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Multi-Hazard Resistant Design of Buildings: A Spectrum-Based Approach for Seismic and Wind Loads

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Abstract:
The building industry suffered serious setbacks in recent natural hazards. These events bring to focus the questions of how the structural engineering profession treats the large uncertainty in the natural hazards and structural capacity and what reliability existing buildings have against future hazards. Although the profession has well recognized the uncertainty of seismic and wind loads, the incorporation of uncertainty in most building code procedures has been limited to the selection of design loads based on return period. Normally, when structures are designed against hazards, it is usual to assume that only one such hazard will act on the structure at a time, and the design is routine. Such designs for different hazards are more often complementary. In the case of buildings situated in a seismic area and located close to a sea coast are liable to be hit by an earthquake and cyclone, though at different times. Nevertheless, the structure is to be designed to resist both these hazardous conditions. In the present study, a mitigation strategy is designed for a 6-storey building that can resist both wind and earthquake by using the response spectrum and velocity spectrum. For this, a multi-hazard spectrum is prepared by bringing the earthquake load and wind load spectrums into a common platform. Modeling and analysis of the building are done by using SAP-2000 software and it is designed for an optimum period for multi-hazards.
Keywords: *Velocity spectrum, Multi-hazard spectrum, Building performance, Response spectrum, SAP-2000 software, Seismic design, Wind-resistant design.*

INTRODUCTION

"Natural Disaster" means a serious disruption of the functioning of a society, causing widespread human, material, or environmental losses due to an earthquake, cyclone, flood, Tsunami, or landslide. While the effects of individual natural disasters on buildings have been extensively researched, the study of the design of structures subjected to multiple hazards has been very limited. The type of damage caused by earthquakes and hurricanes differs, but the social and economic impacts are equally significant. For mid- and high-rise buildings, both high wind events and earthquakes are paramount concerns for their structural design. Earthquakes and high-wind hurricanes can cause significant damage; therefore, the lack of consideration of multi-hazard design in regions with high probabilities of both events could result in high casualties and economic losses.

India faces the threat from a variety of natural hazards such as floods, droughts, landslides, cyclones, earthquakes, and tsunamis. Many regions in India are vulnerable to more than one hazard, such as earthquakes and winds. About 59% of India's land area is under the threat of moderate to severe earthquake shaking intensity VII and higher. On the other hand, India has a long coastline of about 7,516 km of flat coastal terrain, a shallow continental shelf with high population density, and is extremely vulnerable to cyclones, covering 40% of India's land area. Surprisingly, areas like Gujarat, the Andaman Islands, etc., are prone to both earthquakes and cyclones.

Current design practice for regions subjected to wind and earthquakes, structures are designed for loads induced by the wind and, separately, by earthquakes, and the final design is based on the maximum of these two loading conditions. Normally, when structures are designed against hazards, it is usual to assume that only one such hazard will act on the structure at a time, and the design is routine. Such designs for different hazards are more often complementary. In the case of buildings situated in a seismic area and located close to a sea coast, such as the ones in Andaman, they are liable to be hit by an earthquake and a cyclone, though at different times. Nevertheless, the structure is to be designed to resist both these hazardous conditions.

Implicit in this approach is the belief that the separate standards assure risks of expedience of the specified limit states that are essentially identical to the risks inherent in the provisions for regions where only wind or earthquakes occur.

A structure could be subjected to more than one critical type of hazard during its service life, 'Multi-hazard' (Multiple hazards). Multi-hazard or multi-risk could be considered as a superposition of various hazards, e.g., wind and earthquake. Multi-hazard design is a potentially powerful means to achieve structures that meet the requirements of risk consistency for safety metrics, as well as being synergistic through the use of design features appropriate for one hazard that enhance performance for another hazard. In order to assess the multi-hazard risk level of urban areas and their spatial distribution, detailed information is required on the elements at risk, such as buildings, infrastructure, population, and economic activities. The use of detailed building information for the hazard risk assessment is essential.

The need to embrace a multi-hazard approach when designing and retrofitting structures is essential for their protection in these regions. A sound multi-hazard design approach provides an impetus to adopt a performance-based philosophy for design against risk. In this method, the aim is to anticipate and coordinate how the building and its system interact, how mitigation of one hazard can influence the building's vulnerability to others, and how undesirable conditions and conflicts may be avoided or resolved.

1.2. Scope and objective of the project

a) To understand and consider the occurrence of multi-hazard events.

b) Obtain the action of lateral loading on the hypothetical buildings considered.

c) To obtain the natural frequency of the building.

d) To design a mitigation strategy for the building such that it can resist both wind and earthquake efficiently.

LITERATURE REVIEW

2.1. GENERAL

In the present world scenario of design practice for regions subjected to wind and earthquakes, structures are designed for loads induced by the wind and, separately, by earthquakes, and the final design is based on the more demanding of these two loading conditions. Implicit in this approach is the belief that the separate standards assure risks of exceedance of the specified limit states that are essentially identical to the risks inherent in the provisions for regions where only wind or earthquakes occur. This chapter focuses on a brief review of the literature related to multi-hazard-based analysis of structures due to wind and earthquake.

2.2. Methodology For Deriving Multihazard Factors For Design Base Shear Of Buildings For Wind And Earthquake Hazard Buildings For Wind And Earthquake Hazard For Peninsular India by S. Arunachalam et. al. [2013] proposed a methodology to derive the multi hazard factors for base shear for any given return period for wind and earthquake which are derived based on the existing knowledge about the mean annual rate of exceedance of wind and earthquake for a given region. The proposed methodology is demonstrated by deriving a multi-hazard factor for base shear for a twenty-storey building for different cities of peninsular India for wind and earthquake hazards. The sensitivity of multi-hazard base shear for different site classes (site class C, D, E) for earthquake and different terrain categories (Category II and Category III) for wind load is studied. The studies bring out the relative hazard potential of different cities of Peninsular India for wind and earthquake. The proposed methodology provides a scientific platform for the development of probability-based multi-hazard design without deviating much from the existing provisions of codes of practice for earthquake and wind load.

2.3. Analysis and Design of RC Tall Building Subjected to Wind and Earthquake Loads by K. Rama Raju et. al. [2013] presented the response of a tall building under wind and seismic load as per IS codes of practice. Seismic analysis with the response spectrum method and wind load analysis with the gust factor method are used for analysis of a 3B+G+40-storey RCC high-rise building. The building is modeled as a 3D space frame using STAAD. Pro software and observed that the forces found from the analysis in beams and columns using STAAD. Pro are much higher than the results reported INSDAG report. The load cases considered for analysis are not mentioned in the INSDAG report. Safety of the building is checked against allowable Limits prescribed for inter-storey drifts, base shear, accelerations, and roof displacements in codes of practice. After the analysis, the structure is found to be wind and earthquake sensitive, and the roof displacement and inter-storey drifts due to wind and earthquake are exceeding the limits prescribed. While designing, some of the beams and column sections, the limit on the maximum percentage of reinforcement in the member is exceeding the maximum percentage of reinforcement in the member. To satisfy these limits, it is suggested to increase the grade of the concrete from M35 to M60, and the cross sections of the columns and beams also need to be increased.

2.4. Review of Methods To Assess, Design for, and Mitigate Multiple Hazards by Yue Li et. al. [2012] discussed the current state of practice for assessment, design, and mitigation of the impact of multiple hazards on structural infrastructure. Also provides an example of damages and losses resulting from multiple hazards and offers an overview of the differences and similarities between different hazards. Post-disaster surveys, experimental testing, risk assessment and loss estimation, modeling and numerical analysis, to assess the effects of multiple hazards, are discussed. The potential effects of climate change on natural hazard patterns and building/infrastructure damage are presented.

