FINAL DESIGN - CONT. - FRAME 4 - STRONG COLUMN/WEAK BEAM

$$\Sigma Z_c(F_{yc}-f_a)/\Sigma Z_b f_{yb} > 1 \quad \text{(SEAOC 4F5)}$$

Interior column, second floor (Joint "b", p. 13)

W14x68: $A = 20.0$, $Z_c = 115$

$P_{D+L+E} = 166$

$f_a = 166/20.0 = 8.30$

The girder is W18x60, with $Z_b = 123$

$2(115)(36-8.30)/2(123)(36) = 0.72$ which is < 1

Change to Grade 50 steel (make both columns Gr. 50).

$2(115)(50-8.3)/2(123)(36) = 1.08$ which is > 1

Exterior column, second floor (Joint "a")

W14x48: $A = 14.1$, $Z_c = 78.4$

$P_{D+L+E} = 105$ (p.14)

$f_a = 105/14.1 = 7.45$ ksi

The girder is W18x60, with $Z_b = 123$

With one column above the joint and one below,

$2(78.4)(50-7.45)/123(36) = 1.50$ which is > 1

For both columns $f_a < 0.4(36) = 14.41$; therefore, the columns are not required to be stronger than the beams; however, strong column/weak beam design is strongly recommended.

Figure D-3. Continued.

LONGITUDINAL DIRECTION: STEEL BRACED FRAMES

LATERAL FORCES

BUILDING: A-3

DIRECTION: LONGITUDINAL

$Z = 0.40 \quad I = 1.0 \quad R_w = 8$

$C = \frac{1.25(1.2)}{(0.28)^{2/3}} = \cancel{3.50} \rightarrow$ USE 2.75

$W = 1789$ KIPS

$T = 0.02(34)^{3/4}$

$= 0.28$ SEC.

$F_t = 0.07TV = 0\ V^*$

$F_x = (V - F_t)\frac{Wh}{\Sigma Wh} = 1.0\ V\frac{Wh}{\Sigma Wh}$

$V = \frac{ZIC}{R_w}W = \frac{(0.40)(1)(2.75)}{8}W = 0.137W$; SAY $0.14W = 250K$

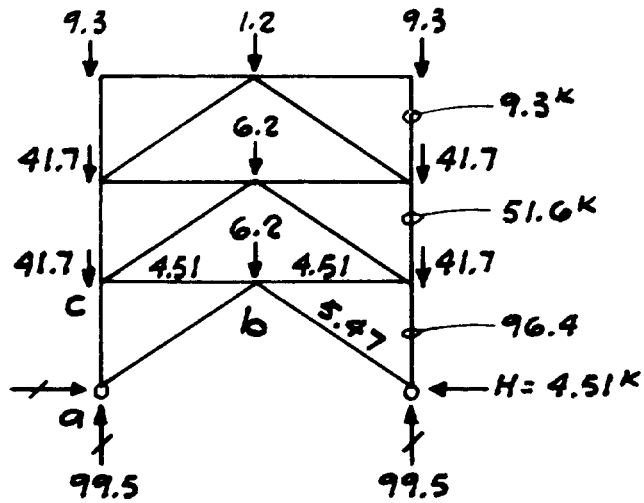
$* F_t = 0$ WHEN $T \leq 0.7$ SEC.

LEVEL (1)	h FT. (2)	Δh FT. (3)	W KIPS (4)	ΣW (5)	(2)×(4) Wh (6)	Wh/ΣWh (7)	F KIPS (8)	Σ(7) V KIPS (9)	(3)×(9) ΔOTM K-FT. (10)	Σ(10) OTM K-FT. (11)	(8)÷(4) F/W (12)	(9)÷(5) V/ΣW (13)
						$F_t = 0$						
R	34		375	375	12,750	0.35	88				.235	.235
		12						88	1056			
3	22		707	1082	15,554	0.43	107			1056	.151	.180
		11						195	2145			
2	11		707	1789	7,777	0.22	55			3201	.078	.140
		11						250	2750			
										5951		
	Σ		1789		36,081	1.00	250					

Figure D-3. Continued.

BRACED FRAME - CONT.

VERTICAL FORCES ALL EXCEPT ROOF LL



TRIB. AREA AT COLUMN

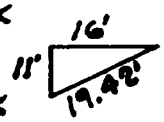
TRANSV. GIRDER : 9' x 32' = 288
EDGE BEAM $\left(\frac{32}{2}+\frac{16}{2}\right)$ x 3' = 72 } 360 SF

TRIB. AREA AT BRACE 16' x 3' = 48 SF

LOADS AT COLUMNS (REDUCED LL) AT BRACES

ROOF (25.7 PSF + 0) x 360 SF = 9.3K x 48 SF = 1.2K
FLOOR (74.5 + 36) x 360 SF = 39.8 } 41.7K x 48 SF = 5.3K } 6.2K
WALL (5.3 PSF x 11') x 32' = 1.9 x 16' = 0.9

BRACE FORCE = $V = \frac{6.2}{2} = 3.1$ $H = \frac{16'}{11'}(3.1) = 4.51^K$



AXIAL FORCE = $\frac{19.4}{11}(3.1) = 5.47^K$

FULL LOADS =

ROOF $P_D = 9.3^K$ $P_L = 7.2^K$

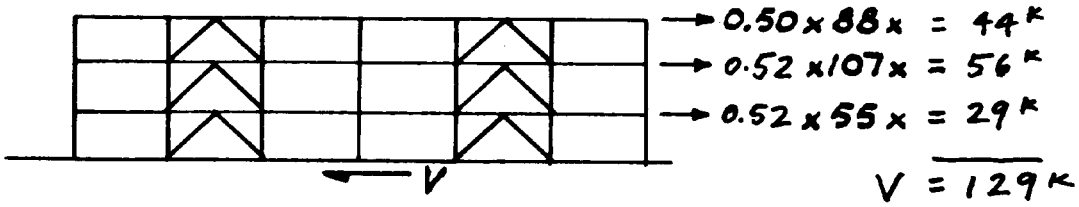
FLOOR $P_D = 28.7^K$ $P_L = 18.0^K$

Figure D-3. Continued.

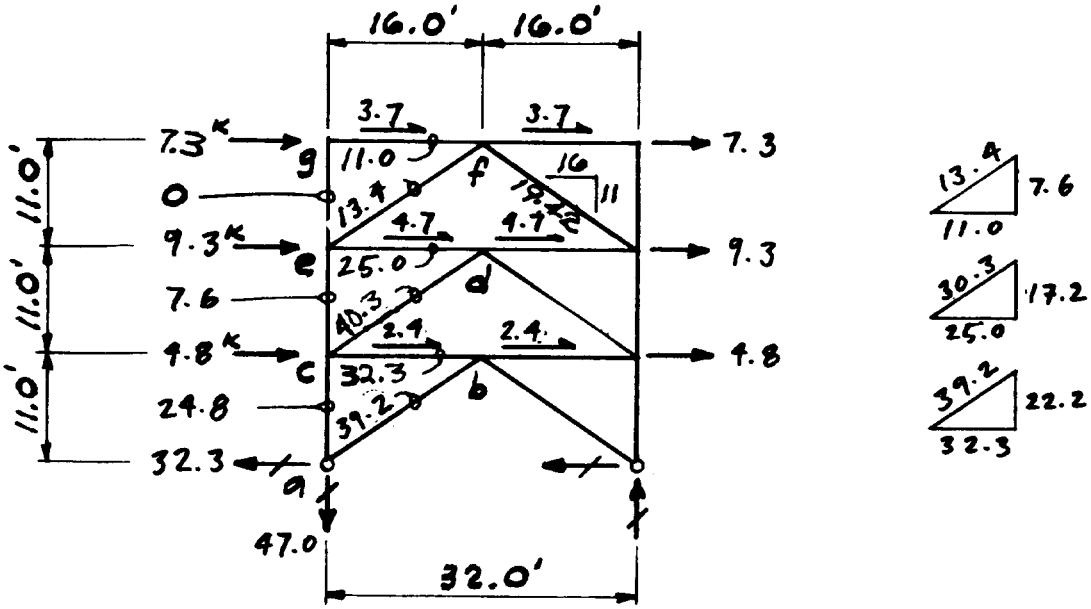
BRACED FRAME - CONT.

