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Methodology & Impact of Vertical Irregularities on Seismic Responses in Tall Buildings: A Comparative Analysis

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Abstract:

The vulnerability of tall buildings to earthquake damage is influenced by numerous factors, including irregularities in their design and construction. Irregular structures, characterized by variations in geometry, mass distribution, and stiffness, are particularly susceptible to seismic-induced damage. In this study, we conduct an in-depth analytical examination of vertical irregularities, encompassing stiffness, mass, and vertical geometry variations. Ten structural models are considered, including an 11-story moment-resisting frame (MRF) building and its derivative models, to investigate the effects of these irregularities. Response spectrum analysis, based on the design response spectrum from NBC 105:2020, is performed on each model, and the building's responses are compared. Our findings reveal significant variations in the responses of tall buildings when different vertical irregularities are introduced. Notably, vertical geometry irregularities have a more pronounced effect on responses compared to mass and stiffness irregularities in specific storeys. This research provides valuable insights into the seismic behavior of tall buildings and their vulnerability to vertical irregularities.

1. Introduction:

A Building with regular configuration is structure which performs against the earthquake. This structure must possess the simple, regular configuration, minimum lateral strength and also stiffness of the structure. Setback buildings are a subset of vertically irregular buildings where there are discontinuities with respect to geometry. The process to determine the response or behavior of a structure under some specified loads or combinations of loads is known as structural analysis. Vertical irregularity are not avoidable in construction of Buildings. However, the behavior of structures with these vertical irregularity during earthquake needs to be studied. By taking adequate precautions, the main objective of Earthquake Engineering is to design and build a structure in such a way that the damage to the structure and its structural components during an earthquake is minimized.

Organization of the study

The thesis is organized as per detail given below:

Chapter 1. Introduce the topic of thesis in brief

Chapter 2. Discusses the literature review related to similar topic by previous authors

Chapter 3. Discussed about theory on buildings and shear wall

Chapter 4. Discuss about methodology followed and obtained values of parameters considered for the study in the thesis for analysis.

Chapter 5. Highlight the results and discussions made among all models.

Chapter 6. Discuss on future scope.

2. Literature Review

Shreyasvi C. and B. Shivakumaraswamy (2015) compared the behavior of regular and re-entrant structures in various seismic zones, utilizing both the response spectrum and time history methods with ETABS. Their findings showed that irregular buildings had higher drift and storey displacement.

Prajapati P. B and Prof. Mayur G. Vanza (2014) compared the seismic response of rectangular, C shape, and L shape structures using SAP 2000. Time history analysis considered accelerograms from Uttarkhasi, Bhuj, and Chamoli, and parameters like deflections at joints and storey shears were compared.

Arunava Das and Priyabrata Guha (2016) compared the behavior of four-storey irregular and regular buildings under earthquake loads, performing time history and pushover analyses with SAP 2000. They used Elecentro acceleration details for the time history method and observed greater displacements in the irregular model from the pushover analysis.

Arvindreddy and R. J. Fernandes (2015) investigated the response of regular and plan irregular structures in Zone V, conducting both static and dynamic methods using ETABS. Their findings indicated that the static method resulted in higher displacements compared to the dynamic method.

3. Seismic methods of analysis

Seismic Methods of Analysis After selecting the structural model, it is possible to perform analysis to determine the seismically induced forces in the structures. The analysis can be performed on the basis of the external action, the behaviour of the structure or structural materials, and the type of structural model selected. The analysis process can be classified. Depending on the nature of the considered variables, the method of analysis can be classified. Based on the type of external action and behaviour of structure, the analysis can be further classified as linear static analysis, linear dynamic analysis, non-linear static analysis, or non-linear dynamic analysis. Linear static analysis or equivalent static analysis can be used for regular structures with limited height. Linear dynamic analysis can be performed in two ways, either by the response spectrum method or by the elastic time-history method. The significant difference between linear static and linear dynamic analyses is the level of the forces and their distribution along the height of the structure. Non-linear static analysis is an improvement over linear static or dynamic analysis in the sense that it allows inelastic behavior of the structure. The method is simple to implement and provides information on the strength, deformation, and ductility of the structure, as well as the distribution of demands. This permits the identification of the critical members that are likely to reach limit states during the earthquake, to which attention should be paid during the design and detailing process. But the non-linear static method is based on many assumptions, which neglect the variation of loading patterns, the influence of higher modes of vibration, and the effect of resonance. In spite of the deficiencies, this method, known as push-over analysis, provides a reasonable estimation of the global deformation capacity, especially for structures that primarily respond according to the first mode.

