Analysis of Moment Resisting Reinforced Concrete Frames for Seismic Response Reduction Factor

Prof. Amit A. Kusanale[1], Mr. Keshav Ramdas Manerikar[2]

[1] *Assistant Professor, Department of Civil Engineering, P.V.P.I.T. Budhgaon, Sangali, India*
[2] *PG Student, Department of Civil Engineering, P.V.P.I.T. Budhgaon, Sangali, India*

Abstract – Dynamic analysis of structures is extensively employed in research across various domains of earthquake analysis, including academic institutions. Until recently, its application in the practical seismic design and evaluation of buildings has not been widely reported. However, contemporary editions of building codes worldwide now advocate for the use of dynamic analysis methods in the seismic design of structures located in regions highly susceptible to seismic events. Traditional methods of earthquake analysis often rely on assumptions and estimated values. Therefore, it is crucial to conduct earthquake analysis of structures using a realistic approach, which can be achieved through the Time History Analysis Method.

Time-history analysis is a step-by-step examination of a structure's dynamic response to a specified loading that Time-history analysis is a step-by-step analysis of the dynamical response of a structure to a specified loading that may vary with time. The analysis may be linear or non-linear. Time history analysis is a kind of dynamic analysis. The recent advancement for analysis and design of high rise structure follow IS 1893:2002 to perform dynamic analysis that is when the lateral load resisting elements are oriented along orthogonal lateral directions the structure would be designed for the effects due to full design of load in one horizontal direction at time. Time history analysis provides linear or nonlinear investigation of dynamic structural response under forces which may vary according to time function. In this type of analysis the structural response is evaluated at a number of subsequent time instants that is time history of structural performance to a given input are obtained as a result.

Key Words: Dynamic analysis, Time History Analysis Method, Seismic design, Earthquake analysis

1. INTRODUCTION

The implementation of advanced control strategies to reduce the impact of seismic forces on building structures provides a promising alternative to conventional earthquake-resistant design methods. In recent years, considerable attention has been devoted to the development and application of various damping mechanisms intended to dissipate seismic energy effectively. Numerous studies have investigated the dynamic response of structures incorporating different types of damping devices, highlighting their potential to enhance structural safety and performance during earthquakes.

Furthermore, extensive numerical modeling has been employed to explore the capacity of dampers to prevent structural collapse under severe ground motions. The finite element method, combined with direct integration techniques, is widely used for simulating the dynamic behavior of structures fitted with supplemental damping systems subjected to continuous or transient excitations.

Energy dissipation devices are now commonly incorporated into both tall and low-rise buildings to complement their inherent earthquake-resistant characteristics. This study focuses on evaluating and comparing the performance of structural systems equipped with passive energy dissipation mechanisms. A passive energy dissipation system operates without the need for an external power supply; instead, it utilizes the relative motion between connected components of the structure to generate resistive forces during dynamic excitation. The amplitude and direction of these control forces depend on the displacement of the attachment points.

By integrating mechanical devices within the structural framework, energy can be dissipated effectively throughout the height of the building. The inclusion of such systems leads to a reduction in lateral drift and associated damage due to increased energy dissipation capacity, while simultaneously enhancing the overall stiffness and strength of the structure, resulting in greater resistance to seismic forces.

1.1 Objective

i. To develop analytical models of the structure with and without supplemental dampers (viscous and viscoelastic) using E-tab software.

ii. This study aims to evaluate the performance of reinforced cement concrete (RCC) structures under three conditions—without dampers, equipped with viscous dampers (VD), and equipped with viscoelastic dampers (VED)—to compare their effectiveness in reducing seismic responses.

iii. This study seeks to analyze how the incorporation of viscous and viscoelastic dampers influences the displacement response of structures under dynamic loading conditions.

iv. This study aims to analyze how the incorporation of viscous and viscoelastic dampers affects the acceleration response of structures subjected to dynamic or seismic loading.

v. Assess the increase in the reserve strength of the structure beyond the design strength due to the presence of damper.

1.3 Proposed Work:-

- Phase Model Definition and Damper Selection
- Define Prototype RC Frames: Select several RC moment-resisting frame models with varying properties
- Design Frames: Design the frames based on standard force-based design (FBD) methods according to relevant seismic codes.
- Select and Model Dampers: Choose the type of dampers (viscous, friction, metallic, etc.) and define their mechanical properties and placement strategy (e.g., diagonal, toggle brace, center span) within the RC frame model
- Create Analytical Models: Develop two main sets of non-linear numerical models

1.4 Sources:-

The 4 unique sources are Material Damping, Structural Damping, Radiation Damping and External Damping.

