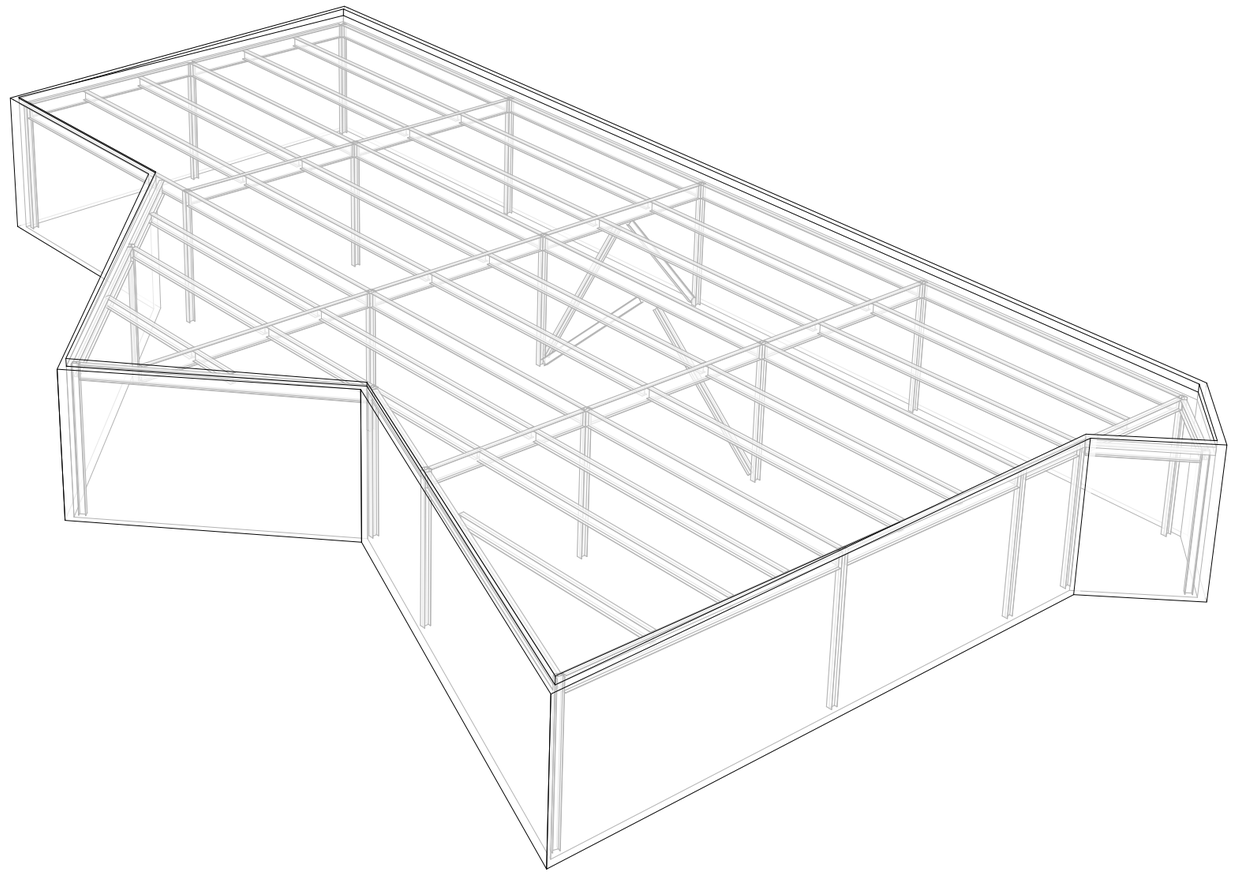
FINAL PROJECT
STRUCTURES III
AMIR MESGAR
FALL 2011



