



Seismic rehabilitation of cathedral towers in Peru

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Abstract

On June 23, 2001, an earthquake of magnitude 8.3 occurred at 1100 Kms south of Lima and about 250 Kms to the west of Arequipa . The four hundred year-old city of Arequipa, whose historical downtown was declared World Cultural Heritage by UNESCO, is known as the “White City” because most of the city is built with a volcanic stone called “sillar”. This kind of stone exhibits a mostly white-gray color. Churches, monasteries and houses are built completely with this stone, which gives the city its characteristic appearance. The cathedral of Arequipa, located in the city’s main square, is the most important neo-classical monument of Peru. It has a 100 m-long façade and two towers at the top of the building having a height of 28 m. Due to the effects of the earthquake of 2001, both towers suffered extensive damage. As a consequence, the left tower partially collapsed, whereas, the right tower remained standing but in an unstable equilibrium condition. This paper describes the emergency work performed to stabilize the right tower with an internal steel structure and the process of strengthening and rebuilding of both towers. The left tower was rebuilt with internal reinforced concrete members. The right tower was strengthened with Carbon Fiber Reinforced Polymer (CFRP) laminates, which were used to provide tensile strength and confinement to the central stone core of the tower. After completing the CFRP installation, carved stones were placed on top of the laminates to keep the original appearance.

1. Introduction

The earthquake of June 23rd, 2001, that affected most of the southern part of Peru, brings out once again the problem of the seismic vulnerability of the cultural heritage of the country. The historical center of the city of Arequipa was

heavily affected by the earthquake, with forty percent of its representative buildings damaged from moderate to severe with partial collapse.

At the time of the earthquake, there was a special office within the municipal government in charge of the historical downtown. This office was created with the economical and technical support of the Spanish Agency for International Cooperation and it was working on a master plan to recover the historical downtown. The Cultural National Institute (INC), with its main office in the capital city of Lima and its branch in Arequipa, is the governmental agency responsible for preserving the national cultural heritage. Later on, the municipal government was in charge of administrating the funds for the task of rehabilitating the public infrastructure creating a new office that was at last the one in charge of the reconstruction project of the Arequipa cathedral.

2. Historical and structural background

The cathedral church of Arequipa was first built in 1629, and is the building that symbolizes the character of the city and its inhabitants. Several reasons make the cathedral the most representative building of the city at a national and international level: it is the highest building in the urban downtown, it is located in the heart of the city, it is built with a volcanic stone from the nearby Misti volcano called "sillar," and it is the silent witness of important historical events.

A main feature of this monument is its towers, 28 meters high above the top of the church body, being an important visual reference in the urban and rural surroundings. It is precisely the slenderness of the towers that makes them vulnerable to the earthquake effects, very common in the region. Along its history, the towers have suffered damage and partial collapses. The 1868 earthquake caused the partial collapse of both towers which were then demolished to rebuild them with the actual architectural configuration (Fig.1). In the 20th century, the earthquakes of 1958, 1960 and 1979, caused cumulative damage until 1983, when the Development Corporation, CORDEAREQUIPA, in agreement with the INC of Arequipa, carried out a joint rehabilitation project for the towers, in light of the fact that: "*...surveys conducted by structural experts show that the towers will not be able to stand another earthquake because of its severe cracking*".

The rehabilitation project was limited to the four pillars and arches directly on the roof of the church. The transition zone was not reinforced resulting in the collapse of the upper part of the left tower during the earthquake of 2001.

For a better understanding of the damage the tower is divided into three bodies: the first body comprises the pillars and arches directly above the roof of the church, until the first friso, it has a height of 6.5 m., each pillar has a cross section of 2.5 by 2.5 m., making this first body very rigid. The second body comprises the transition zone from the first cornisa until the base of the upper arches with an approximate height of 6.7 meters, the footprint diminishes its dimensions from 7 by 7 m. to 4.5 by 4.5 m.. The third body comprises the upper arches until the last cornisa, right before the light metal structure at the top. The approximate height of the third body is 5.1 meters.

