Organization
Introduction

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Design Wind Bracing Between Framing Columns 3N & 4N (Similar between 3O and 4O)

Assumptions : F_y = 50ksi F_u = 65ksi (Typical L shape is A36)

Level 1 to Level 2 - Using same design for L2 to L3 and L3 to L4.

Designed bracing to resist "tension only" due to Wind loads, Row N resists 50% of the East/West wind load and row O resists 50% of the load. Each bracing will not support any gravity loads. Assume that the East/West wind produces 40 psf.

East or West Building Area - from mid height on Level 1 to Level 6
Area between column rows * 3 plus 1 ft each side of columns rows times 3 ½ floors + 3 ft parapet. Add this to the area of Level 6. Multiply all of this by the force

[(3 * 24) + 2] * [(3.5 * 14)+ 3] + 13 (24 +2)
= [(74) * (52)] + [13 * 26] = 3848 + 338 = 4186 ft^2 * 40 $\#/ft^2$ = 167440 or 167.4^k

Each row of columns N and O will withstand ½ of the wind 167.4 / 2 = 83.7^k

Length of the diagonal = $\sqrt{(33^2 + 14^2)} = \sqrt{1285}$ = 35.85 ft

Each diagonal tension force = ratio * force = 35.85'/33' * 83.7^k = 90.9^k(t)

Gross section yielding

$P_n = F_y \times A_g$

LRFD Φ_t=0.90 & Load = 1.6W
Load = $\Phi_t * P_n = \Phi_t * F_y \times A_g$
A_g = Load/ $\Phi_t * F_y$
$A_g = 90.9^k * 1.6/(0.9 * 50ksi)$
$A_g = 145.44^k /45$ ksi = 3.23 in^2

ASD Ω_t=1.67 & Load = 1.0W
Load = $P_n/ \Omega_t = F_y * A_g/\Omega_t$
A_g = Load * Ω_t / F_y
$A_g = 90.9^k * 1.67/50ksi$
A_g = 151.8/50ksi = 3.04in^2

Tensile rupture strength
$P_n = F_u * A_e$

LRFD Φ_t=0.75
Load = $\Phi_t * P_n = \Phi_t * F_u * A_e$
A_e = Load/$\Phi_t * F_u$
$A_e = 90.9^k /0.75 * 65ksi$
$A_e = 90.9^k /48.75ksi = 1.86in^2$

ASD Ω_t=2.00
Load = $P_n/ \Omega_t = F_u * A_e/ \Omega_t$
A_e= Load * Ω_t/F_u
$A_e = 90.9^k * 2/65ksi$
$A_e = 181.8^k /65ksi = 2.80in^2$

$r_{x,y}$ minimum= L/300
$r_{x,y}$= 35.85 ft * 12 in/ft/300

$r_{x,y}$ minimum= L/300
$r_{x,y}$ = 430.2/300 = 1.434

Try L shape bracing:

L6x6x3/8 Wt. 14.9 #/ft $A_g = 4.38\ in^2$ r = 1.87 in

For ¾" A325-N bolts LRFD Capacity 15.9 ^{k}/bolt ASD Capacity 10.6 ^{k}/bolt

	LRFD	ASD
Diagonal Force/bolt Capacity	$145.44^k / 15.9^k = 9.1$ bolts	$90.9^k / 10.6^k = 8.6$ bolts

Use two lines of 5 bolts

Table D3.1 AISC Case 8 Single angle with more than 4 fasteners per line use U = 0.80

$A_e = U * A_n = 0.8\ [4.38\ in^2 - 2(3/8")\ (3/4" + 1/8")] = 0.8\ [4.38 - 0.656] = 0.8 * 3.723 = 2.98\ in^2$

This A_e meets the Tensile Rupture Strength requirements calculated with ASD.

<u>**Level 4 to Level 5**</u> - Both tension and compression.

Designed bracing to resist "both tension and compression" due to Wind loads, Row <u>N</u> resists 50% of the East/West wind load and row <u>O</u> resists 50% of the load. Each bracing will not support any gravity loads. Assume that the East/West wind produces 40 psf.

East or West Building Area - from mid height on Level 4 to Level 6

$[(3 * 24) + 2] * (16/2 + 3) + 13\ (24 + 2)$

$= [(74) * (11)] + [13 * 26] = 814 + 338 = 1152\ ft^2 * 40\ \#/ft^2 = 46080$ or 46.1^k

Each row of columns <u>N</u> and <u>O</u> will withstand ½ of the wind $46.1 / 2 = 23.1^k$

Length of the diagonal $= \sqrt{(33^2 + 14^2)} = \sqrt{1285} = 35.85$ ft

Each diagonal force = ratio * force $= 35.85'/33' * 23.1^k = 25.1^k$ (t & c)

ASD = 25.1^k (t & c) LRFD = $1.6 * 25.1^k = 40.2^k$ (t & c)

AISC Pg 4-54 , KL shown at 36' - HSS 7x7x3/16 ASD= 29.0 LRFD = 43.6

KL/r = 200 r = KL/200 $r = (1)*(35.85')*(12"/')/200 = 2.15"$

Try HSS 7 x 7 x 3/16; $A = 4.67\ in^2$; r=2.77 in; $F_y = 46$ ksi Wt = 17.06 #/ft (AISC pg 1-53)

Check $Kl/r = (1)*(35.85')*(12"/')/2.77 = 155.3$

$F_e = (\pi^2 * 29{,}000) / (155.3)^2 = 11.87\ ksi < .44*(46\ ksi) = 20.2\ ksi$ <u>**ELASTIC**</u>

$F_{cr} = .877\ F_e = .877 * 11.87\ ksi = 10.41\ ksi$ $P_n = F_{cr} \times A_g$

	LRFD $\Phi_c = 0.90$	ASD $\Omega_c = 1.67$
	$\Phi_c \times P_n$	P_n / Ω_c
	$0.9 * 10.41\ ksi * 4.67 = 43.7^k$	$10.41\ ksi * 4.67\ in^2/1.67 = 29.1^k$
Check Required/Capacity	40.2/43.7	25.1/29.1

– Use same HSS 7 x 7 x 3/16 for Level 5 to Level 6

Column Design

Assumptions : F_y = 50ksi F_u = 65ksi
Dead Load : 100 psf
Exterior Wall System : 50 psf (Supported mid-height of each floor)
Live Load : L2, L3, L4 = 120 psf
L5, L6 (L_r) = 60 psf
Wind Load: 40 psf (Service Level)
Seismic Load: None

Exterior non-corner columns - M2, M3, M4, M5, P2, P3, P4, P5, N1, N6, O1, O6
Exterior corner columns - M1, M6, P1, P6 all have lower loads but use same section as Exterior non-corner columns.
Non Rigid Frame Column Splice: Mid-Height between Level 2 and Level 3 only, from this splice to top is single unit.

ASD load cases	D + L	&	D + .75L + .75L_r
LRFD load cases	1.2D + 1.6L + 0.5L_r		

Exterior non-corner Column Area per level 33'(24'/2) = 33' (12') = 396 ft^2

Design Load above Splice for Exterior Non-Corner Columns : $L_3 + L_4 + L_5$
D: [396 ft^2 (100 psf + 100 psf + 100 psf)] + [33' * (14' + 14' + 10') * (50 psf)]
=118,800 + 62,700 =181,500 or 181.5^k
L: [396 ft^2 (120 psf + 120 psf)] = 95,040^k or 95.0^k
L_r : 396 ft^2 (60 psf) = 23,760^k or 23.8^k

ASD load cases
D + L = 181.5^k + 95.0^k = 276.5^k *** USE THIS LOAD ***
D + .75L + .75L_r = 181.5^k + [.75 * (95.0^k)] + [.75 * (23.8^k)]
= 181.5^k + 71.3^k + 17.9^k = 270.7^k

LRFD load cases
1.2D + 1.6L + 0.5L_r = [1.2 * 181.5^k] + [1.6 * (95.0^k)] + [0.5 * (23.8^k)]
= 217.8^k + 152.0^k + 11.9^k = 381.7^k *** USE THIS LOAD ***

Above Splice ASD & LRFD :

AISC 13th Pg 4-14; KL=14'; W14x61 (50 ksi)
ASD P_n/Ω_c = 380^k > 276.5^k OK
LRFD Φ_c x P_n = 572^k > 381.7^k OK

AISC 13th Pg 4-14; KL=14'; W14x53 (50 ksi)
ASD P_n/Ω_c = 267^k not > 276.5^k NOT OK
LRFD Φ_c x P_n = 401^k > 381.7^k OK

Check From Above Splice W14x61 with KL=14'
r_y = 2.45"; Area = 17.9 in^2; AISC Pg 1-22
Kl/r = (1)*(14')*(12"/')/2.45 = 68.6
$F_e = (\pi^2 * 29{,}000) / (68.6)^2$ = 60.82 ksi > .44*(50 ksi)=22.0 ksi **INELASTIC CAN NOT USE EULER**
$F_{cr} = (0.658^{50/60.82})$ *50 = $(0.658^{0.82209})$ * 50= 35.44 ksi $P_n = F_{cr} \times A_g$
LRFD Φ_c=0.90   ASD Ω_c=1.67
Φ_c x P_n P_n/Ω_c

$0.9 * 35.44 \text{ ksi} * 17.9 \text{ in}^2 = 570.9^k$ $35.44 \text{ ksi} * 17.9 \text{ in}^2/1.67 = 379.9^k$

Check Required/Capacity 382/571 277/380

Design Load from Base to Splice : $L_1 + L_2 + L_3 + L_4 + L_5$ (L_1 Supported by Foundation)

D: [396 ft² (100 psf + 100 psf + 100 psf + 100 psf)] + [33' * (14' + 14' + 14' + 10') * (50 psf)]
=158,400 + 85,800 =244,200 or 244.2^k

L: [396 ft² (120 psf + 120 psf + 120 psf)] = $142{,}560^k$ or 142.6^k

L_r : 396 ft² (60 psf) = 23.8^k

ASD load cases $D + L = 244.2^k + 142.6^k = \underline{386.8^k}$ *** USE THIS LOAD ***
$D + .75L + .75L_r = 244.2^k + [.75 * (142.6^k)] + [.75 * (23.8^k)]$
$= 244.2^k + 107.0^k + 17.85^k = 369.1^k$

LRFD load cases $1.2D + 1.6L + 0.5L_r = [1.2 * (244.2^k)] + [1.6 * (142.6^k)] + [0.5 * (23.8^k)]$
$= 293.04^k + 228.15^k + 11.9^k = \underline{533.1^k}$ *** USE THIS LOAD ***

From Base to Splice ASD & LRFD :

AISC 13th Pg 4-14; KL=14'; W14x68 (50 ksi) ASD $P_n/\Omega_c = 425^k > 387^k$ OK
LRFD $\Phi_c \times P_n = 639^k > 533^k$ OK

AISC 13th Pg 4-14; KL=14'; W14x61 (50 ksi) ASD $P_n/\Omega_c = 380^k$ NOT> 387^k NOT OK
LRFD $\Phi_c \times P_n = 572^k > 533^k$ OK

Check From Base to Splice W14x68 with KL=14'

r_y = 2.46"; Area = 20.0 in²; AISC Pg 1-22

$Kl/r = (1)*(14')*(12"/')/2.46 = 68.3$

$F_e = (\pi^2 * 29{,}000) / (68.3)^2 = 61.4 \text{ ksi} > .44*(50 \text{ ksi})=22.0 \text{ ksi}$ **INELASTIC CAN NOT USE EULER**

$F_{cr} = (0.658 \wedge 50/61.4) * 50 = (0.658^{0.81433}) * 50 = 35.56 \text{ ksi}$ $P_n = F_{cr} \times A_g$

LRFD Φ_c=0.90 ASD Ω_c=1.67

$\Phi_c \times P_n$ P_n/Ω_c

$0.9 * 35.56 \text{ ksi} * 20.0 \text{ in}^2 = 640.1^k$ $35.56 \text{ ksi} * 20.0 \text{ in}^2/1.67 = 425.9^k$

Check Required/Capacity 533/640 387/426

CONCLUSION Exterior Columns (Non-Corners) Above Splice use W14x61 (50 ksi)
Exterior Columns (Non-Corners) from Base to Splice use W14x68 (50 ksi)

Design Interior Columns : N_3; N_4; O_3; O_4
Splices 4 ft above L_2 and 4 ft above L_4 ; Column Area per level = 24' * 33' = 792 ft^2

Dead Loads (D)
Above L_4 Splice #1 - (L_5)[792 ft^2 * 100 psf] + (L_6)[396 ft^2 * 100 psf] + (Ext Wall)[13' * 33' * 50 psf]
79,200 + 39,600 + 21,450 = 140,250 or $\underline{140.2^k}$
Above L_2 Splice #2 - 140.2^k + [2 * (792 ft^2 * 100 psf)] = $\underline{298.6^k}$
Base above L_1 - 298.6^k * [792 ft^2 * 100 psf] = $\underline{377.8^k}$

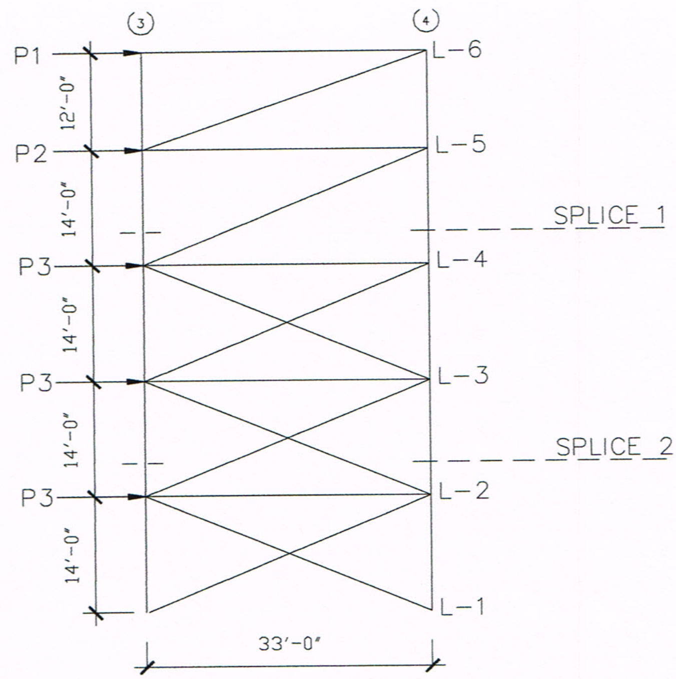
Floor Live Loads (L)
Above L_4 Splice #1 - (L_5)[396 ft^2 * 120 psf] = $\underline{47.5^k}$
Above L_2 Splice #2 - (L_5)47.5^k + (L_3 & L_4) [2 * (792 ft^2 * 120 psf)] = $\underline{237.6^k}$
Base above L_1 - 237.6^k * [792 ft^2 * 60 psf] = $\underline{285.1^k}$

Roof Live Load (L_r)
Above L_4 Splice #1 - [792 ft^2 * 60 psf] = $\underline{47.5^k}$
Above L_2 Splice #2 - = 47.5^k
Base above L_1 - = 47.5^k

P1 : 40psf (12/2 + 1) (24/2 + 1) = 3.6k

P2 : 40psf [(12/2 + 1) (24/2 + 1)
+ (72/2 * (14/2 +3)= 18k

P3 : 40psf (14) (72/2) = 20.2k



FRAMING ELEVATION
FRAMING LINE N & O

Wind Load : Splice #1 : [(3.6^k * 22') + (18^k * 10')] / 33' = $\underline{7.9^k}$
Splice #2 : [(3.6^k * 50') + (18^k * 38') + (20.2^k * (24' + 10')] / 33' = $\underline{47.0^k}$
Base : [(3.6^k * 68') + (18^k * 56') + (20.2^k * (18' + 28' + 42')] / 33' = $\underline{91.8^k}$

ASD load cases D + L ; D + L_r; .6D + W; D + W

Splice #1
D + L ; 140.2^k © + 47.5^k © = 187.7^k ©
D + L_r; 140.2^k © + 47.5^k © = 187.7^k © *** USE THIS LOAD ***
.6D + W; 0.6 * 140.2^k © + 7.9^k (T) = 76.2^k ©
.6D + W; 0.6 * 140.2^k © + 7.9^k © = 92.0^k ©
D + W; 140.2^k © + 7.9^k (T) = 132.3^k ©
D + W; 140.2^k © + 7.9^k © = 148.1^k ©