2.5. Safety of Structures in Strong Winds and Earthquakes: Multi-hazard Considerations by Duthinh and Simiu (2010) pointed out that the regions with significant wind and seismic hazards, risks of exceedance of limit states can be up to twice as high as those for regions where one hazard dominates. The authors explained the nature of the misconception that has led to the development of the current inadequate ASCE design criteria for multi-hazard regions using probabilistic tools, and a simple modification of the ASCE 7 Standard criteria can assure that risks for structures in multi-hazard regions are not higher than risks for structures in regions where one hazard dominates.

2.6. Probability model and solution on earthquake effects combination in along wind resistant design of tall-flexible buildings by Xiao-Jian and Ming (2006) have studied the earthquake effects in combination with wind for a high-rise flexible structure and shown that the total lateral loads in wind resistant design combined with earthquake effects are larger than that in a seismic design involving wind effects. It is also indicated that for a tall-flexible structure, the ignorance of the total force in terms of the wind-resistant design, including earthquake effects, may lead to underestimation.

2.7. Multi-hazard performance assessment of a transfer-plate high-rise building by Zhou and Xu (2007) presented the performance of a typical high-rise building with a thick transfer plate (TP), which is one type of building structure commonly found in Hong Kong. The building is assessed against both earthquake and wind hazards. Seismic and wind-resistant performance objectives are first reviewed based on relevant codes and design guidelines for high-rise buildings. After a brief introduction of the wind-resistant design of the building, various methodologies, including equivalent static load analysis (ESLA), response spectrum analysis (RSA), pushover analysis (POA), linear and nonlinear time-history analysis (LTHA and NTHA), are employed to assess the seismic performance of the building when subjected to frequent earthquakes, design-based earthquakes, and maximum credible earthquakes. The effects of design wind and seismic action with a common 50-year return period are also compared. The results indicate that most of the performance objectives can be satisfied by the building, but there are some objectives, such as the inter-story drift ratio, that cannot be achieved when subjected to frequent earthquakes. It is concluded that, in addition to wind, seismic action may need to be explicitly considered in the design of buildings in regions of moderate seismicity.

2.8. Risk Consistency And Synergy In Multi-Hazard Design by Chiara Crosti [2011] focused on the issue of risk consistency in multi hazard design, and says that, in spite of this difficulty, it is possible to quantify the risks of arriving at a particular lateral drift state for structures exposed to multiple non-simultaneous hazards and to compare them to the risks for the same structures subjected to a single hazard. A second focus is the issue of multi-hazard design synergy. It has been pointed out that redetailing a building to current seismic codes can increase its resistance to blast and that structural efficiency and life-cycle cost are influenced by multi-hazard considerations. The main focus is on risk consistency in the design of buildings under multiple hazards. For this, a case study is considered as a structure satisfying the requirements associated with both hazards. The paper says that a structure exposed to multi-hazards and designed to satisfy code requirements does not necessarily achieve the level of safety implied by the ASCE 7 Standard for individual hazards. This research also shows that for a 10-story steel-framed building used as a case study, the RBS connections, which are intended to enhance ductility in seismic design, produce no appreciable reduction, compared with the WUF-B connections, in the risk of the structure achieving a given lateral drift (corresponding to MCE) under exposure to wind alone or exposure to wind or earthquakes.

2.9. Multi-hazard upgrade decision making for critical infrastructure based on life-cycle cost criteria. Jalayer et al. (2011) conducted a preliminary study aiming to calculate the expected life-cycle cost for a critical infrastructure subjected to more than one hazard in its service lifetime. A methodology is presented that takes into account both the uncertainty in the occurrence of future events due to different types of hazards and also the deterioration of the structure as a result of a series of events. To satisfy life safety conditions, the probability of exceeding the limit state of collapse is constrained to be smaller than an allowable threshold. Finally, the methodology is implemented in an illustrative numerical example, which considers a structure subjected to both seismic hazard and blast hazard in both upgraded and non-upgraded configurations.

2.10. Optimization and Multi-hazard Structural Design by Florian. A et. al. [2009] presented an optimization-based approach to the design of structures in a multi-hazard environment. The advantage of an optimization approach in a multi-hazard context is that it provides an integrative framework allowing the structure to be optimized under constraints associated with all the hazards to which the structure is exposed, thereby achieving the most economical design consistent with the constraints of other types. This approach is illustrated with a simple example by considering earthquakes and the wind as hazards. This basic approach can also be applied to any specified topological configuration, to a high-rise frame structure for which optimization is carried out under a single hazard.

MULTIHAZARD SCENARIO FOR EARTHQUAKE AND WIND HAZARD

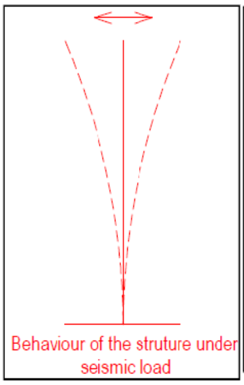
3.1. INTRODUCTION:

There are subtle differences between induced wind and earthquake forces, their magnitude, distribution of forces, and the resulting displacements. Earthquake forces result directly from the distortions induced by the motion of the ground on which the structure rests. The magnitude and distribution of forces are influenced by the properties of the structure and its foundation, as well as the character of the ground motion. Even though wind and earthquake forces are essentially dynamic, there exists a fundamental difference in the manner in which they are induced in a structure. Earthquake forces are principally internal forces resulting from the distortion produced by the internal resistance of the structure to earthquake motions, while wind loads applied as external loads are characteristically proportional to the exposed surface area of a structure. The magnitude of earthquake forces is a function of the mass of the structure and its stiffness, rather than its exposed surface area.

3.2 Contra requirements for earthquake and wind loads

Design of structures against multiple hazards is more than doubly difficult when compared with designing against a single hazard, especially when those multiple hazards are wind and earthquake. Many favorable features of wind-resistant design are unfavorable for earthquake-resistant design and vice versa. Heavy structures resist the wind better, and light structures resist earthquakes better. Flexible structures attract greater wind forces. Stiff structures generally attract greater earthquake forces.

The earthquake force coming onto the structure is proportional to the structure's weight. The wind force coming onto the structure is proportional to the lateral side area obstructing the wind. Soft storey is a critical issue in earthquakes, whereas in wind and tsunamis, it is advantageous. Earthquake design input is taken from the response spectrum, and the wind design input is taken from the force or velocity spectrum. The deflection of the structure for seismic force will be to and fro about the centre of mass, and it causes stress reversal in members. In case of wind forces, the deflection will be about the initial static deflected position, and the to and fro motion is less compared to seismic force and hence less reversal of stresses as shown in figures 3.1 and 3.2.



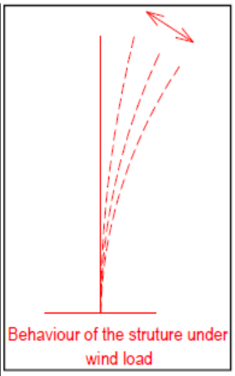
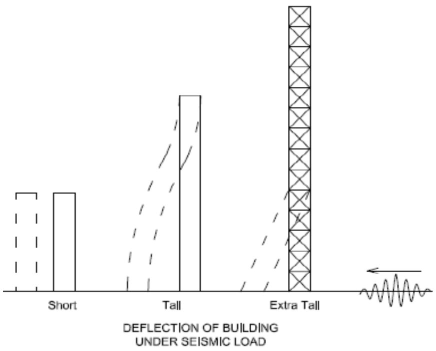


Figure 3.1 Figure 3.2

Fig 3.1 behaviour of structure under seismic loads

Fig 3.2 behaviour of structure under wind loads

The storey displacement will be large at upper floors during seismic events, and the displacement will be parabolic. The maximum deflection of the structure will be around 0.4%. Due to wind forces, the storey displacement at upper floors will be less compared to seismic forces, and the displacement is linear. The maximum deflection of the structure will be around 0.5%. The deflection of the structure for seismic and wind forces is shown in Figures 3.3 &3.4



