N E P A L   N A T I O N A L   B U I L D I N G   C O D E

NBC 105: 2019

COMMENTARY

SEISMIC DESIGN OF BUILDINGS IN NEPAL

Government of Nepal

Central Level Project Implementation Unit (Building)

National Reconstruction Authority

Babar Mahal, Kathmandu, NEPAL

2076

Preface

This Commentary to the National Building Code NBC 105: 2019 Seismic Design of Buildings in Nepal referred to here as “Code”, has been developed after the revision of the NBC 105: 1994. It forms a document intended to facilitate the designers to apply the clauses of the revised Code in the seismic design of buildings in Nepal. It is to be read in conjunction with the revised Code NBC 105: 2019. The purpose of the Commentary is to provide background to the various provisions in the Code, and to highlight approaches satisfying the intent of the Code. The Commentary also, where applicable, further describes differences between the revised and its earlier edition of the Code.

The numbering of the Clauses in the Commentary is identical to that of the Code except that the Commentary Clauses are prefixed with the letter ‘**C**’. For example 3.2.2 refers to that Clause in the Code for Modal Response Spectrum Method (MRSM), whereas C3.2.2 refers to the corresponding commentary Clause. Commentary is not provided for all Clauses in the Code.

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Contents

PART 1 SCOPE AND DEFINITIONS

C1. Title, Scope, Definitions and Notations

C1.1 Title

Nepal National Building Code NBC 105: Seismic Design of Buildings in Nepal is the title of this document. The document is the outcome of the revision of the earlier version of NBC 105: 1994 Seismic Design of Buildings in Nepal.

C1.2 Scope

This code covers the requirements for seismic analysis and design of various building structures to be constructed in the territory of the Federal Republic of Nepal. This code is applicable to all buildings, low to high rise buildings, in general. Requirements of the provisions of this standard shall be applicable to buildings made of reinforced concrete, structural steel, steel concrete composite, timber and masonry. In conjunction to this standard reference to specialist literatures have to be made for design of Base-isolated buildings as well as for buildings equipped and treated with structural control.

This standard is not intended to be used for design of structures other than buildings. However, this standard can be referred for determining the general seismic loading criteria.

C1.3 Definitions

Standard terminologies related to earthquake resistant design of buildings used in this code have been defined in this section. The intent of these definitions is to have a uniform and consistent understanding of the related terms among the engineers using this code.

C1.4 Notations

The following symbols and notation shall apply to the provisions of this section:

a_p	Component Amplification Factor
A_w	Area of Web
b	Maximum horizontal dimension of the building at the particular level measured perpendicular to the direction of loading
CQC	Complete Quadratic Combination
C(T)	Elastic Site Spectra for horizontal loading
$C_d(T)$	Horizontal Design Spectrum
$C_d(T_1)$	Horizontal Base Shear Coefficient
$C_h(T)$	Spectral Shape Factor
$C_s(T)$	Elastic Site Spectra for Serviceability Limit State
$C_v(T)$	Elastic Site Spectra for Vertical Loading
$C_d(T_i)$	Ordinate of the design spectrum for translational period Ti
DL	Design dead load
d*	Displacement of equivalent SDOF system
d_{et}*	Target displacement of a structure with period T*
d_i	Horizontal displacement of the center of mass at level i, ignoring the effect of Torsion
d_y*	Displacement at yield of idealized SDOF system
E	Design earthquake load
ESM	Equivalent Static Method
e_c	Computed distance between the center of mass and the center of rigidity

e_d	Design eccentricity of the seismic force at a particular level
E_c	Modulus of Elasticity of Concrete
E_x	Design earthquake load in Principal direction X
E_y	Design earthquake load in Principal direction Y
F*	Force of equivalent SDOF system
F_i	Lateral force acting at level i
F_p	Design seismic force for parts and components
F_y*	Yield force of idealized SDOF system
g	Acceleration due to gravity. To be taken as 9.81 m/s^2
H	Height from the base to the top of the main portion of the building or the eaves of the building (m)
h_i	Height of the level i from the base considered
h_p	Height of the component
i	level under consideration of the structure
I	Importance factor for the building
I_p	Component Importance Factor
I_g	Section moment of inertia calculated using the gross cross sectional area of concrete
LL	Design live load
m*	Mass of equivalent SDOF system
MRSM	Modal Response Spectrum Method
n	Number of levels in a structure
PGA	Peak Ground Acceleration
R_S	Ductility Factor for Serviceability Limit State

R_μ	Ductility Factor for Ultimate Limit State
SDOF	Single degree of freedom
SRSS	Square root of sum of squares
T*	Time period of idealized equivalent SDOF system
T_1	Approximate Fundamental Period of Vibration
T_c	Corner period corresponding to the end of constant spectral acceleration range
T_i	Fundamental Translation Period of i^{th} mode of vibration
V	Horizontal seismic base shear obtained from equivalent static method
V_R	Combined base shear obtained from modal response spectrum method
W	Seismic weight of the structure
W_i	Seismic weight at level i
W_p	Component weight
Z	Seismic zoning factor
Ωs	Overstrength factor for serviceability limit state
Ωu	Overstrength factor for ultimate limit state
α	Peak spectral acceleration normalized by PGA
μ_p	Component ductility factor

C1.5 UNITS

Unless otherwise noted, this code uses SI units of kilograms, metres, seconds, Pascals and Newtons (kg, m, s, Pa, N).

C2. General Principles

C2.1 PERFORMANCE REQUIREMENTS AND VERIFICATION

The basic philosophy of modem earthquake-resistant design is a two-stage process, the objectives of which can be summarised as follows:

1. A structure should have sufficient strength and stiffness to resist moderate earthquakes so that the frequency of occurrence of significant damage to primary and secondary elements is acceptably low.

2. A structure must be sized and detailed to ensure that the probability of collapse in a severe earthquake, in its useful life, is acceptably low.

The first objective is referred to as **damage limitation objective**. This objective is intended to limit both the number of times a building is likely to incur loss and the cost of repair of damage over the life of a building. The damage limitation objective is verified by consideration of **serviceability limit state.** The **serviceability limit state** for ordinary buildings is based on earthquake ground motions with a return period of approximately less than the design life of the building.

The second objective is referred to as **life safety objective**. This objective is intended to ensure the safety of the building occupants due to structural collapse or failure of elements/components which could be life threatening. It is also intended to prevent failure of building components which are critical to safe evacuation of people from the building after an earthquake. The life safety objective is verified by consideration of **ultimate limit state.** The ultimate limit state for ordinary buildings is based on earthquake ground motions with a return period of 475 years.

C2.2 GENERAL GUIDELINES FOR ARRANGEMENT OF BUILDING STRUCTURAL SYSTEMS

Past earthquakes have repeatedly demonstrated that simple, symmetrical structures have the best seismic performance. The main reasons for this are that the performance is more predictable and that there is less ductility demand in members of structures which do not respond strongly in a torsional mode. Symmetry in both directions is to be encouraged.

The response of wings on buildings with re-entrant angles (such as in H or cruciform-shaped buildings) may be different to the response of the building as a whole, and may produce high local forces. It will generally be more satisfactory to separate the sections of the structure to prevent interaction and to enable simpler analysis techniques to be employed.

It has been proven analytically and demonstrated in practice in many earthquakes that marked changes in rigidity within a structure may lead to the formation of a column-hinge mechanism which will impose excessively severe demands on the ductility required of members at that level.

Henry Degenkolb, a noted earthquake engineer of USA, aptly summarized the immense importance of seismic configuration in his words: “If we have a poor configuration to start with, all the engineer can do is to provide a band-aid - improve a basically poor solution as best as he can. Conversely, if we start-off with a good configuration and reasonable framing system, even a poor engineer can’t harm its ultimate performance too much.”

Therefore, following are the important aspects that need to be followed for good seismic performance of buildings:

1. Structural simplicity
2. Uniformity, symmetry and redundancy
3. Adequate lateral resistance and stiffness
4. Diaphragm action
5. Adequate foundation

C2.3 RESPONSE TO EARTHQUAKE GROUND MOTION

C2.3.1 Ground Motion

The ground motion due to an earthquake is resolved into three orthogonal directions, two in horizontal direction and one in vertical direction. In general, the vertical acceleration component is much lower than the horizontal acceleration components. Further, the factor of safety for gravity loads, e.g., dead and live loads, is usually sufficient to cover the earthquake induced vertical accelerations. Thus, safety during horizontal acceleration is the main concern in seismic design of normal structures.

C2.3.2 Response of Structure

The ground motion affects the response of structures in a number of ways. The ground motion induces inertia forces to the structure. The soil on which the structure is founded may settle, liquefy or slide due to earthquake ground motion. This code assumes that soil on which the structure is founded does not settle, slide or liquefy during an earthquake. This assumption shall be verified by proper geotechnical investigation, at least for important buildings.

C2.3.3 Soil-structure Interaction

If there is no structure, motion of the ground surface is termed as free field ground motion. In normal practice, the free field motion is applied to the structure base assuming that the base is fixed. But this is valid only for structures on rock sites. It may not be an appropriate assumption for soft soil sites. Presence of a structure modifies the free field motion since the soil and the structure interact, and the foundation of the structure experiences a motion different from the free field ground motion. Soil structure interaction (SSI) accounts for this difference between the two motions. The soil structure interaction generally decreases lateral seismic forces on the structure, and increases lateral displacements and secondary forces associated with P-delta effect. For ordinary buildings, the soil structure interaction is usually ignored.

SSI is not to be confused with site effects. Site effects refer to the fact that free field motion at a site due to a given earthquake depends on the properties and geological features of the subsurface soils also.

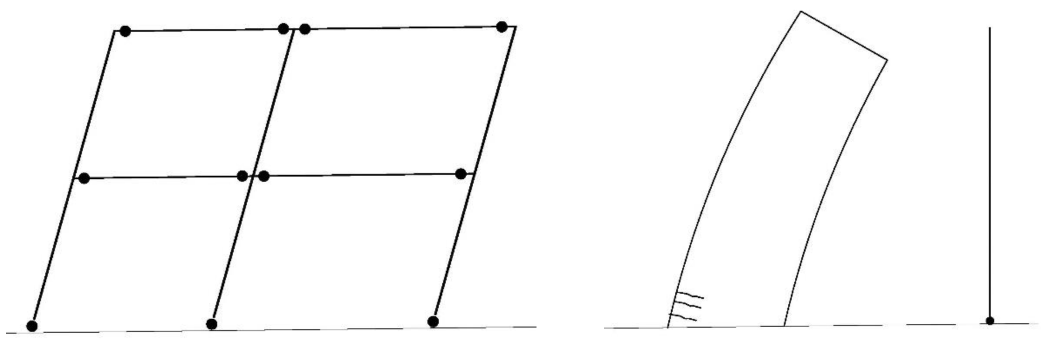
C2.4 CAPACITY DESIGN

The objective of capacity design is to ensure that in the event of a major earthquake (i.e. bigger than the design level earthquake used herein) a ductile failure mechanism can develop, which will enable the structure to survive the earthquake without collapse. This process requires the designer to select a suitable ductile failure mode and then proportion the structure so that other non-ductile failure modes cannot develop.

