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Seismic Vulnerability Assessment of Adamson University Buildings' As-Built using Fragility Curves

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Adamson University

Abstract- Adamson University buildings' age ranges from 10 years to 86 years. The structures' age can be one of the factors of their vulnerability to seismic hazard. Possibility of having different damages after the event of seismic activities can be measured through structural modeling and subjecting the latter to earthquake simulation. In the field of structural reliability, fragility analysis can be used in the assessment of structures. This type of analysis can be carried out by taking the structural performance using nonlinear analyses: Pushover Analysis and Time History Analysis. Five (5) buildings in Adamson University were analyzed and modeled in a structural analysis software based from the developed as-built plans with rebound hammer test results for the compressive strength of concrete. These structural models were subjected through a total of 22 ground motion data (from both Philippine and Japan), with each ground motion data normalized from 0.1g to 1.0g of peak ground acceleration (PGA). The result of fragility analysis has some limitations such as the use of as-built plan of each building assessed. The lowest probability of exceedance of 9% at a PGA of 0.4g as per National Structural Code of the Philippines specification is based from the fragility curve of FRC Building under a "Complete Damage" or damage rank of "As".

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SEISMICVULNERABILITYASSESSMENTOFADAMSONUNIVERSITYBUILDINGSAS-BUILTUSINGFRAGILITYCURVES

Strictly as per the compliance and regulations of:

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Keywords: fragility curve, pushover analysis, time history analysis, probability ofexceedance.

I. Introduction

In the webpage of the Philippine Institute of Civil Engineers, it was stated in one of its Fundamental Canons that: "Civil Engineers shall hold paramount the safety, health and welfare of the public and shall strive to comply with the principles of sustainable development in the performance of their duties (Philippine Institute of Civil Engineers Inc., 2018)." Hazards, vulnerability, and risks affect the safety of a structure and it is a challenge to civil engineers to assess those factors to existing and developing buildings. Furthermore, hazard is the possibility that a damaging phenomenon will likely to occur. Vulnerability is the area or extent that will be subjected to damages after an occurrence of a hazard. The expected losses from the occurrence of a hazard including the property damages, injuries, and loss of lives are the risks.

Nowak & Collins (2000) developed several methods in the analysis of structural reliability. These reliability methods are Monte Carlo Simulation (MCS), Latin Hypercube (LHC), First Order Reliability Method (FORM), Second Order Reliability Method (SORM), Hasofer-Lind Method of reliability index.

Requiso (2013) further enhanced his fragility curves by using nonlinear static analysis and nonlinear dynamic analysis. Two years after, Requiso's research was enhanced further the development of fragility curves by using the two aforementioned nonlinear analysis plus the application of interval analysis (Baylon, 2015). It is notable that the sophistication of these assessments increases the accuracy of the results. The studies of Baylon were followed by more seismic vulnerability assessments in the succeeding years.

One of the major fault lines in the Philippines is the Valley Fault System, which is composed of two sections: the 10km East Valley Fault and the 100km West Valley Fault. The West Valley Fault System (WVFS) traverses through a large portion of Metro Manila which could definitely endanger the lives of people, infrastructures, and buildings. For the past 1400 years, the West Valley Fault has moved 4 times and has a movement interval of 400 years. The last recorded movement of WVFS was on 1658 (Solidum Jr., 2015). The PHIVOLCS warned Metro Manila that the WVFS is ripe for generating a devastating magnitude of no less than 7.2 earthquake in the Richter scale or also called as "The Big One". According to PHIVOLCS, the said earthquake could be experienced in the near future. For that reason, the existing buildings must be assess for structural integrity (PHIVOLCS, JICA, & MMDA, 2004).

The buildings and facilities of Adamson University will be definitely affected if the "Big One" occurs. Using the PHIVOLCS' Fault Finder App, the nearest distance to the West Valley Fault System of Adamson University is 8.9 km as of 2013 mapped, as can be seen in Fig. 1.1 These buildings will be susceptible to such level of damage. As of this writing, a group senior civil engineering students conducted

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seismic assessment of Adamson University buildings with the use of "as-built" plans. The assessment from this study will help provide a seismic evaluation and assess the vulnerability of the structures.

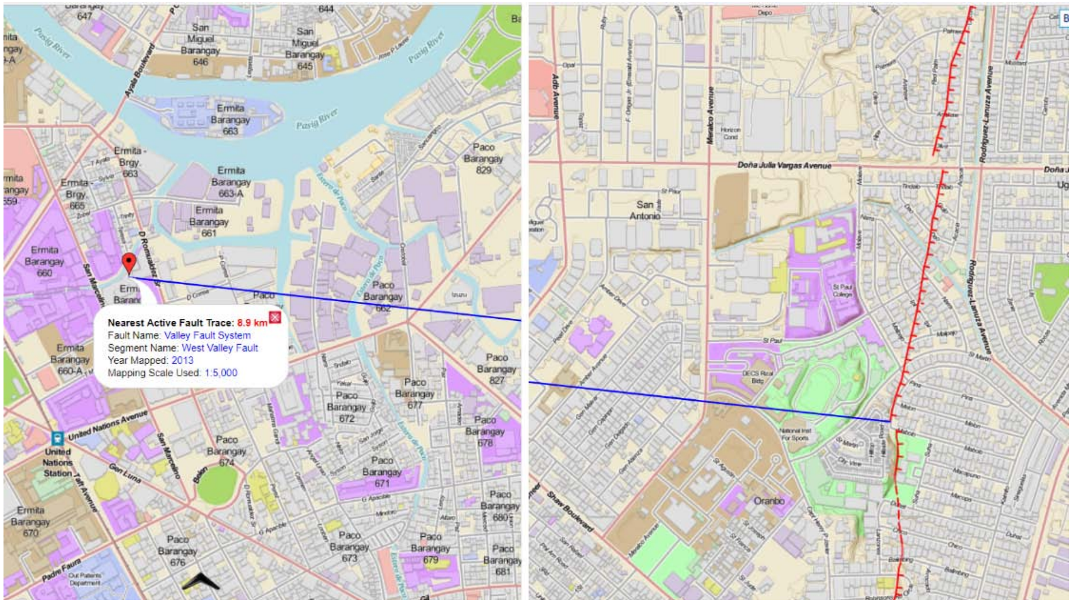


Fig. 1.1 : The nearest distance of the study from the immediate fault, i.e., the West Valley Fault, using PHIVOLCS Faultfinder App.

This study aimed to assess the seismic vulnerability of Adamson University buildings using its As-Built plan and Seismic fragility curve. Specifically, develop a set of as-built plan of the structure, generate sets of fragility curves by determining the damage indices and ranks of the structure from push over analysis and time history analysis using the following parameter: Displacement Ductility, Ultimate Ductility and Hysteretic Energy Ductility, provide with the vulnerability assessment of the structure by using the derived set of fragility curves, and evaluate the different derived seismic fragility curves of Adamson University buildings.

The scope of this study is to assess seismic vulnerability of Adamson University buildings. In Table 1.5, five (5) of the buildings of Adamson University was assessed.

Table 1: Adamson University Buildings

	Name of Building	Year Erected	Main Structural Form
1.	CS Annex (Meralco Building)	1990	Reinforced Concrete Framing with Steel Roof trusses
2.	Cardinal Santos Building (formerly Main Meralco Bldg)	GF & 2F 1936 3F & 4F 1967	Reinforced Concrete Framing with Timber Roof trusses
3.	John Peyboyre Building	1990	Reinforced Concrete Framing with Steel trusses
4.	Francis Regis Clet Building	2000, 2002, 2012	Reinforced Concrete Framing with Roofdeck
5.	Saint Vincent Building	1932	Reinforced Concrete Framing with Timber Roof Trusses

Using Microsoft Excel and Matlab, the researchers obtained the parameters needed in deriving fragility curves. Furthermore, this study limited the seismic assessment methods used in creating fragility curves to Non-linear static analysis (Push Over Analysis) and Non-linear dynamic analysis (Time History Analysis) considering only shear as the mode of failure. Additionally, the structural performance of Adamson University buildings was evaluated under the effects of numerous normalized peak ground acceleration.

The ground motion data were acquired from Incorporated Research Institutions for Seismology (IRIS), the following data that used were listed below:

Table 2: Ground motion Data

Province of Epicenter Location	Date	Magnitude	PGA in g
Batangas	8 April 2017	6.0	0.20
Bohol	15 October 2013	7.2	0.22
Mindoro	15 November 1994	7.1	0.26
Moro Gulf	17 August 1976	8.1	0.50
North Luzon (Baguio)	16 July 1990	7.7	0.72
The Great Hanshin	17 January 1995	6.9	0.82
Ragay Gulf	17 March 1973	7.0	0.95
The Great Tohoku Kanto	11 March 2013	9.0	2.99

Other earthquakes after effects were not within the scope of the study. The basis for selecting these ground motion data was the relative location of earthquakes. This study was also be inclusive of probable cost of damage; reconstruction costs and retrofitting costs. This portion of the study will fall under Net Present Value Approach.

The structural model of Adamson University Buildings was based from As-Built Plans. These plans will be obtained by conducting As-Built/Field Surveys using surveying materials from Adamson University Engineering Department. The resulted As-Built Plans and the ground motion data, which will be acquired from the databases of PHIVOLCS and IRIS, will be used for the generation of seismic analysis.

In connection to modeling of structures, and performing the Nonlinear Static Analysis (Push-Over Analysis) and the Nonlinear Dynamic Analysis (Time History Analysis) the structural software to be used are SAP2000 to be able to determine the parameters needed such as; Ductility Factors, Damage Indices Factors, and Damage Rank, in attaining the fragility functions.

Thereupon, the data gathered from the analysis will then be used to finally plot the fragility curves. Using the fragility curves and by embodying the use of mathematical software, the seismic performance of the structure can be assessed.

II. Methodology

This study utilized the methods of Pushover Analysis (Non-linear static analysis) and Time History Analysis (Non-linear dynamic analysis). Requiso (2013) used the methods of Pushover Analysis (non-linear static analysis) in obtaining the yield displacement and the maximum displacement of the structure. Karim & Yamazaki (2001) used the methods of Time History Analysis (non-linear dynamic analysis) in obtaining the maximum displacement of the structure and for the derivation of the fragility curve. The software used for structural analysis, simulation and design was SAP2000.

