

Seismic Study of Precast Steel-Reinforced Concrete Building using Shake Table Test

Mohammad Arastu, Khalid Moin



Abstract: Precast Steel-Reinforced Concrete (PSRC) structural frame systems for moment-resisting, comprised of Prefabricated Steel (S) girders and Precast Reinforced Cement Concrete (RCC) columns. This structural system has the advantage of inherent stiffness and damping during a seismic event. PSRC's moment-resisting frame system is also known for its construction efficiency, lightweight, and low cost. Earlier investigations have shown PSRC systems helpful in designing and constructing buildings while maintaining ample strength and high ductility during seismic incidents. Despite much previous research, the use of the PSRC structural system in India is still limited. Previous studies have accepted a vital need to test comprehensive structural systems, both experimentally and analytically - to validate the knowledge collected to date and act as evidence of concept for the PSRC moment-resisting frame system. This paper aims to facilitate more recognition and use of the PSRC structural system as a feasible choice for traditional RCC lateral resisting systems. A shake-table test was conducted to evaluate the PSRC building performance during maximum considered earthquake events. The comparative study of experimental and numerical results of the 1/4th scaled building is presented.

Keywords: Composite Structures; Crack; Earthquakes/ Earthquake Loading; Failure/ Failure Mode; Hybrid Structures; Nonlinear Analysis; Structural Analysis

I. INTRODUCTION

Modernizing steel and concrete structures provides attractive alternatives to reinforced concrete systems. PSRC structural systems for moment-resisting comprise Prefabricated Steel (S) girders and Precast Reinforced Cement Concrete (RCC) columns. They have the advantage of inherent stiffness and damping during a seismic event. PSRC's moment-resisting frame system is also known for its construction efficiency, lightweight, and low cost. [4,6].

PSRC frame systems have been shown to retain numerous advantages from economic and construction viewpoints [8,10] compared to RCC or steel frame systems.

RCC columns are nearly ten times more efficient than steel columns in axial strength and axial stiffness [9]. 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 mainly been used for buildings in developed countries' low seismicity areas. In recent years, researchers have attempted to develop seismic design guidelines for PSRC systems in high seismic-risk regions [4,5,6].

The Indian subcontinent has a history of earthquakes. The intensity and high frequency of earthquakes is the Indian plate driving into the Asian region at approximately 47 mm/year [7]. Significant earthquakes like the Jabalpur earthquake (1997), the Chamoli earthquake (1999), the Bhuj earthquake (2001), and the recent Nepal earthquake (2015) have highlighted the need for an extensive study to understand the behavior of PSRC structures under seismic loading. Hence, the performance of such PSRC structures needs to be assessed experimentally and numerically under moderate/severe earthquakes.

One efficient and practical method of assessing the performance of a building under seismic loading is shake-table testing. The present study aims to evaluate the engineering parameters such as natural period, story acceleration, velocity, displacement, and damage pattern of 1/4th scaled PSRC building structures against the horizontal forces produced by scaled El-Centro earthquake [2] using a shake table. One bay and four stories, a three-dimensional building structure, and the scaled 1940 EL-Centro (N.S. component) time history of 0.4g to 2.0g PGA have been used for the study.

II. SCALING AND GEOMETRY OF STRUCTURE

One bay and four stories PSRC structure have been scaled down on a 1/4th scale according to the scale factors obtained from similitude consideration for earthquake loading [1]. The summary of these scale factors is given in Table 1. The structure's overall dimensions have been chosen based on the limitation of the sizes and capacity of the shake table (2m x 2m plan dimensions).

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Table 1: Summary of Scale Factors for Earthquake Response of the Structure

		Dimension	Gravity Force Neglected Prototype Material
Loading	Force	F	S^2
	Pressure	FL^{-2}	1
	Acceleration	LT^{-2}	S^{-1}
	Gravitational acceleration, g	LT^{-2}	Neglected
	Time	T	S
Geometry	Linear dimension	L	S
	Displacement	L	S
	Frequency	T^{-1}	S^{-1}
Material Properties	Modulus of Elasticity	FL^{-2}	1
	Stress	FL^{-2}	1
	Strain	-	1
	Mass Density	$FL^{-4}T^2$	1
	Poison's ratio	-	1

The column center-to-center dimensions were 1750 mm in both directions. The model was fixed at the base with a shake table using a 300x300x8mm base plate and 4-M12 bolts under each column. The base plan and framing plan of the scaled PSRC structure model is shown in [Fig. 1](#).

