

GENERAL REQUIREMENTS

UPDATED STRUCTURAL CALCULATIONS

6C.3.1 - 08



Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 1	
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Project Name:

Northeast Metropolitan Regional Vocational High School

MSBA Module 6 Requirements:

MSBA 90% Construction Documents – Structural Loading Calculations

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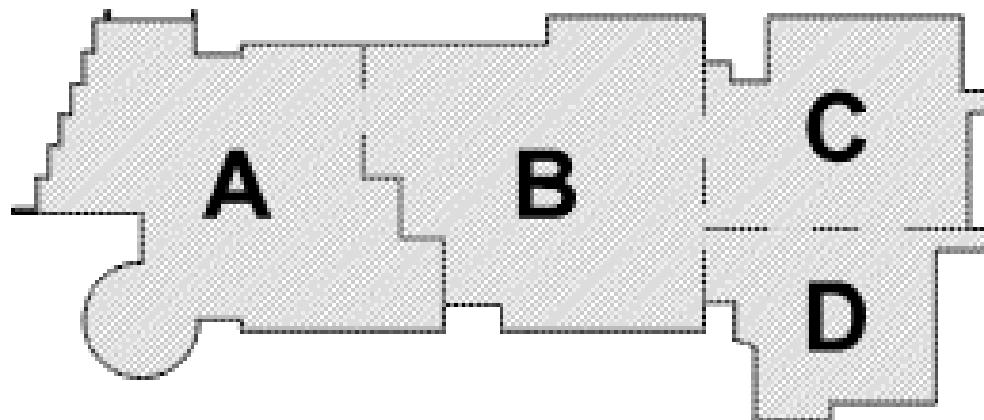
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Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 2	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

Table of Contents

Project Synopsis	3
Design Codes	4
Geotechnical Recommendations for Foundation Analysis and Design	4
Project Materials and Strengths	4
Dead and Live Loading Criteria	5
Snow Loading Criteria	6
Wind Loading Criteria	7
Seismic Loading Calculations	16
Sample Gravity Analysis and Design Calculations	19





Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 3	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

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 academic rooms, Area B is the main academic 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 maintenance 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 suspended 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 maintenance 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.



Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 4	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

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



Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 5	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

Dead and Live Loading Criteria

Design Dead Loads:

Typical Floor Loading on Composite Deck:

5 1/4" Light-Weight Concrete	42 psf
2" x 20-Gauge Composite Steel Deck	3 psf
Mechanical/ Electrical/ Plumbing	10 psf
Miscellaneous	5 psf
	Σ60 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
	Σ35 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
	Σ80 psf

Design Live Loads:

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



Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 6	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

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; $P_g = \mathbf{50.00}$ lb/ft²
Density of snow (Figure 7-1); $\gamma = \min(0.13 \times P_g / 1 \text{ ft} + 14 \text{ lb/ft}^3, 30 \text{ lb/ft}^3) = \mathbf{20.50}$ lb/ft³
Terrain typeSect. 26.7; B
Exposure condition (Table 7-2); Partially exposed
Exposure factor (Table 7-2); $C_e = \mathbf{1.00}$
Thermal condition (Table 7-3); All
Thermal factor (Table 7-3); $C_t = \mathbf{1.00}$
Importance category (Table 1.5-1); III
Importance factor (Table 1.5-2); $I_s = \mathbf{1.10}$
Min snow load for low slope roofs (Sect 7.3.4); $P_{f, \min} = I_s \times 20 \text{ lb/ft}^2 = \mathbf{22.00}$ lb/ft²
Flat roof snow load (Sect 7.3); $P_f = 0.7 \times C_e \times C_t \times I_s \times P_g = \mathbf{38.50}$ lb/ft²



Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 7	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

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);	G _{C_{pi}p} = 0.18
Internal pressure coef -ve (Table 26.11-1);	G _{C_{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
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Pressures and forces

Net pressure;	$p = q \times G_f \times C_{pe} - q_i \times G_{C_{pi}}$;
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Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 8	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

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;

$$A_{vert_w_0} = b \times H = 29140.00 \text{ ft}^2$$

Projected vertical area of roof;

$$A_{vert_r_0} = 0.00 \text{ ft}^2$$

Minimum overall horizontal loading;

$$F_{w,total_min} = p_{min_w} \times A_{vert_w_0} + p_{min_r} \times A_{vert_r_0} = 466.24 \text{ kips}$$

Leeward net force;

$$F_l = F_{w,wB} = -819.4 \text{ kips}$$

Windward net force;

$$F_w = F_{w,wA_1} + F_{w,wA_2} + F_{w,wA_3} + F_{w,wA_4} = 575.6 \text{ kips}$$

Overall horizontal loading;

$$F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total_min}) = 1394.9 \text{ kips}$$

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



Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 9	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

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;

$$A_{vert_w_0} = b \times H = 29140.00 \text{ ft}^2$$

Projected vertical area of roof;

$$A_{vert_r_0} = 0.00 \text{ ft}^2$$

Minimum overall horizontal loading;

$$F_{w,total_min} = p_{min_w} \times A_{vert_w_0} + p_{min_r} \times A_{vert_r_0} = 466.24 \text{ kips}$$