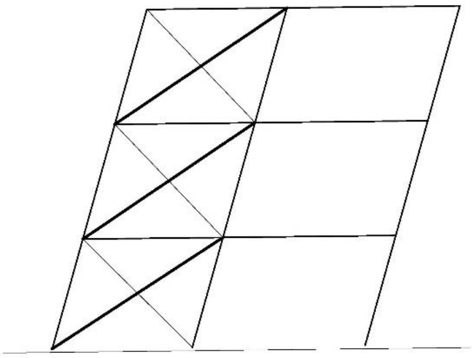
With this arrangement, the strength of the potential inelastic zones limits the structural actions imposed on the other structural members or zones of members.

C2.4.1 Potential Plastic Zones

For multi-story buildings, where the lateral force resistance is provided by moment resisting frames, the selected potential ductile failure mechanism is generally based on the beam-sway mode (figure 2-1 (a)). For buildings where lateral resistance is provided by walls, the selected failure mechanism generally involves the development of plastic hinges at the bases of the walls (figure 2-1 (b)). For buildings, where lateral resistance is developed by braced frames, the failure mechanism involves the braces (figure 2-1 (c)) or eccentric links in the beams between the offset braces (figure 2-1 (d)). A key part of capacity design is to identify the potential inelastic zones and then detail these zones so that they can resist the required deformation without significant loss of strength.



(a) (b)



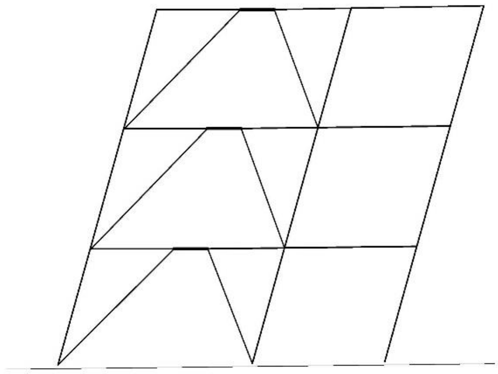


Figure 2-1 Potential plastic hinge locations

C2.4.2 Level of Detailing

The magnitude of the deformation that an inelastic zone can sustain depends on the level of detailing that is used. For example, the deformation that can be sustained by a steel beam depends upon how potential buckling of the inelastic zone is controlled. Thus the required level of constraint against buckling increases with the magnitude of the inelastic deformation that the zone is required to be capable of sustaining.

C2.4.3 Overstrength Actions

To ensure that the intended ductile failure mechanism develops in preference to other failure mechanisms, the maximum likely strength, known as the Overstrength, that each potential inelastic zone can sustain is evaluated. Material Standards define how these overstrengths are calculated. In this calculation, the Over strength should be assessed from the combinations of actions, which allows the most critical actions to be transmitted to the adjacent zones.

C2.5 BASIC ASSUMPTIONS

(a) The probability of occurrence of strong earthquake shaking is low. So is the probability of strong winds. Therefore, the possibility of strong ground shaking and strong wind occurring simultaneously is very low. Thus, it is commonly assumed that earthquakes and winds of very high intensity do not occur simultaneously.

(b) It is difficult to precisely specify the modulus of elasticity of materials such as concrete, masonry and soil because its value depends on factors such as stress level, loading condition (static versus dynamic), material strength and age of material. For such materials, there tends to be large variation in the value of E. Further, the actual concrete strength will be different from the specified value. Modulus of elasticity of masonry has even larger variation than that for concrete. Hence, the code simply allows the modulus of elasticity for static analysis to be used for earthquake analysis also.

PART 2 STRUCTURAL ANALYSIS AND DESIGN

C3. Scope of Analysis

C3.1 STRUCTURAL ANALYSIS METHODS

The simplest and readily-applied method for determining and distributing earthquake induced loads is the Equivalent Static Method. This method should only be applied to simple, regular structures which do not have marked changes in the mass/stiffness ratios of the individual floors.

Other structures which will perform in a less predictable manner should be analyzed using a dynamic method such as the Modal Response Spectrum Method or Numerical Time History Analysis Method. Variations in a structure may cause the horizontal floor deflections and accelerations (and hence the earthquake-induced inertia loads) to be irregular, and hence it is impossible to predict accurately using an empirical shear distribution.

C3.2 APPLICABILITY OF ANALYSIS METHODS

C3.2.1 Equivalent Static Method (ESM)

The Equivalent Static Method is permitted to be used in building having height less than 15 m or time period less than 0.5 seconds regardless of irregularity. Even though higher modes may contribute to the overall response of the structure, regular structures up to 40 meters height are permitted to be analyzed using equivalent static method.

This method provides a simple way to incorporate the effects of inelastic dynamic response into a linear static analysis. This method is applicable to structures without significant discontinuities in mass and stiffness along the height, where the dominant response to ground motions is in the horizontal direction without significant torsion.

C3.2.2 Modal Response Spectrum Method (MRSM)

In irregular structures, higher mode actions have significant influence in the overall performance of the structure. These higher mode actions cannot be modeled effectively by equivalent static method only. Modal response spectrum method is used for capturing these higher mode actions.

C3.2.3 Elastic Time History Analysis

The elastic time history analysis method is included in this version of the code as an alternate to modal response spectrum method. The elastic time history analysis is used as a basis for structural design rather than to predict the behavior of the structure. One of the major advantages of this method over the modal response spectrum method is that signs of the output forces such as bending moments, shear forces and axial forces are preserved. In modal response method, these signs are lost in performing SRSS and CQC combination methods.

C3.2.4 Non-linear Methods

In the present version of the code, apart from the abovementioned linear elastic analyses, non-linear methods (such as non-linear static analysis and non-linear time history analysis) are also introduced. These methods will enable the engineers to verify the performance of existing or retrofitted structures.

Non-linear dynamic analysis is likely to provide the most realistic representation of how a structure will perform during severe earthquake shaking.

C3.3 APPLICATION OF SEISMIC FORCES

Earthquake ground motions invariably have orthogonal components, but the peak ground acceleration does not occur simultaneously in two orthogonal horizontal directions. Hence, this clause permits to consider the design ground motions separately in each perpendicular direction. If at a given instant, the ground motion is in any direction other than orthogonal direction, one can resolve it into the two orthogonal directions, and the building is assumed to be able to resist the oblique ground motion if it is designed for the two orthogonal directions separately.

C3.4 EFFECTIVE STIFFNESS OF CRACKED SECTIONS

Cracking is unavoidable in reinforced concrete structures as well as masonry structures. The presence of cracks complicates the determination of the stiffness (flexural, torsional or axial) of a reinforced concrete member (figure 3-1).

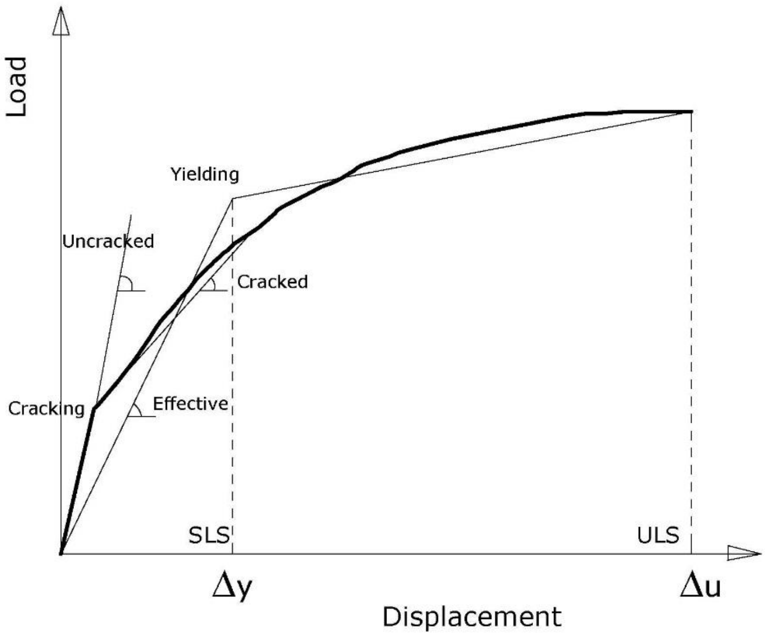


Figure 3-1 Load displacement plot of RC members

Accordingly, certain simplified assumptions are made for the estimation of cracked stiffness. A rational analysis is recommended to be performed in arriving at the elastic flexural and shear stiffness properties of cracked concrete and masonry elements.

C3.5 DESIGN METHODS

Previous version of the code has permitted to use Working stress method as well as limit state method for design of structures. The current version now permits only the Limit State Method (LSM).

C3.6 LOAD COMBINATIONS FOR LIMIT STATE METHOD

C3.6.1 Load Combinations for Parallel Systems

The load combination has been completely revised in the current version of the code. In the previous version, load combination for gravity loading (dead load and live load) was not included. In the proposed load combination, load factors of 1.2 and 1.5 are assigned to dead load and live load, respectively. As dead load tends to be more predictable and uniform, a smaller load factor is adopted. On the other hand, as live load tends to be more variable a relatively larger load factor is adopted.

Where seismic load effect is combined with dead and live load effects, a load factor of unity is assigned to the seismic load. This is different from the load combination factor in previous version of the code where a load factor of 1.25 was used for seismic load. The intention of application of load factor to various load effects is to avoid the failure of structural elements under maximum loads likely to occur during the building's economic life. However, this approach is not appropriate for modern seismic design approach which is based on ductility. It is illogical to assign a seismic load factor greater than unity to a seismic force that has already been reduced from the level corresponding elastic response in lieu of ductility. Applying a seismic load factor greater than unity merely implies a reduction of expected ductility requirement.

C3.6.2 Load Combinations for Non- Parallel Systems

In buildings with non-orthogonal lateral load resisting systems, the lateral load resisting elements may be oriented in a number of directions. Designing for orthogonal directions separately will be un-conservative for elements not oriented along orthogonal directions.

A lateral load-resisting element (frame or wall) offers maximum resistance when the load is in the direction of the element. But in structures with non-orthogonal lateral load resisting systems, it may be tedious to apply lateral loads in each of the directions in which the elements are oriented. For simplicity, the building may be designed for the simultaneous effects due to full design earthquake load in one direction plus 30 percent of design earthquake load along the other horizontal direction.

C4. Seismic Hazard

C4.1 ELASTIC SITE SPECTRA FOR HORIZONTAL LOADING

C4.1.1 Elastic site spectra

The elastic site spectra C (T) for Nepal has been derived from results of a

Probabilistic seismic hazard analysis carried out as a part of this NBC 105 revision project.