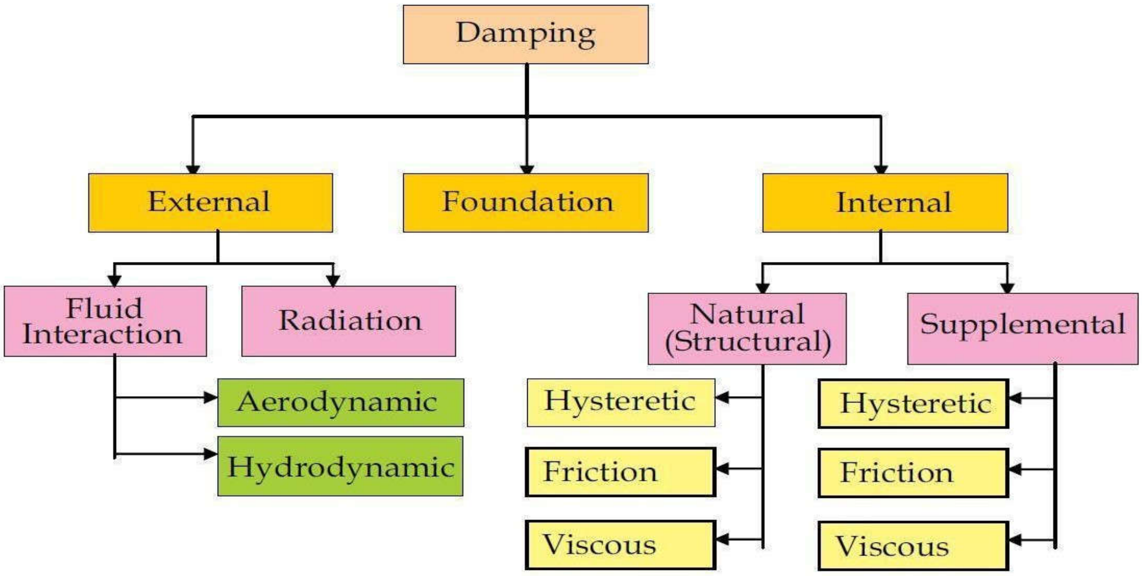

Figure 1: Sources of Damping.

2. Methodology:-

The various strategies of understanding the issue are clarified from wording, hypothesis and definition of the models for getting a reasonable outcome at the end.

3.1.1. Single Degree of Freedom System

A simple single degree of freedom system (a mass, M, on a spring of stiffness k, for example) has the following equation of motion:

$\boldsymbol{M}\ddot{x}+\boldsymbol{K}\boldsymbol{x}=(\boldsymbol{t})$

Where $\ddot{x}$ is the quickening (the twofold subordinate of the removal) and x is the uprooting.

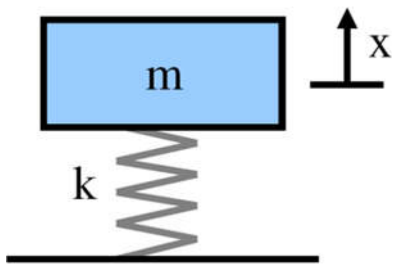

Figure 02: Single degree of freedom system: simple mass spring model

On the off chance that the stacking $(\boldsymbol{t})$ is a Heaviside step work (the unexpected utilization of a consistent burden), the answer for the condition of movement is:

$\boldsymbol{x}$= (F0/K) $[\mathbf{1}-(m\boldsymbol{t})]$

Where $m=\sqrt{}\ (\boldsymbol{k}/\boldsymbol{M})$ and the basic normal recurrence $\boldsymbol{f}=m/\mathbf{2}\boldsymbol{\pi}$

The static avoidance of a solitary level of opportunity framework is:

$x$static=($\boldsymbol{F0}/\boldsymbol{k}$)

Thus, it very well may be composed as the beneath condition, by consolidating the above formulae:

$\boldsymbol{x}$=$\boldsymbol{xstatic}[\mathbf{1}-\mathbf{cos}(m\boldsymbol{t})]$

This gives the (hypothetical) time history of the structure because of a heap F(t), where the bogus supposition that is made that there is no damping.

In spite of the fact that this is too shortsighted to even think about applying to a genuine structure, the Heaviside Step Function is a sensible model for the utilization of numerous genuine burdens, for example, the unexpected expansion of a household item, or the expulsion of a prop to a recently cast solid floor. Notwithstanding, in all actuality loads are never applied promptly - they develop over some stretch of time (this might be extremely short in fact). This time is known as the ascent time.

As the quantity of degrees of opportunity of a structure expands it rapidly turns out to be too hard to even think about calculating the time history physically - genuine structures are examined utilizing non-direct limited component investigation programming.

3.2 Damping

Any genuine structure will disseminate vitality (basically through grinding). This can be demonstrated by adjusting the DAF

$\boldsymbol{DAF}$=$\mathbf{1}$+$\boldsymbol{e}-\boldsymbol{c\pi}$ Where,=($\boldsymbol{DampingCoefficient}/\boldsymbol{CriticalDampingCoefficient}$)

Furthermore, is regularly 2%-10% relying upon the sort of development.

1. Bolted steel = 6%
2. Reinforced concrete = 5%
3. Welded steel = 2%
4. Brick masonry = 10%

For the most part damping would be overlooked for non-transient occasions, (for example, wind stacking or swarm stacking), yet would be significant for transient occasions (for instance, a motivation burden, for example, a quake stacking or bomb impact).

Condition of movement for a solitary level of opportunity would now be able to be composed as

$\boldsymbol{M}\ddot{x}$+$\boldsymbol{Cx}$ $\boldsymbol{Kx}$=$-\boldsymbol{m\ddot{u}}(\boldsymbol{t})$

Where u= uprooting comparative with the ground.