The ground motion data used was acquired from the databases of PHIVOLCS (The Philippine Institute of Volcanology and Seismology) and IRIS (Incorporated Research Institutions for Seismology). PHIVOLCS is a national institution that provides information about volcanic activities, tsunamis, and earthquakes. This institution was mandated to alleviate disasters that may arise from the said activities. While IRIS, is an educational center and a multi-institutional research having investigators from several consulting companies, universities, researchers at different state that provide contributions to; a performance-based earthquake engineering, seismology, risk management and etc., research programs.

This research was designed as shown in Figure 3.1 which intent to generate sets of fragility curves by determining the different parameters from performing the Nonlinear Static Analysis (Push-Over Analysis) and Nonlinear Dynamic Analysis (Time History Analysis) using structural software, SAP 2000. The created As-Built plan was used as the basis for the structural model of Adamson University Buildings.

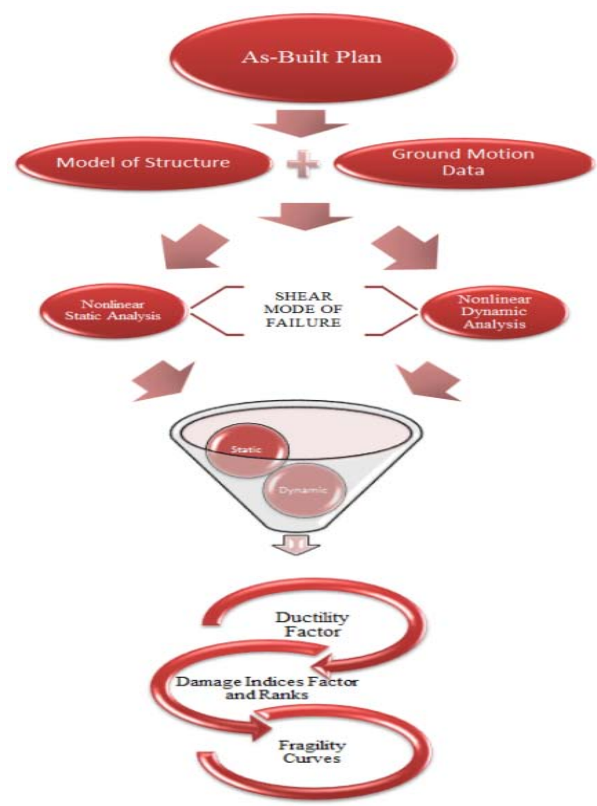


Fig. 3.1: Research Design

In every ground motion data's peak ground accelerations (PGA), the damage indices, one of the parameters, was obtained from 0.10g and 2.0g. In addition, the damage ranks were used as frequencies in solving the occurring probability for different peak ground acceleration (PGA) values.

The data gathered from the probability of occurrence and damage index were used to compute the mean and the standard deviation for the fragility analysis using the lognormal equation.

The seismic fragility curves were developed by plotting the cumulative lognormal probability versus the peak ground acceleration for every damage level.

In forming the As-Built plan for this study. The researchers used unique tools such as Total Station, a surveying measuring tool needed for the determination of the measurements needed for the As-Built Plan and Rebound Hammer to determine the compressive strength of the concrete used in the structure.

Consequently, the researcher used engineering software, Microsoft Excel, SAP2000, AutoCAD, MATLAB to generate the data and a 3D Model of the structure needed for the development of fragility curve.

A Schmidt hammer, also known as a Swiss hammer or a rebound hammer, is a device to measure the elastic properties or strength of concrete or rock, mainly surface hardness and penetration resistance. Ernst Schmidt, a Swiss engineer, invented it. Proceq and TQC worldwide distributed the Schmidt hammer. The hammer measures the rebound of a spring-loaded mass impacted against the surface of the sample. The test hammer will hit the concrete at a defined energy. Its rebound is dependent on the hardness of the concrete and measured by the test equipment. By reference to the conversion chart, the rebound value used to determine the compressive strength. When conducting the test, the hammer was held at right angles to the surface, which in turn should be flat and smooth. The rebound reading affects the orientation of the hammer, when used in a vertical position (on the underside of a suspended slab for example) gravity will increase the rebound distance of the mass and vice versa for a test conducted on a floor slab. The Schmidt hammer is an arbitrary scale ranging from 10 to 100. Schmidt hammers are available from their original manufacturers in several different energy ranges. These include: (i) Type L-0.735 Nm impact energy, (ii) Type N-2.207 Nm impact energy; and (iii) Type M-29.43 Nm impact energy.

The test is also sensitive to other factors:

- Local variation in the sample. To minimize this, it was recommended to take a selection of readings and take an average value.
- Water content of the sample, a saturated material gives different results from a dry one.

Prior to testing, the Schmidt hammer was calibrated using a calibration test anvil supplied by the manufacturer for that purpose. Twelve readings should be taken, dropping the highest and lowest, and then the average of the ten remaining has been taken. Using this method of testing classed as indirect as it does not give a direct measurement of the strength of the material. It simply gives an indication based on surface properties, it is only suitable for making comparisons between samples. This test method for testing concrete is governed by ASTM C805. ASTM D5873 describes the procedure for testing of rock.

In evaluation of the fragility curves, it was observed that there was an increase in every damage rank from various peak ground acceleration. It was also seen if there were a low, moderate or high possibility of structure to be damaged at 0.4g peak ground acceleration as the requirement of the National Structural Code of the Philippines. This also suggest if the structure can resist the different ground motion since a larger earthquake shaking is required to cause a significant damage.

This also generate a seismic report showing the possible damage that might occur within the structure. There is a recommending retrofitting scheme if the structure was subjected to 0.4g ground motion data considered it a high probability of exceedance of damage.

As shown in the Figure, the as built plan was first to establish with the used of as-built plans survey and non-destructive test to assess the seismic vulnerability of the building. These as-built plans were used in deriving fragility curves. Thereafter, the assessment of seismic vulnerability of the building has been performed.

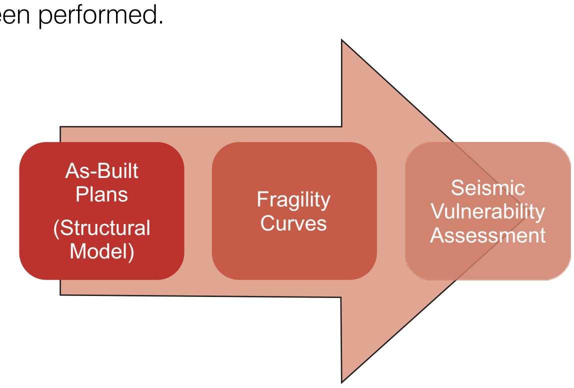


Fig. 3.2.1.1: Research Design of As-Built Structural Modeling

The structural model of Adamson University Buildings is subjected to both nonlinear dynamic analysis and nonlinear static analysis to assess the structure by attaining the fragility curves. In this study, the ratio of normalized peak ground acceleration and the original peak ground acceleration shall be multiplied to the ground motion records. The Normalized Ground

Motion data is scaled up or down from the original ground motion data. (Karim & Yamazaki, 2001).

$$\ddot{u}_{NEW} = A_0\ddot{u}_{SOURCE} \quad (3.1)$$

Where:

$\ddot{u}_{NEW}=A_0\ddot{u}_{SOURCE}$ = the normalized ground motion data

$\ddot{u}_{SOURCE}$ = the source ground motion data

$A_0 = \frac{PGA_{normalized}}{PGA_{source}}$ = a coefficient factor to normalize the source of ground motion

The data gathered from two nonlinear analysis were used in determining the maximum displacement for the static and dynamic case. Furthermore, with the use of SAP2000, the yield displacement for static and dynamic case was determined. Moreover, the results was used for the computation of ductility factors by the following formulas (Karim & Yamazaki, 2001):

Displacement Ductility (μd):

$$\mu_d = \frac{\delta_{\max}^{dynamic}}{\delta_y} \quad (3.2)$$

where:

$\delta_{max(Dynamic)}$ = maximum displacement at the hysteresis model (dynamic)

δ_y= yield displacement from the push-over curve (static)

Ultimate Ductility (μ_u):

$$\mu_u = \frac{\delta max\ (static)}{\delta_{y\ (static)}} \quad (3.3)$$

where:

$\delta_{max(static)}$= displacement at maximum reaction at the push over curve (static)

δ_y= yield displacement from the push-over curve (static)

Hysteretic Energy Ductility (μ_h):

$$\mu_h = \frac{E_h}{E_e} \quad (3.4)$$

Where:

E_h = hysteretic energy (area under the hysteresis model)

E_e= yield energy (area under the push-over curve (static) but until yield point)

The damage indices were calculated and calibrated to their respective rank with the used of the computed values of ductility factors from the equations (3.2),(3.3), and (3.4). The computed values of damage indices was used to obtain the number of occurrence of each damage rank displayed in Figure 3.4.5.1. The formula for the computation of Damage Index was shown in equation (3.5).

$$DI = \frac{\mu_d + \beta\mu_h}{\mu_u} \quad (3.5)$$

where:

β – Cyclic Loading Factor taken as 0.10 for buildings

Table 3.4.5.1: Damage Index and Damage Rank Relationship

(HAZUS, 2003)

Damage Index (DI)	Damage Rank (DR)	Definition
0.00 < DI ≤ 0.14	D	No damage
0.14 < DI ≤ 0.40	C	Slight damage
0.40 < DI ≤ 0.60	B	Moderate damage
0.60 < DI ≤ 1.00	A	Extensive damage
DI ≥ 1.00	As	Complete damage

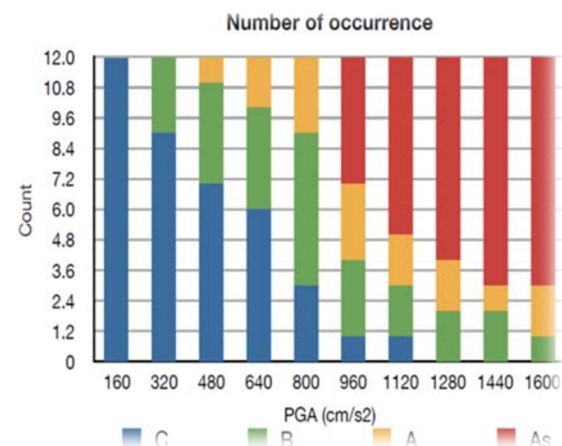


Fig. 3.4.5.1: Number of occurrence in each damage rank (Requiso, 2013)

The damage ratio is the number of occurrence of each damage rank (no, slight, moderate, extensive, and complete) divided by the total number of records. It was plotted against the natural logarithm *(ln)* of (PGA) to determine the mean and standard deviation that will be used to construct the fragility curves.

