

SEMI-RIGID CONNECTION BEHAVIOUR OF STEEL FRAME STRUCTURE UNDER SEISMIC LOAD: A NUMERICAL STUDY USING STAAD-Pro

A project Submitted in Partial fulfilment of the Requirements for the Degree of Master of Technology in Structural Engineering

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ABSTRACT

The stability and seismic response of steel structures with semi-rigid connections are particularly affected during the seismic loading phase. Three-dimensional structural primary deformations are determined during the loading phase because semi-rigid connections possess a certain degree of partial rotational stiffness.

This study uses STAAD-Pro to analyse the effect of semi-rigid connections structure performance of building and inter-storey drift as well as stability of various semi-rigid connection steel frame configurations were evaluated through numerical modelling analysis. The results demonstrate the versatile behaviour of semi-rigid connections. This exerts the need to revise and expand design approaches involving semi-rigid connections to achieve desirable best performance and steel construction relative to cost.

The study presented in this report explores the seismic behaviour of semi-rigid connections in steel-framed structures, aiming to define their real structural influence compared to conventional rigid and pinned connections. Semi-rigid connections exhibit partial rotational stiffness, allowing a transition between rigidity and pinned, which affects the overall structural response under loading conditions.

Semi-rigid connections exhibit partial rotational stiffness, allowing a transition between rigidity and pinned, which affects the overall structural response under loading condition. The research focuses on a 5×5 bay steel frame with varying storey heights (3, 6, 9, and 12 story), every story considers 3m height. Examining the impact of connection rigidity ranging from 0.2 to 0.8 on parameters such as base shear, time period, displacement, inter-storey drift, and span moments. The study highlights the economic advantage and improved the performance of semirigid connections.

Keywords:-Steel frame, Semi-rigid connections, Rigidity factor, Displacement, M- ϕ curve

INTRODUCTION

In steel structural design, connections are critical components that join various structural elements such as beams, columns, and braces to form a cohesive and stable framework.

These connections are essential for transferring loads and ensuring the overall integrity of the structure. Accurate structural analysis hinges on understanding actual joint behavior. Traditional steel assumptions of perfectly flexible (pinned) or fully rigid (fixed) connections often lead to hypothetical estimations. In reality, connections possess rotational rigidity between these extremes, known as semi-rigid connections. This study aims to ascertain their true behavior.

In steel structures, connections are classified based on their ability to resist rotation and transfer bending moments. These classifications rigid, semi-rigid, and pinned (also known as simple or hinge connections) determine how forces and moments are transmitted between structural elements, influencing the overall stability and behaviour of the structure. A semi-rigid defines as connection allows limited rotation under loading while still offering some moment resistance. These connections do not maintain perfect angles between connected members like rigid connections do, but they also don't rotate freely like pinned connections. Semirigid connections strike a balance between strength, flexibility, and economy. Their proper understanding and use can significantly enhance the performance of steel structures, especially where realistic behaviour and cost optimization are key. Behaviour Under Load of Semi-rigid connections- Exhibit non-linear moment-rotation behaviour. Allow redistribution of internal forces during loading, especially beneficial in indeterminate structures. Provide economical and efficient structural systems when properly designed.

Table 1:- Ductility and flexibility fundamental concepts			
Sr.No	Aspect	Flexibility	Ductility
1	Main Concern	Displacement under load	Deformation capacity before failure
2	Design Role	Controls serviceability, dynamic behaviour	Controls seismic performance, failure mode
3	Units	mm/kN, degrees/kNm	Dimensionless ratio (e.g., μ)
4	Failure Type	Not necessarily associated with failure	Important in avoiding brittle failure

Moment –.Rotation. & M– ϕ Behaviour

- In structural engineering, M– ϕ curves (moment–rotation curves) are critical for understanding the semi-rigid behaviour of connections in steel structures. These curves show how a connection resists rotation under an applied moment, offering insight into stiffness, strength, and ductility.
- Moment (M): The bending force applied to a connection (usually in kNm or lb-in).
- Rotation (ϕ): The angular deformation of the connection (radians or degrees).

Moment-Rotation (M– ϕ) Relationship

The M– ϕ curve defines connection response:

$$Mu = S_{j,ini} \cdot \phi(\text{Elastic Phase}).....(1)$$

Nonlinear Model (Realistic):

Many M– ϕ relationships use nonlinear expressions. One popular empirical model:

$$M = \frac{M_u \phi}{\phi_r + \phi} \dots\dots\dots(2)$$

Key Parameters Extracted from M– ϕ Curves

- i) Initial stiffness- Slope at origin, affects structure's lateral stiffness
- ii) Moment capacity (M_u)- Maximum resistible moment
- iii) Rotation capacity (ϕ_u)- Rotation at failure (ductility)
- iv) Plastic rotation (ϕ_{pl})- Useful for energy dissipation, especially in seismic design

Design Standards & Applications Eurocode 3 (EN 1993-1-8):

Offers procedures for defining stiffness, classifying connections (rigid, semi-rigid, pinned).
 AISC 360: U.S. standard for structural steel design (includes M– ϕ testing and modelling guidelines) Nonlinear frame analysis and Progressive collapse studies and seismic performance-based design with partial strength connections

M– ϕ Curves of Various Connections

This analysis is done with the combination of serviceability criteria. While sketching out the basis standards, initial stiffness(C) = $\frac{EI}{L}$ [Standardisation Parameters of Various Connection-IS 800-2007 clause F.4.3.2], is a straight line which is tangential to the moment-rotation curve through the origin. The point at which the straight line bisects the X- axis is called plastic moment of the beam represented by M_p .Also for the rotation it is equal to ϕ_p . The relationship obtained are not in linear model or exponential model but there is a gradual variation from linear to exponential .

Beam Length Selection in Semi-Rigid Joint Classification

Beam length is picked in such a manner that initial stiffness coordinate with that of connection. For a peculiar connection conduct, we have to adopt distinct length of reference though there is significant change in the stiffness of various connection. For the same rationality, 1st place was appreciated to the distribution of the systems. So, assorting the connection according to the beam length have a greater significance because all the structural joints in the domain should match the corresponding length of beam. Like this all the connection behaviour can be distinguished with a single curve of the beam and actual non-dimensional rotation. This can be achieved by $\frac{\phi}{\phi_p}$ of the beam with original length.

Similarly, we can site moment parameter by $\frac{M}{M_p}$. The extent of classification criteria is given [Various Type of Connections as Per IS800 Steel Code fig 32]

Table 2:-Codal Limits (IS – 800 -2007) for Connection Categorization			
Sr.No.	CONNECTION TYPE	STRENGTH	STIFFNESS
1	Rigid	$M' \geq 0.7$	$M' \geq 2.5\theta'$
2	Semi-rigid	$0.7 > M' > .2$	$2.5\theta' > M' \geq 0.5 \theta'$
3	Flexible	$M' \leq 0.2$	$M' \leq 0.5\theta'$
Where, $M' = \frac{M_{ultimate}}{M_{plastic}}$ and $\theta' = \frac{\theta_{Normalized}}{\theta_{plastic rotation}}$			

Most research has shown that the behaviours of connections is important for the strength and displacement characteristics of a structure. Materials, geometrical structures, loading situations, and boundaries are some of the factors that might result in nonlinear structural behaviours. Biradar [2018]]The study of behaviour of partially restrained connections under the effect of seismic load for top and seat angle connection This paper is presented considering top and bottom seat angle with bolted connection. It is to be noted that most of the cases there is a gap of approx. 5mm between column and beam connection details. Hence rotation of the joint is possible when there is load acting on the beam, full force transfer may not take place at the joint. Many researchers have tried to give formulae for stiffness for this partially restrained connection and this paper has tried to analyse the steel frames for pinned, rigid and partially restrained connection with the calculation of relative stiffness of joint. The analyses are carried out using Staad-Pro Software for pinned , rigid and Semi-rigid connection and results are compared and presented in this paper.

