Effect of Structural Stiffness Irregularities on the Blast Performance of High-Rise Buildings during Explosion-Induced Disasters

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Abstract
Rapid urbanization and vertical growth, especially in densely populated countries like India, have increased the demand for high-rise buildings. These structures are more vulnerable to dynamic loads such as earthquakes and wind and often exhibit geometric, mass, or stiffness irregularities, further compromising their performance. Additionally, the growing threat of terrorist attacks has highlighted the need to consider blast loads, which are significantly more intense than conventional design loads. This study investigates the blast response of high-rise reinforced concrete (RC) framed buildings with stiffness irregularities under unconfined surface blasts. A 20-story RC frame with seven bays in each transverse direction is analysed. Irregularities are introduced by varying story heights at three configurations: bottom two stories (HRF-SI1), bottom four stories (HRF-SI2) and top two plus bottom four stories (HRF-SI3).

A regular model without irregularities is also examined for comparison. All models are subjected to surface explosive loads from a 3000 kg TNT charge at standoff distances of 10m, 15m, 20m and 25m (EL1–EL4). Blast parameters are derived from TM-5-1300, considering only the positive phase. Nonlinear time history analysis is conducted using the FNA method in SAP2000, evaluating key response parameters such as displacement, drift, velocity, acceleration and base shear. The results provide insights into how stiffness irregularities affect blast performance, supporting the development of safer and more resilient high-rise structures.

Keywords: High-rise building, Stiffness irregularities, Blast loading, Charge weight, Standoff distance, SAP-2000, Time history analysis.

Introduction

Traditional Indian architecture mostly consists of medium-to-low-rise buildings designed to withstand gravity loads including live and dead loads. In constructing these structures, conventional design approaches are employed, primarily emphasizing the resistance to vertical loads. Lateral loads are only marginally considered. Nevertheless, the construction of high-rise buildings is becoming a daily occurrence due to the advancements in urbanization, construction technology, the need for vertical space and the increase in population. These high-rise buildings are significant in the field of urban planning because they can accommodate residential, commercial and mixed-use developments.

High-rise buildings are considerably more susceptible to lateral dynamic forces than low and mid-rise structures. It is rare to see a high-rise structure that does not possess some structural irregularity. Changes in storey height, material qualities, architectural limitations, or structural layout result in stiffness irregularities. High-rise buildings frequently have stiffness irregularities which increase vulnerability under extreme loading conditions. These loads include seismic, wind and blast loads.

Due to their well-established design methodologies, seismic and wind loads are the most extensively researched lateral loads in structural design. However, the loading conditions are significantly worse when blast loads are present. Blast loads can be caused by either accidental explosions or terrorist attacks that are planned.

Though seismic and wind loads are applied over longer periods and allow energy dissipation, blast loads are nearly instantaneous, leaving the structure little time to respond. The effects of blast loads on buildings can be unpredictable and can have significant structural damage and, in many cases, complete collapse may happen. Also, most traditional ways of designing structures do not specifically consider blast loads. Hence, extreme load conditions like blast loading on high-rise buildings with stiffness irregularities increase lateral displacement, storey drift, velocity, acceleration and base shear distributions. A great number of incidents that have occurred in the past have brought to light the fact that high-rise buildings are susceptible to explosions.

The effects of blast loads on a high-rise building with stiffness irregularities can be significantly more damaging than the effects that a regular building would experience from the same blast loads. As a result, structural engineers are increasingly concentrating their efforts on designing and constructing high-rise buildings with stiffness irregularities to guarantee adequate protection against blast loads.

Review of Literature

This literature review aims to conduct an in-depth analysis of previous research on blast-resistant design, numerical modelling techniques and the impact of geometric irregularities on the performance of structures.

Corley et al[4] provided recommendations for mitigating progressive collapse in new and existing structures, including compartmentalization of structural units, moment-resisting frames and dual systems with special frames. They concluded that seismic detailing improves ductility and recommended jacketing columns, adding walls and adding moment-resisting frames for improved performance.

Woodson and Baylot[27] study investigated the response of structural members like columns, beams and slabs to blast loading in a quarter-scale structure. They used two-storey quarter-scale RC models and numerical simulations to analyse the blast response. The study concluded that slab edge beams, carrying dead weight, prevented column collapse despite severe damage, acting as tensile members and transferring forces to corner columns. Krauthammer and Altenberg[14] have performed research on the assessment of explosion wave's negative phase effects on glass panels. From the research, they concluded that glass panels would exhibit different response at different scaled range and for different charge weights. Alexander and Remennikov[1] study on predicting explosion effects on buildings utilized simplified analytical techniques like Eulerian, Lagrangian, Euler-FCT, ALE and finite element modelling for accurate predictions.

Luccioni et al[16] conducted an analytical study on the failure of a 400kg TNT blast load in a building's entrance hall. They used AUTODYN software to model structural components and analyze the results. The study concluded that simplified assumptions for structure and materials can be made. Luccioni et al[17] studied the impact of mesh size on blast load pressure and impulse distribution in congested urban environments. They used hydro codes and AUTODYN-3D for dynamic analysis, comparing results for different explosive charge positions. The study found that mesh size significantly affects numerical results, with a 10 cm mesh size suitable for wave propagation analysis. They recommended coarser mesh for qualitative responses and a 50 cm mesh size for accurate results. Wu and Hao[28] study examined structure response and damage to surface explosions using ground and air blast pressure. Parametric and numerical studies were conducted, utilizing parameters like explosive weight, standoff distance and structure height. Empirical equations were derived for structural response.

Ngo et al[18] explored the mechanics of blast load and its effects on structures, particularly during vehicle bomb attacks in Oklahoma city. They discussed the SDOF system, blast wave scaling law and blast pressure prediction. The study also highlights the dynamic properties of concrete and steel at high strain rates and the importance of design considerations in public and commercial buildings against extreme events. The study recommends guidelines on abnormal load cases and progressive collapse prevention in current Building Regulations and Design Standards.

Zhu and Lu[30] study explored explosion loads and structural behavior in structures. They found that structures undergo significant plastic deformations and energy absorption, resulting in three types: large inelastic deformation, tear in and transverse shear failure. Mode 1 can transit to mode 2 and 3 with increasing loads. Gebbeken and Doge[7] found that blast wave propagation and reflection depend on the structure's shape and geometry, using AUTODYN software. They found that peak pressure, maximum-impulse, surface area of contact and corners significantly influence column behavior and footing resistance.

Guzas and Earls[8] developed a blast loading process using finite element analysis, incorporating blast parameters from literature and LS-DYNA software to simulate a steel plate and girder response. Wu et al[29] utilized LS-DYNA to study RC square columns, considering parameters like axial load index, residual load index, longitudinal and transverse reinforcement ratios and seismic detailing. Results showed increased transverse reinforcement increases shear resistance, but high shear failure at support end.

Goel et al[9] examined various empirical relations for computing blast load in the form of pressure-time function from air explosions. They recommended using Kinney and Grahm's equations for positive phase parameters, Krauthammer and Altenberg equations for negative phase parameters and Gebbeken and Doge's equation for wave decay parameters. Modified Friedlander's equation was used for pressure-time function computation. Draganic and Sigmund[5] reviewed literature on blast load calculations including parameters, equations, charts and numerical examples. They provided a numerical example of a building subjected to blast load.