INDEX OF DRAWINGS

BOSTON ARCHITECTURAL COLLEGE

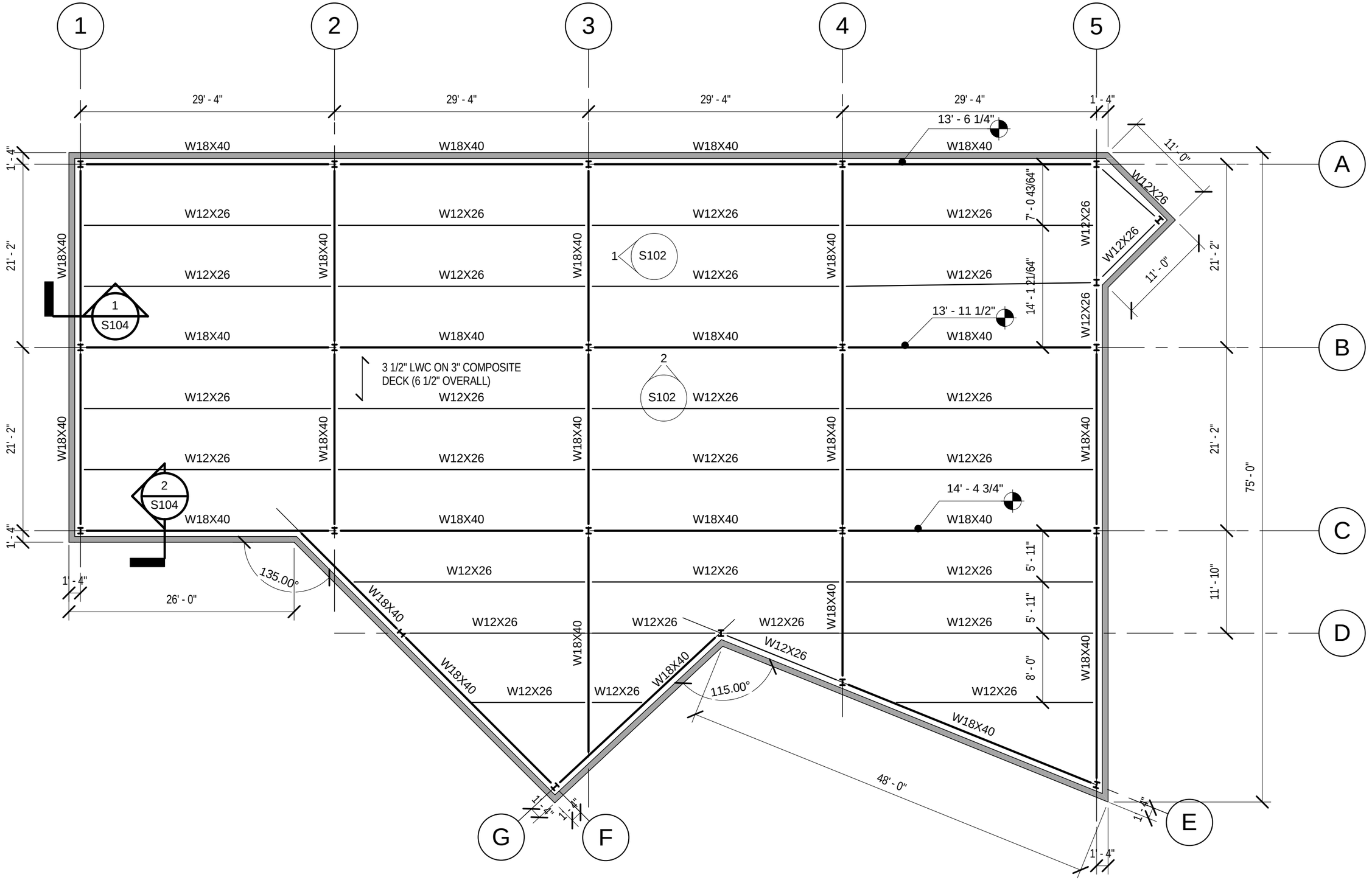
FOOD MARKET - BURN

TM632-7632

COVER SHEET

18 OCTOBER 2011

S100



BOSTON ARCHITECTURAL COLLEGE

FOOD MARKET - BURN

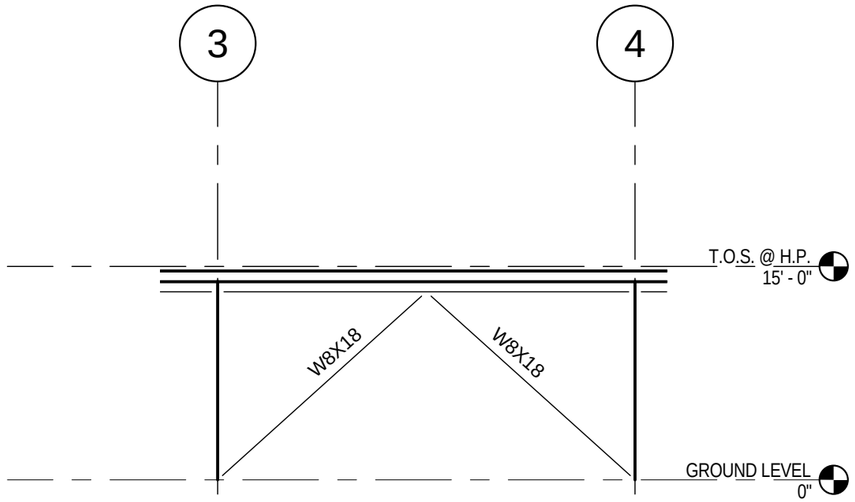
TM632-7632

FRAMING PLAN

18 OCTOBER 2011

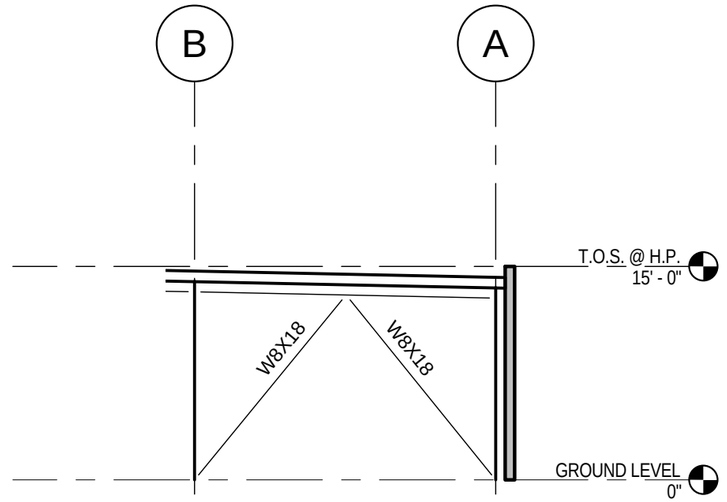
3/32" = 1'-0"

S101