Splice #2
D + L ; 298.6^k © + 237.6^k © = 536.2^k © *** USE THIS LOAD ***
D + L_r; 298.6^k © + 47.5^k © = 346.1^k ©
.6D + W; 0.6 * 298.6^k © + 47.0^k (T) = 132.2^k ©
.6D + W; 0.6 * 298.6^k © + 47.0^k © = 226.2^k ©
D + W; 298.6^k © + 47.0^k (T) = 251.6^k ©
D + W; 298.6^k © + 47.0^k © = 345.6^k ©

Base
D + L ; 377.8^k © + 285.1^k © = 662.9^k © *** USE THIS LOAD ***
D + L_r; 377.8^k © + 47.5^k © = 425.3^k ©
.6D + W; 0.6 * 377.8^k © + 91.8^k (T) = 139.9^k ©
.6D + W; 0.6 * 377.8^k © + 91.8^k © = 318.5^k ©
D + W; 377.8^k © + 91.8^k (T) = 286.0^k ©
D + W; 377.8^k © + 91.8^k © = 469.6^k ©

ASD Analysis

Splice #1	187.7^k @ KL=14	W14x43	Pn= 212^k
Splice #2	536.2^k @ KL=14	W14x90	Pn= 682^k
Base	662.9^k @ KL=14	W14x99	Pn= 751^k

LRFD load cases 1.2D + 1.6L + 0.5L_r ; 1.2D + 0.5L + 1.6L_r ;
0.9D + 1.6W ; 1.2D + 1.6W + 0.5(L + L_r)

Splice #1
1.2D + 1.6L + 0.5L_r; 1.2(140.2^k ©) + 1.6(47.5^k ©) + 0.5(47.5^k ©) = 268.0^k ©
1.2D + 0.5L + 1.6L_r; 1.2(140.2^k ©) + 0.5(47.5^k ©) + 1.6(47.5^k ©) = 268.0^k © * USE THIS LOAD *
0.9D + 1.6W; 0.9(140.2^k ©) + 1.6(7.9^k (T)) = 113.5^k ©
0.9D + 1.6W; 0.9(140.2^k ©) + 1.6(7.9^k ©) = 138.8^k ©
1.2D + 1.6W + 0.5(L + L_r); 1.2(140.2^k ©) + 1.6(7.9^k (T)) + 0.5(47.5^k © + 47.5^k ©) = 203.1^k ©
1.2D + 1.6W + 0.5(L + L_r); 1.2(140.2^k ©) + 1.6(7.9^k ©) + 0.5(47.5^k © + 47.5^k ©) = 228.4^k ©

Splice #2
1.2D + 1.6L + 0.5L_r; 1.2(298.6^k ©) + 1.6(237.6^k ©) + 0.5(47.5^k ©) = 762.2^k ©*USE THIS LOAD *
1.2D + 0.5L + 1.6L_r; 1.2(298.6^k ©) + 0.5(237.6^k ©) + 1.6(47.5^k ©) = 553.1^k ©
0.9D + 1.6W; 0.9(298.6^k ©) + 1.6(47.0^k (T)) = 193.5^k ©
0.9D + 1.6W; 0.9(298.6^k ©) + 1.6(47.0^k ©) = 343.94^k ©
1.2D + 1.6W + 0.5(L + L_r); 1.2(298.6^k ©) + 1.6(47.0^k (T)) + 0.5(237.6^k © + 47.5^k ©) = 425.7^k ©
1.2D + 1.6W + 0.5(L + L_r); 1.2(298.6^k ©) + 1.6(47.0^k ©) + 0.5(237.6^k © + 47.5^k ©) = 576.1^k ©

Base
1.2D + 1.6L + 0.5L_r; 1.2(377.8^k ©) + 1.6(285.1^k ©) + 0.5(47.5^k ©) = 933.3^k © *USE THIS LOAD *
1.2D + 0.5L + 1.6L_r; 1.2(377.8^k ©) + 0.5(285.1^k ©) + 1.6(47.5^k ©) = 671.9^k ©
0.9D + 1.6W; 0.9(377.8^k ©) + 1.6(91.8^k (T)) = 193.1^k ©
0.9D + 1.6W; 0.9(377.8^k ©) + 1.6(91.8^k ©) = 486.9^k ©
1.2D + 1.6W + 0.5(L + L_r); 1.2(377.8^k ©) + 1.6(91.8^k (T)) + 0.5(285.1^k © + 47.5^k ©) = 472.8^k ©
1.2D + 1.6W + 0.5(L + L_r); 1.2(377.8^k ©) + 1.6(91.8^k ©) + 0.5(285.1^k © + 47.5^k ©) = 766.5^k ©

	LRFD			ASD
Splice #1 KL=14	$P_n = 268.0^k$ © W14x43		Splice #1 KL=14	$P_n = 187.7^k$ © W14x43
Splice #2 KL=14	$P_n = 762.2^k$ © W14x82		Splice #2 KL=14	$P_n = 536.2^k$ © W14x90
Base KL-14	$P_n = 933.3^k$ © W14x90		Base KL=14	$P_n = 662.9^k$ © W14x90

Check Above Splice #1 W14x43 with KL=14'

$r_y = 1.89''$; Area = 12.6 in^2; AISC Pg 1-22

$Kl/r = (1)*(14')*(12''/1')/1.89 = 88.9$

$F_e = (\pi^2 * 29{,}000) / (88.9)^2 = 36.2$ ksi > .44*(50 ksi)=22.0 ksi **INELASTIC CAN NOT USE EULER**

$F_{cr} = (0.658$ ^50/36.2) * 50 = $(0.658^{1.38122})$ * 50 = 28.05 ksi $P_n = F_{cr} \times A_g$

	LRFD Φ_c=0.90	ASD Ω_c=1.67
	$\Phi_c \times P_n$	P_n / Ω_c
	0.9 * 28.05 ksi * 12.6 in^2 = 318.1^k	28.05 ksi * 12.6 in^2/1.67 = 211.6^k
Check Required/Capacity	268/318	188/212

Check Above Splice #2 W14x90 with KL=14'

$r_y = 3.70''$; Area = 26.5 in^2; AISC Pg 1-22

$Kl/r = (1)*(14')*(12''/')/3.70 = 45.4$

$F_e = (\pi^2 * 29{,}000) / (45.4)^2 = 138.9$ ksi > .44*(50 ksi)=22.0 ksi **INELASTIC CAN NOT USE EULER**

$F_{cr} = (0.658$ ^50/138.9) * 50 = $(0.658^{0.35997})$ * 50 = 43.01 ksi $P_n = F_{cr} \times A_g$

	LRFD Φ_c=0.90	ASD Ω_c=1.67
	$\Phi_c \times P_n$	P_n / Ω_c
	0.9 * 43.01 ksi * 26.5 in^2 = 1025.8^k	43.01 ksi * 26.5 in^2/1.67 = 682.5^k
Check Required/Capacity	762/1026	536/682

Check Above Base W14x90 with KL=14'

$r_y = 3.70''$; Area = 26.5 in^2; AISC Pg 1-22

$Kl/r = (1)*(14')*(12''/')/3.70 = 45.4$

$F_e = (\pi^2 * 29{,}000) / (45.4)^2 = 138.9$ ksi > .44*(50 ksi)=22.0 ksi **INELASTIC CAN NOT USE EULER**

$F_{cr} = (0.658$ ^50/138.9) * 50 = $(0.658^{0.35997})$ * 50 = 43.01 ksi $P_n = F_{cr} \times A_g$

	LRFD Φ_c=0.90	ASD Ω_c=1.67
	$\Phi_c \times P_n$	P_n / Ω_c
	0.9 * 43.01 ksi * 26.5 in^2 = 1025.8^k	43.01 ksi * 26.5 in^2/1.67 = 682.5^k
Check Required/Capacity	933/1026	663/682

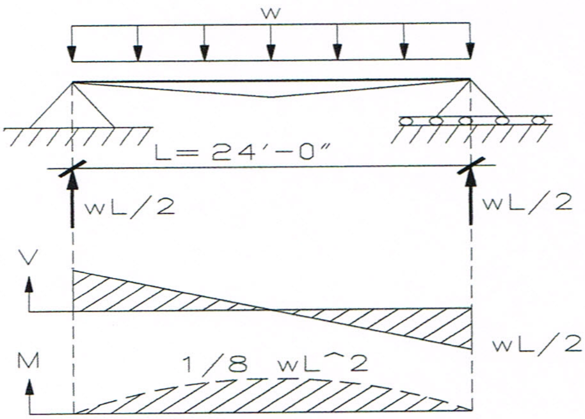
CONCLUSION   Interior Columns (N_3;N_4; O_3; O_4)

Above Splice #1 W14x43

Above Splice #2 to Splice #1 W14x90

Pn= Fcr X Area

Floor Beam Design for North/South Interior Beams Level 2, Level 3 & Level 4



Contributory Width = 33/3 = 11 ft

$M_{max} = wL^2/8$, $V_{max} = wL/2$, $\Delta_{max} = 5wL^4/384EI$ (AISC Pg 3-220)

Deflection Limits, For ASD & LRFD use un-factored loads

$\Delta_{Live} < 24*12/360 = 0.8$ in ; $\Delta_{Total} < 24*12/200 = 1.44$ in

LOADS

w_D=100psf * 11' = $1.1^{k/LF}$

w_L=120psf * 11' = $1.32^{k/LF}$

$w_T = 2.42^{k/LF}$

$w_{nD} = 1.2 * 1.1^{k/LF} = 1.32^{k/LF}$

$w_{nL} = 1.6 * 1.32^{k/LF} = 2.11^{k/LF}$

$w_{nT} = 3.43^{k/LF}$

I_{min} = max of $5w_LL^4*1728/384E*\Delta_{Live}$ or $5 w_T L^4*1728/384E*\Delta_{Total}$

I_{min} = max of $\frac{(5 * 1.32^{k/LF} * 24^4 * 1728)}{(384 * 29{,}000 * 0.8 \text{ in})}$ or $\frac{(5 * 2.42^{k/LF} * 24^4 * 1728)}{(384 * 29000 * 1.44)}$

= max of (425 in^4) or (433 in^4)

$I_{min} = 433 \text{ in}^4$

Designed as fully braced

$M_n = M_p = ZF_y$

LRFD

Load Case 1.2D + 1.6L

$M_{max} = 1/8 * 3.43 * 24^2 = 247.0^{ft\text{-}k}$

$V_{max} = 3.43 * 24 /2 = 41.2^k$

$Z_{required} = 247 * 12"/_{ft} /(.9 * 50ksi) = 65.9in^3$

$I_{required} = 433 \text{ in}^4$

ASD

Load Case D + L

$M_{max} = 1/8 * 2.42 * 24^2 = 174.2^{ft\text{-}k}$

$V_{max} = 2.42 * 24 /2 = 29.0^k$

$Z_{required} = 174.2 * 12"/_{ft} * 1.67 / 50ksi = 69.8in^3$

$I_{required} = 433 \text{ in}^4$

W18x35 A=10.3, d=17.7, t_w=0.300, $Z_{x\text{-}x} = 66.5 > 64$, $I_{x\text{-}x} = 510 > 433$

Check shear: LRFD ($d * t_w$ * ksi) > V_{max} ; ASD ($d * t_w$ * ksi/1.5) > V_{max}

LRFD (17.7 * 0.3 * 50) = 265 > 41.2 ; ASD (17.7 * 0.3 * 50/1.5) = 177 > 29.0

Floor Beam Design for North/South Exterior Beams Level 2, Level 3 & Level 4 (Cont.)

Contributory width floor = 11/2 = 5.5'
Contributory height exterior walls = 14'

w_D=(100psf * 5.5') + (50psf * 14') = $1.25^{k/LF}$
w_L=(120psf * 5.5' = $0.66^{k/LF}$
$w_T = 1.91^{k/LF} < 2.42^{k/LF}$ (Interior)

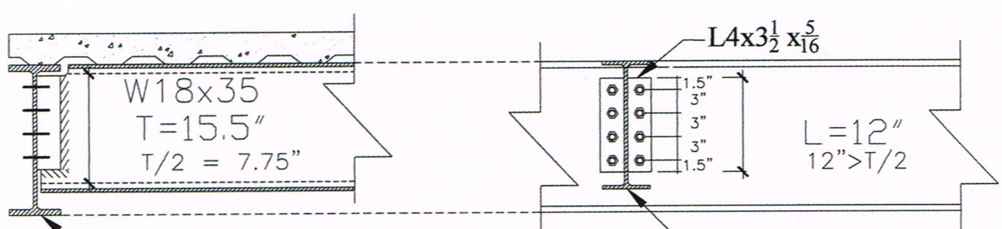
$w_{nD} = 1.2 * 1.25^{k/LF} = 1.50^{k/LF}$
$w_{nL} = 1.6 * 0.66^{k/LF} = 1.06^{k/LF}$
$w_{nT} = 2.56^{k/LF} < 3.43^{k/LF}$ (Interior)

Use W18x35 for both ASD and LRFD

Beam End Connections

¾" A325-N bolts — LRFD Capacity 15.9 k/bolt — ASD Capacity 10.6 k/bolt

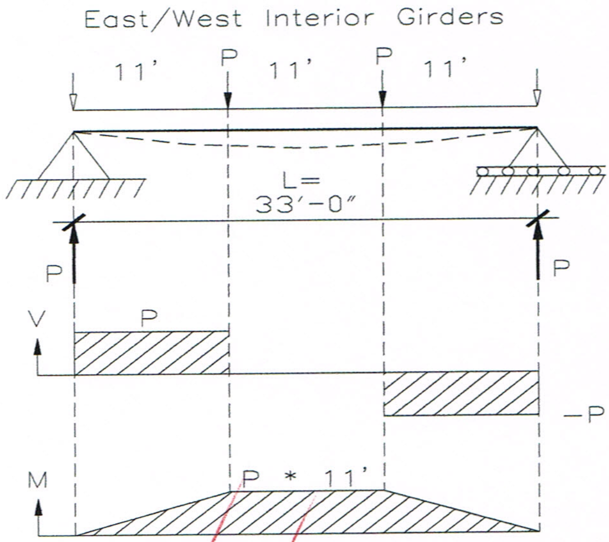
NORTH/SOUTH BEAMS PINN CONNECTION TO EAST/WEST GIRDERS



EAST/WEST GIRDER (NOT YET DESIGNED)
BOLTS ¾" A325-N

8 BOLTS (MINIMUM)
ASD 8 (10.6 k/bolt) = 84.8k > 29.0k
LRFD 8 (15.9 k/bolt) = 12k > 41.2k

East/West Girders Interior Level 2, Level 3 & Level 4



LOADS

P_D=100psf * 11' * 24'= 26.4^k

P_L=120psf * 11' * 24'= 31.7^k

P_T = 58.1^k

Δ_{Live} = 33 * 12/360 = 1.1

Δ_{Total} = 33 * 12/200 = 1.98

I_{min} = max of $0.036\ P_L L^3 * 1728/E * \Delta_{Live}$ or $0.036\ P_T L^3 * 1728/E * \Delta_{Total}$

I_{min} = max of $\frac{(0.036 * 31.7^k * 24^3 * 1728)}{(29{,}000 * 1.1\text{ in})}$ or $\frac{(0.036 * 58.1^k * 24^3 * 1728)}{(29{,}000 * 1.98)}$

= max of (855 in^4) or (870 in^4)

I_{min} = 870 in^4

M_{max} = P (L/3)

V_{max} = P

Δ_{max} = 0.036 PL^3/EI (AISC Pg 3-208)

LRFD

M_{nmax} = (1.2P_D + 1.6P_L) * 33'/3

= [1.2(26.4^k) + 1.6(31.7^k)] * 33/3 = 906.4$^{ft\text{-}k}$

V_{nmax} = (1.2P_D + 1.6P_L)

= [1.2(26.4^k) + 1.6(31.7^k)] = 82.4^k

$Z_{required}$ = 906.4$^{ft\text{-}k}$ *12"/ft /(.9 * 50ksi) = 242in^3

$I_{required}$ = 870 in^4

ASD

M_{max} = P_T (33'/3)