A non-linear dynamic analysis or inelastic time-history analysis is the only method to describe the actual behavior of a structure during an earthquake. The method is based on the direct numerical integration of the differential equations of motion by considering the elasto-plastic deformation of the structural element. The scope of this book limits the discussion to only methods of elastic analysis; namely, the seismic coefficient method, dynamic analysis, and a brief description of the time-history method. These are explained in the sections that follow.

3.1 Basic assumptions

The following assumptions are made in the analysis of earthquake-resistant design of structures.

An earthquake causes impulsive ground motions, which are complex and irregular in character, with each change in period and amplitude lasting for a small duration. Therefore, resonance of the type visualized under steady-state sinusoidal excitations will not occur, as it would need time to build up such amplitudes. However, there are exceptions where resonance-like conditions have been seen to occur between long-distance waves and tall structures founded on deep soft soils.

An earthquake is not likely to occur simultaneously with winds or powerful floods and sea waves. The probability of occurrences of strong earthquake motion along with strong winds and/or maximum sea waves is low. Therefore, it is justified to assume that these hazardous events are not occurring at the same time.

The value of elastic modulus of materials, wherever required, may be taken as the one used for static analysis, unless a more definite value is available for use in such a condition. It may be noted that the values of modulus of elasticity for various construction materials display large variations.

3.2 Methods of elastic analysis

The most commonly used methods of analysis are based on the approximation that the effects of yielding can be accounted for by linear analysis of the building using the design spectrum for inelastic systems. Forces and displacements due to each horizontal component of ground motion are separately determined by analysis of an idealized building having one lateral degree of freedom per floor in the direction of the ground motion component being considered. Such analysis may be carried out by the equivalent lateral force procedure (static method) or response spectrum analysis procedure (dynamic method). Another refined method of dynamic analysis is the elastic time-history method. Both the equivalent lateral force and response spectrum analysis procedures lead directly to lateral forces in the direction of the ground motion component. The main differences between the two methods are in the magnitude and distribution of the lateral forces over the height of the building. The equivalent lateral force method is mainly suited for preliminary design of the building. The preliminary design of the building is then used for response spectrum analysis or any other refined method such as the elastic time-history method.

3.3 Equivalent lateral force method (Seismic coefficient method)

Seismic analysis of most structures is still carried out on the assumption that the lateral force is equivalent to the actual (dynamic) loading. This method requires less effort because, except for the fundamental period, the periods and shapes of higher natural modes of vibration are not required. The base shear, which is the total horizontal force on the structure, is calculated on the basis of the structure's mass, its fundamental period of vibration, and corresponding shape. The base shear is distributed along the height of the structure, in terms of lateral forces, according to the code formula. Planar models appropriate for each of the two orthogonal lateral directions are analyzed separately; the results of the two analyses and the various effects, including those due to torsional motions of the structure, are combined. This method is usually conservative for low- to medium-height buildings with a regular configuration.

