavement, and equipment areas, minimum top of slope setbacks of 25 feet and 10 feet are recommended, respectively.

During construction, slopes should be regularly evaluated for signs of movement, seepage, or an unsafe condition. Soil slopes should be covered for protection from rain, and surface runoff condition.

Stormwater runoff should be diverted away from the slopes. For erosion protection, a protective cover of grass or other vegetation should be established on permanent soil slopes as soon as possible. We recommend using erosion blankets on all slopes greater than 20 feet in height for erosion protection.

5.0 SITE CONSTRUCTION RECOMMENDATIONS

5.1 SUBGRADE PREPARATION

5.1.1 Stripping and Grubbing

The subgrade preparation should consist of stripping vegetation, rootmat, topsoil, existing fill, and soft, loose, or unsuitable materials from the 10-foot expanded building and 5-foot expanded pavement limits, and 5 feet beyond the toe of Engineered Fills. ECS should be retained to verify that topsoil and unsuitable surficial materials have been removed prior to the placement of Engineered Fill or construction of structures.

5.1.2 Removal of Structures and Utilities

At the time of our field work, the site was developed with an existing McDonald's facility. The footprint of the new building will overlap a portion of the existing building. The re-development of the project site will require the demolition and removal of the existing building, associated structures, and utilities. This should include the complete removal of all floor slabs, foundations, walls, under slab utilities, pavements, etc.

Existing utilities that will no longer be in service should be completely removed from the new building footprint. Active utilities should be re-routed around the building, wherever possible, and the abandoned section of utility completely removed from the proposed building area. In the event that active utilities are to remain within the building footprint, these utilities should be reviewed by the project structural engineer for conflicts and clearly identified on the construction plans. Special foundation construction procedures may be required to support the new building foundations over utilities.

Excavations resulting from the removal of the above items should be backfilled with new Engineered Fill as discussed in the following section of this report. This should be observed on a full-time basis by a representative of ECS to document that the unsuitable materials have been removed and that the subgrade is suitable for support of the proposed construction and/or fills.

5.1.3 Proofrolling

Prior to fill placement or other construction on subgrades, the subgrades should be evaluated by ECS. The exposed subgrade should be thoroughly proofrolled with construction equipment having a minimum axle load of 10 tons [e.g. fully loaded tandem-axle dump truck]. Proofrolling should be traversed in two perpendicular directions with overlapping passes of the vehicle under the observation of ECS. This procedure is intended to assist in identifying localized yielding materials.

Where proofrolling identifies areas that are yielding or "pumping," those areas should be repaired prior to the placement of subsequent Engineered Fill or other construction materials. Methods of stabilization include undercutting, moisture conditioning, or chemical stabilization. The situation should be discussed with ECS to determine the appropriate procedure. Test pits may be excavated to explore the shallow subsurface materials to help in determining the cause of the observed yielding materials, and to assist in the evaluation of appropriate remedial actions to create a firm and unyielding subgrade.

5.1.4 Site Temporary Dewatering

The contractor should make their own assessment of temporary dewatering needs based upon the limited subsurface groundwater information presented in this report. Soil sampling is not continuous, and thus soil and groundwater conditions may vary between sampling intervals (typically 5 feet). If the contractor believes additional subsurface information is needed to assess dewatering needs, they should obtain such information at their own expense. ECS makes no warranties or guarantees regarding the adequacy of the provided information to determine dewatering requirements; such recommendations are beyond our scope of services.

Dewatering systems are a critical component of many construction projects. Dewatering systems must be selected, designed, and maintained by a qualified and experienced (specialty or other) contractor familiar with the geotechnical and other aspects of the project. The failure to properly design and maintain a dewatering system for a given project can result in delayed construction, unnecessary foundation subgrade undercuts, detrimental phenomena such as 'running sand' conditions, internal erosion (i.e., 'piping'), the migration of 'fines' down-gradient towards the dewatering system, localized settlement of nearby infrastructure, foundations, slabs-on-grade and pavements, etc. Water discharged from the site dewatering system should be discharged in accordance with local, state and federal requirements.

5.2 EARTHWORK OPERATIONS

5.2.1 Excavation Considerations

Excavation Safety: Excavations and slopes should be made and maintained in accordance with OSHA excavation safety standards. The contractor is solely responsible for designing and constructing stable, temporary excavations and slopes and should shore, slope, or bench the sides of the excavations and slopes as required to maintain stability of both the excavation sides and bottom. The contractor's responsible person, as defined in 29 CFR Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations. ECS is providing this information solely as a service to our client. ECS is not assuming responsibility for construction site safety or the contractor's activities; such responsibility is not being implied and should not be inferred.

Excavatability: Based on the assumed design grades, we anticipate that most of the existing fill and natural soils encountered in the test borings can be removed with conventional earth excavation equipment such as track-mounted backhoes, loaders, or bulldozers.

5.2.2 Existing Man-Placed Fill

Existing undocumented fill materials were encountered in borings B-01 through B-04 to approximate depths ranging from 6 to 8.5 feet below existing grade surface. The undocumented fill contained trace amounts of organics, rock fragments, and construction debris identified as asphalt. In our experience, properly placed fill generally exhibits SPT N-values greater than 8 bpf (blows per foot). The N-values in the fill generally ranged from 0 to 8. It is our opinion that most of the onsite fill material was placed in an uncontrolled manner.

If very soft soils or pockets of debris, organics, stumps, etc., exist within the fill and are not removed during construction, then localized excessive differential settlements could occur in response to new structural loads and the on-going process of volume change which may still occur in the fill. If such non-uniform settlements occur, then moderate distress could result. In consideration of the depth of the fill and the amount of observed organic materials in the fill soils, this material is not acceptable for direct support of any future structures, including buildings, walls, and pavements.

Fill Removal in the Building Area – In consideration of the depth, composition, and lack of compaction, we recommend full depth removal of this material from the planned building pad area.

Fill Removal in Pavement Areas – The fill material within the planned pavement areas should be assessed at the time of construction to determine the depth of required undercuts.

5.2.5 Suitability of On-Site Soils for Reuse as Engineered Fill

On-Site Borrow Suitability: The on-site soils meeting the classifications for recommended Engineered Fill, plus meeting the restrictions on separation distances, organic content, and debris, may be used as Engineered Fill. We anticipate that most of the soils encountered in the borings within the anticipated excavation depths to be suitable for use as Engineered Fill.

Engineered Fill Materials: Materials for use as Engineered Fill should consist of inorganic soils with the following engineering properties and compaction requirements.

ENGINEERED FILL SOIL INDEX PROPERTIES	
Subject	**Property**
Soil Classification	CL, ML, SM, SC, SW, SP, GW, GM or GC
Max. Particle Size	3 inches
LL and PI for Fill in Building and Pavement Areas	LL $\leq$ 40, PI $\leq$ 20
Minimum dry unit weight (in place)	$\geq$ 95 pcf
Max. organic content	4% by dry weight

ENGINEERED FILL COMPACTION REQUIREMENTS	
Subject	**Requirement**
Compaction Standard	Standard Proctor, ASTM D698
Required Compaction	95% of Max. Dry Density (98% in the top 1 foot)
Moisture Content	±3 % points of the soil's optimum value
Loose Thickness	8 inches prior to compaction

Existing Fill: The onsite fill material should be evaluated at the time of construction for its suitability for reuse. We anticipate some of the fill material can be moisture conditioned and reused as Engineered Fill material.

Poor Quality Fill Materials: Poor quality fill materials include materials which do not satisfy the requirements for Engineered Fill materials, such as topsoil, organic materials, debris, debris-laden fill and highly plastic soils such as ELASTIC SILT (MH) and FAT CLAY (CH).

Fill Placement Considerations: Fill materials should not be placed on frozen soils, on frost-heaved soils, and/or on excessively wet soils. Borrow fill materials should not contain frozen materials at the time of placement, and frozen or frost-heaved soils should be removed prior to placement of Engineered Fill or other fill soils and aggregates. Excessively wet soils or aggregates should be scarified, aerated, and moisture conditioned. Fill material should be placed in horizontal lifts. Proper drainage should be maintained during the earthwork phases of construction to avoid ponding of water which can lead to degradation of the subgrade soils.

