SEISMIC DESIGN FOR REINFORCED MASONRY

M.A. Hatzinikolas

SPECIALTY CLINIC
Seismic Design for Reinforced Masonry Buildings

CSCE Annual Conference
Fredericton, N.B.

Introduction

The example given in this "Clinic" is for demonstration only. It can not be taken out of context and applied to other situations without engineering judgment. The purpose of this exercise is to demonstrate various code requirements and some available techniques applicable to the design of mid-rise reinforced masonry structures as it relates to lateral force.

It is assumed that the reader is familiar with the background of the code requirements for lateral load. If not, it is recommended that the participant take the time to become familiar with them prior to attempting to use this information as a design tool.

The example building used to demonstrate the distribution of lateral forces resulting from an earthquake is such as to have no complicating characteristics requiring thought and judgment far beyond the mere application of code requirements and formulae. Although no effort has been spared in an attempt to ensure that all data is factual, the Alberta Masonry Institute does not assume responsibility for errors or oversights resulting from the use of the information contained herein.

Example Building

For the purpose of this example, we will assume an apartment building of 47.6 x 21.6 meters in the plan dimensions and 10 stories high. The building which has no basement is shown in Figure 1. Figure 2 shows the structural elements (shear walls) and Figure 3 shows cross-section through the building. We will assume further that the height of the building is 10 x 2.60 = 26.0 meters (2.6 meters floor to floor).

For this example it is further assumed that all shear walls are 200 mm reinforced grouted concrete block (or giant brick) masonry for floors 7, 8, 9 and 10 and 300 mm for the 1, 2, 3, 4, 5, and 6 floors. For simplicity it is assumed that partition loads act over the entire floor area.

Calculation of Building Weight

For this calculation it is assumed that the walls consist of lightweight concrete blocks and similar grouting material (1850 kg/m^3 or 115 lb/ft^3). The weight of the hollow core slab system is assumed to be 2.69 kN/m^2 (58 psf) based on 200 mm thick slab with 5 mm gypcrete topping.

200 mm walls

weight per meter (foot) of wall per floor

3.6 kN/m x 2.4 m/floor = 8.6 kN/floor

(75 lb/ft x 8 ft/floor = 600 lb/floor)

- 300 mm walls

  weight per meter (foot) of wall per floor
  5.5 x 2.4 = 13.2 kN/floor
  (115 lb/ft x 8 ft/floor = 920 lb/floor)

Note that the wall weights listed in Table 1 are total weights and not weights per floor. For example:

Wall 1: 330.25 = 8.6 x 4 x 9.6
760.30 = 13.2 x 6 x 9.6

Where 4 is the number of floors where 200 mm wall is used and 6 the number of floors where 300 mm wall is assumed.

Weight of all shear walls

= 7540.3 x 17359.9

= 24900 kN or (5597.8 kips)

Take total weight of floor systems (including roof)

W = 10 x (2.64 + 0.15) 47.6 x 21.6

= 28685.6 kN or (6448.8 kips)

Total weight of partitions

W = 10 x (0.95) x 47.6 x 21.6

= 9767.5 kN or (2195.8 kips)

Parapet walls (1 meter high grouted 200 mm thick wall)

[(2 x 47.6) + (2 x 21.6)]3.6

= 498.25 kN or (112 kips)

The total structure weight to be considered for earthquake loading is:

24900 + 28685.6 + 9767.5 + 498.25 = 63851 kN
(5597.8 + 6448.8 + 2195.8 + 112 = 14354.4 kips)

TABLE 1. Total Wall Weight

Walls	Length Meters	W (kN) (200 mm)	W (kN) (300 mm)
1	9.6	330.25	760.3
2	9.6	330.25	760.3
3	7.0	240.80	554.4
4	7.0	240.80	554.5
5	8.2	282.00	649.4
6	8.2	282.00	649.4
7	8.2	282.00	649.4
8	8.2	282.00	649.4
9	2.4	82.50	190.0
10	5.8	199.50	459.3
11	8.2	282.00	649.4
12	2.4	82.50	190.0
13	8.2	282.00	649.4
14	8.2	282.00	649.4
15	7.0	240.80	554.4
16	7.0	240.80	554.4
17	9.6	330.25	760.3
18	9.6	330.25	760.3
19	7.6	261.40	602.0
20	6.8	234.00	538.5
21	6.8	234.00	538.5
22	6.8	234.00	538.5
23	6.8	234.00	538.5
24	7.6	261.40	602.0
25	7.6	261.40	602.0
26	6.8	234.00	538.5
27	6.8	234.00	538.5
28	6.8	234.00	538.5
29	6.8	234.00	538.5
30	7.6	261.40	602.0
		7540.30	17359.9

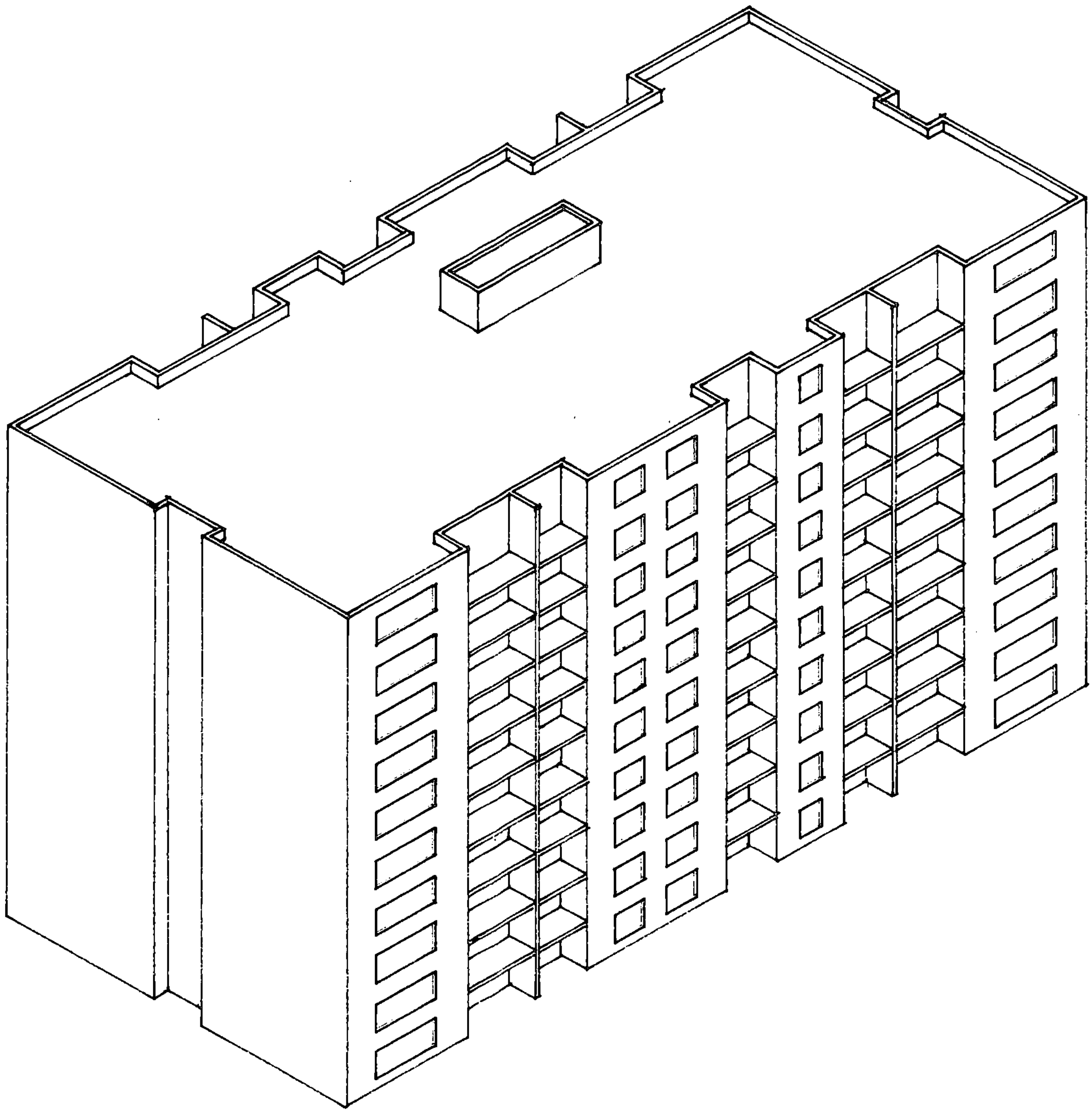


FIGURE I

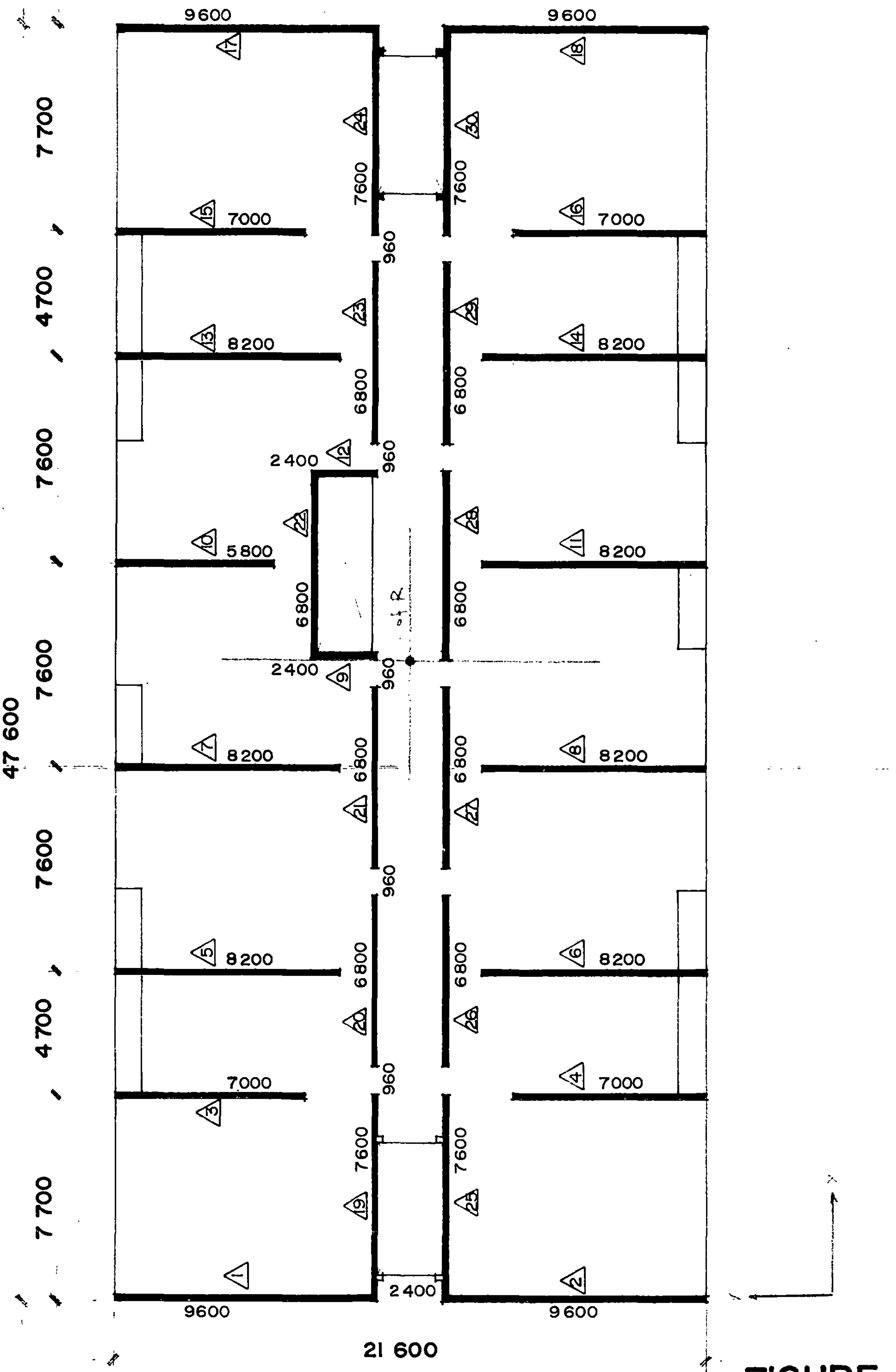

9600
9600
7 700
4 700
7600
7600
7600
4 700
7 700
47 600
7000
8200
5800
8200
8200
7000
7600
6800
960
2400
2400
21 600
9600
9600

