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STRUCTURAL INVESTIGATION OF MALATE CATHOLIC CHURCH

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ABSTRACT: Malate Church underwent several repairs due to damages caused by typhoons, earthquakes and even wars. It was also observed that there was an interventionin the buttresses and walls of the right wing of structure that causes cracks and loosening of the masonry units. The damaged buttresses were sandwiched with steel platesplaced in its interior and exterior faces and were tied together by tendons. Some of these steel reinforcements were corroded, causing further cracks in the masonry around the said reinforcements.

The intent of this structural investigation is to determine the vulnerability of the structure based on the potential hazards due towind and earthquake that may occur at the site. The equivalent forces that may act in the structure due to these hazards are stated in the referral Code, the National Structural Code of the Philippines. To achieve this goal, collection of data at the site including material testing were thoroughly done in order to simulate the approximate behavior of the structure. Also, the structure was mesh into small solid elements approximately 1.0 meter cube and were analyzed using Finite Element Method.

Based on the results of the analysis, the maximum compressive and tensile stresses occurs between the interface of the footing and buttresses. This means that the buttress, as a rigid structural element resist more stress than the walls and cantilevers from the footing. Also, the maximum stress generated in the fabric is due to seismic force acting perpendicular to the plane of the wall. This is because, the wall, though analyzed as a solid element, is slender compared to its height. For seismic force acting parallel to its plane, the behavior is similar to a shear wall thus, produces minimum stress in the fabric.

Finally, the maximum compressive and tensile stresses generated in the fabric are 0.80 MPa and 0.53 MPa, respectively. Based on the test, the ultimate compressive strength of the masonry unit is 4.86 MPa and tensile strength is 0.97 MPa. These values yield a factor of safety equal to 6.0 in compression and 1.8 in tension. These values are greater than the value normally used in the design of buildings with an average of 1.4. Thus, the structure can still resist the forces due to the design loads stated in this report provided, cracks and other damages present in the structure will be restored properly.

1. Introduction

1.1. The Structure

The Malate Catholic Church is Hispano-Moresque style originally built by the Augustinian friars in the 16th century in Ermita, Manila. It is recognized by the National Historic Institute (NHI) as a national historical landmark. This structure was used by the British soldiers as their refuge during the attack in Manila in 1762. After they had left, the church undergoes repair though it was again destroyed by earthquake that hit Manila on June 3, 1863. The church was again rebuilt after it suffers major damages causes by World War II.

Typical to other Heritage Churches in the Philippines, its geometry is composed of a thick wall and buttresses built using adobe stone blocks and mortar. These wall and buttresses are supported by a huge continuous footing trapezoidal in cross-section that is also made of adobe stone blocks and mortar. The depth of the footing is approximately 3.0 meters extending below the existing natural grade line and extending outward the wall face with a slope approximately 1:1. Its geometry generated by STAAD.Pro V8i is shown in Figure 1.

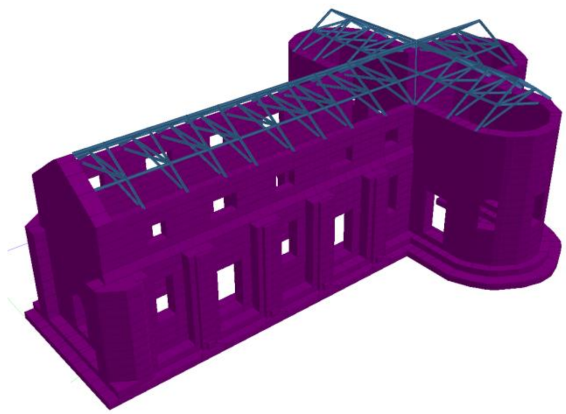

Fig. 1 – Geometry of the Structure

1.2. Intent of the Structural Investigation

The purpose of the structural investigation is to determine the vulnerability of the structure based on wind and earthquake hazards that may occur at the site, as stated in the National Structural Code of the Philippines. The intent of this code is to safeguard against major structural failures and loss of life in the event of major catastrophe, not to limit damage or maintain function of the structure. Figure 2 shows the Seismic Source Map of the Philippines and the location of the building site. Figure 3 shows the Wind Zone Map of the Philippines. The site is located in Zone II with a maximum design wind speed of 200 KPH.

