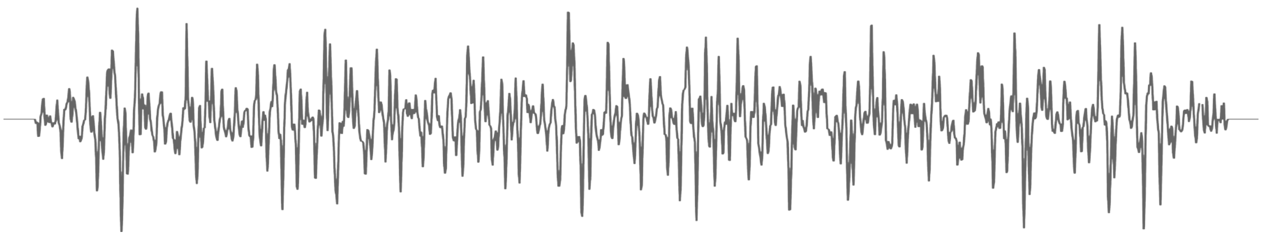
SEISMIC DESIGN GUIDE FOR MASONRY BUILDINGS

Second Edition



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CCMPA
Building Strength

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1 Seismic Design Provisions of the National Building Code of Canada 2015

1.1 Introduction

This chapter provides a review of the seismic design provisions in the 2015 National Building Code of Canada (NBC 2015) as they pertain to masonry. Reference will be made here to NBC 2005 where appropriate to point out changes. Appendix A contains an introduction to the dynamic analysis of structures to assist in understanding the NBC provisions. The original edition of this guideline (Anderson and Brzev, 2009) was produced to address the many fundamental changes in how seismic risk was evaluated between NBC 2005 and CSA S304.1-04, and their previous versions.

The seismic response of a building structure depends on several factors, such as the structural system and its dynamic characteristics, the building materials and design details, and most importantly, the expected earthquake ground motion at the site. The expected ground motion, termed the *seismic hazard*, can be estimated using probabilistic methods, or be based on deterministic means if there is an adequate history of large earthquakes on identifiable faults in the region of the site.

Canada generally uses a probabilistic method to assess the seismic hazard, and over the years, the probability has been decreasing, from roughly a 40% chance (probability) of being exceeded in 50 years in the 1970s (corresponding to 1/100 per annum probability, also termed the 100-year earthquake), to a 10% in 50-year probability in the 1980s (the 475-year earthquake), to finally a 2% in 50-year probability (the 2475-year earthquake) used for NBC 2015. The change was made so that the risk of building failure in eastern and western Canada would be roughly the same (Adams and Atkinson, 2003), as well as to explicitly recognize that an acceptable probability of severe building damage in North America from seismic activity is about 2% in 50 years. Despite the large changes over the years in the probability level for the seismic hazard determination, the seismic design forces have not changed appreciably because other multiplier factors in the NBC design equations have changed to compensate for these higher hazard values. Thus, while the code seismic design *hazard* has been rising over the years, the average seismic *risk* of failure of buildings designed according to the code has not changed greatly, although there can be substantial changes for certain buildings in certain cases.

Seismic design of masonry structures became an issue following the 1933 Long Beach, California earthquake in which school buildings suffered damage that would have been fatal to students had the earthquake occurred during school hours. At that time, a seismic lateral load equal to the product of a seismic coefficient and the structure weight had to be considered in those areas of California known to be seismically active. Strong motion instruments that could measure the peak ground acceleration or displacement were developed around that time, and in fact, the first strong motion accelerogram was recorded during the 1933 Long Beach earthquake. However, in this era the most widely used strong ground motion acceleration record was measured at El Centro during the 1940 Imperial Valley earthquake in southern California. The 1940 El Centro record became famous and is still used by many researchers studying the effect of earthquakes on structures. However, today there are thousands of records to use, and the choice of how many and which ones to consider, and whether to scale the records or modify them somewhat to match the design spectrum is a major consideration in any seismic risk analysis.

With the availability of ground motion acceleration records (also known as acceleration time history records), it was possible to determine the response of simple structures modelled as single degree of freedom systems. After computers became available in the 1960s it was possible to develop more complex models for analysing the response of larger structures. The availability of computers has also had a huge impact on the ability to predict the ground motion hazard at a site, and in particular, on probabilistic predictions of hazard on which the NBC seismic hazard model is based. They also enhanced the ability of engineers to analyse structures both for linear and nonlinear response.

1.2 Design and Performance Objectives

For many years, seismic design philosophy has been founded on the understanding that it would be too expensive to design most structures to remain elastic under the forces that the earthquake ground motion creates. Accordingly, most modern building codes allow structures to be designed for forces lower than the elastic forces, with the result that such structures may suffer inelastic strains and be damaged in an earthquake, but they should not collapse, and the occupants should be able to safely evacuate the building. The past and present NBC editions follow this philosophy, and allow for lateral design forces smaller than the elastic forces, but they also impose detailing requirements so that the inelastic response remains ductile and a brittle failure is prevented, even for larger than expected events.

Research studies have shown that for most structures the lateral displacements or drifts are about the same, irrespective of whether the structure remains elastic or is allowed to yield and experience inelastic (plastic) deformations. This is known as the equal displacement rule, and it will be discussed later in this chapter as it forms the basis for many of the code provisions.

A comparison of building designs performed according to the NBC 2005 and the NBC 2015 will show an increase in design level forces in some areas of Canada, and a decreased level in others. However, it is expected that the overall difference between these designs is not significant.

The NBC 2015 approach to seismic design follows that of previous editions, but its probability seismic hazard has been determined at many more periods, including periods as long as 10 seconds. Previously the hazard for periods longer than 2 or 4 seconds was based on a conservative empirical decay relation. Thus, the probability of severe damage or near collapse remains about 1/2475 per annum, or about 2% in the predicted 50-year life span of the structure, but hopefully with the NBC 2015 spectral values some designs will be more economical.

Work on new model codes around the world is leading to what is described as "Performance Based Design", a concept that is already being applied by some designers working with private or public owners who have concerns that building damage will have an adverse effect on their ability to maintain their business or operations. NBC 2015 only addresses one performance level, that of collapse prevention and life safety, and is essentially mute on serviceability after smaller seismic events that are expected to occur more frequently. Performance based design attempts to minimize the cost of earthquake losses by weighing the costs of repair and lost business against an increased cost of construction. But this usually requires a nonlinear analysis utilizing many earthquake records.

1.3 Seismic Hazard

4.1.8.4.(1)

The NBC 2015 seismic hazard is based on a 2% in 50 years probability (corresponding to 1/2475 per annum), and it is represented by the 5% damped spectral response acceleration, $S_a(T)$, as was the NBC 2005, but the values have changed to reflect new information on the hazard and on spectral values. The response spectrum for each period has the same probability of exceedance, and as such is termed a Uniform Hazard Spectrum, or UHS.

For a specified location NBC 2015 gives the UHS values at nine periods and approximates with straight lines to construct a spectrum, $S_a(T)$, which is termed the hazard spectrum. For many locations in the country, these values are specified in Table C-3, Appendix C to the NBC 2015, along with the peak ground acceleration (PGA) and peak ground velocity (PGV). For other Canadian locations, it is possible to find the values online at:

http://www.earthquakescanada.nrcan.gc.ca/hazard-alea/interpolat/index-en.php

by entering the coordinates (latitude and longitude) of the location. The program does not directly calculate the $S_a(T)$ values, but instead, interpolates them from the known values at several surrounding locations. For detailed information on the models used as the basis for the NBC 2015 seismic hazard provisions, the reader is referred to Adams et al. (2015), Halchuk et al. (2014), and Atkinson and Adams (2013).

As an example, Table 1-1 provides nine spectral acceleration values $S_a(T)$, plus values for PGA and PGV for a Vancouver site. The S_a values and PGA, plotted as the S_a value at T= 0, are shown in Figure 1-1.

Table 1-1. S_a spectral values for Vancouver for the reference ground condition

S_a values for Vancouver (Coordinates 49.2463, -123.1162) Site Class C											
T	0.05	0.10	0.20	0.30	0.50	1.00	2.00	5.00	10.00	PGA	PGV
S_a	0.453	0.688	0.851	0.855	0.758	0.427	0.258	0.081	0.029	0.369	0.555

$S_a(T)$ is defined for Site Class C which consists of very dense soil or soft rock. For other site conditions a Design Spectrum S(T) = F(T) Sa(T) is defined. F(T) is discussed more fully in the next section.

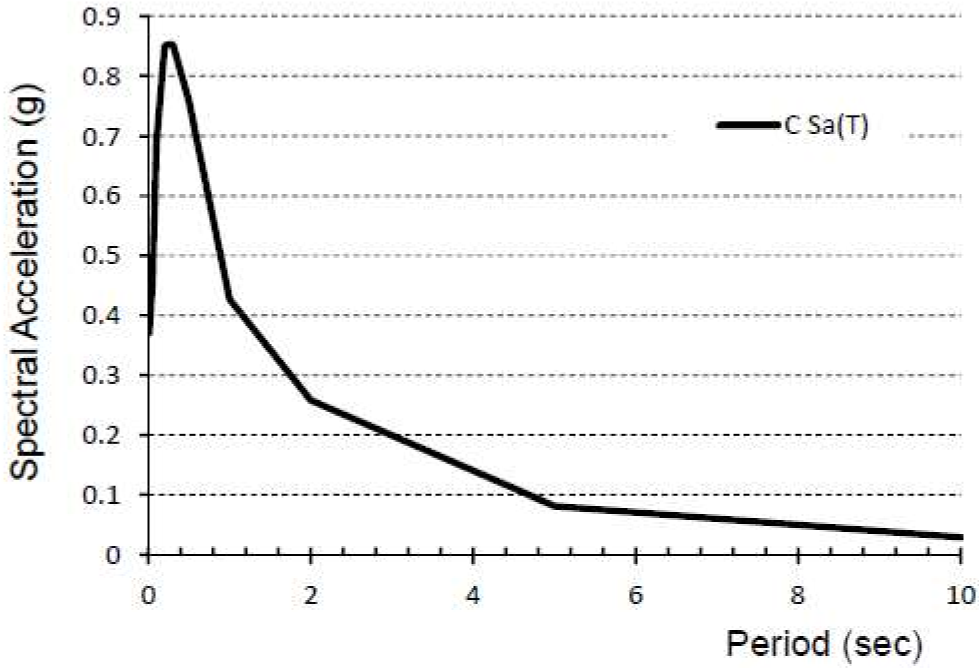

Figure 1-1. Uniform Hazard Spectrum $S_a(T)$ for Vancouver (2% in 50 years probability, 5% damping, Site Class C)

There are limits imposed on the design base shear as discussed in Section 1.6 (NBC 2015 Cl. 4.1.8.11.(2)), which can be demonstrated by plotting S(T) and $S_a(T)$ for Site Class C, as shown in Figure 1-2. These limits affect both the short and long period response and also depend on the type of structure.

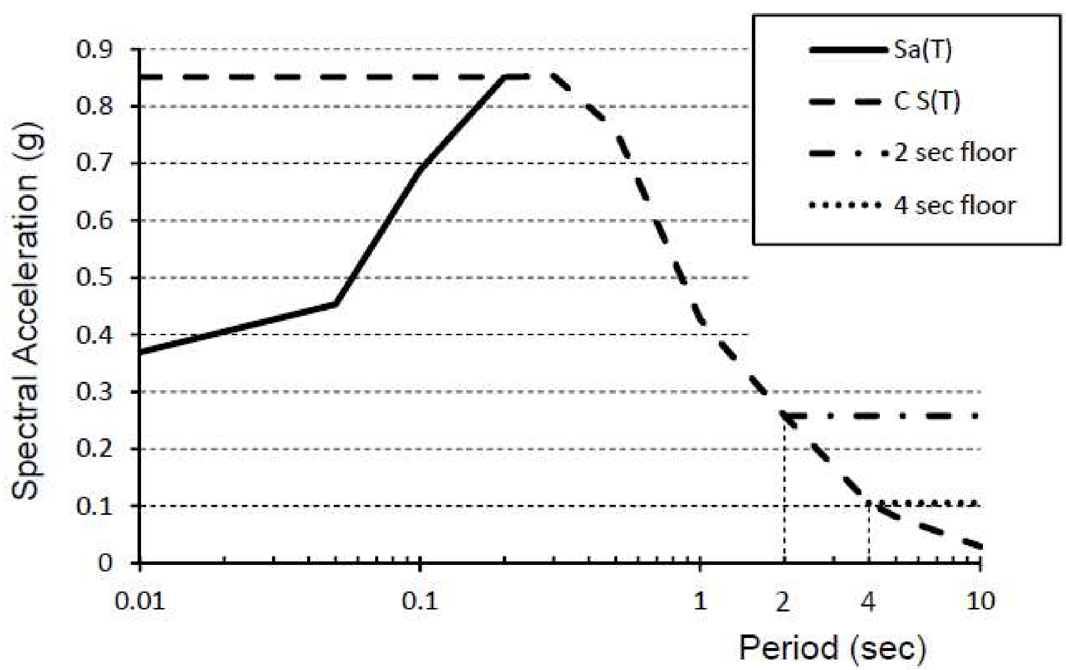

Figure 1-2. Log plot of the UHS Sa(T) and the Design Spectrum S(T) spectrum for Vancouver with limits in the short and long period regions.

