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A Study on Energy Dissipation in RCC Beam Column Joints Using Semi Rigid Connection

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Abstract: Beam column joint failure has been linked to the partial or whole collapse of reinforced structures, according to information gathered during seismic reconnaissance. Under seismic stress, the failure of a single beam has only a limited impact, but the failure of a beam-column junction might lead to the collapse of the whole structure. The resulting economic damage and human casualties are substantial. Hence, keeping the beam and column from coming apart is crucial. Recent earthquakes have caused significant damage to important structures, and this is mostly attributable to a lack of ductility in the beam-column junction. The ductility performance was enhanced by the design of a specific moment resistant frame. There is less moment transmission and greater yielding under lateral load in steel and pre-fabricated constructions that make use of semi-rigid connections. In this project, we use an analytical approach to examine the impact of semi-rigid connections in the RCC columns and beams of reinforced concrete high-rises.

Keywords: RCC Beam, Earthquakes, Column Joint, Concrete, Reinforced.

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1. Introduction

Strong earthquakes often cause inelastic deformations in reinforced concrete framed buildings. Plastic hinges develop in the beams rather than the columns during an earthquake and disperse the quake's energy. As the failure of the columns as well as the beam-column connections may compromise the strength and rigidity of a significant component of the structure, the design strategies should prioritize these areas. [1-2]

In order to ensure that the design strengths of earthquake-resistant columns and connections are maintained during many inelastic deformation cycles, it is crucial to design and specify these elements properly. Joint shear failure, is common in older, non-ductile RC moment-resisting frames during earthquakes. How buildings have reacted to recent earthquakes. Evidence from recent earthquakes suggests that excessive shear stress, lack of concrete confinement, and poor ductility were primary causes of beam column joint failures.[3-4]

When shoddy craftsmanship and risky design are used, the beam column junction becomes the most important section of the building. The flexural members should be able to release the energy they absorb under lateral load so that the structure can withstand the load. The design relies heavily on the ductility of the frame to withstand lateral loads. It's important that the beam-column junction can take a beating until both the beam and the column have taken their maximum load. A structural system's BC joints may be classified into six distinct

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categories: interior, exterior, corner, roof, roof external, and roof corner. A beam column junction is a unique component of a structural framework due to the lower capacity caused by the use of constituent material with low resistance due to low strength. Extreme stress exerted to joints during an earthquake may cause catastrophic injury.[5-7]

Shear pressures will cause the development of compression and tension strains along the joint's diagonal axis. Cracks appear in a diagonal pattern across the center of the concrete as a consequence of tension tensions. The source of shear resistance has changed significantly at this point. As a result of these internal pressures, a diagonal strut forms in the concrete. Beam and column bond forces delivered to the joint core need a truss mechanism. [8-9]

In order to avoid shear failure due to diagonal stress, shear reinforcement inside the horizontal and vertical directions must be applied along the failure plane. Bond stresses in the outer layer's column bars are seen moving into the joint core or being distributed equally throughout the beam's depth. Because of its strength under compression and weakness under tension, concrete's shear resistance mechanism relies heavily on a diagonal compression field. [10]

2. Material and Methods

In this study, we use the software package SAP 2000 to conduct nonlinear static and time-series analysis to determine the seismic performance of a standard four- and seven-story reinforced Concrete (RC) structure. Semi-rigid connections are created by assigning a percentage of fixity to a joint, such as 10%, 20%, 30%, 40%, 50%, 60%, 70%, 80%, or 90%, and afterwards contrasting them with stiff connections. Non-linear static and dynamic analyses are used to compare the efficiency of frames with varying degrees of semi-rigid connection fixity.

- **Description Of Study Frame**

The sum of the dead & live loads operating on the frame may be calculated by measuring its tributary area. The masonry fill is supposed to distribute its own self-weight equally throughout the frame's beams. Using the software program SAP 2000, a linear static analysis was performed to calculate the axial loads, bending moments, and shear forces acting on each part of the frame. For the analysis, we adopt a plane frame analytical model in which the beams and columns are two-nodded frame components, each with six degrees of freedom. We suppose that the Poisson's ratio is 0.2 and the elastic moduli of masonry and concrete are 22360 MPa & 3200 MPa, respectively. Several loads were applied to the frame, and their distribution is shown in Figure.