Denga et. al., [2020]Seismic performance of mid-to-high rise modular steel construction. - A critical reviewThe influence of earthquake becomes critical as the height of the building increases. Hence, this paper presents a state-of-the-art review of the seismic performance of mid-to-high MSC and articulates the key technical issues. The module classification is presented as a brief introduction of MSC, followed by discussion of the structural system. Afterwards, the seismic performance of the lateral force resisting system and recent innovations on the connection system are reviewed in detail, on which the seismic performance of MSC highly depend. The global seismic response analysis methodology, characteristics, failure mode as well as the current design criteria are evaluated, providing a more comprehensive understanding of the seismic performance of Modular steel construction MSC.

Fathizadeh and Dehghani, [2021]Seismic performance assessment of multi-story steel frames with curved damper and semi-rigid connections This paper explore curved steel damper is an innovative energy dissipation device for seismic application. The performance of the curved steel damper has been well studied and experimentally tested. In addition, the behaviour of one story curved damper semi-rigid frame (CDSRMF) has been studied extensively. In this study, the seismic performance of multi-story (3 story, 6 story and 9

story) CDSRMFs are extensively examined using nonlinear static (pushover analysis) and nonlinear dynamic analysis. The results show that the addition of the curved damper has significantly improved the stiffness, strengths and energy dissipation of the CDSRMFs.

Emad A.Elhout [2024]Effect of Beam-Column Connection Types on the Response Modification Factors of Steel Frames In such paper shows the capacity for transferring moment, the response modification factor (R-factor) is an effective parameter used in the seismic design of structures. The influence of the beam-column connection's stiffness factor on the response modification factor did not seem to have been considered in seismic design codes. Consequently, the R-factor under static pushover and dynamic loading is being studied for moment resisting steel frames (MRSFs) with 3-, 6-, and 12-story using three different forms of beam-column connections depending on the connections' stiffness (m). The rigidities of the connections are taken 20, 10, and 5 for rigid, stiff semi-rigid, and flexible semi-rigid connections, respectively. Also, the R-factors were more affected by the rigidity factors for the beam-column connections and the number of story frames. DRAIN-2DX software program was used in the analysis of the structure models. The behaviour of the R-factor value studied is based only on a 1.5% story drift ratio. It is necessary to evaluate the R-factor value of frame buildings with other drift ratios in the future.

Primož and Couchaux [2025]Joints and connections with fasteners for resilient steel structures The aim of this special issue is to provide a comprehensive overview of the latest advances in theoretical and experimental research on joints with mechanical fasteners, thus facilitating the practical application of these solutions. The topic is of great interest to both the academic community and industry professionals. We invite submissions that deal with joints with mechanical fasteners and include experimental, numerical or analytical research on the behaviour of joints with various fasteners such as various types of bolts (blind, injection, anchor bolts), rivets, pins, novel fasteners and joints in self-centring or demountable structures etc.

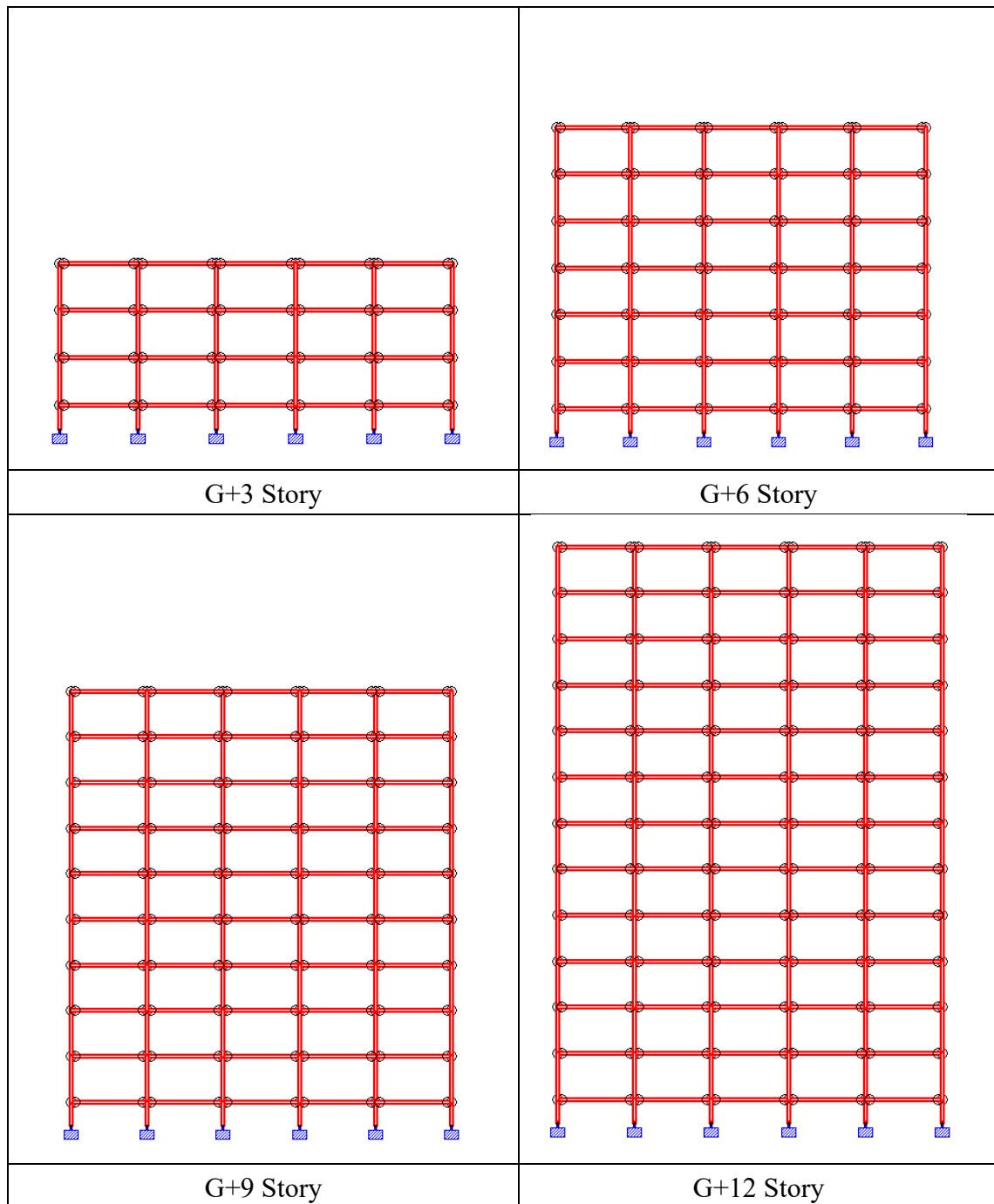
METHODOLOGY AND CALCULATION

Connections are very important factor of any structure. Especially in steel structures. The connections defined the overall ductility of the structures. Theoretically all connections are categorised into two categories, i) Fully rigid ii) fully pinned. However, in practically all connections are partially rigid connection. Thus, to understand the performance of the structures allowing this effect becomes necessary. to understand the problem statement and the parameter that considered for the study. Also it deals with Models considered for solving the problem statement. The overall objective of the work is to assess the effect of rigidity factor (j) on the preference of multi-story structures and to investigate the behaviour of semi-rigid connections under seismic loading, with a specific focus on structural modelling.

The sub objective of the study as follows,

- To investigate the effect on base shear base of the structure.
- To find time period of the structure.
- To calculate the span moments developed in the beam span.

- To find the top storey displacement horizontal displacement of the floor.
- To calculate the Storey drift i.e. difference in sideways movement between two consecutive floors



By their performance directly influenced by their rotational stiffness for this investigation, a square steel frame comprising 5×5 bays in both X and Y directions is modelled. The connection stiffness is varied continuously from 0.8 through 0.2 and additional cases of fully pinned and fully fixed conditions are included. Additionally, the study considers the effect of varying the number of storeys, testing frames of 3, 6, 9, and 12 storeys to assess how story height influences the behaviour of semi-rigid connections. Semi-rigidity is introduced by applying partial moment release at beam ends, simulating realistic connection stiffness. STAAD.Pro software is used as described in the work presented by

[Swati et. al., (2018)] and [Pooja et. al., (2023)]for the analysis, including static linear seismic response spectrum analysis for Aurangabad (Zone II), as per relevant seismic codes.

