Seismic analysis of multi-storey building exposed to Fire

A Thesis Submitted
in Partial Fulfillment of the Requirements
For the degree
of

MASTER OF TECHNOLOGY

In
Structural Engineering

By

Shivansh Pandey
(University Roll No. 1180444011)
Under the Guidance of
Mr SHUBHRANSHU JAISWAL
(Asst. Professor)

BABU BANARASI DAS UNIVERSITY
LUCKNOW

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CERTIFICATE

Certified that SHIVANSH PANDEY (1180444011) has carried out the research work presented in this Thesis entitled **"Seismic analysis of multi-storey building exposed to fire"** for the award of **MASTER OF TECHNOLOGY (Structural Engineering)** from BABU BANARASI DAS UNIVERSITY, LUCKNOW under my supervision. The Thesis embodies results of original work, and studies are carried out by the student himself and the contents of the Thesis do not form the basis for the award of any other degree to the candidate or to anybody else from this or any other University/Institution.

Mr. SHUBHRANSHU JAISWAL

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Lucknow.

DECLARATION

I hereby declare that the Thesis entitled **“Seismic analysis of multi-storey building exposed to fire”** in the partial fulfilment of the requirements for the award of the degree of Master of Technology (Structural Engineering) of **BABU BANARASI DAS UNIVERSITY**, is the record of the own work carried under the supervision and guidance of **Mr SHUBHRANSHU JAISWAL** to the best of my knowledge this Thesis has not been submitted to **BABU BANARASI DAS UNIVERSITY** or any other University or Institute for the award of any degree.

SHIVANSH PANDEY
[1180444011]

ABSTRACT

The purpose of this thesis to study the work that has been done before to observe or measure the effect of high temperature caused by the fire on the reinforced concrete structure. However, reinforced concrete structures are considered to be fire-protected by the reinforcement cover. After exposure to fire, reinforced concrete structures lose strength and durability, but long periods of heat exposure cause physical-chemical changes in concrete properties accompanied by degradation of mechanical strength. To ascertain the integrity of the building, a visual inspection was conducted for all elements (truss, beam, column and slab), followed by a non-destructive test such as rebound hammer test and Ultrasonic pulse velocity test. It is seen that after the exposure of fire the building is usually reconstructed or demolished so by various techniques such as NDT testing method we can do health monitoring of reinforced concrete structure which is damaged by fire. After doing health monitoring we will able to predict the reduced compressive and tensile strength of RCC structure. After health monitoring, we will able to know that the building is safe for re-use after doing repair and retrofitting or else we have to demolish it. If the building is safe for reuse after doing some retrofitting works then the problem is that the building is safe for seismic forces or not. In this thesis, the work is to analyze fire-damaged building at the various temperature on its reduced strength of RCC on ETABS. In this study we will first prepare a model of a building by normal building design material after that we will design three new building models by using reduced strength materials which are predicated before in various research papers at a temperature such as 300°C,500°C,600°C. After that, we will do the seismic analysis of these four building models and do a comparative study of story displacement, story drift, story stiffness.fundamental time period

ACKNOWLEDGEMENTS

I wish to express my deepest gratitude and indebtedness to my supervisors, **Mr Shubhanshu Jaiswal** and the Head of Department **Mr Anupam Mehrotra** for their stimulating ideas, numerous constructive suggestions and guidance, continuous encouragement and invaluable support throughout this study. Without their advice, encouragement and support, this thesis would not be completed.

Finally, I would like to dedicate this research work to my family and friends whose continuous love and support guided me through difficult times.

SHIVANSH PANDEY
(M-Tech. Structural Engineering)

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CHAPTER 1 INTRODUCTION

At present, India is growing at a very fast pace, creating enhanced demand for infrastructure services with population development. Due to the increase in population demand for housing is also increasing, so to fully meet the need for construction of housing and commercial buildings the use of multi-storey buildings is currently very much in demand. These forms of construction need protection for both lives and property, as multi-storey buildings are particularly vulnerable to increased lateral loads due to earthquakes and wind.

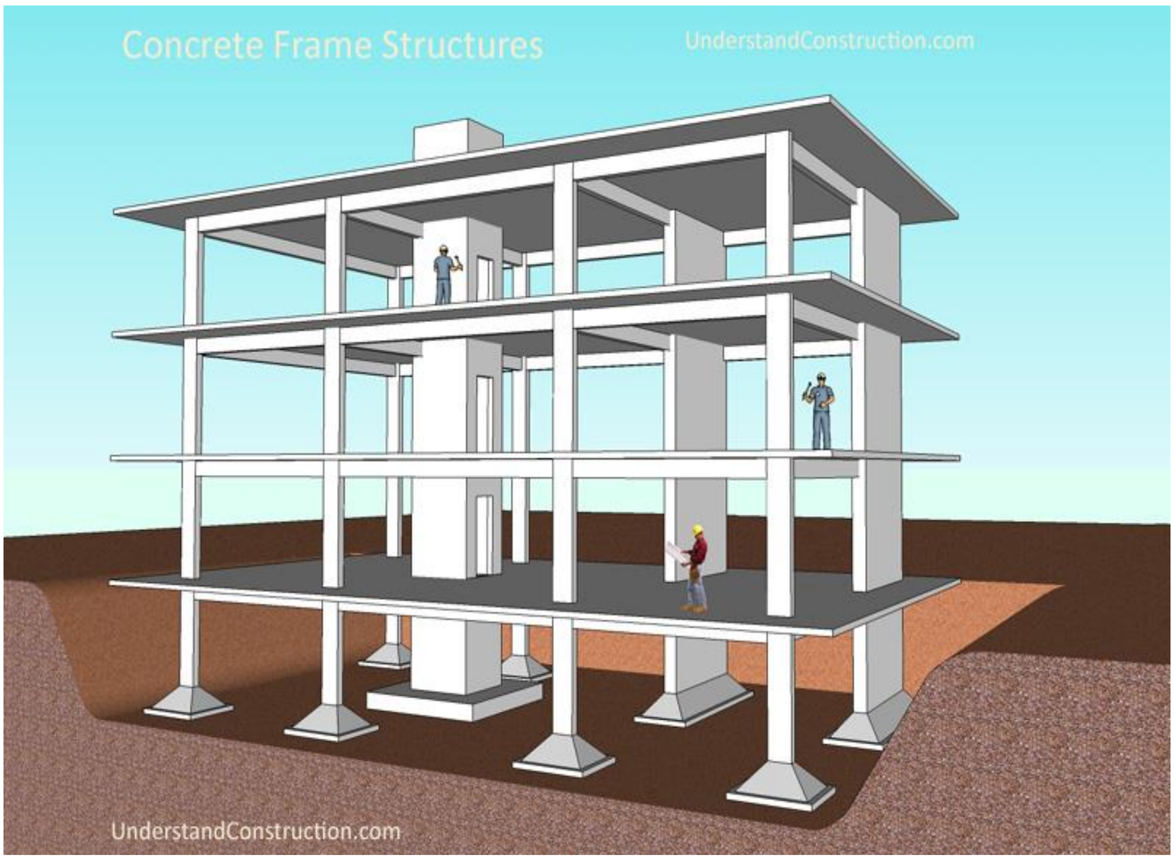

Figure 1.1- Concrete frame structure

Concrete has become a universal building material and more suitable structural forms, such as shear walls and tube structures. The high dead load characteristics are not limited to the height of the concrete building. Therefore, the dead load from concrete appears to be more important in reducing sway deflection and floor-shaking, as well as the issue of instability.

The scale of the building can not be specified in precise terms referring to the height or number of floors. Building to be considered as high when lateral loads are affected by structural analyzes and design. Lateral loads tend to control the structural system and are extremely critical in the overall construction framework as the height of the building rises..

Vertical loading, lateral loading effects on the building are very complex and escalate exponentially with a rise in height. Strength, rigidity and stability were three key considerations to be included in the construction of these structures. There are two ways to meet these requirements, which may be by increasing the size of the member beyond the strength requirement or by changing the shape of the structure into something more rigid and stable.

Reinforced concrete frames are the most common construction practices in India, with an increasing number of high-rise structures adding to the landscape. Several significant Indian cities fall into extremely active seismic areas. These high-rise buildings, particularly in highly sensitive seismic zones, should be studied and engineered for ductility and built with an extra lateral stiffening system to enhance their seismic strength and minimize risk.

It is generally recognized that the seismic design of buildings should meet at least two basic requirements. First, the structure must act elastically and protect relatively brittle non-structural components against minor earthquakes ground shaking. Therefore, the system will have sufficient capacity and elastic stability to limit structural displacements, such as inter-story drift. Second, the structure must not collapse in the event of a major earthquake. Significant damage to the structure and non-structural components is acceptable in this case. To prevent the structure from collapsing and thus minimize the loss of life, it must have a large energy dissipation capacity during a large inelastic period deformation. In general, structural systems with stable hysteretic loops perform well under the large inelastic cyclic loading characteristics of major earthquakes. These robust hysteretic properties of the system can be accomplished such that the structural elements and joints are engineered to provide adequate ductility.

Earthquakes are the most life-threatening and disruptive phenomenon; they are triggered by the rapid release of energy in the Earth's crust that generates seismic waves that occur at different instances with different intensity level. When the earthquake happens, the house collapses and the destruction done by the earthquake. A ground motion which is radiated in all directions from the epicentre. Due to the effect of the earthquake, the building encounters the highest level of displacement, the inertial force caused by the tendency of the building to remain at rest. However, lateral instability is a big concern, when constructing a multi-story building and seismic zones are often regarded when planning a multi-storey structure.

The word 'earthquake' is used to express any seismic occurrence, whether natural or caused by humans, which may produce seismic influence in any particular area. Earthquakes are generally caused by the rupture of geological faults within the earth, but also by other events such as volcanic movements, landslides, mine blasts and nuclear tests.

In Seismic Analysis, we learn that earthquakes are the most volatile, terrifying and unpredictable of all-natural disasters in which life and engineering properties are very difficult to save. Care needs to take charge of every phase in building construction from the foundation stage. As earthquakes occur, the structure is experiencing dynamic motion. Because of the inertia forces that may act in the opposite direction to accelerate the earthquake excitations. This inertia forces, typically called seismic loads, are dealt with by assuming forces outside the structure.

To solve these issues, we need to define the seismic efficiency of different buildings through numerous analytical procedures. This is also to make sure that the different structures withstand seismic incidents. This will also save as many lives as possible. During the earthquake, the performance of the structure depends on many factors, such as stiffness, adequate lateral strength, and simple and regular configurations etc.

1.1-SEISMIC ZONE

The earthquake zoning map of India divides India into 4 seismic zones (Zone 2, 3, 4 and 5). According to the present zoning map, zone 5 expects the highest level of seismicity whereas zone 2 is associated with the lowest level of seismically.

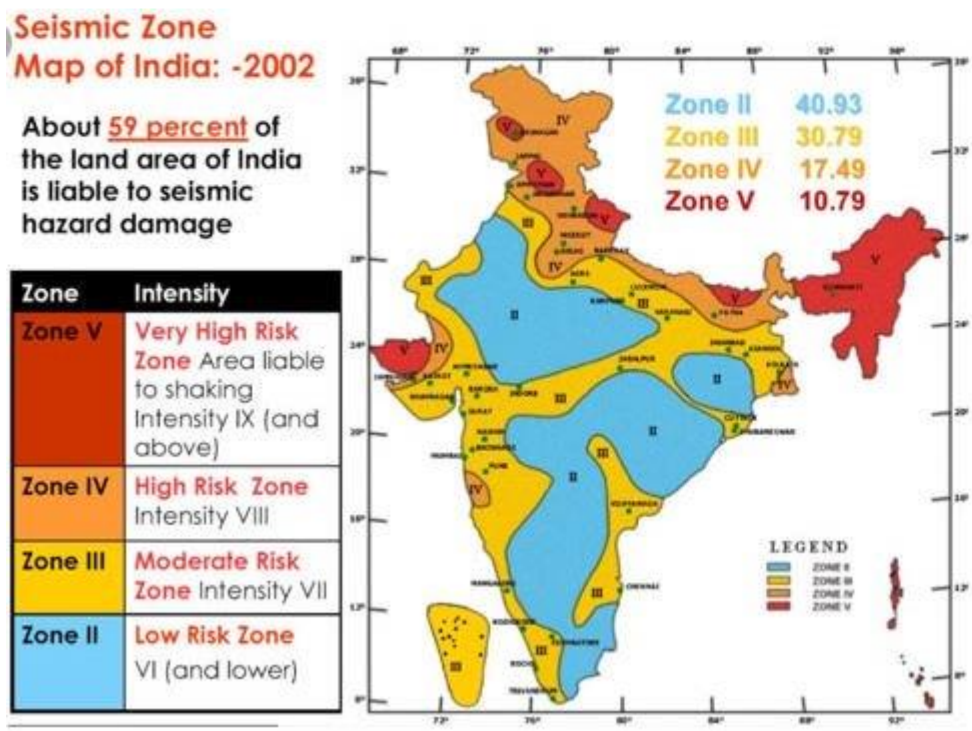

Figure 1.2- Seismic zones in India

Throughout earthquake movements, deformations occur through the elements of both load-bearing structures. As a result of these deformations, internal forces develop across the load-bearing system and result in the building being moved. In order to keep the structure stable and safe from collapse, seismic analysis of the structure must be carried out. In general, steel is used as a construction material to improve ductility. Steel requires less construction time and can be constructed in every season. They are lightweight and can be quickly adjusted according to the needs of the design. There are many known and practised measures to protect against seismic threats. Let’s take a look at some of the earthquake-resistant techniques used by the engineer's world over to minimize the damage to structures due to earthquakes:

1.2 Ways to improved seismic performances of the reinforced concrete frame.

1.2.1 Floating Foundation

The levitating or floating foundation distinguishes the substructure of a building from its superstructure. One method of achieving so is by floating a building over its base on lead-rubber bearings that form a solid lead core surrounded by alternating rubber and steel layers. With the help of steel plates, the bearings are attached to the building and its foundation. And, as an earthquake happens, the floating base will move without raising the frame above it. In Japan, this base insulation device works at a whole different stage. Their design allows the buildings to float in mid-air. The device levitates, holding the house in a cushion of air. The system has built-in sensors for detecting seismic activity, and these sensors communicate with the air compressor that creates a layer of air between the building and its base

1.2.2 Shock Absorption:

Compared to the shock absorbers used in cars, houses do utilize this engineering. The earthquake-resistant equipment allows structures to slow down and raising the frequency of vibratory motions. Ideally, shock absorbers will be mounted at each stage of the building – one end of the beam and the other end of the column. Each one consists of a piston head that moves inside a cylinder full of silicone oil. During earthquakes, the horizontal movement of the building will push the piston against the oil, transforming the mechanical energy from the earthquake to heat.

1.2.3 Rocking Core-Wall:

New high-rise buildings use this methodology to enhance low-cost seismic resistance. To order to perform this job, a reinforced concrete foundation is built down to the centre of the building, flanked by elevator banks. Many modern high-rise buildings use this technique to increase seismic resistance in affordable manner this functions better when used along with the foundation insulation. Elastomeric bearings are built with alternating layers of steel and natural rubber/neoprene for foundation insulation. The resultant bearing has weak horizontal stiffness and vertical stiffness. The mixture is extremely successful, cost-effective and simple to execute.

1.2.4 Pendulum Power:

The pendulum power technique works by suspending a massive mass near the top of the structure. This mass is protected by steel cables and viscous fluid dampers are positioned between the mass and the building it supports. In the event of some seismic activity, the pendulum swings in the opposite direction to overcome the force. Each pendulum is tuned to sync with the natural frequency of the structure, and these systems are called tuned mass dampers. We aim to combat resonance and will the dynamic reaction of the system.

1.2.4 Symmetry, Diaphragms And Cross-Bracing:

In general, symmetry is a standard requirement for seismic designs. Seismic threats in asymmetrical nature are greater. L-shaped, T-shaped and split-level designs can be more physically appealing, but they are often susceptible to torsion. Therefore engineers build symmetrical structures to maintain the forces equally distributed across the framework and to restrict ornamental features such as cornice, cantilever projections etc. The earthquake has a strong lateral impact. Seismic architecture counteracts these effects in both horizontal and vertical dynamic structures. Diaphragms are integral to horizontal structures – such as building floors or roofs. Engineers build each diaphragm on their deck and stabilize it horizontally such that sideways forces of vertical structural components can be spread. With vertical structures, engineers have several approaches.

1.2.5 Base isolation –

The basic idea of the base insulation method is to change the reaction of the building framework so that the earth underneath it may move efficiently without transferring certain forces of motion to the above-mentioned building structure. It is accomplished by removing or isolating the superstructure from its substructure resting on a shaking ground. As a result, the structure has less impact due to the earthquake force.

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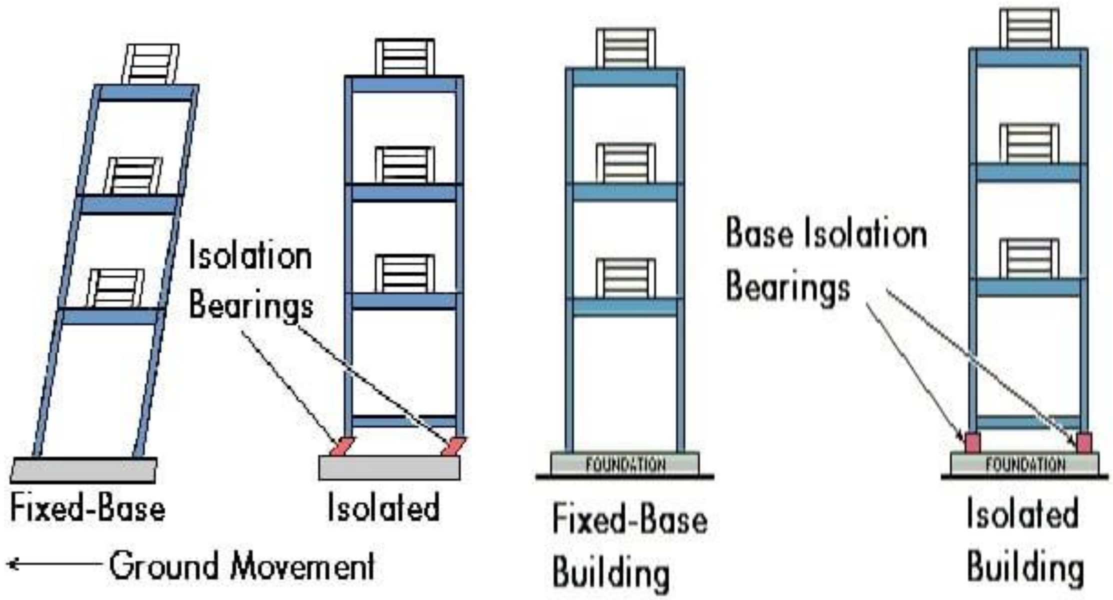


Figure 1.3- RC Residential Building with Base isolation

1.3 OBJECTIVES

The objective of this research is focused on various techniques used to study the seismic behaviour of fire exposed building at the various temperature on its reduced strength of RCC on ETABS.In this study we will first prepare a model of a building by using normal building design material after that we will design three-building models according to new material properties with a reduce strength which is predicted before in my review paper which is based on the study of various literature on a various temperature such as 300 °C, 500°C,600 °C and then we will do seismic analysis these four building models and do a comparative study of story displacement, story drift and story stiffness and fundamental time period behaviour of R.C buildings with seismic zone IV of India using an equivalent static method. The whole design was carried out in ETABs which covers all aspects of structural engineering. More specifically, the salient objectives of this research are:

1) To perform a comparative study of the various seismic parameters.
2) Comparison on the basis of story displacement, storey drift, Storey Stiffness & fundamental time period on four models .

3) The study will provide an approximate idea of how fire exposed building will perform in seismic forces.

In this report, a multi-storey residential building is studied for earthquake and wind load using an equivalent static method on ETABs. This analysis is carried out by considering seismic zone IV, and for this zone, the behaviour assesses by taking the medium soil. A different response for displacements, storey drift, storey stiffness and fundamental time period is plotted for zone IV for a medium type of soil.

1.4 SCOPE OF THE PRESENT STUDY

The study will provide an approximate idea of how fire exposed building will perform in seismic forces and can be used future after exposing to various elevated temperature under seismic forces

2 ORGANIZATION OF THE THESIS

The thesis is organized as per detail given below:

Chapter 1: Introduces to the topic of thesis in brief.

Chapter 2: Discusses the literature review i.e. the work done by various researchers.

Chapter 3: Work methodology for the analysis

Chapter 4: Analysis and Result

Chapter 5: Conclusions

2.2 Introduction

Through the years, many cases of open fire have happened leading to unusual situations such as explosive use, environmental crises, and accidents. Nevertheless, reinforced concrete buildings are classified as fire safety by its cover present over the reinforcement. Long cycles of exposure to elevated temperatures induce Physico-chemical modifications in concrete materials, which lead to a decline in mechanical resistance. The fire tolerance of a concrete building is typically well over its minimum specifications, and thus rehabilitation is favoured over destruction or rebuilding. Observation of the destruction can assess the extent and length of the fire using a variety of test methods and instruments available to evaluate the effect of the fire on both the structures and structural components. Such evaluations, together with the technical analysis, enable for the development and deployment of effective and economical repair knowledge as needed [8]. Non-destructive testing (NDT) techniques play a key role in the assessment of reinforced concrete structural safety systems (SHAs). Deformation of concrete structures, cracks, honeycombs and voids attributable to the operation, wear and tear environmental conditions. These defects can further affect the quality of concrete structures due to corrosion/damage of the steel reinforcement and of the concrete itself. Different NDT methods have been developed to evaluate these defects and, ultimately, to enhance structural protection during structural service life experiments are often performed in which the two measurement methods are used to calculate the compressive power of the concrete structure using the ultrasonic pulse velocity (UPV) and the impact rebound hammer (IRH). High temperatures are responsible for the degradation of the concrete microstructure and the weakening of its vital capacity, and thus this step is an appropriate alternative to UPV and other NDT approaches. The NDT methodology alone is not adequate to predict reinforced concrete structural safety and integrity. Concrete safety assessment shall be carried out with the assistance of the aggregated non-destructive technique. For this analysis, the concrete strength was measured using a combination of ultrasonic pulse velocity and hammer rebound techniques. Yet even less was used to assess the health of fire-damaged buildings.

2.2.3 Nondestructive Testing

Non-destructive (NDT) methods are used for the analysis of large-scale concrete structures. NDT is a test method that does not compromise the supposed overall effectiveness of the member being investigated. This can be achieved before and after construction, repair or commissioning. These assessments shall be carried out during construction to ensure quality control and strength testing in fresh concrete works as well as in existing structures to determine their structural ability and material degradation against time or the environment. Standard control cubes can not determine the strength of the concrete developed on-site, they can not have an accurate measure of the concrete in use during construction. The selection of the investigator is the extraction and examination of the cores taken from the concrete structure.The extraction of cores, though, is costly and can weaken the structure. Researchers have therefore established various NDT methods that allow for the in situ measurement of certain concrete properties from which an estimate of concrete strength can be made. Many of the NDT methods used include a visual examination of concrete buildings, the Schmidt or rebound hammer testing and the UPV check. Other properties can be calculated using NDT and partly destructive measurements such as stiffness, elasticity and compressive strength units, surface hardness and surface absorption, and location of the steel bars, size and distance from the surface. Two tests are typically used in NDT for the safety assessment of the fire-damaged system of ultrasonic pulse velocity and rebound hammer technique.

2.2.4 Rebound hammer testing

The Rebound Hammer is an device that offers the relative compressive strength of a concrete or other construction material depending on its stiffness on the exposed surface, consisting of an exterior frame, a plunger, a hammer mass, a spring, a latching mechanism and a slipping rider and a scale in which the rebound amount is indicated .The measurement required a smooth surface and the contact point of impact must be 20 mm away from the discontinuous edge or other sharp edge. When the plunger is forced towards the concrete surface, the mass falls out from the plunger and retracts to the power of the spring [20]. The hammer impacts into the concrete and the spring touch mass recovers, bringing the driver down the directed scale. The driver shows the distance travelled by mass named the rebound number The rebound amount is calculated on an approximate linear scale of 10 to 100 . The NDT rebound hammer test estimates the compressive strength of the concrete with the aid of the rebound number, which relies on the rebound of the spring-controlled mass and the surface toughness of the concrete It is worth noting that the rebound amount relies solely on the surface state of the concrete and is not linked to internal mechanical properties for which IRH alone is not adequate to quantify the structural compressive power Figure 2.1 shows a cutaway schematic view of the Schmidt or the rebound hammer .

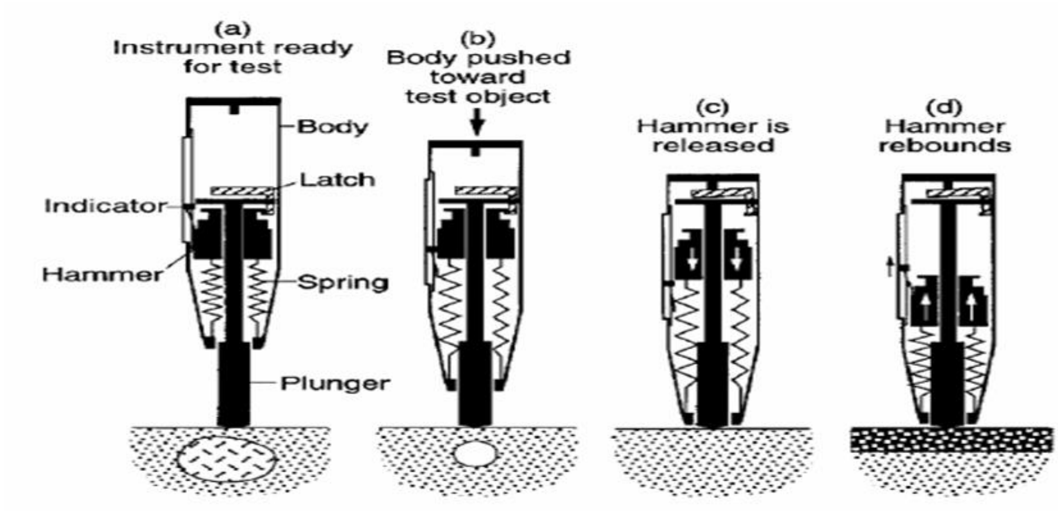


Figure 2.1 Cutaway view of the rebound hammer (2) (Source- Dr Balaji KVGD, V. Sannidha)

2.2.5 Ultrasonic pulse velocity method testing

For more than 60 years, this measuring technique has been used to evaluate the quality of the concrete. Checking equipment is simple to use for construction installations and laboratory experiments. The strength of ultrasonic pulses across the concrete relies on the elastic properties and the composition of the material. Areas of poor elasticity or low density, such as concrete damaged by burning, may also be detected through this method. This consists of a pulse generator, a transmission arm, a receiver arm and a measurement device. The timing of entry of the compression waves, for example. The waves propagating in concrete as quickly as possible are measured at the receiving head with the help of the tool. To prevent measuring mistakes, proper communication is obtained by keeping the heads under steady pressure using a thin layer of communication gel between the heads and the concrete sheet. The heads can be positioned in three separate forms, a direct form seen in Figure 2.2, where the angle between the heads is 180 degrees; the angle of the semi-direct method is 90 degrees, and the angle of the indirect surface system is 0 degrees. The direct path is favoured because the optimum pulse energy is received at the receiving head, but due to the structure of the device, this solution becomes complicated in many situations. The semi-direct technique can be used to avoid the accumulation of reinforcement. The indirect the approach does not provide as good measurements as direct and semi-direct approaches, but it can be used to determine the thickness of the low-quality layer and is utilized in cases when certain approaches are not usable. The heads are then placed close to each other while the low-quality layer is calculated using an indirect method and then moved farther forward. By measuring the time of arrival as a function of the gap between the head, the thickness of the poor layer may be determined in situations where this layer is distinct. If the heads are located next to each other, the waves scatter across the upper layer of the material where there is a wide amount of fine substance. This would generate a low velocity for the waves to pass across both the upper and lower layers as the distance between the heads increases. Shrinkage and delamination fractures should be taken into consideration when assessing fire-damaged concrete in the evaluation of findings

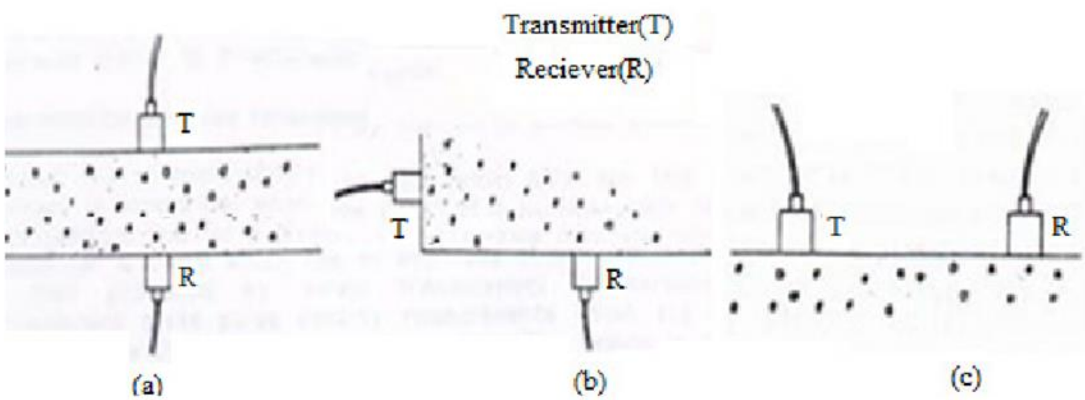


Figure 2.2 direct (a), Semi-direct (b) Indirect (surface) transmission (c)

2.3 LITERATURE REVIEW

2.3.1 General

A brief review of previous studies which focuses on recent contributions related to seismic analysis of multi-story building exposed to fire by various analysis method and past efforts most closely related to the needs of the present work

Ahmed Aseem, Waqas Latif Baloch, Rao Arsalan Khushnood*, Arslan Mushtaq (28 May 2019)- The analysis in this paper shows that IRH and UPV can be used as a reliable tool for predicting the mechanical strength of fire-damaged reinforced concrete construction. Nonetheless, the concrete core extraction is used in this analysis to create a concrete compressive strength relationship based on the Rebound hammer test and the ultrasonic pulse speed test. The microstructural work is also being performed which also informs us of the deteriorated condition of concrete based on scanning electron microscopy and thermal analysis of fire-damaged specimens. Extremely degraded specimens were well associated with lower concrete compressive strength values per microstructural analysis

James W. Jordan, Marc A. Sokol, John H. Stewart (2013) –In this paper, it is seen that the compressive strength of concrete can be greatly reduced from excessive heat, exposure to temperatures below the range of 250° to 300° C (482° to 572° F) are usually not considered to be significant. Shallow cracks may appear in concrete surfaces that reach approximately 300° C (572° F), and deepen with increased temperatures. Sometimes, concrete exposed to this temperature range may exhibit a pinkish hue. At approximately 370° to 430° C (700° to 800° F), concrete loses approximately half of its compressive strength. Cement constituents of the concrete decompose to a whitish powder consistency at approximately 900° C (1650° F). Steel reinforcement within concrete can typically sustain temperatures up to 450° C (840° F) for cold-worked steel, and 600° C (1100° F) for hot-rolled reinforcement steel, and still recover all of its original yield strength upon cooling. Higher temperatures may cause permanent loss of strength and ductility, possibly resulting in excessive deflection or failure of a reinforced concrete structural member

Prestressing steel within concrete has a greater sensitivity to heat exposure than mild steel reinforcement, losing approximately half of its strength at 400° C (750° F). Concrete that has changed colour to a pink or red shade is estimated to have reached a temperature range of 290° to 590° C (550° to 1100° F), which would be below the threshold of permanent damage to mild steel reinforcement. Concrete that has changed colour to a whitish-grey is estimated to have reached a temperature range of 590° to 950° C (1100° to 1740° F)Concrete that has changed to a buff colour is estimated to have exceeded a temperature of 950° C (1740° F). The strength of steel will start to decrease at approximately 430° C (800° F). At 590° C (1100° F) steel loses approximately 50% of its strength and stiffness when compared to normal ambient conditions. At 700° C (1300° F) the strength and stiffness are reduced to approximately 20% of the ambient condition strength and stiffness. These property reductions will likely be temporary, and the steel will regain its strength and stiffness if the temperature of the steel does not exceed 700° C (1,300° F) for longer than 20 minutes

M H Osman, N N Sarbini, I S Ibrahim, C K Ma, M Ismail and M F Mohd(2017)

In this paper, the severe damage is not seen for reinforced concrete elements, except

The spalling of mortar screeding was found in certain parts of column and beams. It is seen that in concrete surface there are no cracks are present. By various test such as IRH, it is seen that the concrete is still unaffected by the fire. In this study, the truss is tested and it was found that the truss is still strong. The maximum deflection is very small. The small deflection tells us is no structural degradation of truss member and connection which proves they are over-designed. After a various lab test, it is seen that the loss of 15% in tensile capacity in the least affected sample and loss of 19% in tensile capacity in most affected places And from other finding it is predicated that steel truss with characteristics strength of 400 N/mm2 to 460 N/mm2 will have strength loss of 30% from the actual strength when it is exposed to fire at temperature 800° C to 1000° C

Taehun Ha , Jeongwon Ko , Sangho Lee , Seonwoong Kim , Jieun Jung and Dae-Jin Kim (2 May 2016)- In this paper, both on-site and laboratory test has been performed. Through visual inspection, it is found that the slabs are mostly effected by fire and has maximum damaged it is because they are located at the highest evaluation of the room where they are exposed to highest temperature during a fire. The strength test on concrete cores from slabs, girders and beams shows the concrete and reinforcement In the girders and beams are less damaged and it is fit for further use but the reinforcement bars in slabs had very large damaged and its structural strength is reduced therefore it could not be used. By finite element analysis, the strength of the concrete structure in both the girders and column having 50mm cover reduced to 60% compared to original strength. In this study, it is advised that the slabs have to be replaced with a fresh one and the girder and beams is to be retrofitted .and the column and wall only needs surface treatment without structure retrofitting

Awoyera, P.O., Akinwumi, I.I., Ede, A.N, Olofinnade, M.O.(August 2014)-
In this study, it is seen that the average ultimate tensile strength of steel is decreased from 592.0 N/mm2 at 30 ° C (86 ° F) to 300.97 N/mm2 at 700 ° C (1292 ° F) for concrete beam having the cover of 20mm. Hence the loss in strength will be 49.2% of its original strength. In this paper, it is seen that the concrete element subjected to the fire at temperature up to 450 ° C are still serviceable. At a temperature of 450 °C, the concrete moisture has been absorbed by the fire and it will cause the cracks in the concrete which is due to expansion and contraction of the constituent material. But the whole structure is still serviceable however building subjected to a temperature above 600 °C is structurally unsafe to use. There is a 70% loss in strength of concrete above 600 °C temperature. In this paper, it is also seen that the value of UPV and RHN will decrease with increase in temperature

P.Srinivasan, A.Cinitha, Vimal Mohan and Nagesh R.Iyer (March 2014)- In this study, it is seen that when the concrete is exposed to the fire at a temperature between 300 °C to 600°C the colour of concrete changes from greenish-grey to pink. After performing UPV test the ultimate stress of the reinforcement changes from 561.5 N/mm2 to 400 N/mm2 at 500°C and yield stress from 461 N/mm2 to 265.0 N/mm2 at 500°C. It is seen that there was

28.8% decrease in ultimate stress of the reinforcements at 500°C and the compressive strength will be 19.15 N/mm2 at 300°Cand 18.50 N/mm2 at 500°C

Ramadan E. Suleiman, Fathi M. Layas Omar F.Labbar and Vail karakale (Janury 2013) In this study it is seen that the maximum effect of fire is on compressive strength of concrete of the fire affected structure. It is seen that at distance 200mm from the heated surface the damage is much smaller hence the damage in the reinforcement is negligible after analysis it is seen that the slab element, ribs, beams and column appear to be still with sufficient reinforcement, however under the capacity of defeated concrete due to fire has to follow the treatment. The treatment is based on chipping away of all concrete which is damaged by fire according to the required strength. For the treatment of slabs, the use of shotcrete in layers of 30 to 44mm thickness is used. For ease construction, the non-shrinkage concrete of maximum size aggregate 10mm with 20 Mpa compressive strength is used in treatment. In this paper, it is seen that compressive strength is reduced 42% of its actual strength at temperature 600 °C.