= 58.1^k (33/3) = 639.1$^{ft\text{-}k}$

V_{max} = P_T

= 58.1^k

$Z_{required}$ = 639.1$^{ft\text{-}k}$ *12"/ft * 1.67 / 50ksi = 256in^3

$I_{required}$ = 870 in^4

<u>W21x101</u> A=29.8, d=21.4, t_w=0.500, ;
$Z_{x\text{-}x}$ = 253 >242, $I_{x\text{-}x}$ = 2420 > 870 ;
Check shear: LRFD (d * t_w * ksi) > V_{nmax} ;
(21.4 * 0.5 * 50) = 535 > 82.4 ;

<u>W21x111</u> A=32.7, d=21.5, t_w=0.550,
$Z_{x\text{-}x}$ = 279 >256, $I_{x\text{-}x}$ = 2670 > 870
ASD (d * t_w * ksi/1.5) > V_{max}
(21.5 * 0.55 * 50/1.5) = 394 > 58.1

East/West Girders Exteriors Level 2, Level 3 & Level 4

Contributory width floor = 24'/2 = 12'
Contributory height exterior walls = 14' * 11'

P_D=(100psf * 12' * 11') + (50psf * 14' * 11') = 20.9^k $P_{nD} = 1.2 * 20.9^k = 25.1^k$
P_L=(120psf * 12' * 11') = 15.8^k $P_{nL} = 1.6 * 15.8^k = 25.3^k$
$P_T = 36.7^k$ $P_{nT} = 50.4^k$

I_{min} = max of $0.036\ P_L L^3 * 1728/E * \Delta_{Live}$ or $0.036\ P_T L^3 * 1728/E * \Delta_{Total}$

I_{min} = max of $\dfrac{(0.036 * 15.8^k * 24^3 * 1728)}{(29{,}000 * 1.1 \text{ in})}$ or $\dfrac{(0.036 * 36.7^k * 24^3 * 1728)}{(29{,}000 * 1.98)}$

= max of (425.9 in^4) or (549.6in^4)

I_{min} = 549.6 in^4

$M_{max} = P\ (L/3)$
$V_{max} = P$
$\Delta_{max} = 0.036\ PL^3/EI$ (AISC Pg 3-208)

LRFD
$M_{nmax} = (1.2P_D + 1.6P_L) * 33'/3$
$= [25.1^k + 25.3^k] * 11' = 554.4^{ft\text{-}k}$
$V_{nmax} = (1.2P_D + 1.6P_L) = 50.4^k$
$Z_{required} = 554.4^{ft\text{-}k} * 12"/_{ft} / (.9 * 50ksi) = 147.8in^3$
$I_{required} = 549.6\ in^4$

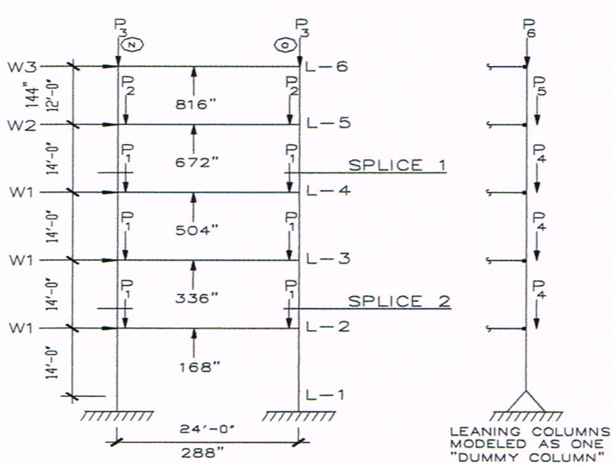
ASD
$M_{max} = P_T\ (33'/3)$
$= 36.7^k\ (11') = 403.7^{ft\text{-}k}$
$V_{max} = P_T = 36.7^k$
$Z_{required} = 403.7^{ft\text{-}k} * 12"/_{ft} * 1.67 / 50ksi = 161.8in^3$
$I_{required} = 549.6\ in^4$

W21x68 A=20.0, d=21.1, t_w=0.4300,
$Z_{x\text{-}x}$ = 160 >147.8, $I_{x\text{-}x}$ = 1480 > 549.6
Check shear: LRFD (d * t_w * ksi) > V_{nmax}
LRFD (21.1 * 0.43 * 50) = 454 > 50.4 ;

W21x73 A=21.5, d=21.2, t_w=0.455,
$Z_{x\text{-}x}$ = 172 >161.8, $I_{x\text{-}x}$ = 1600 > 549.6
ASD (d * t_w * ksi/1.5) > V_{max}
ASD (21.2 * 0.455 * 50/1.5) = 321 > 36.7

RIGID FRAME DESIGN @ $N_2 - O_2$ & $N_5 - O_5$

Resisting North/South Wind Loads & Providing North/South Stability



P-loads – Lump all vertical loads at end nodes, even though some load goes directly to rigid frame beams.

LRFD Load Case 1.6D + 1.2Wind + 0.5L

E = 29,000 ksi

Wind Loads

W_1 = 1.6 * Wind psf [½ length of building * height of exterior wall supported by each floor]

W_1 = 1.6 * 40psf [(½(5 * 33' + 2')) * 14')]

= 1.6 * 40psf [(½ * 167') * 14'] = 1.6 * 40psf * 1169ft² = $\underline{74.8^k}$

W_2 = 1.6 * Wind psf [½ (length of building * ½ height of exterior wall supported by each floor + parapet) + (½ length of the penthouse + 1') * ½ height of exterior wall penthouse]

W_2 = 1.6 * 40psf [[(½(5 * 33' + 2')) * (14/2' + 3')] + [½(3 * 33' + 2') * 12'/2]]

W_2 = 1.6 * 40psf [[(½ * 167') * 10'] + [(50.5' * 6')]] = 1.6 * 40psf [835ft² + 303ft²] = $\underline{72.8^k}$

W_3 = 1.6 * Wind psf [½ length of the penthouse + 1') * ½ height of exterior wall penthouse +1' parapet]

W_3 = 1.6 * 40psf [½(3 * 33' + 2') * (12'/2 + 1')]

W_3 = 1.6 * 40psf [50.5' * 7'] = 1.6 * 40psf * 353.5ft² = $\underline{22.6^k}$

Vertical Loads from 1.2D + 0.5L

P_1 = 1.2 * [24' * 33' * 100psf] + 0.5[24' * 33' * 120psf]

= $95.04^k + 47.52^k = \underline{142.56^k}$

P_2 = 1.2[(24' * 33' * 100psf)+(6'(12' + 16.5') *50psf)] + 0.5 [24' * 33' *60psf]

= 1.2 [$79.2^k + 8.55^k$] + 0.5 [47.5^k] = $105.3^k + 23.7^k = \underline{129.0^k}$

P_3 = 1.2 [(12' * 16.5' * 100psf) + ((6'+1')(12' + 16.5') * 50psf)] + 0.5 [12' * 16.5' * 60psf]

= 1.2 [$19.8^k + 10.0^k$] + 0.5 [11.9^k] = $35.76^k + 5.95^k = \underline{41.7^k}$

All factored load at Level 2, 3, or 4 for 50% of Bldg not directly applied to rigid frame as P_4. Cut the building in two symmetrical halves North to South, each half of building is supported by rigid frame in that area: 72' x 82.5' = 5,940 ft², with perimeter : (2 * 82.5') + 72' = 237' ; Penthouse dimensions 24' x 49.5' = 1,188 ft², with perimeter : (2 * 49.5') + 24' =123'

P_4 = 1.2 [(5,940 ft² * 100 psf) + (14' * 237' * 50psf)] + 0.5 [5,940 ft² * 120 psf] – 2 * P_1

= 1.2 [$594^k + 165.9^k$] + 0.5 [712.8^k] – 2 [142.6^k] = $911.88^k + 356.4^k - 285.2^k = \underline{983.1^k}$

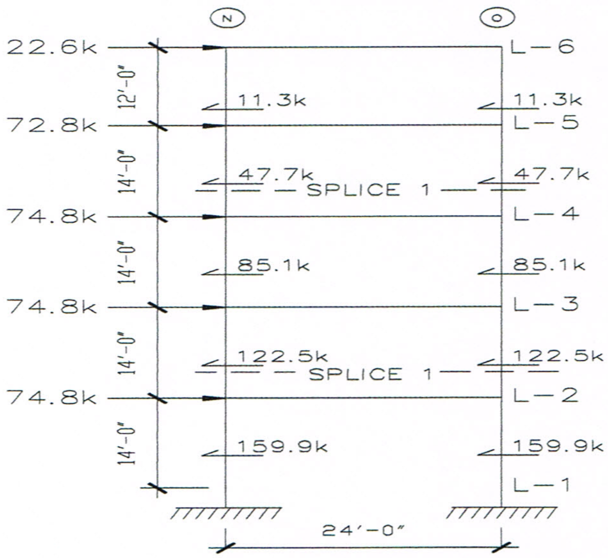
P_5 = 1.2[(5,940 ft² *100 psf)+((7' + 3') *237' *50psf)+(6' *123' *50psf)]+0.5[5,940 ft² *60 psf]– 2 * P_2

= 1.2 [$594^k + 118.5^k + 36.9^k$] + 0.5 [356.4^k] – 2 [129^k] = $899.3^k + 178.2^k - 258.0^k = \underline{819.5^k}$

P_6 = 1.2[(1,188 ft² *100 psf) + ((6' + 1') * 123' * 50psf)] + 0.5[1,188 ft² * 60 psf] – 2 * P_3

= 1.2 [$118.8^k + 43.0^k$] + 0.5 [71.3^k] – 2 [41.7^k] = $194.2^k + 35.7^k - 83.4^k = \underline{146.5^k}$

Preliminary Design – Portal Method, ignoring P–Δ



<u>L_5 – L_6 Columns and L_6 Beam</u>

$M = 12'/2 * 11.3^{k} = 67.8^{ft\text{-}k}$

<u>L_4 – L_5 Columns</u>

$M = 14'/2 * 47.7^{k} = 333.9^{ft\text{-}k}$

<u>L_5 Beam</u>

$333.9^{ft\text{-}k} + 67.8^{ft\text{-}k} = 401.7^{ft\text{-}k}$

<u>L_3 – L_4 Columns</u>

$M = 14'/2 * 85.1^{k} = 595.7^{ft\text{-}k}$

<u>L_4 Beam</u>

$595.7^{ft\text{-}k} + 333.9^{ft\text{-}k} = 929.7^{ft\text{-}k}$

<u>L_2 – L_3 Columns</u>

$M = 14'/2 * 122.5^{k} = 857.5^{ft\text{-}k}$

<u>L_3 Beam</u>

$857.5^{ft\text{-}k} + 595.7^{ft\text{-}k} = 1453.2^{ft\text{-}k}$

<u>L_1 – L_2 Columns</u>

$M = 14'/2 * 159.9^{k} = 1119.3^{ft\text{-}k}$

<u>L_2 Beam</u>

$1119.3^{ft\text{-}k} + 857.5^{ft\text{-}k} = 1976.8^{ft\text{-}k}$

<u>Rigid Frame Beam Selection</u>

Ignoring Axial Force and P–Δ ; Assumed fully braced

$L_6\ Z_{required} = (67.8^{ft\text{-}k}) * (12''/_{ft}) / (.9 * 50ksi) = 18.1\ in^3$	W14x68, Z=115 in^3
$L_5\ Z_{required} = (401.7^{ft\text{-}k}) * (12''/_{ft}) / (.9 * 50ksi) = 107.1\ in^3$	W21x83, Z=196 in^3
$L_4\ Z_{required} = (929.7^{ft\text{-}k}) * (12''/_{ft}) / (.9 * 50ksi) = 247.9\ in^3$	W21x132, Z=333 in^3
$L_3\ Z_{required} = (1453.2^{ft\text{-}k}) * (12''/_{ft}) / (.9 * 50ksi) = 387.5\ in^3$	W30x148, Z=500 in^3
$L_2\ Z_{required} = (1976.8^{ft\text{-}k}) * (12''/_{ft}) / (.9 * 50ksi) = 527.1\ in^3$	W30x191, Z=675 in^3

In addition to the strength requirements limit the L_6 lateral movement to height/150 due to factored loads, including P–Δ, 816" / 150 = 5.44"

Rigid Frame Preliminary Column Design

Below Splice #2 to Column Base ($L_1 - L_2$)

$M_{prelim} = 1119.3^{ft\text{-}k}$
Forces at base of L_1 due to Wind = [W_1 * (distance from base to Level 2 + Level 3 + Level 4) + (W_2 * distance from base to Level 5) + (W_3 * distance from base to Level 6)] / distance between rigid frame columns

$= [74.8^k * (14' + 28' + 42') + (72.8^k * 56') + (22.6^k * 68')] / 24'$
$= [6283.2^{ft\text{-}k} + 4076.8^{ft\text{-}k} + 1536.8^{ft\text{-}k}] / 24' = \underline{495.7^k}$ (due to 1.6W)

$P_{Down} = 3P_1 + P_2 + P_3$ + Axial force from wind overturning
$P_{Down} = 3(142.56^k) + 129.0^k + 41.7^k + 495.7^k = \underline{1094.1^k}$
(Note: $P_{min} = .9P_{Down} = 984.7^k > 495.5^k$ therefore No Uplift)

At this point assuming $M_n = M_{prelim}$
Using this Moment only $Z_{required} = (1119.3^{ft\text{-}k} * 12''/_{ft}) / (.9 * 50ksi) = 298.9\ in^3$
Start with W24x192, A= 56.3 in^2, I = 6260 in^4, r_y = 3.07", r_x = 10.5", Z = 559 in^3

Assuming the weak axis for buckling governs (to be check later)
$Kl/r = (1 * 14' * 12''/_{ft}) / 3.07'' = 54.7$
$F_e = (\pi^2 * 29{,}000) / (54.7)^2 = 95.7\ ksi > .44*(50\ ksi)=22\ ksi$ **INELASTIC CAN NOT USE EULER**
$F_{cr} = (0.658^{\ 50/95.7}) * 50 = (0.658^{0.52247}) * 50 = 40.18\ ksi$

LRFD Φ_c=0.90
$\Phi_c \times P_n$ ($P_n = F_{cr} \times A_g$)
$0.9 * 40.18\ ksi * 56.3\ in^2 = 2035.9^k$
Check Required/Capacity $1094.1^k / 2035.9^k = 0.537$

$M_u/\Phi M_n = (1119.3^{ft\text{-}k} * 12''/_{ft}) / (.9 * 559\ in^3 * 50ksi) = 0.534$
(Not including P–Δ 0.537 + 0.534 = 1.07 > 1 Not OK! $1.07 * 192\ ^{\#}/_{ft} = 205.6\ ^{\#}/_{ft}$

Try W24x229, A= 67.2 in^2, I = 7650 in^4, r_y = 3.11", r_x = 10.7", Z = 675 in^3
$Kl/r = (1 * 14' * 12''/_{ft}) / 3.11'' = 54.0$
$F_e = (\pi^2 * 29{,}000)/(54.0)^2 = 98.1\ ksi > .44*(50\ ksi)=22\ ksi$ **STILL INELASTIC CAN NOT USE EULER**
$F_{cr} = (0.658^{\wedge 50/98.1}) * 50 = (0.658^{0.5096}) * 50 = 40.39\ ksi$

LRFD Φ_c=0.90
$\Phi_c \times P_n$ ($P_n = F_{cr} \times A_g$)
$0.9 * 40.39\ ksi * 67.2\ in^2 = 2443.1^k$
Check Required/Capacity $1094.1^k / 2443.1^k = 0.448$

$M_u/\Phi M_n = (1119.3^{ft\text{-}k} * 12''/_{ft}) / (.9 * 675\ in^3 * 50ksi) = 0.442$
(Not including P–Δ 0.448 + 0.442 = 0.89 < 1 OK!