The parameters such as the mean and standard deviation were needed to construct the fragility curves. These parameters were obtained by plotting the value of damage ratio against the natural logarithm of PGA on a lognormal probability paper. Upon obtaining the values of the standard deviation and mean, equation (3.6) was used to compute for the cumulative probability of occurrence (Pr) of the damage equal or higher than the damage rank.

$$Pr = \Phi[\frac{\{\ln(X) - \lambda\}}{\xi}] \quad (3.6)$$

Where:

Pr – Cumulative Probability

Ø – Standard Normal Distribution

X – Peak Ground Acceleration

λ – Mean

ξ – Standard Deviation

Maximum likelihood estimation is a method that determines values for the parameters of a model. The parameter values are found such that they maximize the likelihood that the process described by the model produced the data that were actually observed (Brooks-Bartlett, 2018).

The probability density of observing a single data point x, which is generated from a Gaussian distribution is given by:

$$P(x;\mu,\sigma)=\frac{1}{\sigma\sqrt{2\pi}}\exp\left(-\frac{(x-\mu)^2}{2\sigma^2}\right)$$

The semi colon used in the notation P(x; μ, σ) is there to emphasize that the symbols that appear after it are parameters of the probability distribution. So it shouldn't be confused with a conditional probability (which is typically represented with a vertical line e.g. P(A| B)).

This family of distributions has two parameters: $\theta=(\mu,\sigma)$; so we maximize the likelihood, $\mathcal{L}(\mu,\sigma)=f(x_1,\ldots,x_n\mid\mu,\sigma)$, over both parameters simultaneously, or if possible, individually.

Since the logarithm function itself is a continuous strictly increasing function over the range of the likelihood, the values which maximize the likelihood will also maximize its logarithm (the log-likelihood itself is not necessarily strictly increasing). The log-likelihood can be written as follows:

$$\log\Big(\mathcal{L}(\mu,\sigma)\Big)=-\frac{n}{2}\log(2\pi\sigma^2)-\frac{1}{2\sigma^2}\sum_{i=1}^{n}(x_i-\mu)^2$$

(Note: the log-likelihood is closely related to information entropy and Fisher information.)

We now compute the derivatives of this log-likelihood as follows.

$$0=\frac{\partial}{\partial\mu}\log\Big(\mathcal{L}(\mu,\sigma)\Big)=0-\frac{-2n(\bar{x}-\mu)}{2\sigma^2}.$$

This is solved by

$$\hat{\mu}=\bar{x}=\sum_{i=1}^{n}\frac{x_i}{n}.$$

This is indeed the maximum of the function, since it is the only turning point in μ and the second derivative is strictly less than zero. Its expected value is equal to the parameter μ of the given distribution,

$$\mathrm{E}\left[\,\widehat{\mu}\,\right]=\mu,$$

which means that the maximum likelihood estimator $\widehat{\mu}$ is unbiased.

Similarly we differentiate the log-likelihood with respect to σ and equate to zero:

$$\begin{aligned}0&=\frac{\partial}{\partial\sigma}\log\left[\left(\frac{1}{2\pi\sigma^2}\right)^{n/2}\exp\left(-\frac{\sum_{i=1}^{n}(x_i-\bar{x})^2+n(\bar{x}-\mu)^2}{2\sigma^2}\right)\right]\\&=\frac{\partial}{\partial\sigma}\left[\frac{n}{2}\log\left(\frac{1}{2\pi\sigma^2}\right)-\frac{\sum_{i=1}^{n}(x_i-\bar{x})^2+n(\bar{x}-\mu)^2}{2\sigma^2}\right]\\&=-\frac{n}{\sigma}+\frac{\sum_{i=1}^{n}(x_i-\bar{x})^2+n(\bar{x}-\mu)^2}{\sigma^3}\end{aligned}$$

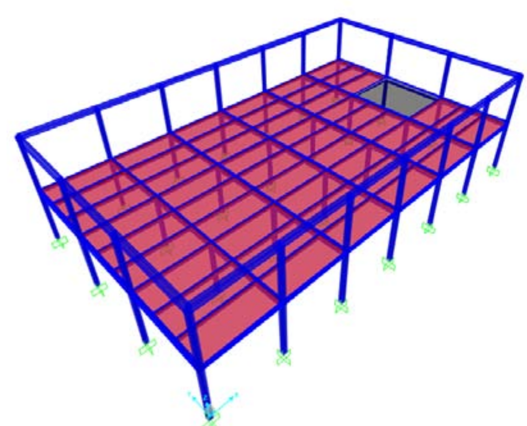
which is solved by

$$\widehat{\sigma}^2=\frac{1}{n}\sum_{i=1}^{n}(x_i-\mu)^2.$$

III. Results and Discussion

It shows the structural plans, structural models, and rendered model using AutoCAD and SAP200 of Adamson University Buildings. In this floor plan, the distances of every column are shown, also the total length of the part of the building. These distances are easily measured by laser meter and every measurement, dimensions of the column are also taken. Thicknesses of the walls are also measured by the use of steel tape. These processes were repeated room by room and in every floor until all the data were completed. Using the data gathered, all the floor plans were drawn by the use of AutoCAD 2013, and the as-built plan was already developed.

CS Annex (Meralco) Building:



Cardinal Santos Building:

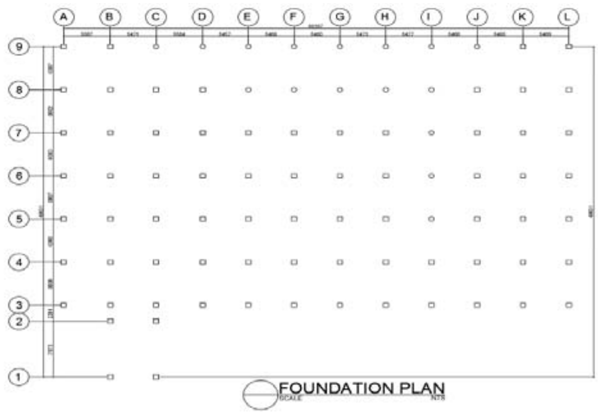


Fig. 4.1.5: Foundation Plan of CS Building

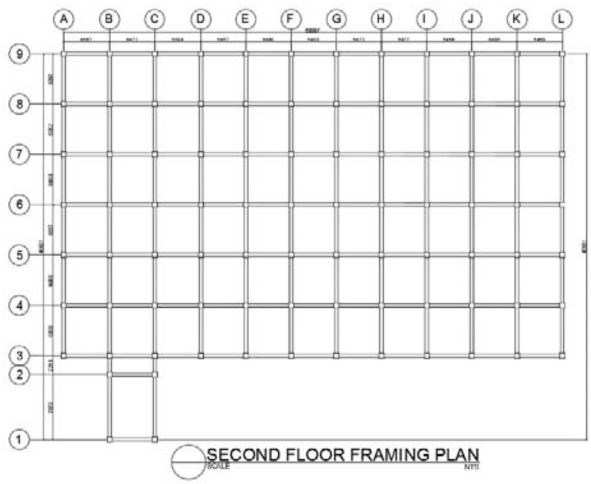


Fig. 4.1.6 2F: Plan of CS Building

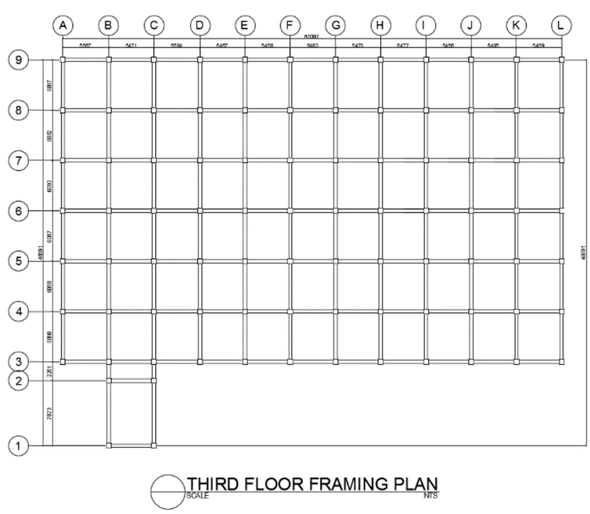


Fig. 4.1.7 3F: Plan of CS Building

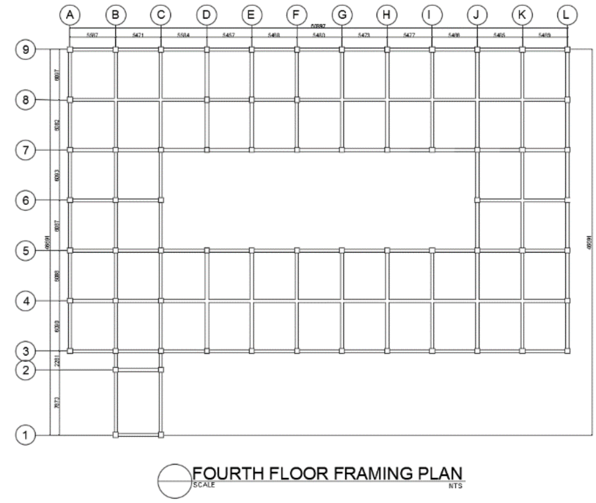


Fig. 4.1.8 4F: Plan of CS Building

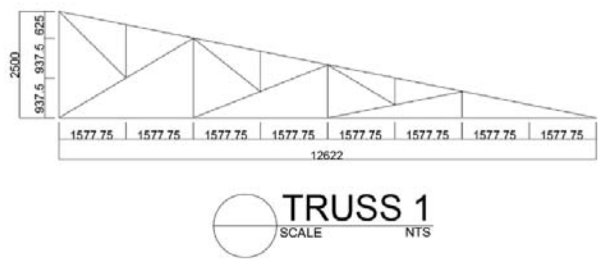


Fig. 4.1.9: Truss Elevation View

John Peyboyre Building:

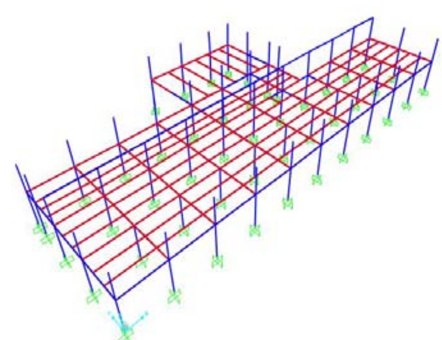


Fig. 4.1.10: Three – Dimensional Model of John Perboyre Building

Father Regis Clet Building:

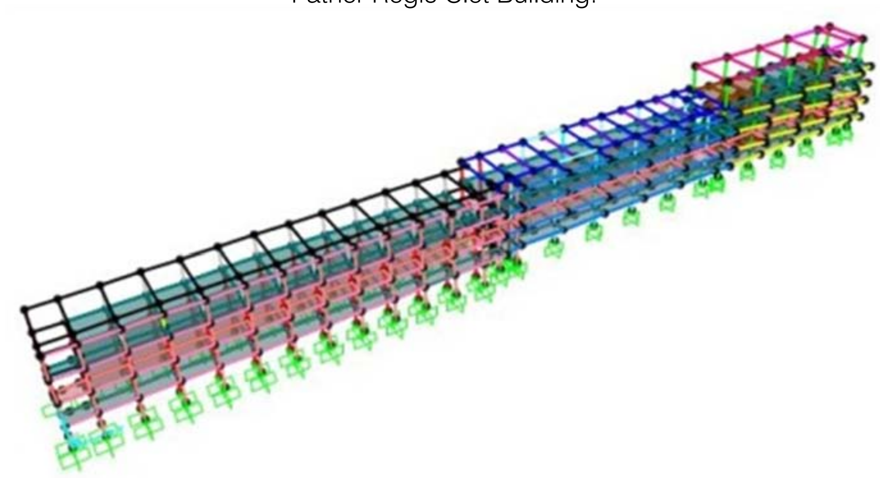


Fig. 4.1.26: Structural Model of FRC Building

Saint Vincent Building:

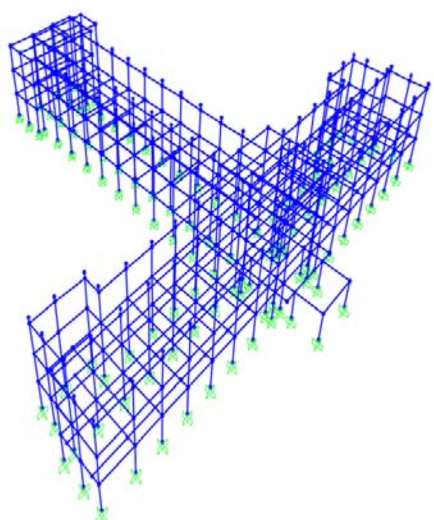


Fig. 4.1.27: Three – Dimensional Model of Saint Vincent Building

IV. Pushover Curve

This pushover curve represents the relationship between the displacement and the base force. The following figures show that the displacement and the base force are directly proportional. The straight line represents the limit of the structure to resist the maximum base force applied. The last point before the linear graph changes into a curved graph is the yielding point. And beyond that curved graph, the structure was completely damaged. The curved part in these graphs are not noticeable because the values and differences are small.

CS Annex (Meralco Building):

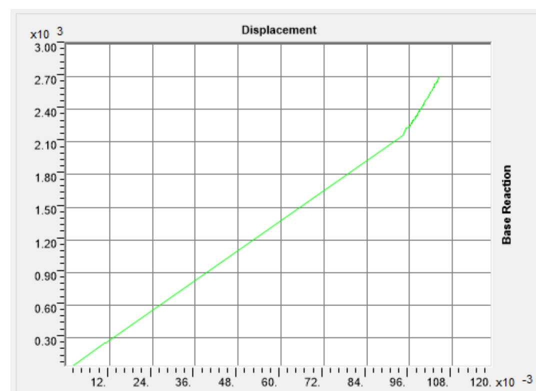


Fig. 4.2.1: Pushover Curve X-Direction

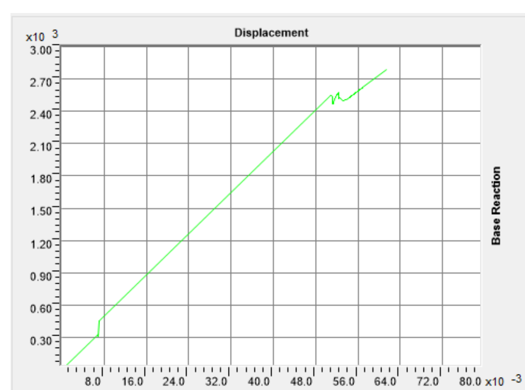


Fig. 4.2.2: Pushover Curve Y-Direction

Cardinal Santos Building:

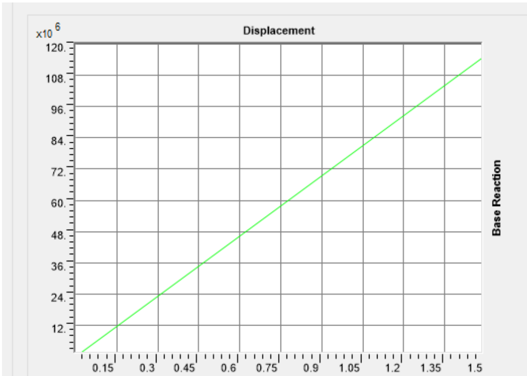


Fig. 4.2.3.: Pushover Curve (X-direction)

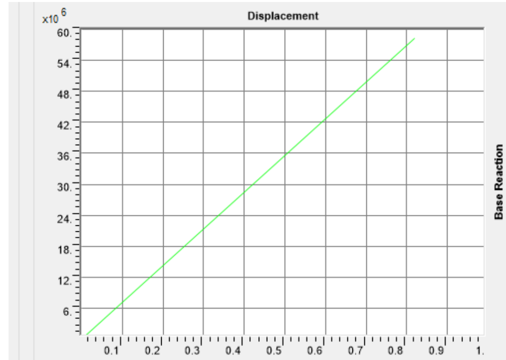


Fig. 4.2.4.: Pushover Curve (Y-direction)

John Peyboyre Building:

Fig. 4.2.5.: Pushover Curve X-direction

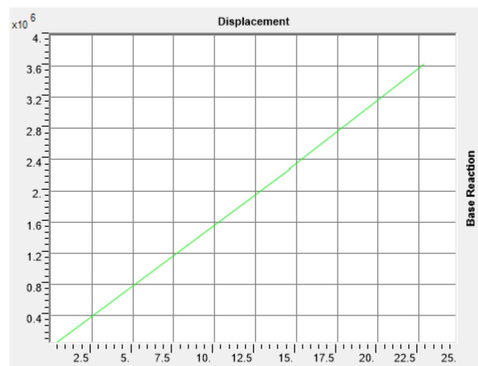


Fig. 4.2.6.: Pushover Curve Y-direction

FRC Building:

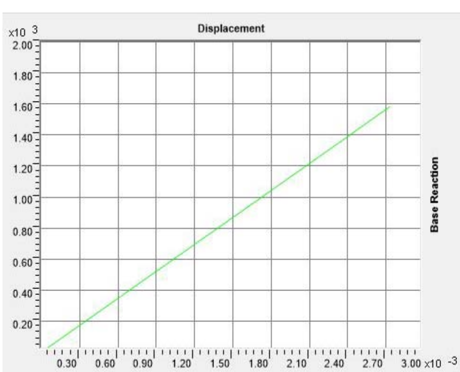


Fig. 4.2.7.: Pushover Curve in X - Direction

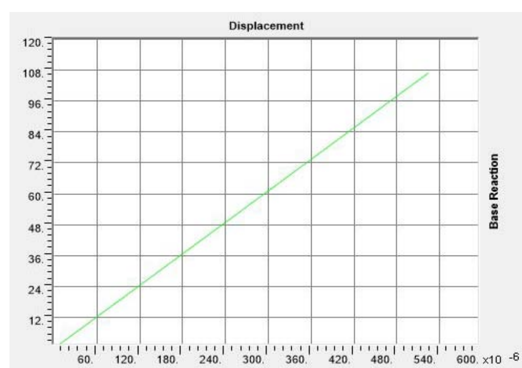


Fig. 4.2.8.: Pushover Curve in Y – Direction

Saint Vincent Building:

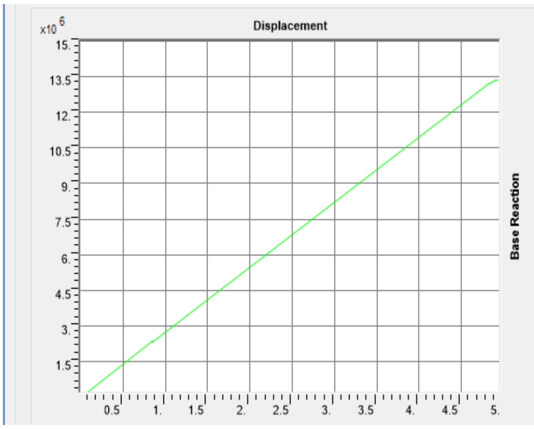


Fig. 4.2.9.: Sample Pushover Curve (X –axis)

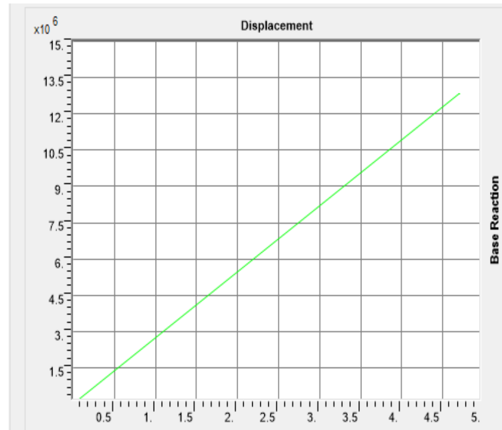


Fig. 4.2.9.: Sample Pushover Curve (Y –axis)

The following charts are the plots that demonstrates how the fitted fragility curves using MLE to find the statistical parameters for the lognormal distribution function. The red dots are the plot of the cumulative values of probability of occurrence based on the damage ratios.