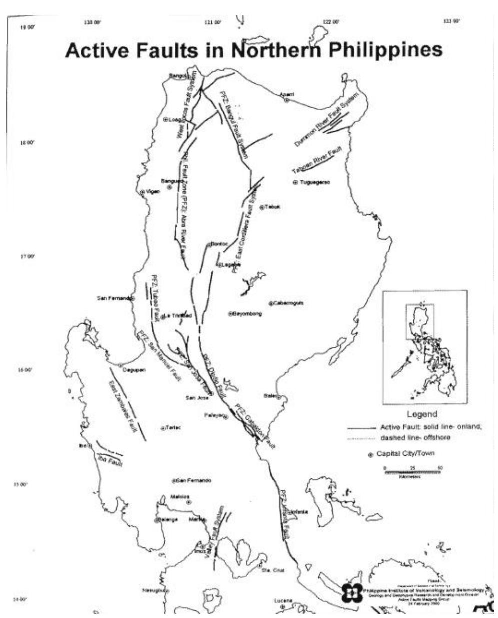

Fig. 2 – Seismic Source Map

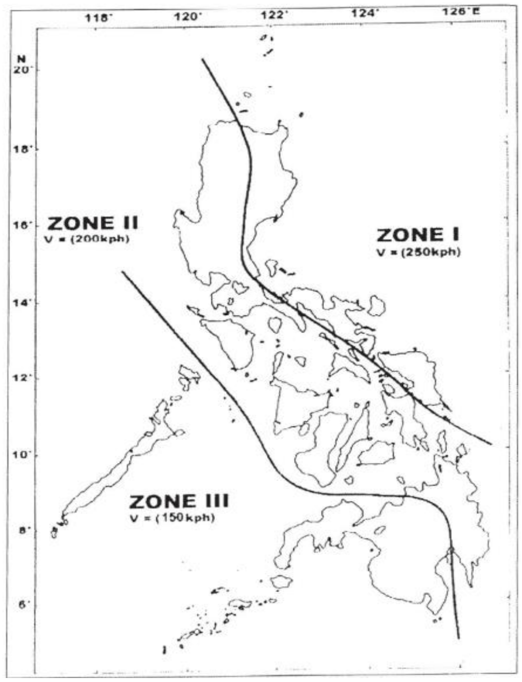

Fig. 3 – Wind Zone Map

1.3. Downside of Historical Structure

According to the United Nations Educational, Scientific, and Cultural Organization (UNESCO), "Heritage is our legacy from the past, what we live with today, and what we pass on to future generations."Over the years, century-old buildings became a landmark of culture, history, and traditions. Some of these edifices were categorically recognized by UNESCO as world's heritage site to acclaim its value and significance. However, these heritage structures are vulnerable to hazards either man-made or environmental such as chemical attacks, biological, thermal, floods, typhoons, and earthquakes.

In line with the structural integrity of heritage structure, P. G. Asteris, and I. P. Giannopoulos mentioned in their article entitled "Vulnerability and Restoration Assessment of Masonry Structural Systems" mentioned that the number one "enemy" of masonry structures composed of stone, bricks, and mortar is always the earthquake forces due to its bad response. However, the main responsibility of protecting the historical structure falls on the Engineers.

High demands are posed on engineers in restoration, according to Paulo B. Laurenco in his paper "Structural Restoration of Monuments: Recommendations and Advances in Research and Practice". This is because modern societies

understands that heritage structure is a landmark which should last forever, and the task of the generation is to deliver it in good shape for the generations to come.

According to S. Pompeu Santos on "Guide for the Structural Rehabilitation of Heritage Buildings", existing buildings are subjected to processes of degradation with time, which leads to a situation in which they became not able to fulfill the purpose for which they were built. Sometimes, there is also the need to improve the condition of the existing building to adapt them to a new function.

1.4. Structural Behavior and Properties of Material

According to the article "Recommendations for the Analysis, Conservation, and Structural Restoration of Architectural Heritage," the actual behavior of a building is usually too complex to model based on the required degree of precessions. The structural model shows how the building transforms actions into stresses and deformations and ensures stability. The behavior of the structure is influence by the quality of construction, material characteristics, action of forces, and the physical-chemical-biological actions.

