



PROGRESSIVE COLLAPSE ANALYSIS OF G+11 STORY STEEL BUILDING BY PUSH OVER ANALYSIS

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Abstract

In order to prevent disproportionate collapse, progressive collapse analysis of reinforced concrete structures evaluates the structural response to abrupt local failures or unusual loading circumstances. The pushover analysis, is used for nonlinear static analysis, which is a structural analysis technique used to assess a building's or other structure's seismic performance. When evaluating the behaviour and capacity of structures exposed to lateral loads, as those caused by earthquakes, it is especially helpful. Push over analysis is used for a Special Moment Resisting Frame (SMRF) and Ordinary Moment Resisting Frame (OMRF); which are defined as, a structural structure called a Special Moment Resisting Frame (SMRF) is frequently utilized in the design of earthquake-resistant buildings to withstand lateral stresses brought on by seismic activity. Because of their high degree of ductility and capacity for energy dissipation, SMRFs are able to withstand considerable deformation during seismic events without collapsing; whereas an Ordinary Moment Resisting Frame (OMRF) is a type of structural system used in building construction to resist lateral forces such as those induced by wind or moderate seismic activity. While not as robust or ductile as Special Moment Resisting Frames (SMRFs), OMRFs still provide adequate resistance to lateral loads in many building applications. As well as the effect of base shear which is a critical concept in structural engineering, particularly in the design and analysis of buildings and other structures subject to seismic activity; which refers to the total horizontal force at the base of a structure due to seismic activity. This force is a result of the ground shaking during an earthquake, which induces inertia forces throughout the structure.

The present project work consists of Progressive Collapse Analysis of G+10 Story RC Building by Push through Analysis for a Special Moment Resisting Frame (SMRF) and Ordinary Moment Resisting Frame (OMRF); and comparison of results & discussion; as well minimum necessary design requirements as per Indian codes IS 456:2000, and IS 1893:2016 (part1) code using this software. For Progressive Collapse and the Analysis General Service Administration (GSA) 2013 guidelines are followed.

Key Words: Special Moment Resisting Frame (SMRF) and Ordinary Moment Resisting Frame (OMRF)

I. INTRODUCTION

In the era of the potential of exponential growth of development which attract the significant constructions of high rise building in the city, the collapse of the single element may cause the collateral damage on the structure. The situation may cause significant damage to the living peoples and the wealth. Considering the past scenario of the collapse of the whole building due to the damage of the one and more element the significance of the study for the progressive collapse of the building is becoming vital.

The Progressive Collapse Analysis may help to ascertain the structures response to the abrupt local failure or unusual loading conditions. Certainly, it emphasizes to review collapse spread in the structure due to the failure of one or more structural element. The main cause of the local failure of the structural element is unintentional action.

Progressive Collapse Overview

To prevent a structure, fail from abrupt local failure the structure need to be design the adequate strength ductility as well as ability to provide the alternate path without sudden collapse of the structure. Following are the key features of the Progressive Collapse as per GSA Design Guideline 2003.

Local Damage Initiation

Progressive Collapse starts at the verge of damage of structural element such as column beam due to local failure or abnormal loading condition.

Load Redistribution

After the failure of the structural element load carried by that structural element could be redistributed to the surrounding structural members.



Figure 1.1 Ronan Point Incident

II Literature Review

Numerous studies have been published in both Indian and international journals, as well as in conference proceedings, addressing similar topics. These publications present a range of findings, conclusions, and analytical formulations related to the progressive collapse of steel buildings. In the present study, several concepts have been reviewed and adopted from this existing body of work. A brief review of selected Indian and international research papers and journals is presented below.

Pagare (2021)

It also evaluates the worst column removal condition for the said G+9 RC building with Shear wall. It also evaluates the sustainability of the shear wall structure provided by Indian Standard code against Progressive Collapse of the structure. The study concludes that, Indian Standard gives the load combinations used are higher than those specified by GSA, which leads to greater predicted structural failure. This indicates that the Indian approach to progressive collapse analysis is more conservative in comparison.

Yolanda (2019)

Used General Service Administration (GSA) 2013 provisions to evaluate the potential progressive collapse of Reinforced Concrete Structure under several critical column removal conditions of building. In each analysis the DCR value of structural element were examined and compared with the acceptance criteria. The classification of regular and irregular building is based on horizontal a vertical configuration of building. It conclude that, the progressive collapse in regular irregular structure does not occur in single column removal case it happens if some of critic columns from structure is removed from the Push Over Analysis.

Mohamed (2013)

The study utilized SAP2000 to evaluate the influence of design provisions for different seismic zones, as per ECP 201-2003, on the susceptibility of structures to progressive collapse. The analysis involved the sudden removal of a primary vertical load-bearing element, and the results were compared with the GSA 2013 guidelines.

Based on various column removal scenarios, the study concluded that structures designed for Seismic Zone 1 are the most prone to progressive collapse when a load-bearing member is eliminated. Furthermore, the ductility requirements incorporated in seismic design contribute to enhancing the performance of existing buildings under progressive collapse conditions caused by accidental explosions.

David (2019)

This study focuses on assessing the effect of column removal duration on the structural response and compares commonly adopted assumptions used in column removal analysis for progressive collapse evaluation. the influence of column removal time within the range of $0.001 < Rt \leq 5$ seconds was examined at three different locations in the structure. For each case, the maximum displacement and rotational responses were recorded. The results indicate that the duration of column removal, when analyzed using time-history functions, significantly affects the structural response. Although the GSA guidelines recommend that the column removal time should be less than one-tenth of the fundamental period of the structure, this criterion still permits a broad range of acceptable values.

III Progressive Analysis of Steel Building

This chapter outlines the methodology adopted for the assessment of the Progressive Collapse of G+11 Story Steel building. The methodology is structured to systematically assess and compare responses including Axial force, Moment, story drift, under standardized loading and design conditions and corner column removal scenario. ETABS software is used for structural modelling and analysis as per IS 800:2007 and IS 1893 (Part 1): 2016.

Analysis Approach

Progressive collapse analysis is performed by removing centre columns of Special Moment Resisting Frame as well as Ordinary Moment Resisting Frame and evaluated the structural response. The structure is analysed to determine the demand on adjacent members and to assess whether they can safely redistribute. The analysis helps in identifying vulnerable areas and understanding capacity of the structure to resist progressive collapse.

Types of Moment Resisting Frame

The Moment Resisting Frame are structural frame in which beam and columns are rigidly connected to resist the lateral load. Following are two types of the Moment Resisting Frame

Ordinary Moment Resisting Frame

As per IS 1893-2016, it is Moment resisting frame design as per IS 800-2007 but not meet ductile requirement as per IS 13920-2016

Special Moment Resisting Frame

As per IS 1893: 2016, it is Moment resisting frame designed and detailed as per IS 800 as well as meeting Following table describe the difference between SMRF and OMRF.

IV Results And Discussion

Frame Situated at Zone-II

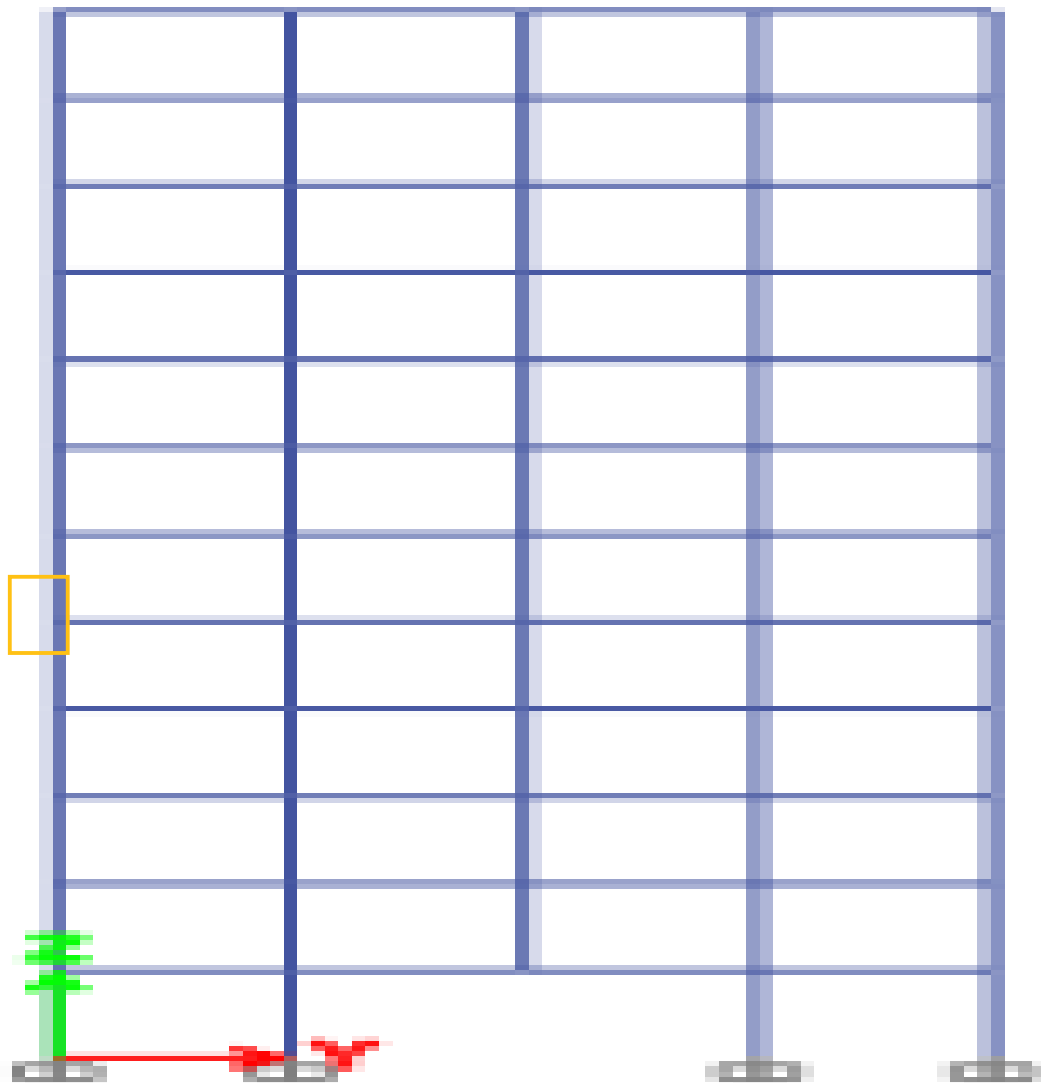


Figure 5.1 Elevation of Building 3-3

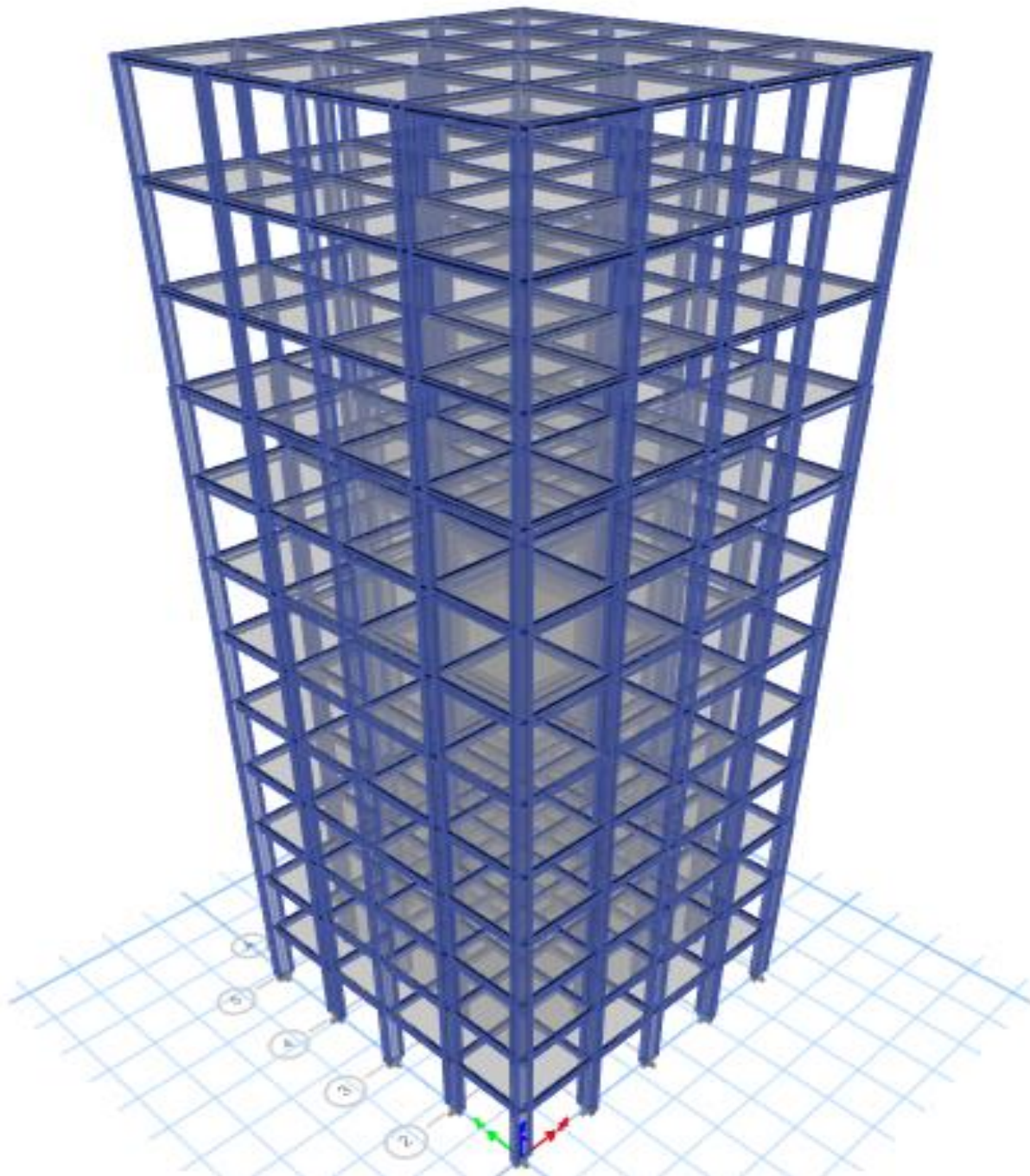
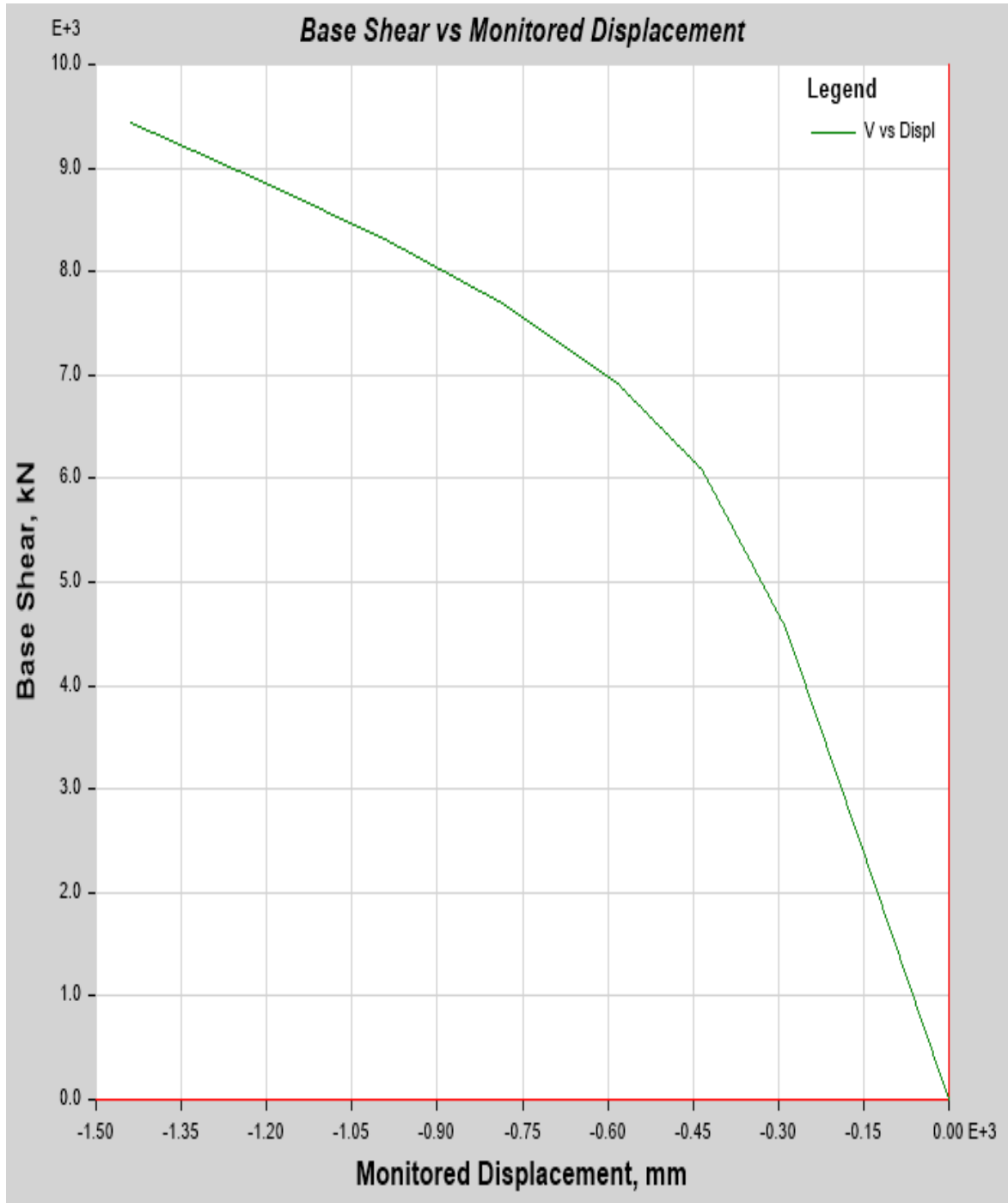