Fujikake and Aemlaor[6] conducted a field blast test on RC column damage, analyzing factors like explosive amount, reinforcement ratios and concrete strength. They found that shear reinforcement significantly impacts residual resistance and the confinement effect of core concrete increases with increased reinforcement. Amy and Hojjat[2] study examined the response of three earthquake-designed framing systems to blast loading, using the applied element method for numerical simulation and plastic hinge analysis. The research concluded that braced frames offer greater blast load resistance. Chen et al[3] studied the dynamic response of a pre-stressed reinforced concrete beam under blast loadings using LS-DYNA. They found that pre-stressing increases blast loading resistance capacities and effectively delays the appearance and growth of flexural cracks in concrete. However, increasing pre-stress levels may increase diagonal shear damage near beam supports. Syed et al[23] compared the blast load performance of seismically designed RC buildings with and without considering seismic load. Results showed better performance for seismically designed buildings and larger spans were more vulnerable.

Nourzadeh et al[19] study compared a 10-storey building's response to earthquake and blast loading conditions. They found that blast load caused larger lateral storey drift than earthquake loading, suggesting global design considerations. Vincent et al[25] found that most researchers examined how charge weight, standoff distance, structural orientation, geometric abnormalities and mass irregularities affect blast-loaded RC frames. The authors also suggested blast-resistant structural design principles. Vincent et al[26] found that the ground floor is most vulnerable to collapse for rigid and flexible bases in a G+11-story reinforced concrete framed building against various unconfined surface blast load intensities and SSI effects.

Pavan Kumar et al[20] investigated the performance of symmetric and vertically uneven reinforced concrete space frame structures under seismic and unconfined surface blast stresses. Nonlinear time history analysis was conducted on the selected building models utilising software based on the applied element method. Pavan Kumar et al[21] studied the performance of symmetric and vertical irregular RC buildings under seismic and unconfined surface blast loads, performing nonlinear time history analysis using AEM based software. Krishna et al[15] conducted a nonlinear dynamic analysis on regular framed structures with varying plan aspect ratios to examine the blast reaction. A blast load equivalent to 2500 kg of TNT at a standoff distance of 10 meters was applied to all buildings as a time history function using ETABS.

Research gaps and future directions: While extensive research has been conducted on blast effects, the critical research gap in understanding the impact of stiffness irregularities in high-rise structures with height variations under explosive loads remains underexplored.

Material and Methods

Geometric details: The geometrical details of various structural components of the considered frames are explained in table 2. Figure 1 shows the typical floor plan of all the cases and figures 2 to 5 show the elevations of all the case studies. Though 3D modelling is more realistic, to avoid lengthy and complex tasks, 2D modelling is followed.

Design Loads: All the considered frames were analysed and designed for gravity and lateral loads as per relevant Indian standard codes of practice [IS:456 – 2000][10], [IS:875 – 1987(Part 1 and 2)][11,12], [IS:1893 – 2016][13]. Tables 3, 4 and 5 present the wall loads, the dead load acting on the slab and the live load acting on the slab respectively.

Explosive Loads: For the present study, the blast response of the models is calculated under unconfined surface explosive loads as shown in table 6. Blast wave parameters are calculated in accordance with the technical manual TM-5-1300[24], considering only the positive phase of the blast wave.

Verification of Case Study on Triangular Impulsive Loads: To verify the response of the RC portal frame against impulsive loads obtained from the computer software package of SAP2000, the manual method suggested by Ray and Penzien[22] is used. One bay one storey framed structure against various triangular impulsive loads (Refer to Figure 6 and Table 7) is considered to verify the results obtained from the time history analysis by SAP2000.

Details of Portal Frame for Verification

Material Properties of the Model:

Grade of Concrete = M20

Grade of Steel = Fe500

Elastic Modulus of Concrete, $E_c = 5000\sqrt{fck} = 22360$ MPa or 22.36×10^6 kPa.

Geometric Details of the Model:

Bay Span = l = 4.75 m

Height of Storey = h = 3.5 m

Size of beam = 0.25 m × 0.25 m

Size of Column = 0.25 m × 0.25 m

$I_b = \frac{bd^3}{12} = 3.25 \times 10^{-4}$ m^4; $I_c = \frac{bd^3}{12} = 3.25 \times 10^{-4}$ m^4;

Mass, m = 5000 X 4.75 = 23750 kg

(Self-weight of the structure is ignored)

Verification: Lateral stiffness of one-bay and one-storied frame with rigid supports $k = \dfrac{\frac{24EI_c^2}{h^4} + \frac{144EI_cI_b}{h^3 l}}{\frac{4I_c}{h} + \frac{6I_b}{l}} = 2617.59$ kN/m

Natural Period $T_N = 2\pi\sqrt{\frac{m}{k}} = 2\pi\sqrt{\frac{23750}{2617590}} = 0.598$ Sec

The ratio of impulse period to natural periods is as follows:

$\frac{t}{T_N} = \frac{0.15}{0.598} = 0.251$ for impulsive Load 1 and 2

$\frac{t}{T_N} = \frac{0.25}{0.598} = 0.418$ for impulsive Load 3 and 4

The maximum response ratio is observed in figure 7:

R_{max} = 1.15 for IL – 1 and 2

R_{max} = 1.63 for IL – 3 and 4

$$\delta_{max} = R_{max}\left(\frac{p_o}{k}\right)$$
$$= 1.15\left(\frac{300}{2617.59}\right) = 0.132 \text{ m for IL – 1}$$
$$= 1.15\left(\frac{600}{2617.59}\right) = 0.264 \text{ m for IL – 2}$$
$$= 1.63\left(\frac{300}{2617.59}\right) = 0.187 \text{ m for IL – 3}$$
$$= 1.63\left(\frac{600}{2617.59}\right) = 0.374 \text{ m for IL – 4}$$

Table 8 compares analysis results against triangular impulsive loads between manual and SAP2000 analysis. The response of a one-bay, one-story SDOF structure subjected to various impulsive load situations, as determined by hand calculations suggested by Ray and Penzien[22], is in satisfactory agreement with the results of the SAP2000.

