SEISMIC VULNERABILITY OF LIMA CATHEDRAL, PERU

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ABSTRACT :

The building of Lima's cathedral started in 1535, with the Spanish foundation of the city. Since then, the structure has experienced at least sixteen major earthquakes and has been reconstructed several times. Some of these reconstructions introduced important structural changes, particularly after the 1746 earthquake, which have reduced its seismic vulnerability, but not completely solved all structural deficiencies.

The Cathedral has an area of 5020m^2. It has three central naves 21m high and two lower lateral naves, corresponding to the chapels. Thick masonry walls and 14 central wood framed pillars support the roof wood structure. The cathedral has two masonry towers with a height of 45m. The total weight of the structure is about 30,400 metric tons.

This paper presents results of the earthquake response analysis of this building carried out within a vulnerability study made at the Japan-Perú Earthquake Engineering and Disaster Mitigation Center (CISMID).

The most vulnerable components of the structure are the two towers and other masonry elements in the front wall and the buttresses located near the lateral threshold. Numerical simulations show that for the design earthquake the displacements are within acceptable limits, but tensile and shear stresses exceed the capacity of the unreinforced masonry.

KEYWORDS: Seismic Vulnerability, Historical Building, Masonry and Wood Structure Evaluation

1. INTRODUCTION

The lot assigned to the Cathedral was the first to be traced by the Spanish conqueror Francisco Pizarro on the day of the foundation of Lima on January 18th, 1535. Since then, at least sixteen major earthquakes have affected the structure to some degree; however, four of them which occurred in 1609, 1630, 1687 and 1746, affected the building to such an extent that required reconstructions which involved changes in the structural configuration of the roof system and the materials used.

Figure 1a shows the original configuration, with masonry vaults which rested on oval arches. After the 1609 earthquake the Cathedral was reconstructed with full centered arches, Figure 1b. Figure 1c shows the flying buttresses which were added after the earthquake of 1687 to improve the lateral stiffness. After the great earthquake of 1746 the pillars and the roof structure were completely rebuilt in wood, Figure 1d. In 1898, a complete reconstruction of the roof structure (vaults, arches and horizontal frame) was carried out. For the first time in the history of the Cathedral, changes in the structure were not due to earthquakes but to the attack of termites and humidity, which had caused great deterioration of the wood elements. Figure 1e depicts the current configuration of the structure. The flying buttresses were removed after 1940.

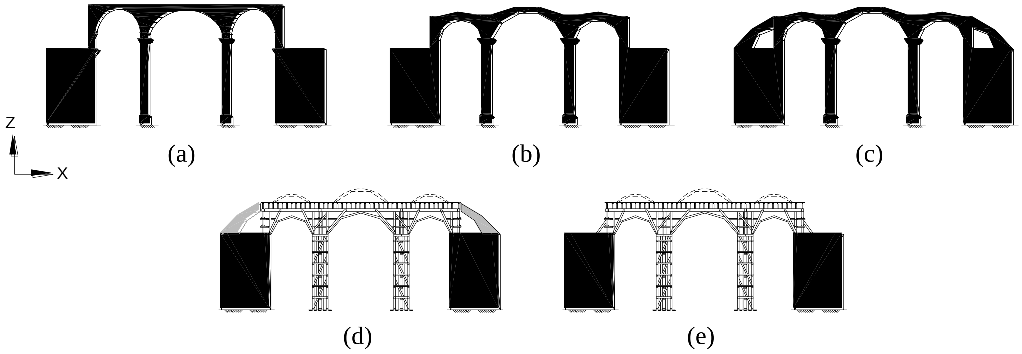


Figure 1. Cross sections of the building showing structural changes in Cathedral. a) Until the earthquake of 1609, b) Before the earthquake of 1687, c) Before the earthquake of 1746, d) After the 1746 earthquake, e) Present configuration

Nowadays, the structure has some deterioration of the wood components, mainly as a result of environmental effects and poor maintenance. There are also structural deficiencies which were never completely corrected in successive reconstructions, mainly the lack of adequate stiffness in longitudinal direction at the roof level and insufficient capacity of the bracing to transfer seismic loads in transverse direction from the roof to the masonry buttresses.

