International Journal of Innovative Technology and Exploring Engineering (IJITEE)
ISSN: 2278-3075 (Online), Volume-12 Issue-7, June 2023

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

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***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 demonstrated that PSRC systems are beneficial in designing and constructing buildings while maintaining sufficient strength and high ductility during seismic events. Despite extensive previous research, the use of the PSRC structural system in India remains limited. Previous studies have recognised the need to test comprehensive structural systems, both experimentally and analytically, to validate the knowledge collected to date and serve as evidence of concept for the PSRC moment-resisting frame system. This paper aims to increase recognition and adoption of the PSRC structural system as a viable alternative to traditional RCC lateral resisting systems. A shake-table test was conducted to evaluate the PSRC building performance during maximum considered earthquake events. A comparative study of the experimental and numerical results for the 1/4-scale building is presented.*

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

I. INTRODUCTION

Modernising steel and concrete structures offers attractive alternatives to traditional 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.

Manuscript received on 01 May 2023 | Revised Manuscript received on 06 May 2023 | Manuscript Accepted on 15 June 2023 | Manuscript published on 30 June 2023.

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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, resulting in substantial reductions in the total building load, foundation costs, and earthquake forces. In previous years, the PSRC structural systems for moment-resisting have primarily been used for buildings in areas of low seismicity in developed countries. 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, such as the Jabalpur earthquake (1997), the Chamoli earthquake (1999), the Bhuj earthquake (2001), and the recent Nepal earthquake (2015), have underscored the need for a comprehensive study to understand the behaviour of PSRC structures under seismic loading. Hence, the performance of such PSRC structures needs to be assessed both experimentally and numerically under moderate to 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. A one-bay, four-story building structure with a three-dimensional design and the scaled 1940 El Centro (N.S. component) time history, ranging from 0.4g to 2.0g PGA, has been utilised in the study.

II. SCALING AND GEOMETRY OF STRUCTURE

One bay and four stories PSRC structure has 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 limitations of the shake table's size and capacity (2m x 2m plan dimensions).

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

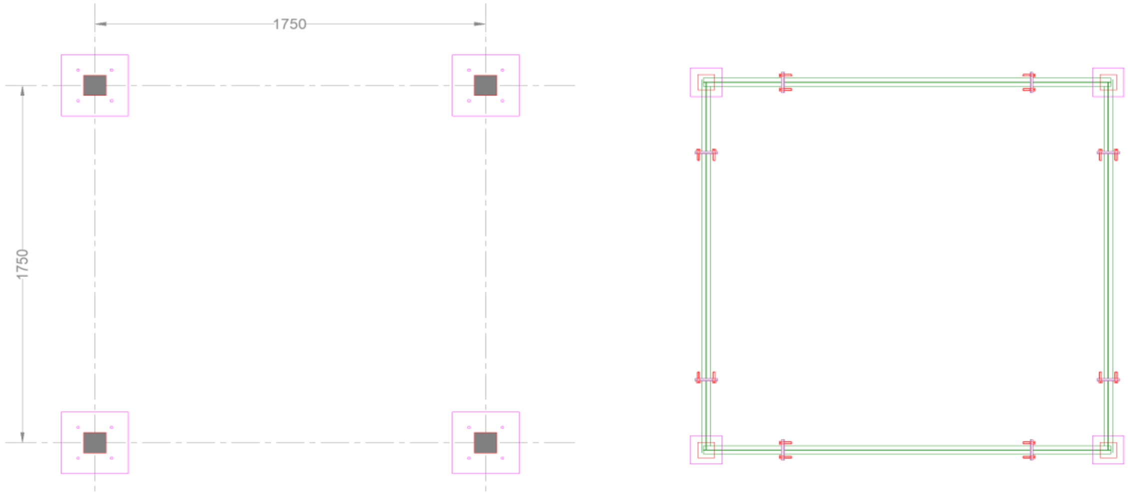

Fig. 1 Base Plan and Framing Plan of PSRC Structure

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 joining prefabricated steel beams and precast RCC columns is shown in Fig. 3. The story height was maintained at 750 mm, centred on the beam on all floors. The column-beam general arrangement for the four-story structure, along with the final 1/4-scale model for testing, is shown in Figs. 4(a) and 4(b).