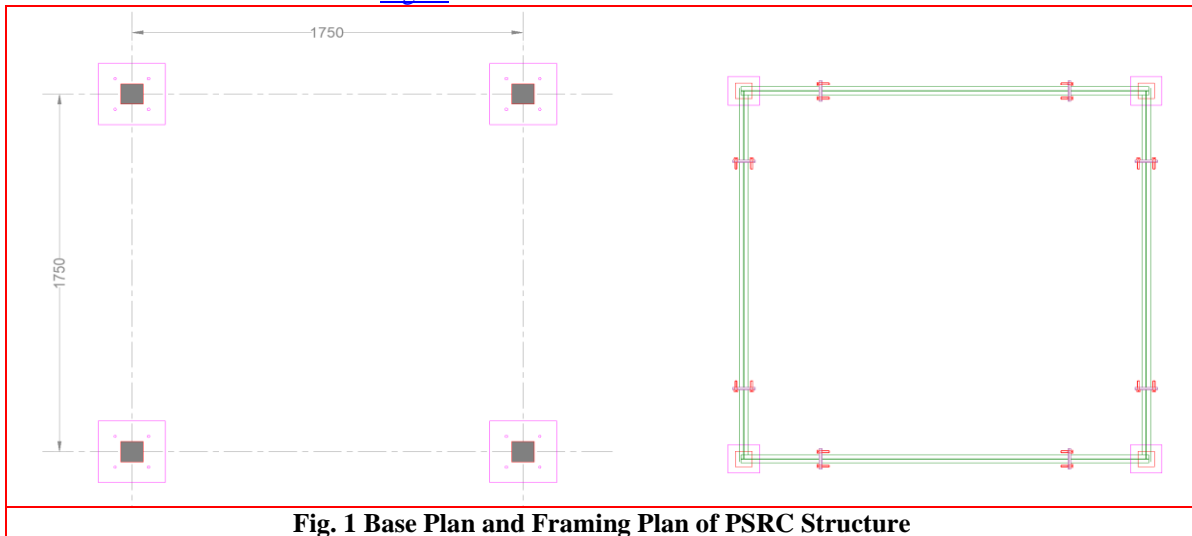


Fig. 1 Base Plan and Framing Plan of PSRC Structure

A Precast RCC square column of 100mmx100mm with four longitudinal bars of 8mm diameter and 2mm diameter stirrups at 50mm c/c along the height was used. The prefabricated steel tubes of 25x25x2mm size were used for the beam at all levels and in both directions. The 300mm length at both ends of the beam was embedded with a Precast RCC column, as shown in [Fig 2](#). The central part of the beam was joined with plates and bolts. The sequence of the joining prefabricated steel beams and precast RCC columns is shown in [Fig 3](#). The story height was kept at 750mm c/c of the beam on all floors. Column-beam general arrangement for the four-story structure and the final 1/4th scaled model for testing is shown in [Fig 4\(a\)](#) and [Fig 4\(b\)](#).

