SEISMIC DESIGN GUIDE FOR MASONRY BUILDINGS

CHAPTER 1

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1 Seismic Design Provisions of the National Building Code of Canada 2005

1.1 Introduction

This chapter provides a review of the seismic design provisions in the 2005 National Building Code of Canada (NBCC 2005). Additionally, there is an introduction to the dynamic analysis of structures to assist in understanding the NBCC provisions. Since there are major changes to the seismic provisions reflected in NBCC 2005, some comparisons will be made to the previous edition of the building code, NBCC 1995, and this is covered in more detail in Appendix A.

In the past, building structures in many areas of Canada did not have to be designed for earthquakes. However, after the NBCC 2005 was issued and adopted by the Provinces, structures in some additional areas must now be designed for earthquakes, especially if the structure is an important or post-disaster building, or if it is located on a soft soil site. Since many engineers in these regions have not had experience in seismic design and now may have to include such design in their practice, this guideline has been prepared to explain the seismic provisions included in the NBCC 2005 and CSA S304.1-04, and to point out the recent changes in these two documents as they pertain to masonry design.

1.2 Background

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.

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 analyzing the response of larger structures. The advent 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 NBCC seismic hazard model is based.

1.3 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 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 NBCC editions follow this philosophy and allow for lateral design forces smaller than the elastic forces, but impose detailing requirements so that the inelastic response remains ductile and a brittle failure is prevented.

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 it is allowed to yield and experience inelastic (plastic) deformations. This is known as the equal displacement rule and will be discussed later in this chapter, as it forms the basis for many of the code provisions.

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, but probably the most important is 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 immediate vicinity 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 NBCC 2005. The latest 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 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 factors in the NBCC 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 seismic *risk* of failure of buildings designed according to the code has not changed greatly.

A comparison of building designs performed according to the NBCC 1995 and the NBCC 2005 will show an increase in design level forces in some areas of Canada and a decreased level in other areas, however it is expected that the overall difference between these designs is not significant (see Appendix A for more details).

The NBCC 2005 has taken a more rational approach towards seismic design than have previous editions, in that the seismic hazard has been assessed for a certain probability related to risk of severe building damage, with the building designed with no empirical or calibrating factors. The real strength of the building has been utilized in the design, so that at this level of ground motion it should not collapse but could be severely damaged. Thus, the probability of severe damage or near collapse is about 1/2475 per annum, or about 2% in the predicted 50-year life span of the structure. When compared to wind or snow loads, which are based on the 1 in 50 year probability of not being exceeded, the 1 in 2475 year probability for seismic design appears inconsistent. However, unlike design for seismic loads, design for wind and snow loads uses load and material performance factors, and so the resulting probability of failure is expected to be smaller than that for earthquakes. Seismic design does use material resistance factors, ϕ factors, in assessing member capacity, but they are effectively cancelled out by the overstrength factor, R_o (which will be described later), used to reduce the seismic forces.

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 owners who have concerns that building damage will have an adverse effect on their ability to maintain their business. NBCC 2005 only addresses one performance level, that of collapse prevention and life safety, and is essentially mute on serviceability during smaller seismic events that are expected to occur more frequently. Performance based design attempts to minimize the cost of earthquake losses by weighing the cost of repair, and cost of lost business, against an increased cost of construction.

1.4 Response of Structures to Earthquakes

1.4.1 Elastic Response

When an earthquake strikes, the base of a building is subject to lateral motion while the upper part of the structure initially is at rest. The forces created in the structure from the relative displacement between the base and upper portion cause the upper portion to accelerate and displace. At each floor the lateral force required to accelerate the floor mass is provided by the forces in the vertical members. The floor forces are inertial forces, not externally applied forces such as wind loads, and exist only as long as there is movement in the structure.

Earthquakes cause the ground to shake for a relatively short time, 15 to 30 seconds of strong ground shaking, although movements may go on for a few minutes. The motion is cyclic and the response of the structure can only be determined by considering the dynamics of the problem. A few important dynamic concepts are discussed below.

Consider a simple single-storey building with masonry walls and a flat roof. The building can be represented by a Single Degree of Freedom (SDOF) model (also known as a stick model) as shown in Figure 1-1a. The mass, M, lumped at the top, represents the mass of the roof and a fraction of the total wall mass, while the column represents the combined wall stiffness, K, in the direction of earthquake ground motion. If an earthquake causes a lateral deflection, Δ, at the top of the building, Figure 1-1b, and if the building response is elastic with stiffness, K, then the lateral inertial force, F, acting on the mass M will be

$$F = K \cdot \Delta$$

When the mass of a SDOF un-damped structure is allowed to oscillate freely, the time for a structure to complete one full cycle of oscillation is called the period, T, which for the SDOF system shown is given by

$$T = 2\pi\sqrt{\frac{M}{K}} \quad \text{(seconds)}$$

Instead of period, the term *natural frequency*, ω, is often used in seismic design. It is related to the period as follows

$$\omega = \frac{2\pi}{T} = \sqrt{\frac{K}{M}} \quad \text{(radians/sec)}$$

Frequency is sometimes also expressed in Hertz, or cycles per second, instead of radians/sec, denoted by the symbol ω_{cps} , where

$$\omega_{cps} = \frac{1}{T} = \frac{\omega}{2\pi}$$

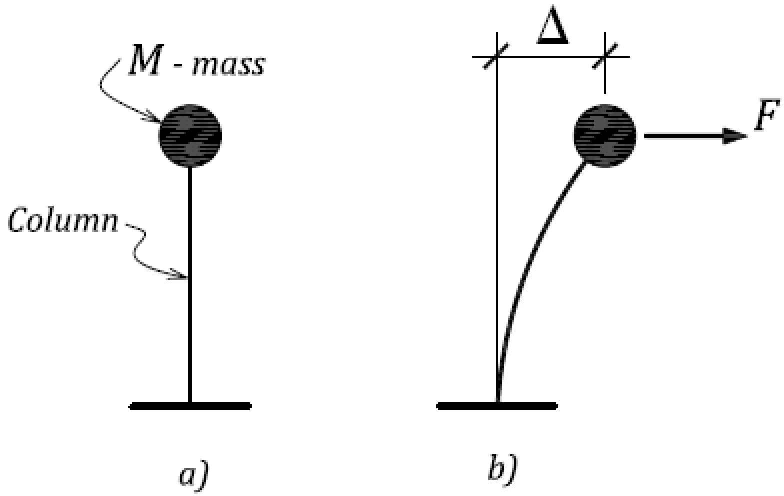


Figure 1-1. SDOF system: a) stick model; b) displaced position.

As the structure vibrates, there is always some energy loss which will cause a decrease in the amplitude of the motion over time - this phenomenon is called *damping*. The extent of damping in a building depends on the materials of construction, its structural system and detailing, and the presence of architectural components such as partitions, ceilings and exterior walls. Damping is usually modelled as viscous damping in elastic structures, and hysteretic damping in structures that demonstrate inelastic response. In seismic design of buildings, damping is usually expressed in terms of a *damping ratio,* β , which is described in terms of a percentage of critical viscous damping. Critical viscous damping is defined as the level of damping which brings a displaced system to rest in a minimum time without oscillation. Damping less than critical, an under-damped system, allows the system to oscillate; while an over-damped system will not oscillate but take longer than the critically damped system to come to rest. Damping has an influence on the period of vibration, *T*, however this influence is minimal for lightly damped systems, and in most cases is ignored for structural systems. For building applications, the damping ratio can be as low as 2%, although 5% is used in most seismic calculations. Damping in a structure increases with displacement amplitude since with increasing displacement more elements may crack or become slightly nonlinear. For linear seismic analysis viscous damping is usually taken as 5% of critical as the structural response to earthquakes is usually close to or greater than the yield displacement. A smaller value of viscous damping is usually used in non-linear analyses as hysteretic damping is also considered.

One of the most useful seismic design concepts is that of the *response spectrum*. When a structure, say the SDOF model shown in Figure 1-1, is subjected to an earthquake ground motion, it cycles back and forth. At some point in time the displacement relative to the ground and the absolute acceleration of the mass reach a maximum, $\Delta_{\max}$ and $a_{\max}$, respectively. Figure 1-2a shows the maximum displacement plotted against the period, T . Denote the period of this structure as T_1. If the dynamic properties, i.e. either the mass or stiffness change, the period will change, say to T_2. As a result, the maximum displacement will change when the structure is subjected to the same earthquake ground motion, as indicated in Figure 1-2b. Repeating the above process for many different period values and then connecting the points produces a plot like that shown in Figure 1-2c, which is termed the *displacement response spectrum.* The spectrum so determined corresponds to a specific input earthquake motion and a specific damping ratio, β. The same type of plot could be constructed for the maximum acceleration, $a_{\max}$, rather than the displacement, and would be termed the *acceleration response spectrum.*

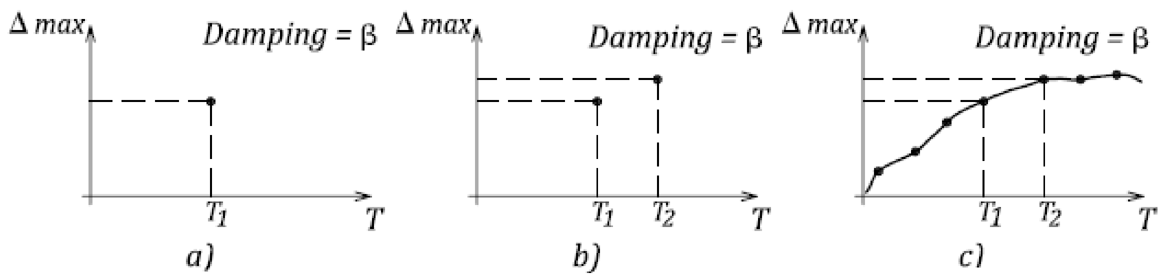


Figure 1-2. Development of a displacement response spectrum - maximum displacement response for different periods T : a) $T = T_1$; b) $T = T_2$; c) many values of T.

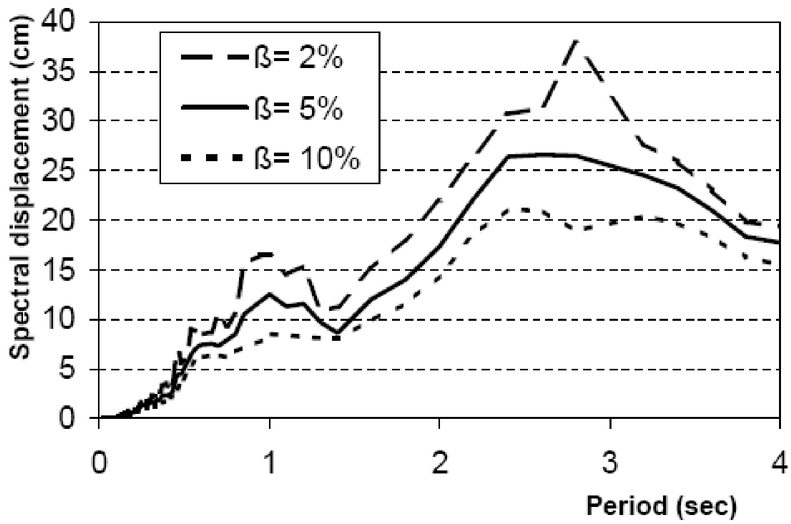
Figure 1-3a shows the displacement response spectrum for the 1940 El Centro earthquake at different damping levels. Note that the displacements decrease with an increase in the damping ratio, β, from 2% to 10%. Figure 1-3b shows the acceleration response spectrum for the same earthquake. For the small amount of damping present in the structures, the maximum acceleration, $a_{\max}$, occurs at about the same time as the maximum displacement, $\Delta_{\max}$, and these two parameters can be related as follows

