International Journal of Scientific and Research Publications, Volume 7, Issue 6, June 2017 308
ISSN 2250-3153

Seismic Strengtheningof Heritage Structure in Mandalay

PyinSaOo*, Dr.AyeMya Cho**

* Department of Civil Engineering, Mandalay Technological University
**Department of Civil Engineering, Mandalay Technological University

Abstract- Recent earthquakes in Myanmar have underlined the need for wide monitoring and safety assessment of cultural heritage structures. The study presents the seismic vulnerability evaluation and seismic strengthening of a specific monumental masonry structure: the main stupa of Kuthodaw Pagoda in Mandalay. The required experimental tests were conducted to determine the material properties of the structure. The probabilistic seismic hazard analysis was done to get ground motion data consistent with local seismic conditions. These seismic waves were used for the input of time history analyses. Comparisons of the expected seismic demand and the capacity of the structure were done to determine the weak parts of the structure under earthquakes. The fragility curves of the structure under consideration with and without external steel rings were developed. The results have shown that the use of external steel rings can significantly reduce seismic vulnerability and effectively strengthen the structure against earthquake.

Index Terms- Heritage Structures, Mandalay, Time History Analyses, Fragility Curves

I. Introduction

Heritage structures perform vital role in nation's history, culture and signify the richness of it. To augment life and enhance strength, their restoration is very important for the future generations to have knowledge about how mankind lived in past ages. The majority of the main structural systems for historical heritage structures or monuments are masonry elements, composed of stone, bricks and mortar. For all types of old historical masonry structures erected in seismic zones of high seismicity, earthquake is always their number one ―enemy due to their very bad response to earthquakes.Earthquake is one of natural phenomenon which cannot be accurately predicted where and when it will happen. Since earthquake force intensity changes with time, then its effect to the structure also changes with time. Earthquakes act as non-harmonic, non-periodic andnon-stationary dynamic load in the form of wave radiation from the source. The wave is then radiated in all directions with the surrounding soil and rocks medium until approaching earth surface and causing vibration. Ground motion received by foundation is then continued to the upper structure, resulting in oscillation at the building as inertial forces.

Recently, it was observed that the frequency of occurrences of earthquakes along the Sagaing fault has increased. Though an earthquake could not be prevented, the loss of life and damage to property could be minimized. Steps could be taken to reduce the damages to existing structures. The estimation of the seismic vulnerability of a heritage structure is a multi-phased process that ranges from the description of earthquake sources to the characterization of structural response, and to the description of measures for seismic protection.

Myanmar is prone to great earthquakes. In 1917, Bago earthquake hit Bago region causing failure of top portion of the Shwemawdaw Pagoda. Recently, the magnitude of 6.8 earthquake struck Myanmar 25 km west of Chauk with a maximum Mercalli intensity of VI and several pagodas and temples in the nearby ancient city of Bagan were damaged. The failures of the structures exampled above indicate that evaluation for seismic safety and strengthening of heritage structures are extremely needed. In this study, the nonlinear seismic analysis of case study structure is done using finite element software ANSYS [1]. The analysis can check whether strengthening of the existing structure is required or not. The evaluation of strengthened structure by using external steel rings is also illustrated.The purpose of this study is to increase seismic resistance of the structure for future anticipated earthquakes without altering their basic structural system.

II. METHODOLOGY

In this study, the nonlinear seismic analysis of case study structure is carried out to determine whether strengtheningfor different levels of ground shaking is required or not. Theexperimental tests are conducted to get the existing properties of the materials. The vibration characteristics of the structure were determined by performing modal analysis. Then transient analysis of the structure is performed considering the appropriate seismic loading and the results before and after strengthening are compared. The general methodology used in this study is illustrated in Figure 1.