Table 3:- Details of Models Considered for Study					
Model Number	Description of Model	STAAD Pro input	Rigidity factor (j)	Height (No. the story)	Number of Models
Model 1	(5x5) bay	0	100% (fixed)	3,6,9,12 storey	4
Model 2	(5x5) bay	0.2	80%	3,6,9,12 storey	4
Model 3	(5x5) bay	0.4	60%	3,6,9,12 storey	4
Model 4	(5x5) bay	0.6	40%	3,6,9,12 storey	4
Model 5	(5x5) bay	0.8	20%	3,6,9,12 storey	4
Model 6	(5x5) bay	1	0%(pinned)	3,6,9,12 storey	4

STAAD PRO software used in the analysis of structures modelling. Model the frame of structures and assign support conditions (fixed). The sectional size of the member will vary according to the variation of the structural models. The STAAD PRO. software will automatically select the most appropriate and economic passing section for the structure .Initially we provided ISHB 450 & ISMB 350 for columns and beams respectively. Semi rigidity can be provided to steel frames by adding springs at the ends of the beams .In software's this can be achieved by releases at the beams .In Staad pro we can define this rigidity in General Specification Partial Moment releases. Here the user should define start and end locations. After defining user can directly assign the releases to the beams to the start and end of the beams .Green colour represents the starting of the beam while blue indicates the ending. However, it works: instead of a full fixity (0% rigid) or a hinge (100% release), you apply a fractional release (e.g., 25% stiffness retained ~75% release means).

Table 4:-Moment Variations vs. Connection Fixity			
Connection Rigidity (j)	Support Moment	Mid-span Moment	Overall Moment Shape
Pinned ($\approx 90\%+$)	Very low	High	Dominant positive mid-span, low support moments
Moderate ($\approx 20-80\%$) semi-rigid	Moderate	Moderate	More balanced moment, reduced peak bending demand
Rigid ($\approx 0\%$)	High	Low	Large negative support moment, small mid-span peak

Since we are performing the seismic analysis of the structure we will consider the loads on the structure. The various loads to be carried by the structure are as follows as per I.S 875-1987 (part 1). The self-weight of the beams, columns, and slab, Walls = 200mm thick Concrete block. Load from wall (super imposed dead load) = 10.2 kN/m and Dead load on the slab $= (0.075 \times 25 + 1) = 2.183 \text{ kN/m}^2$. All the temporary loads acting on the structure constitute imposed loads. For the design calculation of live loads are taken from I.S 875-1987(part 2).i.e. Live load on the slab = 2 kN/m². The wind load designing a flat-roofed, square building in Aurangabad, with a length of 25 m and varying heights of 3 , 6 , 9 , and 12, story building under Terrain Category 3. Here's how to calculate the applied wind load step by step using IS 875-3—we'll interpolate height factors and determine external/internal pressure coefficients. Calculate the design wind speed and pressure (Pd): There is no variation in either for the first 10 m in height, thus the values are constant for the height of the building. The basic wind speed for Aurangabad = 39m/sec Terrain and height factor, k_2 (from Table 2 of IS 875 (Part 3): 2015) = 0.91

The design wind speed:

$$V_z = V_b \times k_1 \times k_2 \times k_3 \times k_4 \quad (\text{Cl.6.3.})$$

$$= 39 (1.0) (0.91) (1.0) (1.0) = 35.49$$

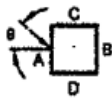
$$P_z = 0.6 \times V_z^2 = 0.6 (37.83)^2 = 0.755 \text{ kN/m}^2 \quad (\text{Cl. 7.2})$$

$$P_d = K_d \times K_a \times K_c \times P_z \quad (\text{Cl 7.2})$$

$$= (0.9) (0.9) (0.9) (0.755) = 0.550 \text{ kN/m}^2$$

Check the minimum wind pressure: $0.7 \times P_z = 0.528 < P_d$, OK.

I. External Pressure Coefficient when wind Direction

Table 5:- From IS 875 3:2015):							
Sr. no.	Building plan ratio	Plan	Wind Angle	C _{pe} for surface			
				A	B	C	D
1	$1 < l/w < 3/2$		0	+0.7	-0.25	-0.6	-0.6
			90	-0.6	-0.5	+0.7	0.1

II. Internal pressure coefficient: C_{pi} = ±0.2 (internal Coefficient as the building opening not more than 5 % From Cl 7. 3.2.1 of IS 875 3:2015).

Table 6:- Wind load calculation $C_{pi} = +0.2$

Height of building	Terrain-3 K2	Area	Pd	Wind	$F=[C_{pe} - (+ C_{pi})] \times A \times Pd = \text{kN}$			
				angle	A	B	C	D
3-story	0.91	225	0.550	0	61.98	-55.78	-99.17	-99.17
9m				90	-86.77	61.98	61.98	-12.40
6-story	1.01	450	0.678	0	152.70	-137.43	-244.32	-244.32
18m				90	-244.32	-213.78	152.70	-30.54
9-story	1.06	675	0.747	0	252.29	-227.06	-403.66	-403.66
27m				90	-403.66	-353.20	252.29	-50.46
12-story	1.15	900	0.879	0	395.93	-356.34	-633.48	-633.48
36m				90	-633.48	-554.30	395.93	-79.19

Table 7:- Wind load calculation $C_{pi} = -0.2$

Height of building	Terrain-3 K2	Area	Pd	Wind	$F=[C_{pe} - (- C_{pi})] \times A \times Pd = \text{kN}$			
				angle	A	B	C	D
3-story	0.91	225	0.55092	0	111.56	-6.20	-49.58	-49.58
9m				90	-49.58	-37.19	111.56	37.19
6-story	1.01	450	0.67866	0	274.86	-15.27	-122.16	-122.16
18m				90	-122.16	-91.62	274.86	91.62
9-story	1.06	675	0.74751	0	454.12	-25.23	-201.83	-201.83
27m				90	-201.83	-151.37	454.12	151.37
12-story	1.15	900	0.87984	0	712.67	-39.59	-316.74	-316.74
36m				90	-316.74	-237.56	712.67	237.56

Seismic loads will be calculated in accordance with IS 1893 – 2016 (PART 1 corresponding to each Zone. Design Horizontal Seismic Coefficient

$$\alpha_h = \frac{Z \times I \times s_a}{2 R \times g} \dots\dots\dots(3)$$

While designing a building or a structural member, the following shall be load combination
 Considered for the appropriate design- i) Ultimate limit state ii) Serviceability limit state

Ultimate limit state load combinations Earthquake Load Combinations in IS 1893-1:2016		Serviceability limit state load combinations	
1	1.5[DL+IL]	1	DL + IL
2	1.5[DL+IL±(EL±x)]	2	DL + 0.8IL ±(EL±x)
	1.5[DL+IL±(EL±z)]		DL + 0.8IL ±(EL±z)
3	1.2[DL+IL±(EL±x)]	3	DL ± (EL±x)
	1.2[DL+IL±(EL±z)]		DL ± (EL±z)
4	0.9DL ±(EL±x)	4	0.9DL ±(EL±x)
	0.9DL ±(EL±z)		0.9DL ±(EL±z)
5	1.2(DL + LL ± (WL±z))	5	DL ± (WL±x)
	1.2 (DL + LL ± (WL±z))		DL ± (WL±z)
6	1.5 (DL ± (WL±x))	6	DL + LL ± (WL±x)
	1.5 (DL ± (WL±z))		DL+LL ± (WL±z)

NOTE :-In seismic load combinations, live loads are often reduced to 25% (i.e., $0.25 \times LL$), especially for storage loads per IS 1893 and IS 875 Part 2.

Run Analysis and View Results, STAAD will compute internal forces and moments.

View Results and Go to Postprocessing mode.

RESULT AND DISCUSSION

Based on the analysis performed in STAAD Pro. for various values of rigidity factors following parameters are evaluated.