$$a_{\max} = \left(\frac{2\pi}{T}\right)^2 \Delta_{\max}$$

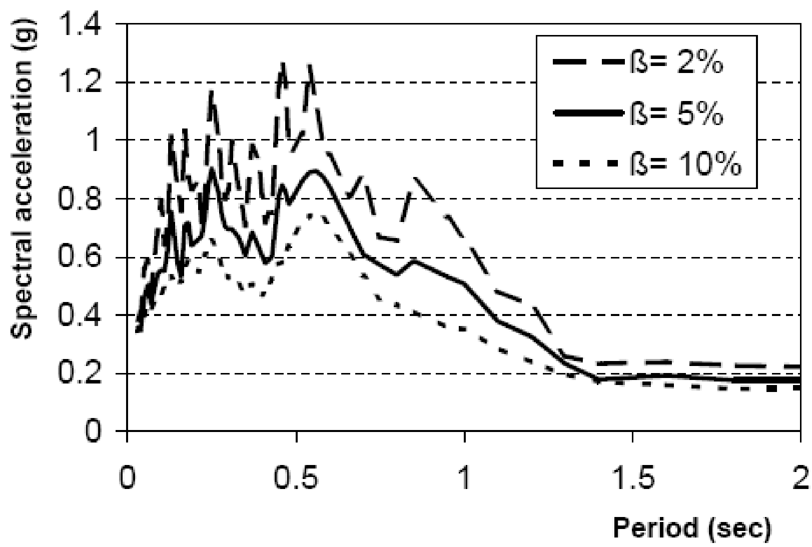
Thus, by knowing the spectral acceleration, it is possible to calculate the displacement spectral values and vice versa. It is also possible to generate a response spectrum for maximum velocity. Except for very short and very long periods, the velocity, $v_{\max}$, is closely approximated by

$$v_{\max} = \left(\frac{2\pi}{T}\right) \Delta_{\max}$$

This is generally called the pseudo velocity response spectrum as it is not the true velocity response spectrum.



a)



b)

Figure 1-3. Response spectra for the 1940 El Centro NS earthquake at different damping levels: a) displacement response spectrum; b) acceleration response spectrum.

The response spectrum can be used to determine the maximum response of a SDOF structure, given its fundamental period and damping, to a specific earthquake acceleration record. Different earthquakes produce widely different spectra and so it has been the practice to choose several earthquakes (usually scaled) and use the resulting average response spectrum as the *design spectrum*. For years, the NBCC seismic provisions have used this procedure where the design spectrum for a site was described by one or two parameters, either peak ground acceleration and/or peak ground velocity, that were determined using probabilistic means.

More recently, probabilistic methods have been used to determine the spectral values at a site for different structural periods. Figure 1-4 shows the 5% damped acceleration response spectrum for Vancouver used in developing the NBCC 2005. This is a uniform hazard response spectrum, i.e., spectral accelerations corresponding to different periods are based on the same probability of being exceeded, that is, 2% in 50 years. This will be discussed further in Section 1.5.1.

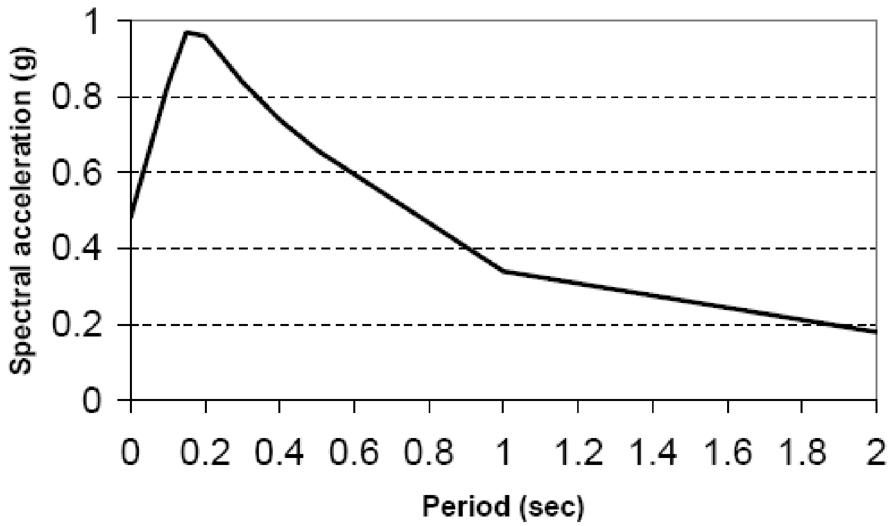


Figure 1-4. Uniform hazard acceleration response spectrum for Vancouver, 2% in 50 year probability, 5% damping.

1.4.2 Inelastic Response

For any given earthquake ground motion and SDOF elastic system it is possible to determine the maximum acceleration and the related inertia force, F_{el}, and the maximum displacement, Δ_{el}. Figure 1-5a shows a force-displacement relationship with the maximum elastic force and displacement indicated. If the structure does not have sufficient strength to resist the elastic force, F_{el}, then it will yield at some lower level of inertia force, say at lateral force level, F_y. It has been observed in many studies that a structure with a nonlinear cyclic force-displacement response similar to that shown in Figure 1-5b will have a maximum displacement that is not much different from the maximum elastic displacement. This is indicated in Figure 1-5c where the inelastic (plastic) displacement, Δ_u, is shown just slightly greater than the elastic displacement, Δ_{el}. This observation has led to the *equal displacement rule*, an empirical rule which states that the maximum displacement that the structure reaches in an earthquake is independent of its yield strength, i.e. irrespective of whether it demonstrates elastic or inelastic response. The equal displacement rule is thought to hold because the nonlinear response softens the structure and so the period increases, thereby giving rise to increased displacements. However, at the same time, the yielding material dissipates energy that effectively increases the damping (the energy dissipation is proportional to the area enclosed by the force-displacement loops, termed hysteresis loops). Increased damping tends to decrease the displacements; therefore, it is possible that the two effects balance one another with the result that the elastic and inelastic displacements are not significantly different.

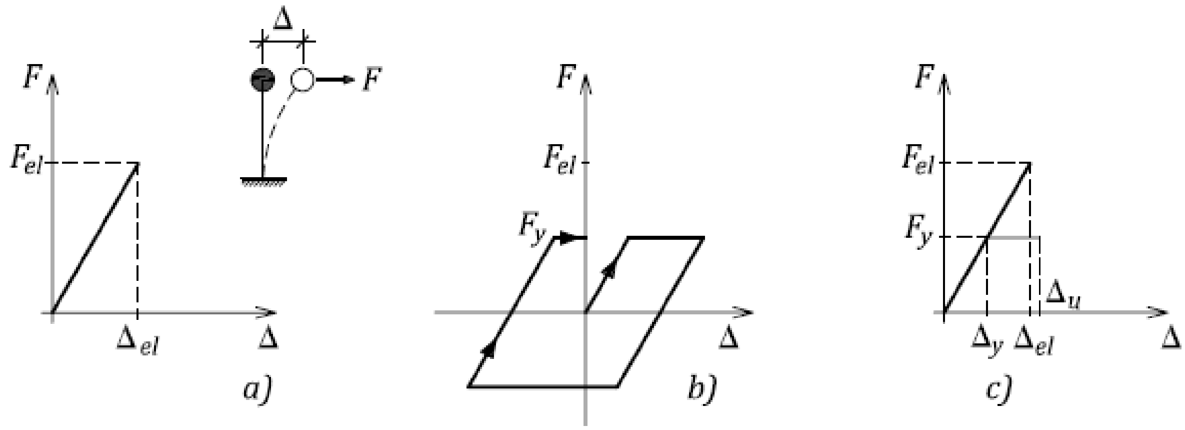


Figure 1-5. Force-displacement relationship: a) elastic response; b) nonlinear (inelastic) response; c) equal displacement rule.

There are limits beyond which the equal displacement rule does not hold. In short period structures, the nonlinear displacements are greater than the elastic displacements, and for very long period structures, the maximum displacement is equal to the ground displacement. However, the equal displacement rule is, in many ways, the basis for the seismic provisions in many building codes which allow the structure to be designed for forces less than the elastic forces. But there is always a trade-off, and the lower the yield strength, the larger the nonlinear or inelastic deformations. This can be inferred from Figure 1-5c where it is noted that the difference between the nonlinear displacement, Δ_u , and yield displacement, Δ_y , which represents the inelastic deformation, would increase as the yield strength decreases. Inelastic deformations generally relate to increased damage, and the designer needs to ensure that the strength does not deteriorate too rapidly with subsequent loading cycles, and that a brittle failure is prevented. This can be achieved by additional "seismic" detailing of the structural members, which is usually prescribed by the material standards. For example, in reinforced concrete structures, seismic detailing consists of additional confinement reinforcement that ensures ductile performance at critical locations in beams, columns, and shear walls. In reinforced masonry structures, it is difficult to provide similar confinement detailing, and so restrictions are placed on limiting the reinforcement spacing, on levels of grouting, and on certain strain limits in the masonry structural components (e.g. shear walls) which provide resistance to seismic loads (see Chapter 2 for more details on seismic design of masonry shear walls).

1.4.3 Ductility

Ductility relates to the capacity of the structure to undergo inelastic displacements. For the SDOF structure, whose force-displacement relation is shown in Figure 1-5c the displacement ductility ratio, μ_Δ , is a measure of damage that the structure might undergo and can be expressed as

$$\mu_\Delta = \frac{\Delta_u}{\Delta_y}$$

The ratio between the maximum elastic force, F_{el} , and the yield force, F_y , is given by the force reduction factor, R , defined as

$$R = \frac{F_{el}}{F_y}$$

If the material is elastic-perfectly plastic, i.e. there is no strain hardening as it yields (see Figure 1-5b), and if Δ_u is equal to Δ_{el}, then it can be shown that μ_Δ is equal to R.

For different types of structures and detailing requirements, most building codes tend to prescribe the R value while not making reference to the displacement ductility ratio, μ_Δ, thus implying that the μ_Δ and R values would be similar.

1.4.4 A Primer on Modal Dynamic Analysis Procedure

The main objective of this section is to explain how more complex multi-degree-of-freedom structures respond to earthquake ground motions and how such response can be quantified in a form useful for structural design. This background should be helpful in understanding the NBCC seismic provisions.

1.4.4.1 Multi-degree-of-freedom systems

The idea of modelling the building as a SDOF structure was introduced in Section 1.4.1, and the dynamic response to earthquake ground motions was developed in terms of a response spectrum. Such a simple model might well represent the lateral response of a single storey warehouse building with flexible walls or bracing system, and with a rigid roof system where the roof comprises most of the weight (mass) of the structure. However, this is not a good model for a masonry warehouse with a metal deck roof, where the walls are quite stiff and the deck is flexible and light relative to the walls. Such a system requires a more complex model using a multi-degree-of-freedom (MDOF) system. A shear wall in a multi-storey building is another example of a MDOF system.

Figure 1-6 shows two examples of MDOF structures. A simple four-storey structure is shown in Figure 1-6a, and a simple MDOF model for this building consists of a column representing the stiffness of vertical members (shear walls or frames), with the masses lumped at the floor levels. If the floors are rigid, it can be assumed that the lateral displacements at every point in a floor are equal, and the structure can be modelled with one degree of freedom (DOF) at each floor level (a DOF can be defined as lateral displacement in the direction in which the structure is being analyzed). This will result in as many degrees of freedom as number of floors, so this building can be modelled as a 4-DOF system. It must also be assumed that there are no torsional effects, that is, there is no rotation of the floors about a vertical axis (torsional effects will be discussed later in Section 1.5.9). The analysis will be the same irrespective of the lateral force resisting system (a shear wall or a frame), aside from details in finding the lateral stiffness matrix for the floor displacements.