Fig. 2 Precast RCC Column and Prefabricated Steel Beam Junction

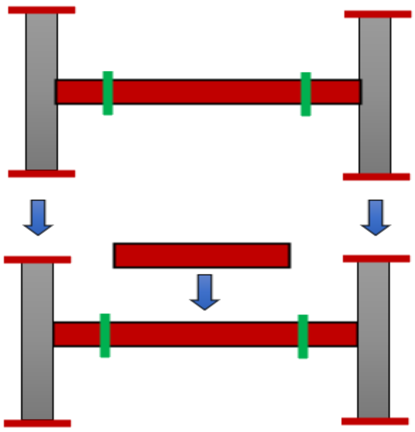

Fig. 3 Sequence of Assembling the Structural Elements

Retrieval Number: 100.1/ijitee.G95750612723
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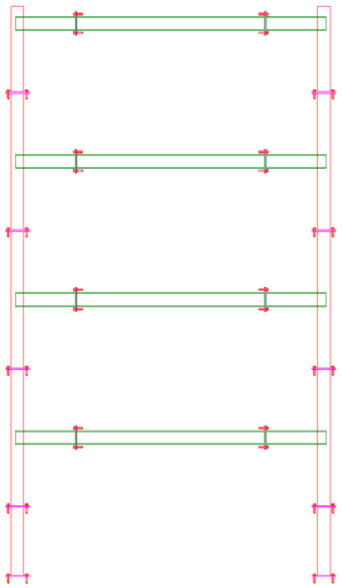

Fig. 4(a) 1/4th PSRC Scaled Model General Arrangement

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 compressive strength of the micro concrete tested was 25 MPa after seven days. The stress-strain curve used for numerical modelling has been considered by Indian standards and is shown in Figs. 5(a) and 5(b).

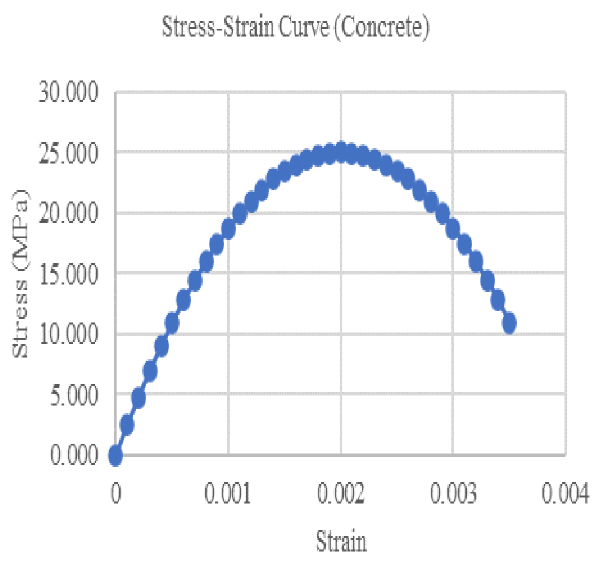

Fig. 5(a) Stress-Strain Curve for Concrete

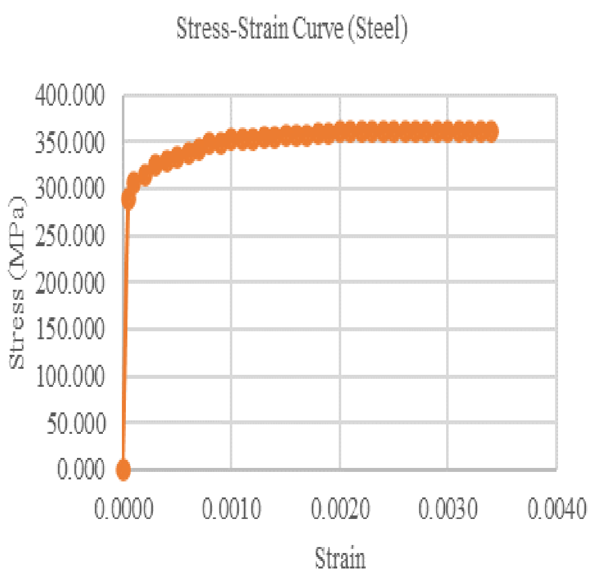

Fig. 5(b) Stress-Strain Curve for Steel

IV. INPUT GROUND MOTION

The 1940 El Centro earthquake scaled time history was used at different peak ground acceleration (PGA) values, as 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 to shake the table.