SEISMIC FORCES

FRAMES SHARE ROOF LOAD ACCORDING TO TRIBUTARY AREA. AT FLOORS THE FRAMES TAKE 0.52 x STORY FORCE (P.12).



APPLY 1/2 FORCE TO EACH BRACED BAY



TYPICAL BRACED BAY

SEISMIC FORCES

DESIGN CHEVRON BRACE MEMBERS FOR 1.5x SEISMIC FORCES.

Figure D-3. Continued.

BRACED FRAMES CONT

MEMBER SIZES

BRACE (FIRST STORY)

CHEVRON BRACE
SEAOC 49

$P_E = \pm 39.2$ DESIGN $F = 1.5 \times 39.2 = 58.8$

$P_D = -5.5$ (p.29)

BRACE FORCE, $P_B = -5.5 \pm 58.8 \begin{cases} +53.3 \\ -64.3 \end{cases}$

$\frac{L}{r} \leq \frac{720}{\sqrt{F_y}} = \frac{720}{\sqrt{46}} = 106$ (SEAOC 49 1a)

TS 6 x 6 x 3/8 $r = 2.27$

$\frac{L}{r} = \frac{228}{2.27} = 100 < 106$ OK

$P_a = 115^K$ AISC ASD p. 3-42

$\beta = 1 \Big/ \left[1 + \frac{Kl/r}{2C_c}\right] = 1 \Big/ \left[1 + \frac{100}{2(112)}\right] = 0.69$ (SEAOC 49 1b)

$(P_a)_{Brace} = 0.69 \times 115 = 79.5 > 64.3$

CHECK $\frac{b}{t}$ OF TUBE WALL

ASD CHAPT. N: $\frac{190}{\sqrt{F_y}} = 28$

$b = 6 - 2\left(\frac{3}{8}\right) = 5\,1/4$

$b/t = \frac{5\,1/4}{3/8} = 14 < 28$

Figure D-3. Continued.

BRACED FRAMES CONT.

COLUMN (SEAOC 4D1)

TRY W14 x 43 FOR COMPATIBILITY WITH OTHER COLUMNS

$$P_c = 1.0\,P_{DL} + 0.7\,P_{LL} + 3\left(\frac{R_w}{8}\right)P_E$$

$$= 1.0\,(66.7) + 0.7\,(43.2) + 3\left(\frac{8}{8}\right)(39.2) = 215^k$$

$L = 10;\ F_y = 36$

ASD p. 3-24

$$P_{allow} = 1.7\,(215) = 366^k > 215 \text{ OK}$$

NOTE:

THIS IS A CHECK FOR COLUMN STABILITY. NO BENDING MOMENTS ARE INCLUDED IN THIS CHECK. FOR SITUATIONS WITH LARGE COLUMN BENDING, A STANDARD COLUMN CHECK INCLUDING BENDING AND CODE LEVEL AXIAL FORCE MAY GOVERN OVER THIS.

COLUMN SPLICES

DESIGN FOR THE FOLLOWING TENSION:

$$P_t = 0.85\,P_{DL+LL} - 3\left(\frac{R_w}{8}\right)P_E$$

$$= 0.85\,(66.7) - 3\left(\frac{8}{8}\right)(39.2) = -60.9^K$$

Figure D-3. Continued.

BRACED FRAME - CONT.

MEMBER DESIGN - CONT.

EDGE BEAMS - FULL LATERAL BRACING BY STEEL DECK. ASSUME WALL LATERAL LOADS TRANSMITTED DIRECTLY TO THE STEEL DECK: NO TORSION OR HORIZONTAL LOAD.

BEAM WITHOUT DIAGONAL BRACE $L = 32'$

BEAM MUST STILL CARRY VERTICAL LOAD EVEN IF THE BRACES SHOULD FAIL IN A LARGE EARTHQUAKE. SEAOC 4G 3a (3)

FLOOR

$$W = \left.\begin{array}{l}(0.745 + .050\ \text{KSF}) \times 3' \times 32' = 12.0 \\ \text{WINDOW}\ \ 0.0053\ \text{KSF} \times 11' \times 32' = 1.9\end{array}\right\} 13.9^{K}$$

W18x40

BEAM WITH BRACE $L = 16'$

$$\left.\begin{array}{l}\text{ROOF USE W14x30} \\ \text{FLOOR USE W18x40}\end{array}\right\} \text{FOR CONSISTENT DETAILS WITH UNBRACED BAYS}$$

$$\text{FLOOR} = 3' \times (74.5 + 50\ \text{PSF}) = 373.5\ \#/'$$

$$\text{WALL} = 5.3\ \text{PSF} \times 11' = 58.3\ \#/'$$

$$w = .432\ K/'$$

$$\text{VERT. } W = .432^{K/'} \times 16' = 6.90^{K}$$

$$M = \frac{6.90 \times 16'}{8} = 13.8^{K'}$$

$$f = \frac{13.8 \times 12}{68.4} = 2.42\ \text{KSI LOW}$$

$$\text{AXIAL } P = +4.51 \pm 32.3 = \begin{cases} +36.8 \\ -27.7 \end{cases}$$

$$f_a = \frac{36.8}{11.8} = 3.12\ \text{KSI LOW}$$

Figure D-3. Continued.