The cut off at low periods may appear to be very conservative, but there are other reasons related to the inelastic response of such short-period structures for the design loads to be conservative in this region. Note that many low-rise masonry buildings may have a fundamental period in the order of 0.2 to 0.3 sec.

1.4 Effect of Site Soil Conditions

4.1.8.4

In NBC 2015, the seismic hazard given by the $S_a(T)$ spectrum has been developed for a site that consists of very dense soil or soft rock, referred to as Site class C by NBC 2015. If the structure is to be located on soil that is softer than this, the ground motion may be amplified, or in the case of rock or hard rock sites, the motion may be de-amplified. NBC 2015 introduces a new site coefficient *F(t)* which is applied to the Site Class C $S_a(T)$ spectrum to account for the local ground conditions. The coefficient depends on the building period and level of seismic hazard, as well as on the site properties, which are described in terms of site classes.

The NBC 2015 site coefficient is more detailed than the foundation factors, F_a and F_v, provided in previous code editions, but should better represent the effect of the local soil conditions on the seismic response.

Table 1-2 excerpted from NBC 2015, describes five site classes, labelled from A to E, which correspond to different soil profiles (note that a sixth class, F, is one that fits none of the first five and would require a special investigation). The site classes are based on the properties of the soil or rock in the top 30 m. Site Class C is the base class for which the site coefficients are unity, i.e. it is the type of soil on which the seismic data used to generate the $S_a(T)$ spectrum is based. The table identifies three soil properties that can be used to identify the site class; the best one being the average shear wave velocity, $\overline{V}_s$, which is a parameter that directly affects the dynamic response. The other classes are Average Standard Penetration Resistance N_{60}, and the Soil Undrained Shear Strength s_u.

NBC 2015 and Commentary J (NRC, 2006) do not discuss the level from which the 30 m should be measured. For buildings on shallow foundations, the 30 m should be measured from the bottom of the foundation. However, if the building has a very deep foundation where the ground motion forces transferred to the building may come from both friction at the base and soil pressures on the sides, the answer is not so clear and may require a site-specific investigation.

Table 1-2. NBC 2015 Site Classification for Seismic Response (NBC 2015 Table 4.1.8.4.-A)

Site Class	Ground Profile Name	Average Properties in Top 30 m, as per NBC Note A-4.1.8.4(3) and Table 4.1.8.4.-A		
		Average Shear Wave Velocity, $\overline{V}_s$ (m/s)	**Average Standard Penetration Resistance, $\overline{N}_{60}$**	**Soil Undrained Shear Strength, s_u**
A	**Hard rock(1)(2)**	$\overline{V}_s > 1500$	Not applicable	Not applicable
B	**Rock(1)**	$760 < \overline{V}_s \leq 1500$	Not applicable	Not applicable
C	**Very dense soil and soft rock**	$360 < \overline{V}_s < 760$	$\overline{N}_{60} > 50$	$s_u > 100$kPa
D	**Stiff soil**	$180 < \overline{V}_s < 360$	$15 \leq \overline{N}_{60} \leq 50$	$50 < s_u \leq 100$kPa
E	**Soft soil**	$\overline{V}_s < 180$	$\overline{N}_{60} < 15$	$s_u < 50$kPa
		Any profile with more than 3 m of *soil* with the following characteristics: ▪ plasticity index: PI > 20 ▪ moisture content: w ≥ 40%; and ▪ undrained shear strength: $s_u < 25$ kPa		
F	**Other soils(3)**	Site-specific evaluation required		

Notes:

(1) Site Classes A and B, hard rock and rock, are not to be used if there is more than 3 m of softer materials between the rock and the underside of footing or mat foundations. The appropriate Site Class for such cases is determined on the basis of the average properties of the total thickness of the softer materials (see Note A-4.1.8.4.(3) and Table 4.1.8.4.-A)

(2) Where $\overline{V}_{s30}$ has been measured in-situ, the F(T) values for Site Class A derived from Tables 4.1.8.4.-B to 4.1.8.4.-G are permitted to be multiplied by the factor $0.04+(1500/\overline{V}_{s30})^{1/2}$.

(3) Other *soils* include:

a) liquefiable *soils*, quick and highly sensitive clays, collapsible weakly cemented *soils*, and other *soils* susceptible to failure or collapse under seismic loading,
b) peat and/or highly organic clays greater than 3 m in thickness,
c) highly plastic clays (PI>75) more than 8 m thick, and
d) soft to medium stiff clays more than 30 m thick.

NBC 2015 Tables 4.1.8.4.-B to -G define a function *F(T)* for each soil class and earthquake strength in terms of PGA. Because of different shapes of the $S_a(T)$ spectrum, mainly between eastern and western sites, the code uses PGA_{ref} rather than PGA in determining the *F(T)* values (NBC Cl.4.1.8.4.4):

$PGA_{ref} = 0.8^{*}PGA$ when the ratio $S_a(0.2)/PGA < 2.0$, otherwise $PGA_{ref} = PGA$.

Note that the foundation factors, F_a and F_v, which were used in NBC 2005 and are still needed for some seismic design parameters, are related to the *F(T)* as follows (NBC Cl.4.1.8.4.7):

$F_a = F(0.2)$ and $F_v = F(1.0)$

Values of F(T) factor as a function of the site class and PGA_{ref} are given in the following tables for T values of: 0.2, 0.5, 1.0, 2.0, 5.0, and 10.0 sec.

Table 1-3. Values of F(0.2) as a Function of Site Class and PGA_{ref} (NBC 2015 Table 4.1.8.4.-B)

Site class	F(0.2)				
	$PGA_{ref} \leq 0.1$	$PGA_{ref} = 0.2$	$PGA_{ref} = 0.3$	$PGA_{ref} = 0.4$	$PGA_{ref} \geq 0.5$
A	0.69	0.69	0.69	0.69	0.69
B	0.77	0.77	0.77	0.77	0.77
C	1.00	1.00	1.00	1.00	1.00
D	1.24	1.09	1.00	0.94	0.90
E	1.64	1.24	1.05	0.93	0.85
F	(1)	(1)	(1)	(1)	(1)

Table 1-4. Values of F(0.5) as a Function of Site Class and PGA_{ref} (NBC 2015 Table 4.1.8.4.-C)

Site class	F(0.5)				
	$PGA_{ref} \leq 0.1$	$PGA_{ref} = 0.2$	$PGA_{ref} = 0.3$	$PGA_{ref} = 0.4$	$PGA_{ref} \geq 0.5$
A	0.57	0.57	0.57	0.57	0.57
B	0.65	0.65	0.65	0.65	0.65
C	1.0	1.0	1.0	1.0	1.0
D	1.47	1.30	1.20	1.14	1.10
E	2.47	1.80	1.48	1.30	1.17
F	(1)	(1)	(1)	(1)	(1)

Table 1-5. Values of F(1.0) as a Function of Site Class and PGA_{ref} (NBC 2015 Table 4.1.8.4.-D)

Site class	F(1.0)				
	$PGA_{ref} \leq 0.1$	$PGA_{ref} = 0.2$	$PGA_{ref} = 0.3$	$PGA_{ref} = 0.4$	$PGA_{ref} \geq 0.5$
A	0.57	0.57	0.57	0.57	0.57
B	0.63	0.63	0.63	0.63	0.63
C	1.0	1.0	1.0	1.0	1.0
D	1.55	1.39	1.31	1.25	1.21
E	2.81	2.08	1.74	1.53	1.39
F	(1)	(1)	(1)	(1)	(1)

Table 1-6. Values of F(2.0) as a Function of Site Class and PGA_{ref} (NBC 2015 Table 4.1.8.4.-E)

Site class	F(2.0)				
	$PGA_{ref} \leq 0.1$	$PGA_{ref} = 0.2$	$PGA_{ref} = 0.3$	$PGA_{ref} = 0.4$	$PGA_{ref} \geq 0.5$
A	0.58	0.58	0.58	0.58	0.58
B	0.63	0.63	0.63	0.63	0.63
C	1.0	1.0	1.0	1.0	1.0
D	1.57	1.44	1.36	1.31	1.27
E	2.90	2.24	1.92	1.72	1.58
F	(1)	(1)	(1)	(1)	(1)

Table 1-7. Values of F(5.0) as a Function of Site Class and PGA_{ref} (NBC 2015 Table 4.1.8.4.-F)

Site class	F(5.0)				
	$PGA_{ref} \leq 0.1$	$PGA_{ref} = 0.2$	$PGA_{ref} = 0.3$	$PGA_{ref} = 0.4$	$PGA_{ref} \geq 0.5$
A	0.61	0.61	0.61	0.61	0.61
B	0.64	0.64	0.64	0.64	0.64
C	1.00	1.00	1.00	1.00	1.00
D	1.58	1.48	1.41	1.37	1.34
E	2.93	2.40	2.14	1.96	1.84
F	(1)	(1)	(1)	(1)	(1)

Table 1-8. Values of F(10.0) as a Function of Site Class and PGA_{ref} (NBC 2015 Table 4.1.8.4.-G)

Site class	F(10.0)				
	$PGA_{ref} \leq 0.1$	$PGA_{ref} = 0.2$	$PGA_{ref} = 0.3$	$PGA_{ref} = 0.4$	$PGA_{ref} \geq 0.5$
A	0.67	0.67	0.67	0.67	0.67
B	0.69	0.69	0.69	0.69	0.69
C	1.00	1.00	1.00	1.00	1.00
D	1.49	1.41	1.37	1.34	1.31
E	2.52	2.18	2.00	1.88	1.79
F	(1)	(1)	(1)	(1)	(1)

Table 1-9 and 1-10 present values of F(PGA) and F(PGV) as a function of the site class and PGA_{ref}.

Table 1-9. Values of F(PGA) as a Function of Site Class and PGA_{ref} (NBC 2015 Table 4.1.8.4.-H)

Site class	F(PGA)				
	$PGA_{ref} \leq 0.1$	$PGA_{ref} = 0.2$	$PGA_{ref} = 0.3$	$PGA_{ref} = 0.4$	$PGA_{ref} \geq 0.5$
A	0.90	0.90	0.90	0.90	0.90
B	0.87	0.87	0.87	0.87	0.87
C	1.00	1.00	1.00	1.00	1.00
D	1.29	1.10	0.99	0.93	0.88
E	1.81	1.23	0.98	0.83	0.74
F	(1)	(1)	(1)	(1)	(1)

Notes: (1) See Sentence 4.1.8.4.(6).

Table 1-10. Values of F(PGV) as a Function of Site Class and PGA_{ref} (NBC 2015 Table 4.1.8.4.-I)

Site class	F(PGV)				
	$PGA_{ref} \leq 0.1$	$PGA_{ref} = 0.2$	$PGA_{ref} = 0.3$	$PGA_{ref} = 0.4$	$PGA_{ref} \geq 0.5$
A	0.62	0.62	0.62	0.62	0.62
B	0.67	0.67	0.67	0.67	0.67
C	1.00	1.00	1.00	1.00	1.00
D	1.47	1.30	1.20	1.14	1.10
E	2.47	1.80	1.48	1.30	1.17
F	(1)	(1)	(1)	(1)	(1)

Notes: (1) See Sentence 4.1.8.4.(6).

Note that the *F(T)*, *F(PGA)*, and *F(PGV)* values depend on the level of seismic hazard as well as the site soil class. For soft soil sites (site classes D and E), motion from a high hazard event would lead to higher shear strains in the soil, which gives rise to higher soil damping and results in reduced site coefficients. The softer the soil, as given by a higher site classification, the larger the site coefficients. For rock and hard rock, the site coefficients will generally be less than unity and are not much affected by the seismic hazard level.

The calculation of *S(T)* values will be illustrated with an example and the resulting spectra for site Classes C and E are given in Table 1-11.

Figure 1-3 shows the design seismic hazard spectrum, $S_a(T)$, for Vancouver for a firm ground site, Class C, and a soft soil site, Class E. Since soil Class C is the reference soil class the *F(T)* values are all unity and the *S(T)* values are the same as the $S_a(T)$ values. The *F(T)* values of site Class E must be interpolated from Tables 4.1.8.4-B to -G.