Table 1
Details of Case Studies on High-Rise RC Frames with Stiffness Irregularities

S.N.	Notation	Description of Case Study	Frame Dimensions (L × B × H)	No. of Stories	Irregular Stories (Usage)	Typical/Irregular Storey Heights	Type of Stiffness Irregularity	Percentage of Irregularity
1	HRF	High-rise RC frame without stiffness irregularity	42m × 42m × 60m	20	None	All stories: 3.0m	None	0%
2	HRF-SI1	**Type-I:** Bottom 2 stories used for parking	42m × 42m × 62.8m	20	Stories 1–2 (Parking)	Stories 1–2: 4.4m, Others: 3.0m	Reduced stiffness at base	10%
3	HRF-SI2	**Type-II:** Bottom 2 stories (Parking), next 2 (Commercial)	42m × 42m × 64m	20	Stories 1–2 (Parking), 3–4 (Commercial)	Stories 1–2: 4.4m, 3–4: 3.6m, Others: 3.0m	Gradual stiffness transition	20%
4	HRF-SI3	**Type-III:** Bottom 2 (Parking), next 2 (Commercial), top 2 (Duplex flats)	42m × 42m × 65.2m	20	Stories 1–2 (Parking), 3–4 (Commercial), 19–20 (Duplex)	Stories 1–2: 4.4m, 3–4 & 19–20: 3.6m, Others: 3.0m	Stiffness variation at top and bottom	30%

Table 2
Geometric details of the frame

S.N.	Parameter	Dimension
1	Size of Bay	6 m × 6 m
2	Number of Bays	7 Bays × 7 Bays
3	Storey Height	3 m, 3.6 m and 4.4 m
4	No of Stories, n	20 Stories
5	Depth of Foundation	2.1 m
6	Size of Beam	300 mm × 600 mm
7	Size of Column	750 mm × 750 mm
8	Thickness of Slab	200 mm

Table 3
Wall loads

S.N.	Wall Thickness (m)	Wall Height (m)	Wall Load (kN/m)
1	0.23	$h_w = 2.4$	0.23×2.4×20 = 11
2	0.23	$h_w = 3.0$	0.23×3.0×20 = 13.8

Table 4
Dead Load Acting on Slab

S.N.	Description	Slab Load (kN/m^2)	Remarks
1	Self-weight	0.2 x 25 = 5	ρ_{rcc} = 25 kN/m^3
2	Unknown force	1	Assumed
3	Floor finish	1	Assumed
Total: 7 kN/m^2			

Table 5
Live Load Acting on Slab

Floor Level	LL (kN/m2)			
	HRF	**HRF-SI1**	**HRF-SI2**	**HRF-SI3**
Ground and 1st Floor	2	5	5	5
2nd and 3rd Floors	2	2	3	3
Remaining Floors	2	2	2	2
Terrace	2	2	2	10

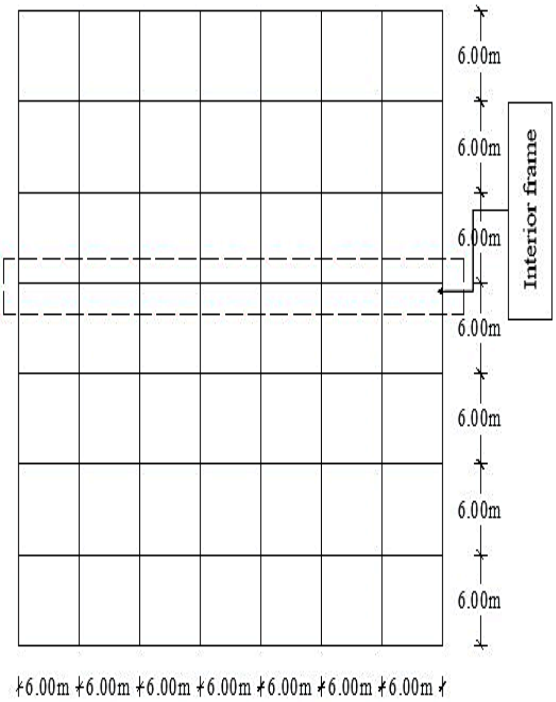

Fig. 1: Typical Floor Plan for HRF, HRF – SI1/SI2/SI3

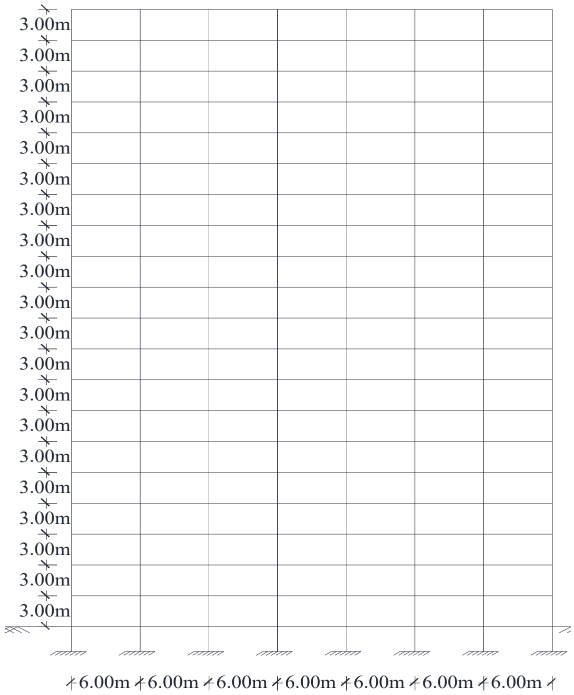

Fig. 2: Elevation of HRF

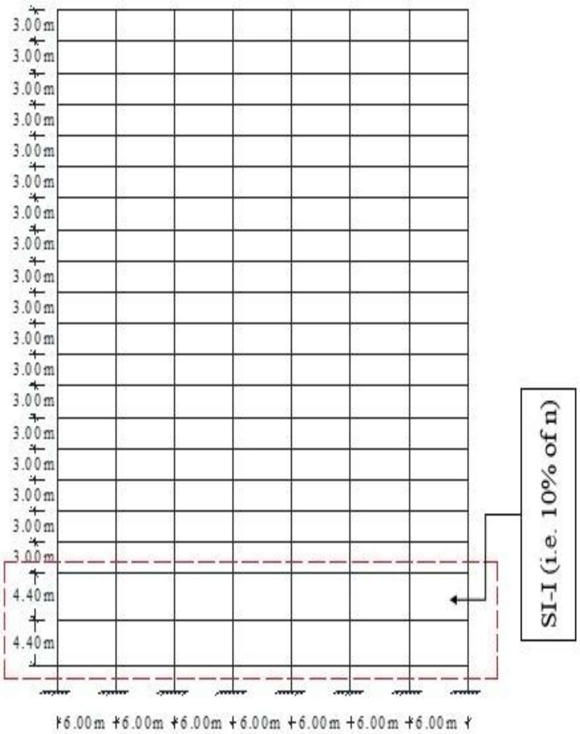

Fig. 3: Elevation of HRF-SI1

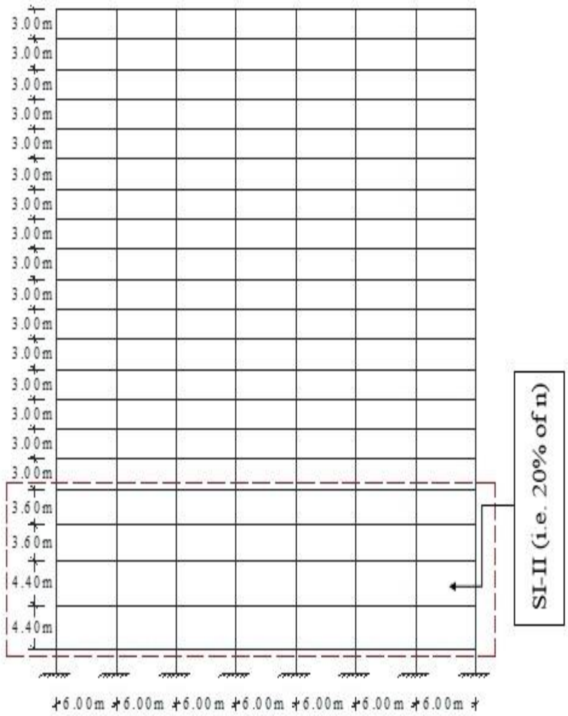

Fig. 4: Elevation of HRF-SI2

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Table 6
Explosive Load Cases

