Academia Journal of Educational Research 5(7): 000-000, July 2017
DOI: 10.15413/ajer.2017.0231
ISSN 2315-7704

Research Paper

Seismic vulnerability analysis and finite element modeling of San Guillermo Parish: A historical coral stone church – in Catmon, Cebu, Philippines

Accepted 18th July, 2017

ABSTRACT

This is an attempt to apply Finite Element Method (FEM) in seismic vulnerability analysis using response spectrum in STAAD Pro V8i software to model the dynamic response of San Guillermo church. A 3D finite element model is constructed based on the technical drawings provided by CHERISH. Plate elements were generated from the imported geometry, afterwards, rectangular and triangular meshing were constructed with an average of 0.30 m × 0.40 meshing. Modal identification analysis was performed to calibrate the computational model and understand how the structure responds dynamically. Dynamic load cases were applied in global X and Z direction in accordance to NSCP 2010. Simulation results showed that the maximum drift ratio is lesser than the allowed drift ratio of 1% as prescribed by FEMA 356 structural performance level (life safety). Shear stress contours suggest that the majority of the church wall will not experience extent of damage when subjected to a representative ground motion having a 10% probability of being exceeded in 50 years and developed for a damping ratio of 0.05. Most of the URM walls were subjected to compressive stress yet its magnitude were lower than the allowable compressive axial stresses prescribed by UBC and ACI 530-88/ASCE 5-88 codes. Majority of the church structural components were expected to present little or no damage under out-of-plane stresses thus, minimal intervention is required.

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Keywords: Unreinforced masonry, church, vulnerability, structural analysis.

INTRODUCTION

Building construction is one oldest form of masonry and Unreinforced masonry (URM) is considered unreliable in seismic prone areas due to damages experienced (Tena-Colunga, 1992). URM structures are those types of buildings that are not braced by reinforcing beams or other type of reinforcements. It is composed of two different materials, masonry units (stone, bricks, or blocks) and mortar (Zeng, 2010). The Bohol earthquake that occurred on October 15, 2013 with recorded magnitude at M_w 7.2 (Lagmay et al. 2013) forces the local government of the Philippines to give extra attention to historical URM structures. It affected the Central Visayas region, particularly Bohol and Cebu.

One of the affected historical church is the San Guillermo Parish in Catmon, Cebu, Philippines (Figures 1 and 2). The church is around 180 years old as of the present time and is considered as one of the heritage sites in Cebu. It experienced the tremor created by the Bohol 2013 earthquake, and as reported by the residents, a number of stone blocks fell off from the structure during the seismic activity making it worthy for an investigation. It is a must to know the current status of its structural integrity and serviceability for the locality's safety and historical preservation.

The church consists of one longitudinal part and a belfry with a total area of 960 m^2 approximately in plan. The main church stands 15 m high while the total height of the belfry is around 23 m. Its primary wall reaches 8 m from the

Figure 1: San Guillermo Parish location (courtesy of Google Map).

Figure 2: San Guillermo church (a) exterior (b) interior facing altar and (c) facing main entrance.

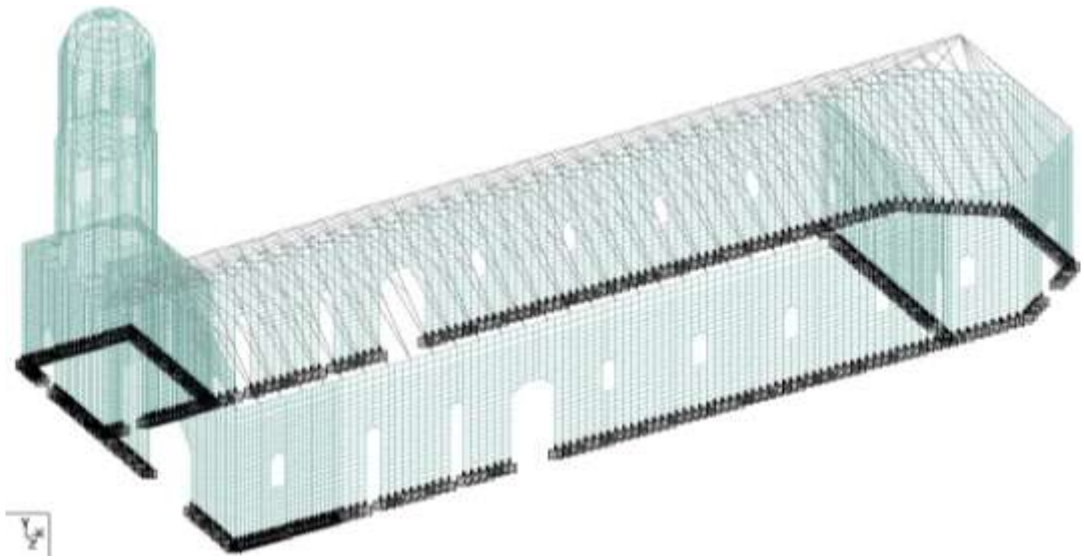

Figure 3: 3D Finite element STAAD model of San Guillermo Church.