Laurenco mentioned that masonry is a heterogeneous material that consists of units and joints. Units include bricks, blocks, ashlars, adobes, irregular stones, and others. Mortar can be clay, bitumen, chalk, lime or cement, glue or other. The huge number of possible combinations of the materials, nature and arrangement of units as well as the characteristic of mortar raises doubt about the term "masonry". Thus, it is very important to consider the properties of the masonry particularly its units and joints.

The bond between unit and mortar is often the weakest link in masonry assemblage. The non-linear response of the joints, which is then controlled by the unit-mortar interface, is one of the most relevant features of masonry behavior. Two different phenomena occur in a unit-mortar interface, one associated with tensile failure and the other one is shear failure. Different test set-ups have been used for the characterization of the tensile behavior on the unit-mortar interface. For the purpose of numerical simulation, direct testing should be used because it allows for the full representation of the stress-displacement diagram and yield the correct strength value.

The provision of International Building Code (IBC)of 2012 relating to the construction, repair, alteration, addition, restoration and movement of structures, and change of occupancy is not mandatory for historic buildings where such buildings are judged by the Building Official to not constitute a distinct life safety hazard.

1.5. Theory of Finite Element Method

Solid elements enable the solution of structural problems involving general three dimensional stresses. There is a class of problems such as stress distribution in concrete dams, soil and rock strata where finite element analysis using solid elements provides a powerful tool.

1.5.1. Theoretical Basis

The solid element used in STAAD is of eight noded isoparametric type as shown in Figure 4. These elements have three translational degrees-of-freedom per node.

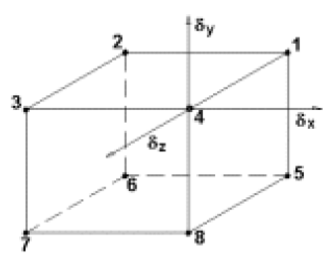

Fig. 4 – Solid Element in STAAD

The stiffness matrix of the solid element is evaluated by numerical integration with eight Gauss-Legendre points. To facilitate the numerical integration, the geometry of the element is expressed by interpolating functions using natural coordinate system, (r,s,t) of the element with its origin at the center of gravity. The interpolating functions are shown below:

$$x = \sum_{i=1}^{8} h_i x_{i,} y = \sum_{i=1}^{8} h_i y_{i,} z = \sum_{i=1}^{8} h_i z_{i,} \tag{1}$$

wherex,y and z are the coordinates of any point in the element and xi, yi, zi, i=1,..,8 are the coordinates of nodes defined in the global coordinate system. The interpolation functions, hi are defined in the natural coordinate system, (r,s,t). Each of r,s and t varies between -1 and +1. The fundamental property of the unknown interpolation functions hi is that their values in natural coordinate system is unity at node, i, and zero at all other nodes of the element. The element displacements are also interpreted the same way as the geometry. For completeness, the functions are given below:

$$u = \sum_{i=1}^{8} h_i u_{i,} v = \sum_{i=1}^{8} h_i v_{i,} w = \sum_{i=1}^{8} h_i w_{i,} \tag{2}$$

where u, v and w are displacements at any point in the element and ui,vi, wi, i=1,8 are corresponding nodal displacements in the coordinate system used to describe the geometry.

Three additional displacement "bubble" functions which have zero displacements at the surfaces are added in each direction for improved shear performance to form a 33x33 matrix. Static condensation is used to reduce this matrix to a 24x24 matrix at the corner joints.

1.5.2. Output of Element Stresses

Element stresses in STAAD may be obtained at the center and at the joints of the solid element.
The items that are printed are:

Normal Stresses : SXX, SYY and SZZ
Shear Stresses : SXY, SYZ and SZX
Principal stresses : S1, S2 and S3.
Von Mises stresses:

Direction cosines : 6 direction cosines are printed, following the expression DC,

$$SIGE = .707\sqrt{(S1-S2)^2 + (S1-S3)^2 + (S3-S1)^2} \tag{3}$$

corresponding to the first two principal stress directions.

2. Methodology

2.1. Ocular Inspection

Ocular Inspection was conducted at the site to document the existing condition of the structure including its non-structural components that may affect the performance of the structural investigation. During the ocular inspection, cracks in the masonry unit were observed in the wall and buttresses located at the right wing of the structure. These cracks were caused by alteration made in the wall and buttresses. Also, noticeable repair were made in some of the walls and buttresses. These interventions were indicated by a material that is not consistent with the old fabric.

