[1] Example 0001 - a simple calculation [0001]

Numbers are fun whatever you do. **First comes one and then comes two!**

```
number of brothers                   | bros = 2
number of sisters                    | siss = 1
height of tall brother               | bro_h = 6.0 ft
height of sister                     | sis_h = 5.8 ft
```

How many siblings do I have?

sum | add up number of brothers and sisters [1.1]

$$bros + siss$$

$$2 + 1$$

$$sum = 3$$

What is the height difference between the tall brother and sister in inches?

dif | subtract heights [1.2]

$$bro_h - sis_h$$

$$6.00 ft - 5.80 ft$$

$$dif = 2.4\ in$$

[end of calc]

[1] Example 0101 - basic template [0101]

This model calculates the mid-span beam moment under uniformly distributed (UDL) floor loads.

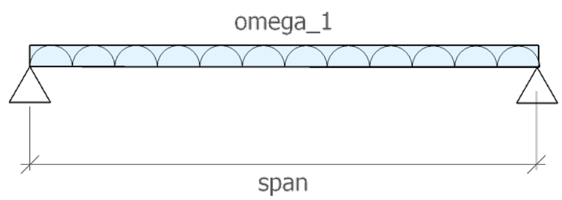

Figure 1.1: Simply supported beam

operation examples:

- [s] section
- [y] sympy and LaTeX symbolic expresssions
- [t] term
- [e] equation
- [#-] format and file operations

reST markup examples:

- lists, **bold**, *italic*
- tables
- raw latex

Notation: (LaTeX block is not processed for UTF calcs)

$$
\begin{aligned}
D_n &= \text{nominal dead load of material or component n} \\
L_n &= \text{nominal live load from action n} \\
DL_n, LL_n &= \text{sum of nominal dead or live loads} \\
l_n &= \text{effective beam span} \\
\omega_n &= \text{factored line load on element n} \\
M_n &= \text{factored bending moment on component n} \\
q_n &= \text{factored area load n} \\
w_n &= \text{tributary width n}
\end{aligned}
$$

[2] Beam Loads and geometry [0101]

Dead and live load contributions to beam UDL

ASCE 7 - 05 Load Effects

Equation No.	Load Combination
16-1	1.4(D+F)
16-2	1.2(D+F+T) + 1.6(L+H) + 0.5(Lr or S or R)
16-3	1.2(D+F+T) + 1.6(Lr or S or R) + (f1L or 0.8W)

Dead loads

```
joists                                | D_1 = 3.8 psf
plywood                               | D_2 = 2.1 psf
partitions                            | D_3 = 10.0 psf
fixed machinery                       | D_4 = 0.5 kips/ft
```

Live loads

```
ASCE7-05                              | L_1 = 40.0 psf
```

Beam tributary width and span

```
distance between beams                | w_1 = 2.0 ft
beam span                             | l_1 = 14.0 ft
```

[3] Maximum bending moment [0101]

DL_1 | Total UDL factored dead load [3.1]

$$1.2 \cdot D_4 + 1.2 \cdot w_1 \cdot (D_1 + D_2 + D_3)$$

$$DL_1 = 0.64 \; kips/ft$$

LL_1 | Total UDL factored live load [3.2]

$$1.6 \cdot L_1 \cdot w_1$$

$$LL_1 = 0.13 \; kips/ft$$

omega_1 | factored UDL [3.3]

$$DL_1 + LL_1$$

$$omega_1 = 0.77 \; kips/ft$$

M_1 | Bending moment at mid-span [3.4]

$$\frac{\omega_1}{8} \cdot l_1^2$$

$$\frac{0.77 kips/ft}{8} \cdot 14.00 ft^2$$

$$M_1 = 18.8 \; ft.kip$$

[4] Symbolic rendering using sympy or LaTeX [0101]

Equation rendered from **SymPy**

[4.1]

$$\sigma = \frac{M}{I} \cdot z$$

Equation rendered from **LaTeX** (expresssion copied from Wikipedia HTML source)

[4.2]

$$\sigma = \frac{Mz}{I} = -zE\,\frac{\mathrm{d}^2 w}{\mathrm{d}x^2}$$

[end of calc]

[1] Example 0201 - Isolation Bearing Properties [0201]

Calculate shape factors, compression stiffness and buckling loads for a rubber laminated seismic isolation bearing

Measured Properties

```
bearing diameter                    | phi = 36.00 in
rubber layer thickness              | t_layer = 0.38 in
initial shear modulus               | G_i = 100.00 psi
shear modulus                       | G = 50.00 psi
total rubber height                 | R_t = 11.00 in
bearing height                      | h_b = 19.00 in
bulk modulus                        | K_1 = 250.00 ksi
measured compression stiffness      | K_cm = 11800.00 kips/in
constant                            | pi = 3.1418
```

Loads

```
applied load                        | P = 3000.00 kips
factored load demand                | P_u = 5000.00 kips
```

Figure 1.1: Bearing Geometry

[2] Basic Calculated Properties [0201]

a_1 | Bearing area [2.1]

$$\frac{\pi}{4} \cdot \phi^2$$

$$\frac{\pi}{4} \cdot 36.00in^2$$

$$a_1 = 1{,}017.94\ in2$$

S_1 | Axial shape factor [2.2]

$$\frac{\phi}{4 \cdot t_{layer}}$$

$$\frac{36.00in}{4 \cdot 0.38in}$$

$$S_1 = 23.7$$

n | Number of rubber layers [2.3]

$$\frac{R_t}{t_{layer}}$$

$$\frac{11.00in}{0.38in}$$

$$n = 29$$

I_1 | Moment of inertia [2.4]

$$0.04875 \cdot \phi^4$$

$$0.04875 \cdot 36.0in^4$$

$$I_1 = 81{,}881\ in4$$

[3] Compression Stiffness [0201]

GK | Stiffness term [3.1]

$$\frac{12}{K_1} \cdot G_i$$

$$\frac{12}{250.00ksi} \cdot 100.00psi$$

$$GK = 0.0048$$

E_c | Compression modulus after Kelly [3.2]

$$K_1 \cdot \left(-\frac{1}{GK^{0.5} \cdot S_1} + 1 + \frac{1}{4 \cdot GK \cdot S_1^2}\right)$$

$$250.000ksi \cdot \left(-\frac{1}{0.005^{0.5} \cdot 23.684} + 1 + \frac{1}{4 \cdot 0.005 \cdot 23.684^2}\right)$$

$$E_c = 120.9\ ksi$$

kv_k | Compression stiffness after Kelly [3.3]

$$\frac{E_c}{R_t} \cdot a_1$$

$$\frac{120.86ksi}{11.00in} \cdot 1017.94in2$$

$$kv_k = 11{,}184.1\ kips/in$$

kv_R | Compresssion stiffness after Robinson [3.4]

$$\frac{6 \cdot G_i \cdot K_1 \cdot S_1^2 \cdot a_1}{R_t \cdot \left(6 \cdot G_i \cdot S_1^2 + K_1\right)}$$

$$\frac{6 \cdot 100.00psi \cdot 250.00ksi \cdot 23.68^2 \cdot 1017.94in2}{11.00in \cdot \left(6 \cdot 100.00psi \cdot 23.68^2 + 250.00ksi\right)}$$

$$kv_R = 13{,}274.7\ kips/in$$

[4] Buckling Load - Pe [0201]

P_e | Buckling capacity after Kelly [4.1]

$$\frac{\pi^2 \cdot E_c \cdot I_1}{3 \cdot R_t \cdot h_b}$$

$$\frac{\pi^2 \cdot 120.86ksi \cdot 81881.28in4}{3 \cdot 11.00in \cdot 19.00in}$$

$$P_e = 155{,}791.2\ kips$$

dc ratio check [4.2]

$$\frac{P_u}{P_e} \leq 1.0$$

$$\frac{5000.00kips}{155791.17kips} \leq 1.0$$

$$0.03 \mathbf{<=} 1.00 - \boldsymbol{ok}$$

[end of calc]

[1] Example 0301 - beam deflection [0301]

Table of tube bending deflections as a function of span and depth when analyzed as a simply supported classical beam with uniform distributed load.

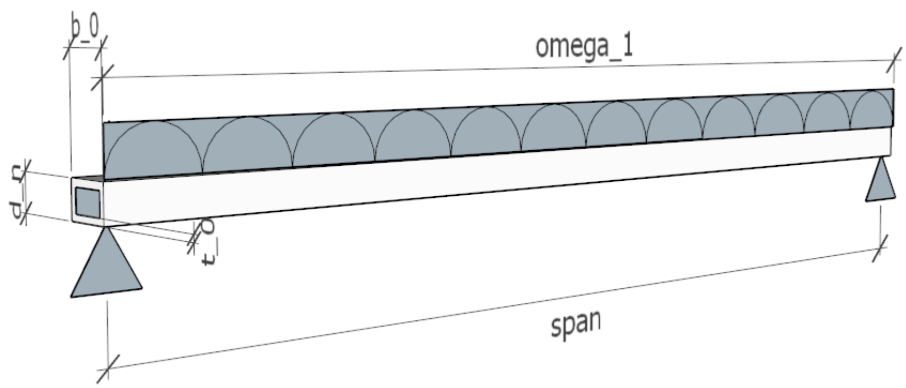

Figure 1.1: Tube geometry

[2] Materials and Geometry [0301]

Calculate tube moment of inertia for an initial condition and a range of depths. Neglect radii at corners.

6061-T6 Aluminum

```
elastic modulus                    | E_0 = 10000.000 ksi
```

Tube Dimensions

```
tube width                         | b_0 = 5.000 in
wall thickness                     | t_0 = 0.125 in
```

Design Load and Span

```
trial clear span                   | span = 6.000 ft
trial tube depth                   | d_0 = 9.000 in
uniform distributed load           | omega_1 = 0.100 kip/in
```

I_9 | Moment of inertia [2.1]

$$\frac{b_0}{12} \cdot d_0^3 - \frac{1}{12} \cdot (b_0 - 2 \cdot t_0) \cdot (d_0 - 2 \cdot t_0)^3$$

$$I_9 = 38.573\ in4$$

Table : I_x [in4] : depth [in] [2.2]

$$I_n = \frac{b_0 d_n^3}{12} - \frac{1}{12} (b_0 - 2t_0) (d_n - 2t_0)^3$$

depth = 9	**depth = 10**	**depth = 11**	**depth = 12**	**depth = 13**
38.57	49.78	62.84	77.87	94.98

[3] Evaluate Deflections [0301]

Evaluate beam deflections for an initial I and span and then a range of values.

Check an initial case

```
initial I                                    | I_1 = 35.000 in4
beam span                                    | span_1 = 10.000 ft
```

delta_1 | beam deflection under uniform load [3.1]

$$\frac{5 \cdot \omega_1 \cdot span_1^4}{384 \cdot E_0 \cdot I_1}$$

$$\frac{5 \cdot 0.100 kip/in \cdot 10.000 ft^4}{384 \cdot 10000.000 ksi \cdot 35.000 in4}$$

$$delta_1 = 0.771\ in$$

Table : Deflections [in] (span [in], I [in4]) [3.2]

$$delta_2 = \frac{5\omega_1 span_2^4}{384 E_0 I_2}$$

span	I = 33	I = 43	I = 53	I = 63	I = 73
72	0.106	0.081	0.066	0.056	0.048
96	0.335	0.257	0.209	0.176	0.151
120	0.818	0.628	0.509	0.429	0.370
144	1.697	1.302	1.056	0.889	0.767
168	3.143	2.412	1.957	1.646	1.421

[end of calc]

[1] Example 0401 - Building and Load Information [0401]

Design Example 1A - Special Concentrically Braced Frame
2006 IBC Structural/Seismic Design Manual, Vol. 3

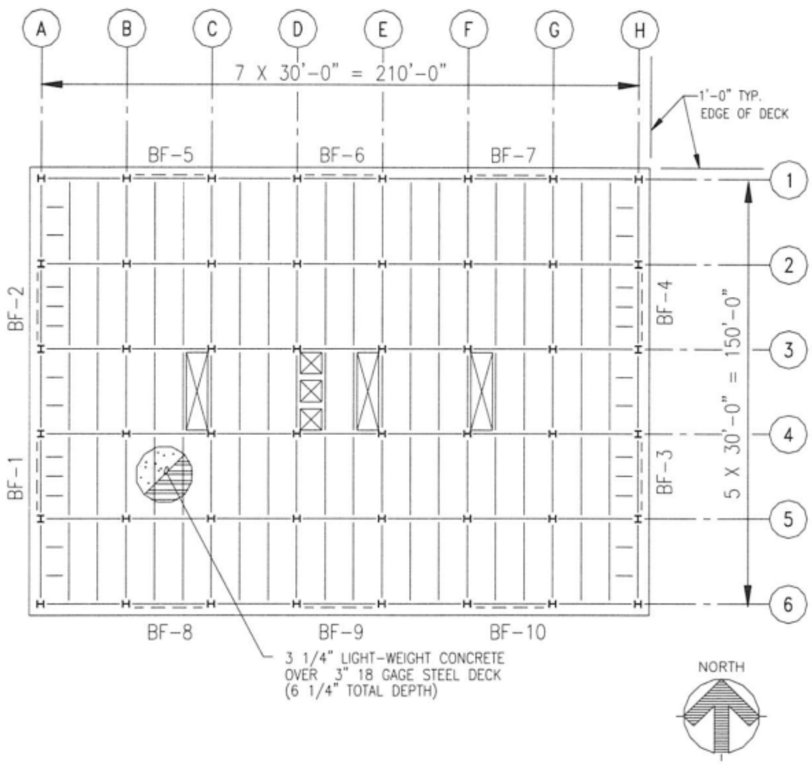

Figure 1A- 2. Typical floor and roof framing plan

Figure 1.1: Floor framing

[2] Roof Dead Loads [0401]

```
Roofing                                    | dl_01 = 6.000 psf
Insulation                                 | dl_02 = 3.000 psf
Steel deck + fill                          | dl_03 = 47.000 psf
Roof framing                               | dl_04 = 8.000 psf
Partition seismic                          | dl_05 = 5.000 psf
Ceiling                                    | dl_06 = 3.000 psf
Mech Elec                                  | dl_07 = 2.000 psf
```

roofwt | total roof weight [2.1]

$$\mathrm{sum}\left(dl_{01} + dl_{02} + dl_{03} + dl_{04} + dl_{05} + dl_{06} + dl_{07}\right)$$

$$\mathrm{sum}\left(6.000 psf + 3.000 psf + 47.000 psf + 8.000 psf + 5.000 psf + 3.000 psf + 2.000 psf\right)$$

$$roofwt = 74.000\ psf$$

[3] Floor / Wall Dead Loads and Live Loads [0401]

Floor Loads

```
Floor covering                    | dl_11 = 1.000 psf
Steel deck + fill                 | dl_12 = 47.000 psf
Framing (beams + cols.)           | dl_13 = 13.000 psf
Partition (ASCE 12.7.2)           | dl_14 = 10.000 psf
Ceiling                           | dl_15 = 3.000 psf
Mech Elec                         | dl_16 = 2.000 psf
```

floorwt | total floor weight [3.1]

$$\text{sum}\left(dl_{11} + dl_{12} + dl_{13} + dl_{14} + dl_{15} + dl_{16}\right)$$

$$\text{sum}\left(1.000psf + 47.000psf + 13.000psf + 10.000psf + 3.000psf + 2.000psf\right)$$

$$floorwt = 76.000\ psf$$

Exterior curtain wall loads steel studs, gypsum board, EIFS skin

```
exterior wall                     | extwallwt = 20.000 psf
```

Live loads (ASCE7-05)

```
roof                              | roof_ll = 20.000 psf
floor                             | floor_ll = 50.000 psf
```

[4] Seismic and structural data [0401]

Mapped spectral response accelerations 5% critical damping Site Class D (IBC 1613.5) Seismic Source Type = A Distance to seismic source < 2 km

```
0.2 sec-response (IBC 22.0)         | S_S = 2.100 G
1.0 sec-response (IBC 11.5.1)       | S_1 = 0.930 G
standard occupancy                  | I_f = 1.0
```

Structural materials

Charpy V-notch toughness for all connections in the seismic-load-resisting system

Weld electrodes E70XX, 20 ft-lb at 0 degrees

```
wide flange,plate ASTM A992, Gr. 50 | F1_y = 50.000 ksi
HSS (round) ASTM A500, Grade B      | F2_y = 42.000 ksi
Plate ASTM A572, Grade 50           | F3_y = 50.000 ksi
lightweight conc fill fc'=3000 psi  | f1_c = 3.000 ksi
```

The calc is not a structural design calculation. The calc user assumes sole responsibility for all inputs and results.

[end of calc]

[1] Seismic Loads [0402]

Design Example 1A - Special Concentrically Braced Frame
2006 IBC Structural/Seismic Design Manual, Vol. 3

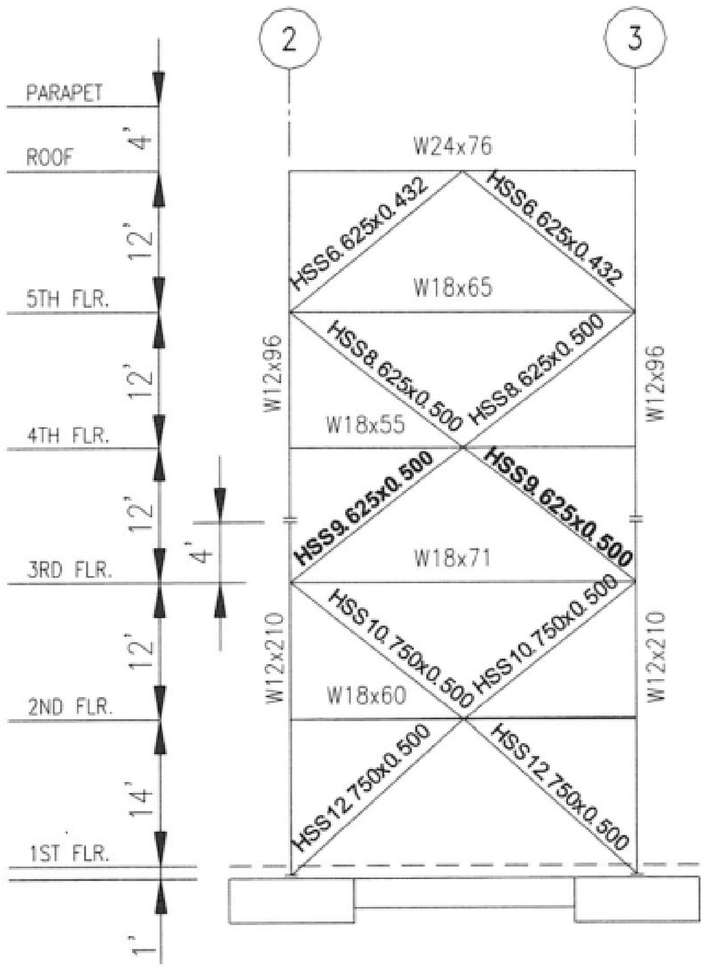

Figure 1A- 3. Elevation of frame BF-2

Figure 1.1: Braced frame elevation

[2] Site ground motion [0402]

Maximum considered earthquake spectral response accelerations for short periods and at 1.0-second period adjusted for site class effects. 5 percent-damped design spectral response accelerations at short periods and at 1.0-second period, SD1, adjusted for site class effects.

```
Eq 11.4-1                          | F_a = 1.0
Eq 11.4-2                          | F_v = 1.5
short period spectral accel        | S_S = 2.1
1 second spectral accel            | S_1 = 0.93
```

S_MS | Eq 11.4-1 (g) [2.1]

$$F_a \cdot S_S$$

$$1.000 \cdot 2.100$$

$$S_M S = 2.100$$

S_M1 | Eq 11.4-2 (g) [2.2]

$$F_v \cdot S_1$$

$$1.500 \cdot 0.930$$

$$S_M 1 = 1.395$$

S_DS | Eq 11.4-3 (g) [2.3]

$$\frac{2}{3} \cdot S_{MS}$$

$$\frac{2}{3} \cdot 2.100$$

$$S_D S = 1.400$$

S_D1 | Eq 11.4-4 (g) [2.4]

$$\frac{2}{3} \cdot S_{M1}$$

$$\frac{2}{3} \cdot 1.395$$

$$S_D 1 = 0.930$$

[3] Base shear coefficients [0402]

The approximate fundamental building period can be computed from the building height and the structural system (IBC 12.2-1).

```
building height (no dimension)      | h_n = 62
building height                     | bldg_ht = 62.000 ft
response mod coef (T 12.2-1)        | R_0 = 6.0
system overstrength factor          | Q_0 = 2.0
deflection amplif factor            | C_d = 5.0
period coefficient - steel          | C_T = 0.02
importance factor                   | I_f = 1.0
```

T_a | code approximate fundamental period [3.1]

$$C_T \cdot h_n^{\frac{3}{4}}$$

$$0.020 \cdot 62^{\frac{3}{4}}$$

$$T_a = 0.442$$

C_Sbasic | basic seismic response coefficient (EQ 12.8-2) [3.2]

$$\frac{I_f}{R_0} \cdot S_{DS}$$

$$\frac{1.000}{6.000} \cdot 1.400$$

$$C_S basic = 0.233$$

C_Smax | coefficient need not exceed (EQ 12.8-3) [3.3]

$$\frac{R_0 \cdot S_{D1}}{I_f \cdot T_a}$$

$$\frac{6.000 \cdot 0.930}{1.000 \cdot 0.442}$$

$$C_S max = 12.627$$

C_Smin | coefficient minimum (EQ 12.8-3) [3.4]

$$0.044 \cdot I_f \cdot S_{DS}$$

$$0.044 \cdot 1.000 \cdot 1.400$$

$$C_S min = 0.062$$

C_S | bounded seismic response coefficient (EQ 12.8-2) [3.5]

$$\min\left(\max\left(C_{Sbasic}, C_{Smin}\right), C_{Smax}\right)$$

$$\min\left(\max\left(0.233, 0.062\right), 12.627\right)$$

$$C_S = 0.233$$

Note that for buildings in Seismic Design Category E or F, and those buildings for which the 1.0-second spectral response, S1, is equal to or greater than 0.6g, the value of the seismic response coefficient, CS, must be at least equal to the following value.