natural grade line and average thickness equal to 0.9 m. Most members of the roof framing are composed of 0.15 × 0.20 m timber cross section, each being spaced by 1.2 m along the length of the church nave.

Over the past decades, draw backs in structural analysis of URM structures has been compensated by the use of advanced computational strategies. Among them, Finite Element Method (FEM) and especially macro-modeling techniques prevailed in the analysis of large masonry structures.

The purpose of this study is to develop a model that can represent the response of URM church caused by seismic excitations and use this model to assess the structural integrity of San Guillermo church. This study evaluates the present condition of the church with coral stone, lime mortar and timber composition with the aid of computer modeling and simulation. The subject church may serve as a representative of most URM coral stone church in the Philippines.

RESEARCH FLOW

Since global behavior of the structure is of great importance, macro-modeling was adopted in the study since most researchers preferred this due to its advantages and reliability (Araújo et al., 2012; Zeng, 2010; Betti and Vignoli, 2008; Maraveas and Tasiouli, 2015; Mele et al., 2001; Ceroni et al., 2007). Technical plans were secured from Conservation and Heritage Research Institute and Workshop (CHERISH). For *in situ* stone properties, experimental results of Caberte et al. (2015) were utilized. The simulations of FEM were done using STAAD Pro V8i(Bentley Systems Incorporated 2007). Findings of this study were used in determining the structural failure through resulting deformations and stresses.

MODELING AND SIMULATION

The floor plan of the church in CAD drawing provided by CHERISH was exported to SketchUP 2016 (Trimble Navigation, 2015) as drawing (.dwg) file for 3D model generation. Then, it was imported to STAAD Pro V8i as .dxf file in order for the latter to organize the model as nodes and beam components. Plate elements were generated from the imported geometry, afterwards, rectangular and triangular mesh (Figure 3) were applied to the created plates considering proper connection between nodes to avoid errors and warnings during analysis. Walls that are perpendicular to each other (that is, belfry to nave wall connection) must have common nodes for continuous typology. The whole 3D model comprises 11,328 plate elements with 12,167 nodes, 479 beams and 70,584 degrees of freedom. A 0.35 m × 0.4 meshing was established which is a lot finer in comparison to Lancioni et al. (2013) work concluding that block sizes does not influence the response significantly. The request of detailed description of the rigid response of San Guillermo church and the need of computational time saving is compromised reasonably.

Timber roof framings were incorporated in the model composed mainly of beam elements having dimensions of 0.15 m × 0.20 m in local Z and Y direction respectively and default density equal to 25 lb/ft3. For member specifications, the top chord of the timber frame is assumed to be continuous and each node connecting the URM wall were released in Y and Z rotations the same with the truss apexes. All web members were also released in the same manner in order to grasp the actual behavior of the