Narendra K. Gosain (Sep2008)-In this paper it is seen that at temperature 290°C there will be no change in colour, physical appearance and strength of concrete. At temperature 290°C to 590°C, the colour changes from pink to red. The surface crazing will occur at temperature 300°C, and deep cracking will occur at temperature 550°C At temperature 290°C to 590°C the concrete condition will be sound but strength significantly started reducing. At temperature 590°C to 950°C the colour of concrete changes to whitish-grey. Spalling will be caused but exposing not more than 25% of reinforcement at temperature 800°C and at temperature 590°C to 950°C the condition of concrete will weak and friable. At temperature above 950°C, the colour of concrete will change to buff and there will be extensive spalling and concrete is very weak and friable

Jeremy Ingham (May 2015) –Through this article, it is shown that the residual compressive strength of structural-quality concrete is not greatly reduced for temperatures up to 300 ° C, whereas the remaining strength can be reduced to only a small fraction of its original value for temperatures above 500 ° C. There will be a decrease in steel strength while the material is at elevated temperatures. It is also possible to recover the yield strength after cooling for temperatures up to 450 ° C for cold-rolled steel and 600 ° C for hot-rolled steel. Higher temperatures may cause permanent loss of strength and ductility after cooling. The impact of excessive temperature is more critical on prestressing steel than on reinforcing steel. At temperatures of 200–400ºC, steel prestressing tendons show considerable loss of strength (>50% loss at about 400°C)

J.S. Kalyana Rama and B.S. Grewal (2015) –In this paper, its is estimated that at a temperature under 250°C the strength of concrete is will not change but temperature above 250°C the strength of concrete reduced by 9.4%, it will reduce further by 25.4% at temperature 500°C and temperature 1100°C the strength will reduce to 84.2% from its original strength before the fire. Concrete, although doesn't melt at a high temperature slightly change in shape will occur and this will cause a significant reduction in strength and cracks will also occur at a higher temperature. It is seen that when the fire is severe then the damage will be a very high extent compression to the fire at a small scale. It is also observed that the accuracy of Schmidt's rebound hammer test has been improved at a high temperature of the concrete. It is seen that the accuracy will be 69% when the test was conducted for cube heated at 500°C. At temperature 1100°C the rebound hammer test will be failed to give a reading for concrete specimen

2.3.2 CONCLUSION OF LETRETURE REVIEW

1. The non-destructive test is a way of testing which does not affect the overall performance of a member's entity under investigation. It could be performed during construction and after maintenance. The IRH and UPV can be used as a reliable method to predict the mechanical strength of the reinforced concrete structure.
2. It is observed that at the high temperature of the concrete the accuracy of Schmidt,s rebound hammer test is improved.
3. At temperature 1100°C the rebound hammer test will be failed to give the reading for concrete specimen

4. It is found that the slabs are mostly effected by fire and has maximum damaged it is because they are located at the highest evaluation of the room where they are exposed to the highest temperature during a fire
5. The strength of steel will start to decrease at approximately 430° C (800° F). At 590° C (1100° F) steel loses approximately 50% of its strength and stiffness when compared to normal ambient conditions. At 700° C (1300° F) the strength and stiffness are reduced to approximately 20% of the ambient condition strength and stiffness. These property reductions will likely be temporary, and the steel will regain its strength and stiffness if the temperature of the steel does not exceed 700° C (1,300° F) for longer than 20 minutes

Heating temperature	Colour change	Mineralogical change	Change in physical appearance	Percentage decrease in compressive strength	Concrete condition
105°C	None	Loss is physically bound water in aggregate and cement	Unaffected	None	Unaffected
120 °C to 163°C	None	Decomposition of gypsum	Unaffected	0-9.4%	
250°C to 350°C	Pink	`Oxidation of iron compounds causing pink /red discolouration of aggregate	Surface crazing(300° C)	200°C -4% 300°C-10% 400°C -20%	250°C to 590°C- Sound but strength significantly reduced
450°C to 550°C	Pink to red	Dehydroxylation of portlandite	Deep cracking (550°C)	500°C-32%	Concrete is not structurally useful after heating in temperature above 500°C to 600°C
573°C	red	5% rise in the quartz volume causing radial cracking in the aggregate around the quartz grains	Popouts over chert or quartz aggregate (575°C)	600°C-52%	

600°C to 800°C	Whitish grey	Carbon dioxide release from carbonates can cause a significant contraction of concrete causing serious microcracking of the cement matrix	Powdered, light-coloured, dehydrated paste (575°C -600°C)	700°C-65% 800°C-82%	590°C to 950°C – Weak and friable
800°C to 1200°C	Whitish grey to buff	Dissociation and intense thermal stress cause the material to disintegrate completely and result in significant microcracking	Spalling, exposing not more than 25% of reinforcing bar surface 800°C and above cause extensive spalling	900°C-91% 1000°C-98.5%	Weak and friable
1200°C	buff	Concrete starts to melt	Extensive spalling	Melted	Very weak
1300°C to 1400°Cs	buff	Concrete melted		melted	

Table 2.1 Conclusion

2.4WORK METHODOLOGY

2.4.1 General

The research methodology was started with problem identification in RC multi-storey buildings exposed to fire. In terms of economic efficiency, it may be a better approach to retrofit the damaged components of the structure, instead of demolishing it partially or completely. This decision must be made based on the result of investigations such as the visual inspection of the damaged structure, tests on the remaining material, and finite element simulations of the structure or its structural components under seismic activity and setting up the objectives and scope of the study. Then all the related background information were collected and studied for the literature review for knowledge updating. The major part of this study was structural modelling and computational analysis using linear static analysis method in ETABs. The results thus obtained then being assessed, interpreted and compared.

The work methodology can be briefly divided into the following:

1) Literature review and problem identification
2) Description of building plan
3) Problem formulation
4) Method of analysis
5) Structural modelling
6) Analysis and results using ETABs software
7) Conclusions and suggestions

An RC Multi-storey building of G+6 storey was analyzed to resist the gravity loads, wind load and earthquake loads using ETABs software. Seismic parameters such as storey drift, storey displacement, storey stiffness and fundamental time period were computed in the analysis phase using ETABs. The result obtained from the analysis was compared between models which is exposed to fire and which is not exposed to fire. The Equivalent static method analysis was used which was most suited to the present problem and was used in the analysis and conclusions were made based on the analysis performed. This is the summary of the work methodology adopted in achieving the target objectives defined.

2.4.2 Description of Building Plan

For analysis, a 7 story high rise building is modelled in ETABs software. The building does not represent any real existing building. RC framed (G+6) multi-storey building having 4 grid line in X and Y direction and spacing between the grid lines in the X direction is 4.5m and in Y direction is 6.5m. The building is 22.5 m high and have a typical story height of 3.5m and bottom storey height is 1.5m. The building is analyzed by Equivalent static, which is a linear static analysis. A dead load of a wall is taken as wall load and parapet wall load which depend upon the wall thickness and the height of the wall. The thickness of the wall is taken as 230 mm for the outer wall and 115mm for inner walls. The unit weight of brick is 20KN/m^3 and height of partition wall will be 3.1m. The live load and the Floor finish dead load are taken as 2 KN/m^2 and 1.5 KN/m^2 according to IS 875:1987 (part 2). All the specifications of the frame are given in Table 1.For first building model

Table 2.2(First building model specification)

1.	Building type	Residential building
2.	No. of storeys	G+6
3.	Bottom storey height	1.5m
4.	Total height	22.5m
5.	Floor height	3.5m
6.	Size of column	400mm*600mm
7.	Size of beam	450mm*300mm
8.	Thickness of slab	130mm
9.	Masonry wall thickness	230mm Outer wall and 115mmfor inner wall
10.	Seismic zone	IV
11.	Importance factor	1
12.	Response reduction factor	5
13.	Soil type	II
14.	Concrete cube compressive strength	30MPa
15.	Grade of steel	Fe500
16.	Damping	5%
17.	Unit weight of PCC	24 kN/m^3
18.	Unit weight of brick	20 kN/m^3

19.	Modulus of Elasticity	24855.58 MPa
20.	Shear Modulus	10356.49 MPa
21.	Live load	2KN/m^2
22.	Floor finish dead load	1.5KN/m^2
23.	IS Code for concrete	IS 456:2000
24.	IS Code for earthquake	IS 1893:2002 (part I)
25.	IS Code for wind	IS 875 :1987
26	Self-weight factor	1
27.	Outer Wall load	14.26 KN/m
28.	Inner wall load	7.13KN/m

Table 2.3 (Second building model specification)

1.	Building type	Residential building
2.	No. of storeys	G+6
3.	Bottom storey height	1.5m
4.	Total height	22.5m
5.	Floor height	3.5m
6.	Size of column	400mm*600mm
7.	Size of beam	450mm*300mm
8.	Thickness of slab	130mm

9.	Masonry wall thickness	230mm Outer wall and 115mmfor inner wall
10.	Seismic zone	IV
11.	Importance factor	1
12.	Response reduction factor	5
13.	Soil type	II
14.	Concrete cube compressive strength	25.5 MPa
15.	Grade of steel	Fe500
16.	Damping	5%
17.	Unit weight of PCC	24 kN/m^3
18.	Unit weight of brick	20 kN/m^3
19.	Modulus of Elasticity	14913.55 MPa
20.	Shear Modulus	6213.9 MPa
21.	Live load	2KN/m^2
22.	Floor finish dead load	1.5KN/m^2
23.	IS Code for concrete	IS 456:2000
24.	IS Code for earthquake	IS 1893:2002 (part I)
25.	IS Code for wind	IS 875 :1987
26	Self-weight factor	1
27.	Outer Wall load	14.26 KN/m
28.	Inner wall load	7.13KN/m

Table 2.4(Third building model specification)

1.	Building type	Residential building
2.	No. of storeys	G+6
3.	Bottom storey height	1.5m
4.	Total height	22.5m
5.	Floor height	3.5m
6.	Size of column	400mm*600mm
7.	Size of beam	450mm*300mm
8.	Thickness of slab	130mm
9.	Masonry wall thickness	230mm Outer wall and 115mmfor inner wall
10.	Seismic zone	IV
11.	Importance factor	1
12.	Response reduction factor	5
13.	Soil type	II
14.	Concrete cube compressive strength	18 MPa
15.	Grade of steel	Fe500
16.	Shear modulus	5385.43 MPa
17.	Unit weight of PCC	24 kN/m^3

18.	Unit weight of brick	20 kN/m^3
19.	Modulus of Elasticity	12935.04 MPa
20.	Damping	5%
21.	Live load	2KN/m^2
22.	Floor finish dead load	1.5KN/m^2
23.	IS Code for concrete	IS 456:2000
24.	IS Code for earthquake	IS 1893:2002 (part I)
25.	IS Code for wind	IS 875 :1987
26	Self weight factor	1
27.	Outer Wall load	14.26 KN/m
28.	Inner wall load	7.13KN/m

Table 2.5 (Forth building model specification)

1.	Building type	Residential building
2.	No. of storeys	G+6
3.	Bottom storey height	1.5m
4.	Total height	22.5m
5.	Floor height	3.5m
6.	Size of column	400mm*600mm
7.	Size of beam	450mm*300mm

8.	Thickness of slab	130mm
9.	Masonry wall thickness	230mm Outer wall and 115mmfor inner wall
10.	Seismic zone	IV
11.	Importance factor	1
12.	Response reduction factor	5
13.	Soil type	II
14.	Concrete cube compressive strength	10.5 MPa
15.	Grade of steel	Fe500
16.	Shear Modulus	4660.43 MPa
17.	Unit weight of PCC	24 kN/m^3
18.	Unit weight of brick	20 kN/m^3
19.	Modulus of Elasticity	11185.13 Mpa
20.	Damping	5%
21.	Live load	$2KN/m^2$
22.	Floor finish dead load	$1.5KN/m^2$
23.	IS Code for concrete	IS 456:2000
24.	IS Code for earthquake	IS 1893:2002 (part I)
25.	IS Code for wind	IS 875 :1987
26	Self weight factor	1
27.	Outer Wall load	14.26 KN/m

28.	Inner wall load	7.13KN/m

2.4.2 Load combination-

Building is analyzed on the basis of Various load combinations in the limit state of design for reinforced concrete structures as per IS 1893:2002(part1).these all are given below:

1) 1.5(DL+IL)

2) 1.2 (DL+IL+EL_x)

3) 1.2 (DL+LL+EL_Y)

4) 1.2(DL+IL-EL_X)

5) 1.2(DL+IL-EL_Y)

6) 1.5(DL+EL_X)

7) 1.5(DL+EL_Y)

8) 1.5(DL-EL_X)

9) 1.5(DL-EL_Y)

10) 0.9DL+1.5EL_X

11) 0.9DL+1.5EL_Y

12) 0.9DL-1.5EL_X

13) 0.9DL-1.5EL_Y

14) 1.2 DL+LL+WL:+X

15) 1.5 DL +WL :+X

16) 1.5 DL +WL :-X

17) 1.5 DL +WL :+Y

18) 1.5 DL +WL :-Y

17) 1.2 DL+LL+WL:-X

18) 1.2 DL+LL+WL:+Y

19) 1.2 DL+LL+WL:-Y

As we know that 1.5(DL+IL) is not the Earthquake load combo. It is purely the gravity load combination. But when we are designing a structure, we need to consider all the different load combinations as specified by the respective design code . So, 1.5 (DL+LL) has nothing to do with the earthquake loading. 1.5(DL+LL) as defined in the IS-1893 code is one of the load combination as specified in IS 456 for the RCC structure. See below the factors these factors are same as IS 456:2000.

Load Combination	Limit State of Collapse			Limit States of Serviceability		
	DL	IL	WL	DL	IL	WL
(1)	(2)	(3)	(4)	(5)	(6)	(7)
DL + IL	1.5		1.0	1.0	1.0	-
DL + WL	1.5 or 0.9[1)]	-	1.5	1.0	-	1.0
DL + IL + WL	1.2			1.0	0.8	0.8

Showing Load combination

2.4.3 Problem Formulation

The study will provide an approximate idea of how fire exposed building will behave in seismic forces and can be used future after exposing to various elevated temperature under seismic forces The analysis was done as per IS Code provision using ETABs software. In this comparison is done for G+6 multi-storey residential building. The seismic data is taken according to the IS 1893(Part 1):2002 for the Zone III as given below in table 2.

Table 2.6 Seismic Data

Serial No	Model Description	
1	Zone	IV
2	Zone Factor	0.24
3	Type of building	Residential
4	Importance Factor	1
5	Soil Type	II
6	Soil Condition	Medium
7	Damping Ratio	5%
8	Response Reduction Factors	5

2.5 Method of Analysis - Seismic analysis may be carried out by:

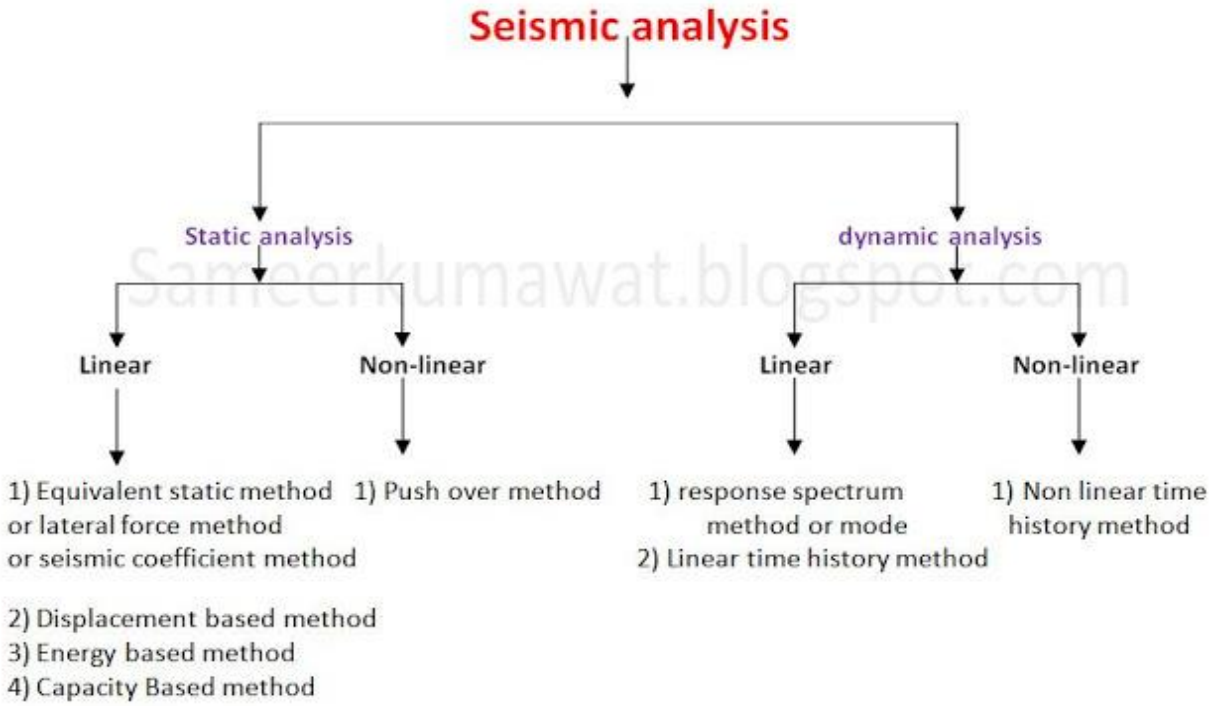


Fig.2.3- Method of Analysis

2.5.1Static Method

The design base shear shall be computed as a whole, and then be distributed along the height of the building based on simple formulas appropriate for the building with regular distributing of mass and stiffness according to IS Code 1893 (part 1): 2002.

2.5.1.2 Equivalent static method

This approach defines a series of forces acting on a building to represent the effect of earthquake ground motion, typically defined by a seismic design response spectrum. It assumes that the building responds in its fundamental mode. For this to be true, the building must be low-rise and must not twist significantly when the ground moves. The response is read from a design response spectrum, given the natural frequency of the building (either calculated or defined by the building code). The applicability of this method is extended in many building codes by applying factors to account for higher buildings with some higher modes, and for low levels of twisting. To account for effects due to "yielding" of the structure, many codes apply modification factors that reduce the design forces (e.g. force reduction factors).

For determination of seismic forces, the country is classified in four seismic zones:

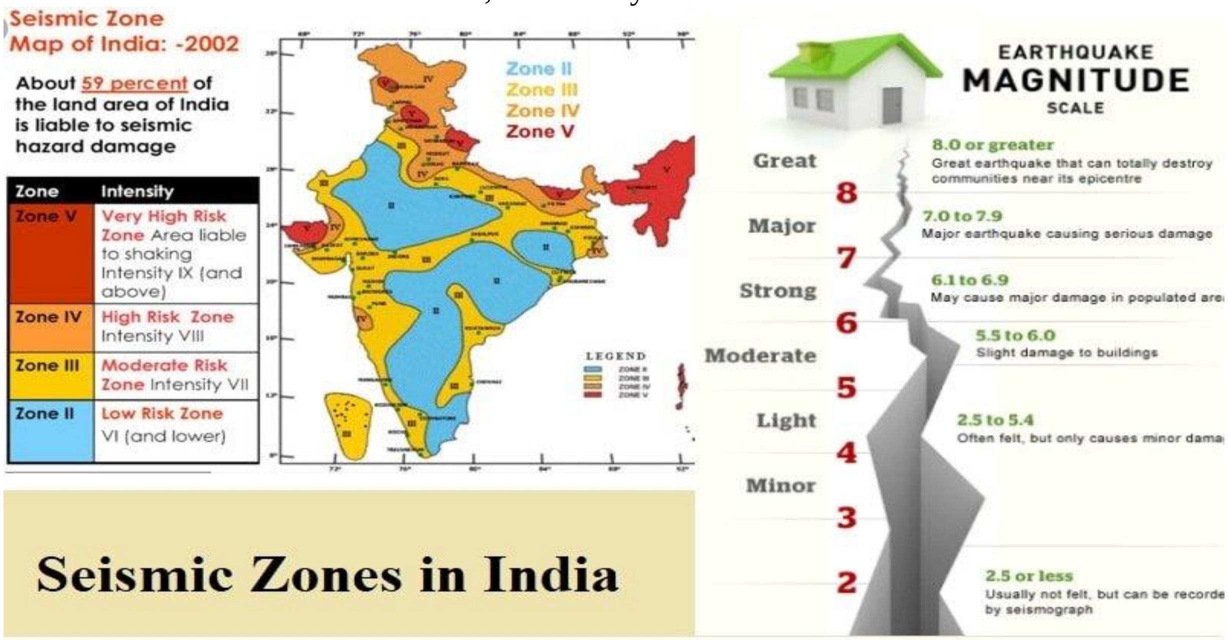


Fig 2.4- seismic zones of India

Each zone has their own zone factor value and as per IS 1893 (Part 1):2002 these values are given below:

Seismic Zone Factor (1)	II (2)	III (3)	IV (4)	V (5)
Z	0.10	0.16	0.24	0.36

As per IS Code 1893(part 1) :2002 the following were the major steps for determining the seismic forces:

For rocky, or hard soil sites

$$\frac{S_a}{g} = \begin{cases} 1 + 15\,T; & 0.00 \le T \le 0.10 \\ 2.50 & 0.10 \le T \le 0.40 \\ 1.00/T & 0.40 \le T \le 4.00 \end{cases}$$

For medium soil sites

$$\frac{S_a}{g} = \begin{cases} 1 + 15\,T; & 0.00 \le T \le 0.10 \\ 2.50 & 0.10 \le T \le 0.55 \\ 1.36/T & 0.55 \le T \le 4.00 \end{cases}$$

For soft soil sites

$$\frac{S_a}{g} = \begin{cases} 1 + 15\,T; & 0.00 \le T \le 0.10 \\ 2.50 & 0.10 \le T \le 0.67 \\ 1.67/T & 0.67 \le T \le 4.00 \end{cases}$$

2.5.1.2.1 Determination of base shear

The total design lateral force or design base shear along any principal dirction shall be determined by the following expression,(clause 7.6.1 of IS 1893 (part 1): 2002)

$$V_b = A_h * W$$

Where, A_h= Design horizontal seismic coefficient for structure

W= Seismic weight of the building

$$A_h = \frac{\left(\frac{Z}{2}\right)\left(\frac{S_a}{g}\right)}{\left(\frac{R}{I}\right)}$$

Where, R=response reduction factor

Z= zone factor

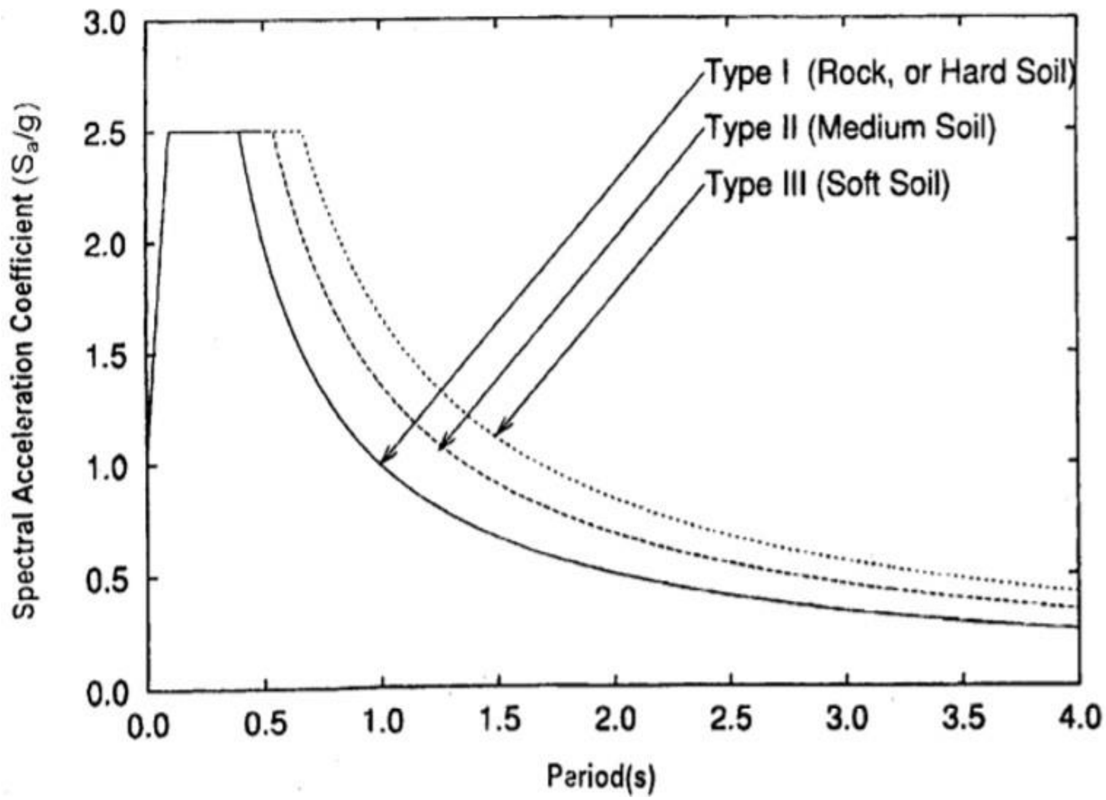
I= importance factor

Sa/g is the average response acceleration coefficient for rock and soil sites as given in

figure 2 of IS 1893:2002 (part 1).The values are given for 5% damping of the structure for S_a/g.

Where T is the fundamental natural period for buildings calculated as per clause 7.6 of IS

1893:2002 (part1) and shown further.



Lateral distribution of base shear

In equivalent lateral force procedure, the magnitude of lateral force is based on the fundamentals period of vibration, IS 1893 (part 1):2002 uses of parabolic distribution of the lateral force along the height of the building as per the following expression:

$$Q_{\mathrm{i}} = \left(\frac{W_{\mathrm{i}} h_{\mathrm{i}}^2}{\sum_{j=1}^{n} W_{\mathrm{j}} h_{\mathrm{j}}^2} \right) V_{\mathrm{B}}$$

Where,

Q_i= Design lateral force at floor i

W_i= seismic weight of the floor i

h_i= height of the floor i from the base

n= number of storeys of the building at which masses are located.

Fundamental natural time period

The approximate fundamental natural period of vibration (T), in seconds, of all other buildings, including moment resisting frame buildings with brick infill panels, maybe estimated by the empirical expression given in clause 7.6.2 of IS 1893(part 1):2002.

$T_a=0.075h^{0.75}$ for RC frame building without brick infill wall

$T_a= 0.085h^{0.75}$ for steel frame building without brick infill wall

$T_a=0.09h/\sqrt{d}$ all other buildings including moment resisting RC frame with brick infill

2.5.2 Nonlinear static analysis

In general, linear procedures are applicable when the structure is expected to remain nearly elastic for the level of ground motion or when the design results in nearly uniform distribution of nonlinear response throughout the structure. As the performance objective of the structure implies greater inelastic demands, the uncertainty with linear procedures increases to a point that requires a high level of conservatism in demand assumptions and acceptability criteria to avoid unintended performance. Therefore, procedures incorporating inelastic analysis can reduce uncertainty and conservatism.

This approach is also known as "pushover" analysis. A pattern of forces is applied to a structural model that includes non-linear properties (such as steel yield), and the total force is plotted against a reference displacement to define a capacity curve. This can then be combined with a demand curve (typically in the form of an acceleration- displacement response spectrum (ADRS). This essentially reduces the problem to a single degree of freedom (SDOF) system. Nonlinear static procedures use equivalent SDOF structural models and represent seismic ground motion with response spectra. Story drifts and component actions are related subsequently to the global demand parameter by the pushover or capacity curves that are the basis of the non-linear static procedures.

2.6 Linear Dynamic Methods

Static procedures are appropriate when higher mode effects are not significant. This is generally true for short, regular buildings. Therefore, for tall buildings, buildings with torsion irregularities, or non-orthogonal systems, a dynamic procedure is required. In the linear dynamic procedure, the building is modelled as a multi-degree-of-freedom (MDOF) system with a linear elastic stiffness matrix and an equivalent viscous damping matrix. The seismic input is modelled using either modal spectral analysis or time history analysis but in both cases, the corresponding internal forces and displacements are determined using linear elastic analysis. In linear dynamic analysis, the response of the structure to ground motion is calculated in the time domain, and all phase information is therefore maintained. Only linear properties are assumed. The analytical method can use modal decomposition as a means of reducing the degrees of freedom in the analysis. The advantage of linear dynamic procedures concerning linear static procedures is that higher modes can be considered. However, they are based on linear elastic response and hence the applicability decreases with increasing nonlinear behaviour, which is approximated by global force reduction factors. The type of linear dynamic methods is as follows-

2.6.1.1Response Spectrum Analysis

Response spectrum analysis is a procedure for calculating the maximum response of a structure when applied with ground motion. Each of the vibration modes that are considered are assumed to respond independently as a single degree of freedom system. Design codes specify response spectra which determine the base acceleration applied to each mode according to its period (the number of seconds required for a cycle of vibration).Having determined the response of each vibration mode to the excitation, it is necessary to obtain the response of the structure by combining the effects of each vibration mode because the maximum response of each mode will not necessarily occur at the same instant, the statistical maximum response, where damping is zero, is taken as a sum of squares (SRSS) of the individual responses.

The results of the response spectrum are all absolute extreme values and so they need to be combined as they do not correspond to any equilibrium state nor they take place at the same time. There are several methods to execute this, one of them being the (SRSS) method, Square root of the sum of squares method. In this method, the maximum response in terms of a given parameter, G (displacement, acceleration, velocity) may be estimated through the square root of the sum of m modal response squares, contributing to global response:

G = Σmn=1 (Gn)2

2.6.2 Nonlinear dynamic analysis

Nonlinear dynamic analysis utilizes the combination of ground motion records with a detailed structural model, therefore is capable of producing results with relatively low uncertainty. In nonlinear dynamic analyses, the detailed structural model subjected to a ground-motion record produces estimates of component deformations for each degree of freedom in the model and the modal responses are combined using schemes such as the square-root-sum-of-squares.

In non-linear dynamic analysis, the non-linear properties of the structure are considered as part of a time-domain analysis. This approach is the most rigorous and is required by some building codes for buildings of unusual configuration or of special importance. However, the calculated response can be very sensitive to the characteristics of the individual ground motion used as seismic input; therefore, several analyses are required using different ground motion records to achieve a reliable estimation of the probabilistic distribution of structural response. Since the properties of the seismic response depend on the intensity, or severity, of the seismic shaking, a comprehensive assessment calls for numerous nonlinear dynamic analyses at various levels of intensity to represent different possible earthquake scenarios.

2.6.2.1Time History Method

It is known as Time history analysis. It is an important technique for structural seismic analysis especially when the evaluated structural response is nonlinear. Time history analysis is a step-by- step analysis of the dynamic response of a structure to a specified loading that may vary with time. A full time history will give the response of a structure over time during and after the application of a load. To find the full time history of a structure's response A linear time history analysis overcomes all the disadvantages of a modal response spectrum analysis provided non linear behaviour is not involving. This method requires greater computational efforts for calculating the response at discrete times. It is used to determine the dynamic response of a structureto arbitrary loading

2.7 Parameters considered for analysis

2.7.1.1	Storey drift
2.7.1.2	Storey displacement
2.7.1.3	Fundamental time period
2.7.1.4	Storey stiffness

2.7.1 Stoey drift- It is the relative displacement of one level relative to other level above or below. According to IS 1893(Part 1):2002 (part 1), the storey drift should not exceed 0.04 mes of relative storey height.

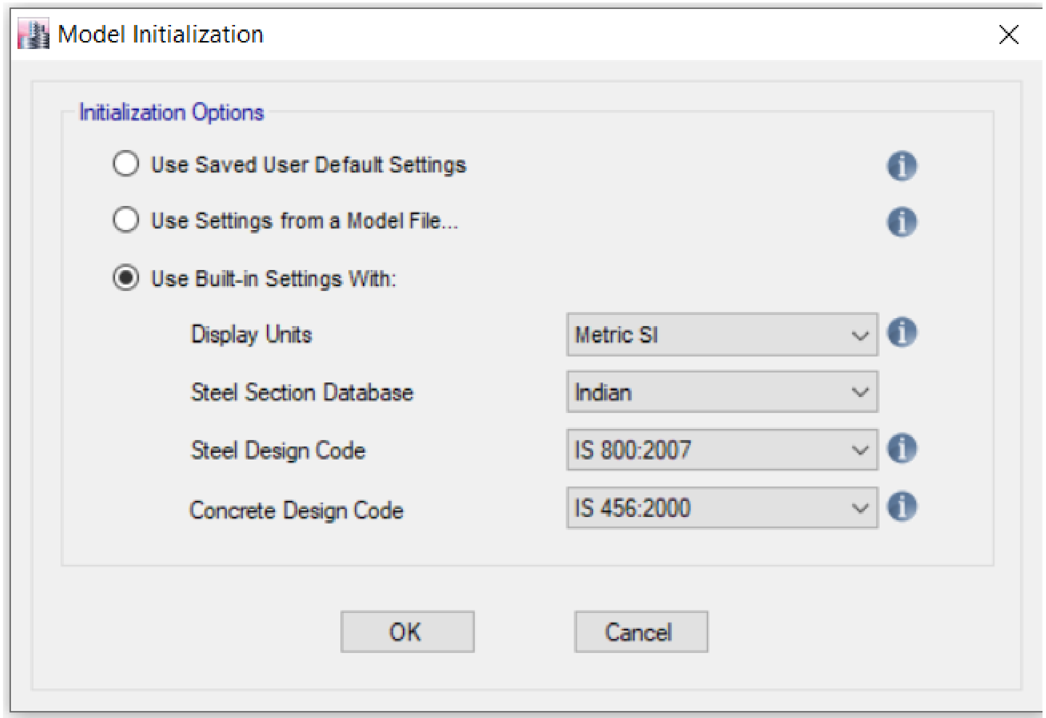
2.7.2 Storey displacement- It is the displacement of each storey with respect to ground level. According to IS 1893 (part1) :2002 the max value of displacement is 1/250 times of storey height with respect to ground.