The calculations to determine $S_a(T)$ for the Class E site in Vancouver are shown below (see NBC Clause 4.1.8.4.9)):

For T≤0.2 sec: $S(0.2) = F(0.2)*S_a(0.2)$ or F(0.5)Sa(0.5), whichever is larger
For T= 0.5 sec: $S(0.5) = F(0.5)*S_a(0.5)$
For T= 1.0 sec: $S(1.0) = F(1.0)*S_a(1.0)$
For T= 2.0 sec: $S(2.0) = F(2.0)*S_a(2.0)$
For T= 5.0 sec: $S(5.0) = F(5.0)*S_a(5.0)$
For T≥10.0 sec: $S(10.0) = F(10.0)*S_a(10.0)$

Table 1-11. Design Spectral Values and F(T) Values for Site Class C and E in Vancouver

S=S_a values for Vancouver (Coordinates 49.2463, -123.1162), Site Class C											
T	0.05	0.10	0.20	0.30	0.50	1.00	2.00	5.00	10.00	PGA	PGV
S=Sa	0.453	0.688	0.851	0.855	0.758	0.427	0.258	0.081	0.029	0.369	0.555
F(T) values for Site Class E											
T	0.05	0.10	0.20	0.30	0.50	1.00	2.00	5.00	10.00	PGA	PGV
F(T)			0.967		1.356	1.591	1.782	2.016	1.917		
S(T) values for Vancouver, Site Class E											
S			0.823		1.028	0.681	0.460	0.163	0.056		

The resulting S(T) design spectra for soil Classes C and E for Vancouver are plotted in Figure 1-3. Note that since F(0.2)*S(0.2) is less than F(0.5)*S(0.5), for Site Class E the S(T) spectra for T≤0.2 is the F(0.5)*S(0.5) value.

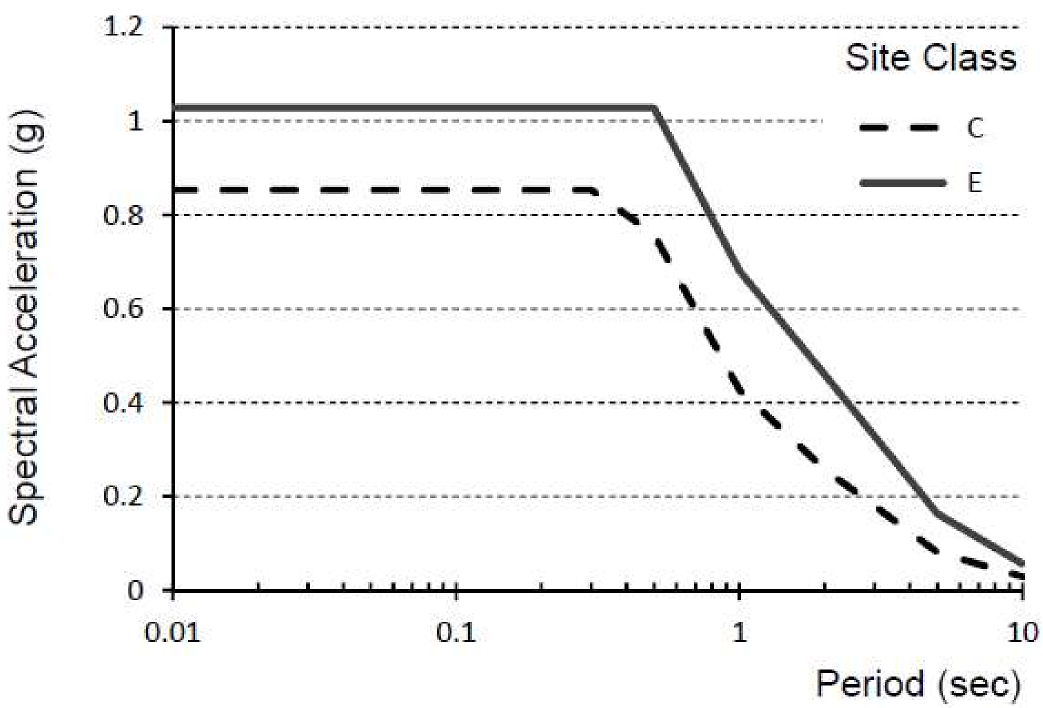

Figure 1-3. NBC 2015 design spectra for Vancouver for site Classes C and E.

1.5 Methods of Analysis

4.1.8.7

NBC 2015 prescribes two methods of calculating the design base shear for a structure. The *dynamic method* is the default method, but the *equivalent static method* can be used if the structure meets any of the following criteria:
(a) is located in a region of low seismic activity where $I_E F_a S_a(0.2) < 0.35$ (I_E is the earthquake importance factor of the structure as defined in Clause 4.1.8.5.(1)), or
(b) is a regular structure less than 60 m in height with period, T_a, less than 2 seconds in either direction (T_a is defined as the fundamental lateral period of vibration of the structure in the direction under consideration, as defined in Clause 4.1.8.11.(3)), or
(c) is an irregular structure, but does not have Type 7 or Type 9 irregularity, and is less than 20 m in height with period, T_a, less than 0.5 seconds in either direction.

The equivalent static method will be described in this section because it likely can be used on the majority of masonry buildings given the above criteria, and notwithstanding, if the dynamic method is used, it must be calibrated back to the base shear determined from the equivalent static analysis procedure. Basic concepts of the modal dynamic analysis method are presented in Appendix A, and further discussion is offered in Section 1.14.

1.6 Base Shear Calculations- Equivalent Static Analysis Procedure

4.1.8.11

The lateral earthquake forces used for design are specified in the NBC 2015, and are based on the maximum (design) base shear V_e of the structure as given by Clause 4.1.8.11, and is the base shear if the structure were to remain elastic. Design base shear, V, is equal to V_e reduced by the force reduction factors, R_d and R_o, (related to ductility and overstrength, respectively; discussed in Section 1.7), and increased by the importance factor I_E (see *Table 1-12* for a description of parameters used in these relations), thus;

$$V = \frac{V_e I_E}{R_d R_o}$$

where $V_e = S(T_a) M_v W$, represents the elastic base shear, M_v is a multiplier that accounts for higher mode shears, and W is the dead load attached to the SFRS, as defined in *Table 1-12*.

The relationship between V_e and V is shown in Figure 1-4. Note that the actual strength of the structure is greater than the design strength because of the overstrength factor R_o.

T_a denotes the *fundamental period* of vibration of the building or structure in seconds in the direction under consideration. The fundamental period of wall structures is given in the NBC 2015 by:

a) $T_a = 0.05(h_n)^{3/4}$, where h_n is the height of the building in metres (Cl.4.1.8.11.3.(c)), or

b) other established methods of mechanics, except that T_a should not be greater than 2.0 times that determined in (a) above (Sub Cl.4.1.8.11.3.(d)(iii). Note the 4 second floor in Fig 1-3.

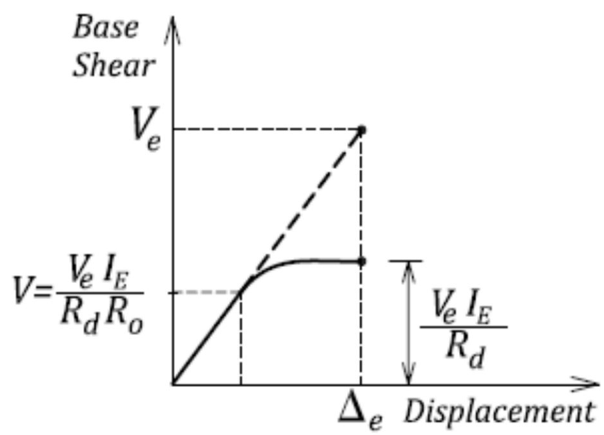

Figure 1-4. Relation between design base shear, V , and elastic base shear, V_e.

The period given by the NBC 2015 in (a) is a conservative (short) estimate based on measured values for existing buildings. Using method (b) will generally result in a longer period, with resulting lower forces, and should be based on stiffness values reflecting possible cracked sections and shear deformations. For the purpose of calculating deflections, there is no limit on the calculated period as a longer period results in larger displacements (a conservative estimate), but it should never be less than that period used to calculate the forces.

NBC 2015 Clause 4.1.8.11.(2) prescribes the following lower and upper bounds for the design base shear, V ;

a) Lower bound:
Because of uncertainties in the hazard spectrum, $S_a(T)$, for periods greater than 2 seconds, the minimum design base shear for walls, coupled walls and wall frame systems should not be taken less than:

$$V_{\min} = \frac{S(4.0) M_v I_E W}{R_d R_o}$$

For moment resisting frames, braced frames, and other systems, the minimum base shear should not be taken less than:

$$V_{\min} = \frac{S(2.0) M_v I_E W}{R_d R_o}$$

b) Upper bound:
Short period structures have small displacements, and there is not a huge body of evidence of failures for very low period structures, provided the structure has some ductile capacity. Thus an upper bound on the design base shear, provided $R_d \geq 1.5$, need not be greater than the larger of:

$$V_{max} = \left(\frac{2S(0.2)}{3}\right)\left(\frac{I_E W}{R_d R_o}\right) \quad \text{and}$$

$$V_{max} = (S(0.5))\left(\frac{I_E W}{R_d R_o}\right)$$

M_v is not included in the above equations as $M_v = 1$ for short periods.

Table 1-12. NBC 2015 Seismic Design Parameters

Design parameter		NBC reference
$S(T)$	the design spectral acceleration that includes the site soil coefficient *F(T)* For T≤0.2 sec: *S(0.2) = F(0.2)*S$_a$(0.2)* or F(0.5)Sa(0.5), whichever is larger For T= 0.5 sec: *S(0.5) = F(0.5)*S$_a$(0.5)* For T= 1.0 sec: *S(1.0) = F(1.0)*S$_a$ (1.0)* For T= 2.0 sec: *S(2.0) = F(2.0)*S$_a$(2.0)* For T= 5.0 sec: *S(5.0) = F(5.0)*S$_a$(5.0)* For T≥10.0 sec: *S(10.0) = F(10.0)*S$_a$(10.0)*	Cl.4.1.8.4(9)
M_v	higher mode factor (see Section 1.8)	Cl.4.1.8.11.(6) Cl.4.1.8.11.(8) Table 4.1.8.11
I_E	importance factor for the design of the structure: 1.5 for post-disaster buildings, 1.3 for high importance structures, including schools and places of assembly that could be used as refuge in the event of an earthquake, 1.0 for normal buildings, and 0.8 for low importance structures such as farm buildings where people do not spend much time. See Table 4.1.2.1 in NBC 2015 Part 4 for more complete definitions of the importance categories. There are also requirements for the serviceability limit states for the different categories.	Cl.4.1.8.5(1) Table 4.1.8.5
W	dead load plus some portion of live load that would move laterally with the structure (also known as seismic weight). Live loads considered are 25% of the design snow load, 60% of storage loads for areas used for storage, and the full contents of any tanks.	Cl.4.1.8.2
R_d =	ductility related force modification factor that represents the capability of a structure to dissipate energy through inelastic behaviour (see Table 1-13 and Section 1.7); *ranges from 1.0 for unreinforced masonry to 3.0 for ductile masonry shear walls.*	Table 4.1.8.9
R_o =	overstrength related force modification factor that accounts for the dependable portion of reserve strength in the structure (see Table 1-13 and Section 1.7); *equal to 1.5 for all reinforced masonry walls.*	Table 4.1.8.9

Note that the design base shear force, V, corresponds to the design force at the ultimate limit state, where the structure is assumed to be at the point of collapse. Consequently, seismic loads are designed with a load factor value of 1.0 when used in combination with other loads (e.g. dead and live loads; see Table 4.1.3.2.-A, NBC 2015). It is also useful to recall that while V represents the design base shear, individual members are designed using factored resistances, ϕR, and since the nominal resistance, R, is greater than the factored resistance, the actual base shear capacity will be approximately equal to VR_o, as shown in Figure 1-4.

1.7 Force Reduction Factors R_d and R_o

4.1.8.9

Table 1-13 (NBC 2015 Table 4.1.8.9) gives the R_d *and* R_o values for the different types of lateral load-resisting systems, which are termed the Seismic Force Resisting Systems, SFRS(s), by NBC 2015 Cl.4.1.8.2. The SFRS is that part of the structural system that has been considered in the design to provide the lateral resistance to the earthquake forces and effects. In addition to providing the R_d *and* R_o values, the table lists height limits for the different systems, depending on the level of seismic hazard and importance factor, I_E.

Table 1-13. Masonry R_d and R_o Factors and General Restrictions(1) - Forming Part of Sentence 4.1.8.9(1)

Type of SFRS	R_d	R_o	Height Restrictions (m) (2)				
			Cases where $I_E F_a S_a(0.2)$				Cases where $I_E F_v S_a(1.0)$ >0.3
			<0.2	≥0.2 to <0.35	≥0.35 to ≤0.75	>0.75	
Masonry Structures Designed and Detailed According to CSA S304-14							
Ductile shear walls	3.0	1.5	NL	NL	60	40	40
Moderately Ductile shear walls	2.0	1.5	NL	NL	60	40	40
Conventional construction - shear walls	1.5	1.5	NL	60	30	15	15
Conventional construction - moment resisting frames	1.5	1.5	NL	30	NP	NP	NP
Unreinforced masonry	1.0	1.0	30	15	NP	NP	NP
Other masonry SFRS(s) not listed above	1.0	1.0	15	NP	NP	NP	NP

Notes: (1) See Article 4.1.8.10.
(2) NP = system is not permitted.
NL = system is permitted and not limited in height as an SFRS; height may be limited in other parts of the NBC.
Numbers in this Table are maximum height limits above grade in m.
The most stringent requirement governs.