The warehouse building shown in Figure 1-6b is another example of a MDOF structure. The walls are treated as a single column with some portion of the wall and roof mass, M_1, located at the top. The roof can be treated as a spring (or several springs) with the remaining roof mass, M_2, attached to the spring(s). How much mass to attach to each degree of freedom, and how to determine the stiffness of the roof, are major challenges in this case.

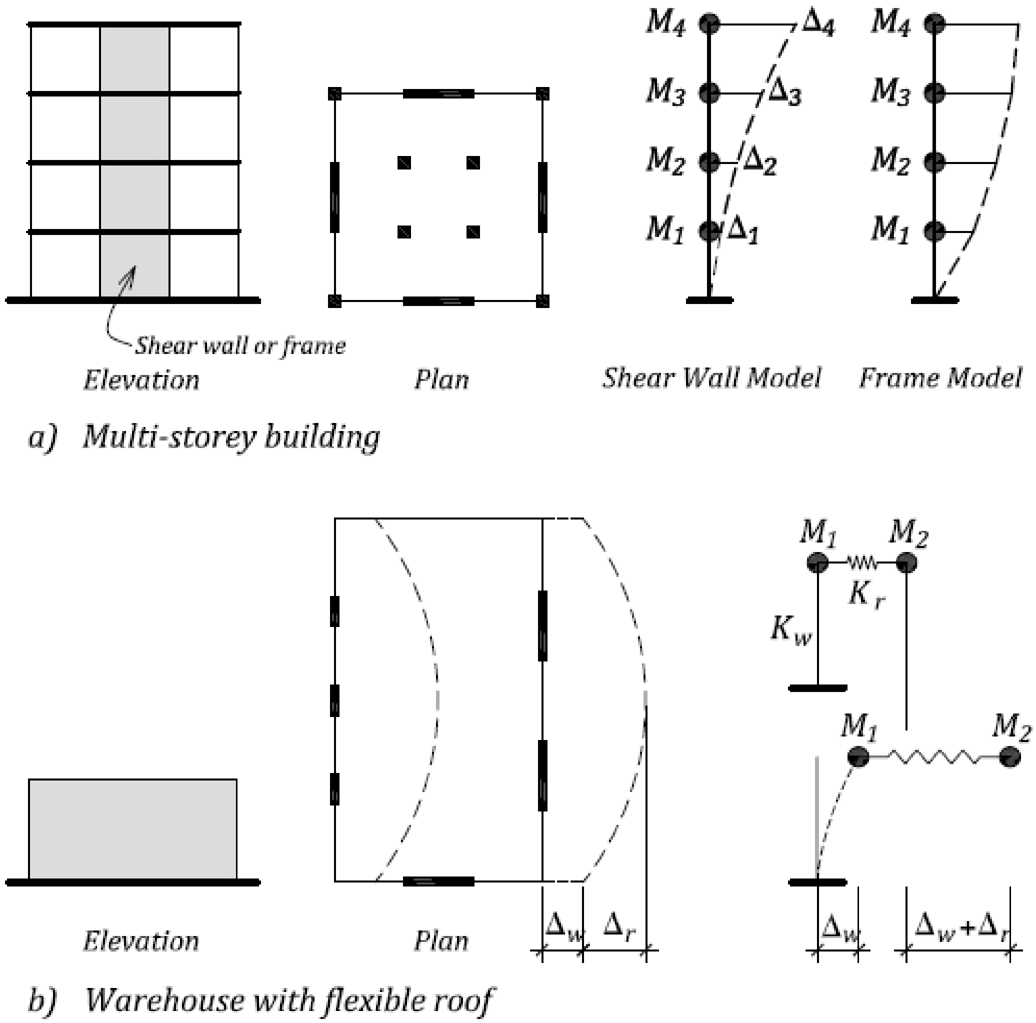


Figure 1-6. MDOF systems: a) multi-storey shear wall building; b) warehouse with flexible roof.

1.4.4.2 Seismic analysis methods

The question of interest to structural engineers is how to determine a realistic seismic response for MDOF systems? The possible approaches are:

- static analysis, and
- dynamic analysis (modal analysis or time history method).

The simplest method is the *equivalent static analysis procedure* (also known as the quasi-static method) in which a set of static horizontal forces is applied to the structure (similar to a wind load). These forces are meant to emulate the maximum effects in a structure that a dynamic analysis would predict. This procedure works well when applied to small, simple structures, and also to larger structures if they are regular in their layout.

NBCC 2005 specifies a dynamic analysis as the default method. The simplest type of dynamic analysis is the *modal analysis method*. This method is restricted to linear systems, and consists of a dynamic analysis to determine the mode shapes and periods of the structure, and then uses a response spectrum to determine the response in each mode. The response of each mode is independent of the other modes, and the modal responses can then be combined to determine the total structural response. In the next section, the modal analysis procedure will be explained with an example.

The second type of dynamic analysis is the *time history method.* This consists of a dynamic analysis model subjected to a time-history record of an earthquake ground motion. Time history analysis is a powerful tool for analyzing complex structures and can take into account nonlinear structural response. This procedure is complex and time-consuming to perform, and as such, not warranted for low-rise and regular structures. It requires an advanced level of knowledge of the dynamics of structures and it is beyond the scope of this document. For detailed background on dynamic analysis methods the reader is referred to Chopra (2007).

1.4.4.3 Modal analysis procedure: an example

Consider a four-storey shear wall building example such as that shown in Figure 1-6a. The building can be modelled as a stick model, with a weight, W , of 2,000 kN lumped at each floor level, and a uniform floor height of 3 m (see Figure 1-7). For simplicity, the wall stiffness and the masses are assumed uniform over the height. This model is a MDOF system with four degrees of freedom consisting of a lateral displacement at each storey level. A MDOF system has as many modes of vibration as degrees of freedom. Each mode has its own characteristic shape and period of vibration. The periods are given in Table 1-1, the four mode shapes are given in Table 1-2 and shown in Figure 1-7. In this example, the stiffness has been adjusted to give a first mode period of 0.4 seconds, which is representative of a four-storey structure based on a simple rule-of-thumb that the fundamental period is on the order of 0.1 sec per floor. Note that the first mode, also known as the *fundamental mode*, has the longest period. The first mode is by far the most important for determining lateral displacements and interstorey drifts, but higher modes can substantially contribute to the forces in structures with longer periods. In this example the mode shapes have been normalized so that the largest modal amplitude is unity.

For linear elastic structures, the equations governing the response of each mode are independent of the others provided that the damping is prescribed in a particular manner. Thus, the response in each mode can be treated in a manner similar to a SDOF system, and this allows the maximum displacement, moment and shear to be calculated for each mode. In the final picture, the modal responses have to somehow be combined to find the design forces (this will be discussed later in this section). Modal analysis can be performed by hand calculation for a simple structure, however, in most cases, the use of a dynamic analysis computer program would be required.

Knowing the mode shapes and the mass at each level, it is possible to calculate the *modal mass* for each mode, which is given in Table 1-1 as a fraction of the total mass of the structure. The modal masses are representative of how the mass is distributed to each mode, and the sum of the modal masses must add up to the total mass. When doing modal analysis, a sufficient number of modes should be considered so that the sum of the modal masses adds up to at least 90% of the total mass. In the example here this would indicate that only the first two modes would need to be considered (0.696 + 0.210 = 0.906).

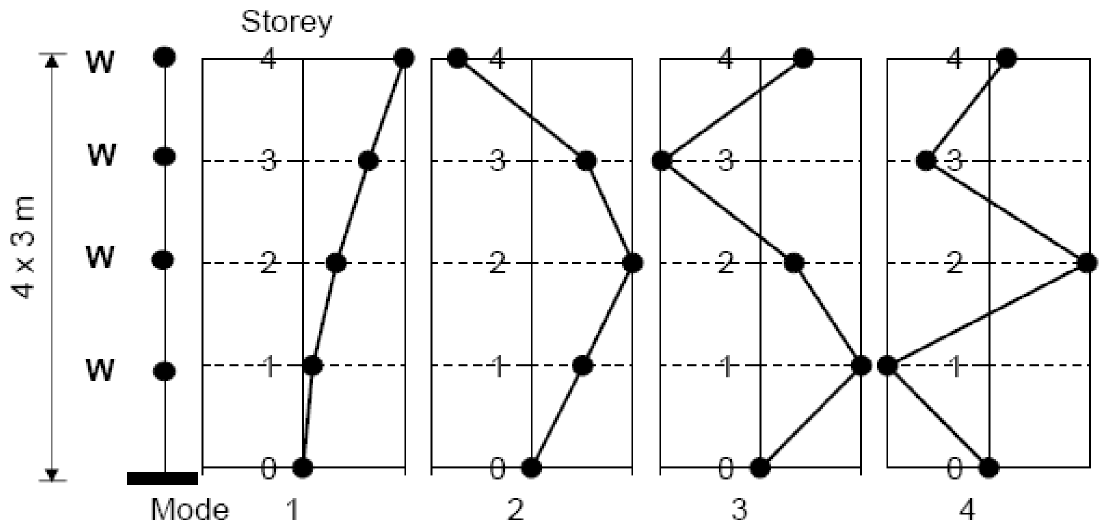


Figure 1-7. Four-storey shear wall building model and modal shapes.

As an example of how the different modes can be used to determine the structural response, Figure 1-8 shows a typical design acceleration response spectrum which will be used to determine the modal displacements and accelerations. The four modal periods are indicated on the spectrum (note that only the first two periods are identified on the diagram; T_1=0.40 and T_2=0.062 sec) and the spectral acceleration S_a at each of the periods is given in Table 1-3.

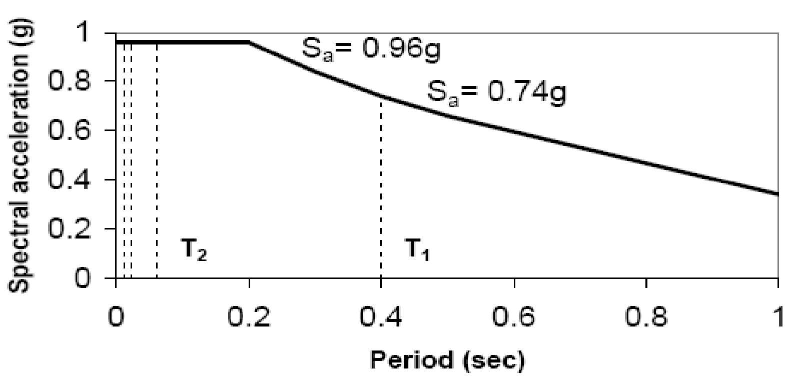


Figure 1-8. Design acceleration response spectrum.

A very useful feature of the modal analysis procedure gives the base shear in each mode as a product of the modal mass and the spectral acceleration S_a for that mode, as shown in Table 1-3. For example, the base shear for the first mode is equal to (8000kN x 0.696) x 0.74 = 4127 kN). Note that the spectral acceleration is higher for the higher modes, but because the modal mass for these modes is smaller, the base shear is smaller. The inertia forces from each floor mass act in the same directions as the mode shape, that is, some forces are positive while others are negative for the higher modes (refer to mode shapes shown in Figure 1-7). It can be seen from the figure that the forces from the first mode all act in the same direction at the same time, while the higher modes will have both positive and negative forces. Thus the base shear from the first mode is usually larger than that from the other modes.

The modal base shears shown in Table 1-3 are the maximum base shears for each mode. It is very unlikely that these forces will occur at the same time during the ground shaking, and they could have either positive or negative signs. Summing the contribution of each mode where all values are taken as positive, known as the absolute sum (ABS) method, produces a very high upper bound estimate of the total base shear. Statistical analyses have shown that the square-root-of-the-sum-of-the squares (RSS) procedure, whereby the contribution of each mode is squared, and the square root is then taken of the sum of the squares, gives a reasonably good estimate of the modal sum, especially if the modal periods are widely separated.