2.2. Data Gathering

2.2.1. As-Built Plans

As-built plan were made using actual measurement and digital scanning of the entire structure. The results were encoded in AutoCAD and were transpired into floor plans, sections, and elevations. The shape of the footing including its dimension was taken by manual digging of the over-burden soil. Based on these procedures, 3D geometry of the structure was established, mesh into small solid elements, and analyzed using Finite Element Method using computer software, STAAD Pro V8i. Figure 5 shows the Front Elevation of the structure, Figure 6 is the Side Elevation, and Figure 7 is the Ground Floor Plan. The total length of the structure is 66.22 meters, width is 40.56 meters, and the height from ground to the top of the masonry is 14.38 meters. The outside dimension of the buttress is 3.69 meters in depth and 3.15 meters in width. The wall thickness is 3.18 meters.

Fig. 5 – Front Elevation

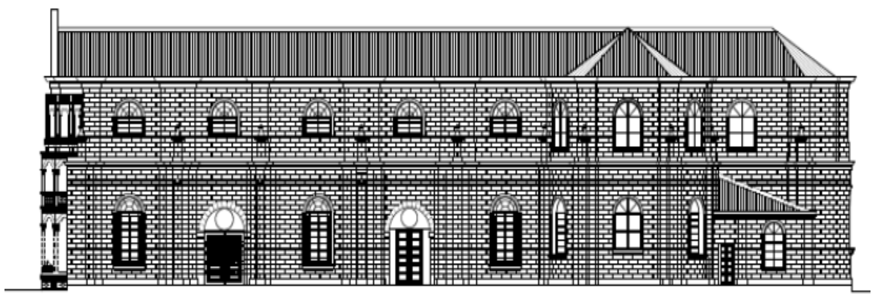

Fig. 6 – Side Elevation

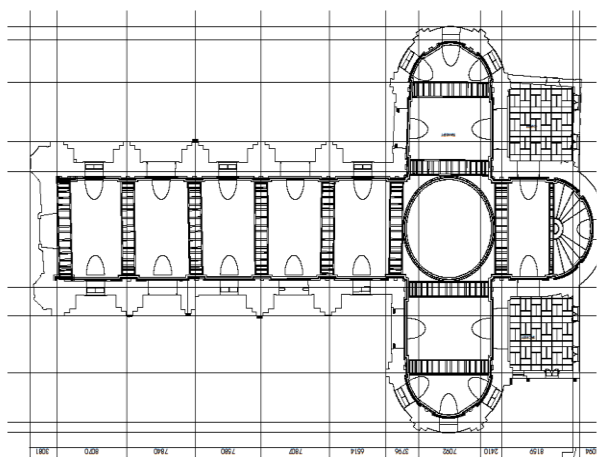

Fig. 7 – Ground Floor Plan

2.2.2. Material Testing

Sample of adobe stone blocks were extracted by coring to determine its compressive strength. Three inches diameter of core samples was used. A total of 21 samples were taken at strategic locations on the exterior side of the structure. The summary of the compressive strength test is presented in Table 1. Based on the results, the ultimate compressive strength of the adobe stone block ranges from 3MPa to 8MPa. The distribution of these values are, 24% is 3MPa, 14% is 4MPa, 29% is 5MPa, 24% is 6MPa, 5% is 7MPa, and 5% is 8MPa. This yields an average value of 4.86 MPa. This value shall be the basis to calculate the factor of safety against the actual stress in compression generated in the fabric due to the design loads.

The tensile strength shall be assumed 20% of its compressive strength. For unreinforced concrete, the tensile strength is calculated as 0.62√f'c that is usually 28% of its compressive strength. But for masonry, some Authors recommend 20% and thus, shall be used in this study.