S.N.	Explosive Load	Charge Weight (TNT)	Standoff Distance (m)
1	EL1	3000 kg	10
2	EL2	3000 kg	15
3	EL3	3000 kg	20
4	EL4	3000 kg	25

Table 7
Triangular Impulsive Load Cases Considered for Verification

Load Case	F (kN)	t (Sec)	t/T_N
IL- 1	300	0.15	0.251
IL- 2	600	0.15	0.251
IL- 3	300	0.25	0.418
IL- 4	600	0.25	0.418

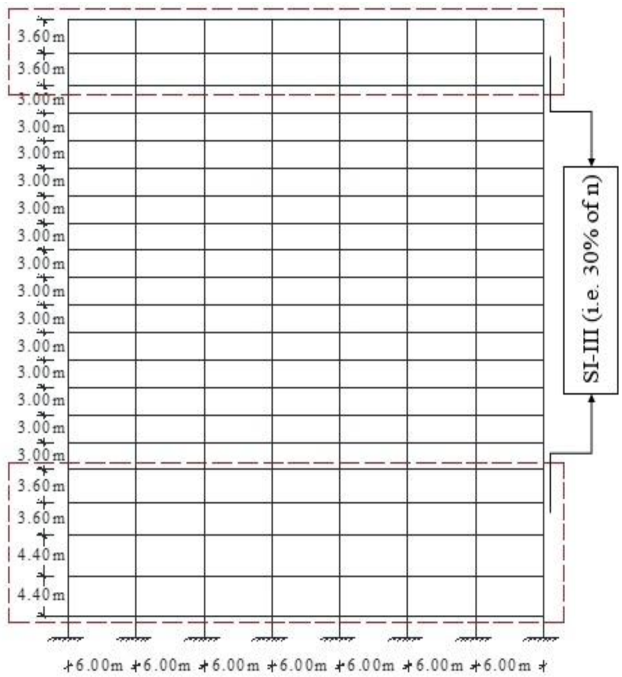


Fig. 5: Elevation of HRF-SI3

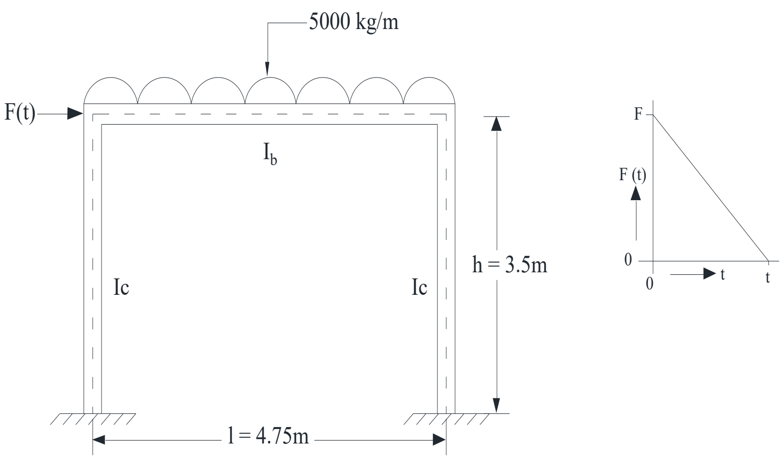


Fig. 6: Single Degree of Freedom Structure subjected to Triangular Impulsive load

Table 8
Results

S.N.	Parameter	SAP2000	Manual	% difference
1	T_N (Sec)	0.599	0.598	+ 0.17%
2	δ_{max} (mm) for IL – 1	130.11	132	- 1.43%
3	δ_{max} (mm) for IL – 2	262.28	264	- 1.83%
4	δ_{max} (mm) for IL – 3	186.11	187	- 0.65%
5	δ_{max} (mm) for IL – 4	372.59	374	- 0.37%

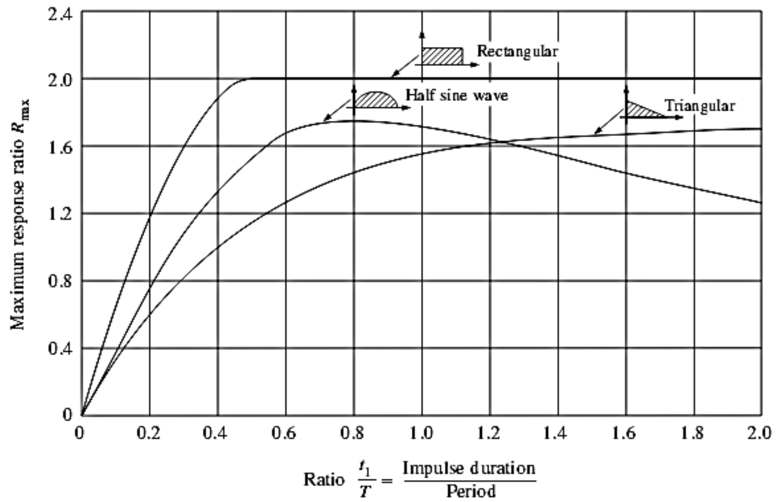


Fig. 7: Displacement – response spectra for three types of impulse[22]

Results and Discussion

Lateral Displacement: Figures 8 to 11 show the explosive displacement response at the roof level of a High-Rise RC framed structure with three different stiffness irregularities compared to a regular structure against different explosive loads EL1, EL2, EL3 and EL4, respectively. The following are the key insights:

High-rise framed structures with stiffness irregularities show higher peak roof lateral displacements than those without irregularities against all the considered blast loads, representing higher flexibility and lower stiffness. From figures 8 to 11, it is observed that increased roof displacement ranging from 5% to 15%, 5% to 8% and 11% to 15% was observed in HRF – SI1, SI2 and SI3 respectively when compared to HRF under varying blast intensities, from lower to higher.

Figure 12 compares the peak roof lateral displacement of all the considered high-rise building models against all explosive load cases. The peak roof displacement reduces as the blast load case changes from EL1 to EL4, indicating that the increase in standoff distance reduces the lateral response. HRF shows a considerable reduction in the lateral displacement for EL3 and EL4 by 5.55% and 11.8% respectively when compared to EL1. However, there has been no considerable change in the lateral displacement of EL2. HRF-SI1 shows a considerable decrease in roof displacement by 5.53%, 12.81% and 17.46% for EL2, EL3 and EL4 respectively compared to EL1. HRF-SI2 shows a considerable reduction in the lateral displacement for EL3 and EL4 by 7.51% and 12.68% respectively when compared to EL1. However, there has been no considerable change in the lateral displacement of EL2. HRF-SI3 shows a considerable reduction in the lateral displacement for EL3 and EL4 by 8.11% and 13.65% respectively when compared to EL1.