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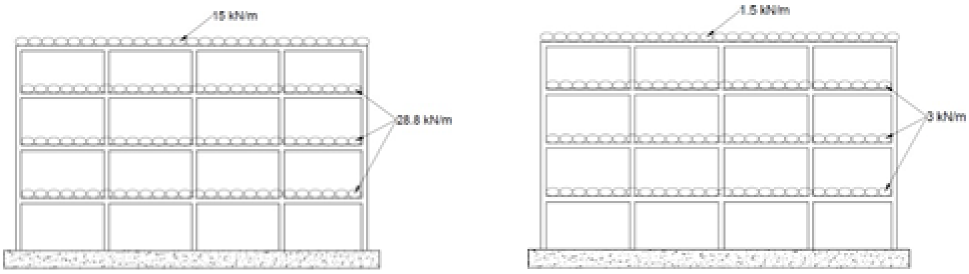

(a) (b)

Figure 1: Specifics of the loads applied on the study's framework (a) Dead weight (b) Load in use

The earthquake load is spread over the building's height, while the gravity loads are dispersed on the beams at different floor levels. Seismic loads are shown distributed throughout the building in the figure.

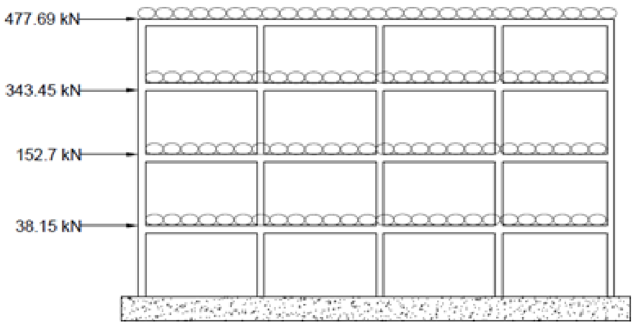

Figure2: Seismic load distribution in the research structure

1. Design of Frame Members

The frame's members are optimized for the critical load combinations, where the axial load, bending moment, and shear force all reach their maximum values, and the other load combinations are used to verify their sufficiency. In this case, we'll pretend that the frame members were built using M20 concrete, which has a required characteristic compressive strength of 20 MPa, and that they were reinforced with TMT bars, which have a required yield strength of 415 MPa. It is assumed that the beam sections are also 300mm x 300mm in size, and that all columns used in the analysis will have square sections of the same dimensions. Shear reinforcement in the column and beam sections was designed in accordance with the requirements of IS 456:2000.Directions of +X, -X, +Y, and -Y earthquake load were taken into account. All 13 load permutations are examined by SAP 2000 due to the sheer volume of data involved.

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Table1: Several Stackings of Responsibility

Combination	Description
1	1.2(DL+LL-EQX)
2	1.5DL+1.5LL
3	1.2(DL+LL-EQY)
4	1.2(DL+LL+EQX)
5	1.2(DL+LL+EQY)
6	1.5(DL-EQX)
7	1.5(DL+EQX)
8	0.9DL+1.5EQX
9	1.5(DL+EQY)
10	0.9DL+1.5EQY
11	1.5(DL-EQY)
12	0.9DL-1.5EQX
13	0.9DL-1.5EQY

A summary of the analysis's force resultants for the critical beam under various load scenarios and load combinations, including the highest values to be employed in design, can be found in Table. Due to the Y-parallel orientation of the considered beam, the 13 load choices in Table are reduced to 7 and the resulting forces are shown in Table. Figure depicts the specifics of the beam's measurements and reinforcing.