The elastic site spectrum for horizontal loading, C(T), is defined as the product of the spectral shape factor, $C_h(T)$, seismic zoning factor, Z and the importance factor. The spectral shape factors $C_h(T)$ for each of the site subsoil classes are normalized by the codified peak ground acceleration for rock. The zoning factor Z is a mapped quantity calculated using probabilistic seismic hazard model which loosely corresponds to the horizontal peak ground acceleration at the bed rock which has a 10% probability of occurrence in 50 years (i.e. which has a return period of 475 years). Zoning factor, Z multiplied by $C_h(T)$ produces the code representation of the 475-year spectrum for the location and site conditions. The importance factor I is the multiplication factor required to produce the code representations of the spectra for return periods other than 475 years, as required for the serviceability limit state or for the ultimate limit state for various combinations of building functionality and design working life.

C4.1.2 Spectral Shape Factor, C_h (T)

Two types of the spectral shape factors, $C_h(T)$ are defined for the equivalent static method and for the modal response spectrum (MRS) or nonlinear time history analysis methods. For the modal response spectrum and non-linear time history analysis, $C_h(T)$ is defined in terms of smooth approximations to the shapes of the estimated hazard spectra for the various site classes, which includes an ascending branch in the low period range (below 0.1s for Soil Categories A,B and C and below 0.5s for Soil Category D) followed by a plateau of a constant maximum value and then finally a nonlinearly descending branch. For the equivalent static method, the sharp ascending branch at short periods is ignored and the constant maximum plateau starts at 0.0s. This is to overcome problems with estimating short fundamental periods accurately, where a small under prediction of the estimated period can otherwise lead to large reduction of design forces (thereby potentially leading to an unsafe structure).

The corner period T_c at the long-period end of the plateau at the peak of the spectrum depends on the site class. For soil type A, B, C and D, the corner period T_c are 0.5 s, 0.7 s, 1.0 s and 2.0 s respectively. For very soft soils found in the lakebed of Kathmandu valley (Soil Category D), the longer range of the maximum spectral plateau is in line with the larger spectral acceleration at longer periods observed in ground motions recorded in 2015 Gorkha earthquake and the subsequent aftershocks (Figure 4-1) .

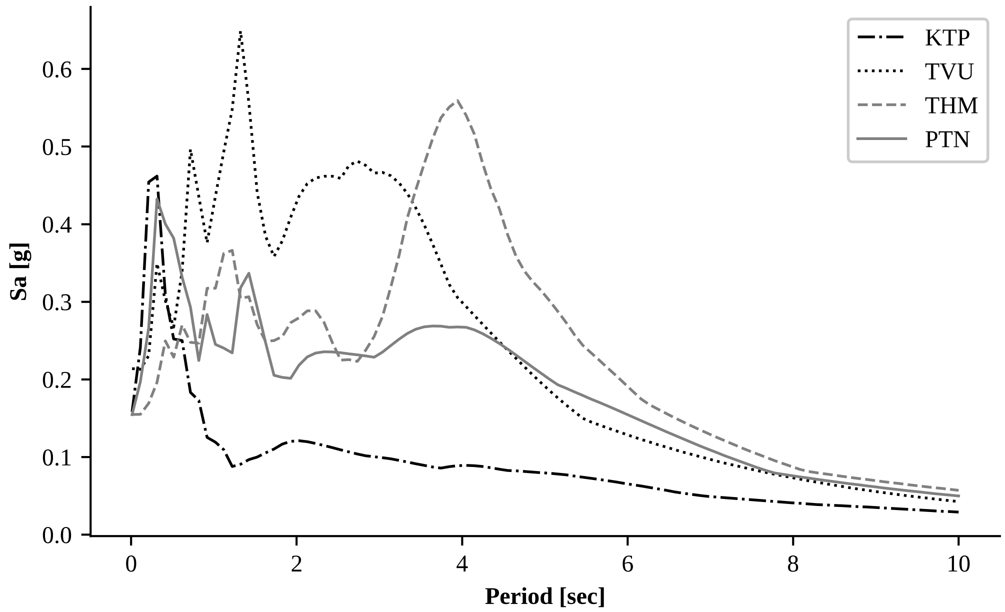


Figure 4-1 Response spectrum of Gorkha earthquake at difference recorded stations

C4.1.3 Site Subsoil Category

Four types of sub soil category are proposed. Very soft soil category is added in addition to previous three categories. This new soil category represents a very soft soil found in Kathmandu valley core where there is deep deposit of clay. The soil types are as follows:

- Stiff or hard soil sites
- Medium soil sites
- Soft soil sites
- Very soft soil sites

Site class definitions consider both soil type and depth, which determine a site's dynamic stiffness and period. These in turn are major factors in determining the site's dynamic response characteristics, along with the impedance contrast with underlying rock, the damping of the soil, and its degree of nonlinearity.

Seismic motions at the surface of a soil deposit can have significantly different characteristics from motions at the underlying bedrock and different types of soil deposits modify the bedrock motions differently. Depending on the depth, shear modulus and plasticity of the soil deposit as well as the intensity, frequency content and duration of the bedrock motions, the seismic motions can be amplified or de-amplified at the ground surface. The local site effect is acknowledged universally in most seismic design codes, but different codes account for this effect differently.

Codes such as IS 1893 (2016), NZS1170.5 (2016), EC8 (2004) and ASEC 7-16 (2016) currently considers a hard to soft soil hierarchy in terms of expected spectral acceleration response. In other words, the specified spectral shape factor reduces as the soil gets harder. For any value of natural period, the elastic design demand for a soft soil is either equal to or greater than (more than three times at some periods) than for a harder soil. However, this is in contrast with the basic structural dynamic principle that stiffer systems attract greater force.

The origin of the notion that soft soils amplify earthquake motions travelling from the bedrock underneath, which appears to be the basis of the local site effect consideration currently adopted in most seismic codes can be tracked to some reported evidences observed in the previous earthquakes; especially the Mexico City earthquake. Nevertheless, there are several evidences which also indicate higher amplification in a rock than on a soil site; especially at lower periods. One such evidence is the statistical study conducted by Seed et al (1976) using 147 records from the western USA and findings of numerical research conducted by Dhakal et al (2013).

A separate verification of this theory was conducted for the soil site available in Nepal. The study has verified this theory and thus the new elastic site spectra were developed based on this verification.

Detailed study of the soil-site response was conducted using the nonlinear site response analysis program DEEPSOIL (Hashash et al., 2016). For this purpose, a suite of ground motions (20 in numbers) is retrieved from PEER NGA-West 2 database and the methodology used for selecting response spectrum compatible suite of ground motion is described in Jayaram et al (2011) is used for arriving at the best suite of ground motion. Figure 4-2 depicts the suite of ground motion compatible with the UHS at 10% probability of exceedence in 50 years.

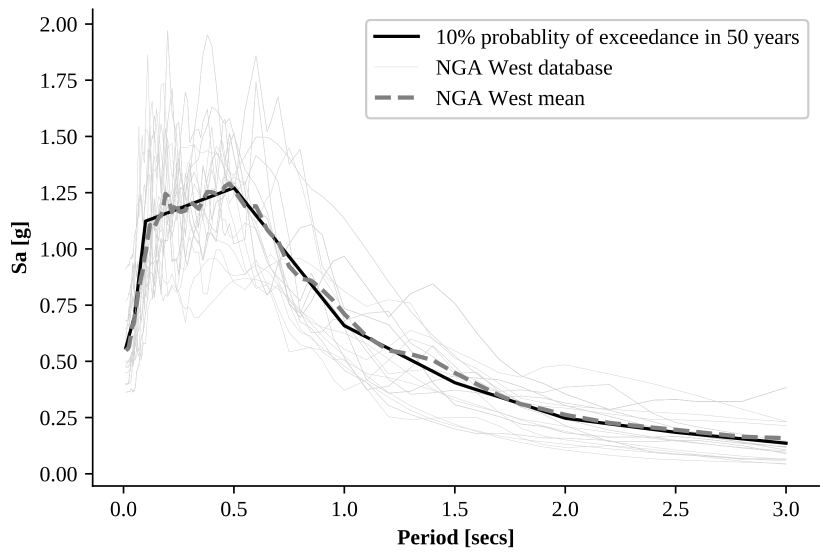


Figure 4-2 Ground motion selection for site response analysis

The response of soil column to a suite of ground motions shown in Figure 4-1 is examined, and observations are depicted in figure 4-2. The average of a suite of ground motion assumed to be recorded at bedrock is denoted by Mean of bedrock in Figure 4-2.

The response of the soil column to each of the ground motion is evaluated and the ratio of free field ground motion to bedrock ground motion termed as amplification factor at each period of interest is evaluated whose average is computed and it is denoted by *Mean of amplification factor.* The UHS at 10% probability of exceedence evaluated at the bedrock is denoted by *10% probability of exceedence in 50 years* and the influence of mean of amplification on this is observed which is represented by *Modified 10%* probability of exceedence *in 50 years*. This *Modified 10% probability of exceedence in 50 years* is represented by a mathematical equation to arrive at the design response spectrum.

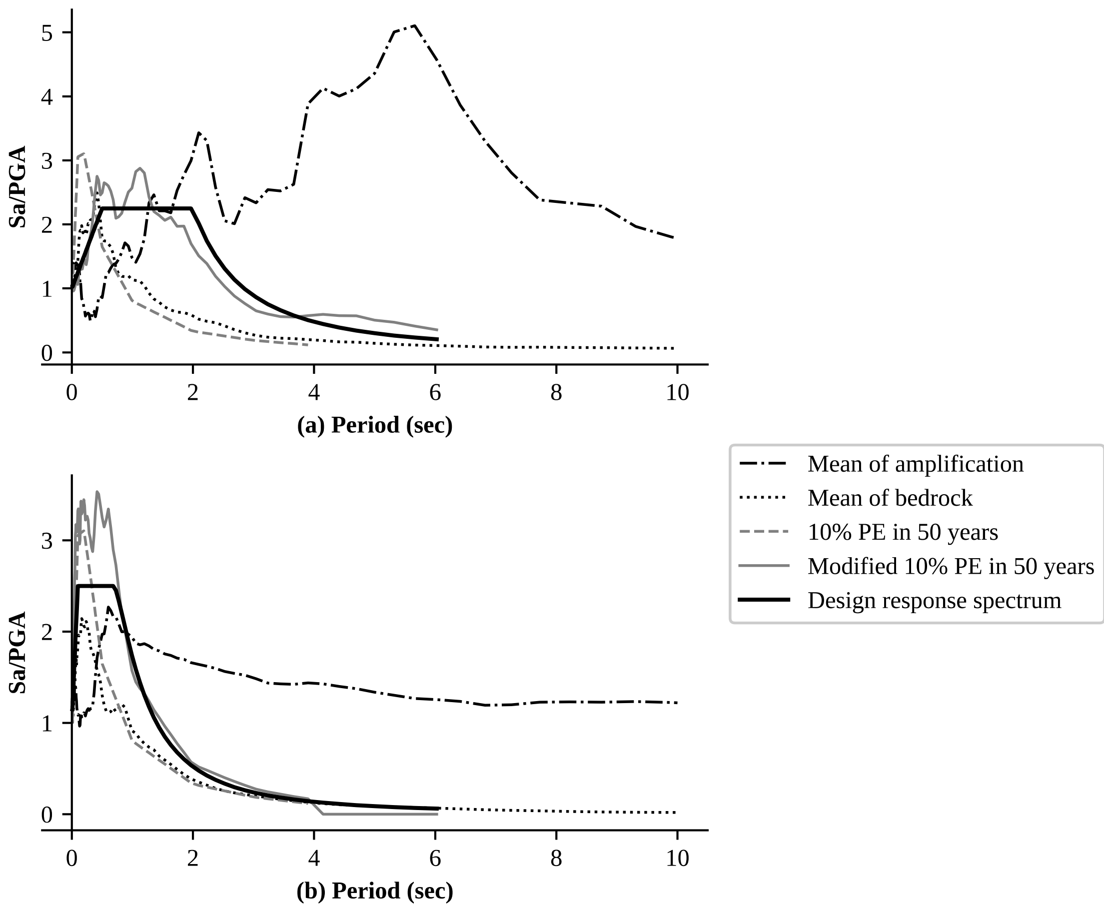


Figure 4-3 Site response results (a) Site D; (b) Site B

The analysis showed that indeed, larger amplification at shorter periods is seen in hard soil sites (Site B) in contrast to the present assumption of higher amplification in soft soil sites.