FIGURE 2

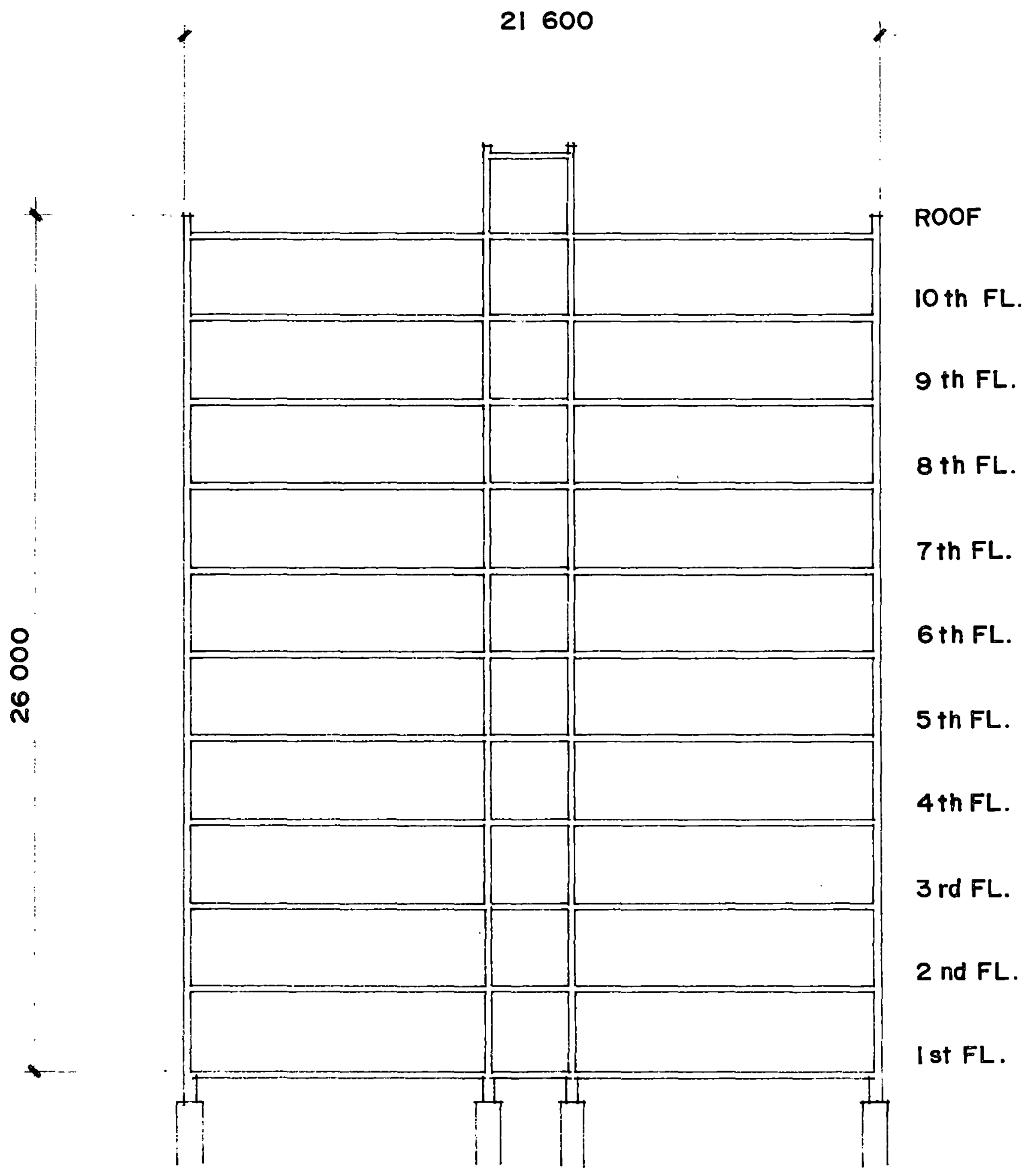

21 600
26 000
ROOF
10 th FL.
9 th FL.
8 th FL.
7 th FL.
6 th FL.
5 th FL.
4 th FL.
3 rd FL.
2 nd FL.
1 st FL.

FIGURE 3

General Design Considerations

1. For bearing wall buildings, it is desirable to carry the bearing walls through uninterrupted from foundation to roof. This means that apartment configuration should "stack" from floor to floor, garages below or "tuck-under" parking should be avoided and public room should be in side buildings or, if necessary, in the top level.
2. Symmetrical placement of shear walls is important. The code requires that walls be designed to resist the effects of a minimum torsional moment of 5% based upon the maximum dimension of the building. Poorly located shear walls which create large eccentricities can cause difficulties in design and result in buildings more vulnerable to earthquake damage than they may seem to be.
3. Walls, where possible, should be approximately equal in length and configured to provide a flange at each end to aid in resisting overturning forces. In our example, we have deliberately avoided the flange condition in order to simplify calculations. In most buildings, the flanged shape for shear walls may not fit a convenient pattern, so this simplifying assumption is not too much different from actual practice. The real world will dictate that all of the ideal conditions for effective seismic resistance will be impossible to achieve. Engineering judgment will dictate how far it is safe to stray from the ideal configuration and what modifications

in design procedures will be required to compensate for the differences.

Our example building has slightly asymmetrical walls in both directions and has identical floor plans on all floors. The positioning of the elevator shaft creates an eccentricity which by observation, is less than the minimum required by code. We will compare them to the minimum required eccentricity and in order to demonstrate the method.

Base Shear

Although both wind and earthquake forces must be evaluated and the most critical one is to be used in evaluating its effects on the building, for this example it is assumed that the building is located in earthquake zone 3 and that seismic load governs. The formula for computing seismic forces is:

$$V = A.S.K.I.F.W. \qquad (1)$$

(Sub-section 4.1.9 "Effects of Earthquakes" of NBC 1977)

$$A = 0.08$$

$$S = \frac{0.5}{3\sqrt{T}} = \frac{0.5}{0.79} = 0.630$$

$$T = \frac{0.05\,h}{\sqrt{D}}$$

$$K = 1.3$$

$$I = 1.0$$

$F = 1.3$

$W = 63851$ kN (14354 kips)

$$T \text{ long direction} = \frac{0.05 \times 26 \times 3.2808}{\sqrt{47.6} \times 3.2808} = 0.34$$

$$T \text{ short direction} = \frac{0.05 \times 26 \times 3.2808}{\sqrt{21.6} \times 3.2808} = 0.50$$

Maximum lateral load (base shear)

$V = 0.08 \times 0.630 \times 1.3 \times 1.0 \times 1.3 \times 63851 = 5438$ kN

($V = 0.08 \times 0.630 \times 1.3 \times 1.0 \times 1.3 \times 14354 = 1222$ kips)

The numerical values of the coefficients in Formula 1 are obtained from NBC 1975 (see Appendix A).

Shear Distribution to the Height of the Building

Since the ratio of the height to the minimum dimension of the structure is less than 3, (h_n/D_s) no concentrated load is applied at the top (see Section 4.1.9(11) of NBC 1977).

The total shear is distributed along the height of the building including the top level in accordance with the following formula:

$$F_x = \frac{V\, w_x h_x}{\sum_{i=1}^{n} w_i h_i} \qquad (2)$$

The total shear in any horizontal plane is distributed to the various elements of the lateral force.

resisting system in proportion to their rigidities with "due regard" to the capacities and stiffnesses of the non-structural elements.

The distribution of the lateral load along the height of the structure is carried out on Table 2 .

Where x is the level being considered, w is the weight of the designated level, h is the height of the designated level and i designates any level of the structure (i = 1 is the first level above base).

Example W_x Calculation

Level 9

$$W_x = 1/10\ [28685.6 + \frac{1}{4} \times 7540.3 + \frac{1}{10} \times 9767.5]$$

$$= 2868.5 + 1885 + 976 = 5729 \text{ kN}$$

Note that:

28685 kN is the total weight of the floor system

7540 kN is the total weight of the 200 mm walls and

9767 kN is the total weight of the partitions.

Distribution of Lateral Loads

Wind and seismic loads are transmitted through the floor to those shear walls parallel to the direction assumed for the wind or earthquake motion (projected section for non-parallel walls). Lateral load distribution in shear walls depends upon the stiffness of the wall. The stiffer

TABLE 2. Distribution of Seismic Load Along the Height of the Building

Level	W_x (kN)	H_x Meters	$W_x h_x$	$\frac{W_x h_x}{\sum_{i=1}^{n} W_i h_i}$	$\frac{W_x h_x}{\sum_{i=1}^{n} W_i h_i} \times V$ (kN)
Roof	3366	26.0	87516	0.11	598.2
10th	5729	23.4	134058	0.17	924.5
9th	5729	20.8	119163	0.15	815.7
8th	5729	18.2	104267	0.13	707.0
7th	5729	15.6	89372	0.11	598.2
6th	6737	13.0	87581	0.11	598.2
5th	6737	10.4	70064	0.09	489.0
4th	6737	7.8	52548	0.07	380.6
3rd	6737	5.2	35032	0.04	217.6
2nd	6737	2.6	17516	0.02	108.8
	59966		797117	1.00	5438.0

Note that 797117 $= \sum_{i=1}^{n} W_i h_i$

$$0.11 = \frac{87516}{797117}$$

$$598 = 5438 \times 0.11$$

the wall, when compared to other walls in the building, the greater share of lateral load it will receive. Stiffness is measured by the deflection of the wall and can be based on shear deflection of flexural deflection walls with length/height (L/h) ratio between 1 and 4 (long-low walls) will deflect as shear walls, exhibiting very little flexural deflection. Tall slender walls with L/h between 1/4 and 1/20 will deflect as flexural members, walls with a 1/h between 1/4 and 1.0 (intermediate walls) will deflect as a combination of shear and flexural resistance.

However, to use either shear or flexural deflection without combining would not present any major error in design. For lateral force distribution, T-beam action may be assumed where a shear wall intersects another wall provided the effective flange width does not exceed a value specified by the applicable code requirements. The effective flange width for the forms of intersection such as L-Z or C-sections is also covered by code provisions (section 4.7.6 of CSA Standard S304-M78).