Table 1-3 shows the base shear values estimated by the two methods and gives an indication of the conservatism of the ABS method for this case (total base shear of 6,462 kN), where the modal periods are widely separated, and use of the RSS method is appropriate since it gives a lower total base shear value of 4,468 kN. Note that there is a third method that is incorporated in many modal analysis programs called the complete-quadratic-combination (CQC) method. This method should be used if the periods of some of the modes being combined are close together, as would be the case in many three-dimensional structural analyses, but for most structures with well-separated periods and low damping, the result of the RSS and CQC methods will be nearly identical (this is true for most two-dimensional structural analyses).

The amplitude of displacement in each mode is dependent upon the spectral acceleration for that mode and its *modal participation factor*, which is a measure of the degree to which a certain mode participates in the response. The value of the modal participation factor depends on how the mode shapes are normalized, and so will not be given here, however the values are smaller for the higher modes with the result that the displacements for the higher modes are generally smaller than those of the first mode. The modal displacements are presented in Table 1-4 (to three decimal places, which is why some values are shown as zero) and plotted in Figure 1-9 for the first two modes as well as the RSS value. In this example, the influence of the two highest modes is very small and has been omitted from the diagram. It is difficult to distinguish between the first mode displacements and the RSS displacements in Figure 1-9; this is characteristic of structures with periods less than about 1 second, such as would be the case for most masonry structures.

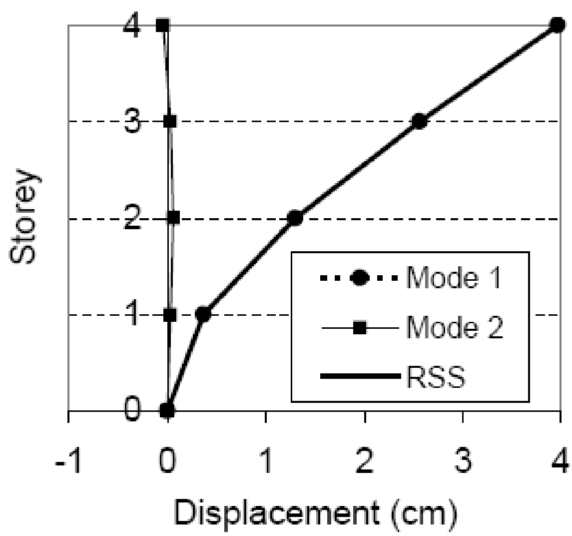
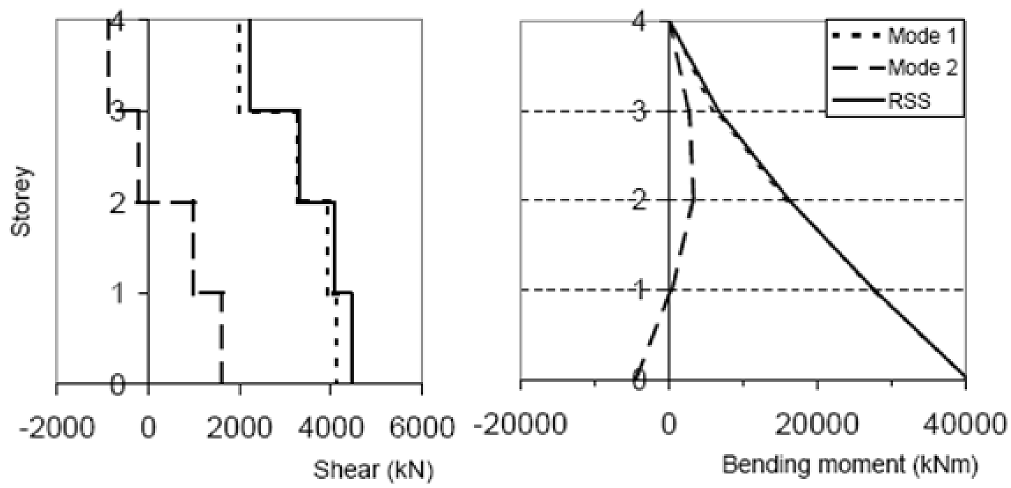


Figure 1-9. Modal displacements.

Modal analysis gives the modal shears and bending moments in each member and these values can be used to generate the shear and moment diagrams. These are summarized in Tables 1-5 and 1-6, and are graphically presented in Figure 1-10. Only the results from the first two modes are shown as the higher modes contribute very little to the response. Except for some contribution to the shears, the second mode is insignificant in contributing to the total values calculated using the RSS method.



a) b)

Figure 1-10. Modal analysis results: a) shear forces; b) bending moments.

The inertia force at each floor for each mode can be determined by taking the difference between the shear force above and below the floor in question. Modal inertia forces along with the RSS values are summarized in Table 1-7, and show that the higher modes at some levels contribute more than the first mode. Note that the sum of the inertia forces for each mode is equal to the base shear for that mode. However, the sum of the RSS values of the floor forces at each level is 6284 kN (obtained by adding values for storeys 1 to 4 in the last column of the table); this is not equal to the total base shear of 4468 kN found by taking the RSS of the base shears in each mode (see Table 1-3). This demonstrates the key rule in combining modal responses: **only primary quantities from each mode should be combined**. For example, if the designer is interested in the shear force diagram for the structure, it is necessary to find the shear forces in each mode and then combine these modal quantities using the RSS method. It is incorrect to find the total floor forces at each level from the RSS of individual modal values, and then use these total forces to draw the shear diagram. Even interstorey drift ratios, defined as the difference in the displacement from one floor to the next divided by the storey height, should be calculated for each mode and then combined using the RSS procedure. It would be incorrect to divide the total floor displacements by the storey height; although in this example since the deflection is almost entirely given by the first mode this approach would be very close to that found using the RSS method.

One of the disadvantages of modal analysis is that the signs of the forces are lost in the RSS procedure and so equilibrium of the final force system is not satisfied. Equilibrium is satisfied in each mode, but this is lost in the procedure to combine modal quantities since each quantity is squared. That is why it is important to determine quantities of interest by combining only the original modal values.

1.4.4.4 Comparison of static and modal analysis results

The equivalent static force analysis procedure, which will be presented in more detail in Section 1.5.4, has been applied to the four storey structure described above for the spectrum shown in Figure 1-8. Table 1-8 compares the results of the two types of analyses. It can be seen that both the base shear and moment given by the modal analysis method is about 75% of that given by the static method. This occurs with short period MDOF structures that respond in essentially the first mode because the modal mass of the first mode for walls is about 70 to 80% of the total mass. The top displacement from the modal analysis is 78% of the static displacement, nearly the same as the ratio of the base moments; this would be expected given that the deflection is mostly tied to the moment.

If the structure is a single-storey, SDOF system, the two analyses methods will give the same result. But for MDOF systems, such as two-storey or higher buildings, dynamic analysis will generally result in smaller forces and displacements than the static procedure.

The floor forces from the two analyses are quite different. The floor forces in the upper storeys obtained by modal analysis are less than the static forces, but in the lower storeys, a reverse trend can be observed. The reason for this is the contribution of the higher modes to the floor forces. It can be seen in Table 1-7, that at the 2^{nd} storey, the second mode contribution is the largest of all the modes. To ensure the required safety level when seismic design is performed using the equivalent static analysis procedure, NBCC 2005 seismic provisions (e.g. Clause 4.1.8.15) provides additional guidance on the level of floor forces to be used in connecting the floors to the lateral load resisting elements.

Table 1-1. Modal Periods and Masses

Mode	Period (sec)	Modal mass/ Total mass
1	0.400	0.696
2	0.062	0.210
3	0.022	0.070
4	0.012	0.024
Sum		1.000

Table 1-2. Mode Shapes

Storey	Mode Shapes			
	1st mode	2nd mode	3rd mode	4th mode
0	0.000	0.000	0.000	0.000
1	0.093	0.505	1.000	-1.000
2	0.328	1.000	0.334	0.969
3	0.647	0.544	-0.972	-0.619
4	1.000	-0.727	0.427	0.175

Note: mode shapes are normalized to a maximum of 1

Table 1-3. Spectral Accelerations, S_a, and Base Shears

Mode	Period (sec)	Spectral Acceleration S_a (g)	Modal mass / Total mass	Base Shear (kN)
1	0.400	0.74	0.696	4127
2	0.062	0.96	0.210	1617
3	0.022	0.96	0.070	534
4	0.012	0.96	0.024	184
Total base shear			ABS	6462
Total base shear			RSS	4468

Note: total weight = 8000 kN

Table 1-4. Modal Displacements

Storey	Modal Displacements (cm)				RSS
	1st mode	2nd mode	3rd mode	4th mode	
Base	0.000	0.000	0.000	0.000	0.00
1	0.367	0.021	0.002	0.000	0.37
2	1.300	0.042	0.001	0.000	1.30
3	2.564	0.023	-0.002	0.000	2.56
4	3.963	-0.031	0.001	0.000	3.96

Table 1-5. Modal Shear Forces

Storey	Shear Forces (kN) 1st mode	2nd mode	3rd mode	4th mode	RSS
0-1	4127	1617	534	-184	4468
1-2	3942	999	-143	204	4074
2-3	3287	-224	-369	-172	3320
3-4	1996	-888	289	68	2205

Table 1-6. Modal Bending Moments

Storey	Bending Moments (kNm) 1st mode	2nd mode	3rd mode	4th mode	RSS
Base	40053	-4511	-931	255	40320
1	27675	339	670	-298	27686
2	15849	3335	240	313	16201
3	5988	2665	-867	-204	6614
4	0	0	0	0	0

Table 1-7. Modal Inertia Forces (Floor Forces)

Storey	Floor Forces (kN) 1st mode	2nd mode	3rd mode	4th mode	RSS
1	185	618	677	-388	1012
2	655	1223	226	376	1455
3	1291	665	-658	-240	1612
4	1996	-888	289	68	2205
Sum	4127	1617	534	-184	4468

Table 1-8. Comparison of Static and Dynamic Analyses Results

Storey	Shear Forces (kN)		Floor Forces (kN)		Moments (kNm)		Deflections (cm)	
	Static	**Modal(1)**	**Static**	**Modal(2)**	**Static**	**Modal(3)**	**Static**	**Modal(4)**
Base			0	0	53280	40320	0	0
	5920	4468						
1			592	1012	35520	27686	0.48	0.37
	5328	4074						
2			1184	1455	19536	16201	1.70	1.30
	4144	3320						
3			1776	1612	7104	6614	3.32	2.56
	2368	2205						
4			2368	2205	0	0	5.11	3.96

Notes: (1) see Table 1-5, last column
(2) see Table 1-7, last column;
(3) see Table 1-6, last column;
(4) see Table 1-4, last column.

1.5 Seismic Analysis According to NBCC 2005

This section presents and explains the relevant seismic code provisions in NBCC 2005. Reference will be made here to NBCC 1995 where appropriate, but Appendix A contains the pertinent 1995 code provisions and a comparison of the design forces from the two codes.

1.5.1 Seismic Hazard

4.1.8.4.(6)

One of the major changes to the seismic provisions between the 1995 and 2005 editions of the NBCC is related to the determination of the seismic hazard. The 1995 code was based on probabilistic estimates of the peak ground acceleration and peak ground velocity for a probability of exceedance of 1/475 per annum (10% in 50 years). For NBCC 2005, the 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, $S_a(T)$. During the NBCC 2005 code development cycle, records became available, and the ability to compute how response spectral values vary with magnitude and distance from source to site greatly improved. Thus, it was possible to compute probabilistic estimates of spectral acceleration for different structural periods, and construct a response spectrum where each point on the spectrum has the same probability of exceedance. Such a spectrum is termed a *Uniform Hazard Spectrum*, or UHS. The acceleration UHS for Montreal is shown in Figure 1-11.