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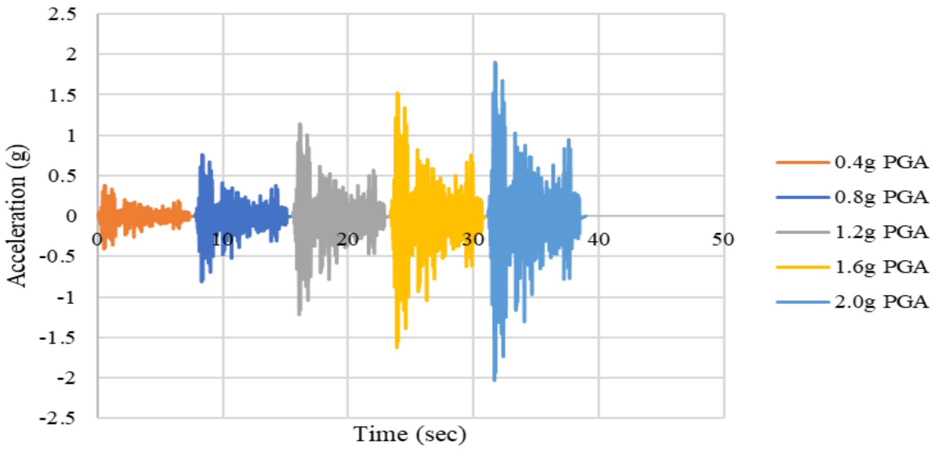

Fig. 6 Scaled El-Centro Time-Histories for different PGA values

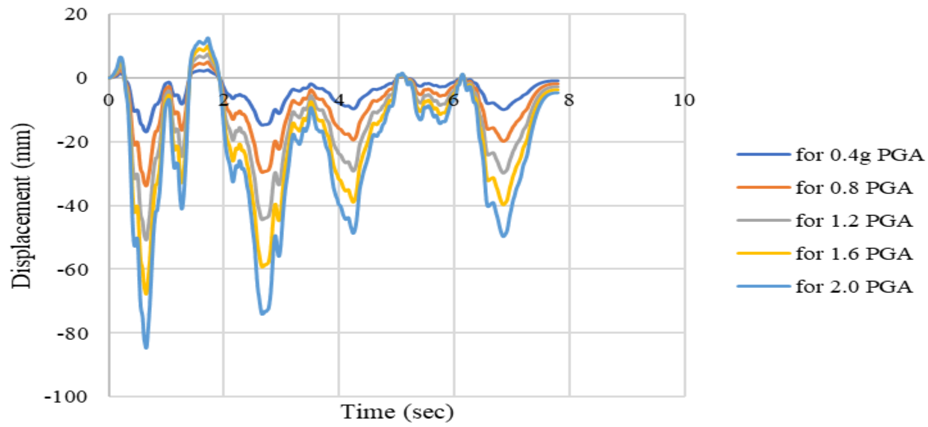

Fig. 7 Scaled Displacement Time-Histories for different El-Centro PGA values

V. SHAKE-TABLE TESTING

The model was subjected to free vibration by pulling it 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 the natural frequency are reported in Table 2.