2.7.3 Fundamental time period- According to IS 1893 (Part 1):2002 it is the first (longest) modal time period of vibration.

2.7.4 Storey stiffness- As per IS 1893(Part 1):2002 the lateral stiffness is less than 70 per cent of that in the storey above or less than 80 per cent of average lateral stiffness of the three storeys above.

Structural Modeling

Software ETABs is used for seismic analysis and to study the behaviour of multistorey building exposed to fire. Different models are made and compared with different parameters of analysis. Complete analysis including structural modelling is performed in this software. For analysis, a 7 storied high rise building is modelled in ETABs software. The building does not represent any real existing building. RC framed (G+6) multi-storey building having 4 grid line in X and Y direction and spacing between the grid lines in the X direction is 4.5m and in the Y direction is 6.5m as shown in figure 2.5. The building is 22.5 m high and has a typical story height of 3.5m and bottom storey height is 1.5m. The building is analyzed by Equivalent static method, which is a linear static analysis. The dead load of the wall is taken as wall load and parapet wall load which depend upon the wall thickness and the height of the wall. the thickness of the wall is taken as 230 mm for the outer wall and 115mm for inner walls. The unit weight of brick is 20KN/m3 and height of partition wall will be 3.1m. The live load and the Floor finish dead load are taken as 2 KN/m2 and 1.5 KN/m2 according to IS 875:1987 (part 2). All the specifications of the frame are given in Table 1.For first building model


Model Initialization
Initialization Options
Use Saved User Default Settings
Use Settings from a Model File...
Use Built-in Settings With:
Display Units
Metric SI
Steel Section Database
Indian
Steel Design Code
IS 800:2007
Concrete Design Code
IS 456:2000
OK
Cancel

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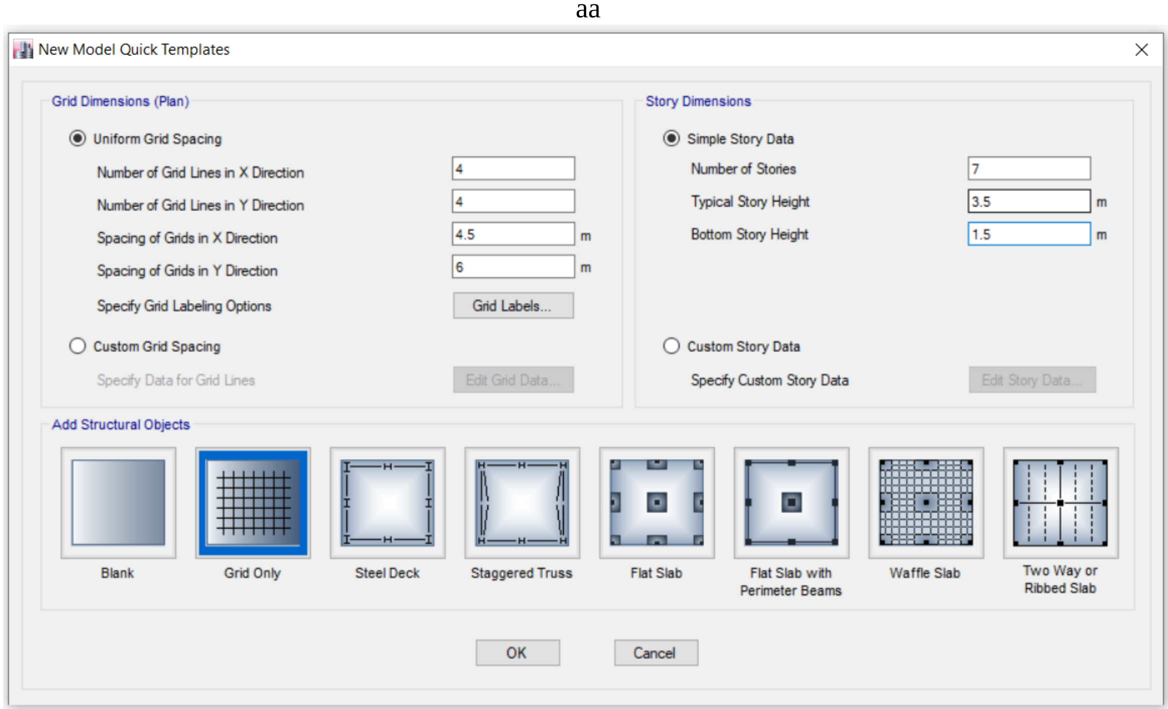
New Model Quick Templates
Grid Dimensions (Plan)
Uniform Grid Spacing
Number of Grid Lines in X Direction
4
Number of Grid Lines in Y Direction
4
Spacing of Grids in X Direction
4.5
m
Spacing of Grids in Y Direction
6
m
Specify Grid Labeling Options
Grid Labels...
Custom Grid Spacing
Specify Data for Grid Lines
Edit Grid Data...
Story Dimensions
Simple Story Data
Number of Stories
7
Typical Story Height
3.5
m
Bottom Story Height
1.5
m
Custom Story Data
Specify Custom Story Data
Edit Story Data...
Add Structural Objects
Blank
Grid Only
Steel Deck
Staggered Truss
Flat Slab
Flat Slab with Perimeter Beams
Waffle Slab
Two Way or Ribbed Slab
OK
Cancel

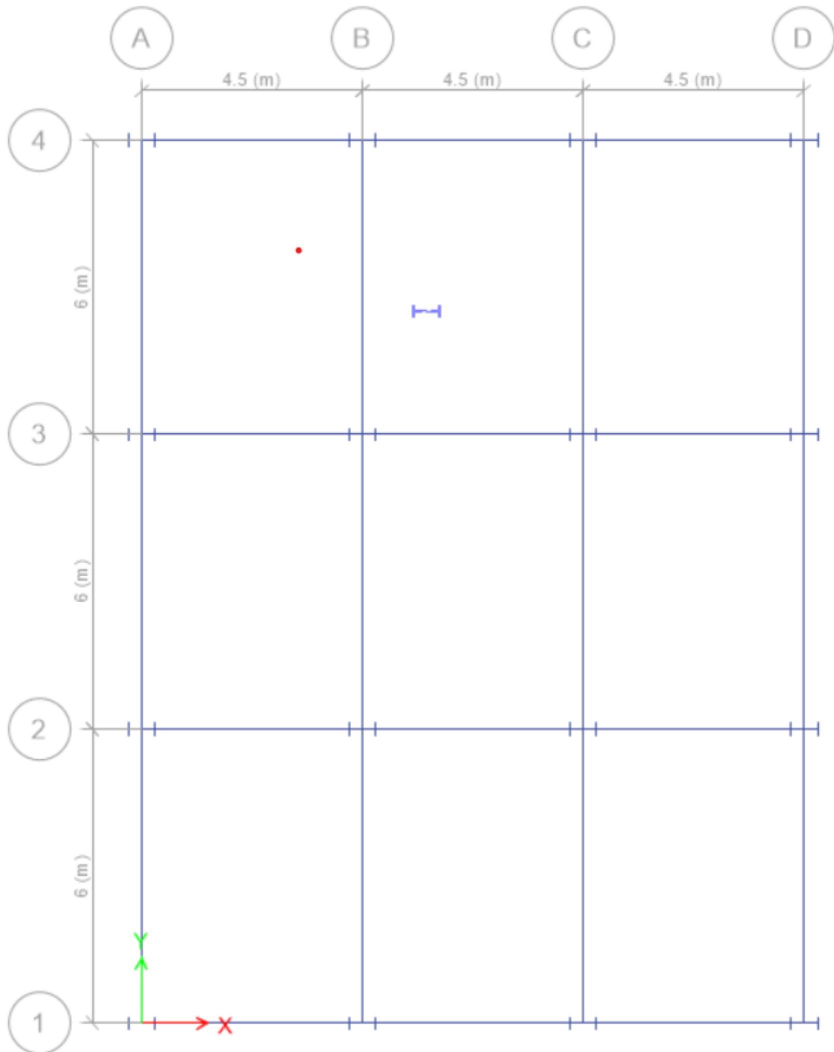

Fig.2.5- Plan of Building , Building dimestions

The four models are selected for the study which looks the same because all has the same element properties all are 7 storey building with the same plan, same sectional properties and same column and beam dimensions but the material properties changes such as concrete cube compressive strength, modulus of elasticity and shear modulus are changed on the bases of a reduce strength which is predicted before in my review paper according to study of various literature at a various temperature such as 300 °C, 600°C,600 °C

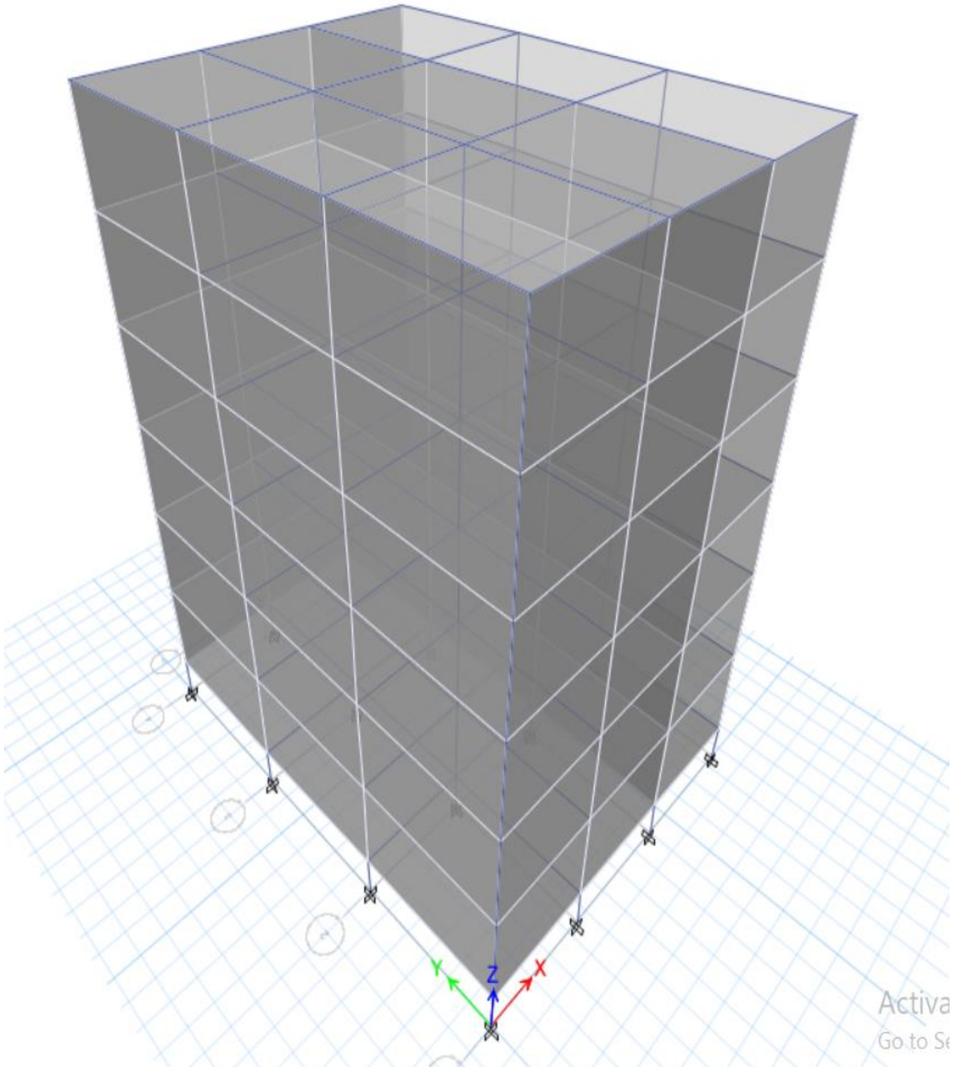

Fig.2.6 - 3d model of Building

3.1 DEFORMED SHAPE

MODEL 1 (Deformed shape)

- In this model, we have applied earthquake load EQ at X+ direction. This is a simple RC framed structure without exposure to fire

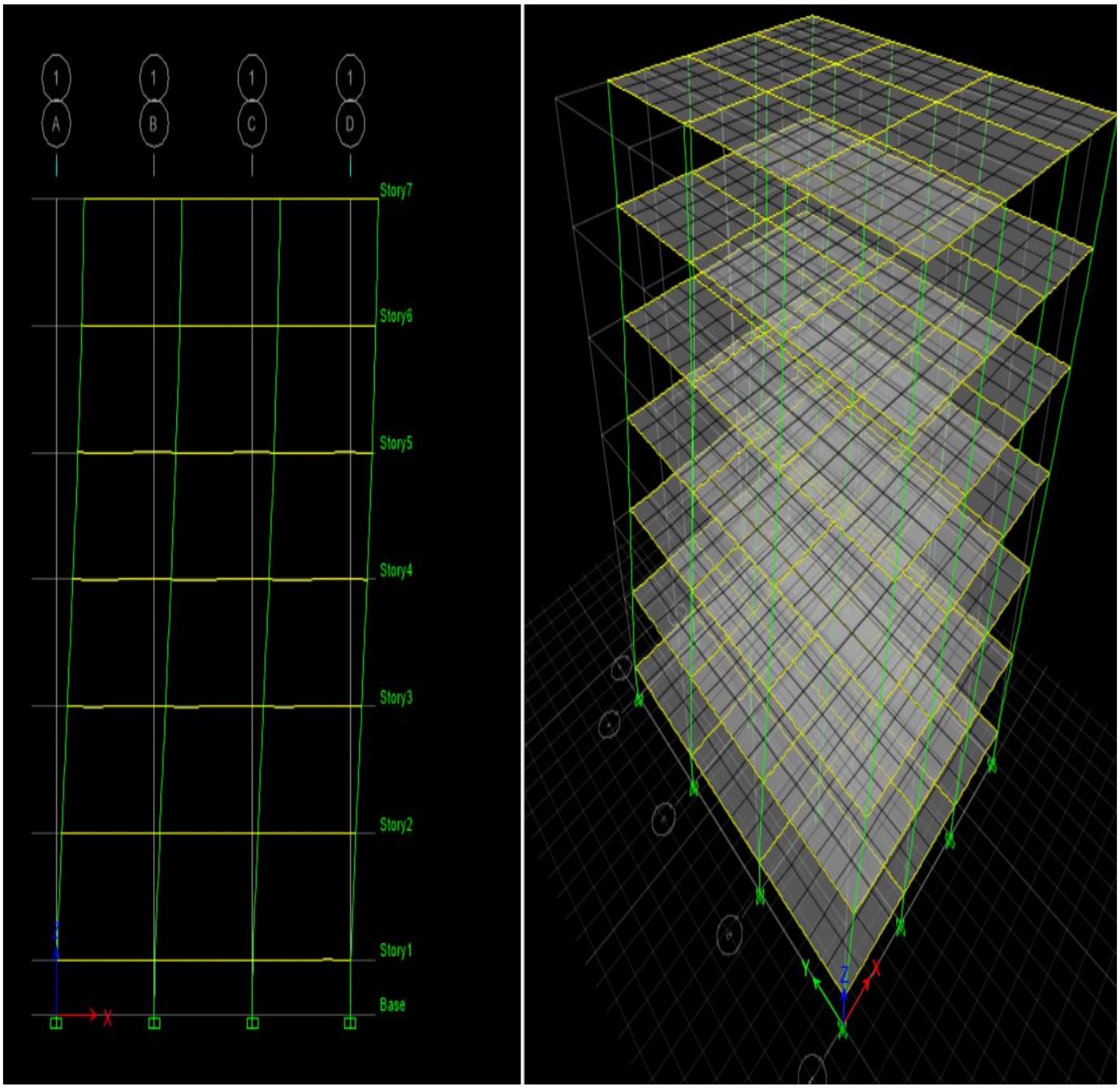

Fig 2.7- 3D and elevation view of model 1 Building

- In this model, we have applied earthquake load EQ at Y+ direction. This is a simple RC framed structure without exposure to fire

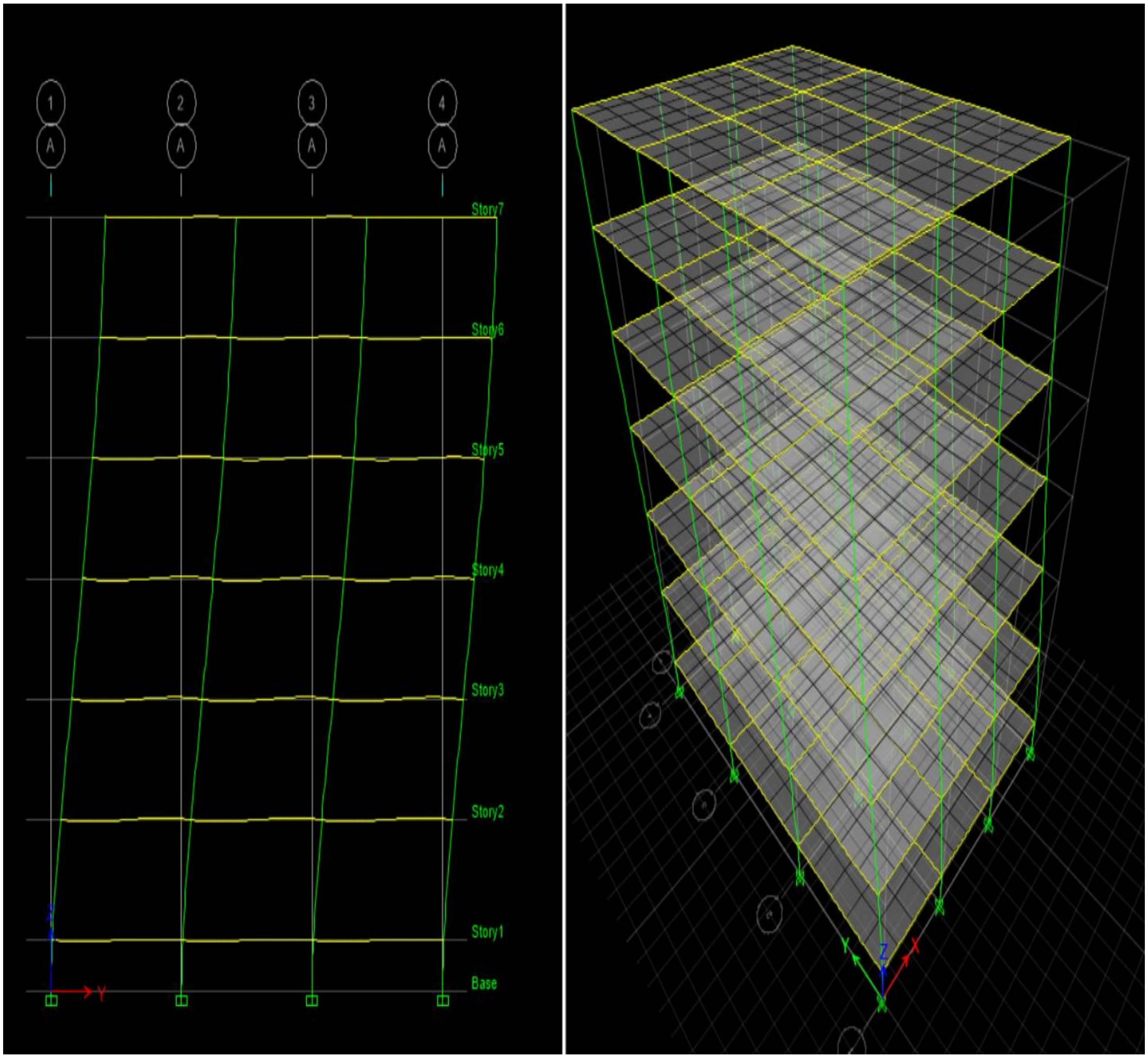

FIG.2.8- 3D and elevation view of Building

MODEL 2- DEFORMED SHAPE OF STRUCTURE DUE TO EARTHQUAKE LOADING

- In this model, we have applied earthquake load EQ at X+ direction. This is a simple RC framed structure which is exposed to fire at temperature 300 °C

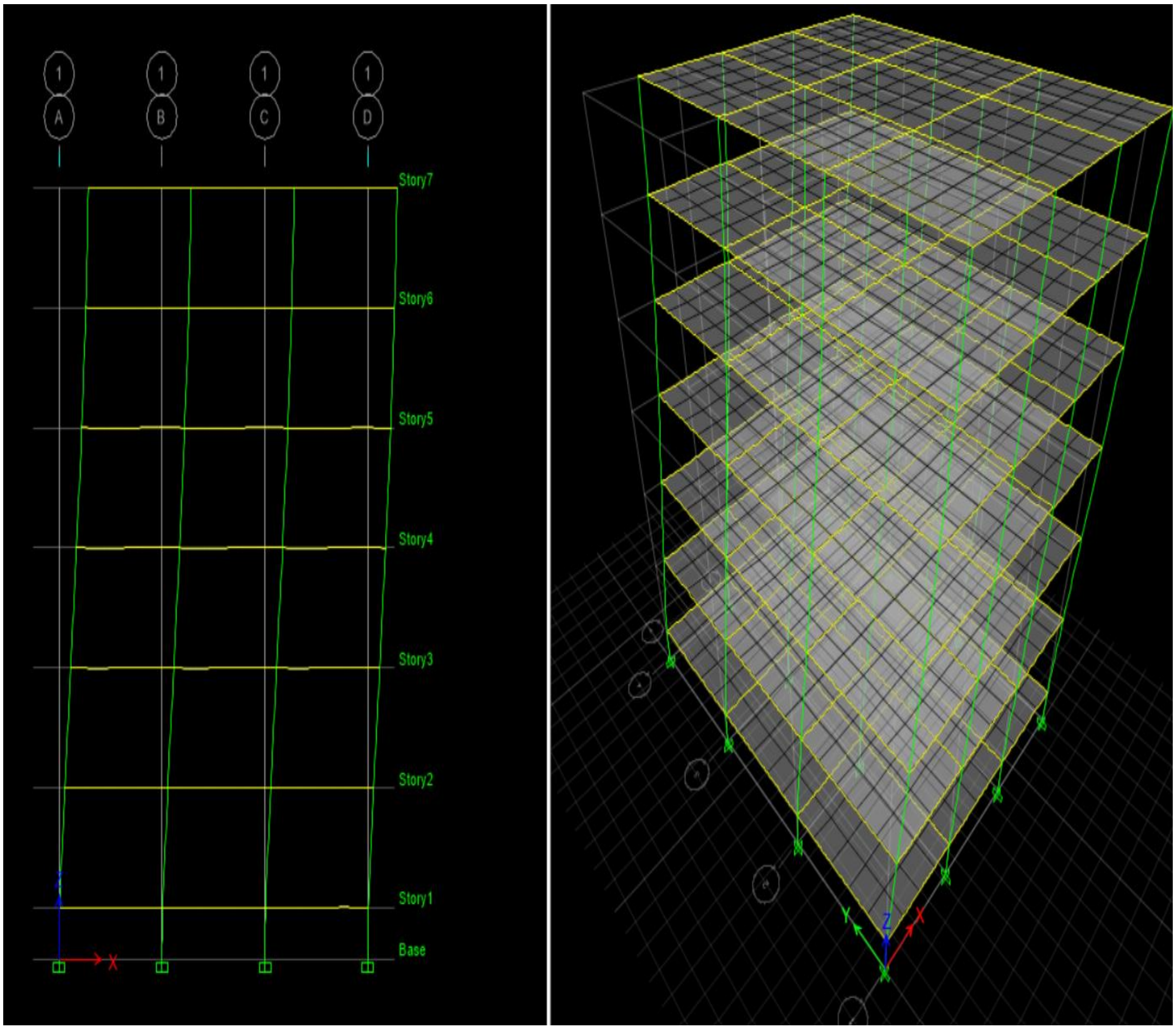

FIG.2.9- 3D View and Elevation view of a building

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- In this model, we have applied earthquake load EQ at Y+ direction. This is a simple RC framed structure which is exposed to fire at temperature 300 °C

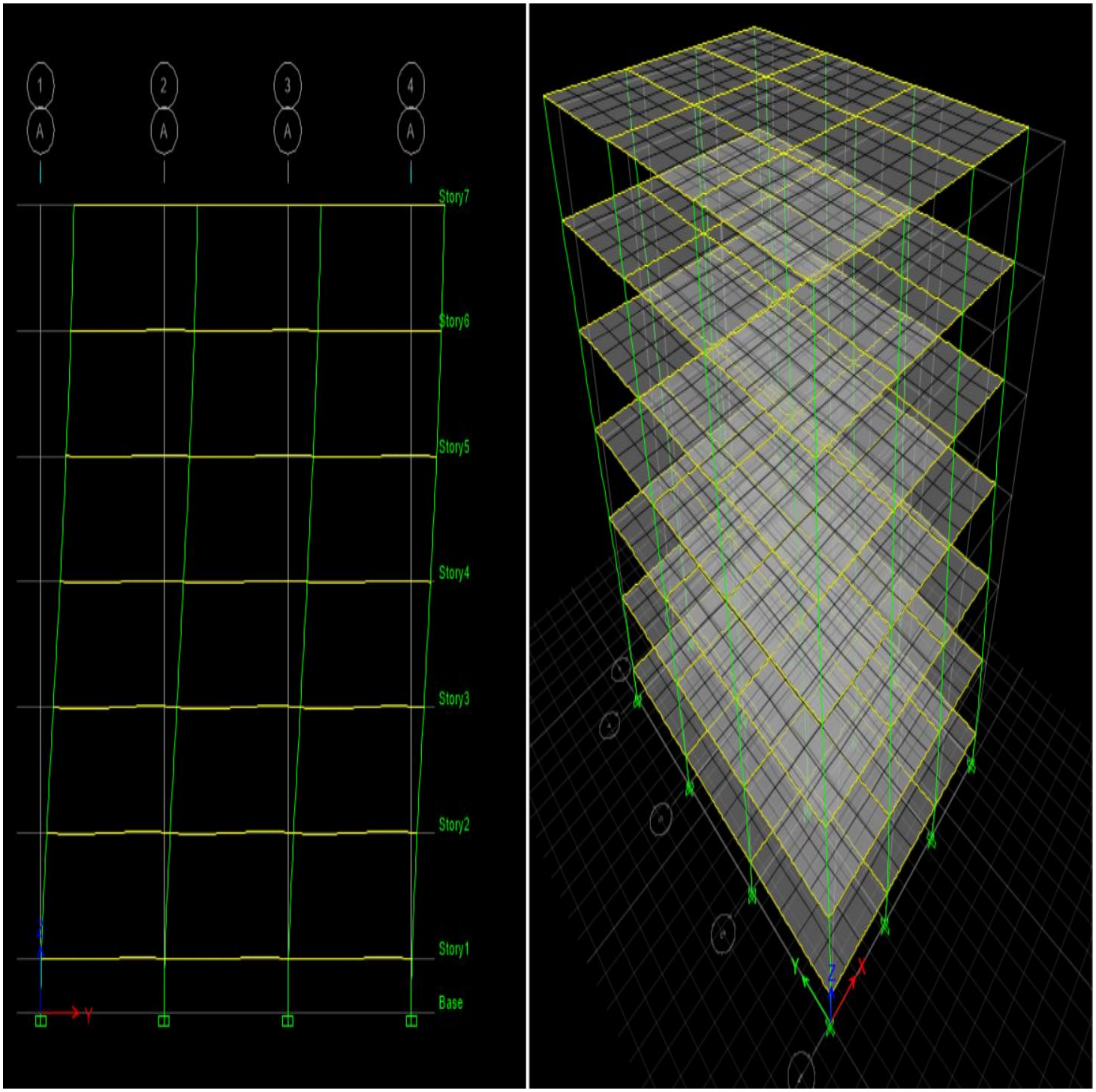


Fig. 2.10- 3D View and Elevation view of a building

MODEL 3- DEFORMED SHAPE OF STRUCTURE DUE TO EARTHQUAKE LOADING

- In this model, we have applied earthquake load EQ at X+ direction. This is a simple RC framed structure which is exposed to fire at temperature 500 °C

.

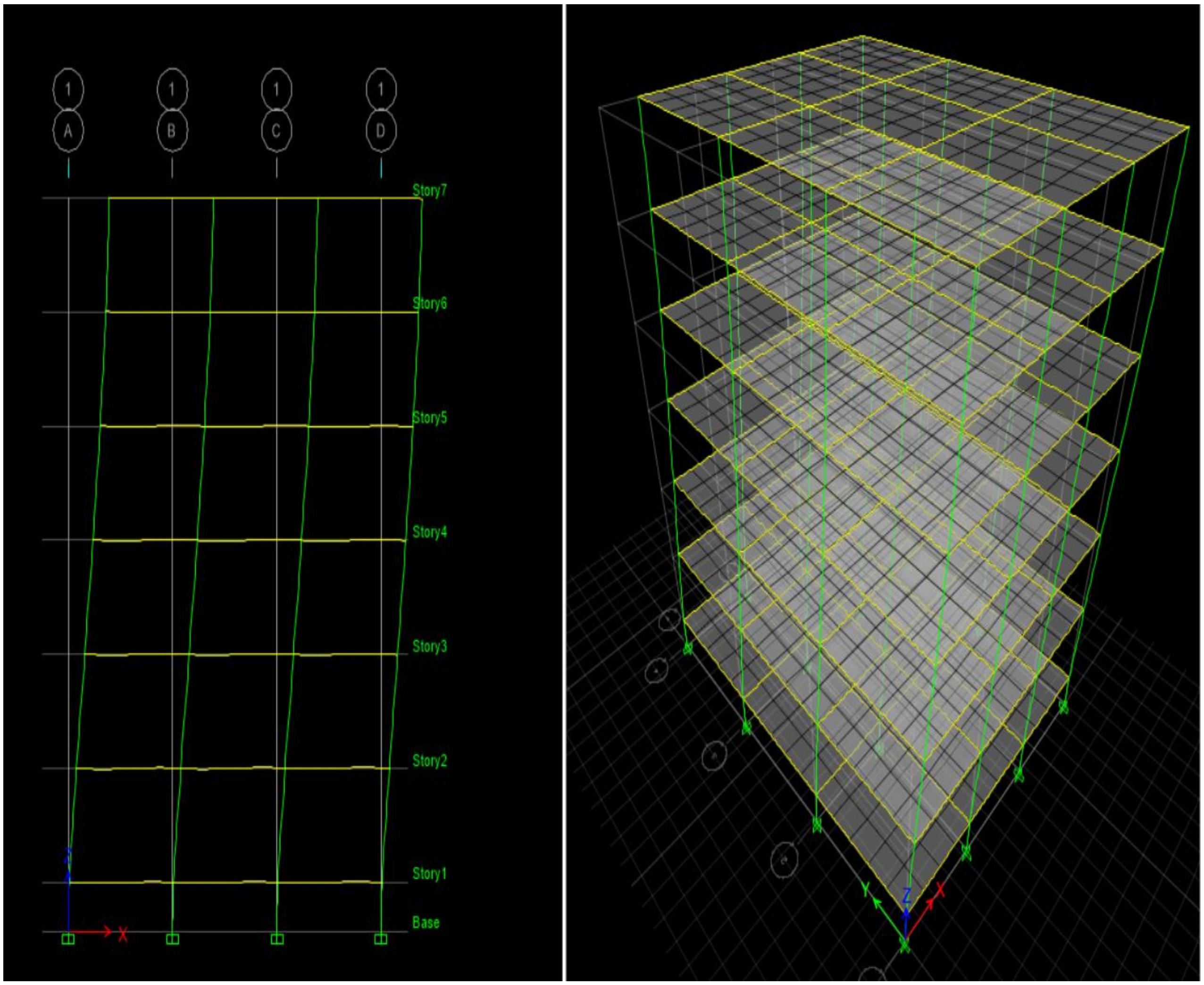


Fig.2.11- 3D View and Elevation view of a building

- In this model, we have applied earthquake load EQ at Y+ direction. This is a simple RC framed structure which is exposed to fire at temperature 500 °C

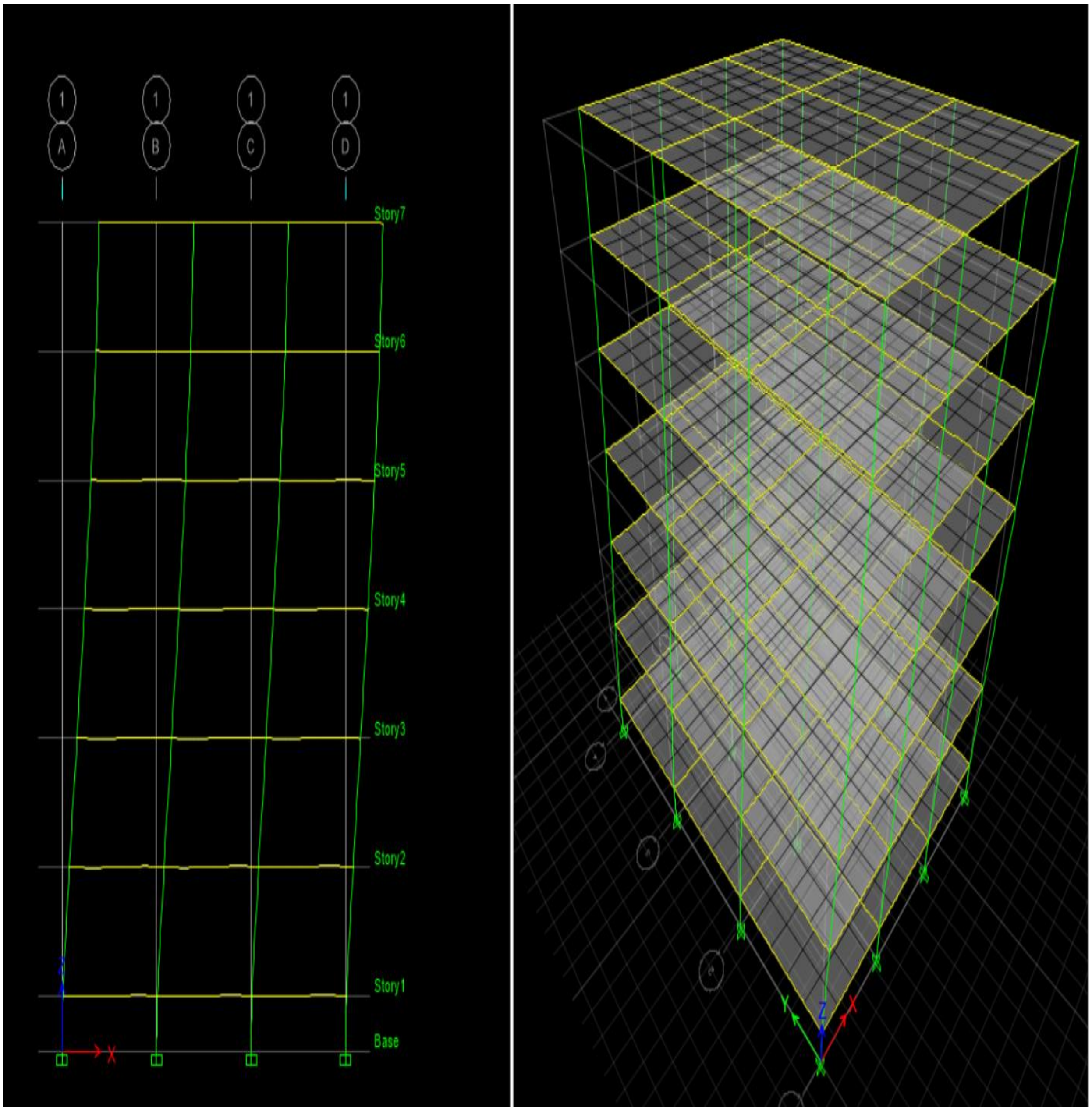


Fig.2.12- 3D View and Elevation view of a building

MODEL 4-DEFORMED SHAPE OF STRUCTURE DUE TO EARTHQUAKE LOADING

- In this model, we have applied earthquake load EQ at X+ direction. This is a simple RC framed structure which is exposed to fire at temperature 600 °C.

