SSRG International Journal of Civil Engineering
ISSN: 2348-8352/ https://doi.org/10.14445/23488352/IJCE-V12I5P115

Volume 12 Issue 5, 197-209, May 2025

Original Article

Preliminary Fragility Assessment of a 19th-Century Baroque Church Bell Tower Using Multiple Time History Analyses

Jeffrey DG. Clemente[1], Francis Aldrine Uy[2], Michael Bautista Baylon[3]

[1]*National University Philippines – Baliwag Campus.*
[1,2]*School of Civil, Environmental, and Geological Engineering, Mapua University, Manila, Philippines.*
[2,3]*Usher Technologies, Philippines.*

[1]*Corresponding Author : jdclemente@nu-baliwag.edu.ph*

Received: 11 March 2025 Revised: 13 April 2025 Accepted: 12 May 2025 Published: 31 May 2025

Abstract *- Philippine structures frequently experience seismic activity, yet the effects of high-intensity earthquakes are often assessed only after they occur. The inadequacy of extensive research on this area, especially for the investigation of structures that were built before systematic building and structural codes were established, poses a problem, especially in heritage preservation and protection. This study aims to develop fragility curves as a preliminary assessment of the seismic vulnerability of the bell tower of Pulilan, Bulacan's Diocesan Shrine and Parish of San Isidro Labrador through Linear Time History Analysis (LTHA), considering limited inelastic data for adobe masonry and destructive testing on the structure. The material properties of tested concrete and adobe samples were integrated into a 3D structural model in Midas Gen to assess the tower's seismic response to scaled 11 ground motion records. The fragility curves from the preliminary assessment show that the bell tower is prone to minor damage to weak PGAs. Considering the required basic design PGA of 0.4g in the Philippines, structural compromise is anticipated, having surpassed the 50% probability threshold at the Life Safety (LS) limit. The findings can serve as a reference for future nonlinear analyses and for the implementation of appropriate retrofitting measures on the tower.*

Keywords *- Adobe masonry, Fragility curves, Heritage conservation, Linear Time History Analysis, Seismic vulnerability.*

1. Introduction

Over the last decade, the Philippines has experienced multiple destructive earthquakes with moment magnitudes (Mw) exceeding 6, including the Mw 7.0 Northwestern Luzon earthquake in July of 2022 [1]. Based on the National Disaster Risk Reduction and Management Council (NDRRMC) Situational Report issued publicly approximately two weeks post-event [2], this event alone inflicted an estimated 46 million U.S. dollars in infrastructure damage. The country's position in the Pacific Ring of Fire further amplifies the probability of earthquakes occurring in more damaging events than before.

Century-old churches in the Philippines are among those at risk of incurring damage when a strong earthquake hits. In 2013, numerous churches across Cebu and Bohol suffered massive, if not complete, damage after a Mw 7.2 earthquake hit the area [3]. Ten churches of significance were identified by the Heritage Conservation Society at that time as damaged by the quake, with 7 in Bohol alone [4]. While many were rebuilt and/or restored and turned over to their respective parishes years after the tremor, this model may end up not being sustainable in the long term. After all, the structural integrity and reliability of these churches, like other historical sites and structures, have depreciated over the years.

Many seismic fragility evaluations have focused on reinforced concrete structures or buildings that have been built for at least a decade. However, the lack of attention paid to heritage and old structures, including Unreinforced Masonry Structures (URMs) in the Philippines [13], perpetuates the need to do so in the name of preserving heritage and furthering research into applicable, appropriate, and ample structural retrofitting strategies that can be applied to existing structures in the country.

This lack of attention furthermore translates into a lack of comprehensive local studies on the matter, which widens the knowledge gap on understanding seismic vulnerability and on potential breakthroughs to more sensible and more modern approaches to resolving concerns. With most analyses usually revolving around the use of nonlinear methods or pushover analyses, research involving the application of linear methods for fragility analysis of structures, especially of a historical nature in the Philippines, has yet to be explored in its entirety.

The lessons learned from previous earthquakes emphasize the significance of early preparations and mitigation measures by civil engineers and the general people in Metropolitan Manila and neighboring provinces such as Bulacan to inspect heritage monuments and structures before an earthquake causes damage. Furthermore, comprehensive studies on historical edifices in the Philippines are scarce, and significant data, even for the component-level analysis of most of these structures, remain unavailable.

The Diocesan Shrine and Parish of San Isidro Labrador in Pulilan, Bulacan, is an excellent example of such a structure. The 19th-century Baroque church, with its bell tower, stands as a testament to the rich cultural and religious history of the region. The current iteration of the bell tower is made up of around 200-year-old adobe bricks for the first floor, while a seven-decade-old reinforced concrete structure sits from the second up to the fourth floor of the structure (Figure 1). As seismic loads are more certainly to affect the highest point in the entire complex, the paper focused on the modelling and displacement measurement for the church's bell tower.


Fig. 1 The main façade of the Diocesan Shrine and Parish of San Isidro Labrador in Pulilan, Bulacan

Given the limited availability of Adobe samples that can be retrieved from the structure, a conservative modelling approach is necessary to avoid compromising the accuracy of the results and capture the research's objectives while at the same time filling the present study gap. Furthermore, the method to be introduced in the research may yield results that need to be interpreted cautiously, considering the inherent uncertainties and the conservative assumptions in the bell tower's structural performance.

Integration of the principles in soil-structure interaction for this research will also be excluded from the focus of the study, as the paper focuses only on the vertical structure in question. Hence, the foundation flexibility of the structure is rather approximated using a pinned boundary condition, considering the absence of a soil study for the site. There will also be no in-depth architectural recommendations (especially cosmetic upgrades) that will be mentioned in this study, unless determined to be an important factor for the improvement of the structural integrity of the building. As such, this research presented potential techniques but did not directly perform the retrofitting itself in response to each and the entire set of seismic data used for the simulation.

The limited availability of inelastic material data constrains the extent of feasible interventions. Further, the status of the structure as a heritage building also limits the extent of samples that can be used and restricts the use of any destructive testing. Hence, the analysis will not fully capture progressive stiffness degradation, energy dissipation, or ultimate failure mechanisms of the adobe structure. Furthermore, the research is intended to be a preliminary assessment of the structure, which the linear time history analysis will serve accordingly.

2. Literature Review

Evaluation techniques for existing structures require a thorough understanding of the building's components. All infrastructure systems, especially those of a public nature, regardless of age, need to undergo routine and regular inspection, maintenance, and monitoring to ensure quality of life and sustainability in the long run [31]. However, there are obvious limitations when visual inspection is the sole mechanism implemented to achieve such. Long-term corrosion or fatigue of structural components, and even natural phenomena like earthquakes, can cause both visible and non-visible damage to civil engineering structures.

Previous studies have already shown that existing structures are often vulnerable even at low-magnitude earthquakes. Such is the case of this research's study site. Considering the nature of these publicly accessible areas, structural health monitoring (SHM) is important to evaluate a building's integrity and provide an overview of necessary repairs or retrofitting. Even before construction, the design and specifications of the different building systems must undergo thorough analysis and diligence in the application or integration of building codes. Hence, effective SHM connects the pre-construction to the post-construction condition of the structure and helps in easily identifying particulars on which most resources for repair and/or retrofitting may be done in a timely fashion.

In the Philippines, the design philosophy of seismic response is based on the base shear. However, high intense acceleration values have been observed in more recent seismic events, which affects the common design ground motion used in the initial design [8]. As such, there is a need for conducting in-depth investigations about their vulnerability and response during earthquakes.