Table 8:-Base Shear of Structure (kN)							
Sr.	Story	Rigidity Factor –(j)					
No.	Number	100% (fixed)	80%	60%	40%	20%	0% (pinned)
1.	3 Story	909.21	909.21	909.21	909.21	909.21	909.21
2.	6 Story	936.89	936.89	936.89	936.89	936.89	936.89
3.	9 Story	979.69	979.69	979.69	979.69	979.69	979.69
4.	12 Story	1021.18	1021.18	1021.18	1021.18	1021.18	1021.18

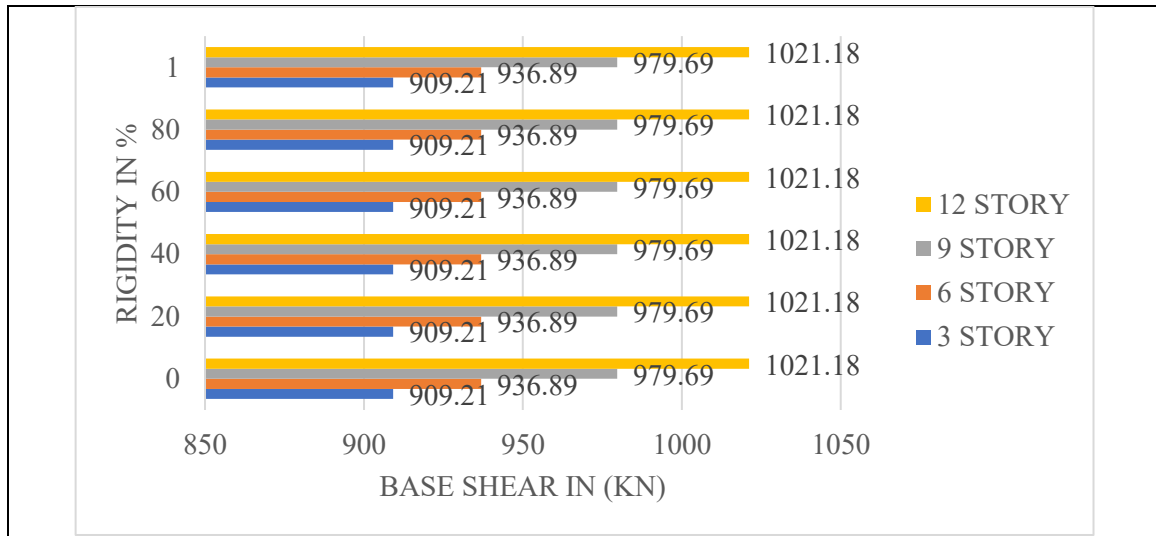


Figure 1:-Base Shear V/s Rigidity Factor-j

From the analysis of results, the following observation is evident: as the number of storeys increases, the base shear also increases. This is because base shear (V_B) is directly proportional to the seismic weight of the building (W). However, there is no significant effect of rigidity factor (j) on the base shear.

Table 9:-Time Period of Structure (Sec)

Sr. No.	Story Number	Rigidity Factor –(j)					
		100% (fixed)	80%	60%	40%	20%	0% (pinned)
1.	3 Story	0.438	0.438	0.438	0.438	0.438	0.438
2.	6 Story	0.696	0.696	0.696	0.696	0.696	0.696
3.	9 Story	0.925	0.925	0.925	0.925	0.925	0.925
4.	12 Story	1.137	1.137	1.137	1.137	1.137	1.137

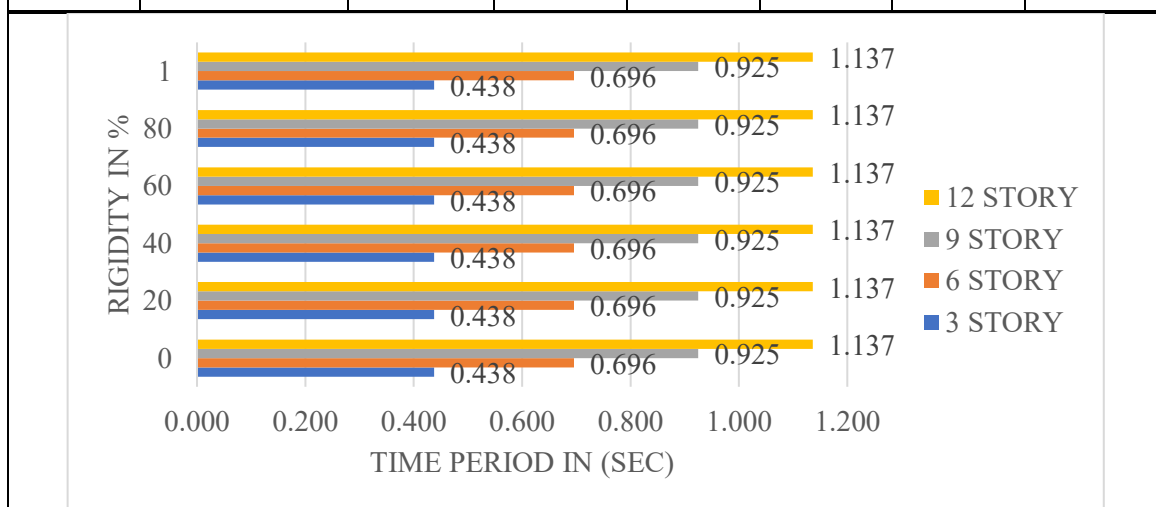


Figure 2:-Time Period V/s Rigidity Factor-j

From the analysis of results, the following observation is evident: as the number of storeys increases, the time also increases. This is because time period (T) is depended upon the height of the building (h), However there is no significant effect of rigidity factor (j) on the time period.

Table 10:-Maximum Bending Moment at Beam- M_z (kN-m)

Sr.	Story	Rigidity Factor –(j)					
No.	Number	100% (fixed)	80%	60%	40%	20%	0% (pinned)
1.	3 Story	135.05	123.64	113.54	135.18	182.29	364.11
2.	6 Story	136.78	125.73	118.32	144.29	201.43	623.16
3.	9 Story	137.87	127.05	121.95	149.48	212.79	919.24
4.	12 Story	91.58	130.59	126.51	154.01	221.93	1242.56
Remak		Maximum moment at beam			Maximum moment at support		

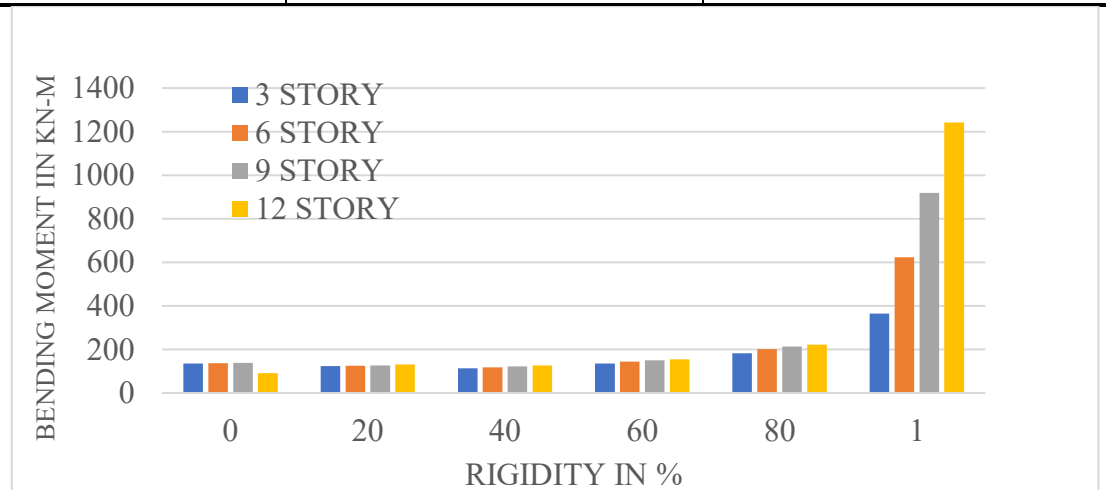


Figure 3:-Maximum Bending V/s Rigidity Factor-j

When the rigidity factor is between 100% to 60%, the connections are more flexible (semi-rigid), allowing the beam ends to rotate more freely. As a result, maximum bending moment occurs at the mid-span of the beam because the supports cannot resist much moment. When the rigidity factor is between 40% to 0%, the connections become stiffer or nearly fixed, restricting beam rotation at the supports. This causes the maximum bending moment to shift toward the supports, since more moment is transferred there due to higher fixity.