Structure selected for seismic analysis

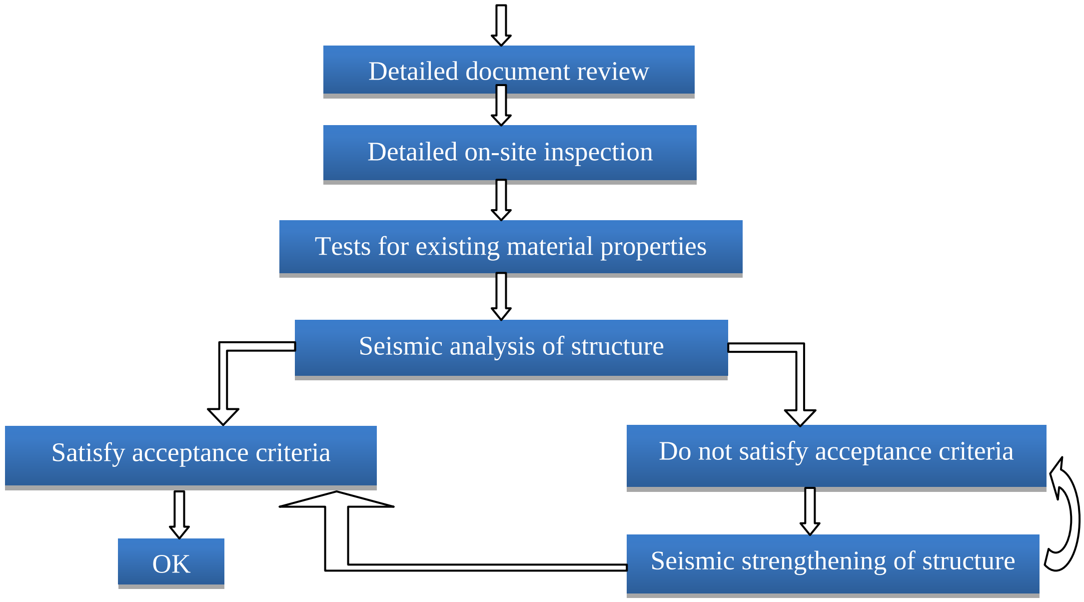


Figure 1: Flow Chart for Methodology

III. PROBABILISTIC SEISMIC HAZARD ANALYSIS

The goal of probabilistic seismic hazard analysis (PSHA) is to quantify the rate or probability of exceeding various ground-motion levels at a site given allpossible earthquakes. Mostly, the seismic hazard levels are determined by the probabilistic seismic hazard analysis (PSHA) [3]. The following three levels are commonly defined for buildings with a design life of 50 years [2]:
1). Serviceability Earthquake (SE),
2). Design Basis Earthquake (DBE), and
3). Maximum Considered Earthquake (MCE).
The definitions of the SE, DBE, and MCE defined by the ATC-40 are as follows:
The Serviceability Earthquake (SE) is defined probabilistically as the level of ground shaking that has a 50% chance of being exceeded in a 50-year period.
The Design Basic Earthquake (DBE) is defined probabilistically as the level of ground shaking that has a 10% chance of being exceeded in a 50-year period.
The Maximum Considered Earthquake (MCE) is defined probabilistically as the level of ground shaking that has a 2% chance of being exceeded in a 50-year period.
Seismic hazard analysis involves estimation of ground motion hazard at a particular area. The following three steps are generally required in the PSHA:
1). Specification of the seismic hazard source model,
2). Specification of the ground motion model, and
3). The probabilistic calculation

1) Seismic Hazard Source Model: The seismic-hazard source model is a description of the magnitude, location, and timing of all earthquakes. In this study, the estimation seismic hazard levels are based on the seismic hazards assessment for Myanmar developed by Myanmar Earthquake Committee (MEC) and Myanmar Geosciences Society (MGS) [9] and bounded Gutenberg-Richter recurrence law.The cumulative distribution function for magnitude of an earthquake can be described as follows [3].

$$F_M(m) = \frac{1-10^{-b(M-M_{min})}}{1-10^{-b(M_{max}-M_{min})}} \quad (1)$$

where
$F_M(m)$ = the cumulative distribution function for M
b = seismic constant
M_{min} = minimum magnitude
M_{max} = maximum magnitude
The probabilities of occurrence of discrete set of magnitudes are computed by the following equation [3].

$$P(M = m_j) = F_M(m_{j+1}) - F_M(m_j) \quad (2)$$

where m_j are the discrete set of magnitudes. Magnitude probabilities for Sagaing fault are tabulated in Table I.