Leeward net force;

$$F_l = F_{w,wB} = -331.8 \text{ kips}$$

Windward net force;

$$F_w = F_{w,wA_1} + F_{w,wA_2} + F_{w,wA_3} + F_{w,wA_4} = 1063.1 \text{ kips}$$

Overall horizontal loading;

$$F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total_min}) = 1394.9 \text{ kips}$$

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



Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 10	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

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;

$$A_{vert_w_90} = d \times H = 12400.00 \text{ ft}^2$$

Projected vertical area of roof;

$$A_{vert_r_90} = 0.00 \text{ ft}^2$$

Minimum overall horizontal loading;

$$F_{w,total_min} = p_{min_w} \times A_{vert_w_90} + p_{min_r} \times A_{vert_r_90} = 198.40 \text{ kips}$$

Leeward net force;

$$F_l = F_{w,wb} = -242.1 \text{ kips}$$

Windward net force;

$$F_w = F_{w,WA_1} + F_{w,WA_2} + F_{w,WA_3} + F_{w,WA_4} = 244.9 \text{ kips}$$

Overall horizontal loading;

$$F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total_min}) = 487.0 \text{ kips}$$

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



Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 11	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

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;

$$A_{\text{vert}_w_{90}} = d \times H = \mathbf{12400.00 \text{ ft}^2}$$

Projected vertical area of roof;

$$A_{\text{vert}_r_{90}} = \mathbf{0.00 \text{ ft}^2}$$

Minimum overall horizontal loading;

$$F_{w,\text{total_min}} = p_{\text{min}_w} \times A_{\text{vert}_w_{90}} + p_{\text{min}_r} \times A_{\text{vert}_r_{90}} = \mathbf{198.40 \text{ kips}}$$

Leeward net force;

$$F_l = F_{w,wB} = \mathbf{-34.7 \text{ kips}}$$

Windward net force;

$$F_w = F_{w,wA_1} + F_{w,wA_2} + F_{w,wA_3} + F_{w,wA_4} = \mathbf{452.4 \text{ kips}}$$

Overall horizontal loading;

$$F_{w,\text{total}} = \max(F_w - F_l + F_{w,h}, F_{w,\text{total_min}}) = \mathbf{487.0 \text{ kips}}$$

Areas C and D

[In accordance with ASCE 7-10]

*Using the directional design method

Building data

Type of roof;

Flat

Length of building;

$b = \mathbf{285.00 \text{ ft}}$

Width of building;

$d = \mathbf{200.00 \text{ ft}}$

Height to eaves;

$H = \mathbf{82.00 \text{ ft}}$

Mean height;

$h = \mathbf{82.00 \text{ ft}}$

General wind load requirements

Basic wind speed;

$V = \mathbf{137.0 \text{ mph}}$

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;

$G_f = \mathbf{0.85}$



Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 12	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

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

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
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.); $q_i = 49.66 \text{ psf}$

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; $F_{w,v} = -2020.62 \text{ kips}$

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

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



Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 13	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

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;

$$A_{vert_w_0} = b \times H = \mathbf{23370.00 \text{ ft}^2}$$

Projected vertical area of roof;

$$A_{vert_r_0} = \mathbf{0.00 \text{ ft}^2}$$

Minimum overall horizontal loading;

$$F_{w,total_min} = p_{min_w} \times A_{vert_w_0} + p_{min_r} \times A_{vert_r_0} = \mathbf{373.92 \text{ kips}}$$

Leeward net force;

$$F_l = F_{w,WB} = \mathbf{-702.2 \text{ kips}}$$

Windward net force;

$$F_w = F_{w,WA_1} + F_{w,WA_2} + F_{w,WA_3} + F_{w,WA_4} = \mathbf{488.4 \text{ kips}}$$

Overall horizontal loading;

$$F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total_min}) = \mathbf{1190.6 \text{ kips}}$$

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;

$$F_{w,v} = \mathbf{76.43 \text{ kips}}$$

Total horizontal net force;

$$F_{w,h} = \mathbf{0.00 \text{ kips}}$$

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



Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 14	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

Overall loading

Projected vertical plan area of wall;

$$A_{\text{vert}_w_0} = b \times H = 23370.00 \text{ ft}^2$$

Projected vertical area of roof;

$$A_{\text{vert}_r_0} = 0.00 \text{ ft}^2$$

Minimum overall horizontal loading;

$$F_{w,\text{total_min}} = p_{\text{min}_w} \times A_{\text{vert}_w_0} + p_{\text{min}_r} \times A_{\text{vert}_r_0} = 373.92 \text{ kips}$$

Leeward net force;

$$F_l = F_{w,wB} = -284.4 \text{ kips}$$

Windward net force;

$$F_w = F_{w,wA_1} + F_{w,wA_2} + F_{w,wA_3} + F_{w,wA_4} = 906.2 \text{ kips}$$

Overall horizontal loading;

$$F_{w,\text{total}} = \max(F_w - F_l + F_{w,h}, F_{w,\text{total_min}}) = 1190.6 \text{ kips}$$