C_Emin | min seismic response coefficient for SDC E and F (EQ 12.8-2) [3.6]

$$\frac{0.5}{R_0} \cdot I_f \cdot S_{DS}$$

$$\frac{0.5}{6.000} \cdot 1.000 \cdot 1.400$$

$$C_E min = 0.117$$

[4] Vertical earthquake effects [0402]

The vertical component of seismic acceleration is also computed using a very low period for the spectral acceleration.

```
dead load                              | DL = 3.000 kip
base shear effects on vertical         | Q_E = 1.000 kip
live load effects                      | f_1L = 2.000 kip
capacity coefficient                   | Omega_0 = 2.0
redundancy factor (IBC 12.3.4)         | rho = 1.0
```

E_p | plus vertical earthquake effect Eq 12.4-2 [4.1]

$$0.2 \cdot DL \cdot S_{DS} + Q_E \cdot \rho$$

$$0.2 \cdot 3.000 kip \cdot 1.400 + 1.000 kip \cdot 1.000$$

$$E_p = 1.840 \; kip$$

E_m | minus vertical earthquake effect Eq 12.4-2 [4.2]

$$-0.2 \cdot DL \cdot S_{DS} + Q_E \cdot \rho$$

$$-0.2 \cdot 3.000 kip \cdot 1.400 + 1.000 kip \cdot 1.000$$

$$E_m = 0.160 \; kip$$

Em_p | maximum vertical earthquake effect Eq 12.4-2 [4.3]

$$0.2 \cdot DL \cdot S_{DS} + \Omega_0 \cdot Q_E$$

$$0.2 \cdot 3.000 kip \cdot 1.400 + 2.000 \cdot 1.000 kip$$

$$Em_p = 2.840 \; kip$$

Em_m | maximum vertical earthquake effect Eq 12.4-2 [4.4]

$$-0.2 \cdot DL \cdot S_{DS} + \Omega_0 \cdot Q_E$$

$$-0.2 \cdot 3.000kip \cdot 1.400 + 2.000 \cdot 1.000kip$$

$$Em_m = 1.160\ kip$$

[end of calc]

[1] Base shear and vertical distribution [0403]

The floor area at each level is 32,224 square feet. The perimeter of the exterior curtain wal

l is 728 feet. The roof parapet height is 4 feet. The curtain-wall weights are distributed to each floor by tributary height. Values are organized from top to bottom (floors 5 to 1).

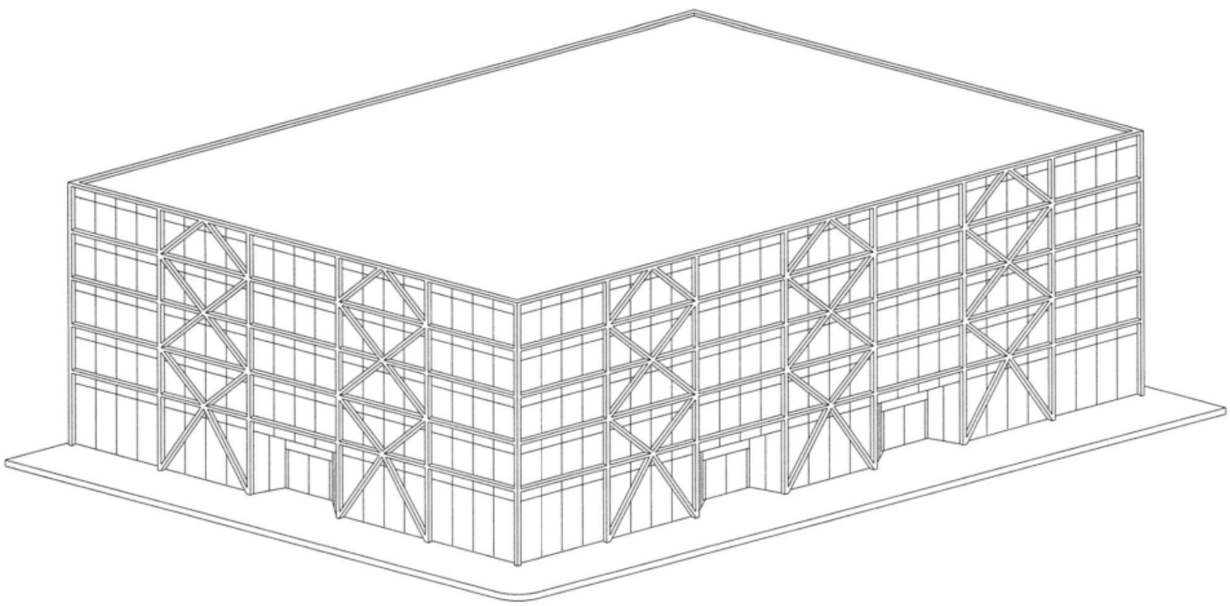

Figure 1A-1. Special concentrically braced frame building

Figure 1.1: Building Structural Scheme

```
wall length [ft]                    | wall_length = 728.0
wall unit weight [psf]              | wallUnitWt = 20.0
floor area  [sf]                    | floor_area = 32224.0

. floor unit weight [psf]           | floor_uweight =
```

. [74. 76. 76. 76. 76.]

```
. wall height [ft]                  | wall_ht =
```

. [10. 12. 12. 12. 13.]

Table - floor weight (kips) [1.1]

$$floorWeight = 0.001 floor_{area} floor_{uweight}$$

lvl = roof	lvl = 5	lvl = 4	lvl = 3	lvl = 2
2385	2449	2449	2449	2449

Table - wall weight (kips) [1.2]

$$wallWeight = 0.001 wallUnitWt wall_{ht} wall_{length}$$

lvl = roof	lvl = 5	lvl = 4	lvl = 3	lvl = 2
146	175	175	175	189

Table - story weight (kips) [1.3]

$$storyWeight = floorWeight + wallWeight$$

lvl = roof	lvl = 5	lvl = 4	lvl = 3	lvl = 2
2530	2624	2624	2624	2638

totalStorywt | sum of story weights (kips) [1.4]

$$KIPS \cdot \operatorname{sum}(storyWeight)$$

$$KIPS \cdot \operatorname{sum}([2530.2624.2624.2624.2638.])$$

$$totalStorywt = 13{,}040\ kips$$

Import loads from model 0402

[2] Seismic Loads [0402]

Design Example 1A - Special Concentrically Braced Frame
2006 IBC Structural/Seismic Design Manual, Vol. 3

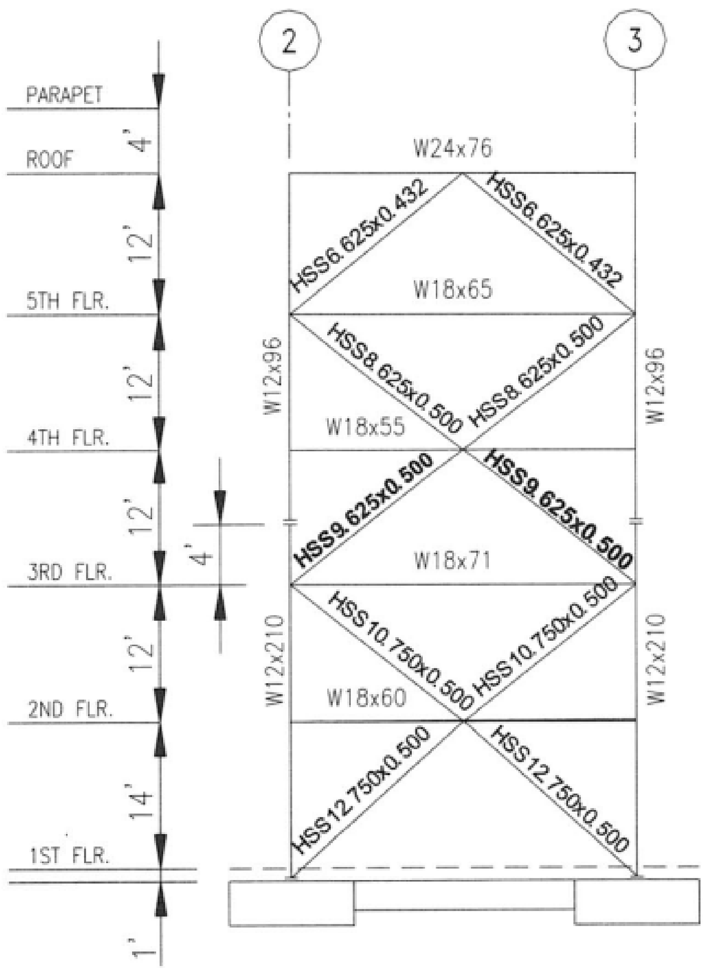

Figure 1A- 3. Elevation of frame BF-2

Figure 2.1: Braced frame elevation

[3] Site ground motion [0402]

Maximum considered earthquake spectral response accelerations for short periods and at 1.0-second period adjusted for site class effects. 5 percent-damped design spectral response accelerations at short periods and at 1.0-second period, SD1, adjusted for site class effects.

```
Eq 11.4-1                           | F_a = 1.0
Eq 11.4-2                           | F_v = 1.5
short period spectral accel         | S_S = 2.1
1 second spectral accel             | S_1 = 0.93
```

S_MS | Eq 11.4-1 (g) [3.1]

$$F_a \cdot S_S$$

$$1.000 \cdot 2.100$$

$$S_M S = 2.100$$

S_M1 | Eq 11.4-2 (g) [3.2]

$$F_v \cdot S_1$$

$$1.500 \cdot 0.930$$

$$S_M 1 = 1.395$$

S_DS | Eq 11.4-3 (g) [3.3]

$$\frac{2}{3} \cdot S_{MS}$$

$$\frac{2}{3} \cdot 2.100$$

$$S_D S = 1.400$$

S_D1 | Eq 11.4-4 (g) [3.4]

$$\frac{2}{3} \cdot S_{M1}$$

$$\frac{2}{3} \cdot 1.395$$

$$S_D 1 = 0.930$$

[4] Base shear coefficients [0402]

The approximate fundamental building period can be computed from the building height and the structural system (IBC 12.2-1).

```
building height (no dimension)      | h_n = 62
building height                     | bldg_ht = 62.000 ft
response mod coef (T 12.2-1)        | R_0 = 6.0
system overstrength factor          | Q_0 = 2.0
deflection amplif factor            | C_d = 5.0
period coefficient - steel          | C_T = 0.02
importance factor                   | I_f = 1.0
```

T_a | code approximate fundamental period [4.1]

$$C_T \cdot h_n^{\frac{3}{4}}$$

$$0.020 \cdot 62^{\frac{3}{4}}$$

$$T_a = 0.442$$

C_Sbasic | basic seismic response coefficient (EQ 12.8-2) [4.2]

$$\frac{I_f}{R_0} \cdot S_{DS}$$

$$\frac{1.000}{6.000} \cdot 1.400$$

$$C_S basic = 0.233$$

C_Smax | coefficient need not exceed (EQ 12.8-3) [4.3]

$$\frac{R_0 \cdot S_{D1}}{I_f \cdot T_a}$$

$$\frac{6.000 \cdot 0.930}{1.000 \cdot 0.442}$$

$$C_Smax = 12.627$$

C_Smin | coefficient minimum (EQ 12.8-3) [4.4]

$$0.044 \cdot I_f \cdot S_{DS}$$

$$0.044 \cdot 1.000 \cdot 1.400$$

$$C_Smin = 0.062$$

C_S | bounded seismic response coefficient (EQ 12.8-2) [4.5]

$$\min\left(\max\left(C_{Sbasic}, C_{Smin}\right), C_{Smax}\right)$$

$$\min\left(\max\left(0.233, 0.062\right), 12.627\right)$$

$$C_S = 0.233$$

Note that for buildings in Seismic Design Category E or F, and those buildings for which the 1.0-second spectral response, S1, is equal to or greater than 0.6g, the value of the seismic response coefficient, CS, must be at least equal to the following value.

C_Emin | min seismic response coefficient for SDC E and F (EQ 12.8-2) [4.6]

$$\frac{0.5}{R_0} \cdot I_f \cdot S_{DS}$$

$$\frac{0.5}{6.000} \cdot 1.000 \cdot 1.400$$

$$C_Emin = 0.117$$

[5] Vertical earthquake effects [0402]

The vertical component of seismic acceleration is also computed using a very low period for the spectral acceleration.

```
dead load                              | DL = 3.000 kip
base shear effects on vertical         | Q_E = 1.000 kip
live load effects                      | f_1L = 2.000 kip
capacity coefficient                   | Omega_0 = 2.0
redundancy factor (IBC 12.3.4)         | rho = 1.0
```

E_p | plus vertical earthquake effect Eq 12.4-2 [5.1]

$$0.2 \cdot DL \cdot S_{DS} + Q_E \cdot \rho$$

$$0.2 \cdot 3.000 kip \cdot 1.400 + 1.000 kip \cdot 1.000$$

$$E_p = 1.840 \; kip$$

E_m | minus vertical earthquake effect Eq 12.4-2 [5.2]

$$-0.2 \cdot DL \cdot S_{DS} + Q_E \cdot \rho$$

$$-0.2 \cdot 3.000 kip \cdot 1.400 + 1.000 kip \cdot 1.000$$

$$E_m = 0.160 \; kip$$

Em_p | maximum vertical earthquake effect Eq 12.4-2 [5.3]

$$0.2 \cdot DL \cdot S_{DS} + \Omega_0 \cdot Q_E$$

$$0.2 \cdot 3.000 kip \cdot 1.400 + 2.000 \cdot 1.000 kip$$

$$Em_p = 2.840 \; kip$$

Em_m | maximum vertical earthquake effect Eq 12.4-2 [5.4]

$$-0.2 \cdot DL \cdot S_{DS} + \Omega_0 \cdot Q_E$$

$$-0.2 \cdot 3.000kip \cdot 1.400 + 2.000 \cdot 1.000kip$$

$$Em_m = 1.160\ kip$$

[6] Design base shear V_1 = C_S * W (Eq 12.8-1) [0403]

V_1 | total base shear [6.1]

$$C_S \cdot totalStorywt$$

$$0.23 \cdot 13039.71 kips$$

$$V_1 = 3{,}043 \ kips$$

[7] Vertical distribution of shear [0403]

The total lateral force (i.e., design base shear) is distributed over the height of the building in accordance with IBC 12.8.3. k is a distribution exponent related to building period as follows: k = 1 for buildings with a period of 0.5 second or less k = 2 for buildings with a period of 2.5 seconds or greater k = is interpolated between these values for buildings with period between 0.5 second and 2.5 seconds

```
. height above ground [ft]          | h_x =
```

. [62 50 38 26 14]

w_x | story weight [kips] [7.1]

```
. w_x =
. [ 2530.2  2623.7  2623.7  2623.7  2638.3]
```

Table - w_x h_x (kip-ft) [7.2]

$$wxhx = h_x w_x$$

lvl = roof	**lvl = 5**	**lvl = 4**	**lvl = 3**	**lvl = 2**
156871	131187	99702	68217	36936

wxhx_sum | sum of w_x h_x (kips) [7.3]

$$wxhx_sum = 492{,}914$$

Vertical distribution coefficient

$$C_{vx} \leq \frac{h_x^k \cdot w_x}{\sum_{i=1}^{n} h_x^k \cdot w_x}$$

Table - Percent vertical load distribution - C_vx [7.4]

$$C_dist = \frac{100 wxhx}{wxhx_{sum}}$$

lvl = roof	**lvl = 5**	**lvl = 4**	**lvl = 3**	**lvl = 2**
32	27	20	14	7

Table - Story Force F_x (kips) [7.5]

$$F_x = \frac{C_{dist} V_1}{100}$$

lvl = roof	**lvl = 5**	**lvl = 4**	**lvl = 3**	**lvl = 2**
968	810	615	421	228

Table - Story Shears (kips) [7.6]

$$V_x = [F_x[0], sum(F_x[0:2]), sum(F_x[0:3]), sum(F_x[0:4]), sum(F_x[0:5])]$$

lvl = roof	**lvl = 5**	**lvl = 4**	**lvl = 3**	**lvl = 2**
968	1778	2394	2815	3043

Vbase | base shear (kips) [7.7]

$$Vbase = 3{,}043\ kips$$

[8] Horizontal distribution of shear [0403]

IBC 12.8.4.2 requires inclusion of an accidental torsional eccentricity, e, equal to 5 percent of the building dimension perpendicular to the direction of force. It is assumed that the four frames in the transverse direction are each 25 percent stiffer than the six in the longitudinal direction because they are more heavily loaded.

```
e/w c.m. distance to frame          | d_ew = 105 ft
n/s c.m. distance to frame          | d_ns = 75 ft
accidental ecc  e/w                 | e_ew = 0.1*d_ew
accidental ecc  n/s                 | e_ns = 0.1*d_ns
rel stiffness-trans frames          | R_i = 1.25
```

[9] Transverse frames-shear plus torsion [0403]

transR_i | rel. torsional resistance - 4 transverse frames [9.1]

$$R_i \cdot d_{ew}^2$$

$$1 \cdot 105 ft^2$$

$$transR_i = 13{,}781\ ft2$$

longR_i | rel. torsional resistance - 6 longitudinal frames [9.2]

$$d_{ns}^2$$

$$75 ft^2$$

$$longR_i = 5{,}625\ ft2$$

SumR_i | sum of rel. torsion resistances [9.3]

$$SumR_i = 88{,}875\ ft2$$

Ttrans_y | Percent torsion force at single transverse frame [9.4]

$$\frac{100 \cdot e_{ew} \cdot transR_i}{SumR_i \cdot d_{ew}}$$

$$\frac{100 \cdot 10.50 ft \cdot 13781.25 ft2}{88875.00 ft2 \cdot 105.00 ft}$$

$$Ttrans_y = 2$$

Tlong_y | Percent torsion force at single transverse frame [9.5]

$$\frac{100 \cdot e_{ew} \cdot longR_i}{SumR_i \cdot d_{ns}}$$

$$\frac{100 \cdot 10.50 ft \cdot 5625.00 ft2}{88875.00 ft2 \cdot 75.00 ft}$$

$$Tlong_y = 1$$

Torsion distribution relation

$$Viy \leq \left(\frac{Ri \cdot di \cdot e_{real}}{\sum_{i=0}^{n} Ri \cdot di^2} + \frac{Riy}{\sum_{i=0}^{n} Riy} + \left| \frac{Ri \cdot di \cdot e_{acc}}{\sum_{i=0}^{n} Ri \cdot di^2} \right| \right) \cdot \sum_{i=0}^{n} Vy$$

Table - Forces at each level at frame BF-2 (kips) [9.6]

$$F_x bf2 = F_x \left(\frac{Ttrans_y}{100} + 0.25 \right)$$

lvl = roof	**lvl = 5**	**lvl = 4**	**lvl = 3**	**lvl = 2**
257	215	163	112	61

Table - Story shears at each level at frame BF-2 (kips) [9.7]

$$V_x bf2 = V_x \left(\frac{Ttrans_y}{100} + 0.25 \right)$$

lvl = roof	**lvl = 5**	**lvl = 4**	**lvl = 3**	**lvl = 2**
257	472	635	747	808

[10] Combined forces (IBC 12.4.2) [0403]

For convenience, the seismic design load combinations can be reformulated to combine the vertical component of the seismic motion with the dead load. Also, as the overstrength factor is not considered in combination with the redundancy factor (and the results of the analysis include the redundancy factor), it is sometimes convenient to recalculate the overstrength factor to account for this difference (if rho > 1.0).

E2 | combined effects - redundancy [10.1]

$$-0.28 \cdot DL + Q_E \cdot \rho$$

$$-0.28 \cdot 3kip + 1kip \cdot 1$$

$$E2 = 0\ kip$$

E1_m | combined effects - overstrength [10.2]

$$0.28 \cdot DL + \Omega_0 \cdot Q_E$$

$$0.28 \cdot 3kip + 2 \cdot 1kip$$

$$E1_m = 3\ kip$$

E2_m | combined effects - overstrength [10.3]

$$-0.28 \cdot DL + \Omega_0 \cdot Q_E$$

$$-0.28 \cdot 3kip + 2 \cdot 1kip$$

$$E2_m = 1\ kip$$

Basic Seismic Load Combinations:

E1_B | 1.2D+ 1.0E + 0.5L (Eq 16-5) [10.4]

$$1.48 \cdot DL + Q_E \cdot \rho + f_{1L}$$

$$1.48 \cdot 3kip + 1kip \cdot 1 + 2kip$$

$$E1_B = 7\ kip$$

E2_B | 0.9 D + 1.0 E (Eq 16-7) [10.5]

$$0.62 \cdot DL + Q_E \cdot \rho$$

$$0.62 \cdot 3kip + 1kip \cdot 1$$

$$E2_B = 3\ kip$$

Special Seismic Load Combinations (Amplified Seismic Load)

E1_M | 1.2D + 1.0Em + 0.5L (Eq 16-22) [10.6]

$$1.48 \cdot DL + 2.0 \cdot Q_E + f_{1L}$$

$$1.48 \cdot 3kip + 2.0 \cdot 1kip + 2kip$$

$$E1_M = 8\ kip$$

E2_M | 0.9D + 1.0Em (Eq 16-23) [10.7]

$$0.62 \cdot DL + 2.0 \cdot Q_E$$

$$0.62 \cdot 3kip + 2.0 \cdot 1kip$$

$$E2_M = 4\ kip$$

The formulae that include the redundancy factor are used for brace design, and those that do not include it are used for column and beam design. The forces that the framing members are likely to see are greater for a frame with larger braces than for one with smaller ones, and the same overstrength factor should be applied to the two frames.