For shear distribution only walls in the direction of the lateral forces can be considered. The overhanging portion of a shear wall is not considered effective in computing the shear resistance of the wall. For our example we can relate stiffness directly to $(h/d)^3$ and distribute the loads to the walls on that basis (shear deflections are neglected). To find actual deflections or drift it is necessary to consider material properties. Although deflection calcula-tions are required to complete most lateral force building designs this part of the design is not discussed here. Referring to Figure 2, walls 1 to 18 resist all lateral forces in the y direction and the remaining walls resist the lateral forces in the x direction. Torsional stresses are resisted by all walls. The center of mass and the center of rigidity are at the same location for every floor because of the building layout. We must calculate both in each direction for one floor only. Consider #8. Center of **mass x direction**.

$$
\begin{aligned}
Px \;=\;& (976.7 + 2868.5)\ \frac{47}{2} + 8.6 \times 7.6\ (3.8 + 43.2)2 \\
& + 8.6 \times 6.8\ [12.10 + 19.7 + 27.3] + (34.9)2 \\
& + 8.6 \times\ [9.6(0.10) \times 2] + 7.0 \times 7.6 \times 2 \\
& + (8.2 \times 12.1 \times 2) + (8.2 \times 19.7 \times 2) + (5.8 \times 27.3) \\
& + (8.2 \times 27.3) + [(8.2)(34.9)2] + (7.0 \times 39.4 \times 2) \\
& + (9.6 \times 46.9 \times 2) + (2.4 \times 23.5) + (2.4 \times 31.10) \\
=\;& 90362 + 6142 + 10994 + 27240 \;=\; 134738
\end{aligned}
$$

$$\bar{X} = \frac{\Sigma Px}{P} = \frac{134738}{5729} = 23.52 \text{ meters}$$

Center of Rigidity in X Direction

$$31.76\ (3.8 + 43.2)2 + 22.75(12.10 + 19.7 + 27.3 + 34.9)2$$

$$= 2985.44 + 4277 = 7274.5$$

$$\bar{X} \quad \Sigma[\frac{1}{\Delta}\ X] \cdot [\frac{1}{\Sigma[\frac{1}{\Delta}]}] = \frac{7274.5}{309} = 23.54 \text{ meters}$$

TABLE 3.

Wall	d_m	H_m	$(h/d)^3$	$\frac{1}{\Delta}$	% Direct Shear	Relative Rigidity
1	9.6	26.0	19.88	63.93	0.0980	0.0666
2	9.6	26.0	19.88	63.93	0.0980	0.0666
3	7.0	26.0	51.18	24.83	0.0380	0.0258
4	7.0	26.0	51.18	24.83	0.0380	0.0258
5	8.2	26.0	31.87	39.88	0.0610	0.0415
6	8.2	26.0	31.87	39.88	0.0610	0.0415
7	8.2	26.0	31.87	39.88	0.0610	0.0415
8	8.2	26.0	31.87	39.88	0.0610	0.0415
9	2.4	26.0	1271.02	1.00	0.0015	0.0001
10	5.8	26.0	90.03	14.12	0.0220	0.0147
11	8.2	26.0	31.87	39.88	0.0610	0.0415
12	8.4	26.0	1271.02	1.00	0.0015	0.0001
13	8.2	26.0	31.87	39.88	0.0610	0.0415
14	8.2	26.0	31.87	39.88	0.0610	0.0415
15	7.0	26.0	51.18	34.83	0.0380	0.0258
16	7.0	26.0	51.18	34.83	0.0380	0.0258
17	9.6	26.0	19.88	63.93	0.0980	0.0666
18	9.6	26.0	19.88	63.93	0.0980	0.0666
				650.30	0.9960	
19	7.6	26.0	40.01	31.76	0.1028	0.0331
20	6.8	26.0	55.85	22.75	0.0736	0.0237
21	6.8	26.0	55.85	22.75	0.0736	0.0237
22	6.8	26.0	55.85	22.75	0.0736	0.0237
23	6.8	26.0	55.85	22.75	0.0736	0.0237
24	7.6	26.0	40.01	31.76	0.1028	0.0331
25	7.6	26.0	40.01	31.76	0.1028	0.0331
26	6.8	26.0	55.85	22.75	0.0736	0.0237
27	6.8	26.0	55.85	22.75	0.0736	0.0237
28	6.8	26.0	55.85	22.75	0.0736	0.0237
29	6.8	26.0	55.85	22.75	0.0736	0.0237
30	7.6	26.0	40.01	31.76	0.1028	0.0331
				309.00	1.0000	0.9970

$$\Sigma \frac{1}{\Delta} = (650.3 + 309) = 959.3$$

$$\frac{63.93}{650.30} = 0.0980 \qquad \frac{63.93}{959.3} = 0.0666$$

$$63.93 = \frac{1271.02}{19.88}$$

TABLE 4.

Wall	R	d	Rd^2	Relative	$\frac{M}{2350}$ kN-m	$\frac{V}{2350}$ kN	Base Shear Direct /1000 kN	Combined /1000 kN
1	0.0666	23.4	36.47	0.1861	437.00	18.69	98.0	116.69
2	0.0666	23.4	36.47	0.1861	437.00	18.69	98.0	116.69
3	0.0258	15.9	6.52	0.0333	78.23	4.92	38.0	42.92
4	0.0258	15.9	6.52	0.0333	78.23	4.92	38.0	42.92
5	0.0415	11.4	5.39	0.0275	64.63	5.67	61.0	66.67
6	0.0415	11.4	5.39	0.0275	64.63	5.67	61.0	66.67
7	0.0415	3.8	0.60	0.0031	7.29	1.92	61.0	62.92
8	0.0415	3.8	0.60	0.0031	7.29	1.92	61.0	62.92
9	0.0001	0.0	0.00	0.0000	0.00	0.00	1.5	1.5
10	0.0015	3.8	0.02	0.0001	0.24	0.06	22.0	22.06
11	0.0415	3.8	0.60	0.0031	7.29	1.92	61.0	62.92
12	0.0001	7.6	0.01	0.0001	0.00	0.00	1.5	1.56
13	0.0415	11.4	5.39	0.0275	64.63	5.67	61.0	66.67
14	0.0415	11.4	5.39	0.0275	64.63	5.67	61.0	66.67
15	0.0258	15.9	6.52	0.0333	78.23	4.92	38.0	42.92
16	0.0258	15.9	6.52	0.0333	78.23	4.92	38.0	42.92
17	0.0666	23.4	36.47	0.1861	437.00	18.67	98.0	116.69
18	0.0666	23.4	36.47	0.1861	437.00	18.67	98.0	116.69
19	0.0331	1.2	0.048	0.0002	0.47	0.40	102.8	103.20
20	0.0237	1.2	0.034	0.0002	0.47	0.40	73.6	74.0
21	0.0237	1.2	0.034	0.0002	0.47	0.40	73.6	74.0
22	0.0237	3.6	0.307	0.0016	3.76	1.04	73.6	74.64
23	0.0237	1.2	0.034	0.0002	0.47	0.40	73.6	74.00
24	0.0331	1.2	0.048	0.0002	0.47	0.40	102.8	103.2
25	0.0331	1.2	0.048	0.0002	0.47	0.40	73.6	74.0
26	0.0237	1.2	0.034	0.0002	0.47	0.40	73.6	74.0
27	0.0237	1.2	0.034	0.0002	0.47	0.40	73.6	74.0
28	0.0237	1.2	0.034	0.0002	0.47	0.40	73.6	74.0
29	0.0237	1.2	0.034	0.0002	0.47	0.40	73.6	74.0
30	0.0331	1.2	0.034	0.0002			102.8	103.2
			196.00	1.000	2349.75			

Note that for wall #1 $\quad 0.1861 = \frac{Rd^2}{\Sigma Rd^2} = \frac{36.47}{196.00}$

437 = 0.1861 x 2350

18.69 = 437/23.4

98 = 1000 x 0.98 (obtained from Table 3.)

As it can be seen from these calculations minimum accidental eccentricity governs. We will proceed with the calculations using 5% accidental eccentricity which for an arbitrary 100 kN load results to a torsional moment of 1000 x 0,05 x 47 = 2350 kN-m. This arbitrary moment will be distributed to the walls in relation to Rd^2 where R = relative rigidity and d distance from the center of rotation. It is assumed that the center of rotation coincides with the geographical center of the building, the introduced error is very small. After the effect of the assured torsional moment are evaluated the calculations will be adjusted to reflect actual loads.

Design of Wall #6

For this exmaple the wall is checked for maximum shear in the 200 mm (8 in.). block and at the ground level where 300 mm (12 in.) block is used.

The forces acting on this wall are obtained from information given in Tables 2 and 4.

Wall #6 from Table 4 is called upon to resist 66.67 kN for every 1000 kN applied to the structure in the lateral direction. The forces acting on this wall are shown in Figure 4.

Note that $39.88 = \frac{65.67}{1000} \times 590.2$

$$61.64 = \frac{66.67}{1000} \times 924.5$$

Total load at 7th floor:

$$V = (39.88 + 61.64 + 54.38 + 47.14 + 39.80)$$

$$= 242.9 \text{ kN}$$

$$M = (39.88 \times 4 \times 2.6) + (61.64 \times 3 \times 2.6) + (54.38 \times 2 \times 2.6)$$

$$+ (47.16 \times 2.6) = 1300 \text{ kN-m}$$

$$P = 93 + (110 \times 4) + (70 \times 4) + (55 \times 4) + (162 \times 4)$$

$$= 1690 \text{ kN}$$

Assuming that the wall is grouted in every second core at this level and that $f'_m = 10$ MPa

$$A_n = (190 \times 1000) - 2.5 \times 16000$$

$$= 150000 \text{ or } 0.15 \text{ m}^2/\text{m}$$

or

$$A_n = 1.23 \text{ m}^2/8.2 \text{ meters of wall}$$

Shear Stress

$$v = \frac{242.9}{1.23} = 0.197 \text{ MPa} < 0.33 \quad \text{OK}$$

Axial Stress

$$= \frac{P}{A} = \frac{1690}{1.23} = 1.373 \text{ MPa}$$

Flexural Stress

$$= \frac{6M}{bt^2} = \frac{6 \times 1300}{8.2^2 \times 0.190} = 0.610 \text{ MPa}$$

Combined Stress

$$= 1.373 + 0.610 = 1.983 < 3.33 \quad \text{OK}$$

$$e = \frac{M}{P} = \frac{1300}{1690} = 0.77 \text{ meters}$$

$$\frac{e}{t} = \frac{0.77}{8.2} = 0.094$$

$$\frac{h}{t} = \frac{2.6}{0.19} = 13.68$$

Allowable Vertical Load

$$P = C_s C_e \; A_n \; f_m = 0.83 \times 0.85 \times 1.23 \times 1000^2 \times 0.225 \times 10$$

$$= 1952 \text{ MPa} < 1690 \text{ OK}$$

In order to calculate flexural reinforcement the live load and the partition load is omitted.

$$P = 1690 - (4 \times 110 + 4 \times 55) = 975 \text{ kN}$$

$$\frac{P}{A} \pm \frac{M}{S} = \frac{975}{1.23} \pm \frac{6 \times 1300}{8.2^2 \times 0.19} = 0.793 + 0.610 \begin{matrix} \nearrow 1.403 \\ \searrow 0.183 \end{matrix}$$

Therefore minimum reinforcement required.

Provide 0.002 A as total reinforcement distributed 1/3 in the horizontal direction and 2/3 in the vertical direction in accordance with section 4.6.8.l of the National Standard of Canada CAN3-S304-M78 Masonry Design and Construction for Building.