3.3 Modal Analysis

A modular examination computes the recurrence modes or common frequencies of a given framework, yet not really its full-time history reaction to a given info. The common recurrence of a framework is reliant just on the firmness of the structure and the mass which takes part with the structure (counting self-weight). It isn't subject to the heap work.

It is helpful to know the modular frequencies of a structure as it permits you to guarantee that the recurrence of any applied occasional stacking won't match with a modular recurrence and subsequently cause reverberation, which prompts huge motions.

The strategy is:

1. Locate the characteristic modes (the shape received by a structure) and normal frequencies

2. Ascertain the reaction of every mode

3. Alternatively superpose the reaction of every mode to locate the full modular reaction to a given stacking

3.3.1 Energy Method

It is conceivable to compute the recurrence of various mode state of framework physically by the vitality technique. For a given mode state of a numerous level of opportunity framework you can locate a "proportional" mass, firmness and applied power for a solitary level of opportunity framework. For basic structures the fundamental mode shapes can be found by examination, yet it's anything but a traditionalist strategy. Rayleigh's rule states:

"The recurrence ω of a discretionary method of vibration, determined by the vitality strategy, is consistently more noteworthy than - or equivalent to - the principal recurrence $m\boldsymbol{n}$."

For an accepted mode shape ū(x), of a basic framework with mass M; bowing firmness, EI (Young's modulus, E, increased constantly snapshot of territory, I); and applied power, F(x):

Equal Mass, $\boldsymbol{Meq}$=∫$\boldsymbol{M}$ū$\boldsymbol{2du}$

Equal Stiffness, $\boldsymbol{keq}$=∫($\boldsymbol{d2}$ū/$\boldsymbol{dx2}$)$\boldsymbol{2dx}$

Equal Force, $\boldsymbol{Feq}$=∫$\boldsymbol{F}$ū$\boldsymbol{dx}$

At that point, as above:

m=√($\boldsymbol{keq}$/$\boldsymbol{Meq}$)

3.3.2 Modal Response

The total modular reaction to a given burden F($\boldsymbol{x}$,$\boldsymbol{t}$) is $\boldsymbol{v}$($\boldsymbol{x}$,$\boldsymbol{t}$)=σ$\boldsymbol{un}$($\boldsymbol{x}$,$\boldsymbol{t}$). The summation can be completed by one of three normal strategies:

- Superpose complete time narratives of every mode (tedious, however accurate)
- Superpose the most extreme amplitudes of every mode (brisk yet preservationist)
- Superpose the square base of the entirety of squares (great gauge for very much isolated frequencies, yet dangerous for firmly separated frequencies)

To superpose the individual modular reactions physically, having determined them by the vitality strategy:

Accepting that the ascent time $\boldsymbol{tr}$ is known (T = 2π/ω), it is conceivable to peruse the DAF from a standard diagram. The static relocation can be determined with $\boldsymbol{ustatic}$= ($\boldsymbol{F1}$,/$\boldsymbol{k1}$,$\boldsymbol{eq}$).

The dynamic dislodging for the picked mode and applied power would then be able to be found from: $\boldsymbol{umax}$=$\boldsymbol{ustaticDAF}$

3.3.3 Modal Participation Factor

For genuine frameworks there is regularly mass taking an interest in the driving capacity, (for example, the mass of ground in a seismic tremor) and mass taking part in inactivity impacts (the mass of the structure itself, $\boldsymbol{Meq}$). The modular investment factor Γ is a correlation of these two masses. For a solitary level of opportunity framework Γ = 1.

F=(σ$\boldsymbol{Mn}$ūn/Σ$\boldsymbol{Mn}$ū$\boldsymbol{n2}$)

3.4 Determined Analysis E-tabs

The investigation and structure of the structure is done utilizing ETABS PC program. The accompanying points depict a portion of the significant regions in the demonstrating.

3.4.1 Defining the slab section

In the current examination, single direction and two-way chunks are given as film type conduct to give in plane firmness/segments are demonstrated as inflexible stomachs by utilizing the unbending structure choice in the side menu task menu by displaying the section as inflexible stomach the majority of the floor is naturally logged knot at their focal point of gravity.

3.4.2 Equality static analysis

The characteristic Period of the structure is determined by the articulation t given in IS 1893:2002 where are h is the stature and d is the base element of the structure the thought about way of vibration. Does the regular time frames for all the models in this strategy is the equivalent the horizontal burden estimation and its circulation around the stature are done according to Seems to be: 1893-1984 the seismic weight is determined utilizing full dead burden + half of live burden.

3.4.3 Response spectrum analysis:-

Reaction range investigation of the structure models is acted in on ETABS. The sidelong burden appropriation created by ETABS reacts to the seismic zone 4 and the 5% damped reaction range given in IS: 1893-2002. In Analysis just a single invariant horizontal burden design was used to speak to the presumable conveyance of inactivity powers forced on the casings during an tremor and the used parallel burden design is depicted as follows. Note that the storey powers are standardized with the Base shear to have an all-out Base shear equivalents to solidarity.

