https://doi.org/10.4322/CINPAR.2022.003

Cinpar 2022 SAKARYA, TURKEY

XVIII CONGRESSO INTERNACIONAL SOBRE PATOLOGIA E REABILITAÇÃO DAS CONSTRUÇÕES

XVIII CONGRESO INTERNACIONAL SOBRE PATOLOGÍA Y REHABILITACIÓN DE LAS CONSTRUCCIONES

XVIII INTERNACIONAL CONFERENCE ON BUILDING PATHOLOGY AND CONSTRUCTIONS REPAIR

17 TO 19TH NOVEMBER 2022

Structural Evaluation and Rehabilitation Proposal of an old building located in a Region of High Seismic Hazard

Roldan, Víctor[1]; Guzmán, Alberto[2]; Palazzo, Gustavo[3]

[1] Universidad Tecnológica Nacional, Facultad Regional Mendoza; CeReDeTeC, Argentina, victor.roldan@frm.utn.edu.ar
[2] Universidad Tecnológica Nacional, Facultad Regional Mendoza; CeReDeTeC, Argentina, victor.roldan@frm.utn.edu.ar
[3] Universidad Tecnológica Nacional, Facultad Regional Mendoza; CeReDeTeC, Argentina, victor.roldan@frm.utn.edu.ar

Abstract: In regions with high seismic vulnerability, there are many old buildings that could be damaged or collapse with a major seismic event. This is the case of a church built in Argentina in 1956, and to date, it has been subject to more than a dozen earthquakes of intensity VI or higher. This situation has not only led to the appearance of pathological manifestations in the building, which allow us to infer abnormal structural behaviour, but has also revealed the need for a comprehensive structural adaptation in accordance with the descriptions of current constructions standards. In this regard, the objective of this work is to show the evaluation methodology carried out for the aforementioned building, as well as the proposed structural intervention strategy in order to achieve its rehabilitation.

Keywords: Structure evaluation, Non-destructive tests in concrete, Structural reinforcement.

1. Introduction

This paper presents the comprehensive study of a building whose construction dates back to 1956. This building presents various pathologies, both structural and constructive. Structural pathologies arise given that it is intended to evaluate the building according to the guidelines of the current constructions reglementation. Due to the year of construction, both the seismic demands taken into account for the design, as well as the reinforcement detail of the reinforced concrete structural elements, do not meet the requirements of current reglementation. As for the constructive pathologies inherent to its use, cracks could be observed in the walls and signs of corrosion in the reinforcement of the slabs, which could compromise the structural safety of the building. For this reason, it is necessary to carry out, first of all, a comprehensive evaluation of the structure that allows qualitatively inferring the current state of conservation. Subsequently, the intervention methodology that manages to correct the detected pathologies, and on the other hand, satisfy the seismic demands imposed by current codes was proposed. The pathologies observed, the different methodologies carried out in the evaluation, the numerical study of the original structure in order to identify the seismic vulnerabilities, and the proposed rehabilitation and strengthening strategies are described below.

2. Building Description

The building under study corresponds to a church, located in an area of high seismic hazard. The year of construction dates back to 1956. The seismically resistant structure in the east-west direction is made up of reinforced concrete frames and masonry walls, and in the north-south direction by reinforced concrete columns and masonry walls, Figure 1.

