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Study of Progressive Collapse of Precast Steel Reinforced Concrete Building

Mohammad Arastu, Khalid Moin

***Abstract:** Progressive collapse starts when any load-carrying elements of the building collapse during extreme loading, such as earthquakes, blasts, or fire. The Precast Steel Reinforced Concrete (PSRC) buildings comprise of precast RCC columns and steel girders. These structural elements are connected to form a moment-resisting frame and are susceptible to progressive collapse. However, this structural system has the advantage of inherent stiffness and damping during lateral loads and is also known for its construction efficiency, lightweight and low cost. Earlier investigations have shown PSRC systems useful in designing and constructing buildings while maintaining ample strength and high ductility during seismic incidents. Despite much previous research, the PSRC structural system's use is limited in high seismic regions. This paper aims to study the progressive collapse of the PSRC building using non-linear dynamic analysis and U.S. General Service Administration (GSA) guidelines during extreme loading. Two structures are studied to validate the performance of progressive collapse of PSRC and RCC structures. Four-story PSRC and RCC buildings are designed according to Indian Codes of practice. Design columns under provisions of the Indian reinforced concrete structures code, and beams are designed according to the Indian steel construction code. Comparative studies of progressive collapse for the two buildings are presented.*

Keywords: Progressive Collapse, Time History, PSRC system, RCC System, Precast

I. INTRODUCTION

The progressive collapse of buildings became a significant issue after the collapse of the 22-story Ronan Point apartment building in 1968 (Figure 1). The structure comprised prefabricated concrete and was destroyed by a gas explosion on the 18th floor [1].

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*Correspondence Author(s)
Mohammad Arastu*, Department of Civil Engineering, Faculty of Engineering & Technology, Jamia Millia Islamia University, New Delhi, India. E-mail: mohd.arastu@gmail.com, ORCID ID: https://orcid.org/0000-0001-9991-7318
Prof. Khalid Moin, Department of Civil Engineering, Faculty of Engineering & Technology, Jamia Millia Islamia University, New Delhi, India. E-mail: mohd.arastu@gmail.com, ORCID ID: https://orcid.org/0000-0002-6547-3357

Figure 1. Ronan Point apartment building after the collapse [16]

PSRC frame systems have retained numerous advantages from economic and construction viewpoints [2] compared to RCC or steel frame systems. RCC columns are nearly ten times more efficient than steel columns in axial strength and axial stiffness [3]. On the other hand, the deck slabs supported on steel girders are significantly lighter than the RCC beam-slab system, leading to significant reductions in the total building load, costs of the foundation, and earthquake forces. In previous years, the PSRC structural systems for moment-resisting have mostly been used for buildings located in low seismicity areas in developed countries. In recent years, researchers have attempted to develop seismic design guidelines for PSRC systems located in high seismic-risk regions [4]. Many researchers have developed testing models of PSRC frames based on a typical theme building devised for the US-Japan program [5], [6], [7], [8]. These studies apply the suggested seismic design specifications for PSRC systems and then assess the seismic performance using non-linear analyses and advanced performance assessment techniques.

Traditional steel frames were also investigated in these studies to benchmark conventional structures' performance compared to the Precast SRC frames. These design studies have shown that the steel beam sizes tend to be similar for the PSRC and steel system and that the main disagreements lie in the RCC column and steel girders connection. Given the additional stiffness provided by the RCC columns, the SRC frames tended to be controlled more by the bare minimum strength requirements. In contrast, lateral drift limitations restricted the steel frames. In general, these studies have shown that the inelastic dynamic response of the PSRC frames is similar to comparably designed steel moment frames.

Cordova [9] designed and tested a full-scale 3-story SRC moment frame. Using the pseudo-dynamic loading technique, this specimen is subjected to a sequence of earthquake motions ranging in hazards from frequent to sporadic events. Using the results of the test specimens and recommendations, trial designs of three case study buildings (3, 6, and 20 stories) are generated, analytically modelled, and subjected to a collection of earthquake ground motions at a range of hazard levels. They Investigate differences between the response of beam-column subassembly and full-scale system testing and evaluate how this affects the interpretations from these tests.

One of the efficient tools for addressing the behaviour of buildings under earthquake loading is Nonlinear Dynamic Time History Analysis. When Nonlinear Dynamic Time History analysis is used carefully, it is widely accepted that it provides useful information that cannot be obtained by linear static or dynamic analysis procedures [10]. This paper aims to study the seismic performance of the PSRC system for buildings compared to RCC buildings. The scaled 1940 EL-Centro (N.S. component) time history of 0.1g to 0.5g PGA has been used for the study.

II. TIME HISTORY ANALYSIS

The structures deform inelastically during the maximum considered earthquake (MCE). Hence structural performance must be checked during the post-elastic behaviour of the structure. Dynamic non-linear analysis (also called Time History Analysis) should be used to evaluate seismic performance because the elastic analysis cannot determine the structure's post-elastic behaviour during such events. Moreover, to estimate the seismically induced needs that exhibit inelastic behaviour, the structures' maximum inelastic displacement demand should be determined adequately.

In the dynamic non-linear analysis method, the ground acceleration time history is applied to the structure. Dynamic equilibrium equations are solved using direct integration methods. Initial conditions are set by continuing the structural state from the end of the previous non-linear gravity analysis. Direct-integration methods are sensitive to time-step size, which should be decreased until results are not affected. Material and geometric nonlinearity, including P-delta effects, have been simulated during non-linear direct-integration time-history analysis.

A scaled time history of 1940 EL-Centro (N.S. component) of 0.1g to 0.5g PGA has been applied to the structure's base (Figure 2, 3, 4, 5 and Figure 6).

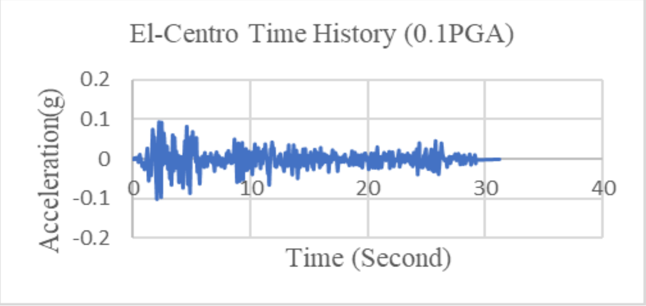

Figure 2. Time History (PGA=0.1g)

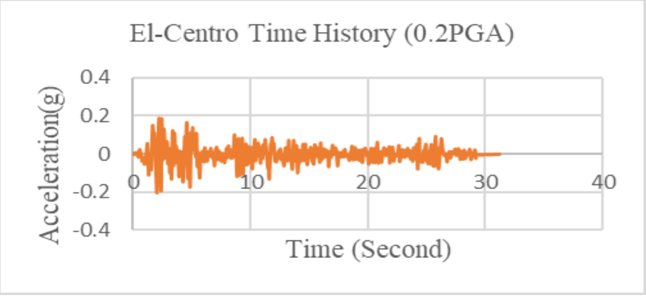

Figure 3. Time History (PGA=0.2g)

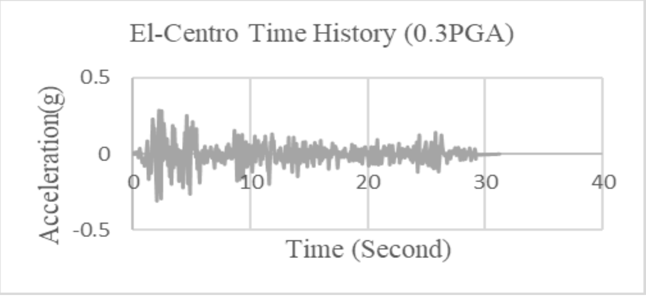

Figure 4. Time History (PGA=0.3g)

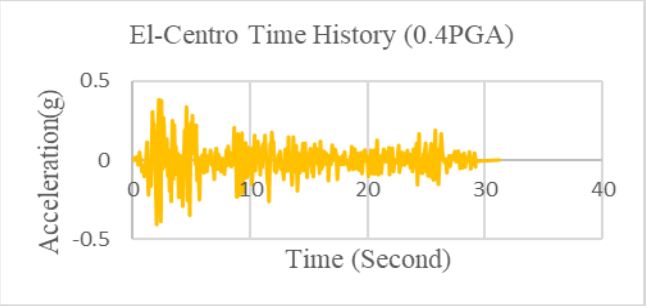

Figure 5. Time History (PGA=0.4g)

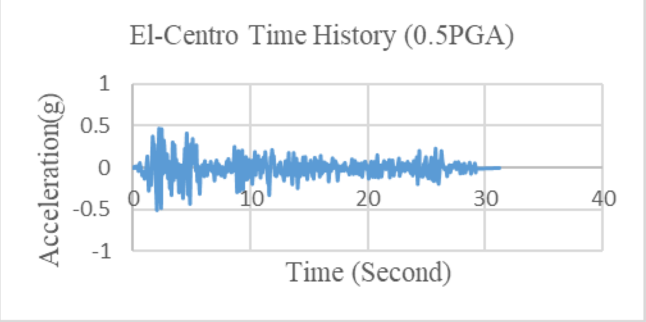

Figure 6. Time History (PGA=0.4g)

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III. SEISMIC PERFORMANCE OF BUILDINGS

The state of damage measures buildings' seismic performance under a certain seismic hazard level. The form of damage is quantified by the roof's drift and the structural elements' displacement. Initially, gravity non-linear analysis is carried out using non-linear dynamic analysis.

Time history analysis gives an insight into the maximum base shear the structure can resist. A building performance level is a combination of the structure's performance levels and the nonstructural components. A performance level describes a limiting damage condition, which may be considered satisfactory for a given building with specific ground motion [11]. The performance of the structure is determined by hinges formation. Various types of plastic hinges: uncoupled/coupled moment, torsion, axial force, and shear hinges are available. After yielding, plastic hinges will form at different locations, indicating the occupant's risk (Figure 7). No hinges will be created before point B, where the structure will show linear behavior, and after that, one or more hinges will start to form. The software will show hinges with the following remarkable indication:

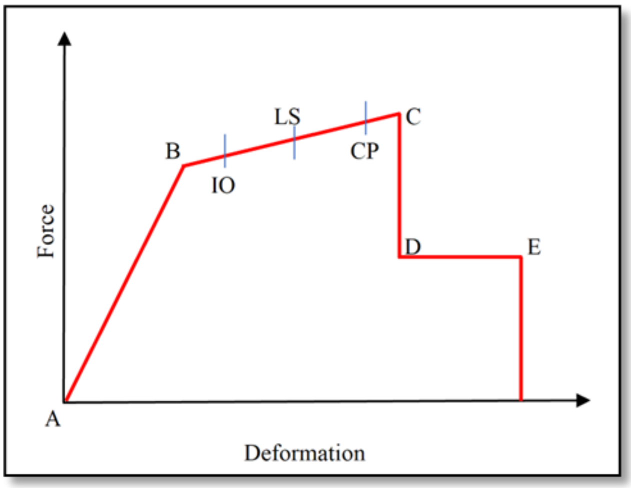

Figure 7. Risk Indicator Curve

Immediate occupancy I.O.: indicates the state of damage in which limited nonstructural damage has occurred. At this stage, the structural elements of the building maintain their original strength and stiffness. The probability of life-threatening injury is very low due to nonstructural damages. Minor repairs of these nonstructural elements can be repaired before re-occupancy [12], [13].