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;

$$F_{w,v} = -1785.24 \text{ kips}$$

Total horizontal net force;

$$F_{w,h} = 0.00 \text{ kips}$$

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;

$$A_{\text{vert}_w_{90}} = d \times H = 16400.00 \text{ ft}^2$$

Projected vertical area of roof;

$$A_{\text{vert}_r_{90}} = 0.00 \text{ ft}^2$$

Minimum overall horizontal loading;

$$F_{w,\text{total_min}} = p_{\text{min}_w} \times A_{\text{vert}_w_{90}} + p_{\text{min}_r} \times A_{\text{vert}_r_{90}} = 262.40 \text{ kips}$$

Leeward net force;

$$F_l = F_{w,wB} = -433.9 \text{ kips}$$



Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 15	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

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; $A_{vert_w_90} = d \times H = \mathbf{16400.00}$ ft²
Projected vertical area of roof; $A_{vert_r_90} = \mathbf{0.00}$ ft²
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



Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 16	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

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 coefficient at 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
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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); $T_a = C_t \times (h_n)^x \times 1 \text{ sec} / (1 \text{ ft})^x = 0.442 \text{ sec}$

Building fundamental period (Sect 12.8.2); $T = T_a = 0.442 \text{ sec}$

Long-period transition period; $T_L = 12 \text{ sec}$

Seismic response coefficient

Seismic force-resisting system (Table 12.2-1); B_BUILDING_FRAME_SYSTEMS
3. Ordinary steel concentrically braced frames



Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 17	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

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);	$C_{s_calc} = S_{DS} / (R / I_e) = \mathbf{0.1026}$
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 = 0.1026
Seismic base shear (Sect 12.8.1)	
Effective seismic weight of the structure;	W = 19660.0 kips
Seismic response coefficient;	C_s = 0.1026
Seismic base shear (Eq 12.8-1);	V = C_s × W = 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 = 0.25
at 1 sec period;	S₁ = 0.08
Site coefficient at 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 × S_S = 0.400
at 1 sec period (Eq. 11.4-2);	S_{M1} = F_v × S₁ = 0.192

Design spectral acceleration parameters (Sect 11.4.4)

at short period (Eq. 11.4-3);	S_{DS} = 2 / 3 × S_{MS} = 0.267
at 1 sec period (Eq. 11.4-4);	S_{D1} = 2 / 3 × 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 = 82 ft
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Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 18	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

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); $T_a = C_t \times (h_n)^x \times 1 \text{ sec} / (1 \text{ ft})^x = 0.545 \text{ sec}$

Building fundamental period (Sect 12.8.2); $T = T_a = 0.545 \text{ sec}$

Long-period transition period; $T_L = 12 \text{ sec}$

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; $W = 9390.0 \text{ kips}$

Seismic response coefficient; $C_s = 0.0903$

Seismic base shear (Eq 12.8-1); $V = C_s \times W = 848.2 \text{ kips}$



Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 19	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

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 $\times 1$ Dead full UDL 0.35 kips/ft Snow full UDL 0.4 kips/ft Roof Live full UDL 0.2 kips/ft
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Load combinations

Load combination 1 - Full	Support A	Dead $\times 1.20$ Live $\times 1.60$ Snow $\times 1.60$ Roof Live $\times 1.60$ Dead $\times 1.20$ Live $\times 1.60$ Snow $\times 1.60$ Roof Live $\times 1.60$
	Support B	Dead $\times 1.20$ Live $\times 1.60$ Snow $\times 1.60$ Roof Live $\times 1.60$

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;	$\delta_{\max} = 1$ in;	$\delta_{\min} = 0$ in
Maximum reaction at support A;	$R_{A_{\max}} = 26$ kips;	$R_{A_{\min}} = 26$ kips
Unfactored dead load reaction at support A;	$R_{A_{\text{Dead}}} = 7.3$ kips	
Unfactored snow load reaction at support A;	$R_{A_{\text{Snow}}} = 7.2$ kips	
Unfactored roof live load reaction at support A;	$R_{A_{\text{Roof Live}}} = 3.6$ kips	
Maximum reaction at support B;	$R_{B_{\max}} = 26$ kips;	$R_{B_{\min}} = 26$ kips
Unfactored dead load reaction at support B;	$R_{B_{\text{Dead}}} = 7.3$ kips	
Unfactored snow load reaction at support B;	$R_{B_{\text{Snow}}} = 7.2$ kips	



Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 20	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

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 $\phi_{ty} = 0.90$
 Resistance factor for tensile rupture $\phi_{tr} = 0.75$
 Resistance factor for compression $\phi_c = 0.90$
 Resistance factor for flexure $\phi_b = 0.90$

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 $A_w = d \times t_w = 9.322$ in²
 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 $\phi_v = 0.90$
 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



Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 21	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

Yielding - Section F2.1

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

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

Design flexural strength; $M_c = \phi_b \times M_n = 502.500 \text{ kips_ft}$

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

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

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]