Table I:Magnitude Probabilities for Sagaing Fault

m_j	$F_M(m_j)$	$P=(M= m_j)$
5	0	0.4381
5.25	0.4381	0.2464
5.5	0.6845	0.1385
5.75	0.823	0.0779
6	0.9009	0.0438
6.25	0.9447	0.0246
6.5	0.9693	0.0139
6.75	0.9832	0.0078
7	0.991	0.0044
7.25	0.9954	0.0025
7.5	0.9978	0.0014
7.75	0.9992	0.0008
8	1	0

Annual rate of exceedance of certain earthquake magnitude for the Sagaing fault shown in Figure 2 is developed by using the seismic historical data of the Sagaing Fault [9] and magnitude probabilities for Sagaing fault.

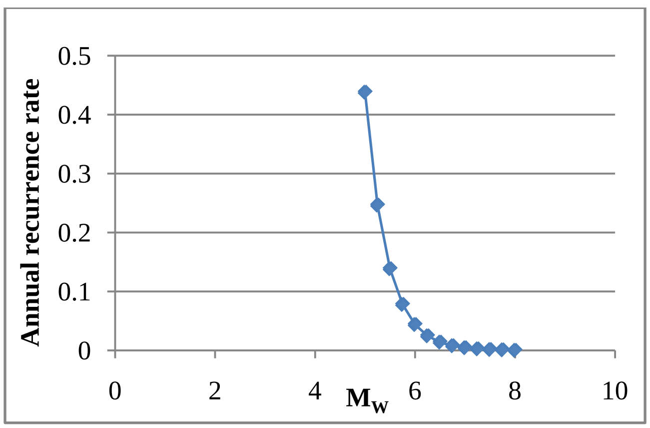


Figure 2: Annual rate of exceedance of certain earthquake magnitude for the Sagaing Fault

2) Ground Motion Model: The ground motion model used in PSHA is referred to as attenuation relationship. The most basic attenuation gives the ground motion level as a function of magnitude and distance, but may have other parameter to allow for a few different site type or style of faulting [11]. Cornell, et al. (1979) proposed the following predictive model for the mean of log peak ground acceleration (in units of g).

$$\ln PGA = -0.152 + 0.859M - 1.803\ln(R + 25) \quad (3)$$

where,
R = distance from the source in kilometer

3) Probabilistic Calculation: The return period, T is defined as according to the following equation.

$$T = \frac{1}{p} \quad (4)$$

For example, a 500 years earthquake has an annual probability of exceedance of 0.002.
The probability of an earthquake with a return period of T being exceeded in n years is given as:

$$p = 1 - (1 - \frac{1}{T})^n \quad (5)$$

In the UBC code, earthquake codes are based on a probability of exceedance of 10 % in 50 years (i.e., p=0.1, n=50 years (i.e., p=0.1, n=50 years). The return periods of three levels of earthquakes can be calculated by Equation 5.

The probability of occurrence in any year for the SE is, therefore, p= $\frac{1}{72}$ = 0.0139. The probability of occurrence in any year for the DBE and MCE are 0.0021 and 0.0004 respectively. Then the associated magnitudes for the three levels of earthquakes can be assumed using Fig. 6. Finally, the peak ground accelerations (PGA) can be estimated using Equation 3 with the nearer source distance of 25 km from the major cities such as Yangon and Mandalay. Using those data, the estimated magnitudes for the three seismic hazard levels can be provided shown in Table II.