Structural analyses usually fall under linear or nonlinear analyses, each with its own strengths and impacts on the eventual understanding of structural behavior under different loads. However, the selection of the type of analysis may impact the eventual simulation of a structural response and, therefore, the realistic scenarios posed by such a simulation. One method that can be implemented to do a vulnerability assessment is the linear time history analysis (LTHA) [6]. This form of linear analysis implements an analysis based on specific ground motion records and measures the dynamic response of the structure under its seismic loading, evaluated individually at each time step [7]. Like eigenvalue and response spectrum analysis, LTHA assumes that the structure behaves linearly and without any change in the existing material properties, nor that the recorded deformations will fall outside the elastic range of the building. LTHA has been compared with another common seismic design approach known as the equivalent static force procedure (ESFP), a simplified method of determining seismic loads on regular structures. However, ESFP has been cited to be lacking in providing a clear image of the fundamental periods of structures during seismic excitation, let alone the failure to consider the multi-directional nature of seismic events [8]. LTHA also provides a more detailed time-dependent structural response assessment, especially for seismic evaluation of structures to specific earthquake events that might pose a similar threat to local scenarios [6, 9]. Furthermore, linear time history analysis is computationally intensive, providing a little more truth to the structure's dynamic response, considering its main reference is actual ground motion data [10].

Conducting linear analysis for structures is usually seen as a preliminary assessment method in preparation or initial check prior to in-depth evaluation for large deformations or post-yield behavior, given that linear analyses do not account for material and geometric nonlinearities in the structure. Hence, seismic performance evaluations also rely on nonlinear analyses, considering that many other variables are considered in these methods.

A popular method under the nonlinear category is the pushover analysis, also known as nonlinear static analysis. Due to its inherent simplicity in the use of seismic loads and ground motion data, it has been used throughout structural and earthquake engineering [25] to capture the inelastic behavior of structures under seismic loads. Aside from pushover analysis, nonlinear time history analysis has also been known as a more realistic method to make seismic demand predictions and performance evaluations of various structures [26]. This method has been acknowledged to present the effects of rigidity changes, including strength-deformation relationships that happen after yield, and make use of the response at discrete time steps to establish how a building behaves in its inelastic format. These types of analyses are crucial, especially in cases of older structures like URMs. Baroque churches, for example, often have an absence of horizontal stiffening diaphragms and possess low material strength required to effectively resist tensile forces. The seismic sensitivity of these structures is further increased by the lack of internal diaphragms and the existence of wide-open areas. Their poor seismic resistance is also a result of their architectural designs, which include arches and vaults along with thin walls [27]. The most severe vulnerability of Baroque churches, however, is the collapse of its bell tower/s. Most of these constructions are tall and slim, a shape that is unfortunately more exposed to damage during an earthquake.

Recent research on the same matter focused on the location and general geometry of the bell towers, in addition to its classification as a URM. In Northern Italy, masonry church bell towers can be seen as either confined (integrated with the church walls) or isolated. Such distinction is important as a study in 2023 showed that while both types of towers suffer significant damage, confined ones experience earlier and more severe cracking [28]. As such, the structural interaction between the tower and the main structure of the church itself zeroes in on a potential difference in the natural and dynamic response of bell towers of different types. Aside from that, the slenderness or bulkiness of the geometrical features of the tower is seen as another factor influencing the structural seismic performance. Seismic loads acting on bell towers with smaller wall thickness, large openings, and small base sections increase the seismic vulnerability of the structure [29]. In Chile, architectural styles add to these identified variables, with churches identified from the colonial era more likely to exhibit high seismic vulnerability. This is due in part to the absence of earthquake-resisting devices or reinforcements and/or the deteriorating quality of the masonry [30]. Nevertheless, studies that aligned with this case still call for further research so that the actual structural weaknesses of the bell tower or belfry are identified properly. It is, therefore, important that simplified procedures are not just made as the primary reference for action.

Therefore, research into the seismic performance of historical Baroque churches and the development of their respective fragility curves is vital not just for the identification of seismic vulnerability but also for potential retrofitting and/or responsive solutions to their present conditions. Beyond the engineering aspect of these structures, these are also treated as invaluable cultural relics that should be preserved for future generations. In addition, further research into the matter leads to a greater understanding of the seismic behavior of complex masonry constructions and their implementation in earthquake engineering [27].

Nevertheless, the development of fragility curves from any method serves as a good and fast way to comprehend the effects of an earthquake on structures. It can also help with providing insights into pre-earthquake preparations and post-tremor recovery efforts and give way to estimate and/or reduce infrastructure damage and human casualties during tremors [11]. Usually defined as the probability of exceeding a known damage state in earthquakes, fragility curves also provide a way to predict potential damage in a seismic event by which many researchers begin with an evaluation criterion. [15]. The predictive nature of the data is also utilized as an indicator for pinpointing the extent of physical damage, whether cosmetic or structural, suffered by the structure in a simulation of the strongest possible quake.

3. Objectives of the Study

This paper's primary objective centered on the utilization of linear time history analysis (LTHA) to develop the fragility curves of the bell tower of the Diocesan Shrine and Parish of San Isidro Labrador. As such, this study was designed to achieve the following specific objectives:

- Analyze the material characteristics of the present adobe and reinforced concrete in the bell tower using available data and nondestructive testing.
- Apply Linear Time History Analysis (LTHA) to develop fragility curves, considering the limitations in inelastic data for adobe and reinforced concrete (RC) portions, including the reliance on nondestructive testing for the reinforced concrete portion.
- Develop fragility curves showing the probability of reaching the different damage levels in consideration of seismic intensity measures.
- Interpret the resulting fragility curves to assess the bell tower's seismic vulnerability and provide insights for future structural evaluation.

4. Materials and Methods

The research employs a case study approach focused on the bell tower of the Diocesan Shrine and Parish of San Isidro Labrador, a 25-meter-high structure built from adobe bricks in the 19th century and subsequently with reinforced concrete for its upper floors in its latest iteration in the 20th century.

Analysis of the structure (or any structure itself) will rely on getting the right data, beginning from the structural plan of the tower, the ground motion data to be used, and the values and properties to be obtained during material testing. The created three-dimensional (3D) version of the structure in the software will then be the basis for all future runs of the analysis. Furthermore, the entire method repeats itself like a cycle for each ground motion data. Any change or correction, therefore, would require the rerun of the entire methodology, beginning from the computerized 3D model onwards. For a better understanding of how the outputs of the paper came, the process for the analysis is laid out in the following flow chart methodology (Figure 2).