For Preliminary Design from Base to Splice 2 use W24x229, A= 67.2 in^2, I = 7650 in^4, r_y = 3.11", r_x = 10.7", Z = 675 in^3

Splice #2 to Splice #1 ($L_2 - L_3$)

M_{L2-L3} Columns = $857.5^{ft\text{-}k}$;
Forces at Splice #2 due to Wind = [W_1 * (distance from Splice #2 to Level 3 + Level 4) + (W_2 * distance from Splice #2 to Level 5) + (W_3 * distance from Splice #2 to Level 6)] / distance between rigid frame columns

= [74.8^k * (7' + 21') + (72.8^k * 35') + (22.6^k * 49')] / 24'
= [$2094.4^{ft\text{-}k}$ + $2548.0^{ft\text{-}k}$ + $1107.4^{ft\text{-}k}$] / 24' = 239.6^k (due to 1.6W)

$P_{Down} = 2P_1 + P_2 + P_3$ + Axial force from wind overturning
$P_{Down} = 2(142.56^k) + 129.0^k + 41.7^k + 239.6^k$ = <u>695.4^k</u>
(Note: $P_{min} = .9P_{Down} = 625.9^k > 239.6^k$ therefore No Uplift)

At this point assuming $M_n = M_{prelim}$
Using this Moment only $Z_{required} = (857.5^{ft\text{-}k} * 12''/_{ft}) / (.9 * 50ksi) = 228.7\ in^3$

Start with W24x146, A= 43.0 in^2, I = 4580 in^4, r_y = 3.01", r_x = 10.3", Z = 418 in^3
Assuming the weak axis for buckling governs (to be check later)
$Kl/r = (1 * 14' * 12''/_{ft}) / 3.01'' = 55.8$
$F_e = (\pi^2 * 29{,}000) / (55.8)^2 = 91.9\ ksi \geq .44*(50\ ksi) = 22.0\ ksi$ **<u>INELASTIC CAN NOT USE EULER</u>**
$F_{cr} = (0.658^{\ 50/91.9}) * 50 = (0.658^{0.54407}) * 50 = 39.82\ ksi$

LRFD Φ_c=0.90
$\Phi_c \times P_n$ ($P_n = F_{cr} \times A_g$)
0.9 * 39.82 ksi * 43.0 in^2 = 1541.0^k
Check Required/Capacity $695.4^k / 1541.0^k$ = 0.451

$M_u/\Phi M_n = (857.5^{ft\text{-}k} * 12''/_{ft}) / (.9 * 418\ in^3 * 50ksi) = 0.547$
(Not including P–Δ 0.451 + 0.442 = 0.998 < 1 <u>OK!</u>

<u>For Preliminary Design from Splice #2 to Splice #1 use W24x146, A= 43.0 in^2, I = 4580 in^4, r_y = 3.01", r_x = 10.3", Z = 418 in^3</u>

Splice #1 to Top ($L_4 - L_5$)

M_{L4-L5} Columns = $333.9^{ft\text{-}k}$;
Forces at Splice #1 due to Wind = [(W_2 * distance from Splice #1 to Level 5) + (W_3 * distance from Splice #1 to Level 6)] / distance between rigid frame columns

= [(72.8^k * 7') + (22.6^k * 19')] / 24'
= [$551.6^{ft\text{-}k}$ + $429.4^{ft\text{-}k}$] / 24' = 40.9^k (due to 1.6W)

$P_{Down} = P_2 + P_3$ + Axial force from wind overturning
$P_{Down} = 129.0^k + 41.7^k + 40.9^k$ = <u>211.6^k</u>
(Note: $P_{min} = .9P_{Down} = 190.44^k > 40.9^k$ therefore No Uplift)

At this point assuming $M_n = M_{prelim}$

Using this Moment only $Z_{required} = (333.9^{ft\text{-}k} * 12''/_{ft}) / (.9 * 50ksi) = 89.04\ in^3$

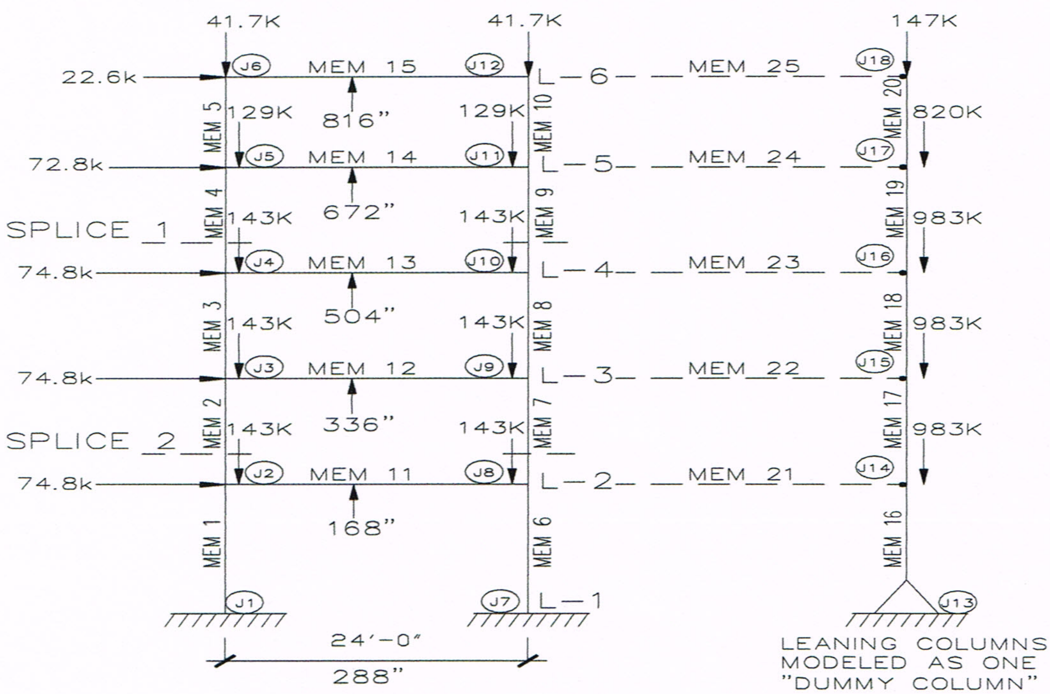
Start with W24x76, $A = 22.4\ in^2$, $I = 2100\ in^4$, $r_y = 1.92''$, $r_x = 9.69''$, $Z = 200\ in^3$
Assuming the weak axis for buckling governs (to be check later)
$Kl/r = (1 * 14' * 12''/_{ft}) / 1.92'' = 87.5$
$F_e = (\pi^2 * 29{,}000) / (87.5)^2 = 37.38\ ksi > .44*(50\ ksi) = 22.0\ ksi$ **INELASTIC CAN NOT USE EULER**
$F_{cr} = (0.658^{50/37.38}) * 50 = (0.658^{1.3376}) * 50 = 28.56\ ksi$

LRFD Φ_c=0.90
$\Phi_c \times P_n$ ($P_n = F_{cr} \times A_g$)
$0.9 * 28.56\ ksi * 22.4\ in^2 = 575.9^k$
Check Required/Capacity $211.6^k / 575.9^k = 0.367$

$M_u/\Phi M_n = (333.9^{ft\text{-}k} * 12''/_{ft}) / (.9 * 200\ in^3 * 50ksi) = 0.445$
(Not including P–Δ $0.367 + 0.445 = 0.812 < 1$ OK!

For Preliminary Design from Splice #1 to Top use W24x76, $A = 22.4\ in^2$, $I = 2100\ in^4$, $r_y = 1.92''$, $r_x = 9.69''$, $Z = 200\ in^3$

Summary of Preliminary Design for computer model :
Computer Model – 18 nodes (joints), 25 members, 15 nodes loaded, none pin ended.



Test the following use of beams and columns in the PLFNL2 program.

L_6 $Z_{required}$ use W14x68, Z=115 in^3, A= 20.0 in^2, I= 722 in^4
L_5 $Z_{required}$ use W21x83, Z=196 in^3, A= 24.3 in^2, I=1830 in^4
L_4 $Z_{required}$ use W21x132, Z=333 in^3, A= 38.8 in^2, I=3220 in^4
L_3 $Z_{required}$ use W30x148, Z=500 in^3, A= 43.5 in^2, I=6680 in^4
L_2 $Z_{required}$ use W30x191, Z=675 in^3, A= 56.3 in^2, I=9200 in^4
Splice #1 to top – W24x76, Z = 200 in^3, A= 22.4 in^2, I = 2100 in^4
Splice #2 to Splice #1 --W24x146, Z = 418 in^3, A=43.0 in^2, I = 4580 in^4
Base to Splice #1 – W24x229, Z = 675 in^3, A= 67.2 in^2, I = 7650 in^4

In addition to the strength requirements limit the L_6 lateral movement to height/150 due to factored loads, including P–Δ, 816" / 150 = 5.44" Computer model calculated 4.77" on Joints 6, 12, & 18 O.K.

PLFNL2 OUTPUT:

```
NUMBER OF JOINTS =    18
NUMBER OF MEMBERS =    25
NUMBER OF JOINTS WITH APPLIED LOAD(S) =     15
MODULUS OF ELASTICITY =     0.290E+05
NUMBER OF PIN END MEMBERS =      0    ( -1 INDICATES TRUSS)
NPD=     1(1 MEANS CALCULATE P-DELTA EFFECTS)
```

JOINT	X	Y	SX	SY (SPRING CONSTANTS)	SRZ
1	0.0000E+00	0.0000E+00	0.100E+31	0.100E+31	0.100E+31
2	0.0000E+00	0.1680E+03	0.000E+00	0.000E+00	0.000E+00
3	0.0000E+00	0.3360E+03	0.000E+00	0.000E+00	0.000E+00
4	0.0000E+00	0.5040E+03	0.000E+00	0.000E+00	0.000E+00
5	0.0000E+00	0.6720E+03	0.000E+00	0.000E+00	0.000E+00
6	0.0000E+00	0.8160E+03	0.000E+00	0.000E+00	0.000E+00
7	0.2880E+03	0.0000E+00	0.100E+31	0.100E+31	0.100E+31
8	0.2880E+03	0.1680E+03	0.000E+00	0.000E+00	0.000E+00
9	0.2880E+03	0.3360E+03	0.000E+00	0.000E+00	0.000E+00
10	0.2880E+03	0.5040E+03	0.000E+00	0.000E+00	0.000E+00
11	0.2880E+03	0.6720E+03	0.000E+00	0.000E+00	0.000E+00
12	0.2880E+03	0.8160E+03	0.000E+00	0.000E+00	0.000E+00
13	0.4000E+03	0.0000E+00	0.100E+31	0.100E+31	0.000E+00
14	0.4000E+03	0.1680E+03	0.000E+00	0.000E+00	0.000E+00
15	0.4000E+03	0.3360E+03	0.000E+00	0.000E+00	0.000E+00
16	0.4000E+03	0.5040E+03	0.000E+00	0.000E+00	0.000E+00
17	0.4000E+03	0.6720E+03	0.000E+00	0.000E+00	0.000E+00
18	0.4000E+03	0.8160E+03	0.000E+00	0.000E+00	0.000E+00

MEMBER	J NODE	K NODE	AREA	INERTIA
1	1	2	0.672E+02	0.765E+04
2	2	3	0.430E+02	0.458E+04
3	3	4	0.430E+02	0.458E+04
4	4	5	0.224E+02	0.210E+04
5	5	6	0.224E+02	0.210E+04
6	7	8	0.672E+02	0.765E+04
7	8	9	0.430E+02	0.458E+04
8	9	10	0.430E+02	0.458E+04
9	10	11	0.224E+02	0.210E+04
10	11	12	0.224E+02	0.210E+04
11	2	8	0.563E+02	0.920E+04
12	3	9	0.435E+02	0.668E+04
13	4	10	0.388E+02	0.322E+04
14	5	11	0.243E+02	0.183E+04
15	6	12	0.200E+02	0.722E+03
16	13	14	-0.100E+03	0.100E+01
17	14	15	-0.100E+03	0.100E+01
18	15	16	-0.100E+03	0.100E+01
19	16	17	-0.100E+03	0.100E+01
20	17	18	-0.100E+03	0.100E+01
21	8	14	-0.100E+03	0.100E+01
22	9	15	-0.100E+03	0.100E+01
23	10	16	-0.100E+03	0.100E+01
24	11	17	-0.100E+03	0.100E+01
25	12	18	-0.100E+03	0.100E+01

MEMBER LOADS

MEMBER	W AT J	W AT K	P1	D1	P2	D2	P3	D3

JOINT	XFORCE	YFORCE	ZMOM
2	0.748E+02	-0.143E+03	0.000E+00
3	0.748E+02	-0.143E+03	0.000E+00
4	0.748E+02	-0.143E+03	0.000E+00
5	0.728E+02	-0.129E+03	0.000E+00
6	0.226E+02	-0.417E+02	0.000E+00
8	0.000E+00	-0.143E+03	0.000E+00
9	0.000E+00	-0.143E+03	0.000E+00
10	0.000E+00	-0.143E+03	0.000E+00
11	0.000E+00	-0.129E+03	0.000E+00
12	0.000E+00	-0.417E+02	0.000E+00
14	0.000E+00	-0.983E+03	0.000E+00
15	0.000E+00	-0.983E+03	0.000E+00
16	0.000E+00	-0.983E+03	0.000E+00
17	0.000E+00	-0.820E+03	0.000E+00
18	0.000E+00	-0.147E+03	0.000E+00

OUTPUT NOT INCLUDING P-DELTA EFFECTS

JOINT DISPLACEMENTS AND REACTIONS

JT	X DISPL	Y DISPL	Z ROT	X REACT	Y REACT	ZM REACT
1	0.161D-27	-0.232D-27	-0.185D-25	-0.161E+03	0.232E+03	0.185E+05
2	0.602D+00	-0.200D-01	-0.376D-02	0.000E+00	0.000E+00	0.000E+00
3	0.168D+01	-0.502D-01	-0.475D-02	0.000E+00	0.000E+00	0.000E+00
4	0.281D+01	-0.775D-01	-0.572D-02	0.000E+00	0.000E+00	0.000E+00
5	0.401D+01	-0.111D+00	-0.481D-02	0.000E+00	0.000E+00	0.000E+00
6	0.466D+01	-0.118D+00	-0.360D-02	0.000E+00	0.000E+00	0.000E+00
7	0.159D-27	-0.967D-27	-0.183D-25	-0.159E+03	0.967E+03	0.183E+05
8	0.596D+00	-0.834D-01	-0.372D-02	0.000E+00	0.000E+00	0.000E+00
9	0.167D+01	-0.176D+00	-0.475D-02	0.000E+00	0.000E+00	0.000E+00
10	0.280D+01	-0.233D+00	-0.570D-02	0.000E+00	0.000E+00	0.000E+00
11	0.400D+01	-0.288D+00	-0.482D-02	0.000E+00	0.000E+00	0.000E+00
12	0.465D+01	-0.299D+00	-0.368D-02	0.000E+00	0.000E+00	0.000E+00
13	0.145D-32	-0.392D-26	-0.378D-02	-0.145E-02	0.392E+04	0.000E+00
14	0.596D+00	-0.227D+00	-0.308D-02	0.000E+00	0.000E+00	0.000E+00
15	0.167D+01	-0.397D+00	-0.473D-02	0.000E+00	0.000E+00	0.000E+00
16	0.280D+01	-0.510D+00	-0.498D-02	0.000E+00	0.000E+00	0.000E+00
17	0.400D+01	-0.566D+00	-0.446D-02	0.000E+00	0.000E+00	0.000E+00
18	0.465D+01	-0.573D+00	-0.306D-02	0.000E+00	0.000E+00	0.000E+00