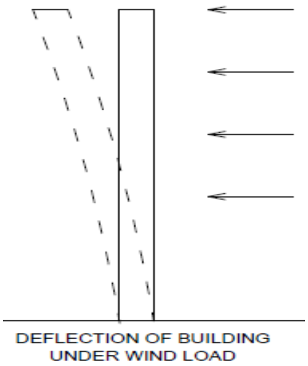
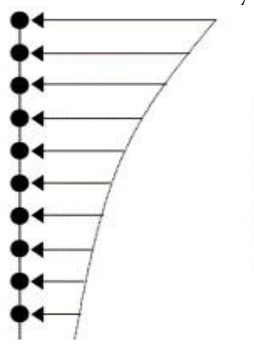


Fig 3.3 Deflection of building under seismic load
Fig 3.4 deflection of building under wind loads

In the case of an earthquake, the input has only a dynamic component, whereas wind has both dynamic and static components. Inelastic design is followed in earthquakes, but mostly the wind design is elastic. The design methods used to protect against the hazards by looking at the ways in which these methods reinforce or are in conflict with one another are compared. Many favorable features of wind-resistant design are unfavorable for earthquake-resistant design and vice versa. Heavy structures resist winds better. Light structures resist earthquakes better. Flexible structures attract greater wind forces. Stiff structures (generally) attract greater earthquake forces. This is a key aspect of multi-hazard design because the similarities and differences in the ways in which hazards affect buildings and how to guard against them demand an integrated approach to natural hazards design. This must be pursued as part of a larger integrated approach to the whole building design problem. Earthquakes are of much shorter duration than strong winds and are treated as transient loading. With earthquake and windstorm hazards having their energy potentials at two different ranges of frequencies, this implies that for a given structure in terms of performance, they may correspond to different damage levels. The structure having a frequency less than 1 Hz is actually advantageous, whereas in the case of wind is critical.



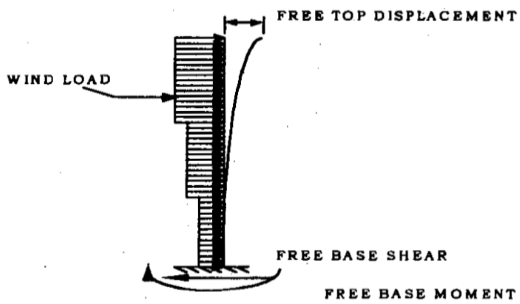


Fig.3.5 Earthquake load distribution over the height of the building.
Fig. 3.6 Wind load distribution over the height of the building.

However, there are many similarities in the effective design and construction of buildings to resist cyclones and earthquakes: Symmetrical and compact shapes are favorable. Connections are of paramount importance. Each critical element must be firmly connected to the adjacent elements. The proposed project scope is to anticipate and coordinate how the building and its systems interact, how mitigation of one hazard can influence the building's vulnerability to others, and how undesirable conditions and conflicts may be avoided or resolved. Arriving at an optimum design requirement and catering to sustainable and economical designs for multi-hazard is an engineering challenge.

3.3. MULTI-HAZARD SPECTRUM

Towards the development of strategies for mitigating the distress of multi-hazard vulnerable buildings, investigations are initiated on the suitability of base isolation for seismically vulnerable buildings, and additionally checked for wind loading. Analytical studies on multi-storied buildings with base isolation to meet contra requirements in a multi-hazard environment with specific reference to earthquakes and strong winds, are performed. The base isolation system design is made in such way that the change in the natural time period of the building by adding the base isolation should help in lowering the seismic forces and also the same shift in the time period concept is effectively used from wind point of view in such way that the time period shift is not in a peak portion of the cross wind force spectrum and then reduces the wind load vulnerability. The multi-hazard spectrum is developed using the Cross wind force spectrum

(AS/NZS 1170.2:2011) and the Earthquake Response spectrum (IS 1893 (Part 1):2002) are suitable brought in a common platform.
The cross-wind force spectrum given for different aspect ratio buildings in AS/NZS 1170.2:2011 is used for generating the multi-hazard spectrum is as shown in Figure 3.7.

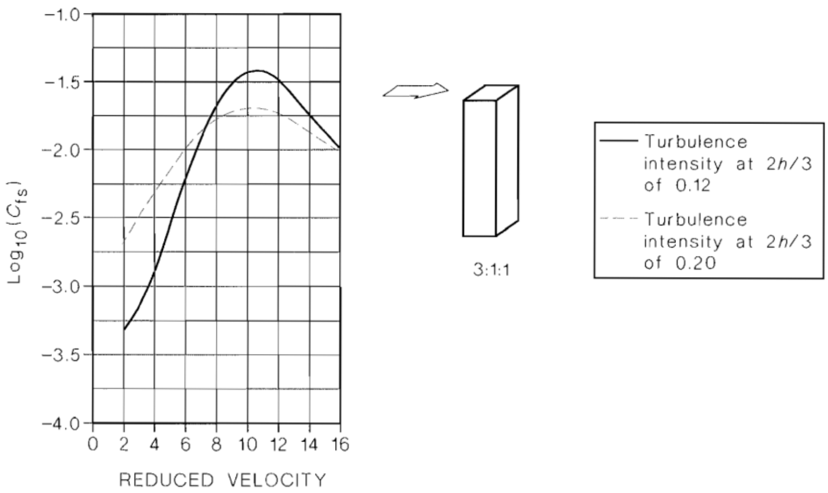


Fig 3.7 crosswind wind force spectrum for a square section
The relation between reduced velocity (V_R) and time period (T) of the building is

$$T = \frac{V_R b(1 + g_v I_h)}{V_{des}}$$

Where,
V_{des} = design wind speed
b = width of the building
I_h = turbulence intensity and
g_v = peak factor for the upwind velocity fluctuations
The multi-hazard spectrum considered for the present building is a tripartite graph consisting of spectral acceleration coefficient (Sa/g), cross-wind force spectrum coefficient Log $_{10}$ (C_{fs}), and Time Period (Sec) is as shown in Figure 3.8. The Design wind speed is taken as 55 m/s

METHODOLOGY
4.1 General
Structural analysis was carried out using the well-known computer program SAP2000. Efficient model formulation and problem solution are achieved by idealizing the building as a system of frames interconnected by floor diaphragms.
The primary purpose of all kinds of structural systems used in building types of structures is to transfer gravity loads effectively. The most common loads resulting from the effect of gravity are dead load, live load, and snow load. Besides these vertical loads, buildings are also subjected to lateral loads caused by wind and earthquake forces. It is important for the structure to have sufficient strength against vertical loads, together with adequate stiffness to resist lateral forces. The static and dynamic structural behaviors of high-rise buildings are governed by the distributions of transverse shear stiffness and bending stiffness per each storey.
4.2 Equivalent Lateral Force Method
The equivalent lateral force method is commonly preferred by design engineers because of its simplicity. It is based on the following assumptions:
1. The effects of yielding on the building structure are approximated using elastic spectral acceleration reduced by a modification factor.
2. A linear lateral force distribution can be used to represent the dynamic response of the building structure.
The following procedure is used for the analysis of building structures using the equivalent lateral load method:
1. Determination of the first natural vibration period.

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2. Determination of the total equivalent seismic load.
3. Determination of design seismic loads acting at storey levels.
4. Determination of points of application of the loads.
5. Analysis of the structural system

4.3. Force Coefficient Method

The following procedure is followed for the analysis of the building structures using the force coefficient method:

1. Examination of the wind speed, topographic features, terrain category, etc., according to the place under study
2. Determination of total equivalent wind load.
3. Determination of wind load distribution and point of application at various storey levels.
4. Analysis of the structural system.