Table 1 – Summary of Compressive Strength Test

Sample	Sample Identification	Compressive Strength (MPa)	Location
01	S,1	5.00	Right Side
02	S,2	6.00	Right Side
03	S,3	5.00	Right Side
04	S,4	4.00	Right Side
05	S,5	5.00	Right Side
06	S,6	3.00	Right Side
07	S,7	6.00	Right Side
08	S,8	3.00	Right Side
09	S,9	4.00	Right Side
10	S,10	5.00	Front Side
11	S,11	6.00	Front Side
12	S,12	5.00	Front Side
13	S,13	6.00	Front Side
14	S,14	8.00	Front Side
15	S,15	7.00	Front Side
16	S,16	3.00	Rear Side
17	S,17	3.00	Rear Side
18	S,18	4.00	Rear Side
19	S,19	3.00	Rear Side
20	S,20	6.00	Rear Side
21	S,21	5.00	Rear Side
	Average:	4.86	

2.3. Design Loadings

The bases of the Structural Investigation are the as-built plans, the results of material testing, and the design loads stated herein. All of these parameters shall be used to model the behaviour of the structure. State-of-the-art computer software shall be used as tool to determine the stresses and deformations under these loads.

2.3.1. Gravity Loads

Gravity load includes the design live load of the roof equal to 1.0 KPa and the weight of the structure computed using a density of stone equal to 16.00 KN/m^3. Other superimposed dead load such as ceiling and weight of other utilities were assumed to be acting at the roof structure equal to 0.30 KPa.

2.3.2. Earthquake Loads

The equivalent earthquake force acting in the structure was auto-generated by the software, STAAD Pro V8i. The seismic force was based on the nearest seismic source, the Valley Fault System, that is approximately 13 kM from the site. The equivalent force acting at the base of the structure was computed based on the formula,

$$V = C_V IW / RT \leq 2.5 C_a IW / R \tag{4}$$

$$\geq 0.11 C_a IW \tag{5}$$

$$\geq 0.80 Z N_V IW / R \tag{6}$$

It is assumed that the Soil Profile Type is Sd, importance factor, I is 1.0, and the numerical coefficient, R is 4.50. The computed base shear is 22780 kN or approximately 22 percent of the total dead weight acting in both major orthogonal directions of earthquake.

2.3.2. Wind Loads

The equivalent design wind force was computed based on the wind speed, V = 200 kph and Exposure D. The design wind pressure was computed as,

$$P = q_h \left[\left(GC_{pf} \right) - \left(GC_{pi} \right) \right] \quad (7)$$

Based on these parameters, the total wind force acting laterally in the structure is 1885 KN with an average pressure of 1.5 KPa and this is less than the seismic force.

2.4. Geometry of the Structure

System hence; a numerical coefficient is equal to 4.5. Also, the interaction between the footing and the soil are neglected thus, pin support will be used in the analysis.

2.5. Method of Analysis

The structural analysis shall consider two important parameters in order to come up with sound engineering conclusions regarding the safety and serviceability of the structure. These are strength versus actual stress and actual deformation versus tolerable deformations. The Factor of Safety shall be computed as the ultimate strength of the material divided by the actual stress generated in the fabric due to the design loads. Also, the relative deformations of the solid elements shall be plotted and shall be compared to the elastic displacement of the solids elements.

The building structure shall be analyzed as solid elements and shall be simulated in three-dimensional space. The geometry of the structure is shown in Figure 1. This structure shall be subjected to design loads stated above to determine the magnitude of stress in the fabric. It is assumed that the structural behavior would be similar to that of a Bearing Wall.

3. Results and Discussions

Based on the results of the analysis, it was noted that the stress occurs in the buttresses are greater than that in the walls. This implies that the buttress being stiffer than the wall will absorb more forces than the walls. Also, the maximum stress in the buttresses occurs within its interface from the footing. This behaviour of the buttress means that it cantilevers from the footing with the action of lateral forces. The illustration of stresses due to combined gravity and lateral forces due to earthquake are presented in Figures 8 to 9. Figure 8 is the stress contour of normal compressive stress due to load combination 0.45DL+0.54EQZ, and Figure 9 is the stress contour of normal tensile stress due to load combination 0.75DL+0.56LL-0.54EQZ.