Design summary

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; $L = 36.000 \text{ ft}$

Beam spacing on one side; $b_1 = 10.000 \text{ ft}$

Beam spacing on other side; $b_2 = 10.000 \text{ ft}$

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

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

Overall depth of slab; $t = 5.250 \text{ in}$

Height of ribs; $h_r = 2.000 \text{ in}$

Centers of ribs; $\text{rib}_{ccs} = 12.000 \text{ in}$

Average width of rib; $w_r = 7.000 \text{ in}$



Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 22	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

Material properties

Concrete

Specified compressive strength of concrete; $f'_c = 4.00$ ksi
Wet density of concrete; $w_{cw} = 125$ lb/ft³
Dry density of concrete; $w_{cd} = 115$ lb/ft³
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; $Z_x = 134.00$ in³
Elastic modulus of steel section; $S_x = 114.00$ in³
Width to thickness ratio; $\lambda_f = b_f / (2 \times t_f) = 6.941$



Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 23	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

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

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

Flange is compact

Depth to thickness ratio (h/t_w); $\lambda_w = \mathbf{54.600}$

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

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

Web is compact

Strength check at construction stage for flexure

Check for flexure

Plastic moment for steel section; $M_p = F_y \times Z_x = \mathbf{558.333 \text{ kip_ft}}$

Resistance factor for flexure; $\phi_b = \mathbf{0.90}$

Design flexural strength of steel section alone; $M_{constr_n} = \phi_b \times M_p = \mathbf{502.500 \text{ kip_ft}}$

Required flexural strength; $M_{constr_u} = \mathbf{157.801 \text{ kip_ft}}$

PASS - Beam bending at construction stage loading

Strength check at construction stage for shear

Web area; $A_w = d \times t_w = \mathbf{9.322 \text{ in}^2}$

Web plate buckling coefficient; $k_v = \mathbf{5.34}$

Depth to thickness ratio (h/t_w); $\lambda_w = \mathbf{54.600}$

Web shear coefficient; $C_{v1} = \mathbf{1.00}$

Resistant factor for shear; $\phi_v = \mathbf{0.9}$

Design shear strength; $V_{constr_n} = \phi_v \times (0.6 \times F_y \times A_w \times C_{v1}) = \mathbf{251.694 \text{ kips}}$

Required shear strength; $V_{constr_u} = \mathbf{17.534 \text{ kips}}$

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; $b = \min(L/8, b_1/2) + \min(L/8, b_2/2) = \mathbf{108.000 \text{ in}}$

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

Diameter of stud anchor; $\text{dia} = \mathbf{0.750 \text{ in}}$

Length of stud anchor after weld; $H_s = \mathbf{3.50 \text{ in}}$

Specified tensile strength of stud anchor; $F_u = \mathbf{65 \text{ ksi}}$

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

Maximum diameter permitted; $\text{dia}_{\max} = 2.5 \times t_f = \mathbf{1.263 \text{ in}}$

PASS - Diameter of stud anchor provided is OK

Point of max. B.M. from nearest support; $L_{BM_near} = \mathbf{18.00 \text{ ft}}$

No. of ribs from points of zero to max moment; $\text{rib}_{\text{numbers}} = \text{int}(L_{BM_near} / \text{rib}_{\text{ccs}} - 1) = \mathbf{17}$

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

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

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

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

Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 24	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

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-hi} 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

Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 25	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

Required shear strength;

$$V_{comp_u} = 43.019 \text{ kips}$$

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; $\Delta_{short_D} = 5 \times w_{constr_D} \times L^4 / (384 \times E_s \times I_x) = 0.5262 \text{ in}$

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

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

Deflection due to construction live load; $\Delta_2 = 5 \times w_{constr_L} \times L^4 / (384 \times E_s \times I_x) = 0.1931 \text{ in}$

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

For short term loading:-

Short term modular ratio; $n_s = E_s / E_c = 11.8$

Depth of neutral axis from top of concrete;

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

$$y_s = 7.051 \text{ in}$$

Moment of inertia of fully composite section;

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

$$I_s = 3875 \text{ in}^4$$

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

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

Deflection due to short term live load; $\Delta_{L_s} = 5 \times r_{L_s} \times w_{comp_L} \times L^4 / (384 \times E_s \times I_{s_eff}) = 0.4059 \text{ in}$

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;

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

$$y_l = 9.653 \text{ in}$$

Moment of inertia of fully composite section;

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

$$I_l = 3212 \text{ in}^4$$

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

Proportion of live load which is long term; $r_{L_l} = 1 - r_{L_s} = 33 \%$

Deflection due to long term live load; $\Delta_{L_l} = 5 \times r_{L_l} \times w_{comp_L} \times L^4 / (384 \times E_s \times I_{l_eff}) = 0.2322 \text{ in}$

Dead load due to parallel wall & superimp. dead; $w_{D_part} = w_{w_par} + (w_{serv} \times (b_1 + b_2) / 2) = 150.0000 \text{ lb/ft}$

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

Wall parallel to span and superimposed dead; $\Delta_4 = 5 \times (w_{D_part}) \times L^4 / (384 \times E_s \times I_{l_eff}) = 0.1055 \text{ in}$