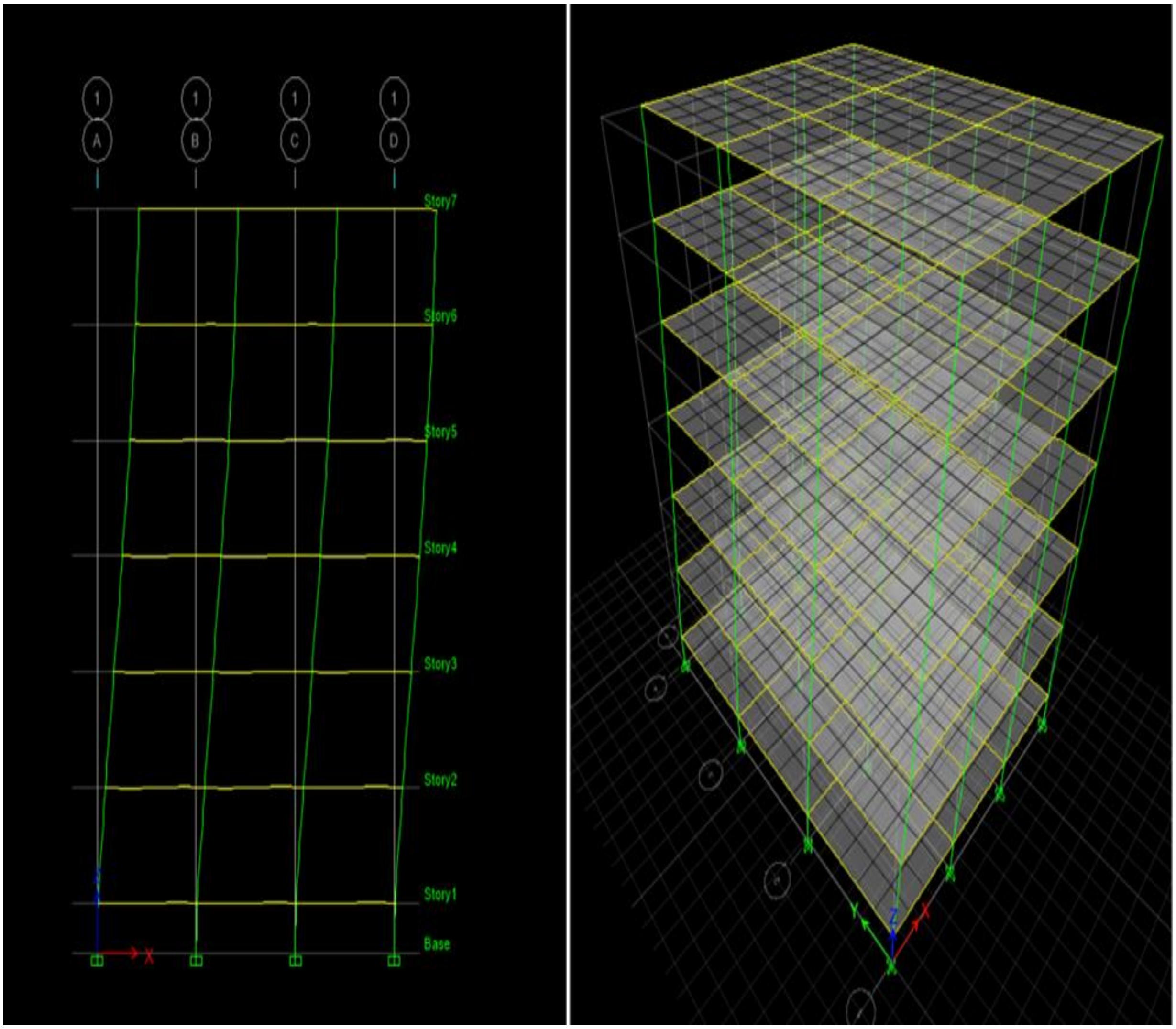


Fig.2.13- 3D View and Elevation view of a building

- In this model, we have applied earthquake load EQ at Y+ direction. This is a simple RC framed structure which is exposed to fire at temperature 600 °C.

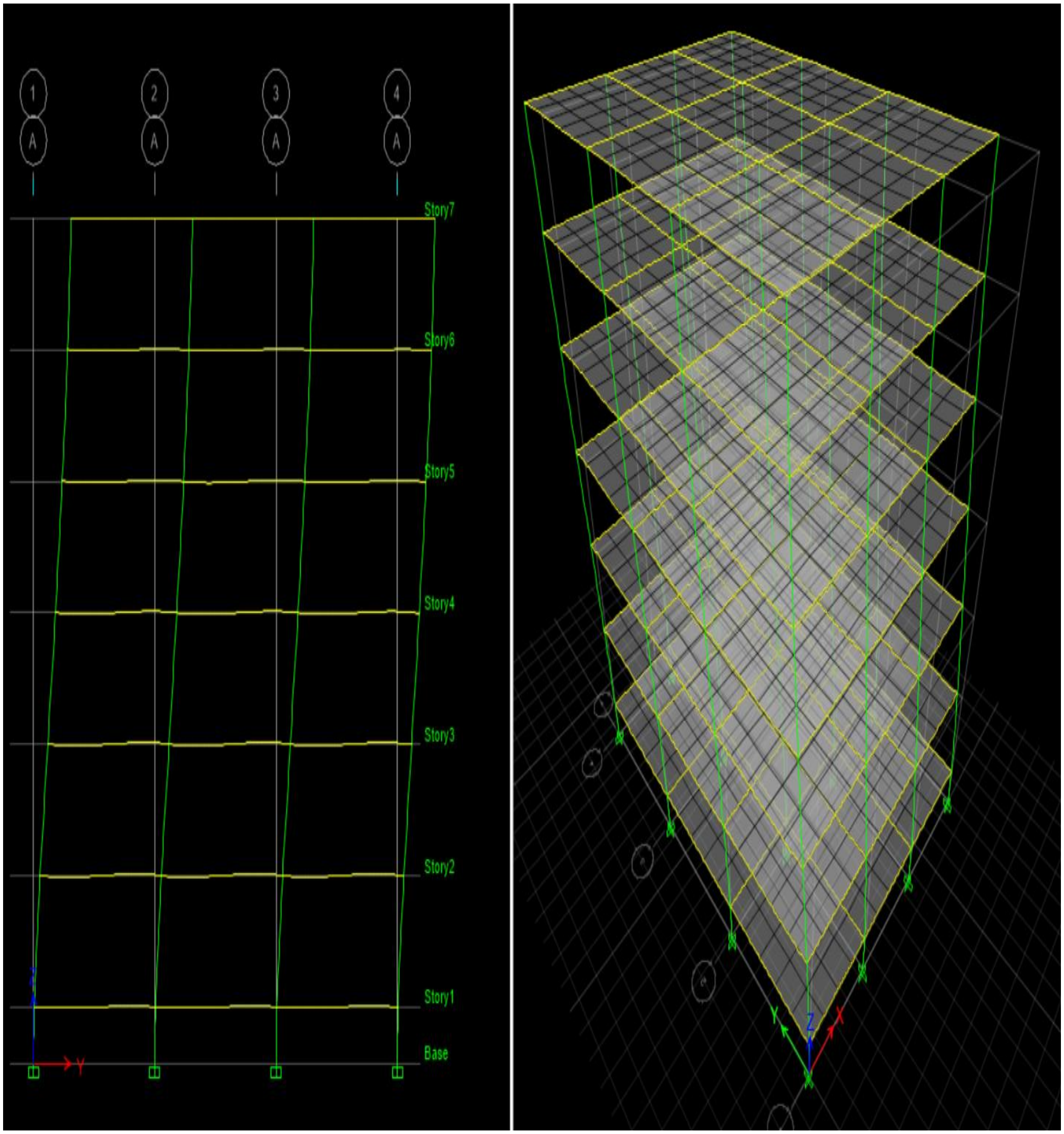


Fig.2.14- 3D View and Elevation view of a building

3.2 Analysis Using ETABs Software

The analysis has been done using ETABs software which involves the following steps:-

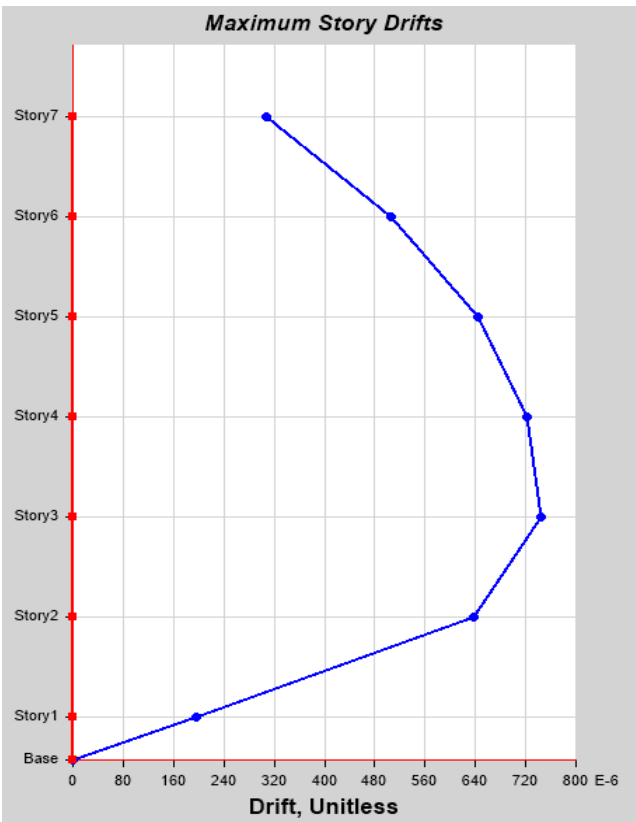
1. Defining Dimensions of the plan
2. Defining the members and material properties
3. Assigning loads and load combinations
4. Run and check the model to find errors
5. Run analysis
6. Extract results and discuss

RESULT AFTER ANALYSIS

4.1 Storey Drift

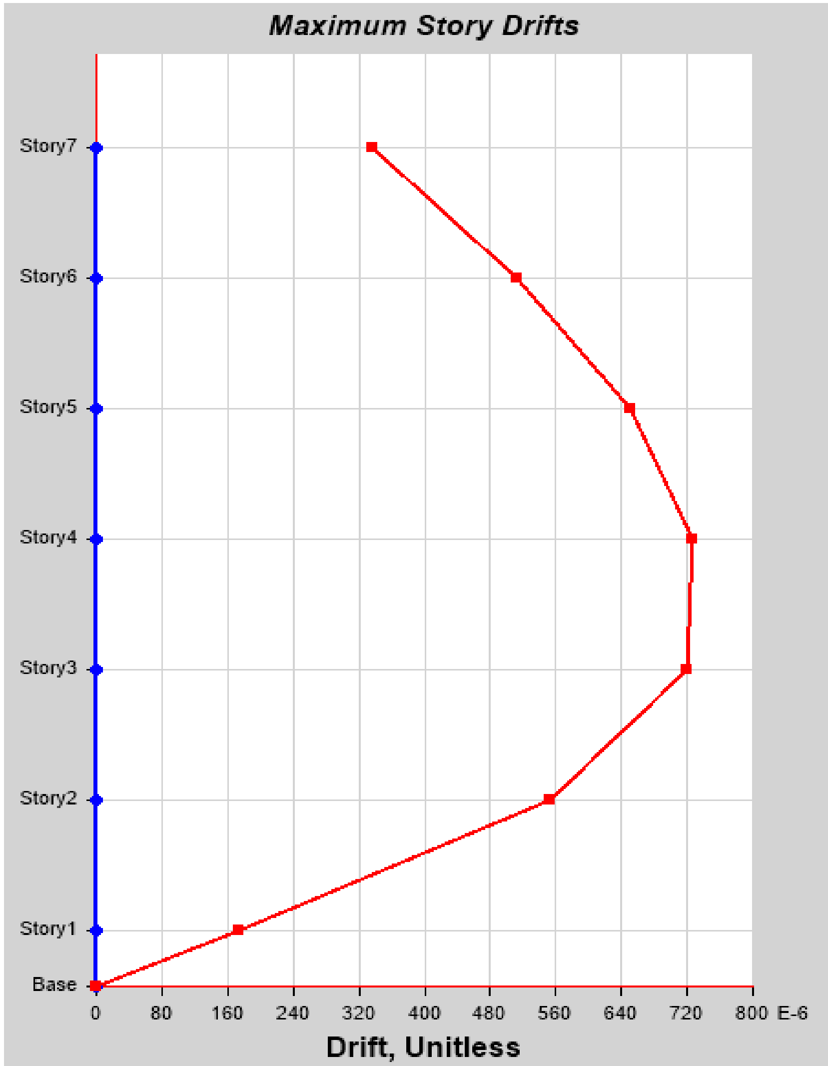
It is the relative displacement of one level relative to other levels above or below. According to IS 1893:2002 (part 1), the storey drift should not exceed 0.004 times of relative storey height.

- **Max. Storey drift comparison in the X direction of model 1**



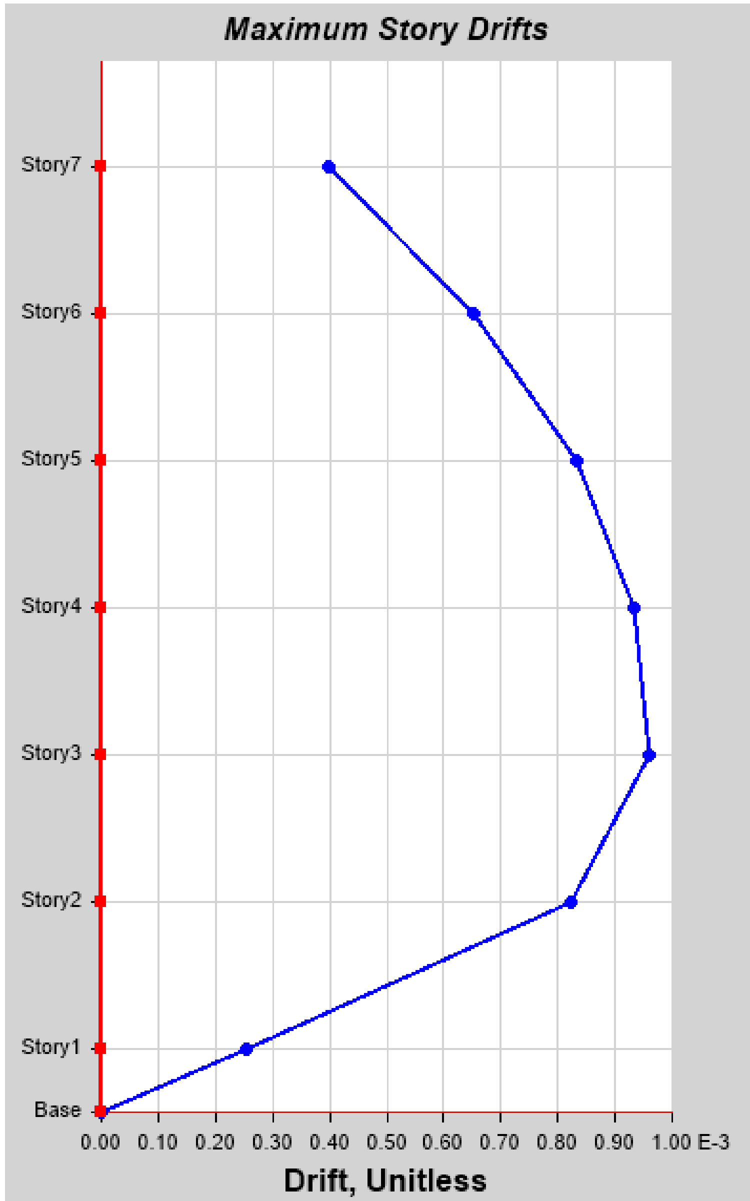
Graph 2.1 -Story Drift of model 1 X direction

- **Max. Storey drift comparison in the Y direction of model 1-**



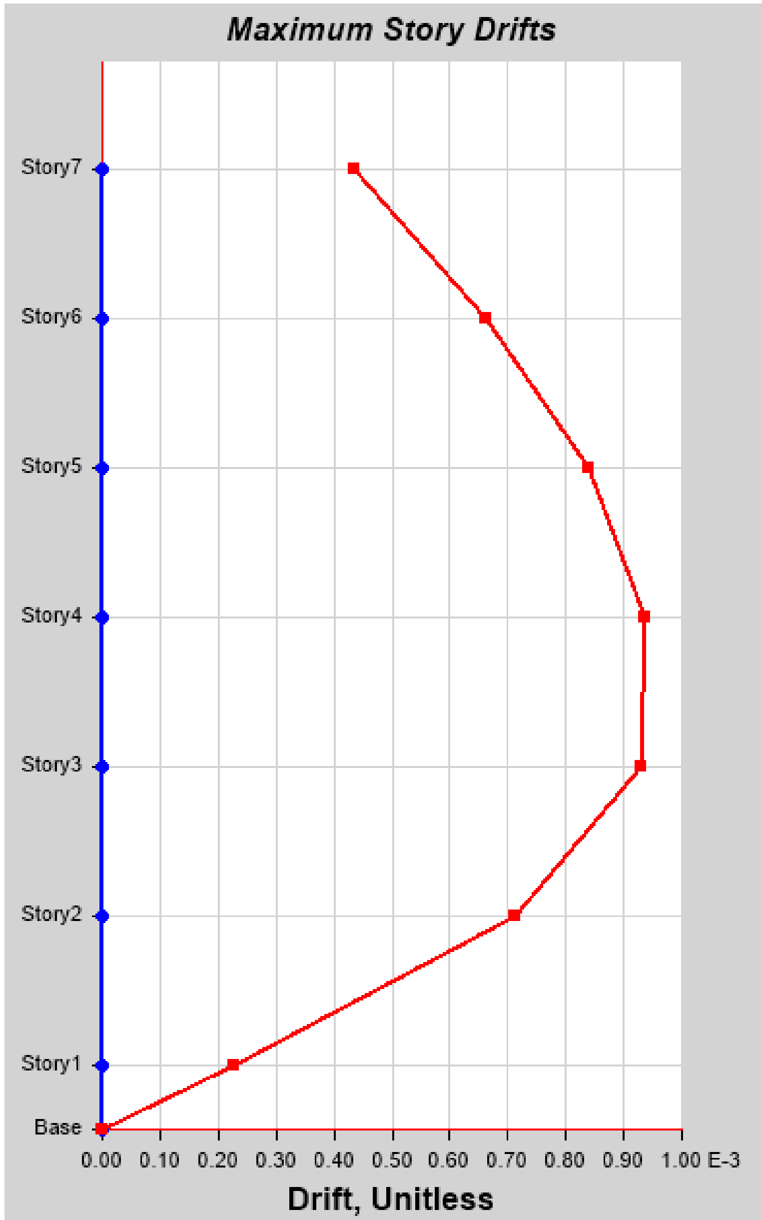
Graph 2.2 - Story Drift of model 1 Y direction

- **Max. Storey drift comparison in the X direction of model 2**



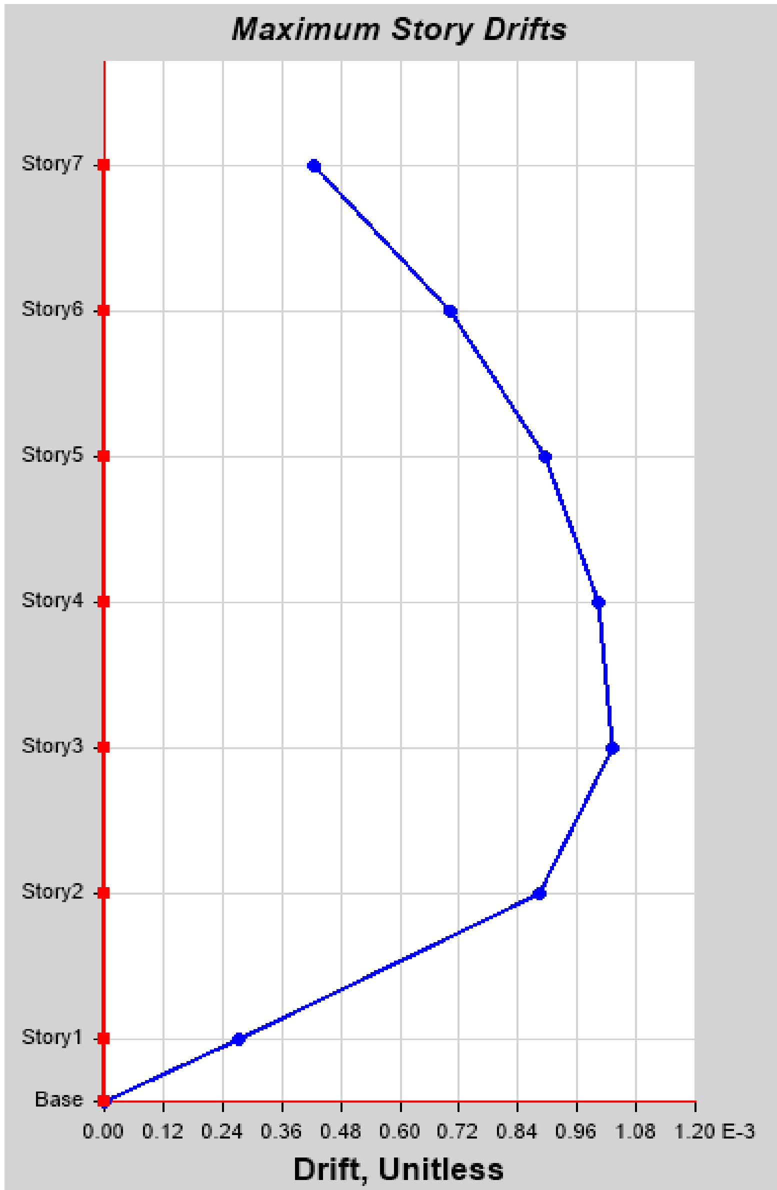
Graph 2.3 Story Drift of model 2 X direction

- **Max. Storey drift comparison in the Y direction of model 2**



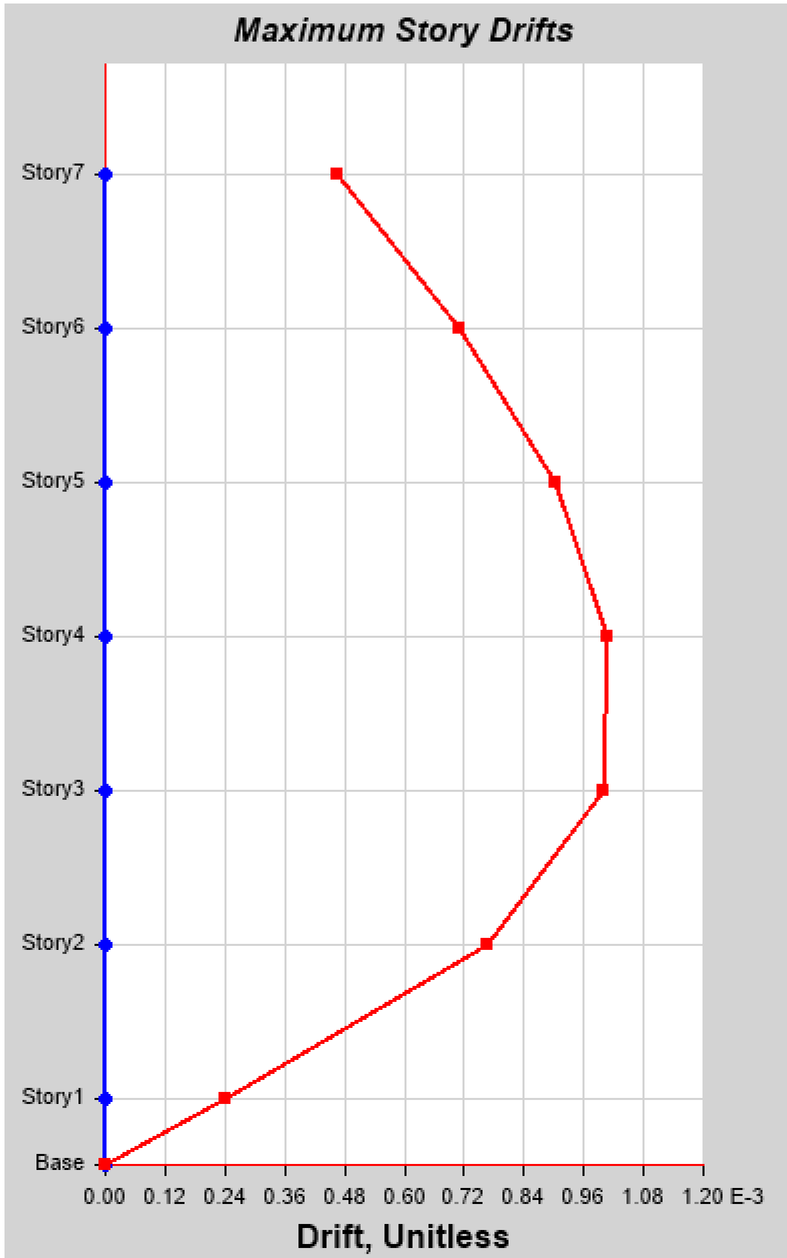
Graph 2.4 Story Drift of model 2 Y direction

- **Max. Storey drift comparison in the X direction of model 3**



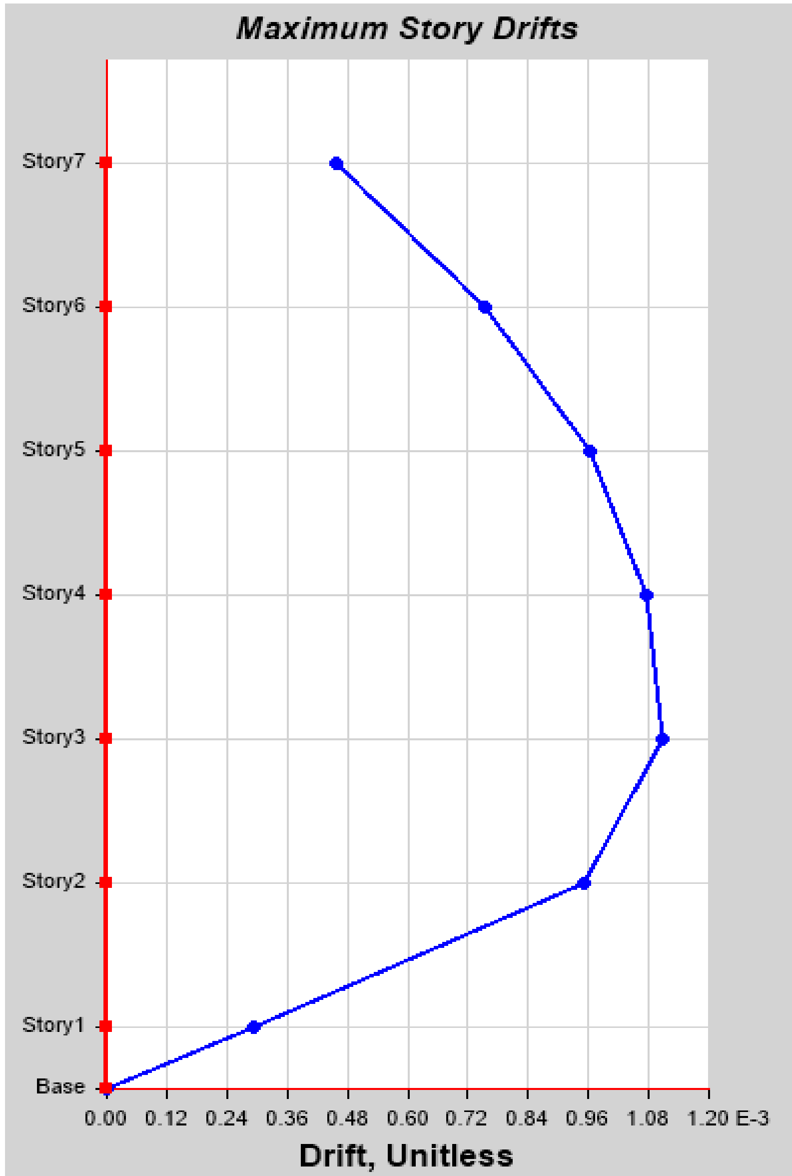
Graph 2.5 Story Drift of model 3 X direction

- **Max. Storey drift comparison in the Y direction of model 3**



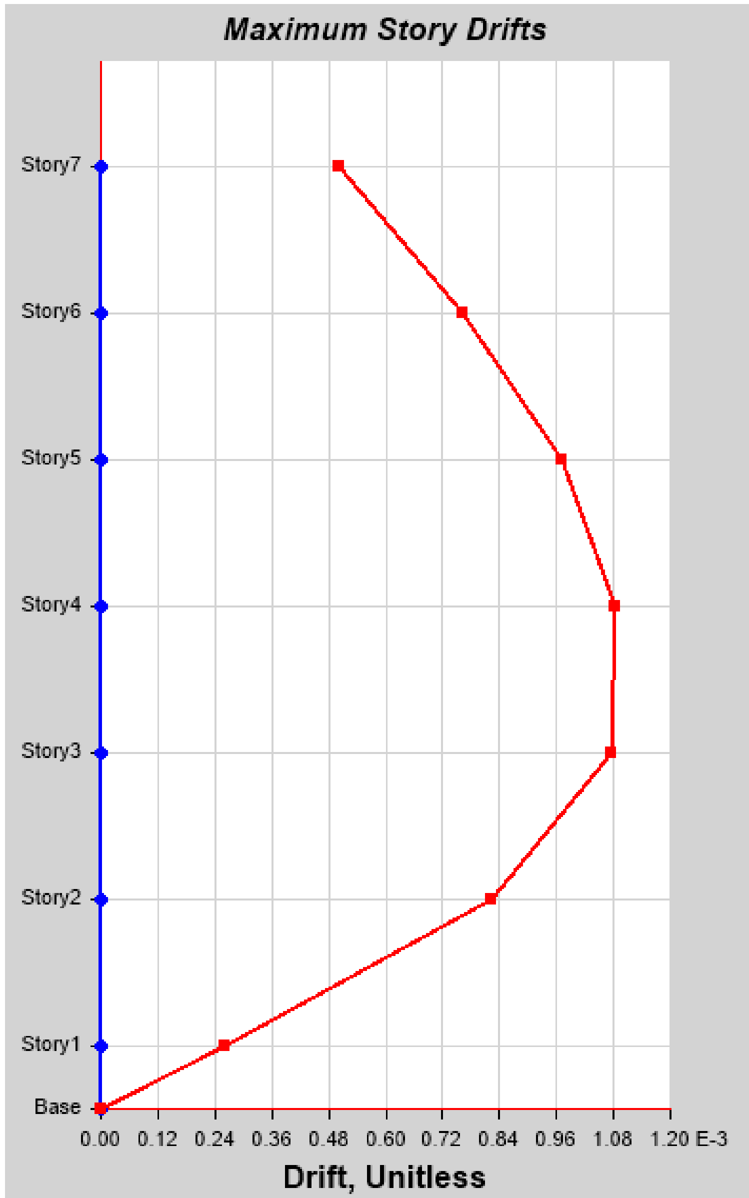
Graph 2.6 Story Drift of model 3 Y direction

- **Max. Storey drift comparison in the X direction of model 4**



Graph 2.7 Story Drift of model 4 X direction

- **Max. Storey drift comparison in Y direction of model 4**

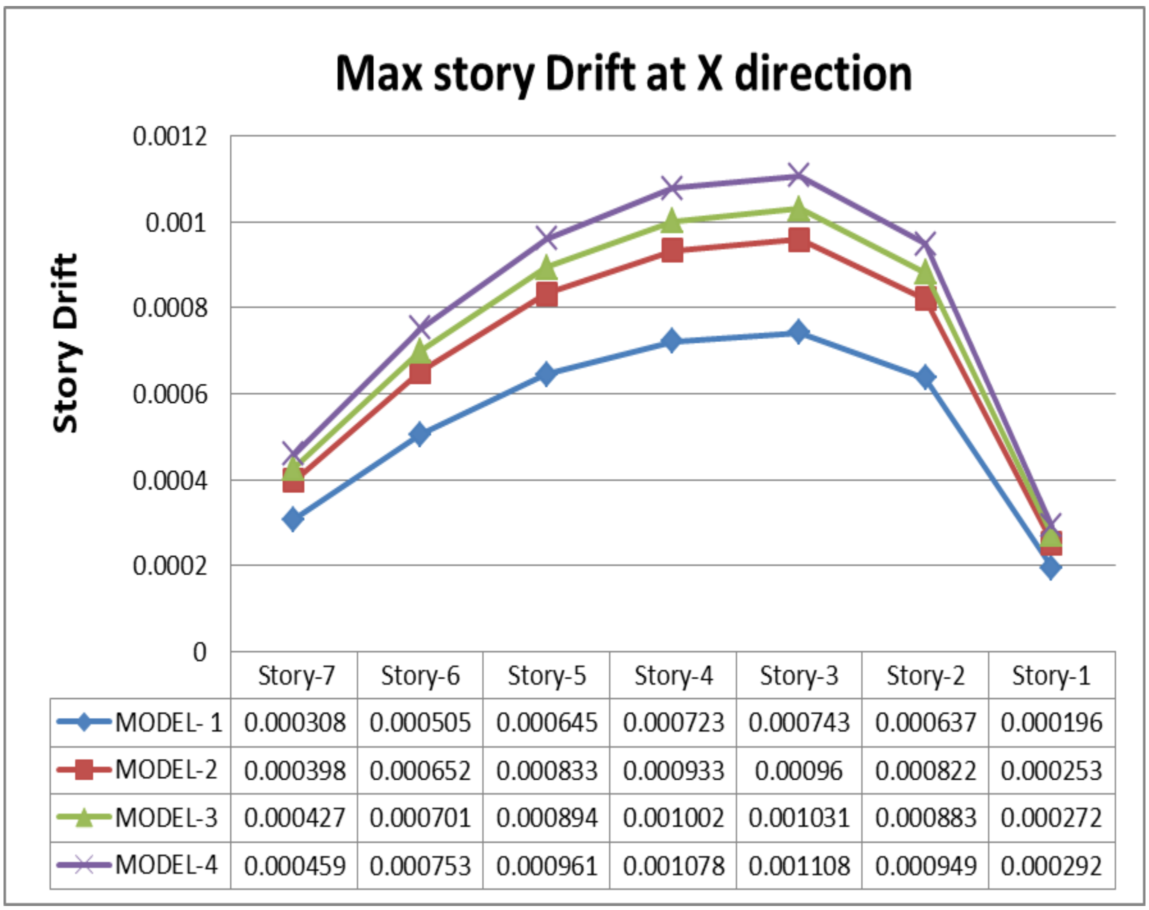


Graph 2.8 Story Drift of model 4 Y direction

Max.Storey drift comparison in x-direction- The table and graph below shows the comparison between the various building models