Table 2: Critical beam member forces for a variety of load configurations

Loadco mb.	Leftend			Centre			Rightend		
	P (kN)	M (kNm)	V (kN)	P (kN)	M (kNm	V (kN)	P (kN)	M (kNm)	V (kN)
5	271.5	-133.7	-149	271.5	86.7	0.5	271.5	-136.7	150
1	-152.9	-145.3	-179.9	-152.9	111.1	7	-152.9	-187.1	195
9	379.8	-134	-147.2	379.8	84.9	-0.4	379.8	-131.9	146.5

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4	-532.1	-98.8	-138.8	-366	75.5	5.4	-532.1	-162.7	160.3
13	428.8	-47.8	-76.2	428.8	49.8	-2.8	428.8	-72.6	85.3
8	-624.7	-90.4	-134.4	-624.7	90.5	12.5	-624.7	-164.5	159.4
12	-575.7	-45.5	-78.1	-575.7	55.4	10.1	-575.5	-105.2	98.2

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The bigger of the two reinforcements planned for the beam-column junction should be used in the column section. This is analogous to the procedure used when designing reinforcements for continuous beams at their points of support.

Computer analysis provides the final moments and shears. Considerations for the design phase:

(a) The long column effect extra moment required by article 39.7 of IS 456:2000.

(b) Article 25.4 of IS 456:2000 specifies the moments owing to minimal eccentricity.

Both shear and moment forces act in two directions on each column. The longitudinal reinforcements conform to IS 456:2000 in terms of their capacity to withstand axial force and biaxial moment. The study summarizes the force results for each load combination in Table. Figure displays the exact column size and reinforcements.

Table 3: Critical column member forces for various load configurations

Loadcomb.	**Leftend**			**Centre**			**Rightend**		
	P (kN)	**M (kNm**	**V (kN)**	**P (kN)**	**M (kNm**	**V (kN)**	**P (kN)**	**M (kNm)**	**V (kN)**
1	-2302	508.3	321.6	-2310	-50.4	321.6	-2318	-609.2	321.6
2	-2911	-0.7	0.2	-2921	-1.1	0.2	-2932	-1.5	0.2
3	-2302	508.3	321.6	-2310	-50.4	321.6	-2318	-609.2	321.6
4	-2356	-509.5	-321.2	-2364	48.6	-321.2	-2373	606.7	-321.2
5	-2356	-509.5	-321.2	-2364	48.6	-321.2	-2373	606.7	-321.2
6	-2219	-636.7	-401.6	-2230	61.1	-401.6	-2240	758.8	-401.6
7	-2219	-636.7	-401.6	-2230	61.1	-401.6	-2240	758.8	-401.6
8	-1477	-577.1	-377.8	-1483	79.3	-377.8	-1490	735.7	-377.8
9	-2151	635.5	401.9	-2161	-62.8	401.9	-2170	-761.1	401.9

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10	-2151	635.5	401.9	-2161	-62.8	401.9	-2170	-761.1	401.9
11	-1573	558.1	365.5	-1580	-76.9	365.5	-1586	-712	365.5
12	-1573	558.1	365.5	-1580	-76.9	365.5	-1586	-712	365.5
13	-1477	-577.1	-377.8	-1483	79.3	-377.8	-1490	735.7	-377.8

Linear and nonlinear analysis are the two most common methods used to assess a building's resistance to earthquakes. Static analysis or dynamic analysis is used in each process. The existence of anomalies and the amount & distribution of inelastic demands on different parts of the lateral-load-resisting system are both ascertained by linear analysis. In the case of irregularly shaped structures, when a nonlinear technique is required to assess the bearing capacity of building elements under static and dynamic loading circumstances, nonlinear analysis is prohibited. In an ideal world, a collection of ground movements indicating source-site features of the area would undergo a nonlinear time history analysis.

Nonlinear static analysis or integration time-history studies are used to verify the stability method for the frame when the percentage of stiffness in the connection is adjusted from 10% to 90%. The main purpose of the nonlinear static analysis was to find out how much of a lateral load the frame could take before it failed. Two-stage pushover analysis is performed to account for material and geometric nonlinearities. The first step involves testing the effectiveness of normal & link column frames while they are subjected to solely gravity loads from dead and live weights. Seismic performance of the non-ductile and reinforced frames is compared using direct-integration time-history analysis with recorded ground movements. The software takes the element masses & applied loads and automatically calculates the total mass at each floor level of the frames. So, the technique for establishing the storey masses does not take into account the decrease in live load. In addition, a viscous damping ratio comparable to 5% was taken into account.

