PROGRESSIVE-COLLAPSE'S OF IRREGULAR RC STRUCTURE USING LINEAR STATIC ANALYSIS

Krishna Reddy K[1], Dr. AdanaGouda[2],

1. PG student of Civil Engineering department of Rao Bahadur Y Mahabaleshwarappa Engineering Collage Ballari, 583101, Karnataka, INDIA.
2. Associate professor at Civil Engineering department of Rao Bahadur Y Mahabaleshwarappa Engineering Collage Ballari, 583101, Karnataka, INDIA

Abstract : *Natural disasters like earthquakes and strong winds have increasingly drawn attention to the vulnerability of existing structures. In structural engineering, the concept of progressive collapse—where a localized failure leads to widespread structural breakdown—has become a critical area of study. Such collapses can result in severe damage, injuries, or fatalities, as seen in events like 9/11.This paper presents a comprehensive review of current research on progressive collapse, emphasizing the need for cost-effective and resilient design strategies. It highlights the importance of further investigation into structural behavior under accidental or deliberate loads. Due to the complexity and cost of full-scale testing, researchers often use scaled-down models and advanced numerical methods like the Finite Element Method (FEM) and Discrete Element Method (DEM) to simulate collapse mechanisms. These approaches help analyze stress progression, material failure, and overall structural response. The study underscores the urgency for innovative design practices and construction techniques that enhance structural safety against progressive collapse.*

Keywords : (*Progressive Collapse ,Irregular RC Structures,Linear Static Analysis,Demand Capacity Ratio (DCR),GSA Guidelines,Seismic and Wind Load Evaluation*)

1. INTRODUCTION

Here Accidental loads—such as explosions, vehicular impacts, or natural disasters—pose unpredictable threats to structural integrity. While non-structural components can often be repaired, failure in load-bearing elements may lead to partial or total collapse. Progressive collapse occurs when a localized failure triggers a chain reaction, compromising the entire structure. The 9/11 incident highlighted the critical need to address this phenomenon in design. The American Society of Civil Engineers (ASCE) and General Services Administration (GSA) emphasize incorporating progressive collapse resistance in structural design. The Alternative Path Method (APM), widely adopted by GSA and DoD, ensures load redistribution when a key element fails. This approach enhances resilience by enabling adjacent components to absorb additional loads. Although designing for progressive collapse increases material use and cost, it significantly improves safety, especially in high-risk zones like urban roadsides. This study investigates the collapse behavior of a conventional building model under the removal of two adjacent columns, simulating scenarios like blasts or vehicular impacts. The analysis follows GSA and Indian code guidelines, aiming to improve structural robustness against unforeseen failures

1.1. History

The concept of progressive collapse gained attention after the 1968 Ronan Point disaster in London, where a gas explosion on the 18th floor caused a partial collapse. Although the damage was localized, it highlighted the vulnerability of structures to accidental loads. A more catastrophic example occurred on September 11, 2001, when the World Trade Center (WTC) Twin Towers collapsed after being struck by airplanes. The initial impact was withstood due to the steel frame, but the intense heat weakened structural members, leading to a chain reaction of floor collapses. WTC-7, a nearby 41-story building, also collapsed later that day due to fire-induced structural failure, despite not being directly hit. These events emphasized the importance of designing buildings to resist progressive collapse, especially under unforeseen loads. They also led to global awareness and revisions in structural design codes, urging engineers to consider robustness, redundancy, and alternative load paths in high-risk structures. For this cause we can hit upon bounty of instances as cited over in which the total structure fell down immediately owing to breakdown of one structura l member the entire structure. Some of the instances for progressive–collapse of the building in olden times of mankind could be rolled as below:

- Ronan–point tower
- Murrah federal–building at Oklahoma
- Sampoorna–departmental Store tower
- Pipers row auto park fall down

- WTC etc.

1.2. Earthquake-Induced Structural RisksNEED OF THE STUDY.

Earthquakes pose a major natural hazard, often resulting in severe structural damage and loss of life. Buildings, especially tall ones, are vulnerable to seismic and wind-induced vibrations. Earthquakes are caused by sudden movements along fault lines, releasing energy as seismic waves that shake the ground. These tremors can trigger secondary hazards like tsunamis, landslides, and soil liquefaction.

To ensure safety, structures must be designed to withstand seismic forces, following standards like IS 1893 Part 1 (2016). This code addresses earthquake-induced forces using the equivalent static lateral force method, where the Design Base Shear is the primary force considered. This force depends on the seismic hazard level of the building's location, represented by the Zone Factor (Z) and structural flexibility factor (Sa/g).

India is divided into four seismic zones—Zone 2 to Zone 5, with Zone 5 experiencing the highest seismic forces. Designing for seismic resistance involves accounting for inertia forces and ensuring that structures can absorb and redistribute energy without collapsing Proper seismic design not only protects lives but also minimizes economic and environmental losses. It is essential to integrate earthquake resilience into both new constructions and retrofitting of existing buildings, especially in high-risk zones

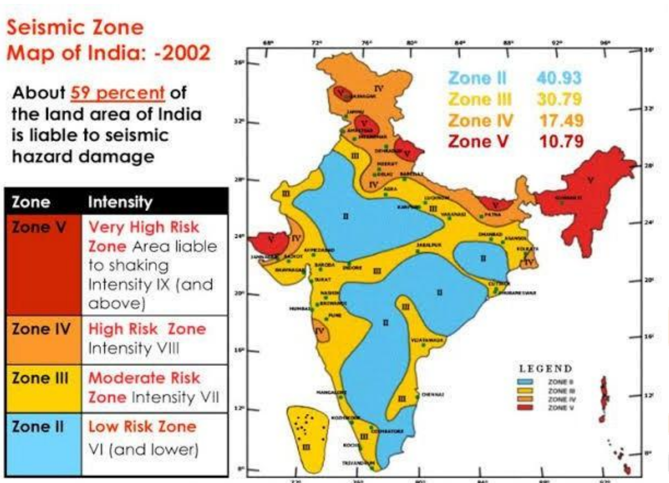


Fig 1:Seismic– zonation & 'intensity' map of India

1.4 Analysis Methods in Structural Engineering

Earthquake analysis is crucial for structures vulnerable to seismic forces, especially multi-storied buildings. While simple, low-rise buildings can be analyzed using static methods, complex or tall structures require dynamic analysis to capture time-dependent ground motion effects. As per IS 1893:2002 (Part 1), both static and dynamic methods are employed.

Earthquake analysis methods are broadly classified into:

1. **Linear Static Analysis** – Simplifies seismic loads into static forces; suitable for regular, low-rise buildings.
2. **Linear Dynamic Analysis** – Uses response spectrum or time-history methods to evaluate elastic behavior under seismic loads.
3. **Nonlinear Static Analysis (Pushover)** – Considers material yielding and provides a capacity curve for performance-based design.
4. **Nonlinear Dynamic Analysis** – Uses direct numerical integration to simulate elasto-plastic deformation under real-time seismic loading.

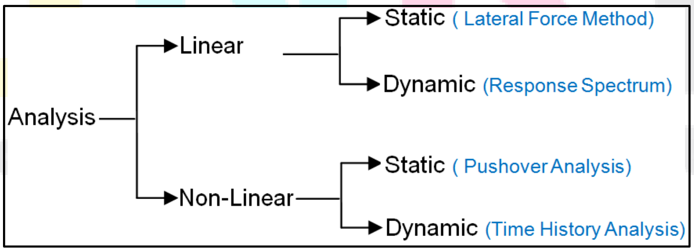


Fig 2:Types of analysis

1.2 Wind Load Structural Analysis

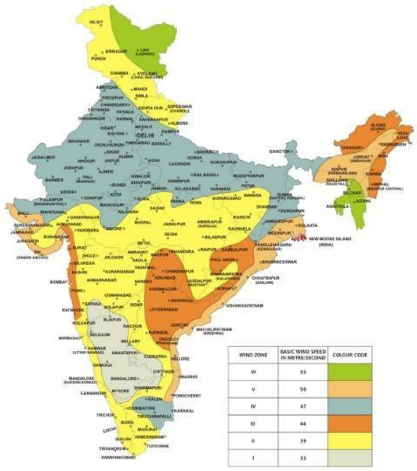
Wind load analysis evaluates how structures respond to wind-induced forces, ensuring safety, stability, and performance. It is especially critical for tall buildings, bridges, towers, and structures in high-wind zones.

headings should be left justified, bold, with the first letter capitalized and numbered consecutively, starting with the Introduction.

1.4.1 Key Components

- Wind Load Evaluation: Based on wind speed, direction, and turbulence.
- Structural Response: Assesses stresses, strains, and deformations.