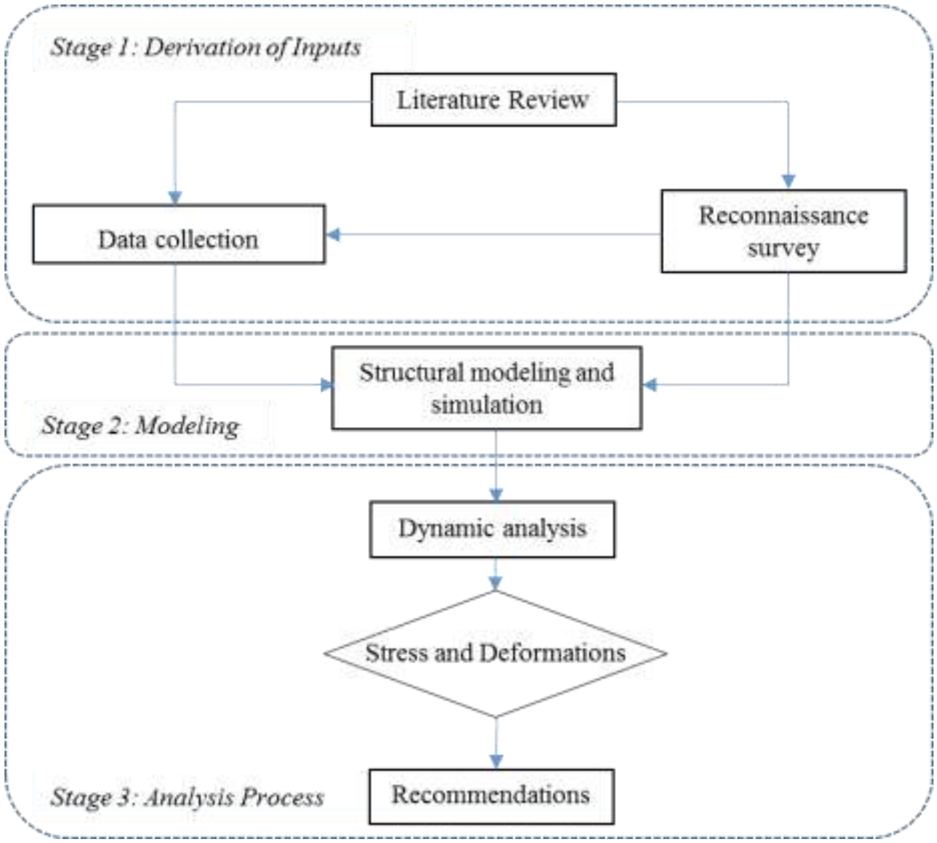

Figure 4: Implementation procedure.

structural roof component (Figure 4). No longitudinal bracing for the timber frame model except at the apex concurrently.

Various models with linear elastic material and Poisson's ratio of 0.20 for URM wall was used by most researchers in the field (Zhou, 2000; Zeng, 2010; Milosevic et al., 2012). These assumptions therefore were adopted for this study. Moreover, in the study entitled "Limits to Poisson's ratio in isotropic materials – general result for arbitrary deformation (Mott and Roland, 2012)" $0.2 \leq v < 0.5$ is the valid range for any isotropic material subjected to arbitrary loading or deformation. This range comports with the value of v for the vast majority of isotropic materials.

Prior to the seismic analysis, an equivalent lateral force procedure (ELF) of UBC 97 was performed in San Guillermo church to determine the scale factors in X and Z directions. Calculating the corresponding mode shapes of a dynamic system is possible since the modal analysis performs Eigen value extraction.

Seismic definition was created using Uniform Building Code of 1997 (UBC, 1997) which was where our NSCP originated. The seismic zone factor of 0.40 was used because the church was located in a zone 4 classification. For essential facilities, an important factor (I) of 1.5 was established as per NSCP 2010 (National Structural Code of the Philippines volume 1 (Buildings, towers, and other vertical structures) 2010). Numerical coefficient representative of the inherent over strength and global ductility capacity of lateral-force-resisting systems or the response modification factor (R) had a value of 4.5, in both X and Z direction. With regards to the soil type, a stiff soil (Sd) profile was considered. Moreover, the near source factor used was 1.0 since the closest distance to a known seismic source is greater than 15 km. The approach for seismic loading was response spectrum (Figure 5) as per section 208.6.5 of NSCP 2010. An elastic design response spectrum was generated using the values of site specific seismic coefficient Ca and Cv, 0.44 and 0.96 respectively. A representative ground motion site-specific design response spectrum based on the geologic, tectonic, seismologic and soil characteristics associated with the specific site having a 10% probability of being exceeded in 50 years and developed for a damping ratio of 0.05 is considered as per NSCP 2010 section 208.6.2.

Assumptions and simplifications

To get the general analysis model, the following assumptions and simplifications were made based on the structural features of San Guillermo church. The assumptions were adopted in order to simplify the model

