Retrofit, using seismic isolation, of the heavily damaged Basílica del Salvador in Santiago, Chile

M. Rendel, C. Lüders, M. Greer, I. Vial & B. Westenenk
SIRVE S.A., Santiago, Chile

J.C. de la Llera
School of Engineering, Pontificia Universidad Católica de Chile, and National Research Centre for Integrated Natural Disaster Management CONICYT/FONDAP/15110017.

F. Pérez, D. Bozzi & F. Prado
School of Architectural Design, Pontificia Universidad Católica de Chile

ABSTRACT: This paper presents the retrofit project of a 150 year-old cathedral that was severely damaged after the March 3rd, 1985 central Chile earthquake, and the February 27th, 2010 Maule earthquake. The project is being developed by a team of architects and structural engineers from two Chilean companies, the latter dedicated to structural engineering and seismic protection of structures. The Basílica del Salvador is located in Santiago and was built towards the end of the 19th century. The unreinforced brick structure is approximately 90m long, 40m wide, 25m high and has suffered extensive damage during major Chilean earthquakes over the past century. Currently, several areas of the structure are partially collapsed and many architectural components are damaged, making the retrofit project very unique and challenging. The overall concept of the retrofit is to recover the original architecture and aesthetics of the church while improving its seismic safety standard to optimal conditions, by introducing seismic isolation between the superstructure and a new underground level. The retrofit project considers two stages: (i) a temporary stabilization phase, which involves adding a steel shoring structure inside the main auditorium to support the partially collapsed columns and unstable exterior walls, and (ii) a permanent phase, which involves the addition of a new underground level, the introduction of seismic isolation to protect the superstructure and the architectural restoration of the superstructure.

1 A BRIEF HISTORY OF THE BASILICA

The Basílica del Salvador was built as a consequence of the fire suffered by the Compañía de Jesús church (Jesuits), which occurred on December 8, 1863, with a death toll of more than 2000. In response to this incident, Bishop Rafael Valentín Valdivieso signed an ordinance on May 2, 1864 for the construction of a new basilica dedicated to the Saviour or “Salvador”. Since then, the basilica has been a building of great religious and civil significance. The construction began on December 1873 and was finished and inaugurated in 1892. After the inauguration it began housing the image of the Virgen del Carmen, the patroness of Chile, in 1938 the church was elevated to the title of “Basilica” and in 1977 it was declared a National Historic Landmark. Figure 1 presents a general view of the Basilica.

Teodoro Burchard, a German architect, was responsible for the design of the basilica. Burchard developed a neo-Gothic building, one of the first examples in the country, and it became an example of modernism. Built of solid masonry without metal or concrete reinforcement with a wooden roof structure, Basílica del Salvador remains one of the few examples of this style in Chile. The 30 stained glass windows, the largest examples in Chile, were developed in 1893 by Mayer'sche Hofkunstanstalt in Munich. The interiors have a polychrome decoration on plaster and canvas for the columns, walls and ceilings, made by the Italian-Chilean artist Aristodemus Lattanzi.
Throughout its history, the basilica has suffered various damage, mainly in the earthquakes of Valparaíso in 1906 (M, 8.2), central Chile in 1985 (M, 7.8), and Maule in 2010 (M, 8.8), with subsequent modifications and repairs. Between 1932 and 1937, the prominent Chilean architect Josué Smith Solar added concrete reinforcement in the area of the narthex and choir and redesigned the north façade incorporating stucco and decorations.

The 1985 earthquake caused extensive damage to the basilica, including collapse of walls, roofs, windows of the transept and two columns of the main atrium. All of these areas were partially rebuilt incorporating reinforced concrete structures. New windows and interior decorations were also added. Additionally, temporary bracing was erected in other areas of the structure during the retrofit after the 1985 earthquake, but was never fully constructed throughout the full structure. The recent 2010 earthquake caused additional damage into the structure, including collapse of walls, columns, arches and ceiling. Due to this damage, the church has been closed since 1985.