C4.1.4 Seismic Zoning Factor (Z)

The Seismic Zoning Factor (Z) represents the peak ground acceleration (PGA) for 475 year return period. It corresponds to the value in g of peak ground acceleration.

The values of Z were determined by probabilistic seismic hazard analysis carried out for under NBC 105 revision project. Contour map of the zoning factor for the whole country was developed as a result of the seismic hazard analysis. The exact value of Z for any specific location can be easily determined from the contour map. Alternatively, values of Z have also been listed in tabular form for selected cities and municipalities. Its range varies from 0.25 to 0.4.

C4.1.5 Importance Classes and Importance Factor (I)

Structures are now explicitly categorized into three Importance classes depending on the consequences of their loss of function. The importance classes are characterized by the Importance factor, I. Importance factor of 1.0 is associated with 475 year return period. Higher values of I represent return periods longer than 475 years. The highest value of importance factor equal to 1.5 is assigned to structures and facilities which are required to remain functional during and after the design earthquake. Similarly, structures which may be used as shelter after an earthquake is also assigned the highest importance factor. Importance factor equal to 1.25 are assigned to important structures that are not necessary to remain functional during and after the design earthquake. However, if there is a necessity to make it functional then the same structure can be designed using an importance factor of 1.5.

C4.2 ELASTIC SITE SPECTRA FOR SERVICEABILITY LIMIT STATE

The elastic site spectrum for Serviceability Limit State is taken as 20% of the elastic site spectra for the Ultimate Limit State.

C4.3 ELASTIC SITE SPECTRA FOR VERTICAL LOADING

In general, vertical ground motions are less intense than the horizontal ground motions. The value of peak vertical ground acceleration generally varies from 1/2 to 2/3 of peak horizontal ground acceleration. A value of 2/3 is adopted here in this code which is based on the work on spectral shapes by Newmark and Hall.

C5. Dynamic Characteristics of Structures

C5.1 PERIODS OF VIBRATION

The code envisions estimating the time period of a building structure using Rayleigh method. Although this is a rigorous method, it may not always to feasible to calculate the time period by this method. Hence, approximate empirical equations have also been proposed so that the engineers can estimate the time period for preliminary initial calculations of time period.

Even though, approximate methods give conservative estimates of time period in general, there are cases when these calculated time period are more than those calculated from Rayleigh method. In such cases, the code proposes to adopt the conservative values among the two methods.

C5.1.1 Rayleigh Method

The Rayleigh method is widely recognized as a method for predicting the fundamental period of vibration of a structure since it is based on methods of structural dynamics and utilizes the actual material and member properties to form a structural stiffness matrix. The method also determines the modal shape and can be used to determine the second and third natural periods and their mode shapes. Nevertheless, there are also a number of empirical methods that have been suggested for use in determining the fundamental natural period of structures. These methods are approximate only since they do not take account of the actual shape and properties of each structure.

C5.1.2 Empirical Equations

A number of approximate empirical methods to assess the fundamental period of the structure have been developed. These approximate methods do not use material and section properties appropriate to the limit state under consideration. Hence it should be noted that these approximate methods can be used for initial estimates in preliminary design, or in structural assessment.

C5.1.3 Amplification of Approximate Period

Generally, the approximate empirical equations predict the fundamental periods that are lower than the corresponding values calculated by analytical methods.

Therefore, to account for this low period values, the approximate fundamental are increased by a factor of 1.25.

C5.2 SEISMIC WEIGHT

The aim of the seismic weight provisions is to estimate the likely weight of the structure at the time of an earthquake. It is apparent that the probability of achieving the maximum design live load simultaneously with the design earthquake forces is extremely low. These provisions allow for a reduction in the live load, but recognize that for high design live loads (e.g., storage), the actual level of live load is likely to be higher as a percentage of the design level than for the normal usage cases (e.g., offices).

The probability of having any live load on roofs at the time of a severe earthquake is considered negligible.

C5.3 DUCTILITY FACTOR

The choice of the structural ductility factor carries with it requirements for design and detailing of the system and these requirements must be met to ensure that the anticipated level of inelastic demand can be reliably sustained. With brittle and nominally ductile structures the inelastic deformation required is such that the normal detailing rules are usually sufficient to enable the required material strain levels to be sustained. The ductility factor gives a measure of the ductility of the structure as a whole.

It is generally accepted that it is not economically feasible to design most structures to resist earthquake-induced loads elastically. For example, a structure subjected to the El Centro 1940 earthquake and having 10 percent damping and a natural period of 0.5 s would have an elastic acceleration response of approximately 0.6 g (see Figure 5-2).

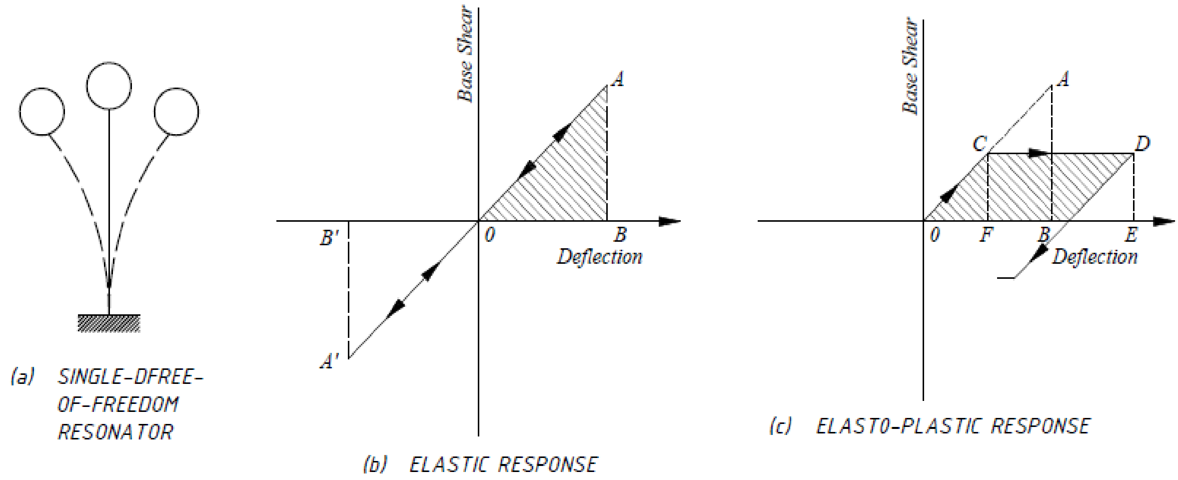


Figure 5-1 Idealized response of a single degree of freedom oscillator

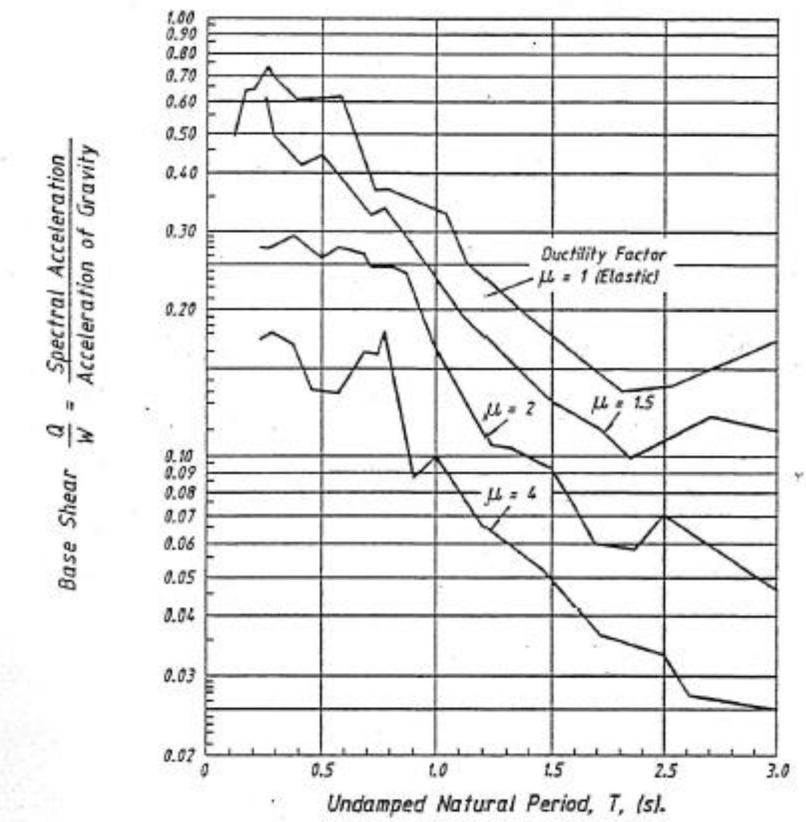


Figure 5-2 Acceleration spectra for Elasto-Plastic systems with 10% critical damping EL Centro earthquake

A Code design load level is therefore selected which will ensure a structure will not be damaged in small or moderate events. However, for large infrequent earthquakes, reliance is placed on the structure performing in a ductile manner to dissipate the earthquake energy and limit the seismic forces.

To illustrate this principle, consider the performance of a single-degree-of-freedom structure shown in Figure 5-1 (a). The purely elastic response is shown in Figure 5-1 (b) where the shaded area OAB underneath the curve is a measure of the stored potential energy when the structure is deflected to deflection B. As the structure vibrates from A through position 0, the energy is converted to· kinetic energy and then back to stored energy at position A ', Consider now the performance of the same resonator if a plastic hinge is allowed to form so that the structure performs in the simplified idealized manner shown in Figure 5-1(c). Point C represents the shear associated with the plastic hinge moment capacity. Instead of the structure responding by deflecting to its full elastic deflection of A, it will proceed along the line C-D until it comes to rest momentarily at D. The velocity energy at 0 has been transformed into stored energy as represented by the area OCDE and the shear has been limited by formation of the plastic hinge. A measure of this ability of the structure to store and dissipate energy is the ratio of the maximum displacement E to the yield displacement F which is termed the displacement ductility factor (μ).

