Structural Engineers 10 Cabot Road, Suite 210 Medford, MA 02155 (781) 396-9007 (781) 396-9008 (FAX) Engineers Design Group Inc.	Project EAST BRIDGEWATER JUNIOR/SENIOR SCHOOL					Job Ref. 2010-059	
	STRUCTURAL CALCULATIONS					Sheet no./rev. 1	
	Calc. by MHP	Date 6/10/2011	Chk'd by CM	Date 6/11/2011		App'd by	Date

Project Name:

EAST BRIDGEWATER
JUNIOR/SENIOR HIGH SCHOOL

Prepared For:

Ai-3 Architects

Prepared By:

Marshall Puffer

Checked By:

Clem McCarey, P.E.

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	EAST BRIDGEWATER JUNIOR/SENIOR SCHOOL				2010-059	
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SHEET INDEX

PROJECT SYNOPSIS AND DESIGN CRITERIA

The project is located at 11 Plymouth Street, East Bridgewater, Massachusetts. The project in question is to be a combined Junior and Senior High School for the community of East Bridgewater. The School will include an academic building and a gym, auditorium and cafeteria building, seperated by an expansion joint. Drawings also include plans for a Waste Water Treatment Plant and a Concession building to be listed as alternates and will not be included in the base bid.

Both buildings will be steel framed supported by isolated concrete footings with concrete slabs on grade and concrete slabs on deck. The roof system will be metal deck. The roof deck is supported by steel beams in the academic building, various low roofs, the band room and the locker rooms. The roof system is supported by steel trusses and infill framing of various depths and configurations at the gym, auditorium and cafeteria.

The lateral resisting system for the academic building will consist primarily of concentrically braced frames with a combined system of masonry shear walls and moment frames at the Metal & Wood Shop exterior wall. The lateral system in the gym/auditorium/cafeteria building is a combination of concentrically braced frames, steel moment frames and masonry shear walls

DESIGN CODES

-Massachussetts State Building Code- Eighth Edition
-ASCE 7-10: Minimum Design Loads for Buildings and Other Structures
-ACI 318-08: Building Code Requirements for Structural Concrete
-ACI 530-08: Building Code Requirements for Masonry Structures
-AISC 360-10: Specification for Structural Steel Buildings
-*Other codes are required by the design codes listed above*

FOUNDATION DESIGN

The foundation design was completed with recommendations from the soils investigation by Pare Corporation located at 10 Lincoln Road Foxboro, Massachusetts. The report found the existing soil to have a maximum net allowable bering pressure of two kips per square foot in the glaciofluvial soil stratum. All footings to have a minimum lateral dimension of 1’-6”. Interior footings are to be a minimum 1’-6” below adjacent slab surface and 4’-0” below grade at exterior conditions.

MATERIALS AND MATERIAL STRENGTH

CONCRETE:	Foundations	3000psi
	Slab-On-Grade	4000psi
	Composite Deck (Light Weight concrete)	3000psi

REINFORCING STEEL:	ASTM A615, Grade 60 ASTM A185 for Welded Wire Fabric
STRUCTURAL STEEL:	ASTM A992, Grade 50
STRUCTURAL CHANNELS:	ASTM A36
STRUCTURAL TUBES:	ASTM A500, Grade B
STRUCTURAL PIPE:	ASTM A53, Grade B or ASTM A501
STEEL PLATES & ANGLES:	ASTM A36
HIGH STRENGTH BOLTS:	ASTM A325-N
METAL DECK:	ASTM A653 (For Galvanized Deck)
CONCRETE MASONRY UNITS:	ASTM C90, Grade N, TYPE I, f'c=2000psi
GROUT:	ASTM C476, f'_c=2500psi
MORTAR:	ASTM C270, Type S, f'_c=1800psi

DEAD AND LIVE LOAD CRITERIA FOR ANALYSIS

DESIGN DEAD LOADS

Typical Composite Floor:

3-1/4” Light Weight Concrete w/ 3” Metal Deck (6-1/4” total thickness)	46psf
20 Gage Composite Metal Deck	3psf
Mechanical/Electrical/Plumbing	5psf
Miscellaneous	5psf
	TOTAL= 59 psf

Running Track Floor

3-1/4” Light Weight Concrete w/ 2” Metal Deck (5-1/4” total thickness)	42psf
20 Gage Composite Metal Deck	3psf
Mechanical/Electrical/Plumbing	5psf
Miscellaneous	5psf
	TOTAL=55psf

Mechanical Roof Pads

4” Normal Weight Concrete w/ 2” Metal Deck (6” total thickness)	63psf
20 Gage Composite Metal Deck	3psf
Mechanical/Electrical/Plumbing (separate from unit weight)	10psf
Miscellaneous	5psf
	TOTAL=81psf

DESIGN DEAD LOADS CONTUNIED…

Typical 3" Steel Roof Deck

3" x 20 Gage Type N Metal Deck	4psf
Roofing and Insulation	5psf
Mechanical/Electrical/Plumbing	5psf
Miscellaneous	5psf
	TOTAL=19psf

Typical 1-1/2" Steel Roof Deck

1-1/2" x 20 Gage Type N Metal Deck	2psf
Roofing and Insulation	5psf
Mechanical/Electrical/Plumbing	5psf
Miscellaneous	5psf
	TOTAL=17psf

MINIMUM DESIGN FLOOR LIVE LOADS *Per ASCE 7-10, Table 4-1*

Classrooms	70 psf	(Reducible)
Corridor	80 psf	(Non-Reducible)
Assembly	100 psf	(Non-Reducible)
Reading Rooms	80 psf	(Reducible)
Open Plan Arears	100 psf	(Non-Reducible)
Stairs	100 psf	(Non-Reducible)
Bathrooms	60 psf	(Non-Reducible)
Storage	125 psf	(Storage)

<u>MINIMUM DESIGN ROOF LIVE LOADS</u> *Per ASCE 7-10, Section 4.9*

Roof	**20 psf**	**(Roof)**

<u>EXTERIOR WALL LOADING</u> *Per ASCE 7-10, Table C3-1*

Solid Masonry Veneer Wall	**48 psf**
Masonry Wall with Window Openings	**40 psf**
Metal Panel/Roof Screen Wall	**15 psf**
Interior Architectural Panel Wall	**15 psf**

SNOW LOAD CRITERIA FOR ANALYSIS

SNOW LOADING (ASCE7-10)

Building details

Roof type — Flat

Ground snow load

Ground snow load — p_g = **45.00** lb/ft^2

Density of snow — γ = min(0.13 × p_g / 1ft + 14lb/ft^3, 30lb/ft^3) = **19.85** lb/ft^3

Terrain type — 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 1-1) — III

Importance factor (Table 7-4) — I_s = **1.10**

Min snow load for low slope roofs (Sect 7.3.4) — $p_{f\_min} = I_s \times 20$ lb/ft^2 = **22.00** lb/ft^2

Flat roof snow load (Sect 7.3) — $p_f = \max(0.7 \times C_e \times C_t \times I_s \times p_g, p_{f\_min})$ = **34.65** lb/ft^2

Drift Calculations Example (Lower Roof over Locker Rooms Areas C and D)

Drift calculations

Balanced snow load height $h_b = p_f / \gamma = \underline{\mathbf{1.75}}$ ft

Length of upper roof $l_u = \underline{\mathbf{104.00}}$ ft

Length of lower roof $l_l = \underline{\mathbf{42.00}}$ ft

Height diff between uppper and lower roofs $h_{diff} = \underline{\mathbf{20.00}}$ ft

Height from balance load to top of upper roof $h_c = h_{diff} - h_b = \underline{\mathbf{18.25}}$ ft

Drift height leeward drift $h_{d\_l} = 0.43 \times (\max(20\text{ ft}, l_u) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft} = \underline{\mathbf{4.01}}$ ft

