

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 extensive previous research, the use of the PSRC structural system is limited in high-seismic regions. This paper aims to investigate the progressive collapse of the PSRC building using nonlinear dynamic analysis and guidelines from the U.S. General Services Administration (GSA) during extreme loading. Two structures are studied to validate the performance of the progressive collapse of PSRC and RCC structures—the Indian Codes of Practice design four-story PSRC and RCC buildings. 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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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, resulting in substantial reductions in the total building load, foundation costs, and earthquake forces. In years, the PSRC structural systems for previous moment-resisting have primarily 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.

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Traditional steel frames were also investigated in these studies to benchmark the performance of conventional structures 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 connection between the RCC column and steel girders. 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 the 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 valuable information that linear static or dynamic analysis procedures cannot obtain [10]. This paper aims to investigate the seismic performance of the PSRC system for buildings in comparison to that of RCC buildings. The scaled 1940 El Centro (N.S. component) time history, which ranges from 0.1g to 0.5g PGA, has been used in 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 nonlinear direct-integration time-history analysis.

A scaled time history of the 1940 El Centro (N.S. component) with PGA ranging from 0.1g to 0.5g has been applied to the structure's base (Figures 2, 3, 4, 5, and $\underline{6}$).

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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 formation of hinges determines the performance of the structure. 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 behaviour, 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 damage. Minor repairs of these nonstructural elements can be made 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 significant hazards from falling debris. 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. A substantial threat of injury may occur due to the collapse of structural

Retrieval Number: 100.1/ijrte.A76170512123 DOI: <u>10.35940/ijrte.A7617.0512123</u> Journal Website: <u>www.ijrte.org</u> 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 the PSRC and RCC structures to study. These consist of a typical steel girder and a Precast RCC column frame building—the Indian Codes of Practice design four-story PSRC buildings. Design columns under provisions of the Indian reinforced concrete structures code, and beams are designed according to the Indian steel construction code.

The yield strength of the longitudinal and transverse bars for RCC beams and columns is 500 N/mm². The compressive strength of the concrete used was 25 MPa at 28 days. The structural steel had a yield strength of 250 N/mm², which was used in the analysis—the stress-strain curve used for the non-linear dynamic analysis (Figures 8 and 9).



Figure 8. Stress-Strain Curve for Concrete



Figure 9. Stress-Strain Curve for Steel

The column centre-to-centre dimensions were 5000 mm in both directions. The model is assumed to be pinned at the base. The Indian Code of Practice has prepared the column and beam details. The 300mm-wide and 400mm-deep beam, with three 16mm-diameter bars at the top and bottom, was used at all levels and in both directions, plus an extra 2T16 at the support.

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The 400mm x 400 mm 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

maintained at 3000mm, centred on the beam on all floors. For the PSRC structural system, steel girders of ISM 300 are considered—the section properties of both frames (Figure 10 and Figure 11).

	B400x300-M25	B400x300-M25	B400x300-M25	B400x300-M25
	B400x300-M25	B400x300-M25	B400x300-M25	B400x300-M25
	B400x300-M25	B400x300-M25	B400x300-M25	B400x300-M25
	B400x300-M25	B400x300-M25	B400x300-M25	B400x300-M25
25	25	25	25	25
M-00	W-00	W-00	W-00	W-00
400x4	400x4	400x4	400x4	400x4
ð	ð	Ŭ.	Ŭ	ٽ ک

Figure 10. RCC Frame Structure Section Properties

ISMB300 ISM300		ISM300	ISM300	ISM300	
	ISMB300	ISMB300	ISMB300	ISMB300	
	ISMB300	ISMB300	ISMB300	ISMB300	
	ISMB300	ISMB300	ISMB300	ISMB300	
C400x400-M25	C400x400-M25	C400x400-M25	C400x400-M25	C400x400-M25	



V. RESULT AND DISCUSSION

The El Centro time history was applied at the base of both structures, ranging from 0.1 g to 0.5 g 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 utilised according to FEMA

Retrieval Number: 100.1/ijrte.A76170512123 DOI: <u>10.35940/ijrte.A7617.0512123</u> Journal Website: <u>www.ijrte.org</u> 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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84



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 the PSRC structure due to the usage of steel girders, and hence, less base shear is computed as compared to the 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 1. Base Reaction for RCC Structure

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 structures for 0.5g to 0.1g PGA (Figures 12, 13, 14, 15 and 16). It has been noted that the maximum base shear is 16% less for the 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



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Figure 15. Base Shear Time History at 0.2g 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 Figures 17, 18, 19, 20 and 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, according to these results.



