GENERAL REQUIREMENTS

STRUCTURAL LOAD CALCULATIONS

6B.3.1 - 06

Project Name:

Northeast Metropolitan Regional Vocational High School

MSBA Module 6 Requirements:

MSBA 60% Construction Documents – Structural Loading Calculations

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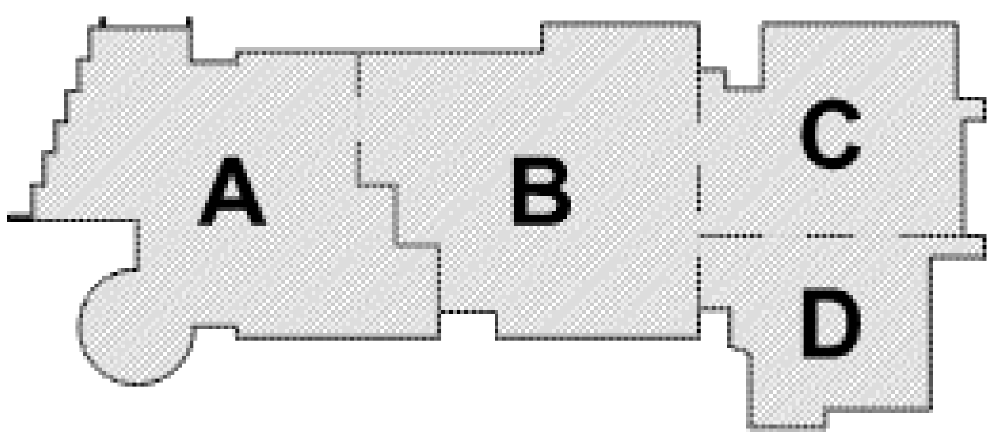
Table of Contents



Project Synopsis

The project is located in Wakefield, Massachusetts. The main building to be constructed consists of a four-story vocational high school near the existing Northeast Metropolitan Regional Vocational High School. The main building is comprised of four wings, named Areas A – D on the project documents. Areas A consists of the cafeteria/kitchen and acedemic rooms, Area B is the main acedemic wing, Area C holds the auditorium, and Area D the gymnasium. Additional buildings to be constructed consist of a two-story locker building, a single story concessions building, and a single story pre-engineered maintainance garage building.

The majority of the structure will be steel-framed, supported by reinforced concrete foundations. The ground floor shall be constructed as a normal-weight concrete slab on grade. Each wing has a mezzanine level above the ground floor level that shall be constructed using precast concrete plank, supported by load-bearing concrete-masonry walls and reinforced concrete foundations. All suspsended floor systems above the mezzanine levels will be constructed as a light-weight concrete slab on steel deck, supported by structural steel beams and girders. The roof system in Areas A and B will consist of steel deck, supported by steel beams and girders; the main roof system in Areas C and D will consist of steel deck, supported by open-web steel joists.

The main structure's lateral force resisting system shall mainly consist of ordinary concentric steel braced frames, comprised of hollow-structural steel members. Reinforced concrete-masonry shear walls will be used throughout the building as well. The structure will have an expansion joint, separating Areas A and B from Areas C and D. The combined lateral force resisting system will be designed to resist the loads imparted on the structure from local wind and seismic forces per applicable design codes.

The two-story locker building structure will consist of structural steel beams, supported by load-bearing concrete-masonry walls and reinforced concrete foundations. The ground floor shall be constructed as a normal-weight concrete slab on grade. The second floor system will be constructed as a light-weight concrete slab on deck, supported by steel beams and girders. The roof system will consist of steel deck, supported by steel beams and girders.

The single-story concessions building will consist of pre-fabricated wood trusses, supported by load-bearing reinforced masonry walls and reinforced concrete foundations. The ground floor shall be constructed as a normal-weight concrete slab on grade. The roof system will consist of plywood sheathing, spanning over wood trusses and masonry walls on all sides.

The maintainance garage building will consist of a pre-engineered steel frame superstructure, supported on reinforced concrete foundations. The ground floor shall be constructed as a normal-weight concrete slab on grade. The roof system will consist of steel deck, supported by continuous steel 'Z'-shaped purlins, spanning between steel frames.

Design Codes

1. Massachusetts State Building Code, 9th Edition
2. International Building Code, 2015 Edition
3. ASCE 7-10: Minimum Design Loads for Buildings and Other Structures
4. ACI 318-14: Building Code Requirements for Structural Concrete
5. ACI 530-13: Building Code Requirements for Masonry Structures
6. AISC 360-10: Specification for Structural Steel Buildings
7. *Other codes as required by the design codes listed above*

Geotechnical Recommendations for Foundation Analysis and Design

The foundation design for this project shall be done with the recommendations from the soils investigations performed by Lahlaf Geotechnical Consulting, Inc from June, 2021. Their report recommended a maximum net allowable bearing pressure of 4,000 pounds per square-foot to be used for the design of the structure's foundations.

Project Materials and Strengths

Concrete:

a.	Foundations	4500 psi
b.	Slab-on-Grade	4000 psi
c.	Composite Slab-on-Steel Deck	4000 psi
d.	Exterior Concrete	5000 psi

Reinforcing Steel:	ASTM A615, Grade 60 ASTM A185 for Welded Wire Reinforcing
Structural Steel:	ASTM A992, Grade 50
Steel Channels:	ASTM A36
Steel Plates, Bars, Angles, etc.:	ASTM A36
Hollow Structural Steel Sections:	ASTM A500, Grade B
Structural Pipes:	ASTM A53, Grade B or ASTM A501
High-Strength Bolts:	ASTM A325-N
Steel Deck:	ASTM A653 (Galvanized Deck)
Concrete-Masonry Units:	ASTM C90, Grade N, Type I, 2000 psi
Grout:	ASTM C476, 2500 psi
Mortar:	ASTM C270, Type S, 1800 psi

Dead and Live Loading Criteria

Design Dead Loads:

Typical Floor Loading on Composite Deck:	
5 ¼" Light-Weight Concrete	42 psf
2" x 20-Gauge Composite Steel Deck	3 psf
Mechanical/ Electrical/ Plumbing	10 psf
Miscellaneous	5 psf
	∑60 psf
Typical Roof Loading on Steel Deck:	
3" x 20-Gauge Type NS or NSA Steel Deck	3 psf
Roofing and Insulation	7 psf
Mechanical/ Electrical/ Plumbing	10 psf
Photovoltaic Panels	15 psf
Miscellaneous	5 psf
	∑35 psf
Roof Loading on Mechanical Roof Pads:	
4 " Normal-Weight Concrete	67 psf
3" x 20-Gauge Composite Steel Deck	3 psf
Mechanical/ Electrical/ Plumbing	10 psf
	∑80 psf

Design Live Loads:

Classrooms with Partitions	40 psf + 15 psf *(Reducible)*
Reading Rooms	60 psf *(Reducible)*
Corridors (First Floor)	100 psf *(Reducible)*
Corridors (Above First Floor)	80 psf *(Reducible)*
Lobbies	100 psf *(Non-Reducible)*
Assembly/Public Gathering Areas	100 psf *(Non-Reducible)*
Stairs	100 psf *(Non-Reducible)*
Storage (Light)	125 psf *(Non-Reducible)*
Storage (Mechanical Equipment)	150 psf *(Non-Reducible)*
Roof (Live)	20 psf *(Non-Reducible)*

Snow Loading Criteria

[In accordance with ASCE7-10]

Building details

Roof type;	Flat
Width of roof;	b = **640.00** ft

Ground snow load

Ground snow load;	P_g = **50.00** lb/ft^2
Density of snow (Figure 7-1);	γ = min(0.13 × P_g / 1ft + 14lb/ft^3, 30lb/ft^3) = **20.50** lb/ft^3
Terrain typeSect. 26.7;	B
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);	III
Importance factor (Table 1.5-2);	I_s = **1.10**
Min snow load for low slope roofs (Sect 7.3.4);	P_{f_min} = I_s × 20 lb/ft^2 = **22.00** lb/ft^2
Flat roof snow load (Sect 7.3);	P_f = 0.7 × C_e × C_t × I_s × P_g = **38.50** lb/ft^2

Wind Loading Criteria

Areas A and B

[In accordance with ASCE7-10]

***Using the directional design method**

Building data

Type of roof; Flat

Length of building; b = **470.00** ft

Width of building; d = **200.00** ft

Height to eaves; H = **62.00** ft

Mean height; h = **62.00** ft

General wind load requirements

Basic wind speed; V = **137.0** mph

Risk category; III

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.27.4.7); 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)
15.00	0.85	34.72
30.00	0.98	40.02
45.00	1.07	43.50
62.00	1.14	46.48

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.); q_i = **46.48** psf

Pressures and forces

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

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

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	62.00	-0.90	46.48	-43.92	14570.00	-639.93
B (-ve)	62.00	-0.90	46.48	-43.92	14570.00	-639.93
C (-ve)	62.00	-0.50	46.48	-28.12	29140.00	-819.38
D (-ve)	62.00	-0.30	46.48	-20.22	35720.00	-722.18