It is apparent from the reformulations that the redundancy factor actually decreases the effective overstrength required of these members and makes unfavorable yield modes more likely. A redundancy factor of 1.3 decreases the effective overstrength factor to 1.54, an extremely low value for a braced frame; use of such a low value in determining column design forces may not be sufficient to preclude column buckling. Designers should not lose sight of the purpose of the required over-strength, even when code provisions permit its reduction.

[end of calc]

[1] Frame analysis [0404]

Small eccentricities occur at the connections to facilitate detailing. These eccentricities result in flexural forces in the framing members, as well as larger drifts than those for frames without eccentricities. It is therefore necessary to include the eccentricities in the frame model. Using the frame forces from Table 1 A4, the frame shown in Figure 1A-5 is analyzed. Columns are modeled as fixed at the base and braces as pinned at each end. Although in-plane rotation of the gusseted beam-to-column connection may induce some flexural forces in the brace, these are small at the elastic drift level. Beams are modeled as having fixed connections at their gusseted connections to the columns; these details include beam-to-column joint flange welds. (AISC 341, 7.2)

[2] Story drifts [0404]

In this design example a second-order analysis has been performed, and the B2 amplification factor is not required. The displacements from the model are converted to design story drifts and compared to the allowable drift from ASCE/SEI 7-05 Table 12.2-1. The story drifts are calculated from the displacements, and divided by the reliability/redundancy factor, rho and I. The drifts from the elastic analysis are multiplied by the displacement amplification factor, Cd. The design story drift for each level of this frame is less than the allowable drift. Because the building design is symmetrical and has rigid diaphragms, the average story drift will not exceed the drift of this frame. Thus, the building will meet the drift limitations of Table 12.12-1.

```
response mod coef (T 12.2-1)       | R_1 = 6.0
```

```
. story height (inch)              | h_sx =
```

. [144 144 144 144 168]

```
. model displacement (inch)        | disp_xe =
```

. [2.2 1.7 1.2 0.8 0.3]

```
. model drift (inch)               | Delta_xe =
```

. [0.5 0.5 0.5 0.4 0.3]

C_d | displacement amplification factor [2.1]

$$0.7 \cdot R_1$$

$$0.7 \cdot 6.00$$

$$C_d = 4.20$$

design story drift (inch) [2.2]

$$delta_x = C_d \Delta_{xe}$$

stry = roof	**stry = 4**	**stry = 3**	**stry = 2**	**stry = 1**
2.1	1.9	2.0	1.8	1.5

allowable story drift (inch) [2.3]

$$delta_A x = 0.02 h_{sx}$$

stry = roof	**stry = 4**	**stry = 3**	**stry = 2**	**stry = 1**
2.9	2.9	2.9	2.9	3.4

[end of calc]

[1] Brace design [0405]

For this example the fourth floor brace will be designed. From the analysis, the earthquake force, rhoP_E, on the brace is known.

```
dl compression                      | PD = 8000.00 kips
ll compression                      | PL = 6.00 kips
eq tension                          | Q_PEt = -328.00 kips
eq compression                      | Q_PEc = 328.00 kips
non-assembly,-parking < 100 psf     | f1 = 0.5
redundancy factor                   | rho = 1.0
pi                                  | pi = 3.14
```

P_dc | max compression 1.2 D + 1.0 E + 0.5L (Eq 16-5) [1.1]

$$1.48 \cdot PD + PL \cdot f_1 + Q_{PEc} \cdot \rho$$

$$1.48 \cdot 8000.00 kips + 6.00 kips \cdot f_1 + 328.00 kips \cdot 1.00$$

$$P_d c = 12{,}171.0\ kips$$

P_dt | max tension 0.9 D + 1.0 (Eq 6-7) [1.2]

$$0.62 \cdot PD + \frac{Q_{PEt}}{f_1} \cdot \rho$$

$$0.62 \cdot 8000.00 kips + \frac{-328.00 kips}{f_1} \cdot 1.00$$

$$P_d t = 4{,}304.0\ kips$$

[2] Brace compression strength [0405]

The distance from work-point to work-point is 17.9 feet. The ductile detailing requirements for SCBF result in the provision of a hinge zone in the connection; it is assumed here that the hinge zone will be at least 9 inches from the work point at each end, resulting in a reduction of the brace length of 18 inches. The brace length = 197 inches. HSS 9.625x0.500 will be used. Section properties are from the 13th Edition LRFD Manual.

```
boundary condition coefficient      | K_1 = 1.0
unbraced lenght                     | l_1 = 197.0 in
radius of gyration                  | r_1 = 3.2 in
elastic modulus                     | E_1 = 29000.0 ksi
yield stress                        | F_y = 42.0 ksi
cross section area                  | A_g = 13.4 in2
HSS section depth                   | D_e = 9.6 in
HSS section thickness               | t_e = 0.5 in
```

Kl_r | Slenderness ratio [2.1]

$$\frac{K_1}{r_1} \cdot l_1$$

$$\frac{1.00}{3.24in} \cdot 197.00in$$

$$Kl_r = 61$$

F_e | Buckling stress [2.2]

$$\frac{\pi^2 \cdot E_1 \cdot r_1^2}{K_1^2 \cdot l_1^2}$$

$$\frac{\pi^2 \cdot 29000.00ksi \cdot 3.24in^2}{1.00^2 \cdot 197.00in^2}$$

$$F_e = 77.3\ ksi$$

F_cr | Critical buckling stress (AISC 360, E3-2) [2.3]

$$0.658^{\frac{F_y}{F_e}} \cdot F_y$$

$$0.658^{\frac{42.00ksi}{77.34ksi}} \cdot 42.00ksi$$

$$F_cr = 33.5\ ksi$$

phiP_c | Compression strength (AISC 360 E3-1) [2.4]

$$0.9 \cdot A_g \cdot F_{cr}$$

$$0.9 \cdot 13.40in2 \cdot 33.46ksi$$

$$phiP_c = 403.5\ kips$$

Check brace slenderness limit (AISC 341-13.2a) [2.5]

$$\frac{K_1}{r_1} \cdot l_1 \leq 4 \cdot \sqrt{\frac{E_1}{F_y}}$$

$$\frac{1.00}{3.24in} \cdot 197.00in \leq 4 \cdot \sqrt{\frac{29000.00ksi}{42.00ksi}}$$

$$\mathbf{60.80 <= 105.07 - ok}$$

Note that under the exception, braces with Kl/r < 200 are allowed if capacity design is used for columns

Check brace width-to-thickness (AISC 341-13.2d, T 1.8.1) [2.6]

$$\frac{D_e}{t_e} \leq \frac{0.44}{F_y} \cdot E_1$$

$$\frac{9.62in}{0.50in} \leq \frac{0.44}{42.00ksi} \cdot 29000.00ksi$$

$$\mathbf{19.25 <= 303.81 - ok}$$

[3] Brace tension strength [0405]

phiP_t | Design tension strength (AISC 360, D1-1) [3.1]

$$0.9 \cdot A_g \cdot F_y$$

$$0.9 \cdot 13.40 in2 \cdot 42.00 ksi$$

$$phiP_t = 506.5\ kips$$

```
tension demand on brace                | P_u = 350.0 kips
```

Check tension strength [3.2]

$$\frac{P_u}{phiP_t} \leq 1.0$$

$$\frac{350.00 kips}{506.52 kips} \leq 1.0$$

$$0.69 \mathbf{<=} 1.00 \boldsymbol{-ok}$$

Equation D1-2, which applies to fracture of the net section, is checked in the design of the connection.

[4] Distribution of force between braces [0405]

AISC 341-13.2c does not permit a major unbalance between braces working in compression and those working in tension to resist lateral loads. The frames in this structure are symmetrical and no single-diagonal frames are used. An elastic analysis shows that the seismic force resisted by compression is identical to that resisted by tension and the frame complies with the requirement. The tension strength of braces is similar in each direction, so that even after degradation of the compression strength of braces there is no significant tendency to accumulate drift in one direction.

[end of calc]

[1] Example 5 - Structure Dynamics [0501]

Example E12-1 from Cough and Penzien *Dynamics of Structures* is used to illustrate application of the *numpy* linear algebra libraries in **on-c-e**. See *numpy* documentation for futher details.

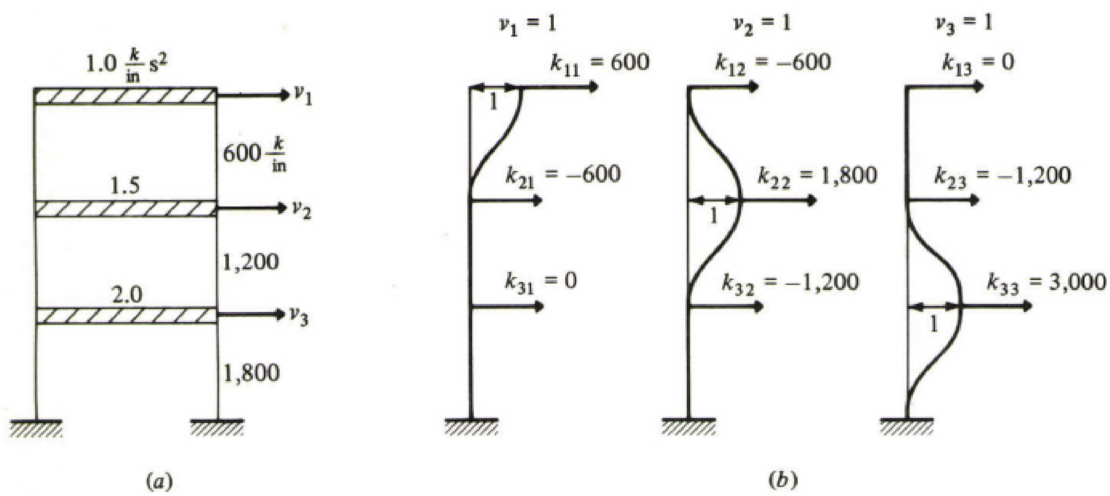

FIGURE E12-1
Frame used in example of vibration analysis: (*a*) structural system; (*b*) stiffness influence coefficients.

Figure 1.1: From Clough and Penzien - page 178

[2] Define mass and stiffness [0501]

units: kips, inches

```
. mass                                  | m_1 =
```

. [[1. 0. 0.] . [0. 1.5 0.] . [0. 0. 2.]]

```
. stiffness                             | k_1 =
```

. [[600. -600. 0.] . [-600. 1800. -1200.] . [0. -1200. 3000.]]

flex | flexibility matrix [2.1]

```
LA.inv(k_1)

. flex =
. [[ 0.0031  0.0014  0.0006]
.  [ 0.0014  0.0014  0.0006]
.  [ 0.0006  0.0006  0.0006]]
```

dyna | dynamic matrix [2.2]

$$\text{inner}\,(flex, m_1)$$

```
. dyna =
. [[ 0.0031  0.0021  0.0011]
.  [ 0.0014  0.0021  0.0011]
.  [ 0.0006  0.0008  0.0011]]
```

[3] Eigenvalue analysis [0501]

eigenvals | eigenvalues [3.1]

```
LA.eig(dyna)[0]

. eigenvals =
. [ 0.0047  0.001   0.0005]
```

omega_1 | natural frequency (secs) [3.2]

$$eigenvals^{-0.5}$$

```
. omega_1 =
. [ 14.5217  31.0477  46.0995]
```

evects_0 | eigenvectors [3.3]

```
LA.eig(dyna)[1]

. evects_0 =
. [[-0.813 -0.739  0.273]
.  [-0.527  0.449 -0.694]
.  [-0.246  0.502  0.666]]
```

normalized eigenvectors [3.4]

return variable: evectors_1
function call: norm_evects(evects_0, eigenvals)
function doc string:

```
normalize and scale eigenvectors
```

function returned:

```
. [[ 1.     0.649  0.302]
.  [ 1.    -0.607 -0.679]
.  [ 1.    -2.542  2.44 ]]
```

[4] Plot mode shapes [0501]

plot mode shapes [4.1]

return variable: plot_1
function call: plot_modes(evectors_1)
function doc string:

```
plot three mode shapes
```

function returned:

```
mode shapes plotted
```

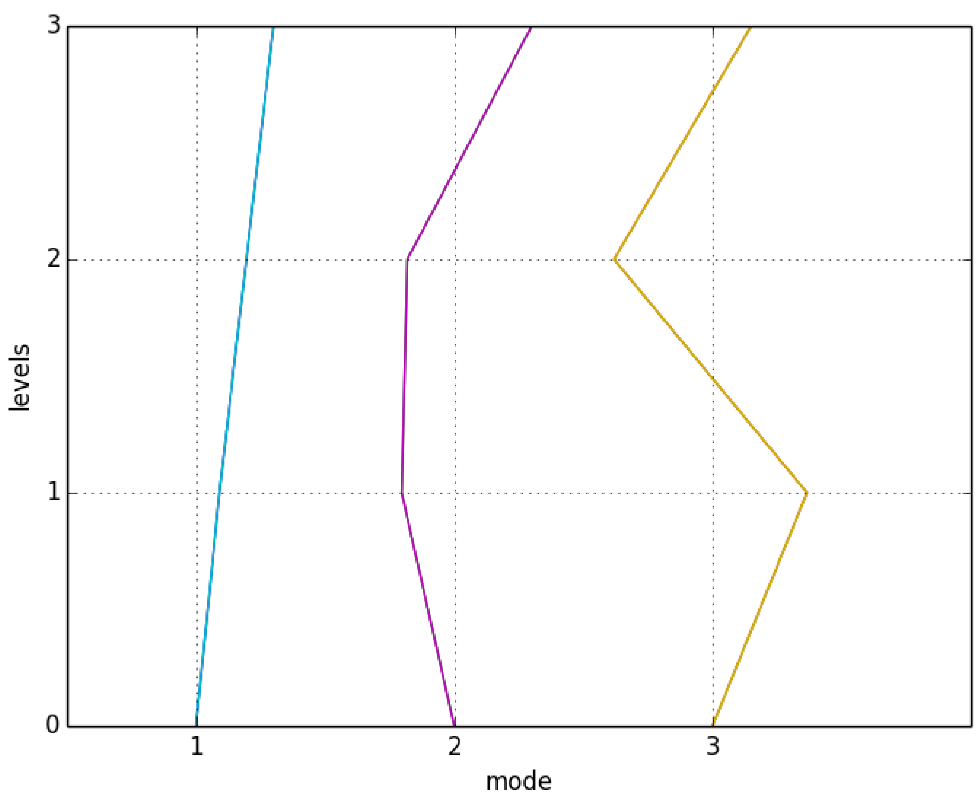

Figure 4.1: Mode Shapes

[end of calc]

[1] Example 0601 - Analyze 3-Bar truss using OpenSees [0601]

OpenSees Primer Example 1.1
Displacement results for two different geometries
Units: kips, in, sec

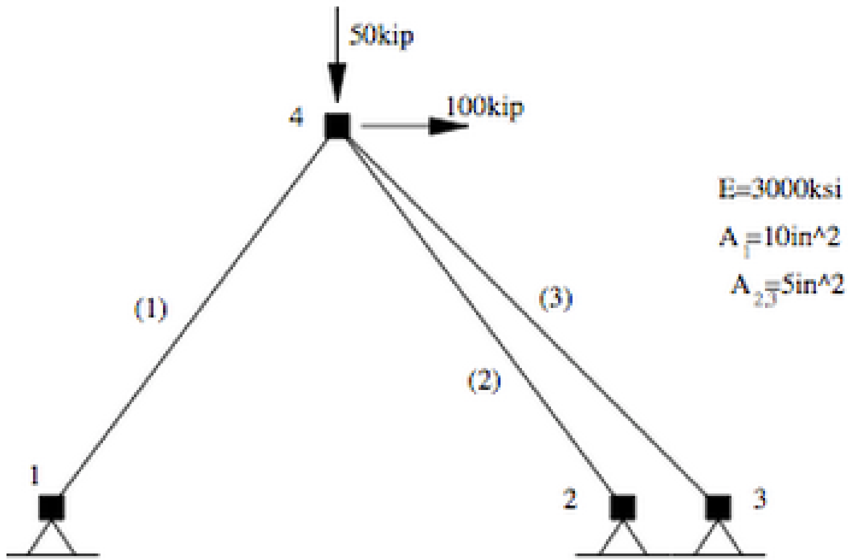

Figure 1.1: Node and element numbers

[2] set up opensees geometry for run 1 [0601]

change node 4 location using a string template
run OpenSees model

```
node 4 x (in)                    | n4x1 = 70.0
node 4 y (in)                    | n4y1 = 45.0
```

c:/opensees/truss.tcl | edit file

```
file: c:/opensees/truss.tcl
[line #]      [replacement line]
```

plot OpenSees geometry - run 1 [2.1]

return variable: plot 1
function call: osplot(”c:/opensees/trusscopy1.tcl”)
function doc string:

```
plot geometry for OpenSees tcl file

function plots geometry using matplotlib

Args:
tcl file (file)

methods called:
ndict - generates node dictionary
edict - generates element dictionary
plotgeo - generates plot and writes to file
```

function returned:

```
plot file 'trusscopy1fig.png' written
```

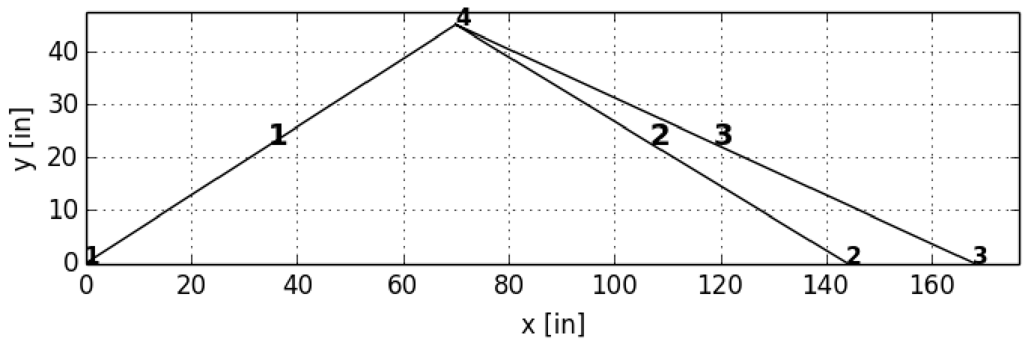

Figure 2.1: Truss geometry - run 1

[3] set up opensees geometry for run 2 [0601]

change node 4 location using a static string - no variable substition
run OpenSees model

c:/opensees/truss.tcl | edit file

```
file: c:/opensees/truss.tcl
[line #]       [replacement line]
```

plot OpenSees geometry - run 2 [3.1]

return variable: plot_2
function call: osplot("c:/opensees/trusscopy2.tcl")
function doc string:

```
plot geometry for OpenSees tcl file

function plots geometry using matplotlib

Args:
tcl file (file)

methods called:
ndict - generates node dictionary
edict - generates element dictionary
plotgeo - generates plot and writes to file
```

function returned:

```
plot file 'trusscopy2fig.png' written
```

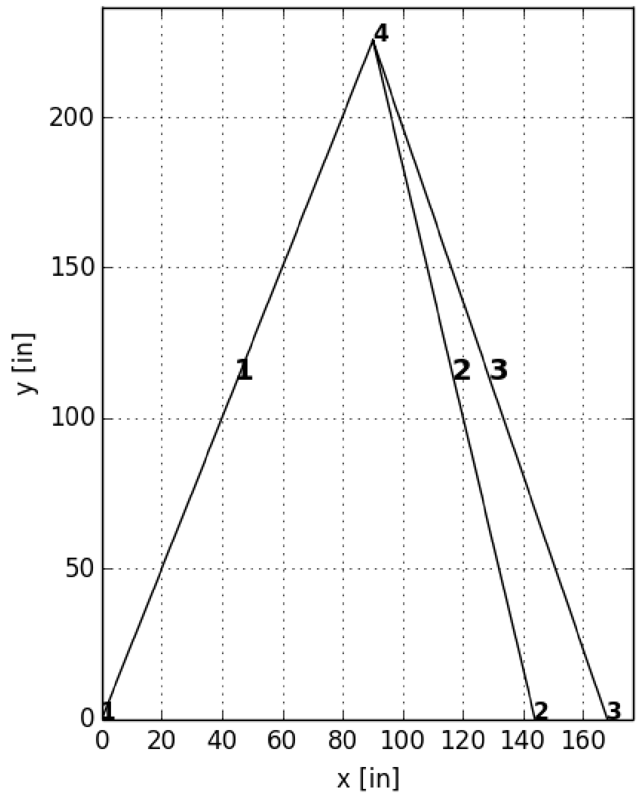

Figure 3.1: Truss geometry - run 2

[4] run models 1 and 2 and write output tables [0601]

deltaxy1 | read data

file: c:/opensees/example_disp1.out
[1. 0.22 -0.33] … [1. 0.22 -0.33]

forceaxial1 | read data

file: c:/opensees/example_force1.out
[4.19 -62.59 -47.31] … [4.19 -62.59 -47.31]

deltaxy2 | read data

file: c:/opensees/example_disp2.out
[1. 3.79 -0.37] … [1. 3.79 -0.37]

forceaxial2 | read data

file: c:/opensees/example_force2.out
[131.57 -80.16 -99.68] … [131.57 -80.16 -99.68]

Table of Displacements-Node 4 (in) [4.1]

$$delta = \left[\left[deltaxy_1\right], \quad \left[deltaxy_2\right]\right]$$

run	= load no.	= x	= y
1	1.00	0.22	-0.33
2	1.00	3.79	-0.37

Table of Axial forces (kips) [4.2]

$$forces = \left[\left[forceaxial_1\right], \quad \left[forceaxial_2\right]\right]$$

run	ele no. = 1	ele no. = 2	ele no. = 3
1	4.19	-62.59	-47.31
2	131.57	-80.16	-99.68

[end of calc]

[1] Example 7 - Stiffness Method [0701]

Examples 3.4 and 3.5 from McQuire and Gallagher *Matrix Structural Analysis* demonstrate applications of the numpy linear algebra library and unit manipulations.

