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PROBABILISTIC SEISMIC ANALYSIS OF TWO INTERCONNECTED IRREGULAR BUILDINGS

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ABSTRACT

The paper carries out a probabilistic seismic performance assessment of two interconnected twelve-story reinforced concrete institutional buildings located in a moderate seismic zone of Bangladesh. The investigation comprehensively covers the complexity of the seismic behavior and the susceptibility of the interconnected structures. The study uses Performance-Based Earthquake Engineering (PBEE) concepts with the aid of time history analysis in the ETABS v.22 software to the review of the key engineering demand parameters with the main focus on interstory drift. To grasp the oscillatory nature of the system, a wind load analysis was carried out on the interconnected structures. The outcome of the investigation revealed that the modular layout caused the system to suffer a significant torsional irregularity due to the finding that the torques arising from wind forces have the potential to cause the biggest difference between the center of mass and center of rigidity. The discovery, which pointed at the divergence between the center of mass and the center of rigidity, thus confirmed the absolute need of a comprehensive 3D analytical model for capturing the complicated coupled seismic response.

The extent of seismic vulnerability was reflected in the numerical depiction of fragility curves that followed from incremental dynamic analysis, which made use of a set of selected ground motions. To the incremental dynamic analysis, ten ground motion records from the PEER database were selected and scaled at various intensity levels (0.2g to 2.0g with increment 0.2g) to evaluate the structural response. The fragility study found the medians for Slight Damage (DS1) to be 0.124g, Life Safe (DS2) at 0.250g, Severe Damage (DS3) at 0.650g, and Collapse (DS4) at 1.050g. The results shows that the structure is exposed to minor serviceability damages at low PGA levels but on the other hand, it has high seismic resilience. The notable distance between the Life Safe and Collapse states, with the collapse capacity being more than four times the Life Safe capacity, is an indication of a substantial nonlinear reserve capacity and ductility. This study thus concludes that though the structural interconnection introduced complex torsional behavior, the system is robust and satisfies both "Life Safe" and "Collapse Prevention" performance objectives

Keywords: *Probabilistic seismic analysis, fragility curve, interconnected buildings, incremental dynamic analysis, interstory drift*