Commentary

NBC 2015 Table 4.1.8.9 identifies the following five SFRS(s) related to masonry construction:

1. Ductile shear walls (new SFRS introduced in NBC 2015)
2. Moderately Ductile shear walls
3. Conventional construction: shear walls and moment resisting frames
4. Unreinforced masonry
5. Other undefined masonry SFRS(s)

Note that Ductile shear walls are assigned the highest R_d value of 3.0, leading to the lowest design forces for masonry structures. The detailing requirements, given in CSA S304 -14, are the most restrictive of all the masonry shear wall types. However, the height limitations imposed by the NBC 2015 are the most liberal, allowing structures up to 60 m in height (approximately 20 storeys) in moderately high seismic regions, and up to 40 m in higher seismic regions.

Moderately Ductile shear walls, R_d = 2.0, have the same height restrictions as Ductile shear walls. They have less restrictive detailing requirements, but have to be designed for larger forces, generally resulting in a stiffer structure with less ductility demand. Moderately ductile shear walls are required for masonry SFRS(s) used in post-disaster buildings, due to the NBC requirement for an R_d = 2.0 for these structures.

Moderately Ductile squat shear walls, those with a height-to-length ratio less than 1, are a separate class of Moderately ductile shear wall. They are allowed higher shear resistance, and less restrictive requirements on the height-to-thickness ratio, when compared to regular Moderately Ductile shear walls.

Conventional construction shear walls and moment-resisting frames both have R_d=1.5, with more onerous height restrictions, but less stringent detailing requirements than Moderately Ductile walls. Masonry moment-resisting frames are limited to low seismic regions and are not discussed in CSA S304-14. Conventional construction is the most common type of shear wall used in typical masonry structures.

Unreinforced masonry construction is only allowed where $I_E F_a S_a(0.2) < 0.35$. It is limited to a height of 15 or 30 m depending on the level of seismic hazard. Unreinforced masonry does not have a good record in past earthquakes, and is assigned $R_d = R_o = 1.0$ values, as there is usually no ductility and brittle failures are a possibility.

The R_o factor in NBC 2015 is an overstrength factor to account for the real resistance capacity of the structure when compared to the factored design resistance. It is made up of 3 components: i) $1/\phi = 1.18 \approx 1.2$, ii) a factor that accounts for the expected yield strength of the reinforcement being above the specified yield strength, and iii) a factor of about 1.1 that recognizes that because of restrictions on possible core locations for the reinforcement in modular masonry walls, the amount of reinforcement is in most cases larger than required. This results in an $R_o = 1.5$ after some rounding of the factors (Mitchell et al., 2003).

A comparison of masonry wall classes contained in NBC 2015 and NBC 2005 is presented in Table 1-14. The class Limited ductility shear walls no longer exists in NBC 2015, and a new class (Ductile shear walls) has been introduced.

Table 1-14. A comparison of NBC 2015 and NBC 2005 Classes of Masonry Walls Based on Seismic Performance Requirements

NBC 2005 Table 4.1.8.9 and CSA S304.1-04	NBC 2015 Table 4.1.8.9 and CSA S304-14	Comments
Unreinforced masonry R_d =1.0 R_o =1.0	***Unreinforced masonry*** R_d =1.0 R_o =1.0	Slight difference in where unreinforced masonry could be used
Shear walls with conventional construction R_d =1.5 R_o =1.5	***Shear walls with conventional construction*** R_d =1.5 R_o =1.5	Changes in seismic reinforcement requirements depending on seismic hazard in S304-14
Limited ductility shear walls R_d =1.5 R_o =1.5	Does not exist	This class was removed from S304-14
Moderately Ductile shear walls R_d =2.0 R_o =1.5	***Moderately Ductile shear walls*** R_d =2.0 R_o =1.5	Seismic design requirements relaxed for low-rise walls in S304-14
Moderately Ductile squat shear walls R_d =2.0 R_o =1.5	***Moderately Ductile squat shear walls*** R_d =2.0 R_o =1.5	No major changes in seismic reinforcement requirements in S304-14
Not included	***Ductile shear walls*** R_d =3.0 R_o =1.5	New class introduced in NBC 2015 and S304-14

1.8 Higher Mode Effects (M_v factor)

4.1.8.11.(6)

In the determination of elastic base shear, V_e, only the first mode spectral value $S(T)$ is used. In longer period structures, higher modes will also contribute to the base shear, and to account for this the M_v factor is introduced. M_v is dependent on the type of SFRS, the fundamental period T_a , and the ratio $S(0.2)/S(5.0)$, and its values are given in Table 1-15. Part of the base shear is assigned to the top modes to ensure that the shear forces in the top of the structure are adequate. Applying larger loads to the top of the structure results in the moments along the height being too large, and so a second factor, J , is introduced to reduce the calculated moments in the lower portion of the structure.

A discussion about the base overturning reduction factor, J , (also shown in Table 1-15) is provided in Section 1.10.

Table 1-15. Higher Mode Factor, M_v, and Base Overturning Reduction Factor, $J^{(1)(2)(3)(4)}$, for Walls and Wall Frame Systems (an excerpt from NBC 2015 Table 4.1.8.11)

S(0.2)/S(5.0)	M_v for $T_a \le 0.5$	M_v for $T_a=1.0$	M_v for $T_a=2.0$	M_v for $T_a \ge 5.0$	J for $T_a \le 0.5$	J for $T_a=1.0$	J for $T_a=2.0$	J for $T_a \ge 5.0$
5	1	1	1	1.25(7)	1	0.97	0.85	0.55(8)
20	1	1	1.18	2.30(7)	1	0.80	0.60	0.35(8)
40	1	1.19	1.75	3.70(7)	1	0.63	0.46	0.28(8)
65	1	1.55	2.25	4.65(7)	1	0.51	0.39	0.23(8)

Notes:
(1) For intermediate values of the spectral ratio S(0.2)/S(5.0), M_v and J shall be obtained by linear interpolation.
(2) For intermediate values of the fundamental lateral period T_a, $S(T_a)*M_v$ shall be obtained by linear interpolation using the values of M_v obtained in accordance with Note (1).
(3) For intermediate values of the fundamental lateral period T_a, J shall be obtained by linear interpolation using the values of J obtained in accordance with Note (1).
(4) For a combination of different seismic force resisting systems (SFRS) not given in Table 4.1.8.11 that are in the same direction under consideration, use the highest M_v factor of all the SFRS and the corresponding value of J.
(7) For fundamental lateral periods, T_a, greater than 4.0 s, use the 4.0s values of $S(T_a)*M_v$ obtained by interpolation between 2.0s and 5.0s using the value of M_v obtained in accordance with Note (1). See 4.1.8.11.(2)(a).
(8) For fundamental lateral periods, T_a, greater than 4.0 s, use the 4.0s values of J obtained by interpolation between 2.0s and 5.0s using the value of J obtained in accordance with Note (1). See Clause 4.1.8.11.(2)(a).

Commentary

For structures with periods T_a greater than 1.0 s (typically, buildings of 10 storeys or higher), the contribution of higher modes to the base shear becomes increasingly important. In the eastern part of Canada, where $S_a(0.2)/S_a(5.0)$ tends to be larger than in the west, and where the $S_a(T)$ spectrum decreases sharply with periods beyond 0.2 seconds, the spectral acceleration for the second and third modes can be high compared to the first mode, hence these modes make a substantial contribution to the base shear. In western Canada, the spectrum does not decrease as sharply with increasing period, and the higher mode shears are less important. The M_v factor is largest for wall structures, ranging in value up to 4.65. This is relevant for high-rise masonry wall structures when compared to frames, because their modal mass for the higher modes is larger and because the difference in periods between the modes is larger.

For periods that fall between the published T_a values it is important to note that interpolation between the two periods should be done on the product $S(T) \cdot M_v$, and not on the individual terms.

Beyond periods of 5 seconds, M_v is assumed constant, although it theoretically could be larger. However, since V_e is conservatively based on the $S(4.0)$ spectral value, it is appropriate to use the 5 second value of M_v.

1.9 Vertical Distribution of Seismic Forces

4.1.8.11.(7)

The total lateral seismic force, V, is to be distributed such that a portion, F_t, is assumed to be concentrated at the top of the building; the remainder $(V - F_t)$ is to be distributed along the height of the building, including the top level, in accordance with the following formula (see Figure 1-5):

$$F_x = (V - F_t) \cdot \frac{W_x h_x}{\sum_{i=1}^{n} W_i h_i}$$

where

F_x – seismic force acting at level x

F_t – a portion of the base shear to be applied, in addition to force F_n, at the top of the building

h_x – height from the base of the structure up to the level x (base of the structure denotes level at which horizontal earthquake motions are considered to be imparted to the structure - usually the top of the foundations)

W_x - a portion of seismic weight, W, that is assigned to level x; that is, the weight at level x which includes the floor weight plus a portion of the wall weight above and below that level.

The seismic weight W is the sum of the weights at each floor; normally this would be the weight of the floors, walls and other rigidly attached masses that would move with the SFRS, hence (Clause 4.1.8.11.(5))

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

Commentary

The above formula for the force distribution is based on a linear first mode approximation for the acceleration at each level. The purpose of applying force F_t at the top of the structure is to increase the storey shear forces in the upper part of longer period structures where the first mode approximation is not correct. For periods less than 0.7 sec, shear is dominated by the first mode and so $F_t = 0$. The F_t force is determined as follows, see Clause 4.1.8.11.(7):

$F_t = 0$ for $T_a \le 0.7$ sec

$F_t = 0.07 T_a V$ for 0.7 < $T_a \le 3.6$ sec

$F_t = 0.25V$ for T_a > 3.6 sec

The remaining force, $V - F_t$, is distributed assuming the floor accelerations vary linearly with height from the base.

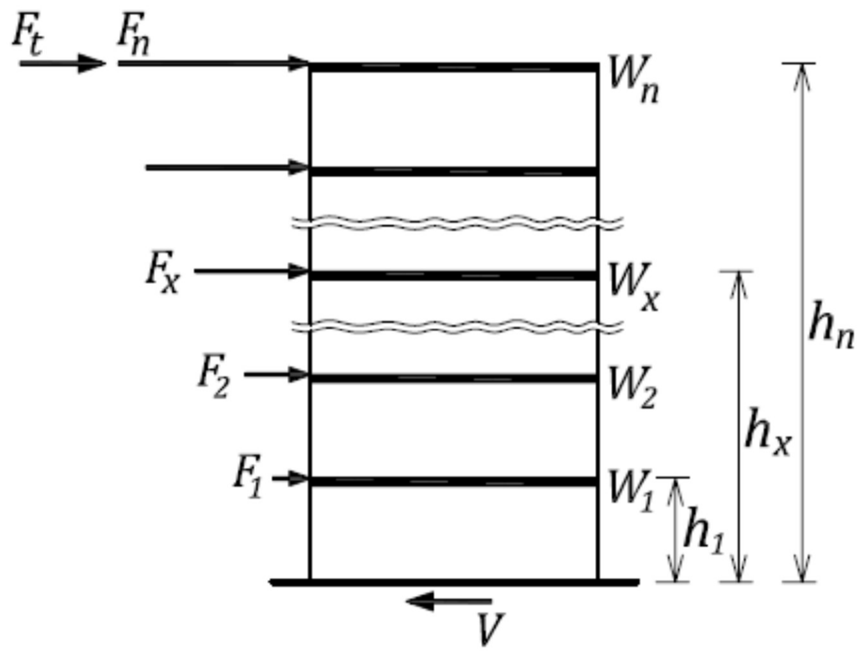

Figure 1-5. Vertical force distribution.

1.10 Overturning Moments (J factor)

4.1.8.11.(6) 4.1.8.11.(8)

While higher mode forces can make a significant contribution to the base shear, they make a much smaller contribution to the storey moments. Thus, moments at each storey level determined from the seismic floor forces, which include the higher mode shears in the form of the F_t factor, result in overturning moments that are too large. Previous editions of the NBC have traditionally used a factor, termed the J factor, to reduce the moments. The value of the J factor and how it is applied over the height of the structure is substantially the same in NBC 2015, but the values are now dependent on T_a.