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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 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 Figures 22, 23, 24, 25 and <u>26</u> 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 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



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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 in Figures 27, 28, 29, 30, 31, 32, 33 and <u>34</u>. 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 was observed in columns in both structures 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 in the columns of the PSRC structure. At 0.5g, the collapse stage is reached at the first story beams in the RCC structure, and no collapse stage is reached in the PSRC structure.



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

90







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. CCONCLUSION

The non-linear dynamic analysis was used to investigate the performance of PSRC structures. Two buildings (PSRC and RCC structures) were modelled 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 was reached 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 "acknowledgement" in American English is without an "e" after the "g." Use the singular heading even if you have many acknowledgements. Avoid expressions such as "One of us (S.B.A.) would like to thank..." Instead, write "F. A. The author thanks" Sponsor and financial support acknowledgements *are placed in the unnumbered footnote on the first page*.

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DECLARATION

REFERENCES

- C. Pearson and N. Delatte, "Ronan Point Apartment Tower Collapse and its Effect on Building Codes," Journal of Performance of Constructed Facilities, vol. 19, no. 2, pp. 172–177, 2005, doi: 10.1061/(ASCE)0887-3828(2005)19:2(172). [CrossRef]
- L. G. Griffis, "Some design considerations for composite-frame structures," Engineering Journal, AISC, pp. 59–64, 1986.
- 3. T. M. Sheikh, J. A. Yura, J. O. Jirsa, and P. M. Ferguson, "MOMENT CONNECTIONS BETWEEN STEEL BEAMS AND CONCRETE COLUMNS AFTER AN INSTALLATION OF STEEL CONNECTIONS," 1987.
- 4. [X. Liang, G. J. Parra-montesinos, and J. K. Wight, "3rd World Conference on Earthquake Engineering SEISMIC BEHAVIOR OF RCS BEAM-COLUMN-SLAB SUBASSEMBLIES DESIGNED FOLLOWING A CONNECTION DEFORMATION-BASED CAPACITY DESIGN APPROACH."
- 5. S. S. F. Mehanny, G. Heger, P. Cordova, and G. G. Deierlein, "SEISMIC DESIGN OF COMPOSITE MOMENT FRAME BUILDINGS-CASE STUDIES AND CODES IMPLICATIONS."
- B. N. Michael Bugeja, S. Member, J. M. Bracci, W. P. Moore Jr, and H. Member, "SEISMIC BEHAVIOR OF COMPOSITE RCS FRAME SYSTEMS," 2000.
- 7. N. Baba and Y. Nishimura, "SEISMIC BEHAVIOR OF RC COLUMN-S BEAM MOMENT FRAMES."
- H. Noguchi and K. Uchida, "Finite Element Method Analysis of Hybrid Structural Frames with Reinforced Concrete Columns and Steel Beams", doi: 10.1061/ASCE0733-94452004130:2328.
- P. P. Cordova and G. G. Deierlein, "VALIDATION OF THE SEISMIC PERFORMANCE OF COMPOSITE RCS FRAMES: FULL-SCALE TESTING, ANALYTICAL MODELING, AND SEISMIC DESIGN," no. 155, 2005.
- M. Inel and H. B. Ozmen, "Effects of plastic hinge properties in nonlinear analysis of reinforced concrete buildings," Eng. Struct., vol. 28, no. 11, pp. 1494–1502, Sep. 2006, doi: 10.1016/j.engstruct.2006.01.017. [CrossRef]
- O. M. A. T. S. J. M.Mouzzoun, "Seismic performance assessment of reinforced concrete buildings using pushover analysis," IOSR Journal of Mechanical and Civil Engineering, vol. 5, no. 1, pp. 44–49, 2003. [CrossRef]
- 12. J. Nicoletti, J. A. Blume, S. Francisco, and R. Wright, "Pennsylvania (representing the Building Officials and Code Administrators International)."
- 13. R. W. Niewiarowski and C. Rojahn, "Seismic Evaluation and Retrofit of Concrete Buildings.

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94