1.4.2 Types of Wind Forces



- Static Wind Loads: Steady pressure acting on the structure.
- Dynamic Wind Loads: Fluctuating forces causing oscillations and vibrations.

1.4.3 Analytical Methods

- Static Analysis: Assumes constant wind forces; suitable for simple structures.
- Dynamic Analysis: Considers time-varying wind effects; ideal for complex or tall structures.

1.4.4 Importance

- Ensures structural safety under wind loads.
- Improves performance, reduces maintenance, and enhances occupant comfort

1.4.5 Applications

- New Construction: Ensures compliance with wind load standards.
- Bridges & Towers: Critical for long-span and slender structures.
- Wind Turbines: Designed to withstand varying wind conditions.
- Retrofitting: Evaluates and strengthens existing structures

1.4.6 Wind Zones & Terrain Categories (IS 875 Part 3)

India is divided into six wind zones:

- Zone 1 (<33 m/s) to Zone 6 (>50 m/s), from low to very high damage potential.
- Terrain categories:
- Category 1: Open, flat areas (e.g., coastlines).
- Category 2: Sparse obstructions (e.g., fields).
- Category 3: Dense urban or industrial zones.
- Category 4: Closely spaced tall buildings or forests

1.4.7 Interference Zones

Z1 to Z4: Ranging from high to insignificant wind interference, based on topography and surrounding obstructions

2. Analysis Process for Progressive Collapse

Progressive collapse analysis evaluates how a structure responds when a key load-bearing element fails, potentially triggering a chain reaction of failures. Four primary methods are used for such analysis:

a) Linear Static Analysis
b) Nonlinear Static Analysis
c) Linear Dynamic Analysis
d) Nonlinear Dynamic Analysis

These methods help assess the structure’s behavior when subjected to loads beyond its material strength. Static methods assume time-independent loading, while dynamic methods consider time-varying effects. Linear analysis assumes elastic behavior, whereas nonlinear analysis accounts for material yielding and large deformations. Although advanced software tools are used, simplifications are often made. For instance, concrete—composed of cement, sand, aggregates, and water—is treated as a homogeneous material, despite its heterogeneous nature. Among the four**, linear static analysis** is the simplest. It assumes small deformations and follows Hooke’s Law, making it suitable for preliminary assessments. This method is particularly useful for calculating the **Demand-Capacity Ratio (DCR),** a key metric in evaluating structural safety under progressive collapse scenarios

2.1 Design Approaches

As per ASCE 7 we have two possibilities for the ‘design approach’. Firstly is direct design approach & secondly indirect design approach. Later approach is essentially works on lowest strength, ductility, continuity of the ‘joint’ that foils progressive– collapse by perk—up the ductility & redundancy. As this method aids to augment the ‘redundancy’ of the structure & as well it will not persuade the abolition of specific lo a d or a precise member. How e ver, this scheme isn’ t advised for conducting the analysis for ‘progressive– collapse of a chosen structure. Hence, we have to go with first method for comprehending progressive– collapse.

2.2 Direct–Design Approach

The Direct–Design Approach focuses on preventing progressive collapse during the structural design phase. This method ensures that even if a part of the structure fails, the overall integrity of the building is maintained. There are two primary strategies under this approach:

• Specific–Local Resistance Approach
This strategy aims to contain and delay the spread of progressive collapse by reinforcing local structural elements. If a member fails—such as a beam or column—the surrounding components are designed to resist and absorb the impact, preventing the failure from propagating further.
• Alternate–Load Path Approach
This approach allows the redistribution of loads through alternative structural paths when a localized failure occurs. For example, if a column fails, the load it was carrying is redirected through adjacent beams or columns, ultimately reaching the foundation. This method reflects real-world behavior, where structures naturally seek alternate paths to maintain stability. r this study secondary data has been collected. From the website of KSE the monthly stock prices for the sample firms are obtained from Jan 2010 to Dec 2014. And from the website of SBP the data for the macroeconomic variables are collected for the period of five years. The time series monthly data is collected on stock prices for sample firmsand relative macroeconomic variables for the period of 5 years. The

data collection period is ranging from January 2010 to Dec 2014. Monthly prices of KSE -100 Index is taken from yahoo finance.

2.3 Permissible Limits

GSA has set the directives that are to be adhered to while performing 'linear static' analysis. It has offered the formula to acquire the 'DCR' values for a structural member.

DCR ≅ Qud/ Que

Where,

Qud = 'Acting force' / demand observed in component or connection [bending moment & axial force for the combined forces]

Que= 'Expected ultimate nonfractured' capacity of a chosen structural component or chosen connection [bending moment & axial force for the combined forces]

As per the instructions supplied by G.S.A, the 'D.C.R' values offered with the higher limits as below

D.C.R for 'flexure failure' of members >= 2

D.C.R for 'shear failure' of members >= 1.5

2.4 Permissible Limits & Analysis Procedure

As previously stated, the 'linear static' analysis technique is the most straightforward & simplest method, whereas the 'non–linear' dynamic analysis approach is the most complex amongst 4 analysis techniques. In this project, we have opted for the 'linear–static' analysis for the selected model, despite the fact that the breakdown of the vertical member is intrinsically dynamic.

For the analysis & design processes, we utilized ETABS software, adhering to IS code provisions in relative to the materials & member properties, in concurrence with GSA guidelines

The step-by-step procedure undertaken during the analysis in the software is detailed below:

Step - 1: as illustrated in the figures on the following pages, the building is modeled using software. Subsequently, various loads, including seismic loads, are allocated to the model. The model is then analyzed using the 'linear–static' method.

Step - 2: the results acquired from software are depicted in the fig. Following this, the D□C□Rs are calculated & tabulated for the columns that are scheduled for removal.

Step - 3: in each instance, two column members were concurrently removed at the 'lower basement floor' level, and the loads on the remaining members are assessed.

The load allocated to the slabs positioned directly above the eradicated column is

$$\mathbf{G_{LF} \rightarrow 2X\ [1.2X\ DL + 0.5X\ LL]}$$

The load allocated to all the other slabs excluding the one positioned directly above the eradicated column is

$$G \rightarrow 1.2X\ DL + 0.5X\ LL$$

Where,

DL→ Dead Load

LL→ Live Load

Step - 4: the results of the analysis are exported to excel sheet. The previously mentioned equation is used to compute the tabular D.C.R values for all applicable columns. A graphical representation is created to facilitate the easy interpretation of the results.

Step - 5: According to the guidelines outlined in the G.S.A, columns with a D.C.R of less than 2.0 are considered safe & competent of withstanding 'progressive–collapse'. In contrast, if the D.C.R is >1, it indicates that the column's 'demand' crosses its 'capacity', proposing that a D.C.R > 1 could upshot in the column's failure. Only when the D.C.R > 2 can anticipate the occurrence of 'progressive collapse'

3. Detailed Data of The Building And Structural Details

Table 1 Material & Loads

Sl. No	Particular	Details
1.	Concrete Grade	M30
2.	Steel Grade	Fe500
3.	Number of cases analysed	3 cases [for zone –2]
4.	**Loads on the building**	
a.	Imposed Load	1.5 kN/sqm
b.	Live Load	5.0/2.0/3.0 kN/sqmm

Table 2 Structural details

Sl. No	Particular	Details
1.	Height b/w floors	3.2 m
2.	Column size	600 x 600 mm
3.	Beam size	400 x 650 mm
4.	Column spacing	Mentioned in the figures in subsequent pages
5.	Slab depth	200 mm
6.	Seismic zones	*II (IS 1893:2002)*
7.	Soil type	II
8.	Response Reduction Factor, R	3
9.	Importance Factor, Ì	1

3.1 Load Considered

Table 3 MAIN WALL UDL

0.200m thk S.C.B UDL →	17.65 X 0.2m →	3.53 kN/Sqm
MortarThk→	0.012m + 0.02m →0.032m thk	
MortarDensity →	20.40	
UDL for Mortar →	0.032 X 20.40	0.65 kN/Sqm
Total		4.18 kN/Sqm
Floor Ht (m)	3.2	
Beam Depth (m)	0.65	
Wall Ht (m)	3.2-0.65 = 2.55	
Main Wall UDL	2.55 m X 4.18	10.66 kN/m

Table 4 PARTITION WALL UDL

0.100m thk S.C.B UDL →	17.65 X 0.1m →	1.77 kN/Sqm
MortarThk→	0.012m + 0.012m →0.024m thk	
MortarDensity →	20.40	
UDL for Mortar →	0.024 X 20.40	0.49 kN/Sqm
Total		2.26 kN/Sqm
Floor Ht (m)	3.2	
Beam Depth (m)	0.65	
Wall Ht (m)	3.2-0.65 = 2.55	
Partition Wall UDL	2.55 m X 2.26	5.76 kN/m