1. INTRODUCTION

Urban development has been compact and led to the increased number of interconnected structures that are frequently connected to each other through common structural and functional features. The seismic security of the buildings is very essential since the interaction between the surrounding buildings would either worsen or reduce in the event of an earthquake taking place (Sharma & Sain, 2024). The seismic strength of the interlocking structures is in a complicated interdependence of the entities such as the development of inter-building links, soil properties underneath the earth, and stress caused by the earthquakes. These hazards are even aggravated in the areas that are vulnerable to seismic activity like Bangladesh, which lie in the boundary of the Eurasian and Indian plates, and create tremendous demands on structural safety. The Bangladesh National Building Code (BNBC, 2021) categorizes the nation into four distinct seismic zones, extending from low risk (Zone 1) to immensely high-risk (Zone 4) regions. Among these, Sylhet and Chattogram are situated within the most seismically vulnerable zone, indicating the highest level of earthquake hazard. The Probabilistic Seismic Analysis (PSA) is a crucial structure for testing the natural differences in the structural performances among the range of seismic intensities. Unlike deterministic methods, PSA takes into consideration the natural variability of ground motion, constitutive behavior, and structural behavior; this process lowers the chances of jointly estimating the likelihood of the damages (Aslani & Miranda, 2005). The fundamentals of Performance-Based Earthquake Engineering (PBEE) are used in this analysis to quantify structural performance by way of fragility and reliability. The modelling and simulation tools, such as ETABS, enable modelling of nonlinear, time-history and modal analysis to enable engineers to model realistic seismic effects and increase the overall safety of the structural design (Hussain et al., 2016). The author is analyzing, in this paper, two, interconnected twelve storey reinforced concrete (RC) educational buildings, which are designed in accordance with the requirements of both the BNBC (2020) and ASCE 7-05. It looks at the critical parameters, relative story drift and lateral deflection - the functions of fragility are employed in measuring the structural solidness in the various seismic areas. The findings are aimed at providing information and enhancing the resilience planning of infrastructure development in the regions where the buildings contact is unavoidable.ETABS, SAP2000 and STAAD pro are a few examples of high-performance computational tools that have transformed the science of seismic analysis due to their ability to enable an engineer to conduct complex dynamic, linear and nonlinear analyses of intricate structural systems (Fajfar, 2018). The probabilistic approaches, in particular PBEE, have received wide acceptance based on the ability to consider randomness of seismic demand and structural resistance (Aslani & Miranda, 2005). In the PBEE framework, fragility curves are very important since they show the likelihood of achieving certain damage states depending on seismic intensity measures (IMs) such as Peak Ground Acceleration (PGA) and spectral acceleration (Sa).Structural connectivity can change the natural frequencies dramatically and, consequently, lead to unpredictable stressing actions and displacement even highlighted by Alwaeli et al. (2017). Linear Time History Analysis (LTHA) has since been reported as a powerful method of analysis to quantify the parameters of fragility with specific reference to the design phase especially quasi-elastic conditions (Lombardi & De Luca, 2020). The location of Pabna is in the central-western region of Bangladesh, which has been identified as seismic Zone 2 with a zone index=0.20, which means moderate seismic exposure. The history of seismic activities documents that Patna has experienced tremors caused by earthquakes within the region such as the Bengal Earthquake (M 7.5) in 1885 whose central distances were near enough to put the structural connectivity of the buildings in the area at risk (Al-Hussaini et al., 2015). The current study is informed by these premises as it uses LTHA on two combined 12-storey academic structures in Pabna. The study is expected to quantify seismic demand, determine displacement control, which is a result of structural integration, and work out fragility curves that will comply with Zone 2 standards by including such seismic parameters as BNBC-2020 and area-specific soil characteristics and stochastic modelling. The research improves the insight into the larger discussion of seismic stability in South Asian mid-rise urban infrastructure and gives viable suggestions to the engineers in the moderately risky fields.

2. METHODOLOGY

In this research, a probabilistic seismic analysis model was used to assess the behavior of two interconnected reinforced concrete (RC) structures when they were earthquake- loaded. The analysis incorporates BNBC 2020 requirements with ASCE 7-05 seismic and wind load design requirements. The education buildings that were treated as the structure bridge between the two interconnected 12-story buildings were simulated with ETABS v22, which allowed the generation of linear dynamic simulations and fragility-based risk assessments.

The following the parameter are used for modeling:

- Details of structure and material Properties.
- Dimensions of structural design.
- Developing fragility curves for probabilistic damage prediction.

2.1 Building Details

2.2.1 Geometry

This structure is a 12-story high-rise building. The full AutoCAD layout plan of the building has been shown in **Error! Reference source not found.** Two multi-story buildings (both are12-story buildings) are placed side by side.

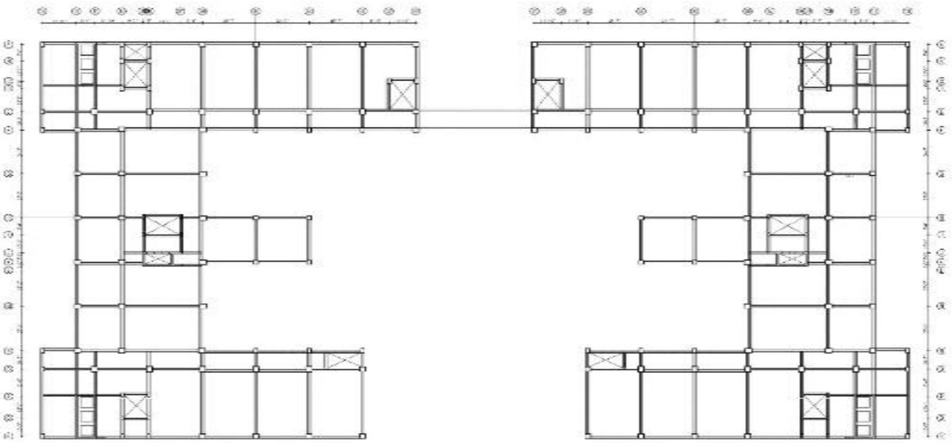


Figure 1: Layout Plan of Structure in AUTOCAD.