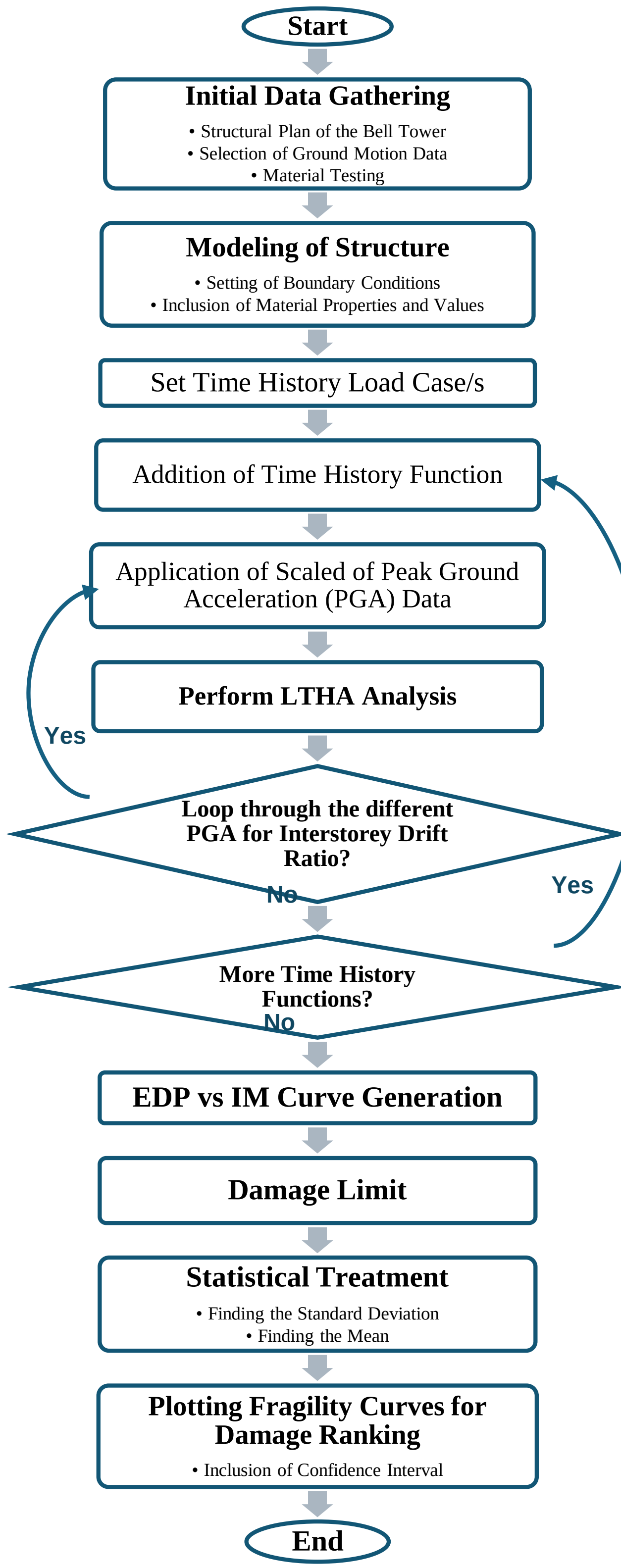


Fig. 2 Flowchart for the entire analysis

4.1. Structural Modelling and Material Testing

To accurately model the bell tower's response to seismic loads, it is essential to determine the material properties of the adobe bricks and concrete used in its construction. Nondestructive testing (NDT) methods will be employed to assess these properties without causing damage to the structure. As such, the rebound hammer test was performed for the concrete areas in accordance with ASTM C805 / EN 12504-2. The hammer was held perpendicular to the test surface, and 10 readings were recorded at each location. Since chipping the plaster was not permitted for the structure, the average rebound number at each location was reduced by 25% [14] to account for the potential influence of the unremoved plaster layer. The adjusted average for each test location was then computed.

Furthermore, for a higher accuracy of the structural behaviour under simulation, the adobe on the first floor of the church's bell tower was also tested. Six (6) cylindrical adobe stone specimens were therefore retrieved from the structure using the core drilling method, following ASTM C42/C42M-20 (Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete). Each of these retrieved cores had an approximate diameter of 4 inches (100 mm) and a height of 8 inches (200 mm). The collected number of samples, however, was made due to the limited availability of workable adobe stones from the bell tower, but nevertheless, the samples were divided into two groups. The first three (3) bricks were tested for compressive strength in normal conditions. The other three stones, comprising the second group, were used to determine the adobe stones' physical properties using ASTM C97/C97M-23 before being dried to a constant mass and tested for compressive strength following ASTM C170/C170M-23. While this sample size of the bricks (n = 6) may limit the comprehensiveness of the statistical analysis involved in this matter, the results will still be able to provide valuable insights into the material's physical and mechanical properties.

On the other hand, with the corresponding architectural and structural plan procured, a three-dimensional (3D) model of the bell tower will be created using Midas Gen, a specialized structural analysis software. This model will incorporate the material properties obtained from the NDT methods and include the effects of the tower's weight and seismic loadings. The modelling process involves creating the structure as an unreinforced masonry model using the finite element analysis (FEA) capabilities of Midas Gen, applying both static and dynamic loads, and setting appropriate boundary conditions to simulate real-world constraints and interactions with the surrounding soil.

The overall engineering aspect of the model also considered assumptions for the elastic modulus of the adobe bricks and Poisson's ratio of both the adobe and concrete parts of the structure. A value of 300fc' for adobe masonry walls was used in accordance with a suggested engineering design for earth buildings [18=5], while the concrete parts used the typical formula of $4700\sqrt{(fc')}$ for its elastic modulus. Conservative estimates for Poisson's ratio were also done for the adobe and concrete material in the model, placing the value at the higher limit of the range of 0.2, considering the usual range of 0.1 to 0.2 for concrete and a test conducted in 2012 for adobe bricks yielding the lower limit [16].

4.2. Ground Motion Selection

The seismic response test on the structural model was done through a linear time history (LTHA) using the selected ground motion records applied at varying intensities. The building's response will be recorded for each intensity level, resulting in a series of sensible fragility curves that show the correlation between the intensity measure (IM) and the engineering demand parameter (EDP).

The study utilized 11 earthquake records (see Table 1) and each of their respective ground motion data to simulate the bell tower's behaviour under seismic excitation. This follows the practices stated in ACI 369.1M-17: Standard Requirements for Seismic Evaluation and Retrofit of Existing Concrete Buildings. At the time of the writing, the researcher made use of the available database within Midas Gen and then cross-checked with the United States Geological Service (USGS) to ensure that all are natural earthquakes and exclude all man-made events.

Table 1. Reference ground motion data

Year of Event	Seismic Event	Magnitude
1989	Loma Prieta	6.9
1940	Elcentro	6.9
1952	Taft Lincoln School	7.5
1971	San Fernando	6.6
1994	Northridge, Santa Monica	6.7
1978	Miyagi – Coast	7.7
2011	Tohoku – Coast	9.1
1995	Hyougoken	7.3
1983	Nihonkai – Central	7.8
1985	Mexico City	8.0
1966	Parkfield - Cholame	6.0

The selection of these ground motion data relied on two things. The first is to ensure the near-realistic scenario that would affect the bell tower considering the projected seismic event around the area. The second criteria lie in maximizing the actual response of the simulated 3D model for an earthquake scenario, allowing the researcher to measure the probability of damage to the building without neglecting any possibility.

On the other hand, it is necessary to establish which damage indices will be used, as these are crucial tools in seismic fragility analysis, providing quantitative measures to assess the extent of damage a structure may experience during an earthquake. The performance limit states, as used by Xue et al. in 2008 [17], will be utilized in this research. As this relies heavily on the percentage drift observed during the application of the loads in the model, the researcher will employ the following formula to get the maximum interstorey drift value:

$$\%drift = \frac{Roof\ displacement}{Building\ height} \times 100\% \quad (1)$$

Which will then be cross-checked with the vertical grid line covered by each %drift value in the performance limit states as shown in Table 2.

Table 2. Performance limit states

Performance Limit State	ISDR%
Operational Performance (OP)	0.5%
Immediate Occupancy (IO)	1.0%
Damage Control (DC)	1.5%
Life Safety (LS)	2.0%
Collapse Prevention (CP)	2.5%

4.3. Development of Fragility Curves

To develop the fragility curve, the following equation [18] will be utilized, as both the mean and the standard deviation are variables required to create the curves. The following is the formula with their respective variable equivalence:

$$P[D/PGA] = \Phi\ \frac{\ln(PGA) - \mu}{\sigma} \quad (2)$$

Where D is the damage state, Φ is the standard normal cumulative distribution function, μ is the mean value of the data, and σ is the standard deviation of the logarithm of the Peak Ground Acceleration (PGA).

This probabilistic model will depict the likelihood of reaching or exceeding different levels of damage based on a given seismic intensity measure. The data obtained from the multiple iterations of the LTHA will be analyzed to generate the respective fragility curves of the bell tower that represent the probability of exceedance for the various damage levels. The results will be interpreted to provide insights into the seismic vulnerability of the structure and to inform the potential need for retrofitting.

5. Results and Discussion

The following are the results for each variable and objective set for the paper.

5.1. Building Model

Using Midas Gen, the model of the bell tower was modelled according to the available architectural and structural plans and details of the building. The structural model of the building contains the key characteristics of the structure, from its adobe brick base up to the concretized upper floors. For reference, the structure belongs to Occupancy Category III of the National Structural Code of the Philippines (NSCP) [19].

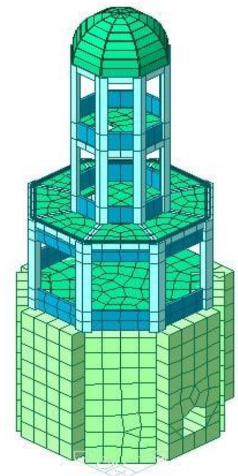


Fig. 3 Three-dimensional model of the bell tower as created in the software

5.2. Physical Properties and Compressive Strength Results of Adobe

As mentioned in the methodology, the accuracy of the assessment also required the determination of the physical properties of the adobe bricks found on the first floor of the bell tower. The provisions stated in the Standard Test Methods for Absorption and Bulk Specific Gravity of Dimension Stone (ASTM C97/C97M-23) were followed in the gathering of the main data for input in the software and for additional calculations regarding the status of the adobe present in the building (see Table 3).