2. Modelling of Semirigid connection

The columns and beams of the frame are modeled as two-noded frame elements, each having six-degrees of freedom, depending on their centre-line dimensions, three translations, and three rotations. Nevertheless, due to plane-frame analysis, only three degrees of freedom are really usable in the software. It is also thought that the columns supporting the bottom floor are fastened there. The stiffness of the member is decreased by achieving partial fixity by releasing a predetermined fraction of moments or shear at the supports.

Eleven RC building models, ranging in height from four to seven stories, were created in SAP 2000. There was a wide range of rigidity and partial fixity in the models, from 0% to 100%. Via either a decrease in member stiffness or an increase in joint rotational stiffness, the moment is dissipated at the supports. Then, the stability technique is utilized to determine the best location for the semi-rigid connection. Table illustrate and describe in full the different 4- and 7-story building models considered.

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Table4: Intricacies of the Scale Model

	Model2	Model1
Bay length in x-direction	6m	5m
Number of Floors	G+6	G+3
Size of the column	0.5×0.5m	0.3×0.3m
Floor Height	3.5m	3m
Depth of the slab	0.12m	0.12m
Size of the beam	0.3×0.4m	0.3×0.3m

For both nonlinear and linear studies, the section and material attributes of each frame member are recorded. Modulus of Elasticity of Concrete Ec = 22360MPa is one of the fundamental material parameters used for linear analysis. Steel has a modulus of elasticity of Es = 210000MPa, concrete has a characteristic strength of fck = 20MPa, and the Poisson's ratio is. Thermal expansion coefficient, The bending yield stress of reinforcement in a cube of concrete is equal to 20 MPa. Nonlinear studies in SAP 2000 need not only the assignment of elastic characteristics, but also the assignment of plastic hinge positions and attributes.

i.Plastic Hinge Properties

The program incorporates nonlinear static and dynamic analysis. For both yielding or post-yielding behavior of frame members, the SAP2000 lumped plasticity model defines discrete plastic hinges at preset positions along the members. Hinge properties are often described using either a force or displacement model. After the yield has been achieved, individuals are expected to perform in a certain way. Beams and columns of present resistant frames exhibit deformation-controlled action during flexural loading, whereas force-controlled action occurs during shear and axial loading. For non-linear behavior, beams with both ends fixed will be allocated moment M3 & shear V2 at both ends, whereas beams with one fixed end and one hinged end will have moment M3 and shear V2 assigned to the fixed end and just shear V2 assigned to the hinged end. Interacting axial moments and forces, or P-M2-M3, were applied to the columns.

ii.Selected Ground Motions

In the current work, we used the recorded ground movements from Northridge as the basis excitations for a nonlinear direct-integration time-history analysis. These earthquakes were chosen because their PGA values were close to 0.36 g, making them part of India's highest seismic i.e. Zone-V according to IS 1893: 2002. In addition, hypocentral distances from of the source are within 30 km of the site, showing a close source-site effect, as shown by the recorded ground movements. The site parameters and earthquake data for the chosen ground

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movements evaluated for linear time history analysis are summarized in Table. Acceleration-time histories of chosen ground movements are shown in Figure.

Table 5: Location information and earthquake data for a subset of tremors

S.No	Earthquake	PGA(g)	Year	Soil	Magnitude (Mw)	Epicentre (km)
1	Northridge	0.37	1994	Alluvium	6.7	27.4

The IS 1893:2002 rock site response spectrum is used as the research's design spectrum. Since the average spectral acceleration of the chosen earthquakes at the fundamental period of the research frame was similar to the design spectrum, and because the chosen earthquakes reflected a variety of source types and site characteristics, the ground movements were not scaled.

3. Results

Pushover analysis & nonlinear time-history analysis are used to analyze the seismic performance of a non-ductile RC frame.