Development of an effective methodology for future repairs and strengthening of upon important historical constructions require an integration of knowledge at least in the areas of seismology, geotechnics, structural engineering, material science, architecture, art as well as social, cultural and economic aspects. This paper presents preliminary results of the earthquake response analysis of this historical building carried out within this process.

2. ABOUT THE CATHEDRAL

The Cathedral of Lima has a built area of about 5020m^2. It has three central naves with heights that reach 21m and two lateral naves, corresponding to the chapels, of 11m. Adjacent to the main structure, there are another church and a museum, which share some walls with the cathedral. The foundation of the building is that of the initial, much heavier, structure of bricks and lime mortar.

The brick masonry walls have variable thicknesses, from 1.20m to 2.40m. Their height is 13m, except the front wall and the buttresses located in the transept area which are 18m high. The cathedral has two front towers, each one of about 100m^2 of area. They are brick masonry structures 45m high, with 2.40m thick walls.

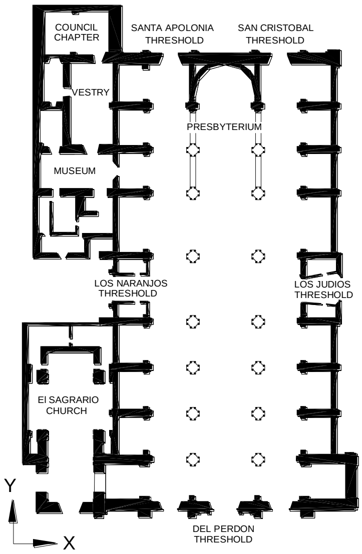
The walls of the transept and the lateral chapels act as buttresses of the longitudinal and rear walls, providing adequate lateral stiffness, as was demonstrated in several earthquakes in the past. This is not the case for the front walls and the two masonry towers, which were severely damaged during every important seismic event.

Fourteen 13m high wood framed pillars consist of eight columns, arranged in a cross shaped section, with horizontal and diagonal elements and an external cover made of wood, cane and plaster (figures 2b, 2d and 2e). These wood columns are not rigidly attached to the foundation; in some cases they penetrate up to 10cm into the foundation, but in others they only rest on top of it.

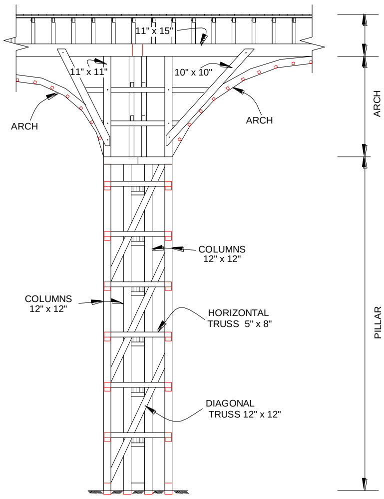
The main roof structure is composed of 26 wood truss vaults and a horizontal wood framework, with full centered wooden arches which are supported directly on the pillars and walls. The 14 lateral vaults that correspond to the chapels and the 4 vaults located next to the rear wall are still made of brick masonry with lime mortar. The roof cover is made of wood and mud, which is enough for the weather conditions in Lima.

(a)

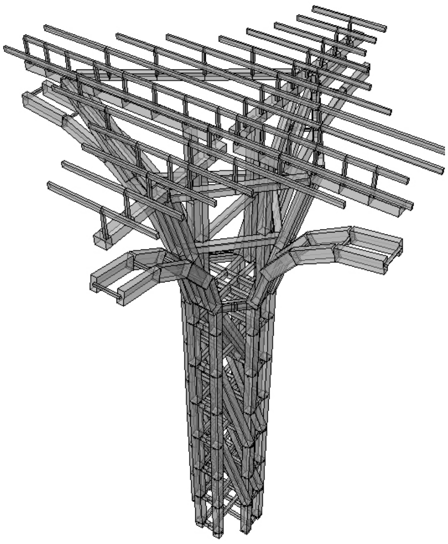
(b)