Life safety level L.S.: indicates the state of damage in which substantial damage to the structural elements has occurred, but some scope against either partial or total structural collapse persists. Many structural elements are severely damaged, but this has not resulted in large falling debris hazards. Injuries may arise at this stage. The overall probability of life-threatening injury is low because low structural damage is expected and feasible to repair the structure [12], [13].

Collapse prevention CP: indicates the state of damage in which the building is on the limit of partial or total collapse. Significant damage to the structure has occurred, like considerable degradation in the stiffness, permanent lateral deformation, and axial strength degradation. The substantial threat of injury may happen due to collapsing of structural debris. The structure may not be practical to repair and is not safe for re-occupancy [12], [13].

IV. DESCRIPTION OF STUDIED STRUCTURES

Two structures are considered to represent PSRC and RCC structures to study. These consist of a typical steel girder and Precast RCC columns frame building. Four-story PSRC buildings are designed according to Indian Codes of practice. Design columns under provisions of Indian reinforced concrete structures code, and beams are designed according to Indian steel construction code.

The longitudinal and transverse bars' yield strength for RCC beams and columns used as 500N/mm^2. The compressive strength of concrete used was 25 MPa at 28 days. The structural steel had a yield strength of 250N/mm^2 used in the analysis. The stress-strain curve used for the non-linear dynamic analysis (Figure 8 and Figure 9).

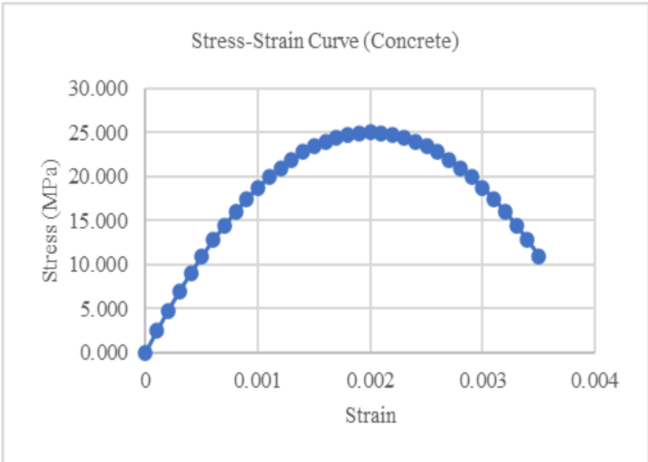

Figure 8. Stress-Strain Curve for Concrete

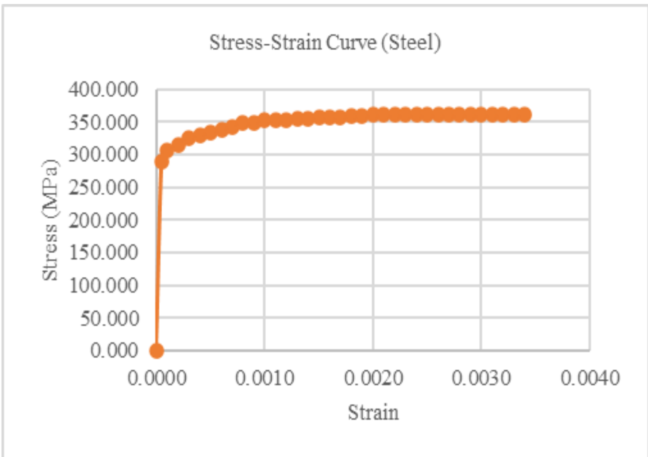

Figure 9. Stress-Strain Curve for Steel

The column center-to-center dimensions were 5000 mm in both directions. The model is assumed to be pinned at the base. The column and beam details have been done as per the Indian Code of Practice. The 300mm wide and 400mm deep beam with 3 bars of 16mm diameter at the top and bottom were used at all levels and in both directions, plus an extra 2T16 at the support.

The 400mm x400mm columns with 8 bars of 20mm diameter and 8mm diameter wire were used as stirrups at 100mm c/c near the beam-column junction and 150mm c/c near the mid-height of the column. The story height was kept as 3000mm c/c of the beam on all floors. For PSRC structural system, steel girders of ISM300 are considered. The section properties of both frames (Figure 10 and Figure 11).

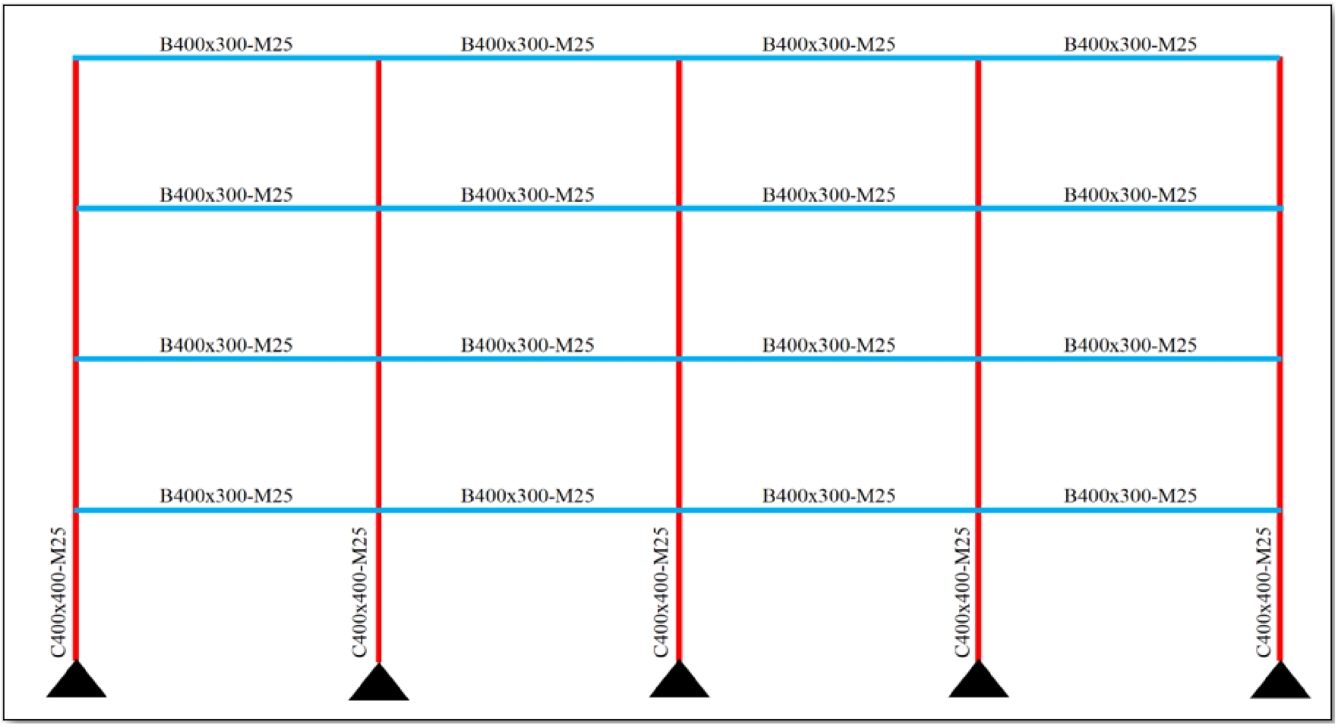

Figure 10. RCC Frame Structure Section Properties

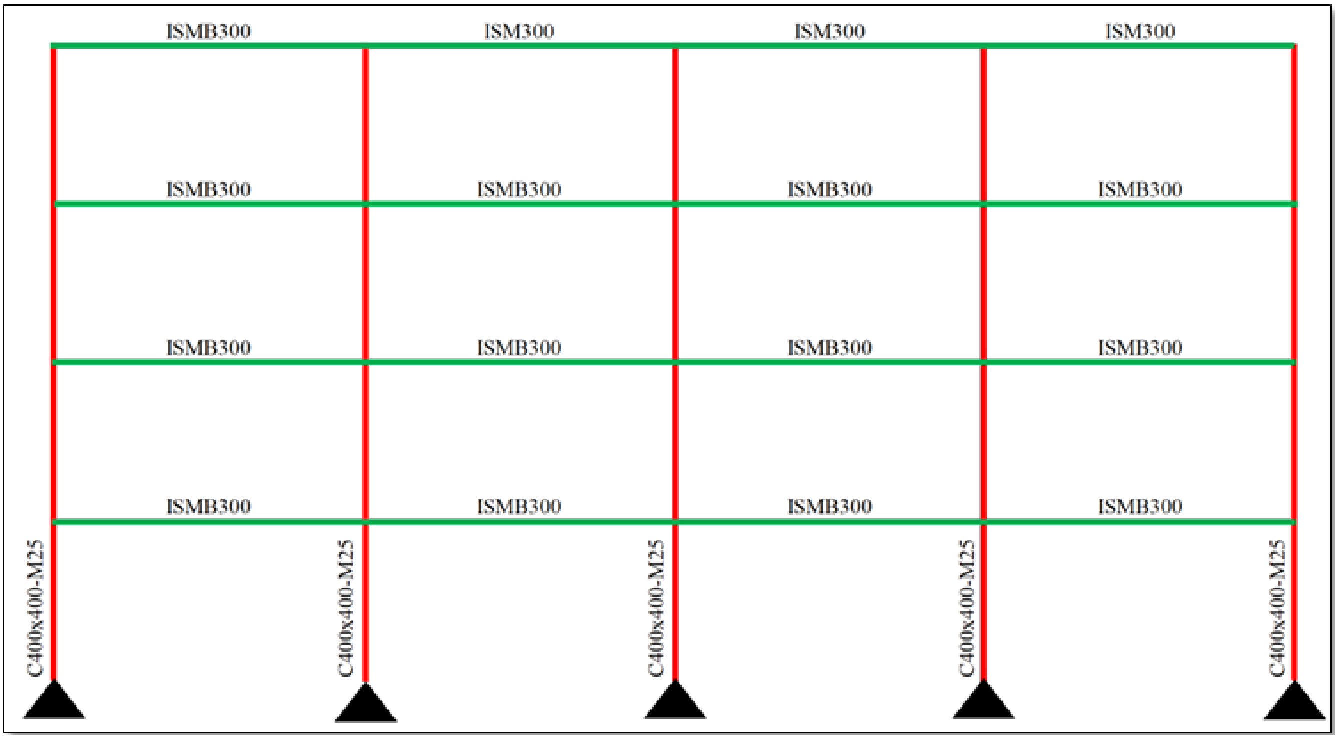

Figure 11. PSRC Frame Structure Section Properties

V. RESULT AND DISCUSSION

The El-Centro time history was applied at the base of both structures from 0.1g to 0.5g PGA. The direction of monitoring the building's behaviour was the same as the ground acceleration direction. For columns, program-defined auto PM2M3 interacting hinges were used at both ends, and for beams, M3 auto hinges were used according to FEMA 356. Column bases are assumed to be hinged at the foundation level. The beams and columns are modelled as non-linear frame elements with lumped plasticity; hinges are defined according to the section properties at both ends of the columns and beams.

