ASSESSMENT OF THE SEISMIC VULNERABILITY OF TYPICAL MEXICAN COLONIAL CHURCHES

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Abstract. During the past two decades, it is growing the interest to study the seismic behavior of old masonry churches. The architectural heritage have been seriously damaged by the earthquake activity. The interest to preserve this kind of structures has motivated the development of novel specific methodologies of analysis and retrofitting, due to old masonry churches have a complex geometries and built with materials with highly nonlinear behavior. The out of plane failure of the façade is very common in many temples, principally in temples that have no towers attached, like the churches localized in many countries of Europe. However, in the old temples built in Mexico, the out-of-plane behavior is generally less important and is only regarded with the detachment of the façade from the nave, but without reach the collapse. To understand the reason that the out-of-plane behavior is less important in old Mexican temples, a study of the longitudinal seismic behavior of this kind of structures was performed. A typical Mexican temple with two tall bell towers was analyzed. Seismic actions was applied to study its effect in longitudinal and transverse directions of the temple. It was concluded that the towers work as buttresses and help to reduce the out-of-plane failure of the façade.
1 INTRODUCTION

In Mexico during the colonial period, architecture was mainly focused on the construction of Catholic religious buildings, because the Spanish conquistadors had the need to evangelize to the so-called New Spain. Monks of various religious orders came to Mexico and dispersed throughout the country, raising temples, monasteries and cities. Many of these buildings still prevail and others have disappeared due to the effect of time, social movements, or by the natural disasters. Of the buildings that still exist are found in areas where earthquakes are frequent. Recent earthquakes have demonstrated the vulnerability of these structures. The damage usually present after the action of strong earthquakes are like those reported by Alcocer et al. [4].

The out of plane failure of the façade is very common in many temples, principally in temples that have no towers attached, like the churches localized in many countries of Europe. Many studies has been developed around of this kind of failure. Analytical or experimental seismic research usually consider the effect of seismic actions in both longitudinal and transversal directions of the temple. However, in the old temples built in Mexico, the out-of-plane behavior is generally less important and is only regarded with the detachment of the façade from the nave, but without reach the collapse. For this reason, many studies only included the effect of the seismic actions in transversal directions and sometimes in vertical direction. To understand the reason that the out-of-plane behavior is less important in old Mexican temples, a study of the longitudinal seismic behavior of this kind of structures was performed. A typical Mexican temple of the center of the country was selected.

2 TYPICAL DAMAGES PATTERNS

In 1999, the center and southwest of Mexico were affected by two earthquakes (June and September) of intermediate magnitude. These earthquakes exceeded the limits established by the spectrum of design and caused significant damage in masonry structures, including more than 1800 historical monuments.

From a statistic study [5] made with the damage surveys of churches of the southwest of Mexico after the 1999 earthquakes, it was possible to find the relationship between the typologies of the churches and their failure mechanisms. The most common damage patterns will be briefly described, based on the studies carried out by Alcocer et al. [4], Meli and Peña [6] and Peña et al. [7].

Vaults, arches and domes are very efficient for resisting their own weight, but they are very sensitive to opening of their support, which introduces tensile stresses and it could produce excessive cracking (Figure 1). Due to the height of the temples, supporting walls and columns experience significant lateral displacements at their tops, during strong ground shaking. Thus, longitudinal cracks in barrel vaults and cracks along the meridians in domes are developed. In most cases, this cracking does not endanger the stability of the roof, giving rise only to a redistribution of stresses. Nevertheless, when large displacements take place, the shape of the elements could change to a less stable configuration, and progressively lend to collapse, especially because of the cumulative effect of subsequent earthquake motions. In this regard, domes are more sensitive than vaults to motions of their supports, which can be enhanced by the flexibility of their drums.
Vertical ground accelerations, which are significant in epicentral areas, play an important role in the damage of the roof, mainly because they tend to increase the thrust of the roof elements on their supports, consequently aggravating their opening and out of plumb.

Bell towers are rather slender and weak elements, in which the effect of ground motion is greatly amplified. Even if they are relatively low and sturdy, in temples of the most seismically active areas their failure is rather common, especially in the vertical elements and in the arches surrounding the belfry (see figure 2).

Facade is typically a tall and heavy wall, poorly connected to the rest of the temple. However, the key damages have shown that the main behavior of these elements is in the plane of the facade. The out-of-plane behavior is generally less important and is only regarded with the detachment of the facade from the nave, but without reach the collapse [4]. In the same way, the slab of the chorus restrains the wall and reduces the facade slenderness. Thus, the shear actions have a significant role in the damage patterns, especially in the joints between the facade and towers [7].

At the other end of the church, the apse has a shape that makes it generally stiffer and less vulnerable than the main facade; sometimes, it shows damage due to the thrust of its dome. Longitudinal walls of the temple commonly show no damage due to in-plane forces. Their out-of-plane stiffness is greatly enhanced by rather heavy buttressing. Nevertheless, it is frequent to notice some out of plumb derived from the thrust of the roof during their vibration under seismic ground motion (Figure 3). Subsequent earthquakes produce a cumulative effect in this regard.

