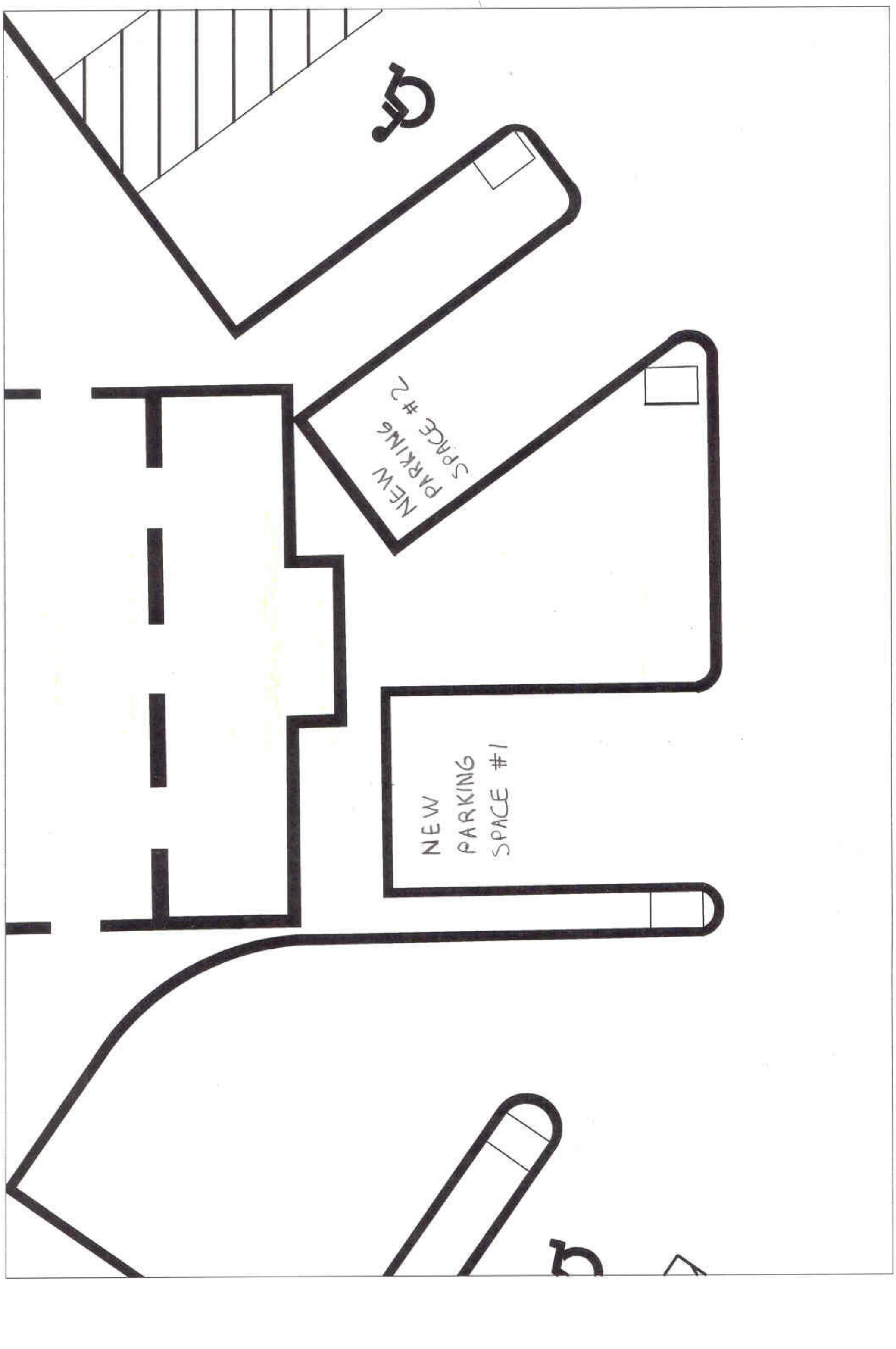
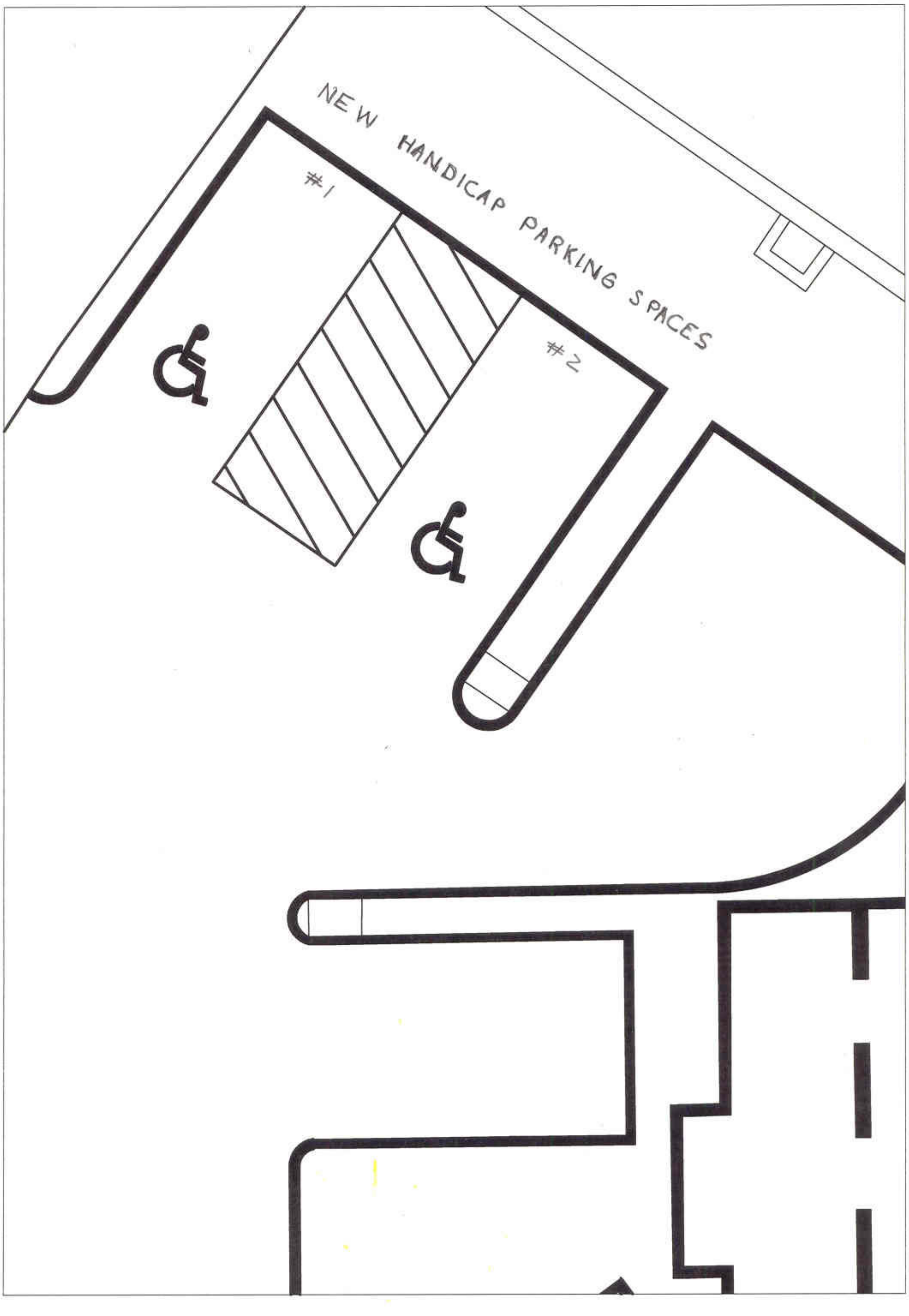
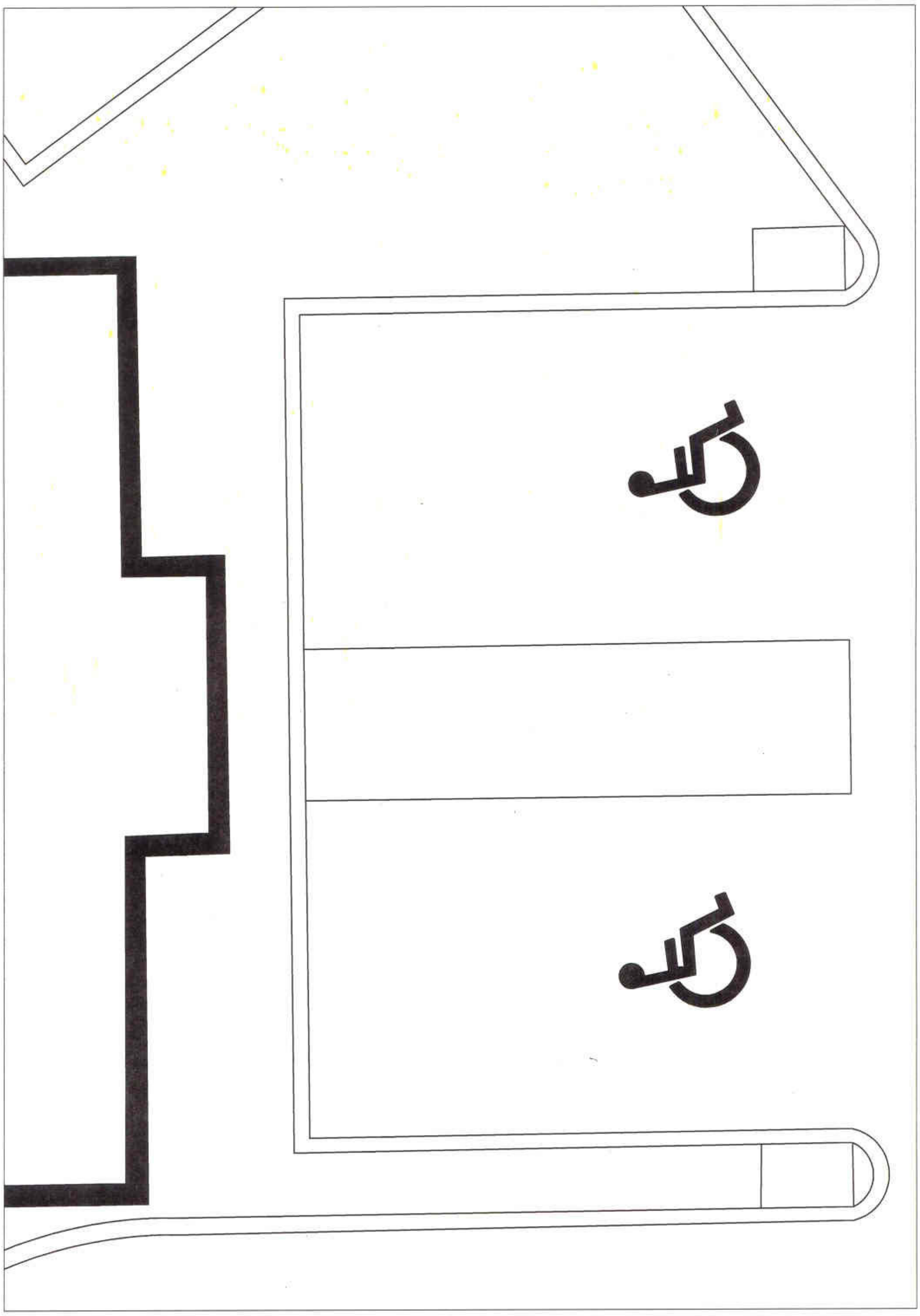
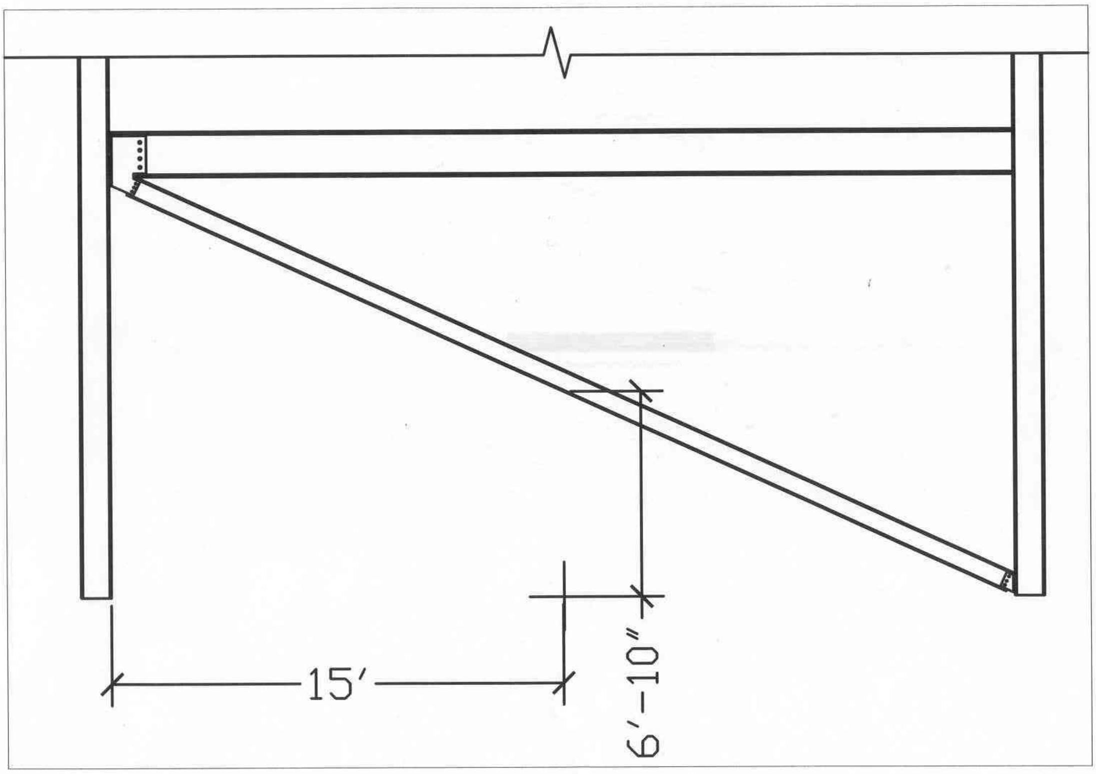
THE APPENDIX