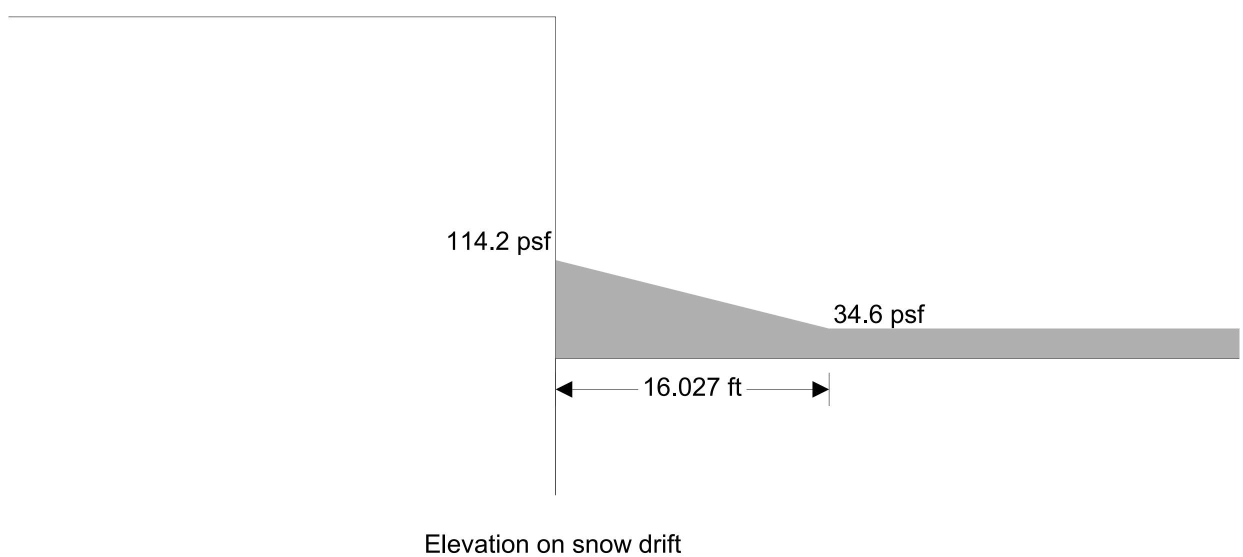
Drift height windward drift $h_{d\_w} = 0.75 \times (0.43 \times (\max(25\text{ ft}, l_l) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft}) = \underline{\mathbf{1.93}}$ ft

Maximum lw/ww drift height $h_{d\_max} = \max(h_{d\_w}, h_{d\_l}) = \underline{\mathbf{4.01}}$ ft

Drift height $h_d = \min(h_{d\_max}, h_c) = \underline{\mathbf{4.01}}$ ft

Drift width $W_d = \min(4 \times h_{d\_max}, 8 \times h_c) = \underline{\mathbf{16.03}}$ ft

Drift surcharge load $p_d = h_d \times \gamma = \underline{\mathbf{79.54}}$ lb/ft^2



Elevation on snow drift

WIND LOAD CRITERIA FOR ANALYSIS AND DESIGN

WIND LOAD (ASCE 7 – 10): ACADEMIC BUILDING

WIND LOAD (ASCE 7 – 05)

Classification summary

Structure is a building

Structure is Rigid

Mean roof height h = **42.0** ft

Horizontal dimension parallel to wind L = **286.0** ft

Horizontal dimension normal to wind B = **175.0** ft

Roof angle θ = **0.0** deg

Wind force resisting element is part of main wind force resisting system

Structure is enclosed

Structure is low rise

Procedure

Occupancy category (table 1-1) Category = **3**

Basic wind speed (sect. 6.5.4) V = **110.0** mph

Region **Hurricane Prone**

Importance factor (table 6-1) I = **1.15**

Exposure category (sect. 6.5.6) **B**

Topographic factor K_{zt} = **1.0**

Design procedure - simplified procedure (Method 1)

Method 1 – simplified procedure to find design wind pressure for main wind force resisting system

Adjustment factor λ = **1.10**

Assuming load case LoadCase = **1**

Simplified design wind pressure for exposure B, at h – 30 ft, and for I – 1.0

Horizontal pressure

Zone A p_{S30_A} = **19.2** psf

Zone B p_{S30_B} = **-10.0** psf

Zone C p_{S30_C} = **12.7** psf

Zone D p_{S30_D} = **-5.9** psf

Vertical pressure

Zone E p_{S30_E} = **-23.1** psf

Zone F p_{S30_F} = **-13.1** psf

Zone G p_{S30_G} = **-16.0** psf

Zone H p_{S30_H} = **-10.1** psf

Overhang

Zone E_{OH} p_{S30_Eoh} = **-32.3** psf

Zone G_{OH} p_{S30_Goh} = **-25.3** psf

Net design wind pressure (eq. 6.1)

Horizontal pressure

Zone A $p_{S\_A} = \lambda \times K_{zt} \times I \times p_{S30\_A}$ = **24.33** psf

Zone B $p_{S\_B} = \lambda \times K_{zt} \times I \times p_{S30\_B}$ = **-12.67** psf

Zone C $p_{S\_C} = \lambda \times K_{zt} \times I \times p_{S30\_C}$ = **16.09** psf

Zone D $p_{S\_D} = \lambda \times K_{zt} \times I \times p_{S30\_D}$ = **-7.48** psf

Vertical pressure

Zone E $p_{S\_E} = \lambda \times K_{zt} \times I \times p_{S30\_E}$ = **-29.27** psf

Zone F $p_{S\_F} = \lambda \times K_{zt} \times I \times p_{S30\_F}$ = **-16.60** psf

Zone G $p_{S\_G} = \lambda \times K_{zt} \times I \times p_{S30\_G}$ = **-20.28** psf

Zone H $p_{S\_H} = \lambda \times K_{zt} \times I \times p_{S30\_H}$ = **-12.80** psf

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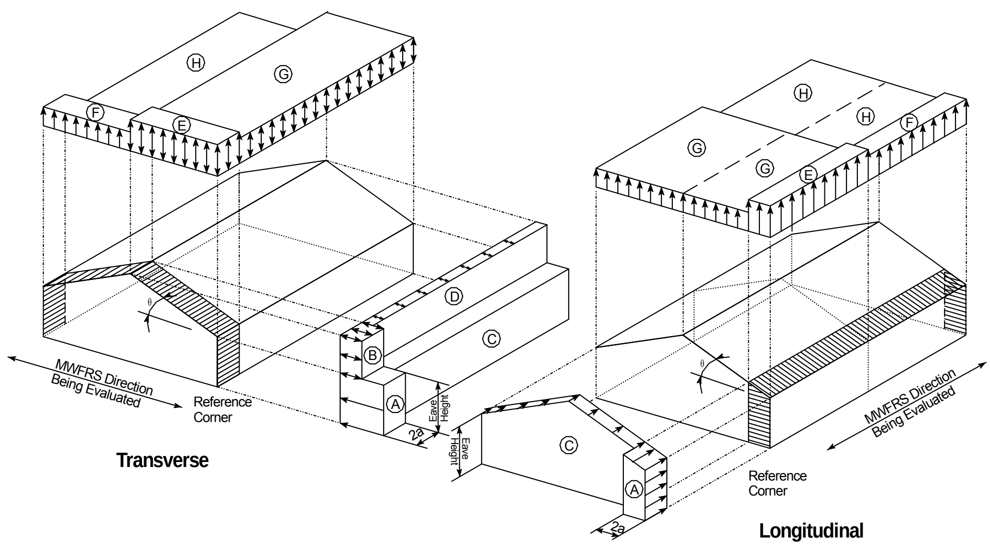
	MHP	6/10/2011	CM	6/11/2011		

Overhang

Zone E_{OH} $p_{S\_Eoh} = \lambda \times K_{zt} \times I \times p_{S30\_Eoh} = \underline{\mathbf{-40.93}}$ psf

Zone G_{OH} $p_{S\_Goh} = \lambda \times K_{zt} \times I \times p_{S30\_Goh} = \underline{\mathbf{-32.06}}$ psf

Note: - As per Section 6.1.4.1, the wind load to be used in the design of the MWFRS shall be not less than 10 psf multiplied by the area of the building or structure projected onto a vertical plane normal to the wind direction.