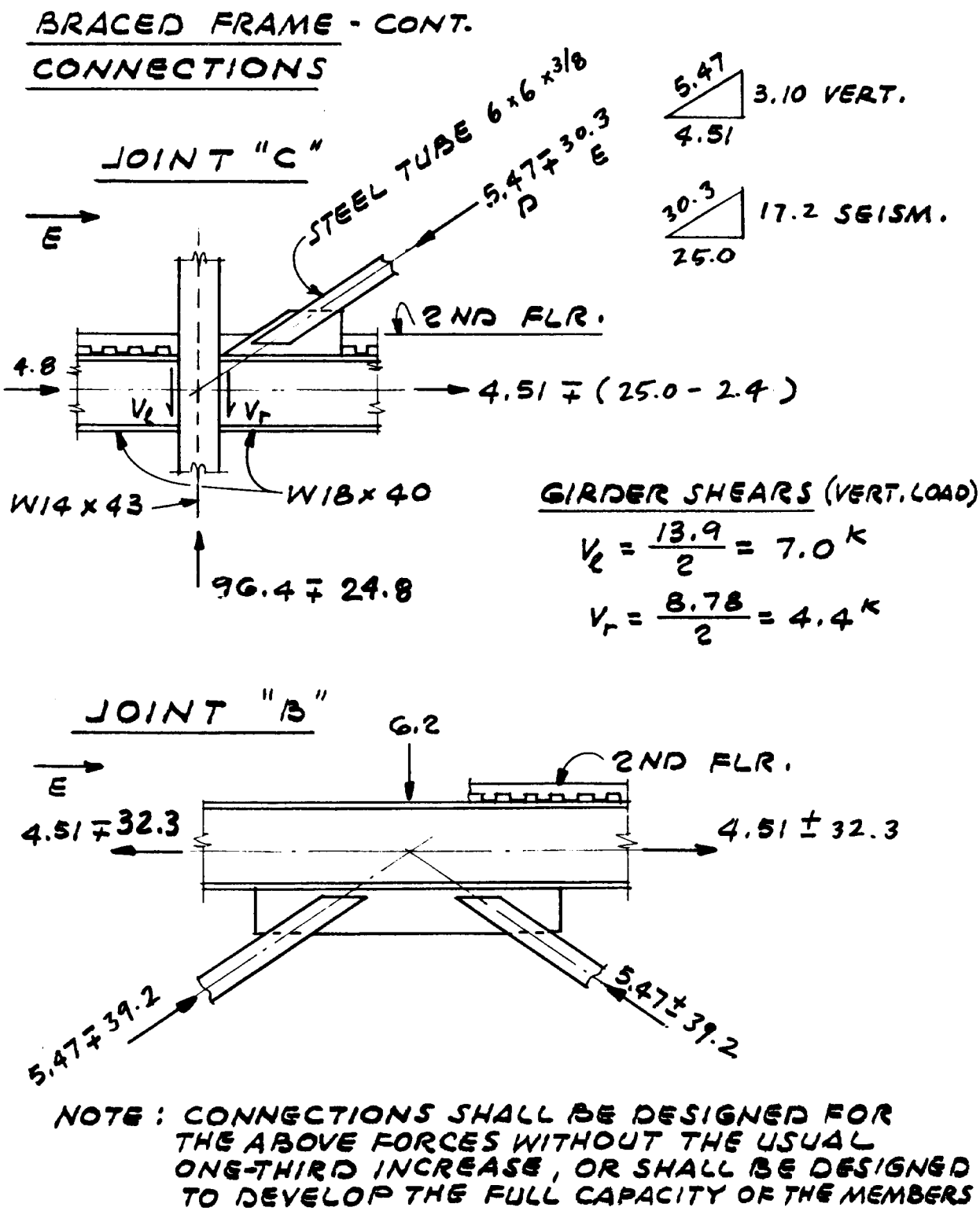


Figure D-3. Continued.

BRACED FRAME - CONT.

CONNECTIONS - CONT.

connections shall be designed for the least of

(1) TENSILE STRENGTH OF THE BRACING

(2) $3\left(\frac{R_W}{8}\right)$ TIMES THE FORCE IN THE BRACE DUE TO PRESCRIBED SEISMIC FORCES.

(3) THE MAX. FORCE WHICH CAN BE TRANSFERRED TO THE BRACE BY THE SYSTEM.

BOTTOM LEVEL

TENSILE STRENGTH OF BRACE

$P_{ST} = F_y A = 46\,(8.08) = 372^k$

3/8 R_W X SEISMIC $\longrightarrow \frac{3}{8}(8)(39.2) = \underline{117.6^k} \longleftarrow$ GOVERNS

$117.6 + 5.47 = \underline{\underline{123.1^k}}$

(p.29)

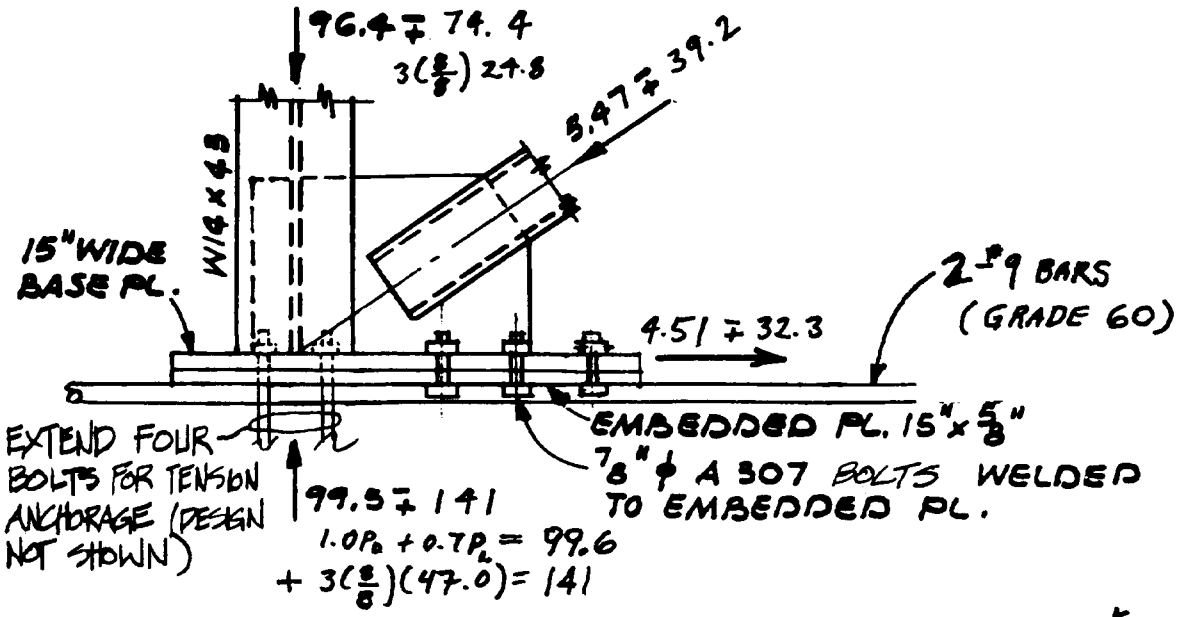
ANCHORAGE TO FOUNDATION

THIS IS A BRACED FRAME CONNECTION. AS SUCH, IT MUST MEET SEAOC 4G2, OR, IF CONSIDERED AS A COLUMN, 4D2. THE RESULT IS THE SAME. SEE NEXT PAGE.

Figure D-3. Continued.

BRACED FRAME - CONT.

JOINT "a"



VERT. BEARING ON CONC. $P = 99.6 + 141 = 241^k$

FOR FOUNDATION MUCH WIDER THAN THE LOADED AREA, ULTIMATE BEARING STRESS:

$\sqrt{A_2/A_1}\ \phi(0.85 f_c')A_1 = 2(0.70)(0.85)(3)(A_1) = 3.57A_1$

REQ'D $A = \dfrac{241}{3.57} = 67.5\ IN^2$ $\quad [(\sqrt{A_2/A_1})_{max} = 2]$

MIN. LENGTH $= \dfrac{67.5}{15} = 4.5''$

SHEAR BOLTS $V = 4.61 + 3(\frac{5}{8})32.3 = 101^k$

USE $10 - \frac{7}{8}''\phi$ A 307 STUDS IN S.S.

STRENGTH: $10 \times 1.7 \times 6.01 = 102^k$

REINFORCING BARS

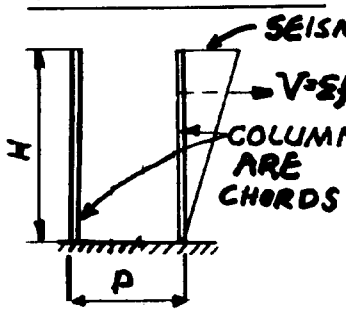
REQ'D $A = 101^k / 0.9(60) = 1.87\ IN^2$ 2-#9

LOCATE BASE PLATES IN A POCKET SO THAT THE #9'S ARE CONFINED WITHIN THE GRADE BEAM BAR CAGE. EXTEND BARS 4'-0 EA. SIDE OF POCKET. WELD TO DEVELOP THE BARS. COMPLY WITH ACI 12.14.3..

Figure D-3. Continued.

BRACED FRAME - CONT.