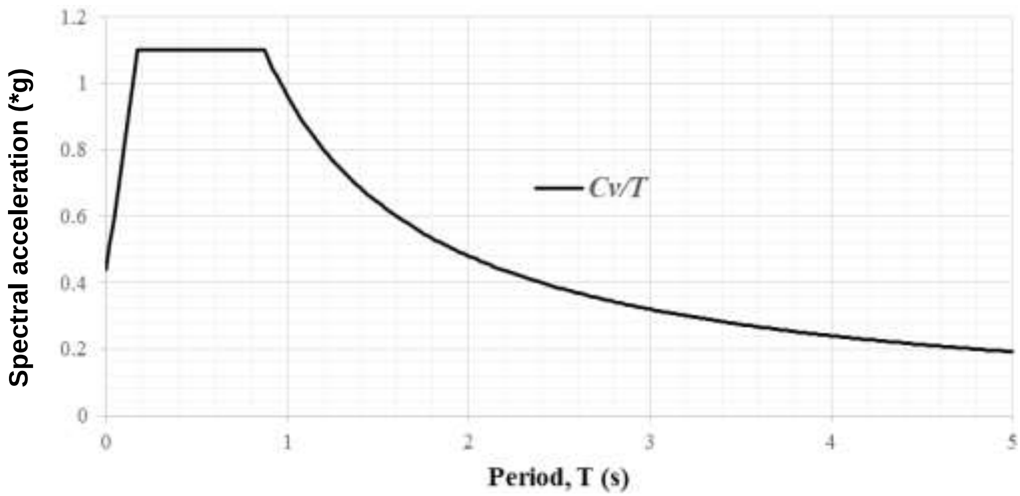

Figure 5: Response spectrum function definition.

Table 1: Material properties of the 3D FE model.

Mechanical Property	URM wall
Density (kg/m^3)	1,761.26
Compressive strength (MPa)	6.02
Young's Modulus (Gpa)	2.40
Poisson's Ratio	0.20

and reduce the calculation time without sacrificing its reliability:

- URM walls are modeled as an isotropic material and the properties of the material are independent of direction (Table 1). The materials that the church is composed of were not homogeneous, making it impossible to model every characteristic. The modeling strategy of Maraveas and Tasiouli (2015) was adopted and the load bearing URM walls simulated with plate elements (finite surface elements), which include both membrane and bending function and are ideal for linear elastic shell analysis representing material isotropy;
- The choir loft and sacristy storage area were not reflected in the model. They do not contribute to the diaphragmatic action of the structure due to their negligible stiffness caused by lack of lateral support and only transfers vertical loads (Maraveas and Tasiouli, 2015);
- Arc type opening are straightened by using triangular plate elements. To do so would increase the complexity of meshing. The finite element meshing was implemented in such a way to take into account recesses and openings in order to realistically simulate the structure geometry;
- Canopy was not included since it does not contribute in resisting seismic load to the main church. A gap between the front façade and the canopy was observed during the reconnaissance survey, thus, negligible seismic pounding is expected to occur.

RESULTS

Structural analysis of URM itself is a huge field that many researchers have been working on. The description of the analysis procedure used in this study is response spectrum (Figure 5). It is an elastic dynamic analysis of a structure utilizing the peak dynamic response of all modes having significant contribution to the total structure response. Peak modal responses are calculated using the ordinates of the appropriate response spectrum curve which correspond to the modal periods. Maximum modal contributions are combined in a statistical manner to obtain an approximate total structure response (Section 208.6.4.1, NSCP, 2010). For the dynamic analysis of structures, the response spectrum analysis is one of the methods that can be utilized (Hancilar et al., 2012). The accidental torsion effects and multi-directional seismic effects were considered in the analysis.

Deformations

One of the most important criteria in assessing an existing structure is the deformation it showcases during the application of loads. The deformation of the church was evaluated through nodal displacements in three global orientations (X, Y, and Z axes). Displacements at the topmost portion of the belfry were determined together