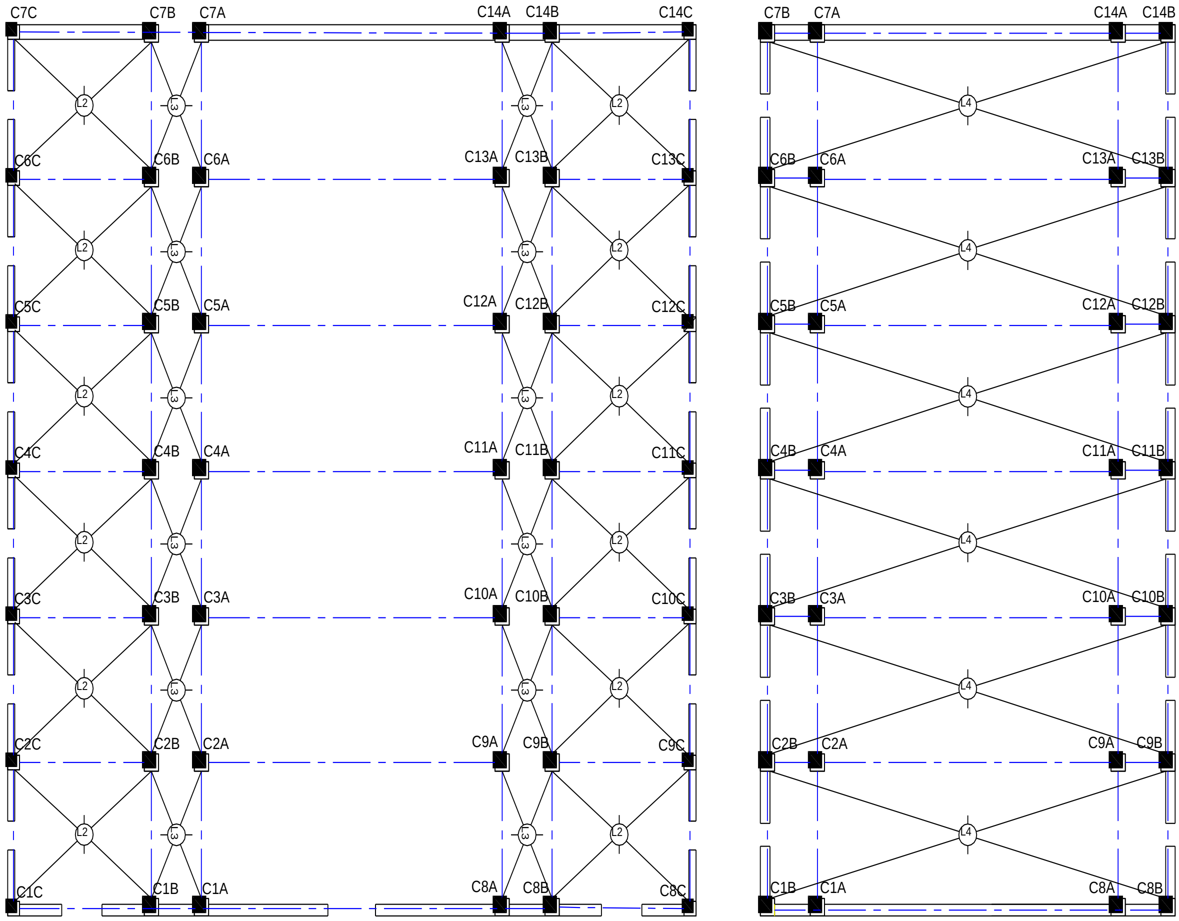

Figure 1: PGround floor and upper floor of the building structure under study.

Its architectural configuration consists of a central auditorium and lateral auditorium above which the tribune sector is located, Figure 2. Although the original architectural project of the church contemplated the construction of the apse of the main altar in the rear sector, the main door and the bell towers in the front sector, this could not be completed. For this reason, the openings corresponding to the first and last reinforced concrete frame in the east-west direction were filled with masonry, Figure 3.

Figure 2: Interior view of the building in study

Figure 3: Rear facade

3. Construction characterization

3.1 Preliminary visual inspection

Firstlyl, a visual inspection was carried out. Through this inspection, the pathologies detailed below could be observed.

. Cracks in the front and rear masonry walls. As mentioned above, both the front and back masonry walls were built on a temporary basis. For this reason, after the construction of the frame columns and beams, the masonry was placed as a filler. As there is no link between the masonry and the reinforced concrete structure, and due to the earthquakes produced in the region, these cracks could then correspond to the seismic action and the lack of wall-structure connectivity, Figure 4. The same would correspond to the rear closing masonry wall condition.

. Cracks in lateral masonry walls. The upstairs side walls, in the grandstand sector, have diagonal cracks in correspondence with the windows, Figure 5.

Figure 4: Cracks in the front facad

Figure 5: Cracks in side wall

. Moisture and corrosion in slabs. Various sectors of the slab are affected by the presence of moisture, Figure 6. In addition, linear cracks were observed in some sectors of the slabs that would coincide with the location of the joists, which could be an indication of a corrosion process in the slab reinforcement.

. Moisture in walls. The walls located above the lateral auditóriums in the tribune sector, show signs of moisture, Figure 7. As they are reinforced masonry walls, this humidity has caused the corrosion of their reinforcement.

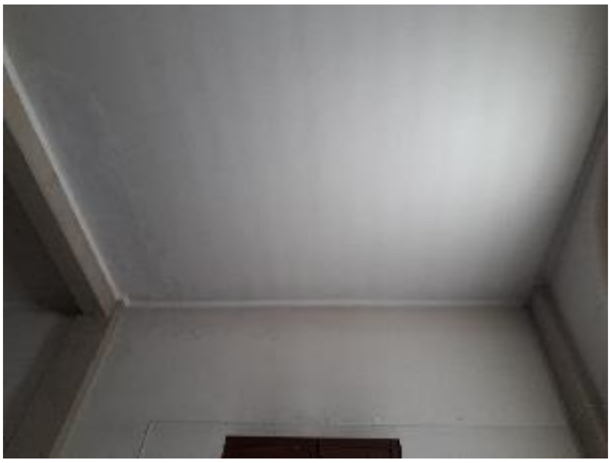

Figura 6: Moisture and cracking in slabs

Figura 7: Moisture in walls

. Non-structural elements. Due to the seismic movements, the large religious images present in the church have suffered displacements.