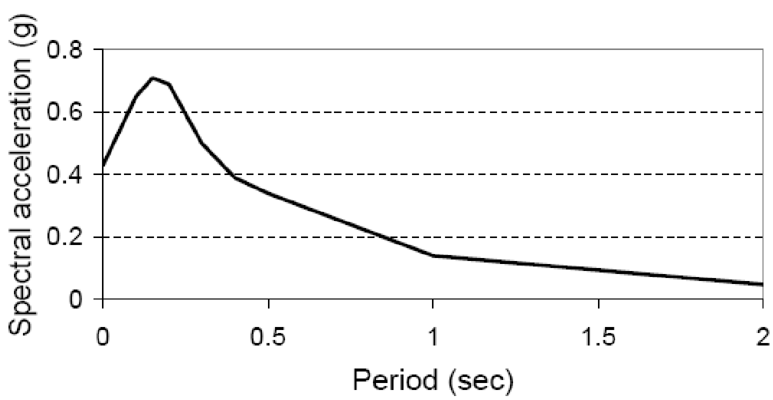


Figure 1-11. Uniform hazard spectrum for Montreal (UHS), 2% in 50 years probability, 5% damping.

For design purposes, the NBCC 2005 does not use the UHS, but rather an approximation given by four period-spectral values which are used to construct a spectrum, $S_a(T)$, which is used as the basis for the design spectrum. For many locations in the country, these values are specified in Table C-2, Appendix C to the NBCC 2005, along with the peak ground acceleration (PGA) for each location, which is used mainly for geotechnical purposes. For other Canadian locations, it is possible to find the values online at:

http://earthquakescanada.nrcan.gc.ca/hazard/interpolator/index_e.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 NBCC 2005 seismic hazard provisions, the reader is referred to Adams and Halchuk (2003).

Figure 1-12 shows the $S_a(T)$ spectrum for Montreal and the corresponding UHS. Since $S_a(T)$ is constructed using only four points (corresponding to different periods), it is an approximation to the UHS, and it also reflects some conservatism in the code. At very short periods $S_a(T)$ is taken to be constant at the $S_a(0.2)$ value, and it does not decrease to the PGA, which is the UHS value at zero period. This may appear to be very conservative, but only a few structures have periods less than 0.2 sec, and there are other reasons related to the inelastic response of such short-period structures, to be conservative in this region. Note that many low-rise masonry buildings may have a fundamental period on the order of 0.2 sec.

The data needed to calculate the UHS values for large periods (over 2 seconds) is not available for all regions in Canada, and so between 2 seconds and 4 seconds, $S_a(T)$ is assumed to vary as $1/T$. Beyond 4 seconds there is even less data, and $S_a(T)$ is assumed to be constant at the $S_a(4)$ value for periods larger than 4 seconds. $S_a(T)$ is defined as the design hazard spectrum for sites located on what is termed soft rock or very dense soil. For sites situated on either harder rock or softer soil the hazard spectrum needs to be modified as discussed below.

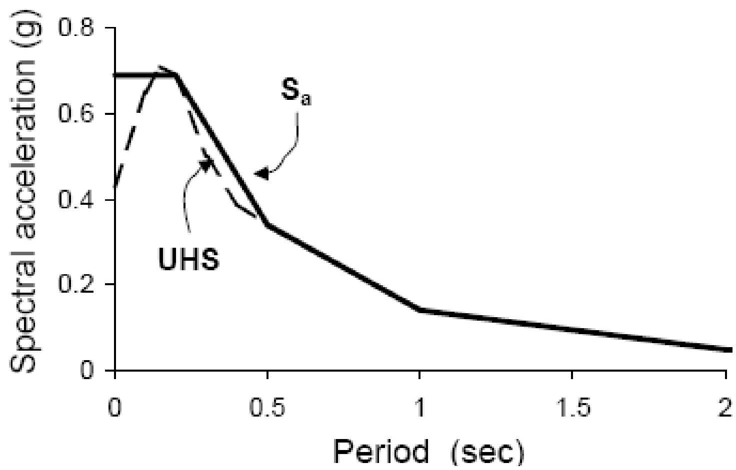


Figure 1-12. $S_a(T)$ and UHS spectrum for Montreal.

1.5.2 Effect of Site Soil Conditions

4.1.8.4

In the NBCC 2005, the seismic hazard given by the $S_a(T)$ spectrum has been developed for a site that consists of either very dense soil or soft rock (Site class C within NBCC 2005). 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 will be de-amplified. In NBCC 2005 two site coefficients are provided to be applied to the $S_a(T)$ spectrum to account for these local ground conditions. The coefficients depend on the building period, level of seismic hazard, as well as on the site properties, which are described in terms of site classes. The NBCC 2005 site coefficients are more detailed than the foundation factor, F, provided in previous code editions, but should better represent the effect of the local soil conditions on the seismic response.

Table 1-9 excerpted from NBCC 2005, describes six site classes, labelled from A to E, which correspond to different soil profiles (note that the seventh class, F, is one that fits none of the first six 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 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 site class determination is based on the weighted average, of the property being considered, in the top 30 m, which for $\overline{V}_s$ would correspond to the average velocity it would take for a shear wave to traverse the 30 m depth. NBCC 2005 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 to determine the accelerations of the building foundation.

Table 1-9. NBCC 2005 Site Classification for Seismic Response (NBCC 2005 Table 4.1.8.4.A)

Site Class	Ground Profile Name	Average Properties in Top 30 m, as per Appendix 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**	$\overline{V}_s > 1500$	Not applicable	Not applicable
B	**Rock**	$760 < \overline{V}_s \le 1500$	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**	$180 < \overline{V}_s < 360$	$15 \le \overline{N}_{60} \le 50$	$50 < s_u \le 100$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: $w \ge 40\%$; and ▪ undrained shear strength: $s_u < 25$ kPa		
F	**Other soils**(1)	Site-specific evaluation required		

Notes:
(1) 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,
d) soft to medium stiff clays more than 30 m thick.

The effect of the site class on the response spectrum is given by the following two site coefficients: F_a, which modifies the spectrum $S_a(T)$ in the short period range (see Table 1-10), and F_v, which modifies $S_a(T)$ in the longer period range (see Table 1-11).

Table 1-10. Values of F_a as a Function of Site Class and $S_a(0.2)$ (NBCC 2005 Table 4.1.8.4.B)

Site Class	Values of F_a				
	$S_a(0.2) \leq 0.25$	$S_a(0.2) = 0.50$	$S_a(0.2) = 0.75$	$S_a(0.2) = 1.00$	$S_a(0.2) = 1.25$
A	0.7	0.7	0.8	0.8	0.8
B	0.8	0.8	0.9	1.0	1.0
C	1.0	1.0	1.0	1.0	1.0
D	1.3	1.2	1.1	1.1	1.0
E	2.1	1.4	1.1	0.9	0.9
F	(1)	(1)	(1)	(1)	(1)

Notes: (1) See Sentence 4.1.8.4.(5).

Table 1-11. Values of F_v as a Function of Site Class and $S_a(1.0)$ (NBCC 2005 Table 4.1.8.4.C)

Site Class	Values of F_v				
	$S_a(1.0) \leq 0.1$	$S_a(1.0) = 0.2$	$S_a(1.0) = 0.3$	$S_a(1.0) = 0.4$	$S_a(1.0) \geq 0.5$
A	0.5	0.5	0.5	0.6	0.6
B	0.6	0.7	0.7	0.8	0.8
C	1.0	1.0	1.0	1.0	1.0
D	1.4	1.3	1.2	1.1	1.1
E	2.1	2.0	1.9	1.7	1.7
F	(1)	(1)	(1)	(1)	(1)

Notes: (1) See Sentence 4.1.8.4.(5).

Note that the F_a and F_v values depend on the level of seismic hazard as well as on 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 reduced surface motion, when compared to a low hazard motion. The softer the soil, as given by a higher site classification, the higher the site coefficients, except for a few F_a values at high hazard level. For rock and hard rock, the site coefficients will generally be less than unity.

The F_a and F_v factors are applied to the $S_a(T)$ spectrum to give $S(T)$, which is the design spectral acceleration for the site. The calculation of $S(T)$ values will be illustrated with an example.

Figure 1-13 shows the design seismic hazard spectrum, $S(T)$, for Vancouver for a firm ground site, Class C, and a soft soil site, Class E. For Vancouver (Granville and 41 Ave): $S_a(0.2)$=0.96g, $S_a(0.5)$=0.66g, $S_a(1.0)$=0.34g, and $S_a(2.0)$=0.17 (see Appendix C, NBCC 2005; note that these values were taken from an earlier version of Table C-2 and are slightly different from the published values).

Interpolating from the values in Table 1-10 for site Class E and $S_a(0.2)$=0.96g, gives F_a=0.932, and from Table 1-11 for $S_a(1.0)$=0.34g, gives F_v=1.82.

The calculations to determine $S(T)$ for the Class E site are (see Clause 4.1.8.4.(6)):

For T=0.2 sec: $S(0.2) = F_a S_a(0.2) = 0.932 \times 0.96 = 0.89$ **S(0.2)=0.89**

For T=0.5 sec:

$S(0.5) = F_v S_a(0.5) = 1.82 \times 0.66 = 1.2$, or

$S(0.5) = F_a S_a(0.2) = 0.932 \times 0.96 = 0.89$, whichever is smaller

Since the smaller value governs, **S(0.5)=0.89**

For T=1 sec: $S(1.0) = F_v S_a(1.0) = 1.82 \times 0.34 = 0.62$ **S(1.0)=0.62**

For T=2 sec: $S(2.0) = F_v S_a(2.0) = 1.82 \times 0.17 = 0.31$ **S(2.0)=0.31**

For T≥4 sec: $S(T) = F_v S_a(2.0)/2 = 1.82 \times 0.17/2 = 0.155$ **S(T≥4.0)=0.155**

The resulting $S(T)$ soil Class C and E design spectra for Vancouver are plotted in Figure 1-13.

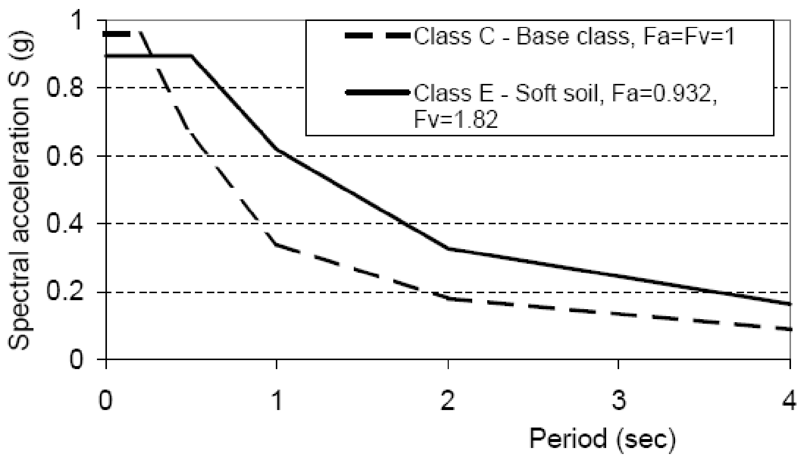


Figure 1-13. S(T) design spectra for Vancouver for site Classes C and E.

1.5.3 Methods of Analysis

4.1.8.7

NBCC 2005 prescribes two methods of calculating the design base shear of a structure. The *dynamic method* is the default method, but the *equivalent static method* can be used if the structure meets <u>any</u> 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)),

(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 irregularity, and is less than 20 m in height with period, T_a, less than 0.5 seconds in either direction (see Section 1.5.10.1 for more details on irregularities).

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 were presented in Section 1.4.4, and a further discussion is offered in Section 1.5.12.