Figure 5.2 Rendering View of Building

After Analyzing The OMRF, EQ. Zone 3 Frame Following Results Are Obtained



Graph 5.1 Pushover Curve Dir.X For OMRF, EQ. Zone 3 Base Shear Vs Monitored Displacement

Step	Monitored Displ (mm)	Base Force (kN)	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	LS-CP	>C P	Total Hinges
0	0	0	1920	0	0	0	0	1920	0	0	0	0	1920
1	-144	2299.6	1920	0	0	0	0	1920	0	0	0	0	1920
2	-285.749	4563.3	1904	16	0	0	0	1920	0	0	0	0	1920
3	-432.918	6089.4	1548	372	0	0	0	1920	0	0	0	0	1920
4	-585.142	6937.4	1396	524	0	0	0	1920	0	0	0	0	1920
5	-791.353	7720.8	1260	660	0	0	0	1642	278	0	0	0	1920
6	-991.47	8312.3	1222	698	0	0	0	1488	432	0	0	0	1920
7	-1211.81	8885.5	1180	740	0	0	0	1388	532	0	0	0	1920

Ste p	Monitore d Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO- LS	LS- CP	LS- CP	>C P	Total Hinges
	mm	kN											
8	-1440	9432.2	1160	760	0	0	0	1270	650	0	0	0	1920

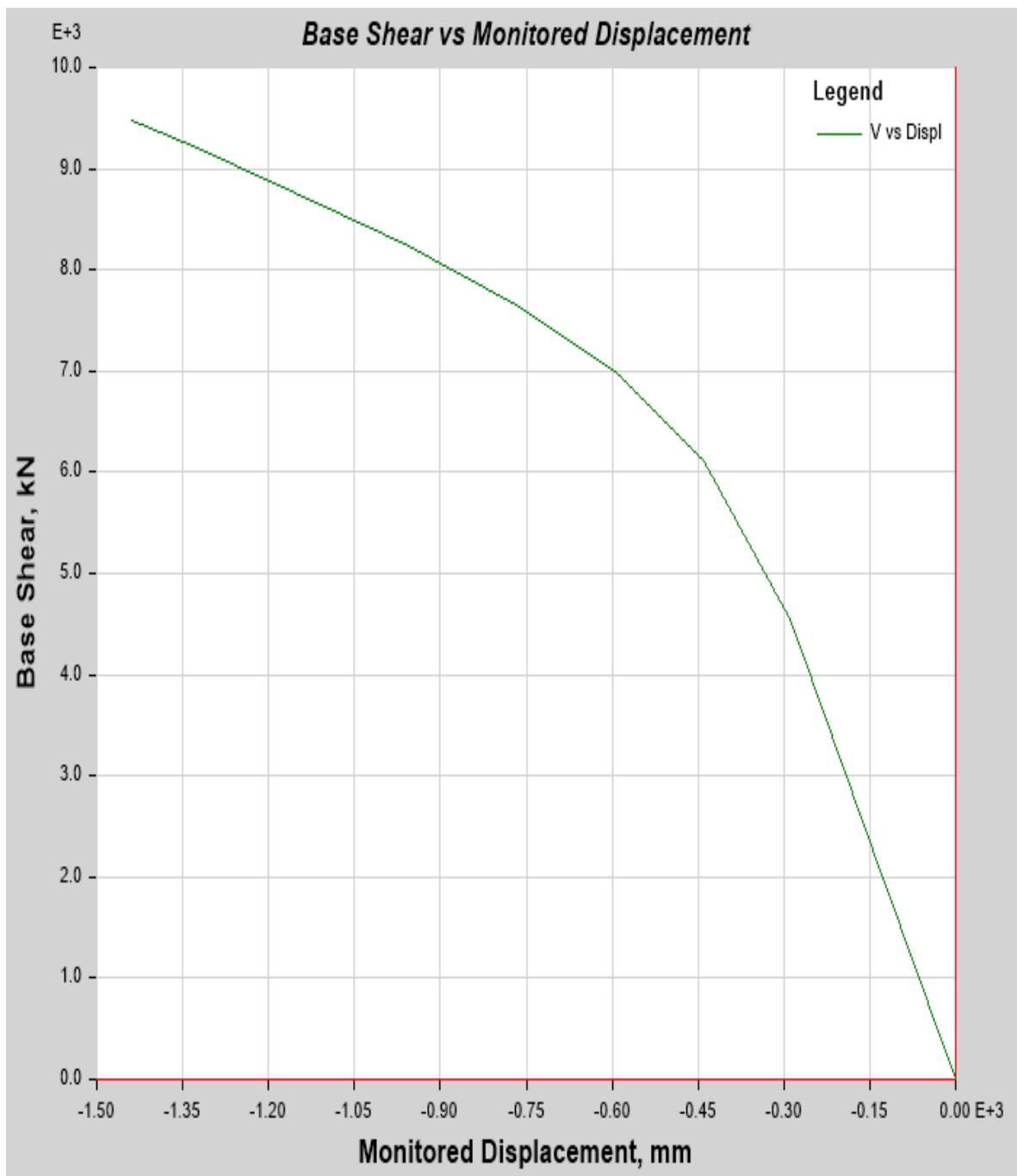
The pushover analysis results show that the base shear increases with monitored displacement, reaching a maximum value of about 9432 kN at 1440 mm displacement.

Plastic hinges initially form in the A–B range and gradually shift toward the B–C range as loading increases.

Most hinges remain within the Immediate Occupancy (IO) to Life Safety (LS) performance levels, and no hinges reach Collapse Prevention (CP). This indicates that the structure has adequate strength, ductility, and satisfactory seismic performance under lateral loading.

After Analyzing The OMRF, Eq. Zone 3 Frame Following Results Are Obtained

For Push-Y Condition



Graph5.2 Pushover Curve Dir. Y For OMRF, Eq Zone 3 Base Shear Vs Monitored Displacement

Step	Monitored Displ mm	Base Force kN	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	LS-CP	>CP	Total Hinges
0	0	0	1920	0	0	0	0	1920	0	0	0	0	1920
1	-144	2257.779	1920	0	0	0	0	1920	0	0	0	0	1920
2	-288	4515.647	1916	4	0	0	0	1920	0	0	0	0	1920
3	-291.12	4564.091	1898	22	0	0	0	1920	0	0	0	0	1920
4	-439.83	6105.485	1552	368	0	0	0	1920	0	0	0	0	1920
5	-592.49	6971.571	1392	528	0	0	0	1920	0	0	0	0	1920
6	-773.92	7678.664	1262	658	0	0	0	1700	220	0	0	0	1920
7	-961.56	8249.811	1230	690	0	0	0	1512	408	0	0	0	1920
8	-1189.3	8861.592	1182	738	0	0	0	1392	528	0	0	0	1920
9	-1333.3	9229.288	1160	760	0	0	0	1302	618	0	0	0	1920
10	-1440	9478.140	1160	760	0	0	0	1274	646	0	0	0	1920

Table 5.2 Base Shear vs Monitored Displacement Push Y case OMRF at Zone 3

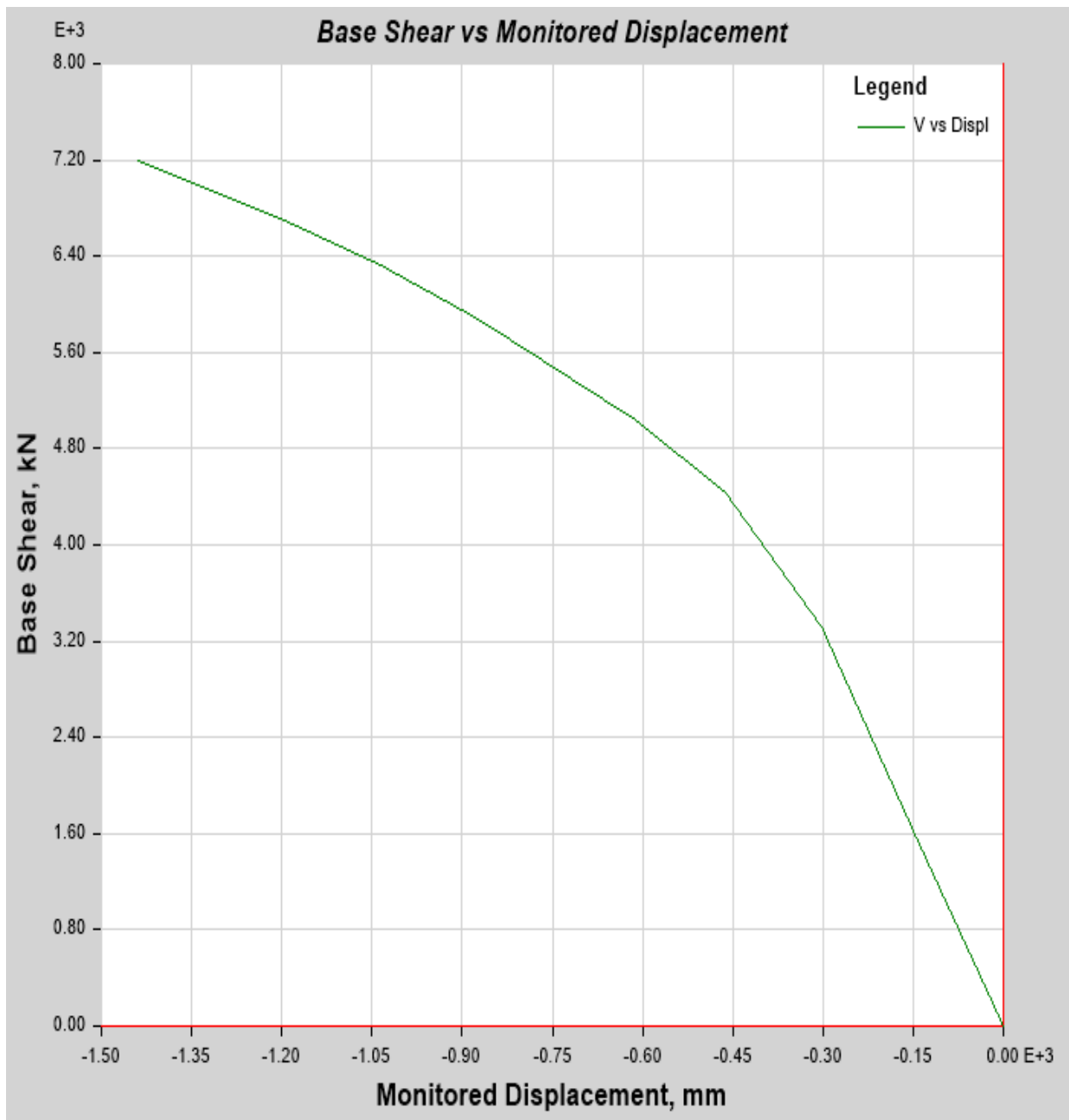
The pushover analysis results show that the base shear increases with monitored displacement, reaching a maximum value of about 9478 kN at 1440 mm displacement.

Plastic hinges initially form in the A–B range and gradually shift toward the B–C range as loading increases.

Most hinges remain within the Immediate Occupancy (IO) to Life Safety (LS) performance levels, and no hinges reach Collapse Prevention (CP). This indicates that the structure has adequate strength, ductility, and satisfactory seismic performance under lateral loadin

After Analyzing The SMRF, Eq. Zone 3 Frame Following Results Are Obtained

for Push-X condition



Graph5.3 Pushover Curve Dir.X For SMRF, Eq.Zone 3Base Shear Vs Monitored Displacement

Step	Monitor ed Displ	Base Force	Step	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total Hinges
	mm	kN											
0	0	0	0	1920	0	0	0	0	1920	0	0	0	1920
1	-144	1573.196	1	1920	0	0	0	0	1920	0	0	0	1920
2	-288	3146.393	2	1920	0	0	0	0	1920	0	0	0	1920
3	-303.057	3310.887	3	1912	8	0	0	0	1920	0	0	0	1920
4	-461.511	4429.213	4	1588	332	0	0	0	1920	0	0	0	1920
5	-610.547	5033.178	5	1444	476	0	0	0	1920	0	0	0	1920
6	-873.699	5869.215	6	1352	568	0	0	0	1592	328	0	0	1920
7	-1036.07	6319.120	7	1262	658	0	0	0	1532	388	0	0	1920
8	-1196.57	6692.247	8	1220	700	0	0	0	1480	440	0	0	1920
9	-1440	7198.981	9	1200	720	0	0	0	1392	528	0	0	1920

Table 5.3 Base Shear vs Monitored Displacement Push X Case SMRF at Zone 3

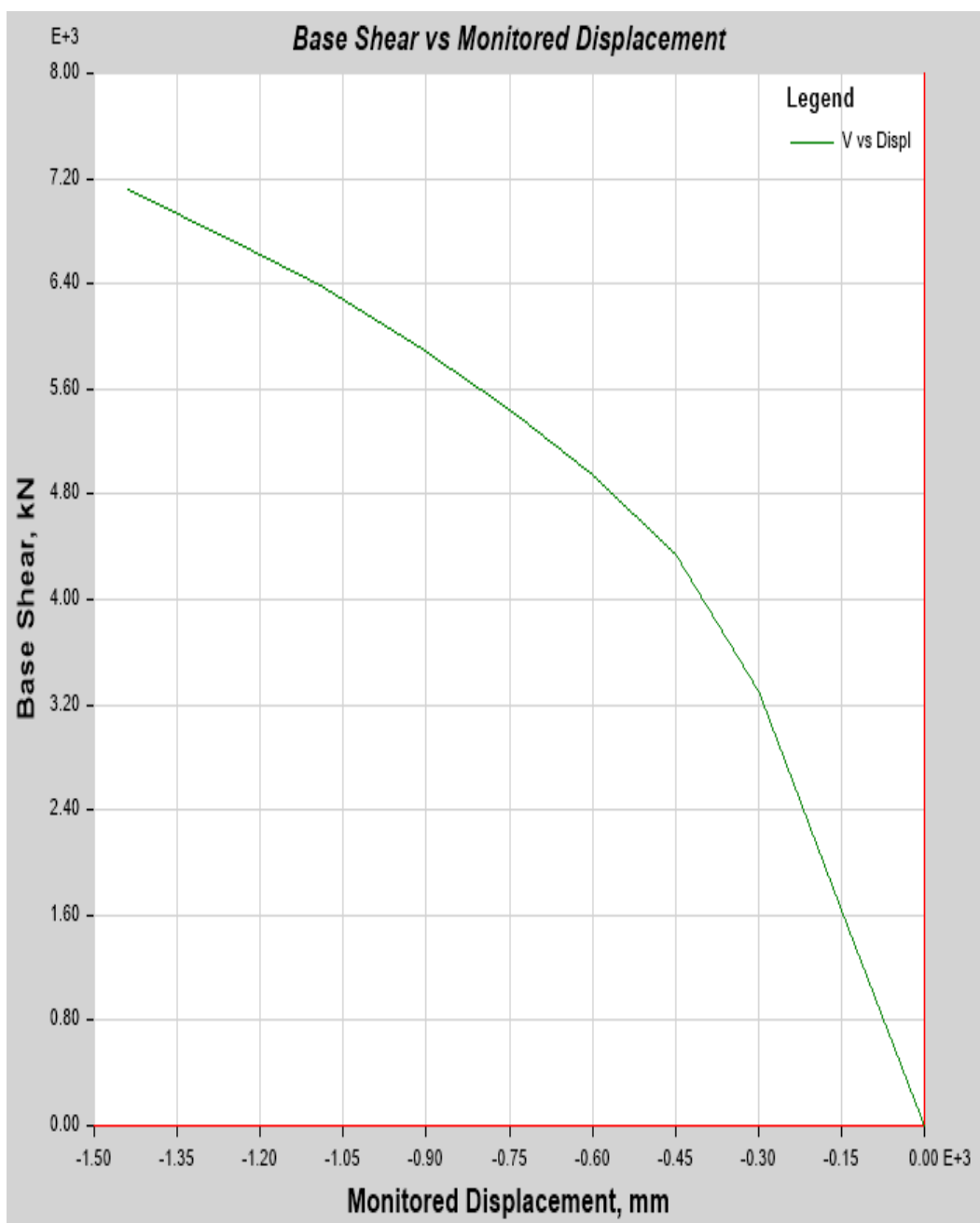
The pushover analysis was carried out for the SMRF structure in Seismic Zone 3 under X-direction loading to evaluate its seismic performance.

The capacity curve shows that the base shear increases with displacement and reaches a maximum value of about 7198 kN at a displacement of 1440 mm. As the displacement increases, plastic hinges gradually form indicating the transition from elastic to inelastic behavior.