Total vertical net force; $F_{w,v}$ = **-2821.42** 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_1	15.00	0.80	34.72	15.24	7050.00	107.44
A_2	30.00	0.80	40.02	18.85	7050.00	132.90
A_3	45.00	0.80	43.50	21.21	7050.00	149.54
A_4	62.00	0.80	46.48	23.24	7990.00	185.68
B	62.00	-0.50	46.48	-28.12	29140.00	-819.38
C	62.00	-0.70	46.48	-36.02	12400.00	-446.65
D	62.00	-0.70	46.48	-36.02	12400.00	-446.65

Overall loading

Projected vertical plan area of wall; $A_{vert\_w\_0} = b \times H =$ **29140.00** ft^2

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

Minimum overall horizontal loading; $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0}$ = **466.24** kips

Leeward net force; $F_l = F_{w,wB}$ = **-819.4** kips

Windward net force; $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3} + F_{w,wA\_4}$ = **575.6** kips

Overall horizontal loading; $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min})$ = **1394.9** 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)	62.00	-0.18	46.48	1.25	14570.00	18.28

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)
B (+ve)	62.00	-0.18	46.48	1.25	14570.00	18.28
C (+ve)	62.00	-0.18	46.48	1.25	29140.00	36.57
D (+ve)	62.00	-0.18	46.48	1.25	35720.00	44.82

Total vertical net force; $F_{w,v}$ = **117.96** 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_1	15.00	0.80	34.72	31.97	7050.00	225.40
A_2	30.00	0.80	40.02	35.58	7050.00	250.86
A_3	45.00	0.80	43.50	37.94	7050.00	267.50
A_4	62.00	0.80	46.48	39.97	7990.00	319.37
B	62.00	-0.50	46.48	-11.39	29140.00	-331.82
C	62.00	-0.70	46.48	-19.29	12400.00	-239.17
D	62.00	-0.70	46.48	-19.29	12400.00	-239.17

Overall loading

Projected vertical plan area of wall; $A_{vert\_w\_0} = b \times H =$ **29140.00** ft^2

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

Minimum overall horizontal loading; $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0} =$ **466.24** kips

Leeward net force; $F_l = F_{w,wB} =$ **-331.8** kips

Windward net force; $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3} + F_{w,wA\_4} =$ **1063.1** kips

Overall horizontal loading; $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **1394.9** kips

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	62.00	-0.90	46.48	-43.92	6200.00	-272.31
B (-ve)	62.00	-0.90	46.48	-43.92	6200.00	-272.31
C (-ve)	62.00	-0.50	46.48	-28.12	12400.00	-348.67
D (-ve)	62.00	-0.30	46.48	-20.22	69200.00	-1399.06

Total vertical net force; $F_{w,v}$ = **-2292.36** kips

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

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	34.72	15.24	3000.00	45.72
A_2	30.00	0.80	40.02	18.85	3000.00	56.55
A_3	45.00	0.80	43.50	21.21	3000.00	63.63
A_4	62.00	0.80	46.48	23.24	3400.00	79.01
B	62.00	-0.28	46.48	-19.53	12400.00	-242.13
C	62.00	-0.70	46.48	-36.02	29140.00	-1049.62
D	62.00	-0.70	46.48	-36.02	29140.00	-1049.62

Overall loading

Projected vertical plan area of wall; $A_{vert\_w\_90} = d \times H =$ **12400.00** ft^2

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

Minimum overall horizontal loading; $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} =$ **198.40** kips

Leeward net force; $F_l = F_{w,wB} =$ **-242.1** kips

Windward net force; $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3} + F_{w,wA\_4} =$ **244.9** kips

Overall horizontal loading; $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) =$ **487.0** kips

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (+ve)	62.00	-0.18	46.48	1.25	6200.00	7.78
B (+ve)	62.00	-0.18	46.48	1.25	6200.00	7.78
C (+ve)	62.00	-0.18	46.48	1.25	12400.00	15.56
D (+ve)	62.00	-0.18	46.48	1.25	69200.00	86.84

Total vertical net force; $F_{w,v}$ = **117.96** kips

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

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	34.72	31.97	3000.00	95.92

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_2	30.00	0.80	40.02	35.58	3000.00	106.75
A_3	45.00	0.80	43.50	37.94	3000.00	113.83
A_4	62.00	0.80	46.48	39.97	3400.00	135.90
B	62.00	-0.28	46.48	-2.79	12400.00	-34.65
C	62.00	-0.70	46.48	-19.29	29140.00	-562.06
D	62.00	-0.70	46.48	-19.29	29140.00	-562.06

Overall loading

Projected vertical plan area of wall; $A_{vert\_w\_90} = d \times H = \mathbf{12400.00}$ ft^2

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

Minimum overall horizontal loading; $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} = \mathbf{198.40}$ kips

Leeward net force; $F_l = F_{w,wB} = \mathbf{-34.7}$ kips

Windward net force; $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3} + F_{w,wA\_4} = \mathbf{452.4}$ kips

Overall horizontal loading; $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = \mathbf{487.0}$ kips

Areas C and D

[In accordance with ASCE 7-10]

***Using the directional design method**

Building data

Type of roof; Flat

Length of building; b = **285.00** ft

Width of building; d = **200.00** ft

Height to eaves; H = **82.00** ft

Mean height; h = **82.00** ft

General wind load requirements

Basic wind speed; V = **137.0** mph

Risk category; III

Velocity pressure exponent coef (Table 26.6-1); $K_d = \mathbf{0.85}$

Exposure category (cl 26.7.3); C

Enclosure classification (cl.26.10); Enclosed buildings

Internal pressure coef +ve (Table 26.11-1); $GC_{pi\_p} = \mathbf{0.18}$

Internal pressure coef –ve (Table 26.11-1); $GC_{pi\_n} = \mathbf{-0.18}$

Gust effect factor; $G_f = \mathbf{0.85}$

Minimum design wind loading (cl.27.4.7); p_{min_r} = **8** lb/ft^2

Topography

Topography factor not significant; $K_{zt} = 1.0$

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

Velocity pressures table

z (ft)	K_z (Table 27.3-1)	q_z (psf)
15.00	0.85	34.72
40.00	1.04	42.47
60.00	1.13	46.15
82.00	1.22	49.66

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.); q_i = **49.66** psf

Pressures and forces

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

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

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	82.00	-0.90	49.66	-46.93	11685.00	-548.40
B (-ve)	82.00	-0.90	49.66	-46.93	11685.00	-548.40
C (-ve)	82.00	-0.50	49.66	-30.05	23370.00	-702.18
D (-ve)	82.00	-0.30	49.66	-21.60	10260.00	-221.65

Total vertical net force; $F_{w,v}$ = **-2020.62** 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_1	15.00	0.80	34.72	14.67	4275.00	62.70
A_2	40.00	0.80	42.47	19.94	7125.00	142.10
A_3	60.00	0.80	46.15	22.44	5700.00	127.93
A_4	82.00	0.80	49.66	24.83	6270.00	155.69
B	82.00	-0.50	49.66	-30.05	23370.00	-702.18

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)
C	82.00	-0.70	49.66	-38.49	16400.00	-631.22
D	82.00	-0.70	49.66	-38.49	16400.00	-631.22

Overall loading

Projected vertical plan area of wall; $A_{vert\_w\_0} = b \times H = \mathbf{23370.00}$ ft^2

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

Minimum overall horizontal loading; $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0} = \mathbf{373.92}$ kips

Leeward net force; $F_l = F_{w,wB} = \mathbf{-702.2}$ kips

Windward net force; $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3} + F_{w,wA\_4} = \mathbf{488.4}$ kips

Overall horizontal loading; $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = \mathbf{1190.6}$ 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)	82.00	-0.18	49.66	1.34	11685.00	15.67
B (+ve)	82.00	-0.18	49.66	1.34	11685.00	15.67
C (+ve)	82.00	-0.18	49.66	1.34	23370.00	31.34
D (+ve)	82.00	-0.18	49.66	1.34	10260.00	13.76

Total vertical net force; $F_{w,v} = \mathbf{76.43}$ kips

Total horizontal net force; $F_{w,h} = \mathbf{0.00}$ 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_1	15.00	0.80	34.72	32.55	4275.00	139.13
A_2	40.00	0.80	42.47	37.82	7125.00	269.48
A_3	60.00	0.80	46.15	40.32	5700.00	229.83
A_4	82.00	0.80	49.66	42.71	6270.00	267.79
B	82.00	-0.50	49.66	-12.17	23370.00	-284.35
C	82.00	-0.70	49.66	-20.61	16400.00	-338.01
D	82.00	-0.70	49.66	-20.61	16400.00	-338.01

Overall loading

Projected vertical plan area of wall; $A_{vert\_w\_0} = b \times H = \mathbf{23370.00}$ ft^2

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

Minimum overall horizontal loading; $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0} = \mathbf{373.92}$ kips

Leeward net force; $F_l = F_{w,wB} = \mathbf{-284.4}$ kips

Windward net force; $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3} + F_{w,wA\_4} = \mathbf{906.2}$ kips