1.5.4 Base Shear Calculations- Equivalent Static Analysis Procedure

4.1.8.11

The lateral earthquake forces used in design are specified in the NBCC 2005, and are based on the maximum (design) base shear, V , of the structure as given by Clause 4.1.8.11. The elastic base shear, V_e , denotes 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 1.5.5), 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, as defined in Table 1-12.

The relationship between V_e and V is shown in Figure 1-14. Note that the actual strength of the structure is greater than the design strength V .

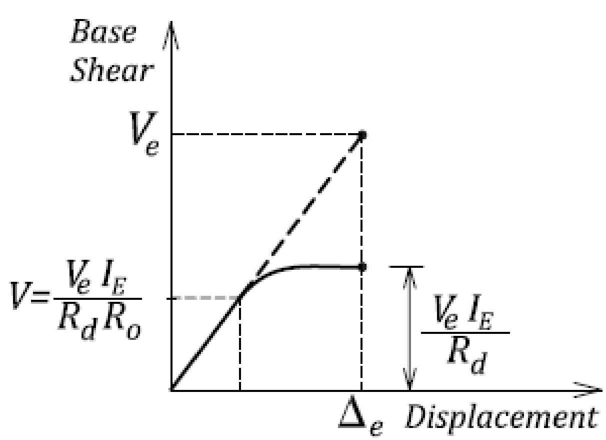


Figure 1-14. Design base shear, V , and elastic base shear, V_e .

NBCC 2005 prescribes the following lower and upper bounds for the design base shear, V :

<u>a) Lower bound:</u>
Because of uncertainties in the hazard spectrum, $S_a(T)$, for periods greater than 2 seconds, the minimum design 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 is given by:

$$V_{max} = \left(\frac{2S(0.2)}{3}\right)\left(\frac{I_E W}{R_d R_o}\right), \text{ provided } R_d \geq 1.5$$

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

Some site specific studies for soil classes E and F, especially those located in high seismic zones, may show spectral values for periods of 0.5 to1.0 seconds to be greater than $2S(0.2)/3$. If this occurs it is recommended that the spectral value used in the short period range not be less than maximum value at the longer period.

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, NBCC 2005). 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 1-14.

T_a denotes the *fundamental period* of vibration of the building or structure in seconds in the direction under consideration (i.e. direction of seismic force). The fundamental period of wall structures is given in the NBCC 2005 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).

The code formula to calculate T_a in (a) is simpler than the corresponding NBCC 1995 equation, in that it is based solely on building height and not on the length of the walls, and the allowance for using a calculated T_a in (b) is usually more liberal than in NBCC 1995. The period given by the NBCC 2005 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.

Table 1-12. NBCC 2005 Seismic Design Parameters

Design parameter		NBCC reference
$S(T)$ =	the design spectral acceleration that includes the site soil coefficients F_a and F_v (see Section 1.5.2) $S(T) = F_aS_a(0.2)$ for T < 0.2 s $= F_vS_a(0.5)$ or $F_aS_a(0.2)$ whichever is smaller for T= 0.5 s $= F_vS_a(1.0)$ for T = 1.0 s $= F_vS_a(2.0)$ for T = 2.0 s $= F_vS_a(2.0)/2$ for T ≥ 4.0 s	Cl.4.1.8.4(6)
M_v =	higher mode factor (see Section 1.5.6)	Cl.4.1.8.11.(5) 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 NBCC 2005 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. This requirement is the same as in the NBCC 1995 except that minimum partition load that need not exceed 0.5 kPa, and that parking garages need not be considered as storage areas.	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 1.5.5); *ranges from 1.0 for unreinforced masonry to 2.0 for moderately 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 1.5.5); *equal to 1.5 for all reinforced masonry walls.*	Table 4.1.8.9

1.5.5 Force Reduction Factors R_d and R_o

4.1.8.9

Table 1-13 (NBCC 2005 Table 4.1.8.9) gives the R_d and R_o values for the different types of masonry lateral load-resisting systems, which are termed the Seismic Force Resisting Systems, SFRS(s), by NBCC 2005 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, Table 1-13 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) (Source: NBCC 2005 Table 4.1.8.9)

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 S304.1							
Moderately ductile shear walls	2.0	1.5	NL	NL	60	40	40
Limited ductility shear walls	1.5	1.5	NL	NL	40	30	30
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 = not permitted.
NL = system is permitted and not limited in height as an SFRS; height may be limited in other parts of the NBCC.
Numbers in this Table are maximum height limits in m.
The most stringent requirement governs.

Commentary

Table 1-13 identifies the following five SFRS(s) related to masonry construction:

1. Moderately ductile shear walls
2. Limited ductility shear walls
3. Conventional construction: shear walls and moment resisting frames
4. Unreinforced masonry
5. Other undefined masonry SFRS(s)

Note that moderately ductile shear walls are assigned the highest R_d value of 2.0, leading to the lowest design forces for masonry structures. The detailing requirements, given in CSA S304.1-04, are the most restrictive of all the masonry shear wall types, but the height limitations imposed by the NBCC 2005 are the most liberal, allowing structures up to 60 m in height (approximately 20 storeys) in moderately high seismic regions. This type of construction would normally only be used in taller structures, but is required for masonry SFRS(s) used in post-disaster buildings. Moderately ductile squat shear walls, those with a height-to-length ratio less than 1, are a separate class of moderately ductile shear walls. They are allowed higher shear resistance, and less restrictive requirements on the height-to-thickness ratio, when compared to regular moderately ductile walls.

Limited ductility shear walls and conventional construction shear walls both have $R_d = 1.5$. The limited ductility walls have more stringent detailing requirements than the conventional construction walls, but the height restrictions imposed by the NBCC 2005 are not as onerous. It is likely that the most common type of masonry shear wall construction used would be conventional construction walls.

Conventional construction moment-resisting frames are also allowed an $R_d = 1.5$, but are not permitted in moderately high seismic regions. CSA S304.1 does not discuss moment frames and they will not be discussed further here as they are rarely, if ever, used in masonry design.

Unreinforced masonry construction is only allowed where $I_E F_a S_a(0.2) < 0.35$, and is limited to a height of 15 m, except that they can go to a height of 30 m if $I_E F_a S_a(0.2) < 0.2$. 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 NBCC 2005 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 above the specified yield strength, and iii) a factor of about 1.1 that recognizes that, because of restrictions on possible locations for the reinforcement in masonry walls, the amount of reinforcement is in most cases larger than that required. This results in an $R_o = 1.5$ after some rounding of the factors (Mitchell et al., 2003).

1.5.6 Higher Mode Effects (M_v factor)

4.1.8.11.(5)

In the determination of elastic base shear, V_e, only the first mode spectral value $S(T)$ is used. To account for the additional base shear that comes from the higher modes, the M_v factor is introduced. M_v depends on the type of SFRS, the fundamental period T_a, and the ratio $S_a(0.2)/S_a(2.0)$. The M_v values assigned by NBCC 2005 are presented in Table 1-14. A discussion about the base overturning reduction factor, J, (also tabled) is provided in Section 1.5.8.

Table 1-14. Higher Mode Factor, M_v, and Base Overturning Reduction Factor, $J^{(1)(2)}$ Forming Part of Sentence 4.1.8.11.(5) (NBCC 2005 Table 4.1.8.11)

$S_a(0.2)/S_a(2.0)$	Type of Lateral Resisting Systems	M_v		J	
		$T_a \le 1.0$	$T_a \ge 2.0$	$T_a \le 0.5$	$T_a \ge 2.0$
< 8.0	**Moment resisting frames or coupled walls (3)**	1.0	1.0	1.0	1.0
	Braced frames	1.0	1.0	1.0	0.8
	Walls, wall-frame systems, other systems (4)	1.0	1.2	1.0	0.7
≥ 8.0	**Moment resisting frames or coupled walls (3)**	1.0	1.2	1.0	0.7
	Braced frames	1.0	1.5	1.0	0.5
	Walls, wall-frame systems, other systems (4)	1.0	2.5	1.0	0.4

Notes:

(1) For values of M_v between fundamental lateral periods, T_a, of 1.0 and 2.0 s, the product $S(T_a) \cdot M_v$ shall be obtained by linear interpolation.

(2) Values of J between fundamental lateral periods, T_a, of 0.5 and 2.0 s shall be obtained by linear interpolation.

(3) A “coupled wall” is a wall system with coupling beams, where at least 66% of the base overturning moment resisted by the wall system is carried by the axial tension and compression forces resulting from shear in the coupling beams.

(4) For hybrid systems, values corresponding to walls must be used or a dynamic analysis must be carried out as per Article 4.1.8.12.

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 $S_a(0.2)/S_a(2.0) \ge 8.0$, 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, and thus, these modes make a substantial contribution to the base shear. In western Canada, where $S_a(0.2)/S_a(2.0) < 8.0$, the spectrum does not decrease as sharply with increasing period, and the higher mode shears are less important when compared to the first mode base shear. It can be noted from Table 1-14 that the M_v factor is largest for wall structures, ranging in value from 1.0 to 2.5. This is relevant for high-rise masonry wall structures, and arises because the modal mass for the higher modes is larger in wall structures than in frames, and because the difference in periods between the modes is larger in wall than in frame structures.

For periods ranging from 1 to 2 seconds, M_v increases but $S(T)$ decreases, and 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 2 seconds, M_v is assumed constant, although it theoretically could be larger. However, since V_e is conservatively based on the $S(2.0)$ spectral value, it is appropriate to use the 2 second value of M_v.

Higher mode effects also affect the overturning moments and the value of J ; this will be discussed in Section 1.5.8.

1.5.7 Vertical Distribution of Seismic Forces

4.1.8.11.(6)

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 1-15):

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

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.

According to NBCC 2005, Sentence 4.1.8.11.(4), the seismic weight W is the sum of the weights at each floor, $W = \sum_{1}^{n} W_i$ (see Table 1-12).

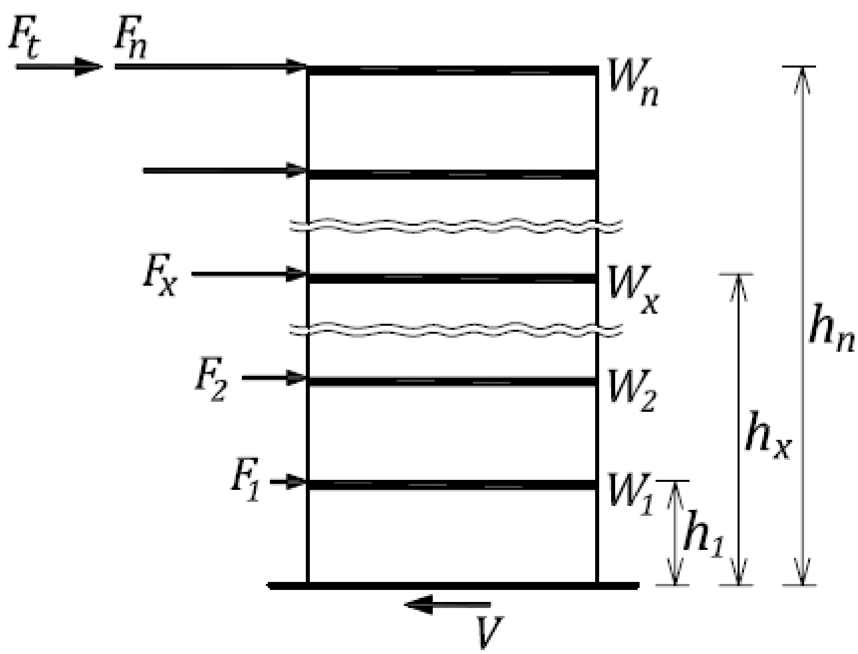


Figure 1-15. Vertical force distribution.