2 STRUCTURAL DAMAGE

Figure 2 shows a brief compilation of the extensive damage in the basilica, both structural and aesthetic. Main damages are related to: (i) collapse of the interior column in the main atrium area (Fig 2a), (ii) large collapse of the exterior arch on the east side of the structure (Fig 2d), (iii) collapse of the interior arches between columns and exterior walls (Fig 2a), (iv) cracking through the bases of the interior columns (Fig 2b), and (v) collapse of the southwest corner of the arcade roof and arches (Fig 2c). There is also widespread damage of the architectural components, such as the ceiling and facades, however these are of little importance structurally and therefore not addressed in detail in this report.
Some damage patterns of the Basílica del Salvador are also present at the Christchurch Cathedral of the Blessed Sacrament (Lester 2012). In both structures, the rotation of walls produced outward displacements at the top of about 100mm to 150mm. On the other hand, the damage in the Basílica del Salvador is more widespread and is present within the core of the building, evident with collapsed walls, columns, arches and ceilings in the main atrium area. In the Christchurch failure the damage appeared to be mainly localized in the front towers and the main dome, which are heavy architectural elements that the Basílica del Salvador does not have. In both buildings the failure patterns are consistent with those known to occur in unreinforced masonry buildings subjected to large earthquakes.

The ongoing retrofit of the Basílica del Salvador has been divided into two phases: (i) a temporary retrofit to stabilize walls and structural elements, and (ii) a permanent retrofit to reach an optimal condition of seismic reliability. The first stage has been developed promptly so as to provide stabilization and avoid additional collapse or damage of structural elements. Conversely, the permanent retrofit considers a more comprehensive and detailed design and will adhere to the following criteria: (i) minimize the seismic demand into the structure –forces and displacements, and (ii) recover the damaged areas using materials that will behave cohesively with the existing elements and have a minimum impact on the original aesthetic. Additionally, a basement beneath the existing structure has been considered, which will provide the support for the seismic isolation system while adding valuable usable space to the building.

3 PROJECT BACKGROUND

3.1 The current state of seismic isolation in Chile

The basilica will be a unique base isolated retrofit project in Latin America, which also adds to more than 60 buildings and structures that currently incorporate seismic protection in Chile. Furthermore, when the 2010 Maule earthquake struck Chile, there were 14 structures with seismic protection technologies installed. These structures demonstrated excellent performance (Moroni 2012) and served to validate the value of these systems to: (i) highly improve the seismic performance of structures, (ii) maximize continuous operation of buildings and industries and, (iii) to protect both structural and non-structural elements. Some of the most relevant seismically isolated structures were: the Nuevo Hospital Militar (50,000m²), Clínica UC San Carlos de Apoquindo (8,000m²), and Edificio San Agustín of the Pontificia Universidad Católica de Chile. Also, recently Chilean engineers were invited to participate in a full scale shake table test at the NEES facility at UCSD for a seismically isolated structure using Chilean designed and manufactured isolator units. Chilean isolators have been fabricated locally since 1995. This development of seismic isolation locally is the result of a joint development between the Pontificia Universidad Católica de Chile and the company VULCO (Weir-Minerals).

3.2 Code guidelines

Chilean codes are mostly based upon well-known methodologies from international codes with adaptation to local seismicity. Chile has two codes that are applicable to this project: NCh 433 (2009) for the design of conventional buildings (mainly residential and commercial) and NCh 2745 (2013) for the design of seismically isolated structures. While it is possible to use the referred codes to determine seismic demands, no specific design criteria or code is available in Chile to design historical structures such as this one. Therefore, this study has considered several experiences worldwide as referential examples and practices (Chiorino 2008, Syngellakis 2013, Walters 2003).