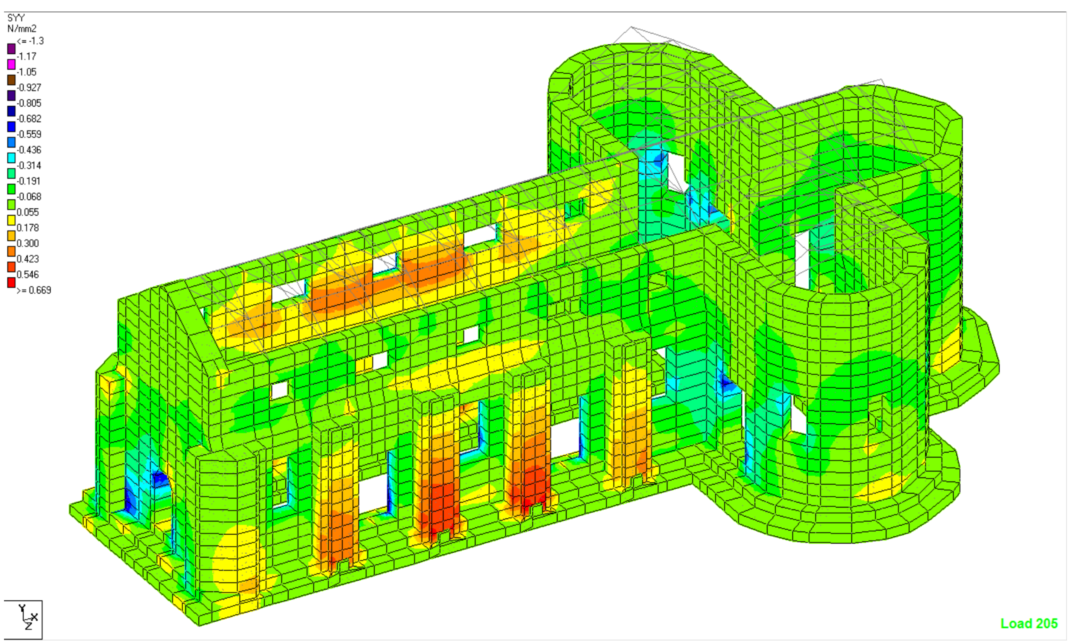

Fig. 8 – Stress Contour of Normal Compressive Stress Due to LC 208 0.45DL+0.54EQZ

Table 2 – Summary of Maximum Normal Compressive Stress at Every Layer

Wall Layer	Governing Load Case	Grid Location	Maximum Compressive Stress (Mpa)	Compressive Strength (Mpa)	Factor of Safety	Conclusion
Walls						
19	206 0.45DL+0.54EQX	L/6	-0.250	4.860	19.440	SAFE
18	206 0.45DL+0.54EQX	K-L/6	-0.132	4.860	36.818	SAFE
17	212 0.75DL+0.54EQZ	L/6	-0.136	4.860	35.735	SAFE
16	205 0.75DL+0.56LL-0.54EQZ	L/6	-0.176	4.860	27.614	SAFE
15	205 0.75DL+0.56LL-0.54EQZ	L/6	-0.251	4.860	19.363	SAFE
14	201 DL+LL	H/6	-0.089	4.860	54.607	SAFE
13	202 0.75DL+0.56LL+0.54EQX	G/5	-0.101	4.860	48.119	SAFE
12	210 0.75DL+0.54EQX	K/5	-0.156	4.860	31.154	SAFE
11	203 0.75DL+0.56LL-0.54EQX	J/6	-0.292	4.860	16.644	SAFE
10	213 0.75DL-0.54EQZ	I/5	-0.249	4.860	19.518	SAFE
9	203 0.75DL+0.56LL-0.54EQX	I/5	-0.332	4.860	14.639	SAFE
8	212 0.75DL+0.54EQZ	J/6	-0.341	4.860	14.252	SAFE
7	212 0.75DL+0.54EQZ	J/6	-0.489	4.860	9.939	SAFE
6	212 0.75DL+0.54EQZ	I/6	-0.339	4.860	14.336	SAFE
5	213 0.75DL-0.54EQZ	K/5	-0.403	4.860	12.060	SAFE
4	212 0.75DL+0.54EQZ	I/6	-0.508	4.860	9.567	SAFE
Buttresses						
14	205 0.75DL+0.56LL-0.54EQZ	L/6	-0.293	4.860	16.587	SAFE
13	205 0.75DL+0.56LL-0.54EQZ	L/6	-0.226	4.860	21.504	SAFE
12	211 0.75DL-0.54EQX	F/2	-0.266	4.860	18.271	SAFE
11	210 0.75DL+0.54EQX	E/9	-0.439	4.860	11.071	SAFE
10	213 0.75DL-0.54EQZ	D/4	-0.349	4.860	13.926	SAFE
9	213 0.75DL-0.54EQZ	D/4	-0.529	4.860	9.187	SAFE
8	203 0.75DL+0.56LL-0.54EQX	F/9	-0.572	4.860	8.497	SAFE
7	205 0.75DL+0.56LL-0.54EQZ	J/7	-0.595	4.860	8.168	SAFE
6	205 0.75DL+0.56LL-0.54EQZ	J/7	-0.681	4.860	7.137	SAFE
5	205 0.75DL+0.56LL-0.54EQZ	J/7	-0.756	4.860	6.429	SAFE
4	205 0.75DL+0.56LL-0.54EQZ	J/7	-0.805	4.860	6.037	SAFE
Foundation						
3	205 0.75DL+0.56LL-0.54EQZ	J/7	-0.826	4.860	5.884	SAFE
2	205 0.75DL+0.56LL-0.54EQZ	I/7	-1.204	4.860	4.037	SAFE
1	205 0.75DL+0.56LL-0.54EQZ	I/7	-0.805	4.860	6.037	SAFE