3.4.4 Multimodal or SRSS lateral load pattern

The heap design considers the impacts of higher methods of vibration for long time significant stretch and unpredictable structures. The horizontal power at any storey is determined as square foundation of whole of squares SRSS mixes of the heap appropriation got from the modular investigation of the structures.

4. Modeling

The investigation in this theory depends on straight and nonlinear examination of RC structures with various regions of building and variable cross segment of segment. This section presents an outline of different boundaries characterizing the computational models, the essential suspicions and the RCC outlines calculation considered for this examination. Exact demonstrating of the nonlinear properties of different basic components is significant in nonlinear examination.

4.1 Design Data

4.1.1 Material Properties:

M25 evaluation of cement and Fe 500 evaluation of Steel are utilized for all sections and light emissions building while M30 is utilized for segments with same evaluation of Steel. Flexible material properties of these materials are taken according to IS 456-2000. The transient modulus of flexibility (Ec) of cement is taken as: $\boldsymbol{E.c}$=$\mathbf{5000}\sqrt{\boldsymbol{fck}}$ Mpa

Where $\boldsymbol{fck}$=characteristic compressive strength of concrete cube

For the Steel rebar with stress and modulus of flexibility is taken according to IS 456-2000.

4.1.2 Structural Elements

The diverse basic components considered are column; beam and slab with variable segments are referenced underneath.

Description of Members used:-

Column Sizes:

4.2 Building Analysis:

In building analysis we used G+8 model.

SCHEDULE OF COLUMN	G+1+2 FLOOR	3+4+5 FLOOR	6+7+8 FLOOR
C8,C9,C10,C22, C23,C31,C32	0.40 X 0.85 M	0.45 X 0.90 M	0.35 X 0.70 M
C2,C3,C4,C5, C40,C41,C42,C43	0.40 X 0.85 M	0.45 X 0.90 M	0.30 X 0.70 M
C1,C6,C39,C44	0.40 X 0.70 M	0.45 X 0.75 M	0.30 X 0.55 M
C7,C12,C13,C15,C16, C17,C19,C20,C25,C27, C28,C29,C33,C34,C35,C38	0.40 X 0.70 M	0.45 X 0.75 M	0.30 X 0.55 M
C8,C9,C10,C22, C23,C31,C32	0.40 X 0.80 M	0.45 X 0.85 M	0.30 X 0.60 M
C8,C9,C10,C22, C23,C31,C32	0.40 X 0.70 M	0.45 X 0.75 M	0.30 X 0.55 M