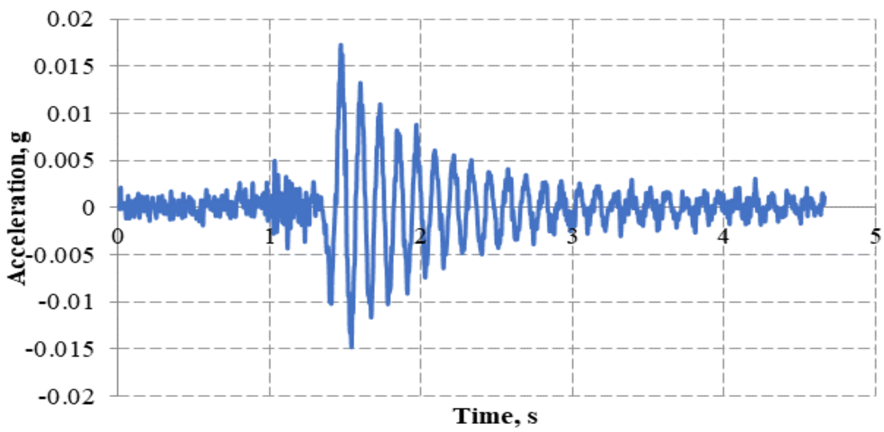

Fig. 8 Records of top story acceleration due to free vibration before testing

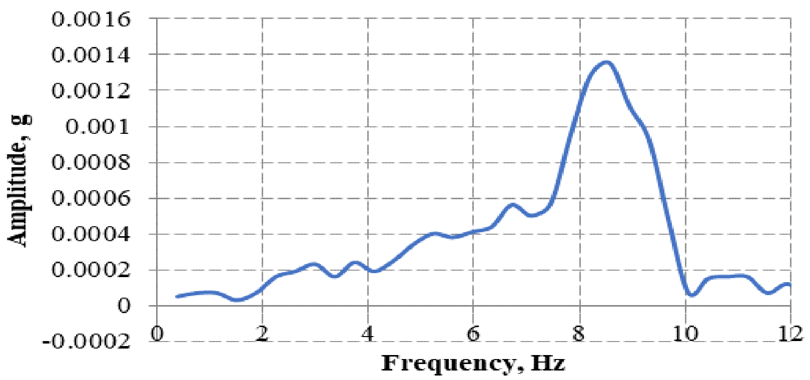

Fig. 9 Fast Fourier Transform of the acceleration record of free vibration before testing

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 maximum acceleration values for the top story 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 the PGA	2.15g	2.65g	2.87g	3.43g	3.97g

It was visually observed 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 the combined action of tensile and compressive forces at 0.6g and were completely damaged at 2.0g, as shown in Figs. 10(a) and 10(b). The steel beam failure occurs at the first and second stories at the steel-to-steel junction, as shown in Figs. 10(c) and 10(d), and no failure is observed at the precast RCC-to-steel junction.

Fig. 10(a) Precast RCC column, Crack from Base at 0.6g

Fig. 10(b) Precast RCC column, Crack from Base at 2.0g

Fig. 10(c) Steel-to-Steel Joint Failure at 2.0g

Fig. 10(d) Steel Beam Failure at 2.0g

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 behaviour of the system. Dynamic nonlinear analysis (also called Time History Analysis) should be used to evaluate seismic performance because elastic analysis cannot determine the structure's post-elastic behaviour 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.

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 nonlinear direct-integration time-history analysis. A scaled time history of the 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 the PGA	2.47g	3.11g	3.30g	3.87g	4.33g

A building's performance level is a combination of the structure's performance level and the non-structural components. A performance level describes a limiting damage condition, which may be considered satisfactory for a given building with specific ground motion [8]. The formation of hinges determines the performance of the structure. Various types of plastic hinges, including uncoupled and 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 exhibits linear behaviour. After that, one or more hinges will start to form.