The J factor values are given in Table 1-15 and illustrated in Figure 1-6. The overturning moment at any level shall be multiplied by the factor J_x where

$$J_x = 1.0 \text{ for } h_x \geq 0.6h_n \text{ and , } J_x = J + (1 - J)(h_x / 0.6h_n) \text{ for } h_x < 0.6h_n$$

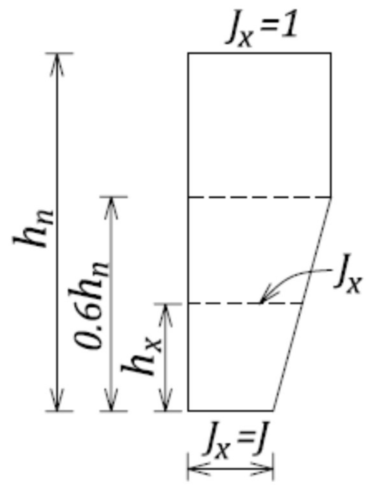

Figure 1-6. Distribution of the J_x factor over the building height.

Commentary

How the J factor and reduced overturning moments are incorporated into a structural analysis is not always straightforward, and it depends on the structural system.

For shear wall structures, the overturning moments can be calculated using the floor forces from the lateral force distribution, and then reduced by the J_x factor at each level to give the design overturning moments. Without applying the J factor, the wall moment capacity would be too high, leading to higher shears when the structure yields, and could result in a shear failure.

For frames, the beam shears and moments and axial loads, resulting from applying the code lateral seismic forces at each floor level, will be too large; but the column shears would not increase. This would essentially result in higher axial loads in the columns, but not increase the shear demand on the structure, and so would be conservative in that the columns would be stronger than necessary, especially in the lower levels. The J factor for frames is usually small, and it is believed that many designers ignore it as it is conservative to do so.

1.11 Torsion

1.11.1 Torsional effects

4.1.8.11.(9)

Torsional effects, that are concurrent with the effects of the lateral forces, F_x, and that are caused by the following torsional moments need to be considered in the design of the structure:

a) torsional moments introduced by eccentricity between the centre of mass and the centre of resistance, and their dynamic amplification, or
b) torsional moments due to accidental eccentricities.

In determining the torsional forces on members, the stiffness of the diaphragms is important. The discussion in Sections 1.11.1 to 1.11.3 considers rigid diaphragms only, while flexible diaphragms are discussed in Section 1.11.4.

Commentary

Torsional effects have been associated with many building failures during earthquakes. Torsional moments, or torques, arise when the lateral inertial forces acting through the centre of mass at each floor level do not coincide with the resisting structural forces acting through the centres of resistance. The *centre of mass*, C_M, is a point through which the lateral seismic inertia force can be assumed to act. The seismic shear is resisted by the vertical elements, and if the resultant of the shear forces does not lie along the same line of action as the inertia force acting through the centre of mass, then a torsional moment about a vertical axis will be created. The *centre of resistance*, C_R, also known as the centre of stiffness, is a point through which the resultant of all resisting forces act provided there is no torsional rotation of the structure. If the centre of mass at a certain floor level does not coincide with its centre of resistance, the building will twist in the horizontal plane about C_R. Torsion generates significant additional forces and displacements for the vertical elements (e.g. walls) furthest away from C_R. Ideally, C_R should coincide with, or be close to C_M, and sufficient torsional resistance should be available to keep the rotations small. Figure 1-7 shows two different plan configurations, one of which has a non-symmetric wall layout (a), and the other a symmetric layout (b). Both plans have approximately the same amount of walls in each direction, but the symmetric building will perform better. The location of the shear walls determines the torsional stiffness of the structure; widely spaced walls provide high torsional stiffness and consequently small torsional rotations. Walls placed around the perimeter of the building, such as shown in Figure 1-7b), have very high torsional stiffness and are representative of low-rise or single-storey buildings. Taller buildings, which often have several shear walls distributed across the footprint of the structure, can also give satisfactory torsional resistance (see Section 1.11.2 for a discussion on torsional sensitivity).

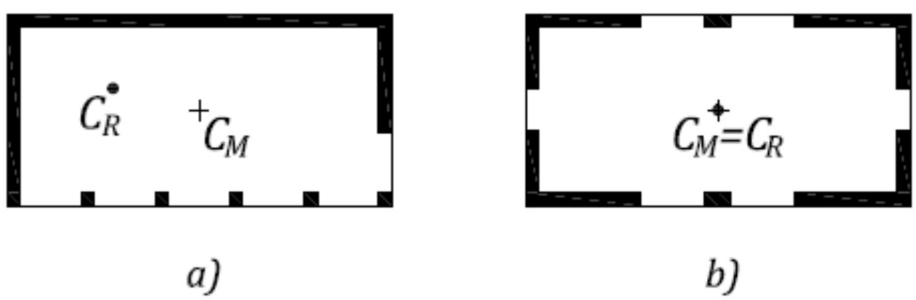

Figure 1-7. Building plan: a) non-symmetric wall layout (significant torsional effects), and b) symmetric wall layout (minor torsional effects).

*Figure 1-8*a) shows a building plan (of a single storey building, or one floor of a multi-storey building), for which the centre of mass, C_M, and the centre of resistance, C_R, do not coincide. The distance between C_R (at each floor) and the line of action of the lateral force (at each floor), which passes through C_M is termed the *natural floor eccentricity*, e_x (note that the eccentricity is measured perpendicular to the direction of lateral load). The effect of the lateral seismic force, F_x, which acts at point C_M, can be treated as the superposition of the following two load cases: a force F_x acting at point C_R (no torsion, only translational displacements, see *Figure 1-8*b), and pure torsion in the form of torsional moment, T_x, about the point C_R, as shown in *Figure 1-8*c). The torsional moment, T_x, is calculated as the product of the floor force, F_x, and the eccentricity e_x.

In addition to the natural eccentricity, the NBC requires consideration of an additional eccentricity, termed the *accidental eccentricity*, e_a. Accidental eccentricity is considered because of possible errors in determining the natural eccentricity, including errors in locating the centres of mass as well as the centres of resistance, additional eccentricities that might come from yielding of some elements, and perhaps from some torsional ground motion.

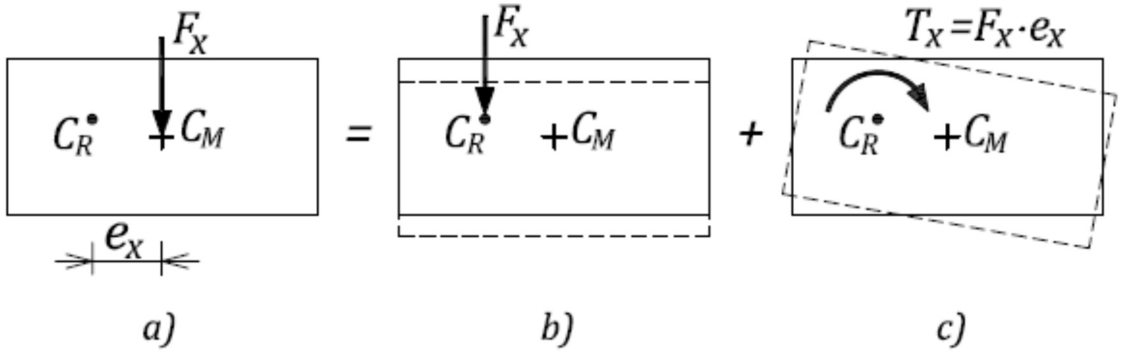

Figure 1-8. Torsional effects a), can be modelled as a combination of a seismic force, F_x, at point C_R (causing translational displacements only) b), and a torsional moment, $T_x = F_x \cdot e_x$ (causing rotation of building plan) about point C_R c).

Finding the centre of resistance, C_R, may be a complex task in some cases. For single-storey structures it is possible to determine a centre of stiffness, which is the same as the C_R. However, in multi-storey structures, C_R is not well defined. For a given set of lateral loads, it is possible to find the location on each floor through which the lateral load must pass, so as to produce zero rotation of the structure about a vertical axis. These points are often called the centres of rigidity, rather than centres of stiffness or resistance, but they are a function of the loading as well as the structure, and so centres of rigidity are not a unique structural property. A different set of lateral loads will give different centres of rigidity. Earlier versions of the NBC (before 2005) required the determination of the C_R location so as to explicitly determine e_x, as it was necessary to amplify e_x (by factors of 1.5 or 0.5) to determine the design torque at each floor level. NBC 2015 does not require this amplification, so the effect of the torque from the natural eccentricities can come directly from a 3-D lateral load analysis, without the additional work of explicitly determining e_x. However, NBC 2015 requires a comparison of the torsional stiffness to the lateral stiffness of the structure to evaluate the torsional sensitivity, and so requires increased computational effort in this regard.

1.11.2 Torsional sensitivity

4.1.8.11.(10)

NBC 2015 requires the determination of a torsional sensitivity parameter, B, which is used to determine allowable analysis methods. To determine B, a set of lateral forces, F_x, is applied at a distance of $\pm 0.1 D_{nx}$ from the centre of mass C_M, where D_{nx} is the plan dimension of the building perpendicular to the direction of the seismic loading being considered. The set of lateral loads, F_x, to be applied can either be the static lateral loads or those determined from a dynamic analysis. A parameter, B_x, evaluated at each level, x, should be determined from the following equation) (Figure 1-9):

$$B_x = \frac{\delta_{\max}}{\delta_{ave}}$$

where

$\delta_{\max}$ - the maximum storey displacement at level x at one of the extreme corners, in the direction of earthquake, and

δ_{ave} - the average storey displacement, determined by averaging the maximum and minimum displacements of the storey at level x.

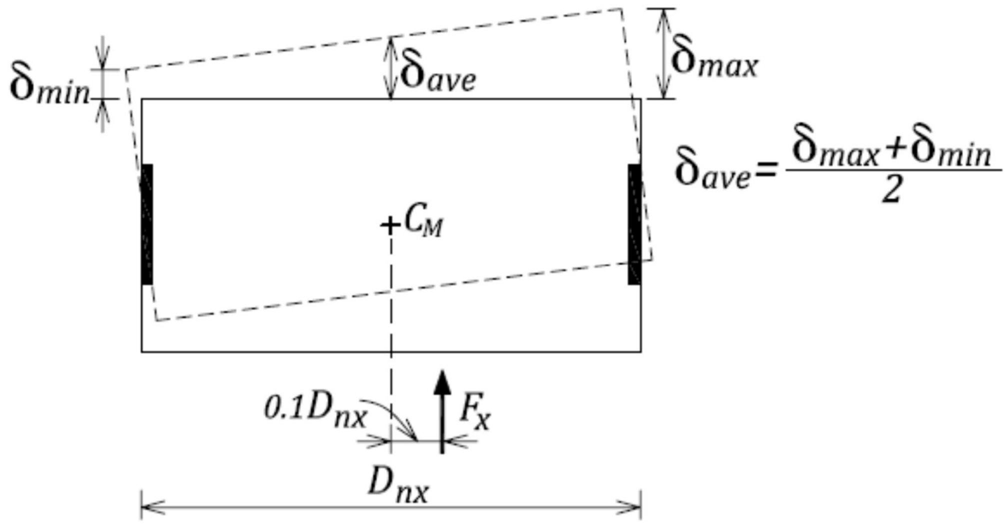

Figure 1-9. Torsional displacements used in the determination of B_x.

The torsional sensitivity, B, is the maximum value of B_x for all storeys for both orthogonal directions. Note that B_x need not be considered for one-storey penthouses with a weight less than 10% of the level below.

Commentary

A structure is considered to be torsionally sensitive when the torsional flexibility compared to the lateral flexibility is above a certain level, that is, when $B > 1.7$. Torsionally sensitive buildings are considered to be torsionally vulnerable, and NBC 2015 in some cases requires that the effect of natural eccentricity be evaluated using a dynamic analysis, while the effect of accidental eccentricity be evaluated statically.

Structures that are not torsionally sensitive, or located in a low seismic region where $I_E F_a S_a(0.2) < 0.35$, can have the effects of torsion evaluated using only the equivalent static analysis. If the structure is torsionally sensitive and located in a high seismic region, a dynamic analysis must be used to determine the effect of the natural eccentricity, but the accidental eccentricity effects must be evaluated statically, and the results then combined as discussed in the next section. A static torsional analysis of the accidental eccentricity, on a torsionally flexible building, will lead to large torsional displacements, and generally to large torsional forces in the elements, and thus may require a change in the structural layout so that the structure is not so torsionally sensitive.

1.11.3 Determination of torsional forces

4.1.8.11.(11)

Torsional effects should be accounted for as follows:

a) if $B \leq 1.7$ (or $B > 1.7$ and $I_E F_a S_a(0.2) < 0.35$), the equivalent static analysis procedure can be used, and the torsional moments, T_x, about a vertical axis at each level throughout the building, should be considered separately for each of the following load cases:

i) $T_x = F_x(e_x + 0.1D_{nx})$, and

ii) $T_x = F_x(e_x - 0.1D_{nx})$.