2 BRACE FRAME @ BEAM

3/32" = 1'-0"



1 BRACE FRAME @ GIRDER

3/32" = 1'-0"

BOSTON ARCHITECTURAL COLLEGE

FOOD MARKET - BURN

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BRACE FRAME ELEVATIONS

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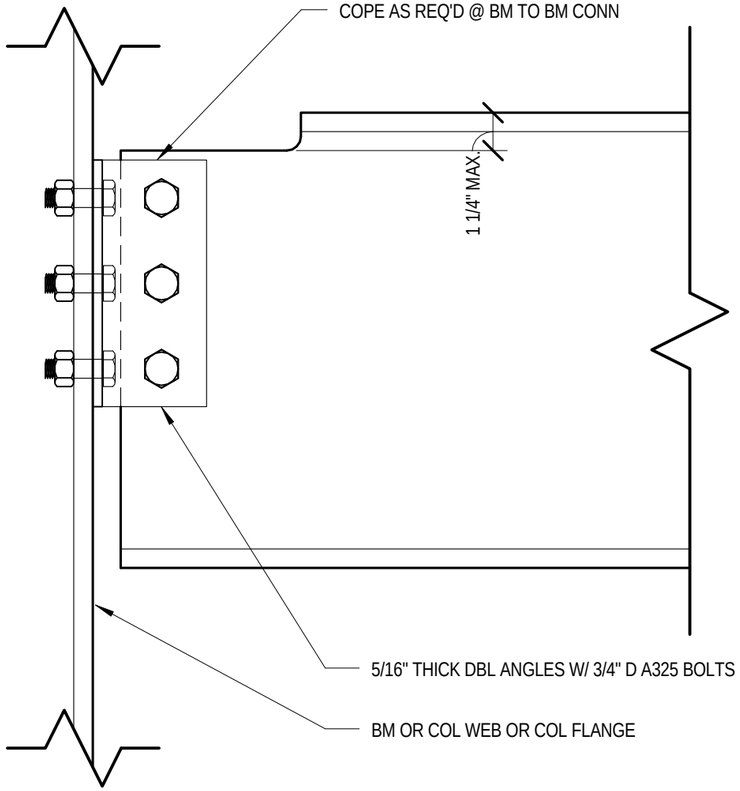
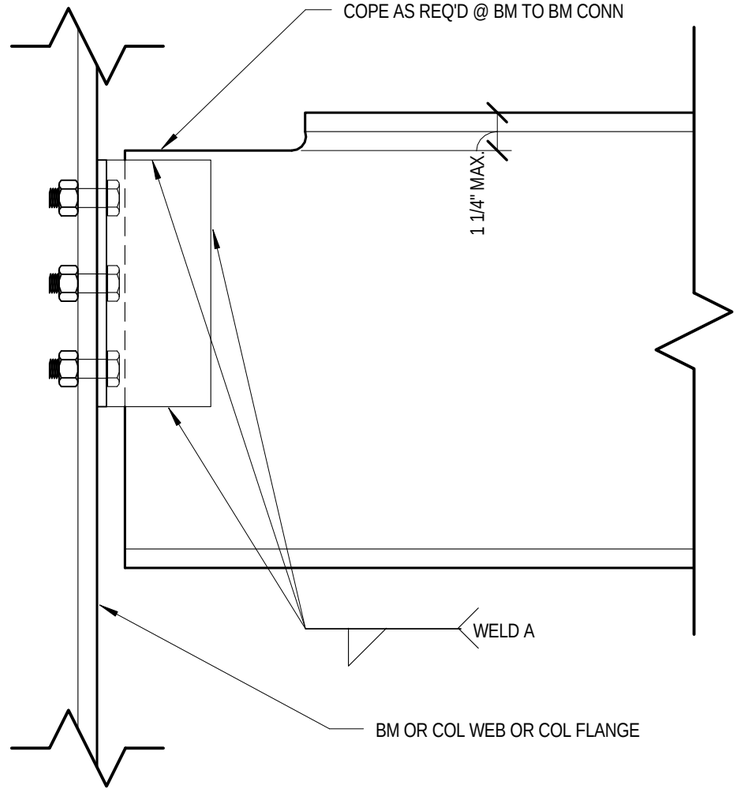
3/32" = 1'-0"