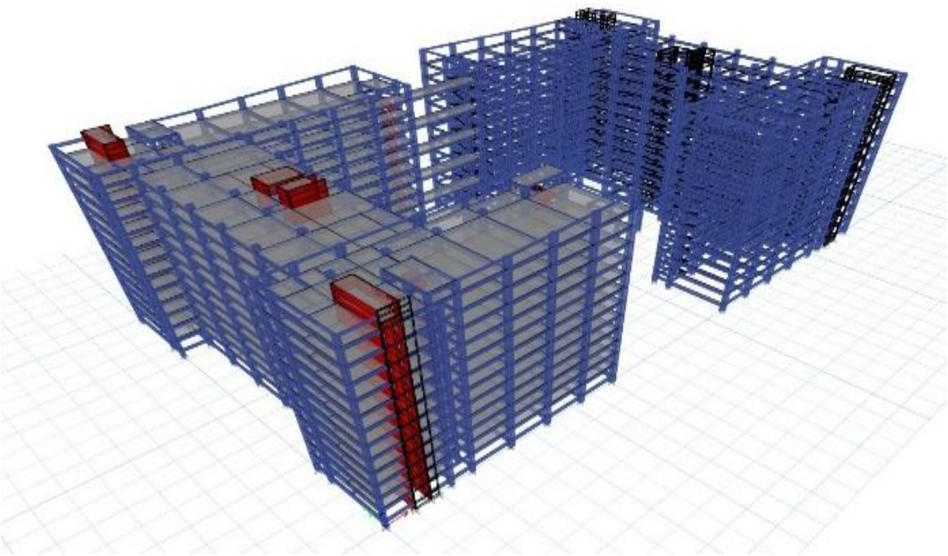


Figure 2: Numerical Model of the Structure

Figure 2 illustrates the numerical model of two interconnected buildings, which has 12 stories and each story 3.65m high. In the modeling all vertical elements are fixed at the bottom level of the structure

2.1.1 Design Parameters

Details of the RC building structure and material properties are shown in **Error! Reference source not found.**. The building G+11 is classified as an educational (Occupancy B) structure with a different dimension plan that is 115.92 meters in the X direction and 67.53 meters in the Y direction, with a bottom floor height of 3.65 meters. These buildings are utilized for habitation in the B category and have the same bottom story height and layout.There are numerous types of beams and columns, with floor-to-column distances ranging from grade beams and rests to more.

Table 1: Details of material properties and RC building structure

Serial No	Parameter	Value
1	Number of stories	GF+11
2	Typical story height	3.6576 m
3	Height of building	49.3776 m
4	Grade of concrete	M35
5	Grade of steel	500W
6	Length of X-direction	115.9254 m
7	Length of Y-direction	67.5386 m
8	Bottom Story height	3.6576 m
9	Building type	Educational (Occupancy B)

The beams, columns, slab of stair and shear walls are of the required dimensions as per the set design. Different beam sizes and column sizes were used to suit various load conditions in both longitudinal and transversal directions, with Table 2 summarizing the same.

1) In this case, it is evident that B7 (304.8 x 304.8 mm) is the smallest beam size which means that the beam is shallow. Nonetheless, the largest beam size, B6 (381 x 914.4 mm) refers to a deep beam. 254 x 508 mm supply-equipped secondary beam.

2) In order to make sure that the verticals are strong enough to carry the weight, we varied the sizes of the columns in this structure. The column sizes include smaller cross-section columns such as C6 (304.8 x 304.8 mm) and bigger columns such as C5 (914.4 x 914.4 mm).