Total Load at Ground Floor

$$v = 242.9 + 39.88 + 32.6 + 25.37 + 14.50 + 7.25$$

$$= 362.50 \text{ kN}$$

$$M = (39.88 \times 10 \times 2.6) + (61.64 \times 9 \times 2.6)$$
$$+ (54.38 \times 8 \times 2.6) + (47.14 \times 7 \times 2.6)$$
$$+ (39.88 \times 6 \times 2.6) + (39.88 \times 5 \times 2.6)$$
$$+ (32.60 \times 4 \times 2.6) + (25.37 \times 3 \times 2.6)$$
$$= 6239.50 \text{ kN-m.}$$

$$P = 2530 \times (6 \times 108) + (5 \times 110) + (5 \times 55) + (162 \times 5)$$
$$= 4705 \text{ kN}$$

$$A_n = 0.73 \times 290 \times 1000 = 0.21 \text{ m}^2 \text{ /m or}$$
$$1.74 \times \text{m}^2 \text{ /8.2 m}$$

$$\text{Shear Stress} = v = \frac{362.5}{1.74} = 0.208 \text{ MPa} < 0.33 \quad \text{OK}$$

$$\text{Axial Stress} = \frac{P}{A} = \frac{4705}{1.74} = 2.704 \text{ MPa} < 3.33 \text{ OK}$$

Provide minimum reinforcement. Check for other load combinations and provide additional reinforcement if needed. All walls in the building must be checked.

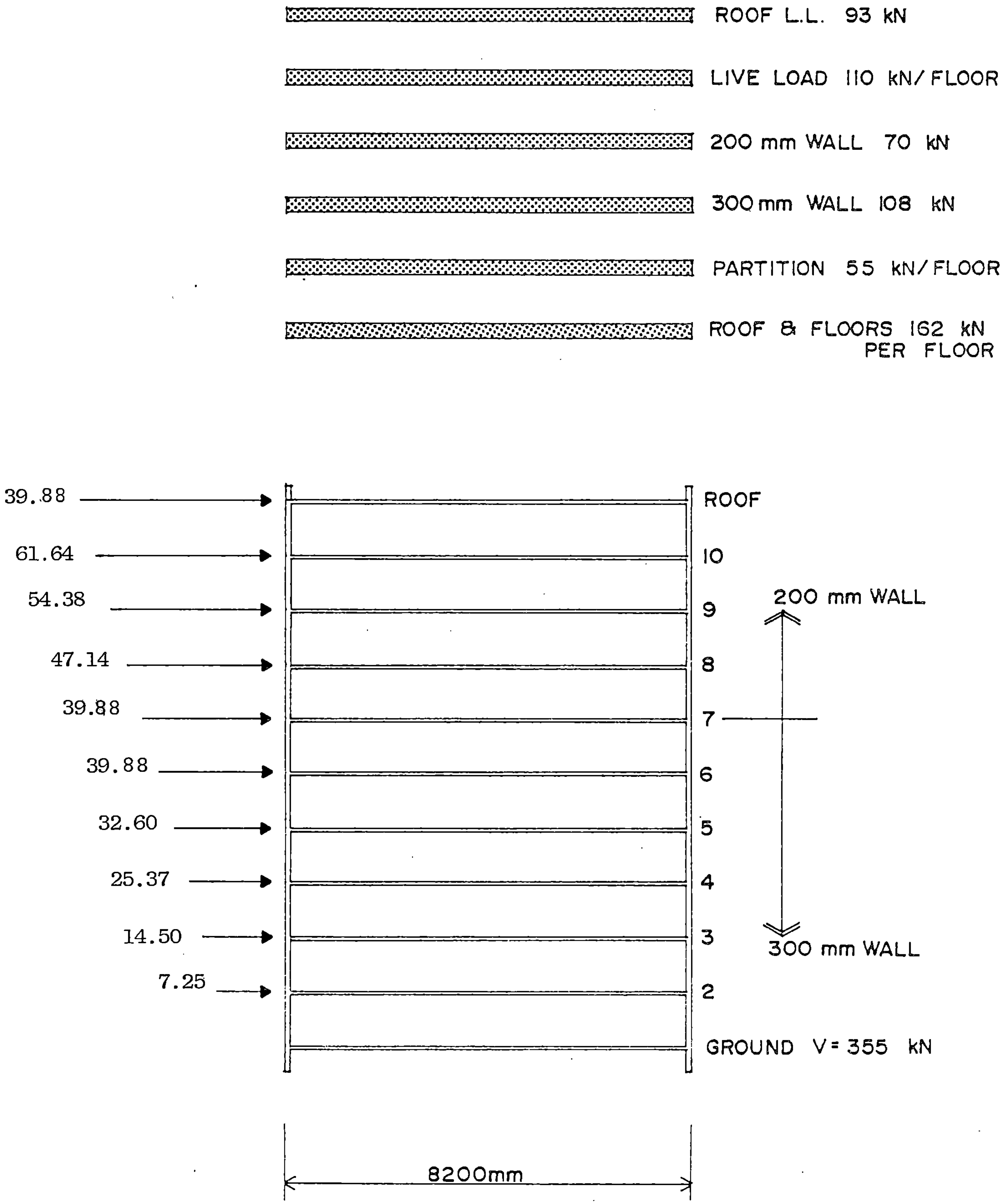


FIGURE 4

LOADS ON WALL #6

SUBSECTION 4.1.9. EFFECTS OF EARTHQUAKES

4.1.9.1.(1) The specified loading due to earthquake motion shall be determined
(a) by the analysis given in this Subsection, or
(b) by a dynamic analysis provided that the acceleration ratio, A, is not less than that given in the Table of Climatic Data in Part 2 of this Bylaw and provided that the dynamically determined value of V is not less than 90 per cent of that determined by the analysis of Clause (a).

(Information on an appropriate dynamic approach consistent with the specified acceleration ratio including a recommended response spectrum and ductility factors can be found in the Commentary on Dynamic Analysis for the Seismic Response of Buildings in NBC Supplement No. 4, "Commentaries on Part 4 1977.")

(2) In this Subsection

A = acceleration ratio = the ratio of the specified horizontal ground acceleration to the acceleration due to gravity.

D = the dimension of the *building* in a direction parallel to the applied forces.

D_n = plan dimension of the *building* in the direction of the computed eccentricity.

D_s = the dimension of the lateral force-resisting system in a direction parallel to the applied forces.

e = computed eccentricity between the centre of mass and centre of rigidity at the level being considered.

e_x = design eccentricity at level x.

F = foundation factor as given in Sentence 4.1.9.1.(9).

F_t = portion of V to be concentrated at the top of the structure as defined in Sentence 4.1.9.1.(11).

F_x = lateral force applied to level x.

h_i, h_n, h_x = the height above the base (i = 0) to level "i", "n" or "x", respectively.

I = importance factor of the structure as described in Sentence 4.1.9.1.(8).

J = numerical reduction coefficient for base overturning moment as defined in Sentence 4.1.9.1.(14).

J_x = numerical reduction coefficient for moment at level "x" as defined in Sentence 4.1.9.1.(15).

K = numerical coefficient that reflects the material and type of construction, damping, ductility and/or energy-absorptive capacity of the structure as given in Sentence 4.1.9.1.(7).

Level i = any level in the *building*, i = 1 first level above the base.

Level n = that level which is uppermost in the main portion of the structure.

Level x = that level which is under design consideration.

M_{tx} = torsional moment at level x.

N = the total number of *storeys* above exterior *grade* to level "n". (N is usually numerically equal to n.)

S = seismic response factor for the structure as defined in Sentence 4.1.9.1.(5).

S_p = horizontal force factor for part or portion of a structure, as given in Table 4.1.9.C.

T = fundamental period of vibration of the *building* or structure in seconds in the direction under consideration.

V = minimum lateral seismic force at the base of the structure.

V_p = lateral force on a part of the structure.

W = *dead load* including the following:
25 per cent of the design snow load specified in Subsection 4.1.7.; for areas used for storage, the full design live load modified according to Sentence 4.1.6.3.(4); the full contents of any tanks.

W_i, W_x = that portion of W which is located at or is assigned to level "i" or "x", respectively.

W_p = the weight of a part or portion of a structure, e.g. cladding, *partitions* and appendages.

(3) Earthquake forces shall be assumed to act in any horizontal direction. Except where required otherwise by the *authority having jurisdiction*, independent design about each of the principal axes shall be considered to provide adequate resistance in the structure for earthquake forces applied in any direction. Direction of forces

(4) The minimum lateral seismic force, V, assumed to act nonconcurrently in any direction on the *building* shall be equal to the product of Lateral seismic force

$$A \cdot S \cdot K \cdot I \cdot F \cdot W$$

where A is the acceleration ratio, given in the Table of Climatic Data in Part 2 of this Bylaw, and the value of this ground acceleration is assumed constant within each seismic zone as defined in the Commentary on Effects of Earthquakes in NBC Supplement No. 4, "Commentaries on Part 4 1977."

(Within seismic Zone 3, peak horizontal ground accelerations corresponding to 1 in 100 probability of annual exceedance can be larger than the assumed constant value. Information concerning the calculation of acceleration for special sites is contained in the Commentary on Effects of Earthquakes in NBC Supplement No. 4, "Commentaries on Part 4 1977.")

(5) The seismic response factor, S, shall be equal to $0.5/(T^{1/3})$ but need not exceed 1.00. Seismic response factor

(6) Except where technical data proves otherwise, the fundamental period, T, in Sentence (5) shall be equal to $0.05h_n/\sqrt{D}$ where h_n and D are in feet, except that where the lateral force-resisting system consists of a moment-resisting space frame which resists 100 per cent of the required lateral forces and the frame is not enclosed by or adjoined by more rigid elements that would tend to prevent the frame from resisting lateral forces, the fundamental period, T, shall equal 0.1 N. Fundamental period

(7) Values of the numerical coefficient, K, shall conform to Table 4.1.9.A. Types of construction

(8) The importance factor, I, shall equal 1.3 for all *post-disaster buildings* and schools, and 1.0 for all other *buildings*. Importance factor

(9) The foundation factor, F, shall conform to Table 4.1.9.B., except that the product FS need not exceed 1. Foundation factor

Table 4.1.9.A.