Table 3. Physical properties of the adobe bricks

Sample No.	Sample 1	Sample 2	Sample 3	Mean
Bulk Density	12.55 kN/m^3	13.27 kN/m^3	13.30 kN/m^3	13.04 kN/m^3
Saturated Density of Sample	13.70 kN/m^3	14.39 kN/m^3	14.35 kN/m^3	14.15 kN/m^3
Dried Density	11.05 kN/m^3	11.58 kN/m^3	11.31 kN/m^3	11.32 kN/m^3
Water Absorption	24%	24%	27%	25%
Porosity	0.27	0.29	0.31	0.29
Void Ratio	0.37	0.40	0.45	0.41
Moisture Content	0.14	0.15	0.18	0.15
Degree of Saturation	64%	69%	77%	70%

Beyond the physical properties, all samples underwent a compressive strength test, albeit under two different conditions. For this case, the compressive strength is an important variable because it allows the researcher to make use of the corresponding values to ensure that the model itself is closest to the real-life scenario without compromising the structural integrity or potentially other factors that might affect the actual performance of the tower under seismic duress. Two sets of adobe samples were tested under different conditions: normal and oven-dried. The results (Table 4) indicate higher compressive strength values for oven-dried samples, with a lower Coefficient of Variation (CoV), suggesting more consistent mechanical properties in controlled conditions.

Table 4. Compressive strength results in two groups

Normal Samples	**1**	**2**	**3**
Compressive Strength	4.01 MPa	2.87 MPa	5.92 MPa
Mean fc'			**4.27**
Coefficient of Variation (CoV)			**36%**
Oven-Dried Samples	**1**	**2**	**3**
Compressive Strength	8.13 MPa	8.50 MPa	8.16 MPa
Mean fc'			**8.26**
Coefficient of Variation (CoV)			**2%**

On these results, it must be noted that a lower Coefficient of Variation (CoV), as seen in the second group, indicates that the compressive strength values are more consistent, compared to a suggestion of greater variability in the numbers by a higher coefficient value. The first group was used during analysis in the assumption of the bricks' true in-situ value. The observed difference can be attributed to the potential effects of the bricks' moisture content under normal conditions vs. the oven-dried samples, which indicates a correlation between that variable and the CoV of the samples themselves.

Another reason might be the generation when the structure was built. The CoV in the grade scope of concrete is between 13% to 15%, and for the same research that investigated tens of buildings in China, the average CoV for structures built in 1970-1971 and 1979-1980, the average CoVs were 16.8% and 14.26% respectively [20].

Furthermore, the lack of extensive samples retrieved from the bell tower due to foreseen and acknowledged limitations has also affected the values, considering that higher numbers of samples might present a much clearer image of the overall compressive strength of the masonry structure. Hence, this noted variability of the paper's compressive strength test shows a potential impact on the overall seismic response of the structure upon analysis.

5.3. Rebound Hammer Testing Results of Reinforced Concrete

A rebound hammer testing (nondestructive testing) was implemented on the concrete areas of the bell tower. Since destructive testing (coring and plaster removal) is not allowed due to heritage building testing limitations, available loose adobe samples were used for the determination of the compressive strength of the upper levels of the structure. The testing follows provisions from ASTM C805 / EN 12504-2 (Standard Test Method for Rebound Number of Hardened Concrete). Tables 5 and 6 are the results from ten (10) test locations distributed evenly across the structure's columns/beams and slab to capture representative data from the bell tower's build.

Table 5. Rebound numbers for the bell tower's column and beams

Location No.	**Average Rebound Number**	**Reduced Rebound Number (-25%)**	**Equivalent Compressive Strength (MPa)**
1	51.10	38.33	39.45
2	49.80	37.35	37.69
3	48.20	36.15	35.55
4	49.80	37.35	37.69
5	53.00	39.75	42.06
6	50.80	38.10	39.04
7	49.60	37.20	37.42
8	48.20	36.15	35.55
9	49.80	37.35	37.69
10	48.20	36.15	35.55
Mean fc'			**37.77**
Coefficient of Variation (CoV)			**5%**

Table 6. Rebound numbers for the bell tower's slab

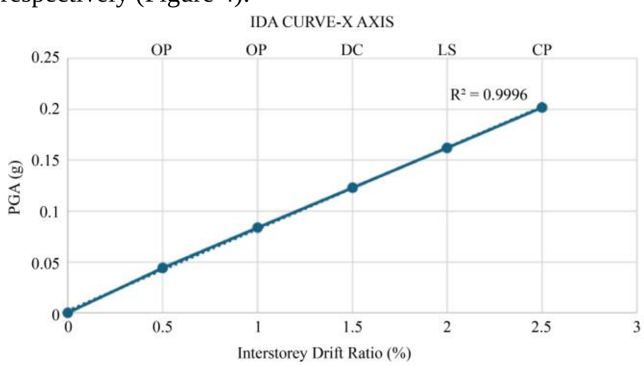
Location No.	**Average Rebound Number**	**Reduced Rebound Number (-25%)**	**Equivalent Compressive Strength (MPa)**
1	37.30	27.98	31.07
2	37.10	27.83	30.93
3	38.90	29.18	32.22
4	33.60	25.20	28.42
5	35.90	26.93	30.07
6	35.10	26.33	29.49
7	35.70	26.78	29.92
8	34.60	25.95	29.13
9	35.70	26.78	29.92
10	35.00	26.33	29.49
Mean fc'			**25.23**
Coefficient of Variation (CoV)			**7%**

After obtaining the adjusted rebound averages for the 10 test locations, the values were converted to an estimated compressive strength (MPa or psi) using the manufacturer's correlation chart for the specific hammer model used. A second average was computed for all locations to determine the equivalent compressive strength for the reinforced concrete. Since the test was performed on a plastered surface, no additional correction factors were applied beyond the applied 25% reduction in rebound values, as explained in the methodology part of this paper. For the slab, however, a correction factor was applied in response to the rebound hammer's orientation.

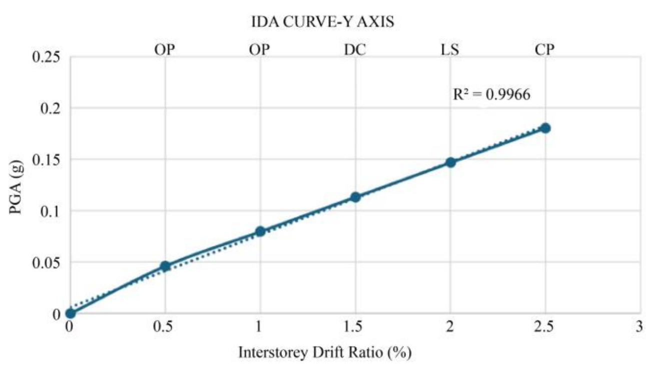
5.4. Engineering Demand Parameter (EDP) vs. Intensity Measure (IM) Curves

Two (2) EDP vs. IM curves were developed for the bell tower, considering the X-axis and Y-axis of the structure. The graph shows a correlation between the intensity measure (peak ground acceleration in g) and the engineering data parameter (the interstorey drift ratio in percent) plotted until the collapse prevention (CP) performance level. The results were gathered using the 11 ground motion data, increasingly scaled by 0.1g via the linear time history analysis (LTHA).

Based on the results, the structure is expected to reach the CP level, using the interstorey drift ratio (ISDR) at 2.5% as the basis, at 0.18g to 0.20g in the Y-axis and X-axis, respectively (Figure 4).



(a)



(b)

Fig. 4 Engineering Demand Parameter (EDP) vs. Intensity Measure (IM) curves for the (a) X-axis, and (b) Y-axis of the bell tower.