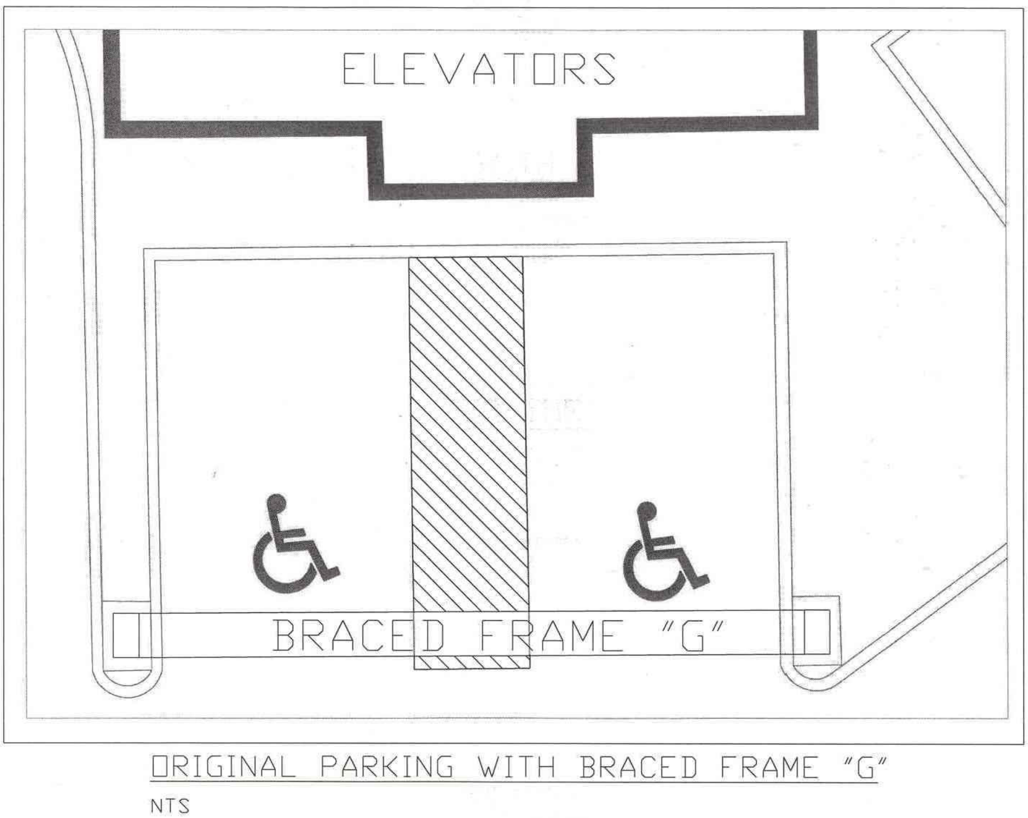
Architectural Redesign


NEW PARKING SPACE #2
NEW PARKING SPACE #1


NEW HANDICAP PARKING SPACES
#1
#2

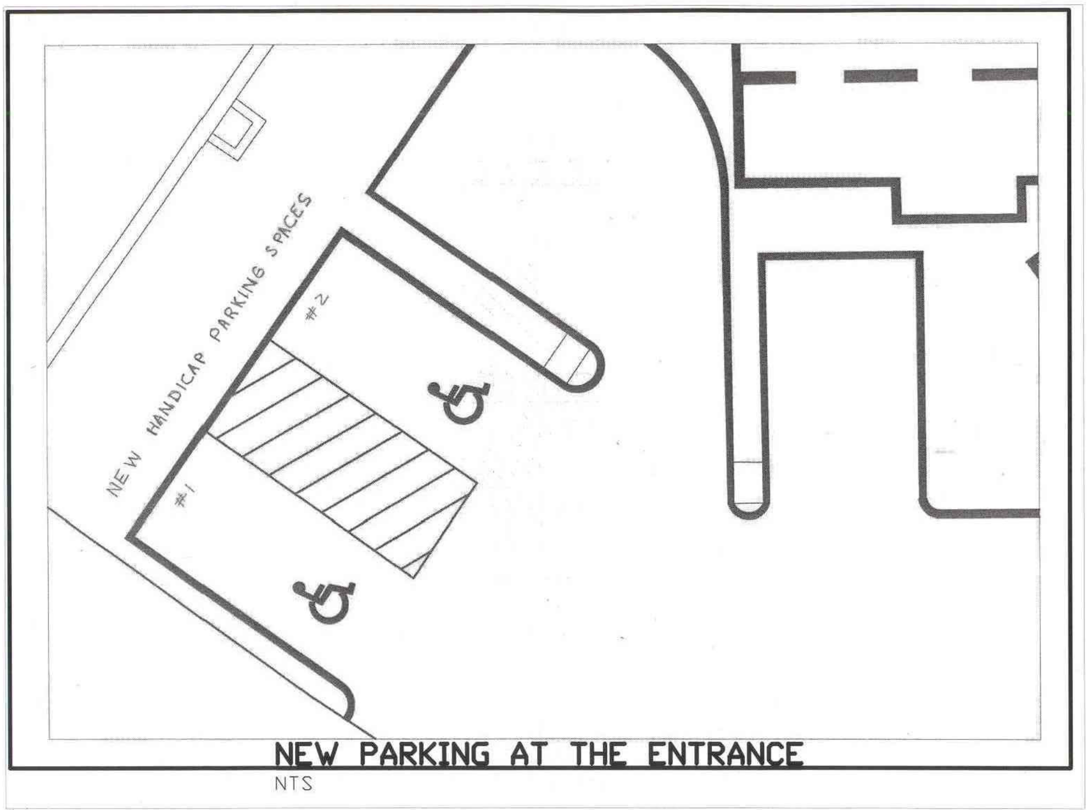


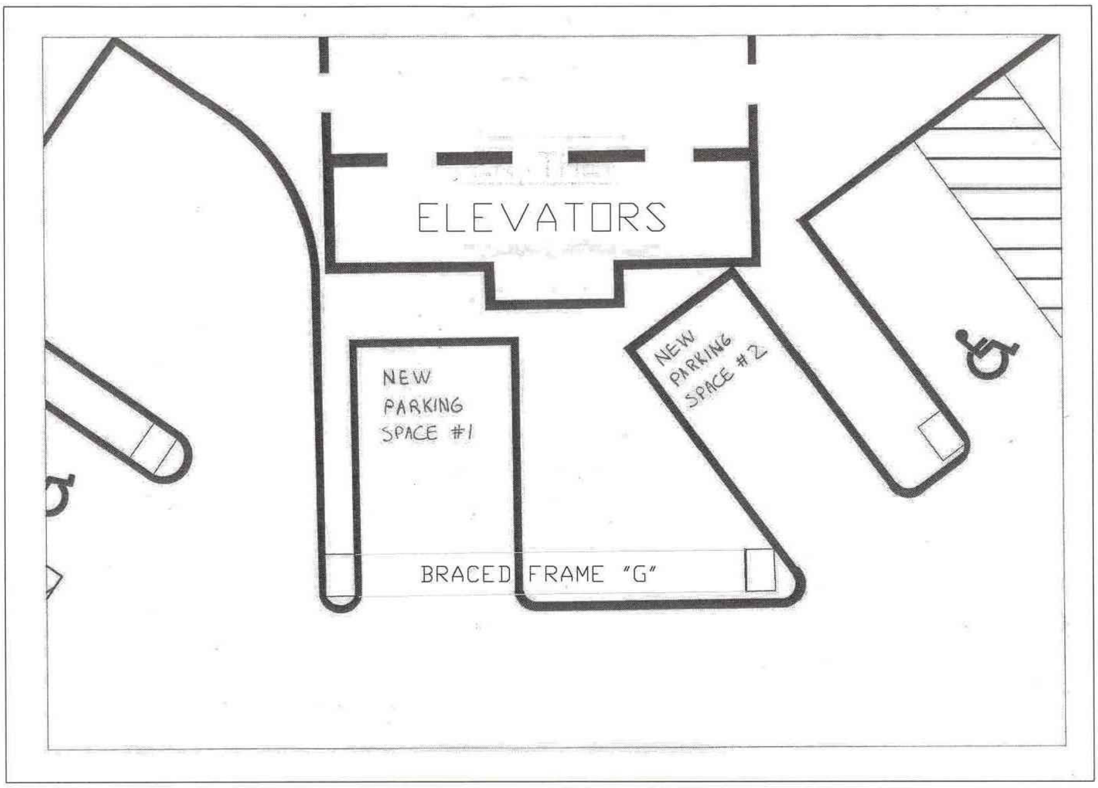




ORIGINAL PARKING WITH BRACED FRAME "G"

NTS

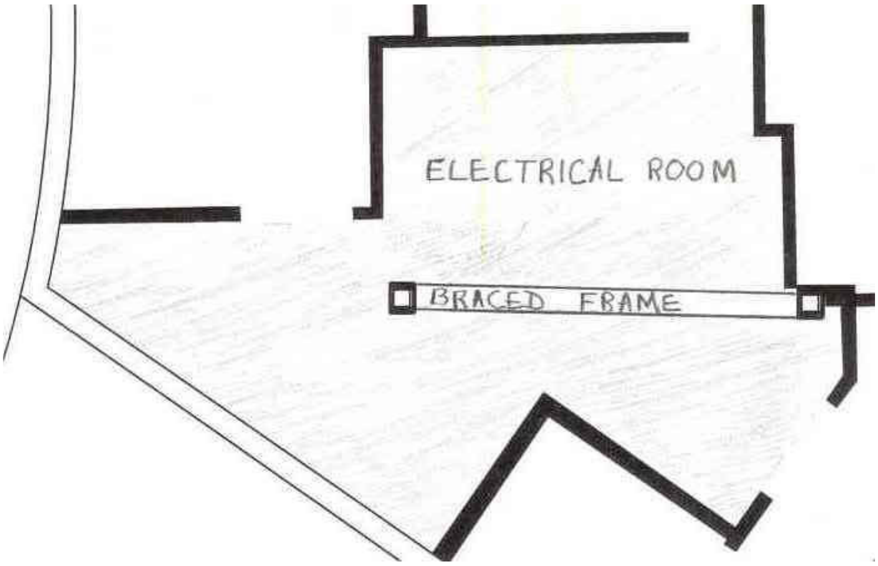

NEW HANDICAP PARKING SPACES
#1
#2
NEW PARKING AT THE ENTRANCE
NTS



NEW PARKING UNDER THE BRACED FRAME "G"

NTS

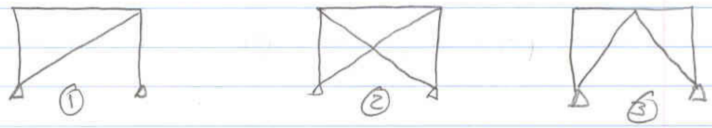
NEW DESIGN


ELECTRICAL ROOM
BRACED FRAME

Braced Frame Analysis

Brased frame Analysis (1)

3 Possibilities:

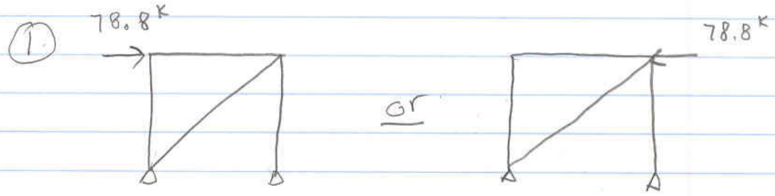


①	②	③
Design for Compressive strength	Design for tension strength or Combination of tension and compression	Design for compression strength.

Results

	①	②	③
weight of Diagonals	1970 lbs or 1257 lbs	540 lbs	1373.5 lbs
# of Additional connections	2	4	3
# of diagonal members	1	2	2
Least Labor Intensive 1-3	1	3	2

Braced frame Analysis: (2)



Calculated on STAAD 2004

Diagonal Axial Force: 87 k (T)	Diagonal Axial Force: **87 k C** ← Controls

USE:

Result: W 8 x 58

- Total weight: (33.1 FT) · 58 lb = 1920 lb

Also Try WT 9 x 38

$\phi P_{nx} = 92.6$ OK

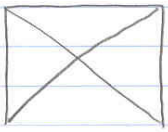
$\phi P_{ny} = 96.1$ OK

$\frac{Kl}{r} < 200$ OK

Tot weight = 33.1 (38) = **1257.8** lbs

Braced Frame Analysis: (3)

(2)



A: Design for Tension; diagonals work one at the time.

B: Design for a combination both diagonals work simultaneously.

A. Diagonal Axial Force = 87^k Tension

Assume the design is control by tension yielding:

$\phi_t P_n = \phi F_y A_g$

$87^k = .9(50)(A)$

$A = 1.93\ in^2$

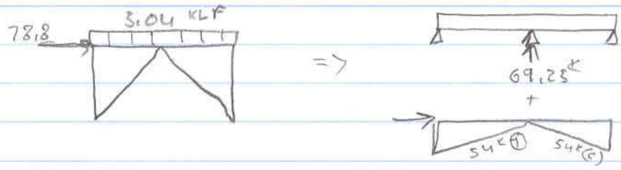
Use 2L 4x4x $\frac{5}{16}$ (2 Angles)

Total weight = $2(8.16\ \frac{lb}{ft})(33.1\ ft)$

= **540.2 lbs**

B. Compressive strength of small, long members is very small; the tension-based design is more economical.

Brace Frame Analysis (4)

③ 

Compression controls

=> Diagonal Axial Force = $54^k + 69.25^k = 123.25^k$

TRY WT 8 X 33.5

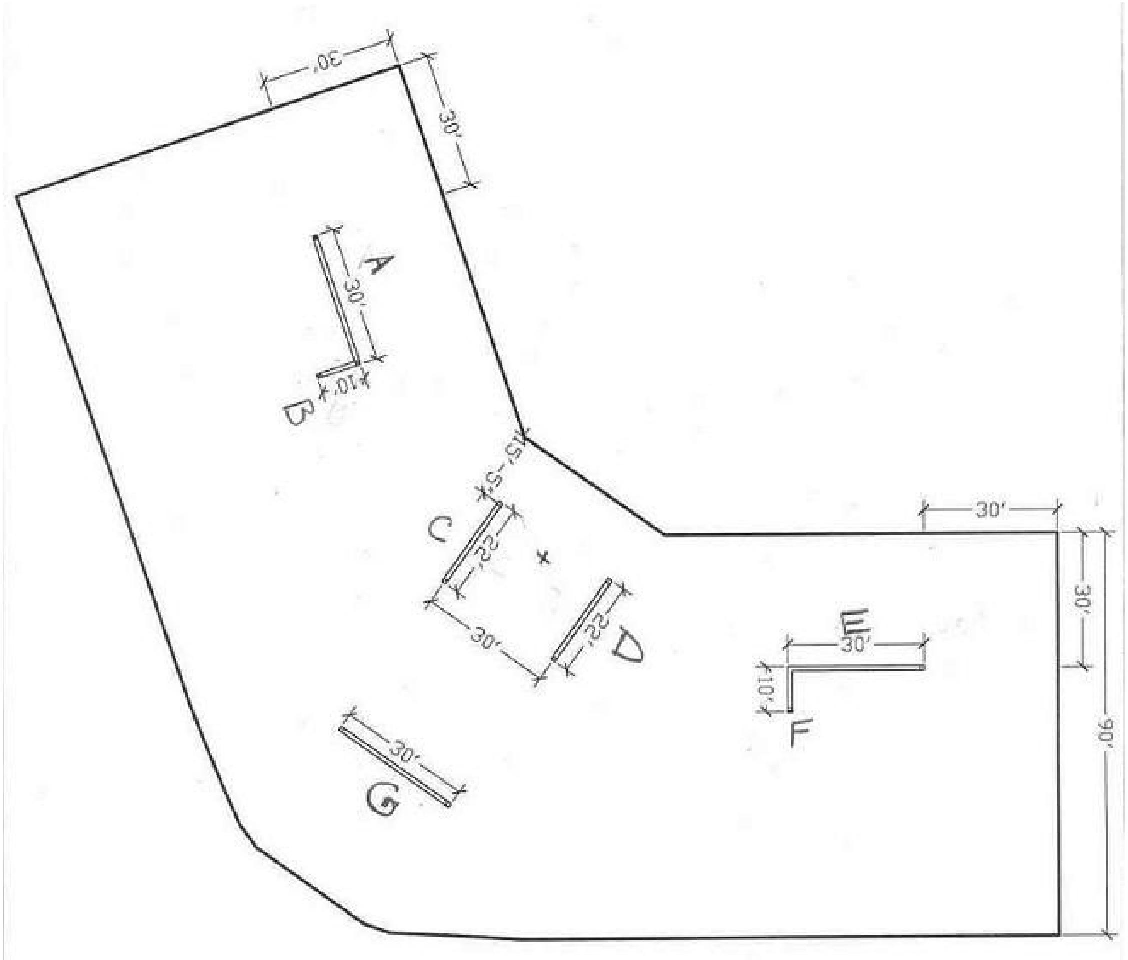
$\phi P_{n\,x} = 174^k$ OK

$\phi P_{n\,y} = 193^k$ OK

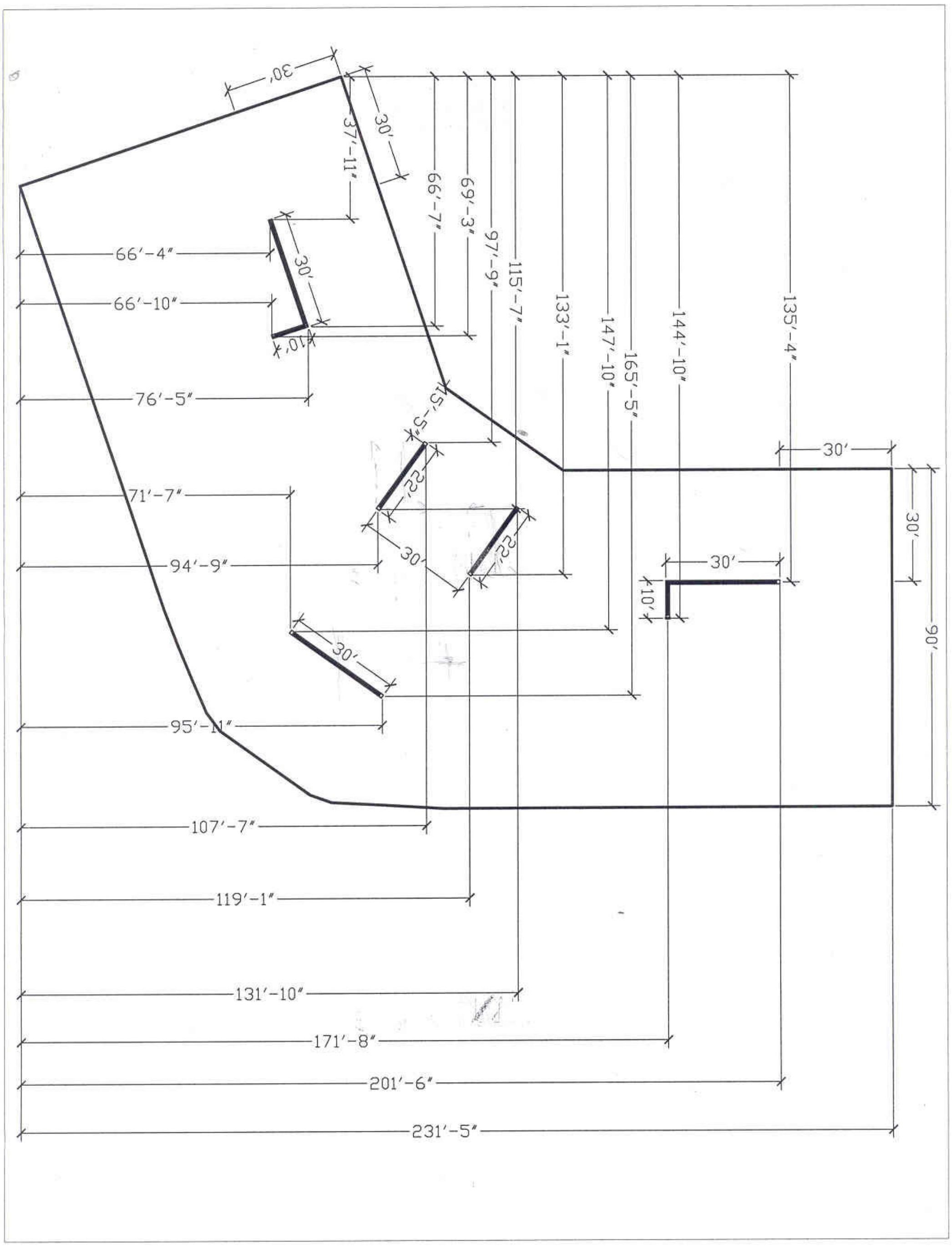
$\frac{Kl}{r} < 200$ OK

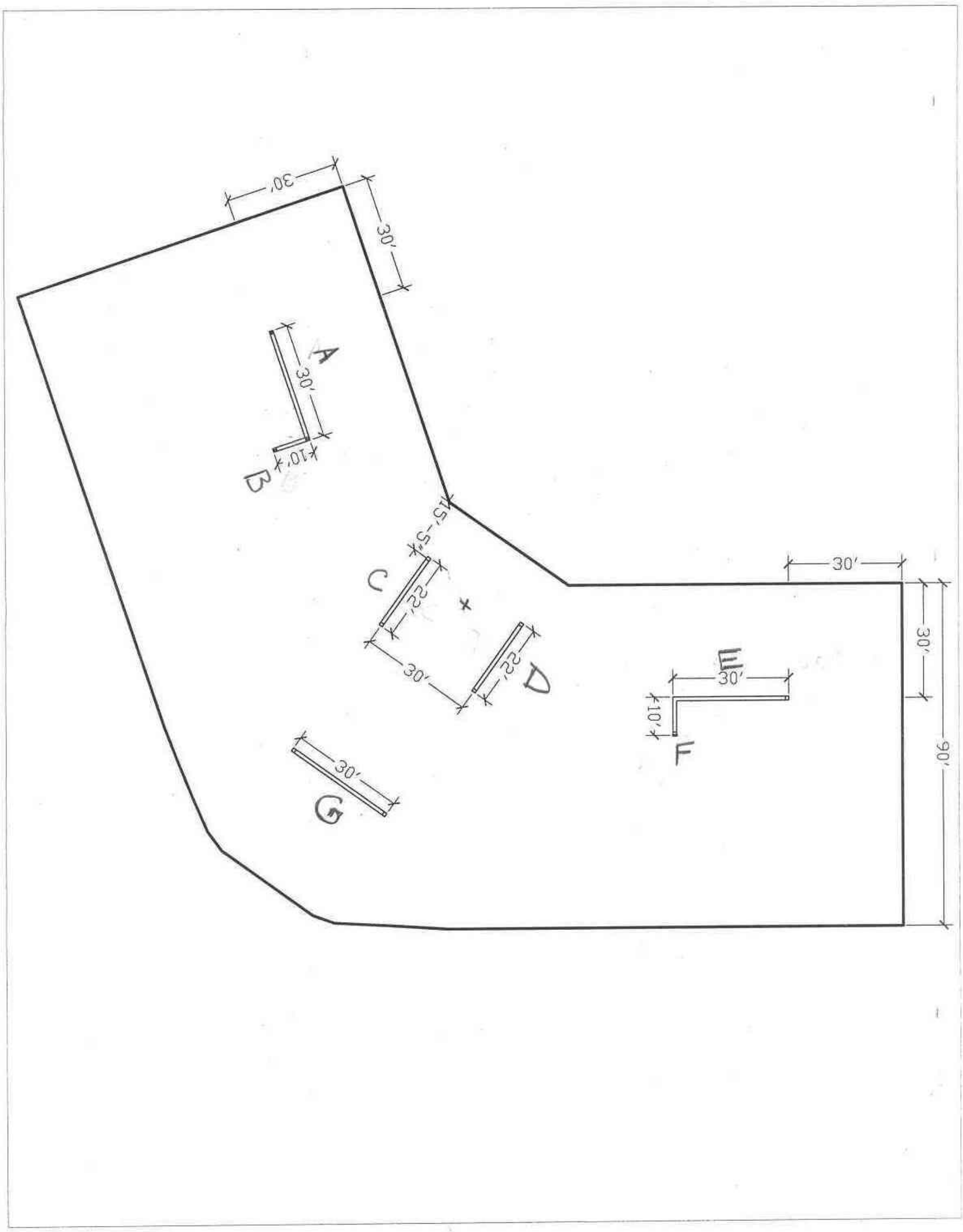
USE **2 WT 8 X 33.5**

TOT WEIGHT = 2(20.5')(33.5) = **1373.5 lbs**



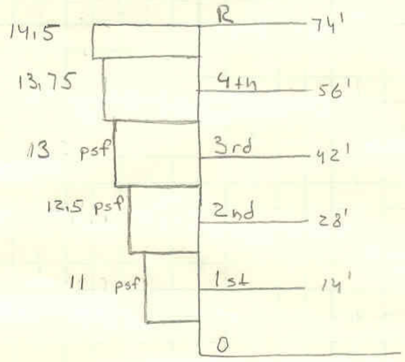
30'
30'
A
30'
10'
B
C
22'
30'
22'
D
30'
30'
E
30'
10'
F
90'
30'
G