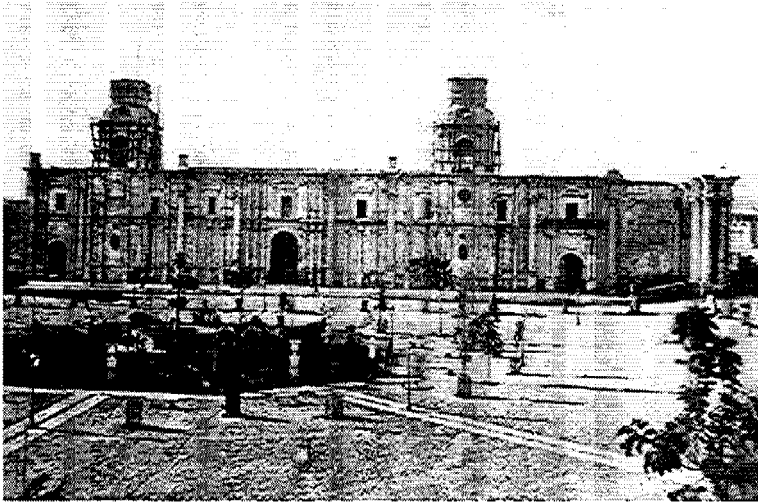


Figure 1: Reconstruction after 1868

In the year 1940, the towers were strengthened with four reinforced concrete (RC) columns in the interior corners of the first body pillars, connected with two RC beams placed diagonally. From these beams, and from brackets at the top of these columns, they were extended out of vertical alignment until the base of the third body where two other crossed beams were placed. Finally, at the top of the third body, a new level of crossed RC beams tied the arches of this body. By 1983, the most damaged parts were the pillars at the first body with cracks that reached widths of 8, 10 cm. and more, including the sillar stones out of position. In the left tower, the cracks were of minor importance than those at the right tower. Additionally, it was reported that in the transition zone, cracks and movements of the sillar stones, which were of greater importance in the left tower, were detected. This tower was the one that collapsed in the earthquake of June 23, 2001.

The strategy for structural reinforcement at that time, consisted of placing two additional RC columns in each of the four bottom pillars. The columns were placed in the right angle corners, leaving without reinforcement the exterior corner in which round sillar pillars existed. An RC collar beam was inserted in each of the four pillars at a height coincident with the base of the arches, and another collar beam was also inserted at a higher level around the whole first body of the tower. In the transition zone between the first and third body there are four steel columns adjacent to the reinforced concrete columns, not having information about when and why these were placed.

3. Damage assessment and structural behaviour in the 2001 Earthquake

Due to their slenderness ratio, and being located at the top of the church main body, the towers of the cathedral can be considered as appendices of the main structure. This implies that their dynamic behavior is almost independent of that of the main structure, but with bigger dynamic amplification (Fig.2). Based on this characteristic, the dynamic analysis of the towers was performed independently of the main structure and with a fixed base as a boundary condition.

The third body, due to its lesser plan dimensions, behaves in a certain way, also as an appendix of the rest of the tower. This brings as a consequence a stress concentration at the base of the third body where, due to the tensile stresses and the horizontal acceleration, the sillar stones fell out of position causing ultimately the collapse of the left tower. This behavior is congruent with the damage observed from past earthquakes, that were documented in 1983. The third body of the left tower collapsed as a rigid body on top of the roof of the church causing a partial roof collapse (Figs. 3 and 4).



Figure 2: Precise moment of Earthquake. June 2001



Figure 3: Left Tower Damage



Figure 4: Right Tower Damage

The right tower also suffered extensive damage, being the most dramatic, the loss of section due to the falling of the sillar stones, finally standing in an unstable equilibrium condition (Fig.5). The bottom pillars are the elements that take the seismic overturning moment and shear forces at the base. In the last earthquake, their behavior was acceptable; however, some small cracks were observed at the base of the pillars just in the exterior corner that was not reinforced.

It can be concluded then, that the weakest part of the tower has been the second body or transition zone, because of cumulative damage from past earthquakes, perhaps not appropriately repaired, and because of the stress concentration immediately under the base of the third body.

Both towers are differently supported, whereas the left tower is completely supported on walls from the bottom of the church, the right tower is partially supported by the roof vault. The two interior pillars of the first body fell onto the vaulted roof of the church causing also some damage to this element.

4. Emergency structural stabilization

Five days after the earthquake and in light of the constant aftershocks, some of them quite strong, an emergency structural intervention was decided upon for the remaining tower. It was decided that a steel structure had to be placed inside the tower and from the bottom to the top of the third body, with the objective of stabilizing it (Fig.5). Six round tubes, 150 mm in diameter, were anchored to the base of the tower and welded to the cross beams at every level, previously covered with steel plates. Lateral bracing consisting of 50 by 50 mm steel angles was placed between the steel tubes in order to get a uniform flexural behavior of the system.

