1.051 Structural Engineering Design

Problem Set 1 Solutions

Problem 1

Wind loads for a frame building

Location: Southern Hawaii
Type: Hospital building
Soil type: Stiff soil (195 ft)
Structural system: Concrete moment frame

Follow wind load calculation procedure given in ASCE 7 – 02

Building height = 65 ft, not a low rise building

Use the Analytical procedure described in ASCE 7–02 Section 6.5

1. Basic wind speed, V, and wind directionality factor, K_d

   V = 105 mph (Figure 6.1)

   K_d = 0.85 (Table 6.4)

2. Importance factor, I

   Building category: IV (Table 1-1)

   I = 1.15 (Table 6-1)

3. Exposure category and velocity pressure exposure coefficient, K_z

   Exposure category: C (Scattered obstructions having low heights, hurricane-prone region, conservative)

h (ft)	K_z
0-15	0.85
20	0.90
35	1.01
50	1.09
65	1.15

4. Topographic factor, K_{zt}

   Not applicable, $K_{zt}=1.0$

5. Gust effect factor

   Assume rigid building, use G = 0.85

6. Enclosure classification

Assume enclosed (information regarding openings not provided)

7. Internal pressure coefficient

GC_{pi} = m0.18 (partially enclosed building)

8. External pressure coefficient, C_p

Assume L/B=1
Windward wall: $C_p = 0.8$
Leeward wall: $C_p = -0.5$
Side walls: $C_p = -0.7$

9. Velocity pressure q_z or q_h

$$q_z = 0.00256 K_z K_{zt} K_d V^2 I$$

height	K_z	K_{zt}	K_d	V	I	q_z
0-15	0.85	1	0.85	105	1.15	23.5
20	0.90	1	0.85	105	1.15	24.8
35	1.01	1	0.85	105	1.15	27.9
50	1.09	1	0.85	105	1.15	30.1
65	1.15	1	0.85	105	1.15	31.7=q_h

10. Design wind pressure

$$p = qGC_p - q_i(GC_{pi})$$

	Windward wall				Leeward wall						
height	q_z (psi)	G	C_p	p_w (psi)	q_h (psi)	G	C_p	p_l (psi)	p (psi)	p_{av} (psi)	F (lb/ft)
0				15.9				-13.5	29.4	29.4	-
15	23.5	0.85	0.8	15.9	31.7	0.85	-0.5	-13.5	29.4	29.4	-
20	24.8	0.85	0.8	16.9	31.7	0.85	-0.5	-13.5	30.4	29.9	532.2
35	27.9	0.85	0.8	18.9	31.7	0.85	-0.5	-13.5	32.4	31.4	484.4
50	30.1	0.85	0.8	20.4	31.7	0.85	-0.5	-13.5	33.9	33.2	507.6
65	31.7	0.85	0.8	21.6	31.7	0.85	-0.5	-13.5	35.1	34.5	258.7

Problem 2

Earthquake loads for the same frame building

(a) Maximum considered earthquake parameters and design spectral response accelerations

Follow the procedure described in ASCE Section 9.4.12 or IBC Section 1615.1

1. Mapped maximum considered earthquake spectral response accelerations at short periods, S_S and at 1-second period, S_1

   $S_S = 267$ (%g)
   $S_1 = 123$ (%g) (ASCE Fig. 9.4.1.1(h))

2. Site class

   Stiff soil, Site class D

3. Maximum considered earthquake spectral response accelerations, adjusted for site Class effects, at short period, S_{MS} and at 1-sec period, S_{M1}

   $S_S = 2.67 > 1.25$ & Site Class D $\Rightarrow$ $F_a = 1.0$

   $S_1 = 1.25 > 0.5$ & Site Class D $\Rightarrow$ $F_v = 1.5$

   $S_{MS} = F_a S_S = 1.0(2.67) = 2.67$
   $S_{M1} = F_v S_1 = 1.5(1.23) = 1.84$

4. Design response spectral accelerations at short period, S_{DS} and at 1-sec period, S_{D1}

   $$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(2.67) = 1.78$$
   $$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(1.84) = 1.23$$

5. Construct the general response spectrum

   $$T_0 = 0.2\frac{S_{D1}}{S_{DS}} = 0.2\left(\frac{1.23}{1.78}\right) = 0.14$$
   $$T_S = \frac{S_{D1}}{S_{DS}} = \left(\frac{1.23}{1.78}\right) = 0.69$$