3.3 Structural concept and materials

The existing structure is constructed of unreinforced masonry, relying heavily on gravity arches and thick wall sections for gravity and lateral support. The building is approximately rectangular in plan. The northern area of the structure, where the main access is located, has a second masonry floor at approximately mid-height and just south of this area is the main atrium area that is open to the roof and ceiling at full height. Two rows of tall brick masonry columns support the roof on both sides of
the centreline while the exterior arched walls support the perimeter of the roof. In the south end, the central area is full height with walls supporting the roof edges whereas the side areas are shorter and also supported on the exterior masonry walls. There is a masonry arch arcade that extends almost the entire perimeter of the structure. On the interior of the structure the arched ceilings are constructed of wood and painted plaster. The roof is comprised of wood trusses spanning transversely (EW direction) across the main atrium and longitudinally across the transept; metal sheeting forms the roof covering.

After the 1985 earthquake some repairs were performed on the structure including the replacement of two brick columns with reinforced concrete columns, the addition of some metal reinforcing for collapsed arches, the addition of concrete strengthening for damaged brick areas and the addition of steel reinforcing ties within the exterior brick arches. The specific material properties considered for the existing structure are shown in Table 1. The final design will include a complete material testing in order to determine exact material properties.

### Table 1. Summary of material properties

<table>
<thead>
<tr>
<th>Material</th>
<th>Compressive Strength (MPa)</th>
<th>Tensile Strength (MPa)</th>
<th>Ultimate Stress (MPa)</th>
<th>Young’s Modulus (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Brick Masonry</td>
<td>1.50</td>
<td>0.10</td>
<td>--</td>
<td>1.50</td>
</tr>
<tr>
<td>Concrete</td>
<td>24.52</td>
<td>--</td>
<td>--</td>
<td>23.41</td>
</tr>
<tr>
<td>Steel</td>
<td>--</td>
<td>245.2</td>
<td>392.3</td>
<td>205.9</td>
</tr>
</tbody>
</table>

Soil data is also a key aspect in the response of this structure and the selection of an appropriate retrofit scheme. The soil mechanics study at the site found that the $V_{30}$ velocity is 564 m/s (shear wave propagation velocity) which is consistent with a type B classification of the soil.

### 4 TEMPORARY REPAIR

The overall idea behind the temporary retrofit is to address the immediate need for reinforcement of the existing structure so as to prevent further structural and architectural damage to this historically sensitive building, and allow safe work conditions for further repair works. There are three main areas of concern: (i) the large collapsed east arch, (ii) the collapsed interior columns and (iii) the collapsed interior arches. If the building is checked without reinforcement, tension stresses are beyond acceptable limits implying the generation of collapse mechanisms. Hence, to prevent this, a steel shoring structure is added to reinforce more susceptible areas and support other eventual unstable and highly stressed zones. For this, an adequate interface between the new and existing structure was considered, that can be easily removed without causing damage to the fragile architectural elements.

A 3D SAP2000 (Computers and Structures 2012) model was generated using initial assumed geometries. The SAP model considers a static force procedure to determine the seismic demand on the structure and reinforcements. Then, according to the NCh 433 code requirements, the seismic response reduction factor ($R$) and seismic coefficient ($C_s$) were determined to be 2 and 0.27, respectively for the masonry structure and 3 and 0.18, respectively for the steel structure. In some areas, due to expected damage, artificial hinges in walls, columns, and arches were added to the model in order to obtain a more demanding loading on the transitory reinforcing structure. The arch crown shell elements were physically separated in order to capture the assumed cracking behaviour at these locations during a seismic event.

The temporary reinforcement structure consists mainly of steel braced frames with steel space trusses that span across the main atrium. The structure acts primarily as a support for the transverse seismic action on the existing structure. A general configuration of the steel structure and its relation to the existing structure is shown in Figure 3. In other zones, different from that of the main atrium, various steel reinforcement elements were considered, including tensors, braces, and section strengthening.
Finally, micro-piles have been considered for the tension and compression force transfer to the foundation soil (each of 76mm diameter and with a 580kN maximum admissible force). Special care was taken to locate the micro-piles in locations where they would not interfere with the underground structure that will be erected during the permanent retrofit. The erection of the temporary retrofit is expected to begin in early 2014, and it should be in place for approximately five (5) years.