The total stored energy at the position of maximum deflection is OCDE. However, as the structure returns to the no-load position, only the portion of energy GDE is returned as velocity energy. This is in contrast to the elastic response where all the stored energy is returned as velocity energy. This elasto-plastic behavior is the basis for the reserve energy technique used for the ductile design of structures.

The relationship between E and B in Figure 5-1(c) depends on the natural structural period of the structure. If the period is greater than 0.7 s, analyses have shown that E is approximately equal to B (i.e., the deflection of the equivalent elastic structure is approximately equal to that of the elasto-plastic structure). This is referred to as the Equal Displacement Principle.

For periods less than 0.3 s, analyses have shown that an Equal Energy Principle applies. That is, the area OAB is equal to the area OCDE.

Using the Equal Displacement Principle, the deflections of the elasto-plastic structure can be estimated by multiplying μ by the displacement at first yield (Δ_y) or as the displacement of the equivalent elastic structure under a lateral load equal to μ multiplied by the load at first yield in the elasto-plastic structure.

Using the Equal Energy Principle, the deflections of the elasto-plastic structure (with ductility capacity μ) can be estimated as the displacement of an equivalent equally stiff elastic structure under a lateral load equal to the product of $\sqrt{2\mu - 1}$ and the load at first yield in the elasto-plastic structure.

The importance of a structure performing in a ductile manner is shown in Figure 5-2 which illustrates the performance of an elasto-plastic single-degree-of-freedom oscillator with 10 percent critical damping when subjected to the 1940 E1 Centro record. The marked reduction in the base shear demand as the ductility factor increases is quite evident.

This philosophy applies to all buildings (although in a more complex manner than represented in Figure 5-1 (a)), with many hinges forming throughout the structure when it is subjected to loads in excess of the Standard-specified level.

It is generally accepted that a ductile structure should be capable of deflecting at least four times the deflection at first yield (i.e., a ductility factor of four) without significant loss of strength. The designer's attention is, however, drawn to the fact that the ductility required of individual structural members will be significantly in excess of the overall structural ductility (Paulay & Priestley, 1992) indicates that member ductility in regular frames may be at least four or five times the structure ductility. Member detailing requirements required by this Standard are specifically included to ensure that, in the majority of cases, adequate member ductility are available for the structure to possess sufficient ductility to cope with large earthquakes in excess of the Standard-specified force levels.

The condition set for satisfactory ductility (i.e., the maintenance, without significant loss, of vertical and lateral load-carrying capacity when subjected to the Standard-specified deflections for several reversals) may be assumed to be satisfied if the requirements of Table 5-2 are complied with. If, however, the designer wishes to carry out an experimental study to prove the ductility capacity of a structure, the above criteria should be met throughout at least four complete reversals of the lateral deflection.

The deflection criterion is not the only factor to be considered. A loss in the energy dissipation capacity of the structure, evidenced by a pinched load-deflection curve (i.e., a reduction in the area within the load-deflection curve under an increasing number of reversals), will also reduce the effectiveness of the seismic-resisting system.

For example, consider Figures 5-3 (a) and (b). Figure 5-3 (a) shows the type of load-deflection characteristic that is to be encouraged. Figure 5-3 (b), however, shows the less desirable pinched characteristic, with a resulting loss in energy dissipation.

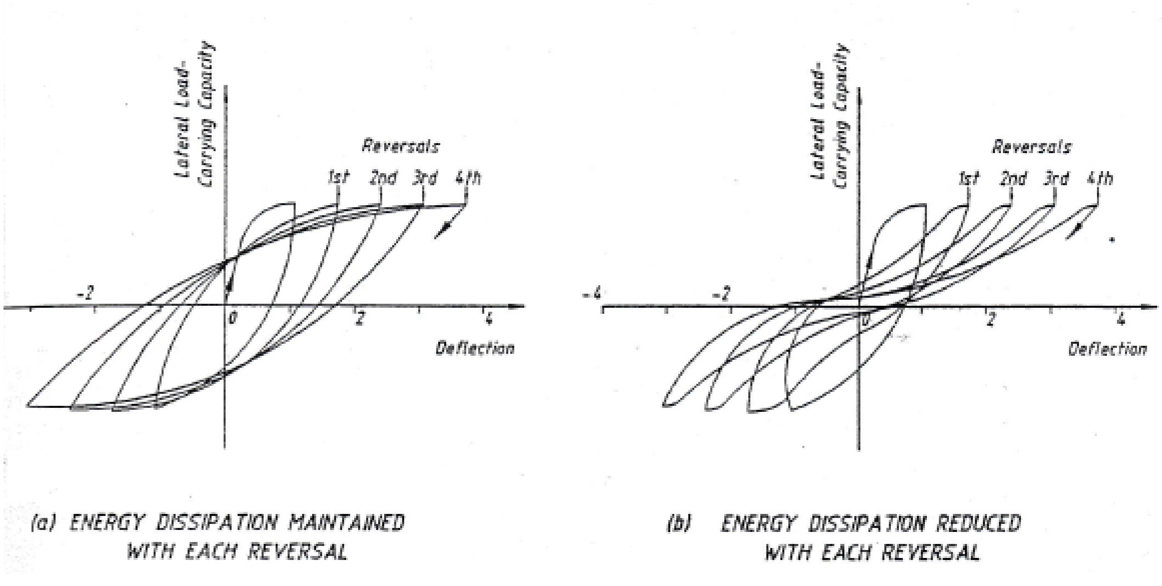


Figure 5-3 Typical load deflection characteristics

The Ductility Factor (R_s) for serviceability limit state is taken as 1 as the structure is expected to remain in elastic range for the considered level of site spectra for this limit state.

C5.4 OVERSTRENGTH FACTOR

Structures tend to have more capacity than accounted for in the analysis. The primary sources of overstrength are:

- Sequential yielding of critical regions
- Material Overstrength (actual vs. specified yield)
- Strain hardening
- Partial safety factors on loads and materials
- Member selection
- Strength contribution of non-structural elements

C5.5 STRUCTURAL IRREGULARITY

The configuration of a structure can significantly affect its performance during a strong earthquake. Configuration can be divided into two aspects, vertical configuration and plan configuration. Past earthquakes have repeatedly shown that structures which have irregular configurations suffer greater damage than structures having regular configurations. This situation prevails even with good design and construction. These provisions are intended to encourage the designer to design structures having regular configurations.

C5.5.1 Vertical Irregularity

Vertical configuration irregularities affect the responses at the various levels and induce loads at these levels that are significantly different from the predominantly first mode distribution assumed in the equivalent static analysis method.

C5.5.1.1 Weak Story

The problem of concentration of energy demand in the resisting elements in a story as a result of abrupt changes in strength capacity between stories has been noted in past earthquakes.

C5.5.1.2 Soft Story

A moment resisting frame structure might be classified as having a soft story irregularity if one story were much taller than the adjoining stories and the resulting decrease in stiffness that would normally occur was not, or could not be, compensated for.

C5.5.1.3 Vertical Geometric Irregularity

The structure may have a geometry that is symmetrical about the vertical axis and still be classified as irregular because of significant horizontal offsets in the vertical elements of the horizontal force resisting system at one or more levels.

C5.5.1.4 In-Plane Discontinuity in Vertical Lateral Force Resisting Element Irregularity

vertical seismic force-resisting elements at adjoining stories that are offset from each other in the vertical plane of the elements resulting in overturning demands on supporting structural elements, such as beams, columns, trusses, walls or slabs are classified as in-plane discontinuity irregularity.

C5.5.1.5 Mass Irregularity

A structure would be classified as irregular if the ratios of mass to stiffness in adjoining stories differ significantly. This might occur when a heavy mass, such as a swimming pool, is placed at one level.

C5.5.1.6 Other Vertical Irregularities

In addition to the above stated reasons, vertical irregularity is also created by unsymmetrical geometry with respect to the vertical axis of the structure. The structure also would be considered irregular if a smaller dimension exists below a larger dimension, thereby creating an inverted pyramid effect.

C5.5.2 Plan Irregularity

C5.5.2.1 Torsion Irregularity

A structure may have symmetrical geometric shape without re-entrant corners or wings but still be classified as irregular in plan because of the distribution of mass or vertical earthquake resisting elements. Torsional effects in earthquakes can occur even where the centers of mass and rigidity coincide. For example, ground motion waves acting on a skew direction with respect to the building axis can cause torsion. Cracking or yielding in an asymmetric fashion also can cause torsion. These effects also can magnify the torsion caused by eccentricity between the centers of mass and rigidity.

C5.5.2.2 Re-entrant Corners Irregularity

A structure having a regular configuration can be square, rectangular, or circular. A square or rectangular structure with minor re-entrant corners would still be considered regular but large re-entrant corners creating a crucifix form would be classified as an irregular configuration. The response of the wings of this type of structure is generally different from the response of the structure as a whole, and this produces higher local forces than would be determined by application of the Standard without modification. Other plan configurations such as H shapes that have a geometrical symmetry would also be classified as irregular because of the response of the wings.

C5.5.2.3 Diaphragm Discontinuity Irregularity

Significant differences in stiffness between portions of a diaphragm at a particular level are classified as irregularities since they may cause a change in the distribution of horizontal earthquake forces to the vertical components and create torsional forces not accounted for in the normal distribution considered for a regular structure.

C5.5.2.4 Out of plane offset Irregularity

Where there are discontinuities in the lateral force resistance path, the structure can no longer be considered regular. The most critical of the discontinuities to be considered is the out of plane offset of vertical elements of the lateral earthquake force resisting elements. Such offsets impose vertical and horizontal load effects on horizontal elements that are, at the least, difficult to provide for adequately.

C5.5.2.5 Other Plan Irregularities

Where vertical elements of the lateral force resisting system are not parallel to, or symmetrical with respect to, major orthogonal axes, the equivalent static force method of analysis cannot be applied as given and, thus, the structure should be considered to be irregular.

There is a type of distribution of lateral force resisting vertical components that, while not being classified as irregular, does not perform well in strong earthquakes. An example is a core wall type building with the vertical components of the horizontal earthquake resisting system concentrated near the center of the structure. Better performance has been observed when these lateral force resisting vertical components are distributed near the perimeter of the structure.

C5.6 DRIFTS AND DISPLACEMENTS

C5.6.1 Determination of Design Horizontal Deflections

The deformation of a structure should be computed using the assumption that its members are highly stressed just prior to the onset of yielding. Any rational method that includes all significant parameters contributing to deformations (such as the extent of cracking in reinforced concrete frame members, the deformation of joint zones, and the cracking of cover concrete in concrete-encased structural steel frames), may be used.

When allowing for the effects of displacement, it is necessary to estimate the likely deformations during design earthquake for appropriate limit state.

The displacements in the design earthquake can be estimated by multiplying those determined using elastic analysis methods for the structure, when subjected to the design lateral loading by the ductility factor (μ) for ultimate limit state. Where nonlinear static or dynamic analysis methods are used, the design displacements can be directly obtained from the analysis; and no further adjustments are needed.

Whereas, for serviceability limit state, the displacement is equal to the displacement calculated from elastic structural analysis.