3.2 Material evaluation

In order to infer the types of steel present in the reinforcement and the quality of the concrete, a series of non-destructive tests were carried out, which are detailed below.

3.2.1 Tests carried out

. Reinforcement Detection

In order to establish the presence and location of reinforcements, they were detected using the electric flow measurement technique using the PROCEQ Profometer 650 AI device (Serial No. UP01-002-1738). Figure 8 shows the results of the procedure carried out. The results obtained were contrasted with the available structure documentation.

. Determination of the sclerometric index (IE)

In order to infer the quality of the concrete, in terms of resistance to surface compression, a NDT technique was used. This technique consist on the interpretation of the rebound or sclerometric indices (IE) obtained on the tested concrete surface, Figure 9. The ELCOMETER 181 Type N equipment (Serial No. 16005120) with an impact energy of 2207 Nm was used according to the provisions of the IRAM 1694 Standard.

. Longitudinal Wave Velocity Determination (V_L)

This NDT technique allows evaluating the homogeneity of the concrete, and with it, its quality, since the speed of the wave is dependent on the density of the concrete and the cavities and/or cracks present in it. The types of measurements carried out were direct, with the transducers on opposite sides of the material, Figure 10. It proceeds in accordance with the provisions of IRAM 1683 Standard. The PROCEQ Pundit PL-2 equipment (Version 200, N Serial No. UP01-001-0554) was used.

Figura 8: Reinforcement detection

Figura 9: Sclerometry test

Figura 10: Ultrasound test

. Structural surveys

Surveys were carried out in beams and columns in order to corroborate the results of the reinforcement detections carried out. The type of steel and its state of conservation could also be observed. The same was carried out on the slabs and the type and state of conservation of the joist reinforcement could be determined.

3.2.2 Number of tests and sectors evaluated

Table 1 shows the elements evaluated. It should be noted that the number of trestss was adjusted based on the uniformity of the results that were obtained.

Table 1 – Tests carried out.

Elemento	Ensayo			
	Detección	Sondeo	Esclerometría	Ultrasonido
C10A	x	x		
C11B	x	x		
C11A	x		x	x
C4A	x		x	x
C5A	x		x	x
C2B	x		x	x
C4B	x		x	x
C13A	x		x	x
C9B	x		x	x
C10B	x		x	x
C3C		x		
C2Asup.	x		x	x
C3Bsup	x		x	x
C4Asup	x		x	x
C6Bsup	x	x	x	x
C10Asup	x	x	x	x
C10Bsup	x	x	x	x
C12Asup	x		x	x
C13Bsup	x		x	x
VC10	x	x		
VC8A-C9A	x	x	x	
VC3A-C4A	x	x	x	
VC9B-C9C	x	x	x	
VC8B-C9B	x	x	x	
VC3B-C3C				
VC3B-VC4B	x	x	x	

3.2.3 Tests results

. Determination of the sclerometric index (IE) and longitudinal wave speed (V_L)

Table 2 shows the results obtained from each sclerometric index test and longitudinal wave velocity test. Using both results, it was possible to correlate the compressive strength of the evaluated concrete, f'_C.

Table 2 – Results of the tests of sclerometric index and longitudinal wave velocity.