In addition, the distribution of maximum normal stresses acting in the solid elements with respect to its elevation from the base of structure is obtained. This is to assess the relationship between the stress in the solid elements to its elevation. These tabulated results are shown in Tables 2 and 3. Also, the results of stresses in the buttress and walls were calculated separately for comparison.

3.1. Buttress

The maximum stress generated in the buttress at the lower most fabric of the structure is 0.805 MPa in compression and 0.537 MPa in tension. These values are less than the ultimate strength of the fabric yielding a factor of safety of 6.03 in compression and 1.81 in tension. At the top most element of the fabric, the stress generated at the buttress is 0.293 MPa in compression and 0.094 in tension. This yields a factor of safety equal to 16.58 in compression and 10.34 in tension. The results above are based on the assumption that the tensile strength is twenty percent of the compressive strength of the material similar to ordinary rocks or unreinforced concrete.

3.2. Walls

The maximum stress generated in the walls at the lower most fabric of the structure is 0.508 MPa in compression and 0.135 MPa in tension. These values are less than the ultimate strength of the fabric yielding a factor of safety of 9.56 in compression and 7.2 in tension. At the top most element of the fabric, the stress generated at the walls is 0.25 MPa in compression and 0.27 in tension. This yields a factor of safety equal to 19.44 in compression and 3.6 in tension. Similarly, these results are based on the assumption that the tensile strength is twenty percent of the compressive strength of the material similar to ordinary rocks or unreinforced concrete.

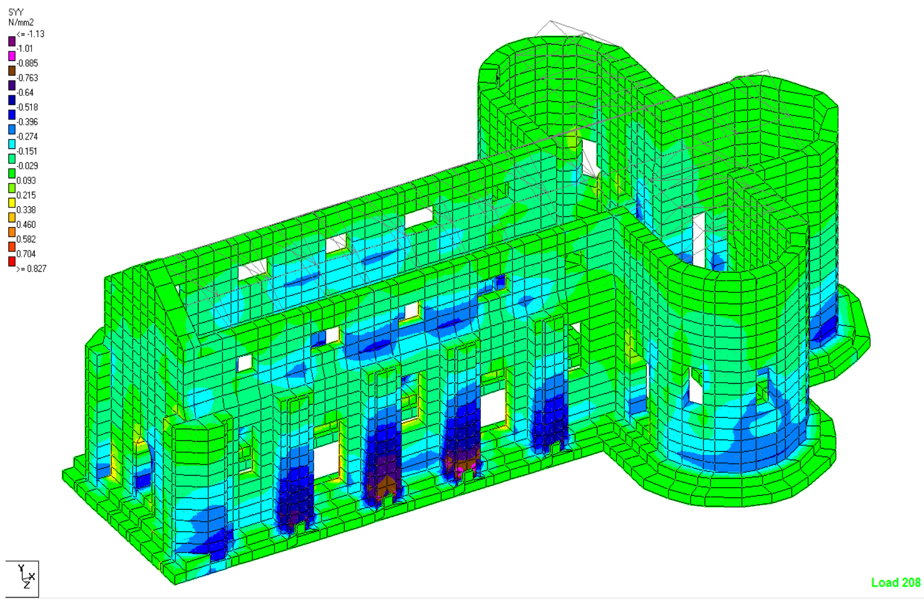

Fig. 9 – Stress Contour of Normal Tensile Stress Due to LC 205 0.75DL+0.56LL-0.54EQZ