C5.6.2 Building Separations

Structures dissipating energy in a ductile post-elastic mode are required to be separated from adjacent buildings in order to avoid hammering (also known as pounding) between them. Such hammering would result in significant damage and a response different to that assumed.

C5.6.3 Inter-Story Deflections

A limit is applied to the inter-story deflection to reduce discomfort to the building occupants and also to reduce secondary moments arising from displacement of the line of the axial load (referred to as the P-Δ effect). This is especially important for flexible structures with heavy axial loads.

C5.7 ACCIDENTAL ECCENTRICITY

Accidental eccentricity of ±0.1b is intended to allow for variations in the calculation of structural properties, variation in distribution of the mass, influence of non-structural components such as partitions in stiffness of the building and also to include the effects of rotation of the ground about vertical axis.

C6. Equivalent Static Method

C6.1 HORIZONTAL BASE SHEAR COEFFICIENT

The inelastic lateral design action coefficients are obtained by dividing the elastic coefficients by the structural ductility factor (R_μ) and Overstrength factor (Ω) through all time periods (Figure 6-1).

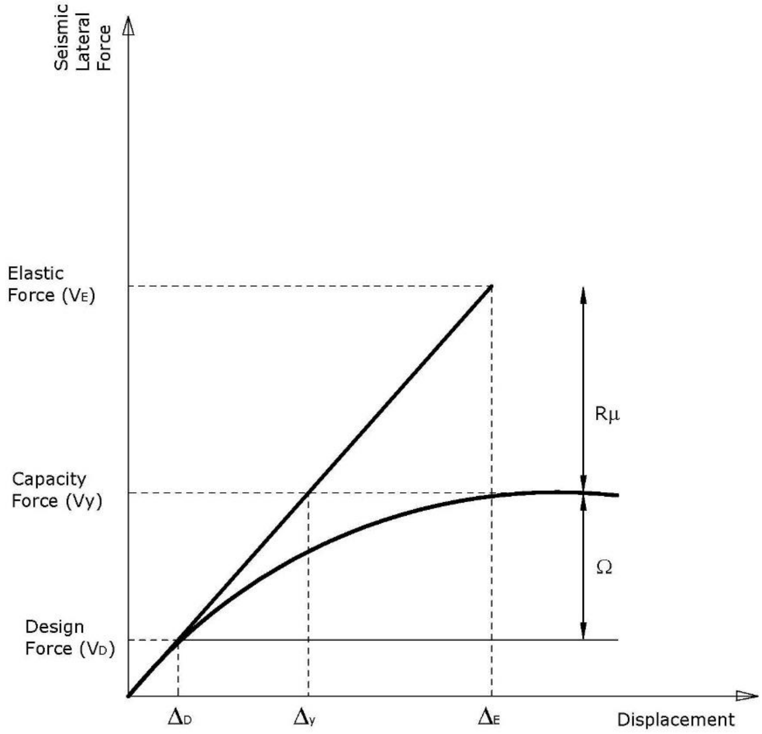


Figure 6-1 Inelastic Force-Deformation Curve

Division by structural ductility for all structural periods is a simplification, essentially based on Equal displacement principle, where it is assumed that inelastic displacements are equal to the displacements that would occur during an elastic response. This is not strictly correct. It is generally accepted that equal energy rule applies to short period structures having time period less than say 0.35 s and that equal displacement rule applies for long period structures having time periods greater than say 0.7 s. A transition zone applies between. Application of equal energy rule has the potential to underestimate the inelastic response for structures with time periods less than 0.7 s. This potential non-conservative approach is considered acceptable given other inaccuracies in the derivation of inelastic response and has been taken for reasons of simplifying the method.

C6.1.1 Ultimate Limit State

The horizontal base shear coefficient (design coefficient), C_d (T_1), is obtained by dividing the elastic site spectra by structural ductility factor (R_μ) and Overstrength factor (Ω).

C6.1.2 Serviceability Limit State

For the serviceability limit state, it is intended that the structure essentially remains elastic. Therefore, structural ductility factor of 1 is used.

C6.2 HORIZONTAL SEISMIC BASE SHEAR

The structure is treated as a single degree-of-freedom system with 100% mass participation in the fundamental mode. The base shear (V) is expressed as a product of the effective seismic weight, W, and the horizontal base shear coefficient, $C_d(T_1)$.

The mass of the structure at or below the level where the ground provides effective horizontal restraint (the base of the structure) is assumed not to contribute to the horizontal seismic shear force at the base of the structure nor at any levels above the base. However, all parts at and below the base should be designed to resist the inertial forces resulting from their masses and the ground acceleration, and the reactions from levels above the base.

C6.3 VERTICAL DISTRIBUTION OF SEISMIC FORCES

The distribution of total horizontal base shear over the height of a building is generally quite complex because these forces are the result of superposition of a number of natural modes of vibration. The relative contributions of these vibration modes to the total forces depends on a number of factors, including shape of the earthquake response spectrum, natural periods of vibration of the structure and shapes of vibration modes which, in turn, depend on the mass and stiffness distribution over the height of the building. This clause provides a reasonable and simple method for determining the horizontal load distribution in buildings with, regular variation of mass and stiffness over the height.

In low and medium rise buildings, fundamental period dominates the response and fundamental mode shape is close to a straight line. For tall buildings, contribution of higher modes can be significant even though the first mode may still contribute the maximum response.

The deformed shape of the structure is a function of the exponent ‘k’. The exponent ‘k’ is intended to approximate the effect of higher modes (figure 6-2).

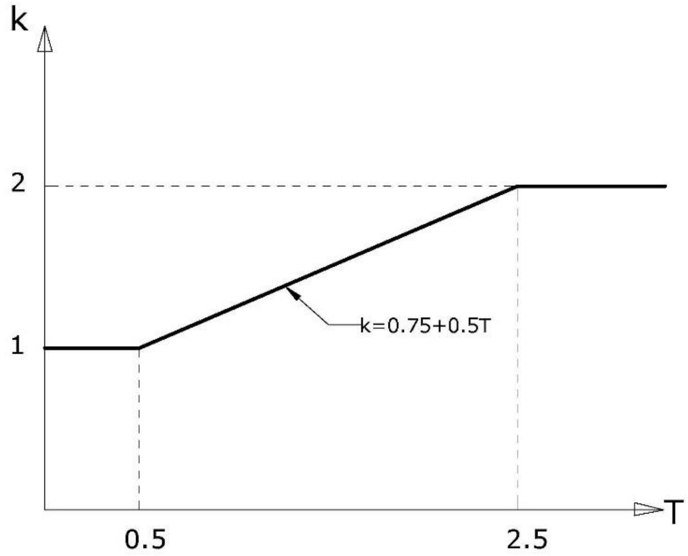


Figure 6-2 Variation of exponent k with time period

C6.4 POINTS OF APPLICATION OF EQUIVALENT STATIC FORCES

It is assumed that the mass of the building is lumped at each floor level. The seismic forces are assumed to act at the center of mass at each floor level.

C7. Modal Response Spectrum Method

C7.1 ULTIMATE LIMIT STATE

The horizontal base shear coefficient is calculated for each of the considered mode separately.

C7.2 CALCULATION OF BASE SHEAR FORCE FOR EACH MODE

The base shear is also calculated for each of the considered mode separately. The vertical distribution of base shear is also calculated separately for each of the considered modes.

C7.3 NUMBER OF MODES TO BE CONSIDERED

Sufficient modes are to be considered so that the summation of effective mass over all modes considered is at least 90% of the total mass. While this results in structural actions and displacements which are slightly less than would be obtained if all modes were included, for practical purposes the difference is negligible.

All modes that are not part of the horizontal load resisting systems shall be ignored in modal combination.

The modal combination shall be carried out only for modes with natural frequency less than 33 Hz; the effect of modes with natural frequencies more than 33 Hz shall be included by the missing mass correction procedure following established principles of structural dynamics.

C7.4 COMBINATION OF MODAL EFFECTS

When the modal responses for different modes are not coupled, the combination may generally be performed according to the Square Root of the Sum of the Squares (SRSS) method:

$$S = \sqrt{\sum_{i=1}^{n} S_i^2}$$

where

S_i = the maximum response quantity in the i th mode of vibration

S = the maximum response quantity under consideration

When the modal responses for different modes are coupled, the combination may be performed using equation for the Complete Quadratic Combination method which is derived from random vibration theory (see Wilson et al., Ref. 10).

$$S = \sqrt{\sum_{i=1}^{n}\sum_{i=1}^{n} S_i \rho_{i,k} S_k}$$

$$\rho_{i,k} = \frac{8\sqrt{\xi_i \xi_k (\xi_i + \xi_k) x^{3/2}}}{(1 - x^2) + 4\xi_i \xi_k x (1 + x^2) + 4(\xi_i^2 + \xi_k^2) x^2}$$

where

ξ_i, ξ_k = the damping ratios for the ith and the kth mode, respectively

x = the ratio of the ith mode natural frequency to the kth mode natural frequency

All modes having significant contribution to the total structural response should be considered for the above Equations.

It is recommended that the CQC method be routinely used as it deals automatically with the problems of closely spaced modes.

C7.5 SCALE FACTOR FOR DESIGN VALUES OF THE COMBINED RESPONSE

The modal base shear (V_R), may be less than the base shear (V) obtained by equivalent static method primarily for two reasons. The fundamental period calculated from modal response spectrum method may be longer than that used in computing V. In modal response spectrum method, the response is not characterized by a single mode, whereas in equivalent static method it is assumed that there will be 100% mass participation in the first mode.

C8. Elastic Time History Analysis

Elastic time history analysis method requires the use of three sets of ground motions, with two orthogonal components in each set. These motions are then modified such that the response spectra of the modified motions closely match the shape of the target response spectrum. Thus, the maximum computed response in each mode is virtually identical to the value obtained from the target response spectrum. The only difference between the Modal response spectrum method and the elastic time history analysis method is that in the former method the system response is computed by statistical combination (SRSS or CQC) of the modal responses and in the latter method, the system response is obtained by direct addition of modal responses or by simultaneous solution of the full set of equations of motion.

C8.1.1 Structural Modeling Requirements

C8.1.1.1 Modeling

Three dimensional models of the structure shall be required for carrying out the analysis.

C8.1.1.2 Gravity Load

Refer to clause 9.3.1 for explanation.

C8.1.1.3 P-Delta Effect

Refer to clause 9.3.1.3 for explanation.

C8.1.1.4 Torsion

Refer to clause 9.3.1.4 for explanation.

C8.1.1.5 Damping

Linear viscous damping shall not exceed 5%.

C8.1.1.6 Below grade Structure elements

Refer to clause 9.3.1.6.

C8.1.2 Ground Motions

Refer to clause C9.3.2 for explanations.

C8.1.3 Evaluation of response quantities

Refer to clause 9.3.3 for explanations.

C8.1.3.1 Inter story drifts

Refer to clause 9.3.3.1 for explanations.

C8.1.3.2 Member strengths

When equivalent static method or modal response spectrum method is used, elastic forces/ stresses are generated. These need to be reduced by the ULS ductility factor R□ for obtaining actual design forces/ stresses.