Table 2: Dimension of structural members

Serial No	Parameter	Value
1	Beam Size	B1- 304.8mm X 609.6mm B2- 304.8mm X 711.2mm B3- 304.8mm X 762mm B4- 304.8mm X 914.4mm B5- 381mm X 762mm B6- 381mm X 914.4mm B7- 304.8mm X 304.8mm SB2- 254mm X 508mm
2	Column Size	C1- 762mm X 762mm C2- 762mm X 914.4mm C3- 508mm X 762mm C4- 508mm X 914.4mm C5- 914.4mm X 914.4mm C6- 304.8mm X 304.8 mm
3	Slab of thickness	127 mm
4	Thickness of stair slab	203.2 mm
5	Thickness of shear wall	254 mm 304.8 mm

2.2 Development of Fragility Curve

The methodology for seismic fragility curve development in this study encompasses several key stages: from seismic input selection to statistical processing.

2.2.1 Analytical methodology

The following sequential steps carry out the development process:

- Ground motion selection: A representative suite of ground motion records is selected that models the wide variations in seismic conditions. They are in a suite taking into account a diversity of seismological characteristics, such as magnitude, source-to-site distance, and local geotechnical settings that comprehensively capture the possible seismic hazards (Eren et al., 2021). In this study 10 ground motion was selected from PEER strong ground motion database (Ancheta et al., 2013).
- Selection of intensity measure: A proper IM (e.g., PGA, Peak Ground Acceleration) is selected for measuring the intensity of the seismic input. The IM is generally used as the independent variable when assessing the structural performance.
- Structural modeling and analysis: A high-fidelity numerical model of the structure is created with the use of finite element software such as ETABS. Incremental Dynamic Analysis (IDA) is applied for establishing the probabilistic relationship between the IM and the structural response. This computationally intensive procedure involves subjecting the structural model to the selected ground motion suite, each record being scaled from 0.2g to 2.0g at 0.2g increment. This way, the whole spectrum of structural behavior is embraced-from the elastic to yielding and up to final collapse (Lombardi & De Luca, 2020). The response is tracked by key Engineering Demand Parameters (EDP) such as inter-story drift ratio. This study was utilized the inter-story drift ratio as damage parameters for the structural performance assessment.
- Fragility function fitting: IDA results, pairs of IM and corresponding EDP, are collected and processed. The data is used to define the IM level at which the structure first reaches pre-defined damage state thresholds (DS1-DS4). Assuming that in each damage state the capacity follows a lognormal distribution, a Cumulative Distribution Function, CDF, is statistically fitted to the data. This CDF is the fragility curve, expressing the probability of exceeding a certain damage state for a given value of the IM.

Table 3: Basic information for selected 10 ground motions

EQ Name (Country)	RSN	Year	Station	M	R_{jb} (km)	R_{rup} (km)	Vs_{30} (m/sec)	5-95% Duration (sec)
Tabas (Iran)	139	1978	Dayhook	7.35	0	13.94	471.53	11.3
Friuli (Italy)	125	1976	Tolmezzo	6.5	14.97	15.82	505.23	4.9
Northridge (USA)	963	1994	Castaic - Old Ridge Route	6.69	20.11	20.72	450.28	9.1
Kobe (Japan)	1107	1995	Kakogawa	6.9	22.5	22.5	312	13.2
Kocaeli (Turkey)	1176	1999	Yarimca	7.51	1.38	4.83	297	15.1
Chi-Chi (Taiwan)	1485	1988	TCU045	7.62	26	26	704.64	13.3
Loma Prieta (USA)	802	1989	Saratoga - Aloha Ave	6.93	7.58	8.5	380.89	9.4
Trinidad (USA)	281	1980	Rio Dell Overpass, E Ground	7.2	76.06	76.26	311.75	12.3
Landers (USA)	879	1992	Lucerne	7.28	2.19	2.19	1369	13.8
Parkfield (USA)	31	1966	Cholame - Shandon Array #8	6.19	12.9	12.9	256.82	13.1

2.2.2 Fragility function

The fragility function, $P(SD_d|IM)$, which defines the probability of exceeding a damage state d given an intensity measure IM, is conventionally modeled using a lognormal CDF (Sam, 2025) (Sam, 2025):

$$P(SD_d|IM) = \emptyset\left(\frac{ln\,(\frac{IM}{\theta_d})}{\beta_d}\right) \qquad 1$$

Where

$P(SD_d|IM)$ = Probability of exceeding a given damage state d `for a given intensity measure IM.