CS Annex (Meralco) Building:

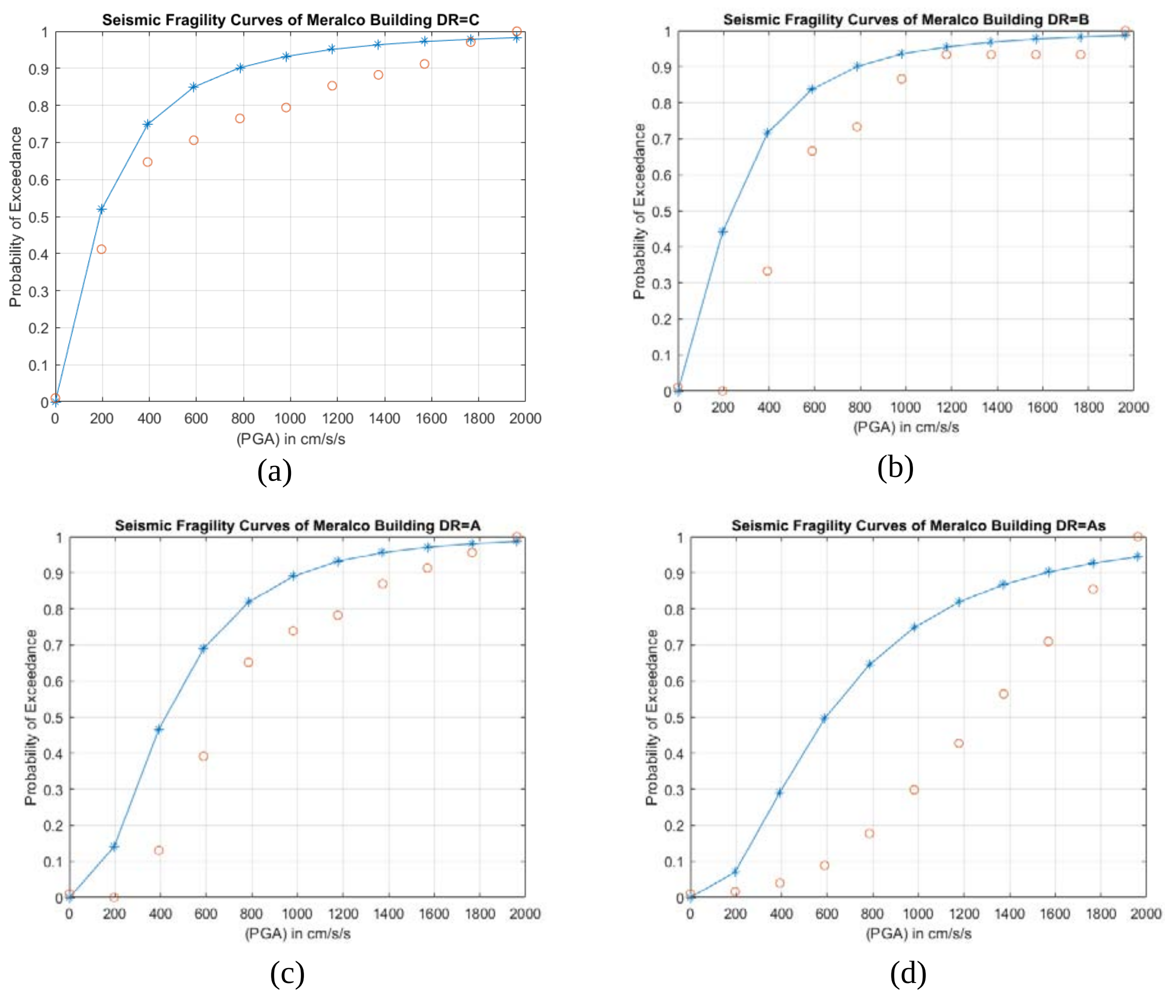
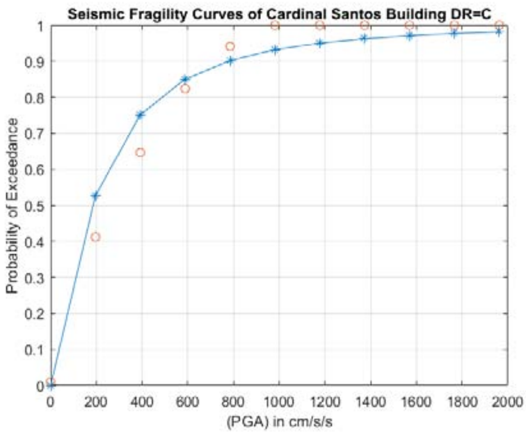
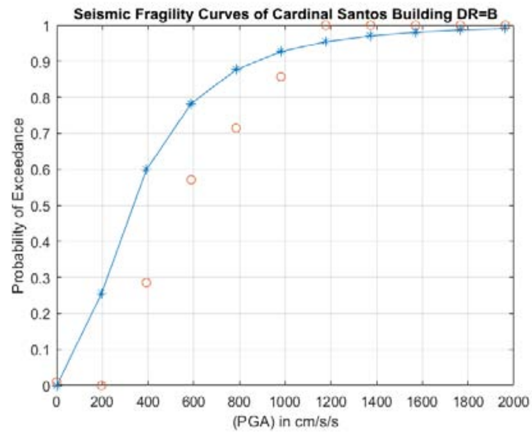


Fig. 4.5.1.1

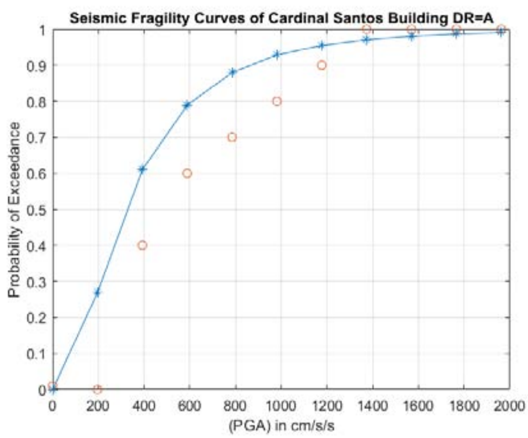
Cardinal Santos Building:



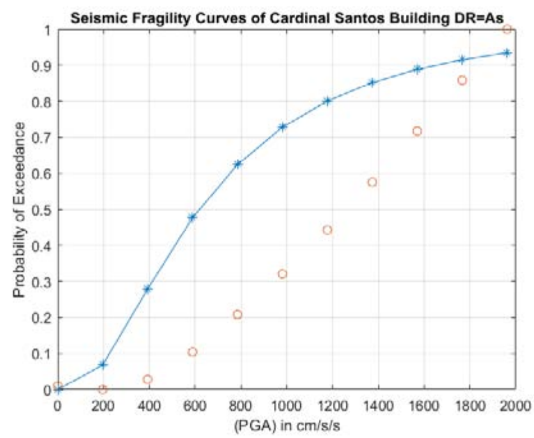
(a)



(b)



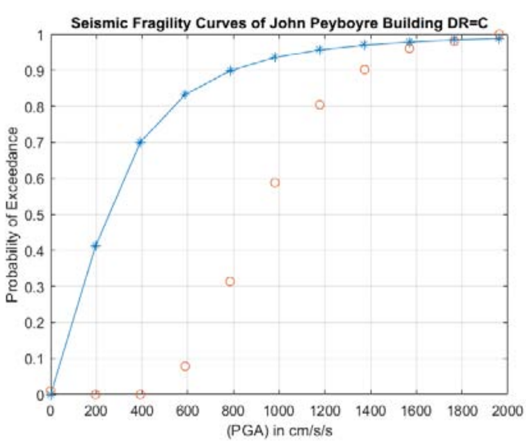
(c)



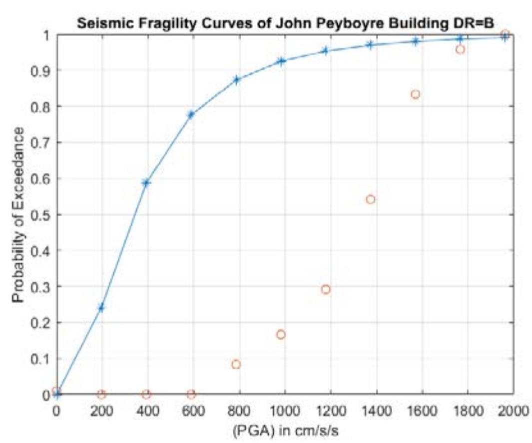
(d)

Fig. 4.5.1.2

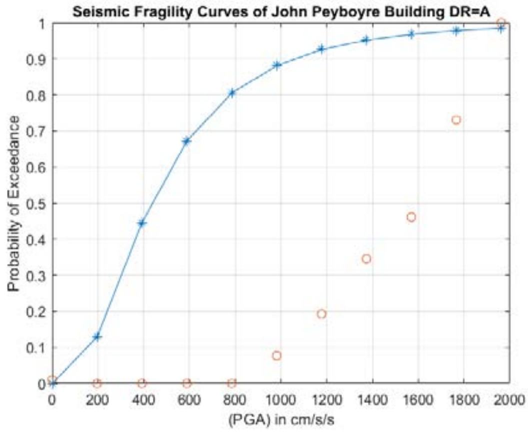
John Peyboyre Building:



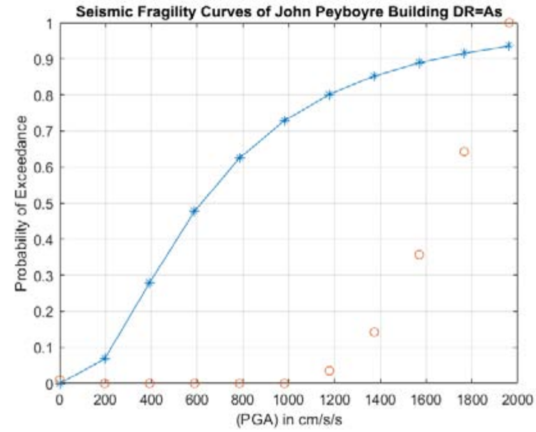
(a)



(b)



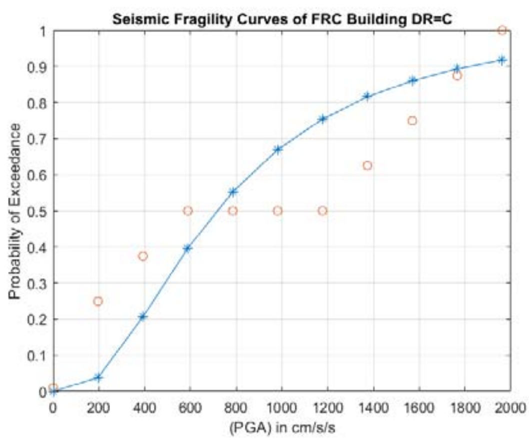
(c)