The analysis required to determine the element forces, for both the lateral load and the above torques, is identical to that required to determine the B factor, where the lateral forces are applied at a distance $\pm 0.1D_{nx}$ from the centre of mass, C_M, as shown by the dashed arrows in Figure 1-10.

b) if $B > 1.7$, and $I_E F_a S_a(0.2) \geq 0.35$, the dynamic analysis procedure must be used to determine the effects of the natural eccentricities, e_x. The results from the dynamic analysis must be combined with those from a static torsional analysis that considers only the accidental torques given by

$T_x = +F_x(0.1D_{nx})$, or

$T_x = -F_x(0.1D_{nx})$

In this analysis, F_x can come from either the equivalent static analysis or from a dynamic analysis.

c) If $B \leq 1.7$ it is permitted to use a 3-D dynamic analysis with the centres of mass shifted by a distance of $\pm 0.05D_{nx}$ (see Clause 4.1.8.12.(4)(b).

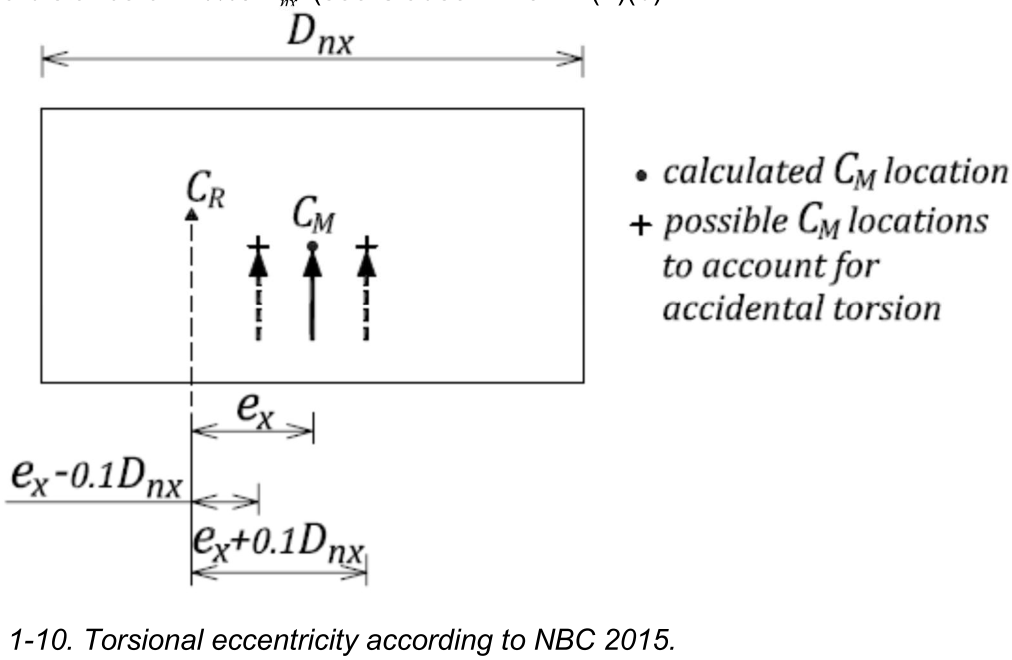

Figure 1-10. Torsional eccentricity according to NBC 2015.

Commentary

When results from a dynamic analysis are combined with accidental torques that use the lateral forces F_x from the equivalent static procedure, the designer should ensure that the analysis is done in a consistent manner, that is, by using either the elastic forces or the reduced design forces (elastic forces modified by $I_E / R_d R_o$). The final force results should be given in terms of the reduced design forces, while the displacements should correspond to the elastic displacements.

If the structure is torsionally sensitive, $B > 1.7$, and if $I_E F_a S_a(0.2) \geq 0.35$, then the member forces and displacements from the accidental eccentricity must be evaluated statically by applying a set of torques to each floor of $\pm F_x(0.1D_{nx})$. The set of lateral forces, F_x, can come from either a static or a dynamic analysis. NBC 2015 is mute on whether the set of lateral static forces should be scaled to match the dynamic base shear (if the dynamic base shear is larger than the static value), and whether the dynamic set should correspond to the set determined with the floor rotations restrained or not restrained (see Section 1.14). It is suggested here that if a set of static forces is used, they should (if necessary) be scaled up to match the base shear from the rotationally restrained dynamic analysis.

The static approach to determine member forces and displacements from the accidental eccentricity is illustrated in Figure 1-11.

If the static forces are to be used, then the following steps need to be followed:

1. The forces F_x are determined using the equivalent static method.
2. Torsional moments at each level are found using the following equations $T_x = +F_x(0.1D_{nx})$, or $T_x = -F_x(0.1D_{nx})$.
3. Displacements and forces due to torsional effects are determined, and combined with the results from the dynamic analysis. Note that, in buildings with larger periods, F_t will cause large rotations and displacements, and the results will probably be conservative.

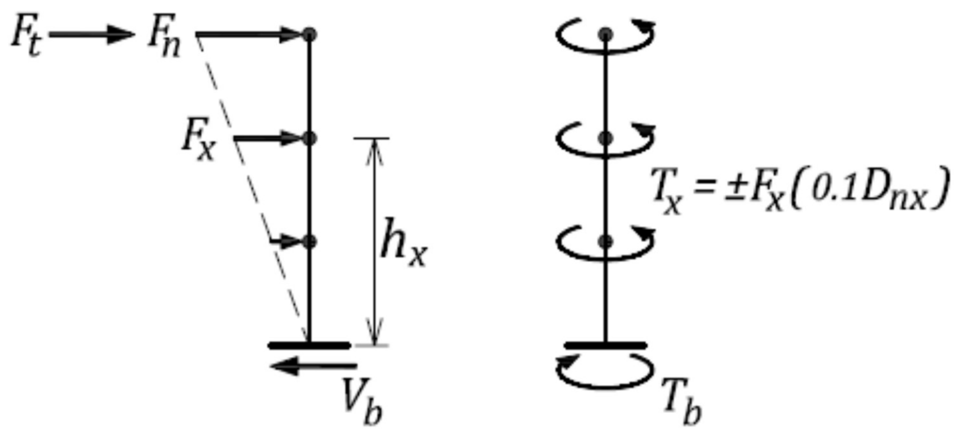

Figure 1-11. Static approach to determine the accidental eccentricity effects (Anderson, 2006).

If a dynamic set of floor forces, F_x, are to be used, they should be scaled, if necessary (as discussed in Section 1.14), to be equal to the design base shear. For the determination of the storey torques, the force F_x at each floor can be determined from the dynamic analysis by taking the difference in the total shear in the storeys above and below the floor in question. These floor forces are not necessarily the correct floor forces (as discussed in Section A.4.3), however the sum of these forces equals the design base shear and they provide a reasonable set of lateral forces to use for the accidental eccentricity calculations. The second and third steps discussed in the previous paragraph are then the same.

If the structure is not torsionally sensitive ($B \leq 1.7$), and a dynamic analysis is being used, it is permissible to account for both the lateral forces and the torsional eccentricity, including the natural and accidental eccentricity, by using a 3-D dynamic analysis and moving the centre of mass by the distance $\pm 0.05D_{nx}$. This would require four separate analyses, two in each direction. In these dynamic analyses the accidental eccentricity is taken as $\pm 0.05D_{nx}$, while in the static application it is taken as $\pm 0.10D_{nx}$. It is thought that the real accidental eccentricity is about $\pm 0.05D_{nx}$, but it would likely be amplified during an earthquake; this is reflected in the results of a dynamic analysis. Thus, $\pm 0.10D_{nx}$ is used in the static case to account for both accidental eccentricity and possible dynamic amplification.

When using a 3-D dynamic analysis for torsional response, it is important to correctly model the mass moment of inertia about a vertical axis. If the floor mass is entered as a point mass at the mass centroid, it will not have the correct mass moment of inertia and the torsional period will be too small. This will have the effect of making the structure appear to be torsionally stiffer than it really is, and could lead to smaller torsional deflections.

When applying the lateral loads in one direction, torsional response gives rise to forces in the elements in the orthogonal direction. For structures with lateral force resisting elements oriented along the principal orthogonal axes, NBC 2015 Cl. 4.1.8.8.(1)(a) requires independent analyses along each axis. For structures with elements oriented in non-orthogonal directions (as shown in Section 1.12.1 for Type 8 Irregularity), an independent analysis about any two orthogonal axes is sufficient in low seismic zones, but in higher zones, it is required that element forces from loading in both directions be combined. The suggested method for combining forces from both directions is the “100+30%” rule that requires the forces in any element that arise from 100% of the loads in one direction be combined with 30% of the loads in the orthogonal direction, see NBC 4.1.8.8.(1)c). Another method is to apply the ‘root-sum-square’ procedure to the forces in each element from 100% of the loads applied in both directions. The two methods usually give close agreement and are based on the knowledge that the probability of the maximum forces from the two directions occurring at the same time is low. For some orthogonal systems, it is possible that the orthogonal forces from the effects of torsion are substantial, and a prudent design may consider combined forces from both directions as described above, especially in high seismic regions.

Note that the NBC requirements are based on an estimate of the elastic properties of the structure. When the structure yields, the eccentricity between the inertia forces acting through the centres of mass and the resultant of the resisting forces based on the capacity of the members, termed the plastic eccentricity, will be different than the elastic eccentricity. In most cases, the plastic eccentricity will be less than the elastic eccentricity. However, there may be cases where some elements are stronger than necessary and do not yield; this could increase the eccentricity when other elements yield, and it should be avoided if possible.

1.11.4 Flexible diaphragms

Diaphragms are horizontal elements of the SFRS whose primary role is to transfer inertial forces throughout the building to the vertical elements (shear walls in case of masonry buildings) that resist these forces. A diaphragm can be treated in a manner analogous to a beam lying in a horizontal plane where the floor or roof deck functions as the web to resist the shear forces, and the boundary elements (bond beams in case of masonry buildings) serve as the flanges in resisting the bending moment. How the total shear force is distributed to the vertical elements of the SFRS will depend on their rigidity compared to the rigidity of the diaphragm. For design purposes, diaphragms are usually classified as rigid or flexible, but that very much depends on the type of structure. Structures with many walls and small individual diaphragms between the walls can be considered as having flexible diaphragms. In large plan structures, such as warehouses or industrial buildings with the SFRS members located around the perimeter, it is more common to assume the diaphragm as being rigid. However the flexibility of the diaphragm may lead to a considerable increase in the period of the structure, and lead to deformations considerably larger than the deformations of the SFRS, in which case a more complex analysis would be required.

In *rigid diaphragms*, shear forces are distributed to vertical elements in proportion to their stiffness. Torsional effects are considered following the approach outlined in Sections 1.11.1 to 1.11.3. Concrete diaphragms, or steel diaphragms with concrete infill, are usually considered rigid.

In *flexible diaphragms*, distribution of shear forces to vertical elements is independent of their relative rigidity; these diaphragms act like a series of simple beams spanning between vertical elements. A flexible diaphragm must have adequate strength to transfer the shear forces to the SFRS members, but cannot distribute torsional forces to the SFRS members acting at right angles to the direction of earthquake motion without undergoing unacceptable displacements. Corrugated steel diaphragms without concrete fill, and wood diaphragms, are generally considered flexible; however, steel and wood diaphragms with horizontal bracing could be considered rigid.

Figure 1-12a) shows the plan view of a simple one storey structure with walls on three sides and non-structural glazing on the fourth side. For an earthquake producing an inertia force, V , the walls provide resisting forces to the diaphragm as shown. The displacement of the diaphragm would be as shown in Figure 1-12b), and it is the size of the displacements that determines whether the diaphragm is considered flexible or rigid. If the displacements are too large to be acceptable, the diaphragm would be classed as flexible, and cannot be used with such a layout of the SFRS. In general, flexible diaphragms require that the SFRS has at least two walls in each direction.

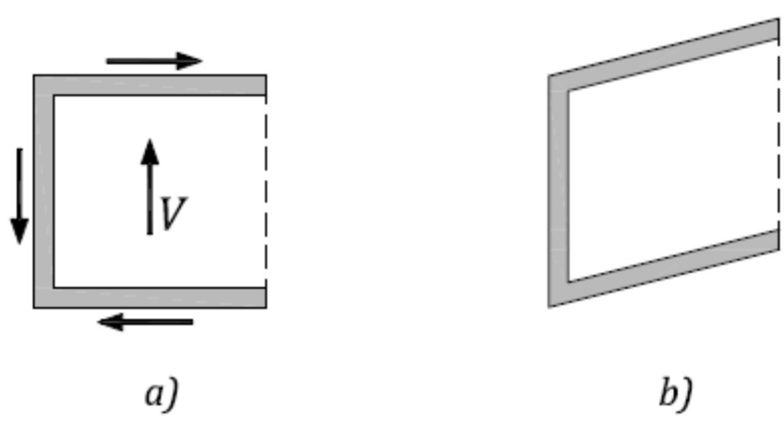

Figure 1-12. Building plan: a) loads on diaphragm; b) displaced shape of a flexible diaphragm.