Subgrade Benching: In fill areas, new soil embankments should be constructed from the bottom up. End dumping from the top of the slope should not be permitted. Fill should not be placed on ground with a slope steeper than 5H:1V. Where steeper slopes exist, the ground should be benched to allow for fill placement on a horizontal surface. Each fill layer should be benched into the existing slope for stability.

5.3 FOUNDATION AND SLAB OBSERVATIONS

Protection of Foundation Excavations: Exposure to the environment may weaken the soils at the footing bearing level if the foundation excavations remain open for too long a time. Therefore, foundation concrete should be placed the same day that excavations are made. If the bearing soils are softened by surface water intrusion or exposure, the softened soils must be removed from the foundation excavation bottom immediately prior to placement of concrete. If the excavation must remain open overnight, or if rainfall becomes imminent while the bearing soils are exposed, a 1 to 3-inch thick "mud mat" of "lean" concrete should be placed on the bearing soils before the placement of reinforcing steel.

Footing Subgrade Observations: We anticipate that most of the soils at the foundation bearing elevation are anticipated to be suitable for support of the proposed structure. It is important to have the Geotechnical Engineer of Record (ECS), or their authorized representative, observe the foundation subgrade prior to placing foundation concrete, to confirm the bearing soils are what was anticipated.

Slab Subgrade Verification: Prior to placement of a drainage layer, the subgrade should be prepared in accordance with the recommendations found in **Section 5.1.3 Proofrolling**.

5.4 UTILITY INSTALLATIONS

Utility Subgrades: The soils encountered in our exploration are expected to be generally suitable for support of utility pipes. The pipe subgrades should be observed and probed for stability by ECS. Loose or unsuitable materials encountered should be removed and replaced with suitable compacted Engineered Fill, or pipe stone bedding material.

Utility Backfilling: The granular bedding material should be at least 4 inches thick, but not less than that specified by the civil engineer's project drawings and specifications. We recommend that the bedding materials be placed up to the springline of the pipe. Fill placed for support of the utilities, as well as backfill over the utilities, should satisfy the requirements for Engineered Fill and Fill Placement.

5.5 ADDITIONAL CONSIDERATIONS

During the cooler and wetter periods of the year, delays and additional earthwork costs should be anticipated. At these times, reduction of soil moisture may need to be accomplished by a combination of mechanical manipulation and the use of chemical additives, such as lime or cement, to lower moisture contents to levels appropriate for compaction. Alternatively, during the drier times of the year, such as the summer months, moisture may need to be added to the soil to provide adequate moisture for successful compaction according to the project requirements.

Measures should also be taken to limit site disturbance, especially from rubber-tired heavy construction equipment, and to control and remove surface water from development areas, including structural and pavement areas.

Exposure to the environment may weaken the soils at the footing bearing level if the foundation excavations remain open for too long a time. Therefore, foundation concrete should be placed the same day that excavations are dug. If surface water intrusion or exposure softens the bearing soils, the softened soils must be removed from the foundation excavation bottom immediately prior to placement of concrete. If the excavation must remain open overnight, or if rainfall becomes imminent while the bearing soils are exposed, we recommend that the foundations be covered or otherwise protected.

Positive site drainage should be maintained during earthwork operations, which should help maintain the integrity of the soil. Placement of fill on the near surface soils, which have become saturated, could be very difficult. When wet, these soils will degrade quickly with disturbance from contractor operations and will be extremely difficult to stabilize for fill placement.

Where unacceptable materials are encountered, they must be evaluated and may need to be undercut and replaced or improved by re-compaction.

The surface of the site should be kept properly graded to enhance drainage of the surface water away from the proposed structure areas during the construction phase. We recommend that an attempt be made to enhance the natural drainage without interrupting its pattern.

The surficial soils contain fines, which are considered moderately erodible. Erosion and sedimentation shall be controlled in accordance with Best Management Practices and current County and State NPDES requirements. At the appropriate time, we would be pleased to provide a proposal for conducting construction materials testing and NPDES services.

6.0 CLOSING

ECS has prepared this report to guide the geotechnical-related design and construction aspects of the project. We performed these services in accordance with the standard of care expected of professionals in the industry performing similar services on projects of like size and complexity at this time in the region. No other representation, express or implied, and no warranty or guarantee is included or intended in this report. ECS is not responsible for the conclusions, opinions, or recommendations of others based on the data in this report.

The description of the proposed project is based on information provided to ECS by McDonald's USA, LLC. If any of this information is inaccurate or changes, either because of our interpretation of the documents provided or site or design changes that may occur later, ECS should be contacted so we can review our recommendations and provide additional or alternate recommendations that reflect the proposed construction.

We recommend that ECS review the final project plans and specifications so we can confirm that those plans/specifications are in accordance with the recommendations of this geotechnical report.

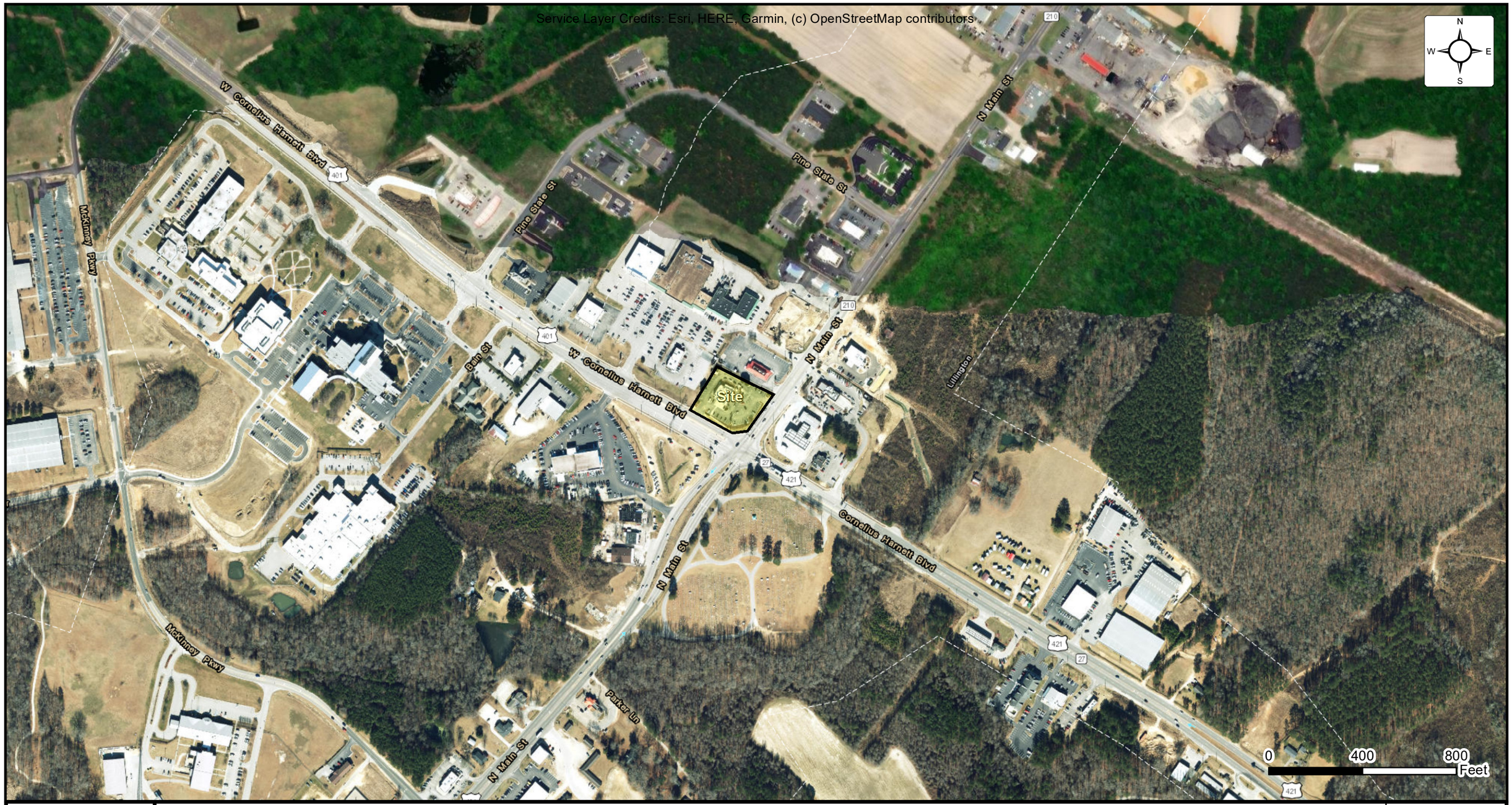
Because undocumented fill is present on this site, the Owner must assess the relative risk that unacceptable material could have been buried in the proposed development area which was not detected in the widely spaced borings.