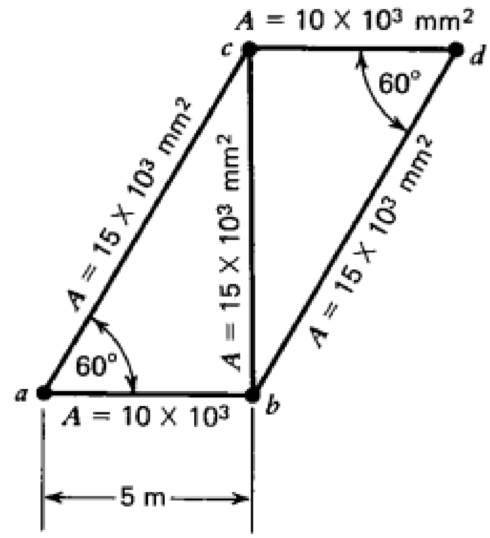

Figure 1.1: Frame geometry

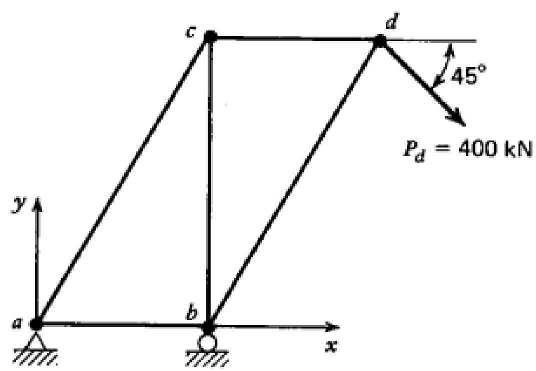

Figure 1.2: Loads

[2] Define geometry and material properties [0701]

units: kN, mm

Element Properties

```
elastic modulus (kN/mm**2)          | e_1 = 200.0
cross section 1 (mm**2)             | a_1 = 10000
cross section 2 (mm**2)             | a_2 = 15000
```

Node coordinates [node number, x, y]

```
. node 0                            | n_0 =
```

[0, 0.0, 0.0]

```
. node 1                            | n_1 =
```

[1, 5000.0, 0.0]

```
. node 2                            | n_2 =
```

[2, 5000.0, 8660.0]

```
. node 3                            | n_3 =
```

[3, 10000.0, 8660.0]

nodes | node coordinates [2.1]

```
. nodes =
. [[0, 0.0, 0.0],
.  [1, 5000.0, 0.0],
.  [2, 5000.0, 8660.0],
.  [3, 10000.0, 8660.0]]
```

Element Connectivity [element number, node1, node2, area, modulus]

elements | element connectivity [2.7]

```
. elements =
. [[1, 0, 1, 10000, 200.0],
.  [2, 1, 3, 15000, 200.0],
.  [3, 2, 3, 10000, 200.0],
.  [4, 0, 2, 15000, 200.0],
.  [5, 1, 2, 15000, 200.0]]
```

Forces and reactions

```
. releases 0 or 1 [node, x, y]      | react =
```

[[0, 1, 1], [1, 0, 1]]

```
. applied forces  [node, x, y]      | forces =
```

[[3, 283.0, -283.0]]

[3] Analyze truss [0701]

Intermediate results may be viewed in standard out by uncommenting print statements in the file *func_stiff.py*.

Direct stiffness analysis [3.1]

return variable: result_1
function call: direct_stiff(nodes, elements, react, forces)
function doc string:

```
executes stiffness functions

  1. structure dictionaries   : node_dict()
  2. global stiffness         : assem_global()
  3. displacements and reacts : displace()
  4. element forces           : axial_force()
  5. tables and plots         : disp_tables(), force_tables()
```

function returned:

```
node        x displacement
 0                 0.0
 1    -0.408487297921
 2      9.81709263995
 3      10.9330799379
node        y displacement
 0                 0.0
 1                 0.0
 2     -2.23184365333
 3     -7.80601753585
node           reaction
 0    x dir      -283.0
 0    x dir    -773.156
 1    x dir    1056.156
element        axial force
 1     -163.394919169
 2     -326.782648882
 3      446.394919169
 4      892.770196745
 5           -773.156
```

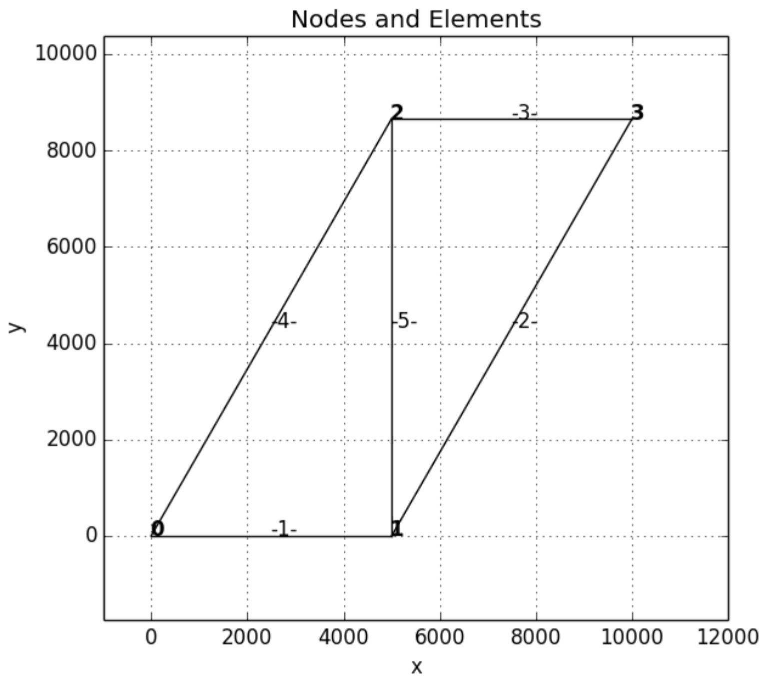

Figure 3.1: Examples 3.4 and 3.5 from McQuire and Gallagher

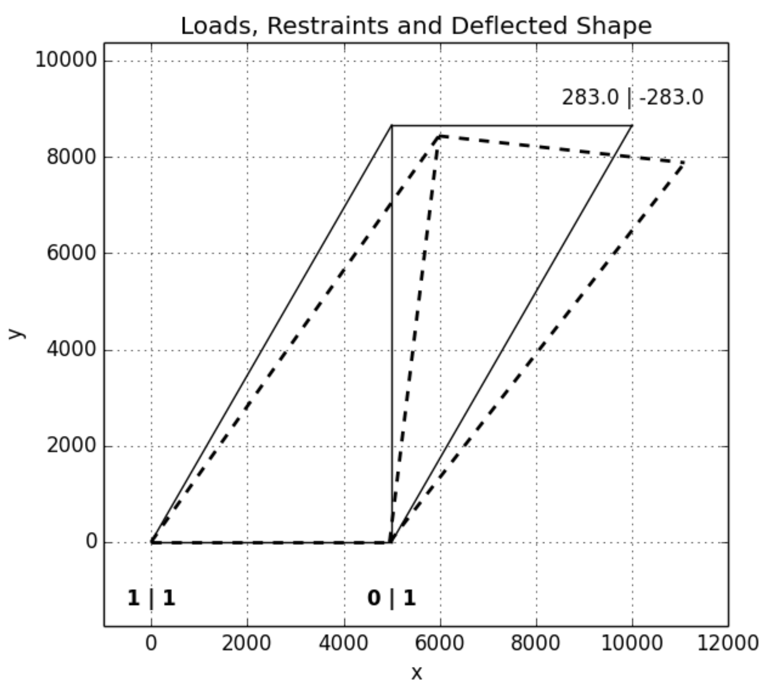

Figure 3.2: Examples 3.4 and 3.5 from McQuire and Gallagher

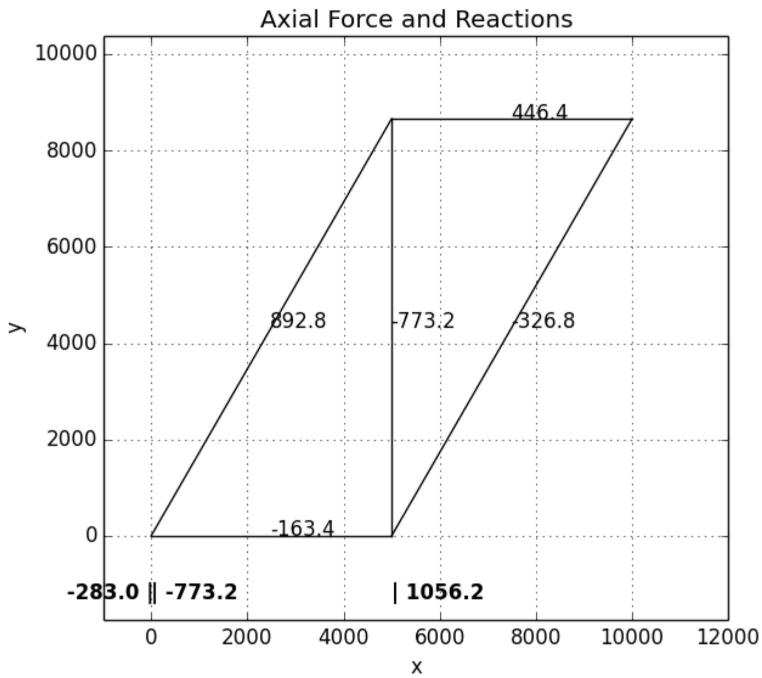

Figure 3.3: Examples 3.4 and 3.5 from McQuire and Gallagher

[end of calc]

```
9/11/2014 7:13:51 PM

==========================================================================================
[1] Example 0001 - a simple calculation                                           [0001]
==========================================================================================
    *Numbers are fun whatever you do.*
    **First comes one and then comes two!**


    number of brothers                    | bros = 2
    number of sisters                     | siss = 1
    height of tall brother                | bro_h = 6.0 ft
    height of sister                      | sis_h = 5.8 ft

How many siblings do I have?

┌---------------------------------------------------------------------------------------┐
                                        sum | add up number of brothers and sisters [1.1]

bros + siss


2 + 1

                                                                                  sum = 3
└---------------------------------------------------------------------------------------┘


What is the height difference between the tall brother and sister in inches?

┌---------------------------------------------------------------------------------------┐
                                                             dif | subtract heights [1.2]

bro_h - sis_h


6.00 ft - 5.80 ft

                                                                             dif = 2.4 in
└---------------------------------------------------------------------------------------┘



[end of calc]
```

```
11/24/2014 2:44:41 AM     onceutf version: 0.4.6

==========================================================================================
[1] Example 0101 - basic template                                                   [0101]
==========================================================================================
    This model calculates the mid-span beam moment
    under uniformly distributed (UDL) floor loads.

Figure 1. Simply supported beam <file: beam.png >


    operation examples:
        - [s] section
        - [y] sympy and LaTeX symbolic expresssions
        - [t] term
        - [e] equation
        - [#-] format and file operations

    reST markup examples:
        - lists, **bold**, *italic*
        - tables
        - raw latex


    **Notation:** (LaTeX block is not processed for UTF calcs)


raw latex directive < directive not shown in UTF calc >

==========================================================================================
[2] Beam Loads and geometry                                                         [0101]
==========================================================================================
    Dead and live load contributions to beam UDL

    **ASCE 7 - 05 Load Effects**

    =============   ==============================================
    Equation No.    Load Combination
    =============   ==============================================
    16-1            1.4(D+F)
    16-2            1.2(D+F+T) + 1.6(L+H) + 0.5(Lr or S or R)
    16-3            1.2(D+F+T) + 1.6(Lr or S or R) + (f1L or 0.8W)
    =============   ==============================================

    **Dead loads**
    joists                              | D_1 = 3.8 psf
    plywood                             | D_2 = 2.1 psf
    partitions                          | D_3 = 10.0 psf
    fixed machinery                     | D_4 = 0.5 kips/ft

    **Live loads**
    ASCE7-05                            | L_1 = 40 psf

    **Beam tributary width and span**
    distance between beams              | w_1 = 2 ft
    beam span                           | l_1 = 14 ft

==========================================================================================
[3] Maximum bending moment                                                          [0101]
==========================================================================================

┌----------------------------------------------------------------------------------------┐
                                                DL_1 | Total UDL factored dead load [3.1]
```

$$1.2\cdot D_4 + 1.2\cdot w_1\cdot(D_1 + D_2 + D_3)$$

```
                                                                      DL_1 = 0.64 kips/ft
└----------------------------------------------------------------------------------------┘
```

┌--┐

LL_1 | Total UDL factored live load [3.2]

$$1.6 \cdot L_1 \cdot w_1$$

LL_1 = 0.13 kips/ft

└--┘

┌--┐

omega_1 | factored UDL [3.3]

$$DL_1 + LL_1$$

omega_1 = 0.77 kips/ft

└--┘

┌--┐

M_1 | Bending moment at mid-span [3.4]

$$\frac{l_1^{2} \cdot \omega_1}{8}$$

$$\frac{14.00\ \mathrm{ft}^{2} \cdot 0.77\ \mathrm{kips/ft}}{8}$$

M_1 = 18.8 ft.kip

└--┘

==

[4] Symbolic rendering using sympy or LaTeX [0101]

==

Equation rendered from **SymPy**

$$\mathrm{sigma} = \frac{M \cdot z}{I}$$

equation <file: latex[4.1].png>

Equation rendered from **LaTeX** (expresssion copied from Wikipedia HTML source)

equation <file: latex[4.2].png>

[end of calc]

```
9/11/2014 7:13:12 PM
==========================================================================================
[1] Example 0201 - Isolation Bearing Properties                                    [0201]
==========================================================================================
    Calculate shape factors, compression stiffness and
    buckling loads for a rubber laminated seismic isolation bearing

    **Measured Properties**
    bearing diameter                     | phi = 36 in
    rubber layer thickness               | t_layer = 0.38 in
    initial shear modulus                | G_i = 100.0 psi
    shear modulus                        | G = 50.0 psi
    total rubber height                  | R_t = 11.0 in
    bearing height                       | h_b = 19 in
    bulk modulus                         | K_1 = 250 ksi
    measured compression stiffness       | K_cm = 11800 kips/in
    constant                             | pi = 3.1418

    **Loads**
    applied load                         | P = 3000 kips
    factored load demand                 | P_u = 5000 kips

Figure 1. Bearing Geometry <file: bearing_geo.png >


==========================================================================================
[2] Basic Calculated Properties                                                    [0201]
==========================================================================================

┌----------------------------------------------------------------------------------------┐
                                                              a_1 | Bearing area [2.1]
```

$$\frac{\pi\cdot\varphi^{2}}{4}$$

$$\frac{pi\cdot 36.00\ in^{2}}{4}$$

```
                                                                  a_1 = 1,017.94 in2
└----------------------------------------------------------------------------------------┘


┌----------------------------------------------------------------------------------------┐
                                                        S_1 | Axial shape factor [2.2]
```

$$\frac{\varphi}{4\cdot t\_layer}$$

$$\frac{36.00\ in}{4\cdot 0.38\ in}$$

```
                                                                         S_1 = 23.7
└----------------------------------------------------------------------------------------┘


┌----------------------------------------------------------------------------------------┐
                                                     n | Number of rubber layers [2.3]
```

```
  R_t
───────
t_layer


11.00 in
────────
0.38 in

                                                                                   n = 29
└---------------------------------------------------------------------------------------┘


┌---------------------------------------------------------------------------------------┐
                                                          I_1 | Moment of inertia [2.4]

        4
0.04875·φ


              4
0.04875·36.0 in

                                                                      I_1 = 81,881 in4
└---------------------------------------------------------------------------------------┘


==========================================================================================
[3] Compression Stiffness                                                          [0201]
==========================================================================================

┌---------------------------------------------------------------------------------------┐
                                                              GK | Stiffness term [3.1]

12·Gᵢ
─────
  K₁

12·100.00 psi
─────────────
  250.00 ksi
                                                                           GK = 0.0048
└---------------------------------------------------------------------------------------┘


┌---------------------------------------------------------------------------------------┐
                                            E_c | Compression modulus after Kelly [3.2]
     ⎛     -0.5             ⎞
     ⎜  GK              1   ⎟
K₁·⎜- ────── + 1 + ──────────⎟
     ⎜    S₁                 2⎟
     ⎝              4·GK·S₁ ⎠

                /           -0.5                    \
                |  0.005                   1        |
250.000 ksi·|- ────────── + 1 + ──────────-------|
                |   23.684                          2|
                \                   4·0.005 ·23.684  /

                                                                      E_c = 120.9 ksi
└---------------------------------------------------------------------------------------┘
```

kv_k | Compression stiffness after Kelly [3.3]

$$\frac{E\_c \cdot a_1}{R\_t}$$

$$\frac{120.86\ \text{ksi} \cdot 1017.94\ \text{in2}}{11.00\ \text{in}}$$

kv_k = 11,184.1 kips/in

kv_R | Compresssion stiffness after Robinson [3.4]

$$\frac{6 \cdot G_i \cdot K_1 \cdot S_1^{2} \cdot a_1}{R\_t \cdot \left(6 \cdot G_i \cdot S_1^{2} + K_1\right)}$$

$$\frac{6 \cdot 100.00\ \text{psi} \cdot 250.00\ \text{ksi} \cdot 23.68^{2} \cdot 1017.94\ \text{in2}}{11.00\ \text{in} \cdot \left(6 \cdot 100.00\ \text{psi} \cdot 23.68^{2} + 250.00\ \text{ksi}\right)}$$

kv_R = 13,274.7 kips/in

===

[4] Buckling Load - Pe [0201]

===

P_e | Buckling capacity after Kelly [4.1]

$$\frac{\pi^{2} \cdot E\_c \cdot I_1}{3 \cdot R\_t \cdot h\_b}$$

$$\frac{\text{pi}^{2} \cdot 120.86\ \text{ksi} \cdot 81881.28\ \text{in4}}{3 \cdot 11.00\ \text{in} \cdot 19.00\ \text{in}}$$

P_e = 155,791.2 kips

dc ratio check [4.2]

$$\frac{P\_u}{P\_e} \le 1.0$$

```
 5000.00 kips
────────────── <= 1.0
155791.17 kips

                                                                         0.03 <= 1.00   ok
└-----------------------------------------------------------------------------------------┘
```


[end of calc]

9/11/2014 7:42:31 PM

[1] Example 0301 - beam deflection [0301]

Table of tube bending deflections as a function of span and depth when analyzed as a simply supported classical beam with uniform distributed load.

Figure 1. Tube geometry <file: tube.png >

[2] Materials and Geometry [0301]

Calculate tube moment of inertia for an initial condition and a range of depths. Neglect radii at corners.

6061-T6 Aluminum

elastic modulus | $E_0 = 10000$ ksi

Tube Dimensions

tube width | $b_0 = 5$ in

wall thickness | $t_0 = 0.125$ in

Design Load and Span

trial clear span | span = 6 ft

trial tube depth | $d_0 = 9$ in

uniform distributed load | $omega_1 = 0.1$ kip/in

I_9 | Moment of inertia [2.1]

$$\frac{b_0 \cdot d_0^3}{12} - \frac{(b_0 - 2 \cdot t_0) \cdot (d_0 - 2 \cdot t_0)^3}{12}$$

I_9 = 38.573 in4

Table : I_x [in4] : depth [in] [2.2]

$$\frac{b_0 \cdot d\_n^3}{12} - \frac{(b_0 - 2 \cdot t_0) \cdot (d\_n - 2 \cdot t_0)^3}{12}$$

depth = 9	depth = 10	depth = 11	depth = 12	depth = 13
38.573	49.785	62.841	77.866	94.984

[3] Evaluate Deflections [0301]

Evaluate beam deflections for an initial I and span and then a range of values.