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A. Base Reaction

The base reactions obtained from the dead load (DEAD), live load (LIVE), and static earthquake load (EQX) for RCC are shown in Table 1, and PSRC are shown in Table 2. It has been noted that there is a 15% reduction in the dead load for PSRC structure due to the usage of steel girders and hence less base shear computed as compared to RCC structure

Table 1. Base Reaction for RCC Structure

Output Case	Case Type Text	Global FX kN	Global FY kN	Global FZ kN	Global MX kN-m	Global MY kN-m	Global MZ kN-m
DEAD	LinStatic	2.22E-15	1.73E-13	14621.612	144903.89	-146653.5	1.84E-12
LIVE	LinStatic	-1.77E-15	6.52E-13	4725	46687.5	-47437.5	6.32E-12
EQX	LinStatic	-939.174	-1.72E-10	-1.09E-12	1.62E-09	-9364.199	9380.1192

Table 2. Base Reaction for PSRC Structure

Output Case	Case Type Text	Global FX kN	Global FY kN	Global FZ kN	Global MX kN-m	Global MY kN-m	Global MZ kN-m
DEAD	LinStatic	-4.02E-14	7.14E-14	12568.76	124375.37	-126125	1.22E-12
LIVE	LinStatic	-1.13E-13	3.08E-13	4725	46687.5	-47437.5	4.58E-12
EQX	LinStatic	-816.003	-3.47E-10	1.28E-13	3.05E-09	-8132.488	8148.4078

B. Base Shear

The base shear time history of both PSRC and RCC structure for 0.5g to 0.1g PGA (Figure 12, 13, 14, 15 and Figure 16). It has been noted that the maximum base shear is 16% less for PSRC structure as compared to the RCC structure at 0.5g, 20% less at 0.4g, 27% at 0.3g, 33% at 0.2g, and 35% at 0.1g. From these results, the change in base shear percentage reduces with an increase in PGA value, and the PSRC structure attracts a lesser amount of earthquake forces than the RCC structure.