Overall horizontal loading; $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = \mathbf{1190.6}$ kips

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	82.00	-0.90	49.66	-46.93	8200.00	-384.84
B (-ve)	82.00	-0.90	49.66	-46.93	8200.00	-384.84
C (-ve)	82.00	-0.50	49.66	-30.05	16400.00	-492.76
D (-ve)	82.00	-0.30	49.66	-21.60	24200.00	-522.80

Total vertical net force; $F_{w,v} = \mathbf{-1785.24}$ kips

Total horizontal net force; $F_{w,h} = \mathbf{0.00}$ kips

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	34.72	14.67	3000.00	44.00
A_2	40.00	0.80	42.47	19.94	5000.00	99.72
A_3	60.00	0.80	46.15	22.44	4000.00	89.77
A_4	82.00	0.80	49.66	24.83	4400.00	109.26
B	82.00	-0.41	49.66	-26.46	16400.00	-433.91
C	82.00	-0.70	49.66	-38.49	23370.00	-899.49
D	82.00	-0.70	49.66	-38.49	23370.00	-899.49

Overall loading

Projected vertical plan area of wall; $A_{vert\_w\_90} = d \times H = \mathbf{16400.00}$ ft^2

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

Minimum overall horizontal loading; $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} = \mathbf{262.40}$ kips

Leeward net force; $F_l = F_{w,wB} = \mathbf{-433.9}$ kips

Windward net force; $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3} + F_{w,wA\_4} = \mathbf{342.8}$ kips

Overall horizontal loading; $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = \mathbf{776.7}$ kips

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (+ve)	82.00	-0.18	49.66	1.34	8200.00	11.00
B (+ve)	82.00	-0.18	49.66	1.34	8200.00	11.00
C (+ve)	82.00	-0.18	49.66	1.34	16400.00	21.99
D (+ve)	82.00	-0.18	49.66	1.34	24200.00	32.45

Total vertical net force; $F_{w,v} = \mathbf{76.43}$ kips

Total horizontal net force; $F_{w,h} = \mathbf{0.00}$ kips

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

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	34.72	32.55	3000.00	97.64
A_2	40.00	0.80	42.47	37.82	5000.00	189.11
A_3	60.00	0.80	46.15	40.32	4000.00	161.29
A_4	82.00	0.80	49.66	42.71	4400.00	187.93
B	82.00	-0.41	49.66	-8.58	16400.00	-140.70
C	82.00	-0.70	49.66	-20.61	23370.00	-481.66
D	82.00	-0.70	49.66	-20.61	23370.00	-481.66

Overall loading

Projected vertical plan area of wall; $A_{vert\_w\_90} = d \times H = \mathbf{16400.00}$ ft^2

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

Minimum overall horizontal loading; $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} = \mathbf{262.40}$ kips

Leeward net force; $F_l = F_{w,wB} = \mathbf{-140.7}$ kips

Windward net force; $F_w = F_{w,wA\_1} + F_{w,wA\_2} + F_{w,wA\_3} + F_{w,wA\_4} = \mathbf{636.0}$ kips

Overall horizontal loading; $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = \mathbf{776.7}$ kips

Seismic Loading Calculations

Areas A and B

[In accordance with ASCE 7-10]

Site parameters

Site class; D

Mapped acceleration parameters (Section 11.4.1)

at short period; S_S = **0.25**

at 1 sec period; S_1 = **0.08**

Site coefficientat short period (Table 11.4-1); F_a = **1.600**

at 1 sec period (Table 11.4-2); F_v = **2.400**

Spectral response acceleration parameters

at short period (Eq. 11.4-1); $S_{MS} = F_a \times S_S =$ **0.400**

at 1 sec period (Eq. 11.4-2); $S_{M1} = F_v \times S_1 =$ **0.192**

Design spectral acceleration parameters (Sect 11.4.4)

at short period (Eq. 11.4-3); $S_{DS} = 2 / 3 \times S_{MS} =$ **0.267**

at 1 sec period (Eq. 11.4-4); $S_{D1} = 2 / 3 \times S_{M1} =$ **0.128**

Seismic design category

Risk category (Table 1.5-1); III

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)

B

Seismic design category; B

Approximate fundamental period

Height above base to highest level of building; h_n = **62** 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); $T_a = C_t \times (h_n)^x \times 1\text{sec} / (1\text{ft})^x =$ **0.442** sec

Building fundamental period (Sect 12.8.2); $T = T_a =$ **0.442** sec

Long-period transition period; T_L = **12** 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); $R = \mathbf{3.25}$

Seismic importance factor (Table 1.5-2); $I_e = \mathbf{1.250}$

Seismic response coefficient (Sect 12.8.1.1)

Calculated (Eq 12.8-2); $C_{s\_calc} = S_{DS} / (R / I_e) = \mathbf{0.1026}$

Maximum (Eq 12.8-3); $C_{s\_max} = S_{D1} / ((T / 1 \text{ sec}) \times (R / I_e)) = \mathbf{0.1114}$

Minimum (Eq 12.8-5); $C_{s\_min} = \max(0.044 \times S_{DS} \times I_e, 0.01) = \mathbf{0.0147}$

Seismic response coefficient; $C_s = \mathbf{0.1026}$

Seismic base shear (Sect 12.8.1)

Effective seismic weight of the structure; $W = \mathbf{19660.0}$ kips

Seismic response coefficient; $C_s = \mathbf{0.1026}$

Seismic base shear (Eq 12.8-1); $V = C_s \times W = \mathbf{2016.4}$ kips

Areas C and D

[In accordance with ASCE 7-10]

Site parameters

Site class; D

Mapped acceleration parameters (Section 11.4.1)

at short period; $S_S = \mathbf{0.25}$

at 1 sec period; $S_1 = \mathbf{0.08}$

Site coefficientat short period (Table 11.4-1); $F_a = \mathbf{1.600}$

at 1 sec period (Table 11.4-2); $F_v = \mathbf{2.400}$

Spectral response acceleration parameters

at short period (Eq. 11.4-1); $S_{MS} = F_a \times S_S = \mathbf{0.400}$

at 1 sec period (Eq. 11.4-2); $S_{M1} = F_v \times S_1 = \mathbf{0.192}$

Design spectral acceleration parameters (Sect 11.4.4)

at short period (Eq. 11.4-3); $S_{DS} = 2 / 3 \times S_{MS} = \mathbf{0.267}$

at 1 sec period (Eq. 11.4-4); $S_{D1} = 2 / 3 \times S_{M1} = \mathbf{0.128}$

Seismic design category

Risk category (Table 1.5-1); III

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)

B

Seismic design category; B

Approximate fundamental period

Height above base to highest level of building; $h_n = \mathbf{82}$ 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 / $(1\text{ft})^x$= **0.545** sec
Building fundamental period (Sect 12.8.2);	$T = T_a$ = **0.545** sec
Long-period transition period;	T_L = **12** 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 = **1.250**
Seismic response coefficient (Sect 12.8.1.1)	
Calculated (Eq 12.8-2);	$C_{s\_calc} = S_{DS} / (R / I_e)$ = **0.1026**
Maximum (Eq 12.8-3);	$C_{s\_max} = S_{D1} / ((T / 1\text{ sec}) \times (R / I_e))$ = **0.0903**
Minimum (Eq 12.8-5);	$C_{s\_min} = \max(0.044 \times S_{DS} \times I_e, 0.01)$ = **0.0147**
Seismic response coefficient;	C_s = **0.0903**
Seismic base shear (Sect 12.8.1)	
Effective seismic weight of the structure;	W = **9390.0** kips
Seismic response coefficient;	C_s = **0.0903**
Seismic base shear (Eq 12.8-1);	$V = C_s \times W$ = **848.2** kips

Sample Gravity Analysis and Design Calculations

1. Sample Steel Roof Beam

[In accordance with AISC360-16 using the LRFD method]

Support conditions

Support A	Vertically restrained	
	Rotationally free	
Support B	Vertically restrained	
	Rotationally free	

Applied loading

Beam loads	Dead self weight of beam × 1	
	Dead full UDL 0.35 kips/ft	
	Snow full UDL 0.4 kips/ft	
	Roof Live full UDL 0.2 kips/ft	

Load combinations

Load combination 1 - Full	Support A	Dead × 1.20
		Live × 1.60
		Snow × 1.60
		Roof Live × 1.60
		Dead × 1.20
		Live × 1.60
		Snow × 1.60
		Roof Live × 1.60
	Support B	Dead × 1.20
		Live × 1.60
		Snow × 1.60
		Roof Live × 1.60