S102



TYPICAL BOLTED/BOLTED CONNECTION



TYPICAL BOLTED/WELDED CONNECTION

BOSTON ARCHITECTURAL COLLEGE

FOOD MARKET - BURN

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TYPICAL STEEL CONNECTIONS

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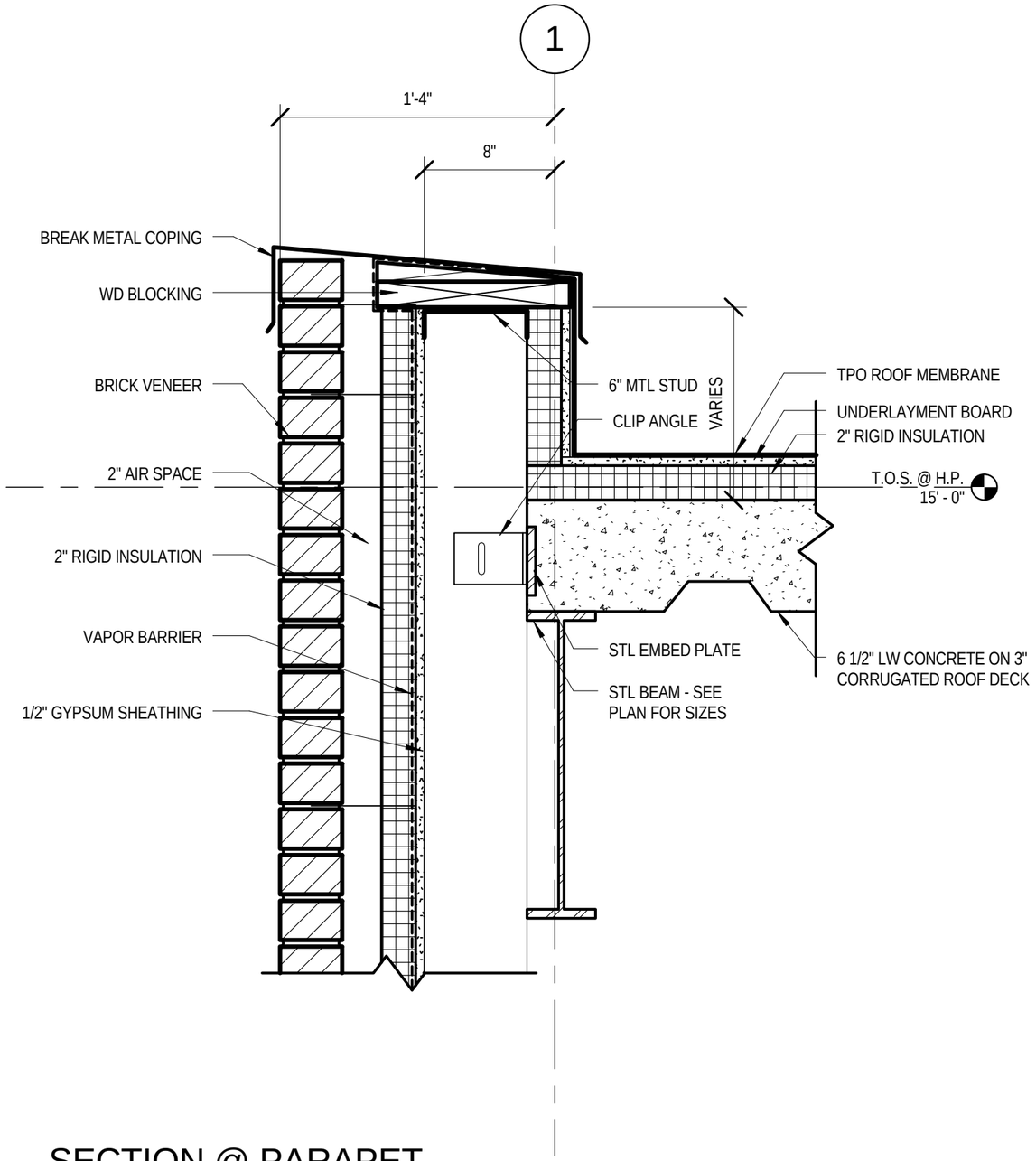
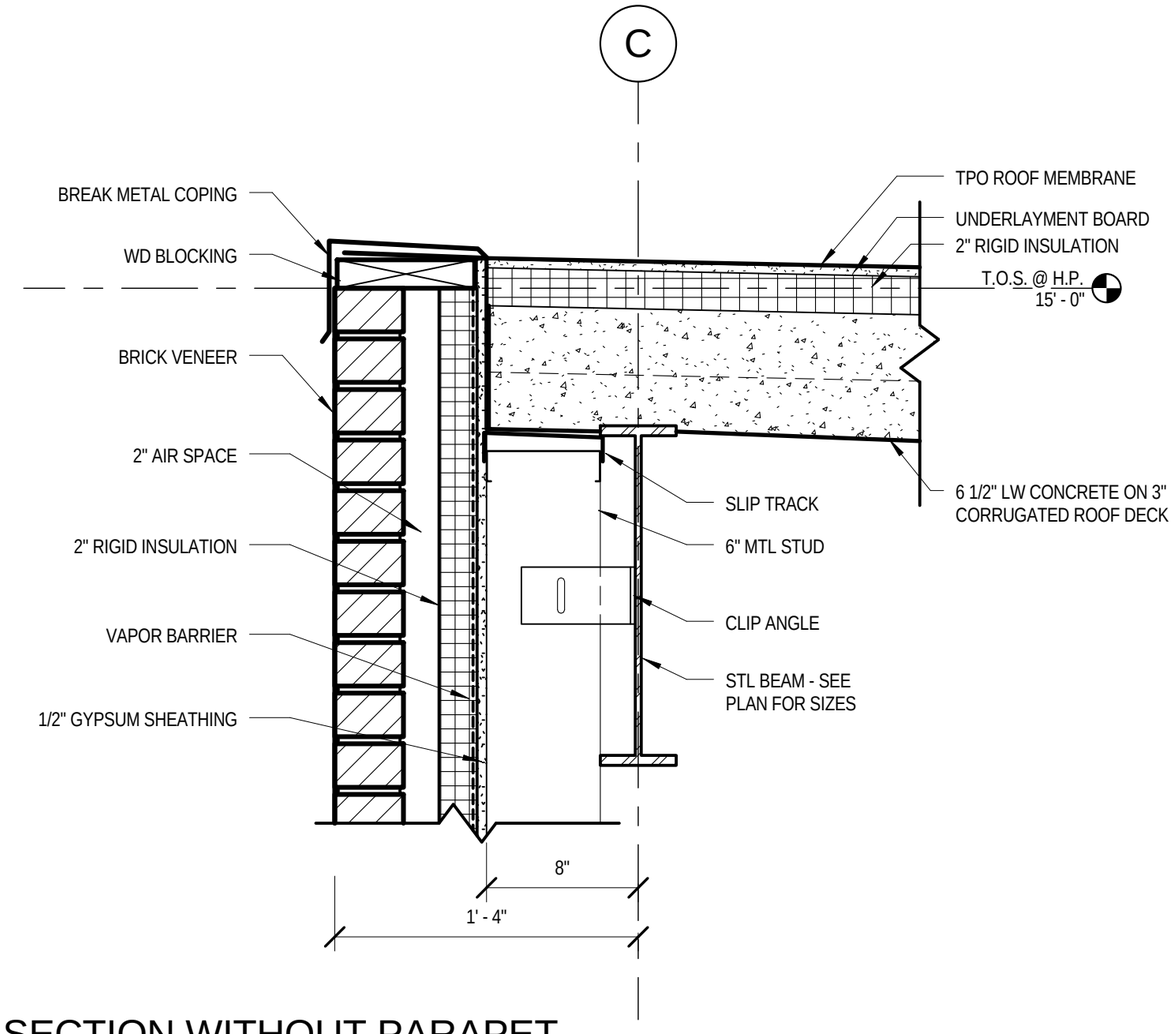
3" = 1'-0"