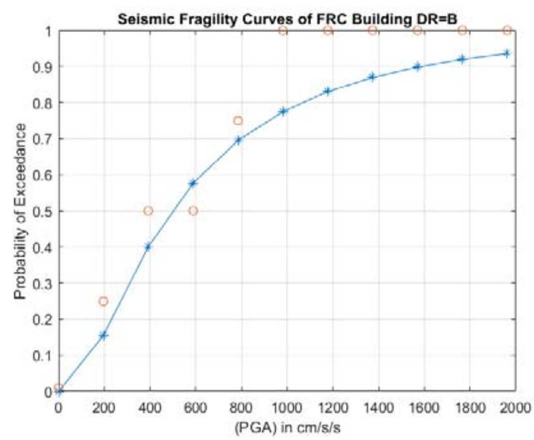
(d)

Fig. 4.5.1.3

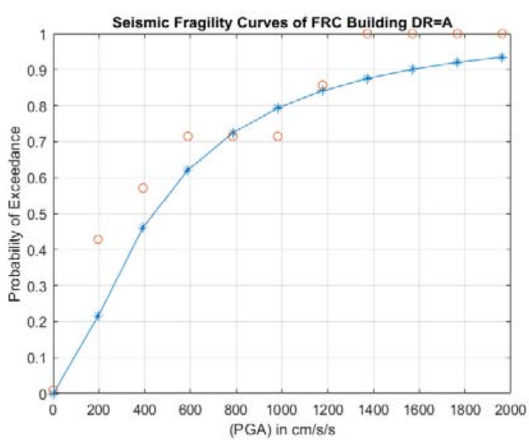
FRC Building:



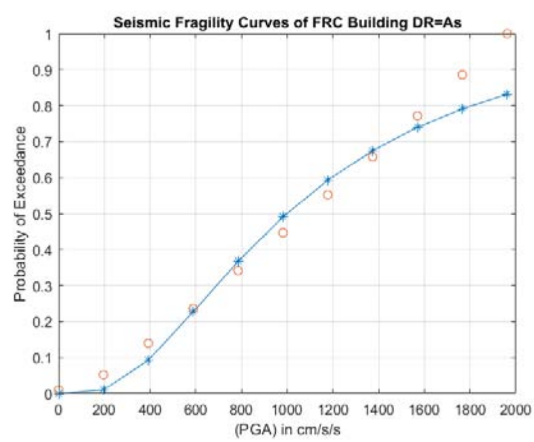
(a)



(b)



(c)



(d)

Fig. 4.5.1.4

Saint Vincent Building:

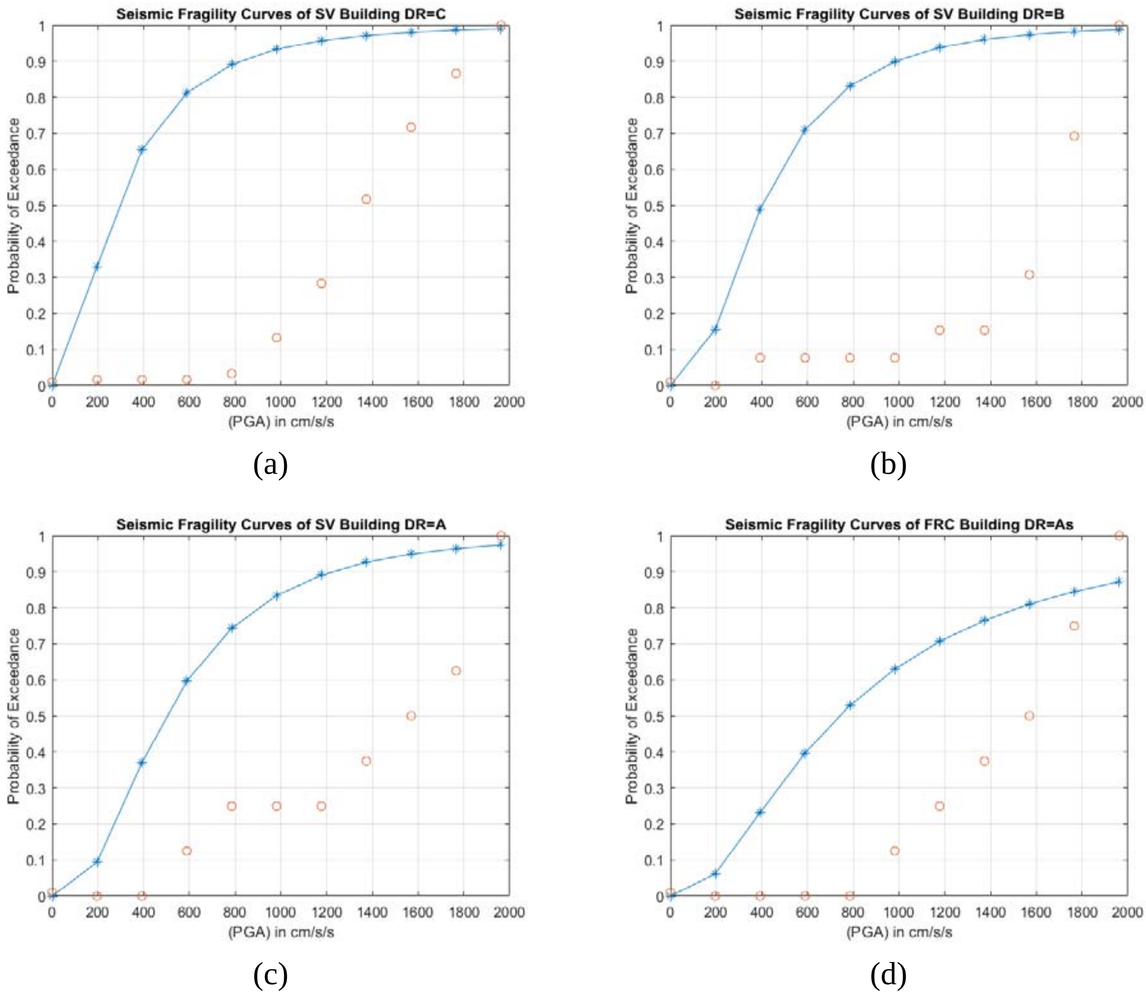


Fig. 4.5.1.5

The following charts are the plot of seismic fragility curves for each building. The different damage ranks can compared. In practice, the damage rank "D" or "No Damage" must not be included in the fragility analysis.

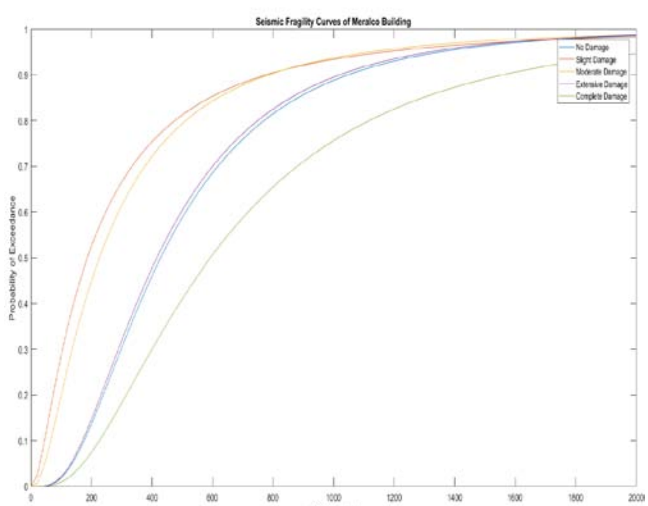


Fig. 4.5.2.1

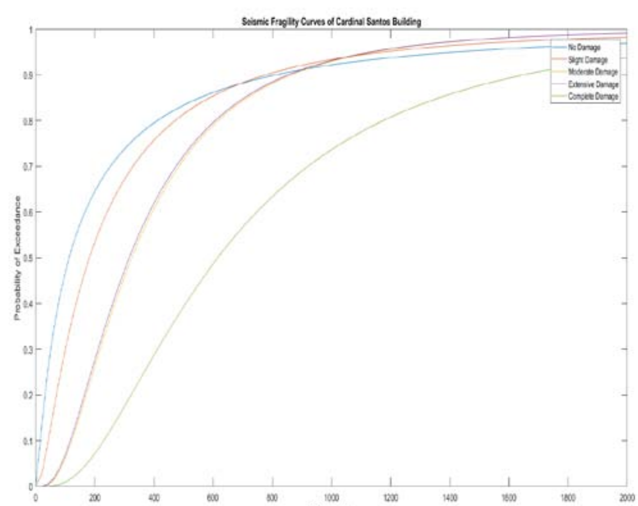


Fig. 4.5.2.2

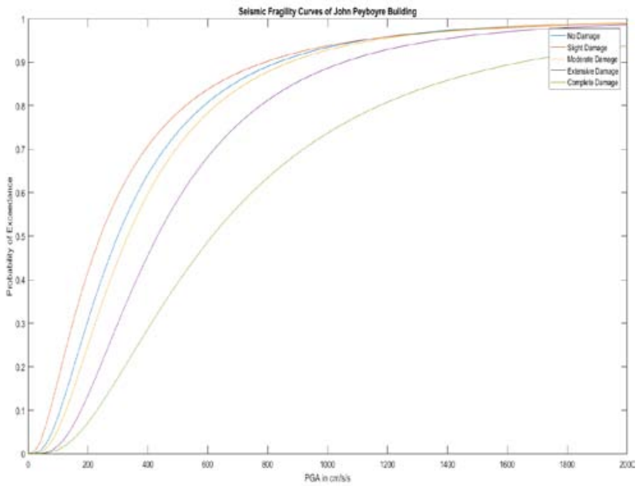


Fig. 4.5.2.3

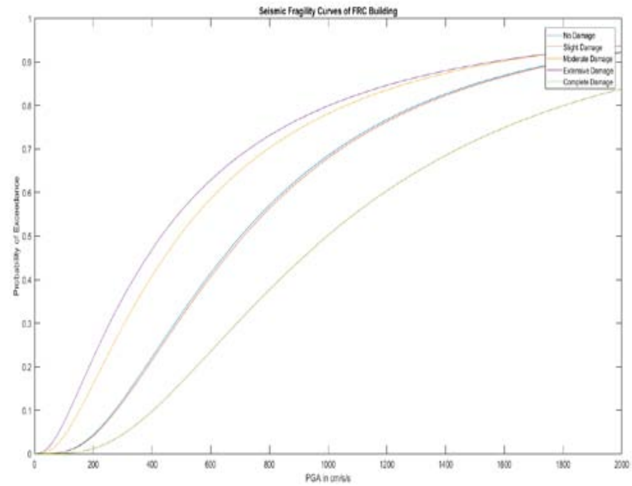


Fig. 4.5.2.4

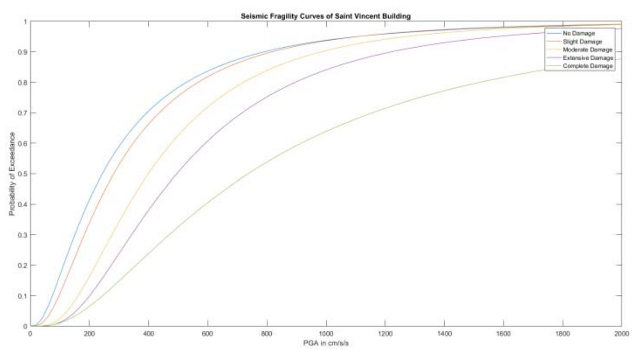


Fig. 4.5.2.5

In order to compare each building's seismic performance, the following charts are needed to which structure is evaluated as least and most resilient as per damage rank.