Wall perpendicular to span; $\Delta_5 = (w_{w_perp} \times (b_1 + b_2) / 2) \times L^3 / (48 \times E_s \times I_{l_eff}) = 0.0000 \text{ in}$

Combined deflections

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

Net total long term deflection; $\Delta_{long} = \Delta_{short_D} + \Delta_{L_s} + \Delta_{L_l} + \Delta_4 + \Delta_5 - \Delta_{camber} = 1.270 \text{ in}$



Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 26	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

Combined short and long term live load deflectn; $\Delta_{live} = \Delta_{L_s} + \Delta_{L_l} = \mathbf{0.638}$ in
 Net long term dead and super imposed dead defln; $\Delta_{dead} = \Delta_{short_D} + \Delta_4 + \Delta_5 - \Delta_{camber} = \mathbf{0.632}$ in
 Post composite deflection; $\Delta_{comp} = \Delta_{L_s} + \Delta_{L_l} + \Delta_4 + \Delta_5 = \mathbf{0.744}$ in
 Allowable max deflection; $\Delta_{Allow} = \mathbf{1.500}$ 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]

Column and loading details

Column details

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

Design loading

Required axial strength; $P_r = \mathbf{250}$ kips; (Compression)

Moment about x axis at end 1; $M_{x1} = \mathbf{0.0}$ kips_ft

Moment about x axis at end 2; $M_{x2} = \mathbf{0.0}$ kips_ft

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

Moment about y axis at end 1; $M_{y1} = \mathbf{0.0}$ kips_ft

Moment about y axis at end 2; $M_{y2} = \mathbf{0.0}$ kips_ft

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

Maximum shear force parallel to y axis; $V_{ry} = \mathbf{0.0}$ kips

Maximum shear force parallel to x axis; $V_{rx} = \mathbf{0.0}$ kips

Material details

Steel grade; **A500 Gr. C**

Yield strength; $F_y = \mathbf{50}$ ksi

Ultimate strength; $F_u = \mathbf{62}$ ksi

Modulus of elasticity; $E = \mathbf{29000}$ ksi

Shear modulus of elasticity; $G = \mathbf{11200}$ ksi

Unbraced lengths

For buckling about x axis; $L_x = \mathbf{240}$ in

For buckling about y axis; $L_y = \mathbf{240}$ in

For torsional buckling; $L_z = \mathbf{240}$ in



Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 27	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

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; $F_{crx} = Q_x \times (0.658^{Q_x \times F_y / F_{ex}}) \times F_y = 41.4$ 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; $F_{cry} = Q_y \times (0.658^{Q_y \times F_y / F_{ey}}) \times F_y = 41.4$ ksi



Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 28	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

Nominal flexural buckling strength;

$$P_{ny} = F_{cry} \times A_g = \mathbf{662.7 \text{ kips}}$$

Design compressive strength (cl.E1)

Resistance factor for compression;

$$\phi_c = \mathbf{0.90}$$

Design compressive strength;

$$P_c = \phi_c \times \min(P_{nx}, P_{ny}) = \mathbf{596.5 \text{ kips}}$$

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;

$$L_x = \mathbf{8 \text{ ft}}$$

Width of footing;

$$L_y = \mathbf{8 \text{ ft}}$$

Footing area;

$$A = L_x \times L_y = \mathbf{64 \text{ ft}^2}$$

Depth of footing;

$$h = \mathbf{24 \text{ in}}$$

Depth of soil over footing;

$$h_{soil} = \mathbf{18 \text{ in}}$$

Density of concrete;

$$\gamma_{conc} = \mathbf{150.0 \text{ lb/ft}^3}$$



Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 29	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

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; $\gamma_{soil} = 120.0$ lb/ft³
Angle of internal friction; $\phi_b = 30.0$ deg
Design base friction angle; $\delta_{bb} = 30.0$ deg
Coefficient of base friction; $\tan(\delta_{bb}) = 0.577$
Design wall friction angle; $\delta_b = 15.0$ 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



Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 30	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

Moments on footing

Moment in x-axis, about x is 0;

$$M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil} + F_{Dsur}) \times L_x / 2) + \gamma_L \times A \times F_{Lsur} \times L_x / 2 + \gamma_D \times (F_{Dz1} \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) + \gamma_S \times (F_{Sz1} \times x_1) = \mathbf{973.5 \text{ kip_ft}}$$

Moment in y-axis, about y is 0;

$$M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil} + F_{Dsur}) \times L_y / 2) + \gamma_L \times A \times F_{Lsur} \times L_y / 2 + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) + \gamma_S \times (F_{Sz1} \times y_1) = \mathbf{973.5 \text{ kip_ft}}$$