Field observations, monitoring, and quality assurance testing during earthwork and foundation installation are an extension of and integral to the geotechnical design recommendation. We recommend that the owner retain these quality assurance services and that ECS be allowed to continue our involvement throughout these critical phases of construction to provide general consultation as issues arise. We would be pleased to provide an estimated cost for these services at the appropriate time.

This report is provided for the exclusive use of McDonald's USA, LLC and their project specific design team. This report is not intended to be used or relied upon in connection with other projects or by other third parties. ECS disclaims liability for any such third-party use or reliance without express written permission.

Appendix A - Drawings and Reports

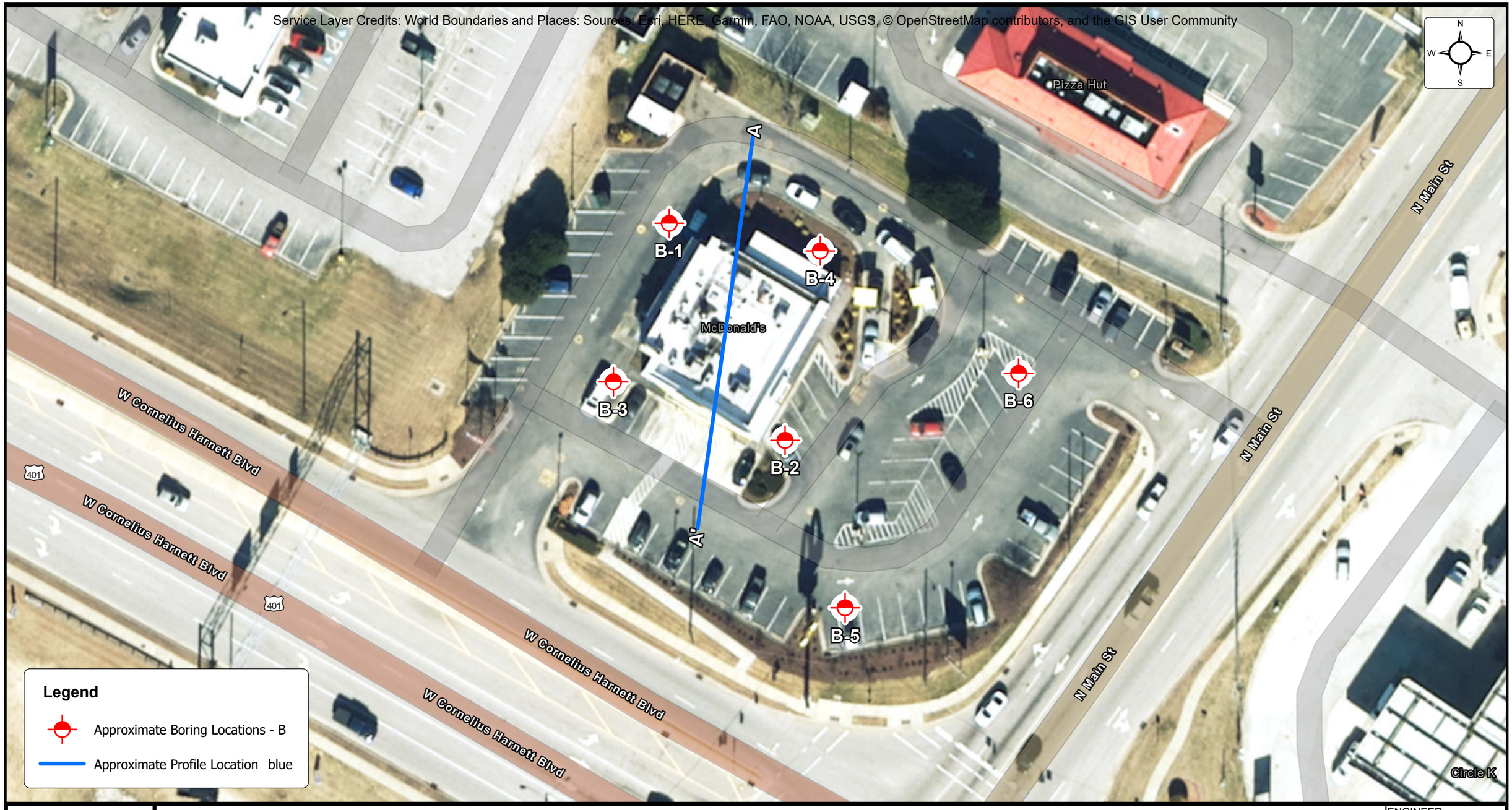
Site Location Diagram
Boring Location Diagram
Subsurface Cross-Section