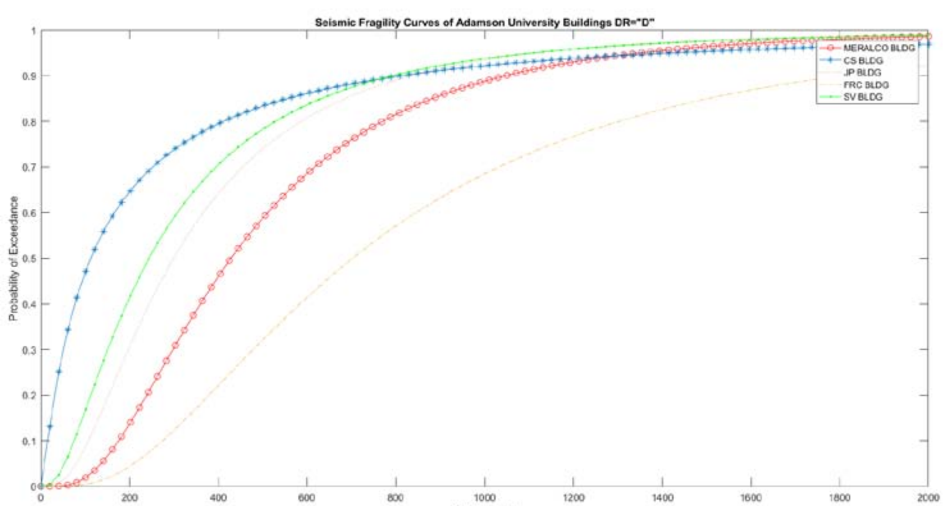


Fig. 4.5.3.1

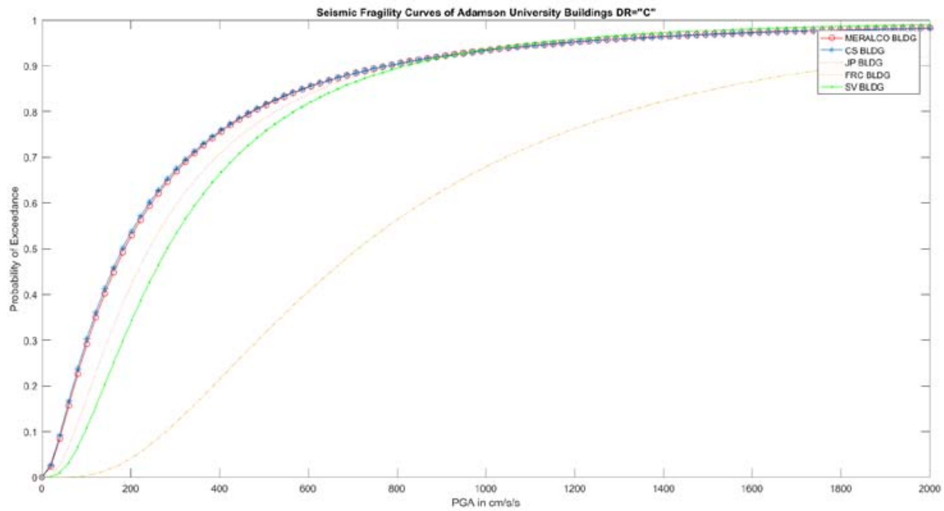


Fig. 4.5.3.2

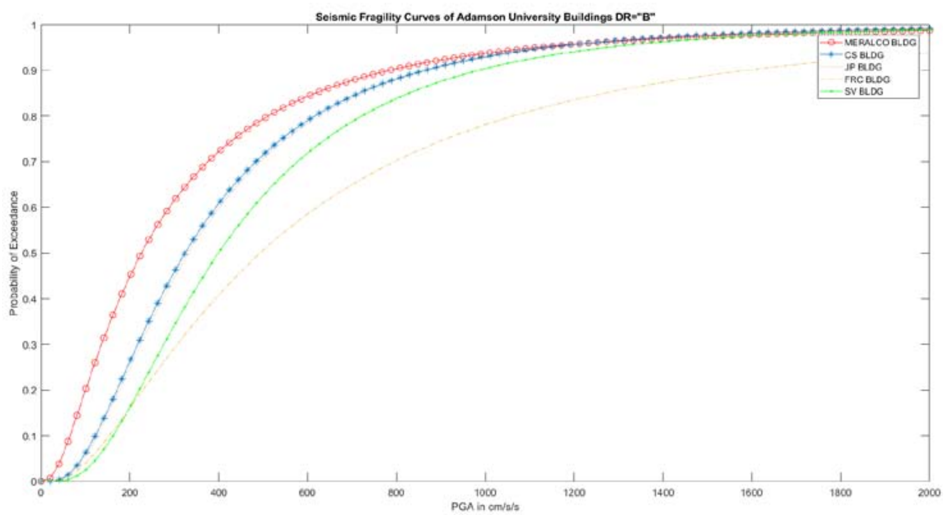


Fig. 4.5.3.3

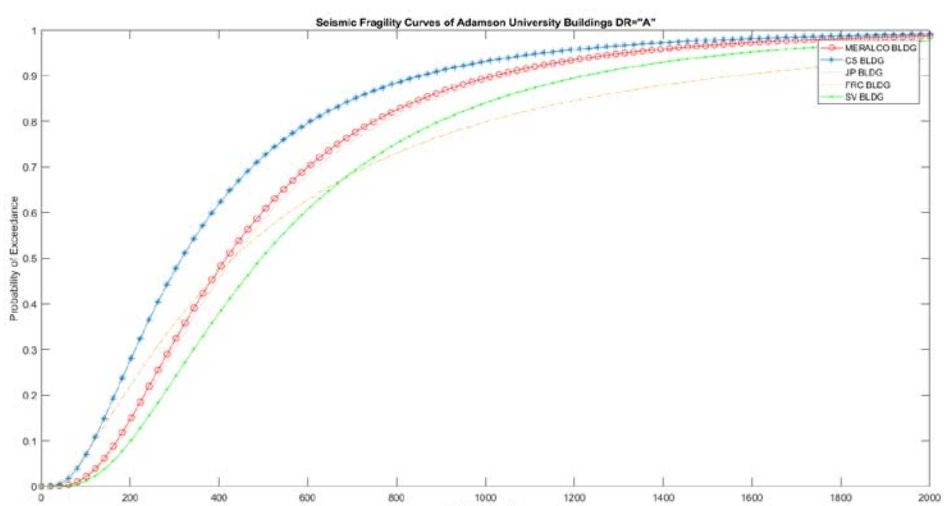


Fig. 4.5.3.4

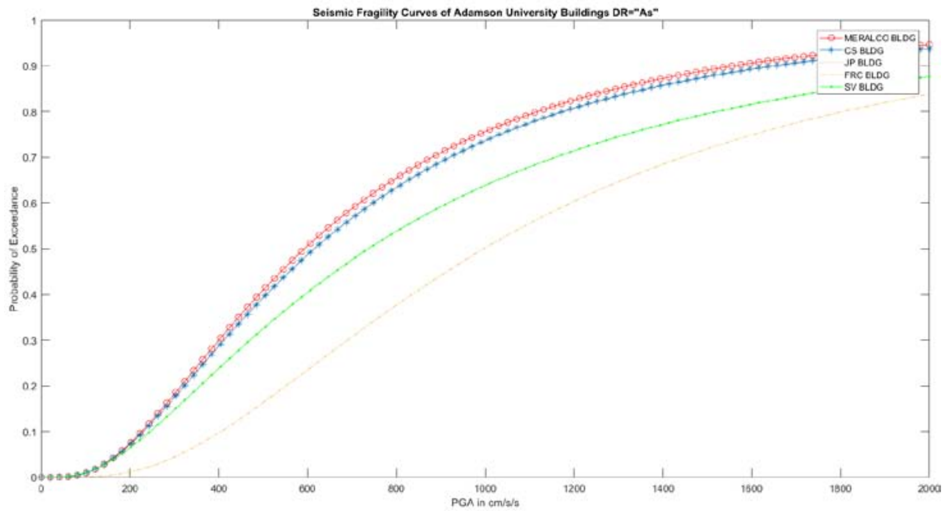


Fig. 4.5.3.5

The table summarizes the values of probability of exceedance at PGA=0.4g or 392.4 cm/s/s for each building per damage rank of "C" or "Slight Damage" to "As" or "Complete Damage".

Table 4.7.1: Summary of Probability of Exceedance of Adamson University Buildings for every Damage Rank

Building	C	B	A	As
CS Annex (Meralco) Building	74.85	71.53	46.66	29.17
CS Building	75.19	59.85	60.99	27.89
JP Building	70.03	58.72	44.58	27.93
FRC Building	20.68	40.07	46.15	9.29
SV Building	65.42	49.01	37.01	23.18

In section 2.3.2 of the National Structural Code of the Philippines (NSCP) Volume 1, the following figure is excerpted:

2.3.2 Seismic Hazard

Ground motion caused by earthquake generates impacts on the structural safety. Structures should be designed to resist the seismic ground motion. In the structural design defined by NSCP, the peak ground acceleration (PGA) with a 10% probability of being exceeded in 50 years (or annual exceedance probability of 0.2%), is defined as the ***basic design PGA***.