DEFLECTION

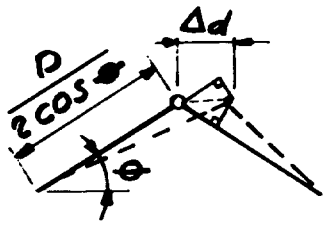


CHORDS: $\Delta_c = \dfrac{11\,VH^3}{60\,EI}$ (BEAM WITH INVERSE TRIANGULAR LOADING)

$$I = 2 \cdot A \cdot \left(\frac{D}{2}\right)^2 = \frac{AD^2}{2}$$

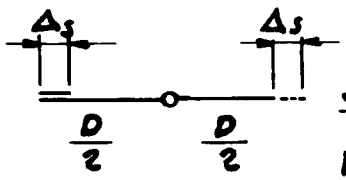
A = AREA OF ONE COLUMN

$$\Delta_c = \frac{11\,VH^3}{60E\,\frac{AD^2}{2}} = \frac{11\,VH^3}{30\,EAD^2}$$



DIAGONALS: $\Delta_d = \dfrac{P\,\frac{D}{2\cos\theta}}{EA\cos\theta} = \dfrac{f_d\,D}{2E\cos^2\theta}$

WHERE f_d IS THE STRESS IN THE DIAG.



STRUTS: $\Delta_s = \dfrac{P \times \frac{D}{2}}{EA} = \dfrac{f_s\,D}{2E}$

WHERE f_s IS THE STRESS IN THE STRUT

$$\Delta = \Delta_c + \Delta_d + \Delta_s$$
$$= \frac{11\,VH^3}{30\,EAD^2} + \Sigma\,\frac{f_d\,D}{2E\cos^2\theta} + \Sigma\,\frac{f_s\,D}{2E}$$

IN COMPUTATION OF DEFLECTIONS USE STRESSES BASED ON SEISMIC FORCES, PAGE 28.

V FROM SHEET 30 ($2 \times 32.3 = 64.6^k$) $\quad D = 32'$

MEMBER FORCES P FROM SHEET 30 $\quad E = 29{,}000$ KSI

A = AREA OF MEMBER $\quad \cos\theta = \dfrac{16}{[11^2+16^2]^{1/2}} = .824$

Figure D-3. Continued.

BRACED FRAME - CONT.
DEFLECTION - CONT.

CHORDS W14 x 43 $A = 12.6\ IN^2$ Δ_c

$$\Delta_c\ (ROOF) = \frac{11}{30}\frac{VH}{EA}\left(\frac{H}{D}\right)^2 = \frac{11}{30}\frac{64.6 \times (33 \times 12)}{29000 \times 12.6}\left(\frac{33}{32}\right)^2 = 0.027''$$

$$\Delta_c\ (3^{RD}) = .55\ \Delta_{ROOF}\ (AT\ 2/3\ H\ UNIFORM\ BEAM) \cong 0.015''$$

$$\Delta_c\ (2^{ND}) = .17\ \Delta_{ROOF}\ (AT\ 1/3\ H\ UNIFORM\ BEAM) \cong 0.005''$$

DIAGONALS 6x6x3/8 TUBE $A = 8.08\ IN^2$ Δ_d

	MEMBER P	$f_d = P/A$	$\frac{f_d D}{2E \cos^2\theta}$	
ROOF				0.100"
3RD	13.4	1.66	0.0162	0.084"
2ND	30.3	3.75	0.0366	0.047"
1ST	39.2	4.85	0.0473	0

STRUTS

		P	A	f_s	$\frac{f_s D}{2E}$	Δ_s
ROOF	W14x30	11.0k	8.85 in²	1.24 ksi	0.0082	0.040"
3RD	W18x40	25.0k	11.8 in²	2.14 ksi	0.014	0.032"
2ND	W18x40	32.3k	11.8 in²	2.74 ksi	0.0181	0.018"

TOTAL DEFLECTION

	Δ_c	Δ_d	Δ_s	Δ
ROOF	0.027	0.100	0.040	0.17"
3RD	0.015	0.084	0.032	0.13"
2ND	0.005	0.047	0.018	0.07"

EST. STIFFNESS OF WALL (A) & (C)

$$K_A = \frac{2\ BRACED\ FRAMES/WALL}{\Delta_R} = \frac{2 \times 64.6}{0.17''/12''/ft} = 9120\ k/FT$$

Figure D-3. Continued.

FINAL PROPERTIES

TRANSVERSE PERIOD - SEAOC FORMULA 1-5
FOR STIFFNESS USE THE F, Δ VALUES FROM THE ANALYSIS FOR FRAME A (P. 19). FOR MASS USE THE STORY WEIGHTS (P. 10)

LEVEL	W	F	Δ	$W\Delta^2$	FΔ
R	375^K	25.0^K	0.1052 FT.	4.15	2.63
3	707	20.3	0.0657	3.05	1.33
2	707	10.4	0.0282	0.56	0.29
				7.76	4.25

FOR THE WHOLE BUILDING (W'S ABOVE) THERE ARE THREE FRAMES; SO USE 3 × FΔ = 12.75

$$T = 2\pi\sqrt{\frac{W\Delta^2}{gF\Delta}} = \sqrt{\frac{7.76\ K\text{-}FT.^2}{32.2\ FT/SEC^2 \times 12.75\ K\text{-}FT}}$$

= 0.86 SEC. (BARE FRAME)

STIFFNESS (SEE P. 11 & 12)

IN FINAL DESIGN, $K_1 = 529\ ^K/_{FT}$ (P. 19), REL. $K_1 = 1$
$K_A = 9120\ ^K/_{FT}$ (P. 38), REL. $K_A = \frac{9120}{529} = 17.2$

FOR FRAME A OR C,
$Kd = 17.2 \times 24 = 414$, $Kd^2 = 9930$ $\Sigma Kd^2 = 39,720$

TORSIONAL SHEAR $= \frac{414}{39,720} \times 2.4\ F_L = 0.02\ F_L$

DESIGN BASED ON 0.02 F_L IS STILL O.K. (p. 12)

Figure D-3. Continued.

LONGITUDINAL DIRECTION: ECCENTRIC BRACED FRAMES

This section of the example illustrates the use of eccentric braced frames as an alternate to the concentric braced frames used in the previous section of the example. It should be noted that EBF's would not be an economical choice for this building because the seismic demands are relatively low. The EBF scheme is not to be compared with the CBF scheme for costs for this building: the EBF scheme is presented here only to illustrate the design procedure.

AISC formulas are from Chapter N of the ASD Manual, 9th edition.

DESIGN PROCEDURE

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2. Floor beams	49-50
3. Braces	51-52
4. Columns	53

Figure D-3. Continued.

A. LATERAL FORCES

1. Base shear

$V = (ZIC/R_w)W$

$Z = 0.4$, $I = 1.0$, $R_w = 10$, $S = 1.5$, $C_t = 0.03$, $h_n = 34$ ft.

$T = C_t(h_n)^{3/4} = 0.03 \times (34)^{3/4} = 0.422$ sec.

$T < 0.70$ sec., therefore $F_t = 0$.

$C = (1.25\ S) / T^{2/3} = (1.25 \times 1.5) / (0.422)^{2/3} = 3.33$

$C > 2.75$, therefore use $C = 2.75$

$C/R_w = 2.75/10 = 0.275 > 0.075$

$W = 1789$ kips

$V = (0.4 \times 1.0 \times 2.75 / 10)\ W = 0.11\ W = 0.11 \times 1789$
$= 197$ kips.