Uplift verification

Vertical force;

$$F_{dz} = \mathbf{243.37 \text{ kips}}$$

PASS - Footing is not subject to uplift

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis;

$$e_{dx} = M_{dx} / F_{dz} - L_x / 2 = \mathbf{0 \text{ in}}$$

Eccentricity of base reaction in y-axis;

$$e_{dy} = M_{dy} / F_{dz} - L_y / 2 = \mathbf{0 \text{ in}}$$

Pad base pressures

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

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

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

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

Minimum base pressure;

$$q_{min} = \min(q_1, q_2, q_3, q_4) = \mathbf{3.803 \text{ ksf}}$$

Maximum base pressure;

$$q_{max} = \max(q_1, q_2, q_3, q_4) = \mathbf{3.803 \text{ ksf}}$$

Allowable Bearing Capacity

Allowable bearing capacity;

$$q_{allow} = q_{allow_Gross} = \mathbf{4 \text{ ksf}}$$

$$q_{max} / q_{allow} = \mathbf{0.951}$$

PASS - Allowable bearing capacity exceeds design base pressure

Footing Design

[In accordance with ACI318-19]

Material details

Compressive strength of concrete;

$$f'_c = \mathbf{4000 \text{ psi}}$$

Yield strength of reinforcement;

$$f_y = \mathbf{60000 \text{ psi}}$$

Compression-controlled strain limit (21.2.2);

$$\epsilon_{ty} = \mathbf{0.00200}$$

Cover to top of footing;

$$c_{nom_t} = \mathbf{3 \text{ in}}$$

Cover to side of footing;

$$c_{nom_s} = \mathbf{3 \text{ in}}$$

Cover to bottom of footing;

$$c_{nom_b} = \mathbf{3 \text{ in}}$$

Concrete type;

Normal weight

Concrete modification factor;

$$\lambda = \mathbf{1.00}$$

Column type;

Concrete



Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 31	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

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;

$$F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil} + F_{Dsur}) + \gamma_L \times A \times F_{Lsur} + \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1} = \mathbf{299.0 \text{ kips}}$$

Moments on footing

Ultimate moment in x-axis, about x is 0;

$$M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil} + F_{Dsur}) \times L_x / 2) + \gamma_L \times A \times F_{Lsur} \times L_x / 2 + \gamma_D \times (F_{Dz1} \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) = \mathbf{1196.1 \text{ kip_ft}}$$

Ultimate moment in y-axis, about y is 0;

$$M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil} + F_{Dsur}) \times L_y / 2) + \gamma_L \times A \times F_{Lsur} \times L_y / 2 + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) = \mathbf{1196.1 \text{ kip_ft}}$$

Eccentricity of base reaction

Eccentricity of base reaction in x-axis;

$$e_{ux} = M_{ux} / F_{uz} - L_x / 2 = \mathbf{0 \text{ in}}$$

Eccentricity of base reaction in y-axis;

$$e_{uy} = M_{uy} / F_{uz} - L_y / 2 = \mathbf{0 \text{ in}}$$

Pad base pressures

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

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

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

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

Minimum ultimate base pressure;

$$q_{umin} = \min(q_{u1}, q_{u2}, q_{u3}, q_{u4}) = \mathbf{4.672 \text{ ksf}}$$

Maximum ultimate base pressure;

$$q_{umax} = \max(q_{u1}, q_{u2}, q_{u3}, q_{u4}) = \mathbf{4.672 \text{ ksf}}$$

Moment design, x direction, positive moment

Ultimate bending moment;

$$M_{u.x.max} = \mathbf{173.679 \text{ kip_ft}}$$

Tension reinforcement provided;

$$9 \text{ No.7 bottom bars (11.1 in c/c)}$$

Area of tension reinforcement provided;

$$A_{sx.bot.prov} = \mathbf{5.4 \text{ in}^2}$$

Minimum area of reinforcement (8.6.1.1);

$$A_{s.min} = 0.0018 \times L_y \times h = \mathbf{4.147 \text{ in}^2}$$

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2);

$$s_{max} = \min(2 \times h, 18 \text{ in}) = \mathbf{18 \text{ in}}$$

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement;

$$d = h - c_{nom_b} - \phi_{x.bot} / 2 = \mathbf{20.562 \text{ in}}$$

Depth of compression block;

$$a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y) = \mathbf{0.993 \text{ in}}$$

Neutral axis factor;

$$\beta_1 = \mathbf{0.85}$$

Depth to neutral axis;

$$c = a / \beta_1 = \mathbf{1.168 \text{ in}}$$

Strain in tensile reinforcement;

$$\epsilon_t = 0.003 \times d / c - 0.003 = \mathbf{0.04982}$$

Minimum tensile strain(8.3.3.1);

$$\epsilon_{min} = \epsilon_{ty} + 0.003 = \mathbf{0.00500}$$

PASS - Tensile strain exceeds minimum required



Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 32	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

Nominal moment capacity;

$$M_n = A_{sx,bot,prov} \times f_y \times (d - a / 2) = \mathbf{541.787 \text{ kip_ft}}$$

Flexural strength reduction factor;

$$\phi_f = \min(\max(0.65 + 0.25 \times (\epsilon_t - \epsilon_{ty}) / (0.003), 0.65), 0.9) = \mathbf{0.900}$$