Check an initial case

initial I | $I_1 = 35.000$ in4

beam span | span_1 = 10.000 ft

delta_1 | beam deflection under uniform load [3.1]

$$\frac{5 \cdot \omega_1 \cdot span_1^4}{384 \cdot E_0 \cdot I_1}$$

$$\frac{5 \cdot 0.100 \text{ kip/in} \cdot 10.000 \text{ ft}^4}{384 \cdot 10000.000 \text{ ksi} \cdot 35.000 \text{ in4}}$$

delta_1 = 0.771 in

Table : Deflections [in] (span [in], I [in4]) [3.2]

$$\frac{5 \cdot \omega_1 \cdot span_2^4}{384 \cdot E_0 \cdot I_2}$$

span	I = 33	I = 43	I = 53	I = 63	I = 73
72	0.106	0.081	0.066	0.056	0.048
96	0.335	0.257	0.209	0.176	0.151
120	0.818	0.628	0.509	0.429	0.370
144	1.697	1.302	1.056	0.889	0.767
168	3.143	2.412	1.957	1.646	1.421

[end of calc]

```
9/11/2014 7:42:16 PM
==========================================================================================
[1] Example 0401 - Building and Load Information                                   [0401]
==========================================================================================
Design Example 1A - Special Concentrically Braced Frame
2006 IBC Structural/Seismic Design Manual, Vol. 3

Figure 1. Floor framing <file: floorplan.png >


==========================================================================================
[2] Roof Dead Loads                                                                [0401]
==========================================================================================

    Roofing                             | dl_01 = 6.0 psf
    Insulation                          | dl_02 = 3.0 psf
    Steel deck + fill                   | dl_03 = 47.0 psf
    Roof framing                        | dl_04 = 8.0 psf
    Partition seismic                   | dl_05 = 5.0 psf
    Ceiling                             | dl_06 = 3.0 psf
    Mech Elec                           | dl_07 = 2.0 psf


┌----------------------------------------------------------------------------------------┐
                                                      roofwt | total roof weight [2.1]

sum(dl01 + dl02 + dl03 + dl04 + dl05 + dl06 + dl07)


sum(6.000 psf + 3.000 psf + 47.000 psf + 8.000 psf + 5.000 psf + 3.000 psf + 2.000 psf)

                                                                   roofwt = 74.000 psf
└----------------------------------------------------------------------------------------┘


==========================================================================================
[3] Floor / Wall Dead Loads and Live Loads                                         [0401]
==========================================================================================

    **Floor Loads**
    Floor covering                      | dl_11 = 1.000 psf
    Steel deck + fill                   | dl_12 = 47.000 psf
    Framing (beams + cols.)             | dl_13 = 13.000 psf
    Partition (ASCE 12.7.2)             | dl_14 = 10.000 psf
    Ceiling                             | dl_15 = 3.000 psf
    Mech Elec                           | dl_16 = 2.000 psf

┌----------------------------------------------------------------------------------------┐
                                                    floorwt | total floor weight [3.1]

sum(dl11 + dl12 + dl13 + dl14 + dl15 + dl16)


sum(1.000 psf + 47.000 psf + 13.000 psf + 10.000 psf + 3.000 psf + 2.000 psf)

                                                                  floorwt = 76.000 psf
└----------------------------------------------------------------------------------------┘


    **Exterior curtain wall loads**
    steel studs, gypsum board, EIFS skin

    exterior wall                       | extwallwt = 20.000 psf

    **Live loads (ASCE7-05)**
    roof                                | roof_ll = 20.000 psf
```

```
    floor                                  | floor_ll = 50.000 psf

===============================================================================================
[4] Seismic and structural data                                                         [0401]
===============================================================================================
    Mapped spectral response accelerations
    5% critical damping
    Site Class D (IBC 1613.5)
    Seismic Source Type = A
    Distance to seismic source < 2 km

    0.2 sec-response (IBC 22.0)            | S_S = 2.100 G
    1.0 sec-response (IBC 11.5.1)          | S_1 = 0.930 G
    standard occupancy                     | I_f = 1.0

    **Structural materials**
    Charpy V-notch toughness for all connections in the seismic-load-resisting system
    Weld electrodes E70XX, 20 ft-lb at 0 degrees

    wide flange,plate ASTM A992, Gr. 50  | F1_y = 50.000 ksi
    HSS (round) ASTM A500, Grade B       | F2_y = 42.000 ksi
    Plate ASTM A572, Grade 50            | F3_y = 50.000 ksi
    lightweight conc fill fc'=3000 psi   | f1_c = 3.000 ksi
```

____

[end of calc]

9/11/2014 8:01:33 PM

```
================================================================================
[1] Seismic Loads                                                        [0402]
================================================================================
    Design Example 1A - Special Concentrically Braced Frame
    2006 IBC Structural/Seismic Design Manual, Vol. 3

Figure 1. Braced frame elevation <file: brace.png >


================================================================================
[2] Site ground motion                                                   [0402]
================================================================================
    Maximum considered earthquake spectral response accelerations for short periods
    and at 1.0-second period adjusted for site class effects. 5 percent-damped
    design spectral response accelerations at short periods and at 1.0-second period,
    SD1, adjusted for site class effects.

    Eq 11.4-1                           | F_a = 1.0
    Eq 11.4-2                           | F_v = 1.5
    short period spectral accel         | S_S = 2.1
    1 second spectral accel             | S_1 = 0.93

┌------------------------------------------------------------------------------┐
                                                   S_MS | Eq 11.4-1 (g) [2.1]

Fₐ·S_S


1.000·2.100

                                                                 S_MS = 2.100
└------------------------------------------------------------------------------┘


┌------------------------------------------------------------------------------┐
                                                   S_M1 | Eq 11.4-2 (g) [2.2]

Fᵥ·S₁


1.500·0.930

                                                                 S_M1 = 1.395
└------------------------------------------------------------------------------┘


┌------------------------------------------------------------------------------┐
                                                   S_DS | Eq 11.4-3 (g) [2.3]

2·S_MS
──────
  3


2·2.100
 ──────
   3

                                                                 S_DS = 1.400
└------------------------------------------------------------------------------┘


┌------------------------------------------------------------------------------┐
                                                   S_D1 | Eq 11.4-4 (g) [2.4]
```

```
2·S_M1
──────
  3


2·1.395
───────
   3

                                                                       S_D1 = 0.930
└─────────────────────────────────────────────────────────────────────────────────┘


==========================================================================================
[3] Base shear coefficients                                                        [0402]
==========================================================================================
    The approximate fundamental building period can be computed from the
    building height and the structural system (IBC 12.2-1).

    building height (no dimension)       | h_n = 62
    building height                      | bldg_ht = 62.000 ft
    response mod coef (T 12.2-1)         | R_0 = 6.0
    system overstrength factor           | Q_0 = 2.0
    deflection amplif factor             | C_d = 5.0
    period coefficient - steel           | C_T = 0.02
    importance factor                    | I_f = 1.0

┌─────────────────────────────────────────────────────────────────────────────────┐
                                          T_a | code approximate fundamental period [3.1]

       3/4
C_T·h_n


        3/4
0.020·62
                                                                        T_a = 0.442
└─────────────────────────────────────────────────────────────────────────────────┘


┌─────────────────────────────────────────────────────────────────────────────────┐
                          C_Sbasic | basic seismic response coefficient (EQ 12.8-2) [3.2]

I_f·S_DS
────────
   R₀


1.000·1.400
  ────────
   6.000

                                                                   C_Sbasic = 0.233
└─────────────────────────────────────────────────────────────────────────────────┘


┌─────────────────────────────────────────────────────────────────────────────────┐
                                   C_Smax | coefficient need not exceed (EQ 12.8-3) [3.3]

R₀·S_D1
───────
 I_f·Tₐ


     6.000·0.930
```

```
    ──────────────────
     1.000·0.442

                                                                    C_Smax = 12.627
└───────────────────────────────────────────────────────────────────────────────────┘


┌───────────────────────────────────────────────────────────────────────────────────┐
                                  C_Smin | coefficient minimum (EQ 12.8-3) [3.4]

0.044·I_f·S_DS


0.044·1.000·1.400

                                                                     C_Smin = 0.062
└───────────────────────────────────────────────────────────────────────────────────┘


┌───────────────────────────────────────────────────────────────────────────────────┐
                    C_S | bounded seismic response coefficient (EQ 12.8-2) [3.5]

min(max(C_Sbasic, C_Smin), C_Smax)


min(max(0.233, 0.062), 12.627)

                                                                        C_S = 0.233
└───────────────────────────────────────────────────────────────────────────────────┘


    Note that for buildings in Seismic Design Category E or F, and those buildings
    for which the 1.0-second spectral response, S1, is equal to or greater than 0.6g,
    the value of the seismic response coefficient, CS, must be at least equal to the
    following value.

┌───────────────────────────────────────────────────────────────────────────────────┐
              C_Emin | min seismic response coefficient for SDC E and F (EQ 12.8-2) [3.6]

0.5·I_f·S_DS
────────────
     R₀


0.5·1.000·1.400
  ────────────
     6.000

                                                                     C_Emin = 0.117
└───────────────────────────────────────────────────────────────────────────────────┘



=====================================================================================
[4] Vertical earthquake effects                                              [0402]
=====================================================================================
    The vertical component of seismic acceleration is also computed using a very low
    period for the spectral acceleration.

    dead load                              | DL = 3.000 kip
    base shear effects on vertical         | Q_E = 1.000 kip
    live load effects                      | f_1L = 2.000 kip
    capacity coefficient                   | Omega_0 = 2.0
    redundancy factor (IBC 12.3.4)         | rho = 1.0

┌───────────────────────────────────────────────────────────────────────────────────┐
```

```
                                        E_p | plus vertical earthquake effect Eq 12.4-2 [4.1]

0.2·DL·S_DS + Q_E·ρ


0.2·3.000 kip·1.400 + 1.000 kip·1.000

                                                                            E_p = 1.840 kip
└-----------------------------------------------------------------------------------------┘


┌-----------------------------------------------------------------------------------------┐
                                       E_m | minus vertical earthquake effect Eq 12.4-2 [4.2]

-0.2·DL·S_DS + Q_E·ρ


-0.2·3.000 kip·1.400 + 1.000 kip·1.000

                                                                            E_m = 0.160 kip
└-----------------------------------------------------------------------------------------┘


┌-----------------------------------------------------------------------------------------┐
                                    Em_p | maximum vertical earthquake effect Eq 12.4-2 [4.3]

0.2·DL·S_DS + Ω₀·Q_E


0.2·3.000 kip·1.400 + 2.000·1.000 kip

                                                                           Em_p = 2.840 kip
└-----------------------------------------------------------------------------------------┘


┌-----------------------------------------------------------------------------------------┐
                                    Em_m | maximum vertical earthquake effect Eq 12.4-2 [4.4]

-0.2·DL·S_DS + Ω₀·Q_E


-0.2·3.000 kip·1.400 + 2.000·1.000 kip

                                                                           Em_m = 1.160 kip
└-----------------------------------------------------------------------------------------┘




[end of calc]
```

9/11/2014 9:35:26 PM

[1] Base shear and vertical distribution [0403]

The floor area at each level is 32,224 square feet. The perimeter of the exterior curtain wal

l is 728 feet. The roof parapet height is 4 feet. The curtain-wall weights are distributed to each floor by tributary height. Values are organized from top to bottom (floors 5 to 1).

Figure 1. Building Structural Scheme <file: bldg.png >

```
    wall length [ft]                     | wall_length = 728.0
    wall unit weight [psf]               | wallUnitWt = 20.0
    floor area  [sf]                     | floor_area = 32224.0
    floor unit weight [psf]              | floor_uweight =
[ 74.  76.  76.  76.  76.]
    wall height [ft]                     | wall_ht =
[ 10.  12.  12.  12.  13.]
```

Table - floor weight (kips) [1.1]

$0.001 \cdot floor_{area} \cdot floor\_uweight$

lvl = roof	lvl = 5	lvl = 4	lvl = 3	lvl = 2
2385	2449	2449	2449	2449

Table - wall weight (kips) [1.2]

$0.001 \cdot wallUnitWt \cdot wall\_ht \cdot wall\_length$

lvl = roof	lvl = 5	lvl = 4	lvl = 3	lvl = 2
146	175	175	175	189

Table - story weight (kips) [1.3]

storyWeight = floorWeight + wallWeight

lvl = roof	lvl = 5	lvl = 4	lvl = 3	lvl = 2
2530	2624	2624	2624	2638

totalStorywt | sum of story weights (kips) [1.4]

$KIPS \cdot sum(storyWeight)$

```
1 kips·sum([ 2530.  2624.  2624.  2624.  2638.])

                                                               totalStorywt = 13,040 kips
└────────────────────────────────────────────────────────────────────────────────────────┘


        Import loads from model 0402

==========================================================================================
[2] Seismic Loads                                                                  [0402]
==========================================================================================
    Design Example 1A - Special Concentrically Braced Frame
    2006 IBC Structural/Seismic Design Manual, Vol. 3

Figure 2. Braced frame elevation <file: brace.png >


==========================================================================================
[3] Site ground motion                                                             [0402]
==========================================================================================
    Maximum considered earthquake spectral response accelerations for short periods
    and at 1.0-second period adjusted for site class effects. 5 percent-damped
    design spectral response accelerations at short periods and at 1.0-second period,
    SD1, adjusted for site class effects.

    Eq 11.4-1                           | F_a = 1.0
    Eq 11.4-2                           | F_v = 1.5
    short period spectral accel         | S_S = 2.1
    1 second spectral accel             | S_1 = 0.93

┌────────────────────────────────────────────────────────────────────────────────────────┐
                                                               S_MS | Eq 11.4-1 (g) [3.1]

Fₐ·S_S


1.000·2.100

                                                                            S_MS = 2.100
└────────────────────────────────────────────────────────────────────────────────────────┘


┌────────────────────────────────────────────────────────────────────────────────────────┐
                                                               S_M1 | Eq 11.4-2 (g) [3.2]

Fᵥ·S₁


1.500·0.930

                                                                            S_M1 = 1.395
└────────────────────────────────────────────────────────────────────────────────────────┘


┌────────────────────────────────────────────────────────────────────────────────────────┐
                                                               S_DS | Eq 11.4-3 (g) [3.3]

2·S_MS
──────
  3

2·2.100
 ──────
```

```
   3

                                                                   S_DS = 1.400
└------------------------------------------------------------------------------┘


┌------------------------------------------------------------------------------┐
                                                    S_D1 | Eq 11.4-4 (g) [3.4]
```

$$\frac{2\cdot S\_M1}{3}$$

$$\frac{2\cdot 1.395}{3}$$

```
                                                                   S_D1 = 0.930
└------------------------------------------------------------------------------┘


==============================================================================
[4] Base shear coefficients                                              [0402]
==============================================================================
    The approximate fundamental building period can be computed from the
    building height and the structural system (IBC 12.2-1).

    building height (no dimension)       | h_n = 62
    building height                      | bldg_ht = 62.000 ft
    response mod coef (T 12.2-1)         | R_0 = 6.0
    system overstrength factor           | Q_0 = 2.0
    deflection amplif factor             | C_d = 5.0
    period coefficient - steel           | C_T = 0.02
    importance factor                    | I_f = 1.0

┌------------------------------------------------------------------------------┐
                                    T_a | code approximate fundamental period [4.1]
```

$$C\_T\cdot h\_n^{3/4}$$

$$0.020\cdot 62^{3/4}$$

```
                                                                    T_a = 0.442
└------------------------------------------------------------------------------┘


┌------------------------------------------------------------------------------┐
                      C_Sbasic | basic seismic response coefficient (EQ 12.8-2) [4.2]
```

$$\frac{I\_f\cdot S\_DS}{R_0}$$

$$\frac{1.000\cdot 1.400}{6.000}$$

```
                                                               C_Sbasic = 0.233
└------------------------------------------------------------------------------┘
```

C_Smax | coefficient need not exceed (EQ 12.8-3) [4.3]

$$\frac{R_0 \cdot S\_D1}{I\_f \cdot T_a}$$

$$\frac{6.000 \cdot 0.930}{1.000 \cdot 0.442}$$

C_Smax = 12.627

C_Smin | coefficient minimum (EQ 12.8-3) [4.4]

$$0.044 \cdot I\_f \cdot S\_DS$$

$$0.044 \cdot 1.000 \cdot 1.400$$

C_Smin = 0.062

C_S | bounded seismic response coefficient (EQ 12.8-2) [4.5]

min(max(C_Sbasic, C_Smin), C_Smax)

min(max(0.233, 0.062), 12.627)

C_S = 0.233

Note that for buildings in Seismic Design Category E or F, and those buildings for which the 1.0-second spectral response, S1, is equal to or greater than 0.6g, the value of the seismic response coefficient, CS, must be at least equal to the following value.

C_Emin | min seismic response coefficient for SDC E and F (EQ 12.8-2) [4.6]

$$\frac{0.5 \cdot I\_f \cdot S\_DS}{R_0}$$

$$\frac{0.5 \cdot 1.000 \cdot 1.400}{6.000}$$

C_Emin = 0.117

[5] Vertical earthquake effects [0402]

The vertical component of seismic acceleration is also computed using a very low

```
    period for the spectral acceleration.

    dead load                            | DL = 3.000 kip
    base shear effects on vertical       | Q_E = 1.000 kip
    live load effects                    | f_1L = 2.000 kip
    capacity coefficient                 | Omega_0 = 2.0
    redundancy factor (IBC 12.3.4)       | rho = 1.0

┌---------------------------------------------------------------------------------------┐
                                E_p | plus vertical earthquake effect Eq 12.4-2 [5.1]

0.2·DL·S_DS + Q_E·ρ


0.2·3.000 kip·1.400 + 1.000 kip·1.000

                                                                       E_p = 1.840 kip
└---------------------------------------------------------------------------------------┘


┌---------------------------------------------------------------------------------------┐
                               E_m | minus vertical earthquake effect Eq 12.4-2 [5.2]

-0.2·DL·S_DS + Q_E·ρ


-0.2·3.000 kip·1.400 + 1.000 kip·1.000

                                                                       E_m = 0.160 kip
└---------------------------------------------------------------------------------------┘


┌---------------------------------------------------------------------------------------┐
                            Em_p | maximum vertical earthquake effect Eq 12.4-2 [5.3]

0.2·DL·S_DS + Ω₀·Q_E


0.2·3.000 kip·1.400 + 2.000·1.000 kip

                                                                      Em_p = 2.840 kip
└---------------------------------------------------------------------------------------┘


┌---------------------------------------------------------------------------------------┐
                            Em_m | maximum vertical earthquake effect Eq 12.4-2 [5.4]

-0.2·DL·S_DS + Ω₀·Q_E


-0.2·3.000 kip·1.400 + 2.000·1.000 kip

                                                                      Em_m = 1.160 kip
└---------------------------------------------------------------------------------------┘




==========================================================================================
[6] Design base shear V_1 = C_S * W (Eq 12.8-1)                                   [0403]
==========================================================================================

┌---------------------------------------------------------------------------------------┐
                                                           V_1 | total base shear [6.1]
```

```
C_S·totalStorywt


0.23·13039.71 kips

                                                                         V_1 = 3,043 kips
└───────────────────────────────────────────────────────────────────────────────────────┘


=========================================================================================
[7] Vertical distribution of shear                                                 [0403]
=========================================================================================
    The total lateral force (i.e., design base shear) is distributed over the height
    of the building in accordance with IBC 12.8.3.
    k is a distribution exponent related to building period as follows:
    k = 1 for buildings with a period of 0.5 second or less
    k = 2 for buildings with a period of 2.5 seconds or greater
    k = is interpolated between these values for buildings with period between 0.5
    second and 2.5 seconds

    height above ground [ft]             | h_x =
[62 50 38 26 14]

┌───────────────────────────────────────────────────────────────────────────────────────┐
                                                     w_x | story weight [kips] [7.1]

w_x =

[ 2530.2  2623.7  2623.7  2623.7  2638.3]
└───────────────────────────────────────────────────────────────────────────────────────┘


┌───────────────────────────────────────────────────────────────────────────────────────┐
                                                      Table - w_x h_x (kip-ft) [7.2]


hₓ·wₓ

============  =========  =========  =========  =========
  lvl = roof    lvl = 5    lvl = 4    lvl = 3    lvl = 2
============  =========  =========  =========  =========
      156871     131187      99702      68217      36936
============  =========  =========  =========  =========
└───────────────────────────────────────────────────────────────────────────────────────┘

┌───────────────────────────────────────────────────────────────────────────────────────┐
                                               wxhx_sum | sum of w_x h_x (kips) [7.3]

                                                                     wxhx_sum = 492,914
└───────────────────────────────────────────────────────────────────────────────────────┘


    Vertical distribution coefficient

           h_x__k·w_x
C_vx = ────────────────
          n
         __
         \ `
          )   h_x__k·w_x
         /_,
        i = 1


┌───────────────────────────────────────────────────────────────────────────────────────┐
```

Table - Percent vertical load distribution - C_vx [7.4]

C_dist = wxhx * (1/wxhx_sum) * 100

lvl = roof	lvl = 5	lvl = 4	lvl = 3	lvl = 2
32	27	20	14	7

Table - Story Force F_x (kips) [7.5]

F_x = C_dist * V_1 / 100

lvl = roof	lvl = 5	lvl = 4	lvl = 3	lvl = 2
968	810	615	421	228

Table - Story Shears (kips) [7.6]

V_x = [F_x[0], sum(F_x[0:2]), sum(F_x[0:3]), sum(F_x[0:4]), sum(F_x[0:5])]

lvl = roof	lvl = 5	lvl = 4	lvl = 3	lvl = 2
968	1778	2394	2815	3043

Vbase | base shear (kips) [7.7]

Vbase = 3,043 kips

[8] Horizontal distribution of shear [0403]

IBC 12.8.4.2 requires inclusion of an accidental torsional eccentricity, e, equal to 5 percent of the building dimension perpendicular to the direction of force. It is assumed that the four frames in the transverse direction are each 25 percent stiffer than the six in the longitudinal direction because they are more heavily loaded.