5 PERMANENT RETROFIT AND RESTORATION

The permanent retrofit involves the addition of a new basement level beneath the existing structure that serves as the support of the seismic isolation for the structure and also a traditional reinforcement of the superstructure. The main concerns for this design are related to construction sequencing and structural compatibility of new and existing materials.

A 3D laser survey was developed for the complete structure since no detailed drawings were available; it was also a powerful tool to precisely locate the extent of collapsed and damaged areas. Figure 4b shows a complete 3D ANSYS model of the basilica that was generated from the 3D laser scan data (Figure 4a). Two cases were considered in order to calculate the dynamic properties of the structure: (i) the undamaged model with original masonry elements, and (ii) the actual damaged state of the basilica. At this stage only linear properties of the material have been considered (see Table 1 for the material properties). The elements considered in the analysis were 3D structural solids of 20 and 10 nodes. The models are comprised of approximately 3 million nodes and 1 million elements, concentrated masses have been added to represent roof and ceiling loads. It is important to note that bonded contacts between elements have been assumed; this assumption is not always fulfilled in historical buildings so special care has to be taken to avoid any possible mechanism.

A very precise construction sequence has been considered in the design, since the new underground level will be built beneath a large-scale and extremely damaged structure. This procedure includes the following steps: (i) perform a plan survey and install the micro-piles, (ii) place jackets at the base of
the walls and columns that will be supported providing a strong connection to the masonry, (iii) pre-stress the micro-piles so that the load transfers from the existing foundations to the micro-piles, (iv) excavate a small space under the columns and walls and build the isolator slab concrete beams, (v) progress with the excavation and add bracing elements between the micro-piles to avoid buckling, (vi) after the excavation reaches the bottom level, build the underground structure, including columns and capitals, and install the isolators, (vii) transfer the load from the micro-piles to the isolators by using flat-jacks, (viii) fill the remaining gap between the isolators and the superstructure beams with high capacity grout, and (ix) cut and remove the micro-piles.

The isolation system considered for this project incorporates elastomeric high damping rubber bearings. The devices shall be designed to provide optimal isolation, reducing the elastic seismic demand by a factor larger than 12. A graphical representation of this reduction is shown in Figure 5 and a preliminary layout of the isolation system is detailed in Figure 6.

![Figure 5](image1.png)

**Figure 5.** Seismic elastic demand reduction from seismic isolation (transverse direction).

![Figure 6](image2.png)

**Figure 6.** Plan and elevation of the seismic isolation design.

The superstructure dynamic properties were obtained from the 3D model. The fundamental modes for the undamaged structure are shown in Figure 7a-c, and for the current damaged state condition in Figure 7d-f. It can be seen from these figures that the critical conditions for the basilica are the lateral displacements of the columns and walls in the main atrium.
For the critical zones, 3D nonlinear ANSYS models were developed to determine stresses, stiffnesses and other mechanical properties of the structure. Specifically one axis has been modelled in order to compare the different strengthening techniques for consideration in the typical arches. The masonry has been modelled using a Drucker Prager yield surface (Drucker & Prager 1956) coupled with a William and Warnke failure criteria (William & Warnke 1974). The material properties for the Drucker Prager surface are: (i) cohesion $c = 0.1\text{MPa}$, (ii) internal friction angle $\phi = 35^\circ$, and (iii) dilatancy angle of $\psi = 30^\circ$. A shear transfer coefficient for an open crack $\beta_\text{t} = 0.1$ and for a closed crack $\beta_\text{c} = 0.6$ has been considered also (see Table 1 for further material properties). The results shown incorporate self-weight and 0.05g horizontal acceleration, since base isolation has been considered. Figure 8 shows the mesh and results for the unreinforced original masonry arch.