Table 5 PARAPET UDL

0.100m thk S.C.B UDL →	17.65 X 0.1m →	1.77 kN/Sqm
MortarThk→	0.012m + 0.012m →0.024m thk	
MortarDensity →	20.40	
UDL for Mortar →	0.024 X 20.40	0.49 kN/Sqm
Total		2.26 kN/Sqm
Wall Ht (m)	1.0	
Parapet UDL	1.0 m X 2.26	2.26 kN/m

3.1 Other Load

Table 6 Live Load Consider

IS : 875 (Part 2) - 1987

TABLE 1 IMPOSED FLOOR LOADS FOR DIFFERENT OCCUPANCIES

(Clauses 3.1, 3.1.1 and4.1.1)

SL No.	Occupancy Classification	Uniformly Distributed Load (UDL)	Concentrated Load
(1)	(2)	(3)	(4)
		kN/m²	kN
i)	RESIDENTIAL BUILDINS		
	a) Dwelling houses:		
	1) All rooms and kitchens	2'0	1·8
	2) Toilet and bath rooms	2·0	—
	3) Corridors, passages, staircases including fire escapes and store rooms	3·0	4.5
	4) Balconies	3.0	1·5 per metre run concentrated at the outer edge
iv)	ASSEMBLY BUILDINGS		
	a) Assembly areas:		
	1) with fixed seats‡	4'0	—
	2) without fixed seats	5'0	3.6
	b) Restaurants (subject to assembly), museums and art galleries and gymnasia	4·0	4.5

Table 7 Live Load At Terrace

TABLE 2 IMPOSED LOADS ON VARIOUS TYPES OF ROOFS

(*Clause* 4.1)

SL No.	Type of Roof	Uniformly Distributed Imposed Load Measuked on PLan Area	Minimum Imposed Load Measured on Plan
(1)	(2)	(3)	(4)
i)	Flat, sloping or curved roof with slopes up to and including 10 degrees		
	a) Access provided	1·5 kN/m²	3.75 kN uniformly distributed over any span of one metre width of the roof slab and 9 kN uniformly distributed over the span of any beam or truss or wall
	b) Access not provided except for maintenance	0·75 kN/m²	1·9 kN uniformly distributed over any span of one metre width of the roof slab and 4·5 kN uniformly distributed over the span of any beam or truss or wall

- ➢ Live Load [kN/ Sqm] ⇔ 5.0 @ upper & ground floor, 2.0 and 3.0 @ Typical floor and 1.5 @ terrace
- ➢ Floor Finish [kN/ Sqm] ⇔ 1.00

Table 8 Load Name, Their Type

Sl. No	Load Name	Load Type
1.	Dead	Linear Static
2.	Live	
3.	Roof Live	
4.	EQX	
5.	EQY	
6.	WLX	
7.	WLY	

Table 9 FACTORED Load COMBINATIONS

SL.No	Factored Load Combinations
1.	1.5[DL + LL]
2.	1.2[DL + LL + EQLX]
3.	1.2[DL + LL - EQLX]
4.	1.2[DL + LL + EQLY]
5.	1.2[DL + LL - EQLY]
6.	0.9DL +1.5EQLX
7.	0.9DL -1.5EQLX
8.	0.9DL +1.5EQLY
9.	0.9DL - 1.5EQLY
10.	1.2[DL + LL + WLX]
11.	1.2[DL + LL - WLX]
12.	1.2[DL + LL + WLY]
13.	1.2[DL + LL - WLY]
14.	0.9DL +1.5WLX
15.	0.9DL -1. WLX
16.	0.9DL +1.5WLY
17.	0.9DL - 1.5WLY
18.	1.2X DL + 0.5X LL
19.	2[1.2X DL + 0.5X LL]

4. OBJECTIVES

- ➢ Executing the analysis for the selected plan using Etabs
- ➢ To discern the consequence of forces due to quake on the formation/model during the progressive– collapse due to quake in zone II.
- ➢ To discover the consequence on a definite column by pulling out certain columns at a range of localities owing to 'load transfer'.
- ➢ Analyzing & comprehending the way the 'load' is conveyed by using Demand Capacity Ratio 'DCR' values for picked columns.
- ➢ To study & understand the progressive– collapse theory & 'DCR' values at elected stories & for chosen 'quake' zone.

4.1 Case Consider

- ➢ 1st Case – Removal of corner & its adjacent columns on grids C7 & D7
- ➢ 2nd Case – Removal of exterior column & its adjacent column on girds E2 & F2
- ➢ 3rd Case – Removal of Interior column & its adjacent column on girds E4 & E5