e/w c.m. distance to frame	d_ew = 105 ft
n/s c.m. distance to frame	d_ns = 75 ft
accidental ecc e/w	e_ew = 0.1*d_ew
accidental ecc n/s	e_ns = 0.1*d_ns
rel stiffness-trans frames	R_i = 1.25

[9] Transverse frames-shear plus torsion [0403]

transR_i | rel. torsional resistance - 4 transverse frames [9.1]

$R_i \cdot d\_ew^{2}$

$1 \cdot 105\ ft^2$

transR_i = 13,781 ft2

longR_i | rel. torsional resistance - 6 longitudinal frames [9.2]

d_ns^2

$75\ ft^2$

longR_i = 5,625 ft2

SumR_i | sum of rel. torsion resistances [9.3]

SumR_i = 88,875 ft2

Ttrans_y | Percent torsion force at single transverse frame [9.4]

$$\frac{100 \cdot e\_ew \cdot transR_i}{SumR_i \cdot d\_ew}$$

$$\frac{100 \cdot 10.50\ ft \cdot 13781.25\ ft2}{88875.00\ ft2 \cdot 105.00\ ft}$$

Ttrans_y = 2

Tlong_y | Percent torsion force at single transverse frame [9.5]

$$\frac{100 \cdot e\_ew \cdot longR_i}{SumR_i \cdot d\_ns}$$

$$\frac{100 \cdot 10.50\ ft \cdot 5625.00\ ft2}{88875.00\ ft2 \cdot 75.00\ ft}$$

Tlong_y = 1

Torsion distribution relation

n

```
       /Ri·di·e_real      Riy      |Ri·di·e_acc |\  \ `
 Viy = |───────────── + ───────── + |────────────||·  )   Vy
       |  n              n         |  n         ||  /_,
       | ___             __        | ___        || i = 0
       | \  `            \ `       | \  `       ||
       |  \        2      )   Riy  |  \        2||
       |  /   Ri·di      /_,       |  /   Ri·di ||
       | /__,           i = 0      | /__,       ||
       \i = 0                      |i = 0       |/
```

```
┌--------------------------------------------------------------------------------------┐
                                  Table - Forces at each level at frame BF-2 (kips) [9.6]

F_xbf2 = F_x * ((1/4) + Ttrans_y/100)
```

lvl = roof	lvl = 5	lvl = 4	lvl = 3	lvl = 2
257	215	163	112	61

```
└--------------------------------------------------------------------------------------┘

┌--------------------------------------------------------------------------------------┐
                            Table - Story shears at each level at frame BF-2 (kips) [9.7]

V_xbf2 = array(V_x) * ((1/4) + Ttrans_y/100)
```

lvl = roof	lvl = 5	lvl = 4	lvl = 3	lvl = 2
257	472	635	747	808

```
└--------------------------------------------------------------------------------------┘
```

```
=========================================================================================
[10] Combined forces (IBC 12.4.2)                                                  [0403]
=========================================================================================
```

For convenience, the seismic design load combinations can be reformulated to combine the vertical component of the seismic motion with the dead load. Also, as the overstrength factor is not considered in combination with the redundancy factor (and the results of the analysis include the redundancy factor), it is sometimes convenient to recalculate the overstrength factor to account for this difference (if rho > 1.0).

```
┌--------------------------------------------------------------------------------------┐
                                                E2 | combined effects - redundancy [10.1]

-0.28·DL + Q_E·ρ


-0.28·3 kip + 1 kip·1

                                                                               E2 = 0 kip
└--------------------------------------------------------------------------------------┘


┌--------------------------------------------------------------------------------------┐
                                            E1_m | combined effects - overstrength [10.2]

0.28·DL + Ω₀·Q_E


0.28·3 kip + 2·1 kip

                                                                             E1_m = 3 kip
```

```
└─────────────────────────────────────────────────────────────────────────────────────┘


┌─────────────────────────────────────────────────────────────────────────────────────┐
                                        E2_m | combined effects - overstrength [10.3]

-0.28·DL + Ω₀·Q_E

-0.28·3 kip + 2·1 kip

                                                                        E2_m = 1 kip
└─────────────────────────────────────────────────────────────────────────────────────┘


    Basic Seismic Load Combinations:

┌─────────────────────────────────────────────────────────────────────────────────────┐
                                            E1_B | 1.2D+ 1.0E + 0.5L (Eq 16-5) [10.4]

1.48·DL + Q_E·ρ + f_1L

1.48·3 kip + 1 kip·1 + 2 kip

                                                                        E1_B = 7 kip
└─────────────────────────────────────────────────────────────────────────────────────┘


┌─────────────────────────────────────────────────────────────────────────────────────┐
                                                E2_B | 0.9 D + 1.0 E (Eq 16-7) [10.5]

0.62·DL + Q_E·ρ

0.62·3 kip + 1 kip·1

                                                                        E2_B = 3 kip
└─────────────────────────────────────────────────────────────────────────────────────┘


    Special Seismic Load Combinations (Amplified Seismic Load)

┌─────────────────────────────────────────────────────────────────────────────────────┐
                                         E1_M | 1.2D + 1.0Em + 0.5L (Eq 16-22) [10.6]

1.48·DL + 2.0·Q_E + f_1L

1.48·3 kip + 2.0·1 kip + 2 kip

                                                                        E1_M = 8 kip
└─────────────────────────────────────────────────────────────────────────────────────┘


┌─────────────────────────────────────────────────────────────────────────────────────┐
                                                E2_M | 0.9D + 1.0Em (Eq 16-23) [10.7]

0.62·DL + 2.0·Q_E

0.62·3 kip + 2.0·1 kip

                                                                        E2_M = 4 kip
└─────────────────────────────────────────────────────────────────────────────────────┘
```

The formulae that include the redundancy factor are used for brace design, and those that do not include it are used for column and beam design. The forces that the framing members are likely to see are greater for a frame with larger braces than for one with smaller ones, and the same overstrength factor should be applied to the two frames.

It is apparent from the reformulations that the redundancy factor actually decreases the effective overstrength required of these members and makes unfavorable yield modes more likely. A redundancy factor of 1.3 decreases the effective overstrength factor to 1.54, an extremely low value for a braced frame; use of such a low value in determining column design forces may not be sufficient to preclude column buckling. Designers should not lose sight of the purpose of the required overstrength, even when code provisions permit its reduction.

[end of calc]

9/11/2014 9:39:07 PM

```
================================================================================
[1] Frame analysis                                                       [0404]
================================================================================
    Small eccentricities occur at the connections to facilitate detailing.
    These eccentricities result in flexural forces in the framing
    members, as well as larger drifts than those for frames without eccentricities. It is
    therefore necessary to include the eccentricities in the frame model.
    Using the frame forces from Table 1 A4, the frame shown in Figure 1A-5 is analyzed.
    Columns are modeled as fixed at the base and braces as pinned at each end. Although
    in-plane rotation of the gusseted beam-to-column connection may induce some
    flexural forces in the brace, these are small at the elastic drift level. Beams are
    modeled as having fixed connections at their gusseted connections to the columns;
    these details include beam-to-column joint flange welds. (AISC 341, 7.2)

================================================================================
[2] Story drifts                                                         [0404]
================================================================================
    In this design example a second-order analysis has been
    performed, and the B2 amplification factor is not required.
    The displacements from the model are converted to design story drifts and compared
    to the allowable drift from ASCE/SEI 7-05 Table 12.2-1. The story drifts are
    calculated from the displacements, and divided by the reliability/redundancy factor,
    rho and I. The drifts from the elastic analysis are multiplied by the displacement
    amplification factor, Cd. The design story drift for each level of this frame is less
    than the allowable drift. Because the building design is symmetrical and
    has rigid diaphragms, the average story drift will not exceed the drift of this frame.
    Thus, the building will meet the drift limitations of Table 12.12-1.

    response mod coef (T 12.2-1)         | R_1 = 6.0
    story height (inch)                  | h_sx =
[144 144 144 144 168]
    model displacement (inch)            | disp_xe =
[ 2.19  1.7   1.24  0.77  0.35]
    model drift (inch)                   | Delta_xe =
[ 0.5   0.45  0.47  0.43  0.35]

┌------------------------------------------------------------------------------┐
                                   C_d | displacement amplification factor [2.1]

0.7·R₁


0.7·6.00

                                                                      C_d = 4.20
└------------------------------------------------------------------------------┘



┌------------------------------------------------------------------------------┐
                                                  design story drift (inch) [2.2]


C_d·Δₓₑ

=============  ==========  ==========  ==========  ==========
  stry = roof    stry = 4    stry = 3    stry = 2    stry = 1
=============  ==========  ==========  ==========  ==========
          2.1         1.9         2.0         1.8         1.5
=============  ==========  ==========  ==========  ==========
└------------------------------------------------------------------------------┘

┌------------------------------------------------------------------------------┐
                                               allowable story drift (inch) [2.3]
```

0.02·h_sx

stry = roof	stry = 4	stry = 3	stry = 2	stry = 1
2.9	2.9	2.9	2.9	3.4

[end of calc]

9/11/2014 9:46:29 PM

```
==========================================================================================
[1] Brace design                                                                   [0405]
==========================================================================================
    For this example the fourth floor brace will be designed. From the analysis,
    the earthquake force, rhoP_E, on the brace is known.

    dl compression                       | PD = 8000 kips
    ll compression                       | PL = 6 kips
    eq tension                           | Q_PEt = -328 kips
    eq compression                       | Q_PEc = 328 kips
    non-assembly,-parking < 100 psf      | f1 = 0.5
    redundancy factor                    | rho = 1.0
    pi                                   | pi = 3.14

┌-----------------------------------------------------------------------------------------┐
                              P_dc | max compression 1.2 D + 1.0 E + 0.5L (Eq 16-5) [1.1]
```

$$1.48 \cdot PD + PL \cdot f_1 + Q\_PEc \cdot \rho$$

$$1.48 \cdot 8000.00\ \text{kips} + 6.00\ \text{kips} \cdot 0.50 + 328.00\ \text{kips} \cdot 1.00$$

```
                                                                     P_dc = 12,171.0 kips
└-----------------------------------------------------------------------------------------┘


┌-----------------------------------------------------------------------------------------┐
                                            P_dt | max tension 0.9 D + 1.0 (Eq 6-7) [1.2]
```

$$0.62 \cdot PD + \frac{Q\_PEt \cdot \rho}{f_1}$$

$$0.62 \cdot 8000.00\ \text{kips} + \frac{-328.00\ \text{kips} \cdot 1.00}{0.50}$$

```
                                                                      P_dt = 4,304.0 kips
└-----------------------------------------------------------------------------------------┘


==========================================================================================
[2] Brace compression strength                                                     [0405]
==========================================================================================
    The distance from work-point to work-point is 17.9 feet. The ductile
    detailing requirements for SCBF result in the provision of a hinge zone
    in the connection; it is assumed here that the hinge zone will be at least
    9 inches from the work point at each end, resulting in a reduction of the brace
    length of 18 inches. The brace length = 197 inches. HSS 9.625x0.500 will be
    used. Section properties are from the 13th Edition LRFD Manual.

    boundary condition coefficient       | K_1 = 1.0
    unbraced lenght                      | l_1 = 197.0 in
    radius of gyration                   | r_1 = 3.2 in
    elastic modulus                      | E_1 = 29000.0 ksi
    yield stress                         | F_y = 42.0 ksi
    cross section area                   | A_g = 13.4 in2
    HSS section depth                    | D_e = 9.6 in
    HSS section thickness                | t_e = 0.5 in

┌-----------------------------------------------------------------------------------------┐
                                                            Kl_r | Slenderness ratio [2.1]
```

$$K_1 \cdot l_1$$

$$\overline{}$$
$$r_1$$

$$\frac{1.00 \cdot 197.00\ in}{3.24\ in}$$

Kl_r = 61

F_e | Buckling stress [2.2]

$$\frac{\pi^2 \cdot E_1 \cdot r_1^2}{K_1^2 \cdot l_1^2}$$

$$\frac{pi^2 \cdot 29000.00\ ksi \cdot 3.24\ in^2}{1.00^2 \cdot 197.00\ in^2}$$

F_e = 77.3 ksi

F_cr | Critical buckling stress (AISC 360, E3-2) [2.3]

$$0.658^{\frac{F\_y}{F_e}} \cdot F\_y$$

$$0.658^{\frac{42.00\ ksi}{77.34\ ksi}} \cdot 42.00\ ksi$$

F_cr = 33.5 ksi

phiP_c | Compression strength (AISC 360 E3-1) [2.4]

0.9·A_g·F_cr

0.9·13.40 in2·33.46 ksi

phiP_c = 403.5 kips

Check brace slenderness limit (AISC 341-13.2a) [2.5]

```
K_1*l_1         / E_1
------- <= 4*  /  ---
  r_1        \/   F_y


                         _____________
1.00·197.00 in          / 29000.00 ksi
────────────── <= 4·   /  ────────────
   3.24 in           \/    42.00 ksi

                                                                     60.80 <= 105.07   ok
└──────────────────────────────────────────────────────────────────────────────────────┘


    Note that under the exception, braces with Kl/r < 200 are allowed if capacity
    design is used for columns

┌──────────────────────────────────────────────────────────────────────────────────────┐
                            Check brace width-to-thickness (AISC 341-13.2d, T 1.8.1) [2.6]

D_e    0.44*E_1
--- <= --------
t_e      F_y

9.62 in    0.44·29000.00 ksi
─────── <= ──────────────---
0.50 in        42.00 ksi

                                                                     19.25 <= 303.81   ok
└──────────────────────────────────────────────────────────────────────────────────────┘


=========================================================================================
[3] Brace tension strength                                                        [0405]
=========================================================================================

┌──────────────────────────────────────────────────────────────────────────────────────┐
                                  phiP_t | Design tension strength (AISC 360, D1-1) [3.1]

0.9·A_g·F_y


0.9·13.40 in2·42.00 ksi

                                                                      phiP_t = 506.5 kips
└──────────────────────────────────────────────────────────────────────────────────────┘


    tension demand on brace              | P_u = 350.0 kips

┌──────────────────────────────────────────────────────────────────────────────────────┐
                                                              Check tension strength [3.2]

 P_u
------ <= 1.0
phiP_t

350.00 kips
─────────── <= 1.0
506.52 kips

                                                                         0.69 <= 1.00   ok
└──────────────────────────────────────────────────────────────────────────────────────┘


    Equation D1-2, which applies to fracture of the net section, is checked in the design
    of the connection.
```

```
==========================================================================================
[4] Distribution of force between braces                                           [0405]
==========================================================================================
    AISC 341-13.2c does not permit a major unbalance between braces working in
    compression and those working in tension to resist lateral loads. The frames in this
    structure are symmetrical and no single-diagonal frames are used. An elastic analysis
    shows that the seismic force resisted by compression is identical to that resisted by
    tension and the frame complies with the requirement. The tension strength of
    braces is similar in each direction, so that even after degradation of
    the compression strength of braces there is no significant tendency to accumulate
    drift in one direction.




[end of calc]
```

```
9/11/2014 9:47:50 PM
==========================================================================================
[1] Example 5 - Structure Dynamics                                                  [0501]
==========================================================================================
    Example E12-1 from Cough and Penzien *Dynamics of Structures* is used to
    illustrate application of the *numpy* linear algebra libraries in **on-c-e**.
    See *numpy* documentation for futher details.

Figure 1. From Clough and Penzien - page 178 <file: frame.png >


==========================================================================================
[2] Define mass and stiffness                                                       [0501]
==========================================================================================
    units: kips, inches

    mass                                  | m_1 =
[[ 1.   0.   0. ]
 [ 0.   1.5  0. ]
 [ 0.   0.   2. ]]
    stiffness                             | k_1 =
[[  600.  -600.     0.]
 [ -600.  1800. -1200.]
 [    0. -1200.  3000.]]

┌----------------------------------------------------------------------------------------┐
                                                         flex | flexibility matrix [2.1]

LA.inv(k_1)

flex =

[[ 0.0031  0.0014  0.0006]
 [ 0.0014  0.0014  0.0006]
 [ 0.0006  0.0006  0.0006]]
└----------------------------------------------------------------------------------------┘


┌----------------------------------------------------------------------------------------┐
                                                             dyna | dynamic matrix [2.2]

inner(flex, m₁)


dyna =

[[ 0.0031  0.0021  0.0011]
 [ 0.0014  0.0021  0.0011]
 [ 0.0006  0.0008  0.0011]]
└----------------------------------------------------------------------------------------┘


==========================================================================================
[3] Eigenvalue analysis                                                             [0501]
==========================================================================================

┌----------------------------------------------------------------------------------------┐
                                                           eigenvals | eigenvalues [3.1]

LA.eig(dyna)[0]

eigenvals =

[ 0.0047  0.001   0.0005]
└----------------------------------------------------------------------------------------┘
```

```
┌-----------------------------------------------------------------------------------┐
                                              omega_1 | natural frequency (secs) [3.2]

         -0.5
eigenvals


omega_1 =

[ 14.5217  31.0477  46.0995]
└-----------------------------------------------------------------------------------┘


┌-----------------------------------------------------------------------------------┐
                                                         evects_0 | eigenvectors [3.3]

LA.eig(dyna)[1]

evects_0 =

[[-0.813 -0.739  0.273]
 [-0.527  0.449 -0.694]
 [-0.246  0.502  0.666]]
└-----------------------------------------------------------------------------------┘



┌-----------------------------------------------------------------------------------┐
                                                         normalized eigenvectors [3.4]

return variable: evectors_1

function call: norm_evects(evects_0, eigenvals)

function doc:
normalize and scale eigenvectors

function return:
[[ 1.     0.649  0.302]
 [ 1.    -0.607 -0.679]
 [ 1.    -2.542  2.44 ]]
└-----------------------------------------------------------------------------------┘


==========================================================================================
[4] Plot mode shapes                                                                [0501]
==========================================================================================

┌-----------------------------------------------------------------------------------┐
                                                                plot mode shapes [4.1]

return variable: plot_1

function call: plot_modes(evectors_1)

function doc:
plot three mode shapes

function return:
mode shapes plotted
└-----------------------------------------------------------------------------------┘


Figure 2. Mode Shapes <file: mode_shapes.png >
```

[end of calc]

```
9/11/2014 9:48:25 PM
==========================================================================================
[1] Example 0601 - Analyze 3-Bar truss using OpenSees                              [0601]
==========================================================================================
    OpenSees Primer Example 1.1
    Displacement results for two different geometries
    Units: kips, in, sec

Figure 1. Node and element numbers <file: truss_1.png >


==========================================================================================
[2] set up opensees geometry for run 1                                             [0601]
==========================================================================================
    change node 4 location using a string template
    run OpenSees model

    node 4 x (in)                        | n4x1 = 70.0
    node 4 y (in)                        | n4y1 = 45.0

┌----------------------------------------------------------------------------------------┐
                                                        c:/opensees/truss.tcl | edit file

file: c:/opensees/truss.tcl
[line #]       [replacement line]
  13      node 4 %n4x1 %n4y1
  55      recorder Node -file example_disp1.out -load -node 4 -dof 1 2 disp
  56      recorder Element -file example_force1.out  -eleRange 1 3 axialForce

└----------------------------------------------------------------------------------------┘



┌----------------------------------------------------------------------------------------┐
                                                      plot OpenSees geometry - run 1 [2.1]

return variable: plot 1

function call: osplot("c:/opensees/trusscopy1.tcl")

function doc:
  plot geometry for OpenSees tcl file

    function plots geometry using matplotlib

    Args:
    tcl file (file)

    methods called:
    ndict - generates node dictionary
    edict - generates element dictionary
    plotgeo - generates plot and writes to file


function return:
plot file 'trusscopy1fig.png' written
└----------------------------------------------------------------------------------------┘


Figure 2. Truss geometry - run 1 <file: trusscopy1fig.png >


==========================================================================================
[3] set up opensees geometry for run 2                                             [0601]
==========================================================================================
    change node 4 location using a static string - no variable substition
    run OpenSees model
```

```
┌-------------------------------------------------------------------------------------┐
                                                        c:/opensees/truss.tcl | edit file

file: c:/opensees/truss.tcl
[line #]       [replacement line]
  13      node 4 90.1 225.5
  55      recorder Node -file example_disp2.out -load -node 4 -dof 1 2 disp
  56      recorder Element -file example_force2.out  -eleRange 1 3 axialForce

└-------------------------------------------------------------------------------------┘


┌-------------------------------------------------------------------------------------┐
                                                      plot OpenSees geometry - run 2 [3.1]

return variable: plot_2

function call: osplot("c:/opensees/trusscopy2.tcl")

function doc:
  plot geometry for OpenSees tcl file

    function plots geometry using matplotlib

    Args:
    tcl file (file)

    methods called:
    ndict - generates node dictionary
    edict - generates element dictionary
    plotgeo - generates plot and writes to file


function return:
plot file 'trusscopy2fig.png' written
└-------------------------------------------------------------------------------------┘


Figure 3. Truss geometry - run 2 <file: trusscopy2fig.png >


=======================================================================================
[4] run models 1 and 2 and write output tables                                   [0601]
=======================================================================================

┌-------------------------------------------------------------------------------------┐
                                                                   deltaxy1 | read data

file: c:/opensees/example_disp1.out

[ 1.        0.224082 -0.327086] ... [ 1.        0.224082 -0.327086]

└-------------------------------------------------------------------------------------┘

┌-------------------------------------------------------------------------------------┐
                                                                forceaxial1 | read data

file: c:/opensees/example_force1.out

[  4.18876 -62.5935  -47.3114 ] ... [  4.18876 -62.5935  -47.3114 ]

└-------------------------------------------------------------------------------------┘

┌-------------------------------------------------------------------------------------┐
                                                                   deltaxy2 | read data
```

file: c:/opensees/example_disp2.out

[1.   3.79084 -0.367795] ... [1.   3.79084 -0.367795]

forceaxial2 | read data

file: c:/opensees/example_force2.out

[131.571   -80.1586 -99.6802] ... [131.571   -80.1586 -99.6802]