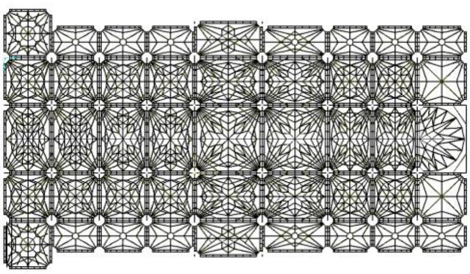
(c)



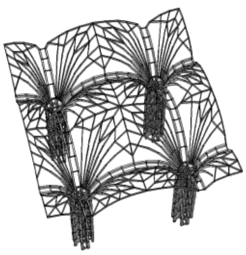
(d)



(e)



(f)



(g)

Figure 2 a) Front view of the Cathedral, b) Internal view of the structure c) Plan view of the structure, d) Elevation of wooden pillar, e) Isometric view of pillar, f) Plan view of wooden truss elements in vaults of building, g) Isometric view of a single vault.

3. STATE OF THE STRUCTURE

(a) (b) (c) (d)

Figure 3 a) Poor maintenance of vaults and buttresses above chapels, b) Cracking in buttress at the crossing of the building, c) Biological attack in wood elements located between the vaults and roof framework d) Cracking in wood element above one pillar.

Defects such as cracks and material deterioration were often repaired aiming only at aesthetics, rather than correcting the structural causes of damage. It is then important to make an exhaustive review of the structural elements and contrast this information with the analysis results.

One of the most visible defects is the lack of verticality of the wood framed pillars. They have horizontal displacement in both directions, with values at the top ranging from 4cm to 15cm (in 13m) with maximum values in the central part of the building. There is no evidence of differential settlement in the walls.

Diagonal cracks were observed in the covers of most pillars. There is also visual evidence of deterioration and cracks in some of the wood elements located between the vaults and the roof framework structure. There is evidence of humidity and termite attack.

Most masonry walls do not have signs of distress, although the towers, the front walls and adjacent areas have always been affected in past earthquakes. However, there are important vertical cracks in the upper portion of the buttresses located in the crossing area of the building.

4. SOIL CONDITIONS

Lima's cathedral stands on firm soil formed from well graded sediments like gravel, sands and pebbles of alluvial nature, coming from resistant rocks like granodiorite, gabbrodiorite, diorite and granite. The thickness of this dry and compacted material is estimated as several hundred of meters. For the purpose of the seismic analysis the soil profile is qualified as rigid, with a characteristic period less than 0.4s.

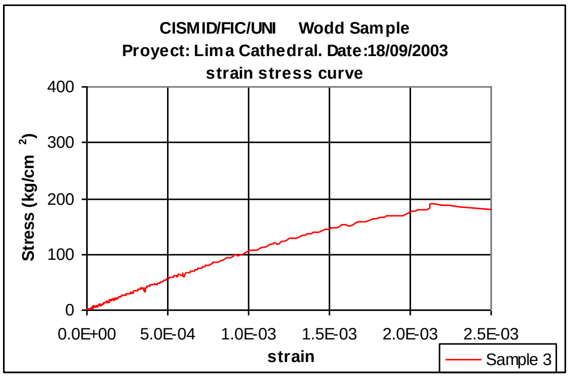
5. MATERIAL PROPERTIES

All walls in the building were made with brick masonry and lime mortar. In order to quantify the strength and stiffness of the brick masonry in compression, tests on piles of 10.8cm x 9.3cm of area were performed. The samples were taken directly from the buttress walls. The density of this material is of the order of 1.7g/cm^3. The average compression strength of the piles is 2.2MPa (22.6kg/cm^2) and its modulus of elasticity is about 1,100MPa (11,300 kg/cm^2).