4.4. Method of analysis:

A G+5-storied building is to be analyzed. The plan consists of 5 bays of span 4m along X direction and 5 bays of span 4m along Y direction with a storey height of 3.3m each. The typical hypothetical building is to be studied for the effect of lateral loading (earthquake and wind forces). A similarly dimensioned fixed-based building and base-isolated building are designed, and the optimum natural frequency of the building is fixed.

4.5. Sequence of work:

The sequence of work done is represented in the following steps

1) Need for study.
2) Collection of Appropriate literature.
3) Analysis and design of a 6-storey building in SAP.
4) Design of a base isolation system for a building to resist earthquake forces.
5) Development of a multi-hazard spectrum.

4.6 Physical Properties:

The material properties of concrete include mass, unit weight, modulus of elasticity, Poisson's ratio, etc. The short-term modulus of elasticity (E_c) of concrete, as per IS 456:2000, is given by

$E_C=5000\sqrt{f_{ck}}$

Where,

f_{ck} = 25 Mpa(Characteristic compressive strength of concrete at 28 days) and

Poison's ratio =0.2

For steel rebar, the properties required are yield stress (f_y) of 415 MPa, modulus of elasticity of 2x105MPa, and Poisson's ratio = 0.3

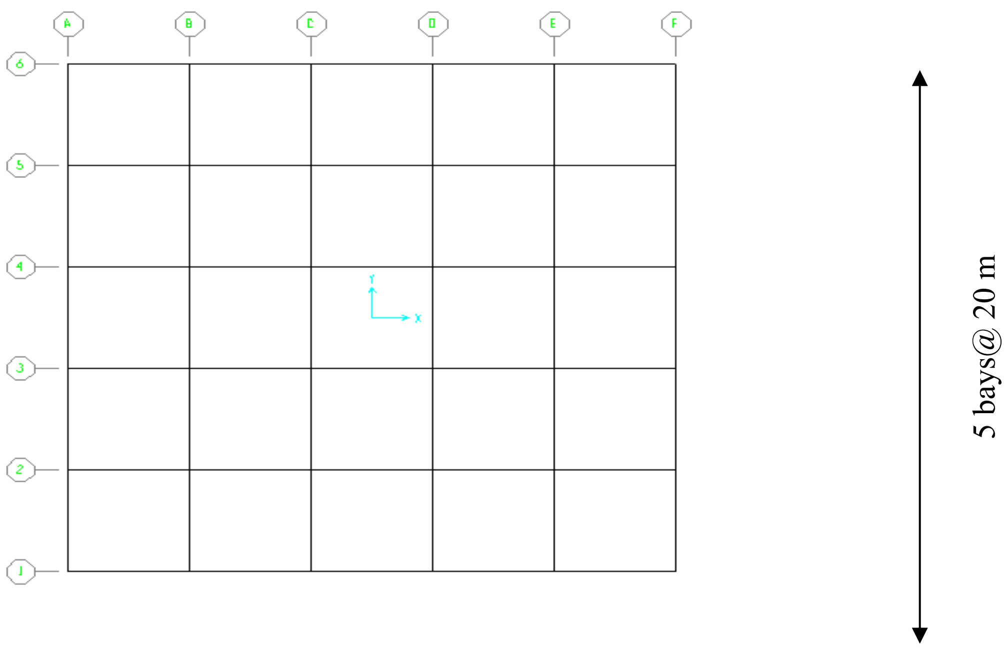

Fig 4.1. Plan of the building

4.7. Design Data

The design data of the building are tabulated in Table 4.1

Table 4.1. Design Data of the Building

Beam	300 x 350 mm
Column	450 x 600 mm
Slab	120mm thick
Live load	2 kN/m^2 at floors
Gravity load	Default (SAP analysis)
Earthquake load	As per IS 1893 part 1: 2002
Wind load	AS per IS 875 part 3: 1987
Type of soil	Type II, Medium as per IS:1893
Zone	V

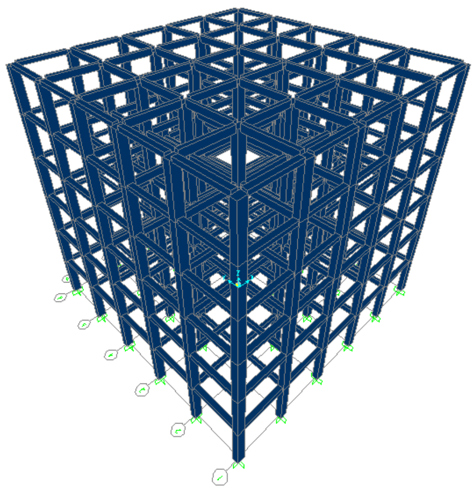

Fig 4.2. Framed building model in SAP2000

4.8. Load analysis

Dead Load = 6 kN/m (internal beams)

= 3 kN/m (external beams)

Seismic Load

Total weight of building, W = 10842kN

Time period, T = 0.7s

Design horizontal seismic coefficient, A_h = 0.07

Design base shear V_B = 376kN

Table5.2.Seismic load calculation.

Storey	Weight W_i (kN)	Height h_i (m)	$W_i{*}h_i^2$ ($kNm^{2)}$)	$Q_i = V_B \frac{W_i h_i^2}{\sum_{1}^{Floors} W_i h_i^2}$ (kN)
6	760	19.8	297950	131.43
5	924	16.5	251559	111.09
4	924	13.2	160997	71.17
3	924	9.9	90561	40
2	924	6.6	40249	17.7
1	924	3.3	10062	44.47

Summation	5380		851378	376

The calculated loads as per the IS codes are assigned to the frame. The structural effect of slabs, diaphragm constraints is taken into account and assigned at each floor level. Rigid beam column joints have been modeled by giving end offsets to the frame elements. The joint restraints at the base are given as fixed, and model analysis is performed to obtain the natural frequency of a fixed-base building. The elevation of a fixed-based building is shown in the Figure below.

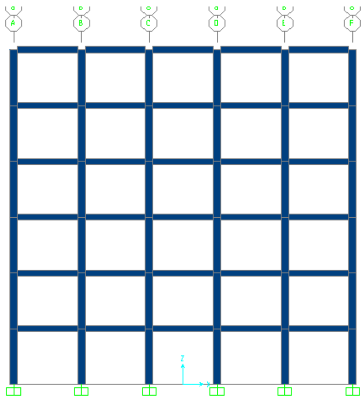

Fig 4.3 elevation of fixed base building

When the natural time period of the building obtained from the analysis is marked on the response spectrum curve, it is observed that the building is highly vulnerable to seismic forces acting on it. To make the structure safe against earthquake loads, the well-known base isolation method is adopted. In seismic isolation, the fundamental aim is to reduce substantially the transmission of the earthquake forces and energy into the structure. This is achieved by mounting the structure on an isolation system with considerable horizontal flexibility so that during an earthquake, when the ground vibrates strongly under the structure, only moderate motions are induced within the structure itself. As the isolator flexibility increases, movements of the structure relative to the ground may become a problem under other vibrational loads applied above the level of the isolator, particularly wind loads.

Plain elastomeric bearings are considered for the building, and isolators are designed for the load acting on the structure.

4.9. Base Isolation Design

Base isolation design includes the size of isolator, height of isolator, number of rubber layers, number of steel plates, mass of isolator, weight of isolator, rotational inertia, horizontal stiffness, and vertical stiffness of isolator.