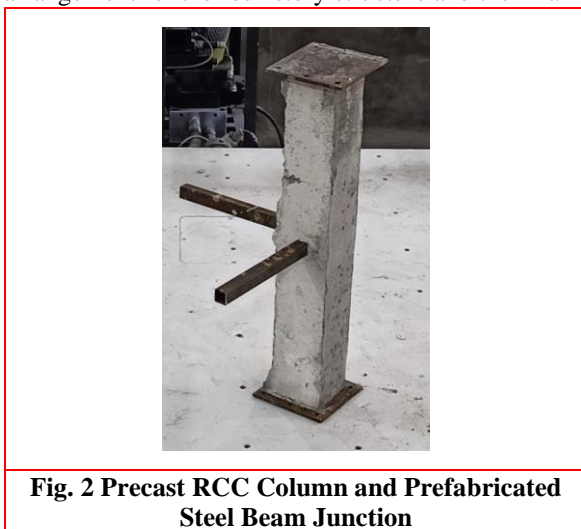


Fig. 2 Precast RCC Column and Prefabricated Steel Beam Junction

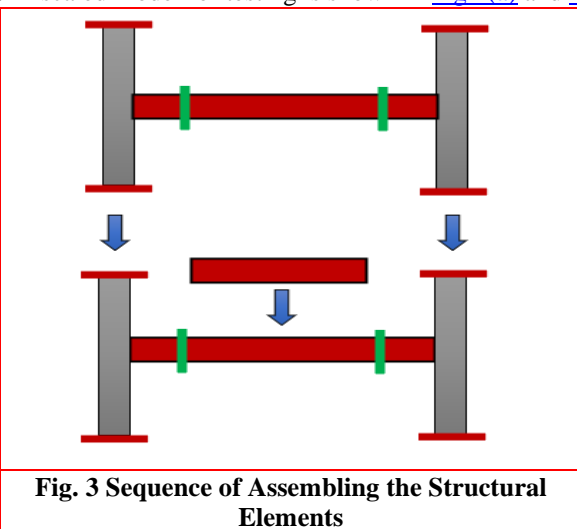


Fig. 3 Sequence of Assembling the Structural Elements

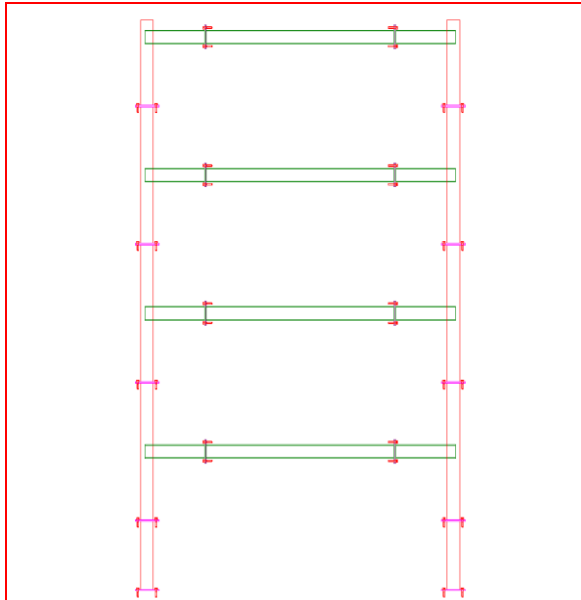


Fig. 4(a) 1/4th PSRC Scaled Model General Arrangement



Fig. 4(b) 1/4th PSRC Scaled Model mounted on Shake Table

III. MECHANICAL PROPERTIES OF MATERIAL

All material properties were tested in the laboratory before the shaking table test. The longitudinal and transverse bars' yield strength was 500N/mm^2 and 250N/mm^2 , respectively. The average tested compressive strength of micro concrete used was 25 MPa in seven days. The stress-strain curve used for numerical modelling has been considered as per Indian standards and shown in Fig. 5(a) and Fig. 5(b).