Table of Displacements-Node 4 (in) [4.1]

delta = [[deltaxy1], [deltaxy2]]

run	= load no.	= x	= y
1	1.00	0.22	-0.33
2	1.00	3.79	-0.37

Table of Axial forces (kips) [4.2]

forces = [[forceaxial1], [forceaxial2]]

run	ele no. = 1	ele no. = 2	ele no. = 3
1	4.19	-62.59	-47.31
2	131.57	-80.16	-99.68

[end of calc]

```
9/11/2014 9:49:09 PM
==========================================================================================
[1] Example 7 - Stiffness Method                                                   [0701]
==========================================================================================
    Examples 3.4 and 3.5 from McQuire and Gallagher *Matrix Structural Analysis*
    demonstrate applications of the numpy linear algebra library
    and unit manipulations.

Figure 1. Frame geometry <file: truss1.png >

Figure 2. Loads <file: truss2.png >


==========================================================================================
[2] Define geometry and material properties                                        [0701]
==========================================================================================
    units: kN, mm

    **Element Properties**
    elastic modulus (kN/mm**2)          | e_1 = 200.0
    cross section 1 (mm**2)             | a_1 = 10000
    cross section 2 (mm**2)             | a_2 = 15000

    **Node coordinates** [node number, x, y]
    node 0                              | n_0 =
[0, 0.0, 0.0]
    node 1                              | n_1 =
[1, 5000.0, 0.0]
    node 2                              | n_2 =
[2, 5000.0, 8660.0]
    node 3                              | n_3 =
[3, 10000.0, 8660.0]

┌---------------------------------------------------------------------------------------┐
                                                            nodes | node coordinates [2.1]

nodes =

[[0, 0.0, 0.0],
 [1, 5000.0, 0.0],
 [2, 5000.0, 8660.0],
 [3, 10000.0, 8660.0]]
└---------------------------------------------------------------------------------------┘


    **Element Connectivity**  [element number, node1, node2, area, modulus]




┌---------------------------------------------------------------------------------------┐
                                                   elements | element connectivity [2.7]

elements =

[[1, 0, 1, 10000, 200.0],
 [2, 1, 3, 15000, 200.0],
 [3, 2, 3, 10000, 200.0],
 [4, 0, 2, 15000, 200.0],
 [5, 1, 2, 15000, 200.0]]
└---------------------------------------------------------------------------------------┘


    **Forces and reactions**
```

```
    releases 0 or 1 [node, x, y]         | react =
[[0, 1, 1], [1, 0, 1]]

    applied forces  [node, x, y]         | forces =
[[3, 283.0, -283.0]]

==============================================================================
[3] Analyze truss                                                       [0701]
==============================================================================
    Intermediate results may be viewed in standard out by uncommenting print
    statements in the file *func_stiff.py*.


┌----------------------------------------------------------------------------┐
                                                  Direct stiffness analysis [3.1]

return variable: result_1

function call: direct_stiff(nodes, elements, react, forces)

function doc:
executes stiffness functions

    1. structure dictionaries   : node_dict()
    2. global stiffness         : assem_global()
    3. displacements and reacts : displace()
    4. element forces           : axial_force()
    5. tables and plots         : disp_tables(), force_tables()


function return:
  node        x displacement
   0                0.0
   1    -0.408487297921
   2      9.81709263995
   3      10.9330799379
  node        y displacement
   0                0.0
   1                0.0
   2     -2.23184365333
   3     -7.80601753585
  node           reaction
   0    x dir      -283.0
   0    x dir    -773.156
   1    x dir    1056.156
  element        axial force
   1     -163.394919169
   2     -326.782648882
   3      446.394919169
   4      892.770196745
   5           -773.156

└----------------------------------------------------------------------------┘


Figure 3. Examples 3.4 and 3.5 from McQuire and Gallagher <file: geom1.png >

Figure 4. Examples 3.4 and 3.5 from McQuire and Gallagher <file: geom2.png >

Figure 5. Examples 3.4 and 3.5 from McQuire and Gallagher <file: geom3.png >
```