2. Story forces and shears

Level	w_i	h_i	w_ih_i	$wh/\sum wh$	F_i	V
R	375	34	12,750	.353	70	
						70
3	707	22	15,554	.431	85	
						155
2	707	11	7,777	.216	42	
						197
	1789		36,081			

3. Horizontal distribution of lateral forces

At the flexible roof diaphragm, the four braced frames each take 0.25 times the story force because of equal tributary areas; at the rigid floor diaphragms, the frames each take 0.30 times the story force because of equal stiffness and consideration of accidental torsion.

Figure D-3. Continued.

B. TYPICAL BEAM SIZING

1. Typical Roof Beam

W12x 19 $d = 12.2"$ $S = 21.3\ in^3$
$t_w = 0.235"$ $I = 130\ in^4$
$Z = 24.7\ in^3$

$l = 32$ ft.

$w = 6' \times (26\ DL + 20\ LL\ psf)$, or 0.276 k/ft.

$M = wl^2/8 = 35.3$ k-ft.

$f_b = M/S = 19.9$ ksi

$\Delta_{LL} = (20/46)5wl^4/384EI = 0.75$ in. $= 1/510$ OK

$V_p \leq 0.55F_y dt_w = 56.8$ kips ASD formula N5-1

2. Typical Floor Beam

W16x36 $d = 15.9"$ $S = 56.5\ in^3$
$t_w = .295"$ $I = 488\ in^4$
$Z = 64\ in^3$

$l = 32$ ft.

$w = 6' \times (75 + 50)$, or 0.750 k/ft.

$M = wl^2/8 = 96$ k-ft.

$f_b = M/S = 20.4$ ksi

$\Delta_{LL} = (50/125)5wl^4/384EI = .54$ in. $= 1/711$ OK

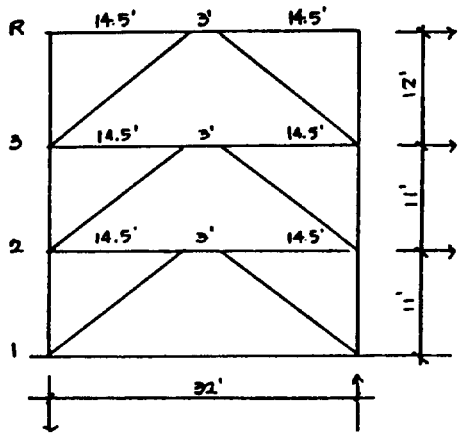
$V_p \leq 0.55F_y dt_w = 92.9$ kips

Figure D-3. Continued.

C. PRELIMINARY FRAME MEMBER SIZING

1. Frame loads

Load factor is 1.3 (ASD Sect. N1)
Distribution factors are 0.25 (roof) and 0.30 (floors)



1.3 (0.25 x 70) = 22.8 kips

1.3 (0.30 x 85) = 33.2 kips

1.3 (0.30 x 42) = 16.4 kips

Link beam shears associated with this factored frame loading:

$(V_p)_R$ = (22.8k x 12') / 32' = 8.55k
$(V_p)_3$ = (56.0k x 11') / 32' = 19.3k
$(V_p)_2$ = (72.4k x 11') / 32' = 24.9k

2. Beam sizes

Size is governed by link-beam strength with a link-beam rotation limit. This example chooses the criterion of 0.06 radians for a clear length of 1.6 M_s/V_s or less, with $M_s = ZF_y$ and $V_s = 0.55F_y dt_w$ (SEAOC 4H2a and 4H3a).

W12x26 at Roof:

M_s = 112 k' (obtained as M_p in the ASD Plastic Design Selection Table)

V_s = 0.55 x 36 x 12.22 x 0.230 = 55.6 kips

Clear length must be ≤ 1.6 x (M_s/V_s)= 3.2' > 3.0

W16x36 at 3rd: (W16x36 also at 2nd)

Ms = 192 k'
Vs = 92.6 k

Clear length must be ≤ 1.6 x (M_s/V_s)= 3.3'

Figure D-3. Continued.

PRELIMINARY FRAME MEMBER SIZING - CONT.

Design braces and columns for forces associated with yield of the following link beams:

R:	W12x26	V_s = 55.6 kips)	{Note: These beam sizes
			{are larger than required
3:	W16x31	V_s = 92.6 kips)	{for the low seismic forces
			{at this example. Beam
2:	W16x31	V_s = 92.6 kips)	{sizes were selected to
			{match typical framing
			{members.

Following are the lateral forces associated with yielding of the link beams:

$12' \times F_R / 32' = 55.6$ kips

$F_R = 148$ kips

$11' \times (F_R + F_3) / 32' = 92.6$ kips

$F_3 = 121$ kips

$F_2 = 0$

Assume forces enter braced bay equally from both sides.

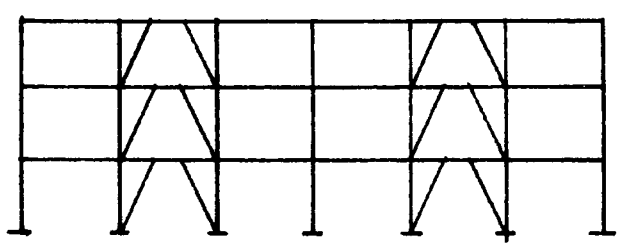


Figure D-3. Continued.

PRELIMINARY FRAME MEMBER SIZING - CONT.

3. Preliminary brace sizes

3rd story:

* brace length = 18.8'
* 1.5 is the load multiplier per SEAOC 4H12
* 148/2 is the horiz. component of the brace force
* 18.8'/14.5' is the geometric multiplier for the axial force

Brace design force = 1.5 x (148k/2 x 18.8/14.5) = 144 k
For use of the table in the AISC manual, the equivalent allowable axial force is

144k/1.7 = 85k

Use TS 7 x 7 x 1/4 P_{all} = 115 kips

2nd and 1st stories: brace length = 18.2'

F_{BR} = 1/1.7 x 1.5 x (270/2 kips x 18.2'/14.5') = 150 kips

Use TS 8 x 8 x 5/16 P_{all} = 186 kips

4. Preliminary column sizing

Size for first story; use that size above

l = 11'; Trib. A = 32' x 12' = 384 sq. ft.

P_{D+L} = 384 (26 + 0 psf)
+ 2 x 384 x (75 + 40 psf)
+ 32' x 34' x 5.3 psf (for the wall)

= 104 kips

With a load multiplier of 1.25 per SEAOC 4H13,

P_E = 1.25 x [(148kips x 23' + 122 x 11)/32']

= 185 kips

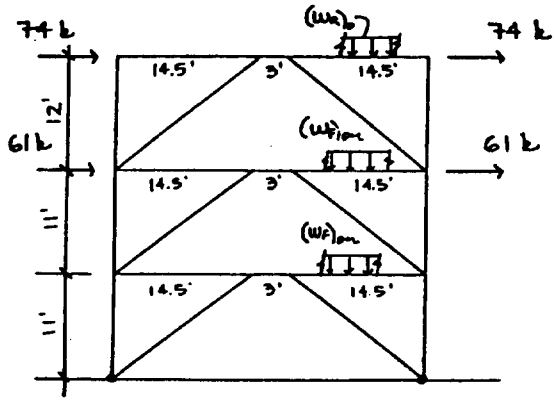
Σ = 289 kips; Equiv. allow. load = 289/1.7 = 170

Use W14x61 P_{all} = 322 kips

Figure D-3. Continued.

D. BRACED FRAME ANALYSIS

Computer input



Load Case I: DL + LL

$(w_R)_D$ = 3'(0.026ksf)
= 0.078 k/ft

$(w_F)_{D+L}$ = 3'(0.075 +0.050)
+ 11'(0.0053) wall
= 0.433 k/ft

Load Case II: Seismic

(at link yield)

E = 29 x 10^3 ksi G = 11.6 x 10^3 ksi

Column bases are pinned; all other joints are rigidly connected.