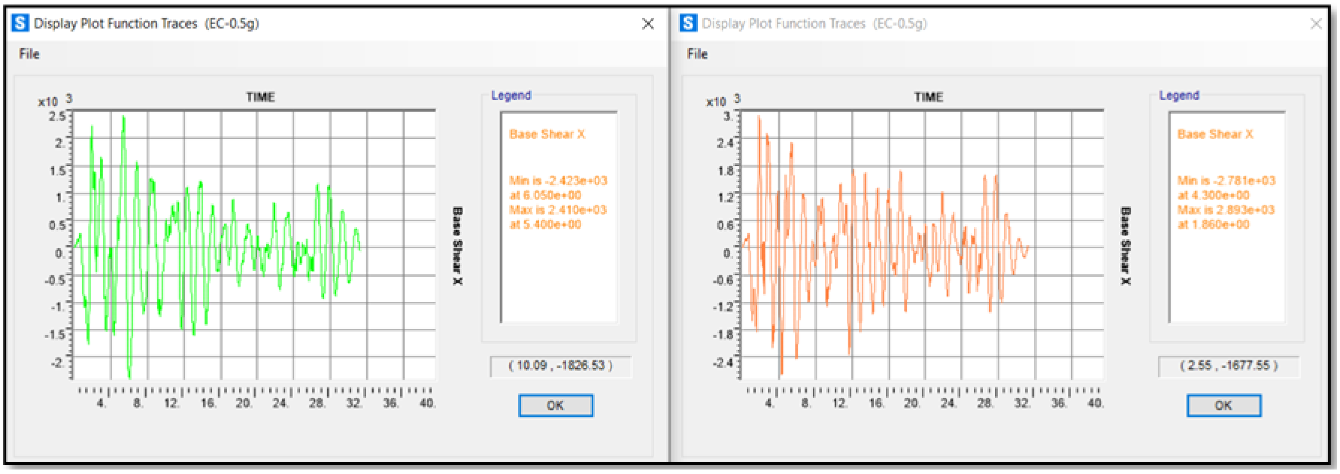

Figure 12. Base Shear Time History at 0.5g PGA for PSRC and RCC Structure

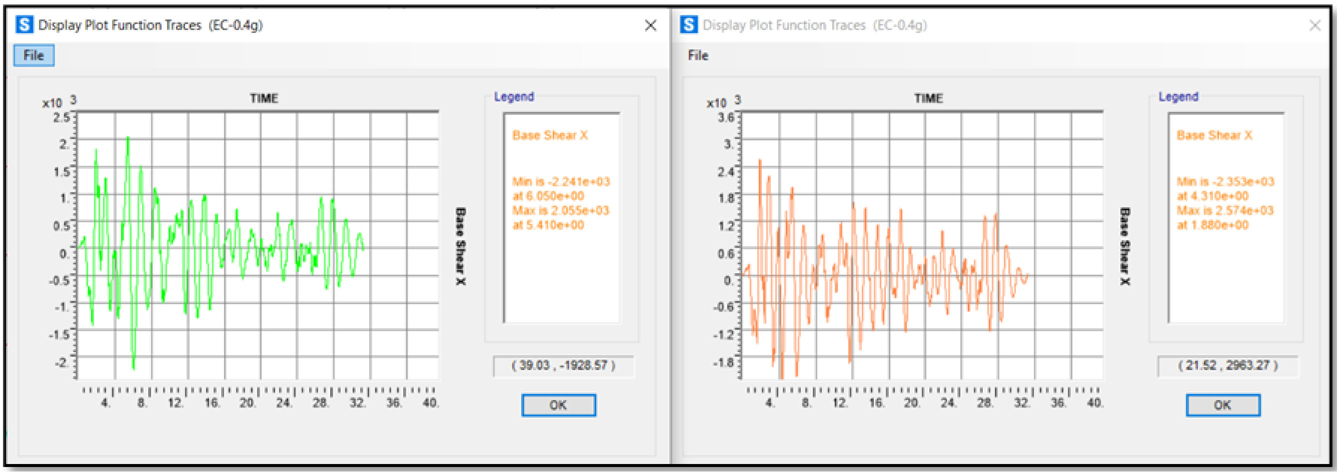

Figure 13. Base Shear Time History at 0.4g PGA for PSRC and RCC Structure

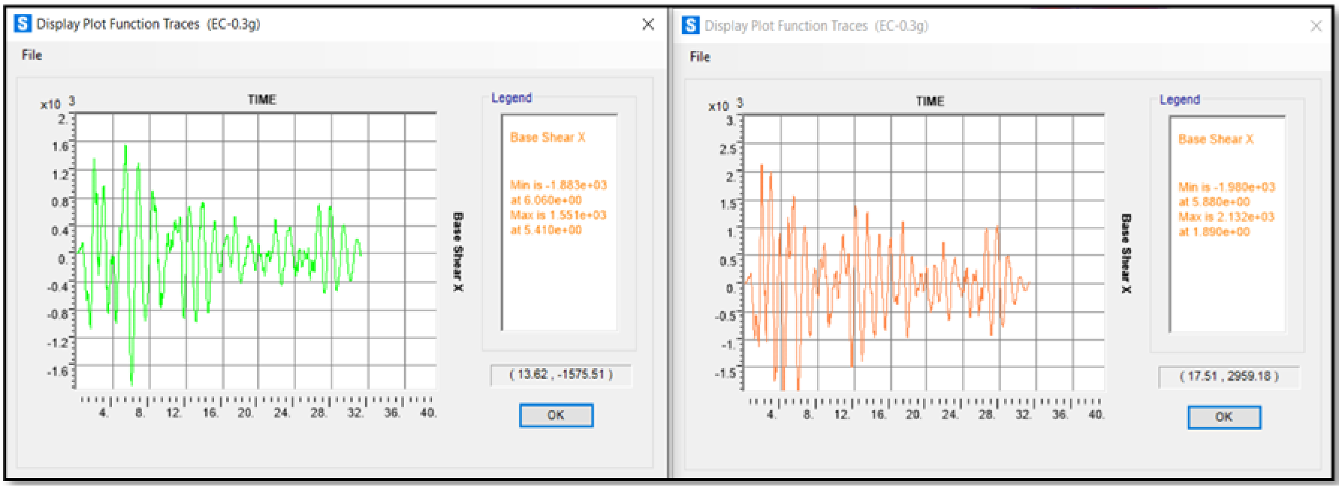

Figure 14. Base Shear Time History at 0.3g PGA for PSRC and RCC Structure

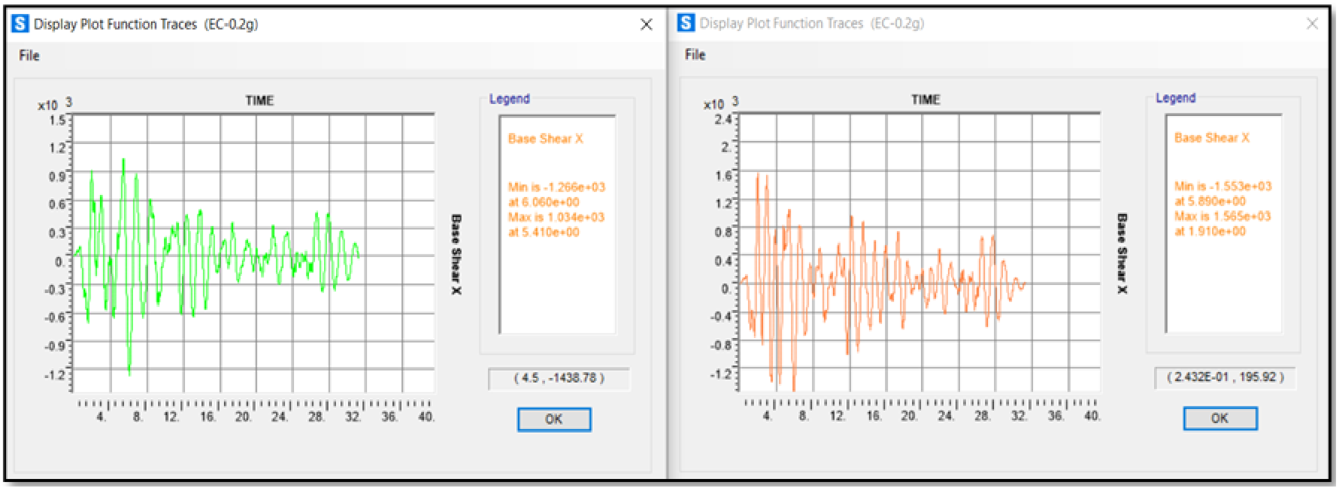

Figure 15. Base Shear Time History at 0.2g PGA for PSRC and RCC Structure

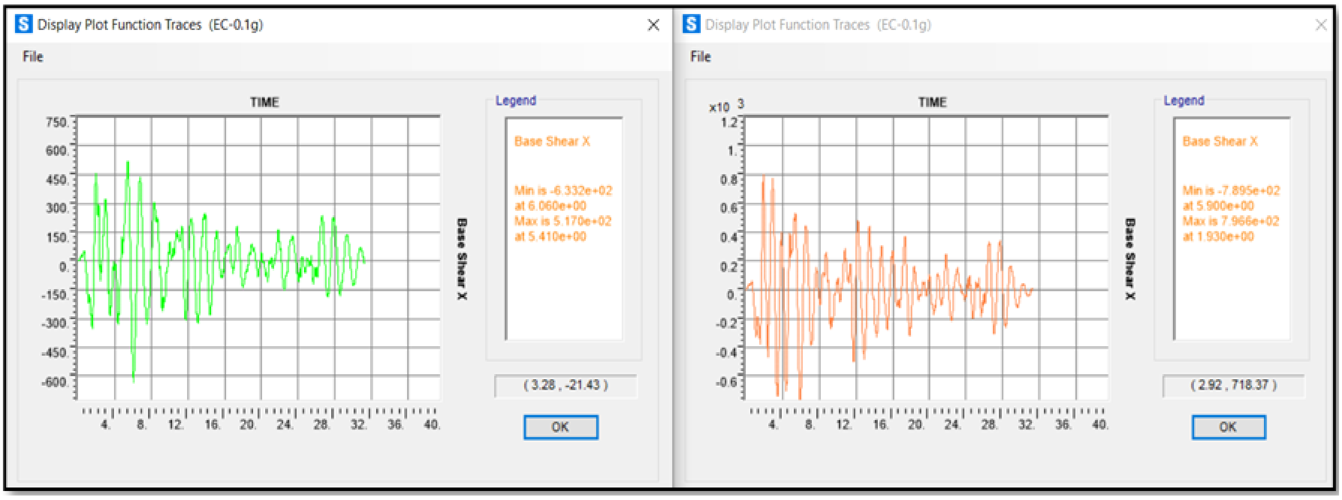

Figure 16. Base Shear Time History at 0.1g PGA for PSRC and RCC Structure

C. Top Story Lateral Displacement

The time histories of the top story lateral displacement for both PSRC and RCC structures at 0.5g to 0.1g PGA are shown in Figure 17, 18, 19, 20 and Figure 21. It has been noted that the maximum top story lateral displacement is 9% less for the PSRC structure as compared to the RCC structure at 0.4g and 0.5g. The top story lateral displacement is the same at 0.3g for both types of structures. The top story lateral displacement is 13% more at 0.1g and 0.2g. The top story lateral displacement is less for the PSRC structure at the higher PGA than the RCC structure from these results.

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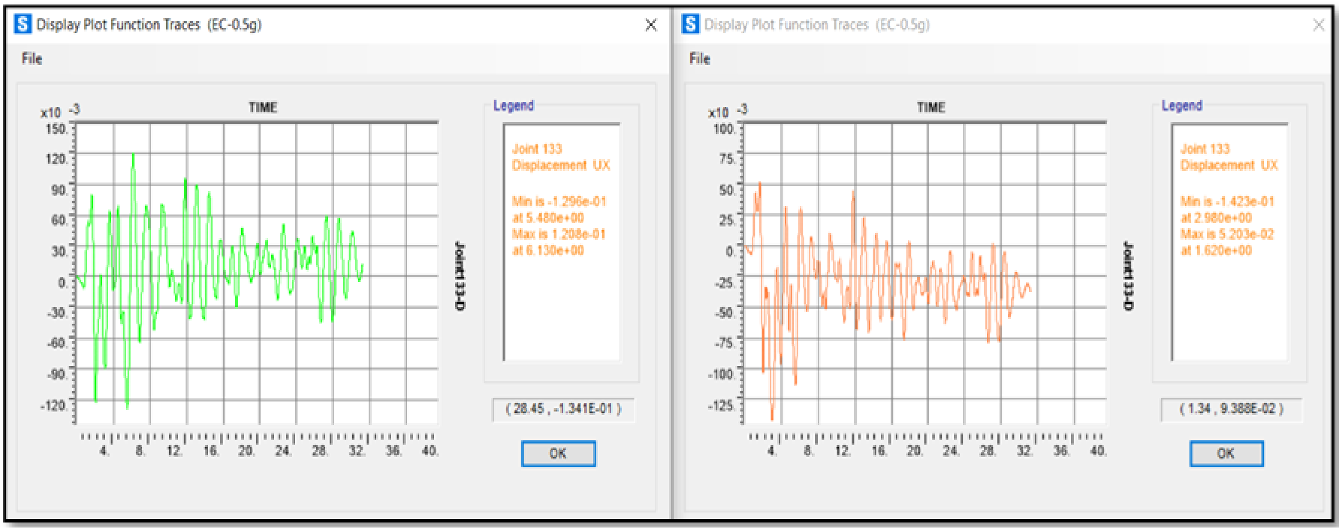

Figure 17. Top Story Displacement Time History at 0.5g PGA for PSRC and RCC Structure

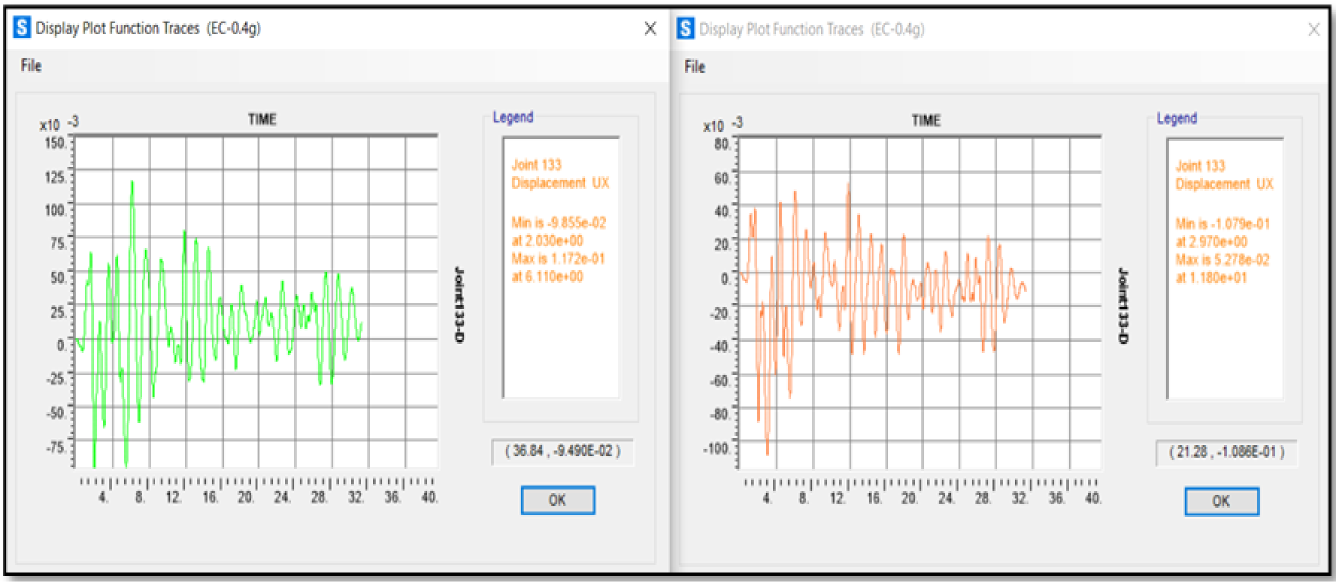

Figure 18. Top Story Displacement Time History at 0.4g PGA for PSRC and RCC Structure

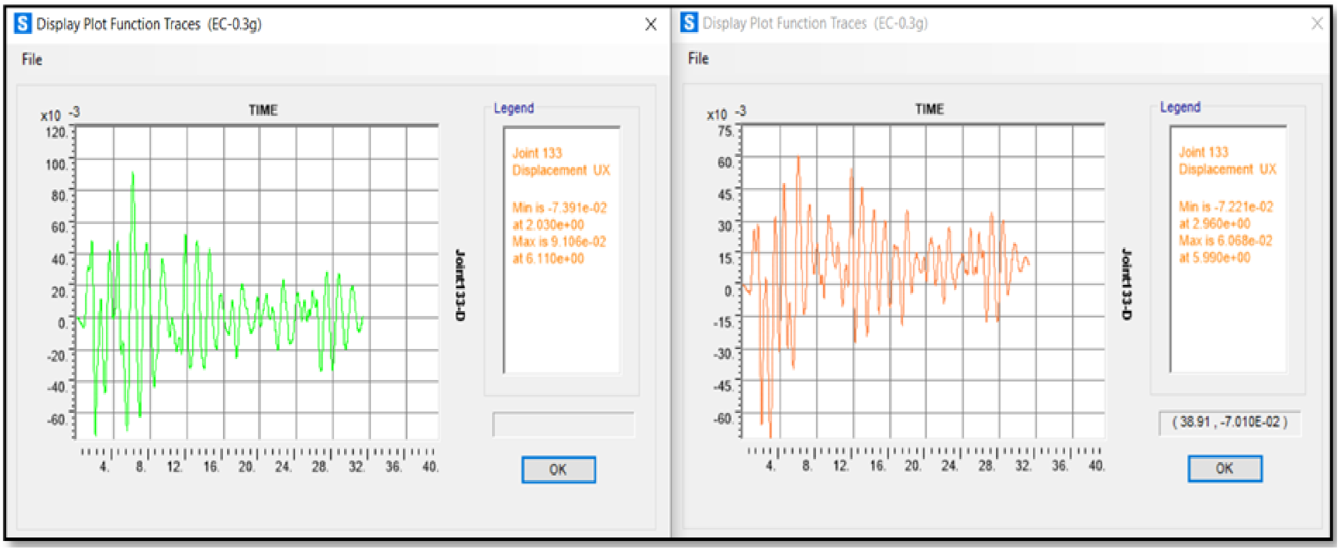

Figure 19. Top Story Displacement Time History at 0.3g PGA for PSRC and RCC Structure