- **Time-History Analysis**

Nonlinear stopwatch time analysis in SAP 2000 is used to assess the frame's behavior under dynamic loading conditions. Because entire damping that couples with modes of vibration is taken into account, direct-integration time-history analysis has several advantages. In order to guarantee the stability and correctness of the solution, smaller time steps are used in the analysis, with the smallest time step being 0.02 s for the specified ground motion. Direct-integration time-history analysis is carried out using the Hilber-Hughes-Taylor (HHT) technique.

- **Peak Displacement**

Time series analysis shows that the Northridge earthquake caused the models of 4- and 7-story buildings to move the most at their highest points. Roof displacement time histories for a 4-story and 7-story structure in reaction to the Northridge ground motion are shown in Figures. For moment releases between 50% and 90%, the deformation is greater and the connection will behave as a hinge, while for moment releases between 10% and 40%, the displacement is small enough that the connection will be stiff. It is determined that a connection with a 50% moment of release is semi-rigid. The joints of a semi-rigid connection are more pliable, allowing for more lateral deformation. When comparing the peak displacement between the rigid and semi-rigid connections, the semi-rigid connection increases it by 29.3 percent in the 4-story model and by 23.4 percent in the 7-story model.

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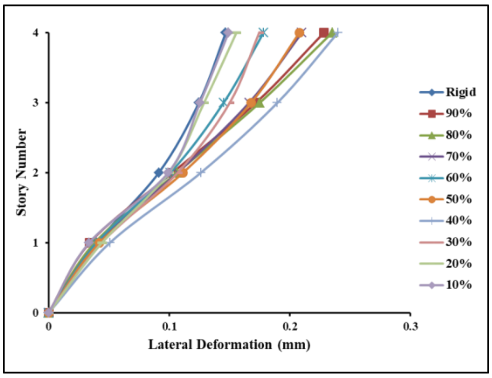

Figure 3: Graph of Lateral Deformation for a Four-Story Building at Various Fixities

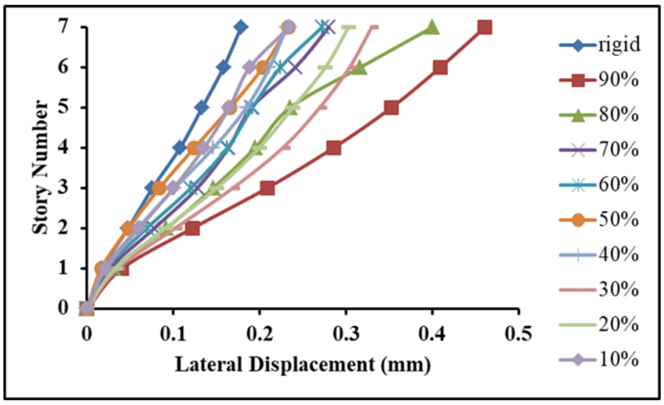

Figure 4: Seven-story building lateral deformation plotted against fixity percentage

The stability method is utilized to enhance the structure's performance. Time-history analysis shows that the rigid frame, semi-rigid, and stability approach (SA) models of the 4-story and 7-story structure both experience a peak displacement during the Northridge earthquake. Time histories of roof displacement for a 4-story and 7-story structure are shown in Figures, respectively. For the Northridge earthquake load scenario for the 4-story frame model, the displacement is effectively reduced by 18.6% for the stability Approach, and for the Northridge earthquake load case for the 7-story structure model, the displacement is effectively reduced by 31.78%.

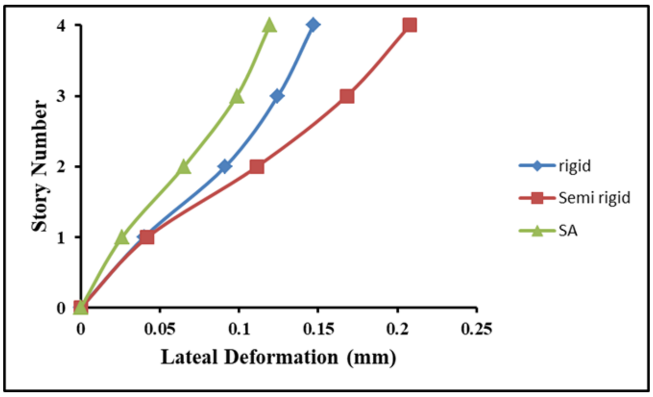

Figure 5: 4-story building's graph of travel time against floor height (Northridge)