Member Properties:

Member	Section	A in^2	A_v in^2	I in^4
Roof beams	W12x26	7.65	2.81	204
Floor beams	W16x36	10.6	4.69	448
Roof braces	TS 7x7x1/4	6.59	---	49.4
Floor braces	TS 8x8x5/16	9.36	---	90.9
Columns	W14x61	17.9	---	640

Figure D-3. Continued.

E. FINAL MEMBER SIZING

1. Roof Beam

W12X26

$A = 7.65$ $t_f = 0.38$
$b_f = 6.5$ $d = 12.2$
$r_x = 5.17$ $t_w = 0.23$
$r_y = 1.51$

a. Link beam stability:

$b_f/2t_f = 8.55 \leq 52/\sqrt{F_y} = 8.67$ (SEAOC 4H1)
$d/t_w = 53.0 \leq 640/\sqrt{F_y} = 107$ (ASD B5.1)

b. Link beam strength:

Assumed design forces at the link:

$V_E = V_S = 56$ k $V_D = 0$
$M_E = 1.5' x V_S = 84$ k' $M_D = 0$
$P_E = 0$ $P_D = 0$

$V_s = 55.6$ k
$M_S = 112$ k'

Using beam flanges only, since link beam strength is governed by shear,

$M_S^* = t_f b_f F_y (d-t_f)/12 = 87.6$ k' > 84 OK

c. Beam strength (check as columns per ASD N4).

Assumed design forces outside of link:

V_E: OK by inspection with link
$M_E = 69.4$ k'*
$P_E = 74$ k (bm.axial @ link yield - see comp.out.)
$(M_D)_u = 1.3 \times 0.9^* = 1.2$ k' $M_u = 69.4 + 1.2 = 70.6$
$(P_D)_u = 1.3 \times 0.4^* = 0.5$ k $P_u = 74 + 0.5 = 74.5$

Unbraced lengths:

$l_x = 14.5'$ $(kl/r)_x = 33.7$
$l_y = 7.3'$ (brace @ midspan) $(kl/r)_y = 58$

$F_a = 17.6$ ksi

$(F_e')_x = 129$ ksi

* from computer analysis

Figure D-3. Continued.

FINAL MEMBER SIZING - cont. Roof Beam - cont.

c. Beam strength - cont.

$P_{CR} = 1.7AF_a$ = 229 k (ASD formula N4-1)
$P_e = 23/12\ A(F_e')_x$ = 1891 k (see N4-1)
$P_y = AF_y$ = 275 k

$$M_m = [1.07 - \frac{(l/r_y)\sqrt{F_y}}{3160}]\ M_s = 108\ k' \leq M_s \quad \text{(ASD N4-5)}$$

Formula N4-2: $P_u/P_{CR} + C_m M_u/(1-P_u/P_e)M_m \leq 1.0$

$74.5/229 + (0.85x70.6)/[(1-(74/1891)) \times 108]$

$= 0.33 + 0.58 = 0.91$ OK

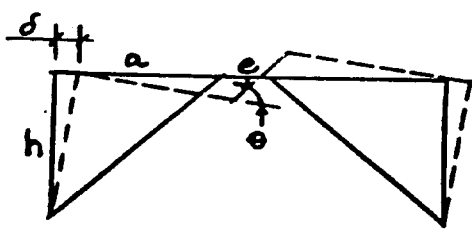
Formula N4-3: $P_u/P_y + M_u/(1.18\ M_p) \leq 1.0;\ M_u \leq M_s$

$74.5/275 + 70.6/(1.18x112)$

$= 0.27 + 0.53 = 0.80$ OK

and $70.6 < 112$ OK

d. Link beam rotation:



θ at $3(R_w/8) \times \delta_{code} \leq 0.06$ rad.

$\delta_{code} \approx [(0.25 \times 70)/148] \times 0.383"^*$
$= 0.0453"$

$3(R_w/8) \times \delta_{code} = 3.75\ \delta_{code} = 0.170"$

$\theta = (1 + 2a/e)(0.170)/h$
$= 0.013$ radians < 0.060 rad.

e. Link beam web:

$V_{code} \leq 0.8\ V_s$ (SEAOC 4H4)

$0.8\ V_s = 44.5$ k

$V_{code} = (0.25x70) \times 12 / 32$
$= 6.56$ k OK

Figure D-3. Continued.

E. FINAL MEMBER SIZING - cont.

2. Floor Beam

W16X36

$A = 10.6$ $t_f = 0.43$
$b_f = 7$ $d = 15.9$
$r_x = 6.51$ $t_w = 0.295$
$r_y = 1.51$

a. Link beam stability:

$b_f/2t_f = 8.14 \leq 52/\sqrt{F_y} = 8.67$
$d/t_w = 53.9 \leq 640/\sqrt{F_y} = 107$

b. Link beam strength:

Assumed design forces at the link:

$V_E = V_S = 92.9$ k $V_{D+L} = 0$
$M_E = 1.5' x V_S = 139$ k' $M_{D+L} = 0$
$P_E = 0$ $P_{D+L} = 0$

$V_s = 92.9$ k
$M_s = 192$ k'
$M_s^* = t_f b_f F_y (d-t_f)/12 = 140 > 139$ OK

c. Beam strength

Assumed design forces outside of link:

V_E: OK by inspection with link
$M_E = 117$ k'*
$P_E = 135$ k
$(M_{D+L})_u = 1.3 \times 6^* = 7.8$ k' $M_u = 117 + 7.8 = 125$
$(P_{D+L})_u = 1.3 \times 5^* = 6.5$ k $P_u = 135 + 6.5 = 141$

Unbraced lengths:

$l_x = 14.5'$ $(kl/r)_x = 26.7$
$l_y = 7.3'$ (brace @ midspan) $(kl/r)_y = 57.6$

$F_a = 17.6$ ksi

$(F_e')_x = 205$ ksi

* from computer analysis

Figure D-3. Continued.

FINAL MEMBER SIZING - cont. Floor Beam - cont.

c. Beam strength - cont.

$P_{CR} = 1.7AF_a$ = 317 k
$P_e = 23/12\ A(F_e')_x$ = 4165 k
$P_y = AF_y$ = 382 k

$$M_m = [1.07 - \frac{(l/r_y)\sqrt{F_y}}{3160}]\ M_s = 184\ k' \leq M_s$$

$P_u/P_{CR} + C_m M_u/(1-P_u/P_e)M_m \leq 1.0$

$= 0.45 + 0.59 = 1.04$ Say OK

$P_u/P_y + M_u/(1.18\ M_s) \leq 1.0;\quad M_u \leq M_s$

$= 0.37 + 0.55 = 0.92$ OK

and $70.6 < 112$ OK

d. Link beam rotation at 3rd Floor:

θ at $3(R_w/8) \times \delta_{code} \leq 0.06$ rad.

$\delta_{code} = [(0.25x70)+(0.30x85)]/(148+122) \times 0.375"^*$

$= 0.0597"$

$3(R_w/8) \times \delta_{code} = 3.75\ \delta_{code} = 0.224"$

$\theta = (1 + 2a/e)(0.224)/h$
$= 0.018$ radians < 0.060 rad.

e. Link beam web:

$V_{code} \leq 0.8\ V_s$

$0.8\ V_s = 74.3$ k

$V_{code} = [(0.25 x70 + 0.30x85 + 0.30x42) \times 11] / 32$
$= 19.1$ k OK

Figure D-3. Continued.