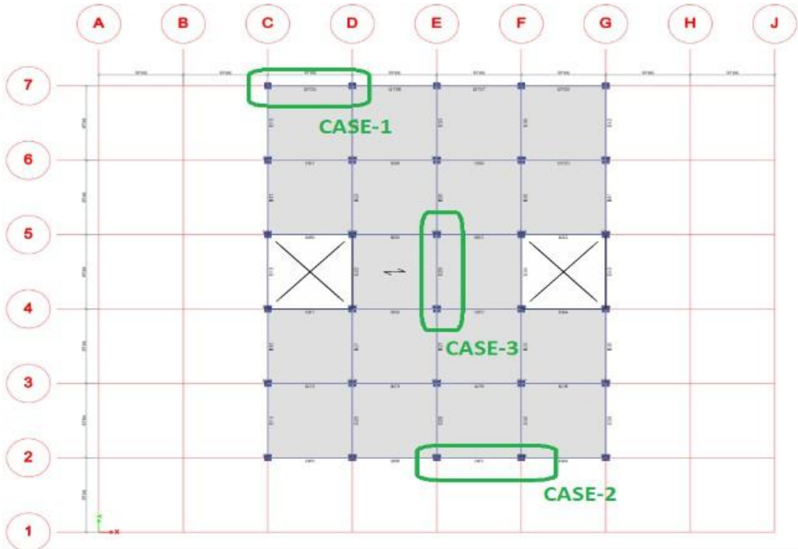


Fig 4: Cases Considered

5. Results

Fig 5: Isometric View of model

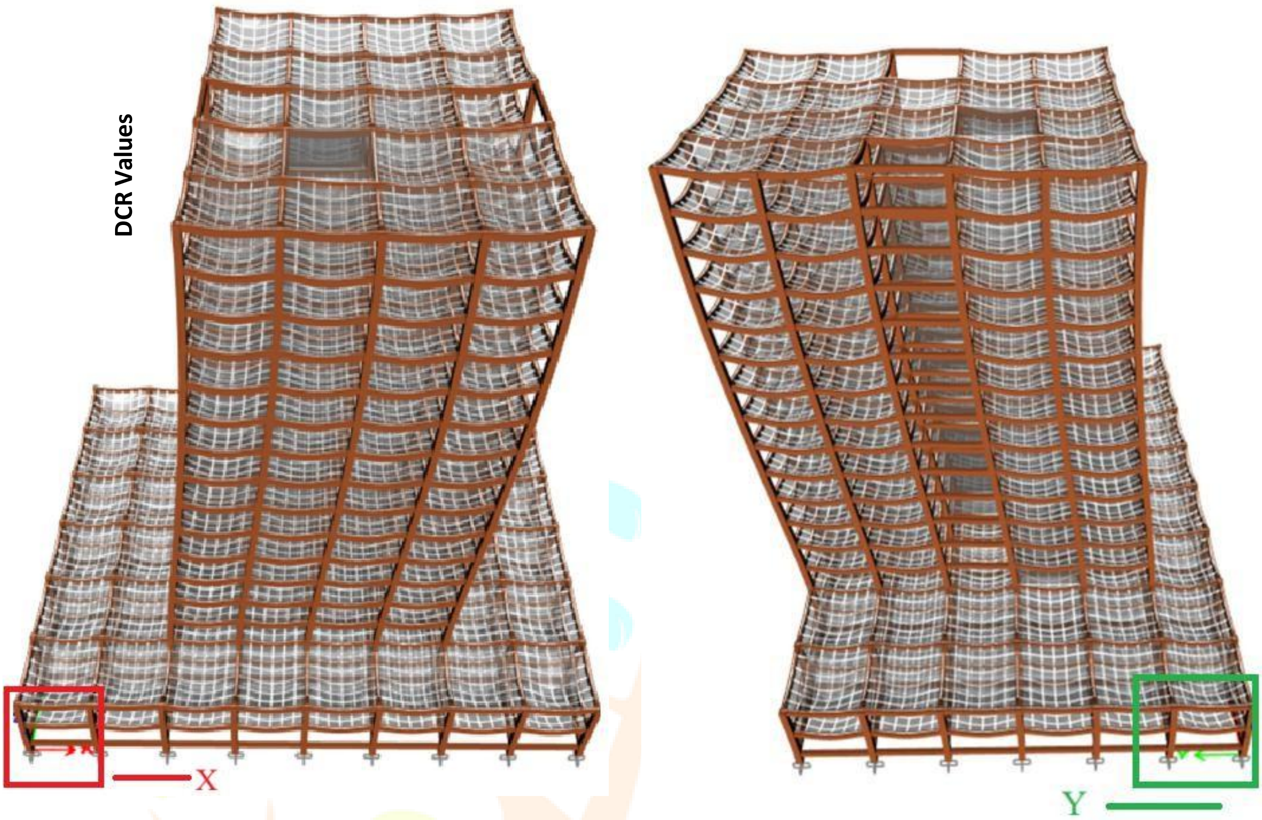


Fig 6: Deflection in X & Y direction

5.1 DCR Value Before Removing Column

Table 10 DCR Value before Removing Column

DCR VALUE BEFORE REMOVING COLUMN							
Floor	Story	C3	C4	C23	C32	C53	C56
TERRACE F	17	0.154	0.149	0.033	0.047	0.153	0.151
14th F	16	0.055	0.055	0.078	0.077	0.055	0.055
13th F	15	0.258	0.257	0.218	0.219	0.257	0.257
12th F	14	0.313	0.313	0.285	0.285	0.313	0.312
11th F	13	0.370	0.324	0.352	0.352	0.334	0.361
10th F	12	0.378	0.318	0.418	0.419	0.327	0.351
9th F	11	0.369	0.319	0.485	0.485	0.330	0.351
8th F	10	0.374	0.329	0.521	0.521	0.338	0.358
7th F	9	0.381	0.339	0.543	0.543	0.352	0.369
6th F	8	0.394	0.344	0.560	0.560	0.359	0.383
5th F	7	0.402	0.353	0.571	0.571	0.366	0.389
4th F	6	0.407	0.362	0.578	0.577	0.378	0.397
3rd F	5	0.414	0.361	0.588	0.590	0.382	0.407
2nd F	4	0.416	0.370	0.607	0.589	0.402	0.425
1st F	3	0.434	0.394	0.630	0.612	0.454	0.482
UPPER B	2	0.441	0.393	0.632	0.630	0.459	0.474
LOWER B	1	0.349	0.306	0.451	0.452	0.334	0.354

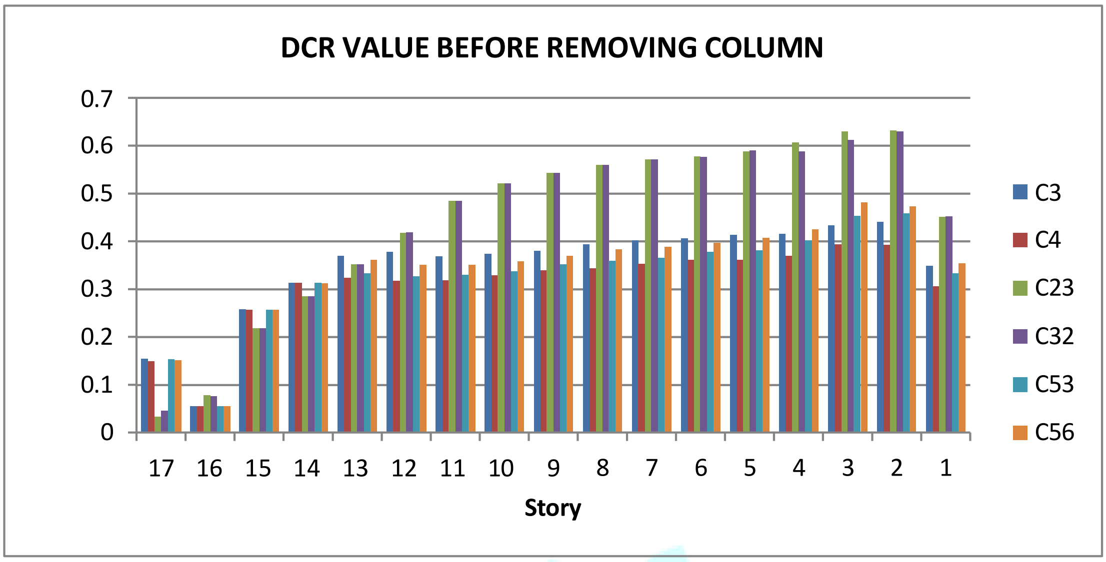


Fig 7 Chart for DCR Value before Removing Column

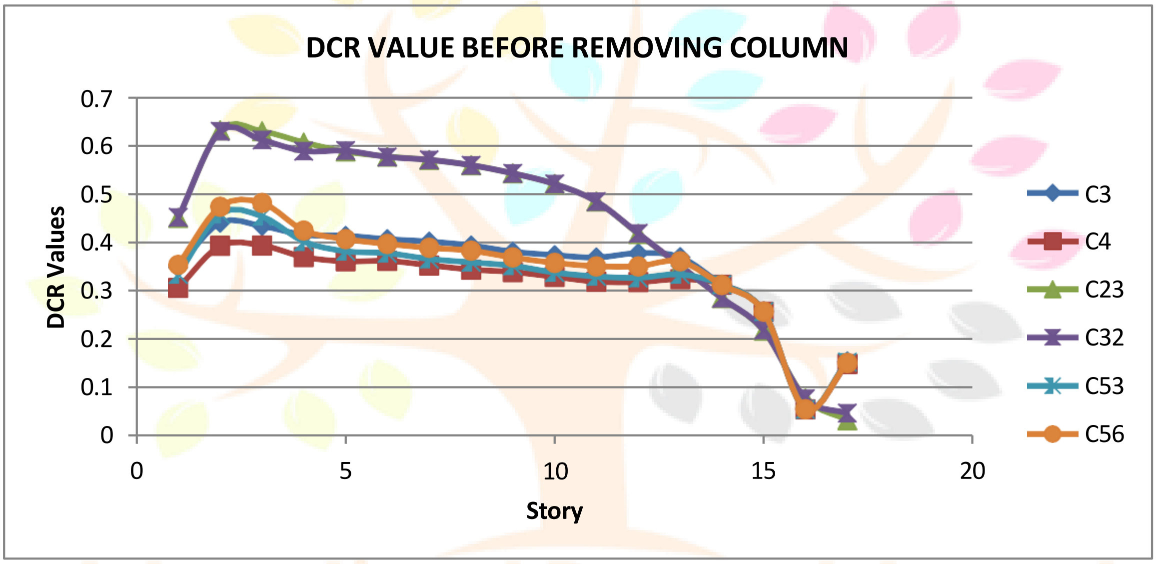


Fig 8 Graph for DCR value before Removing Column

5.2 DCR Value Post Removal of corner Col C3 & C4

Table 11 DCR value post removal of corner column C3 & C4

DCR VALUE POST REMOVAL OF CORNER COL C3 & C4								
Floor	Story	Col Removed	C5	C6	C12	C13	C14	C15
TERRACE F	17	C3 & C4	0.99	0.984	0.98	0.99	0.52	0.739
14th F	16		0.993	0.993	0.997	0.999	0.827	0.853
13th F	15		0.993	0.991	0.995	0.994	0.994	0.994
12th F	14		0.998	0.999	0.996	0.993	0.994	0.995
11th F	13		0.995	0.994	1.000	0.995	0.995	0.999
10th F	12		1.012	0.996	1.021	1.025	0.998	0.999
9th F	11		1.039	0.999	1.05	1.087	1.034	1.001
8th F	10		1.744	0.995	1.745	1.996	1.093	1.035
7th F	9		1.723	1.002	1.725	1.987	1.996	1.086
6th F	8		1.727	1.091	1.715	1.989	1.992	1.947
5th F	7		1.753	1.041	1.731	**2.009**	1.989	1.988
4th F	6		1.814	1.721	1.772	**2.059**	**2.001**	1.994
3rd F	5		1.881	1.712	1.837	**2.127**	**2.035**	1.992
2nd F	4		1.962	1.76	1.943	**2.209**	**2.087**	**2.017**
1st F	3		**2.027**	1.794	**2.118**	**2.314**	**2.153**	**2.065**
UPPER B	2		**2.037**	1.661	**2.131**	***2.446***	**2.252**	**2.135**
LOWER B	1		**2.15**	1.998	**2.218**	***2.456***	**2.222**	**2.124**