C9. Non-linear Static and Dynamic Analysis

C9.1 GENERAL

Actual non-linear behavior of the structural materials and elements are incorporated in the structural analysis model for performing non-linear analysis.

C9.2 NON-LINEAR STATIC ANALYSIS

Nonlinear static analysis converts MDOF models to equivalent SDOF structural models and represent seismic ground motion with response spectra as opposed to ground-motion records. They produce estimates of the maximum global displacement demand. The mathematical model employed in this method accounts directly for effects of material inelastic response, and therefore, the calculated internal forces are reasonable approximations of those expected for the selected Seismic Hazard Level.

C9.2.1 Modeling and Analysis

Gravity loads are to be applied to the nonlinear model first and then lateral load is to be applied. The initial application of gravity load is critical to the analysis, so member stresses and displacements caused by lateral load are appropriately added to the initially stressed and displaced structure.

C9.2.2 Load pattern

The distribution of lateral inertial forces determines relative magnitudes of shears, moments, and deformations within the structure. The actual distribution of these forces is expected to vary continuously during earthquake response as portions of the structure yield and stiffness characteristics change. The extremes of this distribution depend on the severity of the earthquake shaking and the degree of nonlinear response of the structure. More than one seismic force pattern has been used in the past as a way to determine the range of actions that may occur during actual dynamic response. Research in FEMA 440 2005 has shown that multiple force patterns do little to improve the accuracy of nonlinear static procedures and that a single pattern based on the first mode shape is recommended.

C9.2.3 Control node

As the penthouse has a very small footprint and its Center of mass does not in general coincide with the center of mass of the other floors of the building, locating control node at the penthouse will produce erroneous results.

C9.2.4 Capacity curve

The requirement to carry out the analysis to at least 150% of the target displacement is meant to encourage the engineer to investigate likely building performance and behavior of the model under extreme load conditions that exceed the analysis values of the Seismic Hazard Level under consideration. The engineer should recognize that the target displacement represents a mean displacement value for the selected Seismic Hazard Level and that there is considerable scatter about the mean.

C9.2.5 Target displacement

The target displacement is intended to represent the maximum displacement likely to be experienced for the selected Seismic Hazard Level. Because the

There are different methods for computing the target displacement. Capacity spectrum method is documented in ATC-40 1996. Another method known as coefficient method of displacement modification is presented in FEMA 356 2000. A variant of capacity spectrum method known as N2 (Fajfar & Eeri, 2000) Method can also be used. EC8 recommends using N2 method for calculating the target displacement.

C9.3 NON-LINEAR TIME HISTORY ANALYSIS

Non-linear time history analysis shall be carried out through direct numerical integration of the differential equations of ground motion acceleration time histories. The numerical integration time history analysis may be used for all types of structures to verify that the specific response parameters are within the limits of acceptability assumed during design.

Non-linear time history analysis can be used to assess compliance of one or more of the following properties:

a. The strength requirements of the structure are satisfied.

b. The deflections of the structure and its parts do not exceed design values.
c. The ductility demands imposed on members are within acceptable limits as specified in the appropriate material Standard; or
d. The capacity design assumptions regarding the location and distribution of inelastic behavior are consistent with design assumptions.
e. The accelerations and deformations imposed on parts can be ascertained.
f. Any combination of the above.

This procedure can be used to validate design or response assumptions. Its results take precedence over the more general prescriptive requirements which are often introduced in less precise forms of analysis as a simplistic allowance for the complex behavior of a building responding to an earthquake.

Typically, a three-dimensional model of the building will be required. However the method is applicable to two-dimensional analyses if the building is regular in plan, with allowance made for torsional effects.

Non-linear time history analysis provides the ability to model the earthquake effect on the building as it occurs in practice; i.e. the earthquake effects are introduced as input motions at the base of the structure, generating displacements throughout the structure which in turn generate the action effects. It allows the influence of inelastic action in selected elements on the overall structural response to be realistically determined. It also allows the influence of different seismic input motions (e.g. due to soil structure or near-fault effects) on the structural response to be more realistically determined than is possible from the equivalent static or modal response spectrum methods.

The accuracy of output is much more critically dependent on the accuracy of input than for the other methods.

C9.3.1 Structural Modeling Requirements

C9.3.1.1 Modeling

Nonlinear time history analysis is a very powerful analysis method. It has the ability to model a wide variety of nonlinear material behaviors, geometric nonlinearities (including P-delta and large displacement effects), gap opening and contact behavior, and nonlinear viscous damping, and to identify the likely spatial and temporal distributions of inelasticity. However, this method requires larger effort to develop the analytical model, increased time to perform the analysis (which is often complicated by difficulties in obtaining converged solutions), sensitivity of computed response to system parameters, large amounts of analysis results to evaluate, and the inapplicability of superposition to combine live, dead, and seismic load effects. Because the goal of nonlinear response history analysis is to accurately predict the building's probable performance, it is important to include the effects of gravity-load-carrying system and some nonstructural components can add significant stiffness and strength in the analytical model and also to verify that the behavior of these elements will be acceptable

Expected material properties are used in the analysis model, attempting to characterize the expected performance as closely as possible. It is suggested that expected properties be selected considering actual test data for the proposed elements.

The key parameters involved in structural modelling and program execution are the selection of a hysteretic model to represent inelastic cyclic response, damping, elastic and post-elastic strength and stiffness, rigid end blocks, time step and P-delta effects. The hysteretic model needs to realistically represent the important physical characteristics of the element under consideration, for the extent of inelastic demand expected. Important characteristics include:

(a) The rate of strength increase with increased displacement demand in the inelastic range (i.e. the post-elastic stiffness).

(b) The behavior on unloading and displacement reversal (i.e. the unloading stiffness).

(c) The degradation in stiffness and strength expected over successive cycles of loading.

(d) Any development of slackness expected over successive cycles of loading.

C9.3.1.2 Gravity Load

Gravity loads should be determined in a manner consistent with the determination of seismic mass. Gravity loads are to be applied to the nonlinear model first and then ground shaking simulations applied. The initial application of gravity load is critical to the analysis, so member stresses and displacements caused by ground shaking are appropriately added to the initially stressed and displaced structure.

C9.3.1.3 P-Delta Effect

P-delta effects should be realistically included, regardless of the value of the elastic story stability coefficient. When including P-delta effects, it is important to capture not only the second-order behavior associated with lateral displacements but also with global torsion about the vertical axis of the system. Additionally, the gravity load used in modeling P-delta effects must include 100% of the gravity load in the structure.

C9.3.1.4 Torsion

Inherent torsion is actual torsion caused by differences in the location of the center of mass and center of rigidity throughout the height of the structure. Where there is already inherent torsion in the building, additional accidental torsion is not generally a crucial requirement because the building model will naturally twist during analysis, and no additional artificial torsion is required for this twisting to occur. However, for buildings exhibiting torsional irregularities, inclusion of accidental torsion in the nonlinear analysis is required to assist in identification of potential nonlinear torsional instability.

C9.3.1.5 Damping

The traditional damping model is the Rayleigh damping model, in which the computed damping is proportional to the mass and stiffness matrices. There are two variations to the stiffness used; either the initial stiffness (Initial Stiffness

Rayleigh damping) or the tangential stiffness (Tangential Stiffness Rayleigh damping).

When using the Rayleigh damping model, the target elastic damping is set at two modes of vibration. Calculated elastic damping for modes between these two modes will be less than the target; outside of these it will be higher. Care must be taken with the Rayleigh method to avoid the influence of higher modes being diminished by artificially high calculated damping values. This is especially the case for irregular buildings, where a high mode may make a significant contribution to the response.

For regular buildings, when the Rayleigh damping model is used, the target elastic damping should be specified at mode 1 and at a mode number equal or slightly less than the number of stories. The Tangent Stiffness Rayleigh damping model is preferred over the Initial Stiffness Rayleigh damping model.

A uniform damping model, in which the target elastic damping is applied for all modes, overcomes the risk of high mode overdamping but requires more computer running time.

The target elastic damping for mode 1 is typically taken at 5. At the higher mode or modes, it should be taken as 5% or as the mode 1 value, if this is less than 5%.

C9.3.1.6 Below grade Structure elements

Guidelines for Performance-Based Seismic Design of Tall Buildings (2010), NIST GCR 11-917-14 Selecting and Scaling Earthquake Ground Motions for Performing Response History Analyses (2011), NIST GCR 12-917-21 Soil-Foundation-Structure Interaction for Buildings Structures (2012) recommend inclusion of subterranean building levels in the mathematical model. Time-domain nonlinear response-history analysis of the soil-structure system can be performed per Figure 9-1 (a) and input motions for this analysis would be applied at a rock outcrop below the building. Two other options are shown in Figures 9-1 (b) and (c). The coarser model of the two is shown in Figure 9-1(b) wherein the ground motions, either free-field surface motions or foundation input motions, are applied at the underside of the basement. A better model, although poorer than that of Figure 9-1 (a) but computationally more efficient, is shown in Figure 9-1 (c) wherein the soil domain in the vicinity of the building is modeled with springs and dashpots and identical horizontal ground motions, either free-field surface motions or foundation input motions, are input at each basement level in the building.

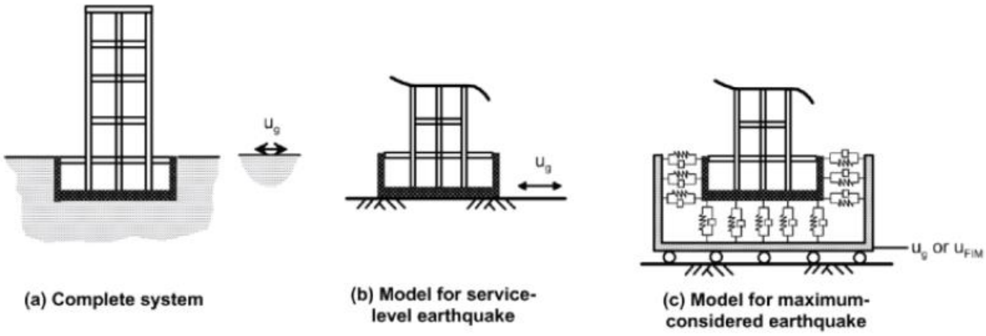


Figure 9-1 Soil-Structure system

C9.3.2 Ground Motions

C9.3.2.1 Number of Ground Motions

The code currently requires using at least three ground motions for 2D analysis and three pairs of ground motions for 3D time history analysis.

Since the design spectra are generally uniform hazard spectra (i.e. an amalgam of the contribution from various events), different records can be expected to significantly contribute over a range of period. The seismological characteristics upon which records are to be selected will generally involve a de-aggregation of the design spectra into at least two period bands so as to establish the seismological signature of records appropriate for use within each band.

Following criteria are used to select the appropriate ground motions:

Source Mechanism: Ground motions from differing tectonic regimes (e.g., subduction versus active crustal regions) often have substantially differing spectral shapes and durations, so recordings from appropriate tectonic regimes should be used whenever possible.