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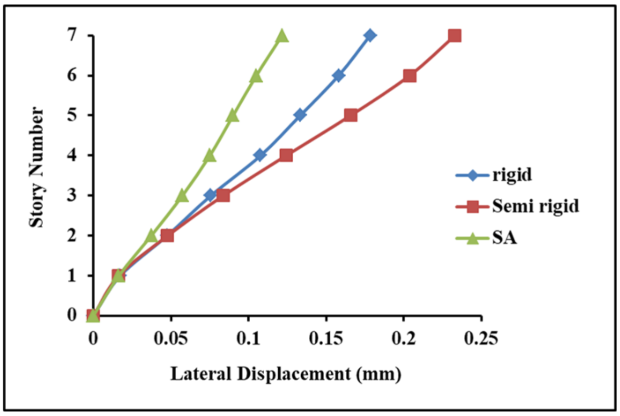

Figure6: Time-distance plot for a 7-story structure

The Northridge earthquake comparison graphs for the three specimens are shown in Figures for 4- and 7-story structures, respectively. The relative stiffness of each storey in the rigid frame, semi-rigid frame, or stability approach frame may be determined from the inter-story displacement. Figure shows that compared to a regular frame or a semi-rigid frame, the stability approach has the least amount of inter-story drift. As can be seen in Figure, the stability strategy benefits from a nearly straight inter-story drift, which increases lateral stiffness across all floors.

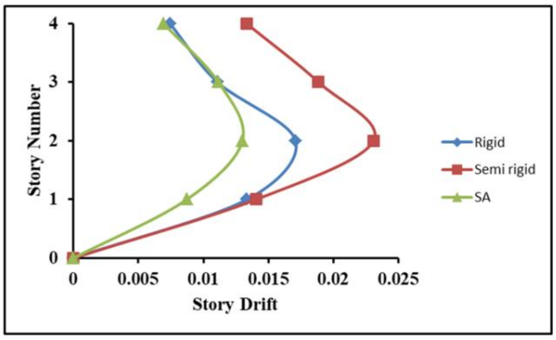

Figure7: Interstoreydrift (Northridge)4-Storey

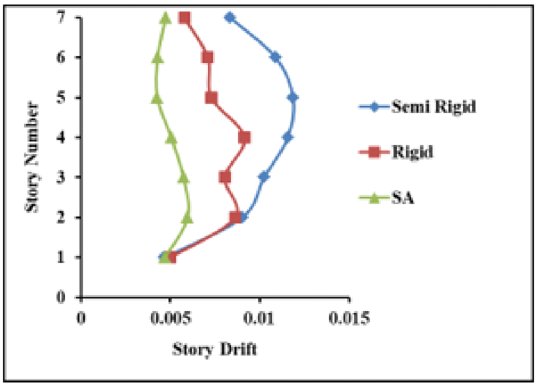

Figure 8: Northridge 7-story inter-floor drift

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- **Pushover Analysis**

ATC-40 (1996) & FEMA 440 provide recommendations for evaluating structural damage (2005). One key goal of performance-based design is the prediction of structural deterioration. Its primary function is to chart the severity of damage (performance goals) for a building, such as its suitability for immediate occupancy, risk to life, and resistance to collapse. In SAP 2000, Figure depicts a generic force-displacement characteristic of a non-degrading beam section .

The Immediate Occupancy (IO) level of performance is the condition after an earthquake that is safe to inhabit, and which virtually preserves the strength and stiffness of the original design.The post-earthquake damage condition, with damaged structural components but a buffer against the commencement of partial or whole collapse, must be specified as the Life Safety Performance Level (LS).Damage to building elements caused by an earthquake is considered to be at the Collapse Prevention Level Of performance (CP) when the building can still bear gravity loads but has no safety margin.