Most hinges remain within the Immediate Occupancy (IO) and Life Safety (LS) performance levels, and no hinges reach the Collapse Prevention (CP) stage. This indicates that the structure possesses adequate strength, stiffness, and ductility. Therefore, the building demonstrates satisfactory seismic performance under the applied lateral load

5.1.4 After Analyzing The SMRF, EQ Zone 3 Frame Following Results Are Obtained

for Push-Y condition



Graph 5.4 Pushover Curve Dir Y For SMRF, EQ Zone 3 - Base Shear Vs Monitored Displacement

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total Hinges
	mm	kN										
0	0	0	1920	0	0	0	0	1920	0	0	0	1920
1	-144	1585.22	1920	0	0	0	0	1920	0	0	0	1920
2	-288	3170.44	1920	0	0	0	0	1920	0	0	0	1920
3	-300.057	3303.16	1904	16	0	0	0	1920	0	0	0	1920
4	-448.132	4339.98	1582	338	0	0	0	1920	0	0	0	1920
5	-599.735	4947.77	1428	492	0	0	0	1920	0	0	0	1920
6	-744.104	5422.025	1374	546	0	0	0	1764	156	0	0	1920
7	-914.493	5917.701	1348	572	0	0	0	1560	360	0	0	1920
8	-1091.315	6380.307	1258	662	0	0	0	1480	440	0	0	1920
9	-1242.969	6718.173	1220	700	0	0	0	1468	452	0	0	1920
10	-1440	7119.142	1200	720	0	0	0	1370	550	0	0	1920

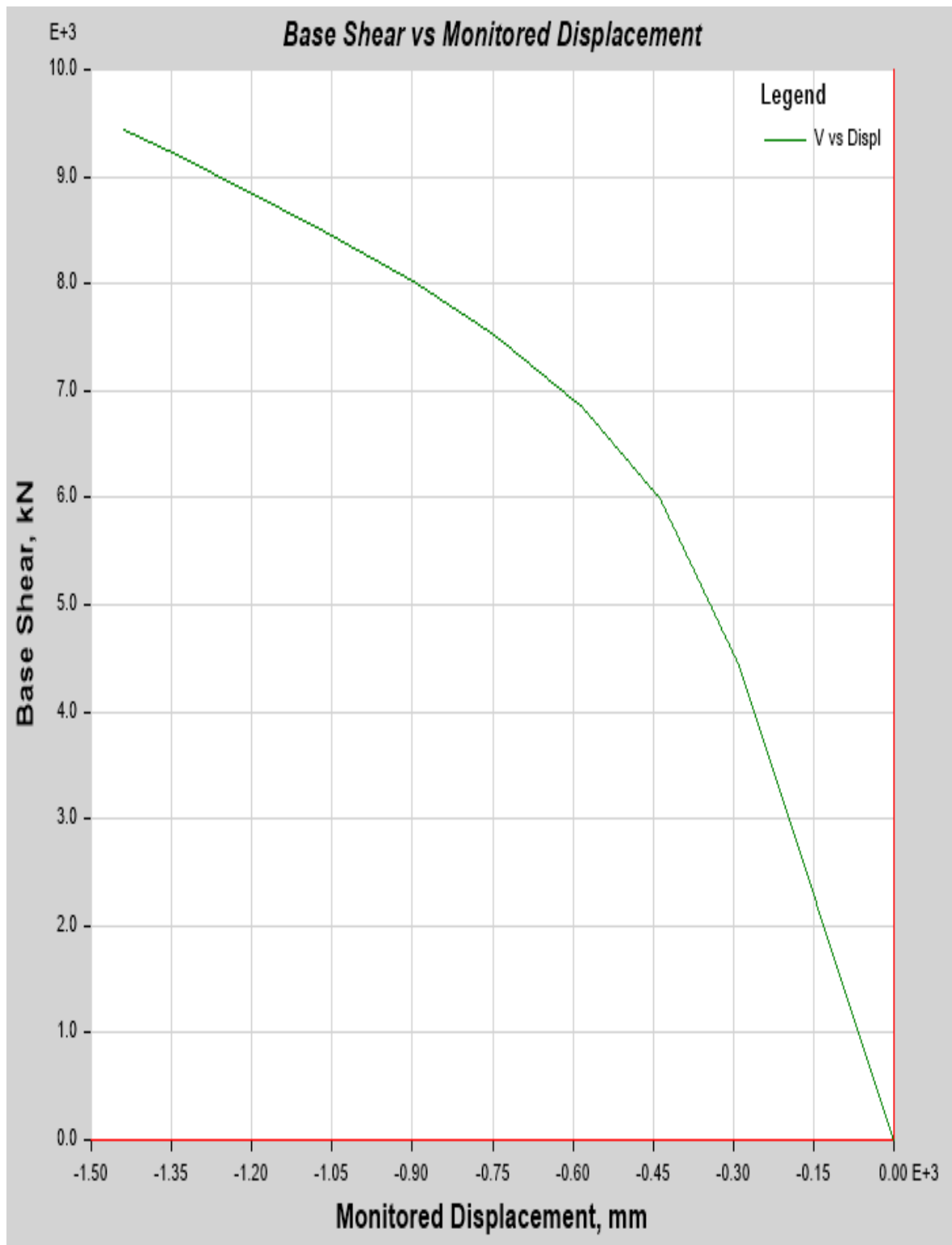
Table 5.4 Base Shear vs Monitored Displacement Push Y case SMRF at Zone 3**Capacity Curve Coordinates (Part 2 of 2)**

The pushover analysis was carried out for the SMRF structure in Seismic Zone 3 under Y-direction loading to evaluate its seismic performance.

The capacity curve shows that the base shear increases with displacement and reaches a maximum value of about 7119 kN at a displacement of 1440 mm. As the displacement increases, plastic hinges gradually form indicating the transition from elastic to inelastic behavior.

Most hinges remain within the Immediate Occupancy (IO) and Life Safety (LS) performance levels, and no hinges reach the Collapse Prevention (CP) stage. This indicates that the structure possesses adequate strength, stiffness, and ductility. Therefore, the building demonstrates satisfactory seismic performance under the applied lateral load.

After Analyzing The SMRF For Progressive Collapse, Eq.Zone 3 Frame Following Results Are Obtained For Push-X Condition



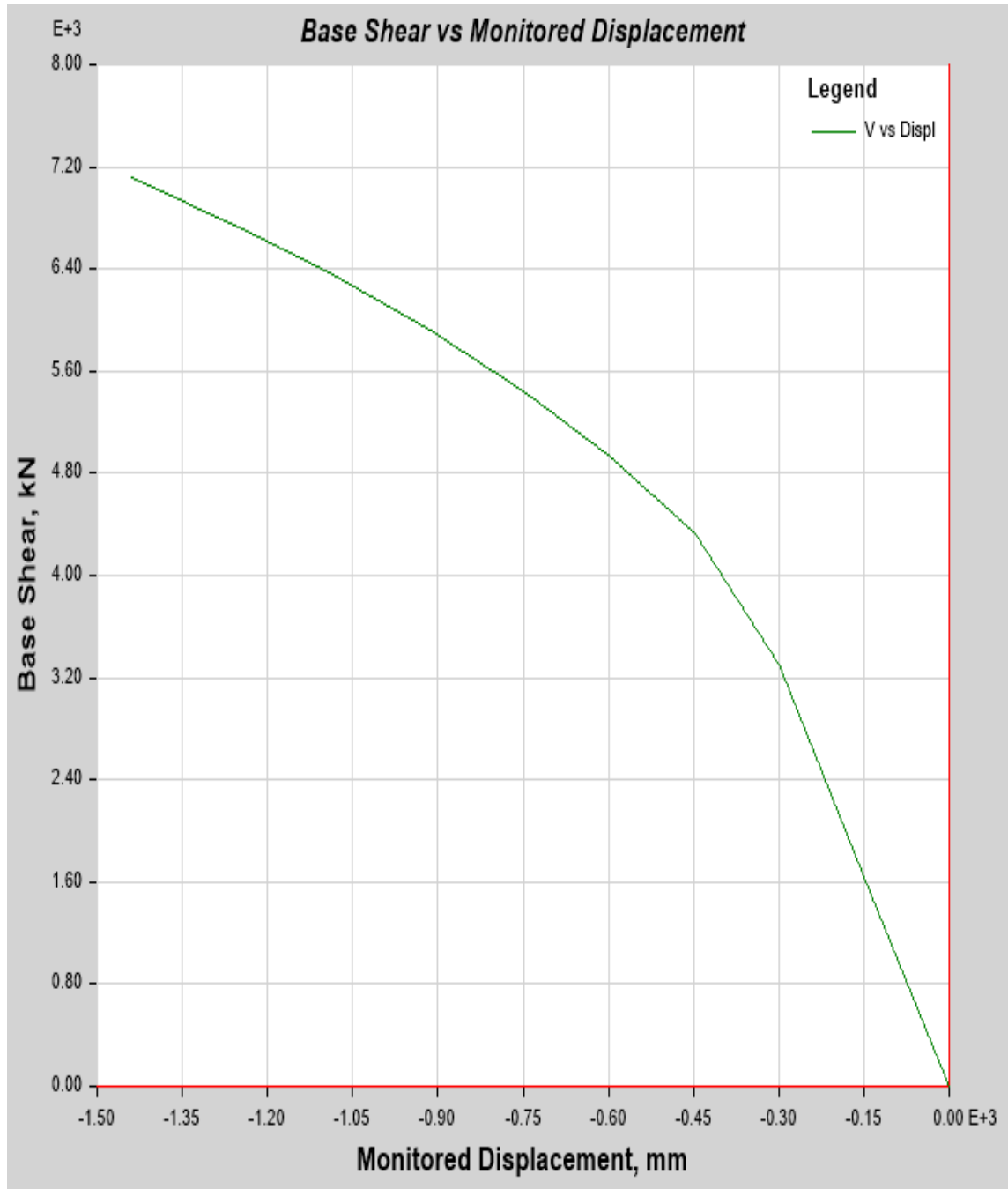
Graph5.5 Pushover Curve Dir X for SMRF, Progressive Collapse Eq Zone 3- Base Shear Vs Monitored Displacement

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total Hinges
	mm	kN										
0	0	0	1920	0	0	0	0	1920	0	0	0	1920
1	-144	2203.403	1920	0	0	0	0	1920	0	0	0	1920
2	-288	4407.315	1918	2	0	0	0	1920	0	0	0	1920
3	-291.144	4455.153	1910	10	0	0	0	1920	0	0	0	1920
4	-437.526	5989.607	1556	364	0	0	0	1920	0	0	0	1920
5	-582.269	6849.507	1408	512	0	0	0	1920	0	0	0	1920
6	-745.933	7520.041	1276	644	0	0	0	1724	196	0	0	1920
7	-898.586	8018.652	1238	682	0	0	0	1560	360	0	0	1920
8	-1079.05	8530.792	1190	730	0	0	0	1458	462	0	0	1920
9	-1338.90	9200.29	1156	764	0	0	0	1294	626	0	0	1920
10	-1440	9438.920	1150	770	0	0	0	1282	638	0	0	1920

Table 5.5 Base Shear Vs Monitored Displacement Push X Case SMRF With Progressive Collapse At Zone 3

The pushover analysis was performed to evaluate the seismic performance of the SMRF structure in Zone 3. The capacity curve shows that the base shear increases with displacement and reaches a maximum value of about 9439 kN. The hinge formation initially occurs in the A–B (elastic) range and gradually progresses to B–C and IO–LS levels as displacement increases. Most hinges remain within acceptable performance limits and none reach the Collapse Prevention (CP) stage.

After Analyzing The SMRF For Progressive Collapse, EQ Zone 3 Frame Following Results Are Obtained For Push-Y Condition



Graph .5.6 Pushover Curve Dir.Y For Smrf, Progressive Collapse Eq.Zone 3 - Base Shear Vs Monitored Displacement

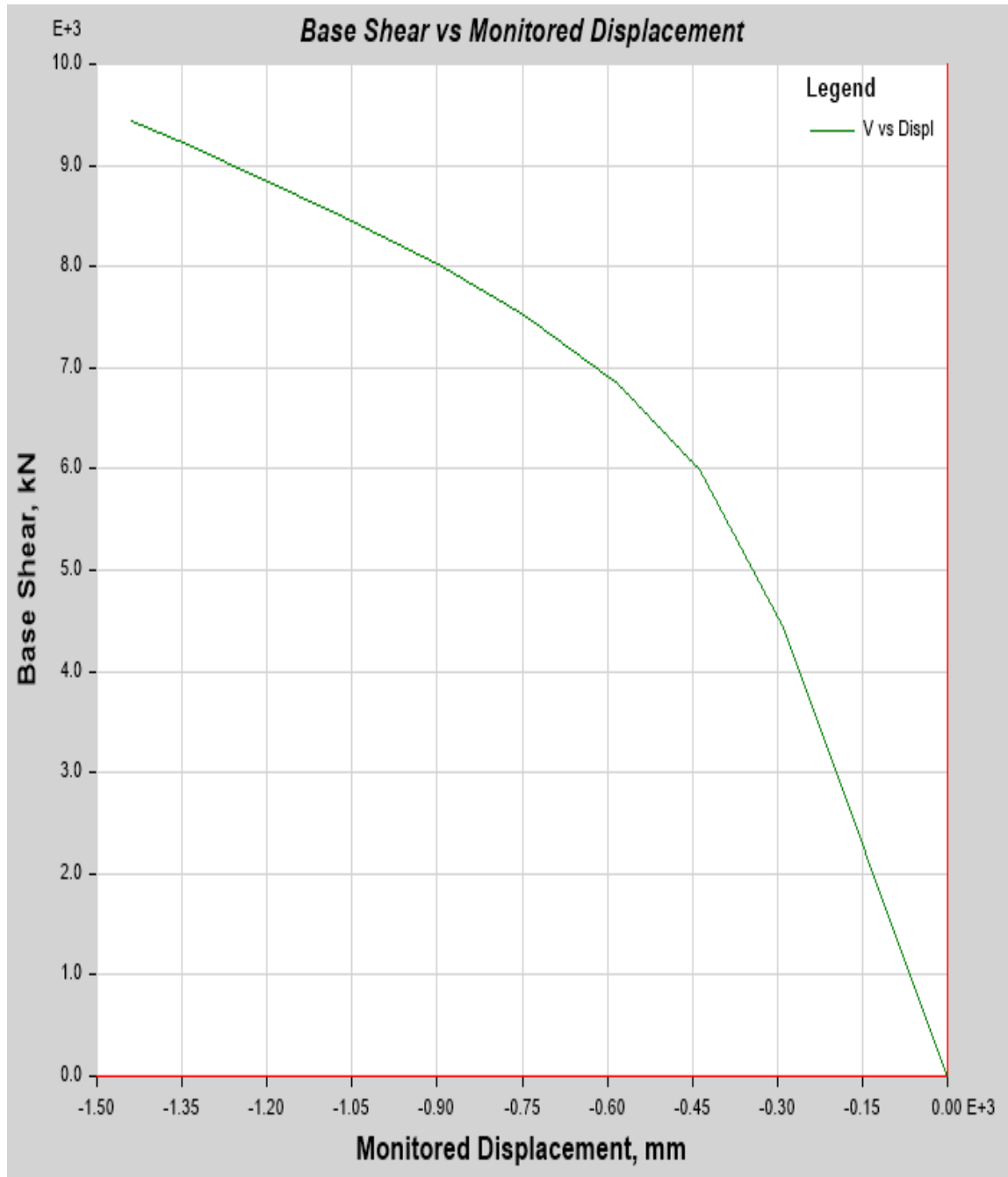
Step	Monitor ed Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS - CP	>CP	Total Hinges
	mm	kN										
0	0	0	1920	0	0	0	0	1920	0	0	0	1920
1	-144	1583.93	1920	0	0	0	0	1920	0	0	0	1920
2	-288	3167.865	1920	0	0	0	0	1920	0	0	0	1920
3	-300.093	3300.880	1904	16	0	0	0	1920	0	0	0	1920
4	-445.607	4323.15	1582	338	0	0	0	1920	0	0	0	1920
5	-597.397	4934.82	1440	480	0	0	0	1920	0	0	0	1920
6	-744.566	5420.61	1374	546	0	0	0	1768	152	0	0	1920
7	-914.97	5915.78	1348	572	0	0	0	1560	360	0	0	1920
8	-1092.35	6379.361	1258	662	0	0	0	1480	440	0	0	1920
9	-1244.36	6717.68	1220	700	0	0	0	1468	452	0	0	1920
10	-1440	7115.021	1200	720	0	0	0	1374	546	0	0	1920

Table 5.6 Base Shear Vs Monitored Displacement Push Y Case SMRF With Progressive Collapse At Zone 3

The pushover analysis was performed to evaluate the seismic performance of the SMRF structure in Zone 3.

The capacity curve shows that the base shear increases with displacement and reaches a maximum value of about 7115 kN. The hinge formation initially occurs in the A–B (elastic) range and gradually progresses to B–C and IO–LS levels as displacement increases. Most hinges remain within acceptable performance limits and none reach the Collapse Prevention (CP) stage.