Analysis results

Maximum moment;	M_{max} = **234.3** kips_ft;	M_{min} = **0** kips_ft
Maximum shear;	V_{max} = **26** kips;	V_{min} = **-26** kips
Deflection;	δ_{max} = **1** in;	δ_{min} = **0** in
Maximum reaction at support A;	R_{A_max} = **26** kips;	R_{A_min} = **26** kips
Unfactored dead load reaction at support A;	R_{A_Dead} = **7.3** kips	
Unfactored snow load reaction at support A;	R_{A_Snow} = **7.2** kips	
Unfactored roof live load reaction at support A;	$R_{A\_Roof\ Live}$ = **3.6** kips	
Maximum reaction at support B;	R_{B_max} = **26** kips;	R_{B_min} = **26** kips
Unfactored dead load reaction at support B;	R_{B_Dead} = **7.3** kips	
Unfactored snow load reaction at support B;	R_{B_Snow} = **7.2** kips	

Unfactored roof live load reaction at support B; $R_{B\_Roof\ Live}$ = **3.6** kips

Section details

Section type; **W 24x55 (AISC 15th Edn (v15.0))**

ASTM steel designation; **A992**

Steel yield stress; F_y = **50** ksi

Steel tensile stress; F_u = **65** ksi

Modulus of elasticity; E = **29000** ksi

Resistance factors

Resistance factor for tensile yielding ϕ_{ty} = **0.90**

Resistance factor for tensile rupture ϕ_{tr} = **0.75**

Resistance factor for compression ϕ_c = **0.90**

Resistance factor for flexure ϕ_b = **0.90**

Lateral bracing

Span 1 has continuous lateral bracing

Classification of sections for local buckling - Section B4.1

Classification of flanges in flexure - Table B4.1b (case 10)

Width to thickness ratio; $b_f / (2 \times t_f)$ = **6.94**

Limiting ratio for compact section; $\lambda_{pff} = 0.38 \times \sqrt{[E / F_y]}$ = **9.15**

Limiting ratio for non-compact section; $\lambda_{rff} = 1.0 \times \sqrt{[E / F_y]}$ = **24.08**; Compact

Classification of web in flexure - Table B4.1b (case 15)

Width to thickness ratio; $(d - 2 \times k) / t_w$ = **54.63**

Limiting ratio for compact section; $\lambda_{pwf} = 3.76 \times \sqrt{[E / F_y]}$ = **90.55**

Limiting ratio for non-compact section; $\lambda_{rwf} = 5.70 \times \sqrt{[E / F_y]}$ = **137.27**; Compact

Section is compact in flexure

Design of members for shear - Chapter G

Required shear strength $V_r = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **26.031** kips

Web area $A_w = d \times t_w$ = **9.322** in^2

Web plate buckling coefficient k_v = **5.34**

Web shear coefficient - eq G2-3 C_{v1} = **1**

Nominal shear strength – eq G6-1 $V_n = 0.6 \times F_y \times A_w \times C_{v1}$ = **279.660** kips

Resistance factor for shear ϕ_v = **0.90**

Design shear strength $V_c = \phi_v \times V_n$ = **251.694** kips

PASS - Design shear strength exceeds required shear strength

Design of members for flexure in the major axis - Chapter F

Required flexural strength; $M_r = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **234.276** kips_ft

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1; $M_{nyld} = M_p = F_y \times Z_x =$ **558.333** kips_ft

Nominal flexural strength; $M_n = M_{nyld} =$ **558.333** kips_ft

Design flexural strength; $M_c = \phi_b \times M_n =$ **502.500** kips_ft

PASS - Design flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead, live, snow and roof live loads

Limiting deflection; $\delta_{lim} = \min(1.5 \text{ in}, L_{s1} / 360) =$ **1.2** in

Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **0.97** in

PASS - Maximum deflection does not exceed deflection limit

2. Sample Composite Steel Floor Beam

[In accordance with AISC 360-16 using the load and resistance factor design method]

<u>Design summary</u>

Overall design status; Pass

Overall design utilisation; 0.847

Description	Unit	Provided	Required	Utilization	Result
Moment, constr	(kip_ft)	502.5	157.8	0.314	PASS
Shear, constr	(kips)	251.69	17.53	0.070	PASS
Moment, comp	(kip_ft)	786.81	387.17	0.492	PASS
Shear, comp	(kips)	251.69	43.02	0.171	PASS
Deflection, constr	(in)	1.5	0.72	0.479	PASS
Deflection, comp	(in)	1.5	1.27	0.847	PASS

Basic dimensions

Beam span; L = **36.000** ft

Beam spacing on one side; b_1 = **10.000** ft

Beam spacing on other side; b_2 = **10.000** ft

Deck orientation; **Deck ribs perpendicular to beam**

Profiles are assumed to meet all dimensional criteria in AISC 360-16

Overall depth of slab; t = **5.250** in

Height of ribs; h_r = **2.000** in

Centers of ribs; rib_{ccs} = **12.000** in

Average width of rib; w_r = **7.000** in

Material properties

Concrete

Specified compressive strength of concrete; f'_c = **4.00** ksi

Wet density of concrete; w_{cw} = **125** lb/ft^3

Dry density of concrete; w_{cd} = **115** lb/ft^3

Modulus of elasticity of concrete; $E_c = w_{cd}^{1.5} \times \sqrt{(f'_c \times 1 \text{ ksi})} / (1 \text{ lb/ft}^3)^{1.5}$ = **2466** ksi

Steel

Specified minimum yield stress of steel; F_y = **50** ksi

Modulus of elasticity of steel; E_S = **29000** ksi

Loading – secondary beam

Weight of slab construction stage; $w_{slab\_constr} = [t - h_r \times (1 - w_r / rib_{ccs})] \times w_{cw}$ = **46.007** psf

Weight of slab composite stage; $w_{slab\_comp} = [t - h_r \times (1 - w_r / rib_{ccs})] \times w_{cd}$ = **42.326** psf

Weight of steel deck; w_{deck} = **3.000** psf

Additional dead load; w_{d_add} = **0.000** psf

Weight of steel beam; w_{beam_s} = **55.000** lb/ft

Weight of construction live load; w_{constr} = **20.000** psf

Superimposed dead load; w_{serv} = **15.000** psf

Weight of wall parallel to span; w_{w_par} = **0.000** lb/ft

Weight of wall perpendicular to span; w_{w_perp} = **0.000** lb/ft ;assumed to be at mid-span.

Floor live load; w_{imp} = **100.000** psf

Lightweight partition load; w_{part} = **0.000** psf

Total construction stage dead load; $w_{constr\_D} = [(w_{slab\_constr}+w_{deck}+w_{d\_add})\times((b_1+b_2)/2)] + w_{beam\_s}$ = **545.069** lb/ft

Total construction stage live load; $w_{constr\_L} = w_{constr} \times (b_1 + b_2) / 2$ = **200.000** lb/ft

Total composite stage dead load(excluding walls); $w_{comp\_D} = [(w_{slab\_comp}+w_{deck}+w_{d\_add}+w_{serv})\times(b_1+b_2)/2]+w_{beam\_s}$ = **658.264** lb/ft

Total composite stage live load; $w_{comp\_L} = (w_{imp} + w_{part}) \times (b_1 + b_2)/2$ = **1000.000** lb/ft;

Design forces – secondary beam

Max ultimate moment at construction stage; $M_{constr\_u} = (1.2 \times w_{constr\_D} + 1.6 \times w_{constr\_L}) \times L^2/ 8$ = **157.801** kips_ft

Max ultimate shear at construction stage; $V_{constr\_u} = (1.2 \times w_{constr\_D} + 1.6 \times w_{constr\_L}) \times L / 2$ = **17.534** kips

Maximum ultimate moment at composite stage;

$M_{comp\_u} = (1.2 \times w_{comp\_D} + 1.6 \times w_{comp\_L}) \times L^2/ 8 + 1.2 \times w_{w\_par} \times L^2/8 + 1.2 \times w_{w\_perp} \times (b_1 + b_2)/2 \times L/4$= **387.166** kips_ft

Maximum ultimate shear at composite stage;

$V_{comp\_u} = (1.2 \times w_{comp\_D} + 1.6 \times w_{comp\_L}) \times L / 2 + 1.2 \times w_{w\_par} \times L / 2 + 1.2 \times w_{w\_perp} \times (b_1 + b_2)/2 \times 1/2$= **43.019** kips

Point of max. B.M. from nearest support; L_{BM_near} = L/2 =**18.00** ft

Steel section check

Trial steel section; **W24X55**

Plastic modulus of steel section; Z_x = **134.00** in^3

Elastic modulus of steel section; S_x = **114.00** in^3

Width to thickness ratio; $\lambda_f = b_f / (2 \times t_f)$ = **6.941**

Limiting width to thickness ratio (compact); $\lambda_{pf} = 0.38 \times \sqrt{(E_S / F_y)}$ = **9.152**

Limiting width to thickness ratio (noncompact); $\lambda_{rf} = \sqrt{(E_S / F_y)}$ = **24.083**

Flange is compact

Depth to thickness ratio (h/t_w); λ_w = **54.600**

Limiting depth to thickness ratio (compact); $\lambda_{pw} = 3.76 \times \sqrt{(E_S / F_y)}$ = **90.553**

Limiting depth to thickness ratio (noncompact); $\lambda_{rw} = 5.70 \times \sqrt{(E_S / F_y)}$ = **137.274**