   Approximate period, T_a

   $T_a = C_t h_n^x$ where $C_t = 0.016$ and $x = 0.9$ (Table 9.5.5.3.2)

   $T_a = 0.016(65)^{0.9} = 0.69$ sec

   or $T_a = 0.1N = 0.1(4) = 0.4$ sec (conservative)

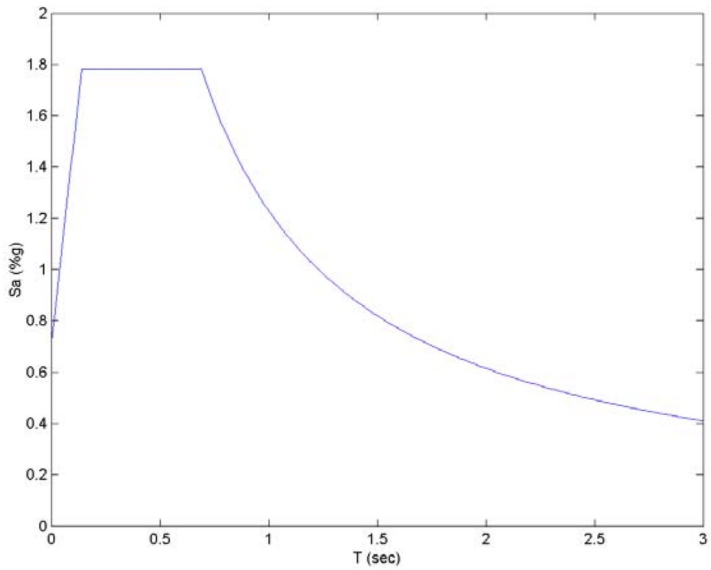
Use $T = T_a = 0.69$ sec (more accurate)

Construct the general response spectrum

$$T \le T_0 \qquad S_a = S_{DS}\left(0.4 + 0.6\frac{T}{T_0}\right) = 1.78\left(0.4 + 0.6\frac{T}{0.14}\right)$$

$$T_0 \le T \le T_s \qquad S_a = S_{DS} = 1.78$$

$$T > T_s \qquad S_a = \frac{S_{D1}}{T} = \frac{0.69}{T}$$



Note that it does not matter which calculated value of T_a is used.

(b) Determine the base shear

Seismic use group:	III	(hospital)
Seismic design category:	F	(SUG III & S_1>0.75g)

$\Rightarrow$ Use equivalent lateral force analysis (Table 9.5.2.5.1)

$V = C_s W$

$W = 3(120) + 60 = 420$ kips

$$C_s = \frac{S_{DS}}{R/I}$$

$R = 3$ (Ordinary moment resisting frame system

$$I = 1.5 \qquad \text{(SUG-III)}$$

$$C_s = \frac{1.78}{(3/1.5)} = 0.89$$

Make checks for max and min values

$$C_s^{max} = \frac{S_{D1}}{T(R/I)} = \frac{1.23}{0.69(3/1.5)} = 0.89 = C_s \qquad \text{OK}$$

$$C_s^{min} = 0.044 S_{DS} I = 0.044(1.78)(1.5) = 0.12 < 0.89 \qquad \text{OK}$$

$$C_s^{min} = \frac{0.5 S_1}{R/I} = \frac{0.5(1.23)}{(3/1.5)} = 0.31 < 0.89 \qquad \text{OK}$$

Base shear

$$V = 0.89(420) = 374 \text{ kips}$$

(c) Vertical distribution of forces

$$F_x = C_{vx} V$$

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

$$T = T_a = 0.69 \text{ sec} \quad \Rightarrow \quad 0.5<0.69<2.5$$

$$k = 1 + \frac{(0.69 - 0.5)}{(2.5 - 0.5)} = 1.1$$

Floor	w_i	h_i	$w_i h_i^k$	F_i
1	120	20	3238	50.4
2	120	35	5993	93.3
3	120	50	8873	138.1
4	60	65	5920	92.2
		Total	24024	374

$$\sum F_i = V \qquad \text{OK}$$

While the wind loads calculated according to IBC is slightly less than those calculated according to UBC-91, the earthquake forces have more than quadrupled. This stems from more accurate seismic mapping of the area, increased importance factor, added safety in the procedure etc.