After Analyzing The OMRF For Progressive Collapse, EQ. ZONE 3 Frame Following Results Are Obtained For Push-X Condition



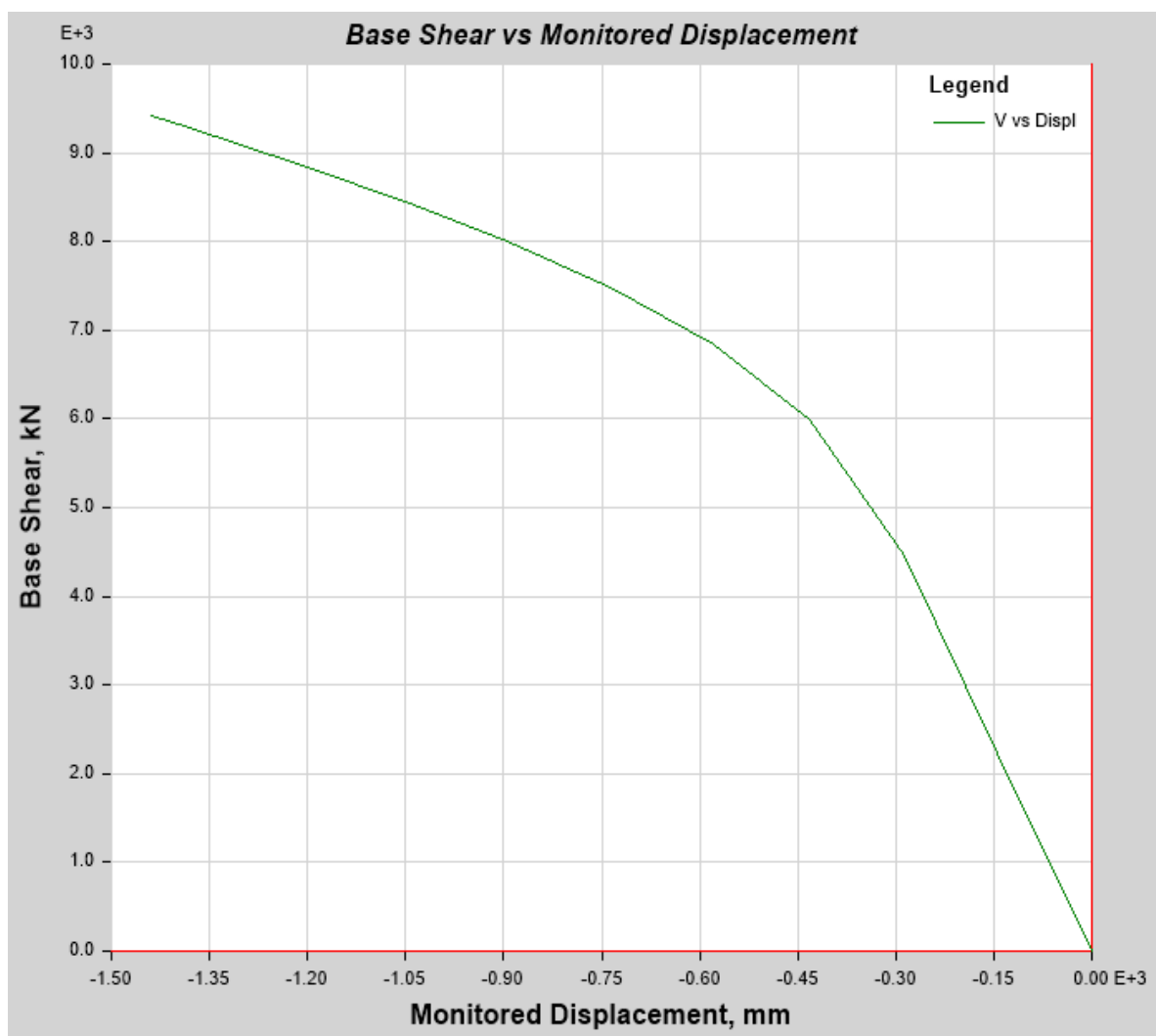
Graph5.7 Pushover Curve Dir.X For OMRF, Progressive Collapse EQ. ZONE 3 - Base Shear Vs Monitored Displacement

Step	Monitored Displ mm	Base Force kN	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>C P	Total Hinges
0	0	0	1920	0	0	0	0	1920	0	0	0	1920
1	-144	2203.403	1920	0	0	0	0	1920	0	0	0	1920
2	-288	4407.315	1918	2	0	0	0	1920	0	0	0	1920
3	-291.144	4455.153	1910	10	0	0	0	1920	0	0	0	1920
4	-437.526	5989.607	1556	364	0	0	0	1920	0	0	0	1920
5	-582.269	6849.507	1408	512	0	0	0	1920	0	0	0	1920
6	-745.933	7520.041	1276	644	0	0	0	1724	196	0	0	1920
7	-898.586	8018.652	1238	682	0	0	0	1560	360	0	0	1920

8	-1079.051	8530.792	1190	730	0	0	0	1458	462	0	0	1920
9	-1338.904	9200.27	1156	764	0	0	0	1294	626	0	0	1920
10	-1440	9438.92	1150	770	0	0	0	1282	638	0	0	1920

The pushover analysis was performed to evaluate the seismic performance of the OMRF structure in Zone 3 for progressive collapse. The capacity curve shows that the base shear increases with displacement and reaches a maximum value of about 9438 kN. The hinge formation initially occurs in the A–B (elastic) range and gradually progresses to B–C and IO–LS levels as displacement increases. Most hinges remain within acceptable performance limits and none reach the Collapse Prevention (CP) stage.

After Analyzing The OMRF For Progressive Collapse ,EQ.ZONE 3 Frame Following Results Are Obtained For Push-Y Condition



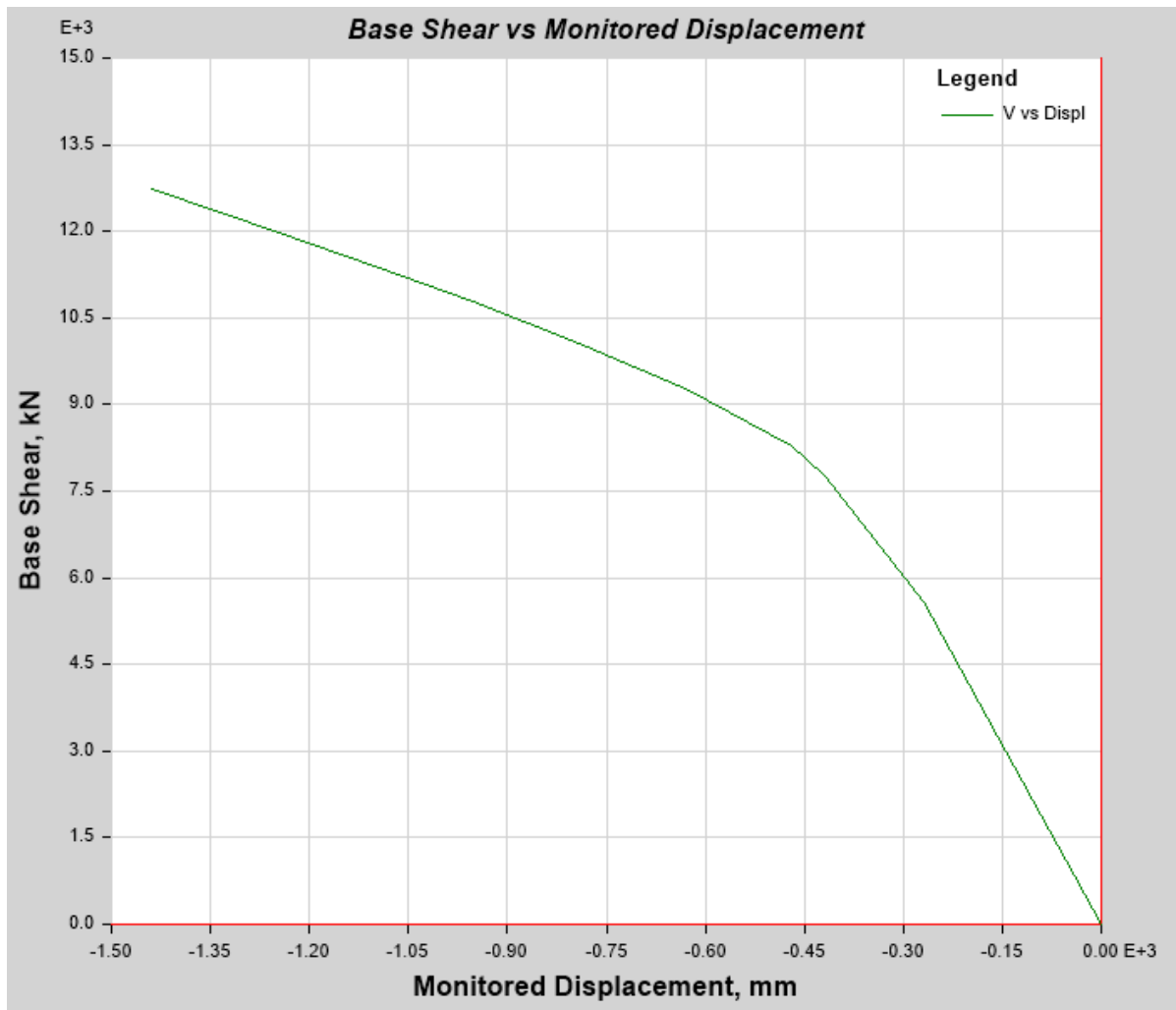
Graph 5.8 Pushover Curve Dir Y For OMRF, Progressive Collapse EQ.ZONE 3 - Base Shear Vs Monitored Displacement

Step	Monitored Displ mm	Base Force kN	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total Hinges
0	0	0	1920	0	0	0	0	1920	0	0	0	1920
1	-144	2230.245	1920	0	0	0	0	1920	0	0	0	1920
2	-287.796	4456.882	1908	12	0	0	0	1920	0	0	0	1920
3	-432.466	5988.464	1572	348	0	0	0	1920	0	0	0	1920
4	-578.954	6843.396	1404	516	0	0	0	1920	0	0	0	1920
5	-736.79	7488.067	1282	638	0	0	0	1726	194	0	0	1920
6	-901.163	8017.508	1244	676	0	0	0	1560	360	0	0	1920
7	-1051.92	8443.056	1190	730	0	0	0	1478	442	0	0	1920
8	-1208.247	8851.543	1170	750	0	0	0	1396	524	0	0	1920
9	-1380.886	9276.087	1148	772	0	0	0	1280	640	0	0	1920
10	-1440	9412.564	1142	778	0	0	0	1278	642	0	0	1920

Table 5.8 Base Shear vs Monitored Displacement Push Y case OMRF with progressive collapse at Zone 3

shover analysis was performed to evaluate the seismic performance of the OMRF structure in Zone 3 for progressive collapse. The capacity curve shows that the base shear increases with displacement and reaches a maximum value of about 9412 kN. The hinge formation initially occurs in the A–B (elastic) range and gradually progresses to B–C and IO–LS levels as displacement increases. Most hinges remain within acceptable performance limits and none reach the Collapse Prevention (CP) stage.

After analyzing the OMRF for progressive collapse with bracing ,EQ.ZONE 3 frame following results are obtained for Push-X condition



Graph5.9 Pushover Curve Dir.X For OMRF, Progressive Collapse With Bracing EQ.ZONE 3 - Base Shear Vs Monitored Displacement

Step	Monitored Displ mm	Base Force kN	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP
0	0	0	1920	0	0	0	0	1920	0	0
1	-144	2988.825	1920	0	0	0	0	1920	0	0
2	-268.315	5569.058	1914	6	0	0	0	1920	0	0
3	-418.347	7775.721	1648	272	0	0	0	1920	0	0
4	-469.244	8278.851	1554	366	0	0	0	1920	0	0
5	-473.758	8308.03	1548	372	0	0	0	1920	0	0
6	-624.415	9255.509	1406	514	0	0	0	1864	56	0
7	-779.552	10010.63	1372	548	0	0	0	1688	232	0
8	-950.569	10773.08	1348	572	0	0	0	1512	408	0
9	-1100.581	11394.23	1304	616	0	0	0	1472	448	0
10	-1245.124	11976.55	1288	632	0	0	0	1444	476	0
11	-1440	12727.26	1272	648	0	0	0	1400	520	0

The pushover analysis was performed to evaluate the seismic performance of the OMRF structure in Zone 3 for progressive collapse. The capacity curve shows that the base shear increases with displacement and reaches a maximum value of about 12227 kN. The hinge formation initially occurs in the A–B (elastic) range and gradually progresses to B–C and IO–LS levels as displacement increases. Most hinges remain within acceptable performance limits and none reach the Collapse Prevention (CP) stage.

This indicates that the structure exhibits ductile behavior and adequate energy dissipation capacity. Therefore, the building shows satisfactory seismic performance and stability under lateral loading.

After Analyzing The OMRF For Progressive Collapse With Bracing ,EQ.ZONE 3 Frame Following Results Are Obtained For Push Y Condition

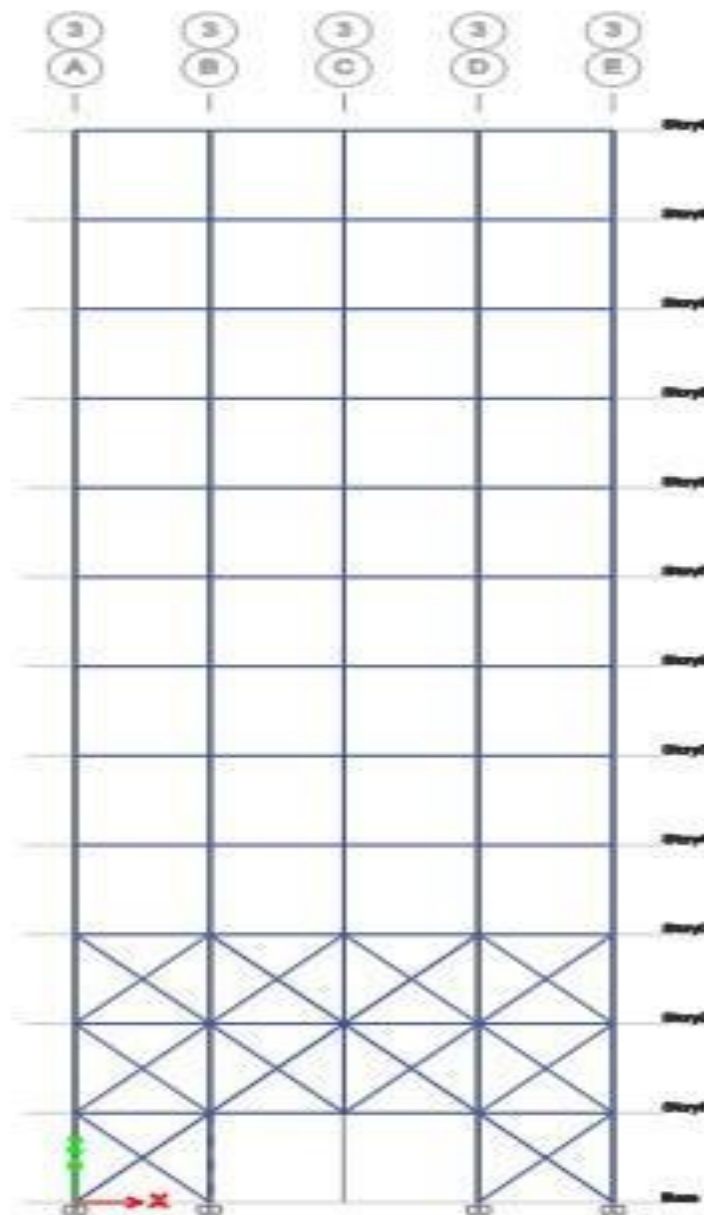
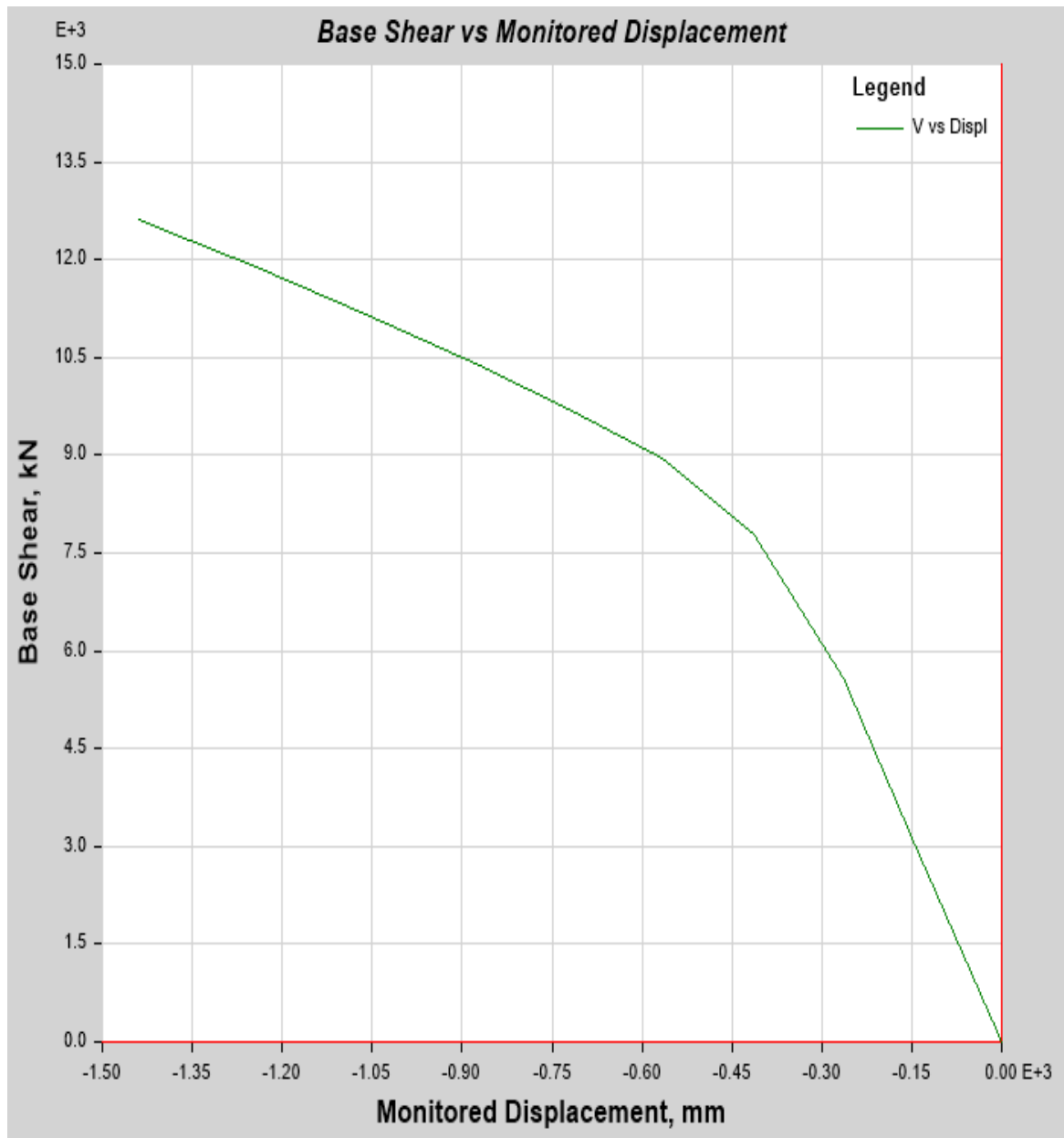