The curves showed the relationship between the likely peak ground acceleration (PGA) as the intensity measure (IM) and the thresholds for each of the performance levels as the engineering demand parameter (EDP) used in this paper. Given that there is an absence of inelastic data, the EDP vs IM curve is linear in nature, considering that the structure remained in the elastic range under LTHA. Further, the coefficient of determination (R^2) values, which fell at 0.9996 and 0.9966 for the X-axis and Y-axis, further proved that the curves for both axes are linear and are predictably close to the actual data points used in the linear regression analysis. The linearity of the graph also relates to the utilization of the available data for the bell tower, as mentioned in the preceding chapter. With the lack of inelastic data, the graph is expected to form such a linear plot, considering that the structure remained in the elastic range under LTHA.

5.5. Fragility Curves

In the provisions of the National Structural Code of the Philippines (NSCP), structures in the Philippines are expected to withstand a peak ground acceleration (PGA) of 0.4g, considering the probability of exceedance of these PGAs in 50 years. This sets the basic design PGA, depending on the Seismic Zone on which the structure sits on, which is in. Seismic Zone 4 is based on the NSCP.

Aside from being prescribed by the NSCP, the setting of the baseline to 0.4g followed existing probabilistic seismic hazard data for the country. In 2020, a probabilistic seismic hazard model of the Philippines was generated in consideration of the tectonics of the archipelago [32]. The results of the model, which was later used among the many references for the seismic design principles in the latest edition of the national structural code, anchored on an analysis of the percentage of the probability of exceedance (PoE) in 10 and 50 years (Figure 5). The study site, which lies very near Manila in the figure, can experience strong shaking under an estimated PGA range between 0.3g to 0.5g, around 50% of the standard acceleration due to Earth's gravity.

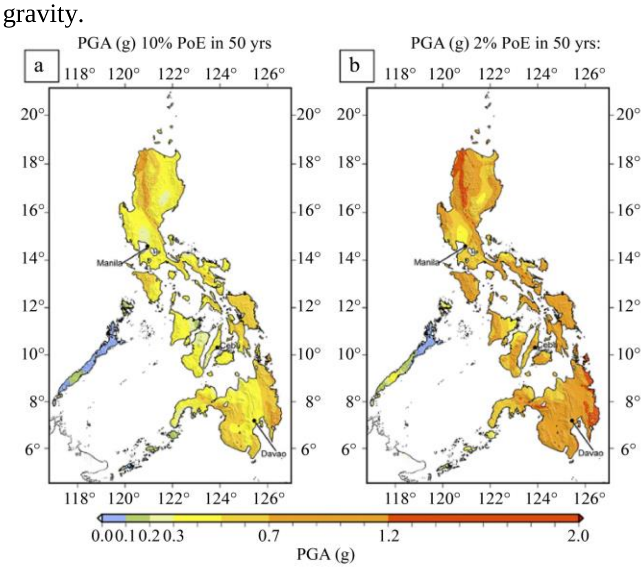
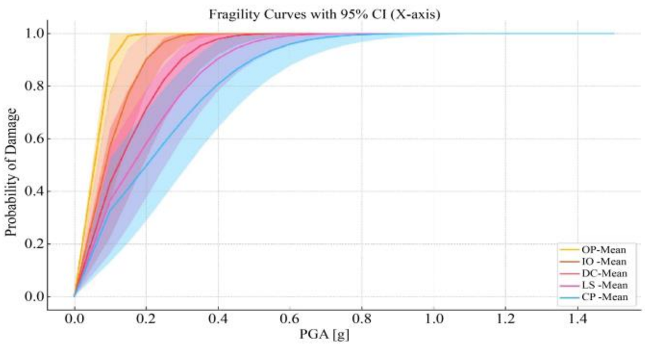
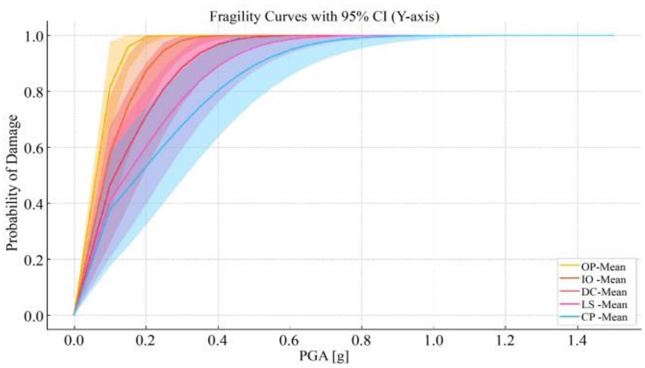


Fig. 5 Map showing the peak ground acceleration (PGA) map for the Philippines with (a) a 10% probability of exceedance, and (b) a 2% probability of exceedance in 50 years [32].

All presented curves were smoothened using a lognormal distribution function, as seen in Equation 2. The fitting process was combined with Maximum Likelihood Estimation (MLE), usually embedded in the fragility curve framework. The fragility estimates for the bell tower are presented in Figure 6.



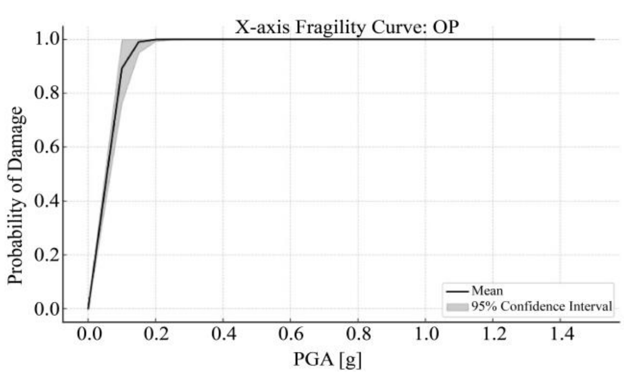
(a)



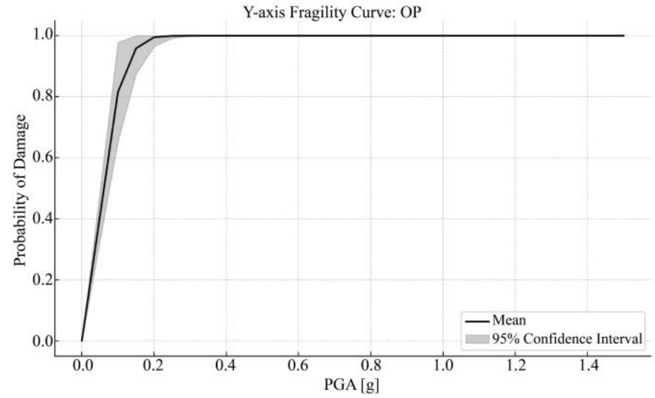
(b)

Fig. 6 Fragility curves for (a) for the X-axis of the structure, and (b) for the Y-axis of the structure.

The results show that at weak ground motions (PGA = 0.2g), there is already a 100% chance in both axes to reach the operational (OP) level (Figure 7). When the immediate occupancy (IO) performance level is checked at the same PGA, the probability of reaching the level is 90% for the X-axis. and 87% for the Y-axis, (Figure 8). At PGA = 0.4g, there is a 100% probability of exceeding both the Operational (OP) and Immediate Occupancy (IO) levels along both axes.