NO OF STOREY	MODEL- 1	MODEL-2	MODEL-3	MODEL-4
Story-7	0.000308	0.000398	0.000427	0.000459
Story-6	0.000505	0.000652	0.000701	0.000753
Story-5	0.000645	0.000833	0.000894	0.000961
Story-4	0.000723	0.000933	0.001002	0.001078
Story-3	0.000743	0.00096	0.001031	0.001108
Story-2	0.000637	0.000822	0.000883	0.000949
Story-1	0.000196	0.000253	0.000272	0.000292

Table 2.7 Max story drift X direction

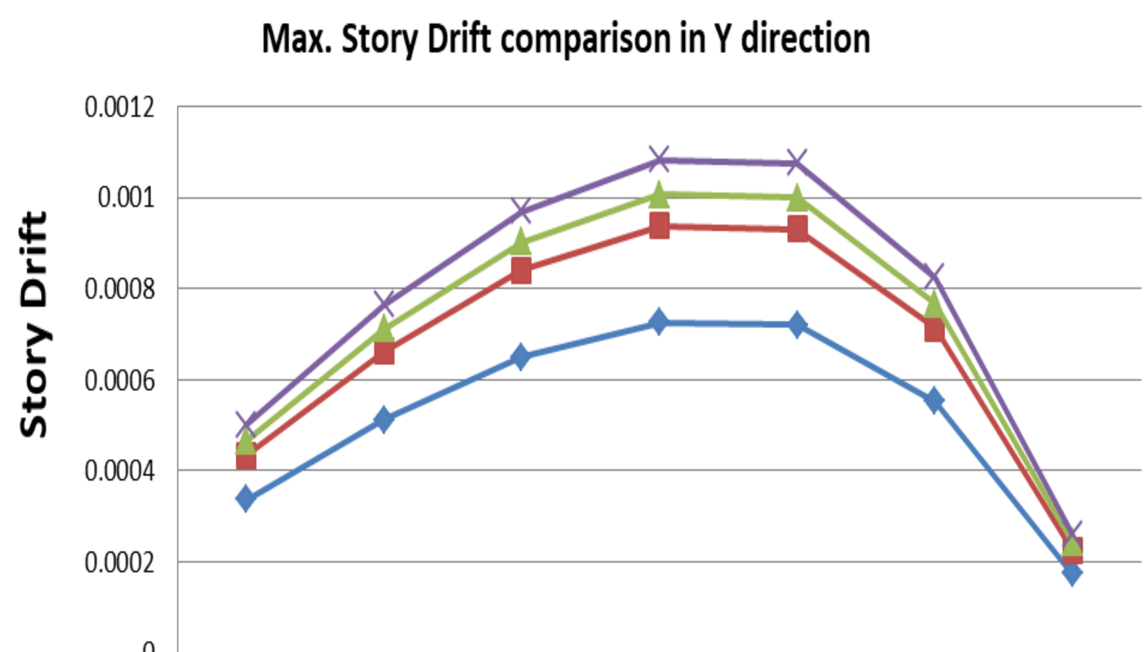


Graph 2.9 Max Story Drift X direction

Max.Storey drift comparison in Y-direction- The table and graph below shows the comparison between the various building models

NO OF STOREY	MODEL- 1	MODEL-2	MODEL-3	MODEL-4
Story-7	0.000336	0.000434	0.000466	0.000501
Story-6	0.000512	0.000662	0.000711	0.000764
Story-5	0.00065	0.00084	0.000902	0.00097
Story-4	0.000726	0.000937	0.001006	0.001082
Story-3	0.000721	0.000931	0.001	0.001075
Story-2	0.000553	0.000714	0.000767	0.000824
Story-1	0.000175	0.000226	0.000243	0.000261

Table 2.8 Max story drift at Y direction



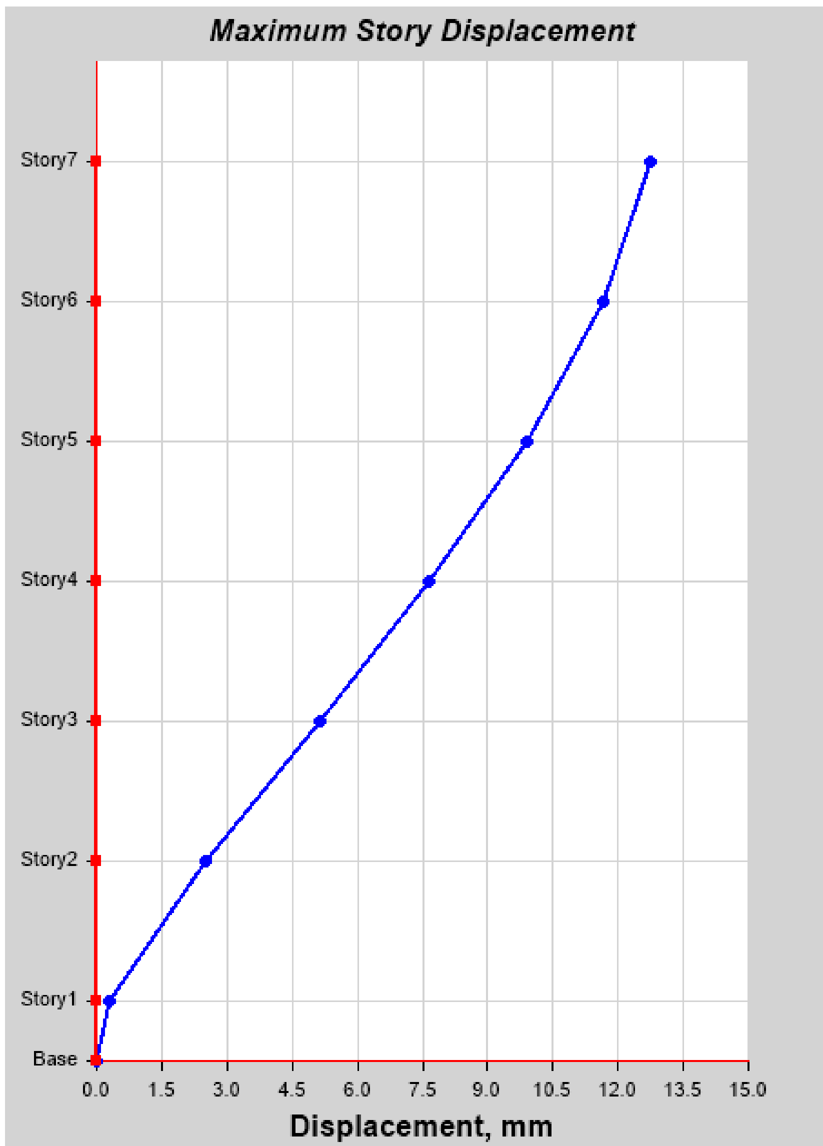
	Story-7	Story-6	Story-5	Story-4	Story-3	Story-2	Story-1
MODEL- 1	0.000336	0.000512	0.00065	0.000726	0.000721	0.000553	0.000175
MODEL-2	0.000434	0.000662	0.00084	0.000937	0.000931	0.000714	0.000226
MODEL-3	0.000466	0.000711	0.000902	0.001006	0.001	0.000767	0.000243
MODEL-4	0.000501	0.000764	0.00097	0.001082	0.001075	0.000824	0.000261

Graph 2.10 Max Story Drift Y direction

4.2 Storey Displacement

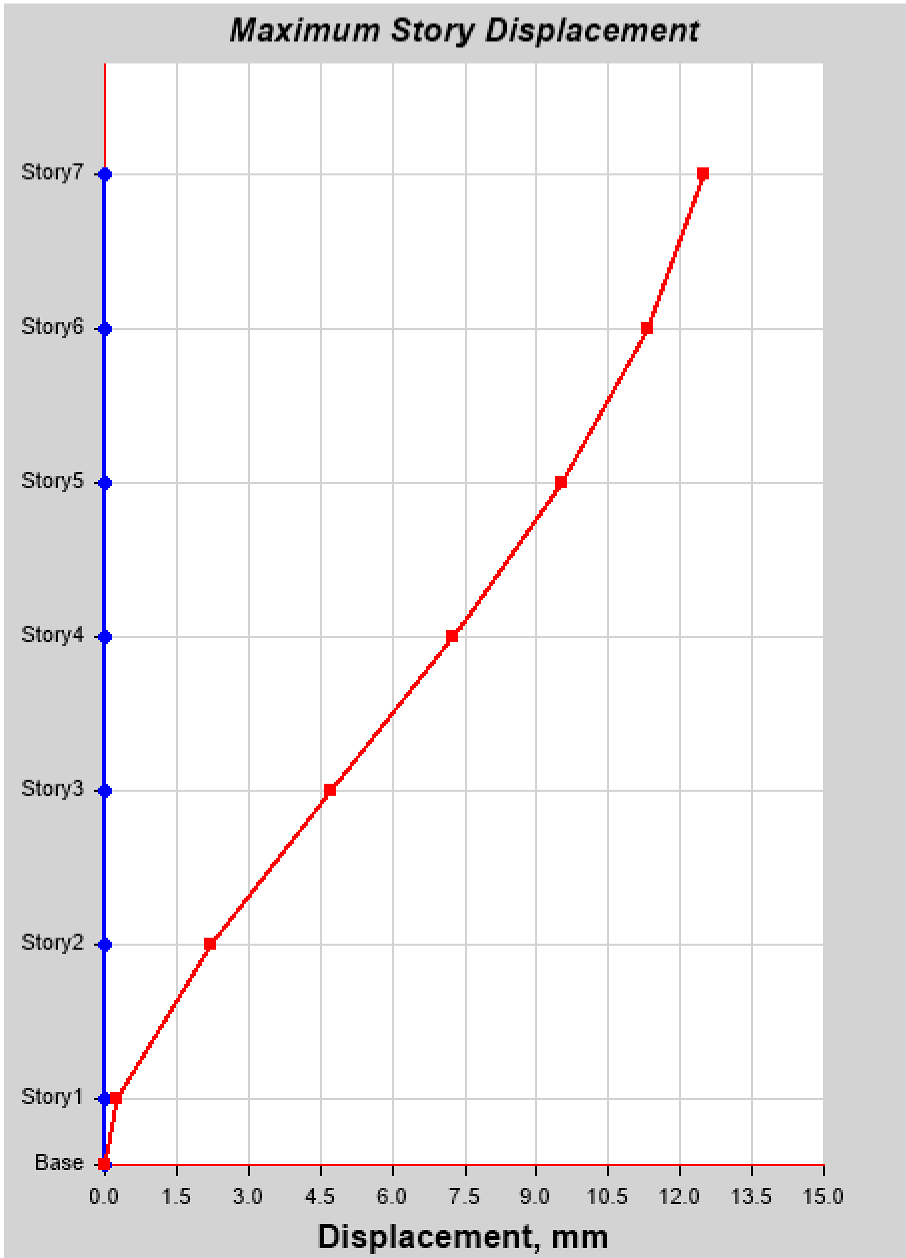
It is the displacement of each storey with respect to ground level. According to IS 1893 (part1):2002 the max value of displacement is 1/250 times of story height with respect to ground.

- **Max story displacement for model 1 at X direction**



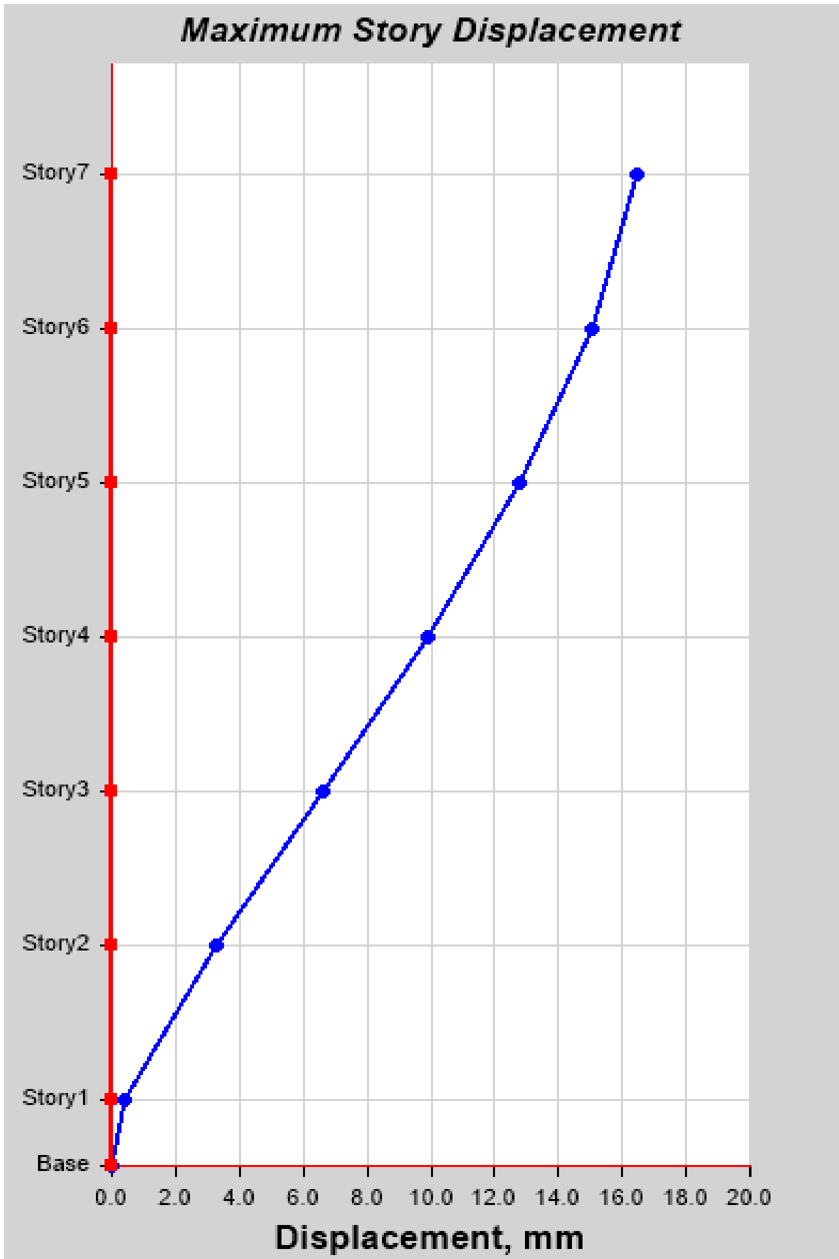
Graph 2.11 Max Story Displacement model 1 x-direction

- **Max story displacement for model 1 at Y direction**



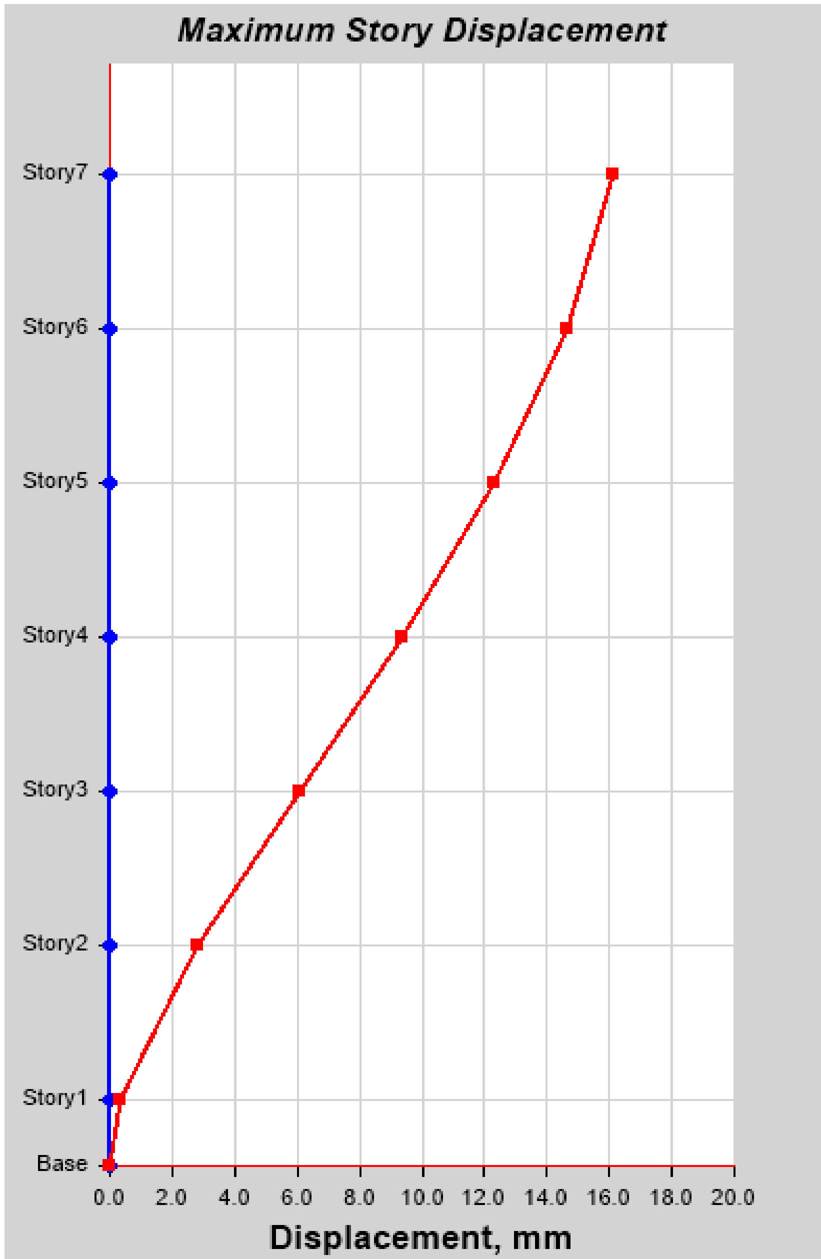
Graph 2.12 Max Story Displacement model 1 Y-direction

- **Max story displacement for model 2 at X direction**



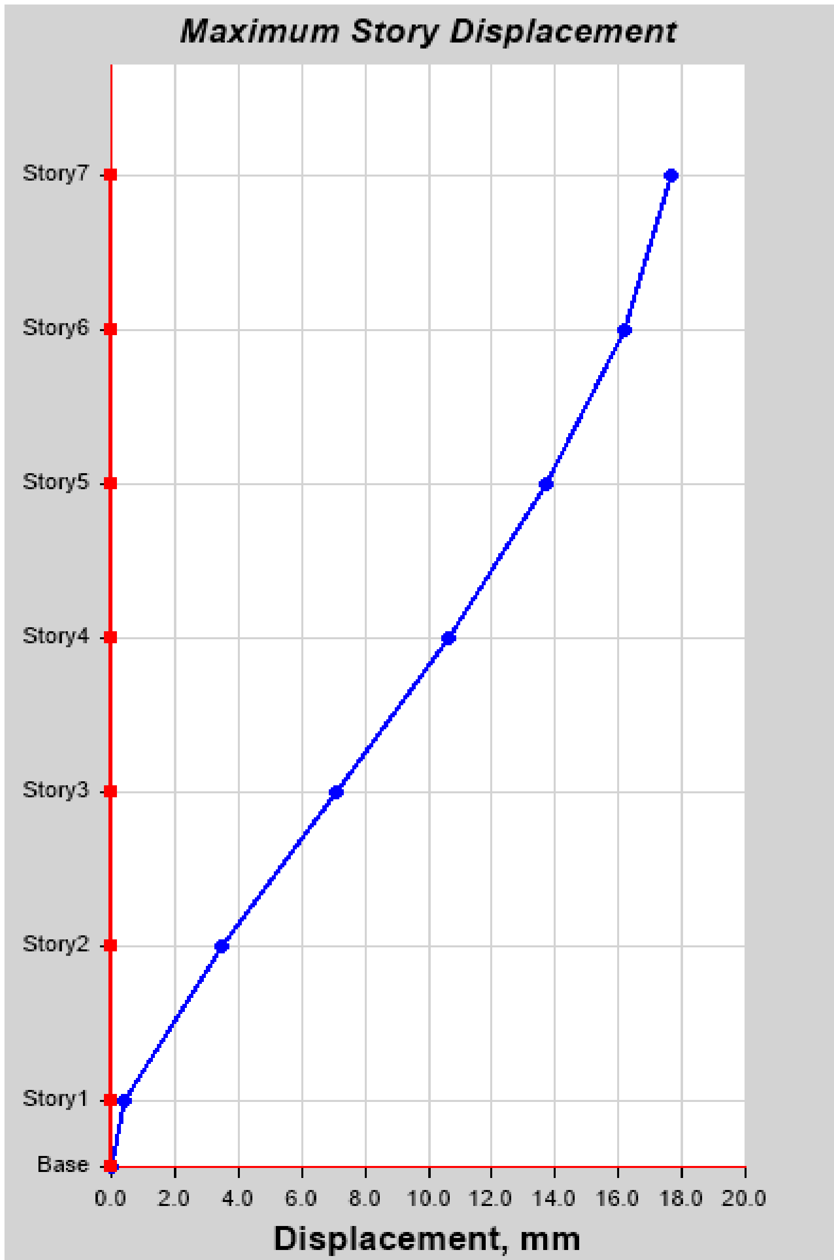
Graph 2.13 Max Story Displacement model 2 x-direction

- **Max story displacement for model 2 at Y direction**



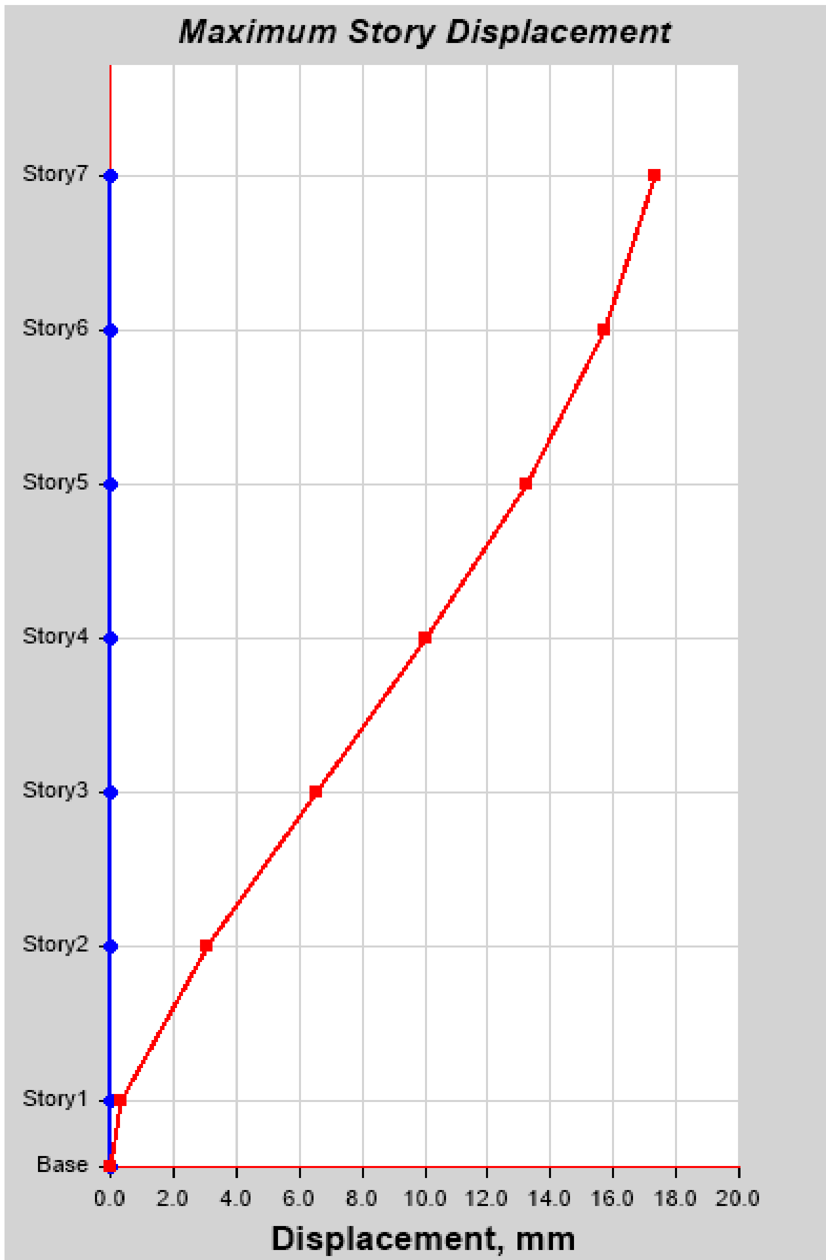
Graph 2.14 Max Story Displacement model 2 Y-direction

- **Max story displacement for model 3 at X direction**



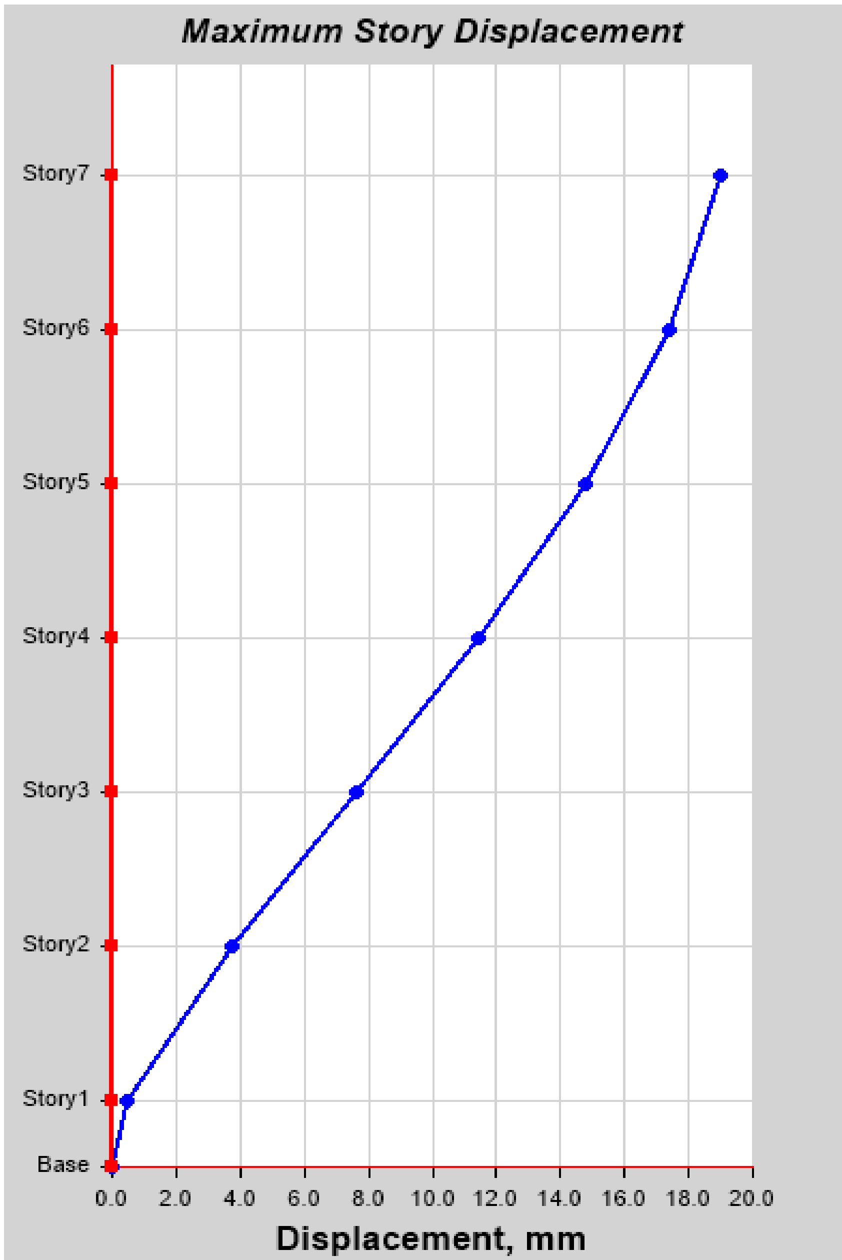
Graph 2.15 Max Story Displacement model 3 x-direction

- **Max story displacement for model 3 at Y direction**



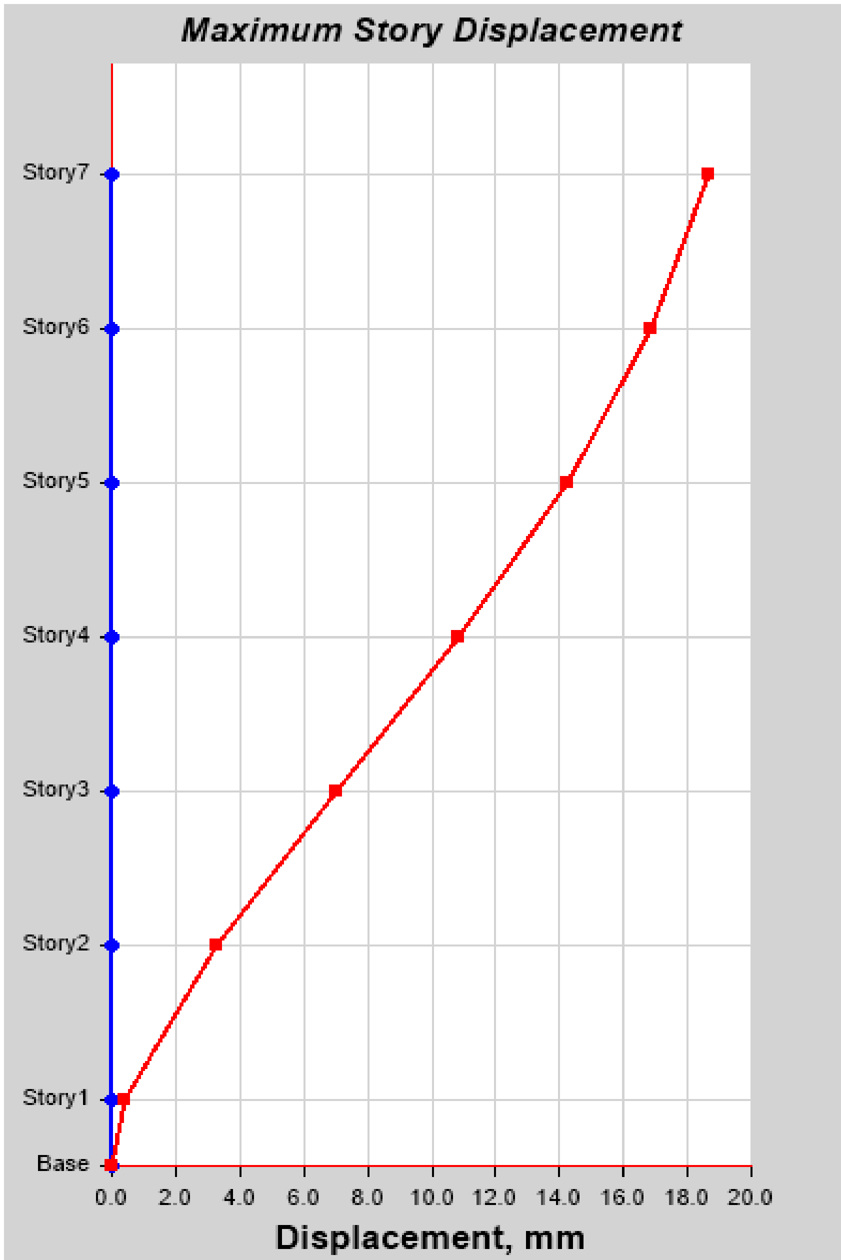
Graph 2.16 Max Story Displacement model 3 at y-direction

- **Max story displacement for model 4 at X direction**



Graph 2.17 Max Story Displacement model 4 x-direction

- **Max story displacement for model 4 at Y direction**

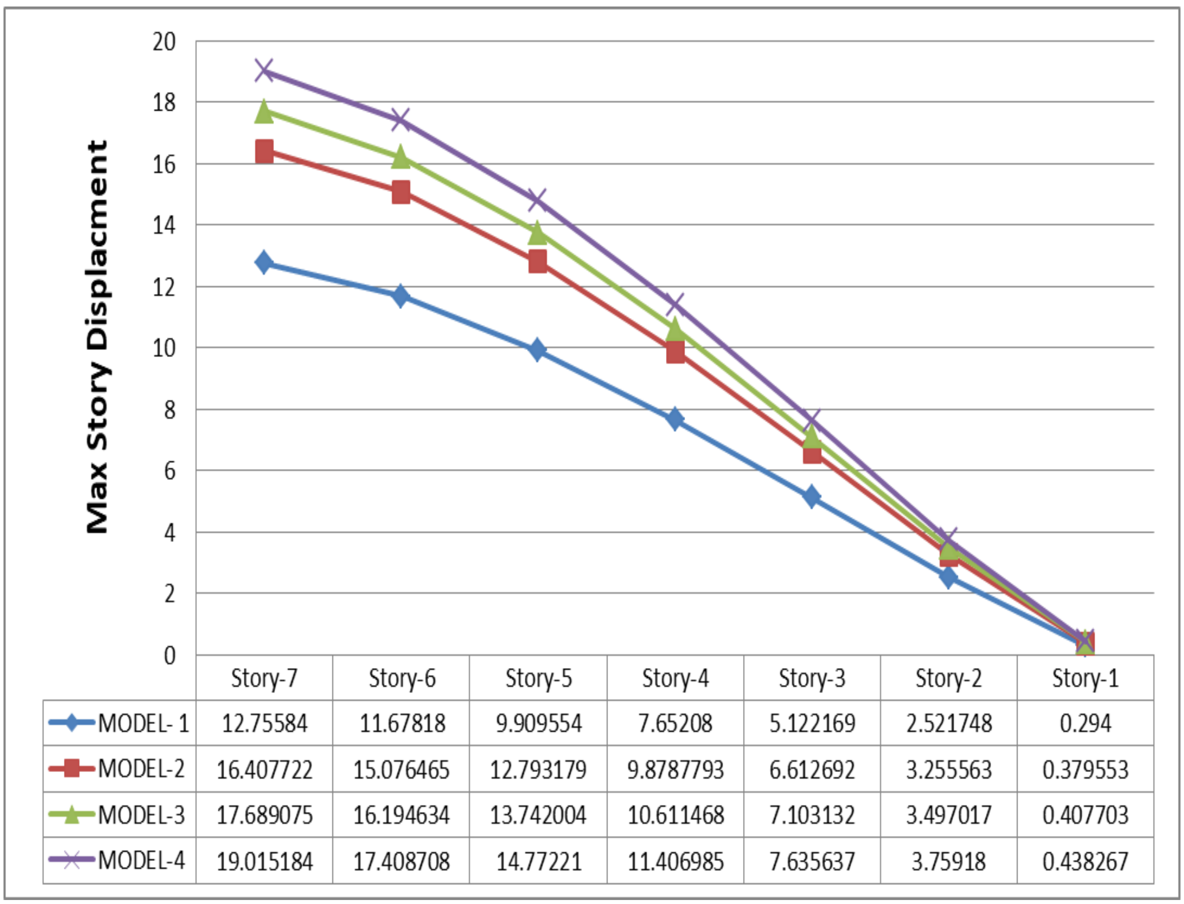


Graph 2.18 Max Story Displacement model 4 y-direction

4.2.1 Max.Storey displacement (mm) comparison in x-direction- The table and graph below shows the comparison of the various models in terms of storey displacement in X direction

NO OF STOREY	MODEL- 1	MODEL-2	MODEL-3	MODEL-4
Story-7	12.75584	16.407722	17.689075	19.015184
Story-6	11.67818	15.076465	16.194634	17.408708
Story-5	9.909554	12.793179	13.742004	14.77221
Story-4	7.65208	9.8787793	10.611468	11.406985
Story-3	5.122169	6.612692	7.103132	7.635637
Story-2	2.521748	3.255563	3.497017	3.75918
Story-1	0.294	0.379553	0.407703	0.438267

Table 2.8 Max story Displacement at X direction

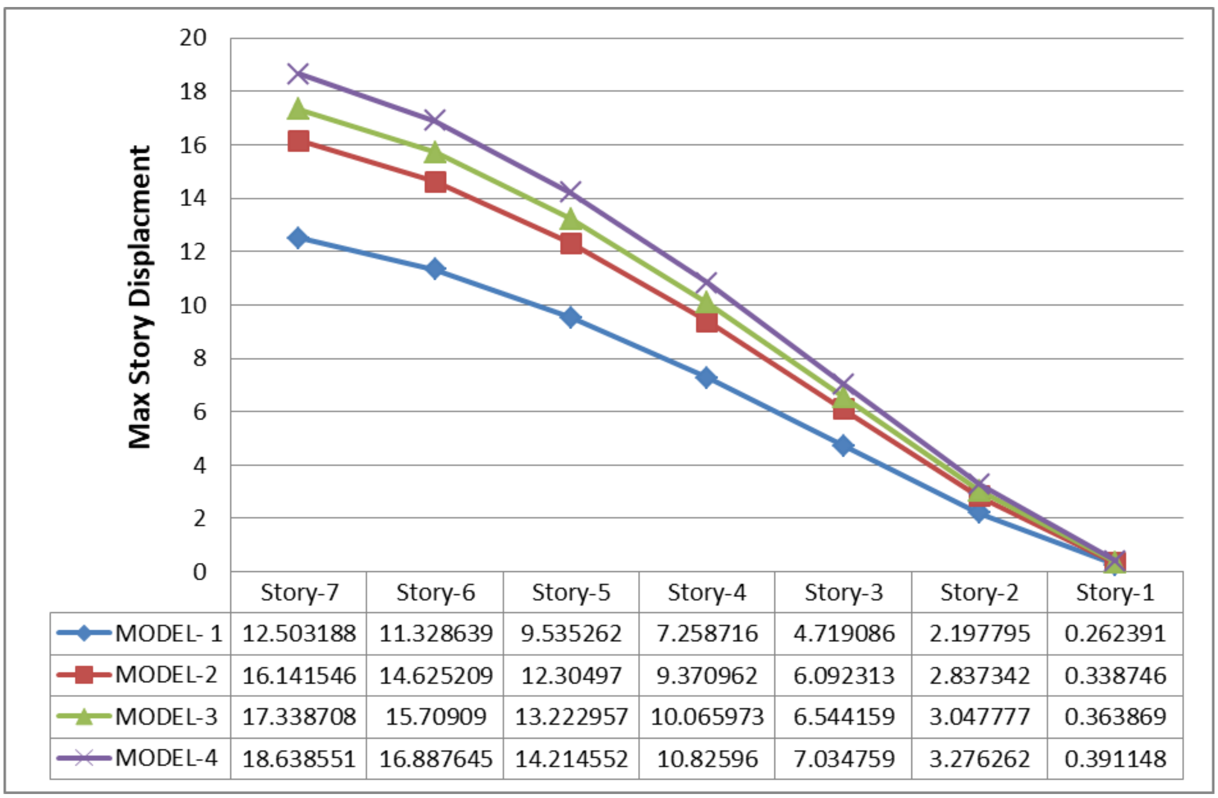


Graph 2.19 Max Story Displacement X-direction

4.2.2 Max. Storey displacement (mm) comparison in Y direction- Table and graph below show the comparison of various models in terms of storey displacement in the Y direction.