MEMBER END ACTIONS

MEMBER	J	K	AX J	SHR J	MOM J	AX K	SHR K	MOM K
1	1	2	0.232E+03	0.161E+03	0.185E+05	-0.232E+03	-0.161E+03	0.855E+04
2	2	3	0.225E+03	0.122E+03	0.111E+05	-0.225E+03	-0.122E+03	0.949E+04
3	3	4	0.202E+03	0.850E+02	0.791E+04	-0.202E+03	-0.850E+02	0.637E+04
4	4	5	0.129E+03	0.480E+02	0.370E+04	-0.129E+03	-0.480E+02	0.436E+04
5	5	6	0.326E+02	0.110E+02	0.278E+03	-0.326E+02	-0.110E+02	0.131E+04
6	7	8	0.967E+03	0.159E+03	0.183E+05	-0.967E+03	-0.159E+03	0.843E+04
7	8	9	0.689E+03	0.123E+03	0.111E+05	-0.689E+03	-0.123E+03	0.949E+04
8	9	10	0.425E+03	0.852E+02	0.791E+04	-0.425E+03	-0.852E+02	0.640E+04
9	10	11	0.212E+03	0.474E+02	0.366E+04	-0.212E+03	-0.474E+02	0.430E+04
10	11	12	0.508E+02	0.116E+02	0.350E+03	-0.508E+02	-0.116E+02	0.132E+04
11	2	8	0.363E+02	-0.136E+03	-0.196E+05	-0.363E+02	0.136E+03	-0.195E+05
12	3	9	0.374E+02	-0.121E+03	-0.174E+05	-0.374E+02	0.121E+03	-0.174E+05
13	4	10	0.378E+02	-0.699E+02	-0.101E+05	-0.378E+02	0.699E+02	-0.101E+05
14	5	11	0.358E+02	-0.322E+02	-0.464E+04	-0.358E+02	0.322E+02	-0.465E+04
15	6	12	0.116E+02	-0.911E+01	-0.131E+04	-0.116E+02	0.911E+01	-0.132E+04
16	13	14	0.392E+04	0.145E-02	0.341E-15	-0.392E+04	-0.145E-02	0.244E+00
17	14	15	0.293E+04	0.309E-01	0.288E+01	-0.293E+04	-0.309E-01	0.231E+01
18	15	16	0.195E+04	0.232E-01	0.199E+01	-0.195E+04	-0.232E-01	0.190E+01
19	16	17	0.967E+03	0.293E-01	0.237E+01	-0.967E+03	-0.293E-01	0.255E+01
20	17	18	0.147E+03	0.138E-01	0.710E+00	-0.147E+03	-0.138E-01	0.127E+01
21	8	14	-0.294E-01	-0.588E-01	-0.346E+01	0.294E-01	0.588E-01	-0.312E+01
22	9	15	0.770E-02	-0.768E-01	-0.431E+01	-0.770E-02	0.768E-01	-0.430E+01
23	10	16	-0.610E-02	-0.797E-01	-0.465E+01	0.610E-02	0.797E-01	-0.427E+01
24	11	17	0.155E-01	-0.599E-01	-0.345E+01	-0.155E-01	0.599E-01	-0.326E+01
25	12	18	0.138E-01	-0.256E-01	-0.159E+01	-0.138E-01	0.256E-01	-0.127E+01

OUTPUT INCLUDING P-DELTA EFFECTS

NUMBER OF CYCLES= 2

JOINT DISPLACEMENTS AND REACTIONS

JT	X DISPL	Y DISPL	Z ROT	X REACT	Y REACT	ZM REACT
1	0.162D-27	-0.223D-27	-0.188D-25	-0.162E+03	0.223E+03	0.188E+05
2	0.613D+00	-0.192D-01	-0.384D-02	0.000E+00	0.000E+00	0.000E+00
3	0.172D+01	-0.487D-01	-0.487D-02	0.000E+00	0.000E+00	0.000E+00
4	0.288D+01	-0.756D-01	-0.587D-02	0.000E+00	0.000E+00	0.000E+00
5	0.411D+01	-0.109D+00	-0.493D-02	0.000E+00	0.000E+00	0.000E+00
6	0.477D+01	-0.116D+00	-0.368D-02	0.000E+00	0.000E+00	0.000E+00
7	0.157D-27	-0.976D-27	-0.185D-25	-0.157E+03	0.976E+03	0.185E+05
8	0.607D+00	-0.841D-01	-0.380D-02	0.000E+00	0.000E+00	0.000E+00
9	0.171D+01	-0.178D+00	-0.486D-02	0.000E+00	0.000E+00	0.000E+00
10	0.287D+01	-0.235D+00	-0.585D-02	0.000E+00	0.000E+00	0.000E+00
11	0.409D+01	-0.290D+00	-0.494D-02	0.000E+00	0.000E+00	0.000E+00
12	0.477D+01	-0.302D+00	-0.376D-02	0.000E+00	0.000E+00	0.000E+00
13	0.149D-32	-0.392D-26	-0.385D-02	-0.149E-02	0.392E+04	0.000E+00
14	0.607D+00	-0.227D+00	-0.313D-02	0.000E+00	0.000E+00	0.000E+00
15	0.171D+01	-0.397D+00	-0.482D-02	0.000E+00	0.000E+00	0.000E+00
16	0.287D+01	-0.510D+00	-0.506D-02	0.000E+00	0.000E+00	0.000E+00
17	0.409D+01	-0.566D+00	-0.453D-02	0.000E+00	0.000E+00	0.000E+00
18	0.477D+01	-0.573D+00	-0.307D-02	0.000E+00	0.000E+00	0.000E+00

MEMBER END ACTIONS

MEMBER	J	K	AX J	SHR J	MOM J	AX K	SHR K	MOM K
1	1	2	0.223E+03	0.162E+03	0.188E+05	-0.223E+03	-0.162E+03	0.865E+04
2	2	3	0.219E+03	0.124E+03	0.113E+05	-0.219E+03	-0.124E+03	0.972E+04
3	3	4	0.200E+03	0.858E+02	0.811E+04	-0.200E+03	-0.858E+02	0.653E+04
4	4	5	0.128E+03	0.483E+02	0.380E+04	-0.128E+03	-0.483E+02	0.448E+04
5	5	6	0.324E+02	0.110E+02	0.277E+03	-0.324E+02	-0.110E+02	0.133E+04
6	7	8	0.976E+03	0.157E+03	0.185E+05	-0.976E+03	-0.157E+03	0.855E+04
7	8	9	0.694E+03	0.121E+03	0.114E+05	-0.694E+03	-0.121E+03	0.972E+04
8	9	10	0.428E+03	0.844E+02	0.811E+04	-0.428E+03	-0.844E+02	0.656E+04
9	10	11	0.213E+03	0.471E+02	0.375E+04	-0.213E+03	-0.471E+02	0.441E+04
10	11	12	0.510E+02	0.115E+02	0.350E+03	-0.510E+02	-0.115E+02	0.135E+04
11	2	8	0.364E+02	-0.139E+03	-0.200E+05	-0.364E+02	0.139E+03	-0.199E+05
12	3	9	0.367E+02	-0.124E+03	-0.178E+05	-0.367E+02	0.124E+03	-0.178E+05
13	4	10	0.374E+02	-0.717E+02	-0.103E+05	-0.374E+02	0.717E+02	-0.103E+05
14	5	11	0.355E+02	-0.330E+02	-0.475E+04	-0.355E+02	0.330E+02	-0.476E+04
15	6	12	0.116E+02	-0.931E+01	-0.133E+04	-0.116E+02	0.931E+01	-0.135E+04
16	13	14	0.392E+04	0.149E-02	0.274E-15	-0.392E+04	-0.149E-02	0.251E+00
17	14	15	0.293E+04	0.319E-01	0.297E+01	-0.293E+04	-0.319E-01	0.239E+01
18	15	16	0.195E+04	0.242E-01	0.208E+01	-0.195E+04	-0.242E-01	0.199E+01
19	16	17	0.967E+03	0.306E-01	0.247E+01	-0.967E+03	-0.306E-01	0.266E+01
20	17	18	0.147E+03	0.148E-01	0.772E+00	-0.147E+03	-0.148E-01	0.136E+01
21	8	14	-0.304E-01	-0.607E-01	-0.357E+01	0.304E-01	0.607E-01	-0.323E+01
22	9	15	0.769E-02	-0.800E-01	-0.449E+01	-0.769E-02	0.800E-01	-0.447E+01
23	10	16	-0.631E-02	-0.834E-01	-0.488E+01	0.631E-02	0.834E-01	-0.447E+01
24	11	17	0.157E-01	-0.632E-01	-0.364E+01	-0.157E-01	0.632E-01	-0.343E+01
25	12	18	0.148E-01	-0.275E-01	-0.172E+01	-0.148E-01	0.275E-01	-0.136E+01

Highlight Key output

Verify Preliminary Design using PLFNL2 P–Δ Output

Beam L_6 (Member 15), W14x68, Z=115 in^3, A= 20.0 in^2, I= 722 in^4, d=14.0", t_w= 0.415"

Axial Force – 11.6^k © $\sigma = {}^P/_A = 11.6/20 = 0.58$ ksi negligible

Shear – 9.31^k ; Check shear: LRFD (d * t_w * ksi) > V_{max}

14" * 0.415" * 50 ksi = $290.5^k > 9.31^k$ Shear O.K.

Max Moment – $1350^{in\text{-}k} = 112.5^{ft\text{-}k}$

$\Phi_c Z F_y = .9 * 115\ in^3 * 50ksi = 5175^{in\text{-}k} > 1350^{in\text{-}k}$ Moment O.K.

Beam L_5 (Member 14), W21x83, Z=196 in^3, A= 24.3 in^2, I=1830 in^4, d=21.4", t_w= 0.515"

Axial Force – 35.5^k © $\sigma = {}^P/_A = 35.5/ 24.3 = 1.46$ ksi negligible

Shear – 33^k ; Check shear: LRFD (d * t_w * ksi) > V_{max}

21.4" * 0.515" * 50 ksi = $551.0^k > 33^k$ Shear O.K.

Max Moment – $4760^{in\text{-}k} = 396.7^{ft\text{-}k}$

$\Phi_c Z F_y = .9 * 196\ in^3 * 50ksi = 8820^{in\text{-}k} > 4760^{in\text{-}k}$ Moment O.K.

Beam L_4 (Member 13), W21x132, Z=333 in^3, A= 38.8 in^2, I=3220 in^4, d=21.8", t_w= 0.650"

Axial Force – 37.4^k © $\sigma = {}^P/_A = 37.4/ 38.8 = 0.96$ ksi negligible

Shear – 71.7^k ; Check shear: LRFD (d * t_w * ksi) > V_{max}

21.8" * 0.650" * 50 ksi = $708.6^k > 71.7^k$ Shear O.K.

Max Moment – $10{,}300^{in\text{-}k} = 858.3^{ft\text{-}k}$

$\Phi_c Z F_y = .9 * 333\ in^3 * 50ksi = 14{,}985^{in\text{-}k} > 10{,}300^{in\text{-}k}$ Moment O.K.

Beam L_3 (Member 12), W30x148, Z=500 in^3, A= 43.5 in^2, I=6680 in^4, d=30.7", t_w= 0.650"

Axial Force – 36.7^k © $\sigma = {}^P/_A = 36.7/ 43.5 = 0.84$ ksi negligible

Shear – 124.0^k ; Check shear: LRFD (d * t_w * ksi) > V_{max}

30.7" * 0.650" * 50 ksi = $997.7^k > 124^k$ Shear O.K.

Max Moment – $17{,}800^{in\text{-}k} = 1{,}483.3^{ft\text{-}k}$

$\Phi_c Z F_y = .9 * 500\ in^3 * 50ksi = 22{,}500^{in\text{-}k} > 17{,}800^{in\text{-}k}$ Moment O.K.

Beam L_2 (Member 11), W30x191, Z=675 in^3, A= 56.3 in^2, I=9200 in^4, d=30.7", t_w= 0.710"

Axial Force – 36.4^k © $\sigma = {}^P/_A = 36.4/ 56.3 = 0.65$ ksi negligible

Shear – 139.0^k ; Check shear: LRFD (d * t_w * ksi) > V_{max}

30.7" * 0.710" * 50 ksi = $1089.8^k > 139^k$ Shear O.K.

Max Moment – $20{,}000^{in\text{-}k} = 1{,}483.3^{ft\text{-}k}$

$\Phi_c Z F_y = .9 * 675\ in^3 * 50ksi = 30{,}375^{in\text{-}k} > 20{,}000^{in\text{-}k}$ Moment O.K.

NOTE: Even though these calculations are substantially larger than required they actually support the lateral deflection @ 4.77" < 5.44" max

Verify Preliminary Column Design

Column Top to Splice #1(Members 4, 5, 9, 10); W24x76, $Z = 200\ in^3$, $A= 22.4\ in^2$, $I_{in\ plane} = 2100\ in^4$, d=23.9", t_w= 0.440", $r_{in\ plane}$ = 9.69", $r_{out\ plane}$ = 1.92"

P_u = Axial Force = 213^k © $\sigma = {}^P/_A = 213/ 22.4 = 9.51$ ksi

Shear = 48.3^k ; Check shear: LRFD (d * t_w * ksi) > V_{max}

22.4" * 0.440" * 50 ksi = $492.8^k > 48.3^k$ Shear O.K.

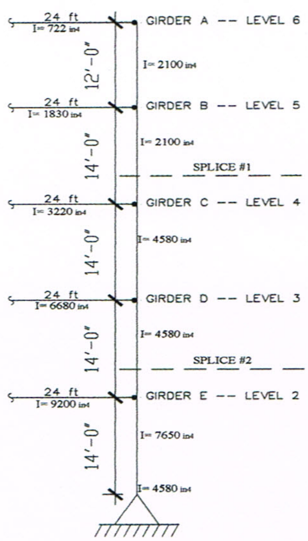
Mu = Max Moment = $4{,}480^{in\text{-}k} = 373.3^{ft\text{-}k}$

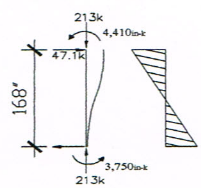
$\Phi_c Z F_y$= .9 * 200 in^3 * 50ksi = $9{,}000^{in\text{-}k} > 4{,}480^{in\text{-}k}$ Moment O.K.

Leaning Column Factor at L_4 to L_5

$$= \frac{2(P_2 + P_3) + P_6 + P_5}{2(P_2 + P_3)} = \frac{2(129^k + 41.7^k) + 146.5^k + 819.5^k}{2(129^k + 41.7^k)} = \frac{341.4^k + 966.0^k)}{341.4^k} = 3.83^k$$

DETERMINE K IN-PLANE FROM NOMOGRAPH





GIRDER B = $\Sigma\, {}^{EI}/_{L\ column} / \Sigma\, {}^{EI}/_{L\ beam} = [{}^{2100}/_{12} + {}^{2100}/_{14}] / [{}^{1830}/_{24}] = 2.95$

GIRDER C = $\Sigma\, {}^{EI}/_{L\ column} / \Sigma\, {}^{EI}/_{L\ beam} = [{}^{2100}/_{14} + {}^{4580}/_{14}] / [{}^{3220}/_{24}] = 3.56$

From Sidesway uninhibited nomograph, (AISC 16.1 – 242): K= 1.9

$K'_{inplane} = K * \sqrt{N} = 1.9 * \sqrt{3.83} = 3.7$

${}^{Kl}/_{r}$ out-of-plane = (1.0 * 14' * ${}^{12"}/_{1'}$) / 1.92" = 87.5 governs

${}^{K'l}/_{r}$ in-plane = (3.7 * 14' * ${}^{12"}/_{1'}$) / 9.69" = 64.1

$F_e = (\pi^2 * 29{,}000) / (87.5)^2 = 37.38$ ksi > .44*(50 ksi)=22 ksi **IN-ELASTIC**

$F_{cr} = (0.658^{50/37.38}) * 50 = (0.658^{1.3376}) * 50$ = 28.56 ksi

LRFD Φ_c=0.90

Φ_c x P_n ($P_n = F_{cr}$ x A_g)

0.9 * 28.56 ksi * 22.4 in^2 = 575.9^k

$^{Pu}/_{\Phi Pn}$ = 213^k © / 575.9^k = 0.370 > 0.20

L_b = 14', L_p = 1.76 * $r_{out\ plane}$ * $\sqrt{E/F_y}$ = 1.76 * 1.92 * √(29,000/50) = 81.38" (6.78')

AISC 16.1-46

$$C_b = \frac{(12.5M_{max})}{(2.5M_{max} + 3M_A + 4M_B + 3M_C)} \leq 3.0$$

= (12.5 * 4410) / ((2.5*4410) + (3*2370) + (4*330) + (3*3750))

= 55125/30705 = 1.79 < 3.0

(From Tbl 3-2) W24x76 = BF_{LRFD} = 22.5^k, ΦM_p = 750$^{ft\text{-}k}$ L_p = 6.78' (81.36")

$\Phi M_n = C_b[\Phi M_p - BF_{LRFD} * (L_b - L_p)] < \Phi M_p$

= 1.79[750$^{ft\text{-}k}$ – 22.5^k(14' – 6.78')] = 1.79 * (587.55$^{ft\text{-}k}$) = 1054.7 > 750$^{ft\text{-}k}$; therefore $\Phi M_n = \Phi M_p$ =750$^{ft\text{-}k}$

$^{Pu}/_{\Phi Pn}$ + ($^8/_9$) * ($^{Mu}/_{\Phi Mn}$) = 0.370 + ($^8/_9$) * (373.3$^{ft\text{-}k}$/ 750$^{ft\text{-}k}$) = 0.370 + 0.4424 = 0.8124 < 1 O.K.