Table 2 – Summary of Maximum Normal Tensile Stress at Every Layer

Wall Layer	Governing Load Case	Grid Location	Maximum Tensile Stress (Mpa)	Tensile Strength (Mpa)	Factor of Safety	Conclusion
Walls						
19	203 0.75DL+0.56LL-0.54EQX	L/6	0.270	0.972	3.600	SAFE
18	211 0.75DL-0.54EQX	K-L/6	0.157	0.972	6.191	SAFE
17	211 0.75DL-0.54EQX	K/6	0.072	0.972	13.500	SAFE
16	207 0.45DL-0.54EQX	K-L/6	0.071	0.972	13.690	SAFE
15	206 0.45DL+0.54EQX	H-I/5	0.107	0.972	9.084	SAFE
14	210 0.75DL+0.54EQX	H/5	0.058	0.972	16.759	SAFE
13	208 0.45DL+0.54EQZ	K-L/6	0.059	0.972	16.475	SAFE
12	210 0.75DL+0.54EQX	H/5	0.060	0.972	16.200	SAFE
11	206 0.45DL+0.54EQX	J/5	0.066	0.972	14.727	SAFE
10	207 0.45DL-0.54EQX	J-K/5	0.110	0.972	8.836	SAFE
9	206 0.45DL+0.54EQX	I/5	0.069	0.972	14.087	SAFE
8	208 0.45DL+0.54EQZ	J/5	0.091	0.972	10.681	SAFE
7	208 0.45DL+0.54EQZ	J/6	0.161	0.972	6.037	SAFE
6	208 0.45DL+0.54EQZ	J/6	0.093	0.972	10.452	SAFE
5	208 0.45DL+0.54EQZ	I/5	0.106	0.972	9.170	SAFE
4	208 0.45DL+0.54EQZ	I/6	0.135	0.972	7.200	SAFE
Buttresses						
14	208 0.45DL+0.54EQZ	L/6	0.094	0.972	10.340	SAFE
13	208 0.45DL+0.54EQZ	L/6	0.067	0.972	14.507	SAFE
12	206 0.45DL+0.54EQX	E-F/2	0.117	0.972	8.308	SAFE
11	209 0.45DL-0.54EQZ	J/4	0.163	0.972	5.963	SAFE
10	209 0.45DL-0.54EQZ	J/7	0.227	0.972	4.282	SAFE
9	209 0.45DL-0.54EQZ	J/7	0.290	0.972	3.352	SAFE
8	208 0.45DL+0.54EQZ	J/7	0.353	0.972	2.754	SAFE
7	208 0.45DL+0.54EQZ	J/7	0.413	0.972	2.354	SAFE
6	208 0.45DL+0.54EQZ	J/7	0.467	0.972	2.081	SAFE
5	208 0.45DL+0.54EQZ	J/7	0.512	0.972	1.898	SAFE
4	208 0.45DL+0.54EQZ	J/7	0.537	0.972	1.810	SAFE
Foundation						
3	208 0.45DL+0.54EQZ	J/7	0.536	0.972	1.813	SAFE
2	208 0.45DL+0.54EQZ	I/7	0.705	0.972	1.379	SAFE
1	208 0.45DL+0.54EQZ	I/7	0.465	0.972	2.090	SAFE

The results show that the fabric is still strong against the expected load generated by earthquake or wind that may occur at the site. The weakness of this fabric is similar to unreinforced rock or concrete that is tension. Nevertheless, these stresses are below than the ultimate strength of the fabric yielding a factor of safety between 1.81 to 16.75 in tension and between 6.03 to 54.60 in compression.

3.3. Conclusion and Recommendations

Based on the results of the structural analysis, and based on the design assumptions and material testing, the Consultant concludes that the structure can sustain the design loads stated in this Final Report without strengthening the original structure. However, all the cracks and excessive deterioration in the masonry unit and mortar should be restored properly. Also, since by nature, this structure has a bad response to earthquake, do not use this structure as evacuation center in times of disaster.

4. References

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