Table 11:-Top Story Displacement-mm (3-Story)

Sr.	Story Height(m)	Rigidity Factor –(j)					
No.		100% (fixed)	80%	60%	40%	20%	0% (pinned)
S3	9m	13.995	17.188	22.316	31.365	51.09	136.929
S2.	6m	10.267	12.436	15.801	21.486	33.246	81.65
S1	3m	5.425	6.403	7.927	10.395	15.231	33.998
GF.	0	0.882	1.022	1.225	1.553	2.158	4.367

Table 11 shows the variation of top-storey displacement (in mm) for a 3-storey structure, ranging from 13.995 mm (for 0 % rigidity, i.e., fully restrained) to 136.929 mm (for 100 % rigidity, i.e., pinned). According to IS 456:2000, under the lateral sway at the top of a building must not exceed $H/500$ (which equals 18 mm here). Therefore, only the 20% (17.188mm) and 0 % (13.995mm) rigid case falls within the permissible limit.

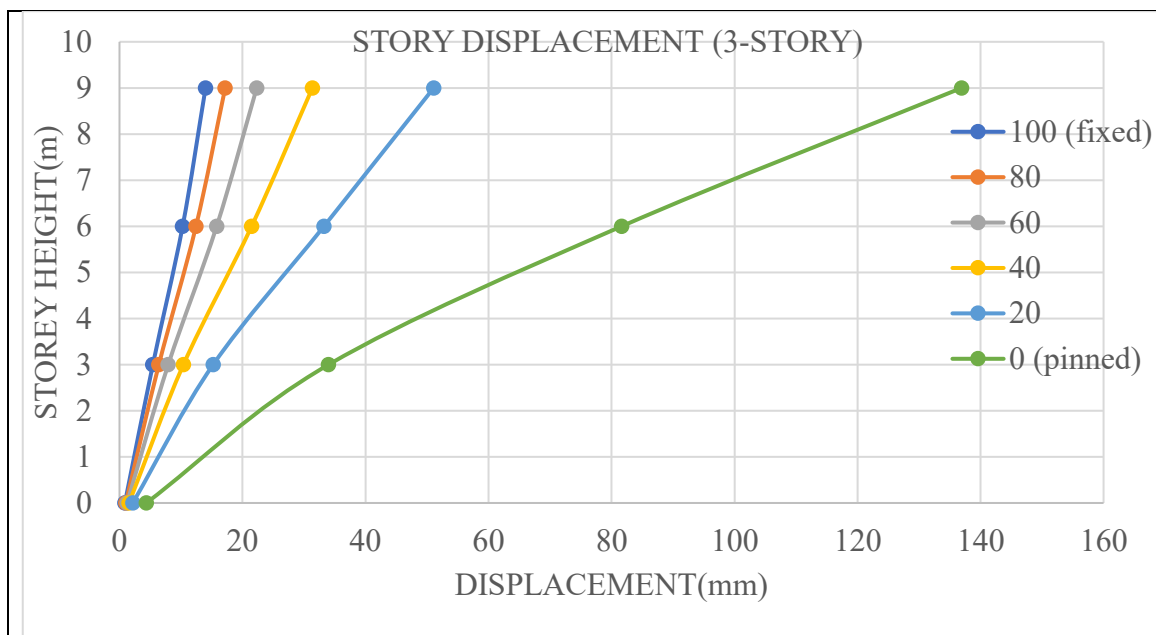


Figure 4:-Displacement V/s Story Height for 3-Story

Table 12:-Top Story Displacement-mm (6- Story)

Sr.	Story Height (m)	Rigidity Factor –(j)					
No.		100% (fixed)	80%	60%	40%	20%	0% (pinned)
S6	18m	29.588	36.812	49.596	73.129	132.096	784.326
S5	15m	26.829	33.499	44.4	64.568	113.232	613.669
S4	12m	22.373	28.274	37.24	53.407	90.982	447.851
S3	9m	17.549	21.687	28.292	39.934	65.983	294.25
S2.	6m	11.713	14.313	18.382	25.371	40.4	162.366
S1	3m	5.789	6.931	8.645	11.552	17.548	62.989
GF.	0	0.914	1.07	1.288	1.659	2.389	7.57

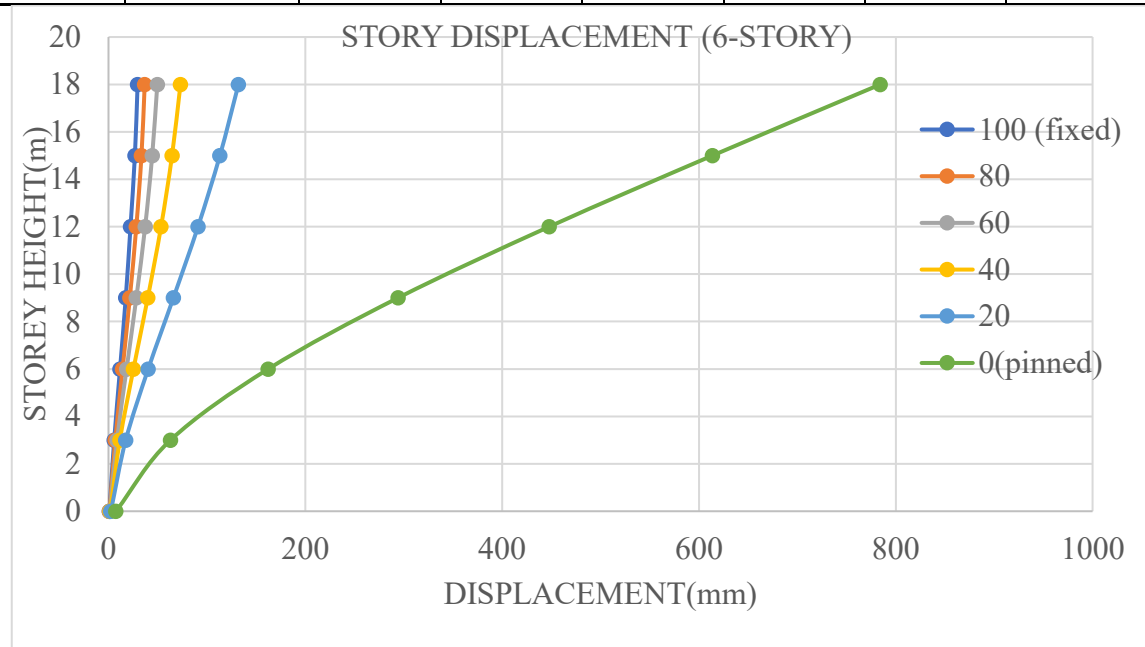


Figure 5:-Displacement V/s Story Height for 6-Story

Table 12 shows the variation of top-storey displacement (in mm) for a 6-storey structure, ranging from 29.588 mm (for 0 % rigidity, i.e., fully restrained) to 784.326mm (for 100 % rigidity, i.e., pinned). According to IS 456:2000, under the lateral sway at the top of a building must not exceed $H/500$ (which equals 36 mm here). Therefore, only the 20% (36.812mm) and 0 % (29.588mm) rigid case falls within the permissible limit.