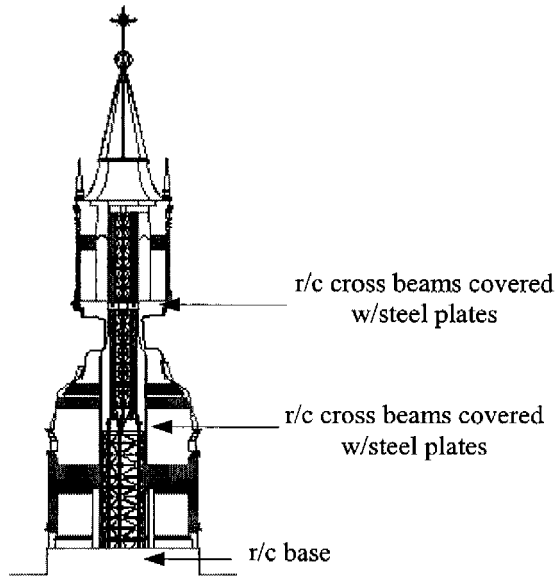


Figure 5: Emergency reinforcement in right tower

5. Structural analysis

The analysis of the tower was performed independently of the church structure and with a fixed end condition. The geometry of the tower was modelled with a commercially available CAD program and the frequencies and vibration modes were computed using a finite element program. The sillar stone, due to its volcanic origin is a lightweight stone, being its unit weight of the order of $1,250 \text{ kg/m}^3$. The total weight of each tower is about 425 tons.

The three first vibration modes were computed giving natural frequencies of 6.3, 10.2 and 11.5 hertz, respectively. The second mode corresponded to torsion effects in the arches of the third body, whereas the first and third modes corresponded to translation. There is enough data available about the mechanical characteristics of the sillar stones but almost no reliable data about the masonry. For this reason, the elasticity modulus of the structure was estimated at 4.9 GPa.

There are no records available from the earthquake in the city of Arequipa itself, but its duration was estimated as more than one minute. The only record available is one located at Ocoña, 150 Km west of Arequipa which shows a peak horizontal acceleration of 0.29g. A static analysis was performed with horizontal forces equivalent to 30, 40 and 50 percent of the weight. Since the sillar masonry can stand compressive stresses of 7.8 kPa, the problem arises when the masonry is subjected to tension stresses, in this case, for all three analysis performed, tension stresses are present in the tower. The location of the tensile stresses are congruent with the damage observed in the tower, the base pillars and the transition zone immediately below the third body.

6. Proposed structural solutions

The structural solutions had to be different for each tower due to their different states of damage. In both towers, the project had an initial solution that was later modified during the construction process. The final solutions as adopted are described in the following sections.

6.1. Left tower

The left tower that had suffered the collapse of the third and part of the second body, had to be rebuilt, and so, the complete reconstruction of these two bodies was proposed. For this, it was necessary to completely dismount the remaining part of the second body until the upper level of the first body. This solution was modified during the construction process by means of horizontal holes through the remaining part of the second body to avoid the dismantling of this part. Through those holes, the proposed reinforcement was placed. (Fig. 6).

In the bottom pillars, an additional column was placed in the exterior corner (Fig.7). This was the only corner left without support from previous interventions. The column was anchored to the existing concrete base and extended all the way up to the second level at which it was connected to a new cross beam through the mentioned holes in the second body (Fig.8). From this level, eight RC columns, with a square section of 25 by 25 cm. and four bars of 16 mm in diameter were placed coincident with the projection of the eight small pillars in the third body.