NO OF STOREY	MODEL- 1	MODEL-2	MODEL-3	MODEL-4
Story-7	12.503188	16.141546	17.338708	18.638551
Story-6	11.328639	14.625209	15.70909	16.887645
Story-5	9.535262	12.30497	13.222957	14.214552
Story-4	7.258716	9.370962	10.065973	10.82596
Story-3	4.719086	6.092313	6.544159	7.034759
Story-2	2.197795	2.837342	3.047777	3.276262
Story-1	0.262391	0.338746	0.363869	0.391148

Table 2.9 Max story Displacement at Y direction



Graph 2.20 Max Story Displacement Y-direction

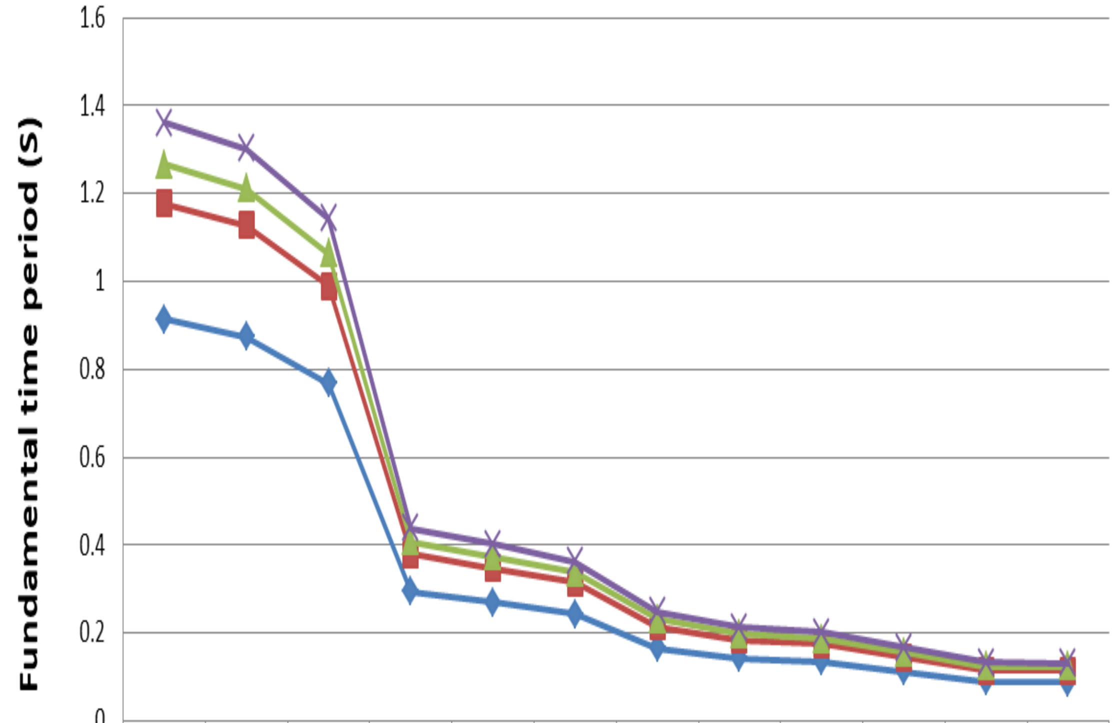
4.3 Fundamental time periods-

Every object has a natural vibration frequency and so has every structure. When a structure is excited by seismic forces, it starts to vibrate. The lowest natural frequency (f) of vibration of a structure corresponds to the longest time period (T) of vibration, as frequency and time period are inversely proportional (T=1/f). This is also referred to as the first mode of vibration or a fundamental period of vibration. The structure will have multiple natural modes of vibration for which frequency will be higher and time period will be shorter than the fundamental period. According to IS 1893(Part 1):2002 it is the first(longest) modal time period of vibration.

4.4.1 Fundamental time period (S) comparison-The table and the graph below shows the comparison of various models at a various temperature in terms of the fundamental time period.

Modal	MODEL- 1	MODEL-2	MODEL-3	MODEL-4
Modal 1	0.913	1.178	1.266	1.36
Modal 2	0.874	1.128	1.212	1.303
Modal 3	0.767	0.99	1.064	1.143
Modal 4	0.294	0.379	0.407	0.438
Modal 5	0.269	0.348	0.374	0.402
Modal 6	0.244	0.315	0.338	0.363
Modal 7	0.166	0.215	0.231	0.248
Modal 8	0.143	0.184	0.198	0.212
Modal 9	0.135	0.174	0.187	0.201
Modal 10	0.112	0.145	0.156	0.167
Modal 11	0.089	0.115	0.124	0.133
Modal 12	0.088	0.114	0.123	0.132

Table 2.10 Fundamental time periods



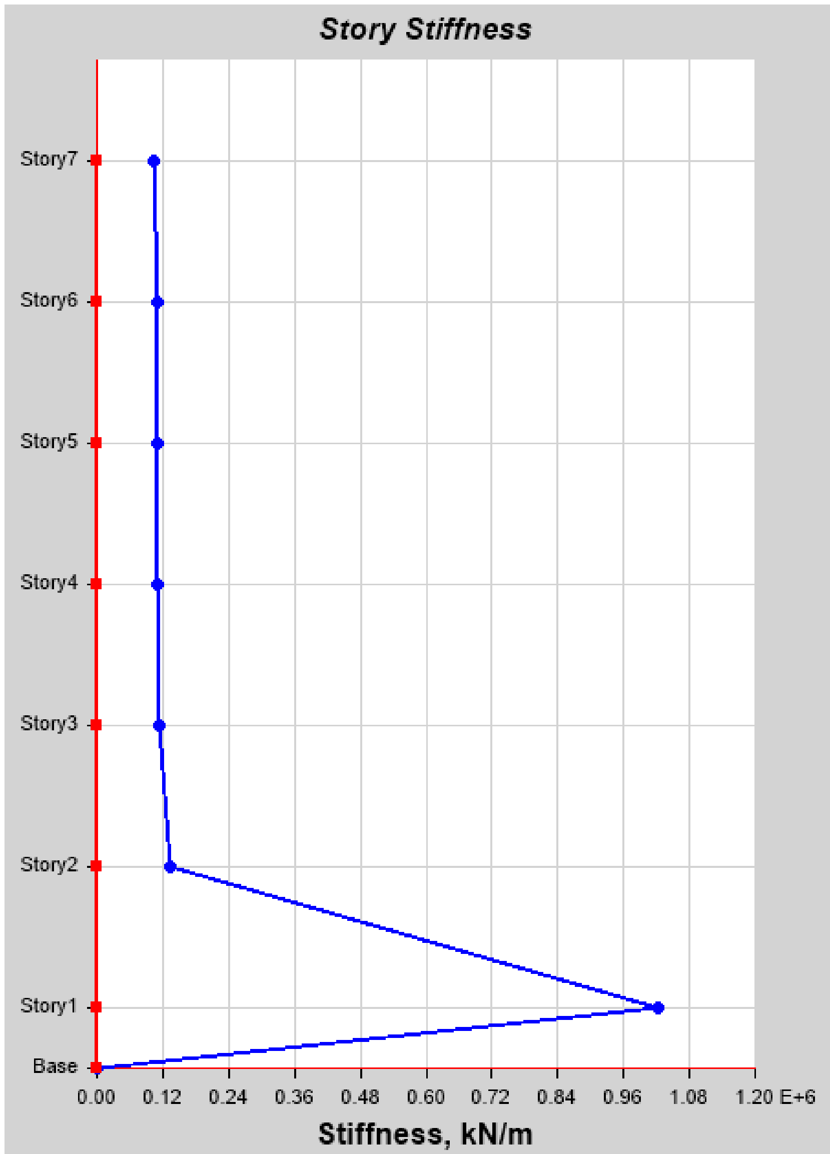
	Modal 1	Modal 2	Modal 3	Modal 4	Modal 5	Modal 6	Modal 7	Modal 8	Modal 9	Modal 10	Modal 11	Modal 12
MODEL- 1	0.913	0.874	0.767	0.294	0.269	0.244	0.166	0.143	0.135	0.112	0.089	0.088
MODEL-2	1.178	1.128	0.99	0.379	0.348	0.315	0.215	0.184	0.174	0.145	0.115	0.114
MODEL-3	1.266	1.212	1.064	0.407	0.374	0.338	0.231	0.198	0.187	0.156	0.124	0.123
MODEL-4	1.36	1.303	1.143	0.438	0.402	0.363	0.248	0.212	0.201	0.167	0.133	0.132

Table 2.20 fundamental time period

4.4 Story stiffness

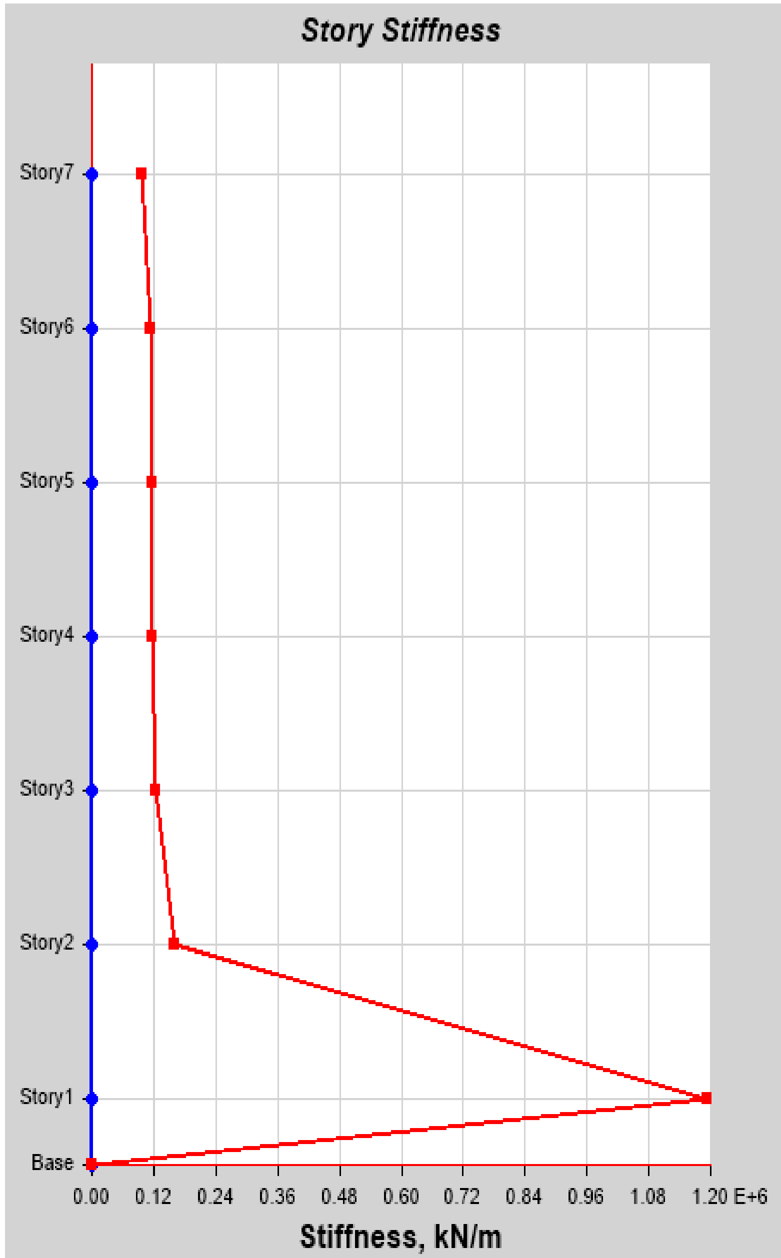
As per IS 1893:2002 the lateral stiffness is less than 70 percent of that in the storey above or less than 80 percent of average lateral stiffness of the three-storey above.

- **Story stiffness model 1 at X direction**



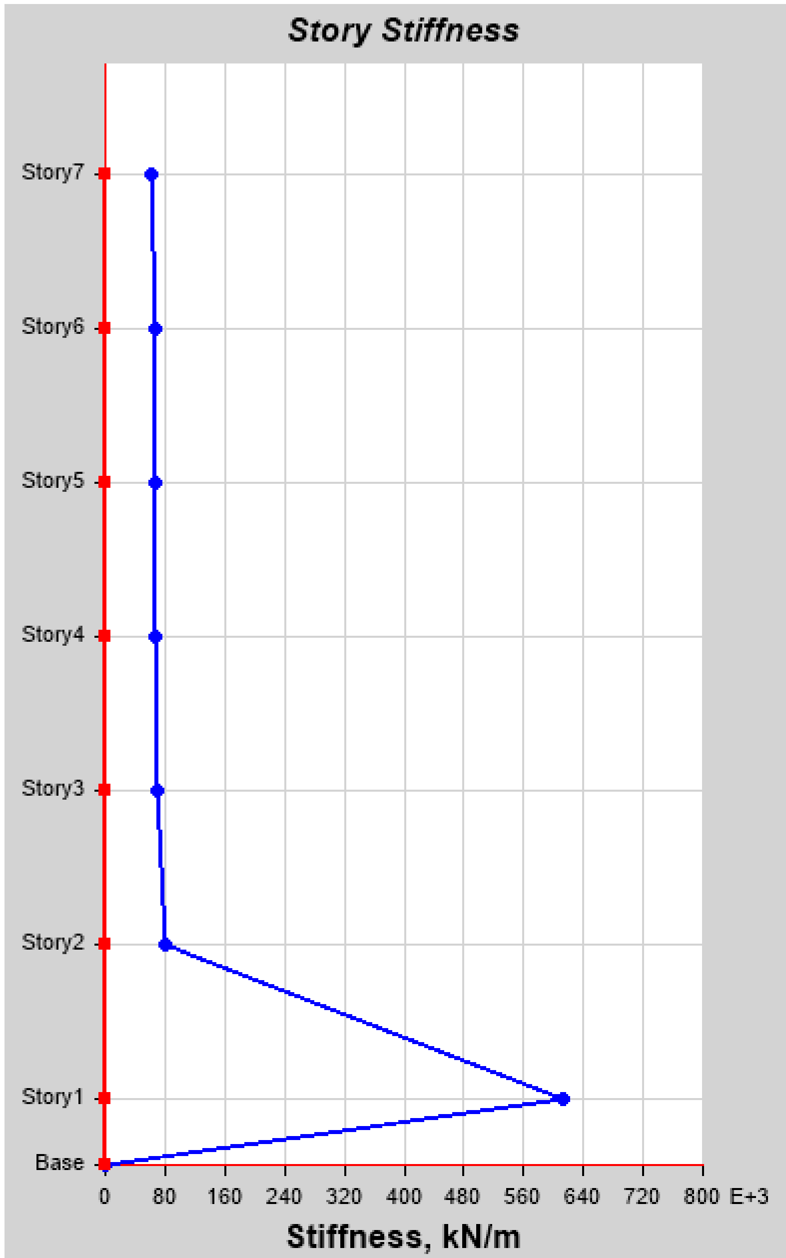
Graph 2.21 Story stiffness model 1 at X direction

- **Story stiffness model 1 at Y direction**



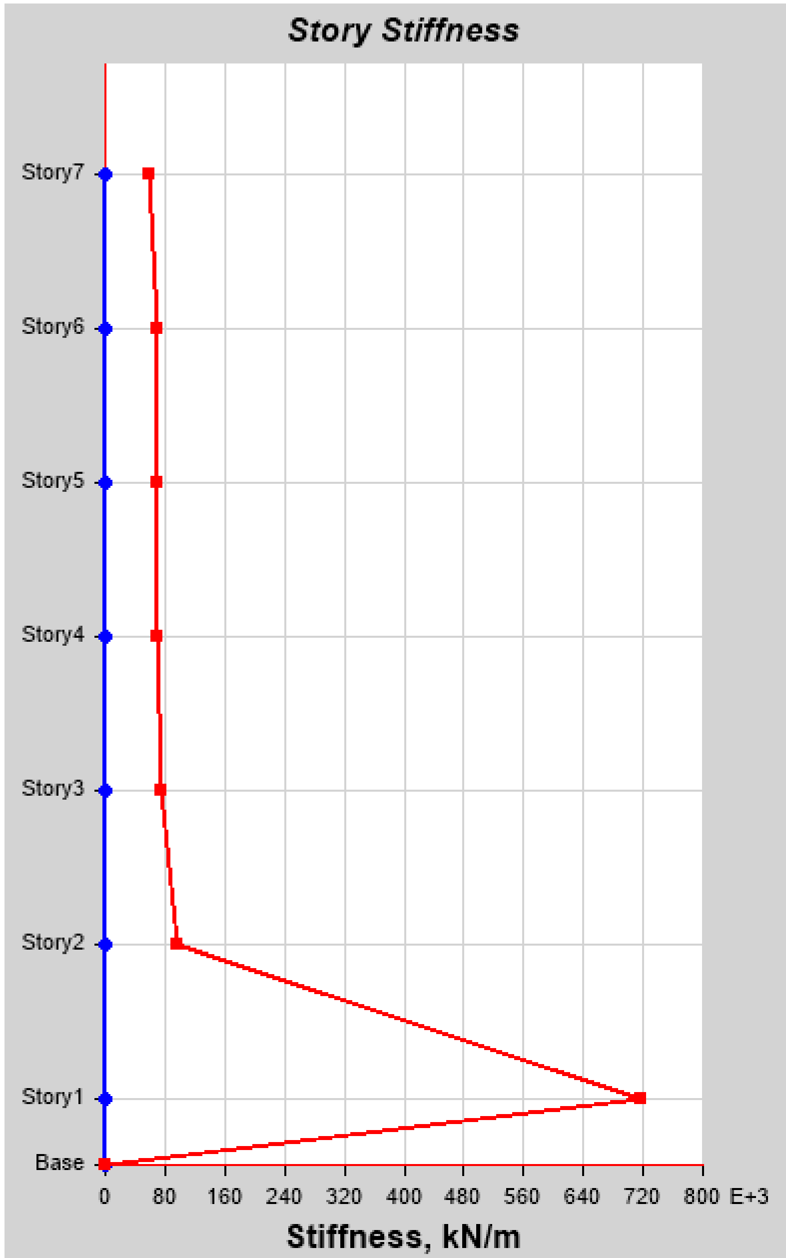
Graph 2.22 Story stiffness model 1 at Y direction

- **Story stiffness model 2 at X direction**



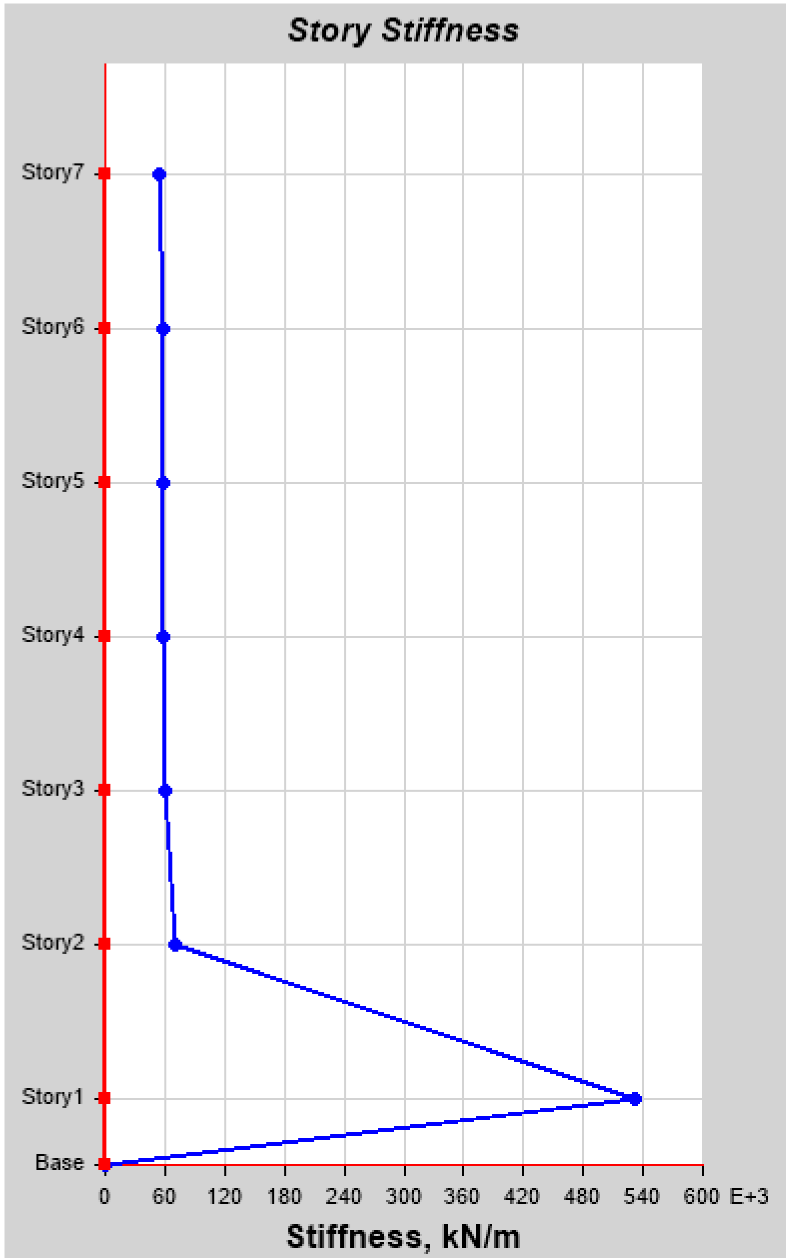
Graph 2.23 Story stiffness model 2 at X direction

- **Story stiffness model 2 at Y direction**



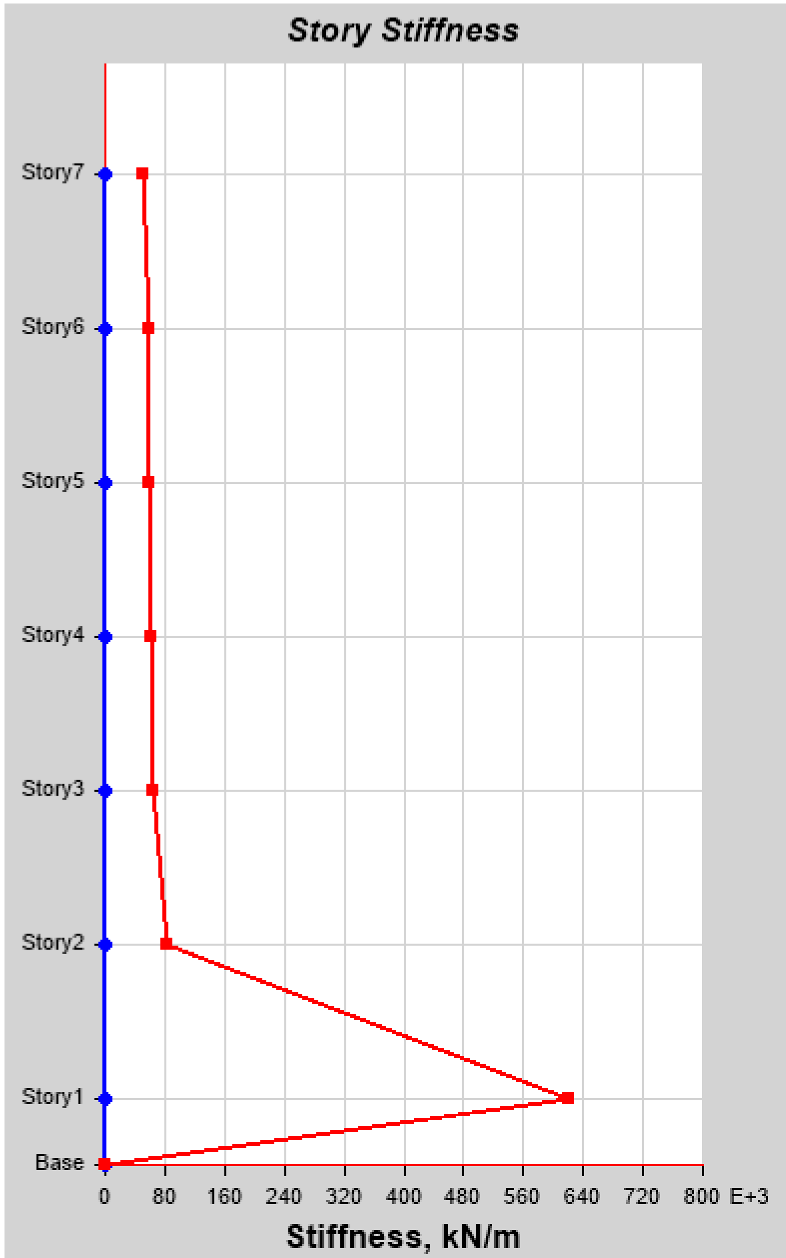
Graph 2.24 Story stiffness model 2 at Y direction

- **Story stiffness model 3 at X direction**



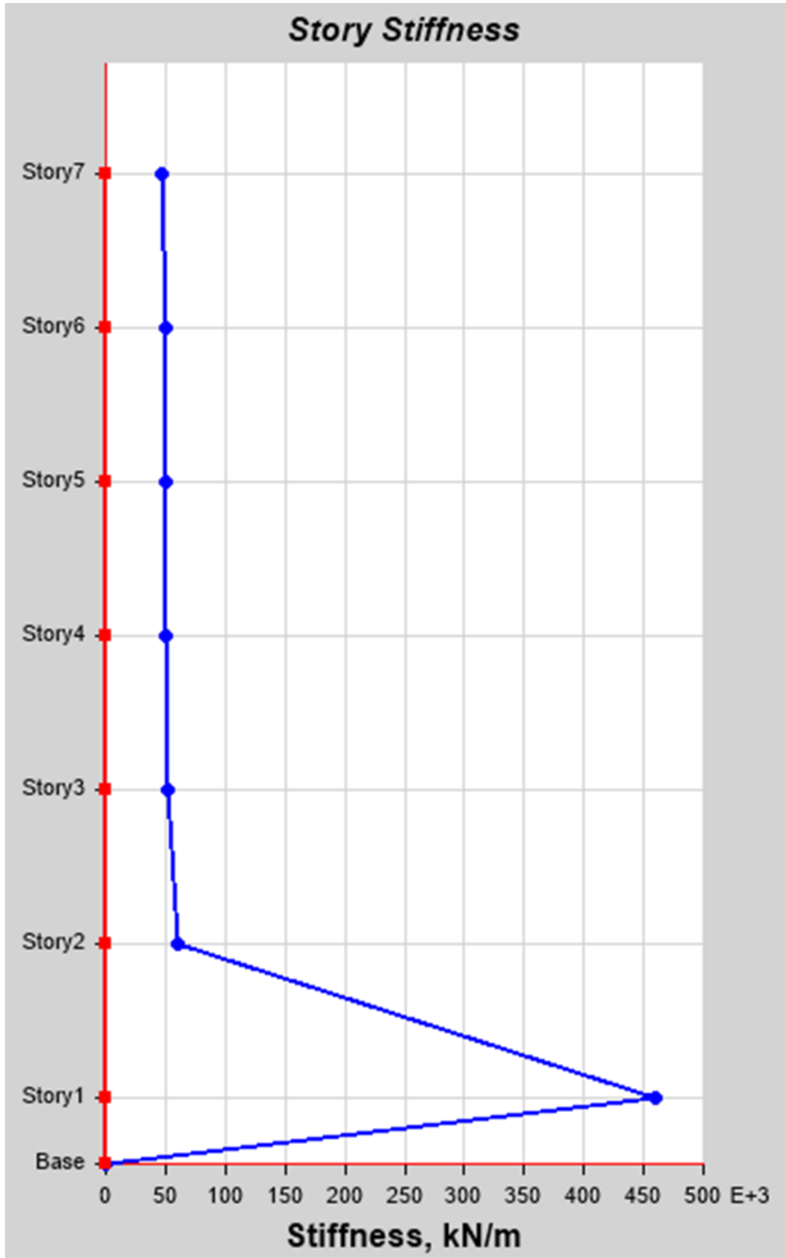
Graph 2.25 Story stiffness model 3 at X direction

- **Story stiffness model 3 at Y direction**



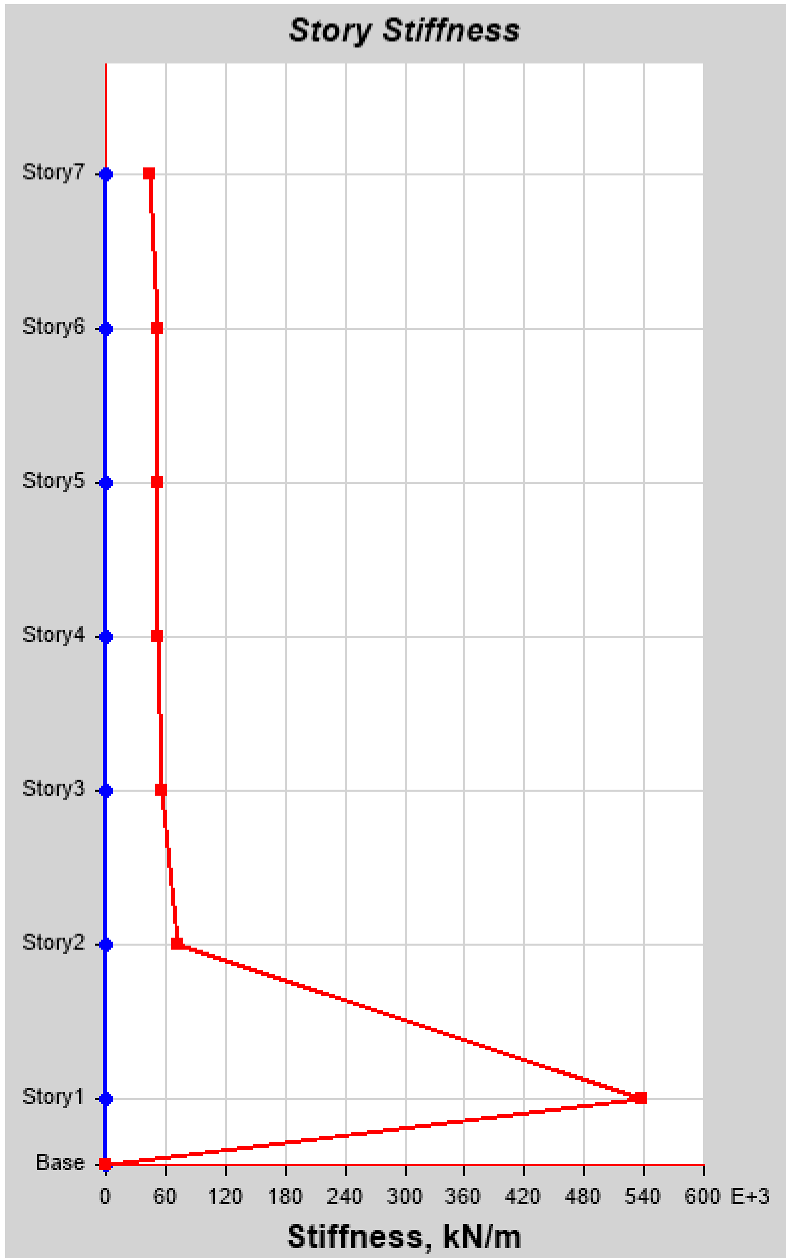
Graph 2.26 Story stiffness model 3 at Y direction

- **Story stiffness model 4 at X direction**



Graph 2.27 Story stiffness model 4 at X direction

- **Story stiffness model 4 at Y direction**



Graph 2.28 Story stiffness model 4 at Y direction

4.4.1 Max. Storey stiffness (kN/m) comparison in x direction-The table and graph below shows the comparison of various models in terms of storey stiffness in X direction.