Forming Part of Sentence 4.1.9.1.(7)

Case[1]	Type or Arrangement of Resisting Elements	Value of K
1	*Buildings* with a ductile moment-resisting space frame[2],[3] with the capacity to resist the total required force.	0.7
2	*Buildings* with a dual structural system consisting of a complete ductile moment-resisting space frame and ductile flexural walls[4] designed in accordance with the following criteria: The frames and ductile flexural walls shall resist the total lateral force in accordance with their relative rigidities considering the interaction of the flexural walls and frames. In this analysis the maximum shear in the frame must be at least 25 per cent of the total base shear.	0.7
3	*Buildings* with a dual structural system consisting of a complete ductile moment-resisting space frame and shear-walls[5] or steel bracing designed in accordance with the following criteria: (a) The shear walls or steel bracing acting independently of the ductile moment-resisting space frame shall resist the total required lateral force. (b) The ductile moment-resisting space frame shall have the capacity to resist not less than 25 per cent of the required lateral force, but in no case shall the ductile moment-resisting space frame have a lower capacity than that required in accordance with the relative rigidities.	0.8
4	*Buildings* with ductile flexural walls[4] and *buildings* with ductile framing systems not otherwise classified in this Table as Cases 1, 2, 3 or 5.	1.0
5	*Buildings* with a dual structural system consisting of a complete ductile moment-resisting space frame with masonry infilling designed in accordance with the following criteria: (a) The wall system comprising the infilling and the confining elements acting independently of the ductile moment-resisting space frame shall resist the total required lateral force. (b) The ductile moment-resisting space frame shall have the capacity to resist not less than 25 per cent of the required lateral force.	1.3
6	*Buildings* (other than Cases 1, 2, 3, 4 and 5) of (a) continuously reinforced concrete, (b) structural steel, and (c) reinforced masonry shear walls.	1.3[6]
7	*Buildings* of unreinforced masonry and all other structural systems except Cases 1 to 6 inclusive and those set forth in Table 4.1.9.C.	2.0

Table 4.1.9.A. (Cont'd)

Case	Type or Arrangement of Resisting Elements	Value of K
8	Elevated tanks plus full contents, on 4 or more cross-braced legs and not supported by a *building*, designed in accordance with the following criteria: (a) The minimum and maximum value of the product SKI shall be taken as 1.2 and 2.5 respectively. (b) For overturning, the factor J as set forth in Sentence 4.1.9.1.(14) shall be 1.0. (c) The torsional requirements of Sentence 4.1.9.1.(15) shall apply.	3.0
Column 1	2	3

Notes to Table 4.1.9.A.:

(1) Explanatory notes on the various cases can be found in the Commentary on Effects of Earthquakes in NBC Supplement No. 4, "Commentaries on Part 4 1977."

(2) A space frame is a 3 dimensional structural system composed of interconnected members laterally supported so as to function as a complete self-contained unit with or without horizontal diaphragms.

(3) A ductile moment-resisting space frame is a space frame that is designed to resist the specified seismic forces and that, in addition, has adequate ductility or energy-absorptive capacity.
(Information on ductile moment-resisting space frames can be found in the Commentary on Effects of Earthquakes in NBC Supplement No. 4, "Commentaries on Part 4 1977.")

(4) A ductile flexural wall is a ductile flexural member cantilevering from the *foundation* consisting of a ductile reinforced concrete wall designed and detailed according to CSA A23.3-1973, "Code for the Design of Concrete Structures for Buildings," Chapter 19, Special Provisions for Seismic Design.

(5) Shear walls may be either flexural walls or shear walls as defined in CSA A23.3-1973, "Code for the Design of Concrete Structures for Buildings," Chapter 19, Special Provisions for Seismic Design.

(6) Except as required by Sentence 4.1.9.3.(1).

(10) The weight, W, of the structure shall be calculated in accordance with the following formula: Weight of structure

$$W = \sum_{i=1}^{n} W_i$$

(11) The total lateral seismic force, V, shall be distributed as follows: Distribution of lateral seismic force

(a) a portion F_t shall be assumed to be concentrated at the top of the structure and equal to $0.004V(h_n/D_s)^2$, except that F_t need not exceed 0.15 V and may be considered as zero for $(h_n/D_s) \leqslant 3$.

(b) the remainder, $V - F_t$, shall be distributed along the height of the *building* including the top level in accordance with the following formula:

$$F_x = (V - F_t)\, W_x h_x / \left(\sum_{i=1}^{n} W_i h_i\right), \text{ and}$$

(c) the total shear in any horizontal plane shall be distributed to the various elements of the lateral force-resisting system in proportion to their rigidities with due regard to the capacities and stiffnesses of the nonstructural elements.

Table 4.1.9.B.

Forming Part of Sentence 4.1.9.1.(9)

Type and Depth of *Soil*[1]	F
Rock, dense and very dense coarse-grained *soils*, very stiff and hard fine-grained *soils;* compact coarse-grained *soils* and firm and stiff fine-grained *soils* from 0 to 50 ft deep	1.0
Compact coarse-grained *soils*, firm and stiff fine-grained *soils* with a depth greater than 50 ft; very loose and loose coarse-grained *soils* and very soft and soft fine-grained *soils* from 0 to 50 ft deep	1.3
Very loose and loose coarse-grained *soils*, and very soft and soft fine-grained *soils* with depths greater than 50 ft	1.5[2][3]
Column 1	2

Notes to Table 4.1.9.B.:

[1] *Soil* depth shall be measured from *foundation* or *pile* cap level. Descriptive terminology relating to the *soils* is as defined in Section 4.2.

[2] Where *soil* deposits are of the order of 300 ft or more, amplification factors greater than those given in the Table may arise in the case of tall *buildings*.

[3] The possibility of ground failure beneath the structure due to excessive settlement or liquifaction of very loose sands and loss of strength of sensitive clays shall be considered.

(12) Parts of *buildings* as described in Table 4.1.9.C. and their anchorage shall be designed for a lateral force, V_p, equal to AS_pW_p, distributed according to the distribution of mass of the element under consideration.

(13) The values of S_p in Sentence (12) shall conform to Table 4.1.9.C.

Overturning

(14) The overturning moment, M, at the base of the structure shall be multiplied by a reduction coefficient, J, where

(a) $J = 1$ where T is less than 0.5,

(b) $J = (1.1 - 0.2T)$ where T is at least 0.5, but not more than 1.5, and

(c) $J = 0.8$ where T is greater than 1.5.

(15) The overturning moment M_x at any level x shall be multiplied by J_x where

$$J_x = J + (1-J)(h_x/h_n)^3$$

The incremental changes in the design overturning moments, in the *storey* under consideration, shall be distributed to the various resisting elements in the same proportion as the distribution of shears in the resisting system. Where other vertical members are provided which are capable of partially resisting the overturning moments, a redistribution may be made to these members if framing members of sufficient strength and stiffness to transmit the required loads are provided. Where a vertical-resisting element is discontinuous, the overturning moment carried by the lowest *storey* of that element shall be carried down as loads to the *foundation*.

Table 4.1.9.C.

Forming Part of Sentence 4.1.9.1.(13)

Category	Part or Portion of *Building*	Direction of Force	Value of S_p
1	All exterior and interior walls except those of category 2 and 3	Normal to flat surface	2
2	Cantilever parapet and other cantilever walls except retaining walls	Normal to flat surface	10
3	Exterior and interior ornamentations and appendages	Any direction	10
4	Towers, *chimneys*, smokestacks, all when less than 10 ft high above the *building*, machinery, fixtures and equipment, pipes, tanks plus contents and penthouses, all when connected to or forming part of a *building*	Any direction	2[1][2]
5	Towers, *chimneys* and smokestacks more than 10 ft high above the *building*	Any direction	3[3]
6	Tanks plus contents when resting on the ground	Any direction	1[2]
7	Floors and roofs acting as diaphragms	Any direction	1[4]
8	Connections for exterior and interior walls and elements, except those forming part of the main structural system	Any direction	25
Column 1	2	3	4

Notes to Table 4.1.9.C.:

[1] When h/D of any *building* is equal to or greater than 5 to 1, increase value by 50 per cent.

[2] The value shall be increased 50 per cent for pipes and containers for toxic or explosive materials, for materials having a flash point below 100°F or for firefighting fluids.

[3] Lower values of S_p may be used if they can be proven by analysis.

[4] Floors and roofs acting as diaphragms shall be designed for a minimum force corresponding to a value of $S_p = 1$ applied to loads tributary from that *storey*, unless a greater force F_x is assigned to the level under consideration as in Sentence 4.1.9.1.(11).

Torsional moments

(16) Torsional moments in the horizontal plane of the *building* shall be computed in each *storey* using the following formula:

$$M_{tx} = \left(V - \sum_{i=1}^{x} F_i\right) e_x$$

(Severe modal coupling may occur in symmetrical or nearly symmetrical structures when the fundamental lateral and torsional periods are nearly equal. Information on this phenomenon is given in the Commentary on Effects of Earthquakes in NBC Supplement No. 4, "Commentaries on Part 4 1977.")

Design eccentricity

(17) The design eccentricity, e_x, in Sentence (16) shall be computed by one of the following equations, whichever provides the greater stresses:

(a) $e_x = 1.5e + 0.05D_n$, or
(b) $e_x = 0.5e - 0.05D_n$

(18) When the maximum design eccentricity exceeds $0.25D_n$,

(a) a dynamic analysis shall be made, or
(b) the adverse effects of torsion as computed in Sentence 4.1.9.1.(16) shall be doubled.

(Information on a dynamic analysis can be found in the Commentary on Dynamic Analysis for the Seismic Response of Buildings in NBC Supplement No. 4, "Commentaries on Part 4 1977.")

Setbacks

(19) The *building* design shall take full account of the possible effects of setbacks.

(A definition of setback together with a recommended design procedure for *buildings* having setbacks is contained in the Commentary on Effects of Earthquakes in NBC Supplement No. 4, "Commentaries on Part 4 1977.")

General provisions

4.1.9.2.(1) Lateral deflections of a *storey* relative to its adjacent *storeys* shall be considered in accordance with accepted practice.

(2) Lateral deflections of a *storey* relative to its adjacent *storeys* obtained from an elastic analysis using the loads given in Sentence 4.1.9.1.(11) shall be multiplied by 3 to give realistic values of anticipated deflections.

(3) All portions of the structure shall be designed to act as integral units in resisting horizontal forces, unless separated by adequate clearances which permit horizontal deflections of the structure consistent with values of deflections calculated in accordance with Sentence 4.1.9.2.(2).

(4) The nonstructural components shall be designed so as not to transfer to the structural system any forces unaccounted for in the design, and any interaction of rigid elements such as walls and the structural system shall be designed so that the capacity of the structural system is not impaired by the action or failure of the rigid elements.

(5) To prevent collision of *buildings* in an earthquake, adjacent structures shall either be separated by twice the sum of their individual deflections obtained from an elastic analysis using the loads given in Sentence 4.1.9.1.(11) or shall be connected to each other.

(6) The method of connection in Sentence (5) shall take into account the mass, stiffness, strength, ductility and anticipated motion of the connected *buildings* and the character of the connection.

(7) The connected *buildings* in Sentence (5) shall be assumed to have a K value equal to that of the least ductile of the *buildings* connected, unless a lower value can be justified by rational analysis.

(8) Except in seismic Zone 0, *pile* footings of every *building* or structure shall be interconnected continuously by ties in at least 2 directions, designed to carry by tension or compression a horizontal force equal to 10 per cent of the larger *pile* cap loading, unless it can be demonstrated that equivalent restraints can be provided by other means.

4.1.9.3.(1) *Buildings* more than 3 *storeys* in height in seismic Zones 2 and 3 shall have a structural system as described in Cases 1, 2, 3, 4, 5 and 6 of Table 4.1.9.A. In addition, for *buildings* in seismic Zone 3 more than 200 ft in height and with a structural system of Case 6 the value of K shall be increased to 2.0.

(2) The design for any structural system which has an assigned value of K of 1 or less, shall ensure that when any member yields the remaining members of the structure shall be capable of resisting 25 per cent of the design seismic force including the effects of torsion.

(3) For *buildings* in Zones 2 and 3 in which discontinuities in columns or shear walls occur, special design provisions shall be made to ensure that failure at the point of discontinuity will not occur before the capacity of the remaining portion of the structure has been realized.