WIND LOAD (ASCE 7 – 10): AUDITORIUM/GYM/CAFETERIA

WIND LOAD (ASCE 7 – 05)

Classification summary

Structure is a building

Structure is Rigid

Mean roof height	h = **39.8** ft
Horizontal dimension parallel to wind	L = **200.0** ft
Horizontal dimension normal to wind	B = **175.0** ft
Roof angle	θ = **0.0** deg

Wind force resisting element is part of main wind force resisting system

Structure is enclosed

Structure is low rise

Procedure

Occupancy category (table 1-1)	Category = **3**
Basic wind speed (sect. 6.5.4)	V = **110.0** mph
Region	**Hurricane Prone**
Importance factor (table 6-1)	I = **1.15**
Exposure category (sect. 6.5.6)	**B**
Topographic factor	K_{zt} = **1.0**

Design procedure - simplified procedure (Method 1)

Method 1 – simplified procedure to find design wind pressure for main wind force resisting system

Adjustment factor λ = **1.09**

Assuming load case LoadCase = **1**

Horizontal pressure

Zone A p_{S30_A} = **19.2** psf

Zone B p_{S30_B} = **-10.0** psf

Zone C p_{S30_C} = **12.7** psf

Zone D p_{S30_D} = **-5.9** psf

Vertical pressure

Zone E p_{S30_E} = **-23.1** psf

Zone F p_{S30_F} = **-13.1** psf

Zone G p_{S30_G} = **-16.0** psf

Zone H p_{S30_H} = **-10.1** psf

Overhang

Zone E_{OH} p_{S30_Eoh} = **-32.3** psf

Zone G_{OH} p_{S30_Goh} = **-25.3** psf

Net design wind pressure (eq. 6.1)

Horizontal pressure

Zone A $p_{S\_A} = \lambda \times K_{zt} \times I \times p_{S30\_A}$ = **24.02** psf

Zone B $p_{S\_B} = \lambda \times K_{zt} \times I \times p_{S30\_B}$ = **-12.51** psf

Zone C $p_{S\_C} = \lambda \times K_{zt} \times I \times p_{S30\_C}$ = **15.89** psf

Zone D $p_{S\_D} = \lambda \times K_{zt} \times I \times p_{S30\_D}$ = **-7.38** psf

Vertical pressure

Zone E $p_{S\_E} = \lambda \times K_{zt} \times I \times p_{S30\_E} =$ **-28.90** psf

Zone F $p_{S\_F} = \lambda \times K_{zt} \times I \times p_{S30\_F} =$ **-16.39** psf

Zone G $p_{S\_G} = \lambda \times K_{zt} \times I \times p_{S30\_G} =$ **-20.02** psf

Zone H $p_{S\_H} = \lambda \times K_{zt} \times I \times p_{S30\_H} =$ **-12.64** psf

Overhang

Zone E_{OH} $p_{S\_Eoh} = \lambda \times K_{zt} \times I \times p_{S30\_Eoh} =$ **-40.41** psf

Zone G_{OH} $p_{S\_Goh} = \lambda \times K_{zt} \times I \times p_{S30\_Goh} =$ **-31.66** psf

Note: - As per Section 6.1.4.1, the wind load to be used in the design of the MWFRS shall be not less than 10 psf multiplied by the area of the building or structure projected onto a vertical plane normal to the wind direction.

SEISMIC CRITERIA FOR ANALYSIS AND DESIGN

SEISMIC FORCES: ACADEMIC BUILDING

Site parameters

Site class E

Mapped acceleration parameters (Section 11.4.1)

at short period S_S = **0.25**

at 1 sec period S_1 = **0.06**

Site coefficientat short period (Table 11.4-1) F_a = **2.500**

at 1 sec period (Table 11.4-2) F_v = **3.500**

Spectral response acceleration parameters

at short period (Eq. 11.4-1) $S_{MS} = F_a \times S_S$ = **0.63**

at 1 sec period (Eq. 11.4-2) $S_{M1} = F_v \times S_1$ = **0.22**

Design spectral acceleration parameters (Sect 11.4.4)

at short period (Eq. 11.4-3) $S_{DS} = 2 / 3 \times S_{MS}$ = **0.42**

at 1 sec period (Eq. 11.4-4) $S_{D1} = 2 / 3 \times S_{M1}$ = **0.15**

Seismic design category

Risk category III

Seismic design category (Table 11.6-1 only) C

Approximate fundamental period

Height above base to highest level of building h_n = **42** 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) $T_a = C_t \times (h_n)^x \times 1\text{sec} / (1\text{ft})^x$= **0.330** sec

Building fundamental period (Sect 12.8.2) $T = T_a$ = **0.330** sec

Long-period transition period T_L = **12** sec

Seismic response coefficient

Seismic force-resisting system (Table 12.14-1)

H. STEEL SYSTEMS NOT SPECIFICALLY DETAILED

1. Steel system not specially detailed for seismic resistance, ex

Response modification factor (Table 12.14-1) R = **3**

Seismic importance factor (Table 11.5-2) I_e = **1.250**

Seismic response coefficient (Sect 12.8.1.1)

Calculated (Eq 12.8-2) $C_{s\_calc} = S_{DS} / (R / I_e)$= **0.174**

Maximum (Eq 12.8-3) $C_{s\_max} = S_{D1} / (T \times (R / I_e))$ = **0.186**

Minimum (Eq 12.8-5,Supp. No. 2) $C_{s\_min} = \max(0.044 \times S_{DS} \times I_e, 0.01)$ = **0.023**

Seismic response coefficient C_s = **0.174**

Seismic base shear (Sect 12.8.1)

Effective seismic weight of the structure W = **10200.0** kips

Seismic response coefficient C_s = **0.174**

Seismic base shear (Eq 12.8-1) $V = C_s \times W$ = **1770.8** kips

Vertical distribution of seismic forces (Sect 12.8.3)

Vertical distribution factor (Eq 12.8-12) $C_{vx} = w_x \times h_x^k / \Sigma(w_i \times h_i^k)$

Lateral force induced at level i (Eq 12.8-11) $F_x = C_{vx} \times V$

Vertical force distribution table

Level	Height from base to Level i (ft), h_x	Portion of effective seismic weight assigned to Level i (kips), w_x	Distribution exponent related to building period, k	Vertical distribution factor, C_{vx}	Lateral force induced at Level i (kips), F_x
1	14.0	4000.0	1.00	0.215	380.8
2	28.0	4000.0	1.00	0.430	761.6
3	42.0	2200.0	1.00	0.355	628.4

SEISMIC FORCES: AUDITORIUM/ GYMNASIUM/ CAFETERIA

Site parameters

Site class E

Mapped acceleration parameters (Section 11.4.1)

at short period S_S = **0.25**

at 1 sec period S_1 = **0.06**

Site coefficientat short period (Table 11.4-1) F_a = **2.500**

at 1 sec period (Table 11.4-2) F_v = **3.500**

Spectral response acceleration parameters

at short period (Eq. 11.4-1) $S_{MS} = F_a \times S_S$ = **0.63**

at 1 sec period (Eq. 11.4-2) $S_{M1} = F_v \times S_1$ = **0.22**

Design spectral acceleration parameters (Sect 11.4.4)

at short period (Eq. 11.4-3) $S_{DS} = 2 / 3 \times S_{MS}$ = **0.42**

at 1 sec period (Eq. 11.4-4) $S_{D1} = 2 / 3 \times S_{M1}$ = **0.15**

Seismic design category

Risk category III

Seismic design category (Table 11.6-1 only) C

Approximate fundamental period

Height above base to highest level of building h_n = **35** ft

From Table 12.8-2:

Structure type All other systems

Building period parameter C_t C_t = **0.02**

Building period parameter x x = **0.75**

Approximate fundamental period (Eq 12.8-7) $T_a = C_t \times (h_n)^x \times 1sec / (1ft)^x$= **0.288** sec

Building fundamental period (Sect 12.8.2) $T = T_a$ = **0.288** sec

Long-period transition period T_L = **12** sec

Seismic response coefficient

Seismic force-resisting system (Table 12.14-1)