Linear Theory of Seismic Isolation:

This can be explained based on the two-mass structural model as shown in Figure 3. The mass m represents the superstructure of the building, and m_b mass of the base floor above the isolation system. The other parameters are structural stiffness (K_s) and damping (C_s), and base isolator stiffness (K_b) and damping (C_b). The absolute displacements of the masses are given by U_s and $U_{b,}$ and the ground displacement is given by U_g. Therefore, the relative displacements are given by

$v_b = U_b - U_g V_s = U_s - U_g$

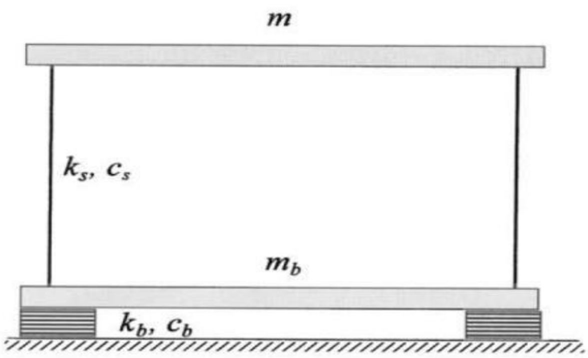

Fig 4.4 Parameters of two degree of freedom isolation system

The basic equations of motion of the two-degree-of-freedom system are

$$m + mb\ \ddot{u}b + m\ddot{u}s + Cb\acute{u}b + KbVs = m + mb\ \ddot{u}g$$
$$m\ddot{u}b + m\ddot{u}s + Cs\acute{u}s + Ksvb = -m\ddot{u}g$$

This can be written in matrix form as

$$\begin{bmatrix} M & m \\ m & m \end{bmatrix}\begin{bmatrix} \ddot{u}b \\ \ddot{u}s \end{bmatrix} + \begin{bmatrix} C_b & 0 \\ 0 & C_s \end{bmatrix}\begin{bmatrix} \acute{u}_b \\ \acute{u}_s \end{bmatrix} + \begin{bmatrix} K_b & 0 \\ 0 & K_s \end{bmatrix} = \begin{bmatrix} M & m \\ m & m \end{bmatrix}\begin{bmatrix} 1 \\ 0 \end{bmatrix}\ddot{u}_g$$

Where M=m+mb

$M\ddot{v}+C\dot{v}+Kv=-Mr\ddot{u}g$

Mass ratio γ is given by

$\gamma = \frac{m}{m+m_b} = \frac{m}{M}$

The natural frequencies ω_b and ω_s are given by

$\omega b2 = K_b \frac{m}{m+m_b}\ \ \omega_s{}^2 = \frac{K_S}{m}$

The damping factors β_b and β_s are given by

$$2\omega b\beta b = \frac{C_b}{m + m_b}\ \ 2\omega b\beta b = \frac{c_s}{m}$$

In terms of these quantities, the basic equation of motion becomes

$\gamma\ddot{u}_s+\ddot{u}_b+2\omega_b\beta_b v_b+\omega_b 2 v_b = -\ddot{u}_g$

$\ddot{u}_s+\ddot{u}_b+2\omega_s\beta_s v_s+\omega_s 2 v_s = -\ddot{u}_g$

Development of Equations

The stiffness of the isolator k is derived from equation of natural frequency.

$$\omega = \sqrt{\frac{k}{m}}$$

Where,

k= stiffness of isolator

m =mass acting on column

$$\frac{2\pi}{T} = \sqrt{\frac{k}{m}}$$
$$(2.6)^2 = \frac{k}{m}$$
$$\frac{k}{m} = 5.84$$

Stiffness K = $5.84 \times \left(\frac{924\times10^3}{9.81}\right)$

= 550643N/m

= 550.643N/mm

It is assumed that the isolator gets deformed, keeping the structure intact. Hence, the maximum spectral displacement of the isolator (IS 1893 (Part 1):2000

$$Sd = \frac{T^2}{4\pi^2} \times s_a \times Z$$

Where,

S_a= Spectral acceleration of the isolator

Z= Zone factor for the maximum considered earthquake

$$Sd = \frac{2.6^2}{4\pi^2} \times 0.6 \times 0.36 \times 9.81$$
$$Sd = 0.362m$$

The thickness of the isolator is obtained by considering the shear strain of the isolator

$$\gamma = \Delta t$$

Where,

Δ = Length of deformation

T = thickness of the isolator

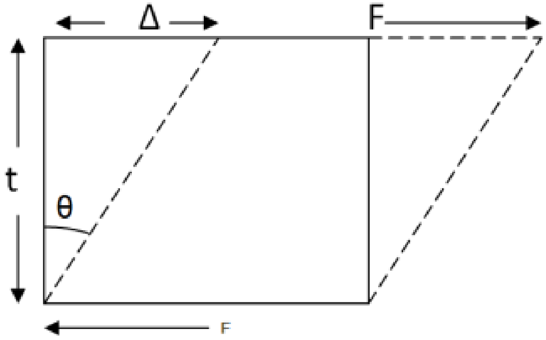

Fig 4.5 behaviour of isolator under shear

In this analysis, γ is assumed as 75%,

Therefore, Δ=t.

Obtained thickness of the rubber is used to find the size of the rubber by stiffness t

$$\frac{S_d}{t} = \gamma$$

$$\frac{0.362}{t} = 0.75$$

Thickness of rubber $t = 0.483\text{mm}$

Horizontal stiffness of the isolator, $K_H = \frac{\text{load}}{\text{deflection}}$

Where,

Load=Shear stress × Cross sectional area

Therefore, $K_H = \frac{GA}{t}$

$$550643 = \frac{0.0009 \times 10^9 \times A}{0.397}$$

Area of isolator rubber $A = 0.295\text{m}^2$

Size of rubber $B = 0.543\text{m}$

LAYERING OF ISOLATOR

To keep the ratio of the horizontal and vertical stiffness equal for the different isolator sets, a parameter called Shape factor, S is introduced to keep the ratio of the horizontal and vertical stiffness equal for the different isolator sets. It is a dimensionless measure of the aspect ratio of the single layer of the elastomer. The shape factor S is given by

$$S = \frac{1}{\sqrt{6}}\frac{f_V}{f_H}$$

Where,

$$f_H = \frac{1}{T} = \frac{1}{2.6} = 0.38\text{HZ}$$

And it is assumed that $f_V = 20\text{H} = 7.69\text{HZ}$

$$S = \frac{1}{\sqrt{6}}\frac{7.69}{0.38} = 8.179$$

The shape factor is also given as

$$S = \frac{\text{Loadedarea}}{\text{Forcefreearea}}$$

$$S = \frac{B}{4t_r}$$

Where,

t_r = Thickness of rubber between two steel plates

$$t_r = \frac{B}{4S} = \frac{0.543}{4 \times 40}$$

$$t_r = 0.013\text{m} = 13.58\text{mm}$$

Number of rubber layers $\text{nt} = \frac{483}{13.58} = 36$

Number of steel plates n $= n - 1 = 36 - 1 = 35$

Total height of the isolator H $= 483 + (35 \times 1) + (20 \times 2) = 558$mm

Mass of the rubber $M_R = \rho V$
$= 930 \times (0.295 \times 0.483)$
$= 132.56$kg

Weight of rubber $W_R = \frac{132.56 \times 9.81}{1000} = 1.3$kN

Mass of steel $M_S = \rho V$
$= 7859 \times (0.295 \times 0.001 \times 35)$
$= 81.05$kg

Weight of steel W = (81.05×9.81)/1000 = 0.79Kn =795N

Total mass M $= 132.56 + 81.05 = 213.5$kg

Total weight W = $1.3 + 0.79 = 2.095$kN $= 2095$N

Rotational inertia RI $= \frac{MR}{12}(l^2 + t^2)$

$$RI_X = RI_Y = \frac{132.6}{12}(0.543^2 + 0.543^2)$$

= 65112487.5 N-mm^2

$$RI = \frac{132.6}{12}(0.543^2 + 0.483^2)$$

$= 58315237.5\text{N} - \text{mm}^2$

Horizontal stiffness $K_H = \frac{GA}{t}$

$$= \frac{0.0009 \times 10^9 \times 0.295}{0.558}$$

$$= 475.8\frac{\text{N}}{\text{mm}}$$

Vertical stiffness $K_V = \frac{E_C A}{t}$

$$= \frac{0.05 \times 10^9 \times 0.295}{0.558}$$

= 26433.7 N/mm

Calculated rubber isolator properties are assigned as link/ support properties in the software, as shown in Figure 5.6, and the base-isolation model analysis is performed for the structure.