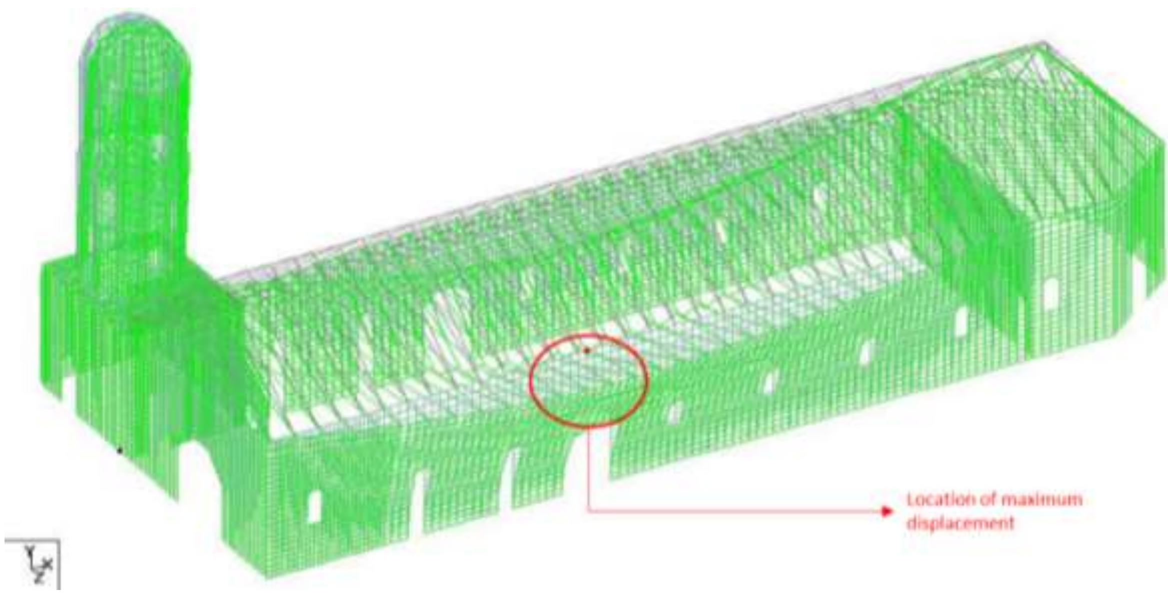

Figure 6: Maximum registered displacement location.

Table 2: Deformations of URM in global direction of San Guillermo Church(X-horizontal, Y-vertical, Z-horizontal).

Load type	Location	Global co-ordinate	Displacement (mm)
Dynamic load X	Belfry dome	X	12.356
	Front façade apex	X	-18.663
	Nave wall	Z	2.438
Dynamic load Z	Belfry dome	Z	17.319
	Front façade apex	X	3.628
	Nave wall	Z	40.507
DL+LL	Belfry dome	Z	6.273
	Front façade apex	X	-3.021
	Nave wall	Z	42.645
DL+LL+Ex/1.4	Belfry dome	Z	7.825
	Front façade apex	X	-16.352
	Nave wall	Z	44.383
DL+LL+Ez/1.4	Belfry dome	Z	18.644
	Front façade apex	Y	-0.969
	Nave wall	Z	71.576

with the apex of the front façade and the central part of nave walls; these chosen locations were the portions of the church that present considerable deformations. Nodal displacements of dynamic loads X and Z and other three service load combinations are presented.

The registered maximum displacement of the URM (Figure 6) that occurred was 71.576 mm located at the topmost portion of the nave wall directly above the door opening, right of the church. The dominant load type is the combination of service dead, live and earthquake load. Table 2 summarizes the displacements experienced by the structure during the simulation including the specific

Table 3: Allowable shear and combined normal stresses by different codes for URM (MPa).

Wall	Allowable shear stresses				Normal stresses				
	UCBC *(max)*	UBC *(ave)*	ACI *(max)*	ABK**(max)*	ACI 530-88		Both+	UBC	
					F_t	F_a	F_b	F_t	F_a
URM	0.10	0.14	0.38	0.56	0	2.30	3.00	0.25	1.83

*Ultimate shear strength criterion +ACI 530-88 and UBC.

locations.

The maximum drift ratio, defined as the drift to height of the church, is about 0.89%, which is less than the allowed drift ratio of 1% prescribed by FEMA 356 structural performance level (life safety) (FEMA, Prestandard and commentary for the seismic rehabilitation of buildings, 2000) which was also adopted in the study "Numerical modeling and experimental investigation of masonry structures" (Zeng, 2010). These observations indicate that the deformation of San Guillermo church is acceptable when subjected to a design seismic in relation to the tremor it experienced during the massive earthquake with epicenter at Bohol last October, 2013. There were no visible URM displacements observed during the site visit after the catastrophe, the structure remains firm and stable in structural engineering's perspective.

Stresses

The provisions for allowable shear stress for unreinforced masonry recommended by different building codes and recommendations, such as ABK Methodology, UCBC, UBC and the ACI 530-88, are next presented. These proposed code values were compared with the maximum shear stresses exhibited by the FE model. Moreover, the provisions for allowable combined stresses for unreinforced masonry recommended by the UBC and the ACI 530-88/ASCE 5-88 building codes were only compared to the normal stresses developed in the model since the UCBC and ABK methodology gives no recommendations to check combined stress states.

Table 3 shows the allowable stresses referred from the study conducted by Tenna-Colunga (1992) on unreinforced masonry building during the Loma Prieta earthquake. These values were used to define the different stress contour regions in the analysis of the results obtained from the FE analyses for better perspective.

Shear stress analysis

The allowable shear stress criterion of the UCBC code (SEAOC, 1990) and the stress contour limits defined by the ultimate shear stress criterion of the ABK Methodology (ABK, A Joint Venture, 1981) were considered since these codes are the current available standards that apply for evaluation of old masonry construction. The ACI 530-88 (ASCE, 1986) and UBC (International Conference of Building Officials, 1988) allowable shear stresses were also considered in all cases because it is an intermediate value between the ABK (the ultimate shear strength criterion) and the UCBC code (the most conservative among the four allowable shear stresses).