H. STEEL SYSTEMS NOT SPECIFICALLY DETAILED

1. Steel system not specially detailed for seismic resistance, ex

Response modification factor (Table 12.14-1) R = **3**

Seismic importance factor (Table 11.5-2) I_e = **1.250**

Seismic response coefficient (Sect 12.8.1.1)

Calculated (Eq 12.8-2) $C_{s\_calc} = S_{DS} / (R / I_e)$= **0.174**

Maximum (Eq 12.8-3) $C_{s\_max} = S_{D1} / (T \times (R / I_e))$ = **0.213**

Minimum (Eq 12.8-5,Supp. No. 2) $C_{s\_min} = \max(0.044 \times S_{DS} \times I_e, 0.01)$ = **0.023**

Seismic response coefficient C_s = **0.174**

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Seismic base shear (Sect 12.8.1)

Effective seismic weight of the structure W = **4000.0** kips

Seismic response coefficient C_s = **0.174**

Seismic base shear (Eq 12.8-1) $V = C_s \times W$ = **694.4** kips

Vertical distribution of seismic forces (Sect 12.8.3)

Vertical distribution factor (Eq 12.8-12) $C_{vx} = w_x \times h_x^k / \Sigma(w_i \times h_i^k)$

Lateral force induced at level i (Eq 12.8-11) $F_x = C_{vx} \times V$

Vertical force distribution table

Level	Height from base to Level i (ft), h_x	Portion of effective seismic weight assigned to Level i (kips), w_x	Distribution exponent related to building period, k	Vertical distribution factor, C_{vx}	Lateral force induced at Level i (kips), F_x
Locker Room Roof	14.0	1100.0	1.00	0.145	100.9
Band Roof	20.0	400.0	1.00	0.075	52.4
Cafe Roof	28.0	700.0	1.00	0.185	128.4
Aud/Gym Roof	35.0	1800.0	1.00	0.594	412.7

SAMPLE CALCULATIONS

I-SECTION MEMBER DESIGN TO LRFD (2001)

Try section - W 21x50

$h = d - 2 \times k$ = **18.72** in t_w = **0.38** in b_f = **6.53** in t_f = **0.54** in

λ_{flange} = b_f_to_2t_f =**6.10** λ_{web} = h_to_t_w = **49.40**

Specified minimum yield stress F_y = **50** ksi

Local buckling classification to LRFD B5.1 for x-axis flexure

(I section)

Flange is COMPACT

Web is COMPACT

DESIGN FOR FLEXURE (X-AXIS)

Flexure about x-axis - yielding limit state to LRFD F1.1

Plastic moment $M_{px} = \min(F_y \times Z_x , 1.5 \times F_y \times S_x)$ = **458.3** kip_ft

Design flexural strength ϕ_b =**0.90** $\phi_b \times M_{px}$ = **412.5** kip_ft

Required flexural strength M_{ux} = **120.0** kip_ft

Pass - yielding (x-axis flexure)

Local buckling under flexure about x-axis (I and channel sections) to LRFD Appendix F1

Resistance factor ϕ_b = **0.9**

Nominal flexural strength for limit state of flange buckling, M_{nx_flange}:-

M_{nx_flange} = if($\lambda_{flange} < \lambda_{p\_flange}$, M_{px}, if($\lambda_{flange} < \lambda_{r\_flange}$, $M_{nx\_flange\_nc}$, 0.1 kip_ft)) = **458.3** kip_ft

Nominal flexural strength for limit state of web buckling, M_{nx_web}:-

M_{nx_web} = if($\lambda_{web} < \lambda_{p\_web}$, M_{px}, if($\lambda_{web} < \lambda_{r\_web}$, $M_{nx\_web\_nc}$, 0.1 kip_ft)) = **458.3** kip_ft

Design flexural strength for local buckling $\phi_b \times \min(M_{nx\_flange}, M_{nx\_web})$ = **412.5** kip_ft

Required flexural strength M_{ux} = **120.0** kip_ft

Pass - local buckling limit state

Flexure (x-axis) - summary

Member is fully braced against lateral-torsional buckling.

Nominal flexural strength $M_{nx} = \min(M_{px}, M_{nx\_flange}, M_{nx\_web}) =$ **458.33** kip_ft

Design flexural strength $\phi_b M_{nx} = \phi_b \times M_{nx} =$ **412.50** kip_ft

Required flexural strength $M_{ux} =$ **120.00** kip_ft

Pass - flexural strength

Deflection to LRFD L3

Maximum deflection due to service loads $\delta_{max} = \max(\mathrm{abs}(\delta_{pos}), \mathrm{abs}(\delta_{neg})) =$ **2.000** in

Effective span of beam for deflection check $L_s =$ **30.0** ft

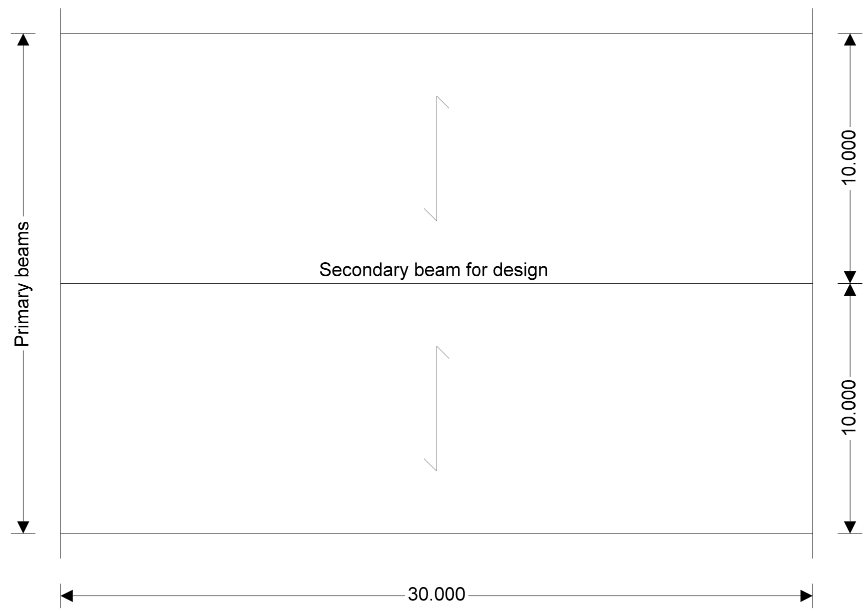
Maximum allowable deflection / span ratio ratio **= 1 / 0**

Fixed limit on maximum deflection $\delta_{lim2} =$ **2.000** in

Governing maximum allowable deflection $\delta_{lim} = \min(\mathrm{ratio} \times L_s, \delta_{lim2}) =$ **2.000** in

Deflection OK

DESIGN OF STEEL COMPOSITE BEAM TO AISC (LRFD)



Basic dimensions

Beam span L = **30.000** 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 360-05

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

Height of ribs h_r = **2.000** in

Centers of ribs rib_{ccs} = **6.000** in

Average width of rib w_r = **2.500** in

Structural Engineers 10 Cabot Road, Suite 210 Medford, MA 02155 (781) 396-9007 (781) 396-9008 (FAX) Engineers Design Group Inc.	Project EAST BRIDGEWATER JUNIOR/SENIOR SCHOOL				Job Ref. 2010-059	
	STRUCTURAL CALCULATIONS				Sheet no./rev. 28	
	Calc. by MHP	Date 6/10/2011	Chk'd by CM	Date 6/11/2011	App'd by	Date

Material properties

Concrete

Specified compressive strength of concrete f'_c = **3.00** ksi

Wet density of concrete w_{cw} = **120** lb/ft^3

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

Modulus of elasticity of concrete $E_c = w_{cd}^{1.5} \times \sqrt{(f'_c \times 1\text{ ksi})} / (1\text{ lb/ft}^3)^{1.5}$ = **2136** 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 $w_{slab\_constr} = [t - h_r \times (1 - w_r / rib_{ccs})] \times w_{cw}$ = **40.833** psf