Web is compact

Strength check at construction stage for flexure

Check for flexure

Plastic moment for steel section; $M_p = F_y \times Z_x$ = **558.333** kip_ft

Resistance factor for flexure; ϕ_b = **0.90**

Design flexural strength of steel section alone; $M_{constr\_n} = \phi_b \times M_p$ = **502.500** kip_ft

Required flexural strength; M_{constr_u} = **157.801** kip_ft

PASS - Beam bending at construction stage loading

Strength check at construction stage for shear

Web area; $A_w = d \times t_w$ = **9.322** in^2

Web plate buckling coefficient; k_v = **5.34**

Depth to thickness ratio (h/t_w); λ_w = **54.600**

Web shear coefficient; C_{v1} = **1.00**

Resistant factor for shear; ϕ_v = **0.9**

Design shear strength; $V_{constr\_n} = \phi_v \times (0.6 \times F_y \times A_w \times C_{v1})$ = **251.694** kips

Required shear strength; V_{constr_u} = **17.534** kips

PASS - Beam shear at construction stage loading

Design of steel anchors

Note - for non-uniform stud layouts a higher concentration of studs should be located towards the ends of the beam

Effective slab width of composite section; $b = \min(L/8, b_1/2) + \min(L/8, b_2/2)$ = **108.000** in

Effective area of concrete flange; $A_c = b \times (t - h_r)$ = **351.00** in^2

Diameter of stud anchor; dia = **0.750** in

Length of stud anchor after weld; H_s = **3.50** in

Specified tensile strength of stud anchor; F_u = **65** ksi

Cross section area of one stud anchor; $A_{sa} = \pi \times dia^2 / 4$ = **0.442** in^2

Maximum diameter permitted; $dia_{max} = 2.5 \times t_f$ = **1.263** in

PASS - Diameter of stud anchor provided is OK

Point of max. B.M. from nearest support; L_{BM_near} = **18.00** ft

No. of ribs from points of zero to max moment; $rib_{numbers} = int(L_{BM\_near} / rib_{ccs} - 1)$ = **17**

No. of ribs with 1 stud per rib; N_{r1} = **17**

No. of ribs with 2 studs per rib; N_{r2} = **0**

No. of ribs with 3 studs per rib; N_{r3} = **0**

Total number of studs; $N_{prov} = N_{r1} + 2 \times N_{r2} + 3 \times N_{r3}$ = **17**

Group effect factor for 1 stud per rib; R_{g1} = **1.00**

Group effect factor for 2 studs per rib; R_{g2} = **0.85**

Group effect factor for 3 studs per rib; R_{g3} = **0.70**

Value of e_{mid-ht} is less than 2 in (51 mm)

Position effect factor for deck perpendicular; R_p = **0.60**

Nom. strength of one stud with 1 stud per rib; $Q_{n1} = \min(0.5 \times A_{sa} \times \sqrt{(f'_c \times E_c)}, R_{g1} \times R_p \times A_{sa} \times F_u)$ = **17.230** kips

Nom. strength of one stud with 2 studs per rib; $Q_{n2} = \min(0.5 \times A_{sa} \times \sqrt{(f'_c \times E_c)}, R_{g2} \times R_p \times A_{sa} \times F_u)$ = **14.645** kips

Nom. strength of one stud with 3 studs per rib; $Q_{n3} = \min(0.5 \times A_{sa} \times \sqrt{(f'_c \times E_c)}, R_{g3} \times R_p \times A_{sa} \times F_u)$ = **12.061** kips

Total strength of provided steel anchors; $S_{sc} = N_{r1}\times Q_{n1} + 2\times N_{r2}\times Q_{n2} + 3\times N_{r3}\times Q_{n3}$ = **292.90** kips

Resistance of concrete flange; $C_{cf} = 0.85 \times f'_c \times A_c$ = **1193.400** kips

Resistance of steel beam; $T_{sb} = A \times F_y$ = **810.000** kips

Beam/slab interface shear force; $C = \min(C_{cf}, T_{sb})$ = **810.000** kips

Strength of studs is less than maximum interface shear force therefore partial composite action takes place

Strength check at partial composite action

Actual net tensile force ; $V_h = C$ = **810.000** kips

Assuming plastic neutral axis at the bottom of the steel beam flange.

Resultant compressive force at flange bottom; $P_{yf} = b_f \times t_f \times F_y$ = **177.003** kips

Net force at steel and concrete interface; $C_{net} = T_{sb} - 2 \times P_{yf}$ = **455.995** kips

PNA is in the web of the I Section

Shear connection force; $F_{shear} = S_{sc}$ = **292.90** kips

Total depth of concrete at full stress; $d_c = F_{shear} / (0.85 \times f'_c \times b)$ = **0.798** in

Depth of compression from top of the steel flange; $t' = A / (2 \times t_w) - b_f \times t_f / t_w - 0.85 \times f'_c / F_y \times b \times d_c / (2 \times t_w) + t_f$ = **4.634** in

Tension

Bottom flange component; $F_{bf} = F_y \times b_f \times t_f$ = **177.003** kips

Moment capacity of bottom flange; $M_{bf} = F_{bf} \times (d - (t_f/2) - t')$ = **276.030** kip_ft

Web component; $F_{web\_t} = F_y \times (A - (2\times b_f \times t_f) - (t'-t_f)\times t_w)$ = **374.450** kips

Moment capacity of web; $M_{web\_t} = F_{web\_t} \times (d - t' - t_f)/2$ = **288.032** kip_ft

Compression

Web component; $F_{web\_c} = F_y \times (t'-t_f)\times t_w$ = **81.545** kips

Moment capacity of web; $M_{web\_c} = F_{web\_c} \times (t' - t_f)/2$ = **14.029** kip_ft

Top flange component; $F_{tf} = F_y \times b_f \times t_f$ = **177.003** kips

Moment capacity of top flange; $M_{tf} = F_{tf} \times (t' - t_f/2)$ = **64.626** kip_ft

Concrete flange component; $F_{cf} = 0.85 \times f'_c \times b \times d_c$ = **292.904** kips

Moment capacity of concrete flange; $M_{cf} = F_{cf} \times (t - d_c/2 + t')$ = **231.518** kip_ft

Design flexural strength of beam; $M_{comp\_n} = \phi_b \times (M_{bf} + M_{web\_t} + M_{web\_c} + M_{tf} + M_{cf})$ = **786.811** kip_ft

Required flexural strength; M_{comp_u} = **387.166** kip_ft

PASS - Beam bending at partial composite stage

Check for shear

Design shear strength; $V_{comp\_n} = V_{constr\_n}$ = **251.694** kips

Required shear strength; V_{comp_u} = **43.019** kips

PASS - Beam shear at partial composite stage loading

Check for deflection (Commentary section I3.1)

Calculation of immediate construction stage deflection;

Deflection due to dead load; $\Delta_{short\_D} = 5 \times w_{constr\_D} \times L^4 / (384 \times E_S \times I_x) =$ **0.5262** in

Amount of beam camber; $\Delta_{camber} =$ **0.000** in

PASS - The camber is less than the construction stage dead load deflection

Deflection due to construction live load; $\Delta_2 = 5 \times w_{constr\_L} \times L^4 / (384 \times E_S \times I_x) =$ **0.1931** in

Net total construction stage deflection; $\Delta_{short} = \Delta_{short\_D} + \Delta_2 - \Delta_{camber} =$ **0.719** in

For short term loading:-

Short term modular ratio; $n_s = E_S / E_c =$ **11.8**

Depth of neutral axis from top of concrete;

$y_s = [b\times (t-h_r)/n_s\times (t-h_r)/2 + A\times (t+d/2)] / [b\times (t-h_r)/n_s+A]$

$y_s =$ **7.051** in

Moment of inertia of fully composite section;

$I_s = I_x + A\times (d/2+t-y_s)^2 + b\times (t - h_r)^3/(12\times n_s) + b\times (t - h_r)/n_s \times (y_s - (t-h_r)/2)^2$

$I_s =$ **3875** in^4

Effective mt of inertia for partially composite; $I_{s\_eff} = 0.75 \times [I_x + \surd(F_{shear} / C) \times (I_s - I_x)] =$;**2151.2**; in^4

Proportion of live load which is short term; $r_{L\_s} =$ **67** %

Deflection due to short term live load; $\Delta_{L\_s} = 5 \times r_{L\_s} \times w_{comp\_L} \times L^4 / (384 \times E_S \times I_{s\_eff}) =$ **0.4059** in

For long term loading:-

Long term concrete modulus as % of short term; $r_{E\_l} =$ **50** %

Long term modular ratio; $n_l = E_S / (E_c \times r_{E\_l}) =$ **23.5**

Depth of neutral axis from top of concrete;

$y_l = [b\times (t-h_r)/n_l\times (t-h_r)/2 + A\times (t+d/2)] / [b\times (t-h_r)/n_l+A]$

$y_l =$ **9.653** in

Moment of inertia of fully composite section;

$I_l = I_x + A\times (d/2+t-y_l)^2 + b\times (t - h_r)^3/(12\times n_l) + b\times (t - h_r)/n_l \times (y_l - (t-h_r)/2)^2$