In determining how the inertia forces are distributed to the SFRS, the flexible diaphragm should be divided into sections, with each section bounded by two walls in the direction of the inertia forces; preferably these two walls will be located on the sides of the section. The inertia forces from each section are then distributed to the SFRS on the basis of tributary areas. Equilibrium must be satisfied, and the diaphragm must have sufficient strength in shear and bending to act as a horizontal beam carrying the loads to the supports. Figure 1-13 shows a flexible roof system supported by three walls in the N-S direction. The roof should be divided into two sections as shown, with the inertia force from section 1 distributed to walls A and B. Section 2 must be considered as a beam with a cantilever end extending beyond wall C. Equilibrium of section 2 then gives rise to a high force in Wall C, with the overhanging portion contributing to a reduction in the force in wall B.

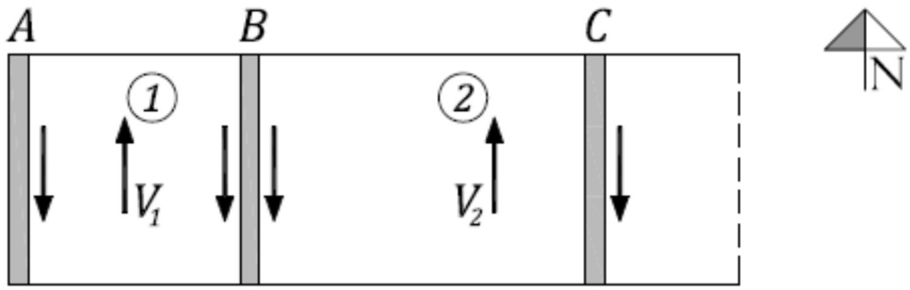

Figure 1-13. Plan view for analysis of flexible diaphragm.

NBC 2015 requires that accidental eccentricity be considered. With rigid diaphragms it is clear how this can be accomplished, as described in the above sections, but trying to account for accidental eccentricity in flexible diaphragms raises several questions about how it is to be applied. NBC 2005 Commentary J, paragraph 179 (NRC, 2006) states that it is sufficient to consider an eccentricity of $\pm 0.05D_{nx}$, where D_{nx} is defined as the width of the building in the direction perpendicular to the direction of the earthquake motion. If the structure consists of a single roof section with supporting walls at each end separated by the distance D_{nx}, moving the centre of mass by $0.05D_{nx}$ would increase the wall reactions by 10%, and the accidental eccentricity requirement would be satisfied. For a structure with several walls in the direction of the earthquake motion, shifting the centre of mass by $\pm 0.05D_{nx}$, which would require moving the centre of mass of each section by this amount, could lead to unrealistic situations, as well as requiring a considerable increase in computational effort. Even flexible diaphragms will have some stiffness, and in many cases will transfer some of the torsional load to the walls perpendicular to the direction of motion. This transfer is ignored when designing for flexible diaphragms, but does provide extra torsional resistance. It is suggested that the wall forces determined without any accidental eccentricity all be increased by 10% to account for the accidental eccentricity. This minimizes the number of calculations required, and it is suggested that it satisfies the intent of NBC 2015.

1.12 Configuration Issues: Irregularities and Restrictions

1.12.1 Irregularities

4.1.8.6

Table 1-16 (same as NBC 2015 Table 4.1.8.6) lists the nine types of irregularity, and the notes to the table refer to the relevant code clauses that consider the irregularity. If a structure has none of the listed irregularities it is considered to be a *regular structure*. A trigger for the NBC 2015 irregularity provisions (Cl.4.1.8.6) is the presence of one of nine types of irregularity in combination with the higher seismic hazard index, that is, $I_E F_a S_a(0.2) > 0.35$.

In NBC 2015 there is a new structural irregularity, Type 9, on ‘gravity-induced lateral demand’ which covers cases where gravity loads could cause the building to yield in one direction only and creates larger displacements than a regular building would undergo. Irregularities are used to trigger restrictions and special requirements, some of which are more restrictive than those in previous codes. See NBC Section 4.1.8.10 for additional restrictive clauses covering structural irregularities.

Table 1-16. Structural Irregularities[1] Forming Part of Sentence 4.1.8.6.(1) (NBC Table 4.1.8.6.)

Type	Irregularity Type and Definition	Notes
1 ***Vertical stiffness irregularity***	Vertical stiffness irregularity shall be considered to exist when the lateral stiffness of the SFRS in a *storey* is less than 70% of the stiffness of any adjacent *storey*, or less than 80% of the average stiffness of the three *storeys* above or below.	(2) (3) (4)
2 ***Weight (mass) irregularity***	Weight irregularity shall be considered to exist where the weight, W_i, of any *storey* is more than 150% of the weight of an adjacent *storey*. A roof that is lighter than the floor below need not be considered.	(2)
3 ***Vertical geometric irregularity***	Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the SFRS in any *storey* is more than 130 percent of that in an adjacent *storey*.	(2) (3) (4) (5)
4 ***In-plane discontinuity in vertical lateral force-resisting element***	An in-plane offset of a lateral-force-resisting element of the SFRS or a reduction in lateral stiffness of the resisting element in the *storey* below.	(2) (3) (4) (5)
5 ***Out-of-plane offsets***	Discontinuities in a lateral force path, such as out-of-plane offsets of the vertical elements of the SFRS.	(2) (3) (4) (5)
6 ***Discontinuity in capacity - weak storey***	A weak storey is one in which the storey shear strength is less than that in the storey above. The *storey* shear strength is the total strength of all seismic-resisting elements of the SFRS sharing the *storey* shear for the direction under consideration.	(2) (3)
7 ***Torsional sensitivity***	Torsional sensitivity shall be considered when diaphragms are not flexible, and when the ratio B>1.7 (see Sentence 4.1.8.11(10)).	(2) (3) (4) (6)
8 ***Non-orthogonal systems***	A non-orthogonal system irregularity shall be considered to exist when the SFRS is not oriented along a set of orthogonal axes.	(2) (4) (7)
9 ***Gravity-Induced Lateral Irregularity***	Gravity-induced lateral demand irregularity on the SFRS shall be considered to exist where the ratio, α, calculated in accordance with Sentence 4.1.8.10.(5), exceeds 0.1 for an SFRS with self-centering characteristics and 0.03 for other systems.	(2) (3) (4) (7)

Notes: (1) One-*storey* penthouses with a weight less than 10% of the level below need not be considered in the application of this table.
(2) See Article 4.1.8.7.
(3) See Article 4.1.8.10.
(4) See Note A-Table 4.1.8.6.
(5) See Article 4.1.8.15.
(6) See Sentences 4.1.8.11.(10), (11), and 4.1.8.12.(4)
(7) See Article 4.1.8.8.

Commentary

The equivalent static analysis procedure is based on a regular distribution of stiffness and mass in a structure. It becomes less accurate as the structure varies from this assumption. Historically, regular buildings have performed better in earthquakes than have irregular buildings. Layouts prone to damage are: torsionally eccentric ones, “in” and “out” of plane offsets of the lateral system, and buildings with a weak storey (Tremblay and DeVall, 2006). For more details on building configuration issues refer to Chapter 6 of Naeim (2001).

Figure 1-14 illustrates the NBC 2015 irregularity types. Note that Types 1 to 6 are vertical (elevation) irregularities, while Types 7, 8 and 9 are horizontal (plan) irregularities.

According to NBC 2015 Clause 4.1.8.7, the structure is considered to be “regular” if it has none of the nine types of irregularity, otherwise it is deemed to be “irregular”. The default method of analysis is the dynamic method, but the equivalent static method may be used if <u>any</u> of the following conditions are satisfied:

a) the seismic hazard index $I_E F_a S_a(0.2) < 0.35$, or
b) the structure is regular, less than 60 m in height, and has a period T < 0.5 seconds in either direction, or
c) the structure is irregular, but does not have Type 7 or 9 irregularity, and is less than 20 m in height with period T < 0.5 seconds in either direction.

For single-storey structures such as warehouses and other low-rise masonry buildings, only irregularity Types 7 and 8 might apply, and these would not likely prevent the use of the equivalent static method.

Type 8 irregularity concerns SFRS(s) which are not oriented along a set of orthogonal axes. The structures with this type of irregularity may require more complex seismic analysis in which seismic loads in two orthogonal directions would need to be considered concurrently. According to Clause 4.1.8.8.(1)(b), where the components of the SFRS are not oriented along a set of orthogonal axes, and the structure is in a low seismic zone ($I_E F_a S_a(0.2) < 0.35$), then independent analysis about any two orthogonal axes is permitted. However, where the components of the SFRS are not oriented along a set of orthogonal axes, and the structure is in a medium or high seismic zone ($I_E F_a S_a(0.2) \geq 0.35$), then the analysis of the structure can be done independently about any two orthogonal axes for 100% of the prescribed earthquake loads in one direction concurrently with 30% of the prescribed earthquake loads acting in the perpendicular direction (see Clause 4.1.8.8.(1)(c). This is so-called “100+30%” rule discussed in Section 1.11.3.

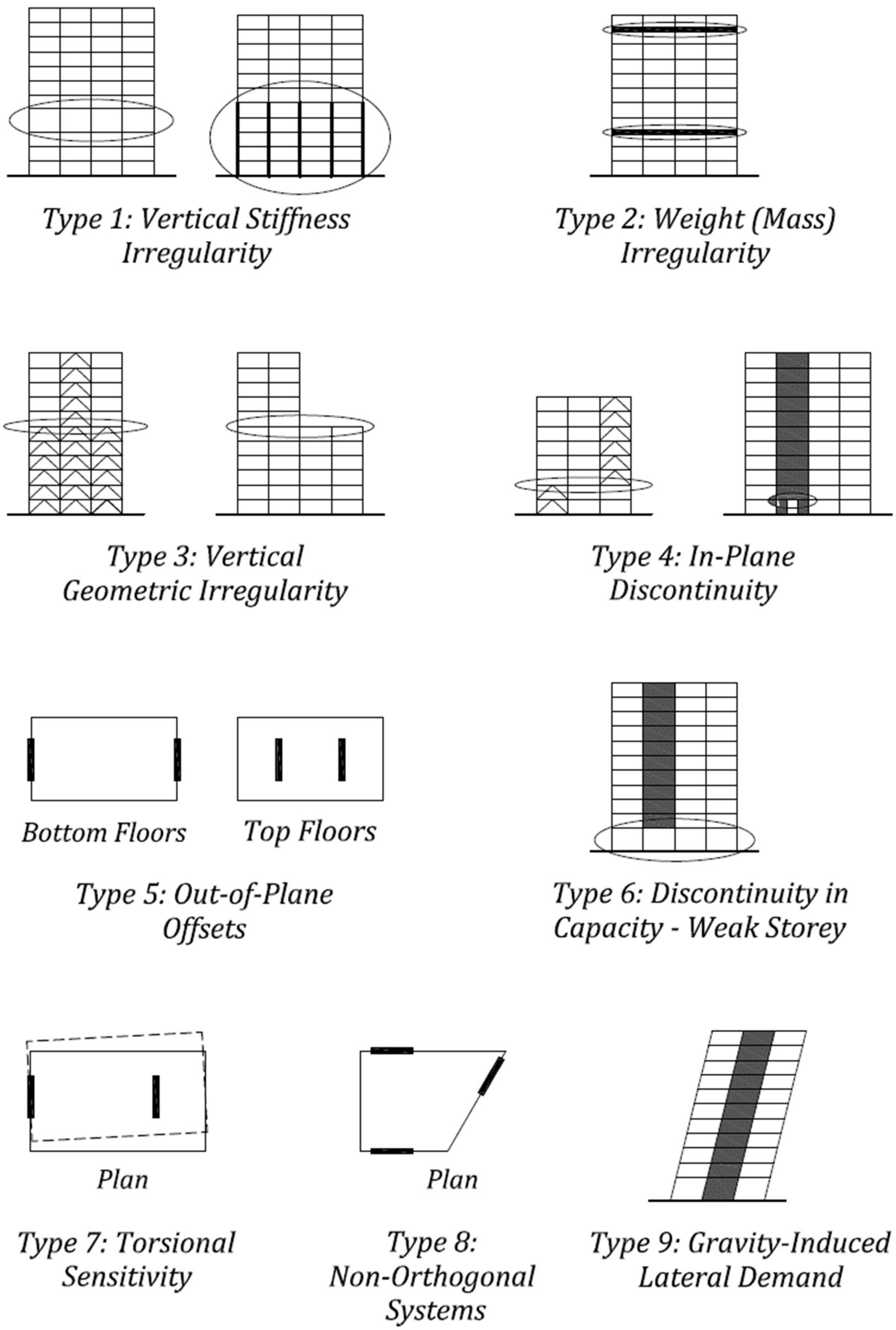

Figure 1-14. Types of irregularity according to NBC 2015 (based on Tremblay and DeVall, 2006).