Weight of slab composite stage $w_{slab\_comp} = [t - h_r \times (1 - w_r / rib_{ccs})] \times w_{cd}$ = **39.132** psf

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

Additional dead load w_{d_add} = **5.000** psf

Weight of steel beam w_{beam_s} = **44.000** lb/ft

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

Superimposed dead load w_{serv} = **25.000** 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 $w_{constr\_D} = [(w_{slab\_constr}+w_{deck}+w_{d\_add})\times((b_1+b_2)/2)] + w_{beam\_s}$ = **532.333** 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) $w_{comp\_D} = [(w_{slab\_comp}+w_{deck}+w_{d\_add}+w_{serv})\times(b_1+b_2)/2]+w_{beam\_s}$ = **765.319** 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 $M_{constr\_u} = (1.2 \times w_{constr\_D} + 1.6 \times w_{constr\_L}) \times L^2 / 8$ = **107.865** kips_ft

Max ultimate shear at construction stage $V_{constr\_u} = (1.2 \times w_{constr\_D} + 1.6 \times w_{constr\_L}) \times L / 2$ = **14.382** kips

Maximum ultimate moment at composite stage

$M_{comp\_u} = (1.2 \times w_{comp\_D} + 1.6 \times w_{comp\_L}) \times L^2/8 + 1.2 \times w_{w\_par} \times L^2/8 + 1.2 \times w_{w\_perp} \times (b_1 + b_2)/2 \times L/4$ = **283.318** kips_ft

Maximum ultimate shear at composite stage

$V_{comp\_u} = (1.2 \times w_{comp\_D} + 1.6 \times w_{comp\_L}) \times L / 2 + 1.2 \times w_{w\_par} \times L / 2 + 1.2 \times w_{w\_perp} \times (b_1 + b_2)/2 \times 1/2$ = **37.776** kips

Point of max. B.M. from nearest support $L_{BM\_near} = L/2$ = **15.00** ft

Steel section check

Trial steel section **W21X44**

Plastic modulus of steel section Z_x = **95.40** in^3

Elastic modulus of steel section S_x = **81.60** in^3

Width to thickness ratio $\lambda_f = b_f / (2 \times t_f)$ = **7.222**

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 = **53.600**

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 $M_p = F_y \times Z_x$ = **397.500** kip_ft

Resistance factor for flexure ϕ_b = **0.90**

Design flexural strength of steel section alone $M_{constr\_n} = \phi_b \times M_p$ = **357.750** kip_ft

Required flexural strength M_{constr_u} = **107.865** kip_ft

PASS - Beam bending at construction stage loading

Strength check at construction stage for shear

Check for shear

Web area $A_w = d \times t_w$ = **7.245** in^2

Web plate buckling coefficient k_v = **5**

Depth to thickness ratio (h/t_w) λ_w = **53.600**

Web shear coefficient $C_v = 1$

Resistant factor for shear ϕ_v =if(λ_w <=2.24×√(E_S/F_y),1.00,□0.9) = **1.0**

Design shear strength $V_{constr\_n} = \phi_v \times (0.6 \times F_y \times A_w \times C_v)$ = **217.350** kips

Required shear strength V_{constr_u} = **14.382** kips

PASS - Beam shear at construction stage loading

Design of shear connectors

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 $b = \min(L/8, b_1/2) + \min(L/8, b_2/2) =$ **90.000** in

Effective area of concrete flange $A_c = b \times (t - h_r) = 292.50$ in^2

Diameter of shear connectors dia = **0.750** in

Length of shear connectors after weld $H_s =$ **3.50** in

Specified tensile stength of stud shear connector $F_u =$ **65** ksi

Cross section area of one shear connector $A_{sc} = \pi \times dia^2 / 4 =$ **0.442** in^2

Maximum diameter permitted $dia_{sc\_max} = 2.5 \times t_f =$ **1.125** in

PASS - Dia of shear connector provided is OK

Point of max. B.M. from nearest support $L_{BM\_near} =$ **15.00** ft

No. of ribs from points of zero to max moment $rib_{numbers} = int(L_{BM\_near} / rib_{ccs} - 1) =$ **29**

No. of ribs with 1 stud per rib $N_{r1} =$ **28**

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 $N_{prov} = N_{r1} + 2 \times N_{r2} + 3 \times N_{r3} =$ **28**

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**

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

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

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

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

Total strength of provided shear connectors $S_{sc} = N_{r1} \times Q_{n1} + 2 \times N_{r2} \times Q_{n2} + 3 \times N_{r3} \times Q_{n3} =$ **482.43** kips

Resistance of concrete flange $C_{cf} = 0.85 \times f'_c \times A_c =$ **745.875** kips

Resistance of steel beam T_{sb} = A × F_y = **650.000** kips

Beam/slab interface shear force C = min(C_{cf}, T_{sb}) = **650.000** 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 = **650.000** kips

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

Resultant compressive force at flange bottom $P_{yf} = b_f \times t_f \times F_y$ = **146.250** kips

Net force at steel and concrete interface $C_{net} = T_{sb} - 2 \times P_{yf}$ = **357.500** kips

PNA is in the flange of I Section

Shear connection force $F_{shear} = S_{sc}$ = **482.43** kips

Total depth of concrete at full stress $d_c = F_{shear} / (0.85 \times f'_c \times b)$ = **2.102** in

Depth of compression from top of the steel flange $t' = A / (2 \times b_f) - 0.85 \times f'_c / F_y \times b \times d_c / (2 \times b_f)$ = **0.258** in

Tension

Bottom flange component $F_{bf} = F_y \times b_f \times t_f$ = **146.250** kips

Moment capacity of bottom flange $M_{bf} = F_{bf} \times (d - (t_f/2) - t')$ = **246.397** kip_ft

Web component $F_{web} = F_y \times (A - (2 \times b_f \times t_f))$ = **357.500** kips

Moment capacity of web $M_{web} = F_{web} \times (((d - 2 \times t_f)/2) + t_f - t')$ = **300.663** kip_ft

Top flange component $F_{tf\_t} = F_y \times b_f \times (t_f - t')$ = **62.465** kips

Moment capacity of top flange $M_{tf\_t} = F_{tf\_t} \times (t_f - t')/2$ = **0.500** kip_ft

Compression

Top flange component $F_{tf\_c} = F_y \times b_f \times t'$ = **83.785** kips

Moment capacity of top flange $M_{tf\_c} = F_{tf\_c} \times t'/2$ = **0.900** kip_ft

Concrete flange component $F_{cf} = 0.85 \times f'_c \times b \times d_c$ = **482.431** kips

Moment capacity of concrete flange $M_{cf} = F_{cf} \times (t - d_c/2 + t')$ = **179.173** kip_ft

Design flexural strength of beam $M_{comp\_n} = \phi_b \times(M_{bf} + M_{web} + M_{tf\_t} + M_{tf\_c} + M_{cf}) =$ **654.870** kip_ft

Required flexural strength $M_{comp\_u} =$ **283.318** kip_ft

PASS - Beam bending at partial composite stage

Check for shear

Design shear strength $V_{comp\_n} = \phi_v \times (0.6 \times F_y \times A_w \times C_v) =$ **217.350** kips

Required shear strength $V_{comp\_u} =$ **37.776** kips

PASS - Beam shear at partial composite stage loading

Check for deflection(AISC 360 – 05, Commentry section I3.1)

Calculation of immediate construction stage deflection

Deflection due to dead load $\Delta_{short\_D} = 5 \times w_{constr\_D} \times L^4 / (384 \times E_S \times I_x) =$ **0.3968** in

Amount of beam camber $\Delta_{camber} =$ **0.250** in

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

Deflection due to construction live load $\Delta_2 = 5 \times w_{constr\_L} \times L^4 / (384 \times E_S \times I_x) =$ **0.1491** in

Net total construction stage deflection $\Delta_{short} = \Delta_{short\_D} + \Delta_2 - \Delta_{camber} =$ **0.296** in