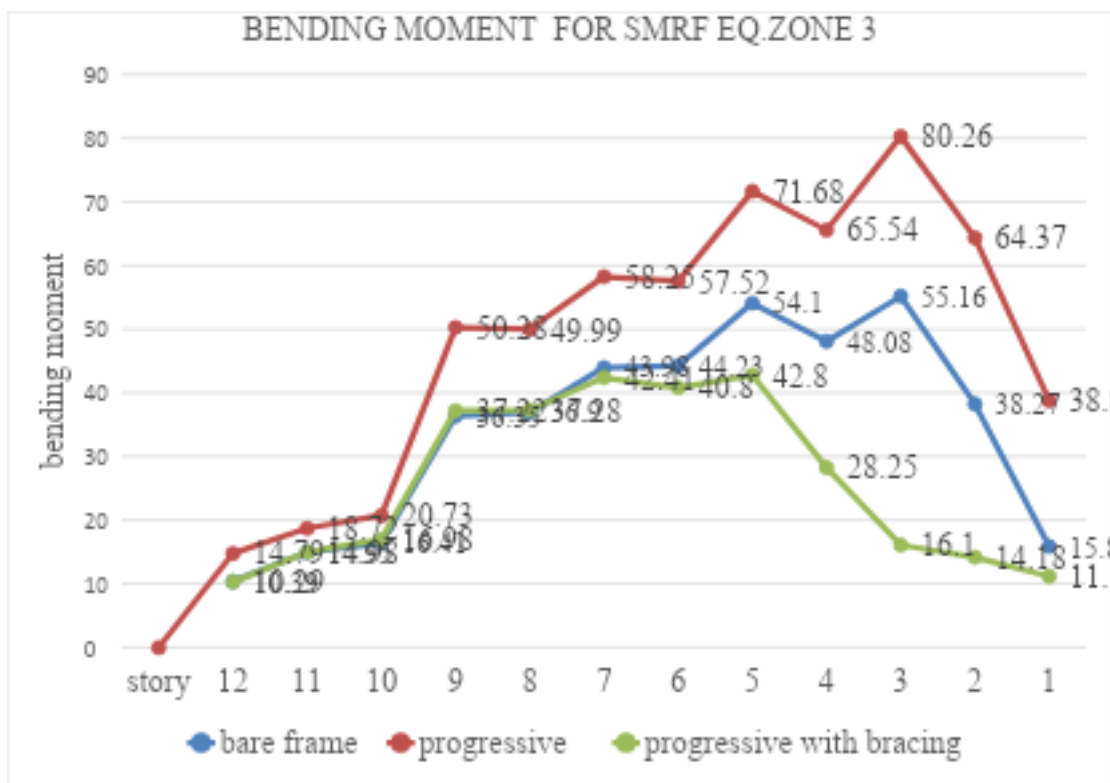
Figure 5.3 Elevation View of Section 3-3 When Bracing Add to The Frame Building



Graph 5.10 Pushover Curve Dir. Y For OMRF, Progressive Collapse With Bracing EQ. Zone 3 - Base Shear Vs Monitored Displacement

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total Hinges
	mm	kN										
0	0	0	1920	0	0	0	0	1920	0	0	0	1920
1	-144	3037.5209	1920	0	0	0	0	1920	0	0	0	1920
2	-264.283	5574.752	1912	8	0	0	0	1920	0	0	0	1920
3	-412.832	7779.9961	1632	288	0	0	0	1920	0	0	0	1920
4	-567.147	8940.4237	1444	476	0	0	0	1920	0	0	0	1920
5	-711.319	9656.7108	1380	540	0	0	0	1768	152	0	0	1920
6	-869.778	10380.6532	1356	564	0	0	0	1522	398	0	0	1920
7	-1060.43	11155.0785	1330	590	0	0	0	1468	452	0	0	1920
8	-1244.074	11885.4666	1296	624	0	0	0	1444	476	0	0	1920
9	-1392.601	12449.6342	1270	650	0	0	0	1416	504	0	0	1920
10	-1440	12625.4627	1268	652	0	0	0	1402	518	0	0	1920

Conclusion for Bending Moment (12-Story Steel Structure)



Graph 5.11 Variation of Bending Moment For SMRF Eq.Zone 3.

Conclusion for Bending Moment (12-Story Steel Structure)

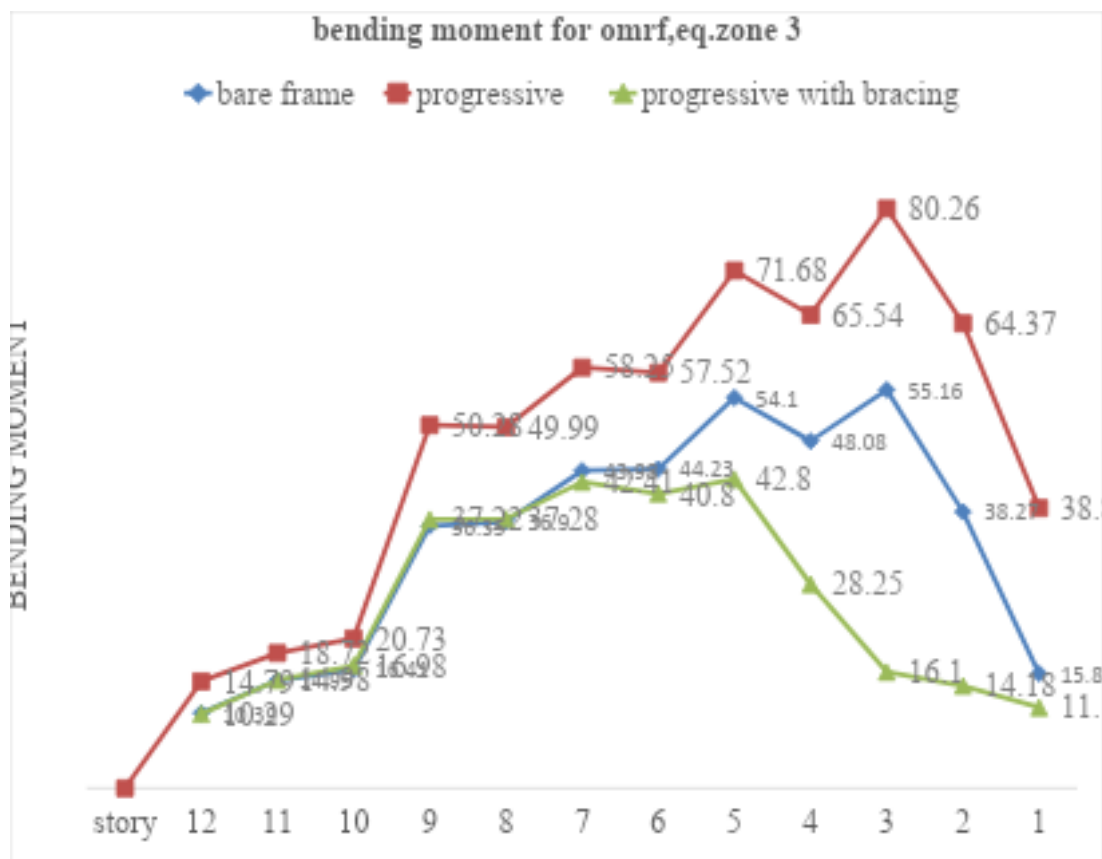
Maximum bending moment occurs in the middle storeys (around 3rd–5th storey). This indicates that seismic forces concentrate in the mid-height of the building rather than at the top.

The progressive collapse for SMRF frame shows the highest bending moments.

The peak value is approximately at around the 3rd storey, which means this configuration experiences the greatest internal forces and demands stronger member design.

The bare frame shows moderate bending moment values.

The progressive collapse with braced frame shows the lowest bending moment values. Overall structural behavior



Graph 5.12 Variation of Bending Moment For OMRF Eq.Zone 3.

Conclusion for Bending Moment (12-Story Steel Structure)

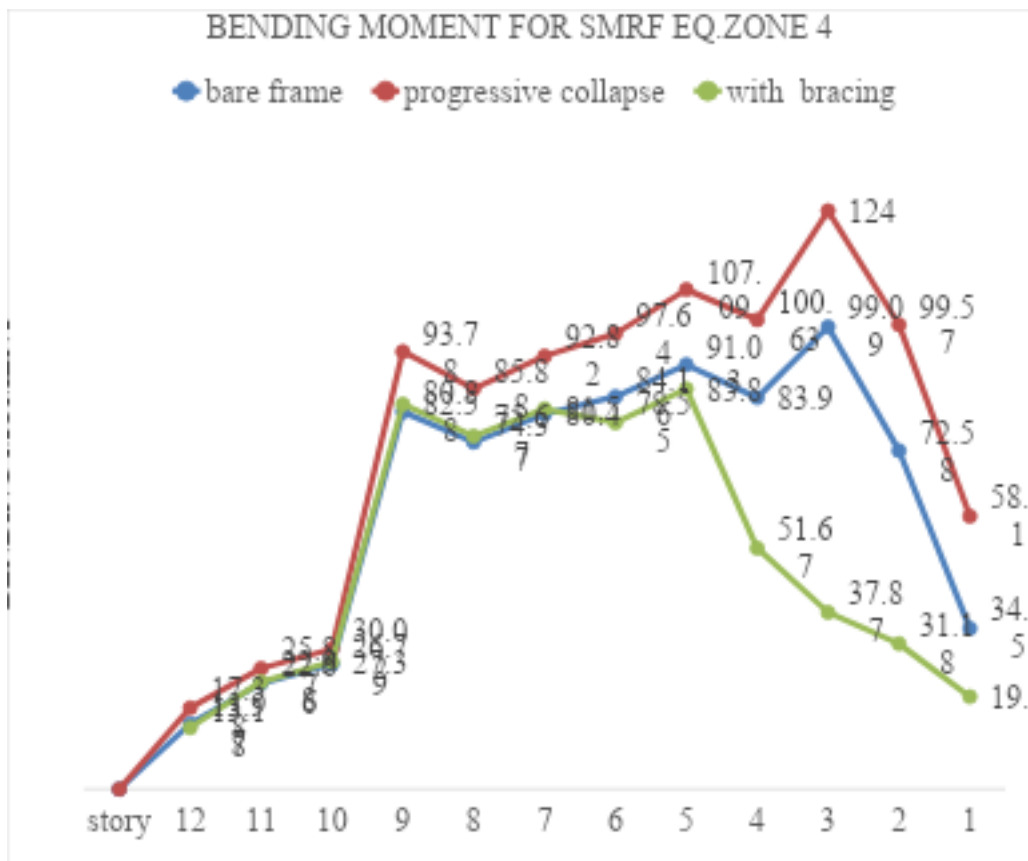
Maximum bending moment occurs in the middle story (around 3rd–5th story). This indicates that seismic forces concentrate in the mid-height of the building rather than at the top.

The progressive collapse for OMRF frame shows the highest bending moments.

The peak value is approximately at around the 3rd story, which means this configuration experiences the greatest internal forces and demands stronger member design.

The bare frame shows moderate bending moment values.

The progressive collapse with braced frame shows the lowest bending moment values. Overall structural behavior



Graph 5.13 Variation of Bending Moment For SMRF Eq.Zone 4.

Conclusion for Bending Moment (12-Story Steel Structure)

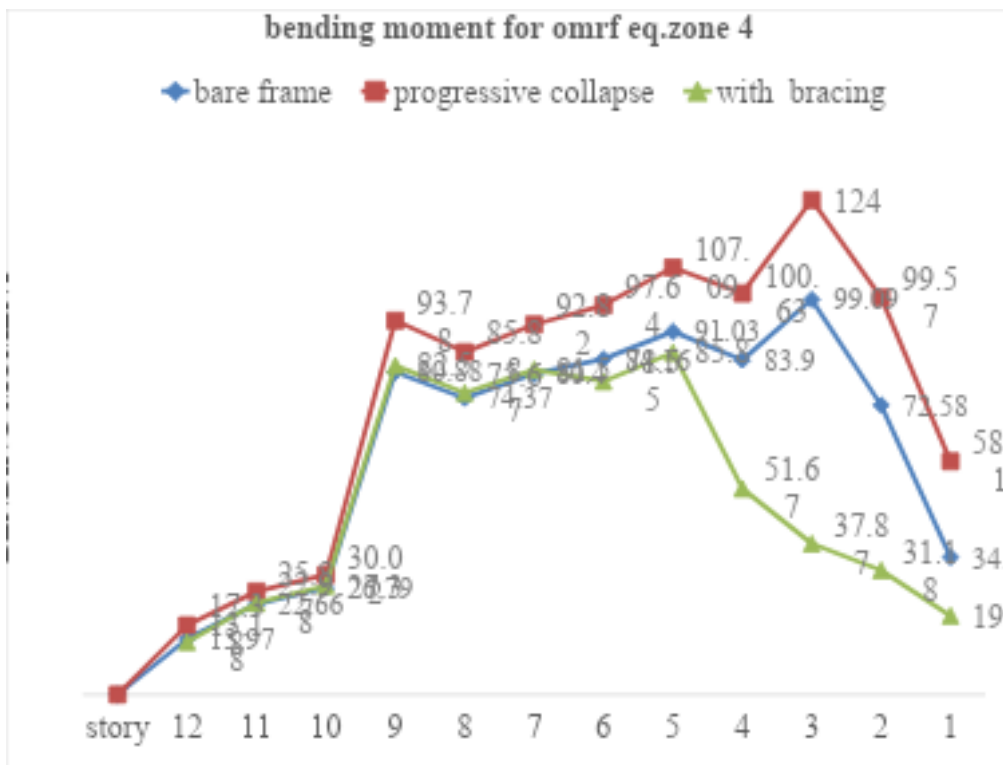
Maximum bending moment occurs in the middle story (around 3rd–5th story). This indicates that seismic forces concentrate in the mid-height of the building rather than at the top.

The progressive collapse for OMRF frame shows the highest bending moments.

The peak value is approximately at around the 3rd story, which means this configuration experiences the greatest internal forces and demands stronger member design.

The bare frame shows moderate bending moment values.

The progressive collapse with braced frame shows the lowest bending moment values. Overall structural behavior



Graph 5.14 Variation of Bending Moment For OMRF Eq.Zone 4.

Conclusion for Bending Moment (12-Story Steel Structure)

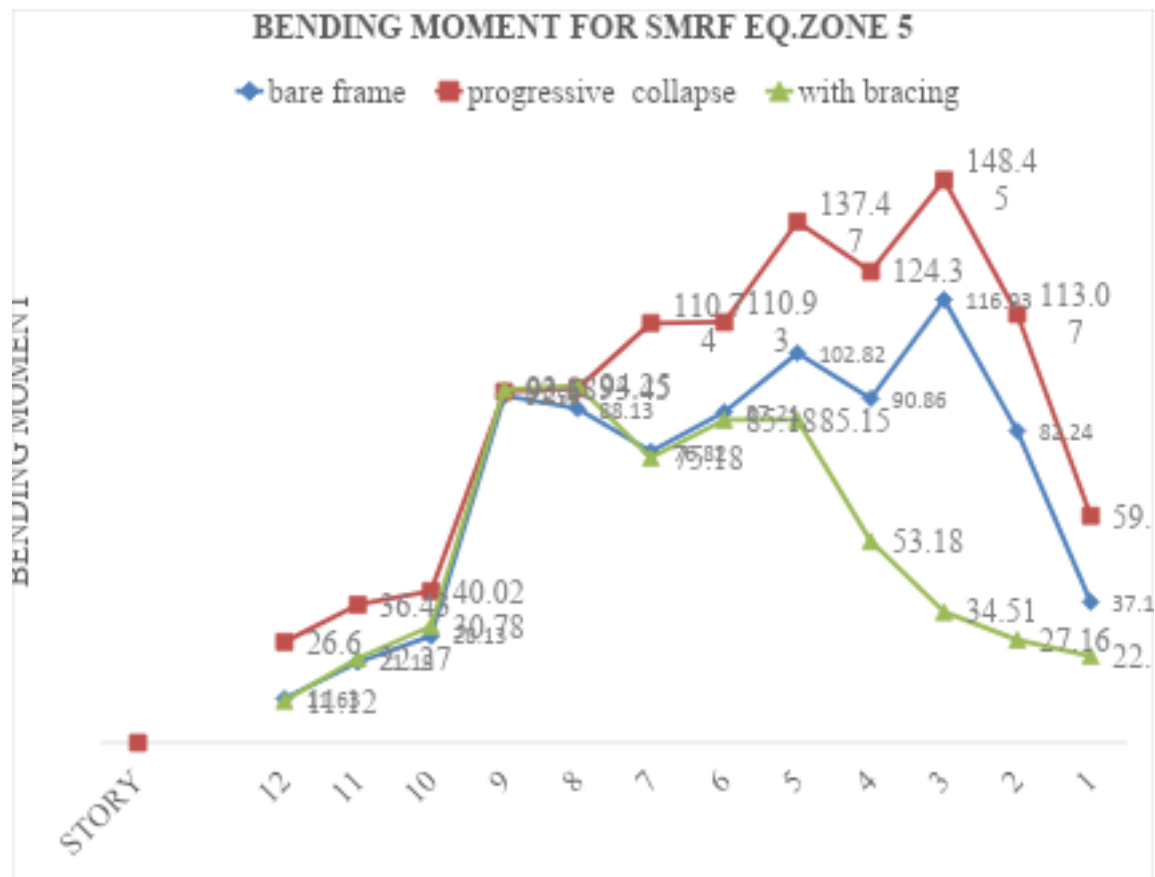
Maximum bending moment occurs in the middle story (around 3rd–5th story). This indicates that seismic forces concentrate in the mid-height of the building rather than at the top.

The progressive collapse for OMRF frame shows the highest bending moments.

The peak value is approximately at around the 3rd story, which means this configuration experiences the greatest internal forces and demands stronger member design.