$I_l =$ **3212** in^4

Effective mt of inertia for partially composite; $I_{l\_eff} = 0.75 \times [I_x + \surd(F_{shear} / C) \times (I_l - I_x)] =$ **1852.1** in^4

Proportion of live load which is long term; $r_{L\_l} = 1 - r_{L\_s} =$ **33** %

Deflection due to long term live load; $\Delta_{L\_l} = 5 \times r_{L\_l} \times w_{comp\_L} \times L^4 / (384 \times E_S \times I_{l\_eff}) =$ **0.2322** in

Dead load due to parallel wall & superimp. dead; $w_{D\_part} = w_{w\_par} + (w_{serv}\times(b_1+ b_2) / 2) =$ **150.0000** lb/ft

Long term deflection due to superimposed dead load (after concrete has cured):-

Wall parallel to span and superimposed dead; $\Delta_4 =5 \times (w_{D\_part}) \times L^4 / (384 \times E_S \times I_{l\_eff}) =$ **0.1055** in

Wall perpendicular to span; $\Delta_5 =(w_{w\_perp} \times(b_1+ b_2) / 2) \times L^3 / (48 \times E_S \times I_{l\_eff}) =$ **0.0000** in

Combined deflections

Net total construction stage deflection; $\Delta_{short} = \Delta_{short\_D} + \Delta_2 - \Delta_{camber} =$ **0.719** in

Net total long term deflection; $\Delta_{long} = \Delta_{short\_D} + \Delta_{L\_s} + \Delta_{L\_l} + \Delta_4 + \Delta_5 - \Delta_{camber} =$ **1.270** in

Combined short and long term live load deflectn; $\Delta_{live} = \Delta_{L\_s} + \Delta_{L\_l}$ = **0.638** in

Net long term dead and super imposed dead defln; $\Delta_{dead} = \Delta_{short\_D} + \Delta_4 + \Delta_5 - \Delta_{camber}$ = **0.632** in

Post composite deflection; $\Delta_{comp} = \Delta_{L\_s} + \Delta_{L\_l} + \Delta_4 + \Delta_5$ = **0.744** in

Allowable max deflection; Δ_{Allow} = **1.500** in

PASS - Deflection less than allowable

Arrangement of steel anchor

Note - for non-uniform stud layouts a higher concentration of studs should be located towards the ends of the beam;

3. Sample Steel Column

[In accordance with AISC360-10 and the LRFD method]

<u>**Column and loading details**</u>

Column details

Column section; **HSS 12x12x3/8**

Design loading

Required axial strength; P_r = **250** kips; (Compression)

Moment about x axis at end 1; M_{x1} = **0.0** kips_ft

Moment about x axis at end 2; M_{x2} = **0.0** kips_ft

Maximum moment about x axis; $M_x = \max(\mathrm{abs}(M_{x1}), \mathrm{abs}(M_{x2}))$ = **0.0** kips_ft

Moment about y axis at end 1; M_{y1} = **0.0** kips_ft

Moment about y axis at end 2; M_{y2} = **0.0** kips_ft

Maximum moment about y axis; $M_y = \max(\mathrm{abs}(M_{y1}), \mathrm{abs}(M_{y2}))$ = **0.0** kips_ft

Maximum shear force parallel to y axis; V_{ry} = **0.0** kips

Maximum shear force parallel to x axis; V_{rx} = **0.0** kips

Material details

Steel grade; **A500 Gr. C**

Yield strength; F_y = **50** ksi

Ultimate strength; F_u = **62** ksi

Modulus of elasticity; E = **29000** ksi

Shear modulus of elasticity; G = **11200** ksi

Unbraced lengths

For buckling about x axis; L_x = **240** in

For buckling about y axis; L_y = **240** in

For torsional buckling; L_z = **240** in

Effective length factors

For buckling about x axis; K_x = **1.00**

For buckling about y axis; K_y = **1.00**

For torsional buckling; K_z = **1.00**

Section classification

Section classification for local buckling (cl. B4)

Critical flange width; $b = b_f - 3 \times t$ = **10.953** in

Critical web width; $h = d - 3 \times t$ = **10.953** in

Width to thickness ratio of flange (compression); $\lambda_{f\_c} = b / t$ = **31.384**

Width to thickness ratio of web (compression); $\lambda_{w\_c} = h / t$ = **31.384**

Width to thickness ratio of flange (major flexure); $\lambda_{f\_fx} = b / t$ = **31.384**

Width to thickness ratio of web (major flexure); $\lambda_{w\_fx} = h / t$ = **31.384**

Width to thickness ratio of flange (minor flexure); $\lambda_{f\_fy} = h / t$ = **31.384**

Width to thickness ratio of web (minor flexure); $\lambda_{w\_fy} = b / t$ = **31.384**

Compression

Limit for nonslender section; $\lambda_{r\_c} = 1.40 \times \sqrt{(E / F_y)}$ = **33.716**

The section is nonslender in compression

Slenderness

Member slenderness

Slenderness ratio about x axis; $SR_x = K_x \times L_x / r_x$ = **50.7**

Slenderness ratio about y axis; $SR_y = K_y \times L_y / r_y$ = **50.7**

Reduction factor for slender elements

Reduction factor for slender elements (E7)

The section does not contain any slender elements therefore:-

Slender element reduction factor; Q = **1.0**

Compressive strength

Flexural buckling about x axis (cl. E3)

Elastic critical buckling stress; $F_{ex} = (\pi^2 \times E) / (SR_x)^2$ = **111.2** ksi

Reduction factor; $Q_x = Q$ = **1.000**

Flexural buckling stress about x axis; $F_{crx} = Q_x \times (0.658^{Qx \times Fy/Fex}) \times F_y$ = **41.4** ksi

Nominal flexural buckling strength; $P_{nx} = F_{crx} \times A_g$ = **662.7** kips

Flexural buckling about y axis (cl. E3)

Elastic critical buckling stress; $F_{ey} = (\pi^2 \times E) / (SR_y)^2$ = **111.2** ksi

Reduction factor; $Q_y = Q$ = **1.000**

Flexural buckling stress about y axis; $F_{cry} = Q_y \times (0.658^{Qy \times Fy/Fey}) \times F_y$ = **41.4** ksi

Nominal flexural buckling strength; $P_{ny} = F_{cry} \times A_g = \mathbf{662.7}$ kips

Design compressive strength (cl.E1)

Resistance factor for compression; $\phi_c = \mathbf{0.90}$

Design compressive strength; $P_c = \phi_c \times \min(P_{nx}, P_{ny}) = \mathbf{596.5}$ kips

PASS - The design compressive strength exceeds the required compressive strength

4. ***Sample Isolated Reinforced Concrete Column Footing***

Footing Analysis

[In accordance with ACI318-19]

Summary results

Description	**Unit**	**Applied**	**Resisting**	**FoS**	**Result**
Uplift verification	kips	243.4			Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	3.803	4	0.951	Pass
Description	**Unit**	**Provided**	**Required**	**Utilization**	**Result**
Moment, positive, x-direction	kip_ft	173.7	487.6	0.356	Pass
Moment, positive, y-direction	kip_ft	173.7	466.3	0.372	Pass
Shear, one-way, x-direction	kips	52.9	104.8	0.505	Pass
Shear, one-way, y-direction	kips	52.9	101.8	0.520	Pass
Shear, two-way, Col 1	psi	73.546	189.737	0.388	Pass
Min.area of reinf, bot., x-direction	in^2	4.147	5.400		Pass
Max.reinf.spacing, bot, x-direction	in	18.0	11.1		Pass
Min.area of reinf, bot., y-direction	in^2	4.147	5.400		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	11.1		Pass

Pad footing details

Length of footing; $L_x = \mathbf{8}$ ft

Width of footing; $L_y = \mathbf{8}$ ft

Footing area; $A = L_x \times L_y = \mathbf{64}$ ft^2

Depth of footing; $h = \mathbf{24}$ in

Depth of soil over footing; $h_{soil} = \mathbf{18}$ in

Density of concrete; $\gamma_{conc} = \mathbf{150.0}$ lb/ft^3

Column no.1 details

Length of column;	l_{x1} = **16.00** in
Width of column;	l_{y1} = **16.00** in
position in x-axis;	x_1 = **48.00** in
position in y-axis;	y_1 = **48.00** in

Soil Properties

Gross allowable bearing pressure;	q_{allow_Gross} = **4** ksf;
Density of soil;	γ_{soil} = **120.0** lb/ft^3
Angle of internal friction;	ϕ_b = **30.0** deg
Design base friction angle;	δ_{bb} = **30.0** deg
Coefficient of base friction;	$\tan(\delta_{bb})$ = **0.577**
Design wall friction angle;	δ_b = **15.0** deg
Passive pressure coefficient (Coulomb);	$K_P = \sin(90 - \phi_b)^2 / (\sin(90 + \delta_b) \times [1 - \sqrt{[\sin(\phi_b + \delta_b) \times \sin(\phi_b) / (\sin(90 + \delta_b))]}]^2)$ = **4.977**
Dead surcharge load;	F_{Dsur} = **25** psf
Live surcharge load;	F_{Lsur} = **100** psf
Self weight;	$F_{swt} = h \times \gamma_{conc}$ = **300** psf
Soil weight;	$F_{soil} = h_{soil} \times \gamma_{soil}$ = **180** psf