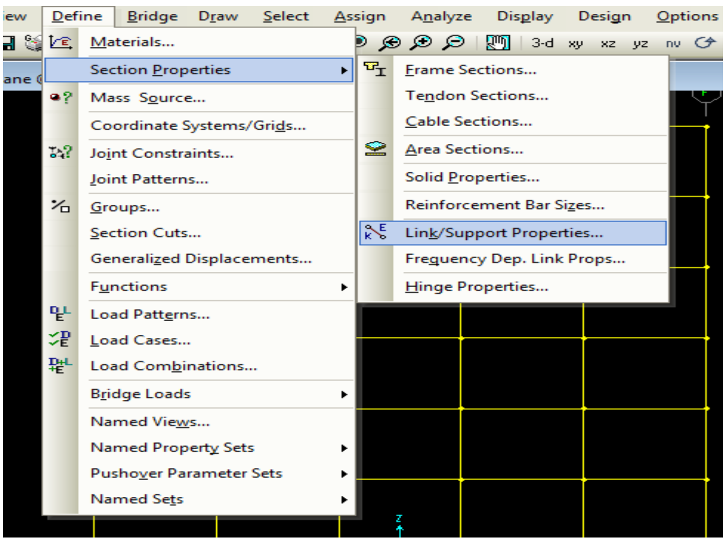

Fig 4.6 base isolation defining as spring element

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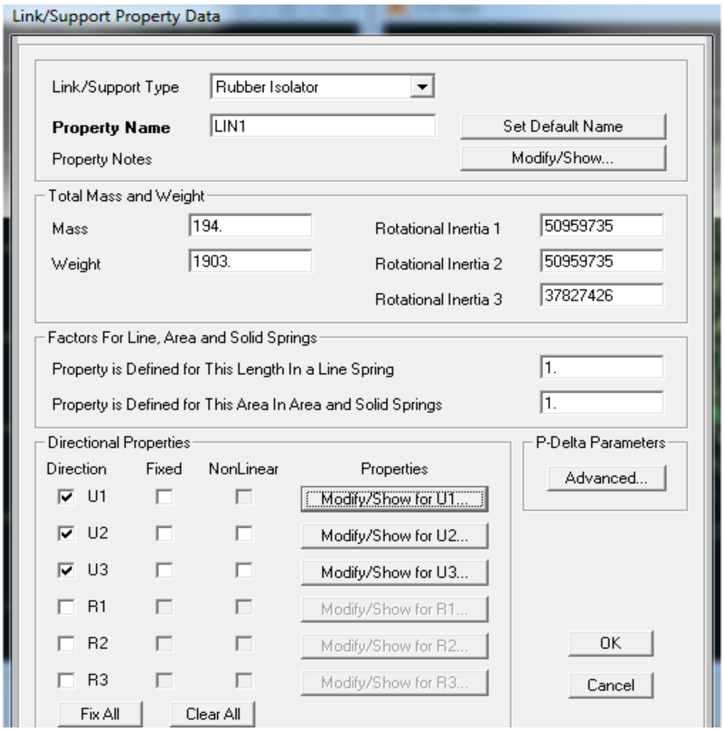

Fig 4.7 dialogue box of base isolation input

After the base isolation design, the response can be clearly seen in the model, and the time period of the building is increased significantly from its fixed-base period. The increased flexibility of the seismically isolated structure is its most important feature and results in the avoidance of resonance and the decrease of the accelerations of the structure at different levels. A very rigid structure has very small relative displacements but accelerations equal to the ground acceleration. Seismic isolation actually takes advantage of the benefits of both the above-mentioned extreme cases, reducing both inter-storey deflections and floor acceleration. Inter-storey deflections are reduced because of the rigid body motion of the superstructure. The superstructure is relatively very stiff compared to the inserted flexibility at the isolation level. The reduction of inter-storey deflections of the structure minimizes or even eliminates structural or non-structural damage.
The observed fixed base time period (T_i) and base isolated time period (T_b) are marked on the multi-hazard spectrum plot as shown in Figure 5.8.

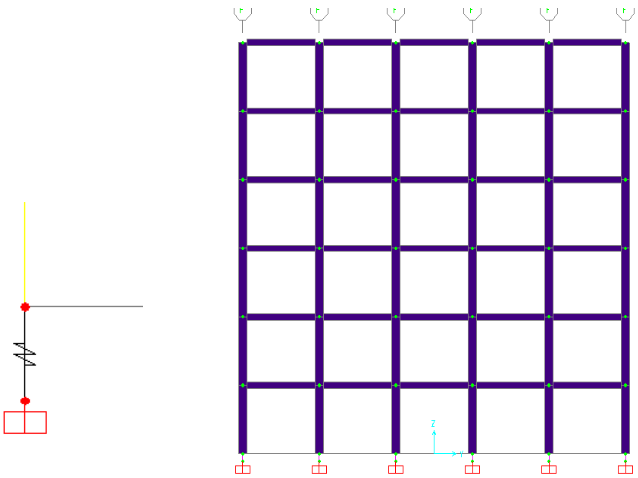
Fig 4.8 elevation of base-isolated building
The response can be clearly seen in the model and the time period of the building is increased significantly from its fixed-base period. The observed fixed base time period (T_i) and base isolated time period (T_b) are marked on the multi-hazard spectrum plot as shown in Figure 5.9.

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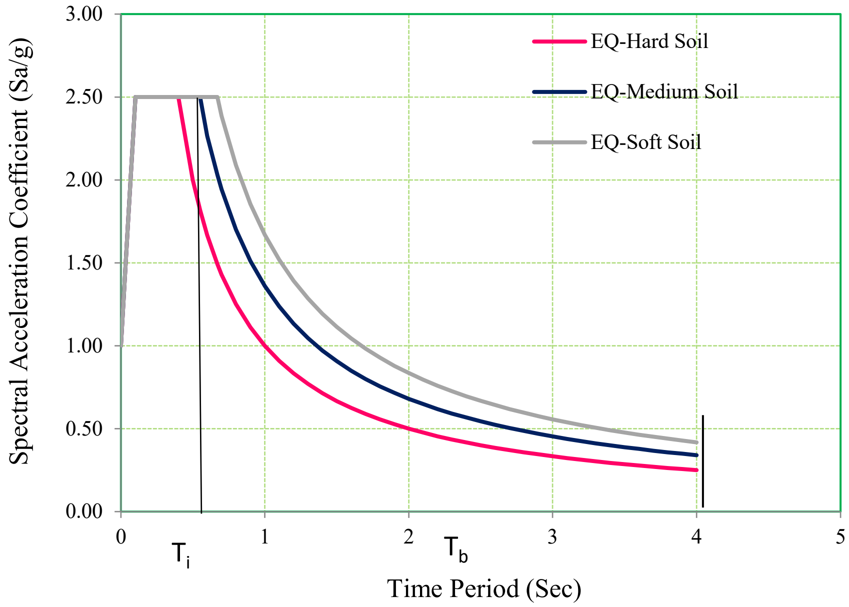

Fig 4.9. Initial and base-isolated time period representation on the Response spectra curve

Analytical studies on multi-storied buildings with base isolation to meet contra requirements in a multi-hazard environment, with specific reference to earthquakes and strong winds, are performed. The base isolation system design is made in such way that the change in the natural time period of the building by adding the base isolation should help in lowering the seismic forces and also the same shift in the time period concept is effectively used from wind point of view in such way that the time period shift is not in a peak portion of the cross wind force spectrum and then reduces the wind load vulnerability. The multi-hazard spectrum developed using Cross wind force spectrum (AS/NZS 1170.2:2011) and Earthquake Response spectrum (IS 1893 (Part 1):2002) is used for further analysis.
The obtained fixed base time period (T_i) and base isolated natural time periods (T_b) are plotted on the multi-hazard spectrum as shown in Figure 4.10.