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 Cl.4.1.8.11.(6):

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

$F_t = 0.07T_aV$ for $0.7 < T_a \leq 3.6$ 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. By establishing the forces at each floor level, the total storey shears can be calculated using statics.

1.5.8 Overturning Moments (J factor)

4.1.8.11.(5)
4.1.8.11.(7)

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 NBCC have traditionally used a factor, termed the J factor, to reduce the moments, but the value of the J factor and how it is applied over the height of the structure is substantially different in NBCC 2005.

The J factor values are given in Table 1-14. Note that for the 2 second period, J is nearly equal to the inverse of M_v, which implies that the overturning moment at the base of the structure is governed by the first mode.

The overturning moment at any level shall be multiplied by the factor J_x (see Figure 1-16), where

$J_x = 1.0$ for $h_x \geq 0.6h_n$ (there is no reduction over the top 40% of the structure), and

$J_x = J + (1 - J)(h_x / 0.6h_n)$ for $h_x < 0.6h_n$ (a linear increase from J at the base to 1.0 at the 60% level).

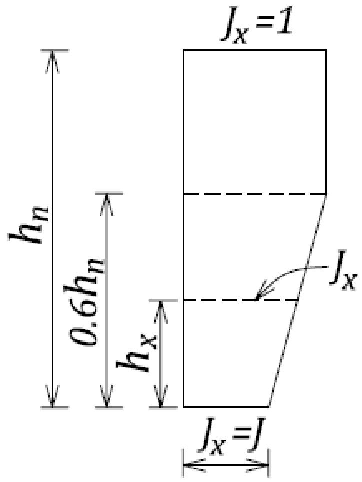


Figure 1-16. Distribution of the J_x factor over the building height.

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 larger, leading to higher shears when the structure yields, and could result in a shear failure.

For frames, the member shears, moments and axial loads, resulting from applying the lateral seismic forces at each floor level, will be too large. 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. The J factor for frames is usually small, and it is believed that many designers ignore it as it is conservative to do so.

1.5.9 Torsion

1.5.9.1 Torsional effects

4.1.8.11.(8)

Torsional effects, that are concurrent with the effects of the lateral forces F_x, and that are caused by the following torsional moments shall 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 1.5.9.1 to 1.5.9.3 considers rigid diaphragms only, while flexible diaphragms are discussed in Section 1.5.9.4.

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 of 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 1-17 shows two different plan configurations, one of which has a non-symmetric wall layout (a), and the other one with 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 1-17b, 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, also give satisfactory torsional resistance (see Section 1.5.9.2 for a discussion on torsional sensitivity).

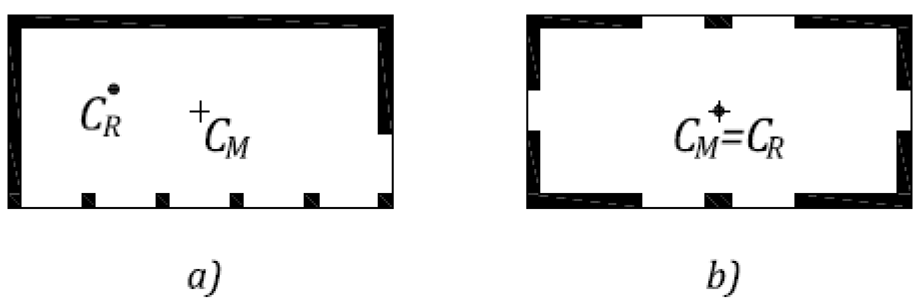


Figure 1-17. Building plan: a) non-symmetric wall layout (significant torsional effects);

b) symmetric wall layout (minor torsional effects).

Figure 1-18a 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 1-18b, and pure torsion in the form of torsional moment, T_x, about the point C_R, as shown in Figure 1-18c. 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 NBCC 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.

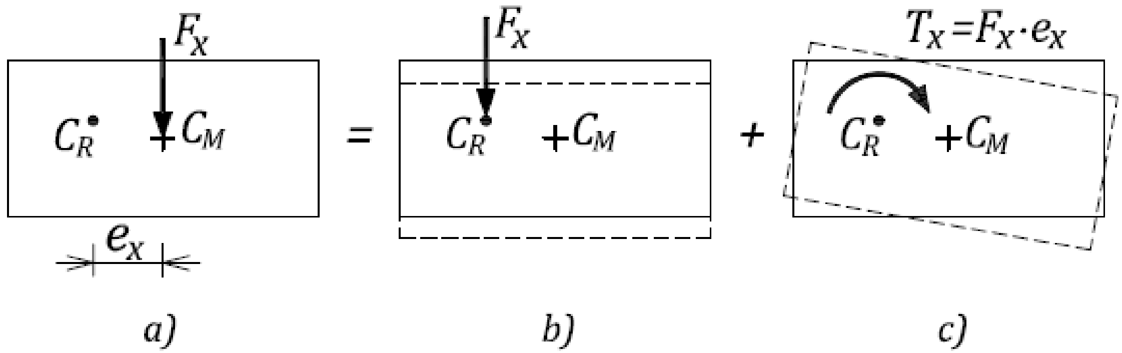


Figure 1-18. Torsional effects can be modelled as a combination of a seismic force, F_x, at point C_R (causing translational displacements only) and a torsional moment, T_x (causing rotation of building plan) about point C_R.

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 NBCC 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. NBCC 2005 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, NBCC 2005 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.

1.5.9.2 Torsional sensitivity

4.1.8.11.(9)

NBCC 2005 requires the determination of a torsional sensitivity parameter, B, which is used to determine possible analysis methods. To determine B, a set of lateral forces, F_x, is applied at a distance of $\pm 0.1D_{nx}$ 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 (see Figure 1-19):

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

where

δ_{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.

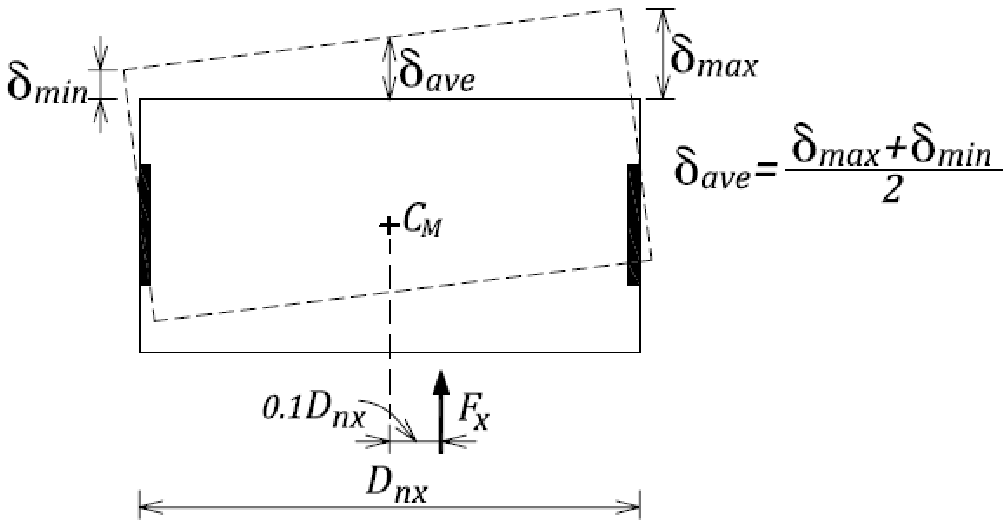


Figure 1-19. Torsional displacements used in the determination of B_x.

The torsional sensitivity, B, is the maximum value of B_x for all storeys for both orthogonal directions. Note that B_x needs 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 NBCC 2005 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 with the dynamic results, as discussed in Section 1.5.9.3. 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.

1.5.9.3 Determination of torsional forces

4.1.8.11.(10)

Torsional effects should be accounted for as follows:

a) if $B \le 1.7$ (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 1-20.

b) if $B > 1.7$, and $I_E F_a S_a(0.2) \ge 0.35$, 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 $B \le 1.7$, it is permitted to use a three-dimensional dynamic analysis with the centres of mass shifted by by a distance of $\pm 0.05D_{nx}$ (see Cl.4.1.8.12.(4)b).

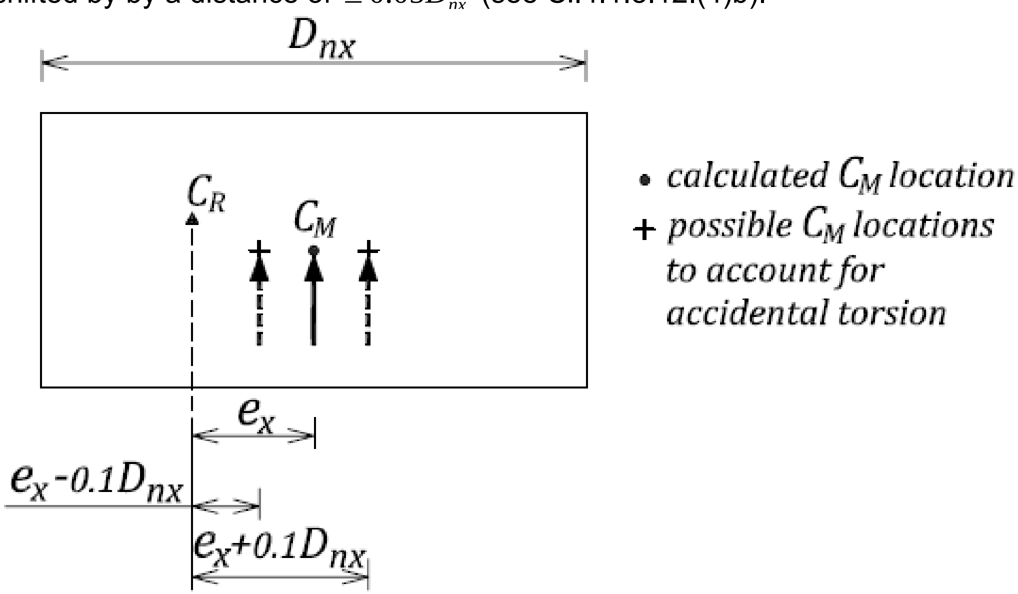


Figure 1-20. Torsional eccentricity according to NBCC 2005.

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. NBCC 2005 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 1.5.12). 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 1-21.

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.

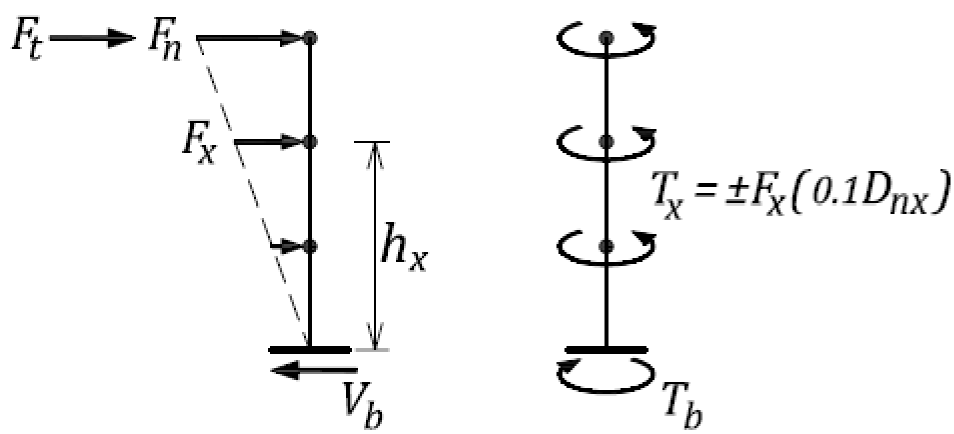


Figure 1-21. Static approach to determine the accidental eccentricity effects (Anderson, 2006).

If a dynamic set of floor forces, F_x, are to be used, they should be scaled, if necessary (as discussed in Section 1.5.12), 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 1.4.4.3), 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 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, NBCC 2005 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 1.5.10.1 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 NBCC 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 NBCC 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.