(4) In seismic Zones 2 and 3, reinforcement conforming to Clause 3.1.19. of CSA S304-1977, "Masonry Design and Construction in Buildings" shall be provided for masonry construction in

(a) *loadbearing* and lateral load-resisting masonry,
(b) masonry enclosing elevator shafts and stairways, or used as *exterior cladding*, and
(c) masonry *partitions*, except for *partitions* which
  (i) do not exceed 40 lb per sq ft in weight, and
  (ii) do not exceed 10 ft in height and are laterally supported at the top.

SUBSECTION 4.1.10. OTHER EFFECTS

4.1.10.1.(1) The minimum specified load applied horizontally and normal to the span at the top of every required *guard* shall be

(a) 40 lb/lineal ft for exterior balconies of individual residential units and a concentrated load of 200 lb applied concurrently,
(b) 100 lb/lineal ft for *exits* and stairs,
(c) 150 lb/lineal ft for *assembly occupancies*, except for grandstands and stadia,
(d) 250 lb/lineal ft for grandstands and stadia including ramps,
(e) 300 lb/lineal ft for vehicle guard rails for parking garages applied 21 in. above the roadway and minimum total load of 2,500 lb uniformly distributed over each vehicle space applied 21 in. above the roadway, and
(f) a 125-lb concentrated load applied at any point for industrial catwalks and other areas where crowding by many people is very improbable.

(2) Individual elements within the *guard*, including solid panels and pickets, shall be designed for 20 psf or 100 lb of concentrated load at any point in the element, whichever results in the more critical loading condition. The loads need not be considered to act simultaneously with the loads provided for in Sentence (1) and (3).

(3) The minimum specified load applied vertically at the top of every required *guard* shall be 100 lb/lineal ft and need not be considered to act simultaneously with the horizontal load provided for in Sentence (1).

MINIMUM EARTHQUAKE FORCES

The NBC specifies that a structure should be designed for a minimum earthquake force given by

$$V = ASKIFW \qquad (2)$$

This is essentially Equation (1) modified to take into account the most important factors involved in the response of buildings to earthquakes. Each of these factors will now be discussed.

Horizontal Design Ground Acceleration A

The factor A is the value of the assigned horizontal design ground acceleration in units of gravity. The value of A as well as the relevant seismic zone are given in the Table of Climatic Data in Part 2 of the NBC. For many sites in Canada the seismic zones are listed in the Table of Design Data for Selected Locations in NBC Supplement No. 1, "Climatic Information for Building Design in Canada 1975."

Values of A that were chosen to correspond with the various seismic zones and zone boundaries are given in Table J-2. For sites other than those listed in that Table, the zone number and hence the appropriate value of A for that site may be obtained from the seismic probability map, Chart 12 in Supplement No. 1 of the NBC which is reproduced as Figure J-1. The acceleration amplitude is assumed to be constant within each zone.

Seismic Coefficient S

The coefficient S is given by the formula

$$S = \frac{0.5}{\sqrt{T}} \qquad (3)$$

which reflects the dependence of seismic spectral acceleration on the fundamental period of the structure, as well as the contributions of the higher modes for tall buildings. In lieu of more accurate estimates, the following empirical formulas can be used for the determination of the fundamental period T for buildings:

$$T = \frac{0.05h_n}{\sqrt{D}} \qquad (4)$$

or

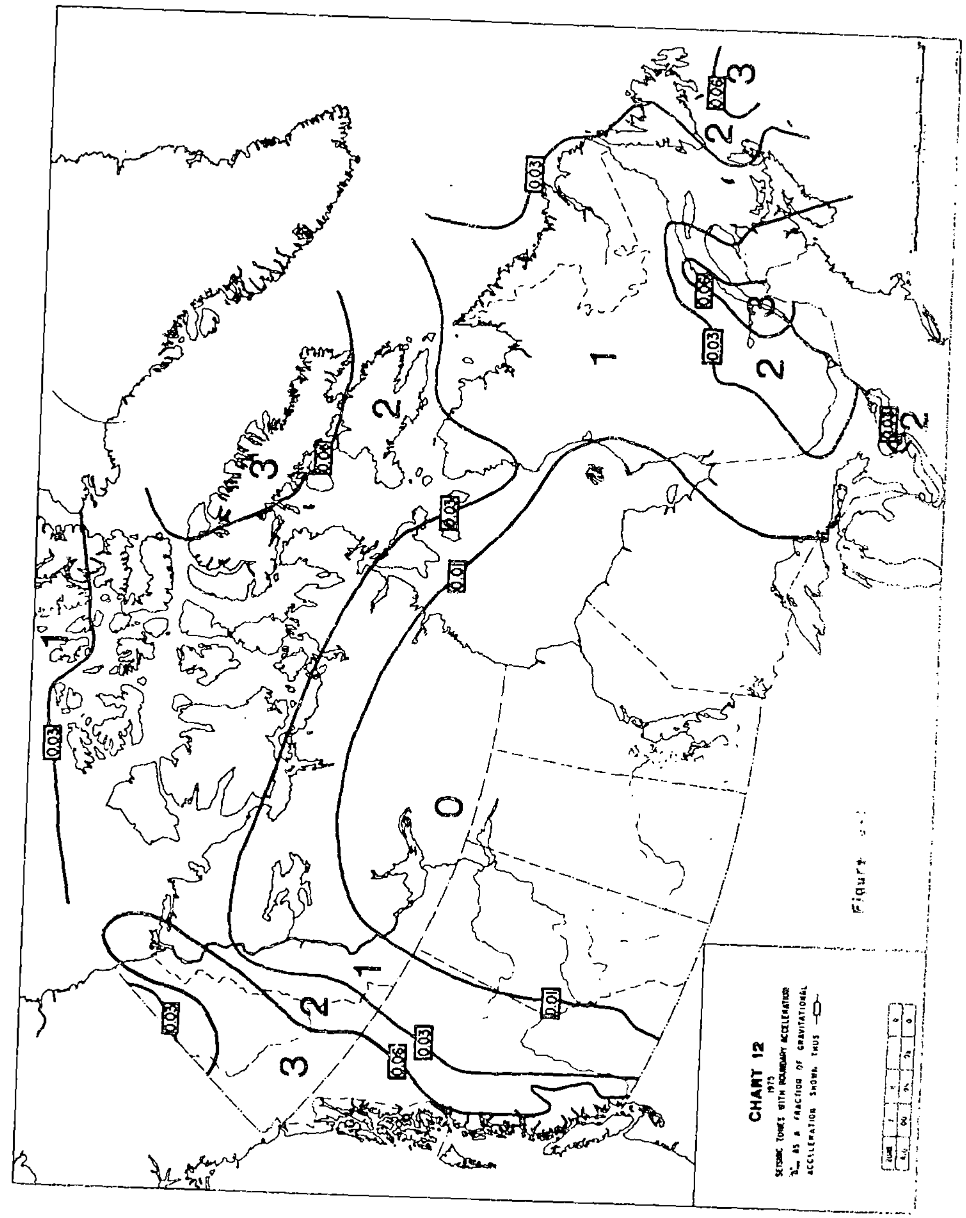
$$T = 0.1\,N \qquad (5)$$

for moment resistant space frames only. The symbols are defined in Sentence 4.1.9.1.(2) of the NBC.

Equation (4) is based on approximately 1600 vibration observations made in 430 buildings, 150 observations on 42 elevated tanks and 250 special observations.[15] In modern multi-storey buildings this period calculation gives values that for the most part are in reasonable agreement with measured values. However, variations in the order of ± 50 per cent have been observed when Equation (5) is applied to framed structures, and similar variations when Equation (4) is applied to mixed shear wall frames and coupled shear wall structures. For pure shear wall structures the natural period is generally overestimated, sometimes by as much as 100 per cent. If the designer wishes, he may determine the period T for a structure by more refined methods of calculation and submit the relevant technical data.

Coefficient K

The coefficient K assigned to different types of structural systems reflects design and construction experience, as well as the evaluation of the performance of structures in major and moderate earthquakes. It endeavours to account for the energy-absorption capacity of the structural system by damping and inelastic action, and the response characteristics of certain types of structures in earthquakes. Types of construction that are recognized to have performed well in earthquakes are assigned lower values of K. The K values of Table 4.1.9.A. of the NBC specifically recognize the following:



(2) The existence of alternate load paths or redundancy of a structural system is a desirable characteristic. This increases the locations where energy can be dissipated and reduces the risk of collapse when individual members should fail or become severely damaged. Some mixed wall-frame systems are therefore given a lower K value than shear wall structures.

(3) Some stiff structural systems have been shown to attract larger base shear forces in the higher modes than those with more frame action. Consequently, shear wall structures are given a higher K value than some mixed wall-frame systems.

(4) The K values assigned to buildings are lower than those assigned to other structures, because buildings are normally endowed with a multiplicity of nonstructural elements and resisting elements not considered in the analysis. Furthermore, buildings generally have higher damping values during large amplitude vibrations than mere skeleton structures.

Buildings incorporating inadequately designed shear walls, unreinforced or inadequately reinforced masonry, precast concrete with nominal connections or structural steel with non-ductile connections, lack adequate ductility for effective seismic performance and the K values are correspondingly increased.

The following should be noted when choosing the K values for the structure:

Cases 1 and 2. Complete moment-resisting ductile space frames with or without ductile flexural walls that qualify for $K=0.7$ need to be analyzed so that realistic load distributions among the various members can be ascertained. This requires a frame analysis for space frames and an interactive analysis for a combination wall-frame system. Detailing requirements as specified in CSA A23.3-1973, "Code for the Design of Concrete Structures for Buildings," including Chapter 19 for concrete, and CSA S16-1969, "Steel Structures for Buildings" for steel, are considered to provide adequate member ductilities. The system ductility factor for these types of structures is approximately 3 to 4 for some visible structural damage and greater for major structural damage.

Case 3. In order to qualify for $K=0.8$, the structure must have a complete ductile moment-resisting space frame and shear walls designed so that the total required lateral force is resisted in accordance with the relative rigidities of the walls and the space frame. In addition, the following must be satisfied:

(a) The complete ductile moment-resisting space frame shall be designed to carry as a system separate from the shear walls the total vertical loads and at least 25 per cent of the total required lateral force, i.e. 25 per cent of $V = ASKIFW$.

(b) The shear walls when acting alone (i.e. independent of the ductile moment-resisting space frame), must be capable of carrying the total lateral force. Concrete shear walls have to be provided at their edges either with encased structural steel elements conforming to CSA G40.21-1973, "Structural Quality Steel" or with built-in concrete columns specially detailed for ductility. While detailing requirements for this Case are not fully covered in CSA A23.3-1973, "Code for the Design of Concrete Structures for Buildings," the recommendations given in the Appendix of ACI 318-71, "Building Code Requirements for Reinforced Concrete" on the seismic design of shear walls provides an adequate design for these elements. For the purpose of the NBC earthquake provisions, tension-diagonal steel bracing systems are also classed as shear walls.

Case 4. $K=1.0$ is required for buildings that consist entirely or mainly of concrete flexural walls designed in a ductile manner according to CSA A23.3-1973, "Code for the Design of Concrete Structures for Buildings," Chapter 19, and all other ductile framing systems that do not qualify for $K=0.7$ or $K=0.8$. This includes wood structures with framing and lateral load resistant elements made of wood, and possessing adequate connections and joint details. Frames with structural steel "K" bracing or with tension-compression diagonal bracing are also considered to form a ductile structural system requiring $K=1.0$.