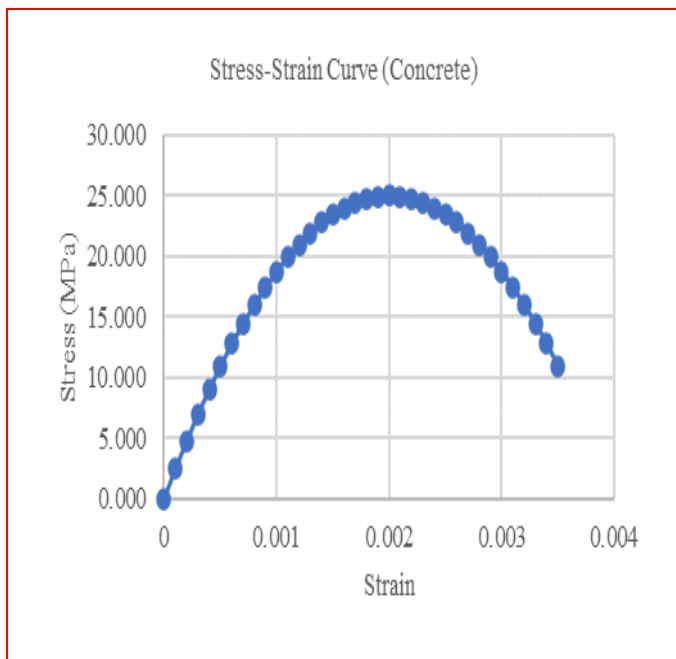


Fig. 5(a) Stress-Strain Curve for Concrete

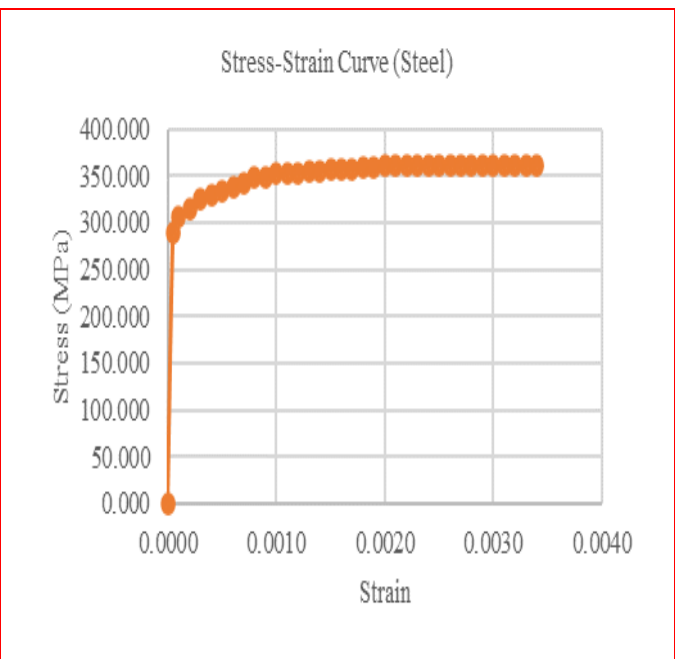


Fig. 5(b) Stress-Strain Curve for Steel

IV. INPUT GROUND MOTION

The 1940 El Centro earthquake scaled time history was used at different peak ground acceleration (PGA) values shown in Fig. 6. The displacement time histories shown in Fig. 7 were generated from scaled El-Centro (N-S Component) time histories and were used for shaking the table.