Table II: Estimated Seismic Hazard Levels

Earthquake Type	Return Period, T (year)	Probability in any year	Estimated Magnitude, M_W	Peak Ground Acceleration, PGA
SE	72	0.0139	6.5	0.2g
DBE	475	0.0021	7.3	0.4g
MCE	2475	0.0004	7.8	0.6g

IV. SEISMIC ANALYSIS AND STRENGTHENING OF MONUMENT

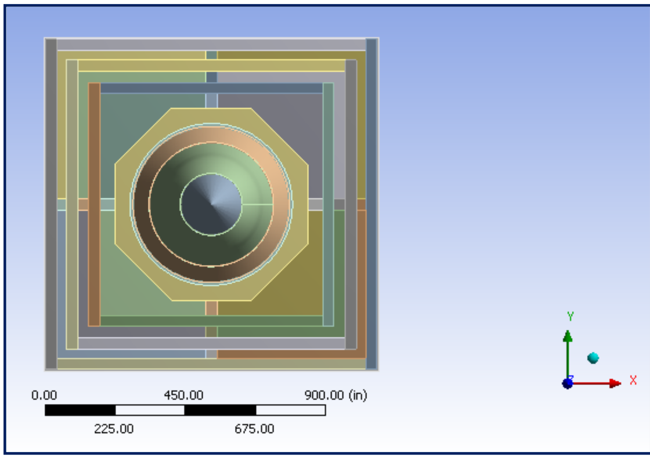
The case studied structure selected in this study is the main stupa of Kuthodaw Pagoda or MahaLawkaMarazein Pagoda located in Mandalay, Myanmar. It was built by KingMindon in 1857. It is one of the structures listed on the Memory of the World Register of the UNESCO. It is the brick masonry structure. It has five terraces: three are square shaped, one is polygonal shaped and one is circular shaped. Overall height is 30.5 m excluding the height of umbrella. The required experimental tests were conducted to determine the existing properties of the materials. Material properties used in the analysis are tabulated in Table III.

Table III: Material Properties

Properties	Masonry
Unit weight (kg/m^3)	1679
Young modulus (MPa)	2344
Poison's ratio	0.25
Ultimate compressive strength (MPa)	3.348
Ultimate tensile strength (MPa)	0.335

A. Finite Element Model

The three dimensional finite element model of the proposed structure is prepared by using ANSYS software. The model is comprised of 2,389 solid elements with 15,208 nodes. Theplan view and the finite element model of the structure are illustrated in Figures 3 and 4 respectively.



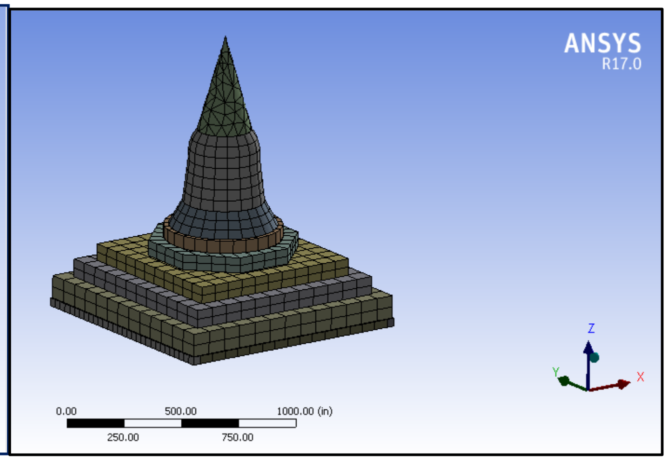


Figure 3: Plan View of StupaFigure 4: Finite Element Model of Stupa

B. Fragility Curves

Evaluating seismic fragility information curves for structural systems involves information on structural capacity, and information on the seismic hazard. Due to the fact that both the aforementioned contributing factors are uncertain toa large extent, the fragility evaluation cannot be carried in a deterministic manner. A probabilistic approach, instead, needs to be utilized in the cases in which the structural response is evaluated and compared against "limit states" that is, limiting values of response quantities correlated to structuraldamage.

Fragility curves can be obtained from a set of data representing the probability that a specific response variable (e.g. displacement, drift, acceleration, damage) exceeds predefined limit states for various earthquake hazards on a specific structure or on a family of structures. The fragility curves distribute damage among Slight, Moderate, Extensive and Complete damagestates.The conditional probability of being in, or exceeding, a particular damage state, ds, given thespectral displacement, S_d, (or other seismic demand parameter) is defined by the following equation.

$$P[ds/S_d] = \emptyset\left[\frac{1}{\beta_{ds}}\ln\left(\frac{S_d}{\bar{S}_{d,ds}}\right)\right] (6)$$

where$S_{d,ds}$ is the median value of spectral displacement at which the building reaches the threshold of damage state, ds, β_{ds} is the standard deviation of the natural logarithm of spectral displacement for damage state, ds, and Ø is the standard normal cumulative distribution function.