1.12.2 Restrictions

4.1.8.10.

Restrictions in NBC 2015 are based on (i) the natural period or height of the building, (ii) whether the building is in a "high" or "low" seismic zone, (iii) irregularities, and (iv) the importance category of the building. These restrictions are outlined below:

1. Except as required by Clause 4.1.8.10.(2)(b), structures with Type 6 irregularity, Discontinuity in Capacity – Weak Storey, are not permitted unless $I_E F_a S_a(0.2) < 0.20$ and the forces used for design of the SFRS are multiplied by $R_d R_o$.
2. Post-disaster buildings shall
   a) not have any irregularities conforming to Types 1, 3, 4, 5, 7 and 9 as described in Table 4.1.8.6, in cases where $I_E F_a S_a(0.2) \geq 0.35$,
   b) not have a Type 6 irregularity as described in Table 4.1.8.6, and
   c) have an SFRS with an $R_d \geq 2.0$.
   d) have no storey with a lateral stiffness that is less than that of the storey above it.
3. For buildings having fundamental lateral periods $T_a \geq 1.0s$, and where $I_E F_v S_a(1.0) > 0.25$, shear walls that are other than wood-based forming part of the SFRS shall be continuous from their top to the foundation and shall not have irregularities of Type 4 or 5 as described in Table 4.1.8.6.
4. Wood construction, see 4.1.8.9 and Note A-4.1.8.10.(4).

5., 6., and 7. Only apply to Irregularity Type 9.

Refer to Section 1.12.1 and Table 1-16 for the list of irregularities identified by NBC 2015.

Commentary

An important restriction for masonry construction concerns post-disaster structures. In other than low seismic regions the structure cannot have irregularity Types 1, 3, 4, 5, or 7; and must have an $R_d \geq 2.0$. Thus masonry post-disaster structures must be designed with Moderately Ductile or Ductile shear walls, and except in low seismic regions (where $I_E F_a S_a(0.2) < 0.35$) the above noted irregularity types should be avoided.

Irregularity Type 6, Discontinuity in Capacity-Weak Storey, is an important restriction for multi-storey structures, and *cannot be present at all in post-disaster structures*. For structures with this type of irregularity, the forces used in the design of the SFRS, except in very low seismic areas, must be multiplied by $R_d R_o$, which implies that the members must remain elastic. This type of irregularity is considered very dangerous, as in past earthquakes many structures with weak storeys have had a total collapse of that storey which has resulted in many deaths. This type of seismic response has often been reported in reinforced concrete frame structures with masonry infill walls which contain more infills in the storeys above the ground floor, leaving the first storey as a weak storey.

1.13 Deflections and Drift Limits

4.1.8.13

Lateral displacement (deflection) limits are prescribed in terms of maximum drift. *Drift* means the lateral deflection of one floor (or roof) relative to the floor below. *Drift ratio* is the drift divided by the storey height between the two floors, and is thus a measure of the distortion of the structure.

The NBC 2015 drift limits are based on the storey height h_s, as follows:

- $0.01 h_s$ for post-disaster buildings
- $0.02 h_s$ for High Importance Category buildings (e.g. schools), and
- $0.025 h_s$ for all other buildings.

Commentary

Since large deflections and drifts due to earthquakes contribute to (i) damage to the non-structural components, (ii) damage to the elements which are not a part of the SFRS, and (iii) P-Delta effects, NBC 2015 provisions have moved in the direction of tightening up the drift limits from the previous versions. Specifically, tighter drift limits for post-disaster or school buildings reflect the importance of these structures.

Drift and drift ratio can be explained on an example of a three-storey building shown in Figure 1-15. The drift in say the second storey is equal to $\Delta_2 - \Delta_1$, where Δ_1 and Δ_2 denote lateral deflections at the first and second floor level respectively. The corresponding drift ratio for that storey is equal to $(\Delta_2 - \Delta_1)/h$, where $h = h_2 - h_1$ (storey height). The average drift ratio for the entire structure is $(\Delta_3)/h$.

Drifts are the elastic deflections and need not be increased by the importance factor I_E as that has already been accounted for in the drift limits. If the equivalent static forces, which are the elastic forces multiplied by $I_E / R_d R_o$, are applied to the elastic structure to calculate deflections, then these deflections must be multiplied by $R_d R_o / I_E$ to get realistic values of the deflections.

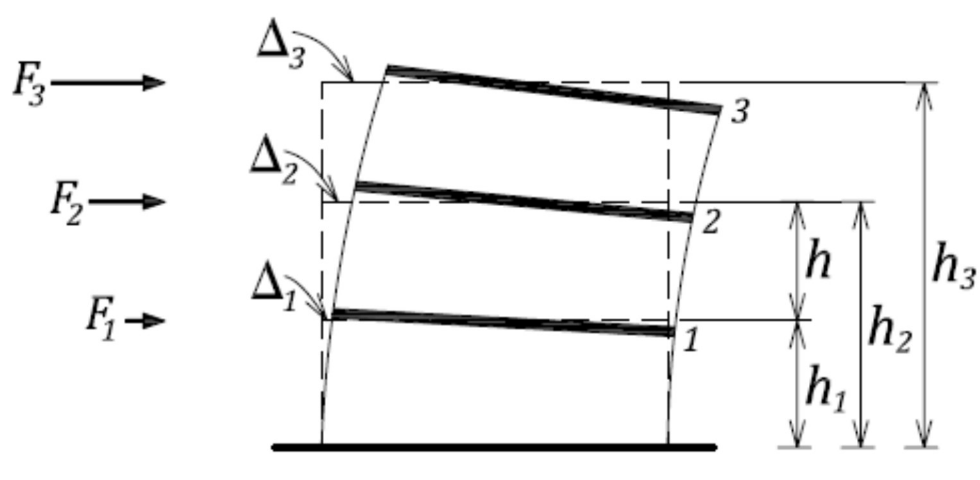

Figure 1-15. Lateral deflections and drift.

In checking drift limits the drift should be taken at the location on the floor which has the maximum deflection. Torsional effects can result in corner deflections being much larger than the deflection at the centre of the floor plan.

Since deflections increase with an increase in the period T, the stiffness used in calculating the deflections should reflect a softening of the structure (before yielding occurs) that might come from cracking of the masonry. The stiffness for squat shear walls should be determined taking into account shear deformation. If the period T determined per NBC provisions (see Section 1.6) is used to determine the seismic forces, the stiffness of the structure used in calculating the deflections should be such that the calculated period would not be less than the NBC period. Many masonry structures are very stiff and the deflections will be well below the code limits, and so displacement calculations will not be critical in many cases.

Drift limits are imposed so that members of the SFRS will not be subjected to large lateral displacements that might degrade their ability to resist the seismic loads, but also to ensure that members that are not part of the SFRS, such as columns that support gravity load only, should not fail during the earthquake. The seismic portion of the code is mute on drift limits for serviceability, however the designer can estimate the structural deflections at different hazard levels, since displacements are roughly proportional to the level of hazard. For example, the drift at an exceedance probability of 1/475 per annum would be about half of that for the 1/2475 per annum design drift because the 1/475 per annum hazard is roughly half the 1/2475 per annum hazard.

1.14 Dynamic Analysis Method

4.1.8.12

In NBC 2015 the default analysis method is the dynamic method. For many structures, even though the equivalent static analysis method could be used according to NBC seismic provisions, dynamic analysis may be used for other reasons. The purpose of this section is not to explain how to use dynamic analysis software, but to give some guidance on scaling or comparing the dynamic results with the results from the static method.

The base shear from a dynamic analysis, determined using the site design spectrum *S(T)*, will give the dynamic elastic base shear, V_e. Since the static analysis method is allowed to reduce the design base shear for short periods, see 4.1.8.11(2)(d), while the dynamic analysis method uses the design spectrum *S(T)*, it is permitted to reduce the dynamic analysis results by the factors $2S(0.2)/3S(T_a)$ or $S(0.5)/S(T_a)$ whichever is larger but ≤1.0, to give V_{ed} for Site Classes A to D (NBC 2015 Sec 4.1.8.12(6)).

NBC 2015 requires that for regular buildings if the base shear from the dynamic method is less than 0.8 times the base shear from the static method, then the dynamic results should be scaled to give 0.8 of the static base shear. If the structure is deemed to be irregular, then the dynamic results should be scaled to 100% of the static results. In essence this means that the dynamic results cannot be less than the static results (or 80% of the static results for regular structures), but if they are larger they should not be reduced to the static values.

If the building is very eccentric, a 3-D dynamic analysis will produce a low total base shear. In that case, it would be very conservative to require that these low base shears be scaled to the static base shear, since the static method of determining the base shear *V* does not consider torsional motion. To make a fair comparison between the static and dynamic results the suggestion is to perform a dynamic analysis with the rotation of the structure restrained about a vertical axis, and then compare the resulting base shear to the static value to determine the amount of scaling required, if any.

Scaling, if necessary, should be applied to the member forces determined from the full 3-D dynamic analysis multiplied by $I_E / R_d R_o$ to give the design member forces. The design displacements are the elastic displacements given by the dynamic analysis, and scaled if necessary. To these design forces and displacements must be added the forces and displacements from accidental torsion.

1.15 Soil-Structure Interaction

For large structures located on soft soil sites the deformation of the soil may have an appreciable influence on the response of the structure. The most common type of soil-structure interaction is based on the flexibility of the soil, which is usually represented by a lateral spring between the foundation and the point where the seismic motion is input, and with a rotational spring at the base of flexural walls. There is a second type of soil-structure interaction, termed the kinematic interaction, which only applies to structures with a very large plan area or a deep foundation, and which will not be discussed further here.

The effect of introducing springs between the point of input motion and the foundation is to increase the period of the structure, which usually reduces the seismic forces but increases the deflections. In the case of a wall structure, the increased deflections may not increase the deformation of the wall since they would arise from displacements and rotations of the foundation, but the rotations would increase the interstorey drifts which would have an influence on other parts of the structure.

For masonry structures, soil-structure interaction will likely only have an influence for slender wall structures with individual footings, where rotation of the footing would have a large effect on the wall displacement. The determination of the soil stiffness should be left to an experienced geotechnical engineer, but it should be recognized that the precision at which the soil stiffness can be estimated is quite low. It is common to consider quite wide upper and lower bounds on the estimated stiffness of the soil springs.

1.16 A Comparison of NBC 2005 and NBC 2015 Seismic Design Provisions

A comparison is presented in Table 1-17 as a reference for the readers who have previously used NBC 2005.

Table 1-17. Comparison of NBC 2005 and NBC 2015 Seismic Design Provisions: Equivalent Static Force Procedure

Provision	NBC 2005	NBC 2015
Analysis method	***Cl.4.1.8.7*** Dynamic method is the default method; static method is restricted to certain structures and seismic hazard.	***Cl.4.1.8.7*** No changes
Seismic force	***Cl.4.1.8.11*** $V = S(T)M_vI_eW / (R_dR_o)$	***Cl.4.1.8.11*** $V = S(T_a)M_vI_eW / (R_dR_o)$
Base response spectrum	***Cl.4.1.8.4*** $S(T)=F_aS_a(T)$ or $F_vS_a(T)$ $S_a(T)$ based on UHS	***Cl.4.1.8.4(9)*** $S(T)=FS_a(T)$ $S_a(T)$ based on UHS for T=0.2 sec, 0.5 sec, 1.0 sec, 2 sec, 5 sec, and 10 sec
Site conditions	***Cl.4.1.8.4*** F_a or F_v Depends on **T** and S_a	***Cl.4.1.8.4(9)*** **F(0.2), F(0.5), F(1.0), F(2.0)** Depends on site class and PGA_{ref}
Importance of structure	***Cl.4.1.8.5*** I_E	***Cl.4.1.8.5*** No changes
Inelastic response	***Cl.4.1.8.9*** R_dR_o Explicit overstrength	***Cl.4.1.8.9*** No changes
MDOF Forces from higher modes	***Cl.4.1.8.11*** M_v multiplier on base shear Depends on period, type of structure and shape of $S_a(T)$	***Cl.4.1.8.11(6)*** No changes
MDOF Distribution of forces	***Cl.4.1.8.11(6)*** F_t Same as NBC 1995	***Cl.4.1.8.11(7)*** No changes
MDOF Overturning forces	***Cl.4.1.8.9(7)*** **J** Revised for consistency with M_v	***Cl.4.1.8.9(6)*** No changes
Eccentricity	***Cl.4.1.8.11(8), (9), and (10)*** $T_x=F_x(e_x \pm 0.1D_{nx})$ Must determine torsional sensitivity	***Cl.4.1.8.11(9), (10), and (11)*** No changes
Irregularities	***Cl.4.1.8.6***	***Cl.4.1.8.6*** There is a new irregularity (Type 9)

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