Currently six different arch retrofit possibilities have been analysed: (i) unreinforced original masonry structure (Figure 9a & g), (ii) masonry arch with ties through the springer of the arch (Figure 9b & h), (iii) masonry arch with ties through the crown of the arch (Figure 9c & i), (iv) reinforced concrete arch (Figure 9d & j), (v) steel truss arch (Figure 9e & k), and (vi) glass fibre reinforced polymer (GFRP) strengthened masonry arch (Figure 9f & l). For cases (ii) and (iii) the ties were designed for the entire lateral thrust from the arch, and the assigned prestress equals the minimum lateral thrust, 19.9kN. The steel arch was designed so that the axial and flexural stiffness are similar to the original masonry arch stiffness. Figure 9 shows the equivalent stress and the equivalent accumulated plastic strain of the different cases analysed. Note that the scale used is the same as in Figure 8 and only half of the axis is
shown.

Figure 9. (a - f) Equivalent stress [Pa] and (g - l) equivalent accumulated plastic strain for different arch strengthening models (noted values correspond to the total plastic strain energy [Joules] at the base and top of the column and the arch).

The tie at the base of the springer (b & h) shows reductions in plastic strain at both the column base and the arch crown, however it has the disadvantage of being disruptive with the original architecture. If the tie is taken upwards (c & i) the arch shows similar strain levels as the original. Furthermore, the GFRP strengthening (f & l) in the extrados and lateral faces of the arch shows insignificant plastic strain reductions, and the reinforced concrete arch (d & j) shows some amplification at the interior column but eliminated stress concentrations at the arch crown and at the exterior column. Finally the steel truss arch (e & k) also suppresses strain issues at the arch crown and shows high strain reductions at the column base and top. The good behaviour of steel is related to the negligible weight of this arch (3,012kgf) compared to the original masonry arch (26,045kgf), which reduces the stress produced by the gravitational deformation of the column-arch structural system. This deformation occurs because the arches are supported on the interior masonry columns, which have small horizontal stiffness to resist the horizontal reaction of the arches.

The reinforcements based on new materials such as steel or concrete and their anchors to the base material, need to be designed carefully in order to avoid the well-known compatibility problems of previous historical retrofitting experiences.

6 CONCLUSIONS

Basílica del Salvador is a unique patrimonial structure in Chile that has a large civic and religious significance. The building is constructed of thick brick masonry walls and columns, relying on the unreinforced mass to provide gravity and lateral support. Several past earthquakes have caused severe structural and architectural damage, leaving the important building in an unusable state and in dire need of retrofit, renovation and restoration.

The basilica is actually in an unstable condition, and prior to the definitive retrofit, a temporary shoring system must be implemented to avoid further damage and to allow safe working conditions. To provide minimal impact on the sensitive structural and architectural elements is of great concern for this temporary phase. This is achieved by providing an adequate interface between the new and existing structure that can be easily removed while allowing stress concentrations to be distributed.

The permanent retrofit considers a base seismic isolation system with high damping rubber bearings, that reduces the elastic seismic demand on the structure by a factor larger than 12. The new
underground basement story will serve as a foundation for the basilica and the seismic isolators, while also creating a useful space beneath the atrium. Superstructure permanent retrofit schemes are also currently being developed and 3D nonlinear ANSYS models have been processed to check configurations and different building materials. Typical arches of the main atrium have been modelled for various configurations including tensor addition, GFRP wrapping, concrete and steel addition. Actually, the final retrofit design is being developed and will incorporate one or more of the investigated schemes. The basilica will become a unique base isolated retrofit project in Latin America and may serve as a pilot case for other patrimonial structures in other countries over the Pacific Rim.

REFERENCES


Drucker, D.C. & Prager, W. 1952. Soil mechanics and plastic analysis for limit design, Quarterly of Applied Mechanics, 10(2), 157-165


Moroni, M., Sarrazin, M. & Soto P. 2012. Behavior of Instrumented Base-Isolated Structures during the 27 February 2010 Chile Earthquake. Earthquake Spectra, 28(S1), S407-S424