$\emptyset$ = Standard normal cumulative distribution function (CDF).

θ_d= Median value of the intensity measure corresponding to the damage state d

β_d = Logarithmic standard deviation (dispersion) for damage state d

2.2.3 Damage states

The discrete DSs adopted in this study are defined in terms of inter-story drift limits, relating structural response and observable physical damage. Performance levels with corresponding drift limits from Ghobarah (2001) are tabulated in Table 1 below.

Table 4: Damage states and corresponding performance levels (adapted from Ghobarah (2001)

Performance level	Damage state	Interstory drift (%)
Immediate Occupancy	None Damage (DS_0)	<0.2
Damage control	Slight Damage (DS_1)	<0.5
Life Safe	Moderate Damage(DS_2)	<1.5
Near Collapse	Severe Damage (DS_3)	<2.5
Collapse Preventation	Collapse (DS_4)	>2.5

3. RESULTS AND DISCUSSIONS

2.3 Characterization of Structural Torsional Response

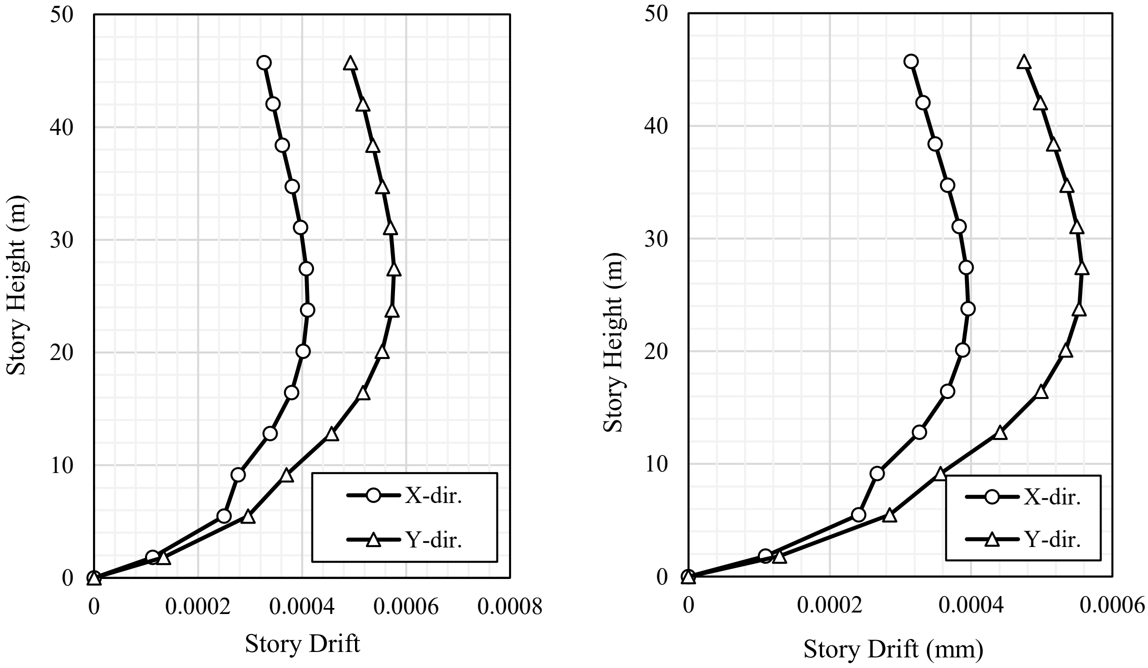
In order to determine the probabilistic seismic hazard, first of all, a wind load analysis analysis was performed to understand the dynamic characteristics of the interconnected building system. The Figure 3 presents the profiles of the building structure resulting from one-directional design wind load that was applied separately in the X and Y directions. Such an analysis plays an essential role in figuring out the source of torsional irregularities that are inherent in the structural system which, in turn, can considerably change its seismic behaviour.