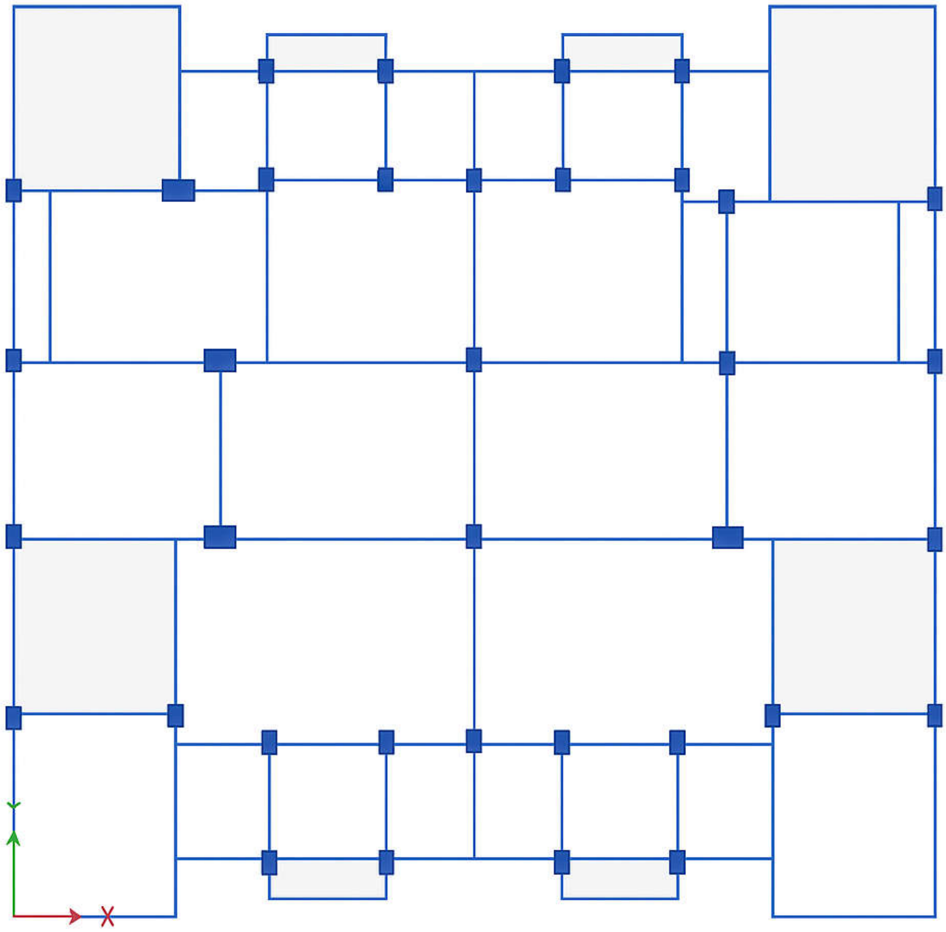

Figure 03: Plan View

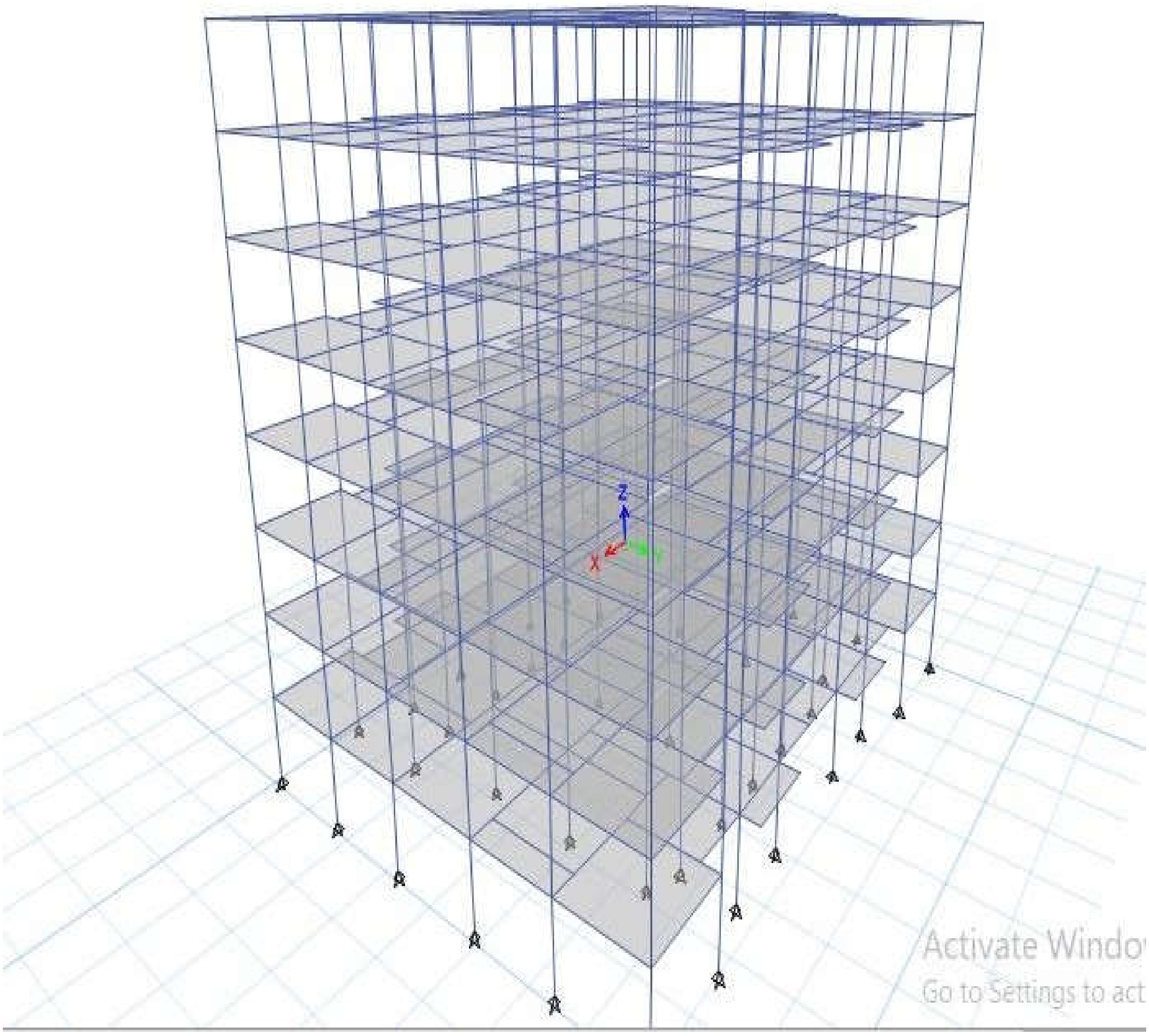

Figure 04: 3D View

4.3 Modelling of damper:

Properties of dampers are provided by deriving it with mass and weight experimentally

Viscous Damper Properties:

Mass - 1850 kg Weight - 0.18 KN

Visco-elastic Damper Properties:

Mass - 2050 kg Weight - 0.22 KN

ETABS MENU=> Define=> Link Properties=> Add new Link=> Link Property Data

4.4 Modeling with Dampers:

Modeling is done with providing damper. Dampers are provided at edges in downward direction. Dampers are provided on every side of the building. Following image shows placing of the damper. This placing is done for viscous damper as well as for viscos-elastic damper.