C. Transient Analysis

Transient dynamic analysis or time-history analysis is a technique used to determine the dynamic response of a structure under the action of any general time-dependent loads. This is used to determine the time varying displacements, stresses, strains and forces as it responds to any combination of static, transient and harmonic loads [1]. The basic equation of motion solved by a transient dynamic analysis is:

$$\{F(t)\} = [M]\{\ddot{u}\} + [C]\{\dot{u}\} + [K]\{u\} \quad (7)$$

where:

[M] = mass matrix
[C] = damping matrix
[K] = stiffness matrix
$\{\ddot{u}\}$ = nodal acceleration vector
$\{\dot{u}\}$= nodal velocity vector
{u} = nodal displacement vector
{F(t)}= load vector

Three methods are available to do transient dynamicanalysis: full, mode- superposition and reduced. In this study,full transient dynamic analysis is used. It is the most general ofthe three methods because it allows all types of nonlinearities tobe included. The transient analysis is carried out using as input, theacceleration time dataconsistent with three seismic hazard levels determined by PSHA. The load combinations according to the ACI codes aseismic regulations [4], shown in Table IV, are used.

Table IV:Loading Case Combinations

Loading Case	Combinations
1	D
2	D+L
3	D+L+(W or E)
4	D+W
5	0.9D+E
Where D = Dead load, L = live load, E = Earthquake load	

In seismic analysis, the structure is divided into eight parts in order to evaluate the stresses subjected to the structure. The eight parts of structure and their respective heights are illustrated in Figure5.

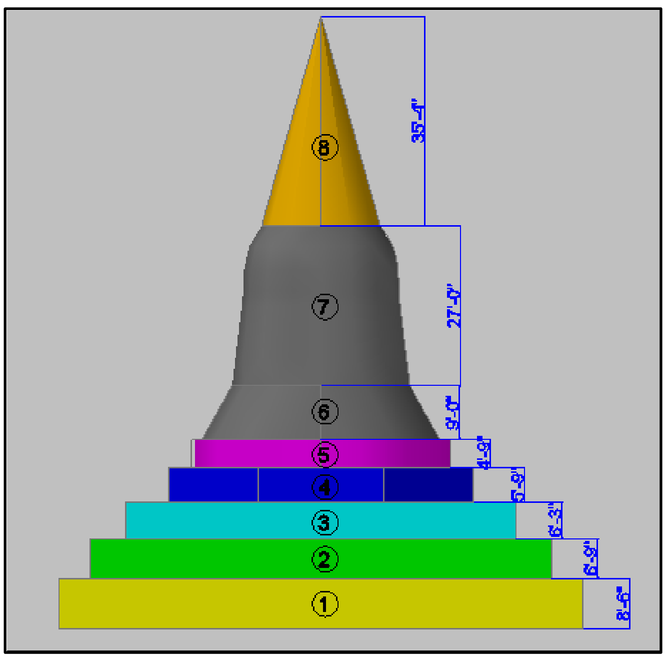


Figure 5: Parts of Structure

According to seismic analysis of the structure, it is observed that the stupa can resist the compressive stresses under three levels of ground excitations. However, for tensile and shear stresses, the calculated demand to capacity ratios exceed 1 under DBE and MCE levels.So, the structure need to be strengthened for these two seismic hazard levels. The compressive stresses subjected to each parts of stupa and calculated demand to capacity ratios are tabulated in Table V.