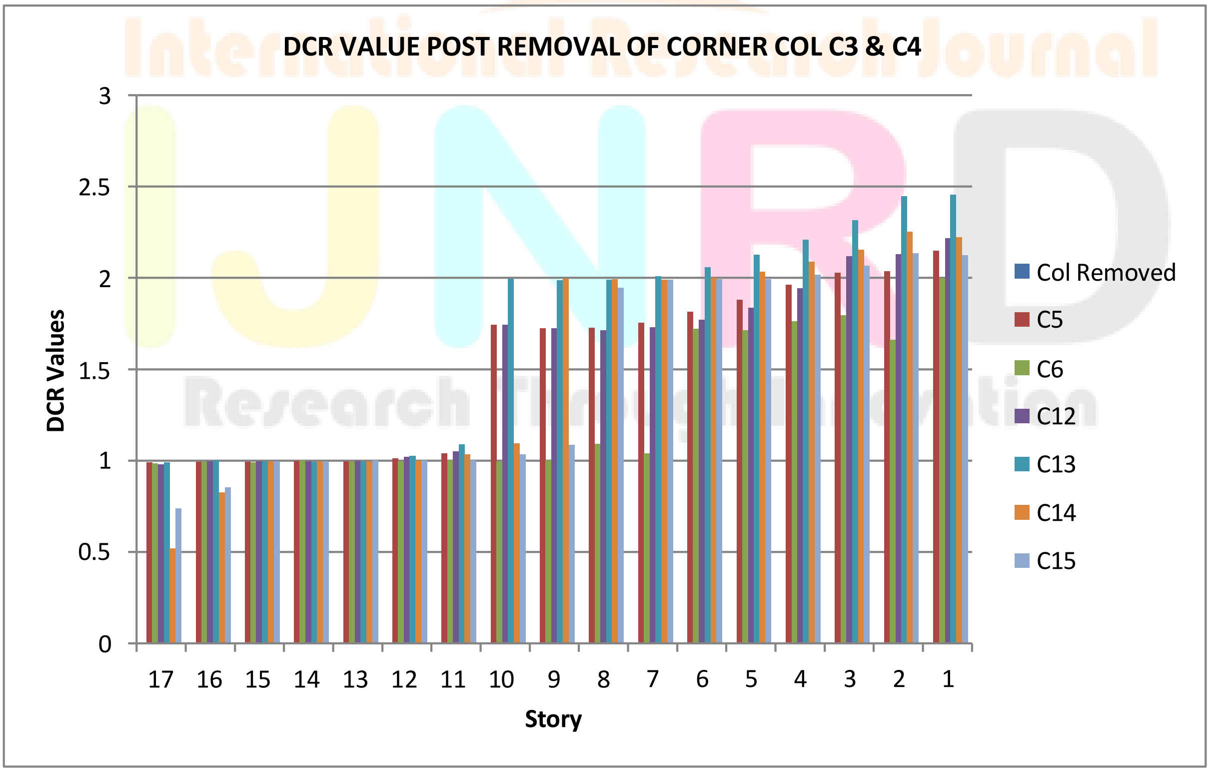


Fig 9 Chart for DCR value post removal of C3 & C4

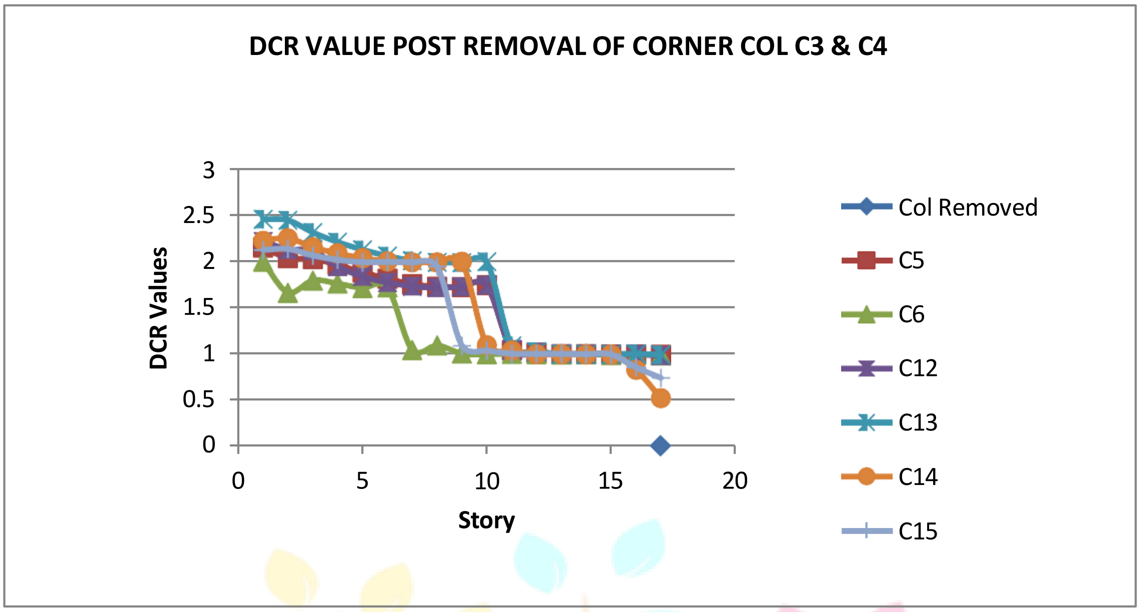


Fig 10 Chart for DCR value post removal of C3 & C4

5.3 DCR Value Post Removal of corner Col C3 & C4

Table 12 DCR value post removal of exterior columnC53 & C56

DCR VALUE POST REMOVAL OF EXTRIOR C53 & C56										
Floor	Story	Col Removed	C47	C50	C59	C48	C51	C54	C57	C60
TERRACE F	17	C53 & C56	1.881	1.986	1.993	0.993	0.860	1.893	1.913	1.864
14th F	16		1.853	1.966	1.957	0.980	1.050	1.819	1.982	1.938
13th F	15		1.925	1.917	1.879	1.000	1.404	1.986	1.985	1.842
12th F	14		1.852	1.908	1.869	0.992	1.751	1.988	1.987	1.871
11th F	13		1.848	1.937	1.867	1.000	1.966	1.985	1.986	1.872
10th F	12		1.822	1.932	1.860	0.994	1.973	1.989	1.987	1.839
9th F	11		1.810	1.940	1.873	0.998	1.999	1.986	1.987	1.863
8th F	10		1.812	1.923	1.817	0.996	1.994	1.986	1.995	1.857
7th F	9		1.801	1.883	1.784	0.994	1.988	1.996	1.989	1.875
6th F	8		1.824	1.905	1.812	0.999	1.990	1.995	1.989	1.878
5th F	7		1.847	1.900	1.778	1.017	1.997	2.034	2.011	1.894
4th F	6		1.860	1.923	1.751	1.045	1.989	2.097	2.062	1.914
3rd F	5		1.878	1.959	1.726	1.080	2.003	2.176	2.132	1.944
2nd F	4		1.942	2.021	1.849	1.122	2.04	2.267	2.214	1.965
1st F	3		1.941	2.069	1.989	1.178	2.081	2.368	2.324	1.989
UPPER B	2		2.013	2.159	2.038	1.263	2.145	2.47	2.43	2.023
LOWER B	1		1.988	2.157	2.015	1.275	2.144	2.504	2.465	1.989