Table 1 shows test results of wood samples from the reticular structures of the pillars. Some samples were rejected due to severe (but non active) termite attack. The wood has an average air-dry density of 0.7g/cm^3. This value corresponds to the limit between structural groups A and B in the Peruvian wood design code. An elastic modulus of 7,355MPa (75,000kg/cm^2).was used in the analyses; which is the minimum specified for species group B. The average compression strength parallel to the grain obtained from the tests was 32.3MPa (329 kg/cm^2). The allowable stress for compression parallel specified for group B in the Peruvian code is 10.8MPa (110 kg/cm^2).

Table 1. Parallel to fiber compression test of wood samples

	Area (cm^2)	Height (cm)	P max (KN)	Strength (Mpa)
Sample 1	5.98	4.94	20.68	34.60
Sample 2	5.88	5.20	15.09	25.67
Sample 3	5.60	5.15	20.51	36.63



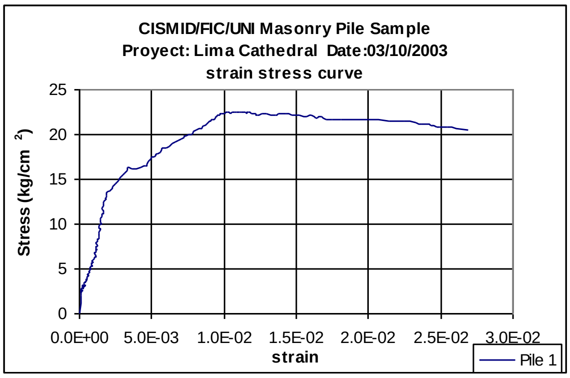


Figure 4 Stress-strain diagrams for a wood sample and a masonry pile

6. MICROTREMOR MEASUREMENTS

Microtremor measurements were used to calibrate the numerical analysis model. Due to the flexibility of the wood vaults and horizontal framework, the microtremor measurements were done over the whole structure. From measurements on the wood horizontal framework between the vaults the average fundamental frequency in transverse (X) direction was found to be 2.6 Hertz (T=0.39s), while in the longitudinal (Y) direction was 3.6 Hertz (T=0.28s). These values correspond to the area located in the crossing of the building. The Fourier transforms obtained from measurements in Y direction near the façade front wall have peaks at 2.6 Hertz (0.38s), which indicates that this wall is insufficiently braced by the rest of the building. At the top of the towers the peaks in Fourier transforms are at 1.8 Hertz (T=0.56s) in both directions. Despite their thick walls, the towers are very flexible in comparison to the main building.

7. STRUCTURAL MODEL

A special effort was made for the development of an appropriate mathematical model for the analysis of this building of complex geometry and multiple materials. The building mainly consists of masonry brick walls and towers, and wood framed pillars, arches, vaults and horizontal roof framework. A linear elastic analysis was considered a reasonable tool for the structural investigation, providing at least a basic understanding of the building response.

Figure 5 depicts the mathematical model for the analysis. 18,788 frame elements were used to represent wood elements from pillars, arches, vaults and the horizontal roof framework. The walls and the roof covers were modeled with 71,166 two dimensional shell elements.

The structural walls of the building are composed of at least two materials: brick and lime mortar. Although they are strictly non-homogeneous and anisotropic, they were modeled as homogeneous and isotropic, with equivalent linear properties based on tests. The walls were supposed fixed at their base. This is consistent with the observation that; even for extreme conditions, there are not vertical tensile stresses at the base of the walls