E. FINAL MEMBER SIZING - cont.

3. Brace size - 3-R

TS 8x8x1/4

$A = 7.59\ in^2$
$F_y = 46$ ksi
$b = d = 8$ in.
$t = 0.25$ in
$r_x = r_y = 3.15$ in.

Assumed design forces:

$P_E = 1.5 \times 99^* = 149$ k
$M_E = 1.5 \times 15^* = 22.5$ k'

$(P_D)_u = 1.3 \times 1\ k^* = 1.3$ k $P_u = 149 + 1.3 = 150$
$(M_D)_u = 0$ $M_u = 22.5 + 0 = 22.5$

Unbraced lengths:

$l_x = l_y = 18.8'$ $(kl/r)_x = (kl/r)_y = 72$

$F_a = 20.7$ ksi

$(F_e')_x = 28.8$ ksi

* from computer analysis

$P_{CR} = 1.7AF_a = 267$ k
$P_e = 23/12\ AF_e' = 419$ k'
$P_y = AF_y = 349$ k
$Z = bd^2/4 - b_1d_1^2/4 = 22.5\ in^3$
$M_S = ZF_y = 86.4$ k'

$$M_m = [1.07 - \frac{(l/r_y)\sqrt{F_y}}{3160}]\ M_S = 79.1\ k' \leq M_S$$

$P_u/P_{CR} + C_mM_u/(1-P_u/P_e)M_m \leq 1.0$

$= 0.56 + 0.27 = 0.83$ OK (Use $C_m = 0.6$)

$P_u/P_y + M/(1.18\ M_S) \leq 1.0;\ M \leq M_S$

$= 0.43 + 0.225 = 0.65$ OK

Width\thickness $= b_1/t = 30 \approx 190/\sqrt{F_y} = 28$ Say OK
Depth\thickness $= d/t = 32 \leq 257/\sqrt{F_y} = 38$ OK

Figure D-3. Continued.

E. FINAL MEMBER SIZING - cont.

4. Brace size - 2-3 and 1-2

TS 8x8x3/8

$A = 11.1\ in^2$
$F_y = 46$ ksi
$b = d = 8$ in.
$t = 0.375$ in
$r_x = r_y = 3.09$ in.

Assumed design forces:

$P_E = 1.5 \times 171 = 257$ k
$M_E = 1.5 \times 22 = 33$ k'

$(P_{D+L})_u = 1.3 \times 6.0\ k^* = 8$ k $\qquad P_u = 257 + 8 = 265$
$(M_{D+L})_u = 1.3 \times 0.6\ k^* = 1$ k' $\qquad M_u = 33 + 1 = 34$

Unbraced lengths:

$l_x = l_y = 18.2'$ $\qquad (kl/r)_x = (kl/r)_y = 71$

$F_a = 20.9$ ksi

$(F_e')_x = 29.6$ ksi

* from computer analysis

$P_{CR} = 1.7AF_a = 394$ k
$P_e = 23/12\ AF_e' = 630$ k'
$P_y = AF_y = 511$ k
$Z = bd^2/4 - b_1d_1^2/4 = 32.7\ in^3$
$M_s = ZF_y = 125$ k'

$$M_m = [1.07 - \frac{(l/r_y)\sqrt{F_y}}{3160}]\ M_s = 115\ k' \leq M_s$$

$P_u/P_{CR} + C_mM_u/(1-P_u/P_e)M_m \leq 1.0$

$= 0.67 + 0.31 = 0.98$ OK (Use $C_m = 0.6$)

$P_u/P_y + M/(1.18\ M_s) \leq 1.0;\quad M \leq M_p$

$= 0.52 + 0.23 = 0.75$ OK

Width\thickness $= b_1/t = 19 \approx 190/\sqrt{F_y} = 28$ Say OK
Depth\thickness $= d/t = 21 \leq 257/\sqrt{F_y} = 38$ OK

Figure D-3. Continued.

E. FINAL MEMBER SIZING - cont.

5. Column size - 1-2

W14x53

$A = 15.6\ in^2$
$b_f = 8\ in.$ $d = 13.9\ in.$
$t_w = 0.37\ in.$ $t_c = 0.66\ in.$
$Z = 87.1\ in^3$
$r_x = 5.89\ in.$ $r_y = 1.92\ in.$

Assumed design forces:

$P_E = 1.25 \times 139 = 174\ k$
$M_E = 1.25 \times 15 = 18.8\ k'$

$(P_{D+L})_u = 1.3 \times 104\ k^* = 135K$ $P_u = 174 + 135 = 265$
$(M_{D+L})_u = 1.3 \times 3.7\ k^* = 4.8\ k'$ $M_u = 18.8 + 4.8 = 24$

Unbraced lengths:

$l_x = l_y = 11'$ $(kl/r)_x = 22$
$(kl/r)_y = 69$

$F_a = 16.5\ ksi$

$(F_e')_x = 309\ ksi$

* from computer analysis

$P_{CR} = 1.7AF_a = 438\ k$
$P_e = 23/12\ AF_e' = 9240\ k'$
$P_y = AF_y = 562\ k$
$M_s = ZF_y = 261\ k'$

$$M_m = [1.07 - \frac{(l/r_y)\sqrt{F_y}}{3160}]\ M_s = 245\ k' \leq M_s$$

$P_u/P_{CR} + C_m M_u/(1-P_u/P_e)M_m \leq 1.0$

$= 0.71 + 0.06 = 0.77$ OK (Use $C_m = 0.6$)

$P_u/P_y + M/(1.18\ M_s) \leq 1.0;\ M \leq M_p$

$= 0.55 + 0.08 = 0.63$ OK

Figure D-3. Continued.

DESIGN EXAMPLE D-4

Dual Bracing System With Concrete Frame

Description of Structure. A two-story Office Building in Zone 4 with a complete reinforced concrete vertical load-carrying space frame. The lateral forces are resisted by a dual system consisting of concrete special moment resisting frames and concrete shear walls. The structural concept is illustrated on Sheet 2. The East-West direction is considered.

Construction Outline.

Roof:
Built-up, 5-ply.
Concrete joists and girders.
Suspended ceiling.

2nd & 3rd Floors:
Concrete joists and girders.
Asphalt tile.
Suspended ceiling.

1st Floor:
Concrete slab-on-grade.

Exterior Walls:
Bearing Walls in concrete and non-bearing, non-shear insulated metal panels.

Partitions:
Non-structural removable drywall, except concrete as structurally required.

Design Concept. The structure is a dual system meeting SEAOC 1D6d, and it qualifies for an R_w-factor of 12 as follows: (1) the SMRF is designed to resist not less than 25 percent of the prescribed lateral forces; (2) the shear wall and the special frame systems are designed to resist the total prescribed lateral forces in proportion to their relative rigidities. In this example it is assumed that the rigidity of the frame system is negligible when compared to the wall system. (Compare stiffness as proportional to the third power of the width of one element: 14^3 for a column is much less that 120^3 for a wall.) Therefore, the wall system is designed for 100 percent of the base shear and the frame system for 25%. The roofs and floors form rigid diaphragms, and the seismic loads to the frames are proportioned according to their stiffnesses and loads to the walls according to theirs. The building is assumed to be symmetrical about both axes so that only accidental torsion is involved.

Discussion. Vertical and lateral forces are pre-computed. (See Example A-5 for a typical computation.) The shear walls in the south wall (Line D) are designed for the given lateral forces. The seismic frames would be designed for 25% of these forces, using the methods of Example A-2. Deformation compatibility is investigated for the nonseismic frames.

Materials. Conc. f'_c = 3,000 psi; reinf. steel f_y = 60,000 psi.

Figure D-4. Dual bracing system.

