```
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==========================================================================================
[1] Shear distribution                                                            [0403]
==========================================================================================
    Import model 0402 for loads


==========================================================================================
[2] 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 >


==========================================================================================
[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_MS = 2.100
-----------------------------------------------------------------------------------------┘

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

$F_v \cdot S_1$

$1.500 \cdot 0.930$

```
                                                                          S_M1 = 1.395
-----------------------------------------------------------------------------------------┘

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

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

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

```
                                                                    S_DS = 1.400
-------------------------------------------------------------------------------┘

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

2·S_M1
──────
  3


2·1.395
───────
   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 | approximate fundamental period [4.1]

       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) [4.2]

I_f·S_DS
────────
   R_0


1.000·1.400
  ─────────
   6.000

                                                                 C_Sbasic = 0.233
-------------------------------------------------------------------------------┘

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

R_0·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) [4.4]

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) [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]

0.5·I_f·S_DS
────────────
     R₀


0.5·1.000·1.400
  ────────────
     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] Base shear and vertical distribution                                            [0403]
==========================================================================================
    The floor area at each level is 32,224 square feet. The perimeter of the exterior
    curtain wall 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).


    floor area  [sf]              | floor_area =
[ 32224.  32224.  32224.  32224.  32224.]
    floor unit weight [psf]       | floor_uweight =
[ 74.  76.  76.  76.  76.]
    wall length [ft]              | wall_length =
[ 728.  728.  728.  728.  728.]
    wall height [ft]              | wall_ht =
[ 10.  12.  12.  12.  13.]
    wall unit weight [psf]        | wall_uweight =
[ 20.  20.  20.  20.  20.]
```

```
---------------------------------------------------------------------------------┐
                                                Table - floor weight (kips) [6.1]

range variables: floor_weight = (1/1000) * floor_area * floor_uweight
equation:

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) [6.2]

range variables: wall_weight = (1/1000) * wall_length * wall_ht * wall_uweight
equation:

0.001·wall_ht·wall_length·wall_uweight

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

---------------------------------------------------------------------------------┐
                                                Table - story weight (kips) [6.3]

range variables: story_weight = floor_weight + wall_weight
equation: story_weight = floor_weight + wall_weight

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

---------------------------------------------------------------------------------┐
                                   total_storywt | sum of story weights (kips) [6.4]

                                                  total_storywt = 13039.712 kips
---------------------------------------------------------------------------------┘

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



---------------------------------------------------------------------------------┐
                                                    V_1 | total base shear [7.1]

C_S·total_storywt


0.233·13039.712 kips

                                                            V_1 = 3042.599 kips
```

Figure 2. Building Structural Scheme <file: bldg.png >

[8] 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] [8.1]

w_x =

[2530.176 2623.744 2623.744 2623.744 2638.304]

Table - w_x h_x (kip-ft) [8.2]

range variables: wxhx = w_x * h_x
equation:

$h_x \cdot w_x$

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

wxhx_sum | sum of w_x h_x (kips) [8.3]

wxhx_sum = 492914

Vertical distribution coefficient

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

Table - Percent vertical load distribution - C_vx [8.4]

range variables: C_dist = wxhx * (1/wxhx_sum) * 100

```
equation: 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) [8.5]

range variables: F_x = C_dist * V_1 / 100
equation: 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) [8.6]

range variables: 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])]
equation: 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
============  =========  =========  =========  =========
-------------------------------------------------------------------------------------┘

-------------------------------------------------------------------------------------┐
                                                      V_base | base shear (kips) [8.7]

                                                              V_base = 3042.599 kips
-------------------------------------------------------------------------------------┘


=======================================================================================
[9] 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 = 105.000 ft
    n/s c.m. distance to frame   | d_ns = 75.000 ft
    accidental ecc  e/w          | e_ew = 0.1*d_ew
    accidental ecc  n/s          | e_ns = 0.1*d_ns
    relative stiffness-trans frames | R_i = 1.25

=======================================================================================
[10] Transverse frames-shear plus torsion                                       [0403]
=======================================================================================



-------------------------------------------------------------------------------------┐
                        transR_i | rel. torsional resistance - 4 transverse frames [10.1]
```