Design moment capacity;

$$\phi M_n = \phi_f \times M_n = \mathbf{487.608 \text{ kip_ft}}$$

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

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force;

$$V_{u,x} = \mathbf{52.918 \text{ kips}}$$

Depth to reinforcement;

$$d_v = h - c_{nom_b} - \phi_{x,bot} / 2 = \mathbf{20.562 \text{ in}}$$

Size effect factor (22.5.5.1.3);

$$\lambda_s = \mathbf{1}$$

Ratio of longitudinal reinforcement;

$$\rho_w = A_{sx,bot,prov} / (L_y \times d_v) = \mathbf{0.00274}$$

Shear strength reduction factor;

$$\phi_v = \mathbf{0.75}$$

Nominal shear capacity (Eq. 22.5.5.1);

$$V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f_c \times 1 \text{ psi})} \times L_y \times d_v, 5 \times \lambda \times \sqrt{(f_c \times 1 \text{ psi})} \times L_y \times d_v) = \mathbf{139.685 \text{ kips}}$$

Design shear capacity;

$$\phi V_n = \phi_v \times V_n = \mathbf{104.764 \text{ kips}}$$

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

PASS - Design shear capacity exceeds ultimate shear load

Moment design, y direction, positive moment

Ultimate bending moment;

$$M_{u,y,max} = \mathbf{173.679 \text{ kip_ft}}$$

Tension reinforcement provided;

$$9 \text{ No.7 bottom bars (11.1 in c/c)}$$

Area of tension reinforcement provided;

$$A_{sy,bot,prov} = \mathbf{5.4 \text{ in}^2}$$

Minimum area of reinforcement (8.6.1.1);

$$A_{s,min} = 0.0018 \times L_x \times h = \mathbf{4.147 \text{ in}^2}$$

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2);

$$s_{max} = \min(2 \times h, 18 \text{ in}) = \mathbf{18 \text{ in}}$$

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement;

$$d = h - c_{nom_b} - \phi_{x,bot} - \phi_{y,bot} / 2 = \mathbf{19.687 \text{ in}}$$

Depth of compression block;

$$a = A_{sy,bot,prov} \times f_y / (0.85 \times f_c \times L_x) = \mathbf{0.993 \text{ in}}$$

Neutral axis factor;

$$\beta_1 = \mathbf{0.85}$$

Depth to neutral axis;

$$c = a / \beta_1 = \mathbf{1.168 \text{ in}}$$

Strain in tensile reinforcement;

$$\epsilon_t = 0.003 \times d / c - 0.003 = \mathbf{0.04757}$$

Minimum tensile strain(8.3.3.1);

$$\epsilon_{min} = \epsilon_{ty} + 0.003 = \mathbf{0.00500}$$

PASS - Tensile strain exceeds minimum required

Nominal moment capacity;

$$M_n = A_{sy,bot,prov} \times f_y \times (d - a / 2) = \mathbf{518.162 \text{ kip_ft}}$$

Flexural strength reduction factor;

$$\phi_f = \min(\max(0.65 + 0.25 \times (\epsilon_t - \epsilon_{ty}) / (0.003), 0.65), 0.9) = \mathbf{0.900}$$

Design moment capacity;

$$\phi M_n = \phi_f \times M_n = \mathbf{466.346 \text{ kip_ft}}$$

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

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force;

$$V_{u,y} = \mathbf{52.918 \text{ kips}}$$



Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 33	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

Depth to reinforcement;	$d_v = h - c_{nom_b} - \phi_{x.bot} - \phi_{y.bot} / 2 = \mathbf{19.687 \text{ in}}$
Size effect factor (22.5.5.1.3);	$\lambda_s = \mathbf{1}$
Ratio of longitudinal reinforcement;	$\rho_w = A_{sy.bot.prov} / (L_x \times d_v) = \mathbf{0.00286}$
Shear strength reduction factor;	$\phi_v = \mathbf{0.75}$
Nominal shear capacity (Eq. 22.5.5.1);	$V_n = \min(8 \times \lambda_s \times \lambda \times (\rho_w)^{1/3} \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_x \times d_v, 5 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} \times L_x \times d_v) = \mathbf{135.694 \text{ kips}}$
Design shear capacity;	$\phi V_n = \phi_v \times V_n = \mathbf{101.77 \text{ kips}}$ $V_{u.y} / \phi V_n = \mathbf{0.520}$