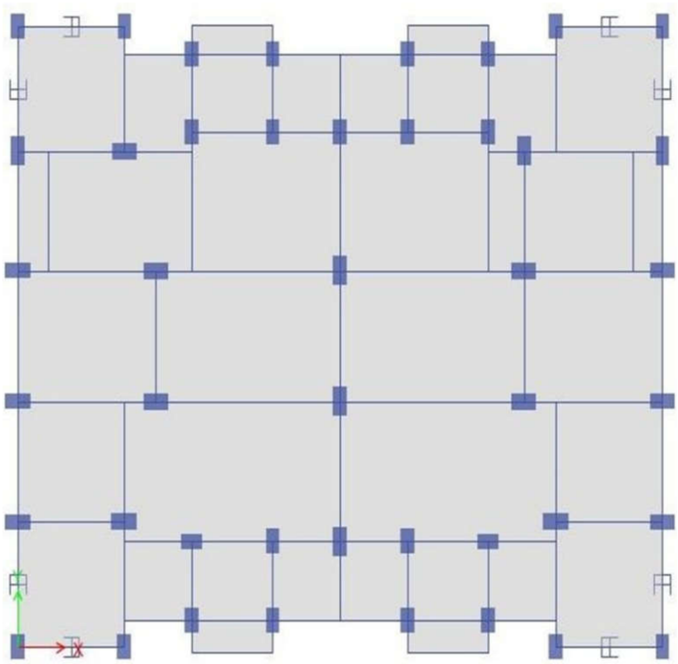

Figure 05: Damper Provided Plan View

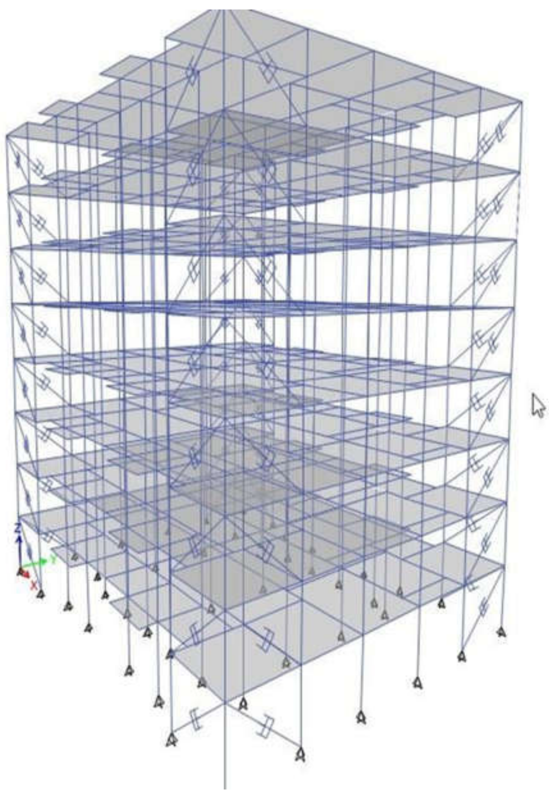

Figure 06: Damper Provided Elevation

5. Analysis & Discussion of Results

The results from the study with and without dampers are shown. The findings are based on the point of the examination. After getting the results, they are compared with the conclusions made from them. In nonlinear cases, the process can either start from zero or continue from a previous case. If it starts from zero, the time work is simply set to zero. If it continues from a previous case, it's assumed that the time work will increase in proportion to its starting value. A long record can be broken into several sequential checks that use the same capacity and time markers. This avoids the need to create multiple different capacities.

5.1Results:

5.1.1 Response Spectrum Curves from Time History: This shows reaction range plots got from time history results at a predefined point for a predetermined time history load case.