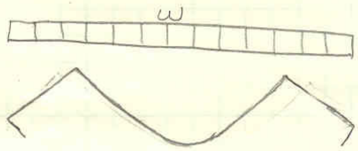

30'
30'
37'-11"
66'-7"
69'-3"
97'-9"
115'-7"
133'-1"
147'-10"
165'-5"
144'-10"
135'-4"
66'-4"
66'-10"
30'
10'
76'-5"
15'-5"
22'
71'-7"
22'
30'
94'-9"
30'
30'
30'
10'
90'
30'
95'-11"
107'-7"
119'-1"
131'-10"
171'-8"
201'-6"
231'-5"



30'
30'
A
30'
10'
B
15'-5"
C
22'
30'
22'
D
30'
30'
E
30'
10'
F
90'
30'
G

Load Distribution (1)





Total distributed wind load at each story height

$w_{ROOF} = 10.5'(14.5 \text{ psf}) = 152.25 \text{ plf}$

$w_4 = 152.25 + 14'(13.75 \text{ psf}) = 344.75 \text{ plf}$

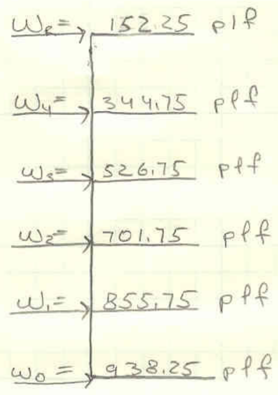
$w_3 = 344.75 \text{ plf} + 14'(13 \text{ psf}) = 526.75 \text{ plf}$

$w_2 = 526.75 \text{ plf} + 14(12.5 \text{ psf}) = 701.75 \text{ plf}$

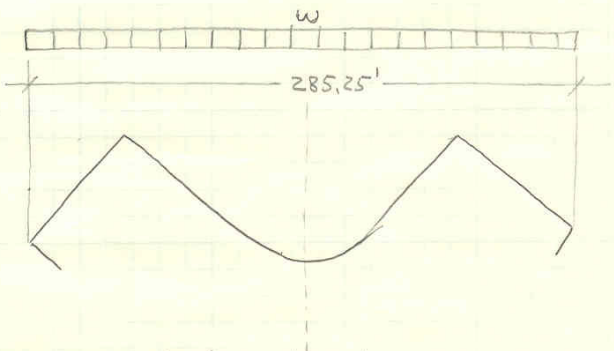
$w_1 = 701.75 \text{ plf} + 14(11 \text{ psf}) = 855.75 \text{ plf}$

$w_0 = 855.75 \text{ plf} + 7.5(11 \text{ psf}) = 938.25 \text{ plf}$

Load Distribution (2)



UNIFORMLY DISTRIBUTED WIND LOAD AT EACH STORY



Total equivalent point Load:

$W_R = 152.25(285.25) = 43.4^K$

$W_4 = 344.75(285.25) = 98.3^K$

$W_3 = 526.75(285.25) = 150.3^K$

$W_2 = 701.75(285.25) = 200.2^K$

$W_1 = 855.75(285.25) = 244.1^K$

Load Distribution (3)

THE CENTER OF RIGIDITY

$$\bar{x} = \frac{15'(22') - 15'(22')}{44} = 0$$

$$\bar{y} = \frac{32.5'(30') + 97.25'(5.88)(2) + 109.58(24.58)(2)}{30 + 2(5.88) + 2(24.58)} = 82.6'$$

POLAR MOMENT OF INERTIA

$$I_p = I_{xx} + I_{yy}$$

$$I_{xx} = \sum \ell_x y^2$$

$$= 30'(50.17)^2 + 24.58'(27.\ )^2(2) + 5.88'(14.67)^2(2)$$

$$= 113{,}879.36\ ft^3$$

$$I_{yy} = \sum \ell_y x^2 = 22'(15^2)2 + 17.58(71.08)^2(2) + 8(61.75)^2(2)$$

$$= 248550.20\ ft^3$$

$$I_p = 113879.36 + 248550.20 = 362429.56\ ft^3$$

Load Distribution (4)

Wind parallel to the $\bar{Y}$ axis

Roof

$W_R = 43.4^k$ Wall Force $= \dfrac{W \ell_i}{\Sigma \ell} + \dfrac{M_T \times \ell}{I_P}$

$M_T = 0$

Wall $A_y = \dfrac{43.4^k(17.59')}{2(17.59)+2(22)+2(8)} + 0 = 8.02^k$

Wall $B_y = \dfrac{43.4(8)}{95.18} + 0 = 3.65^k$

Wall $C = \dfrac{43.4(22)}{95.18} = 8.67^k$

Wall D = Wall C $= 8.67^k$

Wall E_y = Wall A $= 8.02^k$

Wall F_y = Wall B $= 3.65^k$

36.54 ≈ 36.7 OK

Wall $A = \dfrac{8.02}{\cos 55} = 13.98^k$

Wall $B \ \dfrac{3.65}{\cos 35} = 4.46^k$

Wall E = Wall A $= 13.98^k$

Wall F = Wall B $= 4.46^k$

Load Distribution (5)

Wind Parallel to Y-axis

4th story

$W_4 = 98.3^k$ $M_T = 0$

	(k)
Wall A	= 31.67
Wall B	= 10.092
Wall C	= 22.72
Wall D	= 22.72
Wall E	= 31.67
Wall F	= 10.09

3rd story

$W_3 = 150.3$ $M_T = 0$

	(k)
Wall A	= 48.43
Wall B	= 15.42
Wall C	= 34.74
Wall D	= 34.74
Wall E	= 48.43
Wall F	= 15.42

2nd story

$W_2 = 200.2^K$ $M_T = 0$

Wall	Force (k)
A	64.51
B	20.55
C	46.27
D	46.27
E	64.51
F	20.51

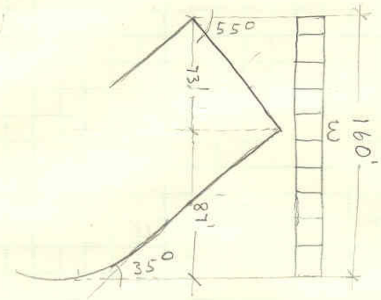
1st story

$W = 244.1^K$ $M_T = 0$

Wall	Force (k)
A	78.65
B	25.05
C	56.42
D	56.42
E	78.65
F	25.05

Load Distribution (6)

Wind Parallel the $\bar{X}$-axis



(K)

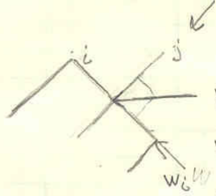
$W_R = 152.25(160) = 24.36$

$W_4 = 344.75(160) = 55.16$

$W_3 = 526.75(160) = 84.28$

$W_2 = 701.75(160) = 112.28$

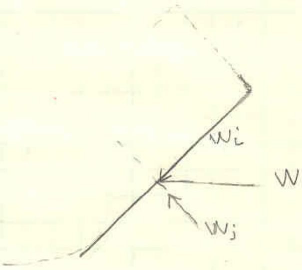
$W_1 = 855.75(160) = 136.92$



$W = 73(152.25) = \boxed{11.114^K}$

$W_i = 11.11 \cos 55 = 6.37^K$

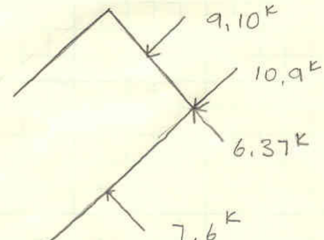
$W_j = 11.114 \sin 55 = 9.10^K$



$W = 87(152.25) = \boxed{13.25^K}$

$W_i = 13.25 \cos 35 = 10.9^K$

$W_j = 13.25 \sin 35 = 7.6^K$



Final Loads

Load Distribution (7)

Roof

$M_T = -11.114^K(41') + 13.25(39.17) = 63.3^{1K}$

$W = 24.36^K$

Direct Forces

Wall A $= \frac{24.36(24.583)}{2(24.583)+2(5.875)+30} / \cos 55 = 11.60^K$

Wall B $= \frac{5.875(24.36)}{90.916} / \cos 35 = 1.92^K$

Wall E $= 11.60 = 11.60$

Wall F $= 1.92$

Wall G $= \frac{30(24.36)}{90.916} = 8.04^K$

Forces due to Torsion

Wall $A_T = \frac{(63.3^{1K})(19.08)(30')}{362429.56\ ft^3} = 0.1^K$ add

Wall $B_T = \frac{(63.3)(47.25)(10')}{362429.56} = 0.08^K$ subtract

Wall C $= \frac{(63.3)(15)(22)}{362429.56} = 0.06^K$

Wall D $= 0.06^K$

} oposite directions

Wall E $= 0.1^K$ add

Wall F $= 0.08^K$ subtract

Wall G $= \frac{63.3(50.17)(30)}{362429.56} = 0.26^K$ add

Total Forces in each Wall:

Wall	Force (K)	Wall	Force (K)
A	11.7	E	11.7
B	1.84	F	1.84
C	0.06	G	8.36
D	-0.06		

Wind parallel to the $\bar{x}$-axis

The distribution of a lateral load on the remaining stories are proportional to the calculated distribution of forces at the roof level.

As shown in a chart below, the wind load case parallel to $\bar{y}$-axis controls the system, with the exeption of the wall G

Distribution of forces at roof level			
Comparison Table I			
X-axis		Y-axis	
wall	Force (K)	Wall	Force (K)
A	11.7	A	13.98
B	1.84	B	4.46
C	0.06	C	8.67
D	0.06	D	8.67
E	11.7	E	13.98
F	1.84	F	4.46
G	8.30	G	0

Load Distribution (9)

wind Parallel to the $\bar{x}$ axis

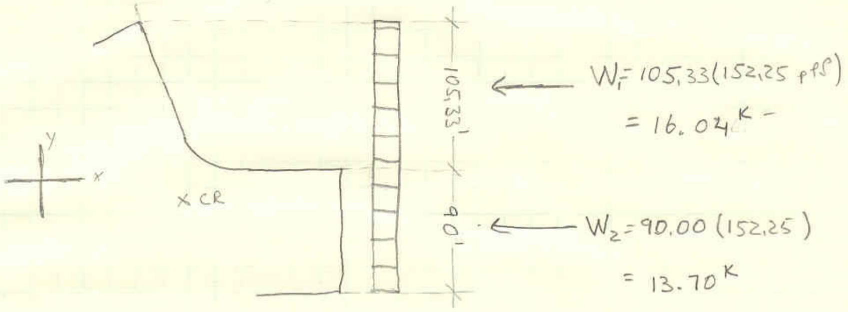
Wall G	
Story	Force (k)
R	8.30
4	18.80
3	28.72
2	38.26
1	46.65

22-141 50 SHEETS
22-142 100 SHEETS
22-144 200 SHEETS
AMPAD

Load Distribution (10) NE - SW

Wind in NE - SW direction

Roof



Moment arms:

W_1: 63.25'

W_2: 34.42'

$M_T = 16.04(63.25) - 13.70(34.42)$

$= \boxed{542.98^{'K}}$

Wall	$Length_x$	$Length_y$
A	9.58'	28.83'
B	9.5'	3.08'
C	12.83	17.83
D	12.83	17.83
E	30'	0
F	0	10'
G	24.33'	17.58'

Wind in the NE-SW direction

$$\text{Walls} = \frac{W\ell}{\Sigma \ell} + \frac{M_T \times \ell}{I_P}$$

Final Distributed Forces in each wall

Wall	Direct shear Force	M_T Force	Total
A	10.377 K	- 0.863	9.51
B	3.00 K	+ 0.872	3.87
C	6.603 K	+ 0.49	7.10
D	6.603 K	- 0.49	6.11
E	9.00 K	- 0.874	8.13
F	0 K	0.877	0.88
G	9.01 K	- 2.25	6.67

Roof Level

Comparison Table 2

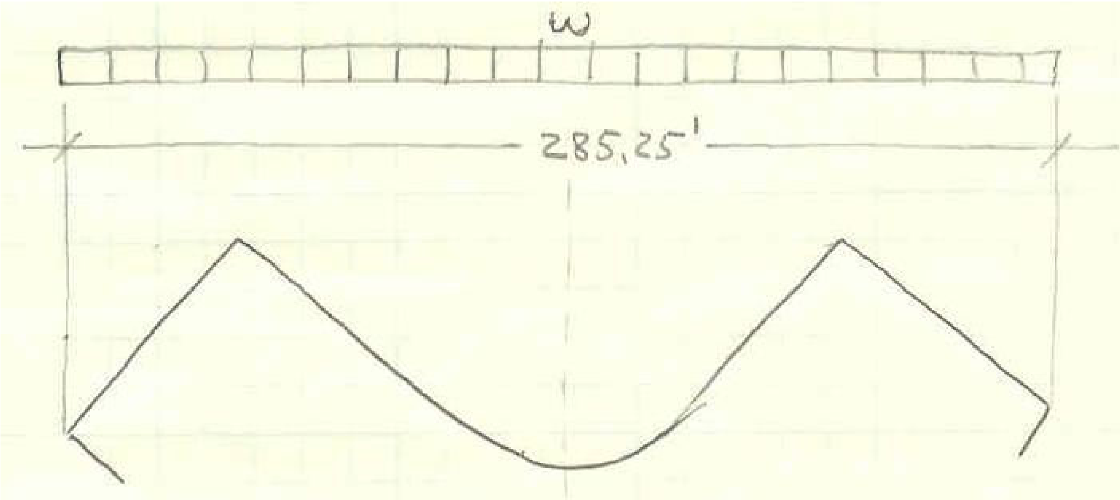
Wall	Load ∥ $\bar{X}$ (K)	Load ∥ $\bar{Y}$ (K)	Load ∥ NE-SW (K)
A	11.7	13.98	9.51
B	1.84	4.46	3.87
C	0.06	8.67	7.10
D	0.06	8.67	6.11
E	11.7	13.98	8.13
F	1.84	4.46	0.88
G	8.30	0	6.67

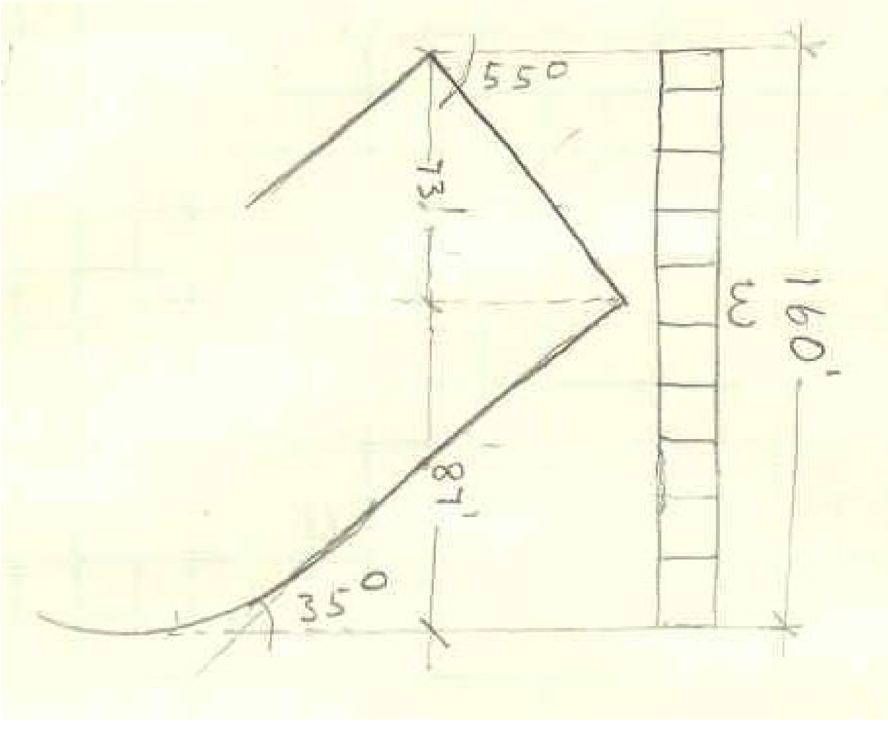
* Underlined are the controlling forces.