In Figure 7 the maximum resulting shear stress developed in the springer of the door openings at rear and front façade are 0.162 MPa. The octagonal base at the second level of the belfry has 0.19 MPa exceeding UCBC and UBC shear values. The front façade had shear value not greater than 0.04 MPa. ACI and ABK shear strength criterion (ultimate) with allowable values of 0.36 and 0.58 MPa were not exceeded, respectively. Only the value of UCBC and UBC were overshadowed during the response spectrum analysis.

Table 4 summarizes the critical shear stress experienced by San Guillermo church. From the aforementioned observations listed, the shear stress contour suggested that the majority of the church wall should not experience extent of damage considering the allowable shear strength criterion of ABK Methodology and ACI 530-88. Critical locations for shear actions were known from the contour plots emphasized where the stress value exceeds the UCBC and UBC criteria. These locations are the springer of the main entrance and the second level of the bell tower. The propagation of damages can safely be concluded to initiate from this vulnerable portions of the church in terms of shear stresses. Critical portions for shear shown may also exhibit poor bonding between stones since shear stress originates mostly within the gaps filled with lime mortar in most URM structures.

Axial stresses

The stress contour regions for the normal stresses were defined to highlight tensile and compressive stress states. In *lieu* of provisions from the UCBC and the ABK methodology the ACI 530-88 and the UBC provisions were considered. Figure 8 shows the axial stress contours in the local X direction. Contour values were selected, negative ones represent compression while positive values are tensile stresses. The UBC code specifies an allowable tensile stress of 0.25 MPa for flexural tension normal to the head joints while ACI 530-88 does not permit any tension to

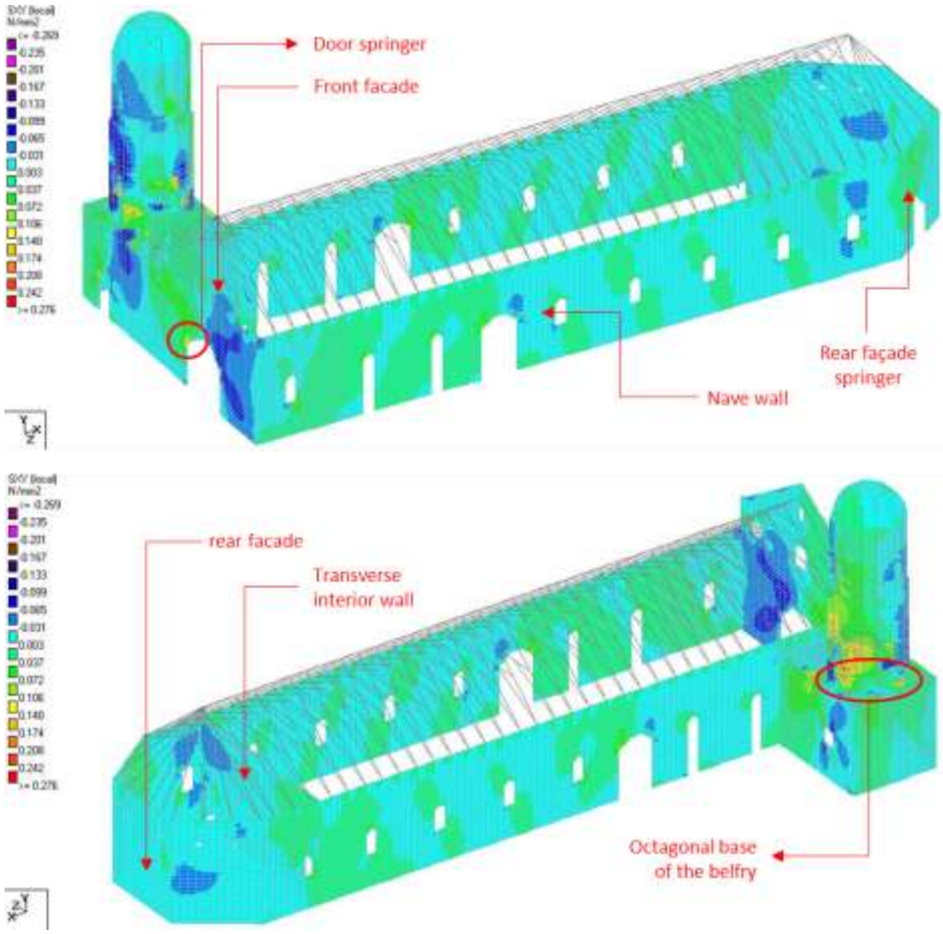

Figure 7: Shear stress in the local XY plane by service load combination DL+LL+Ez/1.4.