3.4 Response spectrum analysis

This method is also known as modal method or mode superposition method. The method is applicable to those structures where modes other than the fundamental one significantly affect the response of the structure. This method is based on the fact that, for certain forms of damping which are reasonable models for many buildings the response in each natural mode of vibration can be computed independently of the others, and the modal responses can be combined to determine the total response. Each mode responds with its own particular pattern of deformation (mode shape), with its own frequency (the modal frequency), and with its own modal damping. The time history of each modal response can be computed by analysis of an SDOF oscillator with properties chosen to be representative of the particular mode and the degree to which it is excited by the earthquake motion. In general, the responses need to be determined only in the first few modes because response to earthquake is primarily due to lower modes of vibration. A complete modal analysis provides the history of response forces, displacements, and deformations of a structure to a specified ground acceleration history. However, the complete response history is rarely needed for design; the maximum values of response over the duration of the earthquake usually suffice. Because the response in each vibration mode can be modeled by the response of an SDOF oscillator, the maximum response in the mode can be directly computed from the earthquake response spectrum. Procedures for combining the modal maxima to obtain estimates (but not the exact value) of the maximum of total response are available. In its most general form, the modal method for linear response analysis is applicable to arbitrary three-dimensional structural systems. However, for the purpose of design of buildings, it can often be simplified from the general case by restricting its application to the lateral motion in a plane. Planar models appropriate for each of two orthogonal lateral directions are analyzed separately and the results of the two analyses and the effects of torsional motions of the structures are combined. Generally, the method is applicable to analysis of the dynamic response of structures, which are asymmetrical or have areas of discontinuity or irregularity, in their linear range of behavior. In particular, it is applicable to analysis of forces and deformations in multi-storey buildings due to medium-intensity ground shaking, which causes a moderately large but essentially linear response in the structure.

3.5 Elastic time-history method

A linear time-history analysis (THA) overcomes all the disadvantages of a modal response spectrum analysis provided non-linear behavior is not involved. This method requires greater computational efforts for calculating the response at discrete times. One interesting advantage of such a procedure is that the relative signs of response quantities are preserved in the response histories. This is important when interaction effects are considered among stress resultants.

3.6 Limitations of equivalent lateral force and response spectrum analysis procedures

The assumptions common to the equivalent lateral force procedure and the response spectrum analysis procedure are as follows:

(a) Forces and deformations can be determined by combining the results of independent analyses of a planar idealization of the building for each horizontal component of ground motion, and by including torsional moments determined on an indirect, empirical basis and

(b) Nonlinear structural response can be determined to an acceptable degree of accuracy, by linear analysis of the building using the design spectrum for inelastic systems.

Both analysis procedures are likely to be inadequate if the dynamic response behaviour of the building is quite different from what is implied by these assumptions, and also if the lateral motions in two orthogonal directions and the torsional motions are strongly coupled. Buildings with large eccentricities at the centers of storey resistance relative to the centers of floor mass, or buildings with close values of natural frequencies of the lower modes and essentially coincident centers of mass and resistance, exhibit coupled lateral-torsional motions. For such buildings independent analyses for the two lateral directions may not suffice, and at least three degrees of freedom per floor-two translational motions and one torsional should be included in the idealized model. The modal method, with appropriate generalizations of the concept involved, can be applied to analysis of the model. Because natural modes of vibration will show a combination of translational and torsional motions, it is necessary while determining the modal maxima to account for two facts: that a given mode might be excited by both horizontal components of ground motion; and modes that are primarily torsional can be excited by translational components of ground motion. Because natural frequencies of a building with coupled lateral torsional motions can be rather close to each other, the modal maxima should not be combined in accordance with the SRSS formula; instead a more general formula should be employed.

3.7 Equivalent lateral force versus response spectrum analysis procedures

Both, the equivalent lateral force procedure and the response spectrum analysis procedure, are based on the same basic assumptions and are applicable to buildings that exhibit dynamic response behavior in reasonable conformity with the implications of the assumptions made in the analysis. The main difference between the two procedures lies in the magnitude of the base shear and distribution of the lateral forces. Although in the modal method the force calculations are based on compound periods and mode shapes of several modes of vibration, in the equivalent lateral force method, they are based on an estimate of the fundamental period and simple formulae for distribution of forces which are appropriate for buildings with regular distribution of mass and stiffness over height. It would be adequate to use the equivalent lateral force procedure for buildings with the following properties seismic force resisting system has the same configuration in all storey and in all floors; floor masses do not differ by more than, say, 30% in adjacent floors; and cross-sectional areas and moments of inertia of structural members do not differ by more than about 30% in adjacent storey's. For other buildings, the following sequence of steps may be employed to decide whether the modal analysis procedure ought to be used.