Use W24x76 for top section of columns in rigid frame

Column Splice #1 to Splice #2 **(Members 2, 3, 7, 8)**; W24x146, Z = 418 in^3, A=43.0 in^2, I = 4580 in^4, d=24.7", t_w= 0.650", $r_{in\ plane}$ = 10.3", $r_{out\ plane}$ = 3.01"

P_u = Axial Force = 694^k © σ = $^P/_A$ = 694/43 = 16.1 ksi

Shear = 124^k ; Check shear: LRFD (d * t_w * ksi) > V_{max}

24.7" * 0.650" * 50 ksi = 802.8^k > 124^k Shear O.K.

M_u = Max Moment = 11,400$^{in\text{-}k}$ = 950$^{ft\text{-}k}$

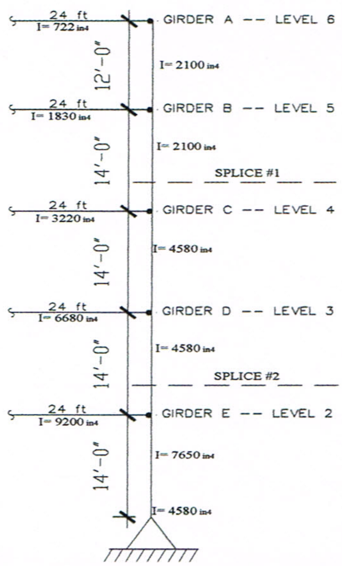
$\Phi_c Z F_y$= .9 * 418 in^3 * 50ksi = 18,810$^{in\text{-}k}$ > 4,480$^{in\text{-}k}$ Moment O.K.

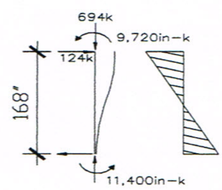
Leaning Column Factor at L_2 to L_3

$$= \frac{2[(P_2 + P_3) + 3(P_1)] + P_6 + P_5 + 3(P_4)}{2[(P_2 + P_3) + 3(P_1)]} = \frac{2[(129^k + 41.7^k) + 3(143^k)] + 146.5^k + 819.5^k + 3(983^k)}{2[(129^k + 41.7^k) + 3(143^k)]} =$$

$$\frac{2[599.7^k] + 3915.0^k}{2[599.7^k]} = \frac{5114.4^k}{1199.4^k} = 4.26^k$$

DETERMINE K IN-PLANE FROM NOMOGRAPH





GIRDER D = $\Sigma\, {}^{EI}/_{L\ column} / \Sigma\, {}^{EI}/_{L\ beam} = [{}^{4580}/_{14} + {}^{4580}/_{14}] / [{}^{6680}/_{24}] = 2.35$

GIRDER E = $\Sigma\, {}^{EI}/_{L\ column} / \Sigma\, {}^{EI}/_{L\ beam} = [{}^{4580}/_{14} + {}^{7650}/_{14}] / [{}^{9200}/_{24}] = 2.28$

From Sidesway uninhibited nomograph, (AISC 16.1 – 242): K= 1.7

$K'_{inplane} = K * \sqrt{N} = 1.7 * \sqrt{4.26} = 3.51$

${}^{Kl}/_{r\ \ out\text{-}of\text{-}plane} = (1.0 * 14' * {}^{12"}/_{1'}) / 3.01" = 55.8$

${}^{K'l}/_{r\ \ in\text{-}plane} = (3.51 * 14' * {}^{12"}/_{1'}) / 10.3" = 57.2$ governs

$F_e = (\pi^2 * 29{,}000) / (57.2)^2 = 87.48 > .44 * (50\ ksi) = 22\ ksi$ **<u>IN-ELASTIC</u>**

$F_{cr} = (0.658^{\,50/87.48}) * 50 = (0.658^{0.57156}) * 50 = 39.36\ ksi$

LRFD Φ_c=0.90

$\Phi_c \times P_n \qquad (P_n = F_{cr} \times A_g)$

$0.9 * 39.36\ ksi * 43.0\ in^2 = 1523.3^k$

${}^{Pu}/_{\Phi Pn} = 694^k$ © $/ 1523.3^k = 0.4556 > 0.20$

$L_b = 14'$, $L_p = 1.76 * r_{out\ plane} * \sqrt{{}^{E}/_{Fy}} = 1.76 * 3.01 * \sqrt{(29{,}000/50)} = 127.58"$ (10.62')

AISC 16.1-46

$$C_b = \frac{(12.5M_{max})}{(2.5M_{max} + 3M_A + 4M_B + 3M_C)} \le 3.0$$

$= (12.5 * 11{,}400) / ((2.5*11{,}400) + (3*4440) + (4*840) + (3*6120))$

$= 142{,}500/63{,}540 = 2.24 < 3.0$

(From Tbl 3-2) W24x146 = $BF_{LRFD} = 25.8^k$, $\Phi M_p = 1570^{ft\text{-}k}$ $L_p = 10.6'$ (127.2")

$\Phi M_n = C_b[\Phi M_p - BF_{LRFD} * (L_b - L_p)] < \Phi M_p$

$= C_b\,[1570^{ft\text{-}k} - 25.8^k(14' - 10.6')] = 2.24 * (1482.28^{ft\text{-}k}) = 3324 > 1570^{ft\text{-}k}$; therefore $\Phi M_n = \Phi M_p = 1570^{ft\text{-}k}$

${}^{Pu}/_{\Phi Pn} + ({}^{8}/_{9}) * ({}^{Mu}/_{\Phi Mn}) = 0.494 + ({}^{8}/_{9}) * (950^{ft\text{-}k} / 1570^{ft\text{-}k}) = 0.4556 + 0.5379 = 0.9935 < 1$ O.K.

<u>Use W24x146 for middle section of columns in rigid frame</u>

<u>Column Splice #2 to Base</u> (Members 1, 6); W24x229, Z = 675 in^3, A= 67.2 in^2, I = 7650 in^4, d=26.0", t_w= 0.960", $r_{in\ plane}$ = 10.7", $r_{out\ plane}$ = 3.11"

P_u = Axial Force = 976^k © $\sigma = {}^P/_A = 976/67.2 = 14.5$ ksi

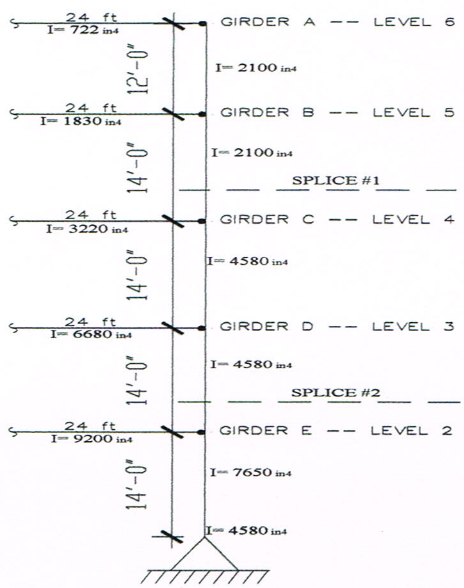
Shear = 162^k ; Check shear: LRFD ($d * t_w *$ ksi) > V_{max}

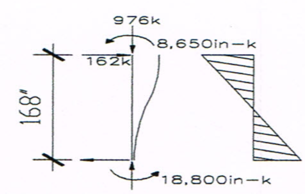
26.0" * 0.960" * 50 ksi = $1248^k > 162^k$ Shear O.K.

M_u = Max Moment = $18{,}800^{in\text{-}k} = 1{,}566.7^{ft\text{-}k}$

$\Phi_c Z F_y = .9 * 675\ in^3 * 50ksi = 30{,}375^{in\text{-}k} > 18{,}800^{in\text{-}k}$ Moment O.K.

DETERMINE K IN-PLANE FROM NOMOGRAPH





$F_{cr} = (0.658^{\,50/65.51}) * 50 = (0.658^{0.7632}) * 50 = 36.33$ ksi

LRFD Φ_c=0.90

$\Phi_c \times P_n$ ($P_n = F_{cr} \times A_g$)

$0.9 * 36.33\ ksi * 67.2\ in^2 = 2197.2^k$

${}^{Pu}/_{\Phi Pn} = 976^k$ © / $2197.2^k = 0.444 > 0.20$

$L_b = 14'$, $L_p = 1.76 * r_{out\ plane} * \sqrt{{}^E/_{Fy}} = 1.76 * 3.11 * \sqrt{(29{,}000/50)} = 131.8"$ (10.98')

AISC 16.1-46

$$C_b = \frac{(12.5M_{max})}{(2.5M_{max} + 3M_A + 4M_B + 3M_C)} \le 3.0$$

= (12.5 * 18,800) / ((2.5*18,800) + (3*1787) + (4*5075) + (3*11937))

= 235,000/108472 = 2.17 < 3.0

(From Tbl 3-2) W24x229 = $BF_{LRFD} = 28.9^k$, $\Phi M_p = 2530^{ft\text{-}k}$ $L_p = 11.0'$

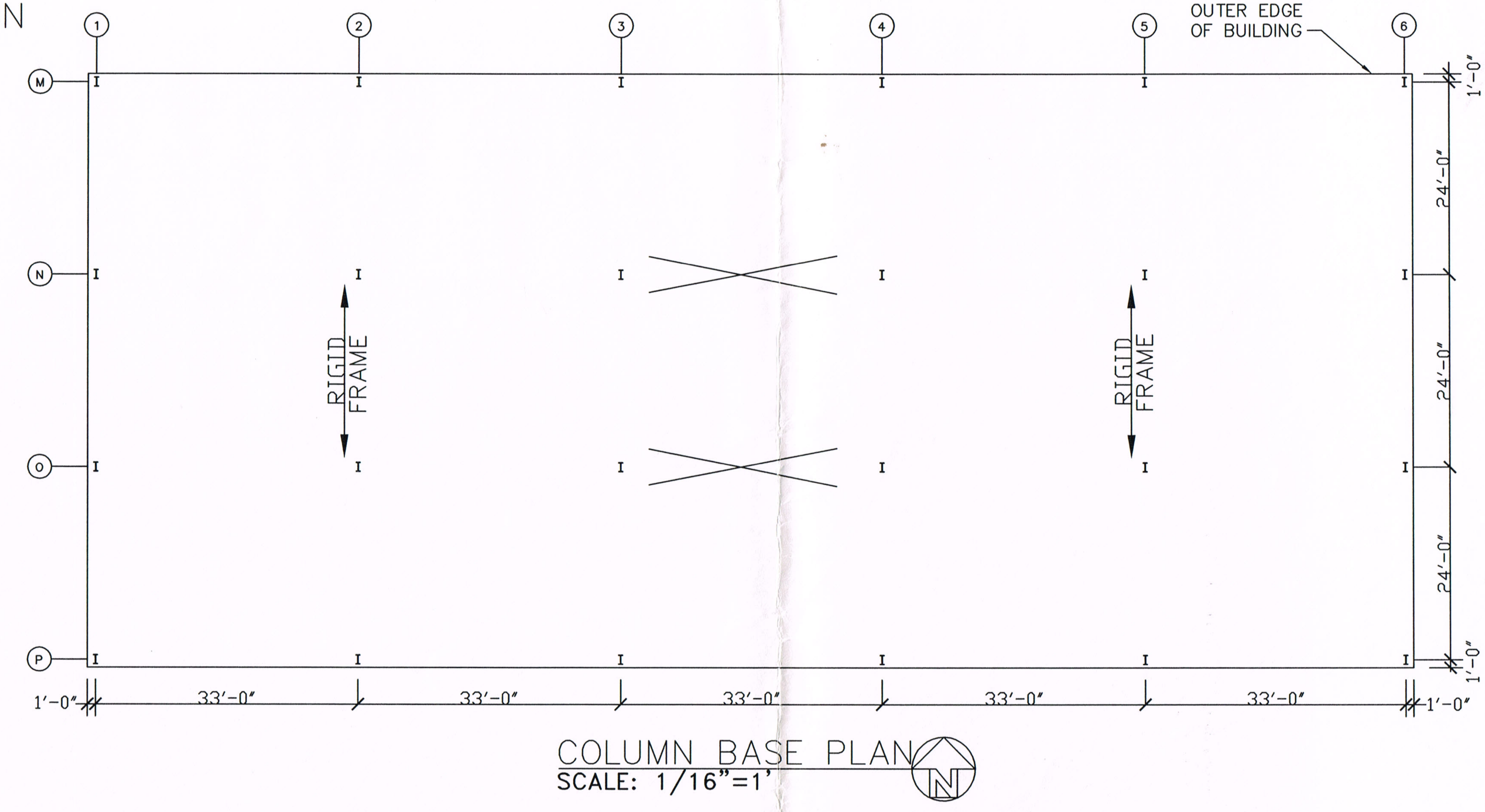
$\Phi M_n = C_b[\Phi M_p - BF_{LRFD} * (L_b - L_p)] < \Phi M_p$

$= 2.17\ [2530^{ft\text{-}k} - 28.9^k(14' - 11')] = 2.17*(2443.3^{ft\text{-}k}) = 5293 > 2530^{ft\text{-}k}$; therefore $\Phi M_n = \Phi M_p = 2530^{ft\text{-}k}$

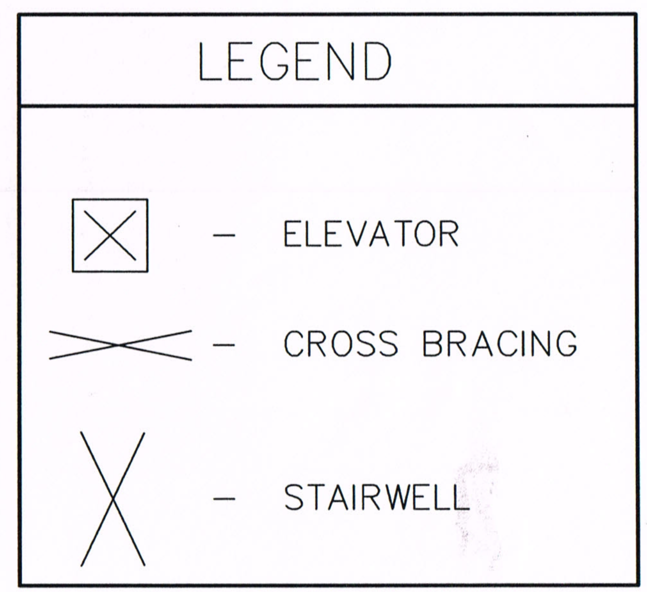
${}^{Pu}/_{\Phi Pn} + ({}^8/_9) * ({}^{Mu}/_{\Phi Mn}) = 0.444 + ({}^8/_9) * (1567^{ft\text{-}k} / 2530^{ft\text{-}k}) = 0.444 + 0.551 = 0.9945 < 1$ **O.K.**