- **Prediction of Structural Damage**

Non-linear static analysis is used to calculate the lateral load bearing capability of the models for the 4-story structure at varying degrees of roof displacement. The lateral force of 107.105 kN, equal to a roof drift of 56 mm, causes the ground-floor columns of a rigid frame to yield flexurally. With a lateral load of 117.89 kN, equivalent to a roof drift of 64 mm, flexural yielding of the ground floor columns was detected for the Semi rigid frame. With a lateral load of 129.124 kN, equivalent to a roof drift of 83 mm, the flexural yielding of a ground storey columns was detected; the maximal lateral strength was obtained at a lateral load of 211.175 kN, resulting in a roof displacement of 206 mm. The damage status of all 4-story building models is shown in Table.

Table6: Condition of 4-story building damaged

Rigid					**SemiRigid**			
Steps	**Grades**	**Baseshear(kN)**	**Disp(m)**		**Steps**	**Grades**	**Baseshear(kN)**	**Disp(m)**
2	IO	107.105	0.05659		2	IO	117.819	0.064602
4	LS	176.303	0.122989		4	LS	175.359	0.124544
4	CP	176.303	0.122989		5	CP	203.415	0.17787
Performance pt		0.078	140.665		Performancept		147.206	0.080

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StabilityApproach			
Steps	**Grades**	**Base shear(kN)**	**Disp(m)**
5	CP	211.175	0.206516
4	LS	182.686	0.147063
2	IO	129.124	0.083315
Performancept			160.12

As a result, it is reasonable to conclude that the stability Approach frame raises the shear strength of the foundation. For four-story building models, the Stability Approach frame was found to perform between IO and LS. Stability Approach models outperform rigid and semi-rigid Frame models in terms of performance.Non-linear static analysis was used to calculate the lateral load bearing capability of the 7-story building models for varying degrees of roof displacement. With a lateral load of 336.839 kN, equivalent to a roof drift of 83.9 mm, flexural cracking of the ground story columns was detected for the normal frame. With a lateral load of 394.032 kN, which is equivalent to a roof drift of 162 mm, flexural yielding of the ground floor columns is seen for a semi-rigid structure.

With a lateral load of 370.34 kN, which is equivalent to a roof drift of 121 mm, flexural buckling of the ground floor columns is detected for the Stability Approach in the frame. The damage status of all 7-story building models is shown in Table. The optimal performance for 7-story building models was located between the IO and LS states in the Stability Approach. Stability Approach models outperform rigid frame and semi-rigid frame ones in terms of performance.

Table7:The 7-story building's current condition of damage

Rigid					**SemiRigid**			
Steps	**Grades**	**Baseshear (kN)**	**Disp(m)**		**Steps**	**Grades**	**Baseshear (kN)**	**Disp(m)**
6	CP	493.878	0.37845		3	IO	394.032	0.162133
5	LS	455.526	0.271404		7	CP	518.321	0.466789
3	IO	336.839	0.083959		6	LS	502.308	0.406627

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Performancept	325.274	0.078		Performancept	340.284	0.116

StabilityApproach			
Steps	**Grades**	**Baseshear (kN)**	**Disp(m)**
7	CP	552.907	0.726757
6	LS	501.848	0.407808
3	IO	370.34	0.121326
Performance		390.23	0.142

4. Conclusion

Rigid frames have been shown to be unreliable for predicting structural damage, lateral strength, and storey drift. When the moment at the joint is reduced by half, a semi-rigid connection is seen; this connection has more lateral strength and drift capacity than the rigid one. By introducing semi-rigid connections in strategic locations, it was discovered that the structure's performance might be improved via a more stable design. In terms of roof displacement, for, and semi-rigid connection, the stability technique demonstrated superior performance.

This paper details the steps involved in designing beams and columns for a four-story structure using the IS 1893: 2002 standard, as well as the analytical assessment of three models—a normal frame, a semi-rigid model, and a stability model for a seven-story building. There is an increase in lateral strength, lateral stiffness, drift capacity, and energy dissipation when a semi-rigid connection is used in conjunction with the Stability method.

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