In the NSCP, seismic hazard is characterised by the seismic zone, proximity of the site to active seismic sources, site soil profile characteristics, and the structure importance factor. The two seismic zones described by NSCP are shown in Figure 2-4. Zone 2 covers only the provinces of Palawan (except Busuanga), Sulu and Tawi-Tawi, and the rest of the country is under Zone 4. The seismic zone factor, *Z*, is specified as follows:

- Zone 2: $Z = 0.2$ and
- Zone 4: $Z = 0.4$.

This means that PGAs with a 10 % probability of being exceeded in 50 years are 0.4 g and 0.2 g for the Zones 4 and 2, respectively. More details can be found in NSCP. The seismic hazard zone related to each local government is listed in Appendix A.

Fig. 4.7.1.: NSCP Specifications on 0.4g PGA for Zone 4

In addition, the Structural Engineers Association of California (SEAOC) has the following excerpt:

> "A structure with a 30 or more years of lifespan is NOT SAFE when subjected to a seismic event of 10% probability of exceedance of collapse or total damage. The structure being more than 50 years old is vulnerable to large magnitude earthquakes."

Based on the table and the SEAOC specifications, only the Father Regis Clet (FRC) Building can still withstand a 0.4g PGA and meets the code specs, that is, a probability of exceedance (POE) of 9.29%. The rest of the buildings are vulnerable to a 0.4g PGA, that is more than twice that of SEAOC specifications.

To summarize, these results indicate that the most probable damage that the structure will sustain will range from Extensive damage to Complete Damage which is significantly below the standards imposed as per NSCP 2010 which states that the structural integrity of the building must withstand 40% of the gravitational acceleration as the peak ground acceleration. Based from the resulting fragility curves, the structure is unsafe for occupancy when subjected to seismic activity, considering only base shear as the mode of failure.

V. Conclusion

The development of as-built plan is only limited in the concrete works of the building. The developed as-built plan includes of elevations, floor plans and other structural plans.

The accuracy of the As-built plans made is mainly dependent on two factors, namely the precision and accuracy of the equipment and human errors. Usage of laser measuring devices and available digital counterparts of surveying equipment yields more reliable and consistent values thus reducing any deviation from the actual measurements.

Based from the ductility parameters derived from undergoing nonlinear static and nonlinear dynamic analyses, all the Damage ranks that were observed in the fragility curve along the x-axis at its longitudinal section are No damage, Slight Damage, Moderate Damage, Extensive Damage and Complete Damage. The probability of sustaining Slight Damage is small until the 0.1g PGA which tends to increase well beyond it. The opposite was observed on the y-axis or transverse section of the building due to having a much steeper incline on the Slight Damage curve suggesting a high probability of transitioning to a higher damage state, making the structure more susceptible to damage when subjected to strong seismic activity along the transverse section.

From what can be gathered from the generated fragility curve, the building has a probability of exceedance for corresponding damage rank. In line with these, it can be inferred that the Adamson University Buildings did not meet the requirement that it must withstand a peak ground acceleration of about 40% of the gravitational acceleration as per National Structural Code of the Philippines 2010.

VI. Recommendation

Using the set of data and conclusions derived from this study, the authors recommended the following:

This study only covers the susceptibility of members to seismic damage and does not deal with the methods to be used for retrofitting (if possible) along with the corresponding cost due to loss in serviceability of the structure and the cost of retrofitting depending on the level of damage. For this reason, incorporation of the Fragility curve reassessed from an economic point of view would develop a more conclusive set of guidelines as to how to approach this problem.

The researchers would recommend doing tests to determine whether the FRC building is applicable for any retrofitting measures that would allow it to withstand the level of ground motion as per NSCP 2010.

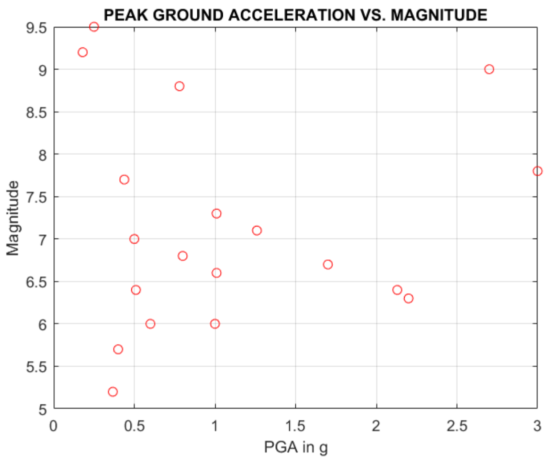
This type of study is only one of the other possible methods of seismic vulnerability assessment. Usage of other methods such as the Monte Carlo Simulations and Latin Hypercube Sampling would be recommended to determine the discrepancy in the results.

The Administrator has to expose and updates through seminars/trainings the staffs of Physical and Facility Office (PFO) on the latest technology in Structural Reliability, specifically, structural assessments of existing structures, by attending the nation's recognized organization, such as Philippine Institute of Civil Engineers and Association of Structural Engineers of the Philippines, Inc.

VII. Implications

Modeling of Structural Fragility relies on damage data generated either from empirical or mechanical methods (Cobum& Spence, 2002). Damage probability functions can be derived from documented post-earthquake damage observations, as with the case in empirical methods (Rota et al,2008). Damage estimates can also be derived using mechanical models such as the use of nonlinear analysis in the area of less earthquake prone regions due to lack of observational data (Karbassi and Nollet, 2013). As for this study, the latter was done using PGA as the primary parameter.

Unlike the Richter Scale or the Moment Magnitude Scale, PGA is not a measure of the total energy of an earthquake but as a measure of the intensity of the acceleration at a geographic point. Hence, PGA is not indicative of any level of magnitude but the probability of achieving higher PGA repeatedly increases as the Magnitude increases. It can be inferred that the probability of achieving a certain PGA repeatedly is directly proportional to the magnitude. Even in smaller magnitudes, high PGA's can be attained though repeated occurrence is much smaller (USGS, 1994).



All form of matter obeys the most basic fundamental principle of physics which is Inertia. Anybody will continue to perpetuate its current state unless influenced by outside force. If it is moving at a certain direction at a certain speed, it will continue to do so unless acted upon by any force and will remain still if let be. Buildings, as it is made of matter, behave in a similar manner.

Earthquakes do not move in one direction, relative to a particular axis. Let us say that the X-axis is the longitudinal section of the building. Ground motion is not inclined to move only at one direction of the longitudinal section may it be to the left or to the right. It goes both ways. One can illustrate it when moving a piece of string back and forth. By doing so, wave like patterns are created. The dips in the waves indicate the sudden change in the direction of the acceleration while the amplitude of the wave is an indication of the magnitude of the change.

This is because of inertial lag. Inertial lag, by definition, is the delay in the response of a flow to the forces acting upon it. Mechanical response of any object is not instantaneous. No object is perfectly rigid. With regards to a structure, the bottom of the column is the first to respond to any form of ground movement before it reaches the top. But unlike the string, columns are sufficiently rigid and are essentially brittle. The greater the acceleration, the greater the inertial lag for every abrupt change in the direction of acceleration. The greater the inertial lag, the higher the probability of any of the structural member to break off.

As so, even though PGA isn't indicative of the total energy being introduced by the ground to the building, it can illustrate the level of damage the structure incurs as it achieves that level of acceleration. These damages accumulate throughout every abrupt periodic change in the direction of the ground acceleration until the accumulated damage exceeds its allowable limit subsequently causing its collapse.

Intensity on the other hand is essentially a qualitative scale of the level of damage which can be described in terms of perceived shaking or potential damage to manmade structures. But it does not indicate the amount of energy released during an earthquake unlike its Magnitude. Despite that, the amount damage structures sustain increases with the increasing magnitude, under the assumption of constant duration (USGS, 1994).

Below is an example of a proposed correlation of the Modified Mercalli Intensity Scale with PGA in Costa Rica.

Table 4.8.1: Proposed ranges of PGA for each Instrumental MMI in Costa Rica (Linkimer, 2008)

Intensity (MM)	PGA Max Range (cm/s^2)	PGA Max Range (% g)	PGA Max Range (cm/s^2)	PGA Max Range (% g)
II	< 4.9	< 0.5	< 5.6	< 0.6
III	4.9-13.3	0.5-1.4	5.6-15.0	0.6-1.5
IV	4.9-13.3	1.4-3.7	15.0-40.3	1.5-4.1
V	36.0-80.3	3.7-8.2	40.3-84.7	4.1-8.6
VI	80.3-146.7	8.2-15.0	84.7-139.6	8.6-14.2
VII	146.7-268.0	15.0-27.3	139.6-230.2	14.2-23.5

This correlation may serve as a good fit for structures in Costa Rica but this relationship is strictly limited to parameters obtained in that particular area. Even then, "locations within the same intensity area will not necessarily experience the same level of damage since damage depends heavily on the type of structure, the nature of the construction, and the details of the ground motion at that site" (USGS, 2012). Also, instrument-only based approach for Generalized Damage Intensity relative to ground motion parameters such as PGA and PGV does not account for the structural characteristics of buildings and, therefore, may not provide useful information about the damage state of the built environment following an earthquake (Tan &Irfanoglu, 2012). Duration also contributes to the damage sustained in a structure. An earthquake could be relatively weak but occur at a long period of time effectively inducing the same amount of damage a relatively strong earthquake can cause in a short amount of time.

Because of these variations, using Damage as an indicator of determining the threshold of Magnitude at which it will be considered unsafe will only yield a probabilistic range of Magnitudes. And in doing so, requires further study about all other parameters influencing the structural integrity of buildings in a particular area which is outside the coverage of this study. The scope of this study is only limited to the development of Fragility Curves to assess the seismic performance of the building in question using PGA as a parameter. That is to say, the purpose of this study is to create a probabilistic model of the level of observational/qualitative material damage in terms of its quantitative counterparts derived from Damage Indices using PGA and does not take in to consideration other factors that could influence the change in the actual incurred damage relative to the theoretical damage.

As it is usually the case in seismic damages, failure of a member is the product of all modes of failures. However, the mode of failure only considered in this study is limited to shear.

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