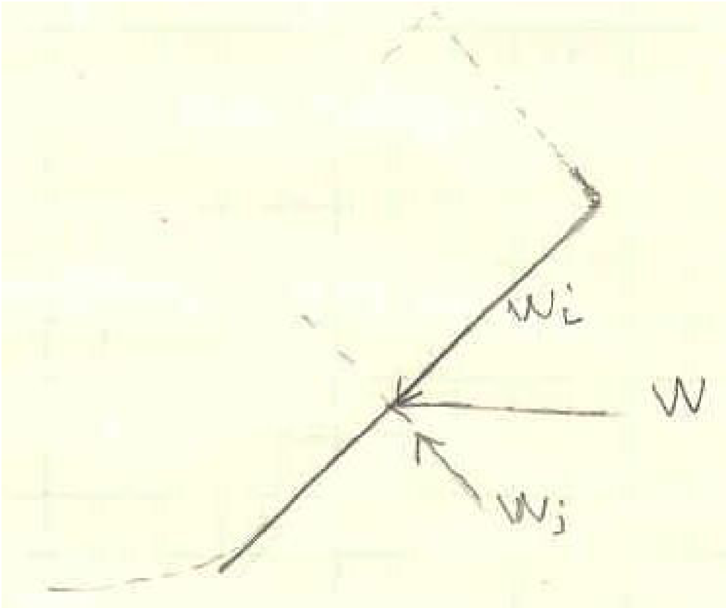
Wind parallel with the NE-SW axis

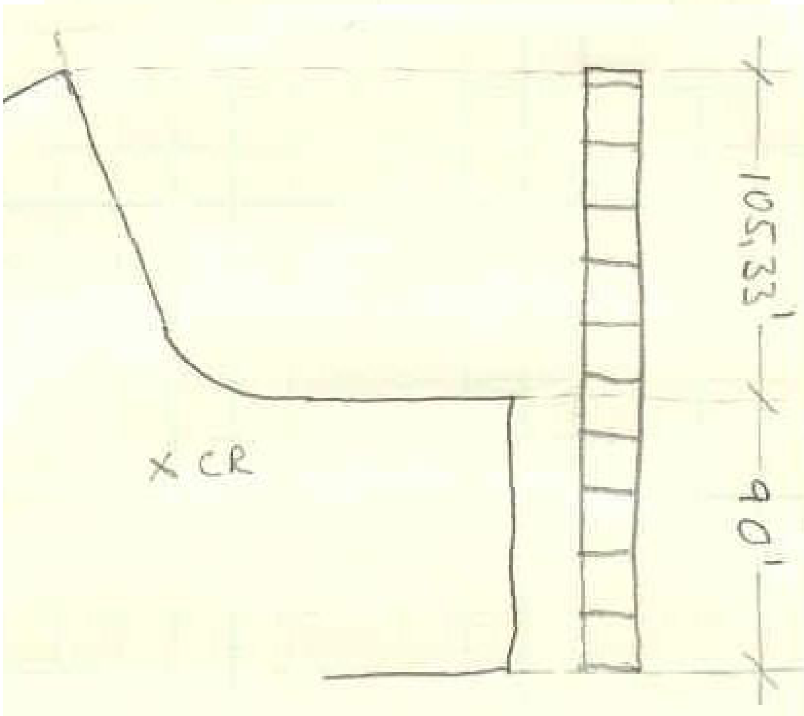
Wall G

Level	W_P (k)	Total Force in wall (k)
R	29.7	6.67
4	67.3	15.30
3	102.9	23.39
2	137.1	31.17
1	167.2	38.01


ω
285.25'


55°
73'
ω
160'
87'
35°


Wi
W
Wj


X CR
105,33'
90'

LOAD DISTRIBUTION TABLES

CASE 1

Resulting Distributed Forces at the top of the frame in kips

Frame	Garage	Level 1	Level 2	Level 3	Level 4
A	*78.65*	*64.51*	*48.43*	*31.67*	*13.98*
B	*25.05*	*20.55*	*15.42*	*10.09*	*4.46*
C	*56.42*	*46.27*	*34.74*	*22.72*	*8.67*
D	*56.42*	*46.27*	*34.74*	*22.72*	*8.67*
E	*78.65*	*64.51*	*48.43*	*31.67*	*13.98*
F	*25.05*	*20.55*	*15.42*	*10.09*	*4.46*
G	0	0	0	0	0

CASE 2

Resulting Distributed Forces at the top of the frame in kips

Frame	Garage	Level 1	Level 2	Level 3	Level 4
A	N/A	N/A	N/A	N/A	11.7
B	N/A	N/A	N/A	N/A	1.84
C	N/A	N/A	N/A	N/A	0.06
D	N/A	N/A	N/A	N/A	0.06
E	N/A	N/A	N/A	N/A	11.7
F	N/A	N/A	N/A	N/A	1.84
G	*46.65*	*38.26*	*28.72*	*18.8*	*8.3*

CASE 3

Resulting Distributed Forces at the top of the frame in kips

Frame	Garage	Level 1	Level 2	Level 3	Level 4
A	N/A	N/A	N/A	N/A	9.51
B	N/A	N/A	N/A	N/A	3.87
C	N/A	N/A	N/A	N/A	7.1
D	N/A	N/A	N/A	N/A	7.1
E	N/A	N/A	N/A	N/A	9.51
F	N/A	N/A	N/A	N/A	3.87
G	N/A	N/A	N/A	N/A	6.67

NOTE: N/A - values that do not control the design

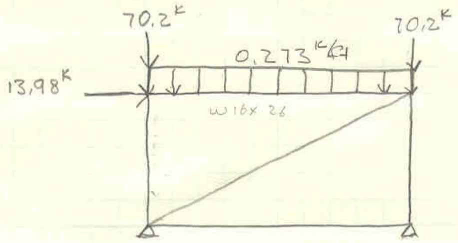
Italicized are the values that control the design

	Steel Memeber design Summary				
		Braced Frame A Size			
Frame	**Level 0**	**Level 1**	**Level 2**	**Level3**	**Level 4**
AB	W12x106	W12x106	W12x106	W12x50	W12x50
BC	W18x35	W18x35	W18x35	W18x35	W16x26
CD	W12x106	W12x106	W12x106	W12x50	W12x50
AC	W8x58	W8x48	W8x35	W8x31	W8x21
		Braced Frame B Size			
Frame	**Level 0**	**Level 1**	**Level 2**	**Level3**	**Level 4**
AB	W12x65	W12x65	W12x65	W12x35	W12x35
BC	W18x35	W18x35	W18x35	W18x35	W21x50
CD	W12x65	W12x65	W12x65	W12x35	W12x35
AC	W8x35	W8x31	W8x28	W8x24	W8x21
		Braced Frame C Size			
Frame	**Level 0**	**Level 1**	**Level 2**	**Level3**	**Level 4**
AB	W12x96	W12x96	W12x96	W12x40	W12x40
BC	W18x40	W18x40	W18x40	W18x40	W18x35
CD	W12x96	W12x96	W12x96	W12x40	W12x40
AC	W8x48	W8x35	W8x31	W8x28	W8x21
		Braced Frame D Size			
Frame	**Level 0**	**Level 1**	**Level 2**	**Level3**	**Level 4**
AB	W12x96	W12x96	W12x96	W12x40	W12x40
BC	W18x40	W18x40	W18x40	W18x40	W18x35
CD	W12x96	W12x96	W12x96	W12x40	W12x40
AC	W8x48	W8x35	W8x31	W8x28	W8x21
		Braced Frame E Size			
Frame	**Level 0**	**Level 1**	**Level 2**	**Level3**	**Level 4**
AB	W12x106	W12x106	W12x106	W12x50	W12x50
BC	W18x35	W18x35	W18x35	W18x35	W16x26
CD	W12x106	W12x106	W12x106	W12x50	W12x50
AC	W8x58	W8x48	W8x35	W8x31	W8x21
		Braced Frame F Size			
Frame	**Level 0**	**Level 1**	**Level 2**	**Level3**	**Level 4**
AB	W12x65	W12x65	W12x65	W12x35	W12x35
BC	W18x35	W18x35	W18x35	W18x35	W21x50
CD	W12x65	W12x65	W12x65	W12x35	W12x35
AC	W8x35	W8x31	W8x28	W8x24	W8x21
		Braced Frame G Size			
Frame	**Level 0**	**Level 1**	**Level 2**	**Level3**	**Level 4**
AB	W12x96	W12x96	W12x96	W12x40	W12x40
BC	W16x26	W16x26	W16x26	W16x26	W16x26
CD	W12x96	W12x96	W12x96	W12x40	W12x40
AC	W8x35	W8x31	W8x31	W8x24	W8x21

Braced Frame Design (1) 4→ Roof

Braced Frame – Wall A

Level: 4 – Roof



Trib Area = (30')(3.5') = 105 ft²

LL = 30 psf P_u = 78 psf

DL = 25 psf

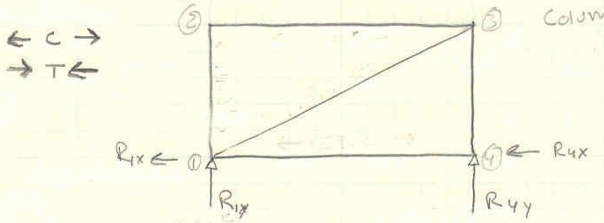
$W_u = 1.2(25)(3.5) + 1.6(30)(3.5)$

$= 273$ lb/ft

$= 0.273$ K/ft

Column = (900 ft²)(78 psf)

$= 70.2^K$

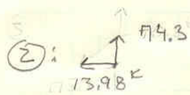


$\Sigma M_{①} = 0 = 14(13.98) + (8.19)(15) + 70.2(30) - 30 R_{4y}$

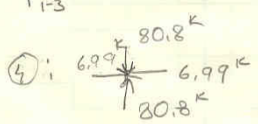
$R_{4y} = 80.8^K$

$R_{1y} = 2(70.2) + 8.19 - 80.8 = 67.77^K$

$R_{1x} = R_{4x} = \frac{13.98}{2} = 6.99^K$



$\frac{74.3 - 67.77}{F_{1-3}} = \frac{14}{33.1}$

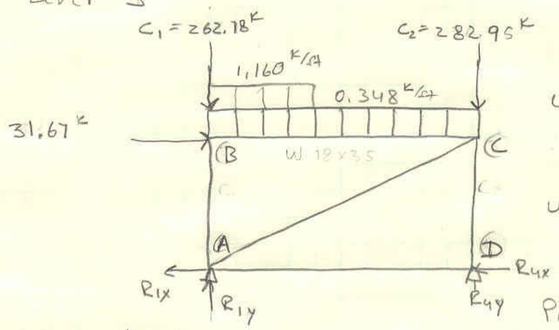


$F_{1-3} = 15.44^K$
$F_{1-2} = 74.3^K$
$F_{2-3} = 13.98^K$
$F_{3-4} = 80.8^K$
$F_{1-4} = 6.99^K$

Braced Frame Design (2) 3→4

Braced Frame - Wall A

Level 3



$w_{u_1} = [1.2(60) + 1.6(100)]1.5'$

$= 0.348\ ^K/ft$

$w_{u_2} = [1.2(60 + 1.6(100)]5'$

$= 1.160\ ^K/ft$

$P_u = 1.2(60) + 1.6(100)$

$= 232$ psf

Column Loads:

$C_1 = \frac{(900\ ft^2)(232\ psf)}{1000} - 14.89^K + 87.77 + 0.5 = 262.18^K$

$C_2 = \frac{(900)(232)}{1000} - 7.15^K + 80.8 + 0.5 = 282.95^K$

$C_1 = (A_T)(P_U)$ - Uniformly Dist loads + Column loads from L4

$\Sigma M_{①} = 0 = 14(31.67^K) + 1.16(10')(5') + 0.348(30)(15) + 282.95(30) - 30 R_{Dy}$

$R_{Dy} = 304.88^K$

$R_{Ay} = 1.16(10) + .348(30) + 282.95 + 262.18 - 304.88$

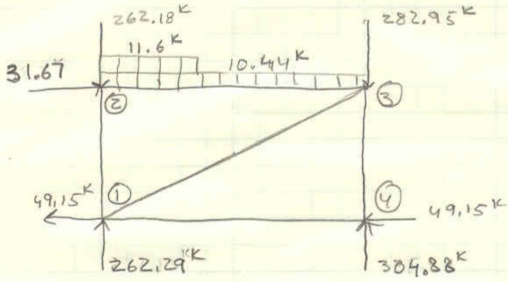
$= 262.29^K$

$R_{Ax} = \frac{98.3}{2} = 49.15^K$

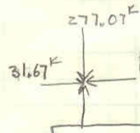
$R_{4x} = \frac{98.3}{2} = 49.15^K$

Braced Frame - Wall A

Level 3



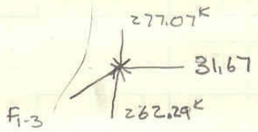
(2):



$F_{2-3} = 31.67^k$ (C)

$F_{12} = 277.07^k$ (C)

(1)



$$\frac{277.07 - 262.29}{F_{1-3}} = \frac{14}{33.1}$$

$F_{1-3} = 34.9^k$ (T)

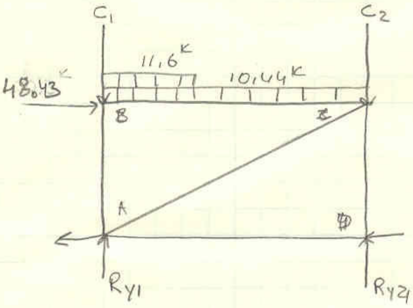
(4) $F_{3-4} = 304.88^k$ C

MEMBER	Force (K)	T or C
F_{1-2}	277.07	C
F_{1-3}	34.9	T
F_{2-3}	31.67	C
F_{3-4}	304.88^k	C

Braced Frame Design (4) Level 2-3

Braced Frame - Wall A

Level 2



$C_1 = 456.7^k$

$C_2 = 507.03^k$

$R_{yA} = 449.00^k$

$R_{yD} = 537.05$

$R_{xA} = 24.2^k$

$R_{xD} = 24.2^k$

MEMBER	axial Force (K)	Direction
A B	472	C
B C	48	C
C D	537	C
A C	53	T

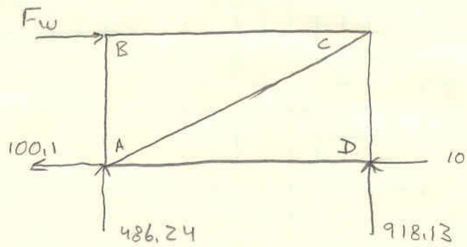
Braced Frame Design (5) Level 1-2

Level 1

$C_1 = 643.41^K$ $R_{Ay} = 628.2^K$

$C_2 = 739.20^K$ $R_{Dy} = 776.5^K$

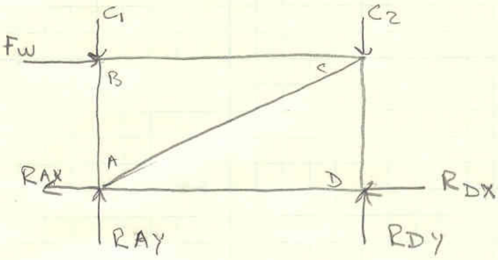
$F_{wind} = 64.51^K$ $R_{Ax} = R_{Dx} = 32.25^K$



MEMBER	AXIAL Force (K)	Direction
AB	658	C
BC	64.5	C
CD	776	C
AC	71.2	T

Broced Frome Design (6) Level 0-1

Level 0



F_W = 78.65 K

C_1 = 822.61 K

C_2 = 978.65 K

R_{Ax} = 39.3 K

R_{Ay} = 800.8 K

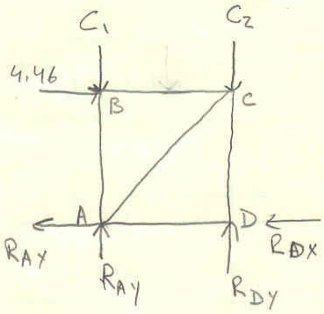
R_{Dx} = 39.3 K

R_{Dy} = 1022.5 K

MEMBER	Axiol Force	Dirrection
AB	837	C
BC	78.7	C
CD	1023	C
AC	86.8	T

Wall B

Level: 14



$C_1 = (5+15)'(30)'(78\ psf) = 46.8^k$

$C_2 = (15')(30')(78\ psf) = 35.1^k$

$F_w = 4.46^k$

$R_{Ax} = 2.23^k$

$R_{Ay} = 40.56^k$

$R_{Dx} = 2.23^k$

$R_{Dy} = 41.34^k$

MEMBER	AXIAL FORCE (K)	DIRECTION
AB	46.8	C
BC	4.46	C
CD	41.34	C
AC	7.67	T

Braced Frame Design (8) Wall B - 3

Level 3

$C_1 = (20')(30')(232 psf) + 40.56^K + 0.5 = 180.26^K$

$C_2 = (15')(30')(232 psf) + 41.34^K + 0.5 = 146.24^K$

$F_\omega = 10.09^K$

MEMBER	AXIAL FORCE (K)	Direction
AB	180.26	C
BC	10.9	C
CD	161.5	C
AC	18.75	T

Braced Frame Design (9) Wall B, Levels 0 – 2

WALL B

MEMBER	Level 2 AXIAL Force (K)	Level 1 AXIAL Force (K)	Level 0 AXIAL Force (K)	Direction
AB	304.7	422.8	533.7	C
BC	15.42	20.55	25.05	C
CD	288	421.662	561.6	C
AC	26.5	35.346	43.1	T
C1	304.7	422.8	533.7	
C2	266.4	392.9	526.7	

AMPAD 22-141 50 SHEETS 22-142 100 SHEETS 22-144 200 SHEETS

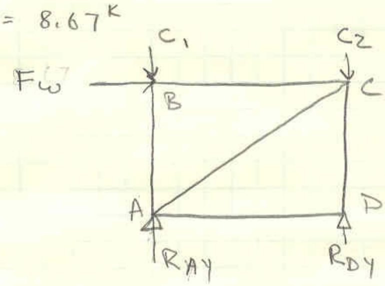
Braced Frame Design (10) Wall C – 4 – Roof→0

Level 4 **WALL C**

$C_1 = (10+11)(15+5)(78) = 32.76^K$

$C_2 = (11+20)(15+10)(78) = 60.45^K$

$F_w = 8.67^K$



Level 3

$C_1 = R_{AY} + (420\ ft^2)(.232\ ksf) + 0.5$

$C_2 = R_{DY} + (775\ ft^2)(.232\ ksf) + 0.5$

$F_w = 22.72^K$

C

WALL C Force Distribution TABLE

MEMBER	Level 4 AXIAL F (K)	Level 3 axial F (K)	Level 2 axial F (K)	Level 1 axial F (K)	Level 0 axial F (K)	Direction
AB	32.76	125.18	208.6	284.5	353	C
BC	8.67	22.72	34.74	46.27	56.42	C
CD	66.0	260.73	463.1	672.9	889.1	C
AC	10.28	26.9	41.2	54.8	66.88	T
C1	32.76	125.2	208.7	284.5	353.0	
C2	60.45	246.3	441.0	643.4	853.2	
Fw	8.67	22.72	34.74	46.27	56.42	

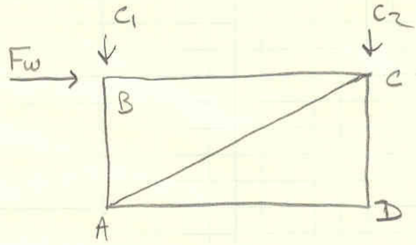
Braced Frame Distribution (11) Wall G – 4-Roof

$A_{TC1} = (20+17)(15) + \frac{(15)(15)}{2} + 7(20) = 807\ ft^2$

$A_{TC2} = 807\ ft^2$

$C_1 = C_2 = (807)(.232) = 187.2^K$ floors

$C_1 = C_2 = (807)(.78) = 62.9^K$ roof



WALL G FORCE DISTRIBUTION TABLE – Axial Force

Member	Level 4 (K)	Level 3 (K)	Level 2 (K)	Level 1 (K)	Level 0 (K)	Direction
AB	62.9	246.7	422.5	591.9	755.2	C
BC	8.3	18.8	28.72	38.26	46.65	C
CD	66.78	266.4	472.4	684.5	901.8	C
AC	9.16	22.28	34.04	45.35	55.29	T
C_1	62.9	246.7	422.5	591.9	755.2	
C_2	62.9	254.4	454.1	660.1	872.2	
Fw	8.3	18.8	28.72	38.26	46.65	

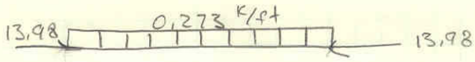
Beam Design

Beam Design (1) Wall A

Wall A

Level: 14 story

Member B-C



$M_{max} = \frac{w\ell^2}{8} = \frac{0.273(30^2)}{8} = 30.71^{1K}$

$P_u = 13.98^K$

$KL_x = 30'$ $\quad B_{1x} = 1.0$

$M_{ux} = (1.0)(30.71) = 30.71^{1K}$

TRY W16x26 $\quad \phi M_p = 166^{1K} \gg 30.71^{1K}$

Tensile Force is too small to be considered.