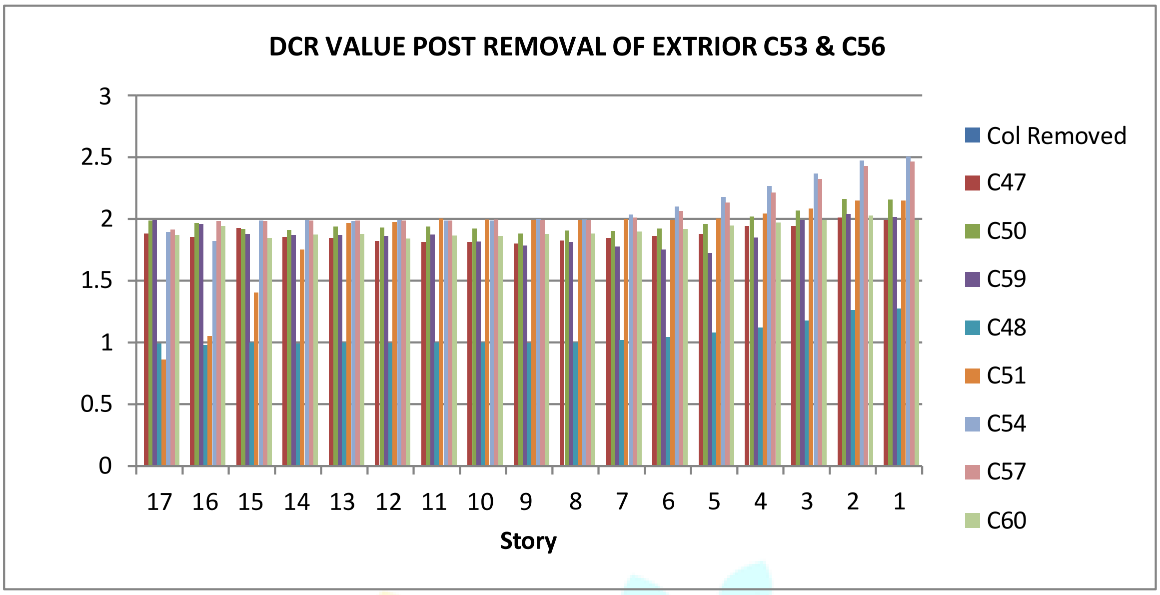


Fig 10 Chart for DCR value post removal of C53 & C56

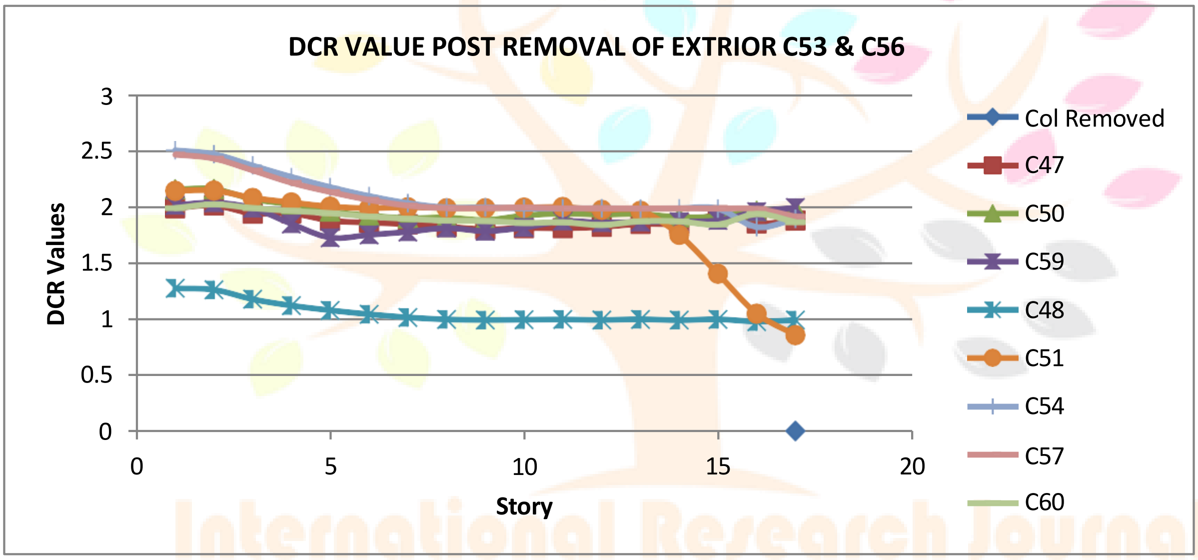


Fig 11 Graph for DCR value post removal of C53 & C56

5.4 DCR value post removal of interior column C23 & C32

Table 13 DCR value post removal of interior column C23 & C32

DCR VALUE POST REMOVAL OF INTERIOR COL C23 & C32												
Floor	Story	Col Removed	C51	C31	C22	C13	C54	C14	C57	C33	C24	C18
TERRACE	17	C23 & C32	0.992	0.992	0.989	0.978	0.987	0.998	0.974	0.99	0.99	0.974
14th F	16		0.993	0.991	0.991	0.993	0.991	0.991	0.999	0.992	0.992	0.999
13th F	15		0.999	0.993	0.993	0.997	0.994	0.994	0.998	0.994	0.994	0.998
12th F	14		0.996	0.998	0.997	0.995	0.994	0.994	0.996	0.993	0.993	0.996
11th F	13		0.998	0.998	0.999	0.998	0.996	0.996	0.996	0.994	0.994	0.996
10th F	12		0.995	0.997	0.998	0.995	1.029	1.029	0.995	1.01	1.01	0.995
9th F	11		1.009	1.03	1.032	1.01	1.086	1.086	1.015	1.053	1.053	1.014
8th F	10		1.052	1.084	1.087	1.053	1.165	1.165	1.062	1.106	1.105	1.061
7th F	9		1.109	1.15	1.152	1.11	1.258	1.255	1.123	1.177	1.176	1.122
6th F	8		1.176	1.223	1.226	1.176	1.334	1.332	1.193	1.254	1.253	1.191
5th F	7		1.248	1.303	1.304	1.248	1.435	1.433	1.267	1.338	1.336	1.266
4th F	6		1.324	1.386	1.388	1.324	1.54	1.536	1.345	1.425	1.424	1.344
3rd F	5		1.403	1.474	1.474	1.403	1.649	1.645	1.427	1.517	1.515	1.425
2nd F	4		1.483	1.569	1.566	1.487	1.763	1.757	1.507	1.616	1.61	1.51
1st F	3		1.568	1.688	1.693	1.58	1.866	1.884	1.597	1.751	1.732	1.6
UPPER B	2		1.653	1.793	1.826	1.672	**2.02**	**2.01**	1.686	1.874	1.866	1.693
LOWER B	1		1.661	1.812	1.837	1.68	**2.018**	**2.048**	1.699	1.887	1.877	1.701