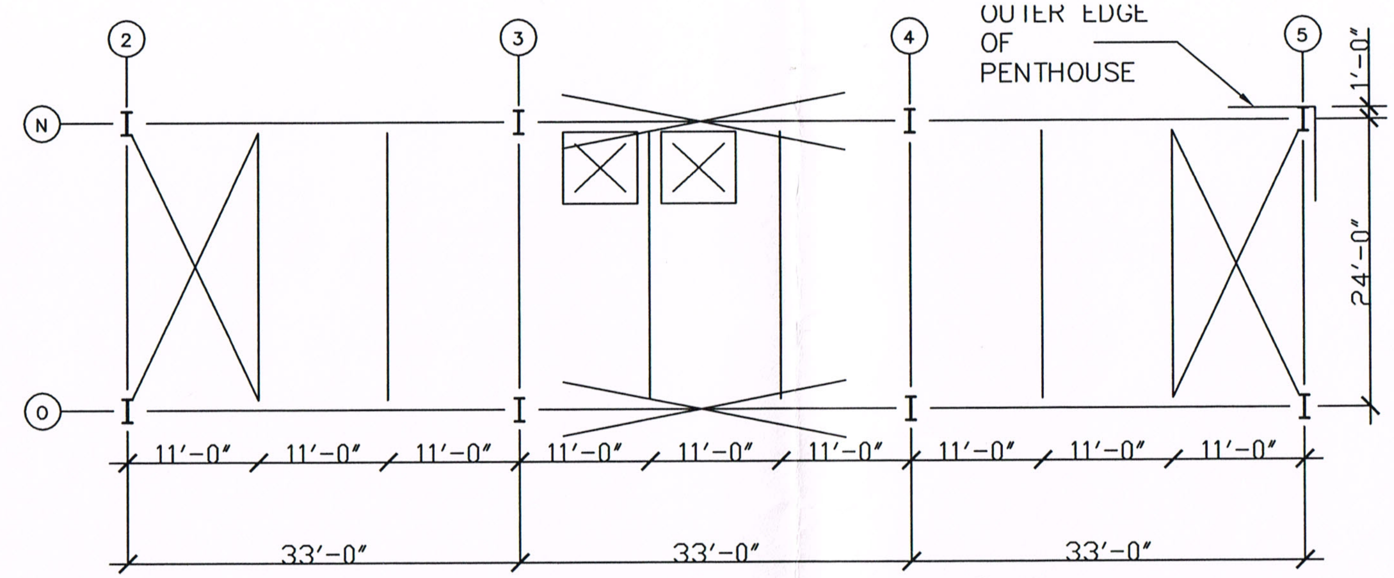
<u>Use W24x229 for bottom section of columns in rigid frame</u>

ENCE 4358 STEEL DESIGN PROJECT
DAVID DAMMON

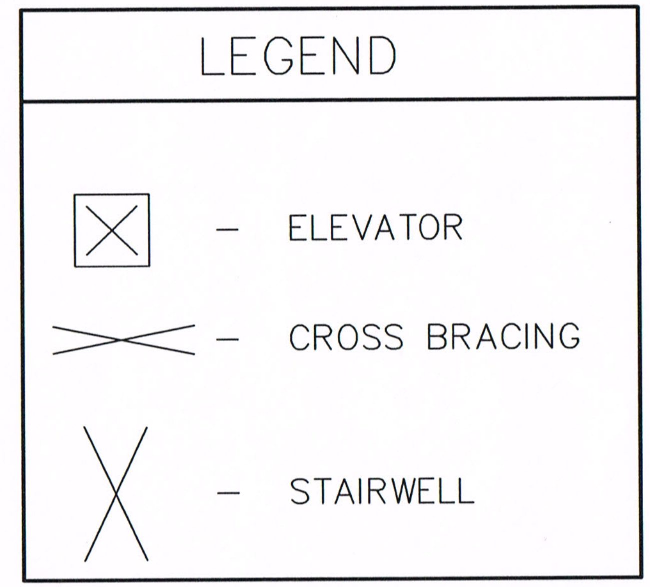


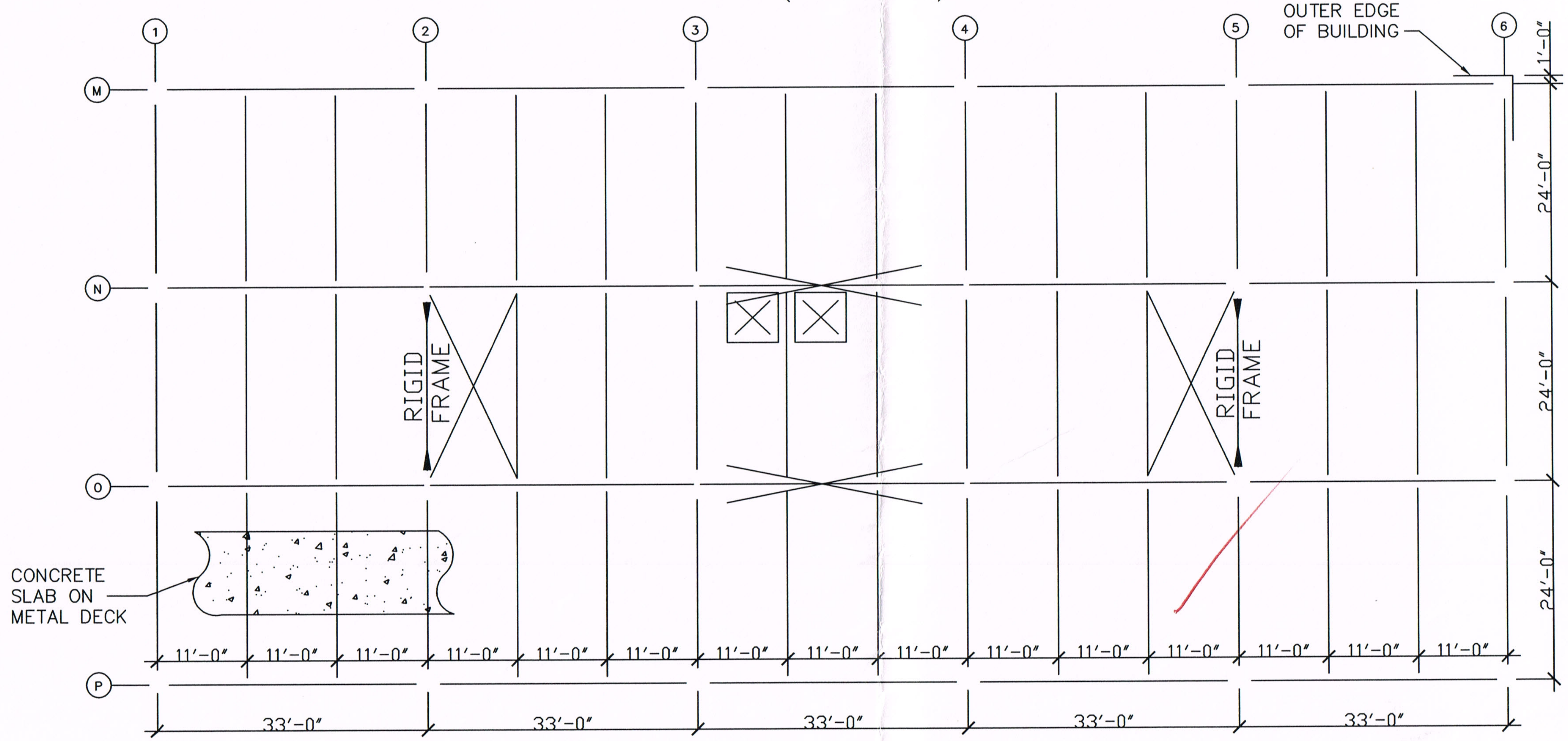
COLUMN BASE PLAN
SCALE: 1/16"=1'





FRAMING PLAN
SCALE: 1/16"=1'
L-6 (PENTHOUSE)





FRAMING PLAN
SCALE: 1/16"=1'
L-2, L-3, L-4, L-5 (MAIN ROOF)

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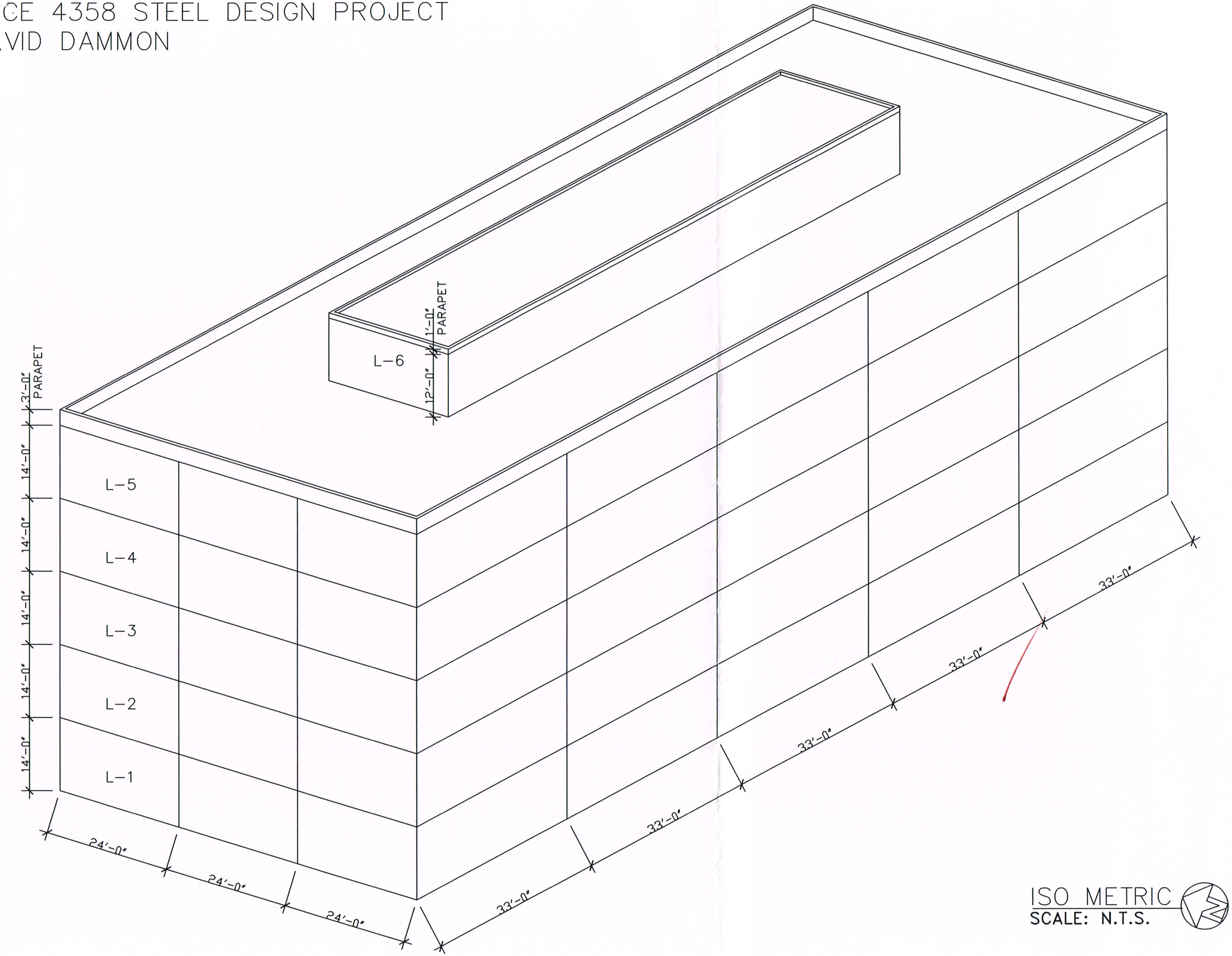

ISO METRIC
SCALE: N.T.S.

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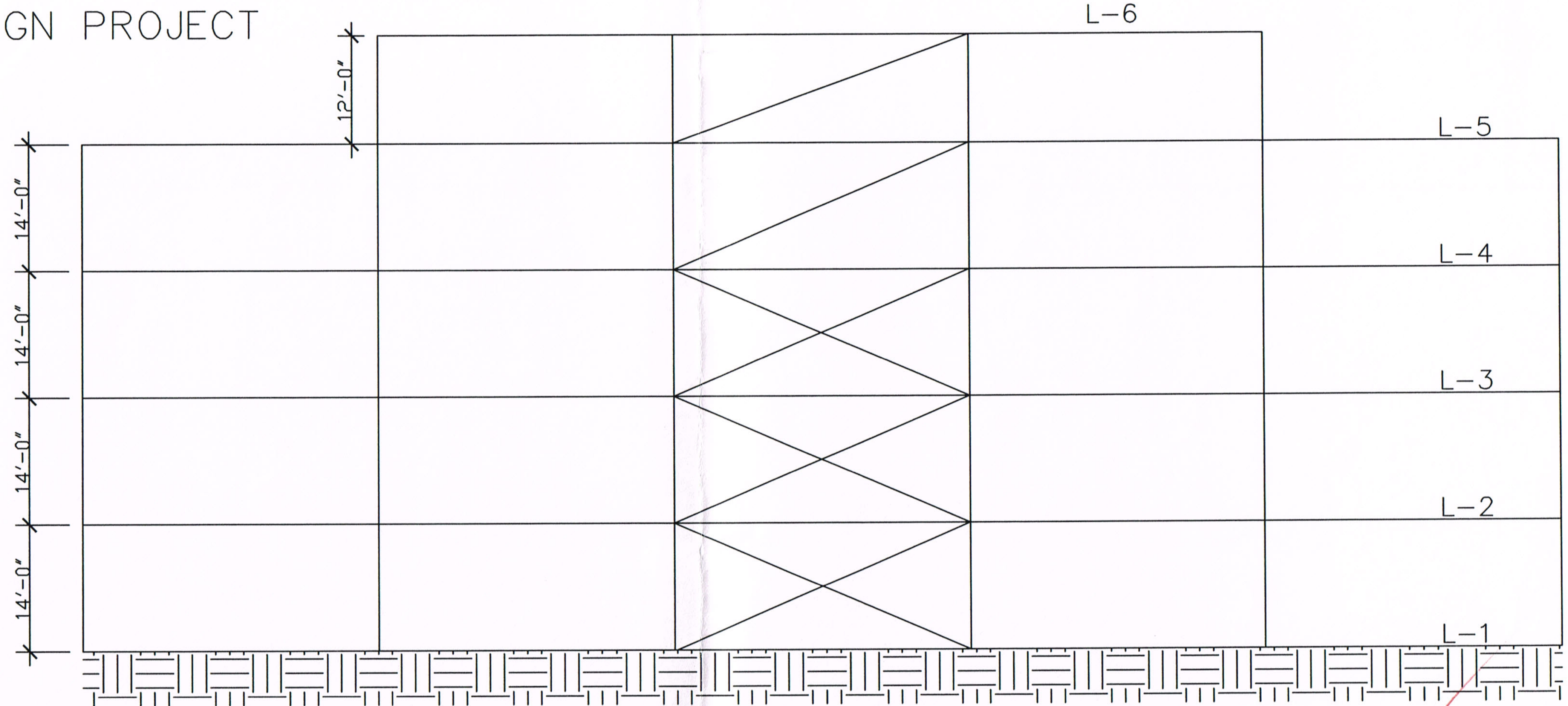
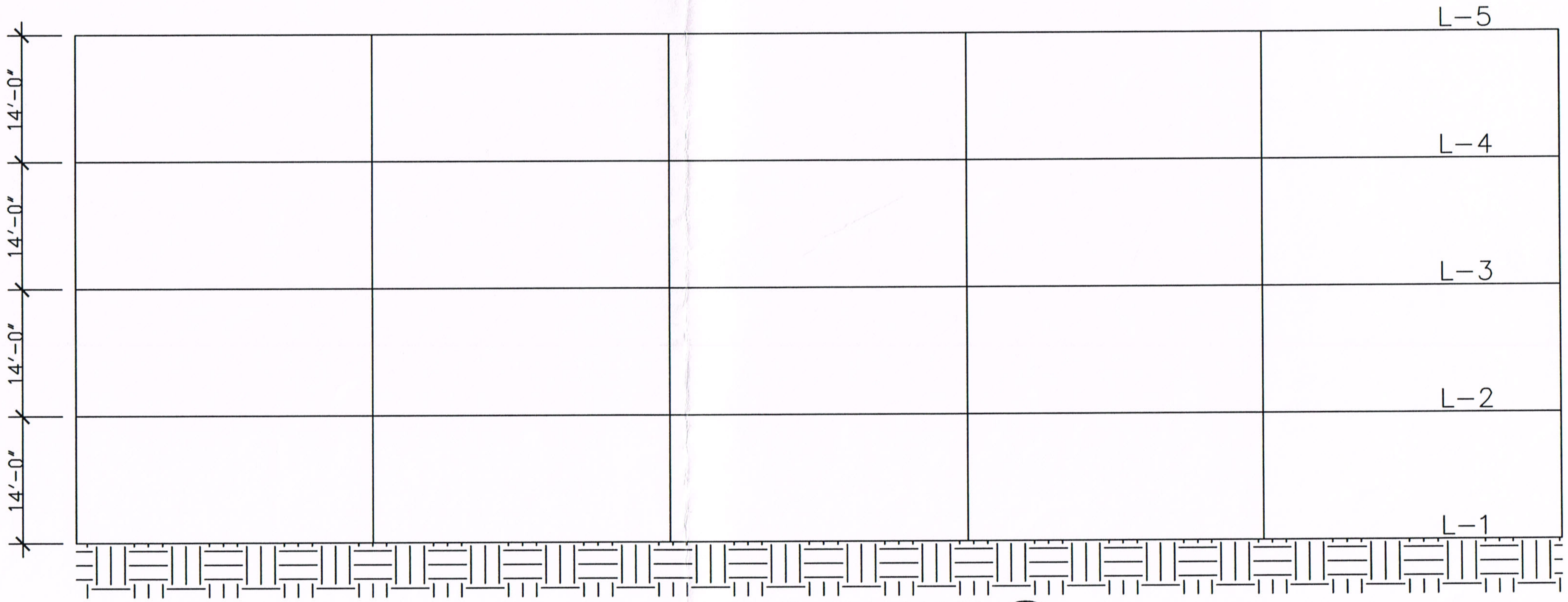