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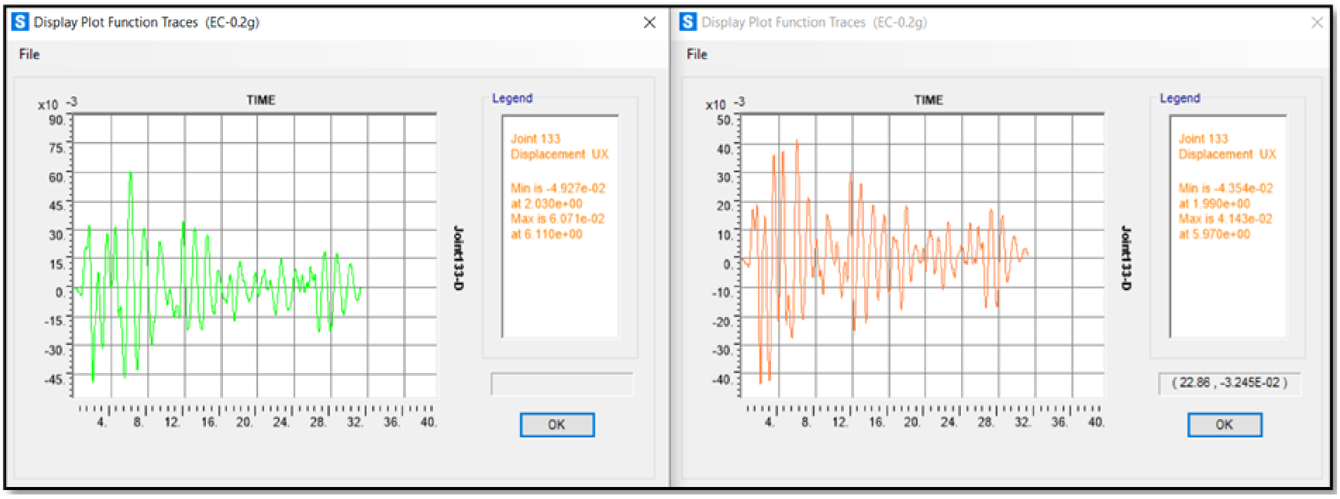

Figure 20. Top Story Displacement Time History at 0.2g PGA for PSRC and RCC Structure

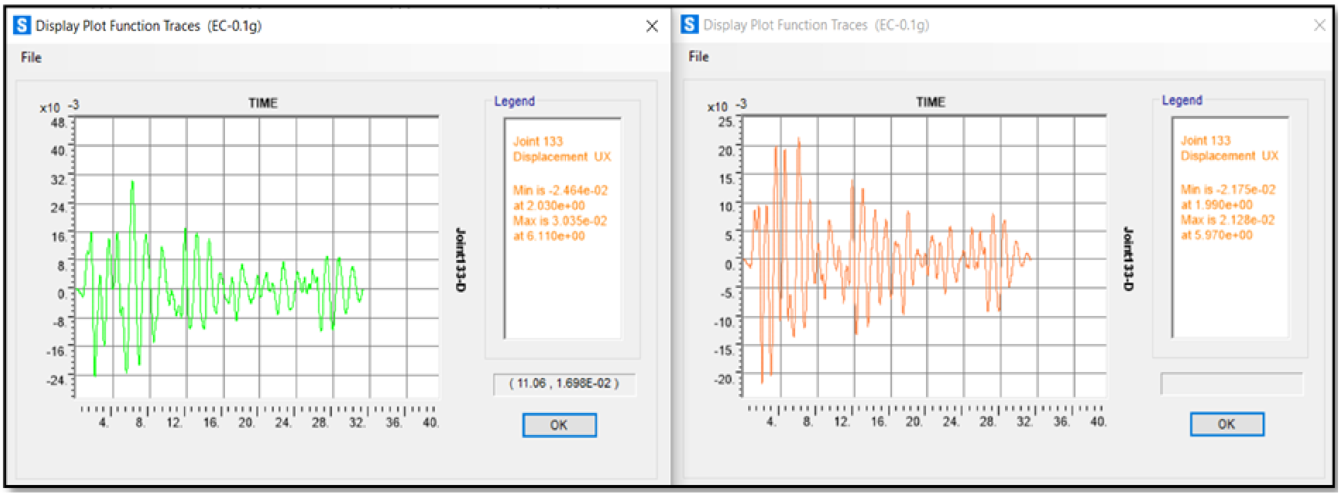

Figure 21. Top Story Displacement Time History at 0.1g PGA for PSRC and RCC Structure

D. Hysteresis Curve

The hysteresis curve indicates the physical characteristics of structures during cyclic loading, including distortion, stiffness degradation, and energy utilization. The hysteresis curves for the PSRC and the RCC structures are shown in Figure 22, 23, 24, 25 and Figure 26 for 0.1g to 0.5g PGA.

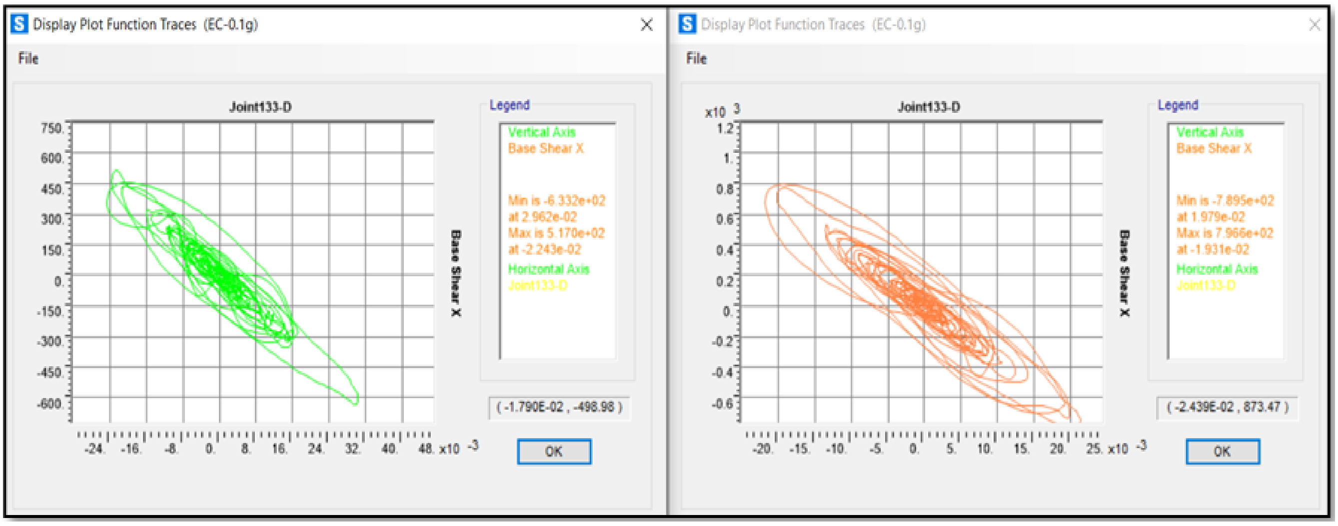

Figure 22. Hysteresis Curve at 0.1g PGA for PSRC and RCC Structure

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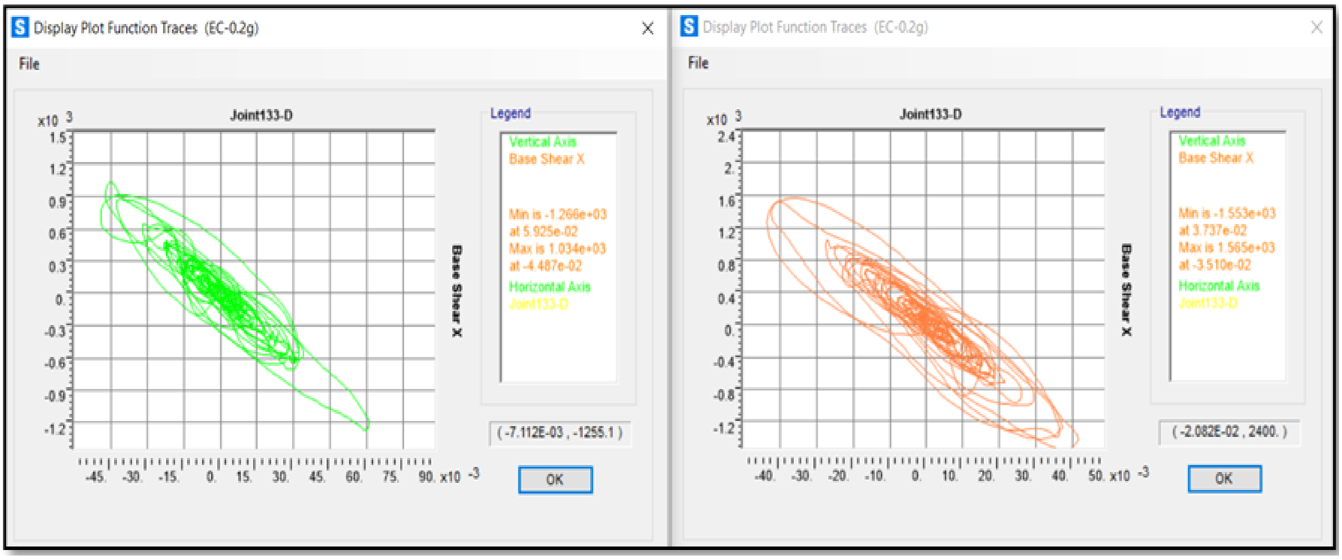