Table 13:-Top Story Displacement-mm (9- Story)

Sr.	Story Height (m)	Rigidity Factor –(j)					
No.		100% (fixed)	80%	60%	40%	20%	0% (pinned)
S9	27m	46.661	58.702	79.107	118.566	223.412	2440.87
S8	24m	44.24	55.627	74.771	111.382	206.592	2079.11
S7	21m	40.847	51.297	68.76	101.842	186.176	1721.02
S6	18m	36.34	45.571	60.981	89.732	161.776	1372.47
S5	15m	30.983	38.771	51.628	75.514	133.996	1041.38
S4	12m	25.045	31.21	41.304	59.797	103.988	737.231
S3	9m	18.728	23.176	30.366	43.286	73.267	470.615
S2.	6m	12.258	14.984	19.313	26.894	43.878	252.849
S1	3m	5.992	7.143	8.967	12.07	18.754	95.715
GF.	0	0.941	1.097	1.326	1.717	2.523	11.223

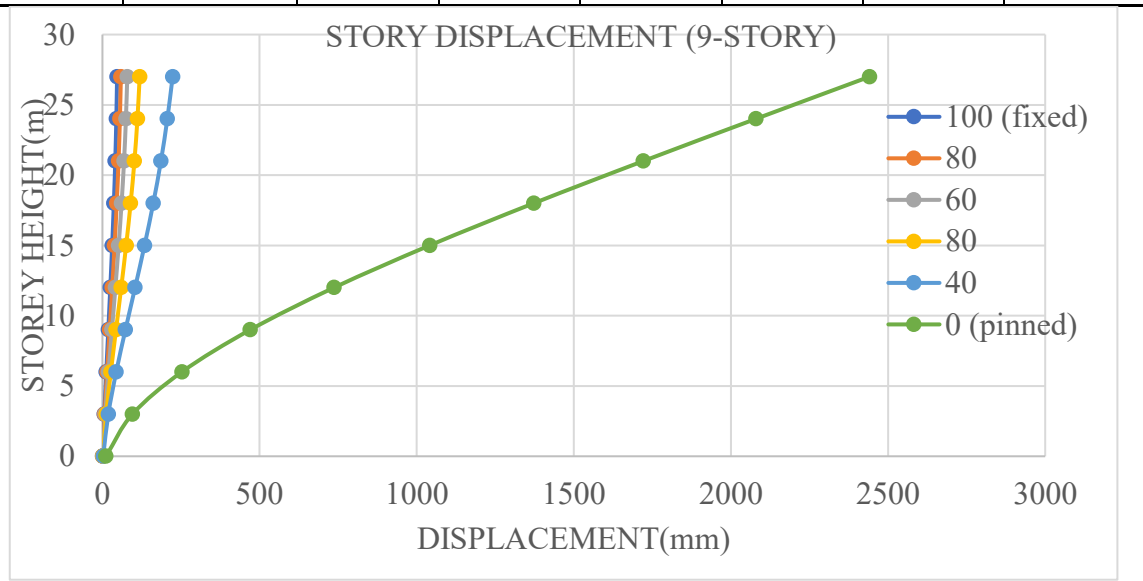


Figure 6:-Displacement V/s Story Height for 9-Story

Table 13 shows the variation of top-storey displacement (in mm) for a 9-storey structure, ranging from 46.661 mm (for 0 % rigidity, i.e., fully restrained) to 2440.87mm (for 100 % rigidity, i.e., pinned). According to IS 456:2000, under the lateral sway at the top of a building must not exceed $H/500$ (which equals 54 mm here). Therefore, only the 20% (58.702mm) and 0%(46.661mm) rigid case falls within the permissible limit.

Table 14:-Top Story Displacement-mm (12- Story)

Sr.	Story Height (m)	Rigidity Factor –(j)					
No.		100% (fixed)	80%	60%	40%	20%	0% (pinned)
S12	36m	65.32	82.13	110.88	167.39	322.82	5667.48
S11	33m	63.04	79.31	106.99	161.05	307.88	5036.24
S10	30m	59.99	75.45	101.69	152.67	289.61	4402.34
S9	27m	55.92	70.34	94.71	141.88	267.34	3776.39
S8	24m	51.01	64.14	86.27	128.94	241.25	3165.20
S7	21m	45.45	57.09	76.66	114.12	211.87	2577.19
S6	18m	39.37	49.38	66.15	98.05	179.91	2021.99
S5	15m	32.93	41.21	54.97	80.89	146.21	1510.23
S4	12m	26.26	32.71	43.36	63.12	111.77	1053.26
S3	9m	19.45	24.06	31.57	45.22	77.85	662.95
S2.	6m	12.64	15.46	19.95	27.90	46.23	351.53
S1	3m	6.11	7.34	9.22	12.46	19.64	131.43
GF.	0	0.95	1.13	1.36	1.77	2.64	15.22

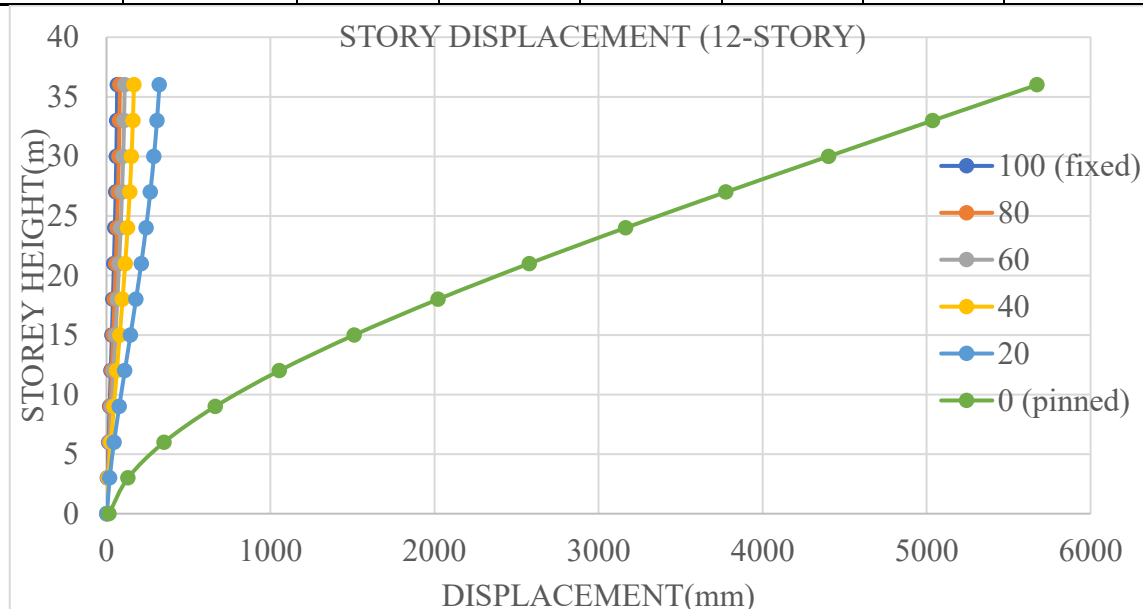


Figure 7:-Displacement V/s Story Height for 12-Story

Table 14 shows the variation of top-storey displacement (in mm) for a 12-storey structure, ranging from 65.32 mm (for 0 % rigidity, i.e., fully restrained) to 5667.48mm (for 100 % rigidity, i.e., pinned). According to IS 456:2000, under the lateral sway at the top of a

building must not exceed $H/500$ (which equals 72 mm here). Therefore, only the 0 % (46.661mm) rigid case falls within the permissible limit.

Table 15:-Inter Story Drift-% (3-Story)

Sr.	Story	Rigidity Factor –(j)					
No.	Height (m)	100% (fixed)	80%	60%	40%	20%	0% (pinned)
S3	9m	0.12427	0.1584	0.21717	0.3293	0.5948	1.84263
S2.	6m	0.1614	0.2011	0.26247	0.3697	0.6005	1.5884
S1	3m	0.15143	0.17937	0.2234	0.29473	0.43577	0.9877
GF.	0	0.0294	0.03407	0.04083	0.05177	0.07193	0.14557

The results from Table 20 show that the storey drift for the 3-storey steel framed structure varies from 0.12% (at 0% rigidity, fully restrained) to 1.84% (at 100% rigidity, pinned). As per IS 1893 (Part 1), the maximum allowable inter-storey drift is 0.004 times the storey height (i.e., 0.4%). Therefore, all the observed drift values for rigidity factors ranging from 20% to 100% are within the permissible limits, indicating that the structure remains safe and stable under seismic loading conditions.