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement;	$d_{v2} = \mathbf{20.125 \text{ in}}$
Shear perimeter length (22.6.4);	$l_{xp} = \mathbf{36.125 \text{ in}}$
Shear perimeter width (22.6.4);	$l_{yp} = \mathbf{36.125 \text{ in}}$
Shear perimeter (22.6.4);	$b_o = 2 \times (l_{x1} + d_{v2}) + 2 \times (l_{y1} + d_{v2}) = \mathbf{144.500 \text{ in}}$
Shear area;	$A_p = l_{x.perim} \times l_{y.perim} = \mathbf{1305.016 \text{ in}^2}$
Surcharge loaded area;	$A_{sur} = A_p - l_{x1} \times l_{y1} = \mathbf{1049.016 \text{ in}^2}$
Ultimate bearing pressure at center of shear area;	$q_{up.avg} = \mathbf{4.672 \text{ ksf}}$
Ultimate shear load;	$F_{up} = \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} + \gamma_D \times A_{sur} \times F_{Dsur} + \gamma_L \times A_{sur} \times F_{Lsur} - q_{up.avg} \times A_p = \mathbf{213.877 \text{ kips}}$
Ultimate shear stress from vertical load;	$v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0 \text{ psi}) = \mathbf{73.546 \text{ psi}}$
Column geometry factor (Table 22.6.5.2);	$\beta = l_{y1} / l_{x1} = \mathbf{1.00}$
Column location factor (22.6.5.3);	$\alpha_s = \mathbf{40}$
Size effect factor (22.5.5.1.3);	$\lambda_s = \mathbf{1}$
Concrete shear strength (22.6.5.2);	$v_{cpa} = (2 + 4 / \beta) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} = \mathbf{379.473 \text{ psi}}$ $v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} = \mathbf{478.828 \text{ psi}}$ $v_{cpc} = 4 \times \lambda_s \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} = \mathbf{252.982 \text{ psi}}$ $v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) = \mathbf{252.982 \text{ psi}}$
Shear strength reduction factor;	$\phi_v = \mathbf{0.75}$
Nominal shear stress capacity (Eq. 22.6.1.2);	$v_n = v_{cp} = \mathbf{252.982 \text{ psi}}$
Design shear stress capacity (8.5.1.1(d));	$\phi v_n = \phi_v \times v_n = \mathbf{189.737 \text{ psi}}$ $v_{ug} / \phi v_n = \mathbf{0.388}$

PASS - Design shear stress capacity exceeds ultimate shear stress load



Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 34	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

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;	$\gamma_{conc} = 150.0$ 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;	$\gamma_{soil} = 120.0$ lb/ft ³
Angle of internal friction;	$\phi_b = 30.0$ deg
Design base friction angle;	$\delta_{bb} = 30.0$ 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



Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 35	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

Wall no.1 loads per linear foot

Dead load in z;
Live load in z;
Snow load in z;

$F_{Dz1} = 2.0$ kips
 $F_{Lz1} = 4.0$ kips
 $F_{Sz1} = 2.0$ 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;
 $F_{R\text{Friction}} = \max(F_{dz}, 0 \text{ kN}) \times \tan(\delta_{bb}) = 3.966$ 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

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

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

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;
 $q_{\text{allow}} = q_{\text{allow_Gross}} = 4$ ksf

$$q_{\max} / q_{\text{allow}} = 0.859$$



Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 36	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

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);	$\epsilon_{ty} = 0.00200$
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;	$\lambda = 1.00$
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

$q_{u1} = F_{uz} \times (1 - 6 \times e_{uy} / L_y) / (L_y \times 1 \text{ ft}) = 5.122$ ksf
 $q_{u2} = F_{uz} \times (1 + 6 \times e_{uy} / L_y) / (L_y \times 1 \text{ ft}) = 5.122$ 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



Project Northeast Metropolitan Regional Vocational High School				Job Ref. 2019-091	
Section MSBA 90% Construction Document Submission				Sheet no./rev. 37	
Calc. by AA	Date 07/20/2022	Chk'd by MD	Date 07/27/2022	App'd by MD	Date 12/19/2022

Moment design, y direction, positive moment

Ultimate bending moment; $M_{u,y,max} = 0.612 \text{ kip_ft}$
Tension reinforcement provided; No.5 bars at 12.0 in c/c bottom
Area of tension reinforcement provided; $A_{sy,bot,prov} = 0.31 \text{ in}^2$
Minimum area of reinforcement (7.6.1.1); $A_{s,min} = 0.0018 \times L_x \times h = 0.259 \text{ in}^2$

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (7.7.2.3); $s_{max} = \min(3 \times h, 18 \text{ in}) = 18 \text{ in}$

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement; $d = h - c_{nom_b} - \phi_{y,bot} / 2 = 8.688 \text{ in}$
Depth of compression block; $a = A_{sy,bot,prov} \times f_y / (0.85 \times f'_c \times L_x) = 0.456 \text{ in}$
Neutral axis factor; $\beta_1 = 0.85$
Depth to neutral axis; $c = a / \beta_1 = 0.536 \text{ in}$
Strain in tensile reinforcement; $\epsilon_t = 0.003 \times d / c - 0.003 = 0.04559$
Minimum tensile strain(7.3.3.1); $\epsilon_{min} = \epsilon_{ty} + 0.003 = 0.00500$

PASS - Tensile strain exceeds minimum required

Nominal moment capacity; $M_n = A_{sy,bot,prov} \times f_y \times (d - a / 2) = 13.112 \text{ kip_ft}$
Flexural strength reduction factor; $\phi_f = \min(\max(0.65 + 0.25 \times (\epsilon_t - \epsilon_{ty}) / (0.003), 0.65), 0.9) = 0.900$
Design moment capacity; $\phi M_n = \phi_f \times M_n = 11.801 \text{ kip_ft}$
 $M_{u,y,max} / \phi M_n = 0.052$

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.