SITE LOCATION DIAGRAM
32-0312 MCDONALD'S LILLINGTON, NC, 01.00.16

102 W CORNELIUS HARNETT BLVD, LILLINGTON, NORTH CAROLINA
MCDONALD'S USA, LLC

ENGINEER	JK01
SCALE	AS NOTED
PROJECT NO.	33:6900
FIGURE	1 OF 2
DATE	6/26/2024

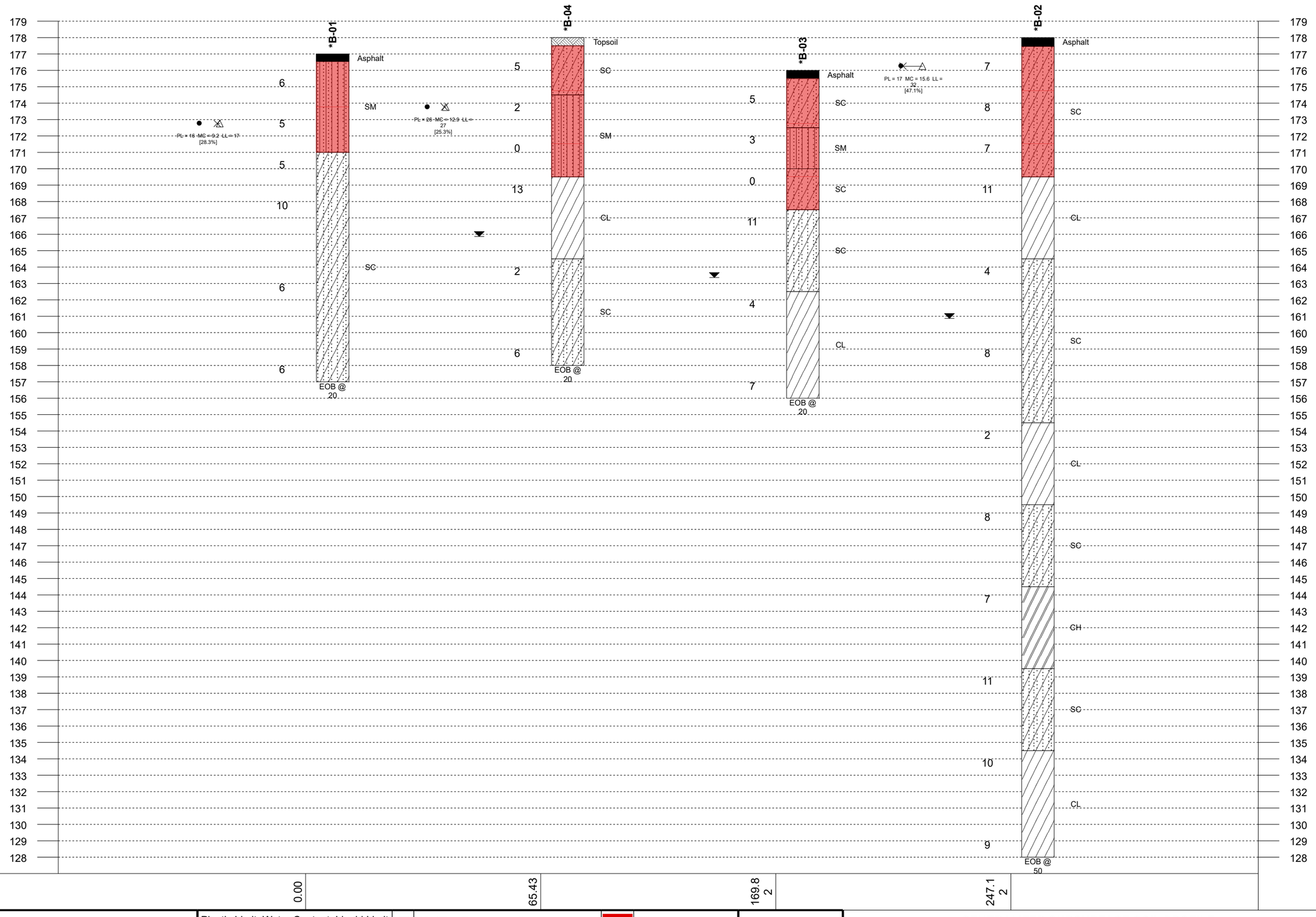


BORING LOCATION DIAGRAM

32-0312 McDonald's Lillington, NC, 01.00.14

102 W Cornelius Harnett Blvd, Lillington, North Carolina
McDonald's USA, LLC

ENGINEER	JK
SCALE	1" = 40'
PROJECT NO.	33:6900
SHEET	2 of 2
DATE	7/15/2024



Legend Key

- Asphalt
- Topsoil
- CLAYEY SAND
- SILTY SAND
- Lean CLAY
- Fat CLAY

Notes:

1- EOB: END OF BORING AR: AUGER REFUSAL SR: SAMPLER REFUSAL.
2- THE NUMBER BELOW THE STRIPS IS THE DISTANCE ALONG THE BASELINE.
3- SEE INDIVIDUAL BORING LOG AND GEOTECHNICAL INFORMATION.
4- STANDARD PENETRATION TEST RESISTANCE (LEFT OF BORING) IN BLOWS PER FOOT (ASTM D1586).

Plastic Limit Water Content Liquid Limit X—●—△	WL (First Encountered)	Fill	
[FINES CONTENT %]	WL (Completion)	Possible Fill	
BOTTOM OF CASING	WL (Estimated Seasonal High Water)	Probable Fill	
LOSS OF CIRCULATION	WL (Stabilized)	Rock	
CALIBRATED PENETROMETER			

GENERALIZED SUBSURFACE SOIL PROFILE

A-A'

32-0312 McDonald's Lillington, NC, 01.00.14

McDonald's USA, LLC

102 W Cornelius Harnett Blvd, Lillington, North Carolina, 27546

Project No: 33:6900 Date: 07/15/2024

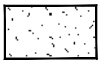
Appendix B – Field Operations

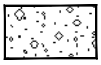
Reference Notes
Exploration Procedures
Boring Logs

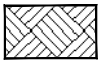
REFERENCE NOTES FOR BORING LOGS

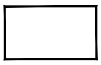
MATERIAL[1,2]

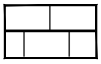

ASPHALT


CONCRETE


GRAVEL

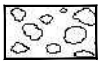

TOPSOIL

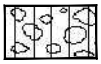

VOID


BRICK

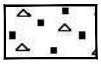

AGGREGATE BASE COURSE

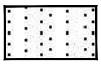

GW **WELL-GRADED GRAVEL**
gravel-sand mixtures, little or no fines

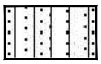

GP **POORLY-GRADED GRAVEL**
gravel-sand mixtures, little or no fines


GM **SILTY GRAVEL**
gravel-sand-silt mixtures

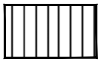

GC **CLAYEY GRAVEL**
gravel-sand-clay mixtures

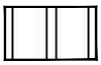

SW **WELL-GRADED SAND**
gravelly sand, little or no fines

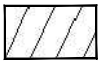

SP **POORLY-GRADED SAND**
gravelly sand, little or no fines


SM **SILTY SAND**
sand-silt mixtures

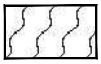

SC **CLAYEY SAND**
sand-clay mixtures

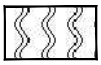

ML **SILT**
non-plastic to medium plasticity

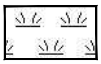

MH **ELASTIC SILT**
high plasticity


CL **LEAN CLAY**
low to medium plasticity


CH **FAT CLAY**
high plasticity


OL **ORGANIC SILT or CLAY**
non-plastic to low plasticity


OH **ORGANIC SILT or CLAY**
high plasticity


PT **PEAT**
highly organic soils

DRILLING SAMPLING SYMBOLS & ABBREVIATIONS

SS	Split Spoon Sampler	PM	Pressuremeter Test
ST	Shelby Tube Sampler	RD	Rock Bit Drilling
WS	Wash Sample	RC	Rock Core, NX, BX, AX
BS	Bulk Sample of Cuttings	REC	Rock Sample Recovery %
PA	Power Auger (no sample)	RQD	Rock Quality Designation %
HSA	Hollow Stem Auger		

PARTICLE SIZE IDENTIFICATION

DESIGNATION		PARTICLE SIZES
Boulders		12 inches (300 mm) or larger
Cobbles		3 inches to 12 inches (75 mm to 300 mm)
Gravel:	Coarse	¾ inch to 3 inches (19 mm to 75 mm)
	Fine	4.75 mm to 19 mm (No. 4 sieve to ¾ inch)
Sand:	Coarse	2.00 mm to 4.75 mm (No. 10 to No. 4 sieve)
	Medium	0.425 mm to 2.00 mm (No. 40 to No. 10 sieve)
	Fine	0.074 mm to 0.425 mm (No. 200 to No. 40 sieve)
Silt & Clay ("Fines")		<0.074 mm (smaller than a No. 200 sieve)

COHESIVE SILTS & CLAYS

UNCONFINED COMPRESSIVE STRENGTH, Q_P[4]	SPT[5] (BPF)	CONSISTENCY[7] (COHESIVE)
<0.25	<2	Very Soft
0.25 - <0.50	2 - 4	Soft
0.50 - <1.00	5 - 8	Firm
1.00 - <2.00	9 - 15	Stiff
2.00 - <4.00	16 - 30	Very Stiff
4.00 - 8.00	31 - 50	Hard
>8.00	>50	Very Hard

GRAVELS, SANDS & NON-COHESIVE SILTS

SPT[5]	DENSITY
<5	Very Loose
5 - 10	Loose
11 - 30	Medium Dense
31 - 50	Dense
>50	Very Dense

RELATIVE AMOUNT[7]	COARSE GRAINED (%)[8]	FINE GRAINED (%)[8]
Trace	≤5	≤5
With	10 - 20	10 - 25
Adjective *(ex: "Silty")*	25 - 45	30 - 45

WATER LEVELS[6]

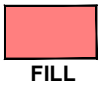

WL (First Encountered)


WL (Completion)


WL (Seasonal High Water)


WL (Stabilized)

FILL AND ROCK


FILL


POSSIBLE FILL


PROBABLE FILL


ROCK

[1]Classifications and symbols per ASTM D 2488-17 (Visual-Manual Procedure) unless noted otherwise.