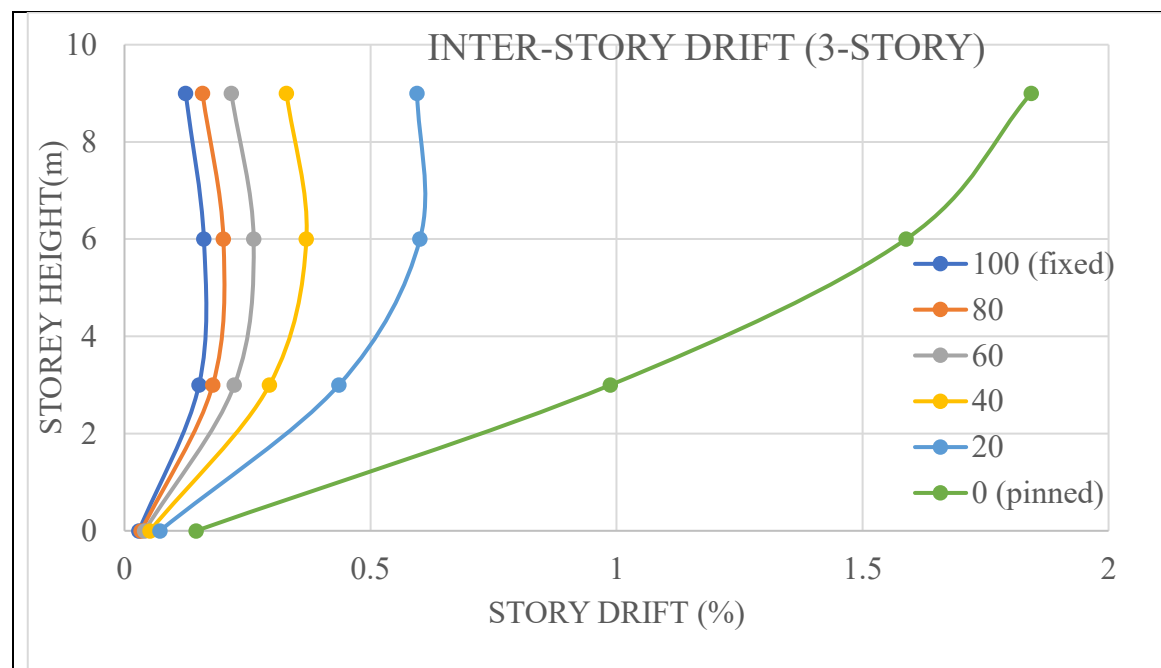


Figure 8:-Story Drift V/s Story Height for 3-Story

Table 16:-Inter Story Drift-% (6-Story)

Sr.	Story Height (m)	Rigidity Factor –(j)					
No.		100% (fixed)	80%	60%	40%	20%	0% (pinned)
S6	18m	0.09197	0.11043	0.1732	0.28537	0.6288	5.68857
S5	15m	0.14853	0.17417	0.23867	0.37203	0.74167	5.52727
S4	12m	0.1608	0.21957	0.29827	0.4491	0.8333	5.12003
S3	9m	0.19453	0.2458	0.33033	0.48543	0.85277	4.39613
S2.	6m	0.19747	0.24607	0.32457	0.46063	0.76173	3.31257
S1	3m	0.1625	0.19537	0.24523	0.32977	0.5053	1.8473
GF.	0	0.03047	0.03567	0.04293	0.0553	0.07963	0.25233

The results from Table 16 show that the storey drift for the 6-storey steel framed structure varies from 0.09% (at 0% rigidity, fully restrained) to 5.68% (at 100% rigidity, pinned). As per IS 1893 (Part 1), the maximum allowable inter-storey drift is 0.004 times the storey height (i.e., 0.4%). Therefore, all the observed drift values for rigidity factors ranging from 20% to 100% are within the permissible limits, indicating that the structure remains safe and stable under seismic loading conditions.

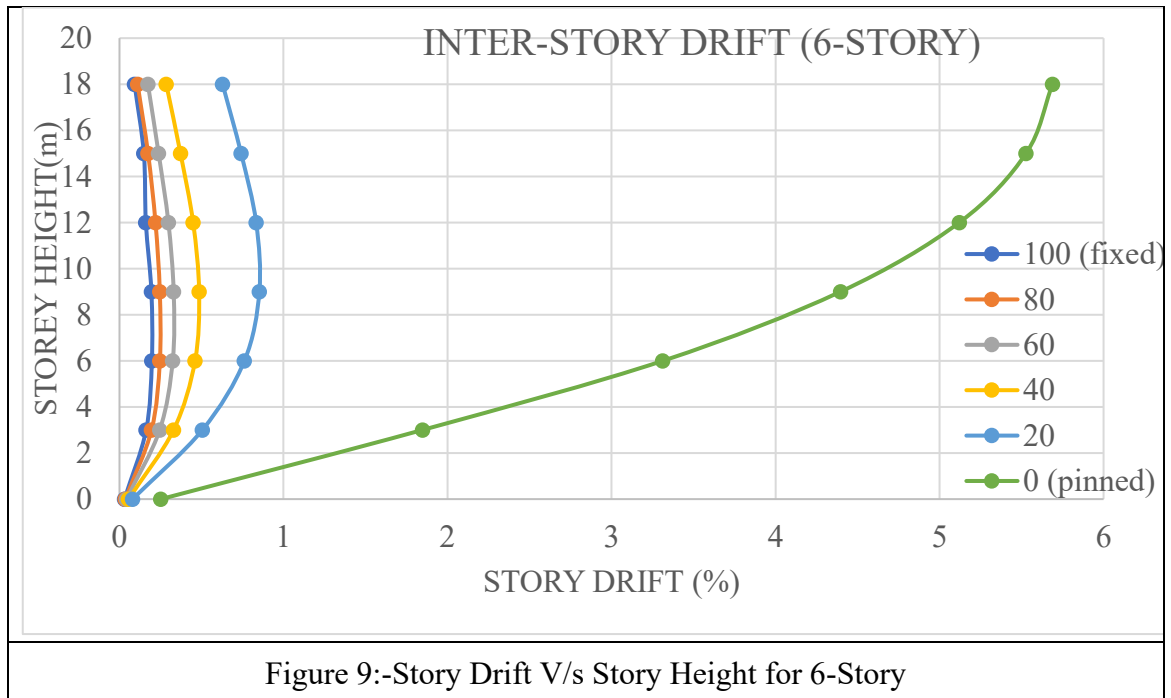


Table 17:-Inter Story Drift-% (9-Story)

Sr.	Story Height (m)	Rigidity Factor –(j)					
No.		100% (fixed)	80%	60%	40%	20%	0% (pinned)
S9	27m	0.0807	0.1025	0.14453	0.23947	0.56067	12.0589
S8	24m	0.1131	0.14433	0.20037	0.318	0.68053	11.9363
S7	21m	0.15023	0.19087	0.2593	0.40367	0.81333	11.6184
S6	18m	0.17857	0.22667	0.31177	0.47393	0.926	11.0363
S5	15m	0.19793	0.25203	0.34413	0.5239	1.00027	10.1382
S4	12m	0.21057	0.2678	0.3646	0.55037	1.02403	8.8872
S3	9m	0.21567	0.27307	0.36843	0.5464	0.97963	7.25887
S2.	6m	0.20887	0.26137	0.34487	0.49413	0.83747	5.2378
S1	3m	0.16837	0.20153	0.2547	0.3451	0.54103	2.8164
GF.	0	0.03137	0.03657	0.0442	0.05723	0.0841	0.3741

The results from Table 22 show that the storey drift for the 9-storey steel framed structure varies from 0.087% (at 0% rigidity, fully restrained) to 12.05% (at 100% rigidity, pinned), which is significantly exceeded with permissible limit. As per IS 1893 (Part 1), the maximum allowable inter-storey drift is 0.004 times the storey height (i.e., 0.4%). Therefore, all the observed drift values for rigidity factors ranging from 20% to 80% are within the permissible limits, indicating that the structure remains safe and stable under seismic loading conditions.