For short term loading:-

Short term modular ratio $n_s = E_S / E_c =$ **13.6**

Depth of neutral axis from top of concrete

$y_s =$ **6.884** in

Moment of inertia of fully composite section

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

$I_s =$ **2445** in^4

Effective mt of inertia for partially composite $I_{s\_eff} = 0.75 \times [I_x + \sqrt{}(F_{shear} / C) \times (I_s - I_x)] =$ **1667.6** in^4

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

Deflection due to short term live load $\Delta_{L\_s} = 5 \times r_{L\_s} \times w_{comp\_L} \times L^4 / (384 \times E_S \times I_{s\_eff}) =$ **0.2525** in

For long term loading:-

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

Long term modular ratio $n_l = E_S / (E_c \times r_{E\_l}) =$ **27.2**

Depth of neutral axis from top of concrete

$y_l =$ **9.267** in

Moment of inertia of fully composite section

I_l = **2003** in^4

Effective mt of inertia for partially composite $I_{l\_eff} = 0.75 \times [I_x + \sqrt{(F_{shear} / C)} \times (I_l - I_x)]$ = **1381.7** 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 $\Delta_{L\_l} = 5 \times r_{L\_l} \times w_{comp\_L} \times L^4 / (384 \times E_S \times I_{l\_eff})$ = **0.1501** in

Dead load due to parallel wall & superimp. dead $w_{D\_part} = w_{w\_par} + (w_{serv} \times (b_1 + b_2) / 2)$ = **250.0000** lb/ft

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

Wall parallel to span and superimposed dead $\Delta_4 = 5 \times (w_{D\_part}) \times L^4 / (384 \times E_S \times I_{l\_eff})$ = **0.1137** 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 $\Delta_{short} = \Delta_{short\_D} + \Delta_2 - \Delta_{camber}$ = **0.296** in

Net total long term deflection $\Delta_{long} = \Delta_{short\_D} + \Delta_{L\_s} + \Delta_{L\_l} + \Delta_4 + \Delta_5 - \Delta_{camber}$ = **0.663** in

Combined short and long term live load deflectn $\Delta_{live} = \Delta_{L\_s} + \Delta_{L\_l}$ = **0.403** in

Net long term dead and super imposed dead defln $\Delta_{dead} = \Delta_{short\_D} + \Delta_4 + \Delta_5 - \Delta_{camber}$ = **0.261** in

Post composite deflection $\Delta_{comp} = \Delta_{L\_s} + \Delta_{L\_l} + \Delta_4 + \Delta_5$ = **0.516** in

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

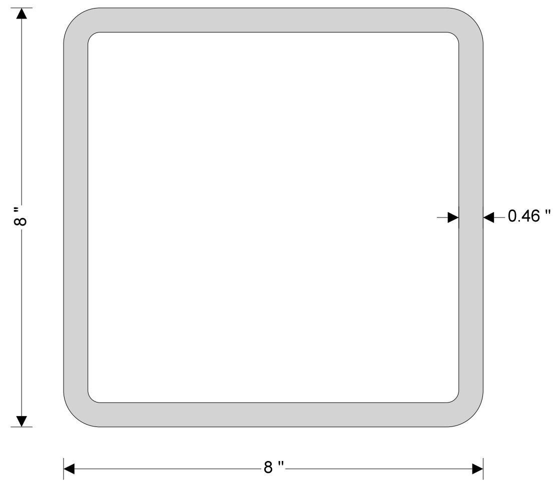
PASS - Deflection less than allowable

Shear connector arrangement

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

STEEL COLUMN DESIGN (AISC 360-05)

In accordance with the LRFD method



Column and loading details

Column details

Column section: **HSS 8x8x0.5**

Design loading

Required axial strength	P_r = **50** kips (Compression)
Moment about x axis at end 1	M_{x1} = **6.0** kips_ft
Moment about x axis at end 2	M_{x2} = **4.0** kips_ft
	Single curvature bending about x axis
Maximum moment about x axis	M_x = max(abs(M_{x1}), abs(M_{x2})) = **6.0** kips_ft
Moment about y axis at end 1	M_{y1} = **6.0** kips_ft
Moment about y axis at end 2	M_{y2} = **4.0** kips_ft
	Single curvature bending about y axis
Maximum moment about y axis	M_y = max(abs(M_{y1}), abs(M_{y2})) = **6.0** kips_ft
Maximum shear force parallel to y axis	V_{ry} = **12.0** kips
Maximum shear force parallel to x axis	V_{rx} = **12.0** kips

Material details

Steel grade	**A500 Gr. B**
Yield strength	F_y = **46** ksi

Ultimate strength F_u = **58** ksi

Modulus of elasticity E = **29000** ksi

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

Unbraced lengths

For buckling about x axis L_x = **192** in

For buckling about y axis L_y = **0** in

For torsional buckling L_z = **300** 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 $b = b_f - 3 \times t$ = **6.605** in

Critical web width $h = d - 3 \times t$ = **6.605** in

Width to thickness ratio of flange (compression) $\lambda_{f\_c} = b / t$ = **14.204**

Width to thickness ratio of web (compression) $\lambda_{w\_c} = h / t$ = **14.204**

Width to thickness ratio of flange (major flexure) $\lambda_{f\_fx} = b / t$ = **14.204**

Width to thickness ratio of web (major flexure) $\lambda_{w\_fx} = h / t$ = **14.204**

Width to thickness ratio of flange (minor flexure) $\lambda_{f\_fy} = h / t$ = **14.204**

Width to thickness ratio of web (minor flexure) $\lambda_{w\_fy} = b / t$ = **14.204**

Compression

Limit for noncompact section $\lambda_{r\_c} = 1.40 \times \sqrt{(E / F_y)}$ = **35.152**

The section is noncompact in compression

Flexure

Limit for compact flange $\lambda_{pf\_f} = 1.12 \times \sqrt{(E / F_y)}$ = **28.121**

Limit for noncompact flange $\lambda_{rf\_f} = 1.40 \times \sqrt{(E / F_y)}$ = **35.152**

Limit for compact web $\lambda_{pw\_f} = 2.42 \times \sqrt{(E / F_y)}$ = **60.762**

Limit for noncompact web $\lambda_{rw\_f} = 5.70 \times \sqrt{(E / F_y)}$ = **143.118**

The section is compact in flexure about the major axis

The section is compact in flexure about the minor axis

Slenderness

Member slenderness

Slenderness ratio about x axis $SR_x = K_x \times L_x / r_x$ = **63.2**

Slenderness ratio about y axis $SR_y = K_y \times L_y / r_y$ = **0.0**

Second order effects

Second order effects for bending about y axis (cl. C2.1b)

Second order effects are already included or do not need to be considered therefore:-

P-δ amplifier $B_{1x} = B_{1y}$ = **1.0**

Required flexural strength (x axis) $M_{rx} = B_{1x} \times M_x$ = **6.0** kips_ft

Required flexural strength (y axis) $M_{ry} = B_{1y} \times M_y$ = **6.0** kips_ft

Shear strength

Shear parallel to the minor axis (cl. G2.1)

Shear area $A_w = 2 \times (d - 3 \times t) \times t$ = **6.143** in^2

Web plate buckling coefficient k_v = **5.0**

Web shear coefficient C_v = **1.000**

Nominal shear strength $V_{ny} = 0.6 \times F_y \times A_w \times C_v$ = **169.5** kips

Design shear strength (cl.G1 & G2.1(a))

Resistance factor for shear ϕ_v = **0.90**

Design shear strength $V_{cy} = \phi_v \times V_{ny}$ = **152.6** kips

PASS - The design shear strength exceeds the required shear strength

Shear parallel to the major axis (cl. G2.1)

Shear area $A_w = 2 \times (b_f - 3 \times t) \times t$ = **6.143** in^2

Web plate buckling coefficient k_v = **5.0**

Web shear coefficient C_v = **1.000**

Nominal shear strength $V_{nx} = 0.6 \times F_y \times A_w \times C_v$ = **169.5** kips

Design shear strength (cl.G1 & G2.1(a))

Resistance factor for shear ϕ_v = **0.90**

Design shear strength $V_{cx} = \phi_v \times V_{nx}$ = **152.6** kips

PASS - The design shear strength exceeds the required shear strength

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 $F_{ex} = (\pi^2 \times E) / (SR_x)^2$ = **71.8** ksi