[2]To be consistent with general practice, "POORLY GRADED" has been removed from GP, GP-GM, GP-GC, SP, SP-SM, SP-SC soil types on the boring logs.

[3]Non-ASTM designations are included in soil descriptions and symbols along with ASTM symbol [Ex: (SM-FILL)].

[4]Typically estimated via pocket penetrometer or Torvane shear test and expressed in tons per square foot (tsf).

[5]Standard Penetration Test (SPT) refers to the number of hammer blows (blow count) of a 140 lb. hammer falling 30 inches on a 2 inch OD split spoon sampler required to drive the sampler 12 inches (ASTM D 1586). "N-value" is another term for "blow count" and is expressed in blows per foot (bpf). SPT correlations per 7.4.2 Method B and need to be corrected if using an auto hammer.

[6]The water levels are those levels actually measured in the borehole at the times indicated by the symbol. The measurements are relatively reliable when augering, without adding fluids, in granular soils. In clay and cohesive silts, the determination of water levels may require several days for the water level to stabilize. In such cases, additional methods of measurement are generally employed.

[7]Minor deviation from ASTM D 2488-17 Note 14.

[8]Percentages are estimated to the nearest 5% per ASTM D 2488-17.

SUBSURFACE EXPLORATION PROCEDURE: STANDARD PENETRATION TESTING (SPT) ASTM D 1586 Split-Barrel Sampling

Standard Penetration Testing, or **SPT**, is the most frequently used subsurface exploration test performed worldwide. This test provides samples for identification purposes, as well as a measure of penetration resistance, or N-value. The N-Value, or blow counts, when corrected and correlated, can approximate engineering properties of soils used for geotechnical design and engineering purposes.

SPT Procedure:

- Involves driving a hollow tube (split-spoon) into the ground by dropping a 140-lb hammer a height of 30-inches at desired depth
- Recording the number of hammer blows required to drive split-spoon a distance of 18-24 inches (in 3 or 4 Increments of 6 inches each)

- Auger is advanced* and an additional SPT is performed
- One SPT typically performed for every two to five feet. An approximate 1.5 inch diameter soil sample is recovered.

**Drilling Methods May Vary*— The predominant drilling methods used for SPT are open hole fluid rotary drilling and hollow-stem auger drilling.

CLIENT:	PROJECT NO.:	BORING NO.:	SHEET:
McDonald's USA, LLC	**33:6900**	**B-01**	**1 of 1**

PROJECT NAME:	DRILLER/CONTRACTOR:
32-0312 McDonald's Lillington, NC, 01.00.14	**Quantex, Inc.**

SITE LOCATION: **102 W Cornelius Harnett Blvd, Lillington, North Carolina, 27546**

LOSS OF CIRCULATION

LATITUDE:	LONGITUDE:	STATION:	SURFACE ELEVATION:	BOTTOM OF CASING
35.418828	**-78.803850**		**177.0**	

DEPTH (FT)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE DIST. (IN)	RECOVERY (IN)	DESCRIPTION OF MATERIAL	WATER LEVELS	ELEVATION (FT)	BLOWS/6" (TCP/MC/SPT-N value)*
					Asphalt Thickness[5.50"]			
	S-1	SS	18	18	(SM FILL) FILL, SILTY SAND, trace gravel, orange and tan, moist, loose			3-3-3 (6)
	S-2	SS	18	18				4-3-2 (5)
5							172	
	S-3	SS	18	18	(SC) CLAYEY SAND, grayish tan to variegated orange- tan- gray, moist, loose			3-2-3 (5)
	S-4	SS	18	18				6-5-5 (10)
10							167	
	S-5	SS	18	18				3-3-3 (6)
15							162	
	S-6	SS	18	18				3-3-3 (6)
20					**END OF BORING AT 20 FT**		157	
25							152	
30							147	

STANDARD PENETRATION BLOWS/FT (10 20 30 40 50)
ROCK QUALITY DESIGNATION & RECOVERY (20 40 60 80 100)
RQD
REC
MC SAMPLER BLOWS/FT (10 20 30 40 50)
TEXAS CONE PENETRATION BLOWS/FT

LIQUID LIMIT
PLASTIC LIMIT
CALIBRATED PENETROMETER TSF (1 2 3 4 5)
WATER CONTENT %
[FINES CONTENT] % (10 20 30 40 50)

9.2 16 17 [28.3%]

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL

▽ WL (First Encountered)	BORING STARTED: **Jun 17 2024**	CAVE IN DEPTH:	
▼ WL (Completion) **DRY**	BORING COMPLETED: **Jun 17 2024**	HAMMER TYPE: **Auto**	
▼ WL (Seasonal High Water)	EQUIPMENT: **Geoprobe 7822DT**	LOGGED BY:	DRILLING METHOD: **Hollow Stem Auger**
▽ WL (Stabilized)			

GEOTECHNICAL BOREHOLE LOG

CLIENT:	PROJECT NO.:	BORING NO.:	SHEET:
McDonald's USA, LLC	**33:6900**	**B-02**	**1 of 2**

PROJECT NAME:	DRILLER/CONTRACTOR:
32-0312 McDonald's Lillington, NC, 01.00.14	**Quantex, Inc.**

SITE LOCATION: **102 W Cornelius Harnett Blvd, Lillington, North Carolina, 27546**

LOSS OF CIRCULATION

LATITUDE:	LONGITUDE:	STATION:	SURFACE ELEVATION:	BOTTOM OF CASING
35.418573	**-78.803685**		**178.0**	

DEPTH (FT)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE DIST. (IN)	RECOVERY (IN)	DESCRIPTION OF MATERIAL	WATER LEVELS	ELEVATION (FT)	BLOWS/6" (TCP/MC/SPT-N value)*
					Asphalt Thickness[6.50"]			
	S-1	SS	18	18	(SC FILL) FILL, CLAYEY SAND, trace gravel, contains significant asphalt, orange to tan, moist, loose			4-3-4 (7)
	S-2	SS	18	18				3-4-4 (8)
5							173	
	S-3	SS	18	18				3-3-4 (7)
	S-4	SS	18	18	(CL) SANDY LEAN CLAY, variegated orange- tan- gray, moist, stiff			4-4-7 (11)
10							168	
	S-5	SS	18	18	(SC) CLAYEY SAND, orange to variegated orange- tan-, moist, very loose to loose	▼		3-2-2 (4)
15							163	
	S-6	SS	18	18				3-4-4 (8)
20							158	
	S-7	SS	18	18	(CL) SANDY LEAN CLAY, red and tan, wet, soft			1-1-1 (2)
25							153	
	S-8	SS	18	18	(SC) CLAYEY SAND, grayish tan, wet, loose			4-4-4 (8)
30							148	

CONT'D ON NEXT PAGE

STANDARD PENETRATION BLOWS/FT (10 20 30 40 50 / 20 40 60 80 100)
ROCK QUALITY DESIGNATION & RECOVERY
RQD
REC
MC SAMPLER BLOWS/FT (10 20 30 40 50)
TEXAS CONE PENETRATION BLOWS/FT

LIQUID LIMIT
PLASTIC LIMIT
CALIBRATED PENETROMETER TSF (1 2 3 4 5)
WATER CONTENT %
[FINES CONTENT] %
(10 20 30 40 50)

17 32 15.6 [47.1%]

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL

▽ WL (First Encountered)	BORING STARTED: **Jun 17 2024**	CAVE IN DEPTH:	
▼ WL (Completion) **17.00**	BORING COMPLETED: **Jun 17 2024**	HAMMER TYPE: **Auto**	
▼ WL (Seasonal High Water)	EQUIPMENT: **Geoprobe 7822DT**	LOGGED BY:	DRILLING METHOD: **Hollow Stem Auger**
▽ WL (Stabilized)			