Case 5. $K=1.3$ for frame buildings where 1 or more bays have infills or reinforced masonry so as to form a wall cantilevering up from the foundations. While the inclusion of isolated infill panels need not affect the K value for the buildings, $K=1.3$ would be required if more than 50 per cent of the building height contains masonry infill panels.

It should be noted that K is to be increased to 2.0 for buildings of Case 6, located in Zone 3 and over 200 ft in height, as described in Table 4.1.9.A.

Case 6. K will be taken as 1.3 for structures without special provisions for ductility in the load-carrying structural system. This includes structures having nominal ductility such as those of continuously reinforced concrete, reinforced masonry shear wall buildings, steel structures which may exhibit a degrading stiffness characteristic, such as frames relying solely on a tension-diagonal lateral bracing system and post and beam wood construction where the earthquake loads are resisted primarily by tension-diagonal steel cross bracing. Continuously reinforced concrete refers to reinforced concrete conforming to CSA A23.3 Chapters 1 to 18. Precast concrete construction may be used in Case 6 provided the reinforcing is made continuous by means of lapped or welded splices in accordance with CSA A23.3-1973. The splices are to be encased with cast-in-place concrete.

Case 7. $K=2.0$ for buildings which exhibit little ductility and damping. This includes unreinforced masonry buildings and unreinforced masonry components. $K=2.0$ also for structures other than buildings that consist of only 1 or a few component parts.

Case 8. Cross-braced towers supporting elevated water tanks require $K=3.0$. This high value is considered appropriate because of the poor performance of such structures in past earthquakes and the special importance of maintaining their integrity in case of fire following an earthquake.

Foundation Factor F

The soil conditions at a given site have been shown to exert a major influence on the amplitudes and nature of the earthquake motions at the ground surface.[16] to [19] In cases where the motions propagate from bedrock to the surface, the soil can amplify the bedrock motions in select frequency ranges about the natural frequencies of the soil layers. A structure located on such a soil system and whose natural frequencies lie close to those of the soil layers would thus undergo more severe motions than one founded on rock. Because of the complexities and some uncertainties involved in the phenomenon, only a rough allowance can be made in code provisions at this time.

The foundation factor in NBC Sentence 4.1.9.1.(9) incorporates 2 soil variables that are known to contribute significantly to motion amplification at the surface: (a) stiffness, and (b) depth. The soil stiffness is characterized by soil types as defined in the foundation section of NBC, and the depth is that of the predominant layer beneath the foundation level of the structure. If more than 1 layer of various soil types are present, the largest value of F should be chosen when the provisions of Sentence 4.1.9.1.(9) are applied to each layer in turn. For many soil layers the value of F is obtained by assuming an "average" soil type over the total depth. In all cases the applicable soil depth is measured from foundation level to the bottom of the layer under consideration. For the purpose of determining the foundation factor F, small lenses of material having lateral dimensions of the order of 200 ft or less can be ignored.

Under certain combinations of soil and structural characteristics, the dynamic loads transmitted by the structure to the soil cause deformations of the ground which can influence the seismic response of the structure.[20],[21] This is commonly known as structure-ground interaction. To consider the influence of flexible soil foundations in the seismic design of buildings, dynamic methods of analysis would be required. In most situations, neglecting the presence of the flexible foundation condition should result in conservative designs.

It should again be emphasized that in addition to the influence of the site conditions on the predominant periods of the ground motion, and thus the amplitude of the seismic forces, the designer should consider the possibility of ground failure beneath the structure due to slides, local fault displacements, liquefactions of sands,[22],[23],[24] and remoulding of sensitive clays. The advice of an experienced geotechnical engineer should be sought for evaluation of the suitability of the site and its possible behaviour under seismic forces and movements.

Importance Factor I

Some structures are designed for essential public services, and it is imperative that these structures be operative after an earthquake. These include buildings that house electrical generating and distribution systems, fire and police stations, hospitals, radio stations and radio towers, telephone exchanges, water and sewage pumping stations, fuel supplies, civil defence buildings and schools. Such structures are assigned an I factor of 1.3. This factor is not intended to cover the design considerations associated with special purpose structures whose failure could endanger the lives of a large number of people or affect the environment well beyond the confines of the building. These would include facilities for the manufacture or storage of toxic materials, nuclear power stations, etc.

VERTICAL ACCELERATIONS

Ground motion during an earthquake is multi-directional, and may contain a substantial vertical component. Outside the epicentral zone the vertical accelerations are generally 30 to 60 per cent of the horizontal accelerations, while in the immediate vicinity of the epicentre the vertical accelerations could be higher.

Under abnormally high vertical accelerations, columns at the upper floors, especially at the roof level, could be adversely affected. However, there is usually sufficient reserve strength in vertical load-carrying members that vertical accelerations can be safely neglected. Certain special structures have been noted[25] where these accelerations may have led to instability or unusual reductions in the factors of safety. When this becomes a governing design consideration, dynamic analysis should be employed.

DISTRIBUTION OF THE BASE SHEAR

The base shear is the algebraic sum of the inertial forces acting on the masses of the structure caused by the seismic motion of the base. The motion of the structure is complex, involving the superposition of a number of modes of vibration about several axes. For translational vibrations, the addition of the spectral modal responses results in a lateral inertial force distribution that is approximately triangular in shape with the apex at the base. As the inertia forces F_x induced at any level x are proportional to the weight W_x at that level, the distribution of seismic forces is approximated by

$$F_x = V\left(\frac{W_x h_x}{\sum_{i=1}^{n} W_i h_i} \right) \tag{6}$$

For a lumped mass system the use of this equation is reasonable for stubby structures, i.e. those with height-to-width ratios less than 3, or those with fundamental periods less than about 1 sec. For more slender buildings, higher forces are induced at the top of the structure[12],[26] due to increasing contributions to top storey amplitudes by all the contributing modes. This redistribution of forces is accounted for by applying part of the base shear as a concentrated force, F_t, to the top of the structure.

It should be noted that V_n, the shear force for the top storey, and V_x, the shear force transmitted to the supporting structure just below the level $i = x$, are given by

$$V_n = F_t + F_{x=n} \tag{7}$$

The dimension D_s should be the plan dimension of the lateral force-resisting system that contributes substantially to resisting the lateral loads.

OVERTURNING MOMENTS

The lateral forces that are induced in a structure by earthquakes give rise to moments which are the product of the induced lateral forces times the distance to the storey level under consideration. There they have to be resisted by axial forces and moments in the vertical load-carrying members. While the base shear contributions of modes higher than the fundamental can be significant, the corresponding modal overturning moments for the higher modes are small. As the equivalent static lateral base shear in the NBC also includes the contributions from higher modes for moderately tall and tall structures, a reduction in the overturning moments computed from these lateral forces appears justified. This is achieved by means of the multiplier J as given in NBC Sentence 4.1.9.1.(14) and shown in Figure J-2. If, however, a structure did respond exclusively in its fundamental mode, the overturning moment at the base would be the sum of the moments corresponding to the forces F_x about the base without any J-factor reductions. A more refined method of accounting for the maximum overturning moments is through the methods of dynamic analysis.

The overturning moment reduction factor J in many building codes has recently been adjusted upwards, as it was realized on the basis of theoretical investigations and recent earthquake experience that small values of J were not justified. Examples of this trend are seen in the California SEAOC, the New Zealand Codes as well as the NBC as shown in Figure J-2.

TORSIONAL MOMENTS

The inertial forces induced in the structure by earthquake ground motions act through the centre of gravity of the masses, e.g. basically at each floor level. If the centre of mass and the centre of rigidity do not coincide because of asymmetrical arrangement of structural elements or uneven mass distributions, torsional moments will arise. The designer should endeavour to make the structural system as symmetrical as possible and should consider the effect of torsion on the behaviour of the structural elements.

A realistic approach to aseismic torsional design should consider the effect of the dynamic magnification[27],[28] of the torsional moments, the effect of simultaneous action of the 2 horizontal components of the ground disturbance, and accidental torsion. Accidental torsional moments are intended to account for the possible additional torsion arising from variations in the estimates of the relative rigidities, uncertain estimates of dead and live loads at the floor levels, addition of wall panels and partitions after completion of the building, variation of the stiffness with time, and inelastic or plastic action. The effects of possible torsional motion of the ground should also be considered. For most practical situations, however, these concepts and effects can only be accounted for by the use of adjustment factors.

The torsional provisions of the NBC deal with the complex nature of torsion and the effect of the simultaneous action of the 2 horizontal ground motion components by increasing or decreasing the computed torsion by 50 per cent, whichever produces the worst effect in a member. The part played by accidental torsion is recognized by specifying an additional torsion due to an eccentricity of 0.05 times the plan dimension in the direction of the computed eccentricity. The NBC also specifies that when the total torsional eccentricity exceeds 25 per cent of the appropriate plan dimension, a dynamic analysis shall be mandatory or the effects of torsion in the static analysis shall be doubled. This accounts for the complexity and importance of the torsional effects under these conditions.

For structural elements to resist torsional moments most effectively, they should preferably be located near the periphery of the building, i.e. some distance from the centre of rigidity. Wall

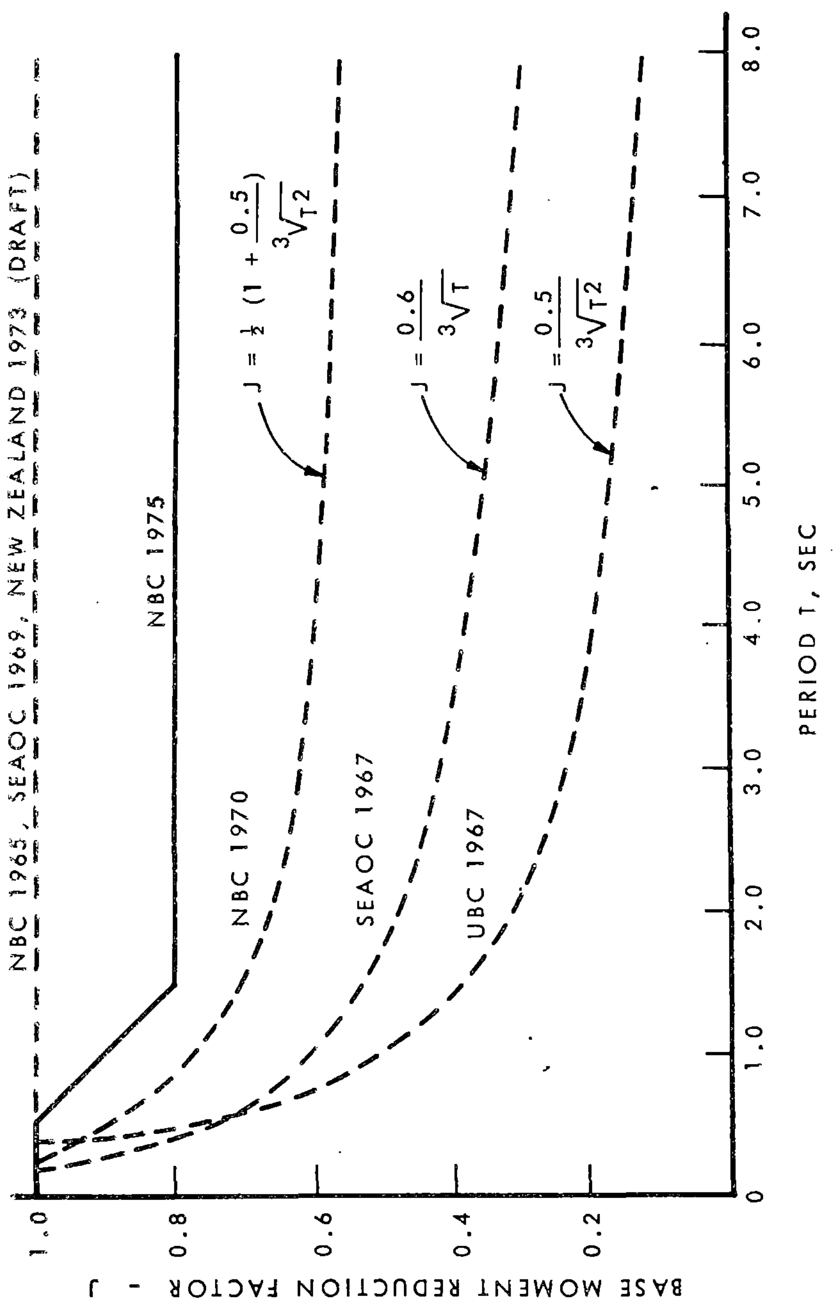


Figure J-2 Variation of the moment reduction coefficient with period

forces are associated with as large a moment arm as possible about the centre of rigidity. In buildings with complete diaphragms such as complete reinforced concrete floor slabs, all elements interconnected by such members can be counted on to resist torsional forces.