The bare frame shows moderate bending moment values.

The progressive collapse with braced frame shows the lowest bending moment values. Overall structural behavior



Graph 5.15 Variation of Bending Moment For SMRF Eq.Zone 5.

Conclusion for Bending Moment (12-Story Steel Structure)

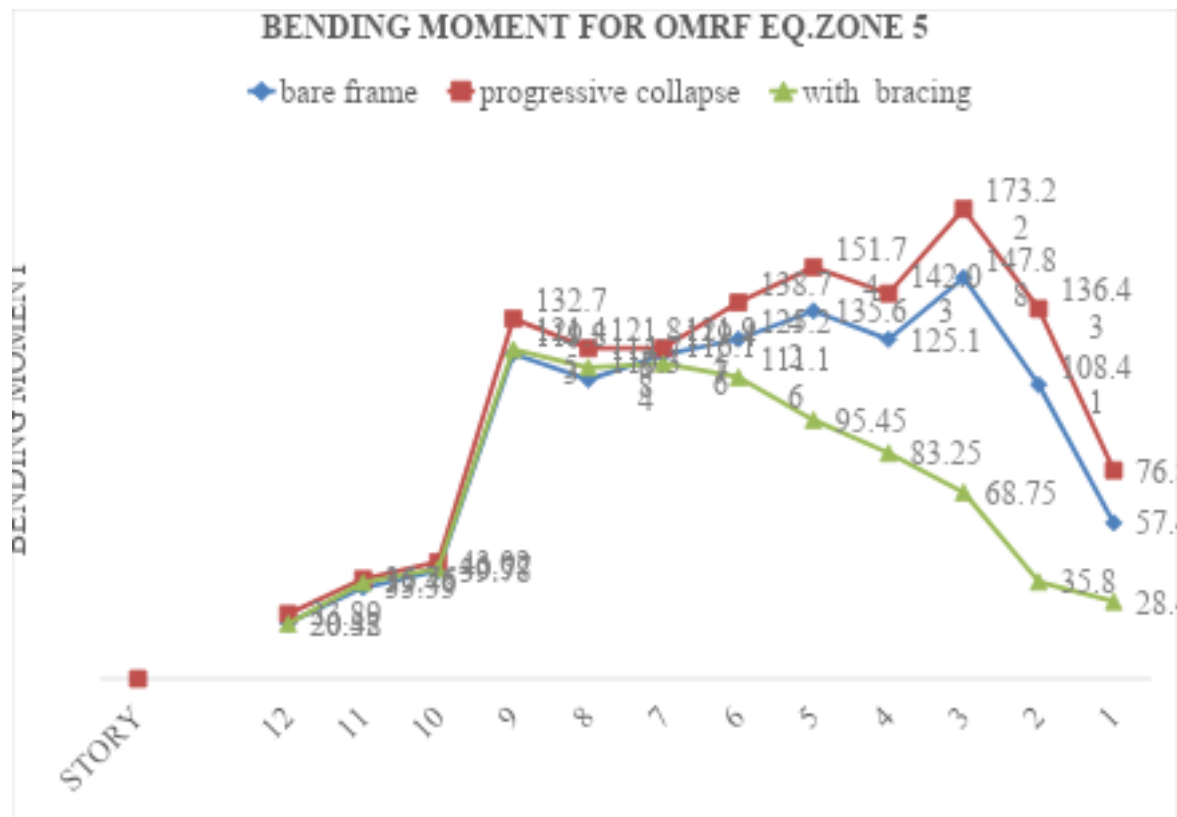
Maximum bending moment occurs in the middle story (around 3rd–5th story). This indicates that seismic forces concentrate in the mid-height of the building rather than at the top.

The progressive collapse for OMRF frame shows the highest bending moments.

The peak value is approximately at around the 3rd story, which means this configuration experiences the greatest internal forces and demands stronger member design.

The bare frame shows moderate bending moment values.

The progressive collapse with braced frame shows the lowest bending moment values. Overall structural behavior



Graph 5.16 Variation Of Bending Moment For OMRF Eq.Zone 5.

Conclusion for Bending Moment (12-Story Steel Structure)

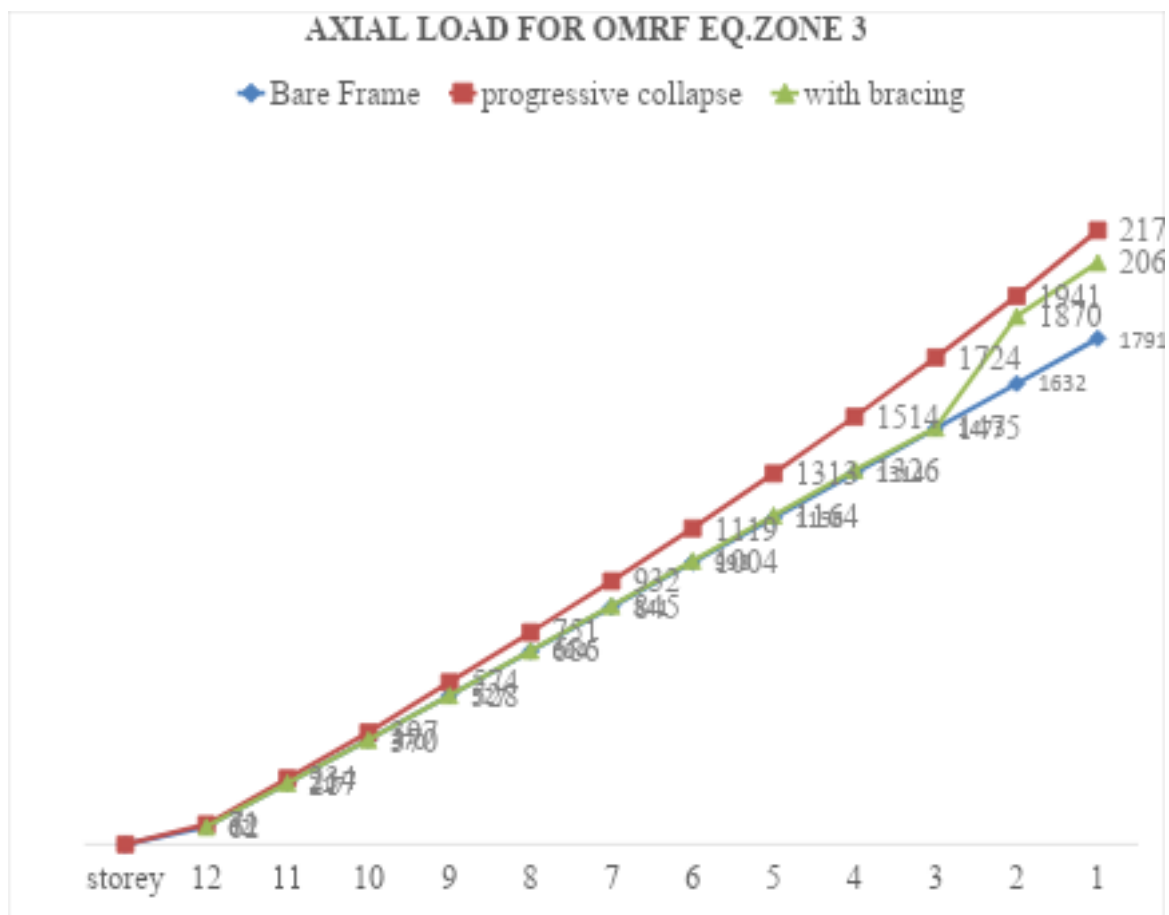
Maximum bending moment occurs in the middle story (around 3rd–5th story). This indicates that seismic forces concentrate in the mid-height of the building rather than at the top.

The progressive collapse for OMRF frame shows the highest bending moments.

The peak value is approximately at around the 3rd story, which means this configuration experiences the greatest internal forces and demands stronger member design.

The bare frame shows moderate bending moment values.

The progressive collapse with braced frame shows the lowest bending moment values. Overall structural behavior



Graph 5.17 Variation of Axial Load For OMRF Eq.Zone 3.

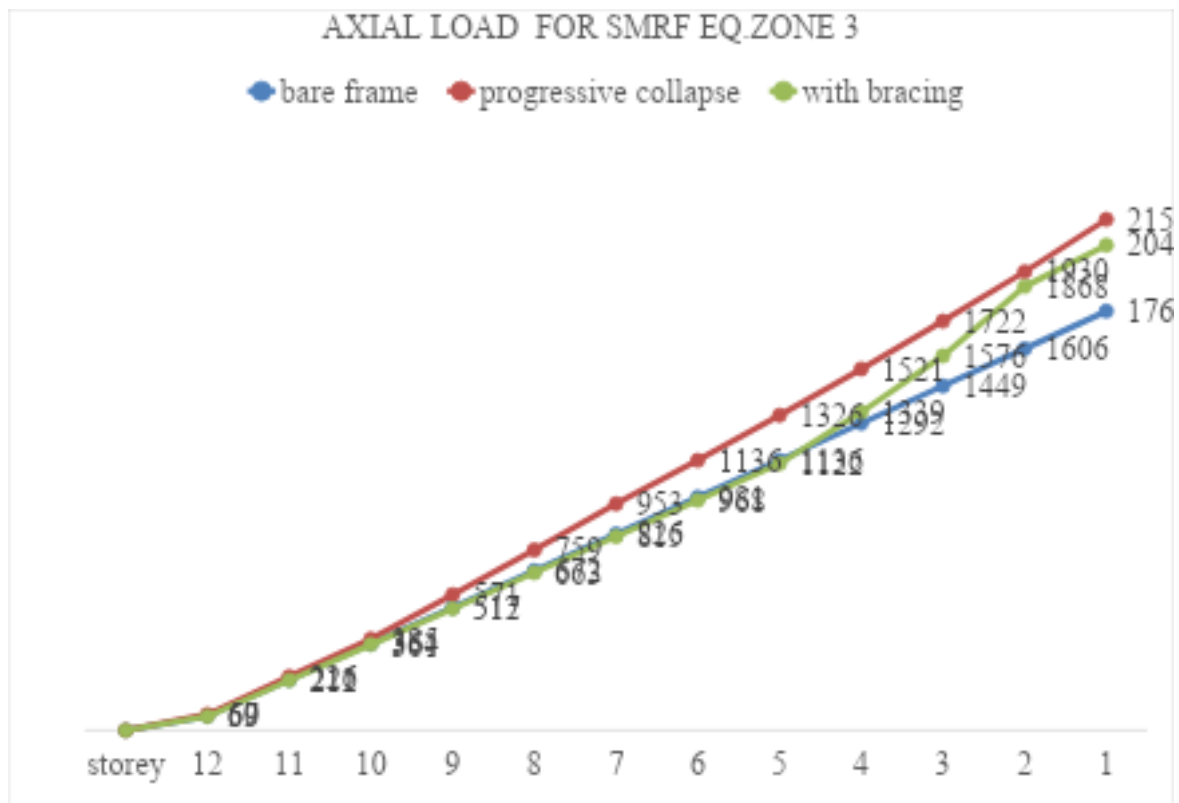
Conclusion for Axial Force (12-Storey Steel Structure)

The axial force in the OMRF structure increases from the top story to the bottom story due to the accumulation of loads.

The progressive collapse condition shows the highest axial forces compared to the other cases.

he braced frame reduces excessive load concentration and improves structural stability.

The bare frame shows comparatively lower axial forces but provides less lateral resistance. Therefore, providing bracing enhances the overall seismic performance and safety of the structure in Zone 3.



Graph 5.18 Variation of Axial Load For SMRF Eq.Zone 3.

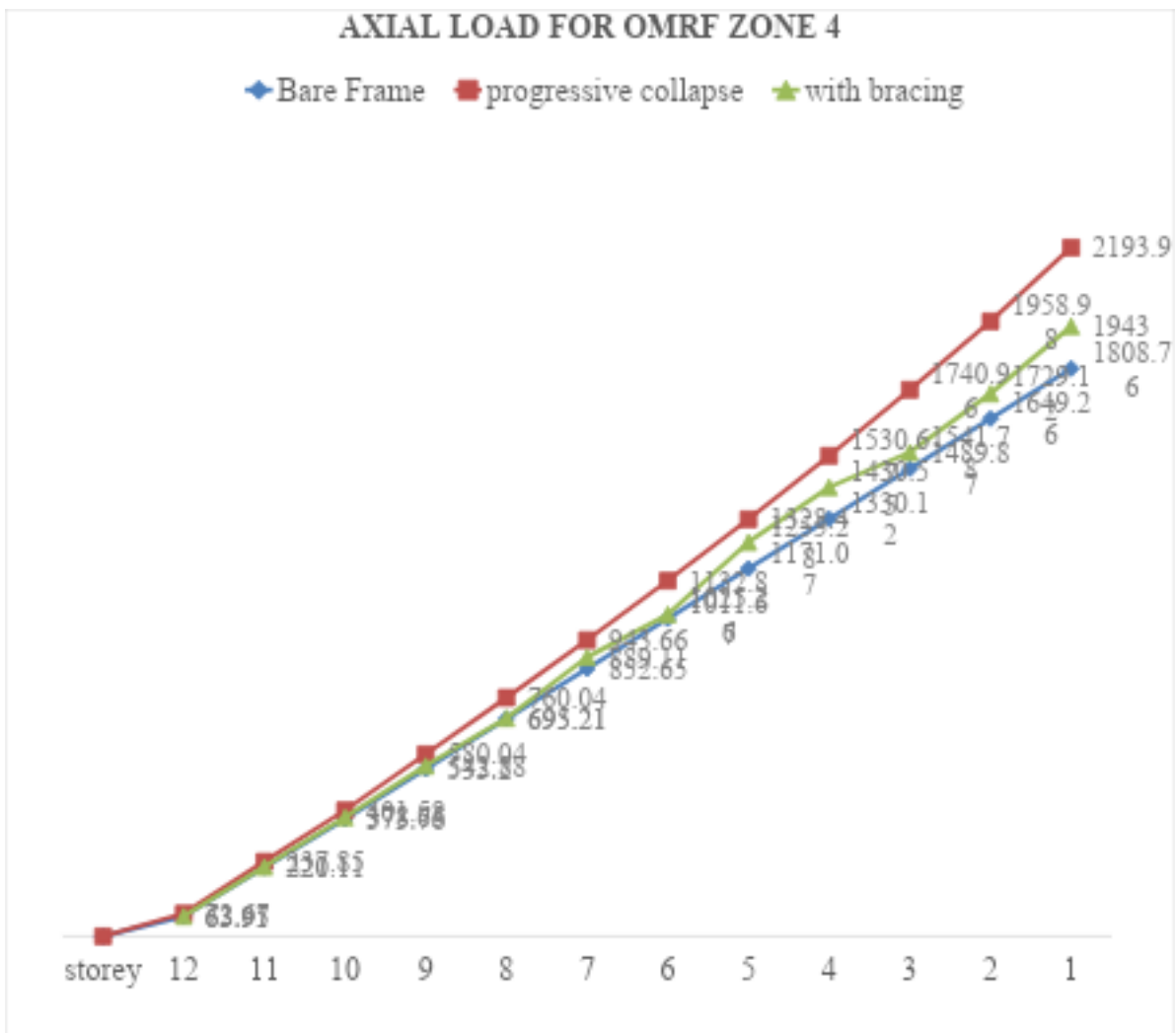
Conclusion for Axial Force (12-Storey Steel Structure)

The axial force in the SMRF structure increases from the top story to the bottom story due to the accumulation of loads.

The progressive collapse condition shows the highest axial forces compared to the other cases.

The braced frame reduces excessive load concentration and improves structural stability.

The bare frame shows comparatively lower axial forces but provides less lateral resistance. Therefore, providing bracing enhances the overall seismic performance and safety of the structure in Zone 3.



Graph 5.19 Variation Of Axial Load For OMRF Eq.Zone 4.

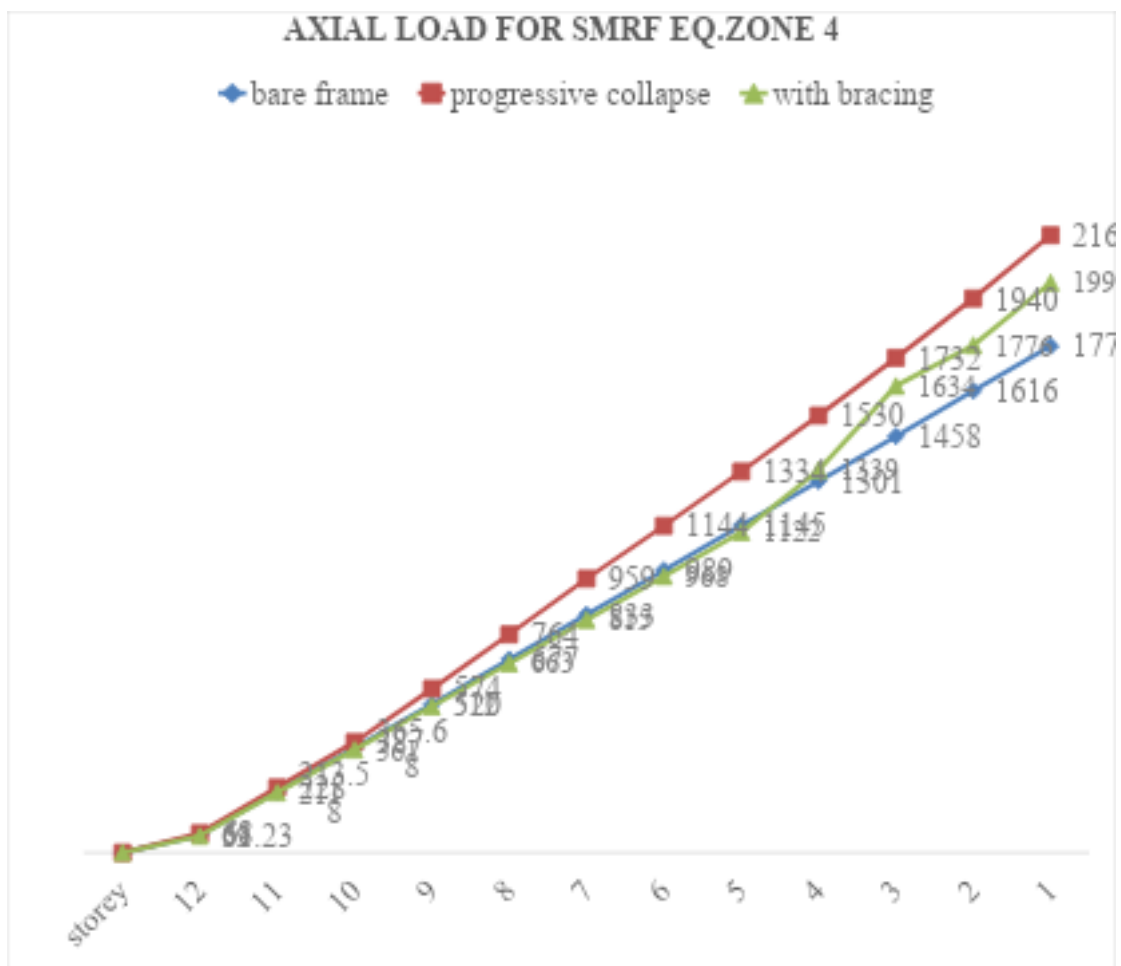
Conclusion for Axial Force (12-Storey Steel Structure)

The axial force in the OMRF structure increases from the top story to the bottom story due to the accumulation of loads.