$I_x = 301\ in^4 \quad E = 29000\ ksi$

$A_{15'} = \frac{5(.273)(30^4)(1728)}{384(29000)(301)} = 0.57\ in < \frac{30(12)}{360} = 1''$

OK

use **W16x26**

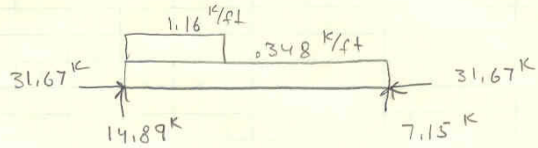
W18x35

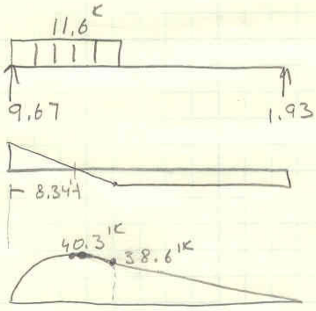
Beam Design (2) Wall A

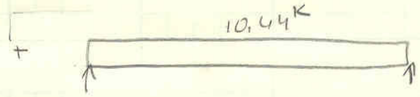
Level 3

Wall A

Beam BC







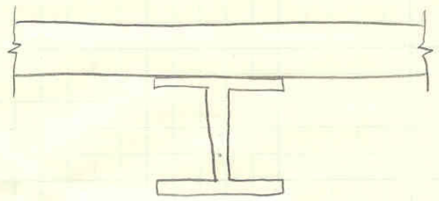
$M_c = \frac{.348(30^2)}{8} = 39.15'^K$

$M_{x=10} = 34.3'^K$

$M_{max} = 38.6 + 34.3 = 73'^K$ EI

TRY W18x35

Nominal Flexural Strength Mn



beff = $\frac{30}{4}$ or 2'

$C_{conc} = (.85)(5)(2')(12)(4'') = 408^K$

$T_{st} = (.50)(10.3) = 515$

$T_{st} > C_{con} \Rightarrow 408 = 50(10.3 - x6)$

PNA = x = 0.357" from TOF

$M_p = 408(2) + 50(6)(.357)\left(\frac{.352}{2}\right) + 50(10.3 - 6(.357))(\approx 10)$

$= 406.3'^K$

Level 3 cont...

$\lambda_c = \frac{1.0(30)}{7.04\pi}\sqrt{\frac{50}{29000}} = 0.0563$

$\phi P_n = 0.85(0.658^{(0.0563)^2})(50)(10.3) = 437.17$

$\frac{P_u}{\phi P_n} = \frac{31.67}{437.17} = 0.07 < 0.2 \Rightarrow H1-1b$

$\frac{31.67^k}{2(437.17)} + \left(\frac{73^{1k}}{406.3^{1k}(.85)}\right) = 0.248 \leq 1.0$ OK

$\Delta = 0.428 < \frac{1}{360}(12) = 1$ OK

TRY

W16x26

$C = 0.85(5)(2)(12)(4) = 408^k$

$T_s = (50)(7.68) = 384$

$a = \frac{384}{0.85(5)(24)}$ 3.76"

$M_p = 50(7.68)(7.85) + 384(2.118) = 3827.6 = 319.0^{1k}$

$\phi M_n = 271.1^{1k}$

$\phi P_n = 325.9$

$\frac{P_u}{\phi P_n} = 0.097 < 0.2 \Rightarrow H1-1b$

$H1-1b \Rightarrow 0.3179 \leq 1.0$ OK

$\Delta =$ NG

USE **W18X35**

Summary of Wall A - Beams B-C

4	W16x26	0	W18x35
3	W18x35		
2	W18x35		
1	W18x35		

Beam Design | 4 | Wall A

Level 2

TRY W18x35 4" conc

$\phi M_n = 345.4^{'K}$

$\phi P_n = 437.17^K$

$\frac{P_u}{\phi P_n} = \frac{48.43^K}{437.17^K} = 0.11 < 0.2 \Rightarrow$ H1-1b

H1-1b = 0.266 < 1.0 OK

Δ OK

Level 1

W18x35 4" conc

$\phi M_n = 345.4^{'K}$

$\phi P_n = 437.17^K$

$\frac{P_u}{\phi P_n} = \frac{64.51}{437.17} = 0.15 < .2 \Rightarrow$ H1-1b

H1-1b = 0.285 < 1.0 OK

Δ OK

Level 0

W18x35 4" conc

$P_u = 78.65$

$\frac{P_u}{\phi P_n} = 0.18 < .2$ use H1-1b

H1-1b ⇒ 0.3 < 1.0 OK

Δ OK

Beam Design | 5 | Wall B

Level 4

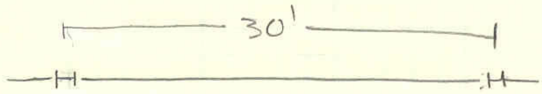
$M_u = 29.25^{1k}$

$P_u = 4.46^{k}$

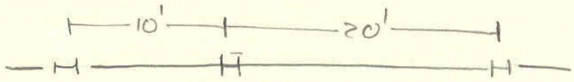
W 21x50

use orignal size

Beam Got stronger with the addition of a column



original Design



New Design

Beam Design (6) Wall B

Level 3 → Level C

use W 18 x 35

$w_u = (0.232)(30) = 6.96^{k/ft}$

$M_u = \frac{6.96(10^2)}{8} = 87^{1k}$

$P_u = 10.09^k$

• Nominal Flex. Strength

$b_{eff} = \frac{10'}{4}$ or $30'$

$= 30''$

$C_{conc} = 0.85(5)(30)(4) = 510^k$

$T_{st} = 515^k$

$a = \frac{-510 + 50(10.3)}{50(6)} = 0.02''$ from TOF

$M_n = 510(2) - 50(6)(.02)\left(\frac{.02}{2}\right) + 50(10.3 - 6(.02))(8.9)$

$= 462.5^{1k}$

$\phi M_n = 393.1^{1k}$

$\phi P_n = 437.7^k$

$\frac{P_u}{\phi P_n} = 0.023 < 0.2 \Rightarrow H1-1b$

$H1-1b = 0.233 < 1.0$ ok

$\Delta \Rightarrow$ ok

use 18 x 35

Beam Design (7) Wall B

Levels 2, 1, 0

W18x35

Level 2

$P_u = 15.42^k$

$\frac{P_u}{\phi P_n} = 0.035 < .2 \Rightarrow H1-1b = 0.239 < 1.0$ ok

Δ = OK

Levels	Beam B-C size
4	W21x50
3	W18x35
2	↓
1	↓
0	W18x35

Wall C

$b_{eff} = 66''$

$a = \frac{515}{.85(5)(66)} = 2.8136$

$M_n = 590(2.95) + 590(8.95) = 585^{1K}$

$\phi M_n = 497.3^{1K}$

$\phi P_n = 501.2$

$M_{p4} = 117.975^{1K}$

$M_{p0\text{-}3} = 350.9^{1K}$

$I_{LB} = 1450 \text{ in}^4$

$\Delta = \frac{5 w \ell^4}{384 E I_{LB}}$

$\frac{\ell}{360} = \frac{22(12)}{360} = 0.733$

Level	Axial Force	Mp	Size	Δ
4	8.67	118	W 18x35	OK
3	22.72	351	W 18x40	0.72
2	34.74	351	W 18x40	0.72
1	46.27	351	W 18x40	0.72
0	56.42	351	W 18x40	0.72

Wall G

$w_E = 0.468$ K/ft

$w = 0.696$ K/ft

$b_{eff} = 36"$

W 16 x 26

$\phi M_n = 290$ 'K

$\phi P_n = 325$ K

$I_{LB} = 730$ in^4

Max Δ = 1"

Roof $M_p = 52.65$ 'K

Floor $M_p = 78.3$ 'K

Eq H1-1b governs

Level	Axial Force	Mp	size	Deflection (in)
4	8.3	52.65	W16x26*	.977
3	18.8	78.3	W16x26	0.6
2	28.72	78.3	W16x26	0.6
1	38.26	78.3	W16x26	0.6
0	46.05	78.3	W16x26	0.6

* Noncomposite Beam

NEW 1

Typical Girder design.

Loads:

LL = 100 psf

DL = 60 psf

LL Reduction

$0.25 + \frac{15}{\sqrt{2(30)^2}} = 0.6 > 0.4$ OK

LL = 100(0.6) = 60 psf

Factored Loads

1.2(60)(30') + 1.6(60)30' = 5040 PLF = **5.04 KLF**

Max Moments & shear

$M = \frac{wL^2}{8} = \frac{5.04(30^2)}{8} = \boxed{567^{1K}}$

$V_u = \frac{wL}{2} = \frac{5.04(30)}{2} = \boxed{75.6^K}$

Composite Beam:

Assume a = 2 in

$Y_2 = Y_{con} - \frac{a}{2} = 4 - \frac{2}{2} = 3$ in

TRY: W 24x62 $\phi_b M_n = 962^{1K}$ @ TFL, $\Sigma Q_n = 915^K$

2

Typical Girder Design

check Y_2:

$$b \le \begin{cases} 2 \times \frac{30}{8} = 7.5' & \text{— controls} \\ 30' \end{cases}$$

$b = 90''$

$$a_{req} = \frac{\Sigma Q_n}{.85 f'c\, b} = \frac{915^k}{.85(3)(90'')} = 3.99''$$

$$Y_2 = 4 - \frac{3.99}{2} = 2 < 3 \quad \boxed{OK}$$

$I_{LB} = 3580$ [LRFD T 5-15]

$$\Delta = \frac{5wL^4}{384EI} = \frac{5(5.04)(30^4)(12^3)}{384(29000)(3580)} = 0.885''$$

$$\frac{30(12)}{360} = 1'' > 0.885'' \quad OK$$

Shear check

$\phi V_n = 275^k > 75.6^k$ OK

check UNSHORED!

Assume LL = 20 psf

(1) $1.4(60)(30) = 2.52$ KLF

(2) $1.2(60)(30) + 1.6(20)(30) = 3.12$ KLF ← controls

$$M_u = \frac{3.12(30)^2}{8} = 351^{1k} < 578^{1k} \quad OK$$

$$V_u = \frac{3.12(30)}{2} = 46.8^k < 275^k \quad OK$$

Typical Girder design

Unshored Girder (W24 x 62) I = 1560

Deflect. check

$$\Delta = \frac{5wL^4}{384EI} = \frac{5(3.12)(30^4)(12^3)}{384(29000)(1560)} = 1.257''$$

$$\frac{30(12)}{360} = 1 < 1.257'' \quad NG$$

TRY W24x68

Shear & Moment capacities are ok
Composite $\Delta < 1''$ ok

UNShored Moments & Shears are ok

$$\Delta = \frac{5wL^4}{384(EI)} = \frac{5(3.14)(30^4)(12^3)}{384(29000)(1830)} = 1 \quad OK$$

USE W24x68

NO changes with respect to Original Design.

Column Design

WALL A

W 12x22

Not slender

$\frac{Kl_x}{r_x} = \frac{1(14)(12)}{5.31} = 31.638$

$\lambda_c = \frac{Kl}{r\pi}\sqrt{\frac{F_y}{E}}$

$\frac{Kl_y}{r_y} = \frac{1(14)(12)}{3.04} = 55.21$

Buckling about Y-Y axis controls

$\phi_c P_n = \phi_c F_{cr} A_g$

$= 0.85(.658^{\lambda_c^2})(50)(21.1) = 717.3^K$

W 12x79

$\phi P_n = 789.8$

W12x106 $\phi P_n = 1071^K > 1022$ ok

W12x50 $\phi P_n = 362.6^K > 305$ ok

Columns

use **W12x106** column spaning from 0 to 3

use **W12x50** from 3 to roof

Colum Design (2) Wall B, C, D, E, F, G

WALL B

TRY W 12x35 $\phi P_n = 183.4^k > 180^k$ ok

TRY W 12x65 $\phi P_n = 647.4 > 561.6$ ok

use:

W12x35 for levels 3 & up

W12x65 for levels 0 to 3

WALL C

W 12x40 ($\phi P_n = 287 > 260.7$) Levels 3 & up

W 12x96 ($\phi P_n = 965 > 889.1$) Levels 0 to 3

WALL G

W12x40 ($\phi P_n = 287 > 266.4$) Levels 3 & up

W12x96 ($\phi P_n = 965 > 901.8$) Levels 0 to 3

WALL D

W 12x40 3+

W 12x96 0-3

WALL E

W 12x50 3+

W 12x106 0-3

WALL F

W 12x35 3+

W 12x65 0-3

22-141 50 SHEETS
22-142 100 SHEETS
22-144 200 SHEETS
AMPAD

Nonbraced Frame Column Design. (1)

30'x30' Bay
DL = 60 psf — Floor
LL = 100 psf
SL = 21 psf
DL = 25 psf — Roof
LL = 30 psf — Roof

LL Reduction:

– FLOOR

$$\frac{LL}{L_0} = 0.25 + \frac{15}{\sqrt{A_I}}$$

$$= 0.25 + \frac{15}{\sqrt{30^2 \times 4}} = 0.375 < \boxed{0.4} \text{ controls}$$

– ROOF

$L_r = 30 R_1 R_2$ $(12 \leq L_r \leq 20)$

$R_1 = 0.6$ $(900\ ft^2)$

$R_2 = 1$ (Flat Roof)

$L_r = 20(0.6)(1) = 18$ psf

Loads:

$DL = 60(4)(30^2) + 25(30^2) = 238.5^K$

$LL = 0.4(100)(30^2)4 + 18(30^2) = 160.2^K$

$SL = 21(30^2) = 18.9^K$

Load Combinations.

$1.4(D)$

$1.2D + 1.6L + 0.5S = \boxed{551.65^K}$ ← controls

Nonbraced Frame Column Design (2)

$P_u = 551.65$

TRY W12x58 [$r_x = 5.28$, $r_y = 2.51$, $A_g = 17.0\,in^2$]

Buckling about y-y axis

$\phi_c P_n = \phi_c F_{cr} A_g$

$$\lambda_c = \frac{KL}{r\pi}\sqrt{\frac{F_y}{E}} = \frac{1(14)(12)}{(2.51)(\pi)}\sqrt{\frac{50}{29000}} = 0.88$$

$$\phi_c P_n = 0.85(.658^{.88^2})(50)(17.0) = 522.5^K \quad NG$$

TRY W12x65

$\phi_c P_n = 647^K > 551.65^K$ OK (LRFD T 4-2)

Use W12x65

* Note: Compare to W12x72 in original design

Aproximately 10% reduction strength wise

Aproximately 10% reduction in size

Reason: Elimination of:

① Wind Moments

* Note: Moment connections are only 40^K resistive, and do not resist Beams FEM. Beams were designed as pin connected.

Foundation Design

Foundation Design - Typical Non-Braced Frame.

$P_D = 238.5^k$

$P_L = 157.5^k$

$P = P_D + P_L + P_S$ 16"

$B \geq \sqrt{\frac{414}{4}} = 10.2$ FT use 11'x11'

$P_u = 1.2(238.5) + 1.6(157.5) + .5(18) = 547$

$q = \frac{547}{11^2} = 4.52$ ksf $= 31.4$ psi

$v_c = 0.75(4)(\sqrt{3000}) = 164$ psi

$$d^2\left(164 + \frac{31.4}{4}\right) + d\left(164 + \frac{31.4}{2}\right)24 = \frac{31.4}{4}\left(132^2 - 24^2\right)$$

$171.85\, d^2 + 4312.8\, d - 132257 = 0$

$d = 17.90"$

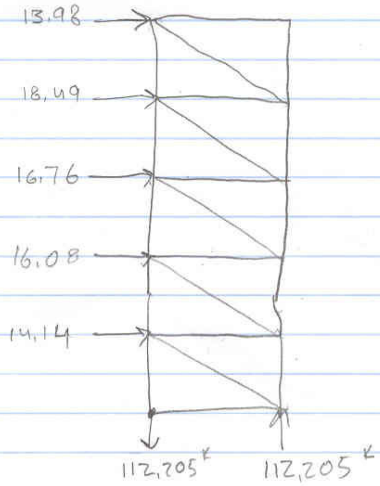
$h = 17.90 + 3 + 0.625 = 21.5"$ use 22"

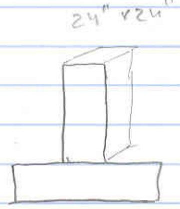
Footing: 10' x 10' x 22"

2

Foundation Design Frame A, E

①





144000

$\frac{L}{L_0} = 1\left(.25 + \frac{15}{\sqrt{4(3600)}}\right) \le .4$ use .4

$L = (100\ psf)(.4) = 40\ psf$

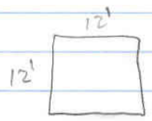
$LL = 40psf(30^2)4 + .5(30)(30^2) = 157{,}500 = 157.5^K$

$DL = 60(30^2)4 + 25(30^2) = 238.5^K$

$P_1 = P_D + P_L + P_W = 238.5 + 157.5 + 112.205 = 508.205^K$

$q_a = 4000\ psf = 4\ ksf$

$B \ge \sqrt{\frac{508.2}{4}} = 11.27'$ use 12'



(2) $1.2(238.5) + 0.5(157.5) + 1.6(112.205) = 544.478^{k}$

~~1.4(238.5) =~~

$1.2(238.5) + 1.6(157.5) + 0.5(21 \times 30^2) = 547.65$ ← controls

$q = \frac{547.65}{144} = 3.8\ ksf = 26.39\ psi$

$v_c = .75(4)\sqrt{3000} = 164\ psi$

$d^2\left(164 + \frac{26.39}{4}\right) + d\left(164 + \frac{26.39}{2}\right)(24) = \frac{26.39}{4}\left(144^2 - 24^2\right)$

$170.6\ d^2 + 4252.68\ d - 133005.6 = 0$

$d = 18.11''$

$h = 18.11 + 3 + 0.625 = \boxed{21.7''}$

USE $h = 22''$

$d = 22 - 3 - .625 = 18.375''$

$\ell = \frac{B - 24''}{2} = \frac{12' - 2'}{2} = 5'$

$M_u = \frac{3.8(5)^2}{2} = 47.5^{1K}$

$a = \frac{A_s(60\ ksi)}{.85(3ksi)(12'')} = 1.96\ A_s$

$M_u = \phi M_n = \phi A_s f_y\left(d - \frac{a}{2}\right)$

$47.5(12) = 0.9\ A_s(60ksi)\left(18.375 - \frac{1.96\ A_s}{2}\right)$

$10.56 = 18.375\ A_s - \frac{1.96 A_s^2}{2}$

$0.98\ A_s^2 - 18.375\ A_s + 10.56 = 0$

$A_s = 0.59 \approx \boxed{0.60\ in^2}$

③ use #7 @ 12" o.c , $A_s = 0.60\ in^2$

$\rho = \frac{A_s}{bh} = \frac{0.60}{(12")(22")} = 0.00227 \geq .0018$ OK

$a = 1.96(A_s) = 1.96(.6) = 1.176"$

$c = \frac{a}{.85} = 1.568"$

$\varepsilon_s = \frac{0.003}{c}(d-c) = \frac{0.003}{1.568}(18.375 - 1.568) = 0.0322 \frac{in}{in} \geq 0.005 \frac{in}{in}$

$\Rightarrow \phi = 0.9$

Spacing ok by inspection

USE (12)#7 each way

$\phi B_n = 0.65(0.85)(3\ ksi)(24")^2 = 955^K$

$\phi B_n = 955\ K \geq B_u = 508.2^K$ OK

$A_{s\,min} = 0.005\ A_{col}$

$= 0.005(24")^2 = 2.88\ in^2 \rightarrow$ does not control

Final Footing:

USE 12#7 each way
SIZE 12' x 12' x 22"

5

FOUNDATION DESIGN

Typical BAY, NON BASED FRAME

Using Gravity Loads obtained earlier in Braced frame foundation design.

LL = 157.5^k (Reduced)

DL = 238.5^k

SL = $21(30^2) = 18.9^k$

$1.2(238.5^k) + 1.6(157.5) + 0.5(18.9) = 547.65$

No moments

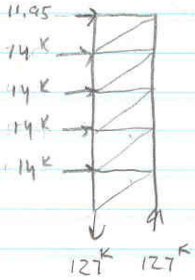
1.2D + 1.6L + 0.5S Load combination controls

So: design is the same as shown in Braced frame foundation design.