Column no.1 loads

Dead load in z;	F_{Dz1} = **75.0** kips
Live load in z;	F_{Lz1} = **100.0** kips
Snow load in z;	F_{Sz1} = **75.0** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-10

1.0D (0.419)

1.0D + 1.0L (0.835)

1.0D + 1.0S (0.712)

1.0D + 0.75L + 0.75S (0.951)

Combination 7 results: 1.0D + 0.75L + 0.75S

Forces on footing

Force in z-axis;	$F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil} + F_{Dsur}) + \gamma_L \times A \times F_{Lsur} + \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1} + \gamma_S \times F_{Sz1}$ = **243.4** kips

Moments on footing

Moment in x-axis, about x is 0; $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil} + F_{Dsur}) \times L_x / 2) + \gamma_L \times A \times F_{Lsur} \times L_x / 2 + \gamma_D \times (F_{Dz1} \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) + \gamma_S \times (F_{Sz1} \times x_1) =$ **973.5** kip_ft

Moment in y-axis, about y is 0; $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil} + F_{Dsur}) \times L_y / 2) + \gamma_L \times A \times F_{Lsur} \times L_y / 2 + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) + \gamma_S \times (F_{Sz1} \times y_1) =$ **973.5** kip_ft

Uplift verification

Vertical force; $F_{dz} =$ **243.37** kips

PASS - Footing is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis; $e_{dx} = M_{dx} / F_{dz} - L_x / 2 =$ **0** in

Eccentricity of base reaction in y-axis; $e_{dy} = M_{dy} / F_{dz} - L_y / 2 =$ **0** in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **3.803** ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **3.803** ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **3.803** ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) =$ **3.803** ksf

Minimum base pressure; $q_{min} = \min(q_1,q_2,q_3,q_4) =$ **3.803** ksf

Maximum base pressure; $q_{max} = \max(q_1,q_2,q_3,q_4) =$ **3.803** ksf

Allowable Bearing Capacity

Allowable bearing capacity; $q_{allow} = q_{allow\_Gross} =$ **4** ksf

$q_{max} / q_{allow} =$ **0.951**

PASS - Allowable bearing capacity exceeds design base pressure

Footing Design

[In accordance with ACI318-19]

Material details

Compressive strength of concrete; $f'_c =$ **4000** psi

Yield strength of reinforcement; $f_y =$ **60000** psi

Compression-controlled strain limit (21.2.2); $\varepsilon_{ty} =$ **0.00200**

Cover to top of footing; $c_{nom\_t} =$ **3** in

Cover to side of footing; $c_{nom\_s} =$ **3** in

Cover to bottom of footing; $c_{nom\_b} =$ **3** in

Concrete type; Normal weight

Concrete modification factor; $\lambda =$ **1.00**

Column type; Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-10

1.4D (0.212)

1.2D + 1.6L + 0.5Lr (0.520)

Combination 2 results: 1.2D + 1.6L + 0.5Lr

Forces on footing

Ultimate force in z-axis; $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil} + F_{Dsur}) + \gamma_L \times A \times F_{Lsur} + \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1}$ = **299.0** kips

Moments on footing

Ultimate moment in x-axis, about x is 0; $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil} + F_{Dsur}) \times L_x / 2) + \gamma_L \times A \times F_{Lsur} \times L_x / 2 + \gamma_D \times (F_{Dz1} \times x_1) + \gamma_L \times (F_{Lz1} \times x_1)$ = **1196.1** kip_ft

Ultimate moment in y-axis, about y is 0; $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil} + F_{Dsur}) \times L_y / 2) + \gamma_L \times A \times F_{Lsur} \times L_y / 2 + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_L \times (F_{Lz1} \times y_1)$ = **1196.1** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis; $e_{ux} = M_{ux} / F_{uz} - L_x / 2$ = **0** in

Eccentricity of base reaction in y-axis; $e_{uy} = M_{uy} / F_{uz} - L_y / 2$ = **0** in

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **4.672** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **4.672** ksf

$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **4.672** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y)$ = **4.672** ksf

Minimum ultimate base pressure; $q_{umin} = \min(q_{u1},q_{u2},q_{u3},q_{u4})$ = **4.672** ksf

Maximum ultimate base pressure; $q_{umax} = \max(q_{u1},q_{u2},q_{u3},q_{u4})$ = **4.672** ksf

Moment design, x direction, positive moment

Ultimate bending moment; $M_{u.x.max}$ = **173.679** kip_ft

Tension reinforcement provided; 9 No.7 bottom bars (11.1 in c/c)

Area of tension reinforcement provided; $A_{sx.bot.prov}$ = **5.4** in^2

Minimum area of reinforcement (8.6.1.1); $A_{s.min} = 0.0018 \times L_y \times h$ = **4.147** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2); $s_{max} = \min(2 \times h, 18\ in)$ = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement; $d = h - c_{nom\_b} - \phi_{x.bot} / 2$ = **20.562** in

Depth of compression block; $a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y)$ = **0.993** in

Neutral axis factor; β_1 = **0.85**

Depth to neutral axis; $c = a / \beta_1$ = **1.168** in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.04982**

Minimum tensile strain(8.3.3.1); $\varepsilon_{min} = \varepsilon_{ty} + 0.003$ = **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity; $M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2) = \mathbf{541.787}$ kip_ft

Flexural strength reduction factor; $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) = \mathbf{0.900}$

Design moment capacity; $\phi M_n = \phi_f \times M_n = \mathbf{487.608}$ kip_ft

$M_{u.x.max} / \phi M_n = \mathbf{0.356}$

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force; $V_{u.x} = \mathbf{52.918}$ kips

Depth to reinforcement; $d_v = h - c_{nom\_b} - \phi_{x.bot} / 2 = \mathbf{20.562}$ in

Size effect factor (22.5.5.1.3); $\lambda_s = \mathbf{1}$

Ratio of longitudinal reinforcement; $\rho_w = A_{sx.bot.prov} / (L_y \square d_v) = \mathbf{0.00274}$

Shear strength reduction factor; $\phi_v = \mathbf{0.75}$

Nominal shear capacity (Eq. 22.5.5.1); $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_y \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_y \times d_v) = \mathbf{139.685}$ kips

Design shear capacity; $\phi V_n = \phi_v \times V_n = \mathbf{104.764}$ kips

$V_{u.x} / \phi V_n = \mathbf{0.505}$

PASS - Design shear capacity exceeds ultimate shear load

Moment design, y direction, positive moment

Ultimate bending moment; $M_{u.y.max} = \mathbf{173.679}$ kip_ft

Tension reinforcement provided; 9 No.7 bottom bars (11.1 in c/c)

Area of tension reinforcement provided; $A_{sy.bot.prov} = \mathbf{5.4}$ in^2

Minimum area of reinforcement (8.6.1.1); $A_{s.min} = 0.0018 \times L_x \times h = \mathbf{4.147}$ in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2); $s_{max} = \min(2 \times h, 18 \text{ in}) = \mathbf{18}$ in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement; $d = h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2 = \mathbf{19.687}$ in

Depth of compression block; $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x) = \mathbf{0.993}$ in

Neutral axis factor; $\beta_1 = \mathbf{0.85}$

Depth to neutral axis; $c = a / \beta_1 = \mathbf{1.168}$ in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003 = \mathbf{0.04757}$

Minimum tensile strain(8.3.3.1); $\varepsilon_{min} = \varepsilon_{ty} + 0.003 = \mathbf{0.00500}$

PASS - Tensile strain exceeds minimum required

Nominal moment capacity; $M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2) = \mathbf{518.162}$ kip_ft

Flexural strength reduction factor; $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9) = \mathbf{0.900}$

Design moment capacity; $\phi M_n = \phi_f \times M_n = \mathbf{466.346}$ kip_ft

$M_{u.y.max} / \phi M_n = \mathbf{0.372}$

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force; $V_{u.y} = \mathbf{52.918}$ kips

Depth to reinforcement; $d_v = h - c_{nom\_b} - \phi_{x.bot} - \phi_{y.bot} / 2 =$ **19.687** in

Size effect factor (22.5.5.1.3); $\lambda_s = $ **1**

Ratio of longitudinal reinforcement; $\rho_w = A_{sy.bot.prov} / (L_x \times d_v) =$ **0.00286**

Shear strength reduction factor; $\phi_v =$ **0.75**

Nominal shear capacity (Eq. 22.5.5.1); $V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_x \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_x \times d_v) =$ **135.694** kips

Design shear capacity; $\phi V_n = \phi_v \times V_n =$ **101.77** kips

$V_{u.y} / \phi V_n =$ **0.520**

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement; $d_{v2} =$ **20.125** in