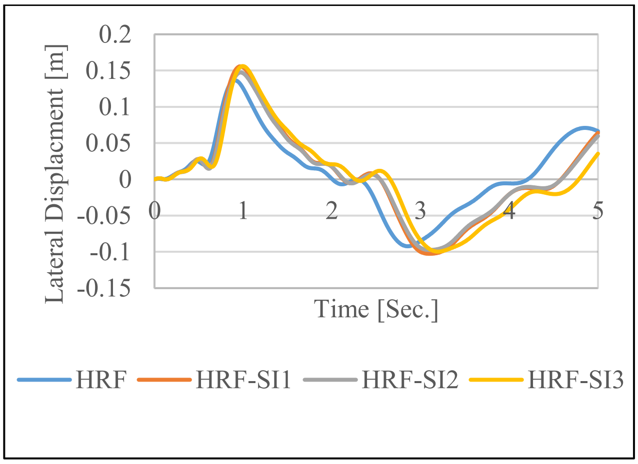


Fig. 8: Roof Displacement of HRF with Different Stiffness Irregularities for EL1

Figures 13 to 16 show the explosive storey drift response of a High-Rise RC framed structure with three different stiffness irregularities compared with a regular structure against different explosive loads EL1, EL2, EL3 and EL4 respectively. The highest storey drift is observed in HRF-SI3, particularly in the upper levels, which suggests that the flexibility is increased by the combination of top and bottom stiffness irregularities. HRF-SI1 and HRF-SI2 showed increased storey drift compared to the regular frame, although to a lesser extent than HRF-SI3.

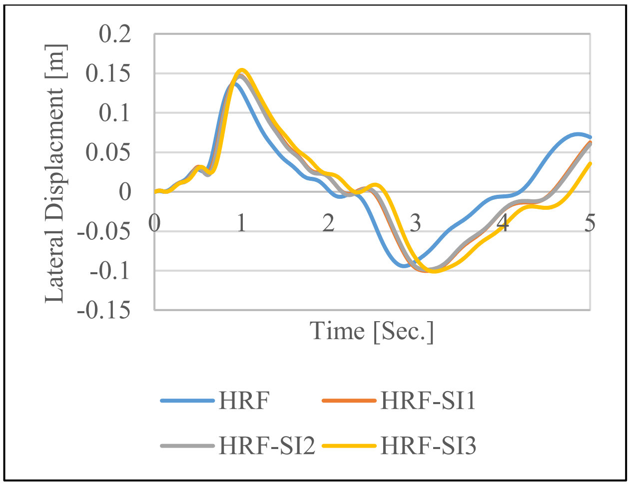


Fig. 9: Roof Displacement of HRF with Different Stiffness Irregularities for EL2

From figures 13 to 16, it is observed that increased storey drift ranging from 49% to 78%, 52% to 59% and 54% to 76% was observed in HRF – SI1, SI2 and SI3 respectively when compared to HRF under varying blast intensities from lower to higher. Figure 17 compares the peak storey drift of all the considered high-rise building models against all blast load cases. The results suggest that stiffness irregularities significantly increase peak roof storey drift during blast loading, with the most severe impacts observed in EL1 and EL2. HRF, it is observed that there is a considerable decrease in the storey drift by 7.09%, 24.14% and 38.28% for EL2, EL3 and EL4 respectively, when compared to EL1. HRF-SI1 shows a considerable decrease in the storey drift by 16.24%, 35.95% and 48.03% for EL2, EL3 and EL4 respectively compared to EL1.

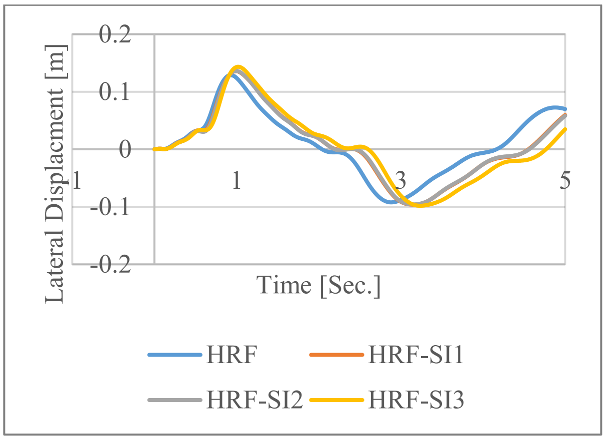


Fig. 10: Roof Displacement of HRF with Different Stiffness Irregularities for EL3

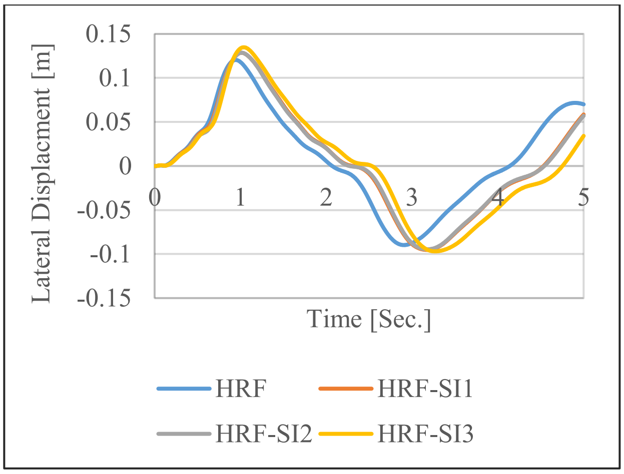
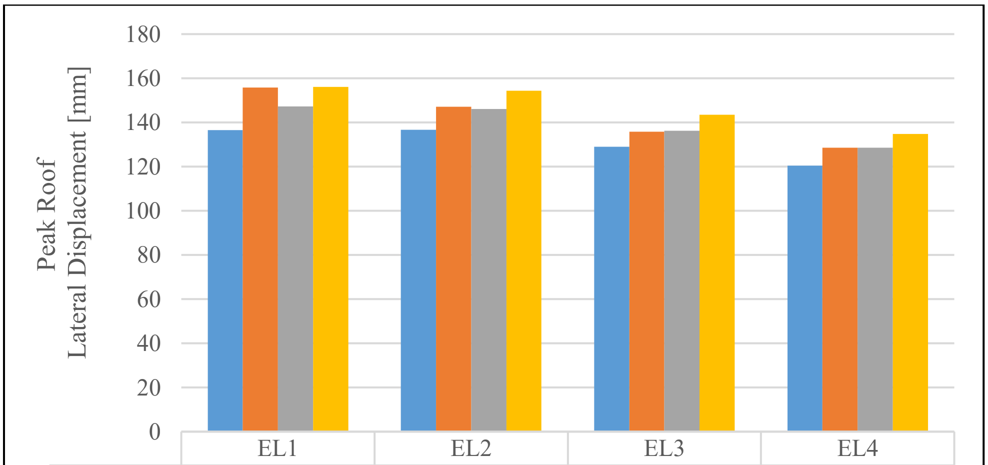


Fig. 11: Roof Displacement of HRF with Different Stiffness Irregularities for EL4



	EL1	EL2	EL3	EL4
HRF	136.51	136.74	128.93	120.4
HRF - SI1	155.76	147.14	135.82	128.56
HRF - SI2	147.31	146.09	136.24	128.62
HRF - SI3	156.09	154.39	143.43	134.77

Fig. 12: Comparison of Peak Roof Lateral Displacement for all High-Rise Building Models under various Explosive Load Cases