USE	(12) #7 each way
size	12' x 12' x 22"

6

Foundation Design Frame C, D



$A_T = 512.5 \; ft^2$

$A_I = 2460 \; ft^2$

DL = 60 psf

$DL_{Roof} = 25$ psf

LL = R · 100 psf

SL = 21 psf

$\frac{LL}{Lo} = .25 + \frac{15}{\sqrt{4(2460)}} = 0.401 \leq 0.4$ OK

$LL = 100(.4) = 40$ psf

$P_D = 60(512.5)4 + 25(512.5) = 135.8^K$

$P_L = 40(512.5)4 + 30(512.5) = 97.4^K$

$P_W = 127^K$

$P = P_D + P_L + P_W = 360.2^K$

$B = \sqrt{\frac{360.2}{4}} = 9.5' \Rightarrow 10'$

$P_U = 1.2(135.8) + 1.6(97.4) + 0.5(21)\left(\frac{512.5}{1000}\right) = 324.2^K$

→ $P_U = 1.2(135.8) + 0.5(97.4) + 1.6(127) = 414.9^K$

$q = \frac{414.9}{10^2} = 4.14 \text{ ksf} = 28.8 \text{ psi}$

$V_c = 164$ psi

$$d^2\left(164 + \frac{28.8}{4}\right) + d\left(164 + \frac{28.8}{2}\right)24 - \frac{28.8}{4}\left(170^2 - 24^2\right) = 0$$

$$171.2\,d^2 + 4281.6\,d - 99532.8 = 0$$

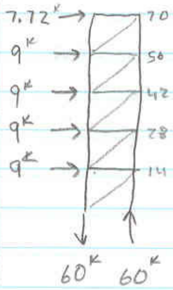
$d = 14.7$

$h = 14.7 + 3 + .625 = 18.3'' \Rightarrow 20''$

Footing: $\boxed{10' \times 10' \times 20''}$

8

Foundation Design - Frame ③



$A_T = 835 \text{ ft}^2$

$A_I = 3015 \text{ ft}^2$

$DL = 60 \text{ psf}$

$DL_{roof} = 25 \text{ psf}$

$LL = R100 \text{ psf}$

$SL = 21 \text{ psf}$

$LL_R = 30 \text{ psf}$

$$\frac{LL}{L_0} = .25 + \frac{15}{\sqrt{3015}} = .39 \Rightarrow 0.4$$

$LL = 0.4(100) = 40 \text{ psf.}$

$P_D = 835(60)(4) + 835(25) = 221.3^k$

$P_L = 835(40)(4) + 835(30) = 158.7^k$

$P_W = 60^k = 60^k$

$P_S = 21(835) = 17.5^k$

$P = 440^k$

$B = \sqrt{\frac{440}{4}} = 10.4' \Rightarrow 11'$

$P_U = 1.2(221.3) + 1.6(158.7) + 0.5(17.5) = \boxed{528.23^k}$

$P_U = 1.2(221.3) + 0.5(158.7) + 1.6(60) = 440.9^k$

$$q = \frac{528.23}{11^2} = 4.3 \text{ ksf} = 30.3 \text{ psi}$$

$V_c = 164 \text{ psi}$

Foundation Design   Frame 6

$$d^2\left(164 + \frac{30.3}{4}\right) + d\left(164 + \frac{30.3}{2}\right)24 - \frac{30.3}{4}\left(132^2 - 24^2\right) = 0$$

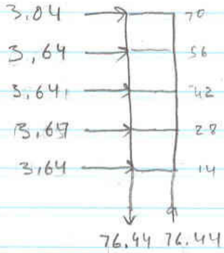
$$171.6\,d^2 + 4299.6\,d - 127623.6 = 0$$

$$d = 17.5''$$

$$h = 17.5 + 3 + .625 = 21.1 \Rightarrow 22''$$

Footing: 11' x 11' x 22"

Foundation Design   Frame B   (1)
Frame F



$A_T = (15)(30) = 450\ ft^2$
$A_I = (30)(60) = 1800\ ft^2$

DL = 60 psf
R_{DL} = 25 psf
LL = 100 psf (R)

$\frac{LL}{L_0} = \left(.25 + \frac{15}{\sqrt{4(1800)}}\right) = 0.42 > 0.4$ ok

LL = 100 (.427) = 42.7 psf

$P_D = 60(450)(4) + 25(450) = 119.25^k$
$P_L = 42.7(450)(4) + 30(450) = 90.36^k$
$P_W = 76.44$

$P = P_D + P_L + P_W = 119.25 + 90.36 + 76.4 = 286^k$

$B \geq \sqrt{\frac{286}{4}} = 8.5'$ use 9'

$P_U = 1.2(119.25) + 1.6(90.36) + 0.5\frac{(21)(450)}{1000} = 292^k$

$P_U = 1.2(119.25) + 0.5(90.36) + 1.6(76.4) = 310.5^k$

$q = \frac{310.5}{9^2} = 3.83$ ksf = 26.62 psi

$v_c = .75(4)(\sqrt{3000}) = 164$ psi

11

Foundation Design Frame B (2)

$$d^2\left(164+\frac{26.6}{4}\right)+d\left(164+\frac{26.6}{2}\right)24-\frac{26.6}{4}\left(108^2-24^2\right)=0$$

$$170.7\,d^2+4255.2\,d-73735.2=0$$

$d = 11.77"$

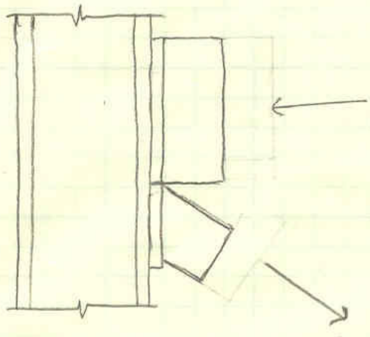
$h = 3 + 11.77 + .625 = 15.4$ use 16"

Footing: 10' x 10' x 16" ✓

Pier : 24" x 24" x 20"

Connection Design

CONNECTION DESIGN (1)



Double Plate

W18 x 35 : T = 15 1/2, t_w = 0.30"

W8 x 58 : T = 5 3/4, t_w = .510"

W12x106 ; t_w = .610"

FRAME A

FLOOR OF LEVEL 1

Connection consists of 2 separate connections: one supports the beam, another connects diagonal brace to the column.

- Beam Connection

Shear stress

FACTORED REACTION R_{uy} = 14.89 K R_{ux} = 78.8 R_u = 80.2 K

TRY 3 Rows of Bolts (TABLE 10-1 LRFD)

- ϕR_n = 94.9 K > 14.89 K OK (STD, 5/16" 3/4" Bolts)(2" Long)
- ϕR_n for uncoped 0.300 in web = 344(.300) = 103.2 > 14.89 OK
- $\phi R_{n\,sup.col}$ = 2(344)(.610) = 419.68 > 14.89 OK (W12x106)

L = 1 1/4 + 1 1/4 + 2(3) = 8 1/2" < 15 1/2" OK

USE (2) L5x3x 5/16 Angle connection with 3 rows of 3/4" Bolts

- Diagonal Connection (@ the column) (T 7-10 LRFD)

- Shear stress: on the column R = 86.6 sin(25°) = 36.62 K

TRY 2 3/4" A325 BOLTS => $f_v = \frac{36.62}{2(2)(0.442)}$ = 20.72 ksi

20.72 ksi < 45 ksi OK

Connection Design 2

Tension stress: @ column

$f_t = \frac{T_u}{N_b A_b} = \frac{86.6(\cos 25)}{4(0.442)} = 44.4$ ksi < 45 ksi OK

Limiting tension:

stress $\phi F_t = 0.75[117 - 2(20.72) \leq 90] = 56.67 > 44.4$ OK

Shear stress on the beam

$f_v = \frac{86.6}{2(3)(.442)} = 32.65$ ksi < 45 ksi OK

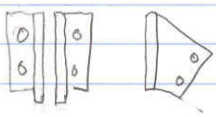
$L = 1\frac{1}{4} + 1\frac{1}{4} + 2(3) = 8.5 > T = 5.75$ NG.

TRY 2 · 7/8" Bolts

$f_v = \frac{86.6}{2(2)(.601)} = 36.02$

$L = 5.5 < 5.75$ OK

USE Double Angles: 2 3/4 A325 Bolts/angle to connect to colum
& 2 7/8 A325 Bolts to connect to beam.



Connection Design

3

Summary:

Typical Beam connection

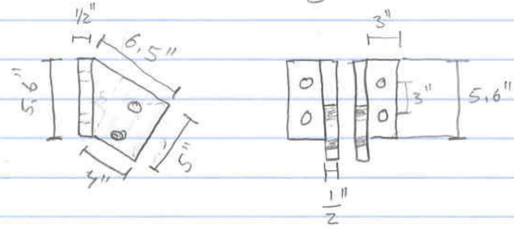
(2) L5 x 3 x $\frac{5}{16}$ x 10" 3 rows of $\frac{3}{4}$" A325 Bolts

Total weight of L = $8.19\left(\frac{10}{12}\right)(2) = \boxed{13.65 \text{ lbs}}$

Diagonal connection

Frame A, Level 0.

(2) sloped Angles: $\frac{1}{2}$' thick



Total weight of Angles = $\left[2(3.5)(5.6)\left(\frac{1}{2}\right) + 2(5)(4)\left(\frac{1}{2}\right) + 2.5(5)\left(\frac{1}{2}\right) - 2.4\right] \times .2835$

$= \boxed{12.32 \text{ lbs}}$

Note: ρ of steel = $\frac{0.2835 \text{ lbs}}{\text{in}^3}$

Computer Analysis

Computer Analysis (1)

K - values:

$K = \frac{F}{\Delta}$ (Δ were obtained from Staad 2004)

- Frame A:
  $K = \frac{1}{0.017} = 58.82 \frac{K}{in}$ (48.193, 33.72)
- Frame B:
  $K = \frac{1}{0.069} = 14.49 \frac{K}{in}$ (8.52, 11.592)
- Frame C
  $K = \frac{1}{0.018} = 55.56 \frac{K}{in}$ (0, 55.56)
- Frame D
  $K = \frac{1}{0.018} = 55.56 \frac{K}{in}$ (0, 55.56)
- Frame E
  $K = \frac{1}{0.017} = 58.82 \frac{K}{in}$ (48.193, 33.72)
- Frame F (8.52, 11.592)
  $K = \frac{1}{0.069} = 14.49 \frac{K}{in}$
- Frame G (47.62, 0)
  $K = \frac{1}{0.021} = 47.62 \frac{K}{in}$

$c^2 = a^2 + b^2$
$c^2 - b^2$

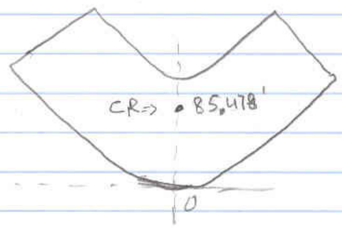
Computer Analysis (2)

The center of Rigidity

$\bar{x} = \boxed{0}$ (symmetry)

$$\bar{y} = \frac{32.5'(47.62) + 97.25'(0.588)(14.49)(2) + 109.58(.819)(58.82)2}{47.62 + (2)(.588)(14.49) + 2(.819)(58.82)}$$

$\bar{y} = \boxed{85.478\ ft}$



$$I_{xx} = 47.62(50.17)^2 + 48.17(27)^2(2) + 8.52(14.67^2)(2)$$
$$= 193,759.9\ \frac{k}{in} \cdot ft^2$$

$$I_{yy} = 55.56(15^2)(2) + 34.47(71.08^2)(2) + 11.592(61.75^2)(2)$$

$$= 461714\ \frac{k}{in} \cdot ft^2$$

$$I_p = 193759.9 + 461714 = \boxed{655474.08}\ \frac{k\ ft^2}{in}$$

Computer Analysis (3)

Direct Forces: $F_D = \dfrac{K_{iy} P_y}{\Sigma K_{iy}}$

Forces due to Torsion: $F_T = \dfrac{K_i d_i}{\Sigma (K_i d_i^2)} M = \dfrac{K_i d_i M}{I_p}$

Resulting Controling forces: in K

Frame	Level				
	Roof	4th	3rd	2nd	1st
A	12,64	28.62	43,76	58,29	71,07
B	3,04	6.90	10,54	14,05	17.13
C	11,95	27,07	41,39	55,14	67,23
D	11,95	27,07	41,39	55,14	67,23
E	12,64	28,62	43,76	58,29	71,07
F	3,04	6,90	10,54	14,05	17,13
G	7,718	17,48	26,7	35,57	43,38

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Job No	Sheet No 1	Rev
Part		
Ref		
By	Date 22-Nov-06	Chd
File Braced Frame Evaluatio	Date/Time 22-Nov-2006 16:22	

Job Title

Client

Job Information

	Engineer	Checked	Approved
Name:			
Date:	22-Nov-06		

Structure Type	SPACE FRAME

Number of Nodes	4	Highest Node	4
Number of Elements	4	Highest Beam	5

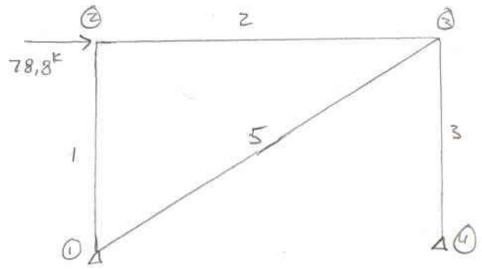
Number of Basic Load Cases	1
Number of Combination Load Cases	0

Included in this printout are data for:

All	The Whole Structure

Included in this printout are results for load cases:

Type	L/C	Name
Primary	1	WIND LOAD



Beam End Forces

Sign convention is as the action of the joint on the beam.

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip·in)	My (kip·in)	Mz (kip·in)
1	1	1:WIND LOAD	0.000	0.000	0.000	0.000	0.000	0.000
	2	1:WIND LOAD	-0.000	-0.000	0.000	0.000	0.000	0.000
2	2	1:WIND LOAD	78.800	0.000	0.000	0.000	0.000	0.000
	3	1:WIND LOAD	-78.800	-0.000	0.000	0.000	0.000	0.000
3	3	1:WIND LOAD	36.773	0.000	0.000	0.000	0.000	0.000
	4	1:WIND LOAD	-36.773	-0.000	0.000	0.000	0.000	0.000
5	1	1:WIND LOAD	-86.958	0.000	0.000	0.000	0.000	0.000
	3	1:WIND LOAD	86.958	-0.000	0.000	0.000	0.000	0.000

Reactions

		Horizontal	Vertical	Horizontal	Moment		
Node	L/C	FX (kip)	FY (kip)	FZ (kip)	MX (kip·in)	MY (kip·in)	MZ (kip·in)
1	1:WIND LOAD	-78.800	-36.773	0.000	0.000	0.000	0.000
4	1:WIND LOAD	-0.000	36.773	0.000	0.000	0.000	0.000

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Job No	Sheet No 1		Rev
Part			
Ref			
By	Date 22-Nov-06		Chd
File Braced Frame Evaluatio	Date/Time 22-Nov-2006 16:05		

Job Title Braced Frame Force Distribution

Client

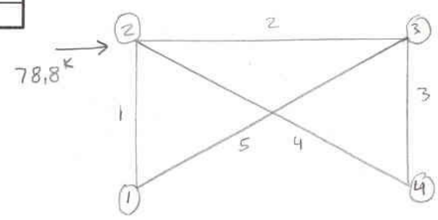
Job Information

	Engineer	Checked	Approved
Name:			
Date:	22-Nov-06		

Structure Type	SPACE FRAME

Number of Nodes	4	Highest Node	4
Number of Elements	5	Highest Beam	5

Number of Basic Load Cases	1
Number of Combination Load Cases	0



Included in this printout are data for:

All	The Whole Structure

Included in this printout are results for load cases:

Type	L/C	Name
Primary	1	Wind Load

Beam End Forces

Sign convention is as the action of the joint on the beam.

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-in)	My (kip-in)	Mz (kip-in)
1	1	1:Wind Load	-25.232	0.000	0.000	0.000	0.000	0.000
	2	1:Wind Load	25.232	-0.000	0.000	0.000	0.000	0.000
2	2	1:Wind Load	24.731	0.000	0.000	0.000	0.000	0.000
	3	1:Wind Load	-24.731	-0.000	0.000	0.000	0.000	0.000
3	3	1:Wind Load	11.541	0.000	0.000	0.000	0.000	0.000
	4	1:Wind Load	-11.541	-0.000	0.000	0.000	0.000	0.000
4	4	1:Wind Load	59.666	-0.000	0.000	0.000	0.000	0.000
	2	1:Wind Load	-59.666	0.000	0.000	0.000	0.000	0.000
5	1	1:Wind Load	-27.292	0.000	0.000	0.000	0.000	0.000
	3	1:Wind Load	27.292	-0.000	0.000	0.000	0.000	0.000

Reactions

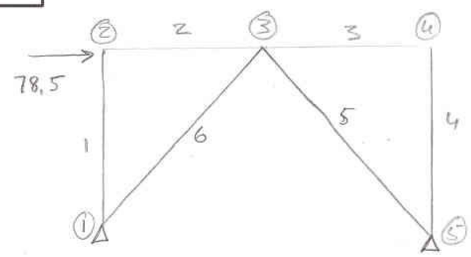
		Horizontal	Vertical	Horizontal	Moment		
Node	L/C	FX (kip)	FY (kip)	FZ (kip)	MX (kip-in)	MY (kip-in)	MZ (kip-in)
1	1:Wind Load	-24.731	-36.773	0.000	0.000	0.000	0.000
4	1:Wind Load	-54.069	36.773	0.000	0.000	0.000	0.000

	Job No	Sheet No 1	Rev
Software licensed to PENN STATE UNIVERSITY	Part		
Job Title	Ref		
	By	Date 22-Nov-06	Chd
Client	File Braced Frame Evaluatio	Date/Time 22-Nov-2006 17:00	

Job Information

	Engineer	Checked	Approved
Name:			
Date:	22-Nov-06		

Structure Type	SPACE FRAME



Number of Nodes	5	Highest Node	5
Number of Elements	6	Highest Beam	6

Number of Basic Load Cases	1
Number of Combination Load Cases	0

Included in this printout are data for:

All	The Whole Structure

Included in this printout are results for load cases:

Type	L/C	Name
Primary	1	Wind Load

Beam End Forces

Sign convention is as the action of the joint on the beam.