The progressive collapse condition shows the highest axial forces compared to the other cases.

The braced frame reduces excessive load concentration and improves structural stability.

The bare frame shows comparatively lower axial forces but provides less lateral resistance. Therefore, providing bracing enhances the overall seismic performance and safety of the structure in Zone 4.



Graph 5.20 Variation of Axial Load For SMRF Eq.Zone 4.

Conclusion for Axial Force (12-Storey Steel Structure)

The axial force in the SMRF structure increases from the top story to the bottom story due to the accumulation of loads.

The progressive collapse condition shows the highest axial forces compared to the other cases.

The braced frame reduces excessive load concentration and improves structural stability.

The bare frame shows comparatively lower axial forces but provides less lateral resistance. Therefore, providing bracing enhances the overall seismic performance and safety of the structure in Zone 4.



Graph 5.21 Variation of Axial Load For OMRF Eq. Zone 5.

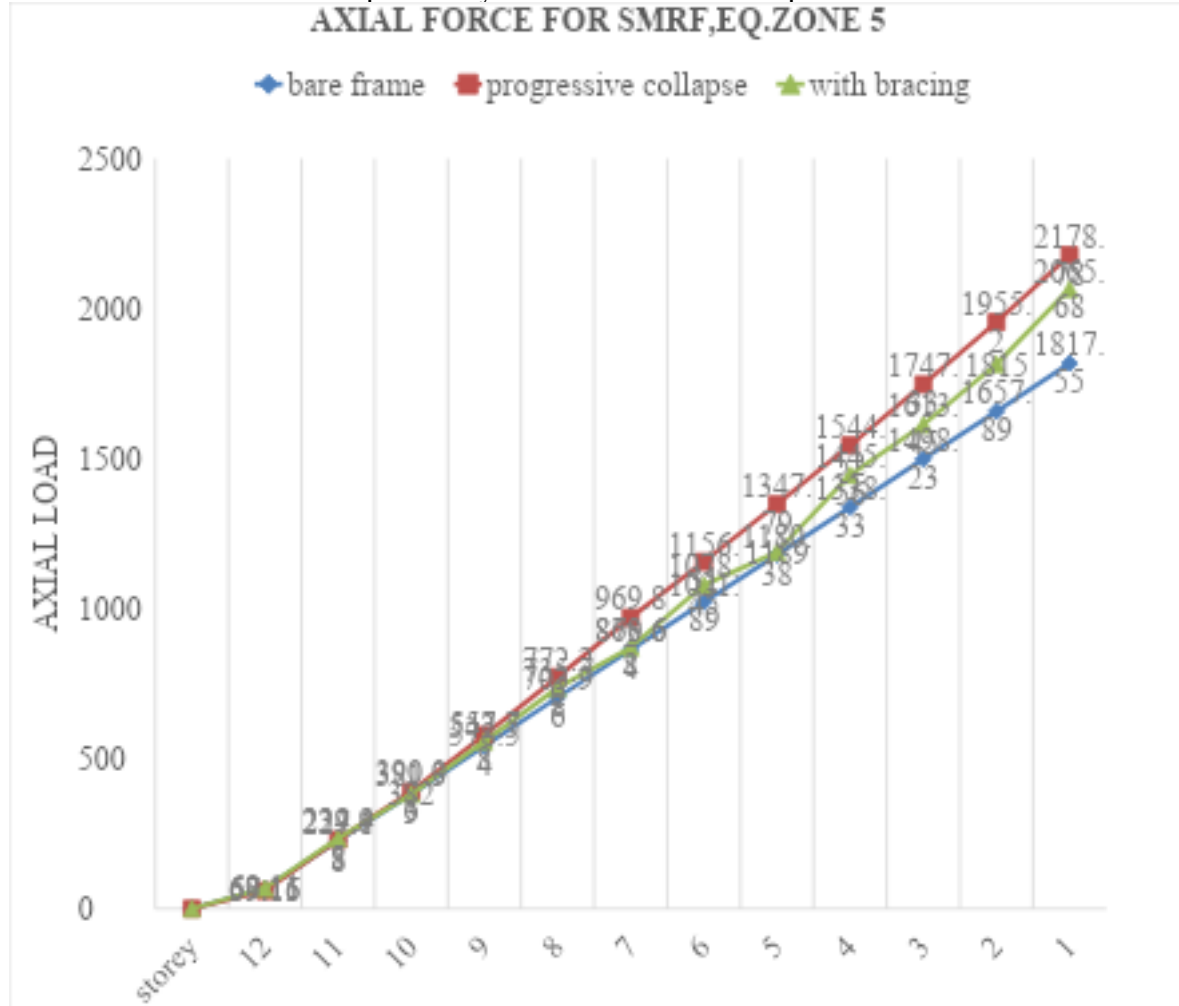
Conclusion for Axial Force (12-Storey Steel Structure)

The axial force in the OMRF structure increases from the top story to the bottom story due to the accumulation of loads.

The progressive collapse condition shows the highest axial forces compared to the other cases.

The braced frame reduces excessive load concentration and improves structural stability.

The bare frame shows comparatively lower axial forces but provides less lateral resistance.



Graph 5.22 Variation of Axial Load For SMRF Eq.Zone 5.

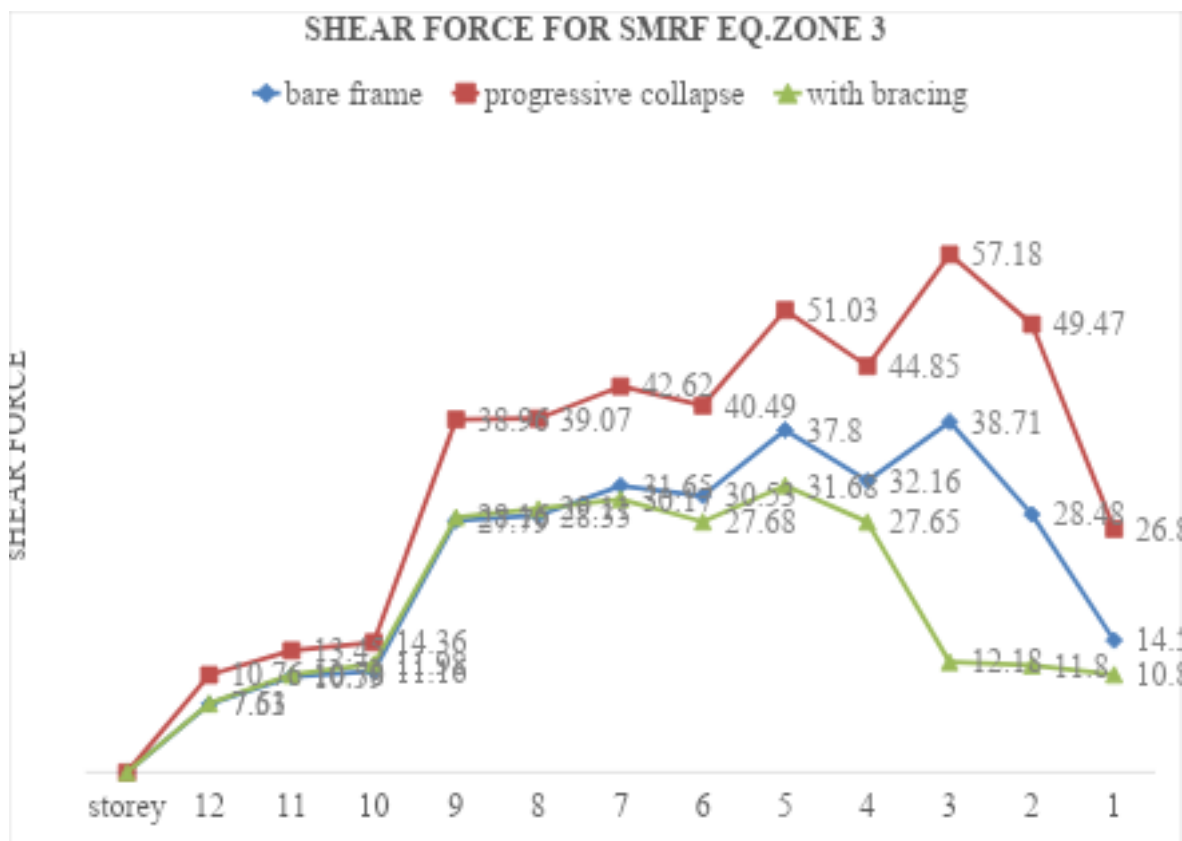
Conclusion for Axial Force (12-Storey Steel Structure)

The axial force in the SMRF structure increases from the top storey to the bottom storey due to the accumulation of loads.

The progressive collapse condition shows the highest axial forces compared to the other cases.

The braced frame reduces excessive load concentration and improves structural stability.

The bare frame shows comparatively lower axial forces but provides less lateral resistance. Therefore, providing bracing enhances the overall seismic performance and safety of the structure in Zone 5.



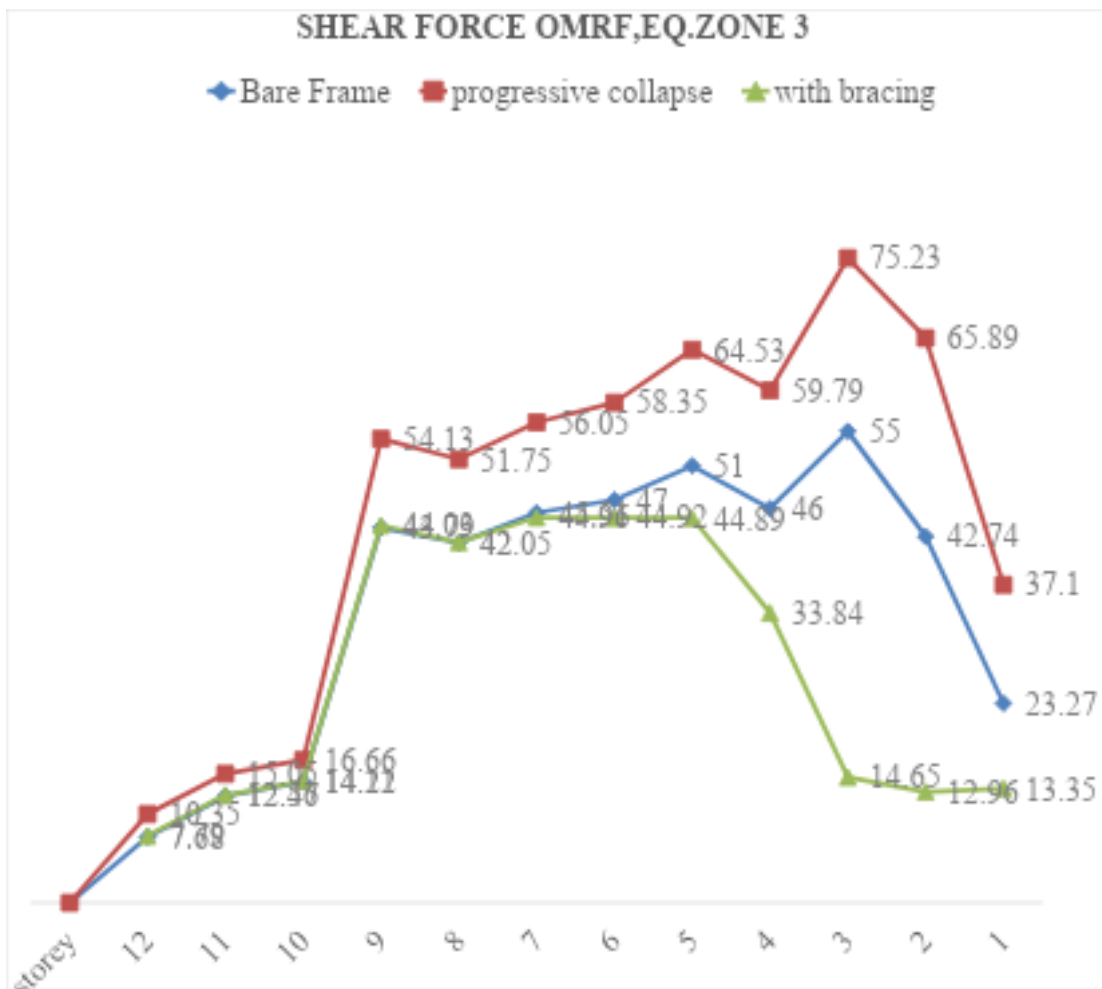
Graph 5.23 Variation of Shear Force For SMRF Eq.Zone 3.

Conclusion for Axial Force (12-Storey Steel Structure)

The analysis of shear force for the SMRF structure in seismic Zone 3 shows that shear force increases from the top story towards the bottom story.

The progressive collapse condition produces the highest shear force compared to the bare frame and braced frame.

The frame with bracing shows reduced shear force, indicating improved lateral stability. Hence, providing bracing in the structure enhances seismic performance and structural safety.



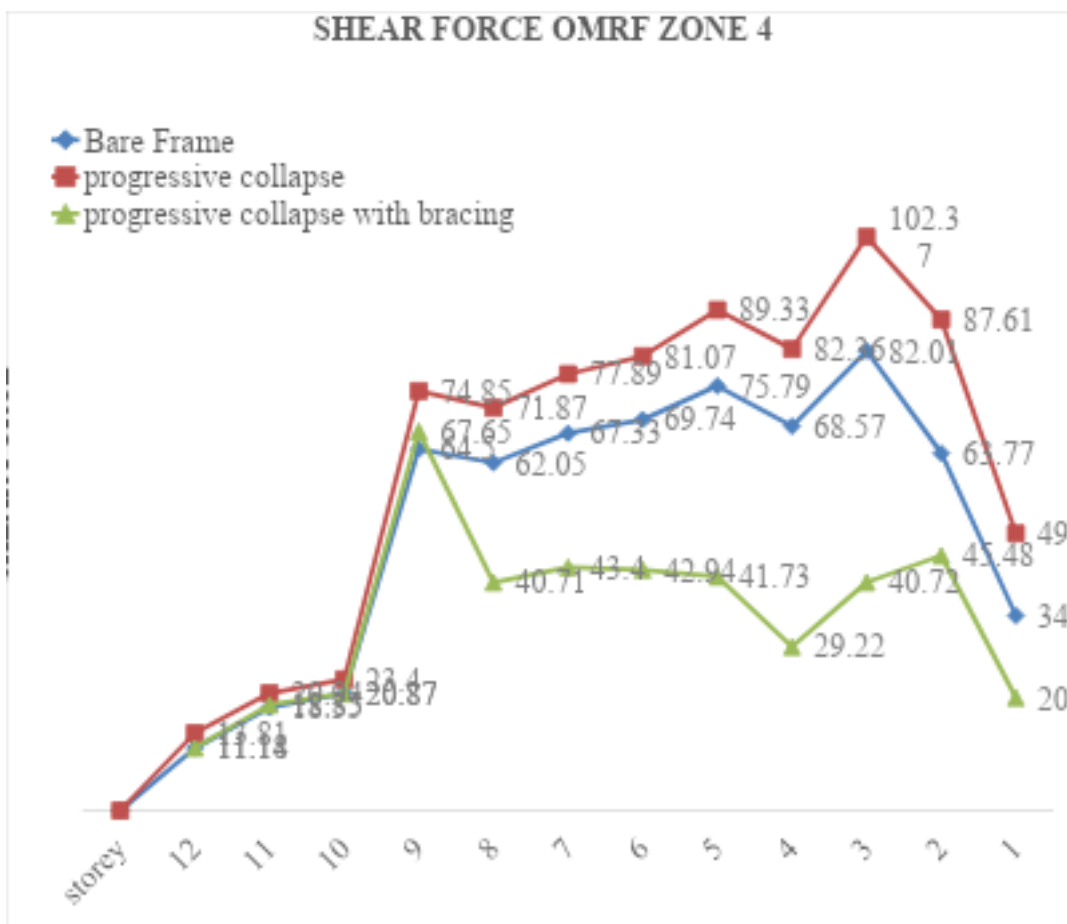
Graph 5.24 Variation of Shear Force For OMRF Eq.Zone 3.

Conclusion for Axial Force (12-Storey Steel Structure)

The analysis of shear force for the SMRF structure in seismic Zone 3 shows that shear force increases from the top story towards the bottom story.

The progressive collapse condition produces the highest shear force compared to the bare frame and braced frame.