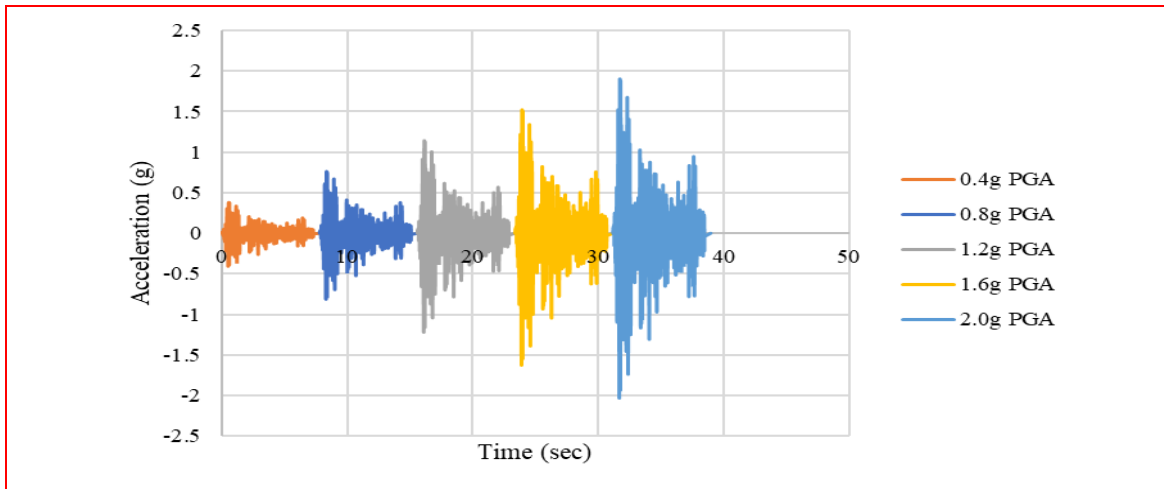


Fig. 6 Scaled El-Centro Time-Histories for different PGA values

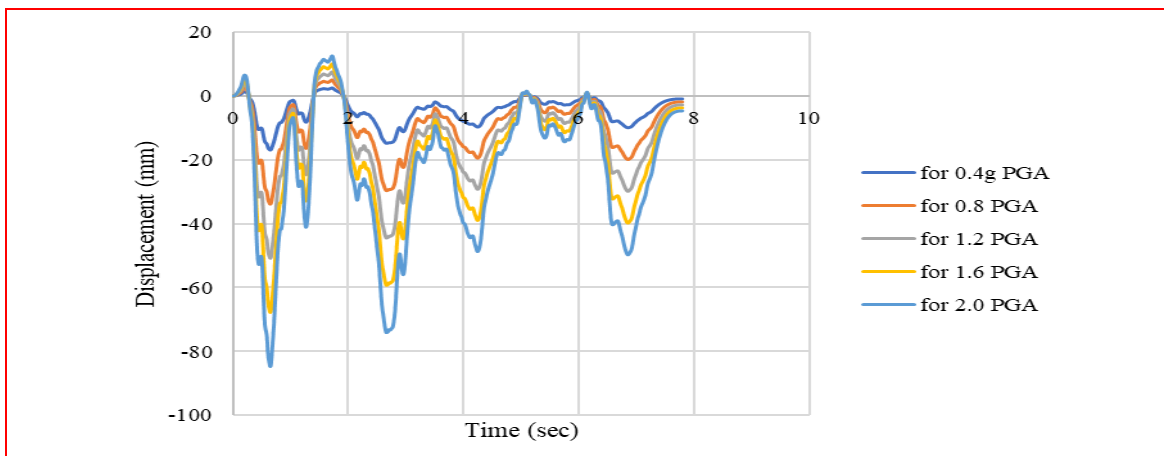


Fig. 7 Scaled Displacement Time-Histories for different El-Centro PGA values

V. SHAKE-TABLE TESTING

The free vibration was given to the model by pulling with the help of a rope. Top-story acceleration was recorded, as shown in Fig. 8. The Fast Fourier Transform of records was carried out using the standard software, as shown in Fig. 9. Before conducting the shake table test, the natural frequency of the structure in the first mode has been evaluated experimentally as 10.70 Hz and damping considered is 0.0417. The ground input motion was given with increased PGA values, and a free vibration test was carried out after each cycle test to calculate the structure’s natural frequency. The results of natural frequency are reported in Table 2.