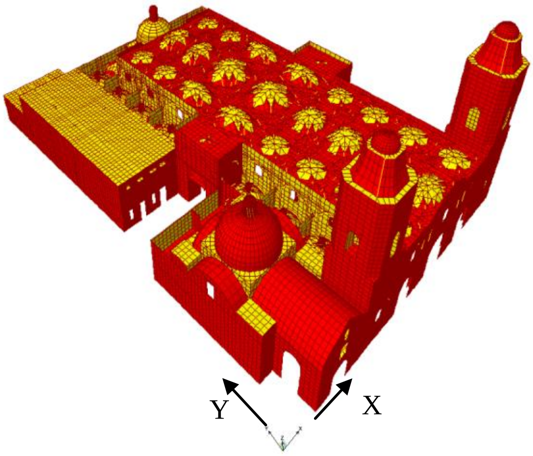


Figure 5 Mathematical model of Lima's cathedral

Special attention was given to the connections between wood elements. These connections are made with spikes or, in many instances, as a direct geometric assembly of appropriately cut wood pieces. They are a significant source of nonlinear behavior. Since the computer program used is limited to linear analysis, a sensibility study was made, comparing the effects obtained after considering different releases at the joints, including the possible uplift of the wood columns in the pillars.

The adequacy of the model was corroborated by its ability to predict not only the results of the microtremor tests but also to identify the portions of the structure which may be more severely stressed by earthquake loads, which are in agreement with the damage observed in past events.

8. NUMERICAL ANALISIS

An elastic analysis of the building was carried out for dead loads, relatively small live loads and earthquake effects. Wind loading is not important in this case. The analysis provided essential information about stress distributions and an estimate of the seismic response of the structure, helping to identify the most vulnerable zones in the building and to interpret the existent damage. The seismic analysis was based on a seudo acceleration spectrum as defined in the Peruvian seismic code, with the following parameters: Z=0.4 (zone factor), U=1.3 (importance factor), S=1.0 (soil factor), $C=2.5T_p/T \leq 2.5$ (dynamic amplification factor), R=3 (reduction factor).

The axial forces in the wood columns of the pillars inside the Cathedral, due to the combined effects of gravity and earthquake loads, reach maximum values of 392kN (40t), which is well below the allowable stress in compression and both local or global buckling loads. The lateral displacements at the roof level imply the rotation of the pillars, with an uplift of some of its wood columns. This uplift causes a redistribution of stresses in the wood framework, but does not imply the collapse of the pillars. However, the design earthquake produces shear stresses in the plaster cover of the pillars as large as 0.28MPa (2.85kg/cm^2). Even for moderate earthquakes the small capacity of this fragile cover is exceeded; this agrees with the observed damage.

None of the analysis could completely explain the large horizontal displacements at the top of the pillars. The analysis predicts maximum lateral displacements of 6.8cm in the area of the cruise and of 7cm in the facade wall, in the transverse (X) and longitudinal (Y) directions, respectively (see figure 9). The observed, permanent, maximum displacements are of the order of 15cm. They are obviously related with the insufficient lateral stiffness at roof level and most probably are the result of accumulated displacements, which were not completely corrected in successive reconstructions of the roof of the cathedral.

The vertical loads on the wood arches are not important, since the vaults are directly supported on the pillars; but earthquake loads produce significant bending of the arches. Hence some connection act as hinges; nevertheless, the arches still support axial loads, for which they have good capacity. The wood vaults perform well because of their low weight, geometry and the confinement provided by the existing wood boards.

Most of the masonry components of the building have earthquake shear stresses which are well below their allowable capacity. However, there are some exceptions, described in what follows.

The towers of the cathedral are flexible (their fundamental vibration period being 0.6 seconds). The maximum shear stress will occur in the zone of the belfry, with values between 0.19MPa and 0.24MPa (2.0kg/cm^2 and 2.5kg/cm^2). These exceed the capacity of the unreinforced masonry. As a matter of fact, the towers have been severely damaged in all major earthquakes in the past, affecting also the adjacent areas of the structure.

The front wall of the cathedral has its major weakness in out of plane direction. This wall is connected to the wood roof structure by four arches, located in pairs at the heights of 7m and 13m, which provide some lateral bracing. The analysis predicts an out of plane displacement of the order of 7cm for the design earthquake. The maximum in plane shear stress in this wall is 0.24MPa (2.5 Kg/cm^2).