S103



1 SECTION @ PARAPET

1 1/2" = 1'-0"



2 SECTION WITHOUT PARAPET

1 1/2" = 1'-0"

BOSTON ARCHITECTURAL COLLEGE

FOOD MARKET - BURN

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ROOF EDGE SECTIONS

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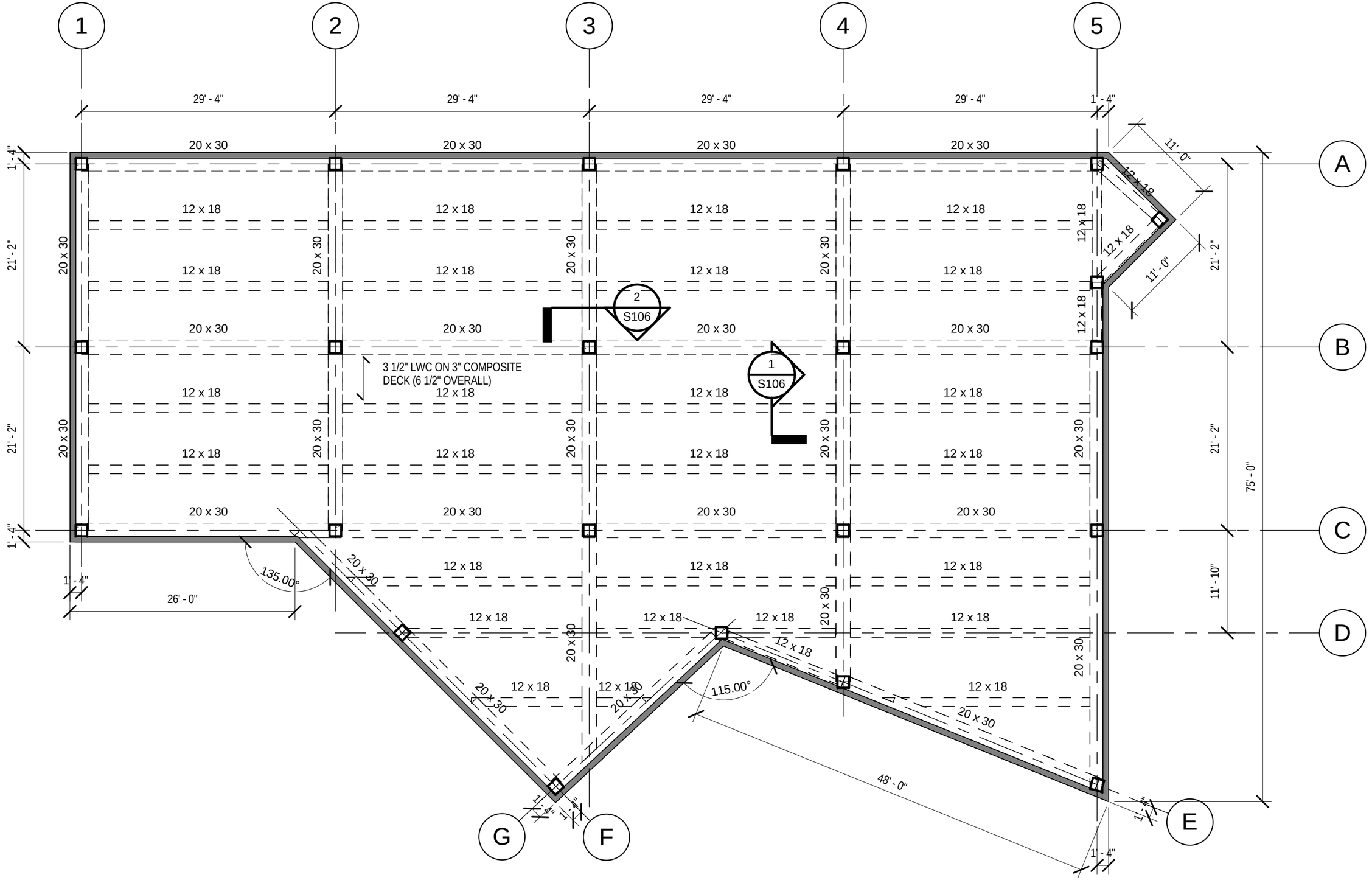
1 1/2" = 1'-0"

S104



BOSTON ARCHITECTURAL COLLEGE

FOOD MARKET - BURN

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ALTERNATE CONCRETE FRAMING

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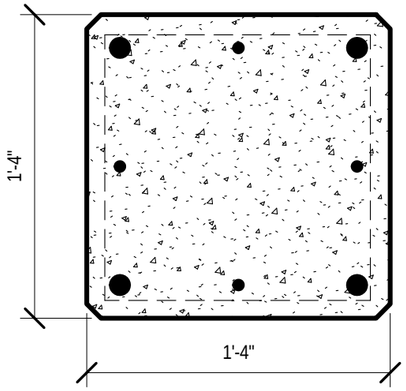
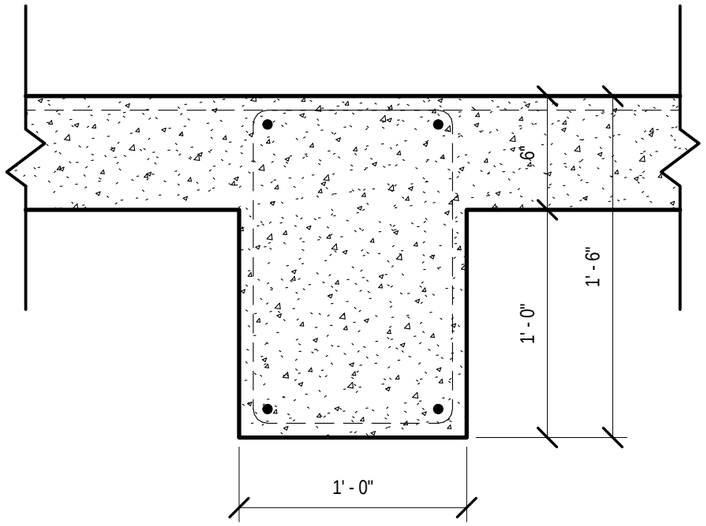
3/32" = 1'-0"