NO OF STOREY	MODEL- 1	MODEL-2	MODEL-3	MODEL-4
Story-7	**102385**	**61431.23**	**53240.97**	**46073.92**
Story-6	**111112**	**66667.12**	**57778.80**	**50000.88**
Story-5	**111465**	**66879.15**	**57962.56**	**50159.91**
Story-4	**111717**	**67030.41**	**58093.65**	**50273.35**
Story-3	**113953**	**68372.01**	**59256.39**	**51279.56**
Story-2	**134610**	**80765.71**	**69997.71**	**60574.94**
Story-1	**1021717**	**613030**	**531299**	**459778**

Table 2.11- Max.Storey stiffness at x direction

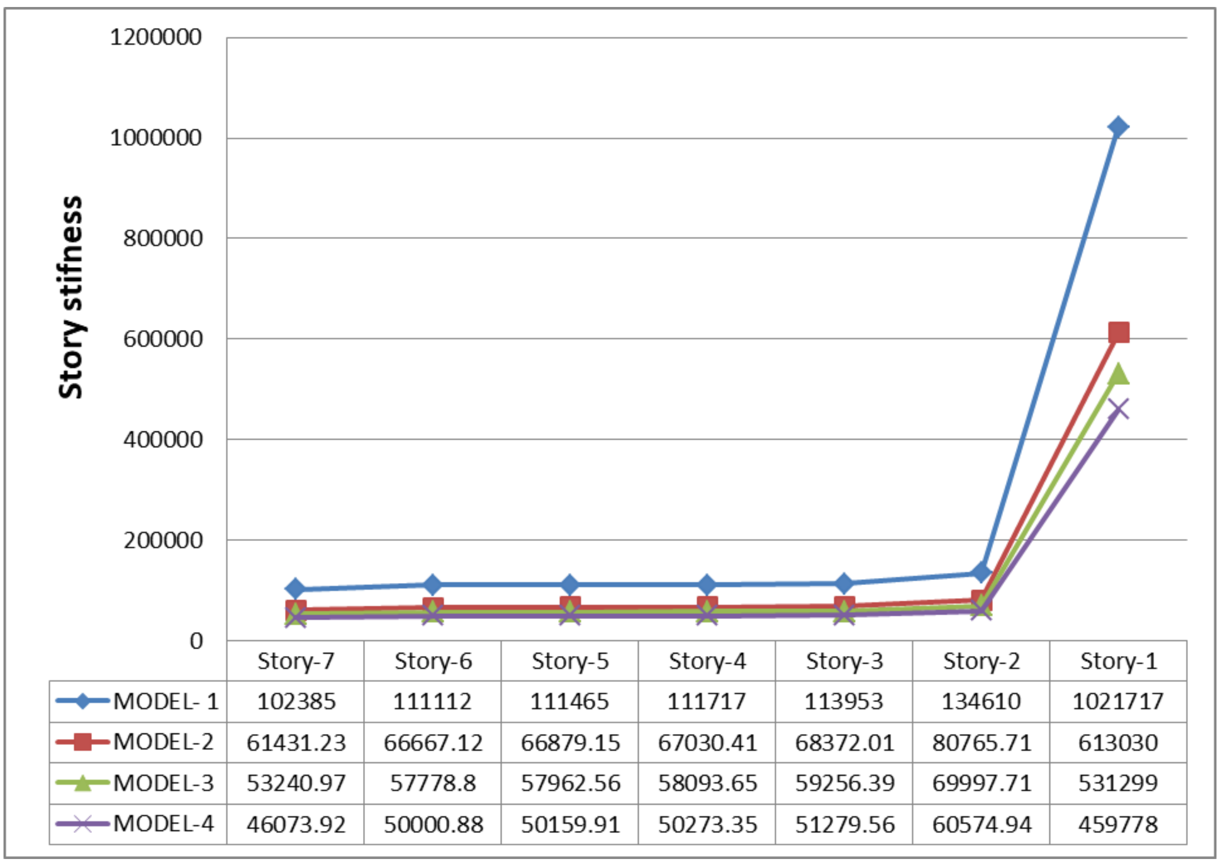
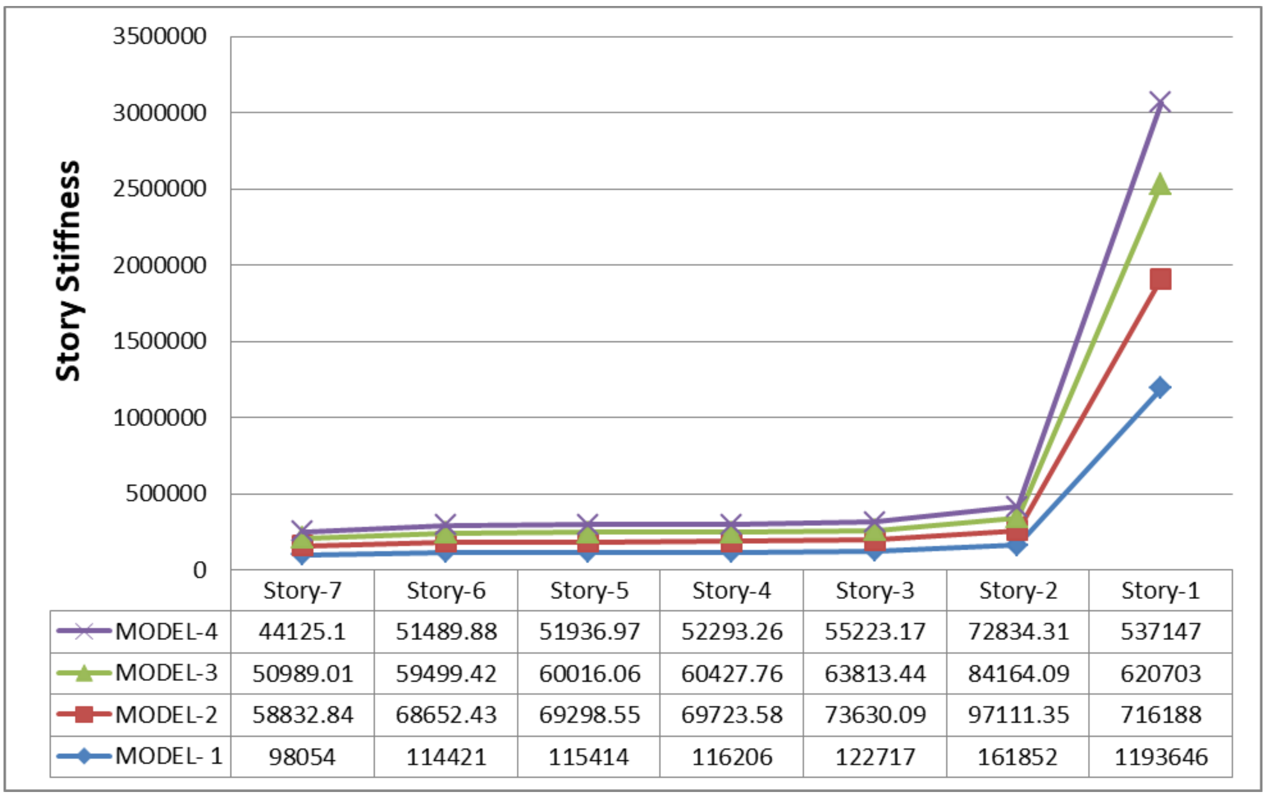


Table 2,29 storey stiffness in x-direction

4.1.1 **Max. Storey stiffness (kN/m) comparison in Y direction**-the table and graph below shows the comparison of various models in terms of storey stiffness in Y direction.

NO OF STOREY	MODEL- 1	MODEL-2	MODEL-3	MODEL-4
Story-7	**98054**	**58832.84**	**50989.01**	**44125.10**
Story-6	**114421**	**68652.43**	**59499.42**	**51489.88**
Story-5	**115414**	**69298.55**	**60016.06**	**51936.97**
Story-4	**116206**	**69723.58**	**60427.76**	**52293.26**
Story-3	**122717**	**73630.09**	**63813.44**	**55223.17**
Story-2	**161852**	**97111.35**	**84164.09**	**72834.31**
Story-1	**1193646**	**716188**	**620703**	**537147**

Table 2.12- Max.Storey stiffness atY direction



Graph 2.29 storey stiffness in Y direction

CHAPTER-5

CONCLUSION

5. CONCLUSIONS

From the above study and results several conclusions can be drawn such as:

1) Concrete elements subjected to temperature up to 300°C or 500°C are still safe for use because at this temperature concrete moistures would have been absolved by fire and cracks will occur due to expansion and contraction of constituent materials but the entire structure will be serviceable. However, buildings subjected to temperatures above 600°C are structurally unsafe. At temperature above 600°C concrete element will have lost about 70 % of its strength
2) The model above 600°C is failed in design check Hence, buildings subjected to temperatures above 600°C are structurally unsafe
3) Max story Drift increases with increase in temperature
4) It is seen that max story drift increases by 22.606% between model1 to model2 ,6.88% in between model 2 and model 3 and 6.94% increases in-between model 3 and model 4
5) Max story Displacement increases with increase in temperature
6) It is seen that max story displacement increases by 23.460% between model1 to model2 ,7.2438% in between model 2 and model 3 and 6.93% increases in-between model 3 and model 4
7) The fundamental time period will increase with the increase in temperature
8) Story stiffness decreases with increase in temperature

9) It is seen that Story stiffness decreases by 40% between model1 to model2 ,13.33% in between model 2 and model 3 and 13.46 % decreases in-between model 3 and model 4

10) The non-destructive test is a way of testing which does not affect the overall performance of a member's entity under investigation. It could be performed during construction and after maintenance. The IRH and UPV can be used as a reliable method to predict the mechanical strength of the reinforced concrete structure

11) It is found that the slabs are mostly effected by fire and has maximum damaged it is because they are located at the highest evaluation of the room where they are exposed to the highest temperature during a fire

12) The strength of steel will start to decrease at approximately 430° C (800° F). At 590° C (1100° F) steel loses approximately 50% of its strength and stiffness when compared to normal ambient conditions. At 700° C (1300° F) the strength and stiffness are reduced to approximately 20% of the ambient condition strength and stiffness. These property reductions will likely be temporary, and the steel will regain its strength and stiffness if the temperature of the steel does not exceed 700° C (1,300° F) for longer than 20 minutes

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APPENDIX

URKUND

Document Information

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STRUCTURAL HEALTH MONITORING OF FIRE DAMAGED STRUCTURE

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ABSTRACT

The purpose of this review paper to study the work that has been done before to observe or measure the effect of high temperature caused by the fire on the reinforced concrete structure. However, reinforced concrete structures are considered to be fire-protected by the reinforcement cover. After exposure to fire, reinforced concrete structures lose strength and durability, but long periods of heat exposure cause physical-chemical changes in concrete properties accompanied by degradation of mechanical strength. To ascertain the integrity of the building, a visual inspection was conducted for all elements (truss, beam, column and slab), followed by a non-destructive test such as rebound hammer test and Ultrasonic pulse velocity test. Such structural checks would provide a comparative comparison of the fire-damaged members with the undamaged one. There is a 70% loss in strength of concrete above 600 °C temperature. Concrete is not structurally useful after heating in temperature above 600°C

Keywords: *Concrete, fire damaged, Ultrasonic pulse velocity, rehabilitation, Rebound hammer test.*

1. INTRODUCTION

Through the years, many cases of open fire have happened leading to unusual situations such as explosive use, environmental crises, and accidents. Nevertheless, reinforced concrete buildings are classified as fire safety by its cover present over the reinforcement. Long cycles of exposure to elevated temperatures induce physico-chemical modifications in concrete materials, which lead to a decline in mechanical resistance. The fire tolerance of a concrete building is typically well over its minimum specifications, and thus rehabilitation is favoured over destruction or rebuilding. Observation of the destruction can assess the extent and length of the fire by means of a variety of test methods and instruments available to evaluate the effect of the fire on both the structures and structural components. Such evaluations, together with the technical analysis, enable for the development and deployment of effective and economical repair knowledge as needed [8]. Non-destructive testing (NDT) techniques play a key role in the assessment of reinforced concrete structural safety systems (SHAs). Deformation of concrete structures, cracks, honeycombs and voids attributable to operation, wear and tear, environmental conditions. These defects can further affect the quality of concrete structures due to corrosion / damage of the steel reinforcement and of the concrete itself. Different NDT methods have been developed to evaluate these defects and, ultimately, to enhance structural protection during structural service lifeExperiments are often performed in which the two measurement methods are used to calculate the compressive power of the concrete structure using the ultrasonic pulse velocity (UPV) and the impact rebound hammer (IRH). High temperatures are responsible for the degradation of the concrete microstructure and the weakening of its vital capacity, and thus this step is an appropriate alternative to UPV and other NDT approaches. The NDT methodology alone is not adequate to predict reinforced concrete structural safety and integrity. Concrete safety assessment shall be carried out with the assistance of the aggregated non-destructive technique. For this analysis, the concrete strength was measured using a combination of ultrasonic pulse velocity and hammer rebound techniques. Yet even less was used to assess the health of fire-damaged buildings [10]

2.0 Nondestructive Testing

Non-destructive (NDT) methods are used for the analysis of large-scale concrete structures. NDT is a test method that does not compromise the supposed overall effectiveness of the member being investigated. This can be achieved before and after construction , repair or commissioning. These assessments shall be carried out during construction to ensure quality control and strength testing in fresh concrete works as well as in existing structures to determine their structural ability and material degradation against time or the environment. Standard control cubes can not determine the strength of the concrete developed on-site, they can not have an accurate measure of the concrete in use during construction. The selection of the investigator is the extraction and examination of the cores taken from the concrete structure .The extraction of cores, though, is costly and can weaken the structure. Researchers have therefore established various NDT methods that allow for the in situ measurement of certain concrete properties from which an estimate of concrete strength can be made. Many of the NDT methods used include visual examination of concrete buildings, the Schmidt or rebound hammer testing and the UPV check. There are other properties that can be calculated using NDT and partly destructive measurements such as stiffness, elasticity and compressive strength units, surface hardness and surface absorption, and location of the steel bars, size and distance from the surface. Two tests are typically used in NDT for the safety assessment of fire-damaged system of ultrasonic pulse velocity and rebound hammer techniques [8].

2.1 Rebound hammer testing

The Rebound Hammer is an device that offers the relative compressive strength of a concrete or other construction material depending on its stiffness on the exposed surface, consisting of an exterior frame, a plunger, a hammer mass, a spring, a latching mechanism and a slipping rider and a scale in which the rebound amount is indicated .The measurement required a smooth surface and the contact point of impact must be 20 mm away from the discontinuous edge or other sharp edge. When the plunger is forced towards the concrete surface, the mass falls out from the plunger and retracts to the power of the spring [20]. The hammer impacts into the concrete and the spring touch mass recovers, bringing the driver down the directed scale. The driver shows the distance travelled by mass named the rebound number The rebound amount is calculated on an approximate linear scale of 10 to 100 . The NDT rebound hammer test estimates the compressive strength of the concrete with the aid of the rebound number, which relies on the rebound of the spring-controlled mass and the surface toughness of the concrete It is worth noting that the rebound amount relies solely on the surface state of the concrete and is not linked to internal mechanical properties for which IRH alone is not adequate to quantify the structural compressive power [10]. Figure 1 shows a cutaway schematic view of the Schmidt or the rebound hammer .

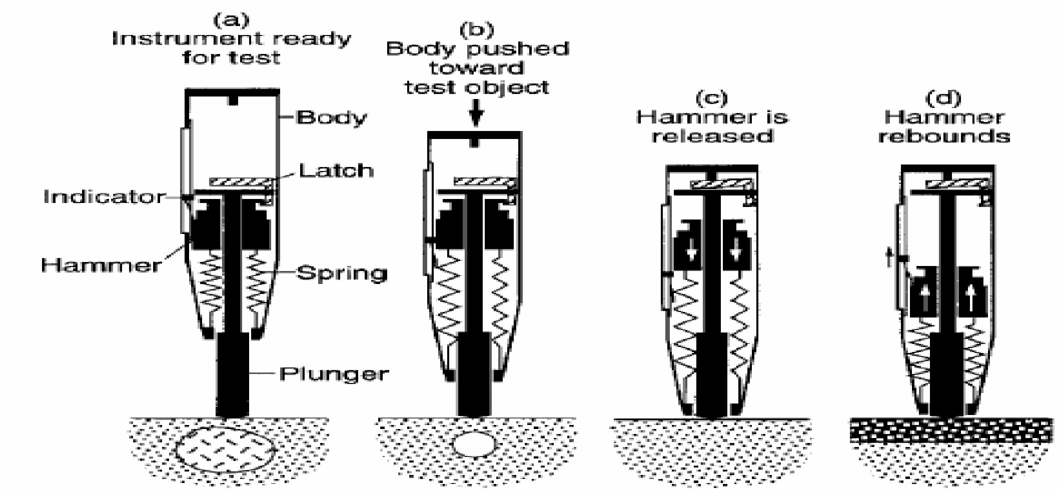

Figure 1 Cutaway view of the rebound hammer (2) (Source- Dr Balaji KVGD, V. Sannidha)

2.2 Ultrasonic pulse velocity method testing

For more than 60 years, this measuring technique has been used to evaluate the quality of the concrete. Checking equipment is simple to use for construction installations and laboratory experiments. The strength of ultrasonic pulses across the concrete relies on the elastic properties and the composition of the material. Areas of poor elasticity or low density, such as concrete damaged by burning, may also be detected through this method. This consists of a pulse generator, a transmission arm, a receiver arm and a measurement device. The timing of entry of the compression waves, for example. The waves propagating in concrete as quickly as possible are measured at the receiving head with the help of the tool. To prevent measuring mistakes, proper communication is obtained by keeping the heads under steady pressure using a thin layer of communication gel between the heads and the concrete sheet. The heads can be positioned in three separate forms, a direct form seen in Figure 2, where the angle between the heads is 180 degrees; the angle of the semi-direct method is 90 degrees, and the angle of the indirect surface system is 0 degrees. Direct path is favored because the optimum pulse energy is received at the receiving head, but due to the structure of the device, this solution becomes complicated in many situations. The semi-direct technique can be used to avoid the accumulation of reinforcement. The indirect approach does not provide as good measurements as direct and semi-direct approaches, but it can be used to determine the thickness of the low-quality layer and is utilized in cases when certain approaches are not usable. The heads are then placed close to each other while the low-quality layer is calculated using an indirect method and then moved farther forward. By measuring the time of arrival as a function of the gap between the head, the thickness of the poor layer may be determined in situations where this layer is distinct. If the heads are located next to each other, the waves scatter across the upper layer of the material where there is a wide amount of fine substance. This would generate a low velocity for the waves to pass across both the upper and lower layers as the distance between the heads increases. Shrinkage and delamination fractures should be taken into consideration when assessing fire-damaged concrete in the evaluation of findings[20]

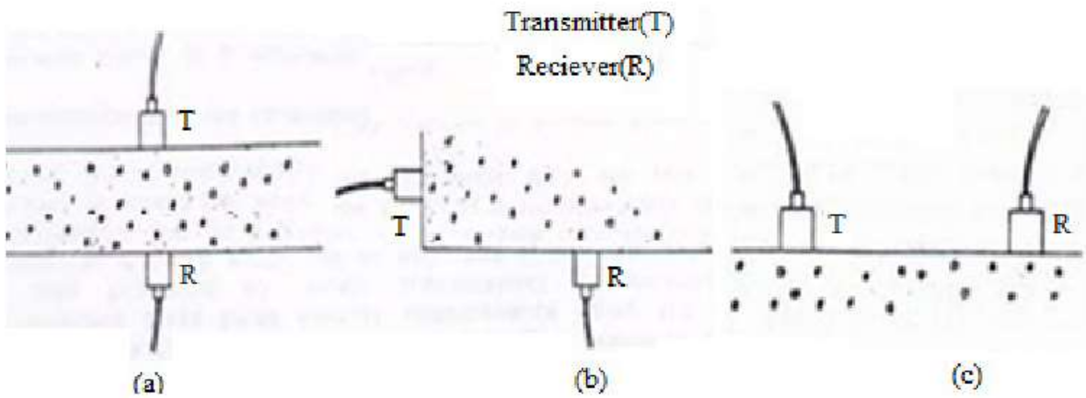

Figure 2 direct (a), Semi-direct (b) Indirect (surface) transmission (c).(source 3)

2. OBJECTIVE

The objective of this paper is to review the work that has been done before to observe or measure varying damage caused by a fire on the reinforced concrete structure by non-destructive testing method

3. LITERATURE REVIEWS

Ahmed Aseem, Waqas Latif Baloch, Rao Arsalan Khushnood*, Arslan Mushtaq (28 May 2019)- The analysis in this paper shows that IRH and UPV can be used as a reliable tool for predicting the mechanical strength of fire-damaged reinforced concrete construction. Nonetheless, the concrete core extraction is used in this analysis to create a concrete compressive strength relationship based on the Rebound hammer test and the ultrasonic pulse speed test. The microstructural work is also being performed which also informs us of the deteriorated condition of concrete based on scanning electron microscopy and thermal analysis of fire-damaged specimens. Extremely degraded specimens were well associated with lower concrete compressive strength values per microstructural analysis

James W. Jordan, Marc A. Sokol, John H. Stewart (2013) –In this paper, it is seen that the compressive strength of concrete can be greatly reduced from excessive heat, exposure to temperatures below the range of 250° to 300° C (482° to 572° F) are usually not considered to be significant. Shallow cracks may appear in concrete surfaces that reach approximately 300° C (572° F), and deepen with increased temperatures. Sometimes, concrete exposed to this temperature range may exhibit a pinkish hue. At approximately 370° to 430° C (700° to 800° F), concrete loses approximately half of its compressive strength. Cement constituents of the concrete decompose to a whitish powder consistency at approximately 900° C (1650° F). Steel reinforcement within concrete can typically sustain temperatures up to 450° C (840° F) for cold-worked steel, and 600° C (1100° F) for hot-rolled reinforcement steel, and still recover all of its original yield strength upon cooling. Higher temperatures may cause permanent loss of strength and ductility, possibly resulting in excessive deflection or failure of a reinforced concrete structural member Prestressing steel within concrete has a greater sensitivity to heat exposure than mild steel reinforcement, losing approximately half of its strength at 400° C (750° F). Concrete that has changed colour to a pink or red shade is estimated to have reached a temperature range of 290° to 590° C (550° to 1100° F), which would be below the threshold of permanent damage to mild steel reinforcement. Concrete that has changed colour to a whitish-grey is estimated to have reached a temperature range of 590° to 950° C (1100° to 1740° F)Concrete that has changed to a buff colour is estimated to have exceeded a temperature of 950° C (1740° F). The strength of steel will start to decrease at approximately 430° C (800° F). At 590° C (1100° F) steel loses approximately 50% of its strength and stiffness when compared to normal ambient conditions. At 700° C (1300° F) the strength and stiffness are reduced to approximately 20% of the ambient condition strength and stiffness. These property reductions will likely be temporary, and the steel will regain its strength and stiffness if the temperature of the steel does not exceed 700° C (1,300° F) for longer than 20 minutes

M H Osman, N N Sarbini, I S Ibrahim, C K Ma, M Ismail and M F Mohd(2017)
In this paper, the severe damage is not seen for reinforced concrete elements, except
The spalling of mortar screeding was found in certain parts of column and beams. It is seen that in concrete surface there are no cracks are present. By various test such as IRH, it is seen that the concrete is still unaffected by the fire. In this study, the truss is tested and it was found that the truss is still strong. The maximum deflection is very small. The small deflection tells us is no structural degradation of truss member and connection which proves they are over-designed. After a various lab test, it is seen that the loss of 15% in tensile capacity in the least affected sample and loss of 19% in tensile capacity in most affected places And from other finding it is predicated that steel truss with characteristics strength of 400 N/mm2 to 460 N/mm2 will have strength loss of 30% from the actual strength when it is exposed to fire at temperature 800° C to 1000° C

Taehun Ha , Jeongwon Ko , Sangho Lee , Seonwoong Kim , Jieun Jung and Dae-Jin Kim (2 May 2016)- In this paper, both on-site and laboratory test has been performed. Through visual inspection, it is found that the slabs are mostly effected by fire and has maximum damaged it is because they are located at the highest evaluation of the room where they are exposed to highest temperature during a fire. The strength test on concrete cores from slabs, girders and beams shows the concrete and reinforcement In the girders and beams are less damaged and it is fit for further use but the reinforcement bars in slabs had very large damaged and its structural strength is reduced therefore it could not be used. By finite element analysis, the strength of the concrete structure in both the girders and column having 50mm cover reduced to 60% compared to original strength. In this study, it is advised that the slabs have to be replaced with a fresh one and the girder and beams is to be retrofitted .and the column and wall only needs surface treatment without structure retrofitting

Awoyera, P.O., Akinwumi, I.I., Ede, A.N, Olofinnade, M.O.(August 2014)-
In this study, it is seen that the average ultimate tensile strength of steel is decreased from 592.0 N/mm2 at 30 ° C (86 ° F) to 300.97 N/mm2 at 700 ° C (1292 ° F) for concrete beam

having the cover of 20mm. Hence the loss in strength will be 49.2% of its original strength. In this paper, it is seen that the concrete element subjected to the fire at temperature up to 450 ° C are still serviceable. At a temperature of 450 °C, the concrete moisture has been absorbed by the fire and it will cause the cracks in the concrete which is due to expansion and contraction of the constituent material. But the whole structure is still serviceable however building subjected to a temperature above 600 °C is structurally unsafe to use. There is a 70% loss in strength of concrete above 600 °C temperature. In this paper, it is also seen that the value of UPV and RHN will decrease with increase in temperature

P.Srinivasan, A.Cinitha, Vimal Mohan and Nagesh R.Iyer (March 2014)- In this study, it is seen that when the concrete is exposed to the fire at a temperature between 300 °C to 600°C the colour of concrete changes from greenish-grey to pink. After performing UPV test the ultimate stress of the reinforcement changes from 561.5 N/mm2 to 400 N/mm2 at 500°C and yield stress from 461 N/mm2 to 265.0 N/mm2 at 500°C. It is seen that there was 28.8% decrease in ultimate stress of the reinforcements at 500°C and the compressive strength will be 19.15 N/mm2 at 300°Cand 18.50 N/mm2 at 500°C

Ramadan E. Suleiman, Fathi M. Layas Omar F.Labbar and Vail karakale (Janury 2013) In this study it is seen that the maximum effect of fire is on compressive strength of concrete of the fire affected structure. It is seen that at distance 200mm from the heated surface the damage is much smaller hence the damage in the reinforcement is negligible after analysis it is seen that the slab element, ribs, beams and column appear to be still with sufficient reinforcement, however under the capacity of defeated concrete due to fire has to follow the treatment. The treatment is based on chipping away of all concrete which is damaged by fire according to the required strength. For the treatment of slabs, the use of shotcrete in layers of 30 to 44mm thickness is used. For ease construction, the non-shrinkage concrete of maximum size aggregate 10mm with 20 Mpa compressive strength is used in treatment. In this paper, it is seen that compressive strength is reduced 42% of its actual strength at temperature 600 **°C.**

Narendra K. Gosain (Sep2008)-In this paper it is seen that at temperature 290°C there will be no change in colour, physical appearance and strength of concrete. At temperature 290°C to 590°C, the colour changes from pink to red. The surface crazing will occur at temperature 300°C, and deep cracking will occur at temperature 550°C At temperature 290°C to 590°C the concrete condition will be sound but strength significantly started reducing. At temperature 590°C to 950°C the colour of concrete changes to whitish-grey. Spalling will be caused but exposing not more than 25% of reinforcement at temperature 800°C and at temperature 590°C to 950°C the condition of concrete will weak and friable. At temperature above 950°C, the colour of concrete will change to buff and there will be extensive spalling and concrete is very weak and friable

Jeremy Ingham (May 2015) –Through this article, it is shown that the residual compressive strength of structural-quality concrete is not greatly reduced for temperatures up to 300 ° C, whereas the remaining strength can be reduced to only a small fraction of its original value for temperatures above 500 ° C. There will be a decrease in steel strength while the material is at elevated temperatures. It is also possible to recover the yield strength after cooling for temperatures up to 450 ° C for cold-rolled steel and 600 ° C for hot-rolled steel. Higher temperatures may cause permanent loss of strength and ductility after cooling. The impact of excessive temperature is more critical on prestressing steel than on reinforcing steel. At temperatures of 200–400ºC, steel prestressing tendons show considerable loss of strength (>50% loss at about 400°C)

J.S. Kalyana Rama and B.S. Grewal (2015) –In this paper, its is estimated that at a temperature under 250°C the strength of concrete is will not change but temperature above 250°C the strength of concrete reduced by 9.4%, it will reduce further by 25.4% at temperature 500°C and temperature 1100°C the strength will reduce to 84.2% from its original strength before the fire. Concrete, although doesn't melt at a high temperature slightly change in shape will occur and this will cause a significant reduction in strength and cracks will also occur at a higher temperature. It is seen that when the fire is severe then the damage will be a very high extent compression to the fire at a small scale. It is also observed that the accuracy of Schmidt's rebound hammer test has been improved at a high temperature of the concrete. It is seen that the accuracy will be 69% when the test was conducted for cube heated at 500°C. At temperature 1100°C the rebound hammer test will be failed to give a reading for concrete specimen

4.CONCLUSION

1. The non-destructive test is a way of testing which does not affect the overall performance of a member's entity under investigation. It could be performed during construction and after maintenance. The IRH and UPV can be used as a reliable method to predict the mechanical strength of the reinforced concrete structure.
2. It is observed that at the high temperature of the concrete the accuracy of Schmidt,s rebound hammer test is improved.
3. At temperature 1100°C the rebound hammer test will be failed to give the reading for concrete specimen
4. It is found that the slabs are mostly effected by fire and has maximum damaged it is because they are located at the highest evaluation of the room where they are exposed to the highest temperature during a fire
5. The strength of steel will start to decrease at approximately 430° C (800° F). At 590° C (1100° F) steel loses approximately 50% of its strength and stiffness when compared to normal ambient conditions. At 700° C (1300° F) the strength and stiffness are reduced to approximately 20% of the ambient condition strength and stiffness. These property reductions will likely be temporary, and the steel will regain its strength and stiffness if the temperature of the steel does not exceed 700° C (1,300° F) for longer than 20 minutes

Heating temperature	**Colour change**	**Mineralogical change**	**Change in physical appearance**	**Percentage decrease in compressive strength**	**Concrete condition**
105°C	None	Loss is physically bound water in aggregate and cement	Unaffected	None	Unaffecte d
120 °C to 163°C	None	Decomposition of gypsum	Unaffected	0-9.4%	
250°C to 350°C	Pink	`Oxidation of iron compounds causing pink /red discolouration of aggregate	Surface crazing(300° C)	200°C -4% 300°C-10% 400°C -20%	250°C to 590°C- Sound but strength significant ly reduced
450°C to 550°C	Pink to red	Dehydroxylati on of portlandite	Deep cracking (550°C)	500°C-32%	Concrete is not structurall y useful after heating in temperatur e above 500°C to 600°C
573°C	red	5% rise in the quartz volume causing radial cracking in the aggregate around the quartz grains	Popouts over chert or quartz aggregate (575°C)	600°C-52%	
600°C to 800°C	Whitish grey	Carbon dioxide release from carbonates can cause a significant contraction of concrete causing serious microcracking of the cement matrix	Powdered, light-coloured, dehydrated paste (575°C -600°C)	700°C-65% 800°C-82%	590°C to 950°C – Weak and friable
800°C to 1200°C	Whitish grey to buff	Dissociation and intense thermal stress cause the material to	Spalling, exposing not more than 25% of reinforcing	900°C-91% 1000°C-98.5%	Weak and friable

		disintegrate completely and result in significant microcracking	bar surface 800°C and above cause extensive spalling		
1200°C	buff	Concrete starts to melt	Extensive spalling	Melted	Very weak
1300°C to 1400°Cs	buff	Concrete melted		melted	

Source (Modify by concrete society TR 68 2008), N.K. Gosain, R.F. Drexler and D. Choudhuri, "Evaluation and Repair of Fire Damaged Buildings", Structure Magazine, Sep. 2008

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Seismic Analysis of Multi-Story Building Exposed to Fire

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Abstract - *It is seen that after the exposure of fire the building is usually reconstructed or demolished so by various techniques such as NDT testing method we can do health monitoring of reinforced concrete structure which is damaged by fire. After doing health monitoring we will able to predict the reduced compressive and tensile strength of RCC structure. After health monitoring, we will able to know that the building is safe for re-use after doing repair and retrofitting or else we have to demolish it. If the building is safe for reuse after doing some retrofitting works then the problem is that the building is safe for seismic forces or not. In this paper, the work is to analyze fire-damaged building at the various temperature on its reduced strength of RCC on ETABS. In this study we will first prepare a model of a building by normal building design material after that we will design three new building models by using reduced strength materials which are predicated before in various research papers at a various temperature such as 300°C,500°C,600°C. After that, we will do the seismic analysis of these four building models and do a comparative study of story displacement, story drift, story stiffness*

Key Words: Concrete, fire damaged, story displacement, story drift, story stiffness, Etab.