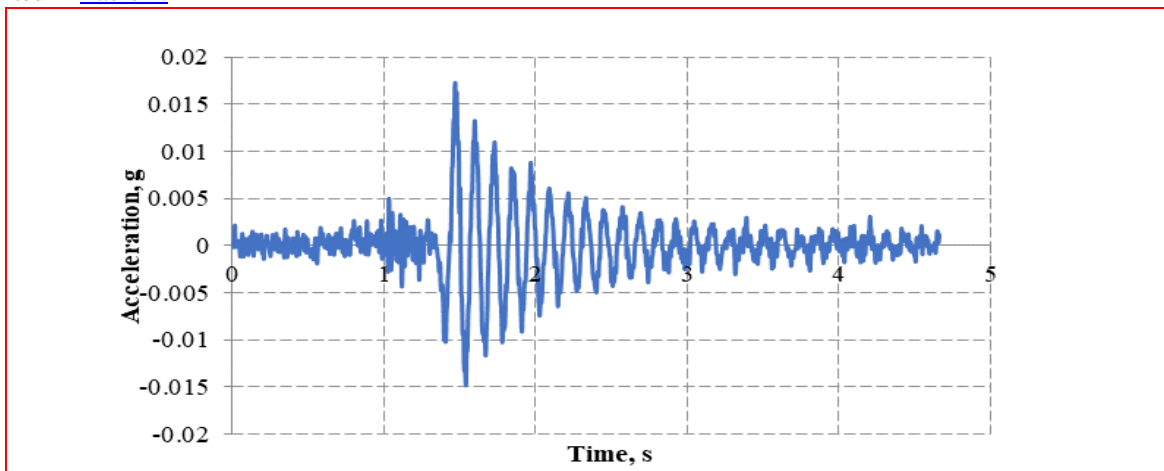


Fig. 8 Records of top story acceleration due to free vibration before testing

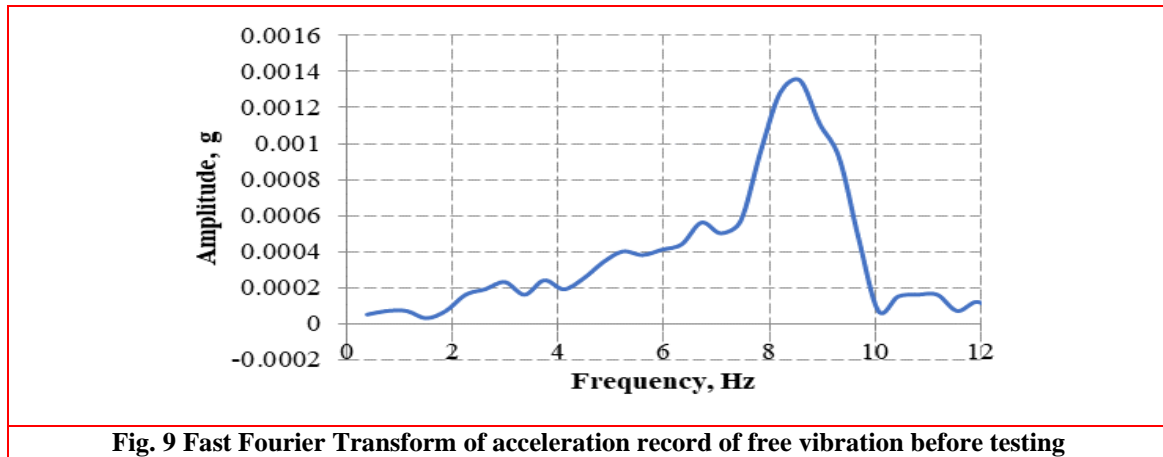


Table 2: Natural Frequencies and Damping Ratio of the model after each cycle of testing

Parameters	0.4g	0.8g	1.2g	1.6g	2.0g	Unit
Natural Frequency:	9.20	8.84	7.45	7.20	6.950	Hz
Damping Ratio:	0.0482	0.046	0.0448	0.0423	0.0478	

The recorded top story maximum acceleration values against each ground acceleration at the base are given in [Table 3](#).

Table 3: Top Story Acceleration of the model after each cycle of testing

Parameters	0.40g	0.80g	1.20g	1.60g	2.00g
Top Story measured PGA	2.15g	2.65g	2.87g	3.43g	3.97g

It was observed visually that no cracks developed within the precast RCC column-Steel beam joints during the complete test from 0.4g to 2.0g. The precast RCC columns started cracking at the base due to tensile and compressive forces at 0.6g and were completely damaged at 2.0g, as shown in [Fig. 10\(a\)](#) and [Fig 10\(b\)](#). The steel beam failure occurs at the first and second story at the steel-to-steel junction, as shown in [Fig. 10\(c\)](#) and [Fig. 10\(d\)](#), and no failure is noticed at the precast RCC to the steel junction.



VI. NON-LINEAR DYNAMIC ANALYSIS

The structures deform inelastically during the maximum considered earthquake (MCE) [2]. Hence structural performance must be checked during the post-elastic behavior of the system. 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 behavior during such events. Moreover, to estimate the seismically induced needs that exhibit inelastic behavior, the structures' maximum inelastic displacement demand should be determined adequately. The dynamic non-linear analysis method applies the ground acceleration time history to the structure. Dynamic equilibrium equations are solved using direct integration methods [2]. Initial conditions are set by continuing the structural state from the end of the previous non-linear gravity analysis.

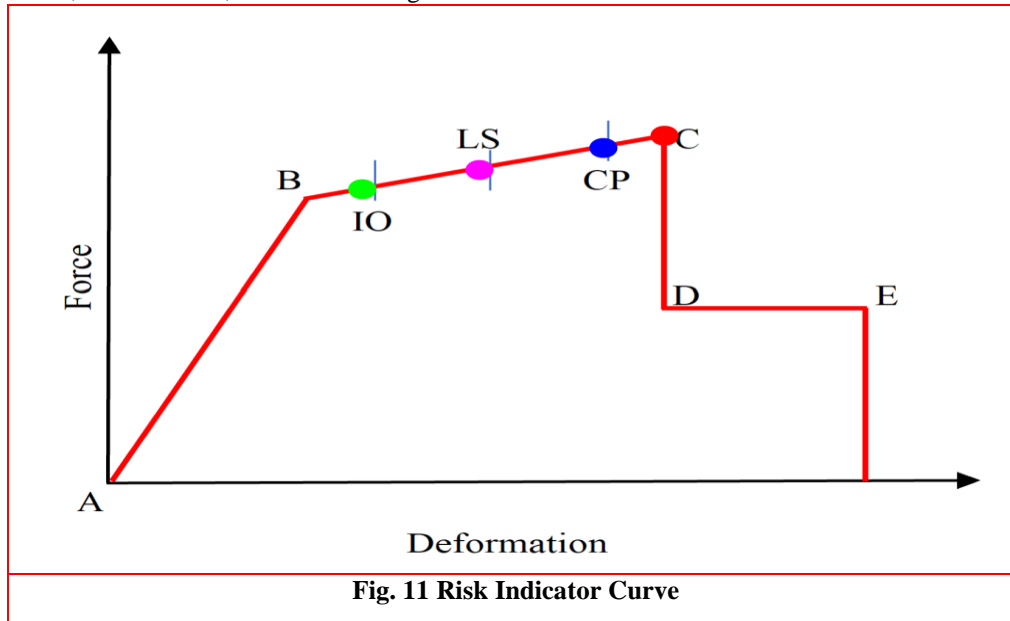
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Direct-integration methods are sensitive to time-step size, which should be decreased until results are unaffected. Material and geometric nonlinearity, including P-delta effects, have been simulated during non-linear direct-integration time-history analysis. A scaled time history of 1940 EL-Centro (N.S. component) from 0.4g to 2.0g PGA with an increment of 0.4g has been applied to the structure's base.