The X-direction response under the load is depicted in the Figure 3(a). The load leads to a drift along the X-direction (solid line with circles) that reaches a maximum of nearly 0.00041 at the 6th story as one would intuitively expect. More importantly, however, this longitudinal load alone also causes a very significant drift in an orthogonal Y-direction (dashed line with triangles) to the structural system. This combined transverse drift is higher than the longitudinal drift at all levels of the building and it attains the maximum of 0.00058 at the 7th story.

The interconnected building behaves in such a manner as confirmed by the Figure 3(b) where the load is applied in the Y-direction. The maximum drift as a result of the Y-direction load is 0.00056 at Story 7 in the Y-direction, while the induced coupled X-direction drift is 0.00040 at Story 6.

The data represented in Figure 3 unveils the most important and evident fact that the interconnected building system shows considerable torsional irregularity. The production of a great drift response in the perpendicular direction (for instance, Y-drift from an X-load) is one of the typical signs that the system whose centre of mass is not at the centre of rigidity. The structural property of the eccentricity of the two interconnected buildings is the reason for the centre of mass and centre of rigidity not coinciding.

The discovery of this point is instrumental in defining the answer to the seismic probabilistic question. The torsionally highly coupled system may turn out to be less safe when the seismic excitation comes, because ground motion can cause the system to vibrate in complicated torsional modes at the resonance frequencies. Stress concentration and uneven drift demands could be the consequences of this phenomenon which cannot be detected by a 2D analysis of the structure. As a result, the torsional behaviour thus explaining the requirement for the subsequent seismic assessment of a complete 3D dynamic model is undoubtedly seen here by the static wind load analysis.



(a) **(b)**

Figure 3: Story drift for wind load; (a) Load in X-direction, (b) Load in Y-direction

2.4 Fragility Analysis

2.5 The fragility parameters define quantitatively the seismic vulnerability of the structure and represent the core outcomes of the IDA and subsequent statistical processing. The median capacity θ_d and the logarithmic standard deviation β_d are reported in Table 5 for each defined DS.

Table 5: Calculated fragility parameters for each damage state

Damage state	Mean, $(\boldsymbol{\theta_d})$	Standard deviation $(\boldsymbol{\beta_d})$
DS_1	0.124	0.150
DS_2	0.250	0.190
DS_3	0.650	0.190
DS_4	1.050	0.188

2.6 The fragility curves depicted in Figure 4 represent the continuous exceedance probability of each damage state across a range of seismic intensities (PGA) that were generated using the parameters from

2.7 Table **5**. The information presented by

Table **5** is essential for understanding the building's seismic performance that has been estimated. The building performance is understood from its median capacity $(\boldsymbol{\theta_d})$ and logarithmic standard deviation $(\boldsymbol{\beta_d})$ parameters. The median capacity that in fact corresponds to a PGA (in $\boldsymbol{g}$) with a probability of 50% of surpassing the damage state is as low as 0.124 for Slight Damage (DS1), which means an initial crack formation at minor damage of the structure with a low-level seismic ground motion is highly probable. The capacity is increased up to 0.250 for Moderate Damage (DS2) and jumps drastically to 0.650 for Severe Damage (DS3) and 1.050 for Collapse (DS4). Such a broad range, especially between the Life Safe (DS2) and Near Collapse (DS3) situations, is indicative of the nonlinear reserve capacity as well as the ductility of the structure being very high. At the same time, the logarithmic standard deviation $(\boldsymbol{\beta_d})$ which is a measure of uncertainty is $\boldsymbol{\beta_d}$ is quite $\boldsymbol{\beta_d}$ consistent and varies from 0.150 to 0.190 in a narrow range. The minimum scatter ($\boldsymbol{\beta_d = 0.150}$) for DS1 activity has the meaning that the beginning of fissuring is an almost completely deterministic event, while normal dispersions for DS2-DS4 (0.188-0.190) denote that the variability is the naturally occurring aspect of seismic analyses. Therefore, we can say, the fragility parameters reveal that the construction is likely to suffer serviceability damage even in a low-magnitude earthquake, nevertheless, it also discloses the presence of a huge remaining capacity to resist severe damage and collapse hence, the structure can be said to feature a high degree of seismic resilience in line with the "Life Safe" and "Collapse Prevention" performance requirements.