Figure 23. Hysteresis Curve at 0.2g PGA for PSRC and RCC Structure

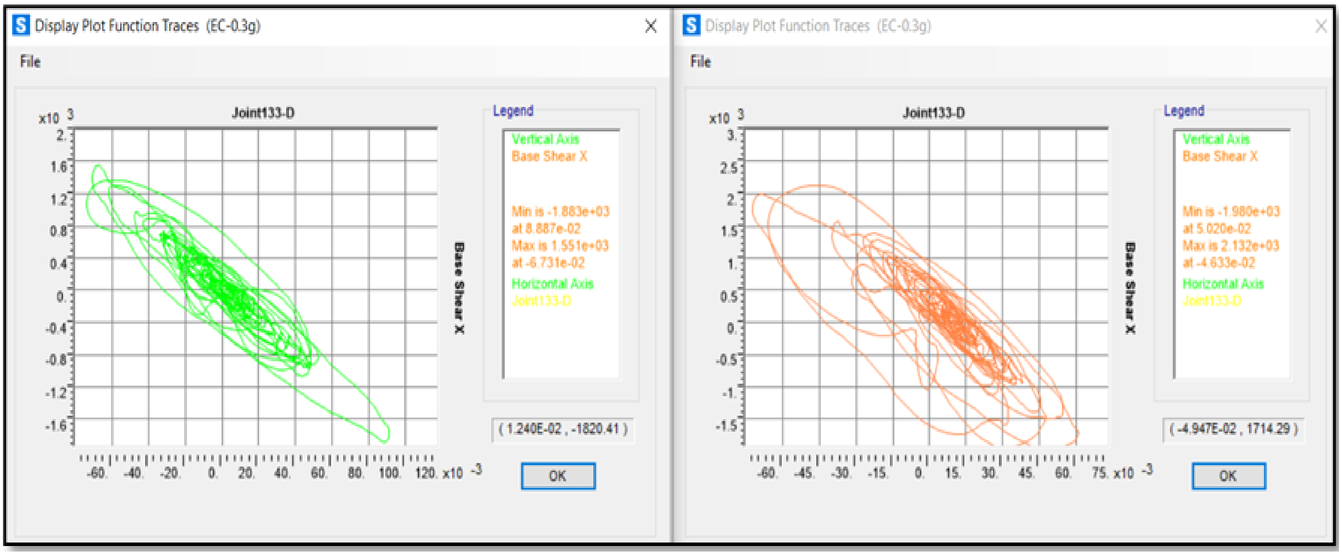

Figure 24. Hysteresis Curve at 0.3g PGA for PSRC and RCC Structure

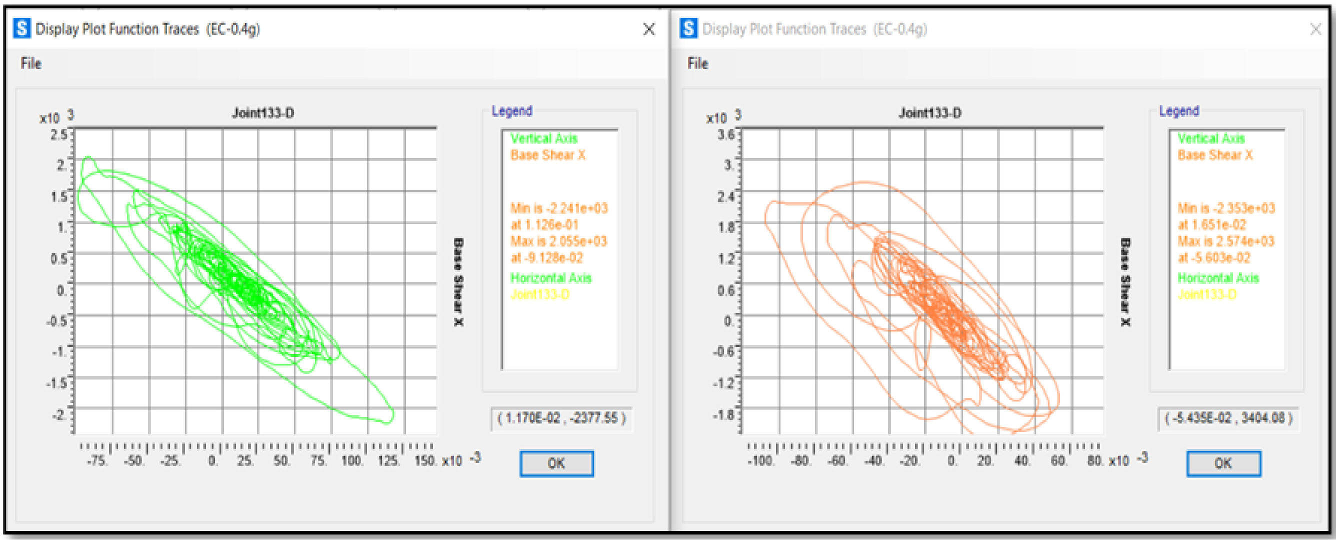

Figure 25. Hysteresis Curve at 0.4g PGA for PSRC and RCC Structure

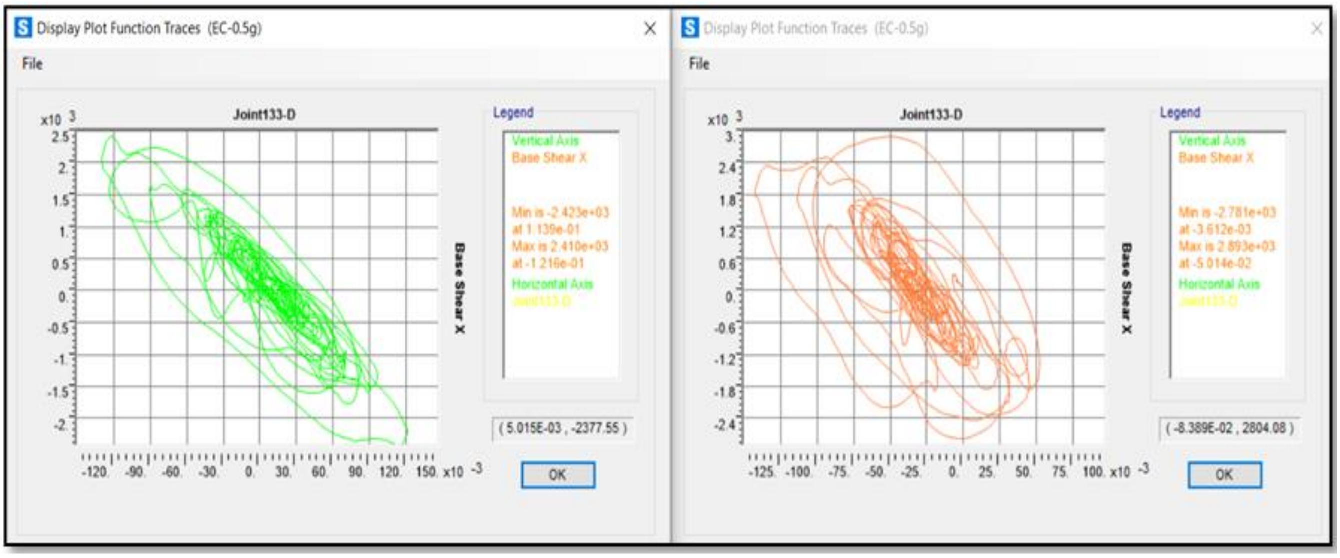

Figure 26. Hysteresis Curve at 0.5g PGA for PSRC and RCC Structure

F. Hinges Formation

The formation of hinges in beams and columns for the RCC and the PSRC structure at 0.2g to 0.5g PGA are shown from Figure 27, 28, 29, 30, 31, 32, 33 and Figure 34. It has been observed that at 0.1g, there is no hinge formation in both structures. At 0.2g, the hinge formation started in beams at the first story level in the RCC structure, and no hinge formation was observed in the PSRC structure. At 0.3g, the hinges propagate in beams at end bays of the second-story level in the RCC structure, and in some beams, hinge formation started at the first story level in the PSRC structure. No hinge formation observed in columns in both the structure up to 0.3g PGA. At 0.4g, the hinge formation propagates in beams from the end bay to the interior bay at the second story and first-story columns in the RCC structure. The hinge formation propagates in all beams at the first story level and column in the PSRC structure. At 0.5g, the collapse stage reaches at first story beams in the RCC structure, and no collapse stage reaches in PSRC structure.