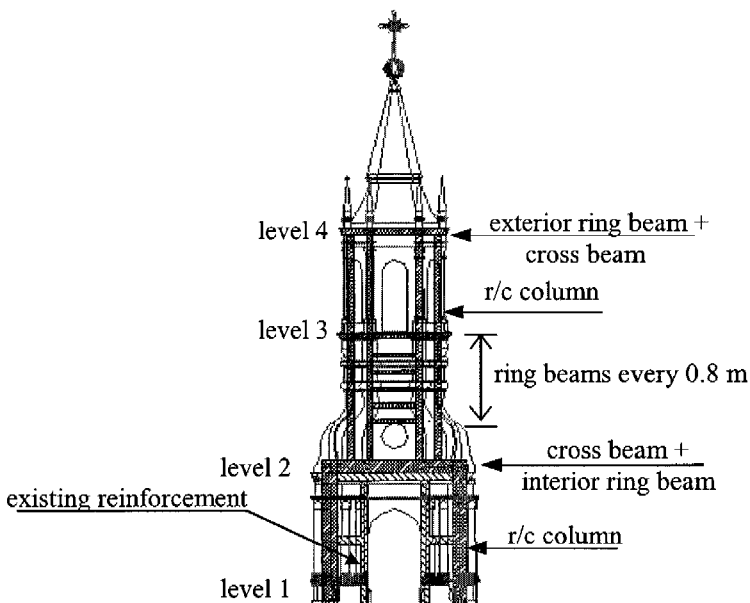


Figure 6: Structural reinforcement in left tower

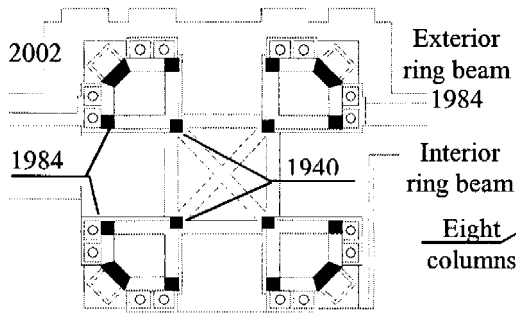


Figure 7: Left Tower Level 1

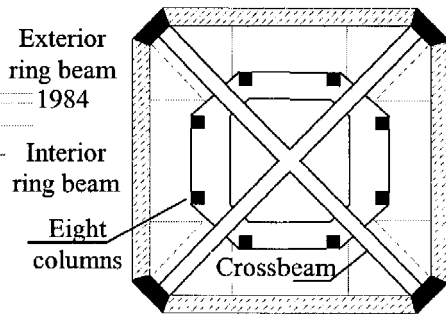
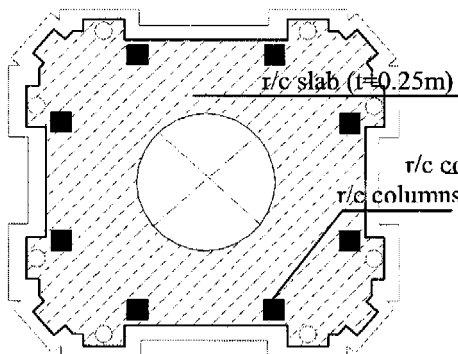
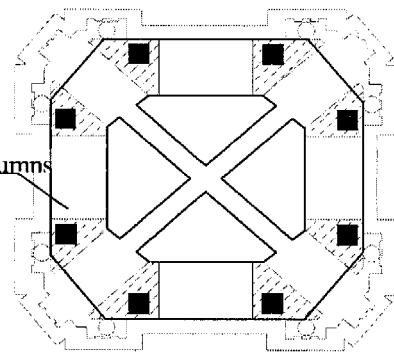


Figure 8. : Left Tower Level 2

Figure 9 : Left Tower Level 3
(Built 2002)Figure 10 : Left Tower Level 4
(Built 2002)

Ring beams were placed every meter until the base of the third body, where a concrete slab was placed (Fig.9). The eight columns continued in the third body inside the small pillars (Fig.10). Around this RC structure, the sillar stones were placed connected between them with vertical and horizontal steel bars.

6.2 Right tower

The right tower suffered extensive damage in the second body, losing part of its mass, and finally staying in an unstable equilibrium condition. For this reason, an internal steel structure was placed in order to stabilize it for possible aftershocks. In a joint coordination meeting with representatives from the Municipality, INC and the contractor, it was decided to leave the internal frame so the dismantling of the second and third body of the tower was not necessary.

The structural intervention consisted on the reinforcing of the bottom pillars with RC columns as described for the left tower, and the rebuilding of the missing mass in the second body. The second body was strengthened with Carbon Fiber Reinforced Polymer (CFRP) laminates, which were used to provide tensile strength and confinement .