1. INTRODUCTION

From the first day of its existence, earthquakes have become a danger to human growth, destroying human lives, property and man-made structures. The effect of dynamic actions on the buildings on account of earthquake forces (lateral forces) are very much important from the structural engineers view point. It is widely accepted that the structural design of buildings will follow at least two specific criteria. First, the system must perform elastically to defend fairly weak non-structural elements against small earthquakes. Therefore, the structure will have good capacity and elastic flexibility to reduce structural displacements., such as interstorey drift, story displacement and fundamental time period . Second, the structure does not fail in the case of a significant earthquake. In this scenario, substantial structural and non-structural harm is acceptable. In order to keep the system from collapsing and thereby reduce the loss of life, it must have a strong energy dissipation capability during large inelastic deformations. In the earthquake design, the building is subjected to a random ground motion or vibration at its base, which causes inertia forces in the building that in turn induce stresses; this is referred to as the displacement type loading also expressed as load-deformation curve of the building or a structure The four essential characteristics in buildings or systems that architects and construction engineers can look at in order to construct an earthquake-resistant building plan are the structural design, lateral stiffness, lateral strength and ductility. Such factors can be addressed by building design IS codes.

1.1 OBJECTIVE

The purpose of this work is to concentrate on the various methods used to test the seismic activity of fire exposed buildings at the various temperature on its reduced strength of RCC on ETABS. In this study we will first prepare a model of a building by using normal building design material after that we will design three-building models according to new material properties with a reduce strength which is predicted before in my review paper which is based on the study of various literature on a various temperature such as 300 °C, 500°C,600 °C and then we will do seismic analysis these four building models and do a comparative study of story displacement, story drift and story stiffness and fundamental time period behaviour of reinforced concrete buildings with seismic zone IV of India using an equivalent static method. The final research was carried out in ETABs addressing all areas of structural engineering. The main objectives of this research are, in particular,:

1) Conduct a comparative analysis of the different seismic parameters;.
2) Comparison based on story displacement, storey drift, Storey Stiffness & fundamental time period on four models.
3) The analysis would have an estimated understanding of how the exposed fire structure would work in the seismic force.

For this study, a multi-storey residential building for earthquakes and wind loads is analyzed using an equivalent static approach for ETABs. The research is carried out by observing the seismic region IV, and for this region, the activity is measured by taking the medium soils. A different response for story displacements, story drift, story stiffness and fundamental time period is plotted for zone IV for a medium type of soil.

1.2 STRUCTURALMODELING

For analysis, the 7-story high-rise building is modelled in ETABs software. The structure is not a true existing structure. RC framed (G+6) multi-storey building having 4 grid line in X and Y direction and spacing between the grid lines in the X direction is 4.5m and in the Y direction is 6.5m. The building is 22.5 m high and has a typical story height of 3.5m and bottom storey height is 1.5m. The building is analyzed by Equivalent static, which is a linear static analysis. A dead load of a wall is taken as wall load and parapet wall load which depend upon the wall thickness and the height of the wall. The thickness of the wall is taken as 230 mm for the outer wall and 115mm for inner walls. The unit weight of brick is 20KN/m^3 and height of partition wall will be 3.1m. The live load and the Floor finish dead load are taken as 2 KN/m^2 and 1.5 KN/m^2 according to IS 875:1987 (part 2).

All the specifications of the frame are given in Table 1.For the first building model

Table -1: (First building model specification)

1.	Building type	Residential building
2.	No. of floors	G+6
3.	Bottom storey height	1.5m
4.	Total height	22.5m
5.	Story height	3.5m
6.	Measurement of column	400mm*600mm
7.	Measurement of beam	450mm*300mm
8.	Thickness of slab	130mm
9.	Masonry wall thickness	230mm Outer wall and 115mmfor inner wall
10.	Seismic zone	IV
11.	Importance factor	1
12.	Response reduction factor	5
13.	Soil type	II
14.	Concrete cube compressive strength	30MPa
15.	Grade of steel	Fe500
16.	Damping	5%
17.	Unit weight of PCC	24 kN/m^3
18.	Unit weight of brick	20 kN/m^3
19.	Modulus of Elasticity	24855.58 MPa
20.	Shear Modulus	10356.49 MPa
21.	Live load	2KN/m^2
22.	Floor finish dead load	1.5KN/m^2
23.	IS Code for concrete	IS 456:2000
24	IS Code for earthquake	IS 1893:2002 (part I)
25.	IS Code for wind	IS 875 :1987
26	Self-weight factor	1
27.	Outer Wall load	14.26 KN/m
28.	Inner wall load	7.13KN/m

Table -2: Second building model specification

1.	Building type	Residential building
2.	No. of floors	G+6
3.	Bottom storey height	1.5m
4.	Total height	22.5m
5.	Story height	3.5m
6.	Measurement of column	400mm*600mm
7.	Measurement of beam	450mm*300mm
8.	Thickness of slab	130mm
9.	Masonry wall thickness	230mm Outer wall and 115mmfor inner wall
10.	Seismic zone	IV
11.	Importance factor	1
12.	Response reduction factor	5
13.	Soil type	II
14.	Concrete cube compressive strength	25.5 MPa
15.	Grade of steel	Fe500
16.	Damping	5%
17.	Unit weight of PCC	24 kN/m^3
18.	Unit weight of brick	20 kN/m^3
19.	Modulus of Elasticity	14913.55 MPa
20.	Shear Modulus	6213.9 MPa
21.	Live load	2KN/m^2
22.	Floor finish dead load	1.5KN/m^2
23.	IS Code for concrete	IS 456:2000
24	IS Code for earthquake	IS 1893:2002 (part I)
25.	IS Code for wind	IS 875 :1987
26	Self-weight factor	1
27.	Outer Wall load	14.26 KN/m
28.	Inner wall load	7.13KN/m

Table -3 Third building model specification

1.	Building type	Residential building
2.	No. of floors	G+6
3.	Bottom storey height	1.5m
4.	Total height	22.5m

5.	Story height	3.5m
6.	Measurement of column	400mm*600mm
7.	Measurement of beam	450mm*300mm
8.	Thickness of slab	130mm
9.	Masonry wall thickness	230mm Outer wall and 115mmfor inner wall
10.	Seismic zone	IV
11.	Importance factor	1
12.	Response reduction factor	5
13.	Soil type	II
14.	Concrete cube compressive strength	18 MPa
15.	Grade of steel	Fe500
16.	Damping	5%
17.	Unit weight of PCC	24 kN/m³
18.	Unit weight of brick	20 kN/m³
19.	Modulus of Elasticity	12935.04 MPa
20.	Shear Modulus	5385.43 MPa
21.	Live load	2KN/m²
22.	Floor finish dead load	1.5KN/m²
23.	IS Code for concrete	IS 456:2000
24	IS Code for earthquake	IS 1893:2002 (part I)
25.	IS Code for wind	IS 875 :1987
26	Self-weight factor	1
27.	Outer Wall load	14.26 KN/m
28.	Inner wall load	7.13KN/m

Table -4 Forth building model specification

1.	Building type	Residential building
2.	No. of floors	G+6
3.	Bottom storey height	1.5m
4.	Total height	22.5m
5.	Story height	3.5m
6.	Measurement of column	400mm*600mm
7.	Measurement of beam	450mm*300mm
8.	Thickness of slab	130mm
9.	Masonry wall thickness	230mm Outer wall and 115mmfor inner wall
10.	Seismic zone	IV
11.	Importance factor	1
12.	Response reduction factor	5
13.	Soil type	II
14.	Concrete cube compressive strength	10.5 MPa
15.	Grade of steel	Fe500
16.	Damping	5%
17.	Unit weight of PCC	24 kN/m³
18.	Unit weight of brick	20 kN/m³
19.	Modulus of Elasticity	11185.13 Mpa
20.	Shear Modulus	4660.43 MPa
21.	Live load	2KN/m²
22.	Floor finish dead load	1.5KN/m²
23.	IS Code for concrete	IS 456:2000
24	IS Code for earthquake	IS 1893:2002 (part I)
25.	IS Code for wind	IS 875 :1987
26	Self-weight factor	1
27.	Outer Wall load	14.26 KN/m
28.	Inner wall load	7.13KN/m

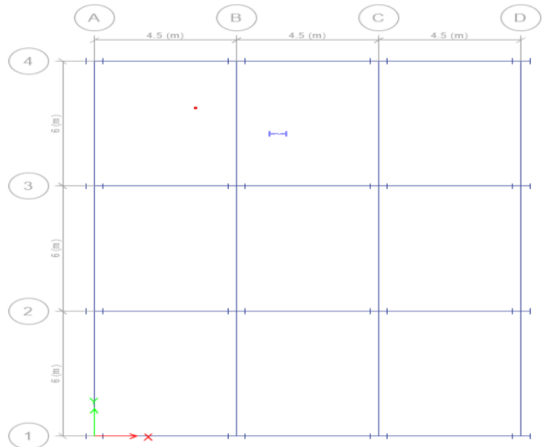

Fig -1: Plan of Building, Building dimensions

2. DEFORMED SHAPE

MODEL 1 DEFORMED SHAPE OF STRUCTURE DUE TO EARTHQUAKE LOADING

• In this model, we have applied earthquake load EQ at X+ direction. This is a simple RC framed structure without exposure to fire

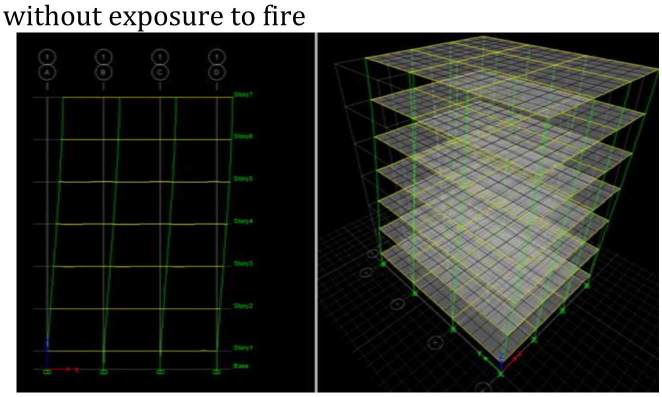

Fig -2 -3D and elevation view of model 1 at X direction

- In this model, we have applied earthquake load EQ at Y+ direction. This is a simple RC framed structure without exposure to fire

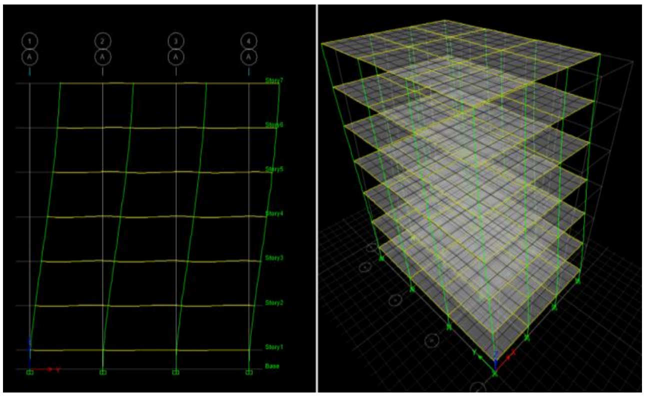

Fig -3- 3D and elevation view of model 1 at Y direction

MODEL 2- DEFORMED SHAPE OF STRUCTURE DUE TO EARTHQUAKE LOADING

• In this model, we have applied earthquake load EQ at X+ direction. This is a simple RC framed structure which is exposed to fire at temperature 300 °C

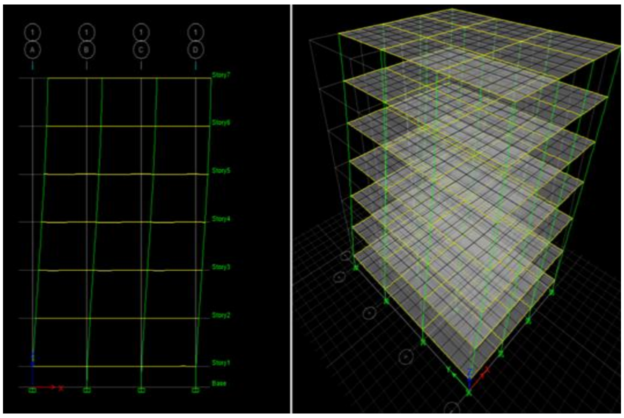

Fig -4 -3D and elevation view of model 2 at X direction

• In this model, we have applied earthquake load EQ at Y+ direction. This is a simple RC framed structure which is exposed to fire at temperature 300 °C

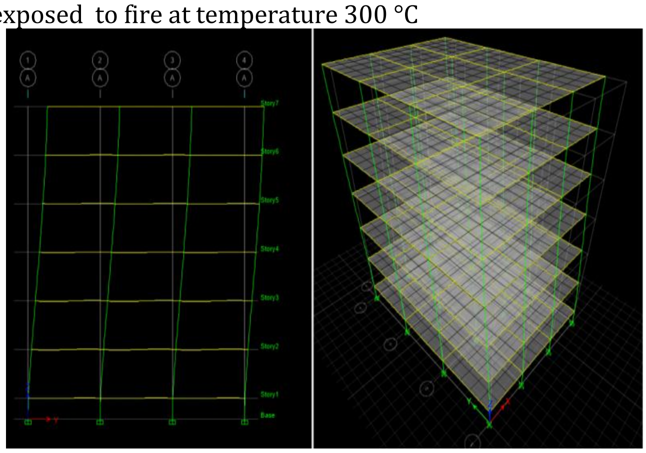

Fig -5 3D and elevation view of model 1 at Y direction

MODEL 3- DEFORMED SHAPE OF STRUCTURE DUE TO EARTHQUAKE LOADING

• In this model, we have applied earthquake load EQ at X+ direction. This is a simple RC framed structure which is exposed to fire at temperature 500 °C

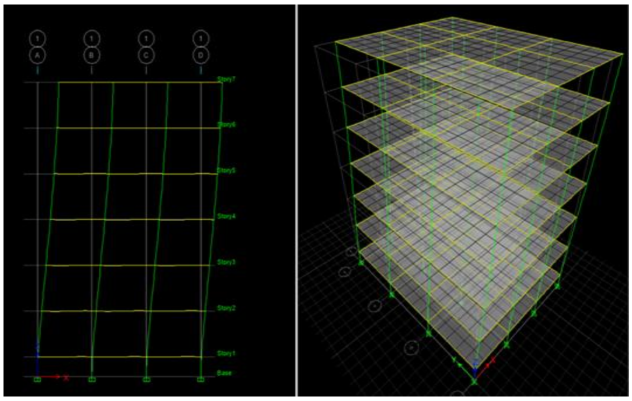

Fig -6 -3D and elevation view of model 3 at X direction

• In this model, we have applied earthquake load EQ at Y+ direction. This is a simple RC framed structure which is exposed to fire at temperature 500 °C

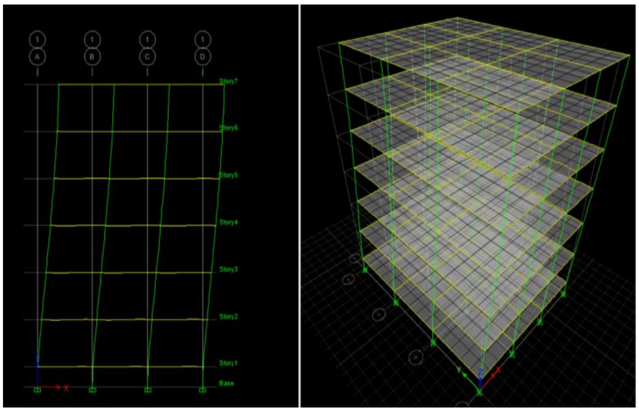

Fig -7 -3D and elevation view of model 3 at Y direction

MODEL 4-DEFORMED SHAPE OF STRUCTURE DUE TO EARTHQUAKE LOADING

•In this model, we have applied earthquake load EQ at X+ direction. this is a simple RC framed structure which is exposed to fire at temperature 600 °C.

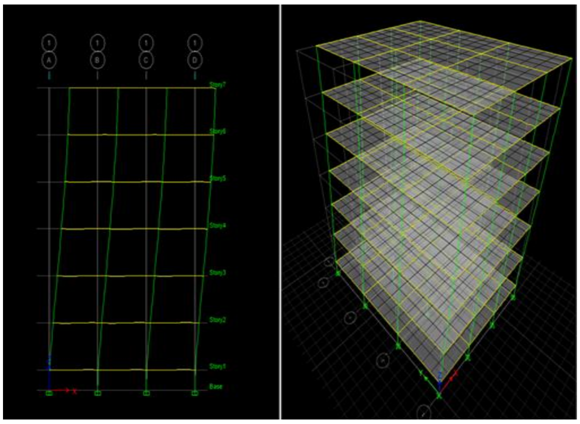

Fig -8 -3D and elevation view of model 4 at X direction

•In this model, we have applied earthquake load EQ at Y+ direction. This is a simple RC framed structure which is exposed to fire at temperature 600 °C.

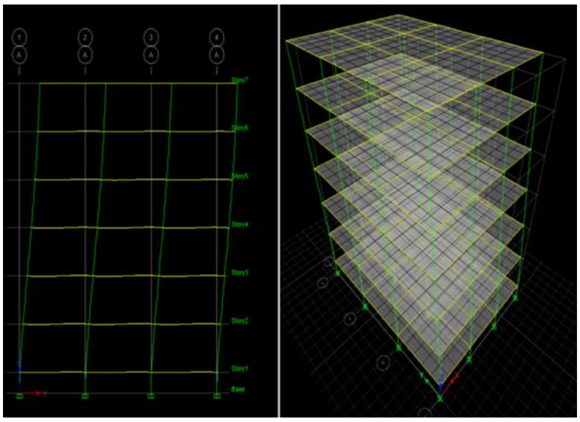

Fig -9 -3D and elevation view of model 4 at Y direction

3. ETABs Overview

ETABS is used as an Engineering software tool for multi-storey construction and design study in buildings. Can be evaluated preliminary to advanced systems under either dynamic or static conditions utilizing ETABS. ETABs are used for seismic analysis and for the evaluation of multi-storey building behaviours which are related to various analytical parameters such as story drift, story displacemen, story stiffness etc. Comprehensive research like structural simulation is done in this program.The analysis was carried out using ETABs software, which involves the following steps:-

1. Defining the dimension of the design
2. Defining the elements and properties of the material
3. Assigning load and load combinations
4. Run and check the model to locate the error
5. Run analysis
6. Extract the findings and analyze them

4. METHOD OF ANALYSIS

Equivalent static method

This method describes a set of forces operating on a building that reflect the impact of earthquake ground motion, usually represented by a seismic design response spectrum. This means that the building reacts in its simple mode of service. To order for this to be valid, the structure must be low-rise and must not be dramatically bent as the ground vibrate. The response is read from the design response spectrum, provided the building's natural frequency (either measured or specified by the building code). The applicability of this approach is generalized in other building codes by adding criteria that qualify for higher buildings with certain higher modes and low rates of twisting.. Such codes add alteration factors that minimize structural forces (e.g. force reduction factors) to compensate for results related to the "yielding" of the framework. For the determination of seismic forces, the country shall be divided into four seismic zones: each zone shall have its zone factor value and, as per IS 1893 (Part 1):2002, these values shall be given below:

Seismic Zone Factor (1)	II (2)	III (3)	IV (4)	V (5)
Z	0.10	0.16	0.24	0.36

Fig -10- Every zone has its own zone factor value and as per IS 1893 (Part 1):2002

Table 4 -As per IS Code 1893(part 1) :2002 the following were the major steps for determining the seismic forces:

Serial No	Model Description	
1	Zone	IV
2	Zone Factor	0.24
3	Type of building	Residential
4	Importance Factor	1
5	Soil Type	II
6	Soil Condition	Medium
7	Damping Ratio	5%
8	Response Reduction Factors	5

5. RESULTS

Storey drift

It is the displacement of one level relative to the other level above or below.It is defined as ratio of displacement of two consecutive floor to height of that floor. It is very important term used for study purpose in seismic engineering.According to Indian standard code 1893:2002 (part 1), the storey drift should not exceed 0.004 times the storey height.

Table 5. Max.Storey drift comparison in x-direction- The table and graph below shows the comparison between the various building models

NO OF STOREY	MODEL-1	MODEL-2	MODEL-3	MODEL-4
Story-7	0.000308	0.000398	0.000427	0.000459
Story-6	0.000505	0.000652	0.000701	0.000753
Story-5	0.000645	0.000833	0.000894	0.000961
Story-4	0.000723	0.000933	0.001002	0.001078
Story-3	0.000743	0.00096	0.001031	0.001108
Story-2	0.000637	0.000822	0.000883	0.000949
Story-1	0.000196	0.000253	0.000272	0.000292

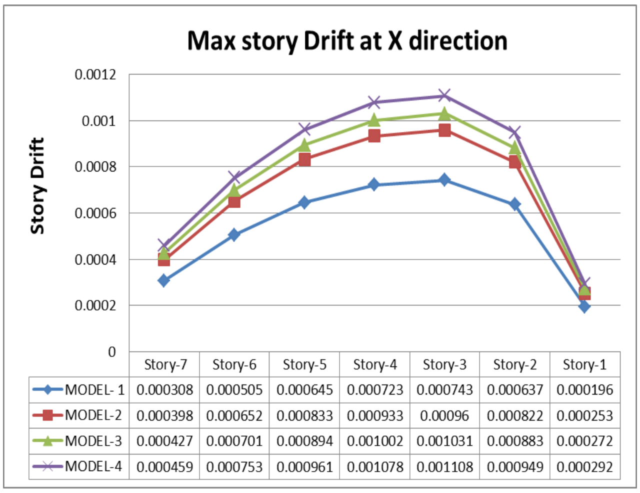

Chart -1 *Max Story Drift X direction*

Table 6 Max.Storey drift comparison in Y-direction- The table and graph below shows the comparison between the various building models

NO OF STOREY	MODEL- 1	MODEL-2	MODEL-3	MODEL-4
Story-7	0.000336	0.000434	0.000466	0.000501
Story-6	0.000512	0.000662	0.000711	0.000764
Story-5	0.00065	0.00084	0.000902	0.00097
Story-4	0.000726	0.000937	0.001006	0.001082
Story-3	0.000721	0.000931	0.001	0.001075
Story-2	0.000553	0.000714	0.000767	0.000824
Story-1	0.000175	0.000226	0.000243	0.000261

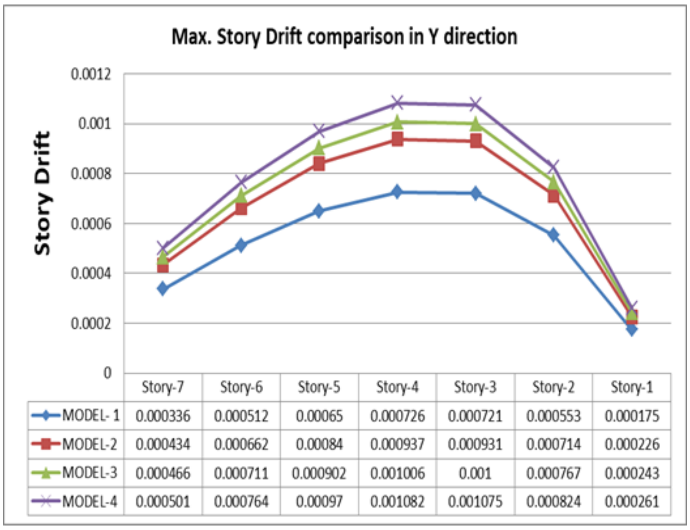

Chart -2 .Max Story Drift Y direction

Storey Displacement

It is the displacement of each floor in relation to the ground level.According to IS 1893 (part1):2002 the max value of displacement is 1/250 times of story height with respect to ground.

Table 7. Max. Storey displacement (mm) comparison in x-direction- The table and graph below shows the comparison of the various models in terms of storey displacement in X direction

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NO OF STOREY	MODEL- 1	MODEL-2	MODEL-3	MODEL-4
Story-7	12.75584	16.407722	17.689075	19.015184
Story-6	11.67818	15.076465	16.194634	17.408708
Story-5	9.909554	12.793179	13.742004	14.77221
Story-4	7.65208	9.8787793	10.611468	11.406985
Story-3	5.122169	6.612692	7.103132	7.635637
Story-2	2.521748	3.255563	3.497017	3.75918
Story-1	0.294	0.379553	0.407703	0.438267

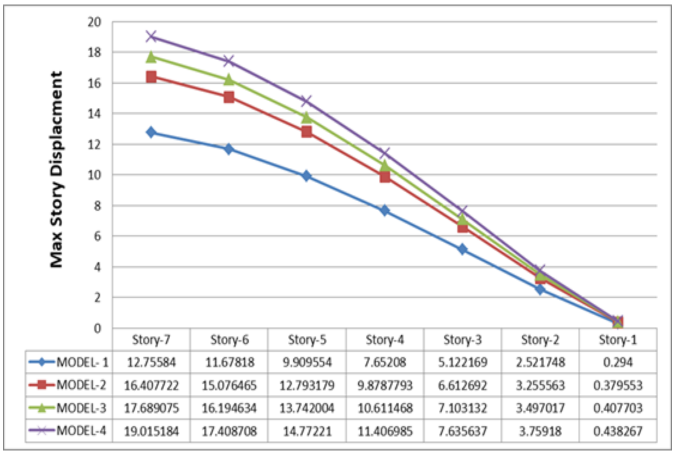

Chart -3 Max Story Displacements X direction

Table8- Max. Storey displacement (mm) comparison in Y direction- Table and graph below show the comparison of various models in terms of storey displacement in the Y direction.

NO OF STOREY	MODEL- 1	MODEL-2	MODEL-3	MODEL-4
Storey-7	12.503188	16.141546	17.338708	18.638551
Storey-6	11.328639	14.625209	15.70909	16.887645
Storey-5	9.535262	12.30497	13.222957	14.214552
Storey-4	7.258716	9.370962	10.065973	10.82596
Storey-3	4.719086	6.092313	6.544159	7.034759
Storey-2	2.197795	2.837342	3.047777	3.276262
Storey-1	0.262391	0.338746	0.363869	0.391148

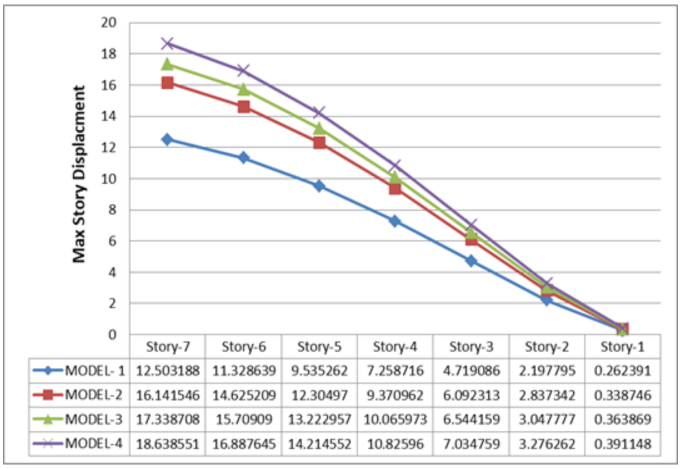

Chart -4 Max Story Displacements Y direction

Fundamental time periods

Every object has a natural vibration frequency and so has every structure. When a structure is excited by seismic forces, it starts to vibrate. The lowest natural frequency (f) of vibration of a structure corresponds to the longest time period (T) of vibration, as frequency and time period are inversely proportional (T=1/f). This is also referred to as the first mode of vibration or a fundamental period of vibration. The structure will have multiple natural modes of vibration for which frequency will be higher and time period will be shorter than the fundamental period. According to IS 1893(Part 1):2002 it is the first(longest) modal time period of vibration

Table 9-Fundamental time period (S) comparison-The table and the graph below shows the comparison of various models at a various temperature in terms of the fundamental time period

Modal	MODEL- 1	MODEL-2	MODEL-3	MODEL-4
Modal 1	0.913	1.178	1.266	1.36
Modal 2	0.874	1.128	1.212	1.303
Modal 3	0.767	0.99	1.064	1.143
Modal 4	0.294	0.379	0.407	0.438
Modal 5	0.269	0.348	0.374	0.402
Modal 6	0.244	0.315	0.338	0.363
Modal 7	0.166	0.215	0.231	0.248
Modal 8	0.143	0.184	0.198	0.212
Modal 9	0.135	0.174	0.187	0.201
Modal 10	0.112	0.145	0.156	0.167
Modal 11	0.089	0.115	0.124	0.133
Modal 12	0.088	0.114	0.123	0.132

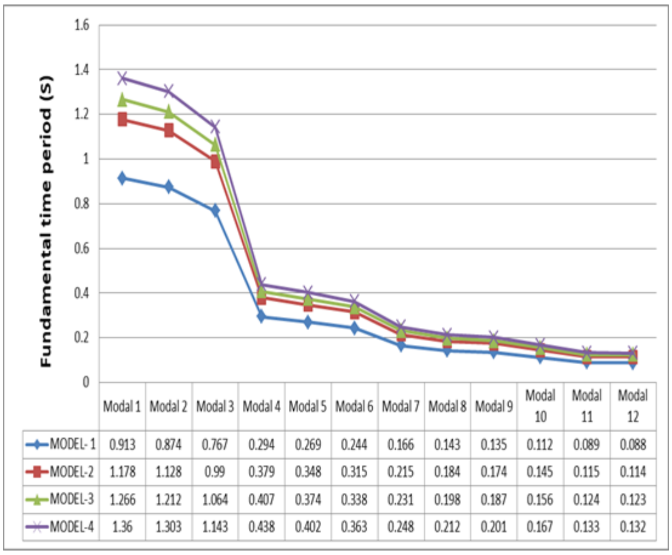

Chart -5 fundamental time period

Story stiffness

As per IS 1893:2002 the lateral stiffness is less than 70 percent of that in the storey above or less than 80 percent of average lateral stiffness of the three-storey above

Table10- Max. Storey stiffness (kN/m) comparison in x direction-The table and graph below shows the comparison of various models in terms of storey stiffness in X direction

NOOF STOREY	MODEL- 1	MODEL-2	MODEL-3	MODEL-4
Storey-7	102385	61431.23	53240.97	46073.92
Storey-6	111112	66667.12	57778.80	50000.88
Storey-5	111465	66879.15	57962.56	50159.91
Storey-4	111717	67030.41	58093.65	50273.35
Storey-3	113953	68372.01	59256.39	51279.56
Storey-2	134610	80765.71	69997.71	60574.94
Storey-1	1021717	613030	531299	459778

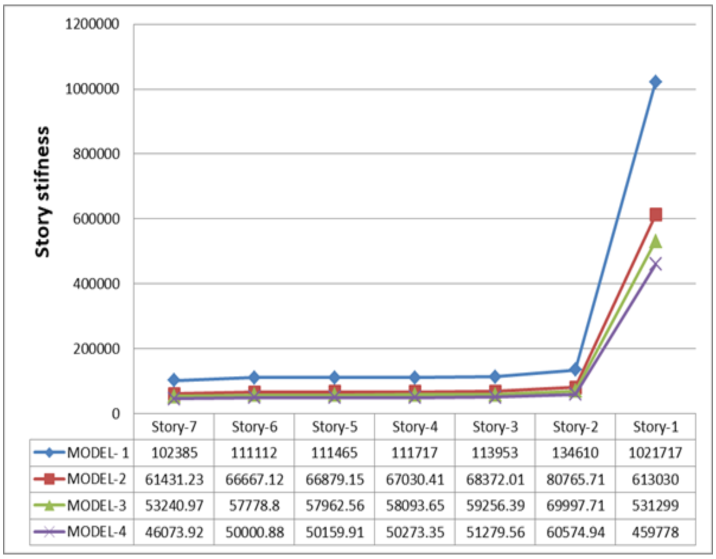

Chart -6. storey stiffness in x-direction

Table11- Max. Storey stiffness (kN/m) comparison in Y direction-the table and graph below shows the comparison of various models in terms of storey stiffness in Y direction.

NO OF STOREY	MODEL- 1	MODEL-2	MODEL-3	MODEL-4
Story-7	98054	58832.84	50989.01	44125.10
Story-6	114421	68652.43	59499.42	51489.88
Story-5	115414	69298.55	60016.06	51936.97
Story-4	116206	69723.58	60427.76	52293.26
Story-3	122717	73630.09	63813.44	55223.17
Story-2	161852	97111.35	84164.09	72834.31
Story-1	1193646	716188	620703	537147

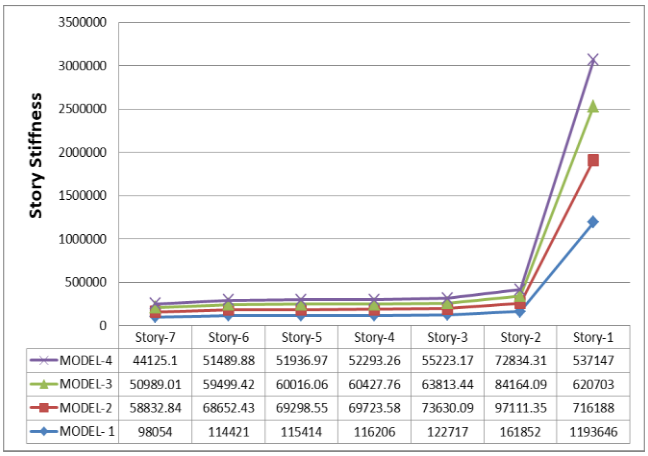

Chart -7. storey stiffness in x-direction

6. CONCLUSIONS

Several conclusions may be taken from the following analysis and findings, such as::

1) Concrete elements subjected to temperature up to 300°C or 500°C are still safe for use because at this temperature concrete moistures would have been absolved by fire and cracks will occur due to expansion and contraction of constituent materials but the entire structure will be serviceable. However, buildings subjected to temperatures above 600°C are structurally unsafe. At temperature above 600°C concrete element will have lost about 70 % of its strength
2) The model above 600°C is failed in design check Hence, buildings subjected to temperatures above 600°C are structurally unsafe
3) Max story Drift increases with increase in temperature
4)) It is seen that max story drift increases by 22.606% between model1 to mode ,6.88% in between model 2 and model 3 and 6.94% increases in-between model 3 and model 4
5) Max story Displacement increases with increase in temperature
6) It is seen that max story displacement increases by 23.460% between model1 to model2 ,7.2438% in between model 2 and model 3 and 6.93% increases in-between model 3 and model 4
7) The fundamental time period will increase with the increase in temperature
8) Story stiffness decreases with increase in temperature
9) It is seen that Story stiffness decreases by 40% between model1 to model2 ,13.33% in between model 2 and model 3 and 13.46 % decreases in-between model 3 and model 4
10) The non-destructive test is a way of testing which does not affect the overall performance of a member's entity under investigation. It could be performed during construction and after maintenance. The IRH and UPV can be used as a reliable method to predict the mechanical strength of the reinforced concrete structure
11) It is found that the slabs are mostly effected by fire and has maximum damaged it is because they are located at the highest evaluation of the room where they are exposed to the highest temperature during a fire
12) The strength of steel will start to decrease at approximately 430° C (800° F). At 590° C (1100° F) steel loses approximately 50% of its strength and stiffness when compared to normal ambient conditions. At 700° C (1300° F) the strength and stiffness are reduced to approximately 20% of the ambient condition strength and stiffness. These property reductions will likely be temporary, and the steel will regain its strength and stiffness if the temperature of the steel does not exceed 700° C (1,300° F) for longer than 20 minutes

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