FRAMING ELEVATION
SCALE: 1/16"=1'
FRAMING LINE N & O



FRAMING ELEVATION
SCALE: 1/16"=1'
FRAMING LINE M & P

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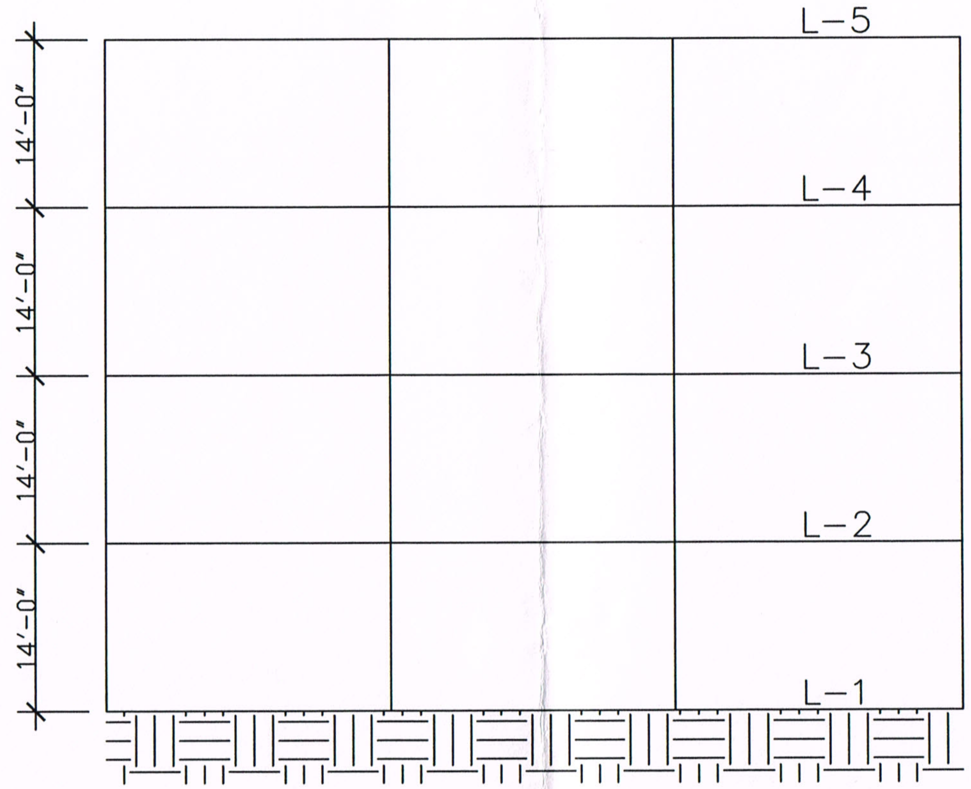
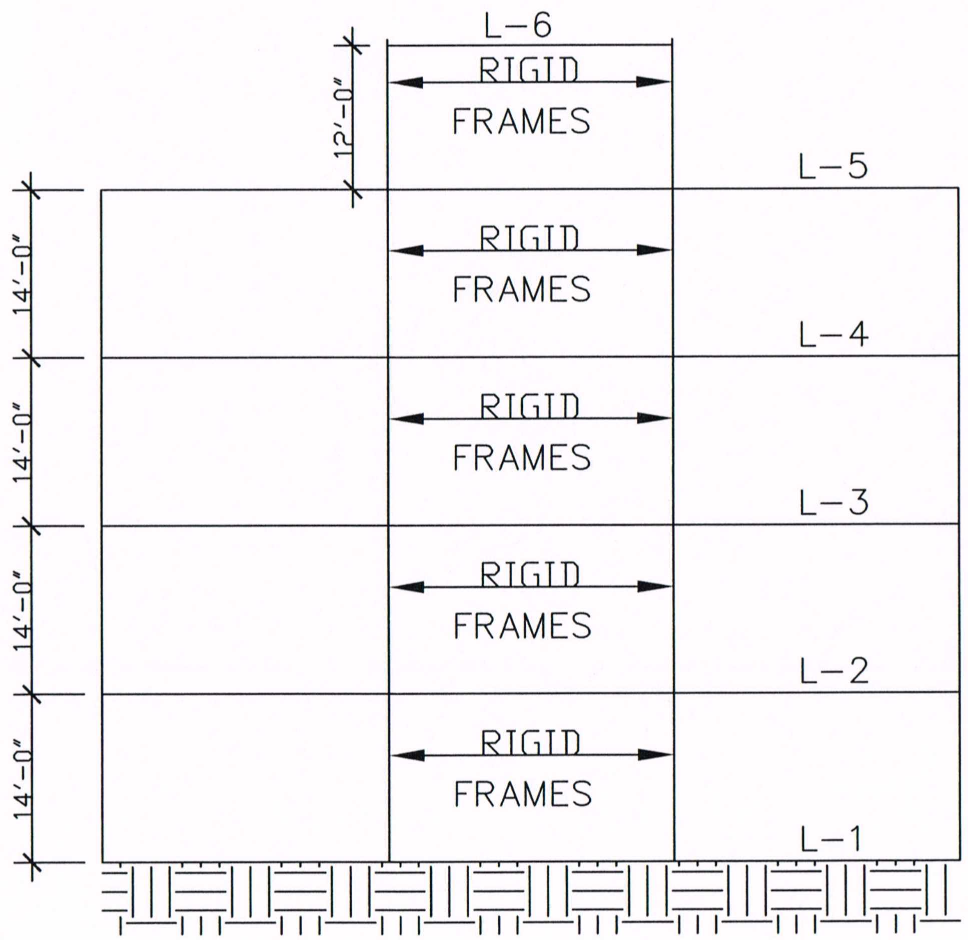
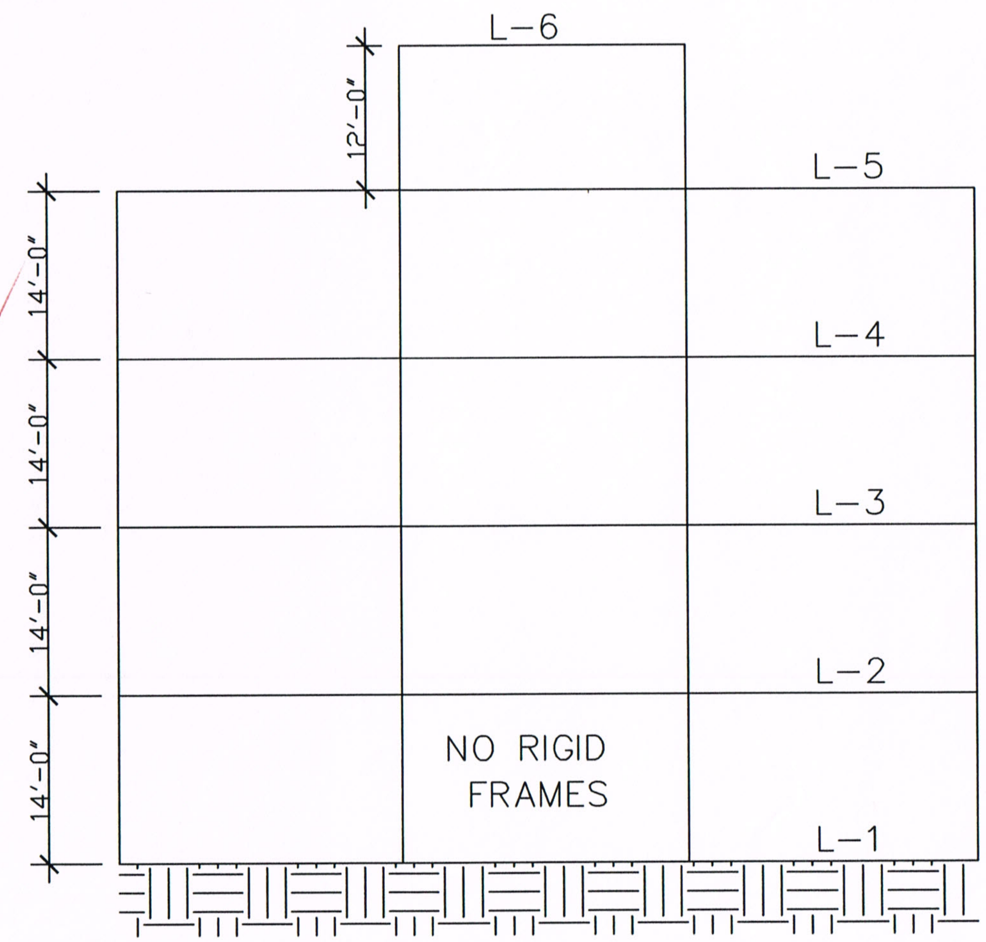


FRAMING ELEVATION
SCALE: 1/16"=1'
FRAMING LINE 1 & 6



FRAMING ELEVATION
SCALE: 1/16"=1'
FRAMING LINE 2 & 5



FRAMING ELEVATION
SCALE: 1/16"=1'
FRAMING LINE 3 & 4

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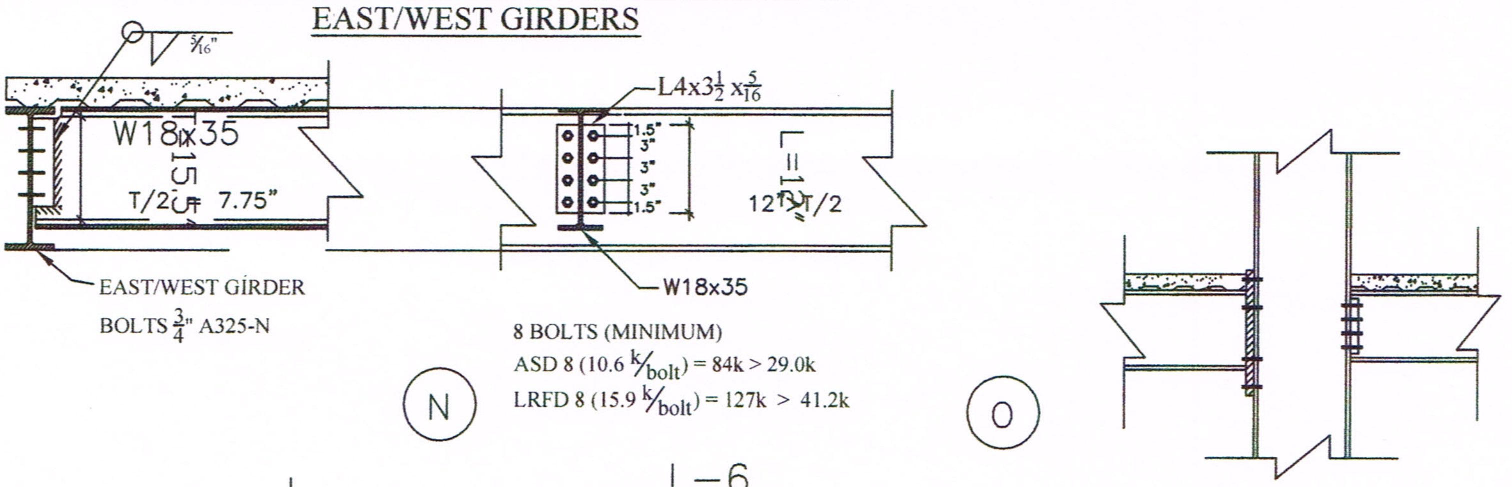
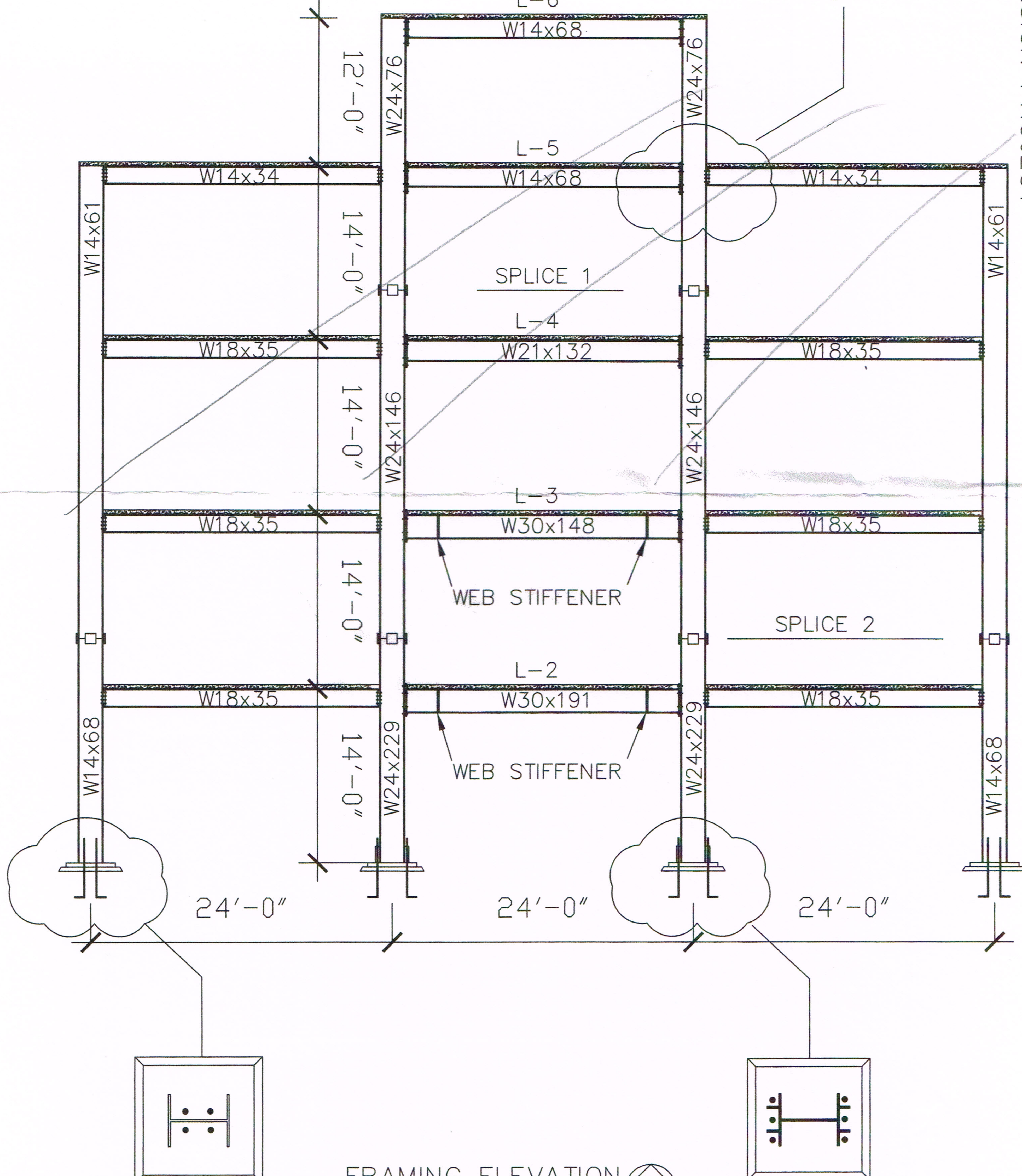
NORTH/SOUTH BEAMS PINN CONNECTION TO EAST/WEST GIRDERS





FRAMING ELEVATION
SCALE: 1/8"=1'
FRAMING LINE 2 & 5