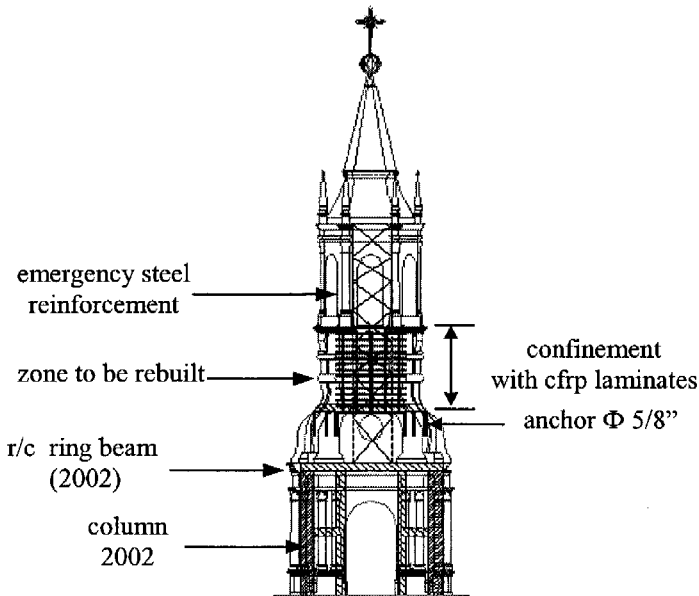
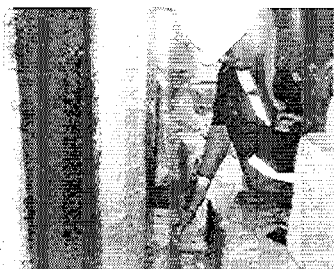


Figure 11 : Structural reinforcement in right tower

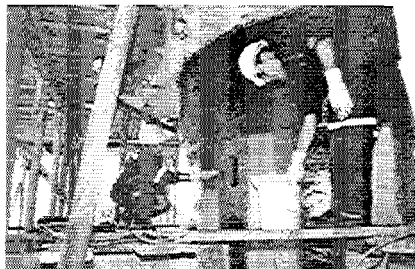
to that section of the tower (Fig.11). The laminates were installed using a manual lay-up technique. The CFRP laminates had a tensile strength of 3.45 GPa and a modulus of elasticity of 206 GPa. Based on the numerical model, normal tensile stresses in the range of 100 Pa, acting on the second body, were considered to determine the amount of FRP reinforcement. Due to the nature of seismic forces, reinforcement was required to be placed vertically along the circumference of the second body. The member was analyzed as a tubular section having an outer diameter of 4.20 m. and an inner diameter of 2.60 m. Trial and error was conducted with different amounts of CFRP reinforcement until the flexural capacity of the strengthened member was larger than the flexural demand associated to a tensile strength of 100 Pa. The strengthening called for vertical 25 cm.-wide CFRP strips spaced at 55 cm. on center around the circumference of the member. The design took into consideration provisions specified by the Committee 440 of the American Concrete Institute (ACI 440, 2002).

The CFRP laminates were anchored around a collar beam at the bottom region of the second body (Fig.12a). The laminates were installed from top to bottom and adhered to the surface of a small ditch. Next, a steel cage was placed in the ditch and concrete was poured.

After the installation of the vertical reinforcement was completed, CFRP laminates were placed in the hoop direction to restrain the lateral expansion of the second body under loads caused by potential seismic events. The laminates were installed along the entire height of the member (Fig.12b). The splice length was 15 cm. which was done at different positions of the height to prevent having



(a) Anchorage of vertical laminates



(b) Hoop reinforcement

Figure 12: Installation of CFRP Reinforcement

a weak area prone to debonding of the laminates. After completing the CFRP installation, carved sillar stones were placed on top of the laminates to keep the original appearance of the structure. These stones were also connected in between with vertical and horizontal steel bars.

7. Concluding remarks

The reconstruction project of the bell towers of the cathedral of Arequipa brought with it some opportunities for improvement with traditional methods and innovative materials, as well as the discovery of knowledge gaps. The sillar stone has been studied thoroughly in universities of Arequipa but the mortar and the joint behavior stone-mortar is still unknown. In the process of gathering information for the project, it was determined that the mortar had undergone changes from the early stages to present times. Originally only sand and lime was used in the mix together with milk and egg shells compared to actual times when sand, lime and cement mix were used. No data is available about the compressive and shear strength of the masonry. On the other hand, this project was a unique opportunity to train young artisans in the practice of carving and bonding the sillar stones, this new acquired ability will be very useful for future interventions in the historic downtown of the city.

References

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