Storey Drift

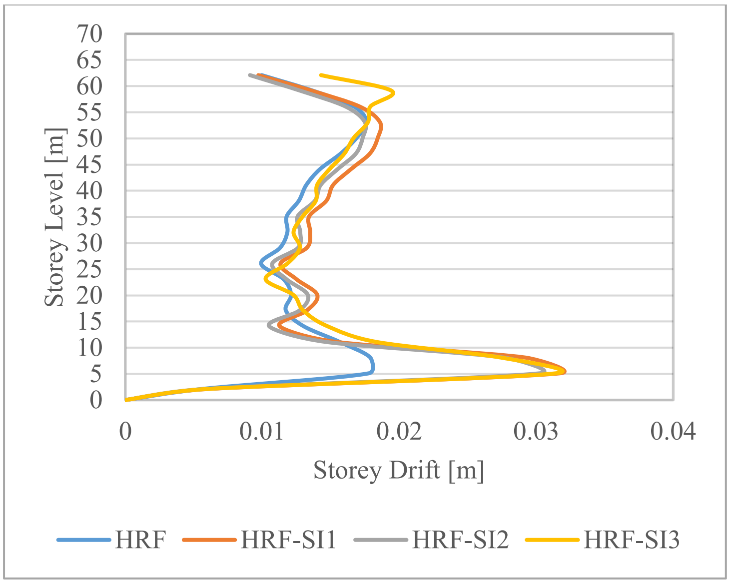


Fig. 13: Storey Drift of HRF with Different Stiffness Irregularities for EL1

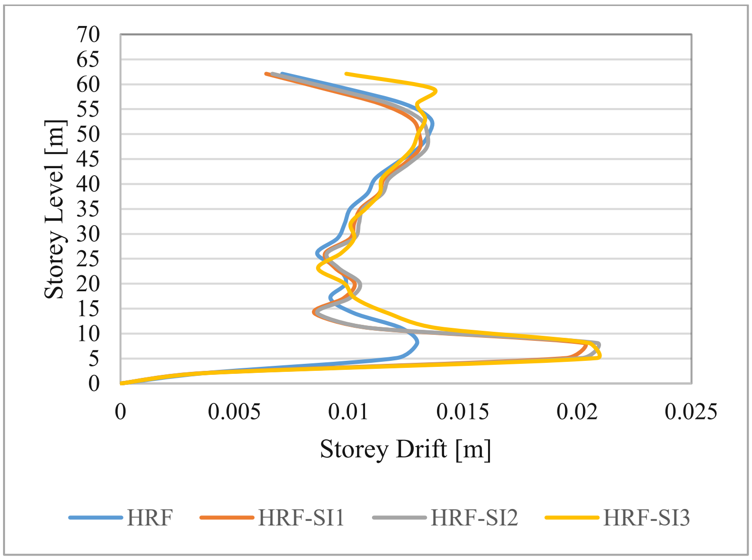


Fig. 14: Storey Drift of HRF with Different Stiffness Irregularities for EL2

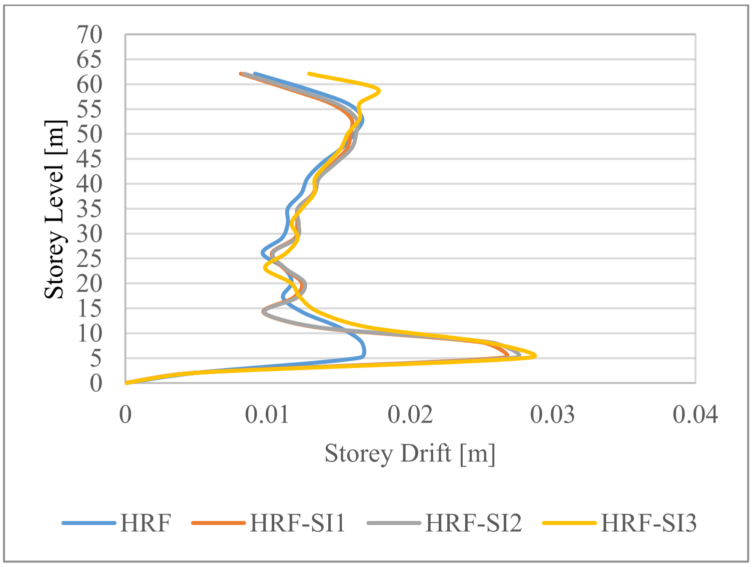


Fig. 15: Storey Drift of HRF with Different Stiffness Irregularities for EL3

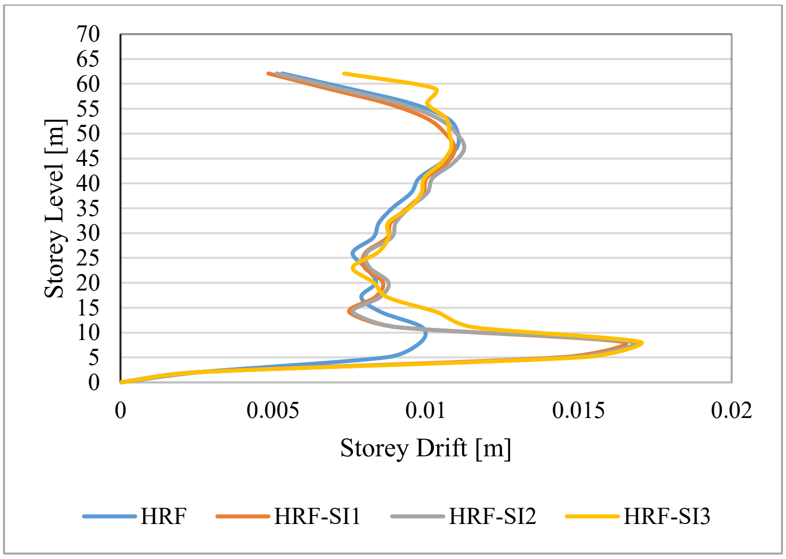
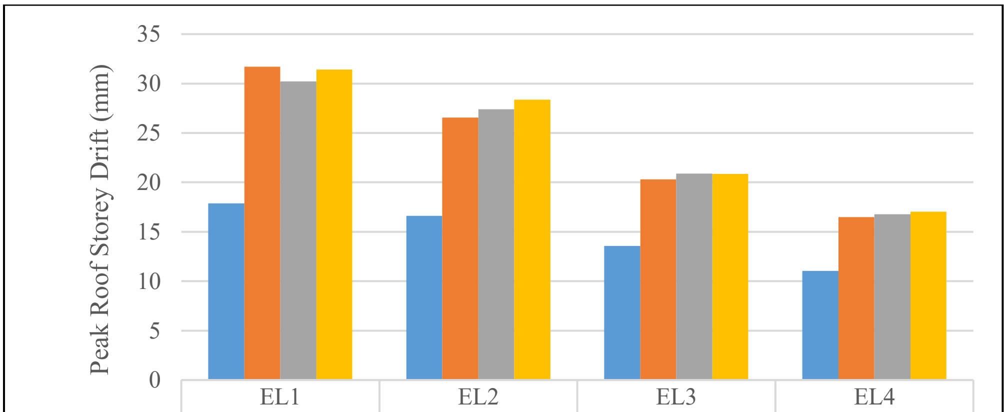