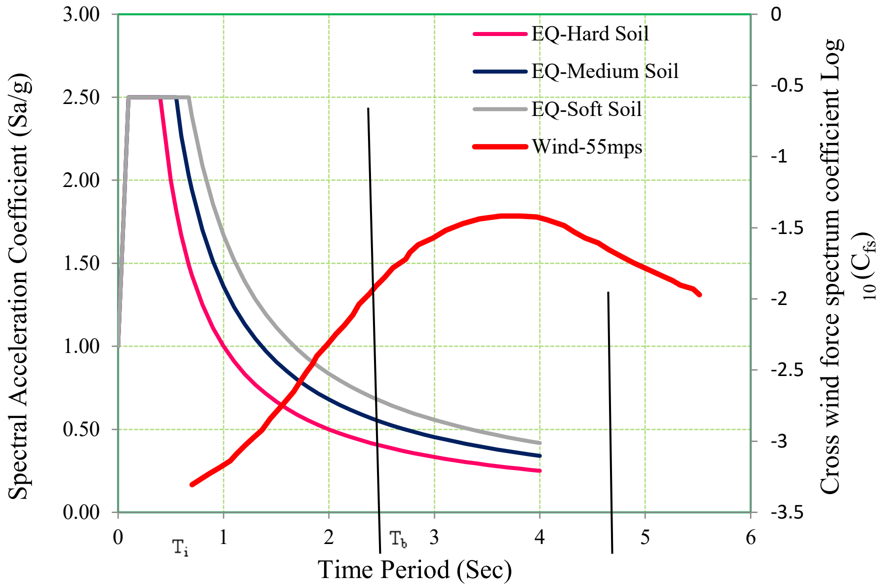

Fig 4.10 Multi-hazard spectrum with fixed and base-isolated time period

RESULTS AND DISCUSSIONS

5.1. GENERAL

In this method, the aim is to anticipate and coordinate how the building and its system interact, how mitigation of one hazard can influence the building's vulnerability to others, and how undesirable conditions and conflicts may be avoided or resolved.

Instead of using a conventional method to design a structure for critical loading, multi-hazard spectrum analysis makes the structure less vulnerable to both seismic and wind loads. A 6-storey moment resisting framed building is modeled and analyzed in SAP2000. When the natural period obtained from this analysis is mapped in the response spectrum curve, it is observed that the building is in the peak range for seismic load, i.e., in the region of maximum spectral acceleration coefficient. So, in order to make the structure safe against seismic loads, isolators are incorporated at the base, and analysis is carried out for this modified structure. From the response spectrum curve, it is evident that the time period of the base-isolated building has increased significantly from its fixed-base period. The structure has become flexible and safe against seismic loads due to an increase in the natural period of the building.
The natural time period of the building for both fixed and base-isolated cases in the X direction, Y direction, and torsion are presented in a tabular form below.

Table 5.1: Time period of fixed base structure

MODE	FIXED BASE TIME PERIOD (T_i)		
	X-DIRECTION	Y-DIRECTION	TORSION
1	0.656	0.58	0.602

Table 5.2: Time period of isolated base structure

MODE	TIME PERIOD OF BASE ISOLATED BUILDING(T_b)		
	X-DIRECTION	Y-DIRECTION	TORSION
1	2.70	2.675	2.658

From the above results, it is evident that the building is flexible and the acceleration of the ground motions cannot be transferred to the structure above, making it safe against seismic loads.
The structure gets deflected due to the loads acting on it. The deflections of the building with and without base isolation are shown in Figures 5.1 and 5.2. It is seen that the base-isolated building moves as a rigid body above the isolators.

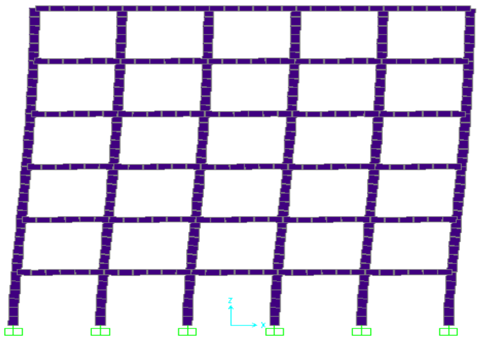

Fig 5.1 deflection of fixed base building

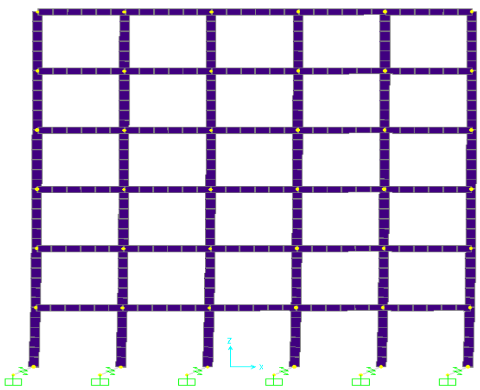

Fig 5.2 deflection of base isolated building

The new amplified time period, when marked in multi hazard spectrum, the structure fell into wind prone region. By tuning the base isolation design, the time period of the structure is optimized where it can resist seismic and wind loads.

The observed fixed base time period (T_i), base isolated time period (T_b) and the optimum targeted time period (T_f) are marked on the multi hazard spectrum plot as shown in Figure 6.3.

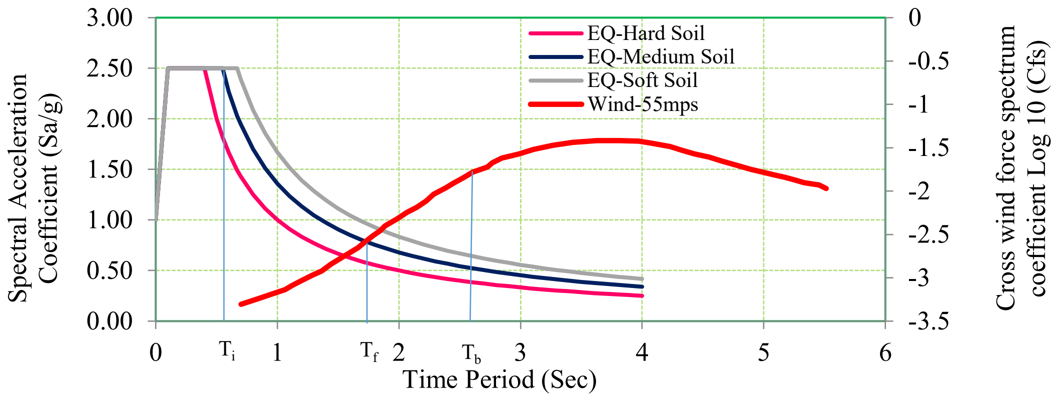

Fig 5.3 time period of the building in different conditions

Isolator design is tuned for the optimum natural time period which is obtained from the multi hazard spectrum.