			Axial	Shear		Torsion	Bending	
Beam	Node	L/C	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-in)	My (kip-in)	Mz (kip-in)
1	1	1:Wind Load	0.000	0.000	0.000	0.000	0.000	-0.000
	2	1:Wind Load	0.000	-0.000	0.000	0.000	0.000	0.000
2	2	1:Wind Load	78.800	0.000	0.000	0.000	0.000	0.000
	3	1:Wind Load	-78.800	0.000	0.000	0.000	0.000	0.000
3	3	1:Wind Load	0.000	0.000	0.000	0.000	0.000	0.000
	4	1:Wind Load	-0.000	0.000	0.000	0.000	0.000	0.000
4	4	1:Wind Load	0.000	0.000	0.000	0.000	0.000	0.000
	5	1:Wind Load	0.000	-0.000	0.000	0.000	0.000	-0.000
5	1	1:Wind Load	-53.895	0.000	0.000	0.000	0.000	0.000
	3	1:Wind Load	53.895	-0.000	0.000	0.000	0.000	0.000
6	3	1:Wind Load	53.895	0.000	0.000	0.000	0.000	0.000
	5	1:Wind Load	-53.895	-0.000	0.000	0.000	0.000	0.000

Reactions

		Horizontal	Vertical	Horizontal	Moment		
Node	L/C	FX (kip)	FY (kip)	FZ (kip)	MX (kip-in)	MY (kip-in)	MZ (kip-in)
1	1:Wind Load	-118.200	-36.773	0.000	0.000	0.000	0.000
5	1:Wind Load	-39.400	36.773	0.000	0.000	0.000	0.000

Cost Comparison

Connection Cost Estimate:

Moment Connection

- (20) 3/4" x 2" Bolts
- 5' 5/16" 0.4#/LF weld
- 52" 5"x3"x 3/8" Angle

Cost Includes Material, Labor, Eq. & O&P

① Bolts = 5.50 $/Bolt

② Weld = 19.30 $/LF

③ Angle = 1.368 $/LB

Time:

① Bolts = 0.067 h/Bolt
② Weld = 0.211 h/LF

Total Cost

Bolts = 20(5.50) = $110
Weld = 19.30(5') = $96.5
Angle = 1.368(8.19)(52/12) = $48.55

Total = $255 ($97.45 for labor)

Total Time: 0.067(20) + 0.211(5) = 2.40 hours

Connection Cost Estimate:

Shear Connection:

# of Bolts = 9 + 6 = 15 Bolts/connection

Cost = $3.50/Bolt

Time = 0.067 hours/Bolt

Two Angles = 13.65 lbs
Two Plates = 12.32 lbs

- Cost of Bolts = (15)(3.50) = $52.5
- Total cost of Bolts including Mat, labor, Eq, O&P
  (15)(5.55) = $ 83.25
- Total cost of angles = 1.35 $/lb × 13.65 lb = $18.5
- Total cost of plates = 21.50$/SF · SF/20.4 lbs · 12.32 lbs

  = $13.0 / 2 plates

Total Cost = $ 83.25 + $18.5 + $ 13.0 = **$114.75** per connection
(39.9 $ for labor)

Total time = 0.067(15) = **1.005 hours**

3

Connection cost Estimate

Shear connection - Non. Braced Frame.

\# of Bolts = 9
Two angles = 13.65 lbs

Cost of Bolts = 5.55 \$/Bolt

Cost of Angles = 1.35 \$/lb

Total cost = 9(5.55) + 1.35(13.65) = **\$68.38**

Total time = 0.067(9) = **0.603 hrs**

Estimate of cost difference between original & new columns.

Labor, Equipment, and O&P, costs difference between the original and new columns are neglegable. The only variable is the material cost.

- Material cost of original = 1.04(72) = 74.88 #/LF
- Material cost of new = 1.04(65) = 67.60 #/LF

Difference = $ 7.28 /LF => ≈ 10% increase

Weight difference; ≈ 36(72)(42) − 36(65)(42) + 36(53)(28) − 36(50)(28)
≈ 13,608 lbs

Δ = 1.04(13608) = $ 14152.32

Total Difference in column cost is **$ 14152.32**

Original Moment Frame Columns: $ 168,779.52

Redesigned Columns : $ 154,627.20

5

Diagonals Cost Estimate

WT9 x 38 => 132.40 ft
WT8 x 33.5 => 796.91 ft

Material = 1.04 \$/lb
Labor = 2.02 \$/LF
Equip = 1.32 \$/LF
O&P = 11%

$$WT9 \times 38 = \left(1.04 \frac{\$}{lb} \times \frac{38\ lb}{LF} + 2.02 + 1.32\right) 132.4$$

= \$5674.66
+ O&P
= 5,674.66 + 5,674.66 (.11) = **\$6,298.88**

$$WT8 \times 33.5 = (1.04(33.5) + 2.02 + 1.32) 796.91$$

= \$31222.93
+ O&P
= 31222.93 + 31222.93(.11) = \$34657.46

Total Cost = **\$40,956**

Time: 0.057(132.4 + 796.91) = 53 hrs

= 371 hours for 1 person

6

Cost Estimate of Additional Columns

W12x65 => 84'
W12x35 => 56'

Material: 1.04 \$/lb
Labor : 2.15 \$/LF
Equip : 1.38 \$/LF
O&P : 11 %

W 12x65 => [1.04(65) + 2.15 + 1.38](84) = 5974.92

5974.92 + .11(5974.92) = 6632.16 \$

W12x35 => [1.04(35) + 2.15 + 1.38] 56 = 2236.08

2236.08 + .11(2236.08) = 2482.05

Total Cost = **\$9,114.2**

Time = 0.057(84+56) = 7.98 (7 people

≈ 55.86 hrs / 1 person

7

Footing Cost Estimate

Using the Interpolation:

Material = 5.50 $/CF

Labor = 4.25 $/CF

Total 9.75 $/CF

FRAME A: $12 \times 12 \times \frac{22}{12} = 264$ CF => $ 2,574

B: $10 \times 10 \times \frac{16}{12} = 133.3$ CF => $ 1,300

: Note Additional Footing + Pier => $ 1,365

C: $10' \times 10' \times \frac{20'}{12} = 166.7$ CF => $ 1,625

D: $ 1,625

E: $ 2,574

F: $ 1,300

G: $11' \times 11' \times \frac{22'}{12} = 221.8$ CF => $ 2,163

Typical Non Braced Frame:

$11' \times 11' \times 22'' = 183$ CF $ 2163

Typical Original: $11' \times 11' \times 28'' = 283.33$ CF $ 2,75375

8

Footing Cost Estimate + Comparison

Original @ Frame B => 10x10x24 => $1950

Original @ Frame C => 8'x8'x28 => $1456

Original @ Frame G => 10'x10'x28" => $ 2,275

Original @ Frame A => 11'x11'x28 => $ 2,753

Original System:

≈ 34	Typical	=> $ 93602
4	@ C+D	=> $ 5824
2	@ G	=> $ 4550
Total		$ 103976

New System:

	≈ 30	Typical	=> $ 64890
	4	A+E	=> $ 10296
New	*2	B+F	=> $ 2730
	4	C+D	=> $ 6500
	2	G	=> $ 4326
	Total		$ 83962

Difference: ≈ $ 15,000

Total Cost comparison,

Cost Difference due to change or addition:

Footing	=> $ 15000	(+)
Connections	=> $ 136035	(+)
Column Size	=> $ 14153	(+)
Diagonals	=> $ 40,956	(-)
New columns	=> $ 9,114	(-)
TOTAL	$ 86,812	

Connections

Moment Connections: $ 191250
Shear Connectionss: $ 55,215

New Design is $86,812 less expensive

10

Impact on schedule,

Original system:

750 connections @ 2.40 hrs/connection

750 (2.4) = 1800 hrs (1 p...

New system:

- 690 connections @ 0.63 hrs/conn

- Diagonals 371 hours (1 person)
- New columns 56 hours
- New footing 40 hours (1 person)

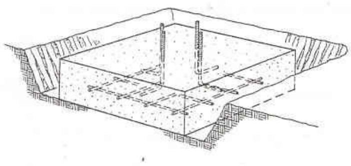
- Δ footing size ≈ 0 hours

Total Time = 690(.63) + 371 + 56 + 40

= 901.7 hrs

Difference ≈ 900 hours (1 person

A1010 Standard Foundations



The Spread Footing System includes: excavation; backfill; forms (four uses); all reinforcement; 3,000 p.s.i. concrete (chute placed); and screed finish.

Footing systems are priced per individual unit. The Expanded System Listing at the bottom shows footings that range from 3′ square x 12″ deep, to 18′ square x 52″ deep. It is assumed that excavation is done by a truck mounted hydraulic excavator with an operator and oiler.

Backfill is with a dozer, and compaction by air tamp. The excavation and backfill equipment is assumed to operate at 30 C.Y. per hour.

Please see the reference section for further design and cost information.

System Components	QUANTITY	UNIT	COST EACH MAT.	COST EACH INST.	COST EACH TOTAL
SYSTEM A1010 210 7100					
SPREAD FOOTINGS, LOAD 25K, SOIL CAPACITY 3 KSF, 3′ SQ X 12″ DEEP					
Bulk excavation	.590	C.Y.		4.51	4.51
Hand trim	9.000	S.F.		6.84	6.84
Compacted backfill	.260	C.Y.		.79	.79
Formwork, 4 uses	12.000	S.F.	7.80	50.16	57.96
Reinforcing, fy = 60,000 psi	.006	Ton	5.61	6.15	11.76
Dowel or anchor bolt templates	6.000	L.F.	5.58	20.76	26.34
Concrete, f′c = 3,000 psi	.330	C.Y.	37.62		37.62
Place concrete, direct chute	.330	C.Y.		6.27	6.27
Screed finish	9.000	S.F.		4.14	4.14
TOTAL			56.61	99.62	156.23

A1010 210	Spread Footings	COST EACH MAT.	COST EACH INST.	COST EACH TOTAL
7090	Spread footings, 3000 psi concrete, chute delivered			
7100	Load 25K, soil capacity 3 KSF, 3′-0″ sq. x 12″ deep	56.50	99.50	156
7150	Load 50K, soil capacity 3 KSF, 4′-6″ sq. x 12″ deep (RA1010 -120)	121	172	293
7200	Load 50K, soil capacity 6 KSF, 3′-0″ sq. x 12″ deep	56.50	99.50	156
7250	Load 75K, soil capacity 3 KSF, 5′-6″ sq. x 13″ deep	193	244	437
7300	Load 75K, soil capacity 6 KSF, 4′-0″ sq. x 12″ deep	98	147	245
7350	Load 100K, soil capacity 3 KSF, 6′-0″ sq. x 14″ deep	245	292	537
7410	Load 100K, soil capacity 6 KSF, 4′-6″ sq. x 15″ deep	150	203	353
7450	Load 125K, soil capacity 3 KSF, 7′-0″ sq. x 17″ deep	390	420	810
7500	Load 125K, soil capacity 6 KSF, 5′-0″ sq. x 16″ deep	193	244	437
7550	Load 150K, soil capacity 3 KSF 7′-6″ sq. x 18″ deep	470	490	960
7610	Load 150K, soil capacity 6 KSF, 5′-6″ sq. x 18″ deep	258	305	563
7650	Load 200K, soil capacity 3 KSF, 8′-6″ sq. x 20″ deep	670	650	1,320
7700	Load 200K, soil capacity 6 KSF, 6′-0″ sq. x 20″ deep	340	380	720
7750	Load 300K, soil capacity 3 KSF, 10′-6″ sq. x 25″ deep	1,250	1,050	2,300
7810	Load 300K, soil capacity 6 KSF, 7′-6″ sq. x 25″ deep	645	635	1,280
7850	Load 400K, soil capacity 3 KSF, 12′-6″ sq. x 28″ deep	1,975	1,550	3,525
7900	Load 400K, soil capacity 6 KSF, 8′-6″ sq. x 27″ deep	895	825	1,720
7950	Load 500K, soil capacity 3 KSF, 14′-0″ sq. x 31″ deep	2,725	2,050	4,775
8010	Load 500K, soil capacity 6 KSF, 9′-6″ sq. x 30″ deep	1,225	1,075	2,300

05 05 Common Work Results for Metals

05 05 23 – Metal Fastenings

05 05 23.05 Anchor Bolts		Crew	Daily Output	Labor-Hours	Unit	Material	2007 Bare Costs Labor	Equipment	Total
0690	42″ long	2 Carp	11	1.455	Ea.	28.50	53.50		82
0700	48″ long		10	1.600		32.50	58.50		91
0710	54″ long		9	1.778		38.50	65		103.50
0720	60″ long		8	2		41.50	73.50		115
0730	66″ long		7	2.286		44.50	84		128.50
0740	72″ long		6	2.667		49	98		147
0990	For galvanized, add					75%			

05 05 23.10 Bolts and Hex Nuts		Crew	Daily Output	Labor-Hours	Unit	Material	Labor	Equipment	Total
0010	**BOLTS & HEX NUTS**, Steel, A307								
0100	1/4″ diameter, 1/2″ long	1 Sswk	140	.057	Ea.	.07	2.36		2.43
0200	1″ long		140	.057		.08	2.36		2.44
0300	2″ long		130	.062		.11	2.54		2.65
0400	3″ long		130	.062		.16	2.54		2.70
0500	4″ long		120	.067		.18	2.76		2.94
0600	3/8″ diameter, 1″ long		130	.062		.12	2.54		2.66
0700	2″ long		130	.062		.15	2.54		2.69
0800	3″ long		120	.067		.20	2.76		2.96
0900	4″ long		120	.067		.25	2.76		3.01
1000	5″ long		115	.070		.31	2.88		3.19
1100	1/2″ diameter, 1-1/2″ long		120	.067		.24	2.76		3
1200	2″ long		120	.067		.27	2.76		3.03
1300	4″ long		115	.070		.41	2.88		3.29
1400	6″ long		110	.073		.56	3.01		3.57
1500	8″ long		105	.076		.73	3.15		3.88
1600	5/8″ diameter, 1-1/2″ long		120	.067		.47	2.76		3.23
1700	2″ long		120	.067		.51	2.76		3.27
1800	4″ long		115	.070		.71	2.88		3.59
1900	6″ long		110	.073		.89	3.01		3.90
2000	8″ long		105	.076		1.29	3.15		4.44
2100	10″ long		100	.080		1.60	3.31		4.91
2200	3/4″ diameter, 2″ long		120	.067		.74	2.76		3.50
2300	4″ long		110	.073		1.02	3.01		4.03
2400	6″ long		105	.076		1.30	3.15		4.45
2500	8″ long		95	.084		1.92	3.48		5.40
2600	10″ long		85	.094		2.50	3.89		6.39
2700	12″ long		80	.100		2.91	4.14		7.05
2800	1″ diameter, 3″ long		105	.076		1.90	3.15		5.05
2900	6″ long		90	.089		2.93	3.68		6.61
3000	12″ long		75	.107		5.50	4.41		9.91
3100	For galvanized, add					75%			
3200	For stainless, add					350%			

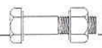
05 05 23.15 Chemical Anchors		Crew	Daily Output	Labor-Hours	Unit	Material	Labor	Equipment	Total
0010	**CHEMICAL ANCHORS**								
0020	Includes layout & drilling								
1430	Chemical anchor, w/rod & epoxy cartridge, 3/4″ diam. x 9-1/2″ long	B-89A	27	.593	Ea.	13.70	19.80	3.89	37.39
1435	1″ diameter x 11-3/4″ long		24	.667		26.50	22.50	4.37	53.37
1440	1-1/4″ diameter x 14″ long		21	.762		50.50	25.50	5	81
1445	1-3/4″ diameter x 15″ long		20	.800		95	26.50	5.25	126.75
1450	18″ long		17	.941		114	31.50	6.15	151.65
1455	2″ diameter x 18″ long		16	1		145	33.50	6.55	185.05
1460	24″ long		15	1.067		190	35.50	7	232.50

05120	Structural Steel	CREW	DAILY OUTPUT	LABOR-HOURS	UNIT	2006 BARE COSTS MAT.	LABOR	EQUIP.	TOTAL	TOTAL INCL O&P	
0010	**COLUMNS, LIGHTWEIGHT**										250
1000	Lightweight units (lally), 3-1/2" diameter	E-2	780	.072	L.F.	2.99	2.79	1.83	7.61	10.10	
1050	4" diameter	"	900	.062	"	4.40	2.42	1.59	8.41	10.75	
5800	Adjustable jack post, 8' maximum height, 2-3/4" diameter				Ea.	32			32	35	
5850	4" diameter				"	51			51	56	
0010	**COLUMNS, STRUCTURAL** R051223-10										260
0020	Shop fab'd for 100-ton, 1-2 story project, bolted conn's.										
0800	Steel, concrete filled, extra strong pipe, 3-1/2" diameter	E-2	660	.085	L.F.	31.50	3.30	2.16	36.96	42.50	
0830	4" diameter		780	.072		35	2.79	1.83	39.62	45.50	
0890	5" diameter		1020	.055		41.50	2.14	1.40	45.04	51	
0930	6" diameter		1200	.047		55	1.82	1.19	58.01	65	
0940	8" diameter		1100	.051		55	1.98	1.30	58.28	65.50	
1100	For galvanizing, add				Lb.	.22			.22	.25	
1300	For web ties, angles, etc., add per added lb.	1 Sswk	945	.008		.95	.34		1.29	1.66	
1500	Steel pipe, extra strong, no concrete, 3" to 5" diameter	E-2	16000	.004		.95	.14	.09	1.18	1.38	
1600	6" to 12" diameter		14000	.004		.95	.16	.10	1.21	1.43	
1700	Steel pipe, extra strong, no concrete, 3" diameter x 12'-0"		60	.933	Ea.	117	36.50	24	177.50	218	
1750	4" diameter x 12'-0"		58	.966		171	37.50	24.50	233	280	
1800	6" diameter x 12'-0"		54	1.037		325	40.50	26.50	392	460	
1850	8" diameter x 14'-0"		50	1.120		575	43.50	28.50	647	740	
1900	10" diameter x 16'-0"		48	1.167		830	45.50	29.50	905	1,025	
1950	12" diameter x 18'-0"		45	1.244		1,125	48.50	31.50	1,205	1,350	
3300	Structural tubing, square, A500GrB, 4" to 6" square, light section		11270	.005	Lb.	.95	.19	.13	1.27	1.52	
3600	Heavy section		32000	.002	"	.95	.07	.04	1.06	1.22	
4000	Concrete filled, add				L.F.	3.47			3.47	3.81	
4500	Structural tubing, sq, 4" x 4" x 1/4" x 12'-0"	E-2	58	.966	Ea.	157	37.50	24.50	219	264	
4550	6" x 6" x 1/4" x 12'-0"		54	1.037		257	40.50	26.50	324	380	
4600	8" x 8" x 3/8" x 14'-0"		50	1.120		555	43.50	28.50	627	715	
4650	10" x 10" x 1/2" x 16'-0"		48	1.167		1,025	45.50	29.50	1,100	1,225	
5100	Structural tubing, rect, 5" to 6" wide, light section		8000	.007	Lb.	.95	.27	.18	1.40	1.72	
5200	Heavy section		12000	.005		.95	.18	.12	1.25	1.49	
5300	7" to 10" wide, light section		15000	.004		.95	.15	.10	1.20	1.40	
5400	Heavy section		18000	.003		.95	.12	.08	1.15	1.35	
5500	Structural tubing, rect, 5" x 3" x 1/4" x 12'-0"		58	.966	Ea.	152	37.50	24.50	214	259	
5550	6" x 4" x 5/16" x 12'-0"		54	1.037		238	40.50	26.50	305	360	
5600	8" x 4" x 3/8" x 12'-0"		54	1.037		345	40.50	26.50	412	480	
5650	10" x 6" x 3/8" x 14'-0"		50	1.120		555	43.50	28.50	627	715	
5700	12" x 8" x 1/2" x 16'-0"		48	1.167		1,025	45.50	29.50	1,100	1,225	
6800	W Shape, A992 steel, 2 tier, W8 x 24		1080	.052	L.F.	25	2.02	1.32	28.34	32.50	
6850	W8 x 31		1080	.052		32.50	2.02	1.32	35.84	40.50	
6900	W8 x 48		1032	.054		50	2.11	1.38	53.49	60	
6950	W8 x 67		984	.057		70	2.21	1.45	73.66	82.50	
7000	W10 x 45		1032	.054		47	2.11	1.38	50.49	56.50	
7050	W10 x 68		984	.057		71	2.21	1.45	74.66	83.50	
7100	W10 x 112		960	.058		117	2.27	1.49	120.76	135	
7150	W12 x 50		1032	.054		52.50	2.11	1.38	55.99	62.50	
7200	W12 x 87		984	.057		91	2.21	1.45	94.66	105	
7250	W12 x 120		960	.058		125	2.27	1.49	128.76	144	
7300	W12 x 190		912	.061		199	2.39	1.56	202.95	224	
7350	W14 x 74		984	.057		77.50	2.21	1.45	81.16	90.50	
7400	W14 x 120		960	.058		125	2.27	1.49	128.76	144	
7450	W14 x 176		912	.061		184	2.39	1.56	187.95	208	
8090	For projects 75 to 99 tons, add				All	10%					
8092	50 to 74 tons, add					20%					
8094	25 to 49 tons, add					30%	10%				