1. Compute lateral forces and storey shears using the equivalent lateral force procedure.

2. Approximate the dimensions of structural members.

3. Compute lateral displacements of the structure as designed in step 2 due to lateral forces in step 1.

4. Compute new sets of lateral forces and storey shears with the displacements computed in this step.

5. If at any storey the recomputed storey shear (step 4) differs from the corresponding original value (step 1) by more than 30%, the structure should be analyzed by the modal analysis procedure. If the difference is less than this value the modal analysis procedure is unnecessary, and the structure should be designed using the storey shears obtained in step 4; they represent an improvement over the results of step 1.

This method for determining modal analysis is efficient as well as effective. It requires far less computational effort than the use of the modal analysis procedure. The seismicity of the area and the potential hazard due to failure of the building should also be considered in deciding whether the equivalent lateral force procedure is adequate. For example, even irregular buildings that may require modal analysis according to the criterion described may be analyzed by the equivalent lateral force procedure if they are not located in higher seismic zones and do not house the critical facilities necessary for post-disaster recovery or a large number of people.

3.8 Nepal Building Code Provision

Nepal National Building Code NBC 105: Seismic Design of Buildings document is the outcome of the revision of the earlier version of NBC 105: 1994 Seismic Design of Buildings in Nepal. 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. For Base-isolated buildings as well as for buildings equipped and treated with structural control can be designed in reference with specialist literatures. Minimum design earthquake forces for buildings, structures or components thereof shall be determined in accordance with the provisions of this standard.

3.8.1 Structural analysis method

The structural analysis for design seismic actions shall be carried out using any one of the following methods:

a) Equivalent Static Method

b) Linear Dynamic Analysis Methods

i. Modal Response Spectrum Method

ii. Elastic Time History Analysis

c) Non-linear Methods

i. Non-linear Static Analysis

ii. Non-linear Time History Analysis

3.8.2 Equivalent Static Method

The Equivalent Static Method may be used for all serviceability limit state (SLS) calculations regardless of the building characteristics. For ultimate limit state (ULS), the Equivalent Static Method may be used when at least one of the following criteria is satisfied:

i. The height of the structure is less than or equal to 15 m.

ii. The natural time period of the structure is less than 0.5 secs.

The structure is not categorized as irregular as per 5.5 and the height is less than 40 m.

3.8.3 Modal Response Spectrum Method

The Modal Response Spectrum Method may be used for all types of structures and the structures where Equivalent Static Method is not applicable. A three-dimensional analysis shall be performed for torsion ally sensitive structures.

3.8.4 Elastic Time History

The elastic 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. A three dimensional analysis shall be performed for torsion ally sensitive structures.

3.8.5 Load Combinations for limit State Method

Load Combinations for Parallel Systems

Where seismic load effect is combined with other load effects, the following load combination shall be adopted.

1.2DL + 1.5LL

DL + λLL + E

Where, λ = 0.6 for storage facilities = 0.3 for other usage

Load Combinations for Non- Parallel Systems

When lateral load resisting elements are not oriented along mutually orthogonal horizontal directions, structure shall 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. In this case, the following load combination shall be adopted.

1.2DL + 1.5LL

DL + λLL + (Ex + 0.3Ey)

DL + λLL + (0.3Ex + Ey)

Where, λ = 0.6 for storage facilities = 0.3 for other usage

4. METHODOLOGY

Planning of Models

The study consists of total of 10 numbers of model. The initial model is a 11 storey RC MRF. Additional 9 models are developed by improvising the initial model. The additional model consists of 3 stiffness irregular building, 3 mass irregular building and 3 vertical geometry irregular buildings. Each model had 5 nos of bays in both directions. The storey height is 3.0m for all models.