Fig. 16: Storey Drift of HRF with Different Stiffness Irregularities for EL4



	EL1	EL2	EL3	EL4
HRF	17.89	16.62	13.57	11.04
HRF - SI1	31.71	26.56	20.31	16.48
HRF - SI2	30.23	27.39	20.88	16.78
HRF - SI3	31.42	28.36	20.86	17.03

Fig. 17: Comparison of Peak Roof Storey Drift for all High-Rise Building models under various Explosive Load Cases

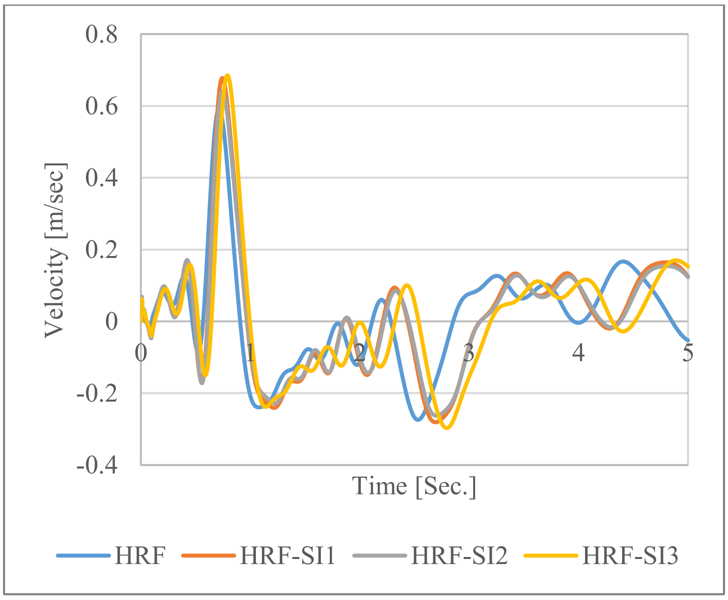


Fig. 18: Roof Velocity of HRF with Different Stiffness Irregularities for EL1

HRF-I2 shows a considerable decrease in the storey drift by 9.39%, 30.93% and 44.49% for EL2, EL3 and EL4 respectively, compared to EL1. HRF-SI3 shows a considerable decrease in the storey drift by 9.73%, 33.61% and 45.79% for EL2, EL3 and EL4 respectively compared to EL1.

Roof Velocity: Figures 18 to 21 show the roof velocity response of a High-Rise RC framed structure with three different stiffness irregularities compared to a regular structure against different explosive loads EL1, EL2, EL3 and EL4, respectively. The time-history response of roof velocity for High-Rise framed buildings under different boundary conditions (EL1, EL2, EL3 and EL4) indicates the impact of stiffness irregularities on structural dynamics. The results demonstrate a significant increase in roof velocity for irregular buildings (HRF-SI1, HRF-SI2 and HRF-SI3) compared with the standard high-rise frame (HRF).

From figures 18 to 21, it is observed that increased roof velocity ranging from 3% to 14%, 5% to 9% and 11% to 16% was observed in HRF – SI1, SI2 and SI3 respectively when compared to HRF under varying blast intensities, from lower to higher.

Roof Acceleration: Figures 22 to 25 show the roof acceleration response of a High-Rise RC framed structure with three different stiffness irregularities compared to a regular structure against different explosive loads EL1, EL2, EL3 and EL4 respectively. The following are the key insights:

The time-history charts show that the acceleration response has abrupt initial peaks, followed by oscillatory decay. The presence of stiffness irregularities increases the duration of high-intensity oscillations, showing that irregular structures are more vulnerable to dynamic amplification induced by blast loading. From figures 22 to 25, it is observed that increased roof acceleration ranging from 8% to 30%, 7% to 29% and 6% to 19% was observed in HRF – SI1, SI2 and SI3 respectively when compared to HRF under varying blast intensities from lower to higher.