Table 4: Evident shear stresses experienced by San Guillermo churchURM wall.

Load type	Location	Shear Stress (MPa)	Remarks
	Nave wall	0.031	< Allowable
	Front façade	0.042	< Allowable
	Rear facade	0.082	< Allowable
	Belfry 1st level	0.036	< Allowable
DL+LL+Ex/1.4	Belfry 2nd level	0.07	< Allowable
	Belfry 3rd level	0.059	< Allowable
	Bell tower dome	0.012	< Allowable
	Door opening springers	0.167	>UCBC and UBC
	Transverse interior wall	0.04	< Allowable
	Nave wall	0.031	< Allowable
	Front façade	0.04	< Allowable
	Rear facade	0.045	< Allowable
	Belfry 1st level	0.039	< Allowable
DL+LL+Ez/1.4	Belfry 2nd level	0.19	>UCBC and UBC
	Belfry 3rd level	0.081	< Allowable
	Bell tower dome	0.031	< Allowable
	Door opening springers	0.162	>UCBC and UBC
	Transverse interior wall	0.03	< Allowable

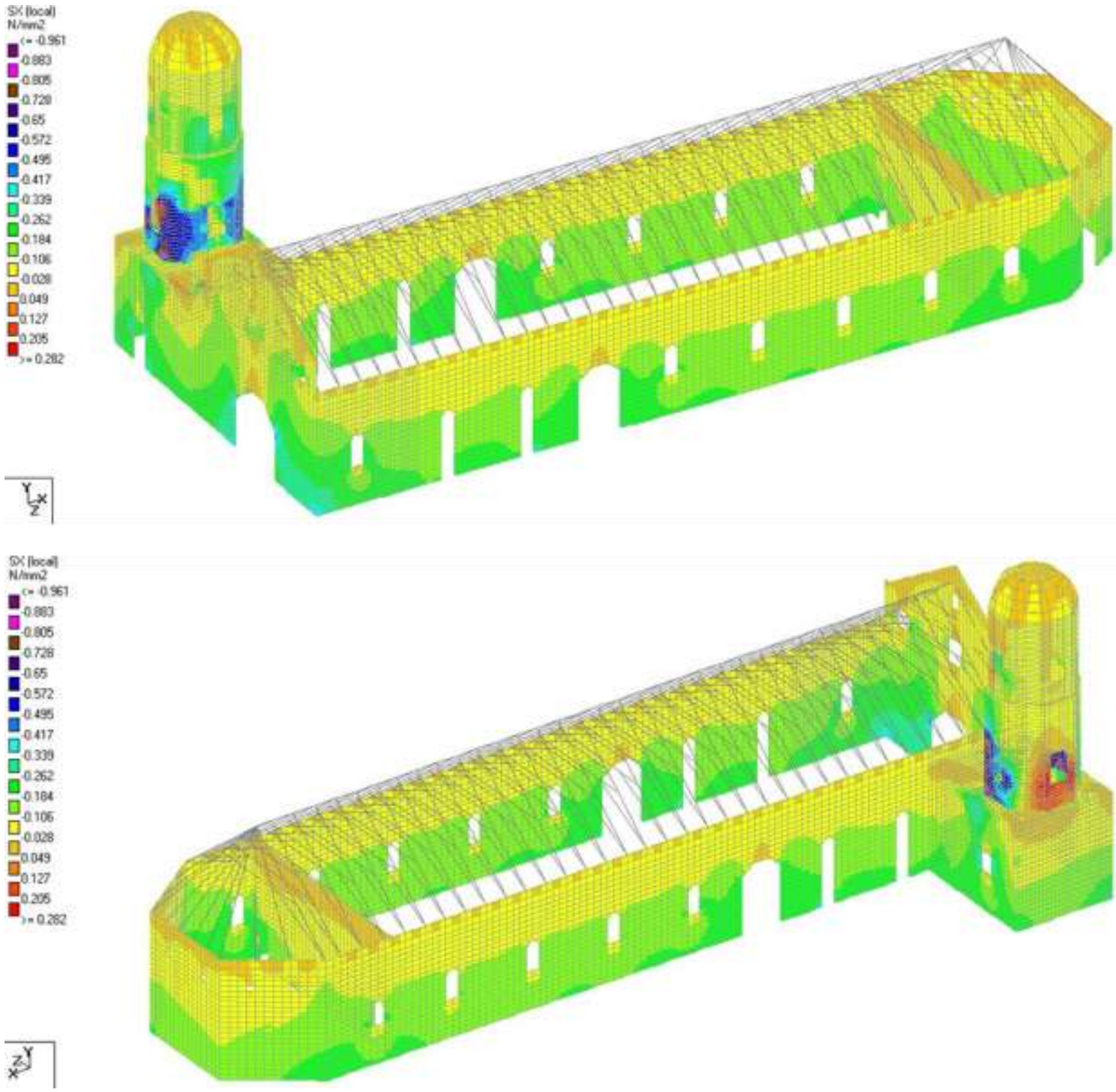

Figure 8: Axial stress in local X by service load combination DL+LL+Ez/1.4.

URM at all. In terms of compression, the ACI and UBC allow values not exceeding 2.3 and 1.83 MPa, respectively (Table 3).

Compression dominates all throughout the URM church when DL+LL combination was considered. Maximum compressive stress of 0.602 MPa was registered at the front of the second level of belfry. Axial stress in local X direction showcases compression of 0.143 MPa at the rear façade door opening springer. For service load combinations DL+LL+Ex/1.4 and DL+LL+Ez/1.4) evident propagation of compressive stresses were observed at the second level of the bell tower with magnitude of not more than 0.909 MPa while its side was in tension of 0.277 MPa. The remaining walls were in compression of not greater than 0.50 MPa. Tensile stress of 0.036 MPa was pin pointed at the rear façade door opening springer.

In light of the earlier observations, tensile stresses were critical when compared to the prescribed values of 0 and 0.25 MPa from ACI and UBC, respectively; these results strongly agrees to the claim that most URM structures are susceptible to tension though for this specific structure only few portion experienced this type of stress. On the other hand, most of the URM walls were subjected to compressive stress and its magnitude were lower than the allowable compression for unreinforced masonry prescribed by the UBC and ACI 530-88 code. These structural components in compression were expected to present little or no damage under out-of-plane stresses. Table 5 sets out the summary

Table 5: Critical portions of San Guillermo church for tensile and compressive stressesof URM wall.