05050 | Basic Metal Materials & Methods

05090 | Metal Fastenings

		Description	CREW	DAILY OUTPUT	LABOR-HOURS	UNIT	2006 BARE COSTS MAT.	LABOR	EQUIP.	TOTAL	TOTAL INCL O&P
080	0480	54" long	2 Carp	12	1.333	Ea.	13.35	47.50		60.85	88.50
	0490	60" long		10	1.600		14.65	57		71.65	105
	0500	1-1/2" diameter x 18" long		22	.727		8.65	26		34.65	50
	0510	24" long		19	.842		10.10	30		40.10	57.50
	0520	30" long		17	.941		11.40	33.50		44.90	64.50
	0530	36" long		16	1		13.10	35.50		48.60	70
	0540	42" long		15	1.067		14.90	38		52.90	75.50
	0550	48" long		13	1.231		16.75	44		60.75	86.50
	0560	54" long		11	1.455		20.50	51.50		72	103
	0570	60" long		9	1.778		22.50	63		85.50	123
	0580	1-3/4" diameter x 18" long		20	.800		13.05	28.50		41.55	59
	0590	24" long		18	.889		15.30	31.50		46.80	66
	0600	30" long		17	.941		17.80	33.50		51.30	71.50
	0610	36" long		16	1		20.50	35.50		56	78
	0620	42" long		14	1.143		23	40.50		63.50	88.50
	0630	48" long		12	1.333		25	47.50		72.50	102
	0640	54" long		10	1.600		31	57		88	123
	0650	60" long		8	2		33.50	71		104.50	148
	0660	2" diameter x 24" long		17	.941		19.50	33.50		53	73.50
	0670	30" long		15	1.067		22	38		60	83
	0680	36" long		13	1.231		24	44		68	94.50
	0690	42" long		11	1.455		27	51.50		78.50	110
	0700	48" long		10	1.600		31	57		88	123
	0710	54" long		9	1.778		37	63		100	139
	0720	60" long		8	2		39.50	71		110.50	155
	0730	66" long		7	2.286		42.50	81.50		124	174
	0740	72" long	▼	6	2.667		46.50	95		141.50	199
	0990	For galvanized, add				▼	75%				
150	0010	**BOLTS & HEX NUTS** Steel, A307									
	0100	1/4" diameter, 1/2" long	1 Sswk	140	.057	Ea.	.07	2.28		2.35	4.19
	0200	1" long		140	.057		.08	2.28		2.36	4.20
	0300	2" long		130	.062		.11	2.46		2.57	4.54
	0400	3" long		130	.062		.16	2.46		2.62	4.60
	0500	4" long		120	.067		.18	2.66		2.84	4.98
	0600	3/8" diameter, 1" long		130	.062		.12	2.46		2.58	4.55
	0700	2" long		130	.062		.15	2.46		2.61	4.59
	0800	3" long		120	.067		.20	2.66		2.86	5
	0900	4" long		120	.067		.25	2.66		2.91	5.05
	1000	5" long		115	.070		.31	2.78		3.09	5.35
	1100	1/2" diameter, 1-1/2" long		120	.067		.22	2.66		2.88	5.05
	1200	2" long		120	.067		.25	2.66		2.91	5.05
	1300	4" long		115	.070		.38	2.78		3.16	5.40
	1400	6" long		110	.073		.52	2.91		3.43	5.80
	1500	8" long		105	.076		.68	3.04		3.72	6.25
	1600	5/8" diameter, 1-1/2" long		120	.067		.45	2.66		3.11	5.30
	1700	2" long		120	.067		.49	2.66		3.15	5.35
	1800	4" long		115	.070		.68	2.78		3.46	5.75
	1900	6" long		110	.073		.85	2.91		3.76	6.20
	2000	8" long		105	.076		1.23	3.04		4.27	6.85
	2100	10" long		100	.080		1.53	3.20		4.73	7.45
	2200	3/4" diameter, 2" long		120	.067		.70	2.66		3.36	5.55
	2300	4" long		110	.073		.97	2.91		3.88	6.30
	2400	6" long		105	.076		1.23	3.04		4.27	6.85
	2500	8" long		95	.084		1.83	3.36		5.19	8.05
	2600	10" long		85	.094		2.38	3.76		6.14	9.35
	2700	12" long	▼	80	.100	▼	2.78	4		6.78	10.25



METALS 5

05090 | Metal Fastenings

			CREW	DAILY OUTPUT	LABOR-HOURS	UNIT	2006 BARE COSTS MAT.	LABOR	EQUIP.	TOTAL	TOTAL INCL O&P
880	0620	Steel, type 6010, 1/8" dia., less than 500#				Lb.	2.09			2.09	[illegible]
	0630	500# to 2,000#					1.88			1.88	[illegible]
	0640	2,000# to 5,000#					1.77			1.77	[illegible]
	0650	Steel, type 7018 Low Hydrogen, 1/8" dia., less than 500#					1.79			1.79	[illegible]
	0660	500# to 2,000#					1.61			1.61	[illegible]
	0670	2,000# to 5,000#					1.51			1.51	[illegible]
	0700	Steel, type 7024 Jet Weld, 1/8" dia., less than 500#					1.89			1.89	[illegible]
	0710	500# to 2,000#					1.70			1.70	[illegible]
	0720	2,000# to 5,000#					1.60			1.60	[illegible]
	1550	Aluminum, type 4043 TIG, 1/8" dia., less than 10#					6.55			6.55	[illegible]
	1560	10# to 60#					5.90			5.90	[illegible]
	1570	Over 60#					5.55			5.55	[illegible]
	1600	Aluminum, type 5356 TIG, 1/8" dia., less than 10#					7.25			7.25	8
	1610	10# to 60#					6.55			6.55	[illegible]
	1620	Over 60#					6.15			6.15	[illegible]
	1900	Cast iron, type 8 Nickel, 1/8" dia., less than 500#					22.50			22.50	[illegible]
	1910	500# to 1,000#					20			20	22
	1920	Over 1,000#					19			19	21
	2000	Stainless steel, type 316/316L, 1/8" dia., less than 500#					10.30			10.30	[illegible]
	2100	500# to 1000#					9.30			9.30	[illegible]
	2220	Over 1000#				▼	8.75			8.75	[illegible]
900	0010	**WELDING STRUCTURAL**									
	0020	Field welding, 1/8" E6011, cost per welder, no oper. engr (R050521-20)	E-14	8	1	Hr.	3.80	42	11.15	56.95	92
	0200	With 1/2 operating engineer	E-13	8	1.500		3.80	59.50	11.15	74.45	118
	0300	With 1 operating engineer	E-12	8	2	▼	3.80	77	11.15	91.95	145
	0500	With no operating engineer, 2# weld rod per ton	E-14	8	1	Ton	3.80	42	11.15	56.95	92
	0600	8# E6011 per ton	"	2	4		15.20	168	44.50	227.70	365
	0800	With one operating engineer per welder, 2# E6011 per ton	E-12	8	2		3.80	77	11.15	91.95	145
	0900	8# E6011 per ton	"	2	8	▼	15.20	310	44.50	369.70	580
	1200	Continuous fillet, stick welding, incl. equipment									
	1300	Single pass, 1/8" thick, 0.1#/L.F.	E-14	150	.053	L.F.	.19	2.24	.59	3.02	4.88
	1400	3/16" thick, 0.2#/L.F.		75	.107		.38	4.47	1.19	6.04	9.80
	1500	1/4" thick, 0.3#/L.F.		50	.160		.57	6.70	1.78	9.05	14.70
	1610	5/16" thick, 0.4#/L.F.		38	.211		.76	8.85	2.35	11.96	19.30
	1800	3 passes, 3/8" thick, 0.5#/L.F.		30	.267		.95	11.20	2.97	15.12	24.50
	2010	4 passes, 1/2" thick, 0.7#/L.F.		22	.364		1.33	15.25	4.05	20.63	33.50
	2200	5 to 6 passes, 3/4" thick, 1.3#/L.F.		12	.667		2.47	28	7.45	37.92	61.50
	2400	8 to 11 passes, 1" thick, 2.4#/L.F.	▼	6	1.333		4.56	56	14.85	75.41	122
	2600	For all position welding, add, minimum						20%			
	2700	Maximum						300%			
	2900	For semi-automatic welding, deduct, minimum						5%			
	3000	Maximum				▼		15%			
	4000	Cleaning and welding plates, bars, or rods									
	4010	to existing beams, columns, or trusses	E-14	12	.667	L.F.	.95	28	7.45	36.40	59.50
920	0010	**STEEL CUTTING**									
	0020	Hand burning, incl. preparation, torch cutting & grinding, no staging									
	0100	Steel to 1/2" thick	E-25	320	.025	L.F.		1.05	.25	1.30	2.17
	0150	3/4" thick		260	.031			1.29	.31	1.60	2.66
	0200	1" thick	▼	200	.040	▼		1.68	.41	2.09	3.47

05120 | Structural Steel

				DAILY	LABOR-		2006 BARE COSTS			
		05120 Structural Steel	CREW	OUTPUT	HOURS	UNIT	MAT.	LABOR	EQUIP.	TOTAL
260	8096	10 to 24 tons, add (R051223-10)				All	50%	25%		
	8098	2 to 9 tons, add					75%	50%		
	8099	Less than 2 tons, add					100%	100%		
300	0010	**CURB EDGING**								
	0020	Steel angle w/anchors, on forms, 1" x 1", 0.8#/L.F.	E-4	350	.091	L.F.	1.61	3.70	.26	5.57
	0100	2" x 2" angles, 3.92#/L.F.		330	.097		5.15	3.92	.27	9.34
	0200	3" x 3" angles, 6.1#/L.F.		300	.107		8.15	4.31	.30	12.76
	0300	4" x 4" angles, 8.2#/L.F.		275	.116		10.55	4.71	.32	15.58
	1000	6" x 4" angles, 12.3#/L.F.		250	.128		15.25	5.20	.36	20.81
	1050	Steel channels with anchors, on forms, 3" channel, 5#/L.F.		290	.110		6.40	4.46	.31	11.17
	1100	4" channel, 5.4#/L.F.		270	.119		6.85	4.79	.33	11.97
	1200	6" channel, 8.2#/L.F.		255	.125		10.55	5.10	.35	16
	1300	8" channel, 11.5#/L.F.		225	.142		14.35	5.75	.40	20.50
	1400	10" channel, 15.3#/L.F.		180	.178		18.65	7.20	.50	26.35
	1500	12" channel, 20.7#/L.F.		140	.229		25	9.25	.64	34.89
	2000	For curved edging, add					35%	10%		
440	0010	**LIGHTWEIGHT FRAMING** (R051223-35)								
	0400	Angle framing, field fabricated, 4" and larger	E-3	440	.055	Lb.	.55	2.22	.20	2.97
	0450	Less than 4" angles (R051223-45)		265	.091		.57	3.68	.34	4.59
	0600	Channel framing, field fabricated, 8" and larger		500	.048		.57	1.95	.18	2.70
	0650	Less than 8" channels		335	.072		.57	2.91	.27	3.75
	1000	Continuous slotted channel framing system, shop fab, min.	2 Sswk	2400	.007		2.95	.27		3.22
	1200	Maximum	"	1600	.010		3.33	.40		3.73
	1250	Plate & bar stock for reinforcing beams and trusses					1.05			1.05
	1300	Cross bracing, rods, shop fabricated, 3/4" diameter	E-3	700	.034		1.14	1.39	.13	2.66
	1310	7/8" diameter		850	.028		1.14	1.15	.11	2.40
	1320	1" diameter		1000	.024		1.14	.97	.09	2.20
	1330	Angle, 5" x 5" x 3/8"		2800	.009		1.14	.35	.03	1.52
	1350	Hanging lintels, shop fabricated, average		850	.028		1.14	1.15	.11	2.40
	1380	Roof frames, shop fabricated, 3'-0" square, 5' span	E-2	4200	.013		1.14	.52	.34	2
	1400	Tie rod, not upset, 1-1/2" to 4" diameter, with turnbuckle	2 Sswk	800	.020		1.24	.80		2.04
	1420	No turnbuckle		700	.023		1.19	.91		2.10
	1500	Upset, 1-3/4" to 4" diameter, with turnbuckle		800	.020		1.24	.80		2.04
	1520	No turnbuckle		700	.023		1.19	.91		2.10
480	0010	**LINTELS**								
	0020	Plain steel angles, under 500 lb.	1 Bric	550	.015	Lb.	.73	.53		1.26
	0100	500 to 1000 lb.		640	.013		.71	.46		1.17
	0200	1,000 to 2,000 lb.		640	.013		.69	.46		1.15
	0300	2,000 to 4,000 lb.		640	.013		.67	.46		1.13
	0500	For built-up angles and plates, add to above					.24			.24
	0700	For engineering, add to above					.10			.10
	0900	For galvanizing, add to above, under 500 lb.					.30			.30
	0950	500 to 2,000 lb.					.27			.27
	1000	Over 2,000 lb.					.22			.22
520	0010	**PIPE SUPPORT FRAMING**								
	0020	Under 10#/L.F.	E-4	3900	.008	Lb.	1.27	.33	.02	1.62
	0200	10.1 to 15#/L.F.		4300	.007		1.25	.30	.02	1.57
	0400	15.1 to 20#/L.F.		4800	.007		1.24	.27	.02	1.53
	0600	Over 20#/L.F.		5400	.006		1.22	.24	.02	1.48
560	0010	**PLATES**								
	0020	For connections & stiffener plates, shop fabricated (R051223-80)								
	0050	1/8" thick (5.1 Lb./S.F.)				S.F.	4.85			4.85
	0100	1/4" thick (10.2 Lb./S.F.)					9.70			9.70

05100 | Structural Metal Framing

05120 | Structural Steel			CREW	DAILY OUTPUT	LABOR-HOURS	UNIT	2006 BARE COSTS MAT.	LABOR	EQUIP.	TOTAL	TOTAL INCL O&P	
560	0300	3/8" thick (15.3 Lb./S.F.) R051223-80				S.F.	14.55			14.55	16	560
	0400	1/2" thick (20.4 Lb./S.F.)					19.40			19.40	21.50	
	0450	3/4" thick (30.6 Lb./S.F.)					29			29	32	
	0500	1" thick (40.8 Lb.S.F.)					39			39	42.50	
600	0010	STRESSED SKIN ROOF & CEILING SYSTEM										600
	0020	Double panel flat roof, spans to 100'	E-2	1150	.049	S.F.	7.60	1.89	1.24	10.73	13	
	0100	Double panel convex roof, spans to 200'		960	.058		12.35	2.27	1.49	16.11	19.15	
	0200	Double panel arched roof, spans to 300'		760	.074		19	2.87	1.88	23.75	28	
640	0010	STRUCTURAL STEEL MEMBERS R051223-10										640
	0020	Shop fab'd for 100-ton, 1-2 story project, bolted conn's.										
	0100	W 6 x 9	E-2	600	.093	L.F.	9.40	3.63	2.38	15.41	19.20	
	0120	x 16		600	.093		16.70	3.63	2.38	22.71	27.50	
	0140	x 20		600	.093		21	3.63	2.38	27.01	32	
	0300	W 8 x 10		600	.093		10.45	3.63	2.38	16.46	20.50	
	0320	x 15		600	.093		15.70	3.63	2.38	21.71	26	
	0350	x 21		600	.093		22	3.63	2.38	28.01	33	
	0360	x 24		550	.102		25	3.96	2.59	31.55	37	
	0370	x 28		550	.102		29.50	3.96	2.59	36.05	41.50	
	0500	x 31		550	.102		32.50	3.96	2.59	39.05	45	
	0520	x 35		550	.102		36.50	3.96	2.59	43.05	49.50	
	0540	x 48		550	.102		50	3.96	2.59	56.55	64.50	
	0600	W 10 x 12		600	.093		12.55	3.63	2.38	18.56	22.50	
	0620	x 15		600	.093		15.70	3.63	2.38	21.71	26	
	0700	x 22		600	.093		23	3.63	2.38	29.01	34.50	
	0720	x 26		600	.093		27	3.63	2.38	33.01	39	
	0740	x 33		550	.102		34.50	3.96	2.59	41.05	47.50	
	0900	x 49		550	.102		51	3.96	2.59	57.55	66	
	1100	W 12 x 14		880	.064		14.65	2.48	1.62	18.75	22	
	1300	x 22		880	.064		23	2.48	1.62	27.10	31.50	
	1500	x 26		880	.064		27	2.48	1.62	31.10	36	
	1520	x 35		810	.069		36.50	2.69	1.76	40.95	46.50	
	1560	x 50		750	.075		52.50	2.90	1.90	57.30	64.50	
	1580	x 58		750	.075		60.50	2.90	1.90	65.30	73.50	
	1700	x 72		640	.088		75	3.40	2.23	80.63	91.50	
	1740	x 87		640	.088		91	3.40	2.23	96.63	108	
	1900	W 14 x 26		990	.057		27	2.20	1.44	30.64	35.50	
	2100	x 30		900	.062		31.50	2.42	1.59	35.51	40.50	
	2300	x 34		810	.069		35.50	2.69	1.76	39.95	45.50	
	2320	x 43		810	.069		45	2.69	1.76	49.45	56	
	2340	x 53		800	.070		55.50	2.72	1.78	60	67.50	
	2360	x 74		760	.074		77.50	2.87	1.88	82.25	92	
	2380	x 90		740	.076		94	2.94	1.93	98.87	110	
	2500	x 120		720	.078		125	3.03	1.98	130.01	145	
	2700	W 16 x 26		1000	.056		27	2.18	1.43	30.61	35.50	
	2900	x 31		900	.062		32.50	2.42	1.59	36.51	41.50	
	3100	x 40		800	.070		42	2.72	1.78	46.50	52.50	
	3120	x 50		800	.070		52.50	2.72	1.78	57	64	
	3140	x 67		760	.074		70	2.87	1.88	74.75	84	
	3300	W 18 x 35	E-5	960	.083		36.50	3.28	1.58	41.36	47.50	
	3500	x 40		960	.083		42	3.28	1.58	46.86	53.50	
	3520	x 46		960	.083		48	3.28	1.58	52.86	60.50	
	3700	x 50		912	.088		52.50	3.46	1.66	57.62	65.50	
	3900	x 55		912	.088		57.50	3.46	1.66	62.62	71	
	3920	x 65		900	.089		68	3.50	1.68	73.18	82.50	
	3940	x 76		900	.089		79.50	3.50	1.68	84.68	95.50	
	3960	x 86		900	.089		90	3.50	1.68	95.18	107