Shear perimeter length (22.6.4); $l_{xp} =$ **36.125** in

Shear perimeter width (22.6.4); $l_{yp} =$ **36.125** in

Shear perimeter (22.6.4); $b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2}) =$ **144.500** in

Shear area; $A_p = l_{x,perim} \times l_{y,perim} =$ **1305.016** in^2

Surcharge loaded area; $A_{sur} = A_p - l_{x1} \times l_{y1} =$ **1049.016** in^2

Ultimate bearing pressure at center of shear area; $q_{up.avg} =$ **4.672** ksf

Ultimate shear load; $F_{up} = \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} + \gamma_D \times A_{sur} \times F_{Dsur} + \gamma_L \times A_{sur} \times F_{Lsur} - q_{up.avg} \times A_p =$ **213.877** kips

Ultimate shear stress from vertical load; $v_{ug} = \max(F_{up} / (b_o \square d_{v2}), 0 \text{ psi}) =$ **73.546** psi

Column geometry factor (Table 22.6.5.2); $\beta = l_{y1} / l_{x1} =$ **1.00**

Column location factor (22.6.5.3); $\alpha_s =$ **40**

Size effect factor (22.5.5.1.3); $\lambda_s = $ **1**

Concrete shear strength (22.6.5.2); $v_{cpa} = (2 + 4 / \beta) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} =$ **379.473** psi

$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} =$ **478.828** psi

$v_{cpc} = 4 \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} =$ **252.982** psi

$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) =$ **252.982** psi

Shear strength reduction factor; $\phi_v =$ **0.75**

Nominal shear stress capacity (Eq. 22.6.1.2); $v_n = v_{cp} =$ **252.982** psi

Design shear stress capacity (8.5.1.1(d)); $\phi v_n = \phi_v \times v_n =$ **189.737** psi

$v_{ug} / \phi v_n =$ **0.388**

PASS - Design shear stress capacity exceeds ultimate shear stress load

5. Sample Continuous Reinforced Concrete Strip Footing

Footing Analysis

[In accordance with ACI318-19]

Summary results

Description	**Unit**	**Applied**	**Resisting**	**FoS**	**Result**
Uplift verification	kips	6.9			Pass
Description	**Unit**	**Applied**	**Resisting**	**Utilization**	**Result**
Soil bearing	ksf	3.435	4	0.859	Pass
Description	**Unit**	**Provided**	**Required**	**Utilization**	**Result**
Moment, positive, y-direction	kip_ft	0.6	11.8	0.052	Pass
Min.area of reinf, bot., y-direction	in^2	0.259	0.310		Pass
Max.reinf.spacing, bot, y-direction	in	18.0	12.0		Pass

Strip footing details - considering a one meter strip

Length of footing; L_x = **1** ft

Width of footing; L_y = **2** ft

Footing area; $A = L_x \times L_y$ = **2** ft^2

Depth of footing; h = **12** in

Depth of soil over footing; h_{soil} = **3.5** in

Density of concrete; γ_{conc} = **150.0** lb/ft^3

Wall no.1 details

Width of wall; l_{y1} = **12** in

position in y-axis; y_1 = **12** in

Soil Properties

Gross allowable bearing pressure; q_{allow_Gross} = **4** ksf;

Density of soil; γ_{soil} = **120.0** lb/ft^3

Angle of internal friction; ϕ_b = **30.0** deg

Design base friction angle; δ_{bb} = **30.0** deg

Coefficient of base friction; $\tan(\delta_{bb})$ = **0.577**

Self weight; $F_{swt} = h \times \gamma_{conc}$ = **150** psf

Soil weight; $F_{soil} = h_{soil} \times \gamma_{soil}$ = **35** psf

Wall no.1 loads per linear foot

Dead load in z; F_{Dz1} = **2.0** kips

Live load in z; F_{Lz1} = **4.0** kips

Snow load in z; F_{Sz1} = **2.0** kips

Footing analysis for soil and stability

Load combinations per ASCE 7-10

1.0D (0.296)

1.0D + 1.0L (0.796)

1.0D + 1.0Lr (0.296)

1.0D + 1.0S (0.546)

1.0D + 1.0R (0.296)

1.0D + 0.75L + 0.75Lr (0.671)

1.0D + 0.75L + 0.75S (0.859)

1.0D + 0.75L + 0.75R (0.671)

Combination 7 results: 1.0D + 0.75L + 0.75S

Forces on footing per linear foot

Force in z-axis; $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1} + \gamma_S \times F_{Sz1}$ = **6.9** kips

Moments on footing per linear foot

Moment in y-axis, about y is 0; $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) + \gamma_S \times (F_{Sz1} \times y_1)$ = **6.9** kip_ft

Uplift verification

Vertical force; F_{dz} = **6.87** kips

PASS - Footing is not subject to uplift

Stability against sliding

Resistance due to base friction; $F_{RFriction} = \max(F_{dz}, 0\ kN) \times \tan(\delta_{bb})$ = **3.966** kips

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in y-axis; $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **0.000** in

Strip base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dy} / L_y) / (L_y \times 1\ ft)$ = **3.435** ksf

$q_2 = F_{dz} \times (1 + 6 \times e_{dy} / L_y) / (L_y \times 1\ ft)$ = **3.435** ksf

Minimum base pressure; $q_{min} = \min(q_1, q_2)$ = **3.435** ksf

Maximum base pressure; $q_{max} = \max(q_1, q_2)$ = **3.435** ksf

Allowable bearing capacity

Allowable bearing capacity; $q_{allow} = q_{allow\_Gross}$ = **4** ksf

q_{max} / q_{allow} = **0.859**

PASS - Allowable bearing capacity exceeds design base pressure

Footing Design

[In accordance with ACI318-19]

Material details

Compressive strength of concrete;	f'_c = **4000** psi
Yield strength of reinforcement;	f_y = **60000** psi
Compression-controlled strain limit (21.2.2);	ε_{ty} = **0.00200**
Cover to top of footing;	c_{nom_t} = **3** in
Cover to side of footing;	c_{nom_s} = **3** in
Cover to bottom of footing;	c_{nom_b} = **3** in
Concrete type;	Normal weight
Concrete modification factor;	λ = **1.00**
Wall type;	Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-10

1.4D (0.015)

1.2D + 1.6L + 0.5Lr (0.047)

1.2D + 1.6L + 0.5S (0.052)

1.2D + 1.6L + 0.5R (0.047)

1.2D + 1.0L + 1.6Lr (0.034)

1.2D + 1.0L + 1.6S (0.051)

1.2D + 1.0L + 1.6R (0.034)

Combination 3 results: 1.2D + 1.6L + 0.5S

Forces on footing per linear foot

Ultimate force in z-axis; $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \square_D \times F_{Dz1} + \square_L \times F_{Lz1} + \square_S \times F_{Sz1}$ = **10.2** kips

Moments on footing per linear foot

Ultimate moment in y-axis, about y is 0; $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) + \gamma_S \times (F_{Sz1} \times y_1)$ = **10.2** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in y-axis; $e_{uy} = M_{uy} / F_{uz} - L_y / 2$ = **0.000** in

Strip base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{uy} / L_y) / (L_y \times 1\ ft)$ = **5.122** ksf

$q_{u2} = F_{uz} \times (1 + 6 \times e_{uy} / L_y) / (L_y \times 1\ ft)$ = **5.122** ksf

Minimum ultimate base pressure; $q_{umin} = \min(q_{u1}, q_{u2})$ = **5.122** ksf

Maximum ultimate base pressure; $q_{umax} = \max(q_{u1}, q_{u2})$ = **5.122** ksf

Moment design, y direction, positive moment

Ultimate bending moment; $M_{u.y.max}$ = **0.612** kip_ft

Tension reinforcement provided; No.5 bars at 12.0 in c/c bottom

Area of tension reinforcement provided; $A_{sy.bot.prov}$ = **0.31** in^2

Minimum area of reinforcement (7.6.1.1); $A_{s.min} = 0.0018 \times L_x \times h$ = **0.259** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (7.7.2.3); s_{max} = min(3 × h, 18 in) = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement; $d = h - c_{nom\_b} - \phi_{y.bot} / 2$ = **8.688** in

Depth of compression block; $a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **0.456** in

Neutral axis factor; β_1 = **0.85**

Depth to neutral axis; $c = a / \beta_1$ = **0.536** in

Strain in tensile reinforcement; $\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.04559**

Minimum tensile strain(7.3.3.1); $\varepsilon_{min} = \varepsilon_{ty} + 0.003$ = **0.00500**

PASS - Tensile strain exceeds minimum required

Nominal moment capacity; $M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2)$ = **13.112** kip_ft

Flexural strength reduction factor; $\phi_f = \min(\max(0.65 + 0.25 \times (\varepsilon_t - \varepsilon_{ty}) / (0.003), 0.65), 0.9)$ = **0.900**

Design moment capacity; $\phi M_n = \phi_f \times M_n$ = **11.801** kip_ft

$M_{u.y.max} / \phi M_n$ = **0.052**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

One-way shear design does not apply. Shear failure plane fall outside extents of foundation.