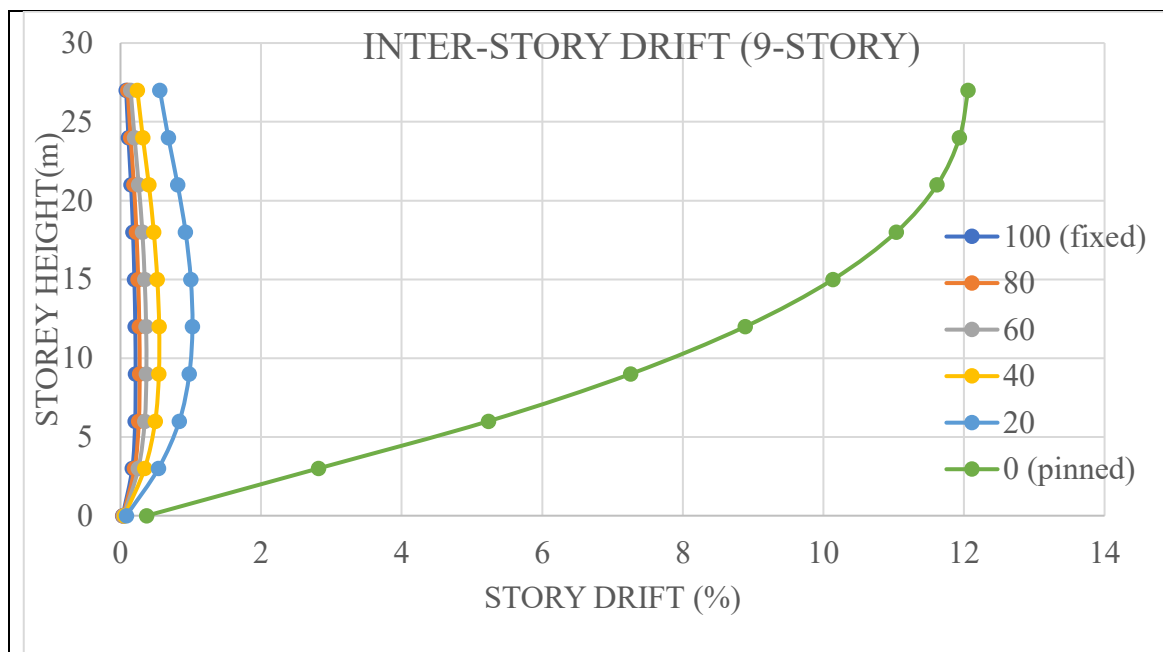
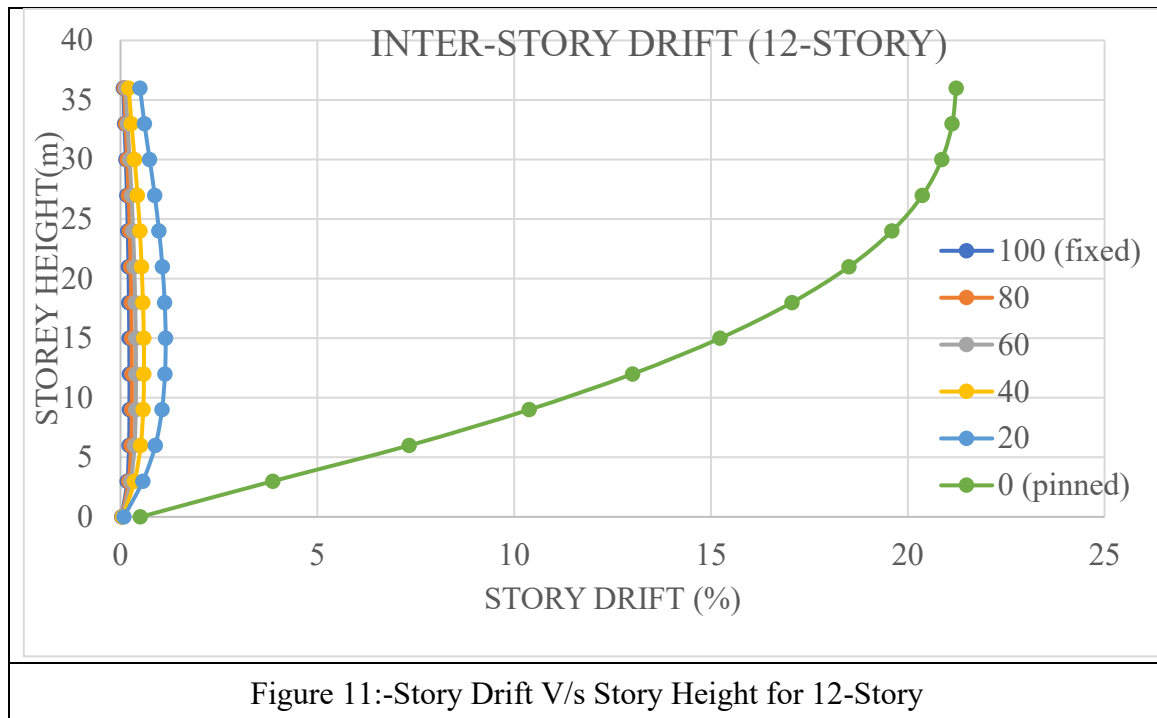


Figure 10:-Story Drift V/s Story Height for 9-Story

Table 18:-Inter Story Drift-% (12-Story)							
Sr.	Story Height (m)	Rigidity Factor –(j)					
No.		100% (fixed)	80%	60%	40%	20%	0% (pinned)
S12	36m	0.07593	0.09407	0.1296	0.21117	0.49823	21.2304
S11	33m	0.10147	0.12877	0.17647	0.2793	0.60897	21.1299
S10	30m	0.1358	0.1703	0.23287	0.3596	0.74227	20.8652
S9	27m	0.1637	0.2066	0.28117	0.43153	0.8695	20.3728
S8	24m	0.1853	0.23503	0.3203	0.49394	0.9793	19.6005
S7	21m	0.07593	0.2568	0.3506	0.53559	1.06537	18.5065
S6	18m	0.20267	0.2726	0.37263	0.57197	1.12333	17.0586
S5	15m	0.2146	0.28307	0.38697	0.59233	1.14807	15.2324
S4	12m	0.22257	0.28833	0.39303	0.59673	1.13063	13.0103
S3	9m	0.22683	0.28677	0.3873	0.57747	1.05407	10.3809
S2.	6m	0.22713	0.27087	0.35743	0.5145	0.88637	7.3365
S1	3m	0.21743	0.207	0.262	0.35647	0.56677	3.87383
GF.	0	0.17223	0.0375	0.04547	0.05893	0.08787	0.50717

The results from Table 23 show that the storey drift for the 12-storey steel framed structure varies from 0.075% (at 0% rigidity, fully restrained) to 21.23% (at 100% rigidity, pinned), which is significantly exceeded with permissible limit. As per IS 1893 (Part 1), the maximum allowable inter-storey drift is 0.004 times the storey height (i.e., 0.4%). Therefore, all the observed drift values for rigidity factors ranging from 20% to 80% are within the permissible limits, indicating that the structure remains safe and stable under seismic loading conditions.



CONCLUSION

Connections are very important factor of any structure. Especially in steel structures. The connections defined the overall ductility of the structures. Theoretically all connections are categorised into two categories, i) Fully rigid ii) fully pinned

However, practically all connections are partially rigid connection. Thus, to understand the performance of the structures allowing this effect becomes necessary. In this work, this investigation is performed. During the investigation very conclusions are made. Some of the important conclusions are summarised as below,

- The Bending moment at the beam end increases as rigidity decreases, this can be up to 62.90% compared to fully rigid connections.
- Connections is not the significant reduction factor on the Base shear of the structure.
- The time period of the structure is not affected by the rigidity of the beam–column connection. The time period increases with the storey height of the building.
- Top storey displacement increases linearly with a decrease in the rigidity of beam–column connections. This can be up to 98.85% compared to fully rigid connection.
- Inter-storey drift increases with an decrease in the rigidity of beam–column connections. This can be up to 99.64% compared to fully rigid connection.
- The equivalent load calculated by the software is 940 kN, while our manual calculation yields 897.609 kN—this difference of approximately 5% is acceptable.

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