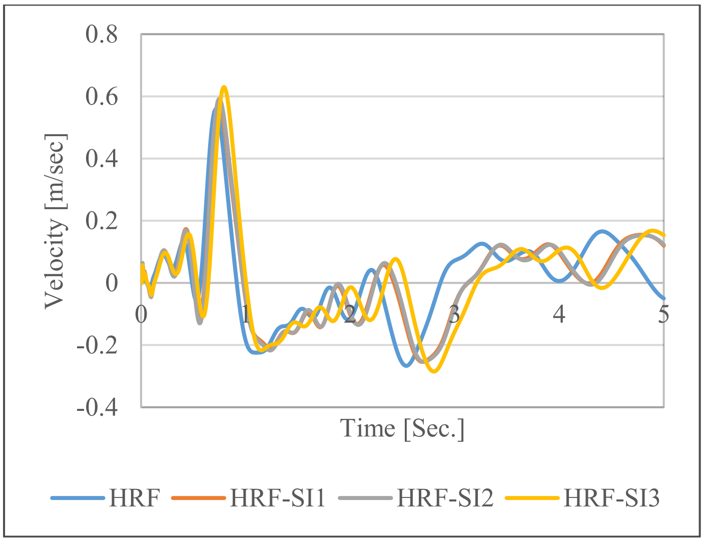


Fig. 19: Roof Velocity of HRF with Different Stiffness Irregularities for EL2

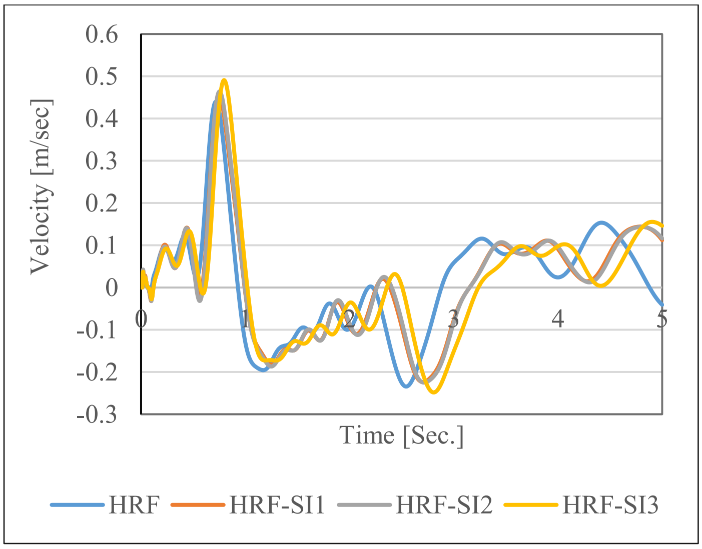


Fig. 20: Roof Velocity of HRF with Different Stiffness Irregularities for EL3

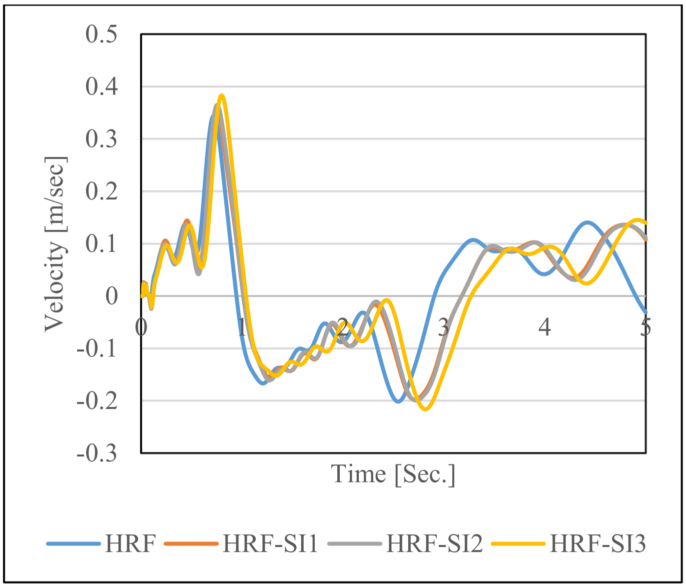


Fig. 21: Roof Velocity of HRF with Different Stiffness Irregularities for EL4

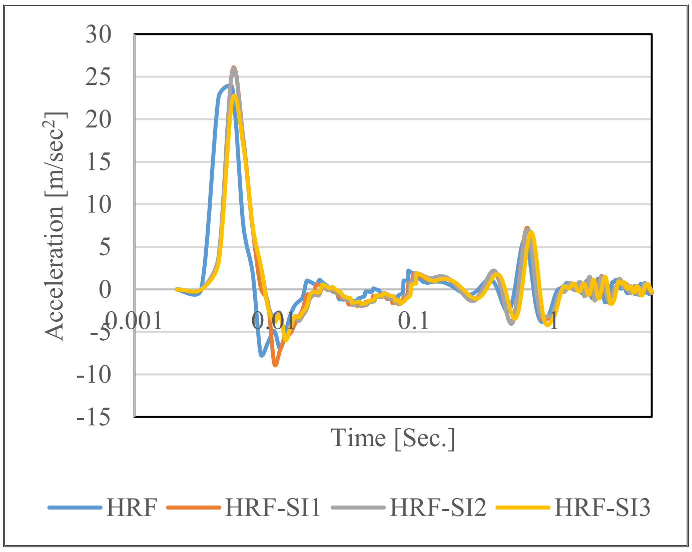


Fig. 22: Roof Acceleration of HRF with Different Stiffness Irregularities for EL1

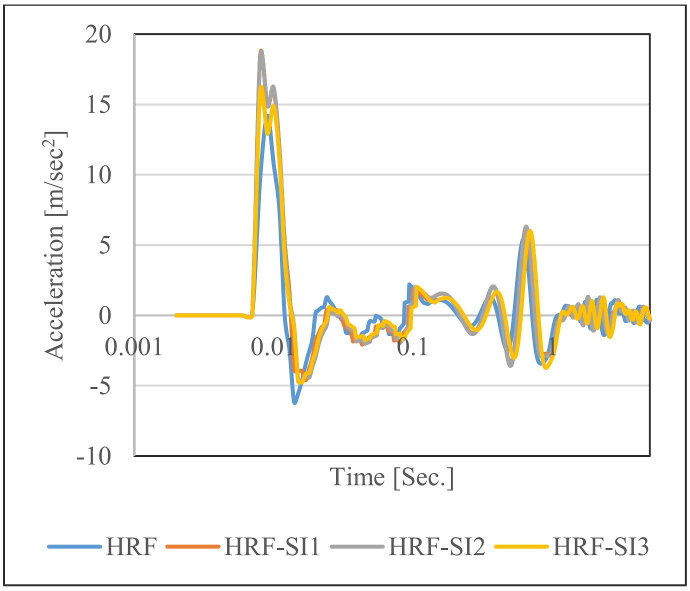


Fig. 23: Roof Acceleration of HRF with Different Stiffness Irregularities for EL2

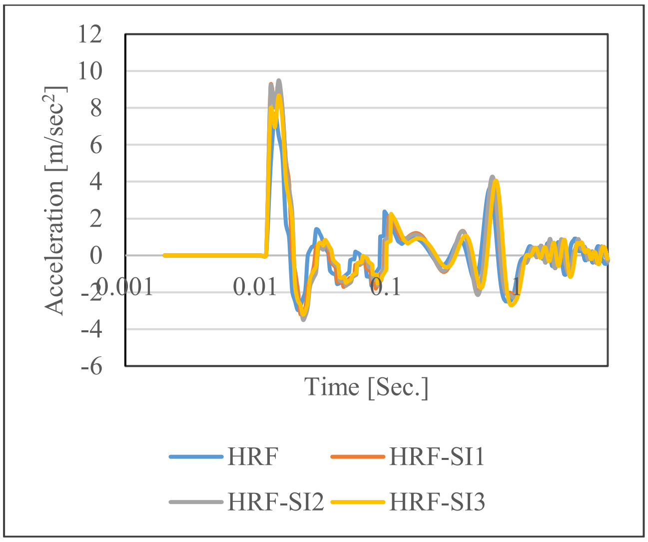


Fig. 24: Roof Acceleration of HRF with Different Stiffness Irregularities for EL3

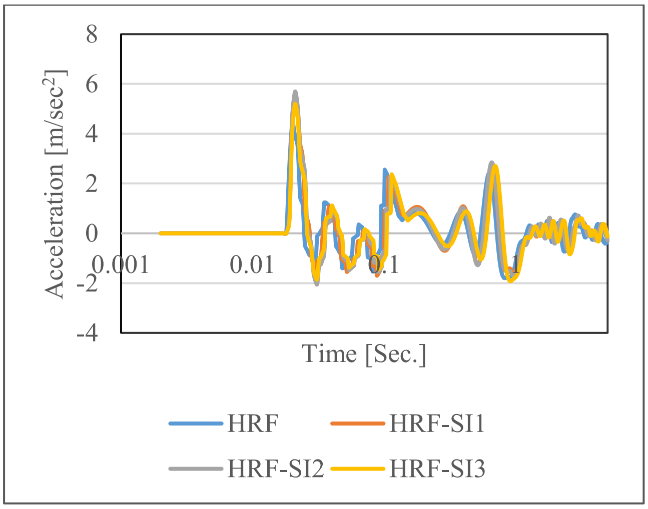


Fig. 25: Roof Acceleration of HRF with Different Stiffness Irregularities for EL4

Conclusion

The major conclusions drawn from the study of the dynamic response of stiffness-irregular RC frames subjected to unconfined surface explosive loads are as follows:

1) Stiffness irregularities significantly increase blast-induced responses such as lateral displacement, storey drift, velocity and acceleration.
2) HRF-SI3 (combined top and bottom irregularities) exhibits the highest overall response, indicating that it is the most vulnerable configuration under blast loading.
3) Roof displacement increased by 5–15% in irregular models compared to the regular frame under varying blast intensities.
4) Storey drift showed a sharp rise, especially in upper storeys, with increases ranging from 49% to 78% in irregular configurations.
5) Roof velocity and acceleration responses were also elevated in irregular models, with peak increases up to 16% (velocity) and 30% (acceleration).
6) Increasing the blast standoff distance (from EL1 to EL4) effectively reduced all response parameters across all models.
7) Regular high-rise frames (HRF) consistently performed better than irregular ones, making them more suitable for blast-resistant design.
8) Although not part of this study, blast loads are fundamentally different from seismic loads in their impulsive and short-duration nature leading to more immediate and critical structural responses, especially in the presence of stiffness irregularities.

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