GEOTECHNICAL BOREHOLE LOG

CLIENT:	PROJECT NO.:	BORING NO.:	SHEET:
McDonald's USA, LLC	**33:6900**	**B-02**	**2 of 2**
PROJECT NAME:	DRILLER/CONTRACTOR:		
32-0312 McDonald's Lillington, NC, 01.00.14	**Quantex, Inc.**		
SITE LOCATION: **102 W Cornelius Harnett Blvd, Lillington, North Carolina, 27546**			LOSS OF CIRCULATION
LATITUDE: **35.418573**	LONGITUDE: **-78.803685**	STATION:	SURFACE ELEVATION: **178.0**	BOTTOM OF CASING

DEPTH (FT)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE DIST. (IN)	RECOVERY (IN)	DESCRIPTION OF MATERIAL	WATER LEVELS	ELEVATION (FT)	BLOWS/6" (TCP/MC/SPT-N value)*
					(SC) CLAYEY SAND, grayish tan, wet, loose			
35	S-9	SS	18	18	(CH) SANDY FAT CLAY, orange and gray, wet, firm		143	3-3-4 (7)
40	S-10	SS	18	18	(SC) CLAYEY SAND, red and tan, wet, medium dense		138	4-5-6 (11)
45	S-11	SS	18	18	(CL) SANDY LEAN CLAY, orange, saturated, stiff		133	5-5-5 (10)
50	S-12	SS	18	18	**END OF BORING AT 50 FT**		128	4-4-5 (9)
55							123	
60							118	

STANDARD PENETRATION BLOWS/FT (10 20 30 40 50 / 20 40 60 80 100)
ROCK QUALITY DESIGNATION & RECOVERY
RQD
REC
MC SAMPLER BLOWS/FT (10 20 30 40 50)
TEXAS CONE PENETRATION BLOWS/FT
LIQUID LIMIT
PLASTIC LIMIT
CALIBRATED PENETROMETER TSF (1 2 3 4 5)
WATER CONTENT %
[FINES CONTENT] % (10 20 30 40 50)

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL

WL (First Encountered)	BORING STARTED: **Jun 17 2024**	CAVE IN DEPTH:	
WL (Completion) **17.00**	BORING COMPLETED: **Jun 17 2024**	HAMMER TYPE: **Auto**	
WL (Seasonal High Water)	EQUIPMENT: **Geoprobe 7822DT**	LOGGED BY:	DRILLING METHOD: **Hollow Stem Auger**
WL (Stabilized)			

GEOTECHNICAL BOREHOLE LOG

CLIENT:	PROJECT NO.:	BORING NO.:	SHEET:
McDonald's USA, LLC	**33:6900**	**B-03**	**1 of 1**

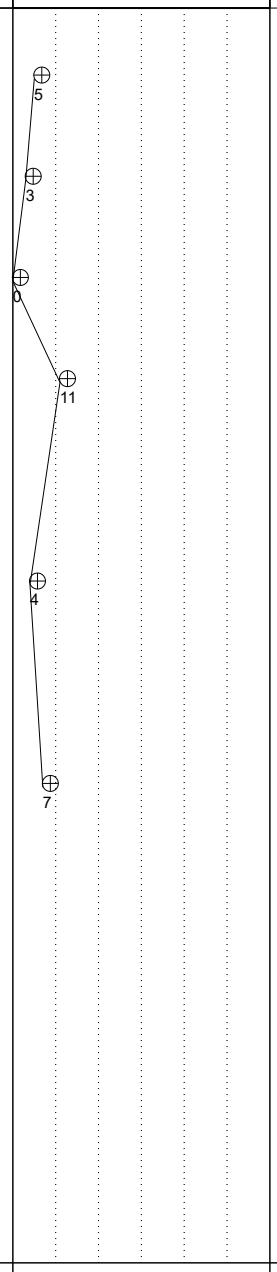
PROJECT NAME:	DRILLER/CONTRACTOR:
32-0312 McDonald's Lillington, NC, 01.00.14	**Quantex, Inc.**

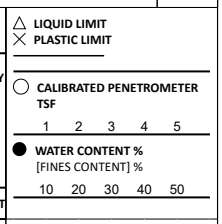
SITE LOCATION: **102 W Cornelius Harnett Blvd, Lillington, North Carolina, 27546**

LOSS OF CIRCULATION

LATITUDE:	LONGITUDE:	STATION:	SURFACE ELEVATION:	BOTTOM OF CASING
35.418642	**-78.803930**		**176.0**	

DEPTH (FT)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE DIST. (IN)	RECOVERY (IN)	DESCRIPTION OF MATERIAL	WATER LEVELS	ELEVATION (FT)	BLOWS/6" (TCP/MC/SPT-N value)*
					Asphalt Thickness[6.00"]			
	S-1	SS	18	18	(SC FILL) FILL, CLAYEY SAND, trace gravel, orange, moist, loose			2-2-3 (5)
5	S-2	SS	18	18	(SM FILL) FILL, SILTY SAND, trace clay, gray, moist, very loose		171	3-2-1 (3)
	S-3	SS	18	18	(SC FILL) FILL, CLAYEY SAND, contains asphalt, brown, moist, very loose			1-WOH-WOH (0)
10	S-4	SS	18	18	(SC) CLAYEY SAND, variegated red- tan-gray, moist, medium dense	▼	166	4-5-6 (11)
15	S-5	SS	18	18	(CL) SANDY LEAN CLAY, gray, moist, soft to firm		161	2-1-3 (4)
20	S-6	SS	18	18	**END OF BORING AT 20 FT**		156	3-3-4 (7)
25							151	
30							146	





THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL

▽ WL (First Encountered)		BORING STARTED: **Jun 17 2024**	CAVE IN DEPTH:
▼ WL (Completion)	**12.50**	BORING COMPLETED: **Jun 17 2024**	HAMMER TYPE: **Auto**
▼ WL (Seasonal High Water)		EQUIPMENT: **Geoprobe 7822DT** / LOGGED BY:	DRILLING METHOD: **Hollow Stem Auger**
▽ WL (Stabilized)			

GEOTECHNICAL BOREHOLE LOG

CLIENT:	PROJECT NO.:	BORING NO.:	SHEET:
McDonald's USA, LLC	**33:6900**	**B-04**	**1 of 1**

PROJECT NAME:	DRILLER/CONTRACTOR:
32-0312 McDonald's Lillington, NC, 01.00.14	**Quantex, Inc.**

SITE LOCATION: **102 W Cornelius Harnett Blvd, Lillington, North Carolina, 27546**

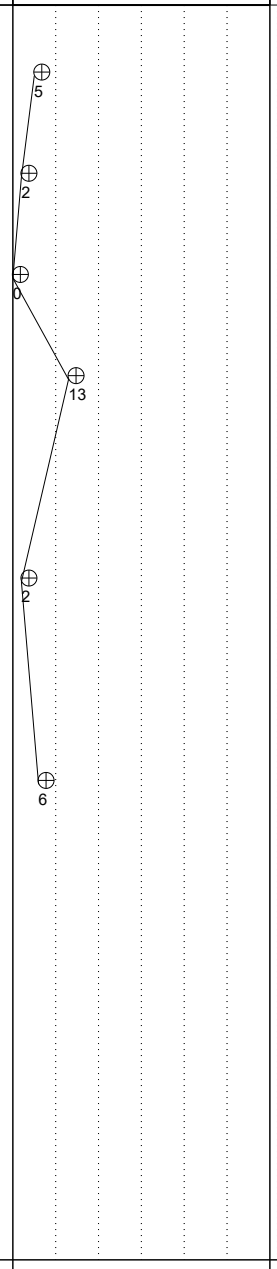
LOSS OF CIRCULATION

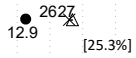
LATITUDE:	LONGITUDE:	STATION:	SURFACE ELEVATION:	BOTTOM OF CASING
35.418795	**-78.803634**		**178.0**	