With the main objective to demonstrate that the failure out of plane of the main facade is not important in most Mexican Colonial churches, in this paper performed a seismic nonlinear Finite Element Analysis on a typical colonial temple subject to strong earthquakes is presented. Also, the effect towers and buttresses on the seismic behavior of the facade is evaluated.
3 FINITE ELEMENT MODELING

3.1 Description of the models

The numerical models were performed by using the software Abaqus [8]. Three models were developed; the first, full model "F", consisting of a nave of 32 m x 20 m with a barrel vault and two towers of 28.5 m (see Figure 4a). The model with diagonal buttresses "DBF" (see Figure 4b), similar to the model "F" but with the difference that the towers are replaced by diagonal buttresses. Finally, the model without towers "WT" (see Figure 4c), with the same characteristics model "F", but with the absence of both towers. The bell tower and vault were considered masonry brick and walls by masonry stone.

Figure 4. FE church models: a) Full “F”; b) with diagonal façade buttresses “DBF”, and c) without towers “WT”
The basic construction material of these historical buildings is a masonry conglomerate composed of stones of different sizes, agglutinated by a lime-sand mortar. Lightweight stones of volcanic origin were preferred by builders. This heterogeneous masonry can be considered a kind of low-strength concrete. An example of the masonry in actual temples is found in Figure 5. The basic mechanical properties of the masonry and of its constitutive materials are given in Table 1.

Table 1 Material properties

<table>
<thead>
<tr>
<th>Material</th>
<th>Mass density [kg/m³]</th>
<th>Young’s Modulus [MPa]</th>
<th>Poisson’s ratio</th>
<th>Compressive strength [MPa]</th>
<th>Tensile strength [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Masonry brick</td>
<td>1580</td>
<td>1200</td>
<td>0.2</td>
<td>4.12</td>
<td>0.29</td>
</tr>
<tr>
<td>Masonry Stone</td>
<td>1600</td>
<td>2000</td>
<td>0.2</td>
<td>3.00</td>
<td>0.21</td>
</tr>
<tr>
<td>Stuffing</td>
<td>1600</td>
<td>1000</td>
<td>0.2</td>
<td>elastic</td>
<td>elastic</td>
</tr>
</tbody>
</table>

3.2 Modal analyses

Eigenvalue analyses of the model shows that, even if largest part of the participating mass corresponds to the fundamental mode of vibration, participation of very high modes cannot be disregarded; this can be attributed to the complex geometry and distribution of mass of this structure. Modal shapes of the modes with greatest participating mass in transverse and longitudinal direction are shown in Figure 6. Fundamental periods of vibration for longitudinal direction for all models are near to 0.15 s, but for transverse direction, fundamental period for model “F” is 0.32 s and for models WT and DBF are 0.27 s. This difference is due to the flexibility of the belfries. It can see
that the first period in both horizontal directions corresponds only to the flexural behavior of the belfries.

![Modal Shapes](image)

<table>
<thead>
<tr>
<th>Model</th>
<th>Transverse</th>
<th>Longitudinal</th>
</tr>
</thead>
<tbody>
<tr>
<td>Full “F”</td>
<td>![Mode 1 T=0.3219 [s]]</td>
<td>![Mode 7 T=0.1584 [s]]</td>
</tr>
<tr>
<td>Without towers “WT”</td>
<td>![Mode 1 T=0.2777 [s]]</td>
<td>![Mode 3 T=0.1562 [s]]</td>
</tr>
<tr>
<td>With diagonal façade buttresses “DBF”</td>
<td>![Mode 1 T=0.2668[s]]</td>
<td>![Mode 3 T=0.1548 [s]]</td>
</tr>
</tbody>
</table>

Figure 6. Transverse and longitudinal modal shapes for the three models.

### 3.3 Time history analyses

For a nonlinear time history analysis, a damping matrix must be defined in order to reproduce damping variation as the stiffness matrix changes. Abaqus does it through the Rayleigh damping. For our purposes, modal damping coefficients were derived from a damping ratio of 7%. Damping ratios measured for ancient masonry stone churches has been estimated to be greater than the typical 5%. From previous studies it has seen that damping varies from 6 to 12% [9]. The two coefficients of damping Rayleigh, $\alpha$ and $\beta$, were computed for the fundamental modes corresponding to lateral and longitudinal vibration. Parameters for Rayleigh damping for the three models are shown in Table 2.
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Table 2 Parameters for Rayleigh damping.