S105



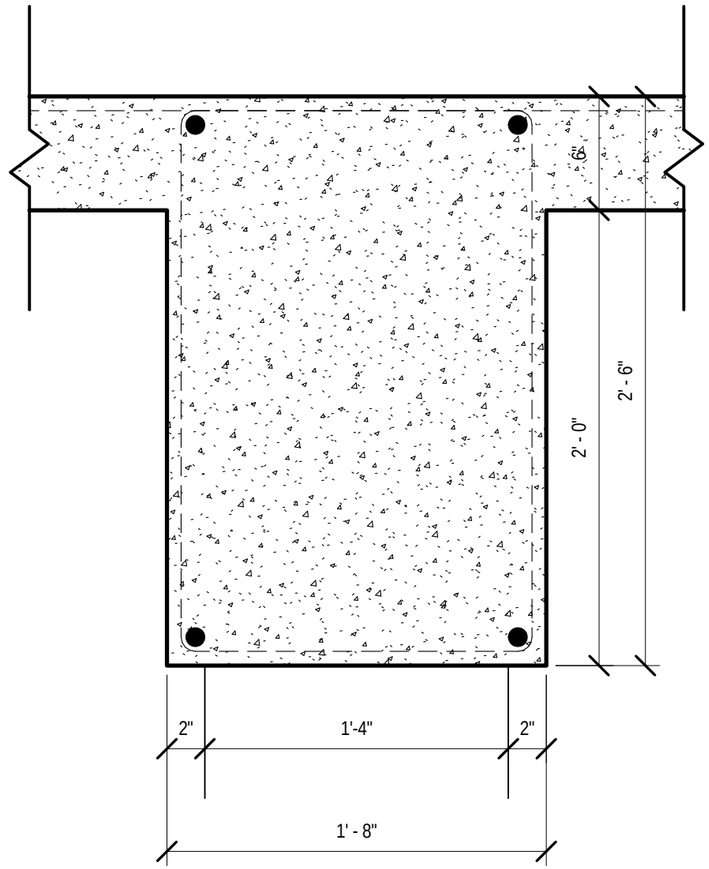
3 TYP. CONC. COLUMN SECTION

1 1/2" = 1'-0"



1 TYP. CONC. BEAM SECTION

1 1/2" = 1'-0"



2 TYP. CONC. GIRDER SECTION

1 1/2" = 1'-0"

BOSTON ARCHITECTURAL COLLEGE

FOOD MARKET - BURN

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TYPICAL CONCRETE DETAILS

18 OCTOBER 2011

1 1/2" = 1'-0"

S106

Typical Steel Beam Calculation:
Find lightest W12 section

Max. Beam Span: 29 Ft
Tributary Width: 7.5 Ft

Loads:

Dead Loads: 65 PSF
Live Loads: 45 PSF
W_D = 7.5 Ft x 65 PSF = 487.5 PLF
W_L = 7.5 Ft x 45 PSF = 337.5 PLF

Load Combinations:

W_D x 1.4 = W_{U1} = 487.5 PLF x 1.4 = 682.5 PLF
W_L x 1.6 + W_D x 1.2 = W_{U2} = (337.5 PLF x 1.6) + (487.5 PLF x 1.2) = 1,125 PLF

W_{U2} Controls

Max Moment:

M_{MAX} = (W_U x Max Span^2) / 8 = (1,125 PLF x (29 Ft)^2)/8 = 118,265/1000 = 118.26 K-Ft

$M_U = \phi$ x M_{MAX}; ϕ = 0.9; F_Y = 50 KSI
$M_U = \phi$ F_Y x $Z_X \Rightarrow$ (M_U x 12") / (ϕ F_Y) = Z_X

Z_X = (118.26 K-Ft x 12") / (0.9 x 50 KSI) = 31.53 In^3

∴ The lightest W12 Beam Choice is **W12x26**

Typical Steel Girder Calculation:
Find lightest W18 section

Max. Beam Span: 22 Ft
Tributary Width: 29 Ft

Loads:

Dead Loads: 65 PSF
Live Loads: 45 PSF
W_D = 29 Ft x 65 PSF = 1,885 PLF
W_L = 29 Ft x 45 PSF = 1,305 PLF

Load Combinations:

W_D x 1.4 = W_{U1} = 1,885 PLF x 1.4 = 2,639 PLF
W_L x 1.6 + W_D x 1.2 = W_{U2} = (1,305 PLF x 1.6) + (1,885 PLF x 1.2) = 4,350 PLF

W_{U2} Controls

Max Moment:

M_{MAX} = (W_U x Max Span^2) / 8 = (4,350 PLF x (22 Ft)^2)/8 = 263,175/1000 = 263.18 K-Ft

$M_U = \phi$ x M_{MAX}; ϕ = 0.9; F_Y = 50 KSI
$M_U = \phi$ F_Y x $Z_X \Rightarrow$ (M_U x 12") / (ϕ F_Y) = Z_X

Z_X = (263.18 K-Ft x 12") / (0.9 x 50 KSI) = 70.18 In^3

∴ The lightest W18 Girder Choice is **W18x40**

Typical Steel Column Calculation:
Find lightest W8 section

Pinned Connection ⇒ K = 1.0

L = 15'-0"
KL = 15'-0" x 1.0 = 15'-0" Effective Length
Dead Loads: 65 PSF
Live Loads: 45 PSF

Tributary Area:

$$\frac{29'\text{-}4" + 29'\text{-}4"}{2} \times \frac{21'\text{-}2" + 21'\text{-}2"}{2} = 610 \text{ SF}$$

Axial Loads:

P_D ⇒ Dead Load = 65 PSF x 610 SF = 39,650 LB

P_L ⇒ Live Load = 45 PSF x 610 SF = 27,450 LB

Design Load Combinations:

P_{U1} = 1.4 x P_D = (1.4 x 39,650 LB)=55,510 LB/1000=55.5 K
P_{U2} = 1.2 x P_D + 1.6 x P_L =(1.2 x 39,650 LB)+(1.6 + 27,450 LB)=91,500 LB/1000=91.5 K
P_{U2} Controls

Sizing:

Try **W8x21**
A = 6.16 in^2 r_x = 3.49 in r_y = 1.26 in

$$\frac{KL}{r_x} = \frac{15' \times 12"}{3.49} = \frac{KL}{r_x} = 51.57 \qquad \frac{KL}{r_y} = \frac{15' \times 12"}{1.26} = \frac{KL}{r_y} = 142.9$$

ϕF_c for 142.9 = 10.4 KSI

$$A_G = \frac{P_U}{\phi F_c} = \frac{91.5 \text{ K}}{10.4 \text{ KSI}} = 8.79 \text{ in}^2 > 6.16 \text{ in}^2$$

∴ **W8x21** is less than adequate.

Typical Column Calculation (cont.):

Try **W8x28**
A = 8.24 in^2 r_x = 3.45 in r_y = 1.62 in

$$\frac{KL}{r_x} = \frac{15' \times 12"}{3.45} = \frac{KL}{r_x} = 52.2 \qquad \frac{KL}{r_y} = \frac{15' \times 12"}{1.62} = \frac{KL}{r_y} = 111.1$$

ϕF_c for 111.1 = 17 KSI

$$A_G = \frac{P_U}{\phi F_c} = \frac{91.5 \text{ K}}{17 \text{ KSI}} = 4.59 \text{ in}^2 > 8.24 \text{ in}^2$$

∴ **W8x24** is more than adequate.

Try **W8x24**
A = 7.08 in^2 r_x = 3.42 in r_y = 1.61 in

$$\frac{KL}{r_x} = \frac{15' \times 12"}{3.42} = \frac{KL}{r_x} = 52.6 \qquad \frac{KL}{r_y} = \frac{15' \times 12"}{1.61} = \frac{KL}{r_y} = 111.8$$

ϕF_c for 111.8 = 17 KSI

$$A_G = \frac{P_U}{\phi F_c} = \frac{91.5 \text{ K}}{17 \text{ KSI}} = 5.4 \text{ in}^2 > 7.08 \text{ in}^2$$

∴ **W8x24** is adequate, and is the lightest W8 shape acceptable.

Design Wind Loads:

Design Wind Speed ⇒ 110 MPH
Building Occupancy Category ⇒ II (>300 Occupants)
Importance Factor ⇒ 1.00 (Occupancy Category II in Hurricane-prone region)
Wind Exposure ⇒ D (next to open water)
Adjustment Factor λ ⇒ 1.47 for 15'-0" Tall Building in Exposure D

Simplified Design Wind Pressure:

Per 110 MPH Wind Speed and 0-5 degree roof angle:

P_{S30} in Zone A ⇒ 19.2 PSF
P_{S30} in Zone C ⇒ 12.7 PSF

Calculations:

$P_S = \lambda \, I \, P_{S30}$

P_SZone A = 1.47 x 1.00 x 19.2 PSF = 28.224 PSF

P_SZone C = 1.47 x 1.00 x 12.7 PSF = 18.669 PSF