$R_i \cdot d\_ew^2$

$1.250 \cdot 105.000\ ft^2$

transR_i = 13781.250 ft2

---

longR_i | rel. torsional resistance - 6 longitudinal frames [10.2]

d_ns^2

$75.000\ ft^2$

longR_i = 5625.000 ft2

SumR_i | sum of rel. torsion resistances [10.3]

$6 \cdot longR_i + 4 \cdot transR_i$

6·5625.000 ft2 + 4·13781.250 ft2

SumR_i = 88875.000 ft2

Ttrans_y | Percent torsion force at single transverse frame [10.4]

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

$$\frac{100 \cdot 10.500\ ft \cdot 13781.250\ ft2}{88875.000\ ft2 \cdot 105.000\ ft}$$

Ttrans_y = 1.551

Tlong_y | Percent torsion force at single transverse frame [10.5]

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

$$\frac{100 \cdot 10.500\ ft \cdot 5625.000\ ft2}{88875.000\ ft2 \cdot 75.000\ ft}$$

Tlong_y = 0.886

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) [10.6]

range variables: F_xbf2 = F_x * ((1/4) + Ttrans_y/100)
equation: 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) [10.7]

range variables: V_xbf2 = array(V_x) * ((1/4) + Ttrans_y/100)
equation: 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

[11] 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).

E1 | combined effects - redundancy [11.1]

0.28·DL + Q_E·ρ

0.28·3.000 kip + 1.000 kip·1.000

E1 = 1.840 kip

E2 | combined effects - redundancy [11.2]

-0.28·DL + Q_E·ρ

```
-0.28·3.000 kip + 1.000 kip·1.000

                                                                            E2 = 0.160 kip
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-----------------------------------------------------------------------------------------┐
                                                 E1_m | combined effects - overstrength [11.3]

0.28·DL + Ω₀·Q_E

0.28·3.000 kip + 2.000·1.000 kip

                                                                          E1_m = 2.840 kip
-----------------------------------------------------------------------------------------┘

-----------------------------------------------------------------------------------------┐
                                                 E2_m | combined effects - overstrength [11.4]

-0.28·DL + Ω₀·Q_E

-0.28·3.000 kip + 2.000·1.000 kip

                                                                          E2_m = 1.160 kip
-----------------------------------------------------------------------------------------┘

    Basic Seismic Load Combinations:

-----------------------------------------------------------------------------------------┐
                                                     E1_B | 1.2D+ 1.0E + 0.5L (Eq 16-5) [11.5]

1.48·DL + Q_E·ρ + f_1L

1.48·3.000 kip + 1.000 kip·1.000 + 2.000 kip

                                                                          E1_B = 7.440 kip
-----------------------------------------------------------------------------------------┘

-----------------------------------------------------------------------------------------┐
                                                         E2_B | 0.9 D + 1.0 E (Eq 16-7) [11.6]

0.62·DL + Q_E·ρ

0.62·3.000 kip + 1.000 kip·1.000

                                                                          E2_B = 2.860 kip
-----------------------------------------------------------------------------------------┘

    Special Seismic Load Combinations (Amplified Seismic Load)

-----------------------------------------------------------------------------------------┐
                                                  E1_M | 1.2D + 1.0Em + 0.5L (Eq 16-22) [11.7]

1.48·DL + 2.0·Q_E + f_1L

1.48·3.000 kip + 2.0·1.000 kip + 2.000 kip

                                                                          E1_M = 8.440 kip
-----------------------------------------------------------------------------------------┘

-----------------------------------------------------------------------------------------┐
```

E2_M | 0.9D + 1.0Em (Eq 16-23) [11.8]

$0.62 \cdot DL + 2.0 \cdot Q_E$

$0.62 \cdot 3.000\ \text{kip} + 2.0 \cdot 1.000\ \text{kip}$

$E2_M = 3.860\ \text{kip}$

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.