<table>
<thead>
<tr>
<th>Model</th>
<th>(\alpha)</th>
<th>(\beta)</th>
</tr>
</thead>
<tbody>
<tr>
<td>F</td>
<td>1.5495</td>
<td>0.0031</td>
</tr>
<tr>
<td>WT</td>
<td>2.2746</td>
<td>0.0017</td>
</tr>
<tr>
<td>DBF</td>
<td>2.3321</td>
<td>0.0017</td>
</tr>
</tbody>
</table>

Acceleration time histories applied to the three models in the transverse and longitudinal directions were derived from a strong ground motion record obtained in June 15 of 1999 near the epicenter of the great Tehuacan earthquake (Ms 6.7). This motion was selected because it produced severe damages to these structures, mainly in churches located in Cholula, a town near to the epicenter. The accelerogram and its acceleration response spectrum for 7% of damping are shown in Figure 7. One peak of the spectrum is near of the first period of vibration of the structure. The maximum acceleration is 2.3 m/s\(^2\) (0.24 g). The duration of the record is 65 s. The frequency content of record give rise to large spectral amplitudes, not only for the fundamental period of the structure corresponding to its undamaged state, but also for the longer periods that could be expected once the structure is damaged by the earthquake. The horizontal motion was applied, first, in the transverse direction of the model, and next, this same horizontal motion was applied in the longitudinal direction. This was realized to study the failure out of plane of the façade.

4 RESULTS OF NUMERICAL ANALYSES

4.1 Damage Patterns

Damage patterns obtained from the numerical simulation for the three models are shown in Figures 8 and 9. Damages of the pilasters of the belfries of the model "F" are several (see Figure 8a). This demonstrate the vulnerability of these elements to the effect of strong earthquakes that may lead to imminent collapse. The façade shows diagonal cracks and vertical cracks between connections with towers, which are caused by the inertial forces that induce the tower bell. The longitudinal walls show a number of cracks in the middle of the nave, principally in the parapet. This is due to the push of the vault over the walls and eventually they may not withstand. The apse is undamaged, this confirms the high stiffness of these elements. The vault shows the greatest damage in areas close to the façade.

Figure 7. Seismic action
Facade model "WT" shows damages similar to the model "F" but with less magnitude (see Figure 8b). Longitudinal walls are exposed to greater cracking. The vault shows a slightly higher cracking. The apse is undamaged. The "DBF" cracking pattern is more intense than the model "WT" but not be greater than that observed in the model "F". The cracking of the walls is similar to the model "WT". The apse is undamaged (see Figure 8c).
When the Strong Longitudinal Motion was applied, only the bell tower model "F" shows cracks that can lead to a condition of imminent collapse. The lower body of the towers is separated from the rest of the nave. However, this does not become critical condition. All models are observed only small diagonal cracks in windows that are due to shear stresses. Either model shows the possibility of failure out of the plane of the facade.

Figure 9. Damages patterns for the three models. Only Strong Longitudinal Motion
4.2 Displacements

Figure 10 shows the displacements of points 1, 2 and 3 of the three models produced by the action of seismic movements applied. Figure 4 shows the location of these points. The displacements of points 1, 2 and 3 in the longitudinal direction during the intense phase of the longitudinal motion were about the same for the three models. The absence or presence of towers or towers replacing buttresses showed no differences in the seismic behavior of the structures in this direction.

The displacements of point 1 in the transverse direction during the intense phase of the transverse motion showed some differences but it is difficult to rate which was higher than the other. The displacement of point 2 in the transverse direction during the intense phase of the transverse movement for "WT" and "DBF" models were similar and lower than the model "F". The displacement of point 3 of the model "F" in the transverse direction during the intense phase of the transverse motion was lower than those of the other models. For both seismic actions applied of the three models, displacements of points 1, 2 out of the plane of the facade were up to 50% lower than those produced in the plane.

4.3 Comparison of the behavior of the three models

Clearly, the greatest damage occurs by the action of the earthquake in the transverse direction of the churches. The action of the earthquake in the longitudinal direction is not critical. Damage was accentuated in the model “F”, by the inertial forces produced by the bell towers and its effects are transmitted to the connecting parts. So try to separate the towers of the facade. If the belfry came to collapse, it is likely that demand resistance of the facade was reduced significantly. For the WT model, the absence of the towers reduced the shear forces acting on the facade, but increase the damages in the longitudinal walls because the towers work like buttresses.

In DBF model, the diagonal buttresses reduced the damage of the longitudinal walls but increased shear forces acting on the façade although less than in the model “F”.

5 CONCLUSIONS

The results of the numerical analysis have shown that the failure out of the plane on the facades is not critical in this type of Churches, for even applying strong seismic movements in the direction perpendicular to this, the facade is not affected by these movements. One possible reason is that the frequency of the fundamental mode of longitudinal vibrated is far from the area of high spectral ordinates the movement applied. This behavior is consistent with the damage observed after intense earthquakes in this kind of structures.

Moreover, it was shown that the presence of the lower body of the towers helps to reduce the lateral movements of the front, however, this also induces greater shear force that cannot withstand the facades and the damages are more intense. If the model does not have towers, the damages are lesser. A similar behavior was observed in the model with the diagonal buttresses but less intense.
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Figure 10. Displacements of the points 1 (top), 2 (medium) and 3 (bottom), for the three models

a) Only Strong Transverse Motion  b) Only Strong Longitudinal Motion
REFERENCES