5.2 Comparative graphs between viscous and visco-elastic damper

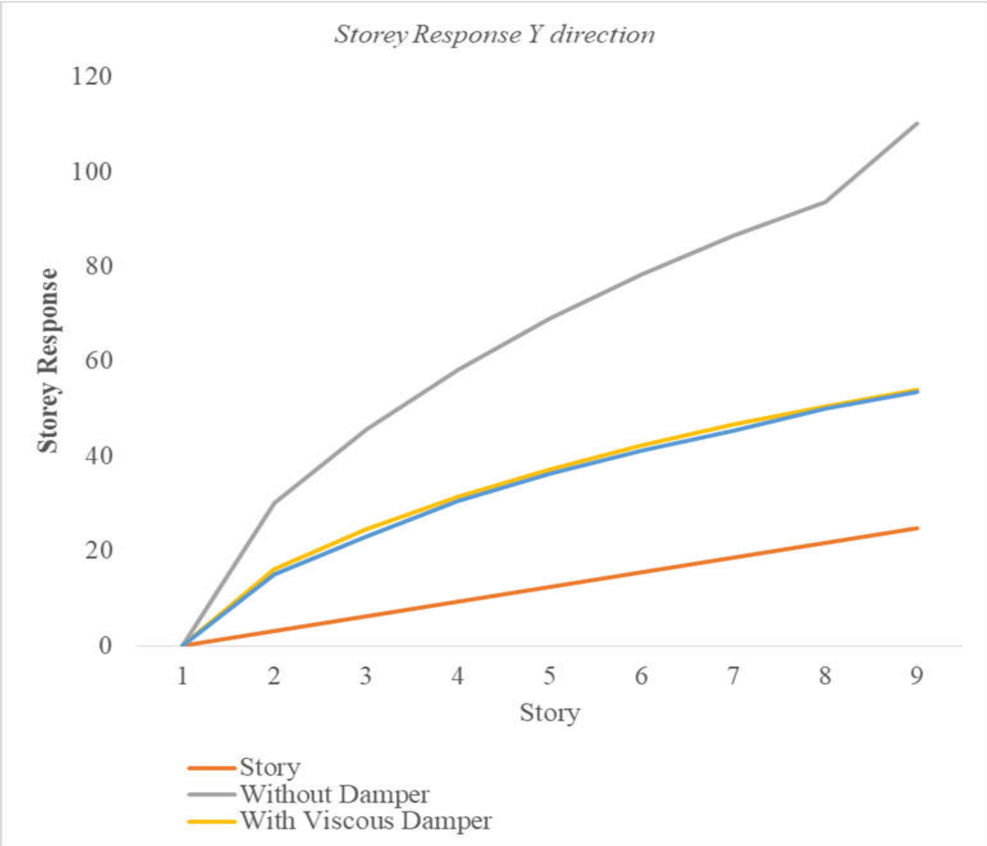

Figure 08: Damper Graph of Storey Response in y direction

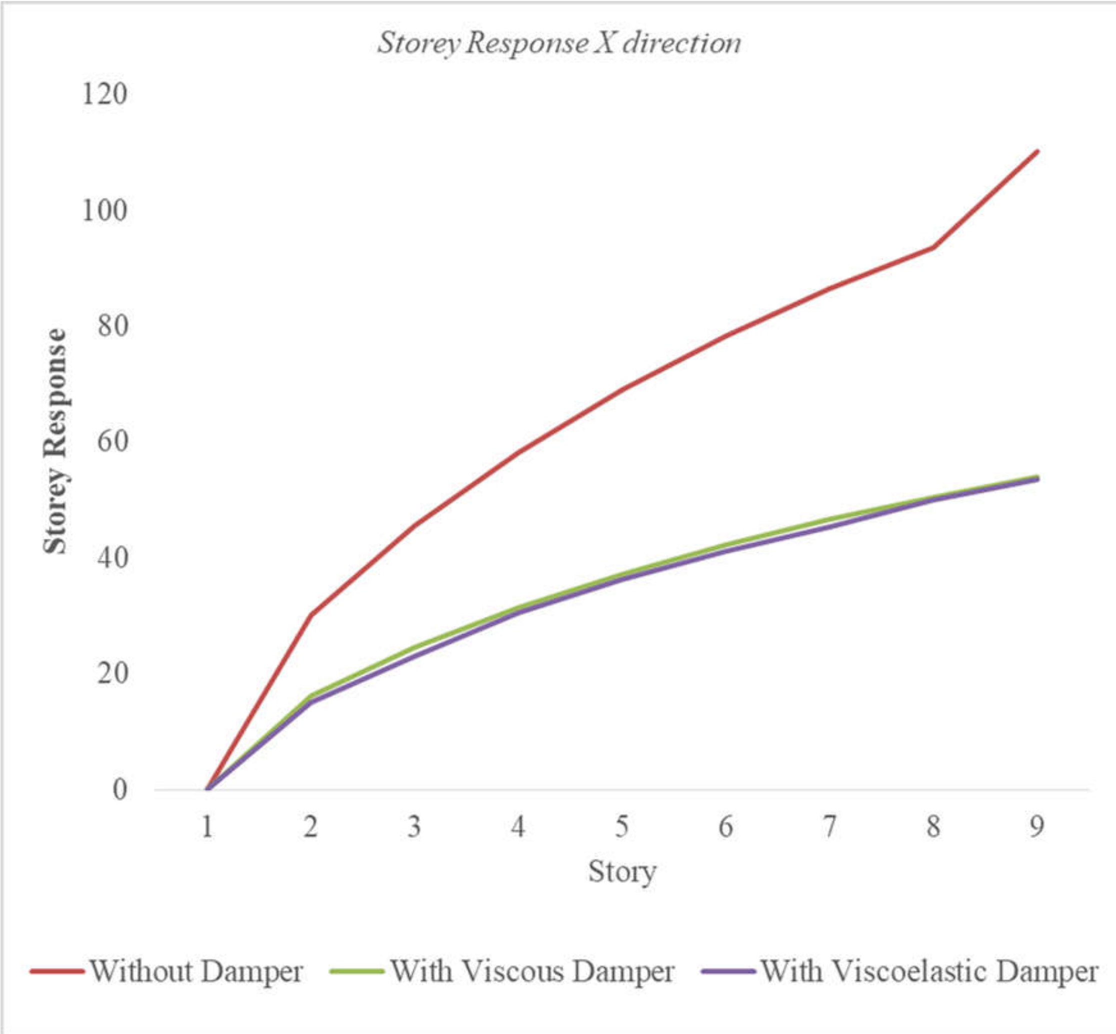

Figure 08: Damper Graph of Storey Response in x direction

3. CONCLUSIONS

It is concluded that the maximum displacement reduces from 87.52 mm (without dampers) to 45.22 mm (with viscoelastic dampers), resulting in a reduction of approximately 48.32%.

For the analyzed G+8 structure, incorporating viscoelastic dampers resulted in approximately a 48.32 % reduction in displacement, demonstrating their effectiveness in minimizing lateral movements.

Displacement responses are lower in systems equipped with viscoelastic dampers dampers than in those using viscoelastic dampers

The storey drift observed in all buildings remains within the permissible limit of 0.004H, as specified by design codes.