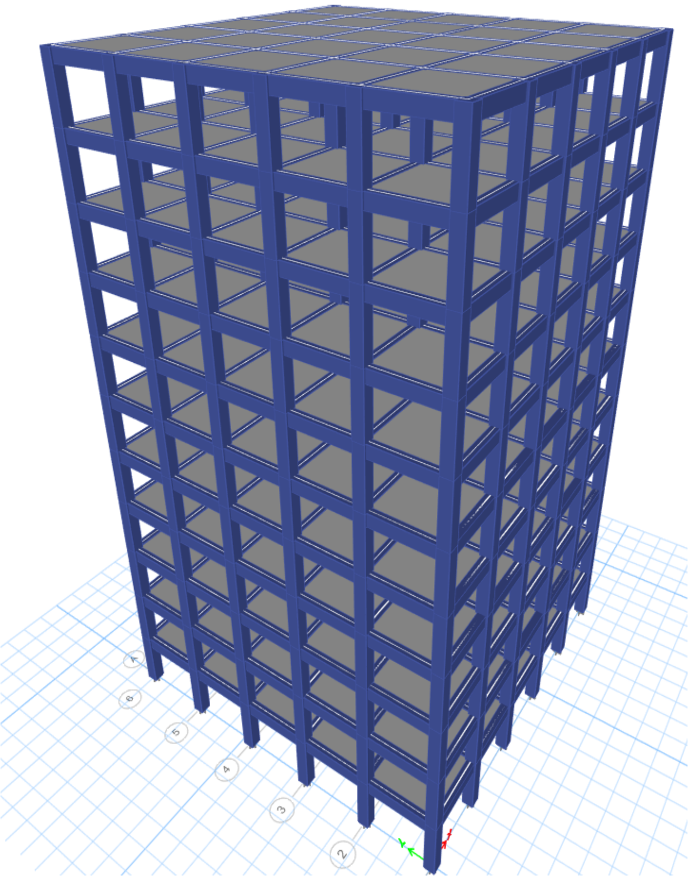

Figure 1 Model rendered in ETABS

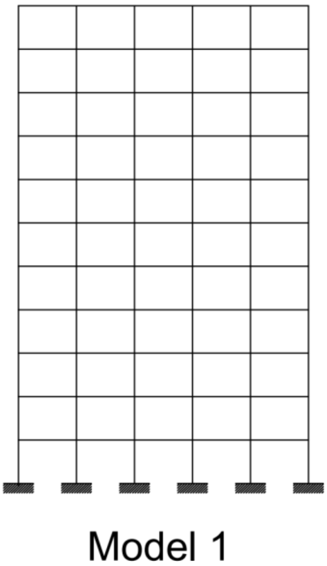

Figure 2 Initial Model (11 Storey)