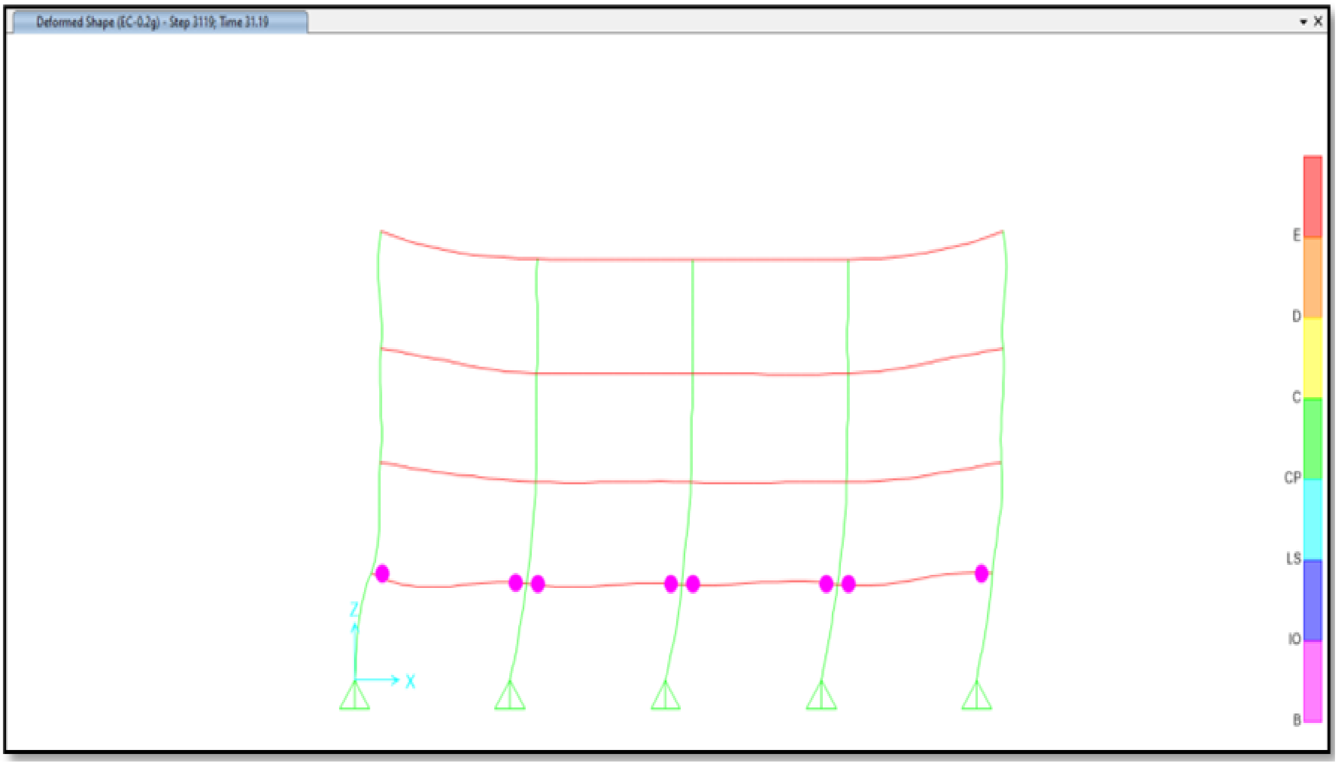

Figure 27. Hing Formation at 0.2g PGA for RCC Structure

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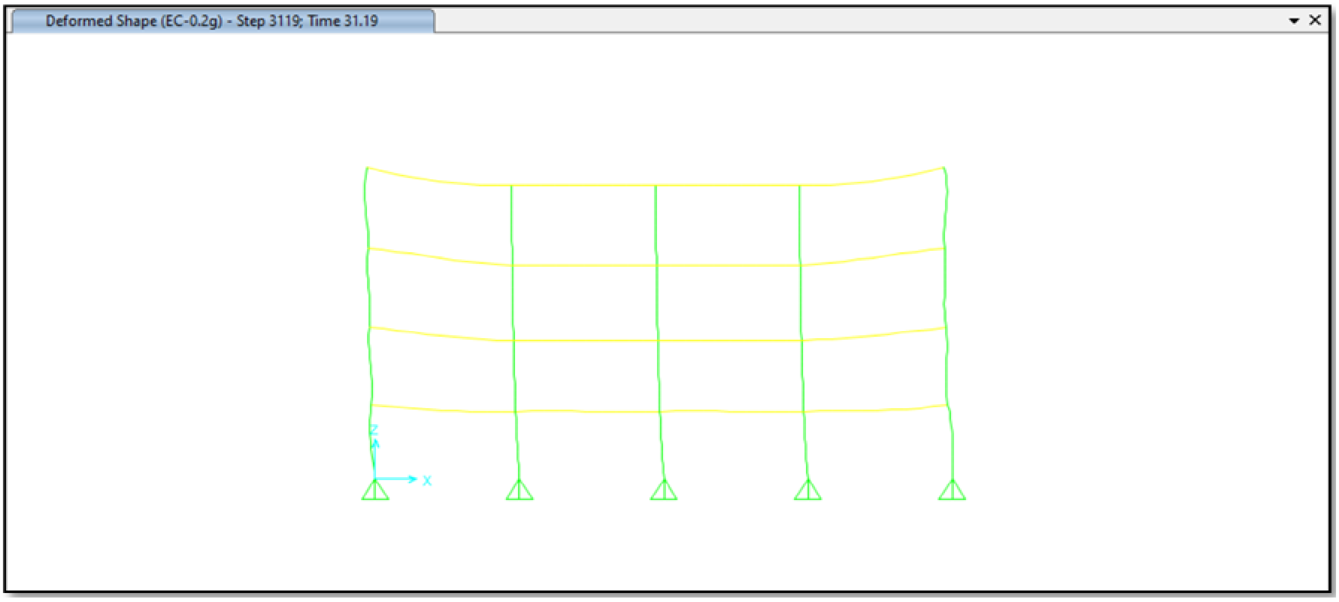