The system With Viscoelastic Dampers provides the lowest initial acceleration (e.g., 0.87{mm/sec2} at X=4) compared to the Undamped system (1.27 {mm/sec}2).

REFERENCES

1. Jayadeep K S, Pramod Kumar Mahato, Dharmendra Kumar Sah, Baiju Prasad Gupta, Paon Thangjam (2022)

2. "Seismic Analysis of Multi-Storey RCC Building with and without Viscous Dampers Sharjuban, Jisha P (2019)

3. "Seismic Response Reduction Factor Evaluation for Irregular RC Structures"Visuvasam Joseph Anton, (2017)"

4. Assessment of seismic response reduction factor for moment resisting RC frames" Constantinous. M. C., Soong, T. T., and Dargush, G. F. (1998).

5. "Passive energy dissipation systems for structural design and retrofit", Monograph No. 1, Multidisciplinary Center for Earthquake Engineering Research, Buffalo, New York.

6. Chang, K. C., Soong, T. T., Lai, M. L., and Nielsen, E. J. _1993_. "Development of a design procedure for structures with added viscoelastic Trevor e Kelly and t k dutta "optional use of VE dampers in frames for seismic forece",ASCE J.Strutural engineering

7. T. Soong and G. F, Dargush John Wiley Sons Chichester, Passive energy dissipation in Structural engineering, Volume6, Issue 1 June 1999.

8. Trever. E. Kelly, S.E, "In Structure damping and energy Dissipation guidelines" Volume 4, July 2001.

9. Hanson RD, Soong TT, Seismic design with supplemental energy dissipation devices. Monograph No. 8, EERI Oakland, 2001.

10. M.D. Symans and M.C.Contstaniou, " Passive fluid viscous damping system for seismic energy dissipation," ISET Journal of Earthquake technology.,Vol.35,pp.185-206, Dec 1998.

11. K.C. Chang , Y.Y. Lin and M, "Seismic Analysis and Design of Stucture with Viscoelastic Dampers," ISET Journal of Earthquake Technology., Vol.35,pp.143-166, Dec 1998.

12. T.T. Soong and B.F. Spencer, "Supplemental energy dissipation: state-of-the-art and state- of-the practice," Engineering Structures., Vol.24, pp.243-259,2002.

13. Robert J. MCNAMARA and Douglas P. Taylor, "Fluid viscous dampers for high-rise buildings," – The structural design of tall and special buildings, Vol.12, pp.145-154,2003.

14. Lyan-Ywan Lu, "Predictive control of seismic structures with semi-active friction dampers," –Earthquake Engineering Structures-Dynamics, Vol.33,pp.647-668,2004

15. Carlos A. Martínez, Oscar Curadelli, María E.Compagnoni, Optimal Design Of Passive Viscous Damping Systems For Buildings Under Seismic Excitation, Journal of Constructional Steel Research 90 253–264 (2013)

16. V. S. Balkanloua , M. R. Bagerzadeh , B. B. Azarc and A. Behraveshd , Evaluating Effects of Viscous Dampers on optimizing Seismic Behavior of Structures, International Journal of Current Engineering and Technology ISSN 2277 – 4106 (2013).

17. Devesh P, Bharat B. Qualitative review of seismic response of vertically irregular building frames. ISET Journal of Earthquake Technology. 2006 Dec; 43(4):121–32.

18. Goud SS, Ramancharla PK. Rationalizing Response Reduction Factor R for better Performance of Reinforced Concrete Framed Buildings. National Conference on Recent Research Advances in Civil Engineering, India, 2014.

19. Chaulagain H, Rodrigues H, Spacone E, Guragain R, Mallik R, Varum H. Response reduction factor of irregular RC buildings in Kathmandu valley. Earthquake Engineering and Engineering Vibration. 2014 Apr; 13(3):455–70.

20. Sococol, I.; Mihai, P.; Toma, I.-O.; Olteanu-Dontov, I.; Venghiac, V.-M. Stress-Strain Relation Laws for Concrete and Steel Reinforcement Used in Non-linear Static Analytical Studies of the Moment Resisting Reinforced Concrete (RC) Frame Models. Bull. Polytech. Inst. Jassy Constr. Archit. Sect. 2021.

21. Rizwan, M.; Ahmad, N.; Khan, A.N. Seismic performance assessment of reinforced concrete moment resisting frame with low strength concrete. Structures 2021,

22. Patel,B. and Shah,D. (2010), 'Formulation of Response Reduction Factor for RCC Framed Staging of Elevated Water tank using static pushover analysis, Proceedings of the World Congress on Engineering, Vol. III, London, U.K.

23. Standard, Indian. "Criteria for earthquake resistant design of structures." Bureau of Indian Standards, Part 1 (1893)

24. Misal, D. J., and M. A. Bagade. "Study of seismic behaviour of multi-storied RCC buildings resting on sloping ground and considering bracing system." International Journal of Engineering Research 5.3 (2016): 690-697.

25. Abdi, H., Hejazi, F., Saifulnaz, R., Karim, I. A., and Jaafar, M.S. (2015). "Response modification factor for steel structure equipped with viscous damper device." International Journal of Steel Structures, 15 (3), 605-622.