The buttresses in the crossing zone next to “Los Judíos” threshold are very flexible due to their considerable mass and 18m height. When treated as separate structures, these buttresses have a fundamental period of about 0.45 seconds, which is larger than the 0.39 s obtained for the complete building. This means that they do not effectively restrain the lateral displacement of the roof structure, but rather impair its performance.

9. CONCLUSIONS

Successive reconstructions of Lima’s cathedral introduced important structural changes, which have reduced its seismic vulnerability, but not completely eliminated some structural deficiencies. One such deficiency is in the lateral stiffness of the roof, which is linked with the lateral displacement at the top of the pillars and the cracking of their plaster covers. The lack of adequate maintenance and the elimination of the flying buttresses have also resulted in a deterioration of the Cathedral’s seismic performance. An adequate plan to protect and replace some structural wood elements is necessary.

However, the most vulnerable parts of the structure are the two towers and other masonry elements in the front wall and the buttresses located near the lateral threshold. The two towers and adjacent walls have been severely damaged in every important earthquake in the past. Numerical analysis shows that tensile and shear stresses in the towers due to the design earthquake exceed the material capacity. The out of plane instability of the front wall and the structural damage in the buttresses at the lateral threshold are other aspects that require corrective measures.

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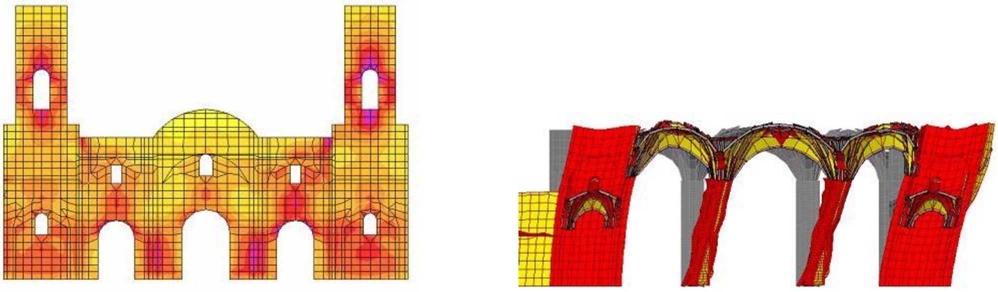

(a) (b)

Figure 6 a) Shear stress concentration in facade wall and towers b) Deformed shape of the structure due to earthquake component in transverse (X) direction.

Figure 7 Vibration modes: (a) 16th mode T=0.6s (b) 31st mode T=0.45s (c) 34th mode T=0.36s

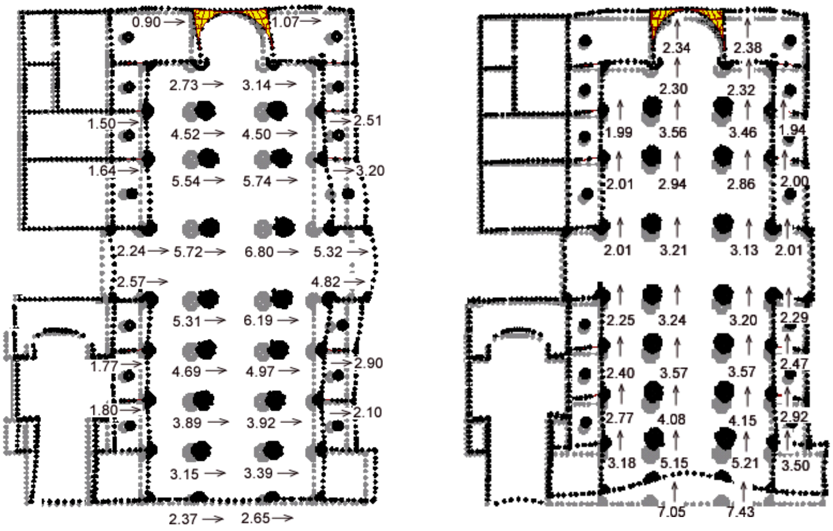

Figure 8 Lateral displacements at the top of the pillars due to earthquake components in transverse (X) and longitudinal (Y) directions.