DEPTH (FT)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE DIST. (IN)	RECOVERY (IN)	DESCRIPTION OF MATERIAL	WATER LEVELS	ELEVATION (FT)	BLOWS/6" (TCP/MC/SPT-N value)*
					Topsoil Thickness[6.00"]			
	S-1	SS	18	18	(SC FILL) FILL, CLAYEY SAND trace gravel, contains rock fragments, orange and brown, moist, loose			3-3-2 (5)
	S-2	SS	18	18	(SM FILL) FILL, SILTY SAND, trace clay, trace organics, trace gravel, contains slight roots, brown and gray, moist, very loose			2-1-1 (2)
5	S-3	SS	18	18			173	1-WOH-WOH (0)
	S-4	SS	18	18	(CL) SANDY LEAN CLAY, variegated red-tan- gray, moist, stiff	▼		4-5-8 (13)
10							168	
	S-5	SS	18	18	(SC) CLAYEY SAND, orange and gray, wet to moist, very loose to loose			1-1-1 (2)
15							163	
	S-6	SS	18	18				3-3-3 (6)
20					**END OF BORING AT 20 FT**		158	
25							153	
30							148	

STANDARD PENETRATION BLOWS/FT
10 20 30 40 50
20 40 60 80 100
ROCK QUALITY DESIGNATION & RECOVERY
RQD
REC
MC SAMPLER BLOWS/FT
10 20 30 40 50
TEXAS CONE PENETRATION BLOWS/FT

LIQUID LIMIT
PLASTIC LIMIT
CALIBRATED PENETROMETER TSF
1 2 3 4 5
WATER CONTENT %
[FINES CONTENT] %
10 20 30 40 50





THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL

▽ WL (First Encountered)	BORING STARTED: **Jun 17 2024**		CAVE IN DEPTH:
▼ WL (Completion) **12.00**	BORING COMPLETED: **Jun 17 2024**		HAMMER TYPE: **Auto**
▼ WL (Seasonal High Water)	EQUIPMENT: **Geoprobe 7822DT**	LOGGED BY:	DRILLING METHOD: **Hollow Stem Auger**
▽ WL (Stabilized)			

GEOTECHNICAL BOREHOLE LOG

CLIENT:	PROJECT NO.:	BORING NO.:	SHEET:
McDonald's USA, LLC	**33:6900**	**B-05**	**1 of 1**

PROJECT NAME:	DRILLER/CONTRACTOR:
32-0312 McDonald's Lillington, NC, 01.00.14	**Quantex, Inc.**

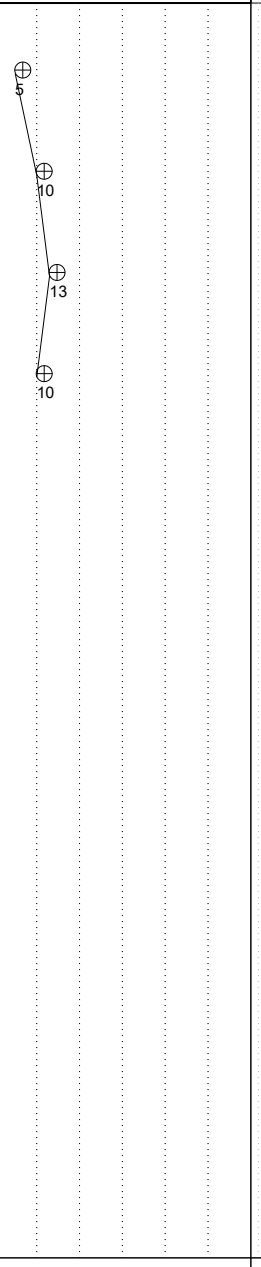
SITE LOCATION: **102 W Cornelius Harnett Blvd, Lillington, North Carolina, 27546**

LOSS OF CIRCULATION

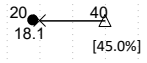
LATITUDE:	LONGITUDE:	STATION:	SURFACE ELEVATION:	BOTTOM OF CASING
35.418377	**-78.803599**		**177.0**	

DEPTH (FT)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE DIST. (IN)	RECOVERY (IN)	DESCRIPTION OF MATERIAL	WATER LEVELS	ELEVATION (FT)	BLOWS/6" (TCP/MC/SPT-N value)*
					Asphalt Thickness[9.00"]			
	S-1	SS	18	18	(CL) SANDY LEAN CLAY, brown to variegated red- tan- gray, moist, firm to stiff			3-2-3 (5)
	S-2	SS	18	18				6-5-5 (10)
5							172	
	S-3	SS	18	18				3-5-8 (13)
	S-4	SS	18	18	(SC) CLAYEY SAND, orange, moist, loose			5-5-5 (10)
10					**END OF BORING AT 10 FT**		167	
15							162	
20							157	
25							152	
30							147	

STANDARD PENETRATION BLOWS/FT (10 20 30 40 50 / 20 40 60 80 100)
ROCK QUALITY DESIGNATION & RECOVERY
RQD
REC
MC SAMPLER BLOWS/FT (10 20 30 40 50)
TEXAS CONE PENETRATION BLOWS/FT



LIQUID LIMIT
PLASTIC LIMIT
CALIBRATED PENETROMETER TSF (1 2 3 4 5)
WATER CONTENT %
[FINES CONTENT] % (10 20 30 40 50)



THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL

▽ WL (First Encountered)	BORING STARTED: **Jun 17 2024**	CAVE IN DEPTH:	
▼ WL (Completion) **DRY**	BORING COMPLETED: **Jun 17 2024**	HAMMER TYPE: **Auto**	
▼ WL (Seasonal High Water)	EQUIPMENT: **Geoprobe 7822DT**	LOGGED BY:	DRILLING METHOD: **Hollow Stem Auger**
▽ WL (Stabilized)			

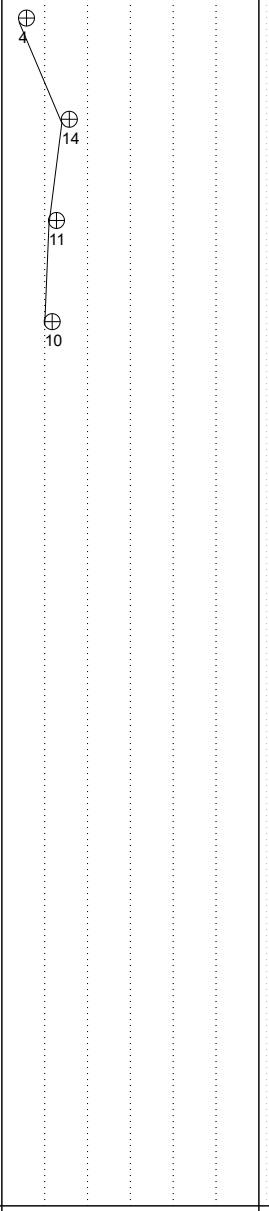
GEOTECHNICAL BOREHOLE LOG

CLIENT:	PROJECT NO.:	BORING NO.:	SHEET:
McDonald's USA, LLC	**33:6900**	**B-06**	**1 of 1**
PROJECT NAME:	DRILLER/CONTRACTOR:		
32-0312 McDonald's Lillington, NC, 01.00.14	**Quantex, Inc.**		
SITE LOCATION:			LOSS OF CIRCULATION
102 W Cornelius Harnett Blvd, Lillington, North Carolina, 27546			
LATITUDE:	LONGITUDE:	STATION:	SURFACE ELEVATION:
35.418651	**-78.803350**		**178.0**

BOTTOM OF CASING

DEPTH (FT)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE DIST. (IN)	RECOVERY (IN)	DESCRIPTION OF MATERIAL	WATER LEVELS	ELEVATION (FT)	BLOWS/6" (TCP/MC/SPT-N value)*
					Asphalt Thickness[5.00"]			
	S-1	SS	18	18	(CL) SANDY LEAN CLAY, trace silt, brown to variegated red- tan- gray, moist, soft to stiff			3-2-2 (4)
	S-2	SS	18	18				5-6-8 (14)
5							173	
	S-3	SS	18	18				4-5-6 (11)
	S-4	SS	18	18	(SC) CLAYEY SAND, orange, moist, loose			4-5-5 (10)
10					**END OF BORING AT 10 FT**		168	
15							163	
20							158	
25							153	
30							148	