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

Flexural buckling stress about x axis $F_{crx} = Q_x \times (0.658^{Qx \times Fy/Fex}) \times F_y$ = **35.2** ksi

Nominal flexural buckling strength $P_{nx} = F_{crx} \times A_g$ = **474.9** kips

Flexural buckling about y axis (cl. E3)

Unbraced length about y axis equals zero therefore:-

Flexural buckling stress $F_{cry} = F_y$ = **46.0** ksi

Nominal flexural buckling strength $P_{ny} = F_{cry} \times A_g$ = **621.0** kips

Design compressive strength (cl.E1)

Resistance factor for compression ϕ_c = **0.90**

Design compressive strength $P_c = \phi_c \times \min(P_{nx}, P_{ny})$ = **427.4** kips

PASS - The design compressive strength exceeds the required compressive strength

Flexural strength about the major axis

Yielding (cl. F7.1)

Nominal flexural strength $M_{nx\_yld} = M_{px} = F_y \times Z_x =$ **143.7** kips_ft

Design flexural strength about the major axis (cl. F1)

Resistance factor for flexure $\phi_b =$ **0.90**

Design flexural strength $M_{cx} = \phi_b \times M_{nx\_yld} =$ **129.4** kips_ft

PASS - The design flexural strength about the major axis exceeds the required flexural strength

Flexural strength about the minor axis

Yielding (cl. F7.1)

Nominal flexural strength $M_{ny\_yld} = M_{py} = F_y \times Z_y =$ **143.7** kips_ft

Design flexural strength about the minor axis (cl. F1)

Resistance factor for flexure $\phi_b =$ **0.90**

Design flexural strength $M_{cy} = \phi_b \times M_{ny\_yld} =$ **129.4** kips_ft

PASS - The design flexural strength about the minor axis exceeds the required flexural strength

Combined forces

$M_{ry} / M_{cy} < 0.05$ - Moments exist primarily in one plane therefore check combined forces in accordance with clause H1.3.

In-plane instability (cl. H1.3(a))

Available comp. strength in plane of bending $P_{ci} = \phi_c \times P_{nx} =$ **427.4** kips

Member utilization (eqn H1-1) $UR_i = P_r / (2 \times P_{ci}) + M_{rx} / M_{cx} =$ **0.105**

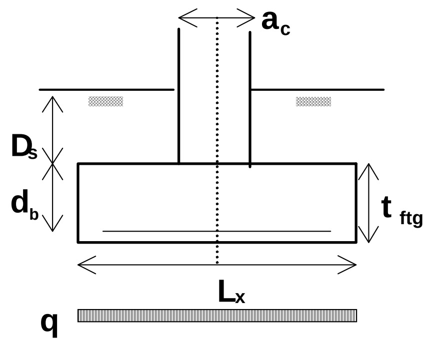
Out-of-plane buckling (cl. H1.3(b))

Available comp. strength out of plane of bending $P_{co} = \phi_c \times P_{ny} =$ **558.9** kips

Member utilization (eqn H1-2) $UR_o = P_r / P_{co} + (M_{rx} / M_{cx})^2 =$ **0.092**

PASS - The member is adequate for the combined forces

ISOLATED FOOTING DESIGN



Square Footing
(y axis into page)

SQUARE FOOTING SIZE – INDIVIDUAL COLUMN (AXIAL LOAD ONLY)

Column details

Column width, x — a_c = **6.000** in

Column width, y — b_c = **6.000** in

Loading details

Column axial dead load — P_{dl} = **45.000** kips

Column axial live load — P_{ll} = **40.000** kips

Total column load (unfactored) — $P_n = P_{dl} + P_{ll}$ = **85.000** kips

Axial load acting downward - OK

Soil details

The allowable increase in bearing pressure — q_a = **6.000** ksf

Depth of soil above top of footing — D_s = **1.500** ft

Density of soil — ρ_s = **120** lb/ft^3

Footing details

Thickness of footing t_{ftg} = **16.000** in

Concrete density ρ_c = **150** lb/ft^3

Increase in pressure due to weight of footing $q_{ftg} = (\rho_c - \rho_s) \times t_{ftg}$ = **0.040** ksf

Materials

Yield strength of tension reinforcement f_y = **60** ksi

Concrete strength f'_c = **3.000** ksi

Footing size

Available net increase in bearing pressure $q_s = q_a - q_{ftg}$ = **5.960** ksf

Footing size reqd $L_x = L_y = \sqrt{(P_n / q_s)}$ = **3.776** ft

Say footing size 5.000 ft × 5.000 ft × 16.000 in

Footing area OK

SQUARE FOOTING USD DESIGN FORCES – INDIVIDUAL COLUMN (AXIAL LOAD ONLY)

Footing details

Thickness of footing t_{ftg} = **16.000** in

Concrete cover d_c = **3.000** in

Trial bar diameter d_{bar} = **0.750** in

Structural depth to reinforcement

$d_b = t_{ftg} - d_c - d_{bar}/2$ = **12.625** in

$d_{bx} = d_b$

$d_{by} = d_{bx} - d_{bar}$ = **11.875** in

Design forces

Column service loads

Column axial dead load P_{dl} = **45.000** kips

Column axial live load P_{ll} = **40.000** kips

Total column load (unfactored)

$P_n = P_{dl} + P_{ll}$ = **85.000** kips

Total column load (factored)

$P_u = 1.2 \times P_{dl} + 1.6 \times P_{ll} = \underline{\mathbf{118.000}}$ kips

Axial load acting downward - OK

Actual soil pressure under base

Ultimate net design bearing pressure

$q_u = P_u / L_x^2 + 1.2 \times q_{ftg} = \underline{\mathbf{4.768}}$ ksf

Assume the base is a cantilevered slab with uniform load q_u ksf

Ultimate moment at column face, x direction

ACI 15.4.2a

Column width, x $a_c = \underline{\mathbf{6.000}}$ in

$M_{ux} = q_u \times L_x \times (L_x/2 - a_c/2)^2/2 = \underline{\mathbf{60.3}}$ kip_ft

Ultimate moment at column face, y direction

Column width, y $b_c = \underline{\mathbf{6.000}}$ in

$M_{uy} = q_u \times L_x \times (L_x/2 - b_c/2)^2/2 = \underline{\mathbf{60.3}}$ kip_ft

One-way (beam) shear

$x_d = \max((L_x/2 - a_c/2 - d_b), 0 \text{ in}) = \underline{\mathbf{14.375}}$ in

$V_{ux} = q_u \times L_x \times x_d = \underline{\mathbf{28.6}}$ kips

$y_d = \max((L_x/2 - b_c/2 - d_b + d_{bar}), 0 \text{ in}) = \underline{\mathbf{15.125}}$ in

$V_{uy} = q_u \times L_x \times y_d = \underline{\mathbf{30.0}}$ kips

Two-way shear at d/2 from column face

ACI 11.12.1.2

Area within ftg - OK

Perimeter at d/2

$b_o = 2 \times (a_c + b_c + 2 \times d_b) = \underline{\mathbf{74.500}}$ in

$V_{up} = q_u \times (L_x^2 - (a_c + d_b) \times (b_c + d_b)) = \underline{\mathbf{107.714}}$ kips

Note - it is assumed that edge distances permit this to be a valid failure mechanism

Footing reinforcement - along X axis

Depth to tension steel along X axis $d_{bx} = \underline{\mathbf{12.625}}$ in

Ultimate moment at column face $M_{ux} = \underline{\mathbf{60.345}}$ kip_ft

Area of reinforcement required

β_1 = if(f'$_c$ < 4 ksi, 0.85, max(.65, 0.85 - 0.05 × (f'$_c$ - 4 ksi) / 1 ksi)) = $\underline{\mathbf{0.850}}$