In core-type buildings where all stiffening elements are located in a central core away from the periphery, accidental torsion and torsional ground motion are particularly significant. In odd- and irregularly-shaped buildings, such as the L-shaped building, and in buildings with the core located at one side or corner, large torsional oscillations are induced by horizontal ground motion. These are some examples of torsion situations that should be avoided in building layouts. Torsional effects should also be evaluated for parts of structures relative to the whole. For example, it is important that the torsional effects of projecting wings on buildings be considered in relation to the motion of the building as a whole.

When the torsional frequency of a structure is close to one of the translational frequencies of the structure, large torsional amplifications can occur,[29,30] even in symmetrical buildings. Such cases should be analyzed dynamically. It is, however, not yet feasible to identify such situations simply, without actually performing a detailed modal analysis.

SETBACKS

A setback is considered to be a sudden change in plan dimension or a sudden change in stiffness along the height of a building. Only the case of sudden changes in plan dimensions will be treated here. The effects of major changes in stiffness are best investigated by dynamic methods.

The following guidelines are extracted from the 1967 Edition of the SEAOC Code Commentary.[31] These are considered most suitable for the majority of cases encountered in practice.

The problem of seismic design for buildings with setbacks is rather complex because of the many factors and variables involved. Setbacks in practice can be symmetrical or asymmetrical about the base portion in one or both axes. Towers and bases can vary in types of construction, the amount of the setback and the height of the tower as compared to that of the base. Also, the relative masses of the tower and base portions will influence the dynamic behaviour of the entire structure.

In consideration of the many variables involved in this problem, mathematical, experimental and judgment-type factors have all been employed in arriving at the following recommended procedure. While many setbacks are 3-dimensional in geometry, it is considered satisfactory that the 2 dimensions in the plane under consideration be used.

Terminology

The following definitions and symbols are employed in this Subsection of the Commentary as well as in the associated diagram, Figure J-3:

- Setback = a change in either or both plan dimensions of a building from one storey to another,
- Base = the portion of a building below a setback level,
- Tower = the portion of a building above a setback level,
- b = the width in feet of the base parallel to the direction under consideration,
- t = the width in feet of the tower parallel to the direction under consideration,
- H = the height in feet of the entire building (base plus tower),
- H_t = the height in feet of the tower only on the face parallel to the direction under consideration.

$\frac{H}{d}$	$\left(\frac{H}{d}\right)^3$	$\triangle$ FIXED END $.0833\left(\frac{H}{d}\right)^3 + 0.25\left(\frac{H}{d}\right)$	$\triangle$ CANTILEVER $0.333\left(\frac{H}{d}\right)^3 + 0.25\left(\frac{H}{d}\right)$
0.1	0.001	0.0251	0.0253
0.2	0.008	0.0507	0.0527
0.3	0.027	0.0773	0.0840
0.4	0.064	0.1053	0.1214
0.5	0.125	0.1354	0.1668
0.6	0.216	0.1680	0.2221
0.7	0.343	0.2036	0.2896
0.8	0.512	0.2427	0.3710
0.9	0.729	0.2857	0.4685
1.0	1.000	0.3333	0.5840
1.1	1.331	0.3859	0.7196
1.2	1.728	0.4440	0.8772
1.3	2.197	0.5080	1.0588
1.4	2.744	0.5786	1.2665
1.5	3.375	0.6561	1.5023
1.6	4.096	0.7412	1.7368
1.7	4.913	0.8343	2.0659
1.8	5.832	0.9358	2.3949
1.9	6.859	1.0464	2.7159
2.0	8.000	1.1664	3.1720
2.1	9.261	1.2964	3.6182
2.2	10.648	1.4370	4.1064
2.3	12.167	1.5885	4.6388
2.4	13.824	1.7515	5.2172
2.5	15.625	1.9265	5.8438
2.6	17.760	2.1294	6.5818
2.7	19.683	2.3146	7.2491
2.8	21.952	2.5286	8.0320
2.9	24.389	2.7567	8.8709
3.0	27.000	2.9910	9.7500

When $P = 1 \times 10^6$

$E_m = 1.2 \times 10^6$

$t = 1''$

Eq. for fixed end walls —

$$\triangle_F = \frac{P}{E_m t}\left[\left(\frac{H}{d}\right)^3 + 3\left(\frac{H}{d}\right)\right]$$

and for cantilever walls.

$$\triangle_C = \frac{P}{E_m t}\left[4\left(\frac{H}{d}\right)^3 + 3\left(\frac{H}{d}\right)\right]$$

becomes

$$\triangle_F = 0.0833\left(\frac{H}{d}\right)^3 + 0.25\left(\frac{H}{d}\right)$$

$$\triangle_C = 0.333\left(\frac{H}{d}\right)^3 + 0.25\left(\frac{H}{d}\right)$$

4.6.8 Reinforcement

4.6.8.1 Load-Bearing and Shear Walls

4.6.8.1.1 Reinforced masonry load-bearing and shear walls shall be reinforced horizontally and vertically with steel having a minimum area calculated in conformance with the following formulae:

$$A_v = 0.002\ A_g\alpha$$
$$A_h = 0.002\ A_g\ (1 - \alpha)$$

where

A_v = area of vertical steel per metre of wall, mm^2

A_h = area of horizontal steel per metre of wall, mm^2

α = reinforcement distribution factor varying from 0.33—0.67, as determined by the designer

4.6.8.1.2 The horizontal and vertical reinforcing steel shall be spaced not more than six times the wall thickness nor more than 1.2 m apart, whichever is less. (Wire reinforcement in the mortar joints may be considered as required horizontal reinforcement.)

4.6.8.1.3 Horizontal reinforcement shall be provided at the top of every masonry foundation wall, at the bottom and top of every wall opening, in the course immediately below the roof and floor levels, and at the top of every parapet wall.

4.7 Shear Walls

4.7.1 A plain masonry shear wall shall be designed so that no part of the wall is in tension.

4.7.2 Reinforced masonry shear walls shall be designed in conformance with Clauses 4.6.7.3, 4.6.7.4, and 4.6.7.7.

4.7.3 The maximum horizontal shear stress in a shear wall, v_{sw}, shall not exceed the value:

$$(v \text{ or } v_m) + 0.3f_{cs}$$

where
v or v_m = the allowable applicable shear stress

4.7.4 In computing the shear resistance of a shear wall, flanges or projections formed by intersecting walls shall be neglected.

4.7.5 Except as provided in Clauses 4.7.8, 4.7.9, and 4.7.10, where a masonry shear wall intersects a load-bearing masonry wall or walls to form T- or I-sections, the effective flange width shall not exceed one-sixth of the total wall height above any cross-section of the wall, and its overhanging width on either side of the shear wall shall not exceed six times the thickness of the intersected wall.

4.7.6 Except as provided in Clauses 4.7.8, 4.7.9, and 4.7.10, where a masonry shear wall intersects a load-bearing masonry wall or walls to form L-, C- or Z-sections, the effective overhanging flange width shall not exceed 1/16 of the total wall height above any cross-section of the wall, nor six times the thickness of the intersected wall.

4.7.7 Limits to effective flange width in Clauses 4.7.5 and 4.7.6 may be increased on the basis of a detailed analysis of the actual stress distribution in the shear wall.

4.7.8 Where wall intersections described in Clauses 4.7.5, 4.7.6, and 4.7.7 are bonded so that at least 50 per cent of the units of one wall are embedded in the other wall, the vertical shear stress at the intersection shall not exceed the allowable shear stress in Clause 4.5.1.1 for shear walls.

4.7.9 Toothed joints shall not be used in shear walls.

4.7.10 Where wall intersections described in Clauses 4.7.5, 4.7.6, and 4.7.7 are bonded by concrete or grout completely filling vertical keyways, recesses, or a combination of these, to provide a bond at least equivalent to the masonry in Clause 4.7.8, the vertical shear stress at the intersection shall not exceed the allowable shear stress in Clause 4.5.1 for shear walls. The compressive strength of concrete or grout used to bond the intersection shall be at least equal to that of the masonry. The minimum horizontal reinforcement across the vertical intersection shall be equivalent in strength to at least two steel wires of at least 3.8 mm diameter spaced 400 mm vertically.

4.7.11 Rigid steel connectors such as anchors, rods, or bolts may be utilized to bond wall intersections in Clauses 4.7.5, 4.7.6 or 4.7.7, except in portions of reinforced masonry shear walls in which the flanges contain tensile steel and are subject to axial tension under load.

4.7.12 When rigid steel connectors in Clause 4.7.11 are used:

(a) Connectors shall be embedded in mortar or grout;

(b) Vertical masonry joints at the intersections shall be completely filled with mortar or grout;

(c) The bearing stress of steel connectors on masonry shall not exceed the allowable bearing stress in Clause 4.5.1 due to the action of the vertical shear at the intersection. In determining this bearing stress, the eccentricity of the shear load shall be provided for in the design;

(d) The allowable bond stress shall not exceed the values given in Tables 6 and 7 due to the action of axial forces resisted by the connectors, except that embedment length shall be at least 450 mm or 72 times the thickness or diameter of the connector on each side of the intersection, whichever is the greater length;

(e) Adequate anchorage shall be provided by hooks or rigid cross-pieces that are fully embedded in mortar or grout in the horizontal joints or vertical voids in the masonry where the embedment length is limited or where there is not sufficient thickness of masonry to bond the connector;

(f) Maximum shear stress in steel connectors shall not exceed 35 MPa;

(g) Maximum thickness or diameter of steel connectors shall not exceed one-half the thickness of the mortar or grout spaces in which they are embedded; minimum strap thickness shall be 5 mm and minimum bar size shall be #10;

(h) Vertical spacing of steel connectors shall not exceed 600 mm; and

(i) Where steel connectors bond walls exposed to weather or in contact with ground, they shall be galvanized after fabrication, or equivalent corrosion protection shall be provided.

4.7.13 When floors or roofs are designed to transmit horizontal forces to walls, the anchorage of the floor or roof to the wall shall be designed to resist the horizontal force.

4.7.14 Steel anchors to resist shear force shall be designed in conformance with Clause 4.5.4.