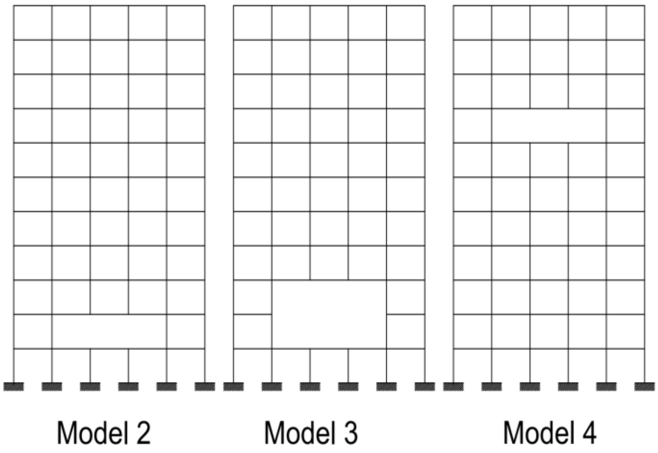

Figure 3 Stiffness Irregular Building Models

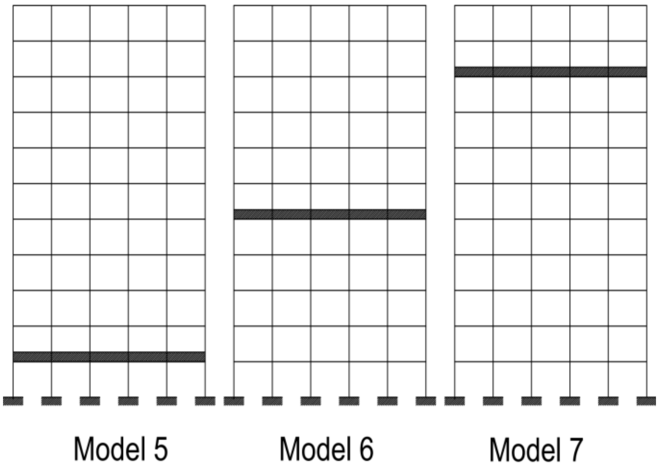

Figure 4 Mass Irregular Building Models

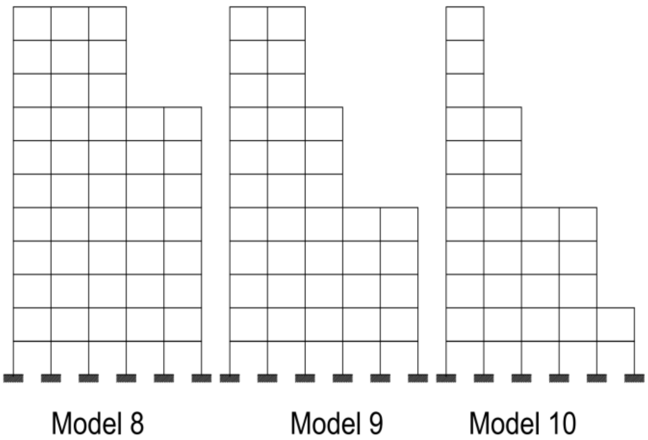

Figure 5 Vertical Geometry Irregular Building Models

The detail of each model is described in figures above and the table below.

Table 1 Details of Models

SN	Model Name	Description	Remarks
1	Model 1	Ten Storey RC Framed Building	
2	Stiffness Irregular Building		
	Model 2	Case I	1-2 Storey No Column (4 Columns)
	Model 3	Case II	1-3 Storey No Column (4 Columns)
	Model 4	Case III	7-8 Storey No Column (4 Columns)
3	Ten Storey RC Framed Building		
	Model 5	Case I	Live Load at Story 1 increased by 2 times
	Model 6	Case II	Live Load at Story 5 increased by 2 times
	Model 7	Case III	Live Load at Story 9 increased by 2 times
4	Vertical Geometry Irregular Building		
	Model 8	Case I	Vertical Geometry Irregularity 1
	Model 9	Case II	Vertical Geometry Irregularity 2
	Model 10	Case III	Vertical Geometry Irregularity 3

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4.1.1 Loads

Dead loads

Brick masonry : Unit Weight 19.2KN/m^3

Finishes (Floor Finishes) : 1.5 KN/m^2

Reinforced Concrete Elements : Unit Weight 25KN/m^3

Live load : 3 KN/m^2 on all floors except roof.

Lateral loads : Earthquake Loads as per NBC:105:2020

4.1.2 Lateral load

Equivalent static method is used to calculate the lateral forces at each storey level as per NBC: 105:2020 and time period of the modes is calculated by using ETABS 2016 software. Following parameters were considered in calculating the lateral forces in the structures.

Location = Birendranagar, Surkhet

Zone factor (Z) = 0.35

Importance factor (I) = 1

Response Reduction Factor (R) = 5(SMRF)

Soil Type = C

Load Combination considered in the analysis are mentioned above and for Dynamic Analysis addition combination is considered.

For Regular

DL+0.3LL+REX

DL+0.3LL+REY

For Irregular

DL+0.3LL+REX +0.3REY

DL+0.3LL+REY+0.3REX

4.1.3 Material properties

Concrete grade : M25 for beam and Slab

M25for Column

Steel grade : Fe 500

Modulus of Elasticity of concrete (Ec) : 5000√fck N/mm^2

Modulus of Elasticity of Steel (Es) : $2x10^5$ N/mm^2

4.2 Element dimensions

A 150mm thick slab is considered for all building models. The column dimension is kept as 600mmx600mm and that of beam is kept as 400mmx600mm.

Seismic Load Calculation

Coefficient Calculation:

Based on NBC 105:2020, Criteria for earthquake resistant design of structures, calculation of earthquake loads is done using seismic coefficient method:

The design horizontal seismic coefficient,

$C_d(Ti) = \frac{C(Ti)}{R_\mu x \Omega_u}$

Where,

$C(T_i)$ = Elastic Site Spectra at period (T_i)

R_μ = Ductility Factor

Ω_u = Over Strength Factor

The approximate fundamental natural period of vibration (T_i) in seconds, of moment-resisting frame buildings with brick infill panels,

may be estimated using the empirical expression:

$T_i = 0.075{*}h^{0.75}$

Where,

h = Height of building in meters

$T_a = 0.075{*}h^{0.75}$

$= 0.075{*}34.925^{0.75}$

=1.07 sec

Time period shall be increased by 1.25.