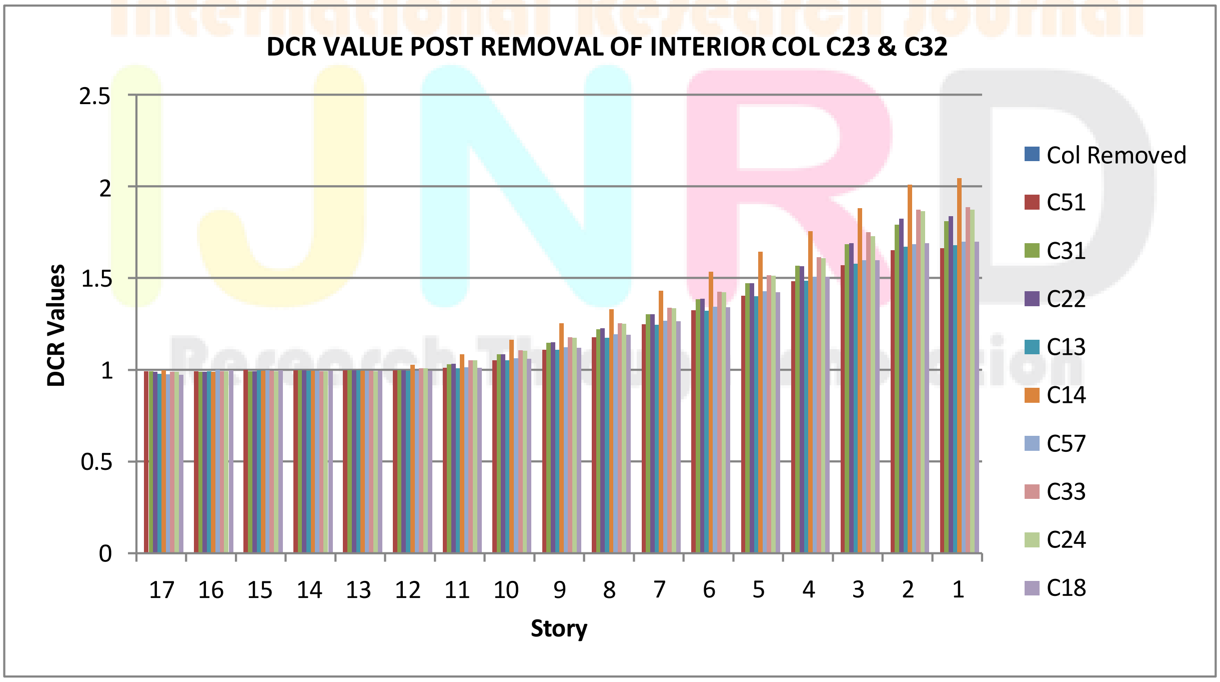


Fig 12 Chart for DCR value post removal of C23 & C32

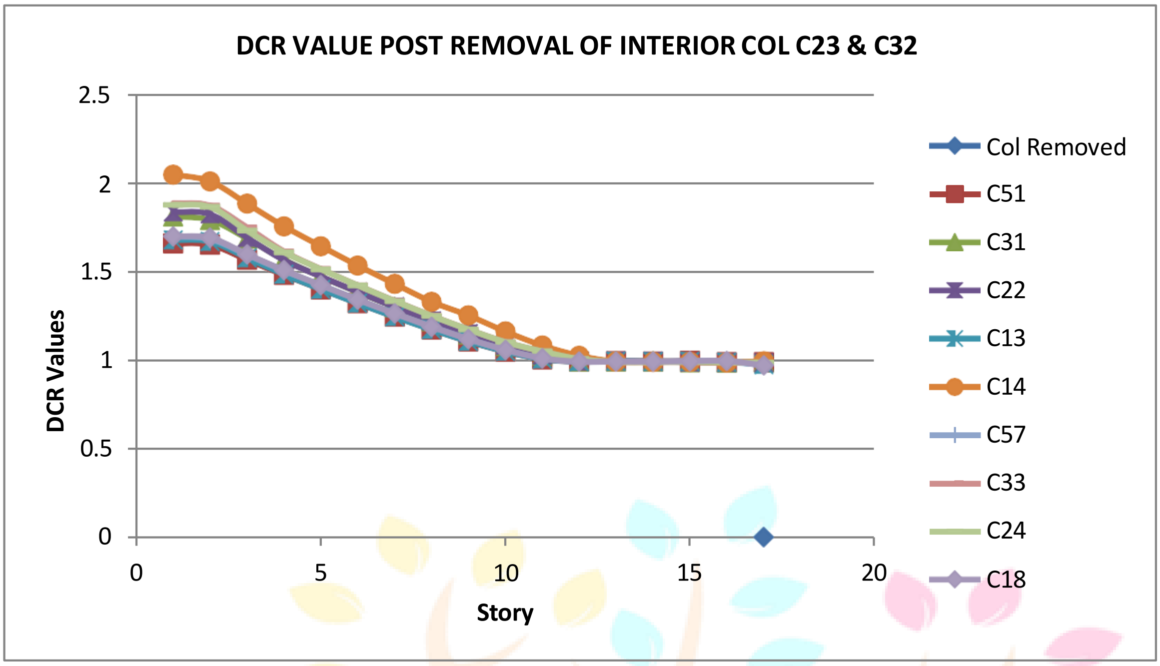


Fig 13 Graph for DCR value post removal of C23 & C32

5.5 Story displacement

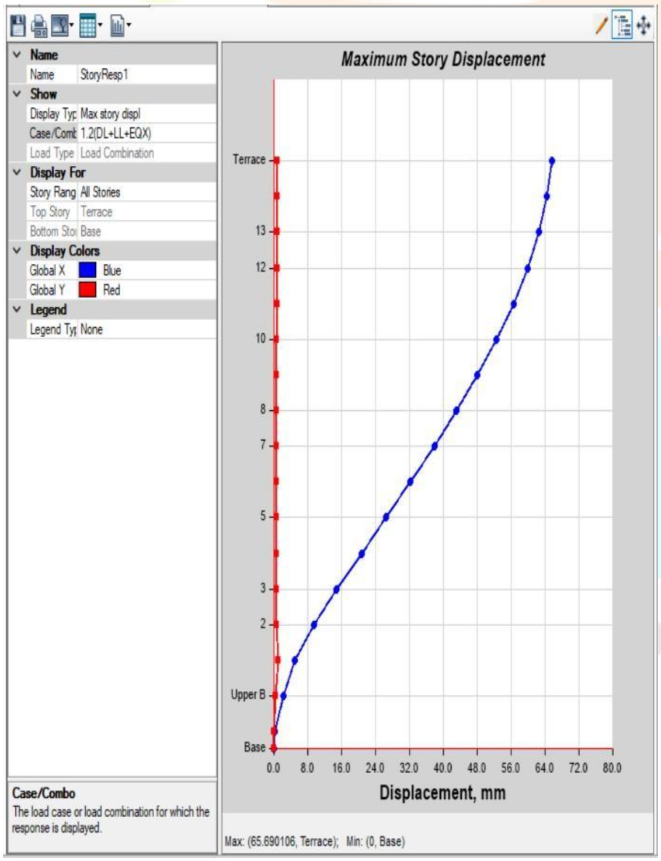


Fig 14 Story displacement before removing column for 1.2[D L+ LL+ `EQ LX]

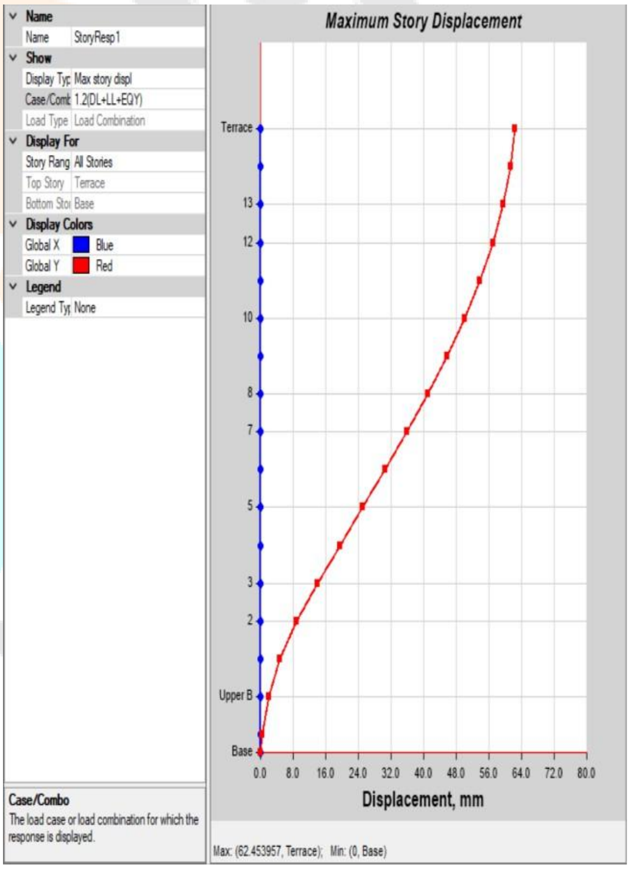


Fig 15 Story displacement before removing column for 1.2[D L+ LL+ `EQ LY]

5.6 Story drift

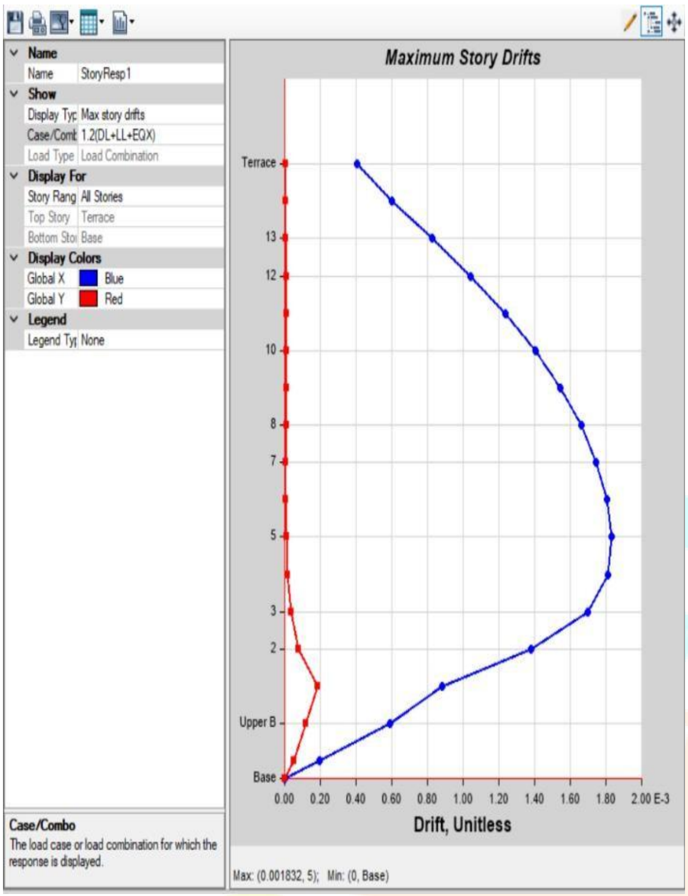


Fig 16 Story drift before removing column
for 1.2[D L+ LL+ `EQ LX]