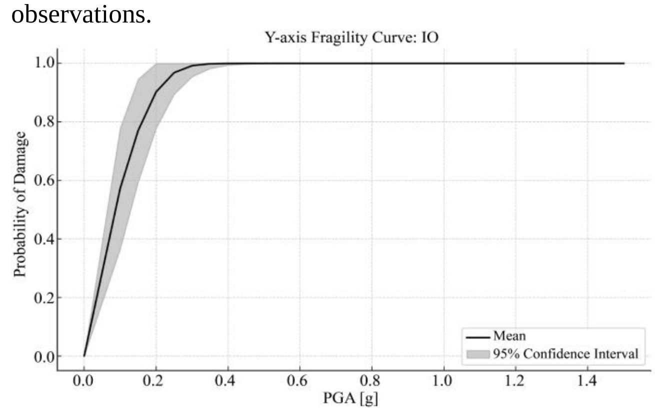
(a)



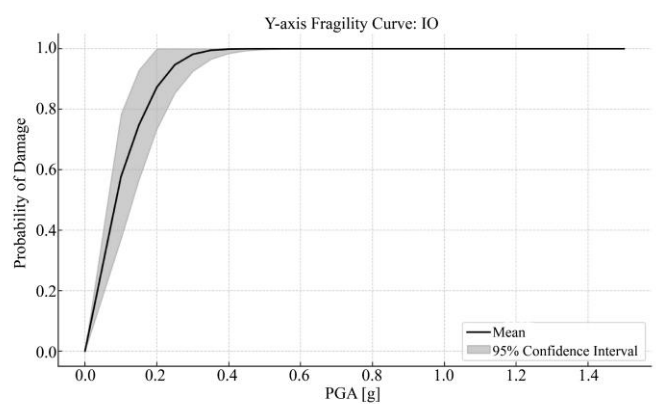
(b)

Fig. 7 Fragility curves under the operational performance level for (a) For the X-axis of the structure, and (b) For the Y-axis of the structure

For reference, the damage states used by Xue et al. are also the same as the ones utilized under FEMA 356 [21], which defines the levels based on their characteristic of negligible damage state and light damage to the structure, respectively. It is worth noting that in both cases, the confidence interval range has increased in width. This behavior can also be seen in the succeeding individual fragility curves for each performance level. Nevertheless, the trends remain mathematically sound despite these observations.

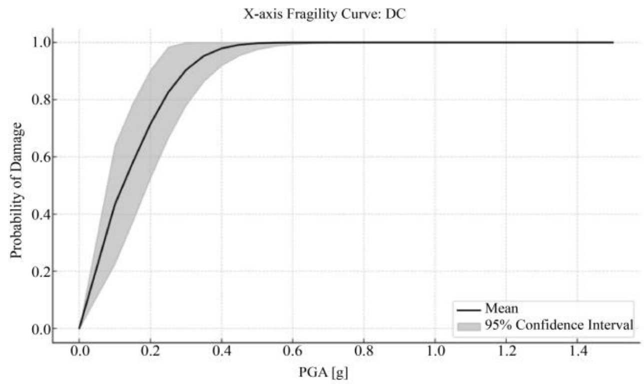


(a)

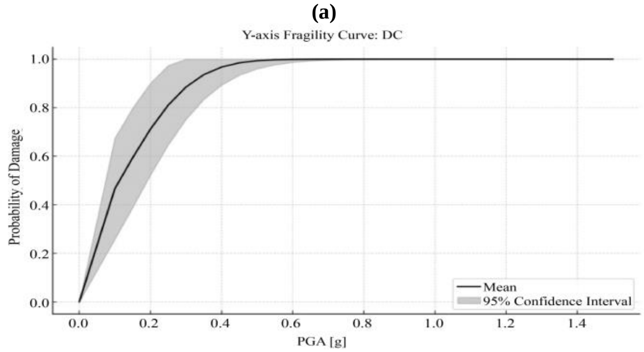


(b)

Fig. 8 Fragility curves under the Immediate Occupancy (IO) limit state for (a) For the X-axis, and (b) For the Y-axis of the structure.



(a)

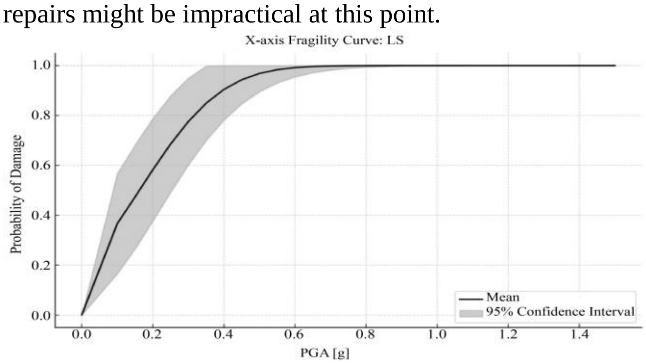


(b)

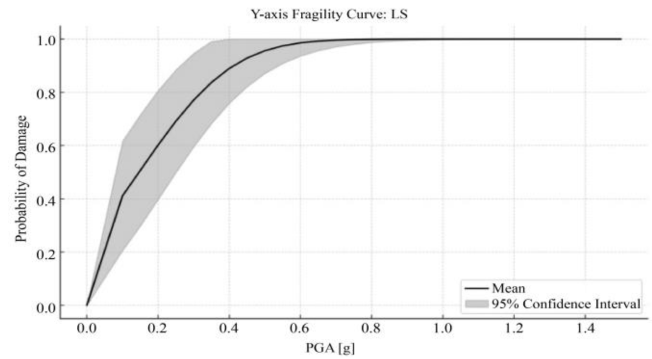
Fig. 9 Fragility curves under the damage control limit state for (a) For the X-axis of the structure, and (b) For the Y-axis of the structure.

At PGA=0.2g, the structure can have a 72% and 71% probability of damage in the X- and Y-axis under the damage control (DC) limit. Additionally, a 97% chance at the X-axis and 98% at the Y-axis to reach the same level at PGA = 0.4g exists (Figure 9). The DC performance limit means the structure is expected to be non-functional in the aftermath of the earthquake, but the damages are repairable at a reasonable cost, and the structure itself has only suffered moderate damage to its systems.

The structure, however, has a chance of 58% and 60% in the X- and Y-axis to exceed the life safety (LS) performance level. This is further reinforced by a 90% chance in the X-axis, and 89% Y-axis exists under the LS limit at PGA = 0.4g (Figure 10). Life safety performance levels mean that the building has suffered from significant damage and that repairs might be impractical at this point.



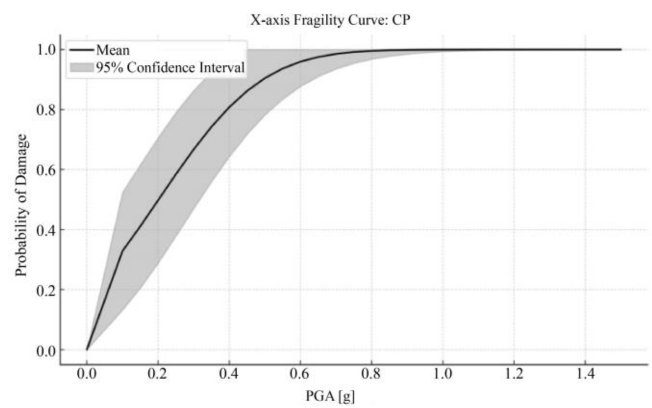
(a),.



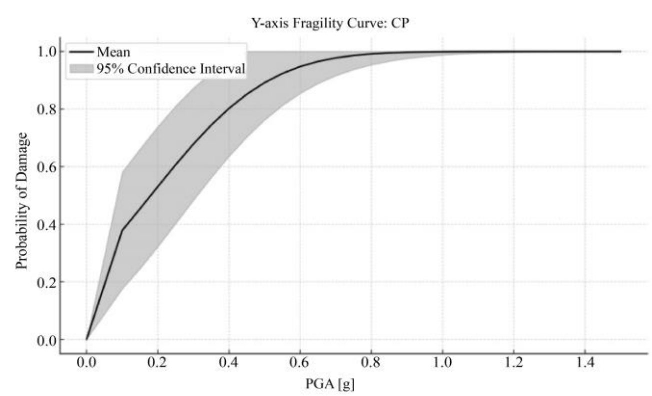
(b)

Fig. 10 Fragility curves under the Life Safety performance level for (a) for the X-axis of the structure, and (b) For the Y-axis of the structure.

The probability of damage under the Collapse Prevention (CP) limit ranges from 50% to 53% at PGA=0.2g, aligning with trends observed in the EDP vs. IM curves. Furthermore, an 81% probability of damage on the X-axis and 80% probability of damage on the Y-axis exists when measured against the NSCP's Seismic Zone 4 basic design PGA of 0.4g (Figure 11). This performance level means the structure will incur severe damage, and there is a substantial chance that the building is in a near-collapse state.



(a)



(b)

Fig. 11 Fragility curves under the collapse prevention performance level for (a) For the X-axis of the structure, and (b) For the Y-axis of the structure.