5.2. Final Isolator Design

The stiffness (k) of the isolator is derived from the following equation.

$$\omega = \sqrt{\frac{k}{m}}$$

Where,
k= stiffness of isolator
m =mass acting on column
m = Pu = 924kN
ω = natural frequency

$$\frac{2\Pi}{T} = \sqrt{\frac{k}{m}}$$

$$\left(\frac{2\pi}{1.7}\right)^2 = \frac{k}{m}$$

$$\frac{k}{m} = 13.66$$

Stiffness $K = 13.66 \times \left(\frac{924\times10^3}{9.81}\right)$

=1286632N/m
=1286.632N/mm

Maximum spectral displacement of isolator

$$Sd = \frac{T^2}{4\pi^2} \times s_a \times Z$$

Where,
S_a= Spectral acceleration of the isolator
Z= Zone factor for maximum considered earthquake

$$Sd = \frac{1.7^2}{4\pi^2} \times 0.6 \times 0.36 \times 9.81$$

$$Sd = 0.155m$$

$$\frac{S_d}{t} = ɣ$$

$$\frac{0.155}{t} = 0.75$$

Thickness of rubber $t = 0.206\text{mm}$

Horizontal stiffness of the isolator, $K_H = \frac{\text{load}}{\text{deflection}}$

Where,

Load=Shear stress × Cross sectional area

Therefore, $K_H = \frac{GA}{t}$

$$1286632 = \frac{0.0009 \times 10^9 \times A}{0.206}$$

Area of isolator rubber A = 0.295m^2

Size of rubber B = 0.543m

The shape factor $S = \frac{1}{\sqrt{6}}\frac{f_V}{f_H}$

Where,

$$f_H = \frac{1}{T} = \frac{1}{1.7} = 0.588\text{HZ}$$

And it is assumed that $f_V = 20H = 11.76\text{HZ}$

$$S = \frac{1}{\sqrt{6}}\frac{11.76}{0.588} = 8.179 \cong 10$$

The shape factor is also given as

$$S = \frac{\text{Loaded area}}{\text{Forcefreearea}}$$

$$S = \frac{B}{4t_r}$$

Where,

t_r= Thickness of rubber between two steel plates

$$t_r = \frac{B}{4S} = \frac{0.543}{4 \times 10}$$

$$t_r = 0.0135\text{m} = 13.57\text{mm}$$

Number of rubber layers $nt = \frac{206}{13.57} = 16$

Number of steel plates $n = n - 1 = 16 - 1 = 15$

Total height of the isolator $= 206 + (16 \times 1) + (20 \times 2) = 261$mm

Mass of the rubber $M_R = pV$

$$= 930 \times (0.295 \times 0.206)$$

$$= 56.51\text{kg}$$

Weight of rubber $W_R = \frac{56.51 \times 9.81}{1000} = 0.554\text{kN}$

Mass of steel $M_S = pV$

$$= 7859 \times (0.295 \times 0.001 \times 15)$$

$$= 34.73\text{kg}$$

Weight of steel $W_S = \frac{34.73 \times 9.81}{1000} = 0.340\text{kN}$

Total mass M = 56.51+ 34.73 = 91.246kg

Total weight W = 0.554+0.340=0.895kN = 895N

Rotational inertia $RI = \frac{MR}{12}(l^2 + t^2)$

$$RI_X = RI_Y = \frac{56.1}{12}(0.543^2 + 0.543^2)$$

$= 27769861 \text{ N-mm}^2$

$$RI_Z = \frac{56.1}{12}(0.543^2 + 0.206^2)$$

$$= 15883312 \text{ N} - \text{mm}^2$$

Horizontal stiffness $K_H = \frac{GA}{t}$

$$= \frac{0.0009 \times 10^9 \times 0.295}{0.261}$$

$$= 1017.24 \frac{N}{mm}$$

Vertical stiffness $K_V = \frac{E_C A}{t}$

$$= \frac{0.05 \times 10^9 \times 0.295}{0.261}$$

$= 56513.4 \text{N/mm}$

From the base isolated design made for the optimum time period of the building it can be observed that The height of the isolator, area of isolator reduced comparatively from the prior base isolation design and the stiffness of the isolator increased. It can also observed that the final isolator design is a moderate design both for seismic and wind loads.
Calculated rubber isolator properties are assigned as link/ support properties in the software and the base-isolation model analysis is performed for the structure.
The natural time period of the building for optimum time period of building in X direction, Y direction and torsion are presented in a tabular form below.

Table 5.3: Time period of tuned base isolated structure

MODE	FINAL OPTIMUM TIME PERIOD OF BUILDING(T_f)		
	X-DIRECTION	Y-DIRECTION	TORSION
1	1.602	1.573	1.542

The time period obtained for seismic modeling is modified by using base isolation technique and the optimum time period is obtained which is safe for both wind and seismic loading.

CONCLUSIONS

Many regions in India are vulnerable to multi-hazards such as earthquake and winds. The need to embrace a multi hazard approach when designing structures is essential for their protection in these regions. In this method, the aim is to anticipate and coordinate how the building and its systems interact, how mitigation of one hazard can influence the building's vulnerability to others, and how undesirable conditions and conflicts may be avoided or resolved.
In the present study, a new methodology is introduced for a multi hazard design of structures for the locations where seismic and wind loads are critical and require an optimum design. A 6 storey moment resisting frame is considered for the demonstration of the methodology and the computer aided software SAP2000 is used for the analysis and design. The well known seismic control technique base isolation is adopted to reduce the seismic response of the building and to prevent the structures from earthquake damage. The most popular elastomeric bearings which consist of thin rubber sheets bonded onto thin steel plates and combined with an energy dissipation mechanism are used for base isolation.
From the base isolation analysis results, it is observed that the use of base isolation technique increases the flexibility of the structure and results in the avoidance of resonance and the decrease of the accelerations of the structure at different levels. It reduces both inter-storey deflections and floor acceleration. The inter-storey deflections are reduced because of the rigid body motion of the superstructure and the superstructure is relatively very stiff compared to the inserted flexibility at the isolation level. The reduction of inter-storey deflections of the structure minimizes or even eliminates structural or non-structural damage. In addition, the lengthening of the fundamental period due to the inserted flexibility reduces the accelerations significantly. This is an important achievement, in particular for buildings with properly dimensioned seismic isolators can be used to control the distribution and the way of transfer of the lateral forces. For a particular value of axial load which is to be transferred, the area

of the isolator increases, as an isolator with the rubber of lesser value of shear modulus is used. The diameter of isolator decreases as the desired value of time period increases, provided the soil condition and the rubber used are the same. Buildings situated on soft soils need isolator with a bigger diameter than situated on hard rocky soils. Both thickness and diameter increases as greater earthquake prone area are chosen.

Seismic isolation may be used for new structures in combination with methods of construction which are usually not suitable for earthquake resistance design since the induced seismic forces are substantially decreased without the need for high ductility. It is evident from this discussion that seismic isolation is an efficient design scheme which successfully addresses earthquake loadings, and not only provides safety but also prevents damage. Seismic isolation is particularly useful for low to medium rise buildings which happen to have their fundamental frequencies within the dangerous for resonance range of dominant earthquake frequencies, critical facilities, such as emergency response centers, hospitals, fire-stations utilities and communication centers should remain operational even during the strongest earthquake. The disruption of the operation of such essential, for the public interest, facilities may prevent by using seismic isolation of enhancing their earthquake capacity by reducing the seismic loads that they may experience during a powerful earthquake. It is also very useful, and is probably the only currently available technology that can be used, to seismically upgrade historic structures or to protect very sensitive equipment and the valuable contents of a building. In the former cases, seismic isolation can decrease the seismic vulnerability of a historic structure without impairing its distinctive architectural characteristics, by reducing the induced seismic forces.

The considered isolation design is tuned to make the structure safe against wind loads by reducing the height and area of the isolator. The stiffness of the isolator increases as the dimension of the isolator and thus the flexibility of the isolator is adjusted to the optimum requirement of the seismic and wind-resistant design. The optimum natural time period of the structure is observed from the multi-hazard spectrum, which is a tripartite plot made using the wind spectrum and response spectrum by bringing them on a common platform. This method avoids laborious calculations and time, unlike conventional designs for multi-hazard design of structures. Thus, there is a scope for future development of the multi-hazard-resistant design. So this method can be adopted for future research work, keeping this as a reference.

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