Load type	Location	Axial stress (MPa) -compression + tension	Remarks
DL+LL+Ex/1.4	Nave wall	-0.176	<Allowable
	Front façade	-0.044	<Allowable
	Rear facade	-0.467	<Allowable
	Belfry 1st level	-0.143	<Allowable
	Belfry 2nd level	-0.043	<Allowable
	Belfry 3rd level	-0.115	<Allowable
	Bell tower dome	-0.197	<Allowable
	Door opening springers	-0.255	<Allowable
	Rear façade door opening springer	-0.095	<Allowable
	Transverse interior wall	-0.177	<Allowable
DL+LL+Ez/1.4	Nave wall	-0.274	<Allowable
	Front façade	-0.182	<Allowable
	Rear facade	-0.062	<Allowable
	Belfry 1st level	-0.024	<Allowable
	Belfry 2nd level	-0.458	<Allowable
	Belfry 3rd level	-0.194	<Allowable
	Bell tower dome	0.039	>Aci
	Door opening springers	-0.113	<Allowable
	Rear façade door opening springer	-0.199	<Allowable
	Transverse interior wall	-0.244	<Allowable

of critical locations of the church where the manifestation of compressive axial stresses obviously dominated.

CONCLUSIONS

Simulation results show that the maximum drift ratio is lesser than the allowed drift ratio of 1% as prescribed by FEMA 356 structural performance level (life safety). Shear stress contours suggest that the majority of the church wall will not experience extent of damage when subjected to a representative ground motion having a 10% probability of being exceeded in 50 years and developed for a damping ratio of 0.05 as per NSCP, 2010 section 208.6.2 considering the ultimate shear criterion. Tensile stresses were critical when compared to the prescribed value of 0 and 0.25 MPa from ACI and UBC, respectively; these results strongly agrees to the claim that most URM structures are susceptible to tension though for this specific structure only few component property experienced this type of stress. Most of the URM walls are subjected to compressive stress yet its magnitude are lower than the allowable compressive axial stresses prescribed by UBC and ACI 530-88/ASCE 5-88 codes. Majority of San Guillermo church's structural component are expected to present little or no damage under out-of-plane stresses, thus, minimal intervention is required.

ACKNOWLEDGEMENTS

The authors would like to take this opportunity to gratefully express their appreciation and indebtedness particularly to the Engineering Research for the Development of Technology (ERDT) Scholarship Program of the Department of Science and Technology (DOST) and Engr. Ricardo L. Fornisis for his advice and helpful comments.

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