The confidence interval (CI) for the fragility curves also emphasizes the presence of uncertainty in the true probability of damage occurring at each PGA (Figure 12). Set at a 95% confidence interval, the uncertainty range can be seen in the shaded regions in the graph, which widened as it reached the higher PGAs. This is due once again to the linear assumptions taken in the use of the LTHA, with the certainty more seen at lower damage states compared to the higher damage states. The absence of workable inelastic data from the materials of the tower affects the value range as it reaches the maximum probability projections. As such, the combination of the LTHA and absence of inelastic data, therefore explains the increase in uncertainty range at higher damage states.

Nevertheless, the CI range shows that the trend is well within the identifiable upper and lower limits of the defined uncertainty range. However, the observed results, inclusive of the confidence interval must not be taken as a flaw or loophole in the accuracy of the projections for each condition. Rather, the CI must be taken as the likelihood of the real probability of damage falling between the values contained in the region, with the linear path expected to be the most likely considering the statistical treatment employed for the gathered data. In any case, the graph was able to show a clear trend on the probabilities that can be considered for further applications or studies in the matter. Furthermore, the steepness of the curves shows that the preliminary assessment of the structure connects it to the observed high probabilities of minor damage being incurred at low PGAs. The outcome also correlates with the inherent brittleness of the adobe material present in the tower, considering the differences in the material properties of the masonry compared to reinforced concrete. The linear analysis that was also implemented affected the curves, considering the established limitations for using so in the previous chapters.

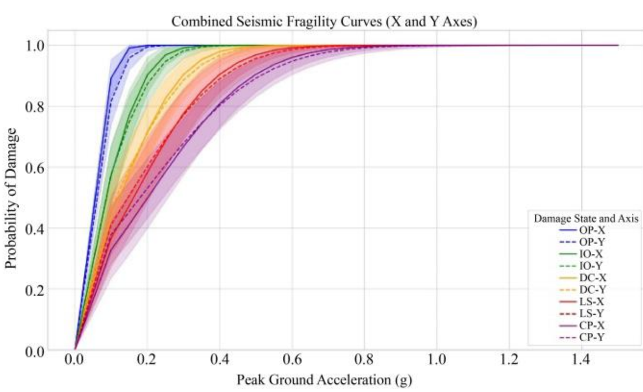


Fig. 12 Fragility curves with probability of exceedance measurement using the mean data from the X-axis and Y-axis, including the 95% confidence interval

6. Summary and Conclusion

A fragility curve for the X- and Y-axis of the bell tower was generated using the values retrieved from the linear time history analysis (LTHA). Eleven varying ground motion data were made as a seismic reference toward the projected probability of damage at different peak ground accelerations (PGA) during the analysis. Considerations for the available data on the physical properties of the adobe and concrete present in the bell tower were also made part of the input for the structural model and proper analysis.

The absence of established and well-documented research on the material properties of the tower and the use of LTHA on them had to be filled through several tests and calculations. Considering the heritage status of the building and the lack of obtainable extensive samples for testing, a nondestructive method via the rebound hammer testing was implemented for the concrete upper levels of the tower, while six (6) adobe samples from the lower levels of the structure were retrieved from the loose samples given by the church for the identification of its physical properties. Compressive strength for both materials was computed and integrated with the model created in Midas Gen for analysis. The results of the analysis show that the LTHA yielded a linear probability of scenarios in the research's preliminary assessment of the bell tower's seismic vulnerability. Both fragility curves showed how the tower is already susceptible at 0.2g in reaching the collapse prevention (CP) performance level, with PGAs influencing the preceding levels. The curves also give an insight into the projected moderate to heavy damages to be incurred starting and beyond 0.4g for the structure. A noticeable linearity of the graphs can be traced back to the fact that there was no inelastic data available for inclusion in the overall computerized assessment, which is why the influence of variables such as post-cracking behaviour, strain softening, and hysteretic response are not present in the final output.

The 0.4g threshold aligns with seismic provisions in the latest National Structural Code of the Philippines, indicating that the bell tower is at moderate damage risk even under best-case conditions. However, as mentioned in the previous chapters, the analysis was aligned to conservative estimations, which means that the results might be a little higher or overestimated than the actual scenario when an earthquake transpires in the area. Nevertheless, the fragility curves show a clear image that the bell tower of the chosen study site is highly susceptible to moderate and especially high-intensity tremors.

6.1. Recommendations

It is recommended that future researchers conduct further investigation into the inelastic properties of the adobe and concrete present in the structure to help establish the nonlinear behaviour of the bell tower in succeeding analyses. As the researcher set this as a current limitation for this paper, the Soil-Structure Interaction (SSI) may be explored as another additional variable in the structural response of the tower during an earthquake. This would tackle whether the foundation of the bell tower itself may influence the stability of the structure during seismic events. Aside from that, the basic idea of whether the type of soil present in the area affects the overall behaviour of the tower under seismic duress is another topic that can be integrated into further studies related to foundation design and the overall fragility of structures. Given the high probability of damage at low PGAs, exploring retrofitting strategies is essential to improve the bell tower's seismic resilience. Considering that the tower is an almost 70-year-old structure, future researchers may explore the following retrofitting techniques for the structure and whether those strengthening interventions are appropriate and feasible. Further studies should also look at whether they can bring to reduce the effects of seismic excitation on the bell tower and the potential constraints for implementation (e.g., budget, heritage conservation guidelines):

- Implementing an ultra-high-performance concrete (UHPC) or carbon fiber-reinforced polymer (CFRP) as a retrofitting solution, particularly for the concrete upper floors of the bell tower, can be among the methods that can be explored. Studies indicate that CFRP retrofitting enhances the ultimate load capacity of reinforced concrete beams by 56.6%, though UHPC is preferable for improving displacement, toughness, and fracture energy [22]. Additionally, further research on CFRP demonstrates that incorporating a higher elastic modulus generally resulted in an increased CFRP/steel bond, leading to enhanced load-bearing and flexural performance of retrofitted steel components [23], variables that are of importance in terms of assessing the entire strength and performance of the tower in future seismic events.
- Using longitudinal and transversal ties for the masonry portion of the structure. A study conducted in Central Italy focused on masonry churches and common interventions applied as a response to seismic events in the area. These strategies included wooden insertions, metal ties, and buttresses. In the sample sites of the paper, however, restraining tie rods became more acceptable in terms of providing adequate connections between the existing building systems and elements to improve the global and local response of the masonry [24].

Furthermore, owing as well to the fact that the Philippines is among many countries suffering from the effects of climate change, future researchers can also endeavour to study whether prevailing or persistent environmental factors experienced in the country affect the quality and properties of the existing materials in a structure. Such an idea would help in further understanding whether climate change plays an active role in potential accelerated material degradation and how such can affect the fragility of a structure and long-term conservation plans for historical buildings.

Acknowledgments

The corresponding author sincerely thanks the National University Philippines - Baliwag Campus for allowing the researcher to use their software for the analysis and expresses his appreciation to Bulacan State University for providing the nondestructive testing (NDT) equipment used in this study. Their support and resources were invaluable in completing this research.