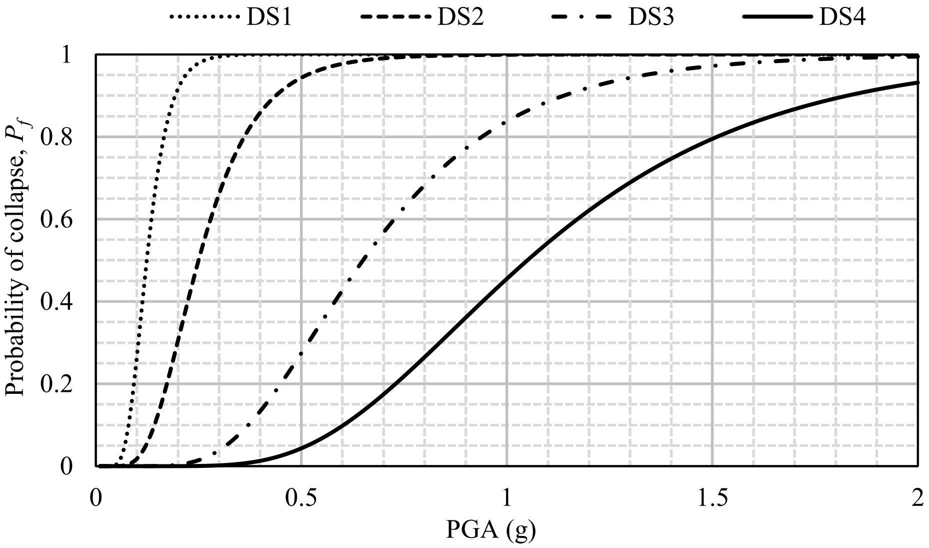


Figure 4: Fragility curves

4. CONCLUSIONS

This research involved the execution of a seismic probabilistic analysis of the interconnected buildings system with the aim of pinpointing its structural weaknesses as well as the system's seismic performance. The probabilistic analysis was evaluated based on fragility analysis in which IDA was utilized for a set of selected ground motion. The analysis was performed using a 3D finite element model, and the investigation yielded two primary conclusions.

- The first point to note is that the wind load analysis revealed that the interconnected building system exhibits significant inherent torsional irregularity. This is apparent from the large drift caused in the direction perpendicular to the load when the load was applied in one of the main directions. It shows that the structural center of mass and the center of rigidity do not coincide. The result confirmed that it is absolutely necessary to use a full 3D dynamic model to trace the complex coupled torsional-lateral seismic response that a 2D model would give the wrong answer. The structure showed high stiffness under the design wind load despite the irregularity with the drifts being very low and within the limits set by the code.

- The second point to highlight is that the fragility analysis helped in quantifying the structure's seismic resilience. It was found as a result of the research that DS1, slight, serviceability-level damage, could occur with a very high probability even at a low-intensity ground motion (median capacity of 0.124g). More importantly, however, the calculation of the structural margin of safety revealed that the risk of absolute failure is very low indeed. There is a significant difference between the median capacities for the Life Safe state (DS2, 0.250g) and the Collapse state (DS4, 1.050g), respectively. The considerable distance between these two points - with the collapse capacity nearly four times as much as the Life Safe capacity - indicates that the structure retains a large amount of nonlinear reserve capacity and ductility

Finally, the interconnected building system is a torsionally irregular one, but it still has a strong seismic performance. It can continue to dissipate the seismic energy effectively and it retains a large part of its structural integrity well beyond the point of first damage, thus, the "Life Safe" and "Collapse Prevention" performance objectives are met.

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DECLARATION OF USE OF AI

The authors declare that no AI tools were used in the preparation of this manuscript.

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