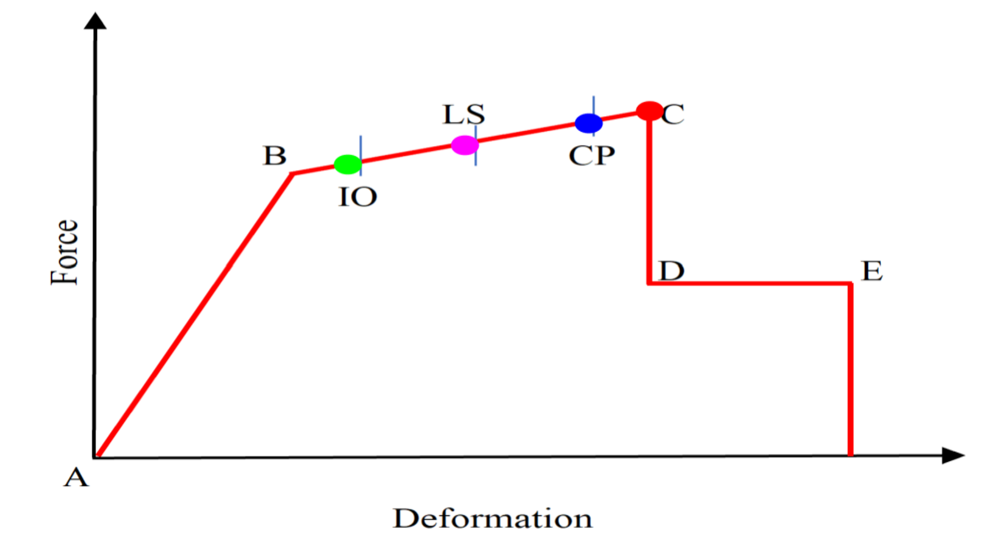

Fig. 11 Risk Indicator Curve

The El Centro time history was applied at the base of both structures, ranging from 0.4g to 2.0g PGA in increments of 0.4g. The direction of monitoring the building's behaviour was the same as the direction of the ground acceleration. For columns, program-defined auto PM2M3 interacting hinges were used at both ends and for beams, M3 auto hinges were utilised 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 an acceleration of 0.4g, and the first formation occurs at the base of the column at 0.6g; subsequently, the formation of beams occurs in the beams. The structure does not reach the collapsed state at an acceleration of 2.0g. The hinge formation patterns at different PGA values are shown in Fig. 12(a) to Fig. 12(d).

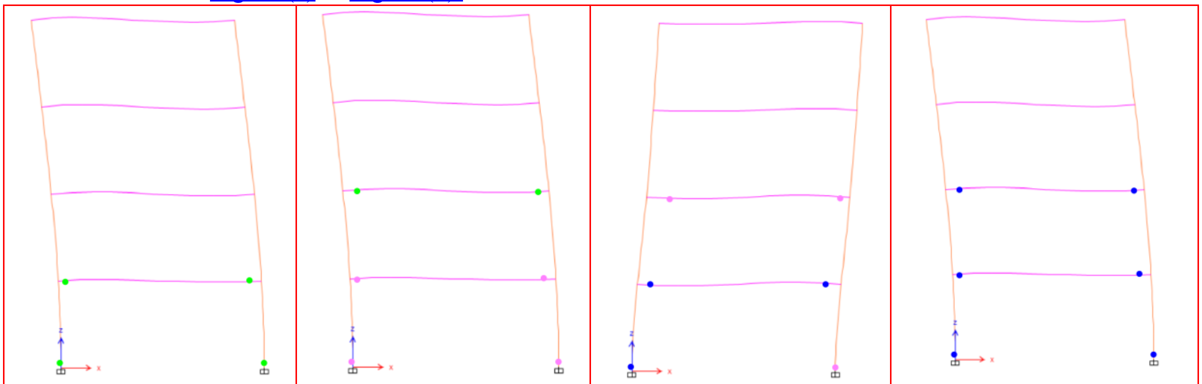

Fig. 12 (a) PGA 0.8g	Fig. 12 (bb) PGA 1.2g	Fig. 12 (c) PGA 1.6g	Fig. 12 (d) PGA 2.0g

VII. CONCLUSIONS

It was visually observed 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 began to crack at the base due to the combined effects of 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 observed at the precast RCC-to-steel junction. The non-linear dynamic analysis was also used to investigate the performance of Precast Steel-Reinforced Concrete structures. The numerical results indicate that the top-story acceleration values recorded during the shake table test are consistent, with an approximately 15% difference from those obtained through 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

Funding/ Grants/ Financial Support	No, I did not receive it.
Conflicts of Interest/ Competing Interests	No conflicts of interest to the best of our knowledge.
Ethical Approval and Consent to Participate	No, the article does not require ethical approval or consent to participate, as it presents 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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