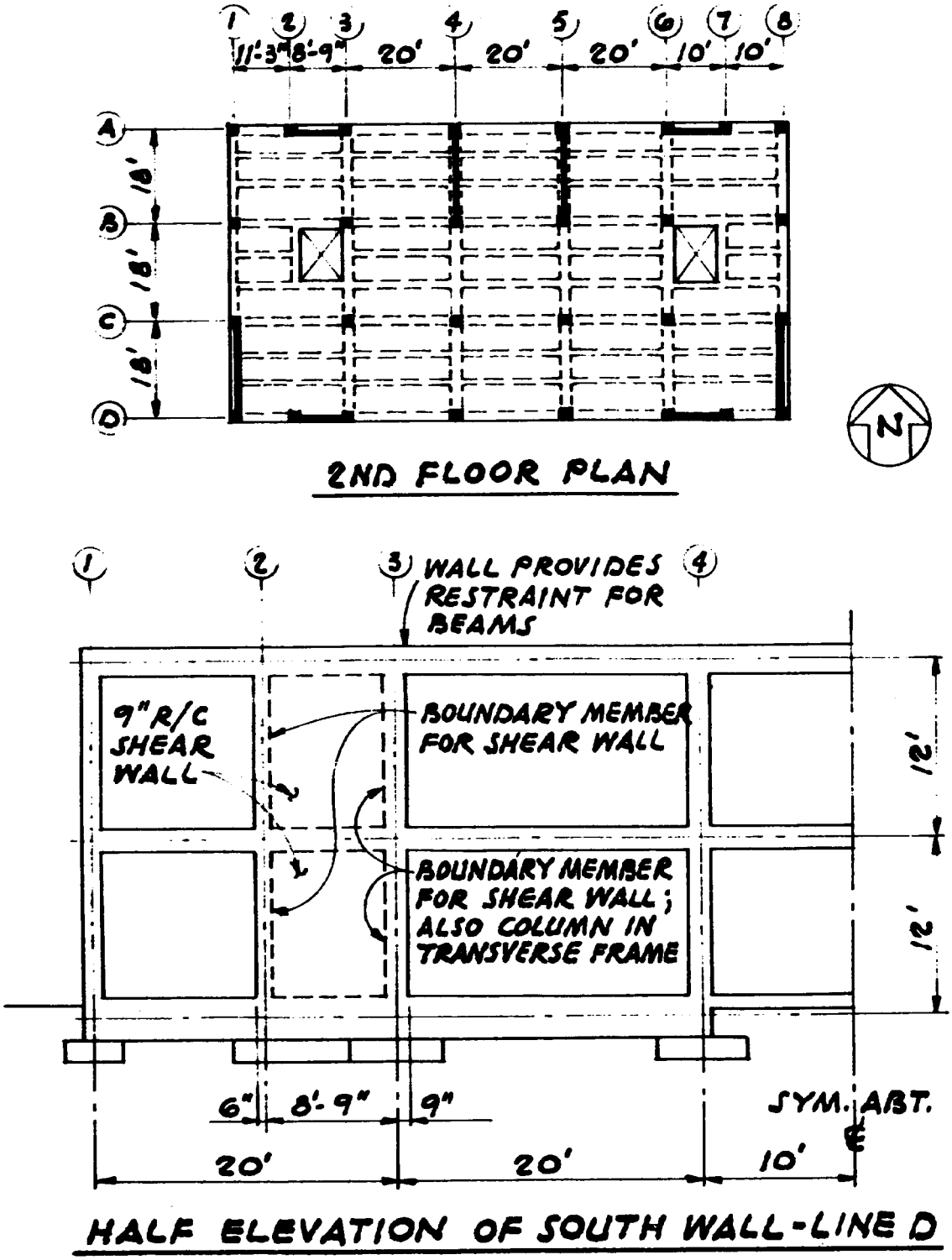
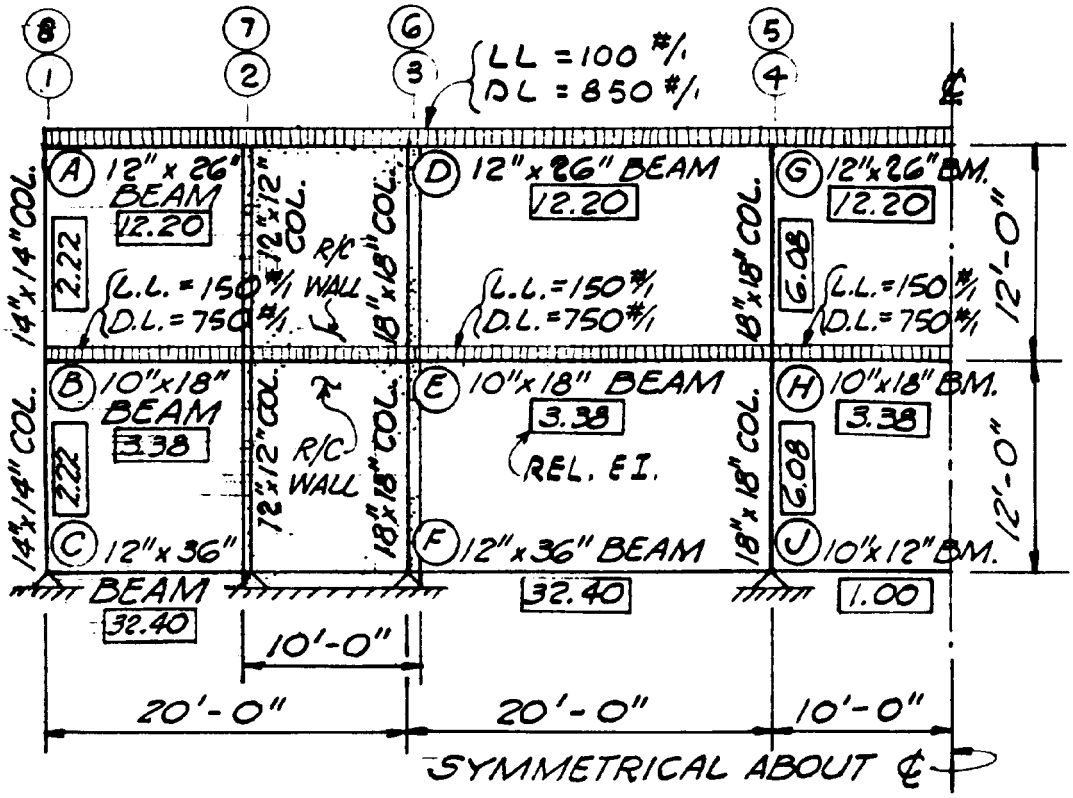


Figure D-4. Continued.



MEMBERS AND LOADS - SOUTH WALL - LINE D

FRAME PROVISIONS

The two interior frames (Lines B and C) will be designed as special moment resisting frames to carry 25% of the total required lateral force. See Example A-2. This example will deal only with Lines A and D which have shear walls that carry 100% of the lateral force. Deformation compatibility (SEAOC 1H2d) must be investigated for the vertical load-carrying frames on Lines A and D (see p. 11).

As an alternate, the interior frames could be designed for vertical load only (with an investigation for deformation compatibility), and the lateral forces would be carried by ductile moment resisting space frames on Lines A and D. In these frames there is a choice concerning the columns on Lines 2 and 7: the columns may be treated as columns with adjacent girders of 10' span, or they may be treated as boundary members for the shear walls. In the latter case, the girders must still be designed, together with the columns, for the actual 10' spans, but they must also be designed to span 20' from 1 to 3 and from 6 to 8 in case the shear walls and boundary members fail.

Figure D-4. Continued.