References

[1] Jeffrey S. Perez et al., "Impacts and Causative Fault of the 2022 Magnitude (M_w) 7.0 Northwestern Luzon Earthquake, Philippines," *Frontiers in Earth Science*, vol. 11, pp. 1-15, 2023. [CrossRef] [Google Scholar] [Publisher Link]

[2] "*Situational Report for Magnitude 7.0 Earthquake in Tayum, Abra*," National Disaster Risk Reduction and Management Council (NDRRMC), 2022. [Publisher Link]

[3] "*Final Report Effects of Magnitude 7.2 Sagbayan, Bohol Earthquake*, National Disaster Risk Reduction and Management Council (NDRRMC), pp. 1-42, 2013. [Publisher Link]

[4] Pia Ranada, Cultural Agencies to Restore Damaged Heritage Structures, Rappler, 2013. [Online]. Available: https://www.rappler.com/philippines/41417-national-historical-commission-statement/

[5] Henremagne C. Peñarubia, Probabilistic Seismic Ground Motion Hazard Analysis: A Tool for Disaster Risk Management, Seminar on Earthquake Engineering and Disaster Risk Management, pp. 1-23, 2019. [Online]. Available: http://www.rsucivilengineering.weebly.com/uploads/7/7/6/4/77644974/psgmha_atool4disasterriskmitigation_081919_4rsu.pdf

[6] Luca Lombardi, Flavia De Luca, and John Macdonald, "Design of Buildings through Linear Time-History Analysis Optimising Ground Motion Selection: A Case Study for RC-MRFs," *Engineering Structures*, vol. 192, pp. 279-295, 2019. [CrossRef] [Google Scholar] [Publisher Link]

[7] Alessia Di Cuia et al., "Linear Time-History Analysis for EC8 Design of CBF Structures," *Procedia Engineering*, vol. 199, pp. 3522-3527, 2017. [CrossRef] [Google Scholar] [Publisher Link]

[8] P. Mortazavi, J. Stephenson, and J. Elliot, "The Benefits of Linear Time-History Analysis in Consulting Practice and the Precision of the Code Calculated Period," *6th World Conference on Earthquake, 16WCEE 2017, National Information Centre of Earthquake Engineering*, 2017. [Google Scholar]

[9] Luca Lombardi, and Flavia De Luca, "Derivation of Fragility Curves at Design Stage through Linear Time-History Analysis," *Engineering Structures*, vol. 219, pp. 1-28, 2020. [CrossRef] [Google Scholar] [Publisher Link]

[10] Hasan Dilek, and Ali Sadeghpour, "A Comparison of Different Linear and Non-linear Structural Analysis Methods," *ENG Transactions*, vol. 3, pp. 1-9, 2022. [CrossRef] [Google Scholar] [Publisher Link]

[11] Fadzli Mohamed Nazri, *Seismic Fragility Assessment for Buildings due to Earthquake Excitation*, Springer Singapore, pp. 1-114, 2018. [CrossRef] [Google Scholar] [Publisher Link]

[12] Jhon Philip Camayang, Orlean Dela Cruz, and Rhommel Grutas, "Integrating Building- and Site-Specific and Generic Fragility Curves into Seismic Risk Assessment: A PRISMA-Based Analysis of Methodologies and Applications," *CivilEng*, vol. 5, no. 4, pp. 1011-1041, 2024. [CrossRef] [Google Scholar] [Publisher Link]

[13] D.D'Ayala et al., "Multi–hazard Risk Assessment of Priority Cultural Heritage Structures in the Philippines," *6th Asia Conference on Earthquake Engineering (6ACEE)*, Cebu City, Philippines, pp. 1-13, 2016. [Google Scholar] [Publisher Link]

[14] Samia Hannachi, and M. Nacer Guetteche, "Review of the Rebound Hammer Method Estimating Concrete Compressive Strength on Site," *Proceedings of International Conference on Architecture And Civil Engineering*, Dubai, pp. 118-127, 2014. [CrossRef] [Google Scholar] [Publisher Link]

[15] Dora Silveira et al., "Mechanical Properties of Adobe Bricks in Ancient Constructions," *Construction and Building Materials*, vol. 28, no. 1, pp. 36-44, 2012. [CrossRef] [Google Scholar] [Publisher Link]

[16] Dora Silveira, Humberto Varum, and Aníbal Costa, "Influence of the Testing Procedures in the Mechanical Characterization of Adobe Bricks," *Construction and Building Materials*, vol. 40, pp. 719-728, 2013. [CrossRef] [Google Scholar] [Publisher Link]

[17] Qiang Xue et al., "The Draft Code for Performance-Based Seismic Design of Buildings in Taiwan," *Engineering Structures*, vol. 30, no. 6, pp. 1535-1547, 2008. [CrossRef] [Google Scholar] [Publisher Link]

[18] Yasser E. Ibrahim, and Mostafa M. El-Shami, "Seismic Fragility Curves for Mid-Rise Reinforced Concrete Frames in Kingdom of Saudi Arabia," *The IES Journal Part A: Civil & Structural Engineering*, vol. 4, no. 4, pp. 213-223, 2011. [CrossRef] [Google Scholar] [Publisher Link]

[19] Association of Structural Engineers of the Philippines (ASEP), *National Structural Code of the Philippines*, 7th ed., Quezon City, Philippines, 2016. [Publisher Link]

[20] Shan Wu, "Statistical Analysis on Concrete Strength," *Modern Civil and Structural Engineering*, vol. 2, no. 4, pp. 63-71, 2018. [CrossRef] [Google Scholar] [Publisher Link]

[21] Mohd. Zameeruddin Mohd. Saleemuddin, and Keshav K. Sangle "Seismic Damage Assessment of Reinforced Concrete Structure using Non-Linear Static Analyses," *KSCE Journal of Civil Engineering*, vol. 21, pp. 1319-1330, 2016. [CrossRef] [Google Scholar] [Publisher Link]

[22] Yishuo Huang et al., "Reinforced Concrete Beams Retrofitted with UHPC or CFRP," *Case Studies in Construction Materials*, vol. 17, pp. 1-11, 2022. [CrossRef] [Google Scholar] [Publisher Link]

[23] Mohamadreza Delzendeh Moghadam, Abbas Fathi, and Omar Chaallal, "Retrofitting of Steel Structures with CFRP: Literature Review and Research Needs," *Applied Sciences*, vol. 14, no. 13, pp. 1-20, 2024. [CrossRef] [Google Scholar] [Publisher Link]

[24] Giorgia Cianchino et al., "An Overview of the Historical Retrofitting Interventions on Churches in Central Italy," *Applied Sciences*, vol. 13, no. 1, pp. 1-32, 2022. [CrossRef] [Google Scholar] [Publisher Link]

[25] Emrah Meral, Bayram Tanik Cayci, and Mehmet Inel, "Comparative Study on the Linear and Nonlinear Dynamic Analysis of Typical RC Buildings," *Construction Magazine*, vol. 23, no. 3, pp. 587-607, 2024. [CrossRef] [Google Scholar] [Publisher Link]

[26] Ahmet Demir, Ali Haydar Kayhan, and Mehmet Palanci, "Response- and Probability-Based Evaluation of Spectrally Matched Ground Motion Selection Strategies for Bi-Directional Dynamic Analysis of Low- to Mid-Rise RC Buildings," *Structures*, vol. 58, 2023. [CrossRef] [Google Scholar] [Publisher Link]

[27] Sergio Lagomarsino, and Serena Cattari, "PERPETUATE Guidelines for Seismic Performance-Based Assessment of Cultural Heritage Masonry Structures," *Bulletin of Earthquake Engineering*, vol. 13, pp. 13-47, 2014. [CrossRef] [Google Scholar] [Publisher Link]

[28] Marco Valente, "Earthquake Response and Damage Patterns Assessment of Two Historical Masonry Churches with Bell Tower," *Engineering Failure Analysis*, vol. 151, pp. 1-23, 2023. [CrossRef] [Google Scholar] [Publisher Link]

[29] Marco Valente, "Seismic Vulnerability Assessment and Earthquake Response of Slender Historical Masonry Bell Towers in South-East Lombardia," *Engineering Failure Analysis*, vol. 129, 1-29, 2021. [CrossRef] [Google Scholar] [Publisher Link]

[30] Patryk Kot et al., "Recent Advancements in Non-Destructive Testing Techniques for Structural Health Monitoring," *Applied Sciences*, vol. 11, no. 6, pp. 1-28, 2021. [CrossRef] [Google Scholar] [Publisher Link]

[31] Nuria Chiara Palazzi et al., "Preliminary Assessment on Seismic Vulnerability of Masonry Churches in Central Chile," *International Journal of Architectural Heritage*, vol. 14, no. 6, pp. 829-848, 2020. [CrossRef] [Google Scholar] [Publisher Link]

[32] H.C. Penarubia et al., "Probabilistic Seismic Hazard Analysis Model for the Philippines," *Earthquake Spectra*, vol. 36, no. 1, pp. 44-68, 2020. [CrossRef] [Google Scholar] [Publisher Link]