Table V. Demand to Capacity Ratios for Compressive Stresses before Strengthening

Parts	Stress(psi)			Strength (psi)	DCR		
	SE	DBE	MCE		SE	DBE	MCE

1	10.618	71.069	79.712	485.59	0.022	0.146	0.164
2	10.618	71.069	79.712	485.59	0.022	0.146	0.164
3	10.618	71.069	79.712	485.59	0.022	0.146	0.164
4	10.618	71.069	79.712	485.59	0.022	0.146	0.164
5	10.618	71.069	79.712	485.59	0.022	0.146	0.164
6	10.618	71.069	79.712	485.59	0.022	0.146	0.164
7	10.618	71.069	79.712	485.59	0.022	0.146	0.164
8	10.618	71.069	79.712	485.59	0.022	0.146	0.164

The tensile stresses subjected to each parts of stupa and calculated demand to capacity ratios are shown in Table VI.

Table VI: Demandto Capacity Ratios for Tensile Stresses before Strengthening

Parts	Stress(psi)			Strength (psi)	DCR		
	SE	DBE	MCE		SE	DBE	MCE
1	12.695	110.69	417.3	48.559	0.26	2.28	8.59
2	12.695	110.69	119.09	48.559	0.26	2.28	2.45
3	12.695	110.69	119.09	48.559	0.26	2.28	2.45
4	12.695	201.58	119.09	48.559	0.26	4.15	2.45
5	12.695	201.58	119.09	48.559	0.26	4.15	2.45
6	24.351	201.58	218.5	48.559	0.50	4.15	4.50
7	24.351	201.58	218.5	48.559	0.50	4.15	4.50
8	12.695	110.69	119.09	48.559	0.26	2.28	2.45

The shear stresses subjected to each parts of stupa and calculated demand to capacity ratios are described in Table VII.

Table VII: Demand to Capacity Ratios for Shear Stresses before Strengthening

Parts	Stress(psi)	Strength (psi)	DCR

	SE	DBE	MCE		SE	DBE	MCE
1	33.661	179.94	194.43	126	0.27	1.43	1.54
2	16.952	60.221	97.431	126	0.13	0.48	0.77
3	16.952	90.15	97.431	126	0.13	0.72	0.77
4	16.952	90.15	97.431	126	0.13	0.72	0.77
5	16.952	90.15	97.431	126	0.13	0.72	0.77
6	16.952	90.15	97.431	126	0.13	0.72	0.77
7	16.952	90.15	97.431	126	0.13	0.72	0.77
8	5.812	30.292	32.767	126	0.05	0.24	0.26

D. Seismic Strengthening of Structure

Strengthening of heritage structure is a difficult compromise between requirements of structural theory and conservation principles. The aim of strengthening is to preserve the historical structure for generations to come. Structural assessment and remedial interventions on structural systems of the historical structure require special considerations to retain the architectural integrity and historical authenticity [6]. Various techniques are available for strengthening of structure. The type and quality of masonry materials and the structural integrity of the structure are the main criteria to be considered when choosing the methods of strengthening. Taking into account these criteria, the use of external steel rings is selected as the most suitable mean for seismic strengthening of the structure. The isometric view of strengthened structure with external steel rings is shown in Figure 6.

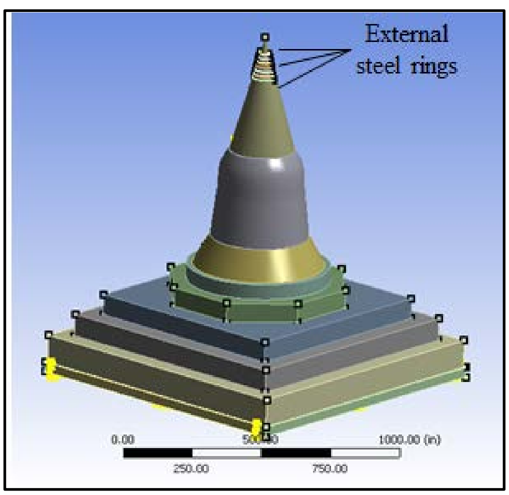


Figure 6: Isometric View of Strengthened Structure

In this study, the structure is strengthened to satisfy the acceptance criteria for MCE level of ground excitation, the most severe of three seismic hazard levels. The compressive stresses subjected to each parts of stupa and calculated demand to capacity ratios after strengthening are tabulated in Table VIII.