Figure 28. No Hing Formation at 0.2g PGA for PSRC Structure

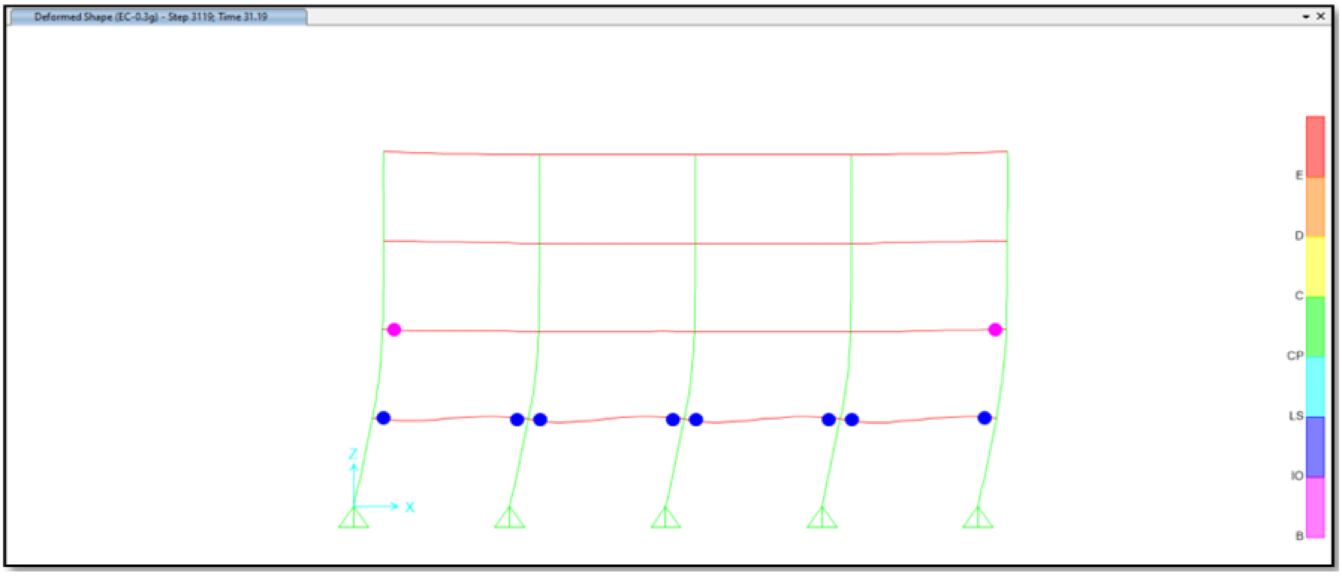

Figure 29. Hing Formation at 0.3g PGA for RCC Structure

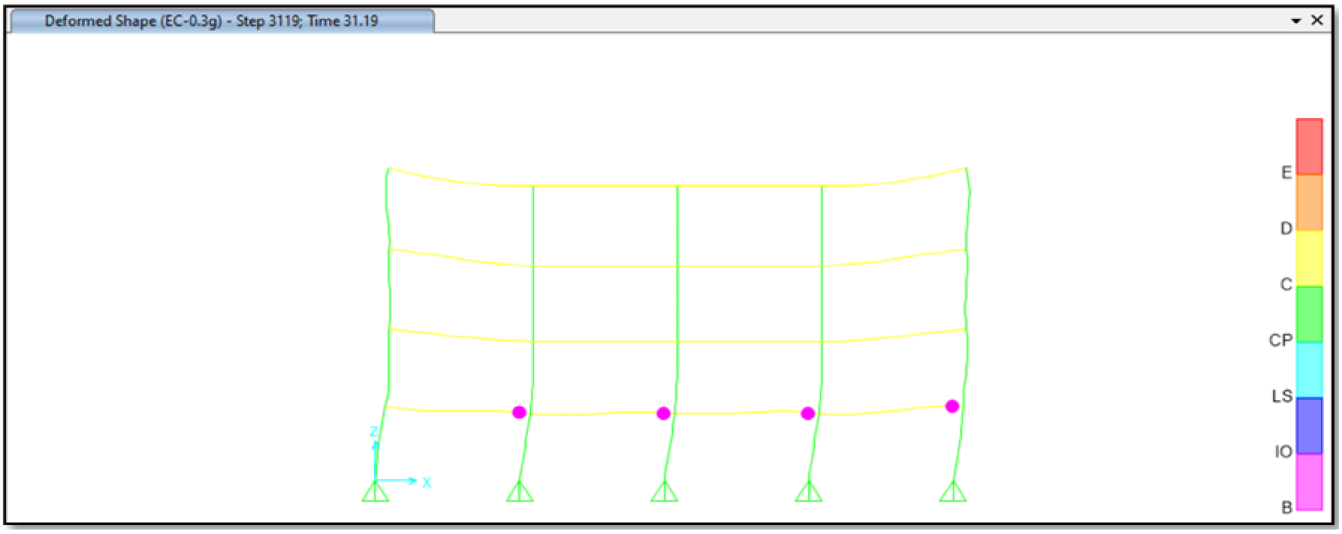

Figure 30. Hing Formation at 0.3g PGA for PSRC Structure

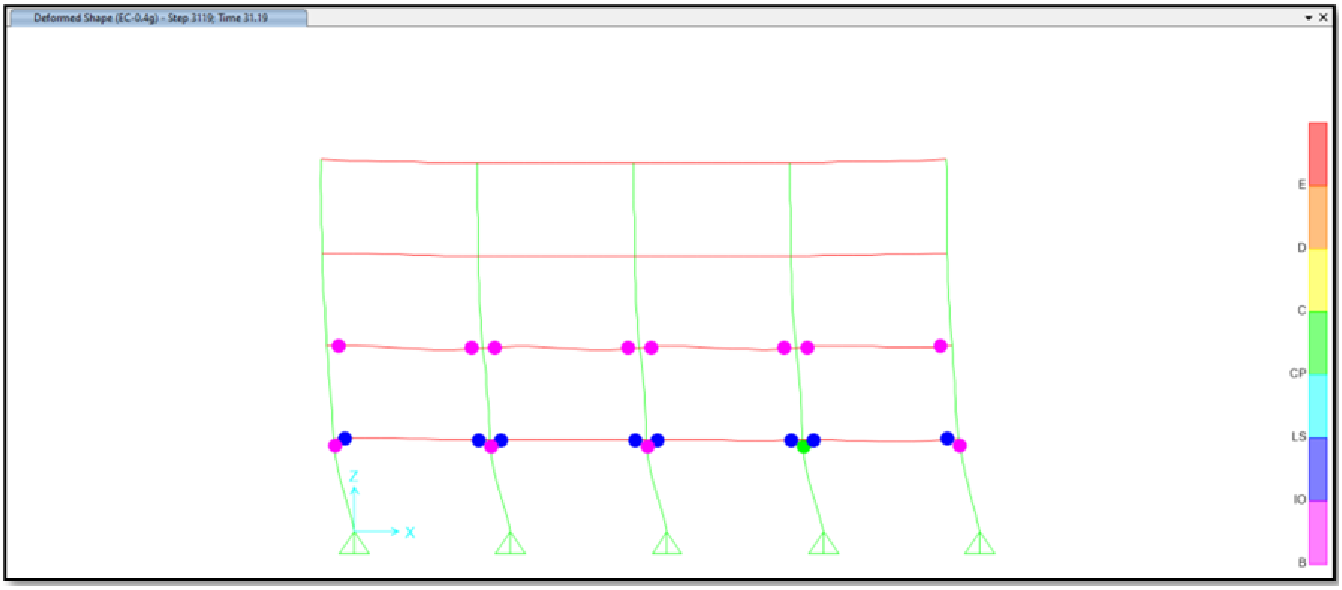

Figure 31. Hing Formation at 0.4g PGA for RCC Structure

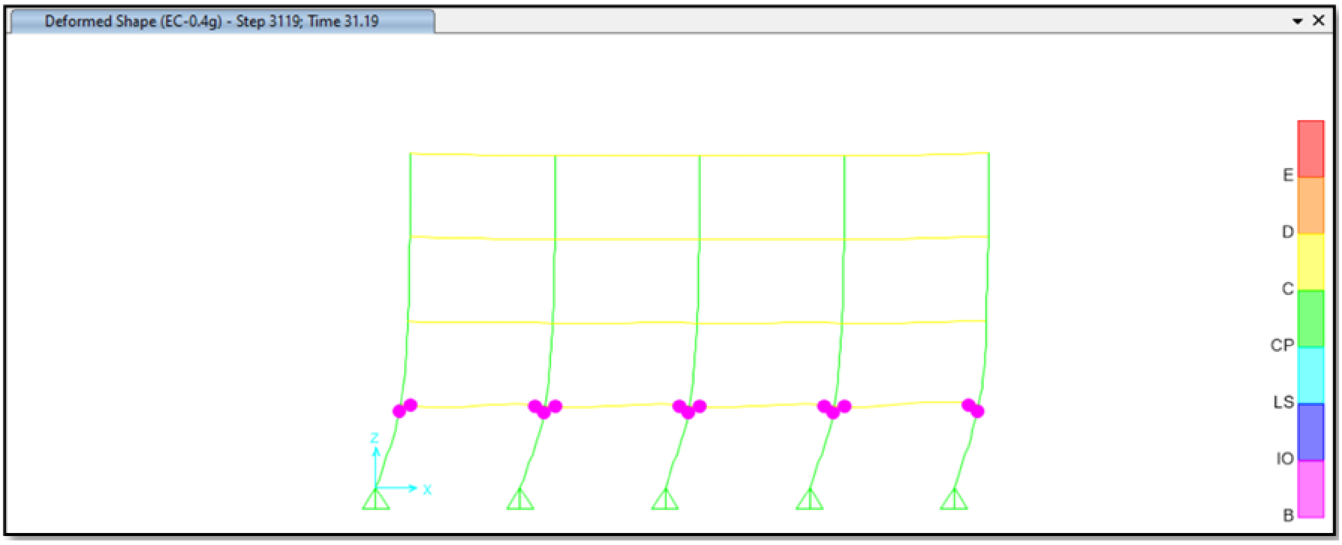

Figure 32. Hing Formation at 0.4g PGA for PSRC Structure

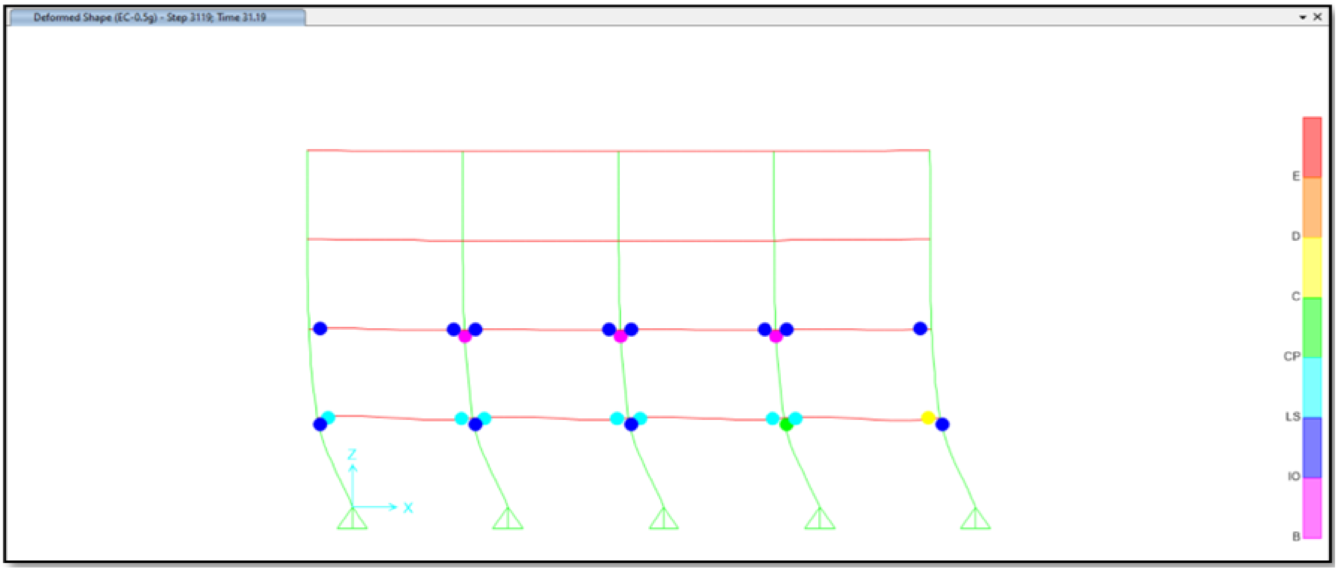

Figure 33. Hing Formation at 0.5g PGA for RCC Structure

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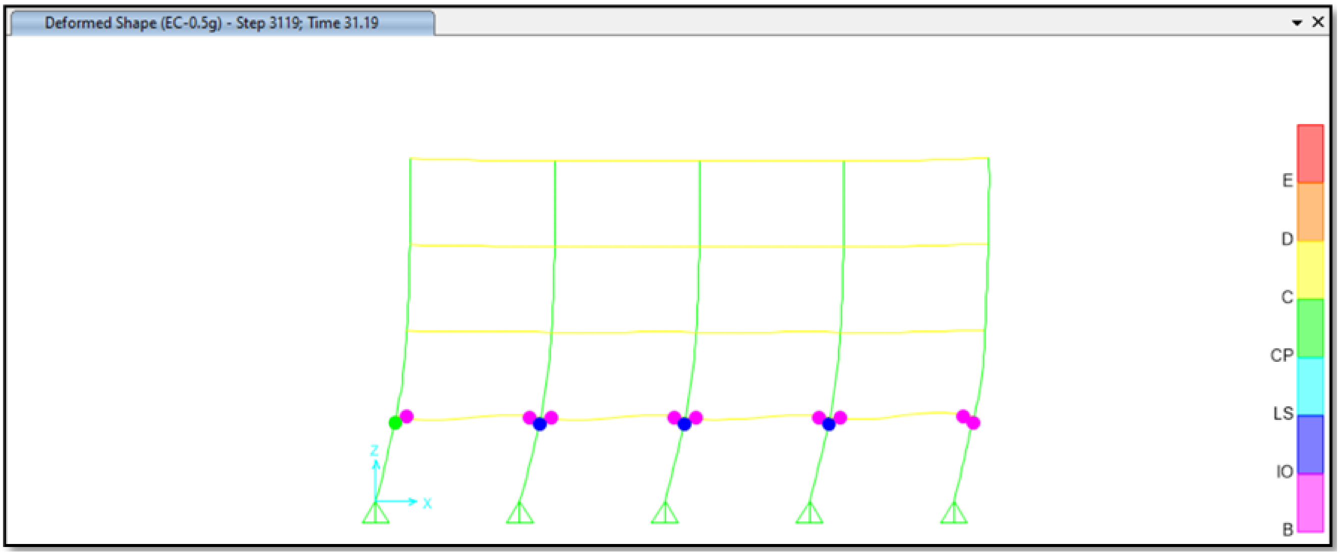

Figure 34. Hing Formation at 0.5g PGA for PSRC Structure

VI. CONCLUSION

The non-linear dynamic analysis was used to investigate the performance of PSRC structures. Two buildings (PSRC and RCC structures) were modeled to represent structures in seismic zone IV. A comparison of the PSRC structural system with the RCC structural system has been presented. The study results show that 15% less base shear in the PSRC structure than the RCC structure and no collapse stage reach in the PSRC structure while the RCC structure beams attained the collapse stage at 0.5g PGA. Hence, the PSRC structure performs better than the RCC structures and can be used in high seismic regions without loss of integrity during extreme earthquakes.

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It is optional. The preferred spelling of the word "acknowledgment" in American English is without an "e" after the "g." Use the singular heading even if you have many acknowledgments. Avoid expressions such as "One of us (S.B.A.) would like to thank... ." Instead, write "F. A. Author thanks" *Sponsor and financial support acknowledgments are placed in the unnumbered footnote on the first page.*

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AUTHORS PROFILE

Mohammad Arastu pursuing PhD from Jamia Millia Islamia University, completed his M.Tech. (Earthquake Engineering) in 2013 with Gold Medal and B.Tech. (Civil) in 2009. He is a member and charted engineer with the Indian Institute of Engineers (India). He has more than ten years of experience in structural analysis and design of Residential, Commercial, Industrial, Hospital, and Airports projects at national and international levels.

Professor Khalid Moin completed his PhD in 1997 at IIT Roorkee, M.Tech. in structures from IIT Delhi in 1990 and B.Sec. Engg. (civil) from AMU in 1985. He has more than 37 years of teaching experience at the UG and PG level, more than 29 years of research experience and more than 20 years of consultancy experience. The major areas of research are Structures, structural dynamics and Earthquake Engineering. He is also a member of the Indian Society of Earthquake Technology, IIT Roorkee, the Member of Building Committee, Jamia University, and Member of Task Force Committee for finalising the XII Five-year Plan of the University. He has organised more than nine national and international conferences/symposia/workshops.

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