1.5.9.4 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 clearly 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 1.5.9.1 to 1.5.9.3. 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 1-22a 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 1-22b, 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 such as shown in Figure 1-17b.

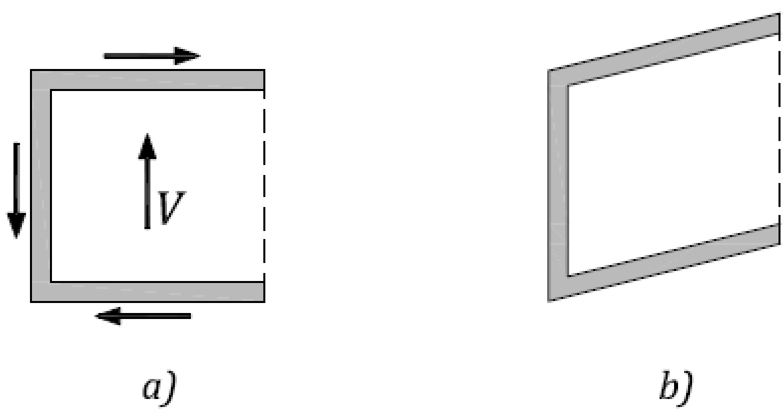


Figure 1-22. Building plan: a) loads on diaphragm; b) displaced shape of a flexible diaphragm.

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 1-23 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.

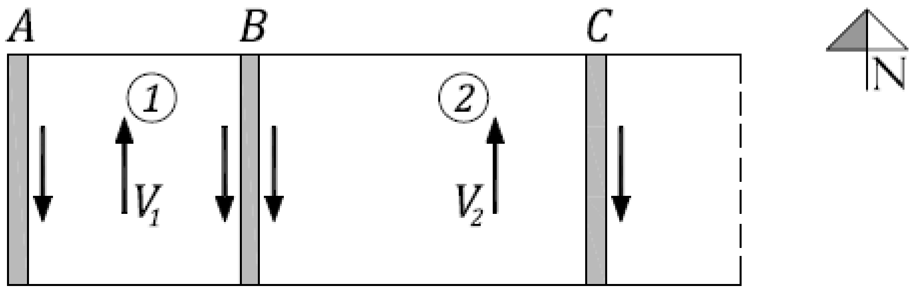


Figure 1-23. Plan view for analysis of flexible diaphragm.

NBCC 2005 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. NBCC 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 NBCC 2005.

1.5.10 Configuration Issues: Irregularities and Restrictions

1.5.10.1 Irregularities

4.1.8.6

New definitions of structural irregularities represent a substantial change in NBCC 2005. There are eight different types of irregularity, and these are used to trigger restrictions and special requirements, some of which are more restrictive than those in previous codes.

Table 1-15 (same as Table 4.1.8.6 in NBCC 2005) lists the eight types of irregularity, and the notes to the table refer to the relevant code clauses that consider the irregularity. The NBCC 2005, Commentary J (NRC, 2006) provides an expanded description of each type of irregularity. If a structure has none of the listed irregularities it is considered to be a *regular structure*. A trigger for the NBCC 2005 irregularity provisions (Cl.4.1.8.6) is the presence of one of eight types of irregularity in combination with the higher seismic hazard index, that is, $I_E F_a S_a(0.2) > 0.35$.

Table 1-15. Structural Irregularities[1] Forming Part of Sentence 4.1.8.6.(1) (NBCC 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***	Out-of-plane offsets are 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.	(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(9)).	(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.	(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 Appendix A.
(5) See Article 4.1.8.15.
(6) See Sentences 4.1.8.11.(9), (10), 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 1-24 illustrates the NBCC 2005 irregularity types. Note that Types 1 to 6 are vertical (elevation) irregularities, while Types 7 and 8 are horizontal (plan) irregularities.

According to NBCC 2005 Clause 4.1.8.7, the structure is considered to be “regular” if it has none of the eight 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:

- the seismic hazard index $I_E F_a S_a(0.2) < 0.35$, or
- the structure is regular, less than 60 m in height, and has a period T < 0.5 seconds in either direction, or
- the structure is irregular, but does not have Type 7 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 1.5.9.3.

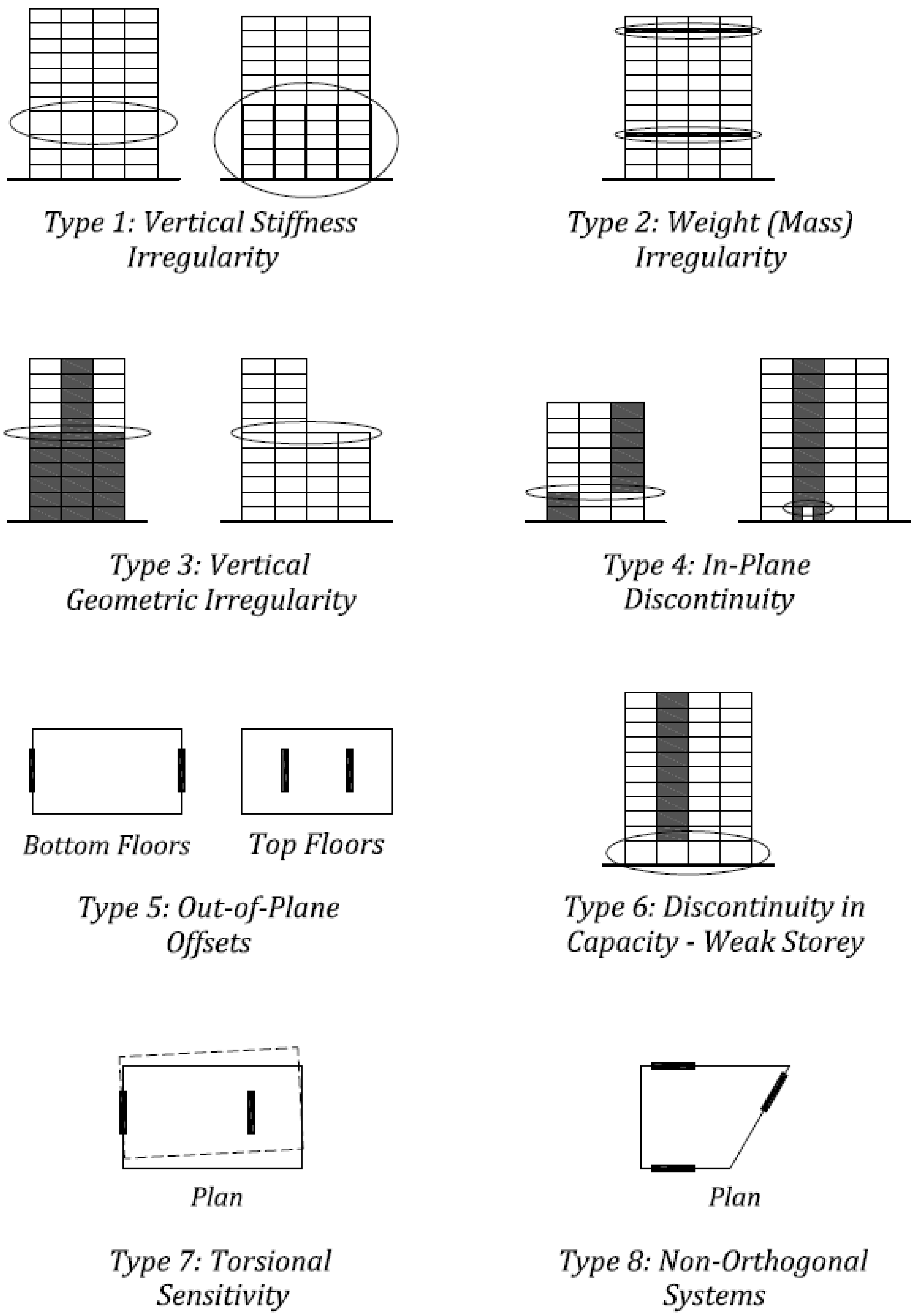


Figure 1-24. Types of irregularity according to NBCC 2005 (Tremblay and DeVall, 2006).

1.5.10.2 Restrictions

4.1.8.10.

Restrictions in NBCC 2005 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, and 7 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 $R_d \geq 2.0$.
3. For buildings having fundamental lateral periods $T_a > 1.0s$, and where $I_E F_a S_a(0.2) > 0.25$, walls 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.

Note that Table 1-15 in this document is the same as NBCC 2005 Table 4.1.8.6.

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 shear walls (with $R_d = 2.0$), 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.

1.5.11 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 NBCC 2005 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 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, NBCC 2005 provisions have moved in the direction of tightening up the drift limits from the previous versions. NBCC 2005 drift limits are more restrictive than those stated in NBCC 1995 because they apply to displacements based on a 1/2475 year return period event, whereas the NBCC 1995 uses the 1/475 year event (DeVall, 2003). 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 1-25. 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.

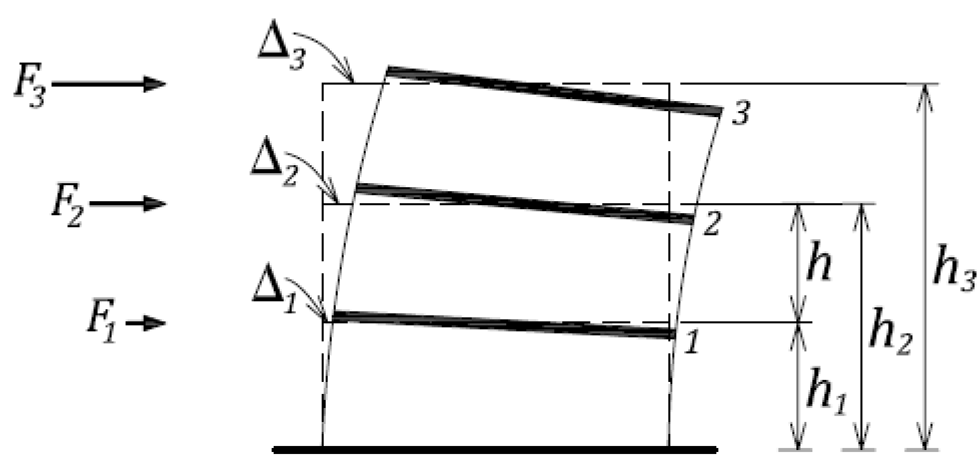


Figure 1-25. Lateral deflections and drift.

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 NBCC provisions (see Section

1.5.4) 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 NBCC 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.

1.5.12 Dynamic Analysis Method

4.1.8.12

In NBCC 2005 the default analysis method is the dynamic method. For many structures, even though the equivalent static analysis method could be used according to NBCC 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, will give the dynamic elastic base shear, V_e. NBCC 2005 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. The comparison can be made on the basis of the elastic base shear, V_e, or the design base shear, V, but must be the same for both analyses.

Since the static analysis method is allowed to reduce the design base shear by a factor of two-thirds in the short period range while the dynamic analysis method must use the design spectrum *S(T)*, it is very unlikely that for short period structures the base shear determined using the dynamic method would ever be less than that given by the static method, let alone less than the 80% value allowed for regular buildings. This is an inconsistency in the code as it adversely impacts the results from a dynamic analysis for short period structures, but not for longer period structures. It is anticipated that the NBCC code, or at least the commentary to the code, will be changed for the next edition due out in 2010, allowing the base shear from a dynamic analysis be evaluated using a spectrum where the short period values are reduced by one third.

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.

1.5.13 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 rotations of the foundation, but they would increase the interstorey drifts which would have an influence on other parts of the structure. While it is not so apparent, the larger deflections may lead to larger inelastic deformations and larger ductility demands. However, at the small ductilities used in masonry design this is most likely not to be a concern.

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.