Table VIII: Demand to Capacity Ratios for Compressive Stresses after Strengthening

Parts	Stress(psi)	Strength (psi)	DCR

	MCE		MCE
1	45.484	485.59	0.094
2	45.484	485.59	0.094
3	45.484	485.59	0.094
4	45.484	485.59	0.094
5	45.484	485.59	0.094
6	45.484	485.59	0.094
7	45.484	485.59	0.094
8	45.484	485.59	0.094

The tensile stresses subjected to each parts of stupa and calculated demand to capacity ratios after strengthening are shown in Table IX.

Table IX: Demand to Capacity Ratios for Tensile Stresses after Strengthening

Parts	Stress(psi)	Strength (psi)	DCR
	MCE		MCE
1	111.5	48.559	2.30
2	46.259	48.559	0.95
3	46.259	48.559	0.95
4	46.259	48.559	0.95
5	46.259	48.559	0.95
6	62.57	48.559	1.29
7	62.57	48.559	1.29
8	46.259	48.559	0.95

The shear stresses subjected to each parts of stupa and calculated demand to capacity ratios after strengthening are described in Table X.

Table X: Demand to Capacity Ratios for Shear Stresses after Strengthening

Parts	Stress(psi)	Strength (psi)	DCR
	MCE		MCE
1	62.607	126	0.50

2	12.552	126	0.10
3	37.579	126	0.30
4	37.579	126	0.30
5	37.579	126	0.30
6	37.579	126	0.30
7	37.579	126	0.30
8	12.552	126	0.10

Figure 7 shows the fragility curves of the existing structure for MCE level of ground excitation and Figure 8 shows the fragility curves of the strengthened structure.

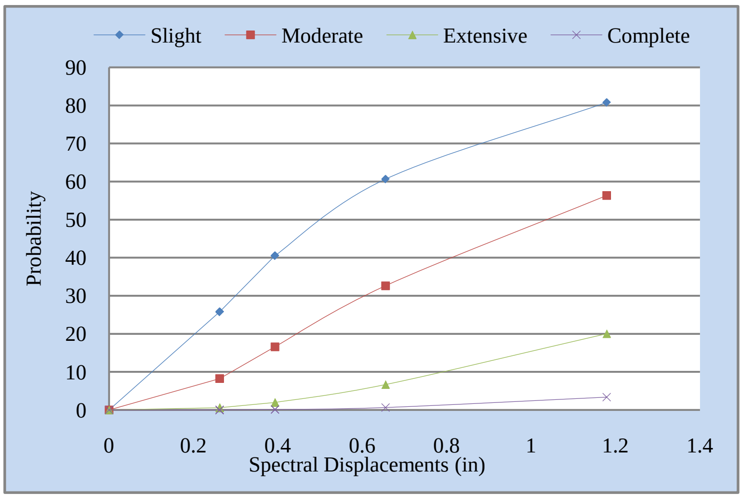


Figure 7: Fragility Curves of Existing Structure

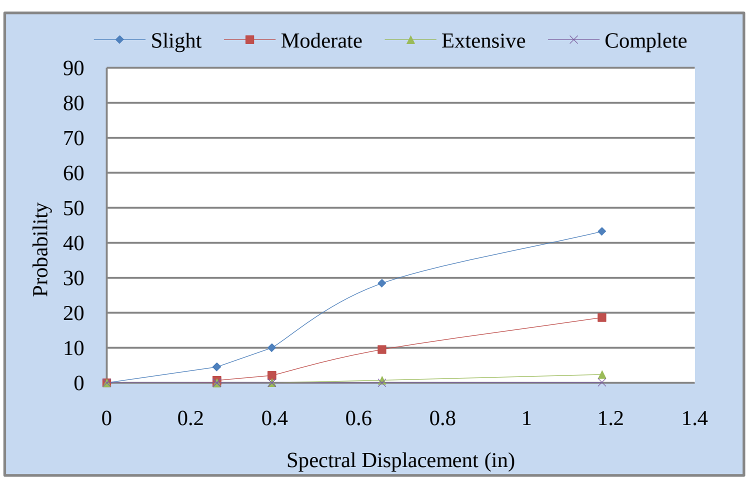


Figure 8: Fragility Curves of Strengthened Structure

The effect of the external steel rings on the response of existing heritage structure can be seen in Fig.7 and 8.The probabilities of damageare reduced from 81% to 44% for slight damage state, from 57% to 19% for moderate damage state, from 21% to 3% for extensive damage state and from 4% to 0.14% for complete damage state. These are considerable reductions which indicate that the use of external steel rings is effective in strengthening of the structure.