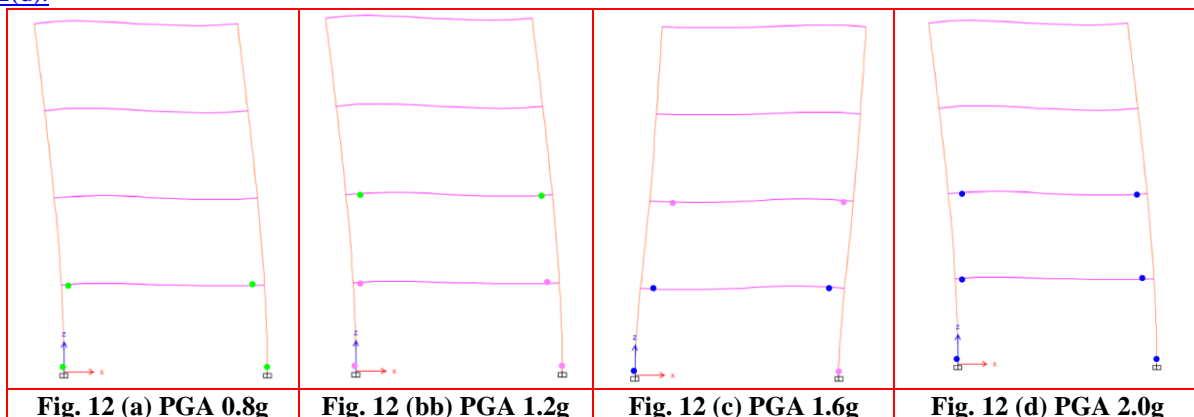
Table 4: Top Story Acceleration of the model after each cycle of testing

Parameters	0.40g	0.80g	1.20g	1.60g	2.00g
Top Story measured PGA	2.47g	3.11g	3.30g	3.87g	4.33g

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 [8]. The performance of the structure is determined by hinge 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 (Fig. 11). 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 El-Centro time history was applied at the base of both structures from 0.4g to 2.0g PGA with an increment of 0.4g. The direction of monitoring the building's behavior 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 [3]. Column bases are assumed to be fixed at the base 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. From the numerical analysis, it has been observed that there is no hinge formation at 0.4g acceleration, and the first thing forms at the base of the column at 0.6g; subsequently, the formation of beams occurs in beams. The structure does not reach the collapsed state at 2.0g acceleration. The hinge formation patterns at different PGA values are shown in Fig. 12(a) to Fig. 12(d).



VII. CONCLUSIONS

It was observed visually that no cracks developed within the precast RCC column-Steel beam joints during the complete test from 0.4g to 2.0g. The precast RCC columns started cracking at the base due to tensile and compressive forces at 0.6g and were completely damaged at 2.0g. The steel beam failure occurred at the first and second story at the steel-to-steel junction, and no failure was noticed at the precast RCC to the steel junction. The non-linear dynamic analysis was also used to investigate the performance of Precast Steel-Reinforced Concrete structures. The numerical results show that the top story acceleration values recorded from the shake table test are the same with approx. 15% difference from those obtained from numerical analysis. The hinge formation patterns almost match the failure pattern observed during the shake table test. The study concludes that the Precast Steel-Reinforced Concrete Building can also be used in high seismic zones.

DECLARATION

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Ethical Approval and Consent to Participate	No, the article does not require ethical approval and consent to participate with evidence.
Availability of Data and Material/ Data Access Statement	Not relevant.
Authors Contributions	All authors have equal participation in this article.

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