T=1.07*1.25=1.346sec

I = 1 (for Residential building)

Z = 0.3

$C_d(Ti) = \frac{C(Ti)}{R_\mu x \Omega_u} = \frac{0.75}{1.5x4} = 0.125$

$V_B = C_d(Ti)$ x W

$(C_s(T)=0.2{*}C_h(T)=0.2{*}0.75=0.15$

$C_d(Ti) = \frac{Cs(T)}{\Omega_s} = 0.15/1.25 = 0.12$

$V_B = \frac{C(Ti)}{R_\mu x \Omega_u} xW$

$T = 0.075 h^{\frac{3}{4}}$ For RCC frame building

Where,

V_B = Base shear

$C_d(T_i)$ = Design horizontal acc. spectrum

Z = Zone Factor

I = Importance Factor

$C(T_i)$ = Elastic Site Spectra at period (T_i)

R_μ = Ductility Factor

W = Seismic Weight of building

T_i = Fundamental time period of i^{th} mode of vibration

Ω_u = Over Strength Factor

h = Height of Building in m

d = Base Dimension at Plinth level

The design acceleration response spectra based on NBC 105:2020, is used for the analysis of the present building. Location of Building is Birendranagar, Surkhet. The seismic zone factor is 0.35. The horizontal base shear coefficient at ULS and SLS is calculated as 0.144 and 0.138 respectively. The plot of basic response spectrum is given below.

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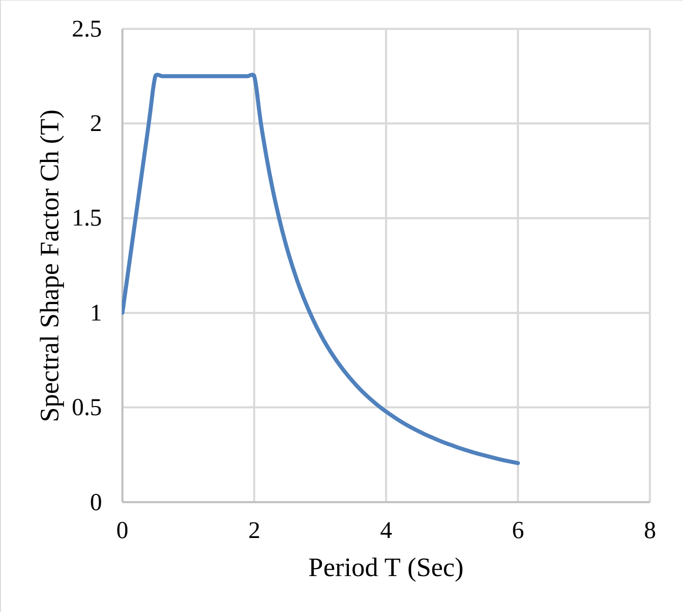

Figure 6 Design Response Spectrum

Conclusion:

This study underscores the critical importance of considering vertical irregularities in the design and construction of tall buildings. We have observed that the introduction of discontinuities in lower storeys can cause significant changes in building displacement, while similar discontinuities in upper storeys have a relatively lesser effect. Additionally, in mass irregular buildings, a higher mass in the upper storeys leads to greater top displacement. Furthermore, a decrease in the number of bays in upper storeys results in reduced top deformation in the case of vertical geometry irregular buildings. The drift of storeys with vertical element discontinuities increases significantly, emphasizing the need for careful design and construction practices. Interestingly, base shear does not significantly vary with stiffness and mass irregularities in specific storeys, but it exhibits notable variation in vertical geometry irregular tall buildings. Moreover, time period increases significantly with mass and stiffness irregularities in upper storeys, while a reduction in the number of bays in upper storeys leads to a significant decrement in the time period of tall buildings. These findings can inform better engineering practices and seismic-resistant design guidelines for tall buildings, contributing to the safety and resilience of urban infrastructure.

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