Magnitude: Earthquake magnitude is related to the duration of ground shaking, so using ground motions from earthquakes with appropriate magnitudes should already have approximately the appropriate durations.

Fault Distance: The distance is a lower priority parameter to consider when selecting ground motions. Studies investigating this property have all found that time history analyses performed using ground motions from different site-to-source distances but otherwise equivalent properties produce practically equivalent demands on structures (NIST GCR 11-917-14, 2011).

When the required number of recorded ground motions is not available, it is permitted to generate artificial simulated ground motions.

C9.3.2.2 Scaling of Ground Motions

The scaling procedures are necessary to ensure the ground motion records selected match those intended for design as reflected in the published design spectra and that each record is applied to the building in a manner which reflects the most adverse conditions within the building. The ground motion is represented by two horizontal components in each record. The direction of attack is generally considered to be random, although the relative intensity of the motion within each component should be maintained. The intent of the analysis procedure is that earthquake attack should be considered about the weakest axis of the building so as to ensure adequate strength, ductility and stiffness are provided about all axes.

The procedure outlined matches (as nearly as practicable) the target or design spectrum with the more severe component of each ground motion.

The reduction of the elastic site spectra by the factor Ωu to obtain the target spectra acknowledges that system effects are also present within the actual structure that are not accounted for within the engineering model. Such effects, in combination with the requirement that the model be considered under the combined orthogonal ground motions of the selected records, is expected to result in computer demands and displacements that align more closely to those expected in service.

The determination of scale factor is undertaken over a period range of interest that includes the fundamental period of the building, T1. This fundamental period should be calculated using the response spectrum method or the Rayleigh method. The target spectrum (elastic site hazard spectrum) is a uniform hazard spectrum (i.e. an amalgamation of the contributions from various events with different spectral shapes).

Because of this, no single ground motion record selected will be a close match for the target spectrum over the period range of interest. The period range for scaling of ground motions is selected such that the ground motions accurately represent the target spectra at the structure's fundamental response periods, periods somewhat longer than this to account for period lengthening effects associated with nonlinear response and shorter periods associated with a higher mode response Koopaee et al. (2016).

The lower bound period, Tn is supplemented with an additional requirement that the lower bound also should capture the periods needed for 90% mass participation in both directions of the building. This requirement ensures that when used for tall buildings and other long-period structures, the ground motions are appropriate to capture response in higher modes that have significant response.

Unless the foundation system is being explicitly designed using the results of the time history analyses, the above 90% modal mass requirement pertains only to the superstructure behavior; the period range does not need to include the very short periods associated with the subgrade behavior.

The requirement of scale factor to be between 0.33 and 3 ensures that the amplitude of the selected records are sufficiently similar.

C9.3.2.3 Application of Ground Motions

The most adverse response of a parameter will usually occur when the motion is in one or other of the two orthogonal structural axes. Alternative orientations of the ground motion may be required when the structural axes are not clearly defined or when the dynamic response of the structure when excited about a different axis may produce more severe effects.

C9.3.2.4 Analysis time step

The time step should generally be no greater than T1/100, where T1 is the period associated with the first mode of vibration.

For analyses involving impact (building pounding, rocking walls or uplifting foundations), the time step will need to be significantly lower and a starting value of T1/1000 is recommended.

If convergence is not obtained with a particular time step, reduce by a factor of 2 and re-run. Once convergence is obtained, make a further reduction and compare the peak results for the target response parameter. If they are within 5%, the longer time-step (which requires fewer computers running time) is satisfactory.

C9.3.3 **Evaluation of response quantities**

Less than seven ground motions are not sufficient to accurately characterize either mean response or the record-to-record variability in response. Therefore, for such cases maximum values of the response quantities from these ground motions needs to be used. Large number of ground motions more than 7 serves the objective of predicting more reliable mean structural response quantities.

C9.3.3.1 Inter story drifts

The design inter-story deflection is record dependent and so is based on the maximum value obtained for each record.

C9.3.3.2 Member strengths

The critical inelastic deformation demands that need to be checked are as follows:

1. Hinge rotations in beams and columns leading to significant strength/stiffness degradation
2. Deformations of non-ductile gravity beam-to-column connections
3. Axial deformations (tension/compression) in braces
4. Deformations of non-ductile slab–column connections in reinforced concrete gravity systems
5. Tensile strains in longitudinal wall reinforcement
6. Compression strains in longitudinal wall reinforcement and concrete
7. Flexural hinging or shear yielding of coupling beams
8. Soil uplift and bearing deformations in shallow foundations (when modeled in-elastically)
9. Tensile pullout deformations or compression bearing deformations of pile foundations (when modeled in-elastically)

C10. Parts and Components

C10.1 GENERAL

The satisfactory seismic performance of non-structural elements in a building is as important as the performance of the structure itself for two reasons. Firstly, the non-structural elements of a building may account for a significant proportion of its cost and is therefore worth protecting against earthquake damage. Secondly, the failure of any of these items may present either an immediate threat to the building's occupants, or it may prevent evacuation of the building and/or operation of emergency services (such as fire sprinklers) immediately following an earthquake.

Friction force due to gravity to resist horizontal forces in earthquakes shall be limited as there may be a vertical component of the ground motion which could seriously reduce such friction resistance, thereby allowing the element or system to move under the horizontal forces that are simultaneously acting.

C10.2 SERVICE CUT-OFFS

Some facilities, such as those involved in chemical processing or employing gas supplies or high energy sources, may present an excessive threat to the public unless shut down in large earthquakes. The ATC3 provisions (also known as the NBS 510 code) specify a cut-off at a ground acceleration of 0.2 times gravity in the most seismic areas of California and this has been used as the limit for this requirement. Shut-down should also occur if there is a failure detected within the system (eg, a pressure drop in a process).

C10.3 DESIGN SEISMIC FORCE

The seismic design force for a component is derived as a product of the weight of the component, the component Importance Factor, the component ductility factor, the component amplification factor, and the seismic zoning factor.

The expression $\left(1+\frac{h_p}{H}\right)$ is included to allow, in part, for the amplification of the ground motion by the structure. Some studies indicate that very high amplifications may occur. The given relationship should provide a reasonable estimate of the amplification to be expected, without the calculation being unduly complicated. Designers should be aware that elements in the top portions of structures may be subject to large accelerations.

C10.3.1 Component Amplification Factor

Component amplification factor (ap) represents the dynamic amplification of the component relative to the fundamental time period of the structure. Recent researches have indicated that the amplification of the component response is dependent on the difference between the natural period of the structure and component (Figure 10-1). If the natural period of component is closer to the natural period of the structure, resonance will take place and component response is highly amplified.

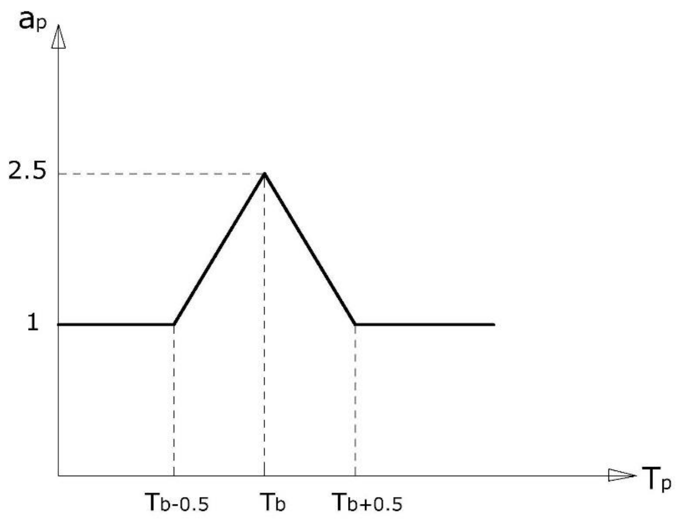


Figure 10-1 Component amplification factor- time period curve

C10.3.2 Component Ductility Factor

Component ductility factor (μp) represents the ductility and energy dissipation capacity of the components and its connections. It represents the energy absorption capability of a component and its attachments depend on both Overstrength and deformability. Components with low deformability are assigned a smaller value. The value of component ductility factor for components with high deformability shall be determined by research.

C10.3.3 Component Importance Factor

The component Importance Factor (Ip) implies performance levels for specific cases.

Components with Importance factor equal to 1.5 are expected to remain in place, sustain limited damage, and when necessary, function after an earthquake. For example, fire sprinkler piping systems are assigned with Importance Factor of 1.5 in all structures because these essential systems should function after an earthquake.

Components with importance factor equal to 1.0, i.e., noncritical nonstructural components are expected to sustain minimal damage in a minor earthquake shaking, some damage affecting its functionality in moderate earthquake shaking and major damage without significant falling hazard and loss of functionality in design earthquake shaking.

.Egress stairways are assigned an Ip of 1.5 as well, although in many cases the design of these stairways is dictated by differential displacements, not inertial force demands.

The component Importance Factor is intended to represent the greater of the life-safety importance of the component and the hazard-exposure importance of the structure. It indirectly influences the survivability of the component via required design forces and displacement levels, as well as component attachments and detailing.

C10.4 OTHER REQUIREMENTS

a) A major deficiency in many seismic designs is in the adequacy of details for connections. The design of these should take into account not only the loads induced directly by an earthquake, but also the effects of interaction with other elements of the structure. Lateral movement of the structure may induce additional loads unless adequate separations are provided. It must be recognized, however, that the specified separations may not be adequate for a very large earthquake. Therefore, it is felt that the connections for ornamentations, veneers, appendages and exterior panels should also be ductile and adequately connected in case the separation gaps close. The closing of separation gaps may induce significant additional loads and may possibly yield the fixings.

b) The seismic weight of containers and the like may significantly influence the performance of the structure, and therefore must be considered in the structural analysis.

c) Hanging or swinging lights present a high risk in that they are readily excited in an earthquake and may move violently. Should this movement cause the fixing to fail, the safety cable will offer protection for the building occupants.

d) Failure of suspended systems has occurred in many earthquakes. This type of damage can be a life hazard to occupants and can add to the potential for panic. The functions of important buildings, such as telephone exchanges and hospitals have been disrupted by failure of suspended ceilings, their integral lighting fixtures, and hanging light fixtures. It is therefore recommended that in important areas, where the loss of ceiling tiles cannot be tolerated (such as in hospital operating theatres) the ceiling should be fixed rigidly to the structure. The failure of suspended ceiling systems can be caused not only by failure of members and connections, but also by local or cumulative deformations which allow elements to drop between supports. The ceiling framing should be constructed in such a manner that all joints and connections are positively and mechanically fixed in order to avoid disconnection under dynamic effects. The connections should preferably be such that components will fail prior to the joints.

e) Non-structural components such as rigid masonry or concrete walls can significantly alter the response of a structure. Such components shall be treated as structural and needs to be included in the structural model itself.

f) The restraint system designed for the contents of museums and similar items of historical or artistic value needs to be so as not to alter its historic value. Special kinds of system that blends well with the historic fabric need to be devised, for that special advice should be obtained for detailing such restraints.

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