The frame with bracing shows reduced shear force, indicating improved lateral stability. Hence, providing bracing in the structure enhances seismic performance and structural safety.



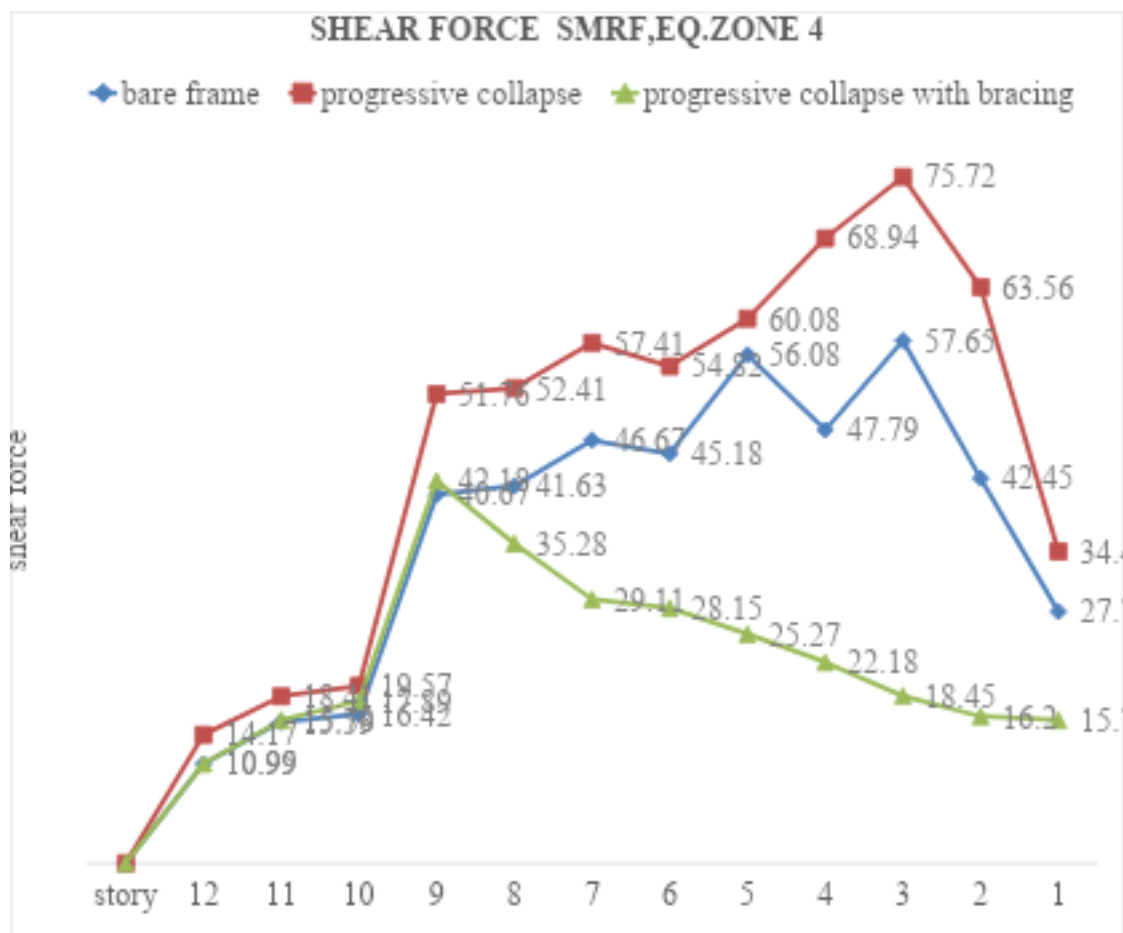
Graph 5.25 Variation of Shear Force For OMRF Eq.Zone 4.

Conclusion for Axial Force (12-Storey Steel Structure)

The analysis of shear force for the OMRF structure in seismic Zone 3 shows that shear force increases from the top story towards the bottom story.

The progressive collapse condition produces the highest shear force compared to the bare frame and braced frame.

The frame with bracing shows reduced shear force, indicating improved lateral stability. Hence, providing bracing in the structure enhances seismic performance and structural safety.



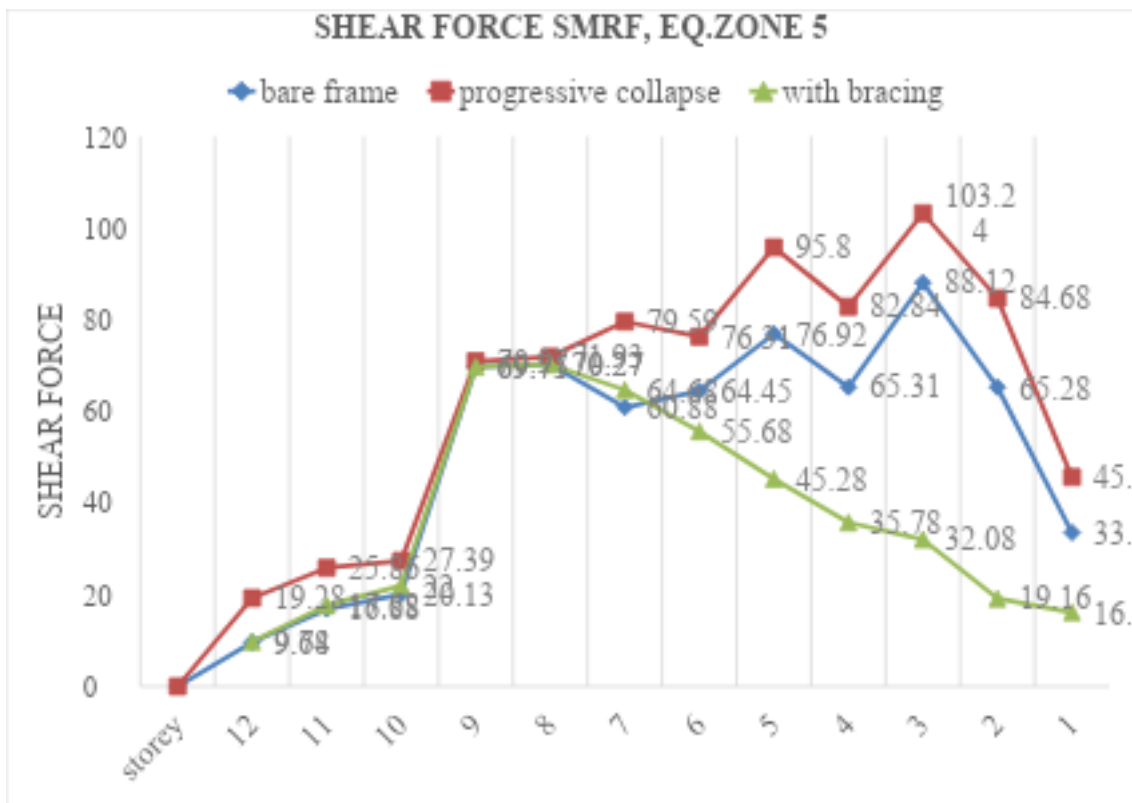
Graph 5.26 Variation of Shear Force For SMRF Eq.Zone 4.

Conclusion for Shear Force (12-Storey Steel Structure)

The analysis of shear force for the OMRF structure in seismic Zone 4 shows that shear force increases from the top story towards the bottom story.

The progressive collapse condition produces the highest shear force compared to the bare frame and braced frame.

The frame with bracing shows reduced shear force, indicating improved lateral stability. Hence, providing bracing in the structure enhances seismic performance and structural safety.



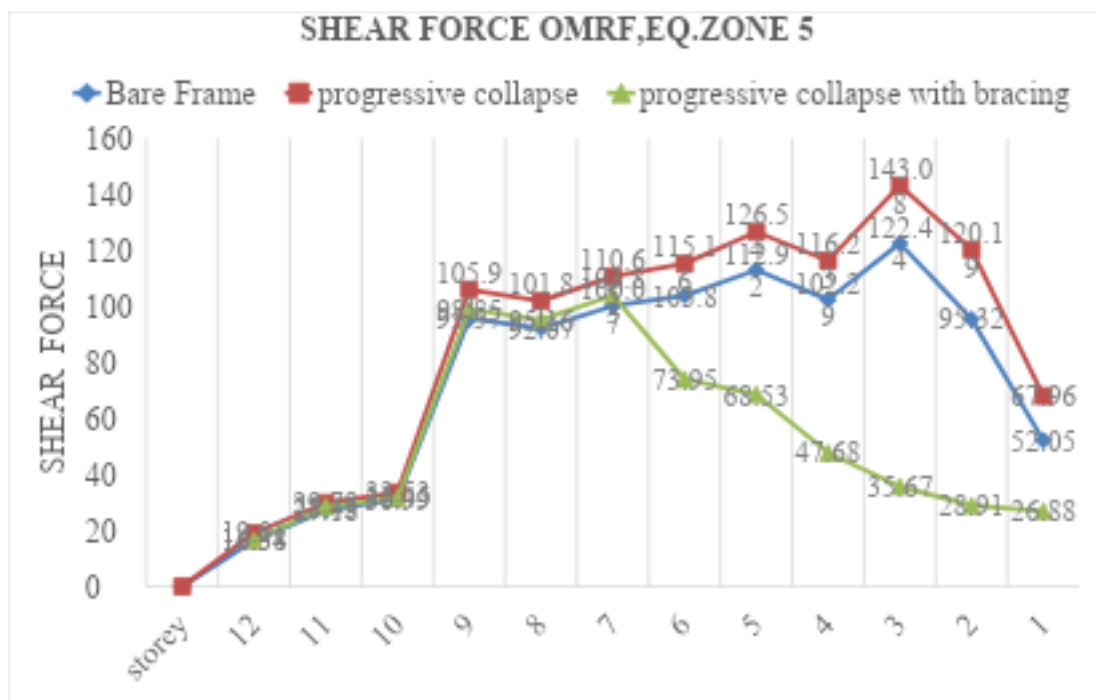
Graph 5.27 Variation of Shear Force For SMRF Eq.Zone 5.

Conclusion for Shear Force (12-Storey Steel Structure)

The analysis of shear force for the SMRF structure in seismic Zone 5 shows that shear force increases from the top story towards the bottom story.

The progressive collapse condition produces the highest shear force compared to the bare frame and braced frame.

The frame with bracing shows reduced shear force, indicating improved lateral stability. Hence, providing bracing in the structure enhances seismic performance and structural safety.



Graph 5.28 Variation of Shear Force For SMRF Eq.Zone 5

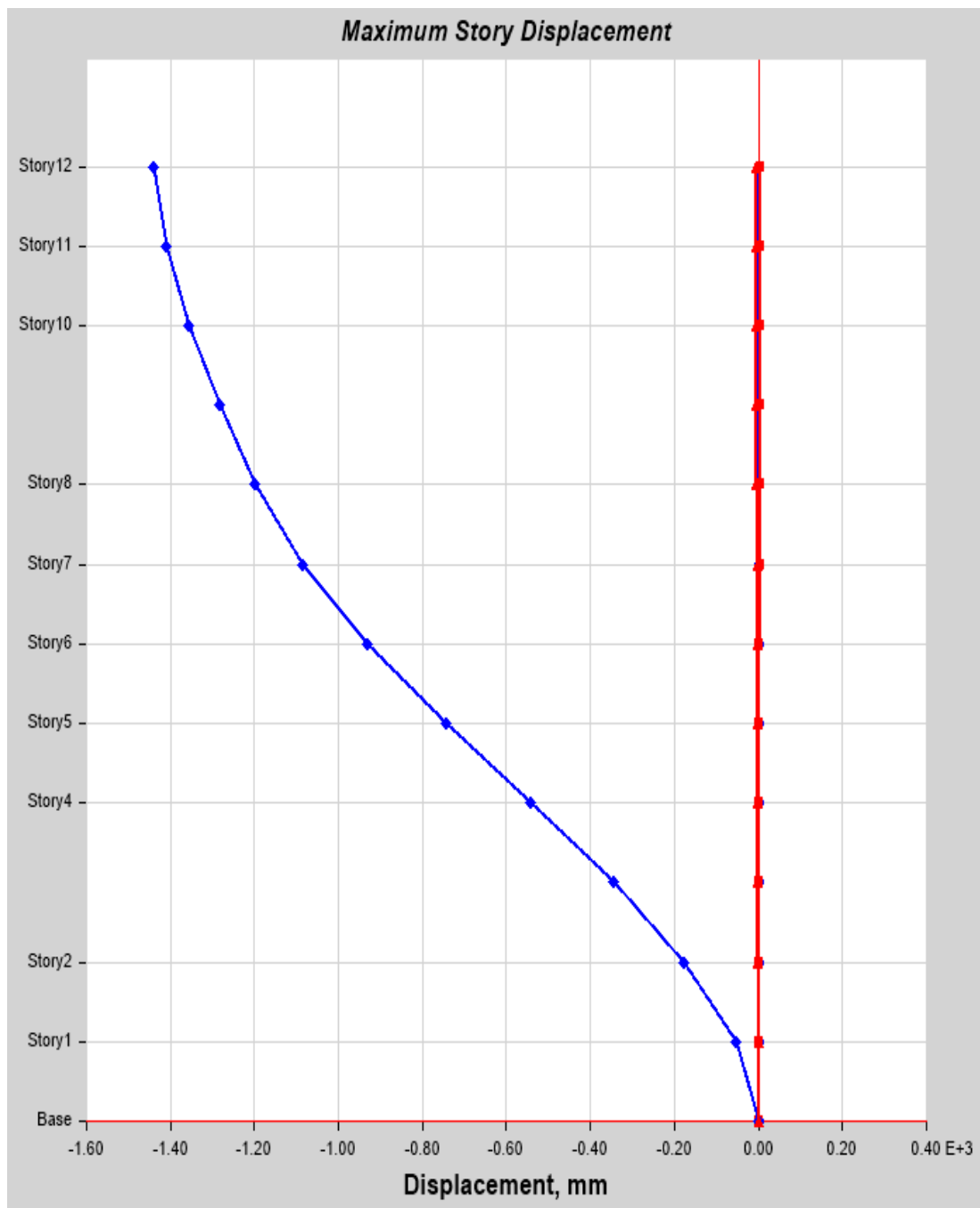
Conclusion for Shear Force (12-Storey Steel Structure)

The analysis of shear force for the OMRF structure in seismic Zone 4 shows that shear force increases from the top story towards the bottom story.

The progressive collapse condition produces the highest shear force compared to the bare frame and braced frame.

The frame with bracing shows reduced shear force, indicating improved lateral stability. Hence, providing bracing in the structure enhances seismic performance and structural safety.

After Analyzing The OMRF EQ.ZONE 3 Frame Following Results Are Obtained
For Push-Condition



Graph 5.29 Variation of Displacement For OMRF Eq.Zone 3.

Story	Elevation	Location	X-Dir	X-Dir	Y-Dir	Y-Dir
	m		Max	Min	Max	Min
	m		mm	mm	mm	mm
Story12	36	Top	0	-1440.014	4.074	-4.415
Story11	33	Top	0	-1409.693	4.028	-4.366
Story10	30	Top	0	-1355.877	3.892	-4.224
Story9	27	Top	0	-1281.755	3.7	-4.027
Story8	24	Top	0	-1199.569	3.49	-3.799
Story7	21	Top	0	-1084.835	3.177	-3.463
Story6	18	Top	0	-930.134	2.773	-3.027
Story5	15	Top	0	-742.17	2.253	-2.461
Story4	12	Top	0	-540.507	1.715	-1.873
Story3	9	Top	0	-344.984	1.138	-1.245
Story2	6	Top	0	-176.009	0.618	-0.681
Story1	3	Top	0	-51.623	0.205	-0.238
Base	0	Top	0	0	0	0

V CONCLUSION

1. The main objective of this project is to know the behavior of Progressive Collapse after sudden loss of the column due to the any incident of seismic forces, bomb blast, or any other reason.
2. The pushover analysis results indicate that the OMRF system develops a higher base shear capacity compared to the SMRF system. However, the SMRF structure provides better ductility and energy dissipation due to special seismic detailing.
3. In both systems, plastic hinges mainly remain within the Immediate Occupancy (IO) and Life Safety (LS) performance levels, with no hinges reaching the Collapse Prevention (CP) stage. Therefore, the SMRF system demonstrates better seismic performance and reliability under earthquake loading conditions
4. The bending moment distribution in the multi-story steel structure shows that maximum bending moments occur around the mid-height of the building.
5. As the seismic zone increases from Zone 3 to Zone 5, the bending moment values also increase due to higher seismic forces.
6. The progressive collapse condition produces the highest bending moments among the analyzed systems.
7. Braced frame systems show lower bending moments compared to bare frames because bracing increases lateral stiffness. Overall, adding bracing improves seismic performance and reduces structural demand.
8. Overall, adding bracing significantly improves structural performance, reducing bending moments by approximately 20–40% compared to bare frames, and enhancing the building's resistance to seismic loads.
9. The axial load in the 12-storey steel structure increases gradually from the top storey to the bottom due to the accumulation of gravity and seismic loads.

10. The progressive collapse condition produces the highest axial forces because of load redistribution after element failure.
11. Introducing bracing improves structural performance by reducing load concentration and increasing stiffness.
12. Higher seismic zones result in greater axial forces, with SMRF generally experiencing larger forces than OMRF. Overall, bracing enhances the stability and seismic safety of the structure.
13. The study analyzed the shear force behavior of SMRF and OMRF frames for a 12-storey steel structure under different seismic zones and structural conditions.
14. Results show that the progressive collapse condition produces the highest shear forces, while the bare frame shows moderate values.
15. The inclusion of bracing significantly reduces shear force by improving lateral stiffness and load redistribution. Shear force demand also increases as seismic zone intensity increases from Zone 3 to Zone 5. Therefore, bracing systems are recommended to enhance structural stability and seismic performance.

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