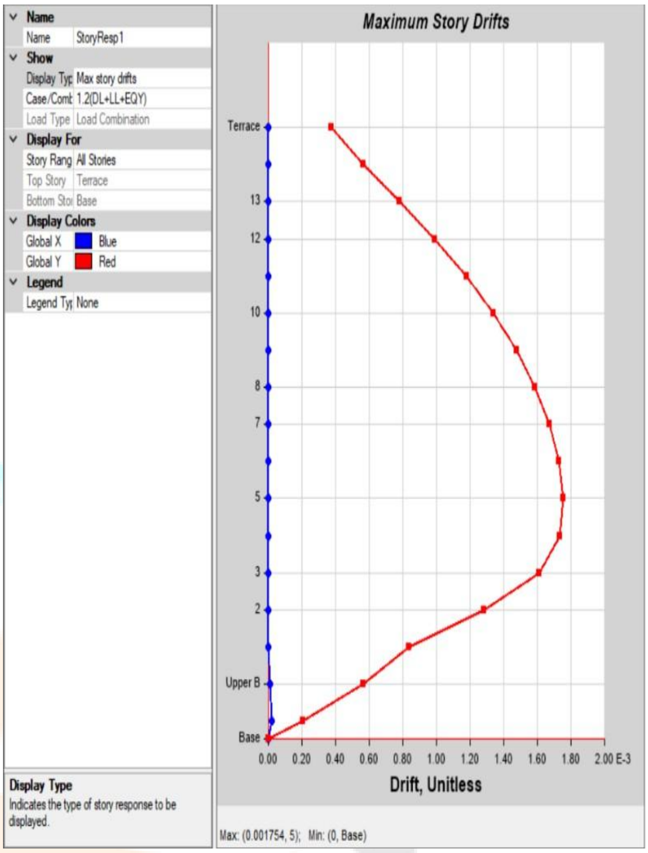


Fig 17 Story drift before removing column
for 1.2[D L+ LL+ `EQ Ly]

5.7 Story drift

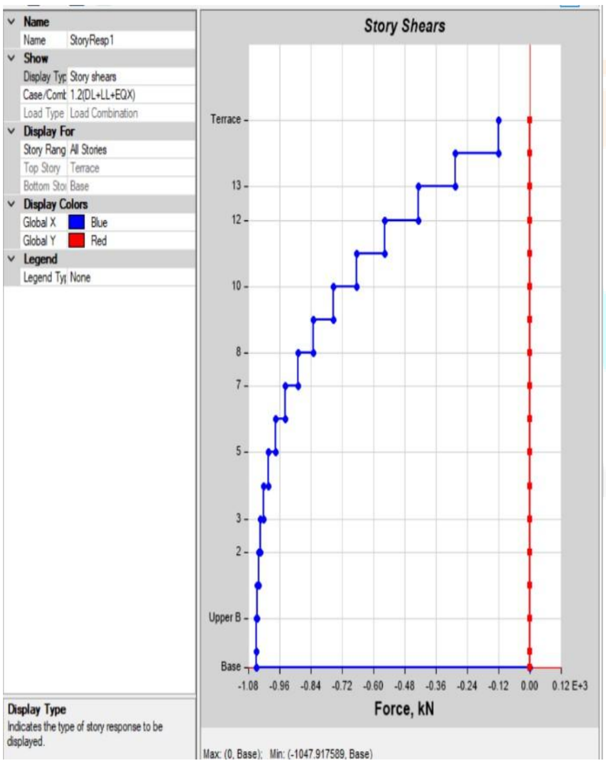


Fig 18 Story Shear before removing column
for 1.2[D L+ LL+ `EQ LX]

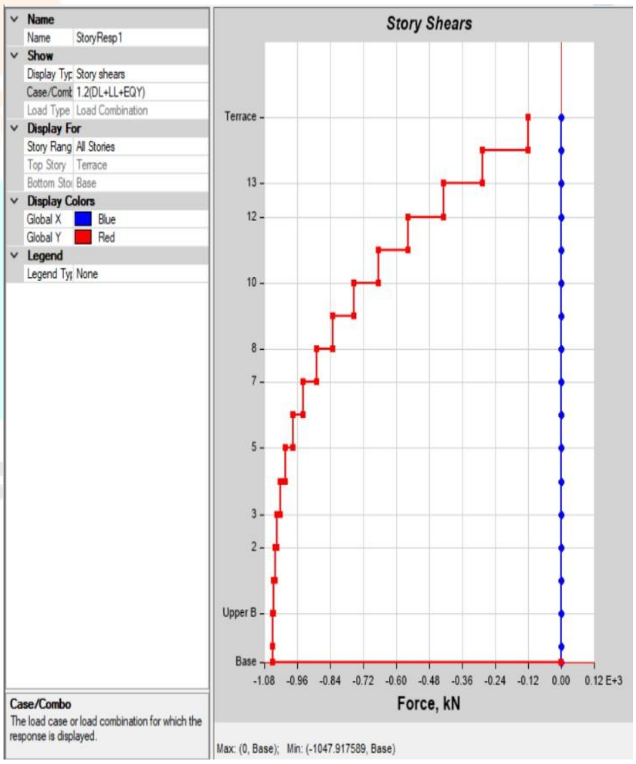


Fig 19 Story Shear before removing column
for 1.2[D L+ LL+ `EQ Ly]

6. CONNCLUSION

A study was conducted on a 17-storey structure to assess the impact of progressive collapse by analyzing the Demand-Capacity Ratio (DCR) of columns surrounding those confiscated for simulation. Key observations include:

1. Column Failures Adjacent to Confiscated Columns: Columns immediately next to the confiscated ones exhibited DCR values exceeding the critical limit of 2.0, indicating failure at multiple levels—especially in the lower basement and occasionally in upper floors.
2. Beam Failures Due to Excessive Shear: Beams connected to the confiscated columns also failed due to shear stresses surpassing permissible limits.
3. Case-wise Observations:
   - Case 1 (Corner Columns C3 & C4): 5 columns failed across 23 levels.
   - Case 2 (Exterior Columns C53 & C56): 7 columns failed across 27 levels.
   - Case 3 (Interior Columns C23 & C32): 2 columns failed across 4 levels.
4. Comparative Severity: Cases 1 and 2 showed more severe structural impact compared to Case 3, though all cases demonstrated significant vulnerability.
5. Preventive Measures:Failures can be mitigated through design precautions such as protective barricades to prevent vehicular impact on critical columns.

REFERENCES

[1] SaumiaMeenathethilAlex ,SreedeviLekshmi "Study of Progressive– collapse Analysis of Flat Slab Building" International Journal of Science and Research {I¸J¸S¸R} I¸S¸S¸N {Online}: 2319–7064¸ 2015

[2] Mr. Muralidhara G. B , Mrs. Swathi Rani K. S , Mr. MeleseWorku "Seismic Parametric Study on Different Irregular Flat Slab Multi–Story Building" International Journal of Engineering Research & Technology {I¸J¸E¸R¸T} Vol¸ 5 Issue 04¸ April–2016

[3] Kevin A¸ Giriunas and HalilSezen Progressive– collapse analysis of an existing building ohio state university May–2009

[4] Seweryn Kokot, Armelle Anthoine, Paolo Negro and Goergesolomos "Static & dynamic analysis of a reinforced concrete flat slab frame building for progressive– collapse" A¸I¸S¸C 40:–205-217 July–2012

[5] Mohamed zanjir, "Vulnerability of buildings with flat plates and flat slabs to progressive– collapse" university of Ottawa April–2012.

[6] Rakshith KG¸ Radhakrishna¸ "Progressive Collapse Analysis Of Reinforced Concrete Framed Structure"¸ International Journal of Research in Engineering and Technology, ISSN: 2321–7308¸ Nov– 2013.

[7] Ram Shankar Singh¸ Yusuf Jamal¸ Meraj A. Khan, "Progressive Collapse Analysis of Reinforced Concrete Symmetrical And Unsymmetrical Framed Structures By Etabs", International Journal of Innovative Research in Advanced Engineering (I.J.I.R.A.E) ISSN– 2349– 2763, Issue –12, Volume– 2 (December –2015)

[8] Syed Asaad Mohiuddin Bukhari, Shivaraju GD, Ashfaque AhmedKhan. "International Journalof Engineering Sciences & ResearchTechnology", International Journal of Engineering Sciences & ResearchTechnology, I.S.S.N: 2277–9655, June–2015.

[9] Yash Jain,Dr. V. D.Patil, "ProgressiveCollapse Assessment ofa Multi-StoreyRC Framed StructureUsing Non–Linear StaticAnalysis Technique", Yash Jain Journalof Engineering Researchand Application¸ I¸S¸S¸N :– 2248–9622, Vol¸ 8, Issue ,July– 2018.

[10] Yihai Bao¸ SashiK. Kunnath. "Simplified progressive collapse simulation ofRC framewall structures"¸ EngineeringStructures–2010.

[11] ZHANG Peng ¸ CHENBaoxu, "Progressive CollapseAnalysis of ReinforcedConcrete Frame Structures inLinear Static AnalysisBased onG¸S¸A", IEEE– 2012.