V. CONCLUSIONS

This study has presented seismic vulnerability evaluation and seismic strengthening of a brick masonry cultural heritage structure located in Mandalay by using the finite element analysis. The performance of the structure is evaluated in terms of both stresses limitations and fragility curves. According to the results of the seismic analysis of the existing structure under study, the structural configuration of the stupa satisfies the compressive stress limitations, as is usual for masonry structures and the stresses exceed the strength for tension and shear. The more vulnerable parts of the pagoda are inverted bell shaped portion and top spherical cone. By using external steel rings, the stresses subjected to the structure due to earthquake can be reduced to acceptance criteria and the probabilities of damage states can also be reduced to considerable values. Thus, the use of external steel rings can significantly reduce seismic vulnerability of the structure, leading to an effective method in strengthening of heritage structuresagainst earthquake.

ACKNOWLEDGMENT

The author is deeply gratitude to Dr.Nilar Aye, Professor, Head of Department of Civil Engineering, Mandalay Technological University, for her kind permission, and giving useful comments. The author would like to express grateful thanks to Dr.AyeMya Cho, Associate Professor, Department of Civil Engineering, Mandalay Technological University, for accepting and guiding through the whole period of studying for this paper, thoroughly proof-reading this paper and giving useful remarks on it.

REFERENCES

[1] ANSYS User’s Manual Vol. I. Swanson Analysis Systems, Inc. 1994..

[2] ATC 1996. "Seismic Evaluation and Retrofit of Concrete Buildings", Volume 1, ATC-40 Report, Applied Technology Council, Redwood City, California. H. Poor, *An Introduction to Signal Detection and Estimation*. New York: Springer-Verlag, 1985, ch. 4.

[3] Baker, J.W. 2008. ―An Introduction to Probabilistic Seismic Hazard Analysis (PSHA)‖. US Nuclear Regulatory Commission.

[4] Building Code Requirements for Masonry Structures (ACI 530-02/ASCE 5-02/TMS 402-02) reported by the Masonry Standards Joint Committee (MSJC)

[5] Chayanon Hansapinyo, "Seismic Performances of Brick Masonry Inverted Bell-Shaped Chedi ", APCBEE Procedia 9 (2014) 217 – 221

[6] Handbook on Seismic Retrofit of Buildings, Central Public Works Department and Indian Building Congress(April 2007)

[7] Chorpa, A.K.. "Earthquake Dynamic of Structure.", arthquake Engineering Research Institute, USA, 2006.

[8] Hazard Profile of Myanmar- July 2009

[9] Myo Thant, Nwai Le` Ngal, Soe Thura Tun, Maung Thein, Win Swe and Than Myint. 2012. ― Seismic Hazard Assessment forMyanmar‖. Myanmar Earthquake Committee (MEC) and Myanmar Geosciences Society (MGS).

[10] NEHRP. 1997. Guidelines for Seismic Rehabilitation of Building. FEMA 273. Federal Emergency Management Agency, Washington, D.C.

[11] Tint Lwin, "A Simplified Seismic Performance Based Design Method For Steel Moment Resisting Frames Using Building Design Data Bank",2015.

AUTHORS

First Author – Ms. PyinSaOo, Ph.DCandidate, Department of Civil Engineering, Mandalay Technological University.
Email – pyinsaoo11@gmail.com
Second Author – Dr. Aye Mya Cho, Professor, Department of Civil Engineering, Mandalay Technological University.
Email – amcho.civil@gmail.com