$\omega_t = 0.319 \times \beta_1 = \underline{\mathbf{0.271}}$

$R_u = \omega_t \times (1 – 0.588 \times \omega_t) = \underline{\mathbf{0.228}}$

$R_{reqdx} = M_{ux} / (f'_c \times d_{bx}^2) / L_y = \underline{\mathbf{0.033417}}$

Section dimensions are OK to be tension-controlled

Footing requiring tension steel only – bars in X direction

$J_x = \text{sqrt}(\max(.25 - R_{reqdx} / 0.85 / 2.,0)) + .5 = \underline{\mathbf{0.9799}}$

Area of tension steel required

$A_{sx\_reqd} = M_{ux} / (0.90 \times f_y \times J_x \times d_{bx} \times L_y) = \underline{\mathbf{0.29}}$ in^2 /ft

Minimum ratio of tension reinforcement for temperature and shrinkage

$\rho_{min} = \underline{\mathbf{0.001800}}$

Thickness of footing

$t_{ftg} = \underline{\mathbf{16.000}}$ in

Total area of concrete per foot width

$A_c = t_{ftg} \times 12 \text{ in} / 1 \text{ ft} = \underline{\mathbf{192.000}} \text{ in}^2/\text{ft}$

$A_{s\_minx} = \underline{\mathbf{0.50}} \text{ in}^2/\text{ft}$

$A_{s\_43} = (4/3 \times A_{sx\_reqd})$

$\beta_{bx} = \text{if}(L_y/L_x>1, 2 / (L_y / L_x + 1) \times L_y / L_x, 1) = \underline{\mathbf{1.000}}$

$A_{sx\_req} = \max(A_{sx\_reqd}, A_{st\_minx}) \times \beta_{bx} = \underline{\mathbf{0.38}} \text{ in}^2 /\text{ft}$

Tension steel provided

<u>Provide Size #4 @ 12 in centers</u>

$A_{sx} = \underline{\mathbf{0.20}} \text{ in}^2/\text{ft}$

$d_{bar} = \underline{\mathbf{0.500}} \text{ in}$

$a_x = A_{sx} \times f_y / (0.85 \times f'_c) = \underline{\mathbf{0.385}} \text{ in}$

$c_x = a_x / \beta_1 = \underline{\mathbf{0.038}}$

$\varepsilon_{ty} = 0.003 \times ((d_{bx} – c_x) / c_x) = \underline{\mathbf{0.081}}$

<u>Pass - Ductility OK at ultimate strength.</u>

<u>Insufficient area of tension steel provided</u>

<u>Check maximum spacing</u>

<u>Spacing of bars - OK</u>

<u>Check minimum area of steel</u>

<u>Area of steel < min - NG</u>

<u>Check of nominal cover and thickness of footing</u>

Effective depth to bottom outer tension reinforcement

$d_{bx} = \underline{\mathbf{12.6}} \text{ in}$

AI 15.7

Footing thickness > minimum - OK

Cover to outer tension reinforcement

$d_{cov} = t_{ftg} - d_{bx} - d_{bar} / 2 = \underline{\mathbf{3.1}}$ in

Permissible minimum nominal cover to all reinforcement

$c_{min} = \underline{\mathbf{3.000}}$ in

Cover over outer steel - OK

Footing reinforcement - along Y axis

Depth to tension steel along Y axis $d_{by} = \underline{\mathbf{11.875}}$ in

Ultimate moment at column face $M_{uy} = \underline{\mathbf{60.345}}$ kip_ft

Area of reinforcement required

β_1 = if(f'$_c$ < 4 ksi, 0.85, max(.65, 0.85 - 0.05 × (f'$_c$ - 4 ksi) / 1 ksi)) = $\underline{\mathbf{0.850}}$

$\omega_t = 0.319 \times \beta_1 = \underline{\mathbf{0.271}}$

$R_u = \omega_t \times (1 - 0.588 \times \omega_t) = \underline{\mathbf{0.228}}$

$R_{reqdy} = M_{uy} / (f'_c \times d_{by}^2) / L_x = \underline{\mathbf{0.028529}}$

Section dimensions are OK to be tension-controlled

FOOTING REQUIRING TENSION STEEL ONLY – BARS IN Y DIRECTION

J_y = sqrt(max(.25 - R_{reqdy} / 0.85 / 2.,0)) + .5 = $\underline{\mathbf{0.9829}}$

Area of tension steel required

$A_{sy\_reqd} = M_{uy} / (0.90 \times f_y \times J_y \times d_{by} \times L_x) = \underline{\mathbf{0.23}}$ in^2 /ft

Minimum ratio of tension reinforcement for temperature and shrinkage

ρ_{min} = **0.001800**

Thickness of footing

t_{ftg} = **16.000** in

Total area of concrete per foot width

$A_c = t_{ftg} \times 12$ in / 1 ft = **192.000** in^2/ft

A_{s_miny} = **0.47** in^2/ft

$A_{s\_43y} = (4/3 \times A_{sy\_reqd})$

A_{st_miny} = **0.35** in^2/ft

β_{by} = = **1.139**

A_{sy_req} = = **0.39** in^2 /ft

Tension steel provided

Provide Size #5 @ 12 in centers

A_{sy} = **0.31** in^2/ft

d_{bar} = **0.625** in

$a_y = A_{sy} \times f_y / (0.85 \times f'_c)$ = **0.602** in

$c_y = a_y / \beta_1$ = **0.059**

$\varepsilon_{ty} = 0.003 \times ((d_{by} - c_y) / c_y)$ = **0.047**

Pass - Ductility OK at ultimate strength.

Insufficient area of tension steel provided

Check maximum spacing

Spacing of bars - OK

Check minimum area of steel

Area of steel < min - NG

Check of nominal cover and thickness of footing

Effective depth to bottom outer tension reinforcement

d_{by} = **11.9** in

Footing thickness > minimum - OK

Cover to outer tension reinforcement

$d_{cov} = t_{ftg} - d_{by} - d_{bar} / 2 =$ **3.8** in

Permissible minimum nominal cover to all reinforcement

$c_{min} =$ **3.000** in

Cover over outer steel OK

ONE-WAY (BEAM) SHEAR RESISTANCE OF FOOTING - X AXIS (ACI 11.12, 15.5)

Transverse width of footing $L_y =$ **3.776** ft

Depth to tension steel $d_{bx} =$ **12.625** in

Design ultimate shear forces

Ultimate shear at 'd' from column face $V_{ux} =$ **28.558** kips

Concrete strength $f'_c =$ **3.000** ksi

Shear capacity of concrete

$V_{cx} = 2 \times \sqrt{(f'_c)} \times L_y \times d_{bx} =$ **62.674** kips

$V_{sx} = 0$ kips

$\phi V_{nx} = 0.75 \times (V_{cx} + V_{sx}) =$ **47.006** kips

One-way shear capacity - OK

ONE-WAY (BEAM) SHEAR RESISTANCE OF FOOTING - Y AXIS (ACI 11.12, 15.5)

Longitudinal length of footing $L_x =$ **5.000** ft

Depth to tension steel $d_{by} =$ **11.875** in

Design ultimate shear forces

Ultimate shear at 'd' from column face $V_{uy} =$ **30.048** kips

Concrete strength $f'_c =$ **3.000** ksi

Shear capacity of concrete

$V_{cy} = 2 \times \sqrt{(f'_c)} \times L_x \times d_{by} =$ **78.050** kips

$V_{sy} = 0$ kips

$\phi V_{ny} = 0.75 \times (V_{cy} + V_{sy}) =$ **58.538** kips

One-way shear capacity - OK

TWO-WAY (Punching) shear CHECK (ACI 11.12.2)

Tension steel resisting bending

Total length of shear perimeter at d/2 from column face

b_o = **74.500** in

Depth to tension steel d_b = **12.625** in

Max punching shear force V_{up} = **107.714** kips

Concrete strength f'_c = **3.000** ksi

Shear capacity of concrete

β_c = **1.000**

$\alpha_s = 40$

factor = **4.000**

V_{cp} = **206.067** kips

V_{sp} = 0 kips

$\phi V_{np} = 0.75 \times (V_{cp} + V_{sp}) =$ **154.550** kips

Two-way shear capacity - OK