STANDARD PENETRATION BLOWS/FT
10 20 30 40 50
20 40 60 80 100
ROCK QUALITY DESIGNATION & RECOVERY
RQD
REC
MC SAMPLER BLOWS/FT
10 20 30 40 50
TEXAS CONE PENETRATION BLOWS/FT



LIQUID LIMIT
PLASTIC LIMIT
CALIBRATED PENETROMETER TSF
1 2 3 4 5
WATER CONTENT %
[FINES CONTENT] %
10 20 30 40 50

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL

▽ WL (First Encountered)	BORING STARTED: **Jun 17 2024**		CAVE IN DEPTH:
▼ WL (Completion) **DRY**	BORING COMPLETED: **Jun 17 2024**		HAMMER TYPE: **Auto**
▼ WL (Seasonal High Water)	EQUIPMENT: **Geoprobe 7822DT**	LOGGED BY:	DRILLING METHOD: **Hollow Stem Auger**
▽ WL (Stabilized)			

GEOTECHNICAL BOREHOLE LOG

Appendix C – Laboratory Testing

Laboratory Testing Summary
Plasticity Chart

Laboratory Testing Summary

Sample Location	Sample Number	Depth (ft)	^MC (%)	Soil Type	Atterberg Limits			**Percent Passing No. 200 Sieve	Moisture - Density		CBR (%)		#Organic Content (%)
					LL	PL	PI		<Maximum Density (pcf)	<Optimum Moisture (%)	0.1 in.	0.2 in.	
B-01	S-2	3.5-5.0	9.2	SM	17	NP	NP	28.3					
B-02	S-1	1.0-2.5	15.6	SC	32	17	15	47.1					
B-04	S-2	3.5-5.0	12.9	SM	27	NP	NP	25.3					
B-05	S-1	1.0-2.5	18.1	SC	40	20	20	45.0					

Notes: See test reports for test method, ^ASTM D2216-19, *ASTM D2488, **ASTM D1140-17, #ASTM D2974-20e1 < See test report for D4718 corrected values

Definitions: MC: Moisture Content, Soil Type: USCS (Unified Soil Classification System), LL: Liquid Limit, PL: Plastic Limit, PI: Plasticity Index, CBR: California Bearing Ratio, OC: Organic Content

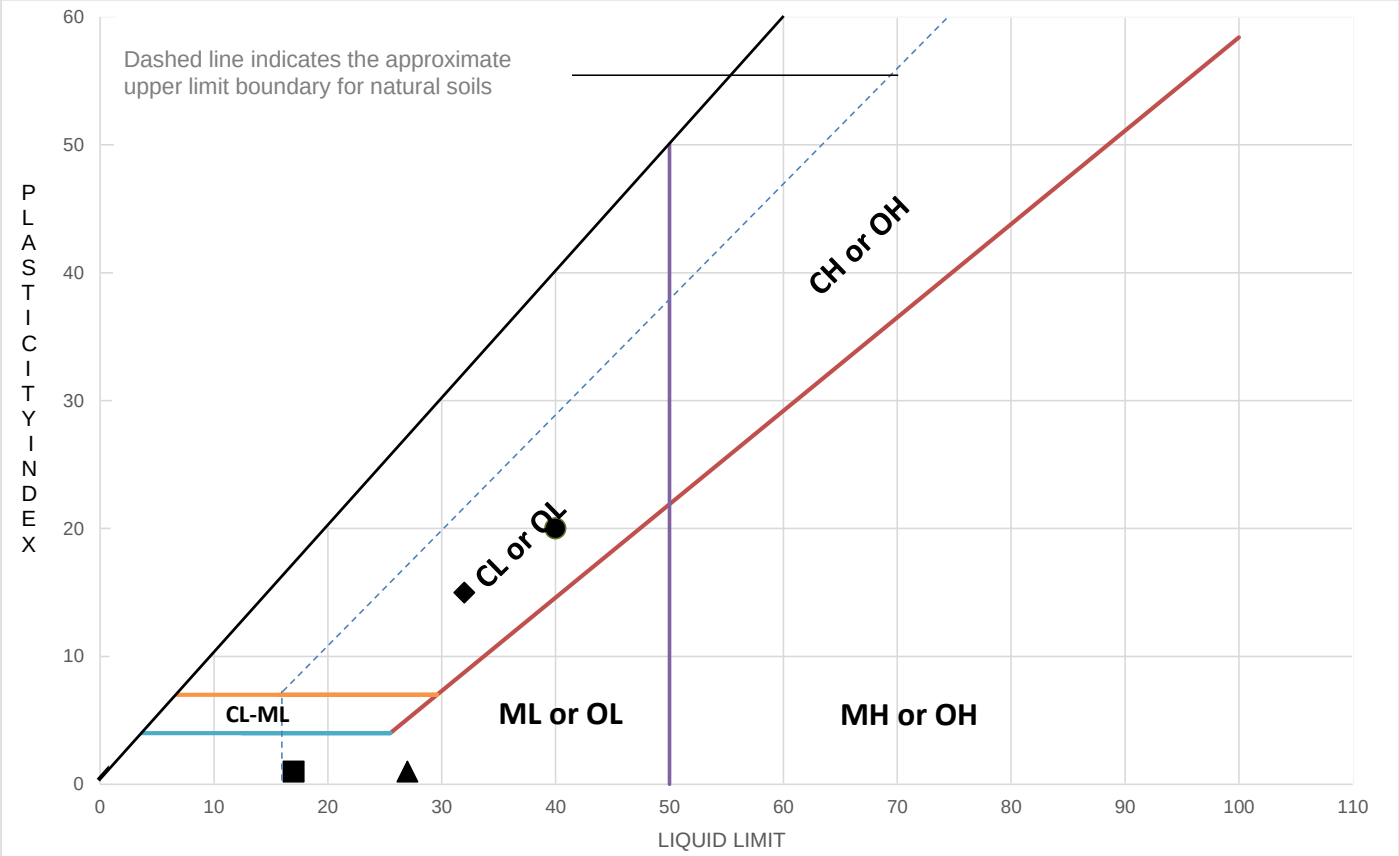
Project: 32-0312 McDonald's Lillington, NC, 01.00.16
Client:

Project No.: 33:6900
Date Reported: 7/10/2024

Office / Lab	Address	Office Number / Fax
ECS Southeast LLC - Raleigh	5260 Greens Dairy Road Raleigh, NC 27616	(919)861-9910 (919)861-9911

Tested by	Checked by	Approved by	Date Received
acreech	acreech	acreech	6/26/2024

LIQUID AND PLASTIC LIMITS TEST REPORT



TEST RESULTS (ASTM D4318-10 (MULTIPOINT TEST))

	Sample Location	Sample Number	Sample Depth (ft)	LL	PL	PI	%<#40	%<#200	AASHTO	USCS	Material Description
■	B-01	S-2	3.50-5.00	17	NP	NP		28.3			(SM) SILTY SAND, Reddish Yellow
◆	B-02	S-1	1.00-2.50	32	17	15		47.1			(SC) CLAYEY SAND, Yellowish Red
▲	B-04	S-2	3.50-5.00	27	NP	NP		25.3			(SM) SILTY SAND, Dark Brown
●	B-05	S-1	1.00-2.50	40	20	20		45.0			(SC) CLAYEY SAND, Brown

Project: 32-0312 McDonald's Lillington, NC, 01.00.16
Client:

Project No.: 33:6900
Date Reported: 7/10/2024

Office / Lab	Address	Office Number / Fax
ECS Southeast LLC - Raleigh	5260 Greens Dairy Road Raleigh, NC 27616	(919)861-9910 (919)861-9911

Tested by	Checked by	Approved by	Date Received
acreech	acreech	acreech	6/26/2024