[end of calc]

```
[s] Example 0001 - a simple calculation
    *Numbers are fun whatever you do.*
    **First comes one and then comes two!**


    [t] number of brothers          | bros = 2
    [t] number of sisters           | siss = 1
    [t] height of tall brother      | bro_h = 6.0*FT
    [t] height of sister            | sis_h = 5.8*FT

How many siblings do I have?

    [e] add up number of brothers and sisters #- 10
        sum = bros + siss

What is the height difference between the tall brother and sister in inches?

    [e] subtract heights #- 11
        dif = bro_h - sis_h

#- format | 3,3  | 1.0
#- 10 | 2,2 | FT |
#- 11 | 2,1 | IN |
```

```
[s] Example 0101 - basic template
    This model calculates the mid-span beam moment
    under uniformly distributed (UDL) floor loads.

    #- 01 insert beam figure

    operation examples:
        - [s] section
        - [y] sympy and LaTeX symbolic expresssions
        - [t] term
        - [e] equation
        - [#-] format and file operations

    reST markup examples:
        - lists, **bold**, *italic*
        - tables
        - raw latex


    **Notation:** (LaTeX block is not processed for UTF calcs)

.. raw:: latex
`
  \vspace{-12mm}
  \openup 1em
  \begin{align*}
  %
  \bm{D_n}
  \hspace{2mm} &= \hspace{1mm}
  \textrm{nominal dead load of material or component n}\\
  %
  \bm{L_n}
  \hspace{2mm} &= \hspace{1mm}
  \textrm{nominal live load from action n}\\
  %
  \bm{DL_n, LL_n}
  \hspace{2mm} &= \hspace{1mm}
  \textrm{sum of nominal dead or live loads}\\
  %
  \bm{l_n}
  \hspace{2mm} &= \hspace{1mm}
  \textrm{effective beam span}\\
  %
  \bm{\omega_n}
  \hspace{2mm} &= \hspace{1mm}
  \textrm{factored line load on element n}\\
  %
  \bm{M_n}
  \hspace{2mm} &= \hspace{1mm}
  \textrm{factored bending moment on component n}\\
  %
  \bm{q_n}
  \hspace{2mm} &= \hspace{1mm}
  \textrm{factored area load n}\\
  %
  \bm{w_n}
  \hspace{2mm} &= \hspace{1mm}
  \textrm{tributary width n}
  %
  \end{align*}
  \openup -1em
`

[s] Beam Loads and geometry
    Dead and live load contributions to beam UDL

    **ASCE 7 - 05 Load Effects**

    =============   ==============================================
```

```
    Equation No.    Load Combination
    =============   ==============================================
    16-1            1.4(D+F)
    16-2            1.2(D+F+T) + 1.6(L+H) + 0.5(Lr or S or R)
    16-3            1.2(D+F+T) + 1.6(Lr or S or R) + (f1L or 0.8W)
    =============   ==============================================

    **Dead loads**
    [t] joists          | D_1 =   3.8*PSF
    [t] plywood         | D_2 =   2.1*PSF
    [t] partitions      | D_3 =  10.0*PSF
    [t] fixed machinery | D_4 =   0.5*KLF

    **Live loads**
    [t] ASCE7-05        | L_1 = 40*PSF

    **Beam tributary width and span**
    [t] distance between beams  | w_1 = 2*FT
    [t] beam span               | l_1 = 14*FT

[s] Maximum bending moment

    [e] Total UDL factored dead load    #- 01
        DL_1 = 1.2 * (w_1 *(D_1 + D_2 + D_3) + D_4)

    [e] Total UDL factored live load    #- 01
        LL_1 = 1.6 * w_1 * L_1

    [e] factored UDL    #- 01
        omega_1 = DL_1 + LL_1

    [e] Bending moment at mid-span    #- 02
        M_1 = omega_1 * l_1**2 / 8

[s] Symbolic rendering using sympy or LaTeX

    Equation rendered from **SymPy**
    [y] p | sigma = M*z / I

    Equation rendered from **LaTeX** (expresssion copied from Wikipedia HTML source)
    [y] x | \sigma = \frac{Mz}{I} = -zE ~ \frac{\mathrm{d}^2 w}{\mathrm{d} x^2}

#- formateq  | 3,3  | 1.5
#- 01  | 2,2  | KLF     | 2
#- 02  | 2,1  | KIP*FT  | 3

#- fileop
#- 01 | f | beam.png | Simply supported beam | 50 |

# This file contains a on-c-e public domain template (the template).
# The template is distributed under the CC0 1.0 Public Domain Dedication
# at http://creativecommons.org/publicdomain/zero/1.0/
# The template is not a structural design calculation.
# The template user assumes sole responsibility for all inputs and results.
```

```
[s] Example 0201 - Isolation Bearing Properties
    Calculate shape factors, compression stiffness and
    buckling loads for a rubber laminated seismic isolation bearing

    **Measured Properties**
    [t] bearing diameter                 |   phi = 36*IN
    [t] rubber layer thickness           |   t_layer = .38*IN
    [t] initial shear modulus            |   G_i = 100.0*PSI
    [t] shear modulus                    |   G = 50.0*PSI
    [t] total rubber height              |   R_t = 11.0*IN
    [t] bearing height                   |   h_b = 19*IN
    [t] bulk modulus                     |   K_1 = 250*KSI
    [t] measured compression stiffness   |   K_cm = 11800*KIPS/IN
    [t] constant                         |   pi = 3.1418

    **Loads**
    [t] applied load                     |   P = 3000*KIPS
    [t] factored load demand             |   P_u = 5000*KIPS

    #- 01 bearing figure

[s] Basic Calculated Properties

    [e] Bearing area #- 12
        a_1 = pi * phi**2 / 4

    [e] Axial shape factor #- 11
        S_1 = phi / (4*t_layer)

    [e] Number of rubber layers #- 14
        n = R_t / t_layer

    [e] Moment of inertia #- 15
        I_1 = .78*(phi / 2.)**4

[s] Compression Stiffness

    [e] Stiffness term #- 21
        GK = 12*G_i / K_1

    [e] Compression modulus after Kelly #- 22
        E_c = K_1*(1 - (1/(S_1 * GK**.5)) + (1/(4*GK * S_1**2)))

    [e] Compression stiffness after Kelly #- 23
        kv_k = E_c*a_1 / R_t

    [e] Compresssion stiffness after Robinson #- 24
        kv_R = 6*G_i * S_1**2 * a_1*K_1  / ((6*G_i * S_1**2 + K_1)*R_t)

[s] Buckling Load - Pe

    [e] Buckling capacity after Kelly #- 31
        P_e = pi**2 * E_c*I_1*h_b / (h_b**2 * 3*R_t)

    [c] dc ratio check |  ok | 2
        P_u / P_e  |  <=  | 1.0

#- format | 3,3 | 1.0
#-  11  | 2,1  |          | 3
#-  12  | 2,2  | IN**2    | 3
#-  13  | 2,2  |          | 3
#-  14  | 2,0  |          | 3
#-  15  | 1,0  | IN**4    | 3
#-  21  | 2,4  |          | 3
#-  22  | 3,1  | KSI      | 3
#-  23  | 2,1  | KIPS/IN  | 3
```

```
#-  24  | 2,1  | KIPS/IN  | 3
#-  31  | 2,1  | KIPS     | 3

#- file
#- 01 | f | bearing_geo.png | Bearing Geometry | 60 |

# This file contains a on-c-e public domain template (the template).
# The template is distributed under the CCO 1.0 Public Domain Dedication
# at http://creativecommons.org/publicdomain/zero/1.0/
# The template is not a structural design calculation.
# The template user assumes sole responsibility for all model inputs and results.
```

```
[s] Example 0301 - beam deflection

    Table of tube bending deflections as a function of span and depth
    when analyzed as a simply supported classical beam with uniform
    distributed load.

        #- 01 insert figure

[s] Materials and Geometry
    Calculate tube moment of inertia for an initial condition and a range of
    depths.  Neglect radii at corners.

    **6061-T6 Aluminum**
    [t] elastic modulus  | E_0 = 10000*KSI

    **Tube Dimensions**
    [t] tube width       | b_0 = 5*IN
    [t] wall thickness   | t_0 = .125*IN

    **Design Load and Span**
    [t] trial clear span          | span = 6*FT
    [t] trial tube depth          | d_0 = 9*IN
    [t] uniform distributed load  | omega_1 = 0.1*KIP/IN

    [e] Moment of inertia  #- 22
            I_9 = (b_0*(d_0**3)-(b_0 - 2*t_0)*(d_0 - 2*t_0)**3) / 12

    [a] Table : I_x [in4] : depth [in]   #- 20
            d_n = arange(9,14,1)
            I_n = (b_0*(d_n**3)-(b_0 - 2*t_0)*(d_n - 2*t_0)**3) / 12

[s] Evaluate Deflections
    Evaluate beam deflections for an initial I and span and then a range of values.

    **Check an initial case**
    [t]  initial I   | I_1 = 35*IN**4
    [t]  beam span   | span_1 = 10*FT

    [e] beam deflection under uniform load    #- 23
            delta_1 = 5 * omega_1 * span_1**4 / (384 * E_0 * I_1)

    [a] Table : Deflections [in] (span [in], I [in4])   #- 24
            I_2 = arange(33,83,10)
            span_2 = arange(6,16,2)*12
            delta_2 = 5 *omega_1 * span_2**4 / (384 * E_0 * I_2)

#- format | 2  | 1.0
#- 22  | 2  | IN**4   | 2
#- 20  | 2  | depth   |
#- 23  | 2  | IN      | 3
#- 24  | 2  | I       | span

#- file
#- 01 | f | tube.png | Tube geometry | 85 |
```

```
[s] Example 0401 - Building and Load Information
| Design Example 1A - Special Concentrically Braced Frame
| 2006 IBC Structural/Seismic Design Manual, Vol. 3

        #- 01 f insert floorplan figure

[s] Roof Dead Loads

    [t] Roofing                         | dl_01 = 6.0*PSF
    [t] Insulation                      | dl_02 = 3.0*PSF
    [t] Steel deck + fill               | dl_03 = 47.0*PSF
    [t] Roof framing                    | dl_04 = 8.0*PSF
    [t] Partition seismic               | dl_05 = 5.0*PSF
    [t] Ceiling                         | dl_06 = 3.0*PSF
    [t] Mech Elec                       | dl_07 = 2.0*PSF


    [e] total roof weight   #- 01
        roofwt = sum(dl_01+dl_02+dl_03+dl_04+dl_05+dl_06+dl_07)

[s] Floor / Wall Dead Loads and Live Loads

    **Floor Loads**
    [t] Floor covering                      | dl_11 = 1.0*PSF
    [t] Steel deck + fill                   | dl_12 = 47.0*PSF
    [t] Framing (beams + cols.)             | dl_13 = 13.0*PSF
    [t] Partition (ASCE 12.7.2)             | dl_14 = 10.0*PSF
    [t] Ceiling                             | dl_15 = 3.0*PSF
    [t] Mech Elec                           | dl_16 = 2.0*PSF

    [e] total floor weight   #- 01
        floorwt = sum(dl_11+dl_12+dl_13+dl_14+dl_15+dl_16)

    **Exterior curtain wall loads**
    steel studs, gypsum board, EIFS skin

    [t]  exterior wall  | extwallwt = 20.0*PSF

    **Live loads (ASCE7-05)**
    [t]  roof           | roof_ll = 20.0*PSF
    [t]  floor          | floor_ll = 50.0*PSF

[s] Seismic and structural data
    Mapped spectral response accelerations
    5% critical damping
    Site Class D (IBC 1613.5)
    Seismic Source Type = A
    Distance to seismic source < 2 km

    [t] 0.2 sec-response (IBC 22.0)            | S_S = 2.1*G
    [t] 1.0 sec-response (IBC 11.5.1)          | S_1 = 0.93*G
    [t] standard occupancy                     | I_f = 1.0

    | **Structural materials**
    | Charpy V-notch toughness for all connections in the seismic-load-resisting system
    | Weld electrodes E70XX, 20 ft-lb at 0 degrees

    [t] wide flange,plate ASTM A992, Gr. 50   | F1_y = 50*KSI
    [t] HSS (round) ASTM A500, Grade B        | F2_y = 42*KSI
    [t] Plate ASTM A572, Grade 50             | F3_y = 50*KSI
    [t] lightweight conc fill fc'=3000 psi    | f1_c = 3*KSI

#- format | 3  | 1.0
#- 01  | 2  | PSF   | 3
# -02  | 3  | PSF   | 3
```

```
#- file
#- 01 | f | floorplan.png | Floor framing | 80 |

# This file contains a on-c-e public domain template (the template).
# The template is distributed under the CC0 1.0 Public Domain Dedication
# at http://creativecommons.org/publicdomain/zero/1.0/
# The template is not a structural design calculation.
# The template user assumes sole responsibility for all inputs and results.
```

```
[s] Seismic Loads
    | Design Example 1A - Special Concentrically Braced Frame
    | 2006 IBC Structural/Seismic Design Manual, Vol. 3

        #- 01 insert frame figure

[s] Site ground motion
    Maximum considered earthquake spectral response accelerations for short periods
    and at 1.0-second period adjusted for site class effects. 5 percent-damped
    design spectral response accelerations at short periods and at 1.0-second period,
    SD1, adjusted for site class effects.

    [t] Eq 11.4-1                     | F_a = 1.0
    [t] Eq 11.4-2                     | F_v = 1.5
    [t] short period spectral accel | S_S = 2.1
    [t] 1 second spectral accel     | S_1 = 0.93

    [e] Eq 11.4-1 (g) #- 01
        S_MS = F_a*S_S

    [e] Eq 11.4-2 (g) #- 01
        S_M1 = F_v*S_1

    [e] Eq 11.4-3 (g) #- 01
        S_DS = (2 * S_MS) / 3

    [e] Eq 11.4-4 (g) #- 01
        S_D1 = (2 * S_M1) / 3

[s] Base shear coefficients
    The approximate fundamental building period can be computed from the
    building height and the structural system (IBC 12.2-1).

    [t] building height (no dimension)          | h_n = 62
    [t] building height                         | bldg_ht = 62*FT
    [t] response mod coef (T 12.2-1)            | R_0 = 6.0
    [t] system overstrength factor              | Q_0 = 2.0
    [t] deflection amplif factor                | C_d = 5.0
    [t] period coefficient - steel              | C_T = 0.02
    [t] importance factor                       | I_f = 1.0

    [e] code approximate fundamental period #- 02
        T_a = C_T * h_n ** (3/4)

    [e] basic seismic response coefficient (EQ 12.8-2) #- 02
        C_Sbasic = S_DS / (R_0 / I_f)

    [e] coefficient need not exceed (EQ 12.8-3) #- 02
        C_Smax = S_D1 / (I_f * T_a/ R_0)

    [e] coefficient minimum (EQ 12.8-3) #- 02
        C_Smin = 0.044 * S_DS * I_f

    [e] bounded seismic response coefficient (EQ 12.8-2) #- 02
        C_S = min(max(C_Sbasic, C_Smin),C_Smax)

    Note that for buildings in Seismic Design Category E or F, and those buildings
    for which the 1.0-second spectral response, S1, is equal to or greater than 0.6g,
    the value of the seismic response coefficient, CS, must be at least equal to the
    following value.

    [e] min seismic response coefficient for SDC E and F (EQ 12.8-2)
        C_Emin = 0.5 * S_DS / (R_0 / I_f)


[s] Vertical earthquake effects
```

```
    The vertical component of seismic acceleration is also computed using a very low
    period for the spectral acceleration.

    [t] dead load                                    | DL = 3*KIP
    [t] base shear effects on vertical               | Q_E = 1*KIP
    [t] live load effects                            | f_1L = 2*KIP
    [t] capacity coefficient                         | Omega_0 = 2.0
    [t] redundancy factor (IBC 12.3.4)               | rho = 1.0

    [e] plus vertical earthquake effect Eq 12.4-2  #- 02
        E_p = rho * Q_E + 0.2 * S_DS * DL

    [e] minus vertical earthquake effect Eq 12.4-2  #- 02
        E_m = rho * Q_E - 0.2 * S_DS * DL

    [e] maximum vertical earthquake effect Eq 12.4-2  #- 02
        Em_p = Omega_0 * Q_E +  0.2 * S_DS * DL

    [e] maximum vertical earthquake effect Eq 12.4-2  #- 02
        Em_m = Omega_0 * Q_E -  0.2 * S_DS * DL

#- format | 3    |  1.0
#-    01  | 2    |       | 3
#-    02  | 2    |       | 3
#-    03  | 3    |       | 1


#- file
#- 01 | f | brace.png | Braced frame elevation | 80 |
```

```
[s] Base shear and vertical distribution
    The floor area at each level is 32,224 square feet. The perimeter of the exterior
    curtain wal

    l is 728 feet. The roof parapet height is 4 feet. The curtain-wall
    weights are distributed to each floor by tributary height.  Values are organized
    from top to bottom (floors 5 to 1).


        #- 02 insert building figure


    [t] wall length [ft]        | wall_length = 728.0
    [t] wall unit weight [psf]  | wallUnitWt = 20.0
    [t] floor area  [sf]        | floor_area = 32224.0
    [t] floor unit weight [psf] | floor_uweight = array([74.0] + [76.0]*4)
    [t] wall height [ft]        | wall_ht = array([10.0] + [12.0]*3 + [13.0])

    [a] Table - floor weight (kips)  #- 90
        floor = ['roof',5,4,3,2]
        floorWeight = (1/1000) * floor_area * floor_uweight

    [a] Table - wall weight (kips)   #- 90
        floor = ['roof',5,4,3,2]
        wallWeight = (1/1000.) * wall_length * wall_ht * wallUnitWt

    [a] Table - story weight (kips)   #- 90
        floor = ['roof',5,4,3,2]
        storyWeight = floorWeight + wallWeight

    [e] sum of story weights (kips)   #- 03
        totalStorywt = sum(storyWeight)*KIPS

        Import loads from model 0402

        #- 01 import model

[s] Design base shear V_1 = C_S * W (Eq 12.8-1)

    [e] total base shear  #- 23
        V_1 = C_S  * totalStorywt

[s] Vertical distribution of shear
    The total lateral force (i.e., design base shear) is distributed over the height
    of the building in accordance with IBC 12.8.3.
    k is a distribution exponent related to building period as follows:
    k = 1 for buildings with a period of 0.5 second or less
    k = 2 for buildings with a period of 2.5 seconds or greater
    k = is interpolated between these values for buildings with period between 0.5
    second and 2.5 seconds

    [t] height above ground [ft]      | h_x = array([62, 50, 38, 26, 14])

    [e] story weight [kips]  #- 21
            w_x = array(storyWeight)

    [a] Table - w_x h_x (kip-ft)  #- 90
            floor = ['roof',5,4,3,2]
            wxhx = w_x * h_x

    [e] sum of w_x h_x (kips)    #- 01
            wxhx_sum = sum(wxhx)

    Vertical distribution coefficient

    [y]  C_vx = (w_x * h_x__k) / Sum(w_x * h_x__k ,(i, 1, n))
```

```
    [a] Table - Percent vertical load distribution - C_vx   #- 90
        floor = ['roof',5,4,3,2]
        C_dist = wxhx * (1/wxhx_sum) * 100

    [a] Table - Story Force F_x (kips)   #- 90
            floor = ['roof',5,4,3,2]
            F_x = C_dist * V_1 / 100

    [a] Table - Story Shears (kips)   #- 90
            floor = ['roof',5,4,3,2]
        V_x = [F_x[0], sum(F_x[0:2]), sum(F_x[0:3]), sum(F_x[0:4]), sum(F_x[0:5])]

    [e] base shear (kips)   #- 01
            Vbase = sum(F_x)

[s] Horizontal distribution of shear
    IBC 12.8.4.2 requires inclusion of an accidental torsional eccentricity, e, equal
    to 5 percent of the building dimension perpendicular to the direction of force.
    It is assumed that the four frames in the transverse direction are each 25
    percent stiffer than the six in the longitudinal direction because they are more
    heavily loaded.

    [t] e/w c.m. distance to frame          | d_ew = 105*FT
    [t] n/s c.m. distance to frame          | d_ns = 75*FT
    [t] accidental ecc  e/w                 | e_ew = 0.05 * 2*d_ew
    [t] accidental ecc  n/s                 | e_ns = 0.05 * 2*d_ns
    [t] rel stiffness-trans frames          | R_i = 1.25

[s] Transverse frames-shear plus torsion

    [e] rel. torsional resistance - 4 transverse frames  #- 03
            transR_i = R_i * d_ew**2

    [e] rel. torsional resistance - 6 longitudinal frames  #- 03
            longR_i = d_ns**2

    [e] sum of rel. torsion resistances
            SumR_i = 4*transR_i + 6*longR_i

    [e] Percent torsion force at single transverse frame  #- 23
            Ttrans_y = 100 * (e_ew / d_ew) * transR_i / SumR_i

    [e] Percent torsion force at single transverse frame  #- 23
            Tlong_y = 100 * (e_ew / d_ns)  * longR_i / SumR_i

    Torsion distribution relation

    [y] Viy = Sum(Vy,(i,0,n))*(Riy/Sum(Riy,(i,0,n))+(e_real*Ri*di)/(Sum(Ri*di**2,(i,0,n)))
                      + Abs((e_acc*Ri*di)/(Sum(Ri*di**2,(i,0,n)))))

    [a] Table - Forces at each level at frame BF-2 (kips)  #- 90
            floor = ['roof',5,4,3,2]
            F_xbf2 = F_x * ((1/4) + Ttrans_y/100)

    [a] Table - Story shears at each level at frame BF-2 (kips)  #- 90
            floor = ['roof',5,4,3,2]
            V_xbf2 = array(V_x) * ((1/4) + Ttrans_y/100)

[s] Combined forces (IBC 12.4.2)
    For convenience, the seismic design load combinations can be reformulated to
    combine the vertical component of the seismic motion with the dead load. Also,
    as the overstrength factor is not considered in combination with the
    redundancy factor (and the results of the analysis include the redundancy factor),
    it is sometimes convenient to recalculate the overstrength factor to account
    for this difference (if rho > 1.0).
```

```
    [e] combined effects - redundancy  #- 13
            E2 = rho * Q_E - 0.28 * DL

    [e] combined effects - overstrength  #- 13
            E1_m = Omega_0 * Q_E + 0.28 * DL

    [e] combined effects - overstrength  #- 13
            E2_m = Omega_0 * Q_E - 0.28 * DL

    Basic Seismic Load Combinations:

    [e] 1.2D+ 1.0E + 0.5L (Eq 16-5)  #- 13
            E1_B = 1.48*DL + rho*Q_E + f_1L

    [e] 0.9 D + 1.0 E (Eq 16-7)  #- 13
            E2_B = 0.62*DL + rho*Q_E

    Special Seismic Load Combinations (Amplified Seismic Load)

    [e] 1.2D + 1.0Em + 0.5L (Eq 16-22)  #- 13
            E1_M = 1.48*DL + 2.0*Q_E + f_1L

    [e] 0.9D + 1.0Em (Eq 16-23)  #- 13
            E2_M = 0.62*DL + 2.0*Q_E

    The formulae that include the redundancy factor are used for brace design, and
    those that do not include it are used for column and beam design. The forces that
    the framing members are likely to see are greater for a frame with larger braces
    than for one with smaller ones, and the same overstrength factor should be applied
    to the two frames.

    It is apparent from the reformulations that the redundancy factor actually
    decreases the effective overstrength required of these members and makes
    unfavorable yield modes more likely. A redundancy factor of 1.3 decreases the
    effective overstrength factor to 1.54, an extremely low value for a braced frame;
    use of such a low value in determining column design forces may not be sufficient
    to preclude column buckling. Designers should not lose sight of the
    purpose of the required overstrength, even when code provisions permit its
    reduction.

#- format | 3,3 |  1.0
#-    03| 0,0    |       | 3
#-    13| 0,0   |       | 3
#-    23| 2,0    |       | 3
#-    33| 3,0    |       | 3
#-    02| 0,0    |       | 2
#-    12| 1,0    |       | 2
#-    22| 2,0    |       | 2
#-    32| 3,0    |       | 2
#-    01| 0,0   |       | 1
#-    11| 1,0     |       | 1
#-    21| 1,0   |       | 1
#-    31| 3,0    |       | 1
#-    90| 0,0    | lvl   | 3
#-    91| 2,0    | lvl   | 3
#-    92| 0,0    | frm   | 3


#- file
#- 01 | i | 0402.seismic.txt | | |
#- 02 | f | bldg.png | Building Structural Scheme  | 100 |
```

```
# The template is not a structural design calculation.
# The template user assumes sole responsibility for all inputs and results.
```

```
[s] Frame analysis
    Small eccentricities occur at the connections to facilitate detailing.
    These eccentricities result in flexural forces in the framing
    members, as well as larger drifts than those for frames without eccentricities. It is
    therefore necessary to include the eccentricities in the frame model.
    Using the frame forces from Table 1 A4, the frame shown in Figure 1A-5 is analyzed.
    Columns are modeled as fixed at the base and braces as pinned at each end. Although
    in-plane rotation of the gusseted beam-to-column connection may induce some
    flexural forces in the brace, these are small at the elastic drift level. Beams are
    modeled as having fixed connections at their gusseted connections to the columns;
    these details include beam-to-column joint flange welds. (AISC 341, 7.2)

[s] Story drifts
    In this design example a second-order analysis has been
    performed, and the B2 amplification factor is not required.
    The displacements from the model are converted to design story drifts and compared
    to the allowable drift from ASCE/SEI 7-05 Table 12.2-1. The story drifts are
    calculated from the displacements, and divided by the reliability/redundancy factor,
    rho and I. The drifts from the elastic analysis are multiplied by the displacement
    amplification factor, Cd. The design story drift for each level of this frame is less
    than the allowable drift. Because the building design is symmetrical and
    has rigid diaphragms, the average story drift will not exceed the drift of this frame.
    Thus, the building will meet the drift limitations of Table 12.12-1.

    [t] response mod coef (T 12.2-1)   | R_1 = 6.0
    [t] story height (inch)            | h_sx = array([12]*4+[14]) * 12
    [t] model displacement (inch)      | disp_xe = array([2.19, 1.70, 1.24, 0.77, 0.35])
    [t] model drift (inch)             | Delta_xe = array([0.50, 0.45, 0.47, 0.43, 0.35])

    [e] displacement amplification factor #- 23
        C_d = R_1 * 0.7

#page

    [a] design story drift (inch)  #- 91
        story = ['roof',4,3,2,1]
        delta_x = C_d * Delta_xe

    [a] allowable story drift (inch)  #- 91
        story = ['roof',4,3,2,1]
        delta_Ax = .020 * h_sx


#- format |  2,2  |  1.0
#- 23  |  2,2    |       | 3
#- 33  |  3,3    |       | 3
#- 02  |  0,0    |       | 2
#- 90  |  0,0    | lvl   | 3
#- 91  |  1,0    | stry  | 3
#- 92  |  0,0    | frm   | 3
```

```
[s] Brace design
    For this example the fourth floor brace will be designed. From the analysis,
    the earthquake force, rhoP_E, on the brace is known.

    [t] dl compression                         | PD = 8000*KIPS
    [t] ll compression                         | PL = 6*KIPS
    [t] eq tension                             | Q_PEt = -328*KIPS
    [t] eq compression                         | Q_PEc = 328*KIPS
    [t] non-assembly,-parking < 100 psf        | f1 = 0.5
    [t] redundancy factor                      | rho = 1.0
    [t] pi                                     | pi = 3.14

    [e] max compression 1.2 D + 1.0 E + 0.5L (Eq 16-5) #- 13
        P_dc = 1.48*PD + PL*f1 + rho*Q_PEc

    [e] max tension 0.9 D + 1.0 (Eq 6-7)  #- 13
        P_dt = 0.62*PD + rho*Q_PEt/f1

[s] Brace compression strength
    The distance from work-point to work-point is 17.9 feet. The ductile
    detailing requirements for SCBF result in the provision of a hinge zone
    in the connection; it is assumed here that the hinge zone will be at least
    9 inches from the work point at each end, resulting in a reduction of the brace
    length of 18 inches. The brace length = 197 inches. HSS 9.625x0.500 will be
    used. Section properties are from the 13th Edition LRFD Manual.

    [t] boundary condition coefficient  | K_1 = 1.0
    [t] unbraced lenght                 | l_1 = 197*IN
    [t] radius of gyration              | r_1 = 3.24*IN
    [t] elastic modulus                 | E_1 = 29000*KSI
    [t] yield stress                    | F_y = 42*KSI
    [t] cross section area              | A_g = 13.4*IN**2
    [t] HSS section depth               | D_e = 9.625*IN
    [t] HSS section thickness           | t_e = 0.500*IN

    [e] Slenderness ratio #- 03
        Kl_r = K_1 * l_1 / r_1

    [e] Buckling stress #- 43
        F_e = pi**2 * E_1 / (K_1*l_1/r_1)**2

    [e] Critical buckling stress (AISC 360, E3-2) #- 43
        F_cr = F_y * 0.658**(F_y/F_e)

    [e] Compression strength (AISC 360 E3-1) #- 13
        phiP_c = 0.9 * A_g *F_cr

    [c] Check brace slenderness limit (AISC 341-13.2a) | ok | 2
        K_1 * l_1/r_1  | <= | 4 * sqrt(E_1/F_y)

    Note that under the exception, braces with Kl/r < 200 are allowed if capacity
    design is used for columns

    [c] Check brace width-to-thickness (AISC 341-13.2d, T 1.8.1) | ok | 2
        D_e  / t_e | <= | 0.44*E_1 / F_y

[s] Brace tension strength

    [e] Design tension strength (AISC 360, D1-1) #- 13
        phiP_t = 0.9 * F_y * A_g

    [t] tension demand on brace | P_u = 350*KIPS

    [c] Check tension strength | ok | 2
        P_u / phiP_t | <= | 1.0
```

```
    Equation D1-2, which applies to fracture of the net section, is checked in the design
    of the connection.

[s] Distribution of force between braces
    AISC 341-13.2c does not permit a major unbalance between braces working in
    compression and those working in tension to resist lateral loads. The frames in this
    structure are symmetrical and no single-diagonal frames are used. An elastic analysis
    shows that the seismic force resisted by compression is identical to that resisted by
    tension and the frame complies with the requirement. The tension strength of
    braces is similar in each direction, so that even after degradation of
    the compression strength of braces there is no significant tendency to accumulate
    drift in one direction.


#- format | 3,3 |  1.0
#- 03| 2,0   |       | 3
#- 13| 2,1   | KIPS  | 3
#- 23| 2,1   |       | 3
#- 43| 2,1   |  KSI  | 3


# This file contains a generic on-c-e model template (the template).
# The template is distributed under the CCO 1.0 Public Domain Dedication
# http://creativecommons.org/publicdomain/zero/1.0/
# The template is not a structural design calculation.
# The template user assumes sole responsibility for all model inputs and results.
```

```
[s] Example 5 - Structure Dynamics
    Example E12-1 from Cough and Penzien *Dynamics of Structures* is used to
    illustrate application of the *numpy* linear algebra libraries in **on-c-e**.
    See *numpy* documentation for futher details.

        #- 01 insert frame plot

[s] Define mass and stiffness
    units: kips, inches

    [t] mass        | m_1 = array([[1.0,0,0],[0,1.5,0],[0,0,2.0]])
    [t] stiffness   | k_1 = 600*array([[1,-1,0.0],[-1,3,-2],[0,-2,5]])

    [e] flexibility matrix  #- 01
        flex = LA.inv(k_1)

    [e] dynamic matrix  #- 01
        dyna = inner(flex, m_1)

[s] Eigenvalue analysis

    [e] eigenvalues  #- 01
        eigenvals = LA.eig(dyna)[0]

    [e] natural frequency (secs)  #- 02
        omega_1 = 1 / (eigenvals**.5)

    [e] eigenvectors    #- 03
        evects_0 = LA.eig(dyna)[1]

        #- 02 import functions from script

    [f] normalized eigenvectors
        norm_evects(evects_0, eigenvals) |  evectors_1

[s] Plot mode shapes

    [f] plot mode shapes
        plot_modes(evectors_1) | plot_1

        #- 03 insert mode plot
#page

#- format | 2,2 | 1.0
#- 01 | 4,4 |  | 2
#- 02 | 4,2 |  | 2
#- 03 |     |  | 2

#- file
#- 01 | f | frame.png |  From Clough and Penzien - page 178 | 90 |
#- 02 | s | eigen.py  |  | |
#- 03 | f |  mode_shapes.png | Mode Shapes | 90 |
```

```
[s] Example 0601 - Analyze 3-Bar truss using OpenSees
    | OpenSees Primer Example 1.1
    | Displacement results for two different geometries
    | Units: kips, in, sec

        #- 01 insert figure

[s] set up opensees geometry for run 1
    | change node 4 location using a string template
    | run OpenSees model

    [t] node 4 x (in)   |  n4x1 = 70.0
    [t] node 4 y (in)   |  n4y1 = 45.0

        #- 02 read functions from script
        #- 03 edit opensees file

    [f] plot OpenSees geometry - run 1
        osplot("c:/opensees/trusscopy1.tcl") | plot 1

        #- 04 insert figure

[s] set up opensees geometry for run 2
    | change node 4 location using a static string - no variable substition
    | run OpenSees model

        #- 05 edit opensees file

    [f] plot OpenSees geometry - run 2
        osplot("c:/opensees/trusscopy2.tcl") | plot_2

        #- 06 insert figure

[s] run models 1 and 2 and write output tables

        #- 07 run opensees model 1
        #- 08 read displacements
        #- 09 read forces
        #- 10 run opensees model 2
        #- 11 read displacements
        #- 12 read forces

    [a] Table of Displacements-Node 4 (in) #- 02
        clabel = ['load no.', 'x', 'y']
        runa = [1,2]
        delta = [[deltaxy1], [deltaxy2]]

    [a] Table of Axial forces (kips) #- 03
        elements = [1,2,3]
        runb = [1,2]
        forces = [[forceaxial1], [forceaxial2]]


#- format | 3,2 | 1.0
#- 01 | 3,3 |         | 3
#- 02 | 2,2 |         | run
#- 03 | 2,2 | ele no. | run


#- file
#- 01 | f | truss_1.png                       | Node and element numbers | 65 |
#- 02 | s | plot_osgeom.py                    | | |
#- 03 | e | c:/opensees/truss.tcl             | copy1 | |
#-   13 | node 4 %n4x1 %n4y1
#-   55 | recorder Node -file example_disp1.out -load -node 4 -dof 1 2 disp
#-   56 | recorder Element -file example_force1.out  -eleRange 1 3 axialForce
```

```
#--
#- 04 | f | trusscopy1fig.png                 | Truss geometry - run 1 | 80 |
#- 05 | e | c:/opensees/truss.tcl             | copy2 | |
#-   13 | node 4 90.1 225.5
#-   55 | recorder Node -file example_disp2.out -load -node 4 -dof 1 2 disp
#-   56 | recorder Element -file example_force2.out  -eleRange 1 3 axialForce
#--
#- 06 | f | trusscopy2fig.png                 | Truss geometry - run 2| 50 |
#- 07 | o | opensees_run1.cmd                 | | |
#- 08 | r | c:/opensees/example_disp1.out    | deltaxy1    | * |
#- 09 | r | c:/opensees/example_force1.out   | forceaxial1 | * |
#- 10 | o | opensees_run2.cmd                 | | |
#- 11 | r | c:/opensees/example_disp2.out    | deltaxy2    | * |
#- 12 | r | c:/opensees/example_force2.out   | forceaxial2 | * |
```

```
[s] Example 7 - Stiffness Method
    Examples 3.4 and 3.5 from McQuire and Gallagher *Matrix Structural Analysis*
    demonstrate applications of the numpy linear algebra library
    and unit manipulations.

        #- 01 insert geometry figure
        #- 02 insert loads figure

[s] Define geometry and material properties
    units: kN, mm

    **Element Properties**
    [t] elastic modulus (kN/mm**2)  | e_1 = 200000*MPA.asUnit(KN/MM**2).asNumber()
    [t] cross section 1 (mm**2)     | a_1 = 10000*(MM**2).asNumber()
    [t] cross section 2 (mm**2)     | a_2 = 15000*(MM**2).asNumber()

    **Node coordinates** [node number, x, y]
    [t] node 0      | n_0  = [0,   0.,      0.]
    [t] node 1      | n_1  = [1,   5000.,   0.]
    [t] node 2      | n_2  = [2,   5000.,  8660.]
    [t] node 3      | n_3  = [3,   10000., 8660.]

    [e] node coordinates        #- 02
        nodes = [n_0, n_1, n_2, n_3]

    **Element Connectivity**  [element number, node1, node2, area, modulus]
    [e]  element 1   #- 01
        el_1 = [1,0,1,a_1,e_1]

    [e] element 2    #- 01
        el_2 = [2,1,3,a_2,e_1]

    [e] element 3    #- 01
        el_3 = [3,2,3,a_1,e_1]

    [e] element 4    #- 01
        el_4 = [4,0,2,a_2,e_1]

    [e] element 5    #- 01
        el_5 = [5,1,2,a_2,e_1]

    [e] element connectivity  #- 02
        elements = [el_1, el_2, el_3, el_4, el_5]

    **Forces and reactions**

    [t] releases 0 or 1 [node, x, y] | react =  [[0, 1, 1],[1, 0, 1]]

    [t] applied forces  [node, x, y] | forces = [[3, 283., -283.]]

[s] Analyze truss
    Intermediate results may be viewed in standard out by uncommenting print
    statements in the file *func_stiff.py*.

        #- 03 read function file

[f] Direct stiffness analysis
    direct_stiff(nodes, elements, react, forces) | result_1

        #- 04 geometry figure
        #- 05 geometry figure
        #- 06 geometry figure

#- format | 3,3 | 1.0
#- 01 | 2,2 |     | 0
#- 02 | 2,2 |     | 1
```

```
#- file
#- 01 | f | truss1.png | Frame geometry | 45 |
#- 02 | f | truss2.png | Loads          | 45 |
#- 03 | s | func_stiff.py | | |
#- 04 | f | geom1.png | Examples 3.4 and 3.5 from McQuire and Gallagher | 60 |
#- 05 | f | geom2.png | Examples 3.4 and 3.5 from McQuire and Gallagher | 60 |
#- 06 | f | geom3.png | Examples 3.4 and 3.5 from McQuire and Gallagher | 60 |
```