Elemento	Esclerometría									IE	V_L (m/s^2)	f'_c (Mpa)
C11A	40	44	44	30	38	36	42	40	40	39	4150	33
	34	42	36	44	48	36	46	38	36	40		
C4A	30	32	32	44	36	44	30	30	34	35	3400	17
	32	38	32	32	40	32	32	34	36	34		
C5A	34	34	32	30	44	30	40	44	36	No válido	3115	13
	36	30	30	34	28	28	30	30	28	30		
C2B	32	40	30	30	32	34	32	28	34	32	2905	9
	32	28	34	34	34	28	28	26	30	30		
C4B	32	42	30	36	42	32	36	40	30	No válido	3125	13
	36	30	42	32	34	34	34	32	30	34		
C13A	46	42	42	36	38	40	38	42	48	41	3785	29
	40	36	38	40	44	32	36	38	36	38		
C9B	30	30	30	34	35	35	32	36	32	33	3600	19
	36	34	30	40	36	34	44	32	42	36		
C10B	35	42	38	32	44	38	34	36	40	38	3400	21
	44	38	40	48	48	38	42	40	40	42		
C2Asup.	36	40	40	40	42	42	36	42	31	39	3213	13
	44	46	50	48	40	44	44	38	46	44		
C3Bsup	38	44	42	38	42	42	44	44	40	42		
C4Asup	40	44	36	46	36	42	34	36	36	39	3316	19
	42	46	34	44	36	46	36	38	40	40		
C6Bsup	30	34	28	34	38	30	42	32	36	34		
C10Asup	30	36	38	30	30	30	34	34	32	33	3081	13
	28	30	34	38	32	38	38	32	34	34		
C10Bsup	28	32	28	26	26	28	28	28	28	28		
C12Asup	46	32	46	44	38	48	38	42	40	42	3544	26
	38	40	32	40	44	38	44	44	36	40		
C13Bsup	46	38	38	42	44	46	34	34	38	No válido		
VC8A-C9A	34	34	28	44	40	42	42	42	44	39		
VC3A-C4A	32	34	28	36	36	34	40	36	34	34		
VC9B-C9C	30	34	30	30	32	30	30	30	32	31		
VC8B-C9B	42	40	42	42	44	44	40	40	44	42		
VC3B-VC4B	38	32	42	40	36	40	42	32	34	37		

. Results of surveys in beams and columns

It was possible to infer that the reinforcements were made of AL-220 type steel bars. Regarding the conservation condition, it was observed that the reinforcements were in a good state of conservation, showing no signs of corrosion. Table 3 presents the observations determined.

Tablea 3 – Results of concrete surveys.

Elemento	A° long. Ø (mm)	A° transv. Ø (mm)	A° transv. sep. (mm)	Recub. r (mm)	Condición
C10A	30	8	200	40	No corroida
C11B	26	8	200	20	No corroida
C3C	20	6			No corroida
C6Bsup	26	8	200	20	No corroida
C10Asup	30	8	200	10	No corroida
C10Bsup	26	8	200	0	No corroida
VC10	26	8	200	0	No corroida
VC8A-C9A	12	6	200	30	No corroida
VC3A-C4A	12	6	200	45	No corroida
VC9B-C9C	20	6	200	5	No corroida
VC8B-C9B	18	6	200	0	No corroida
VC3B-C4B	20	6	200	16	No corroida

. Results of slabs survey

It was determined that the slabs are of the ribbed type. These slabs generally present the problem that the rib reinforcements do not have an adequate concrete coating, which means that they are exposed to a premature onset of corrosive processes. Thus, in the mechanical surveys it was observed that the reinforcements were in an active corrosion process, Figure 11. These reinforcements were made up of AL-220 type steel bars. On the other hand and as a consequence of the active corrosive processes mentioned above, in some sectors of the slabs the ceramic elements that compose it were cracked and with a sudden risk of detachment.

Figure 11: Slab reinforcement corrosion

4. Evaluation of the capacity to seismic actions of the primitive structure

The seismic evaluation for existing structures was carried out according to Chapter 11 of the INPRES - CIRSOC 103 - Part I (2018) standard. As a result of this evaluation, the construction is classified and the exceptions that are admitted for this building with respect to the requirements that the standard demand for new constructions are determined. In this classification procedure it is necessary to determine a quantitative measure that is the construction safety index r. This index is calculated by multiplying by 100 the ratio between the nominal shear base capacity of the construction, with respect to the shear base demand. The safety of the construction will be sufficient if r is > 100, low if it is between 80 and 100, limit if it is between 60 and 80, and insufficient if this index is < 60. In order to determine the aforementioned shear capacities, the starting point is analysis of the loads on the building. A computational model of this buiding should also be made, considering the properties of the materials analyzed in Section 3 of this paper.

4.1 Computational simulation

For the structural evaluation, a finite element software was used. The structural elements columns and beams were modeled using two-node frame-type linear finite elements, while the walls and slabs were modeled with four-node shell-type surface finite elements. The reduction in stiffness of the walls due to cracking was considered, as established in the INPRES - CIRSOC 103 - Part III (2018) standard, where it is indicated that the reduction of the cross-sectional area corresponds to Ae = 0.6 Ag, while the reduction of the moment of inertia at Ie = 0.4 Ig.
For the masonry walls a normal-type mortar (N) was considered, the result is then an average stress f'm = 1.75 Mpa, and with it, an elasticity modulus E = 2100 Mpa and a shear modulus G = 420 Mpa.
The 3D model of the building considered in the structural analysis is presented in Figure 12, and the lines that define the structural planes are shown in Figure 13.

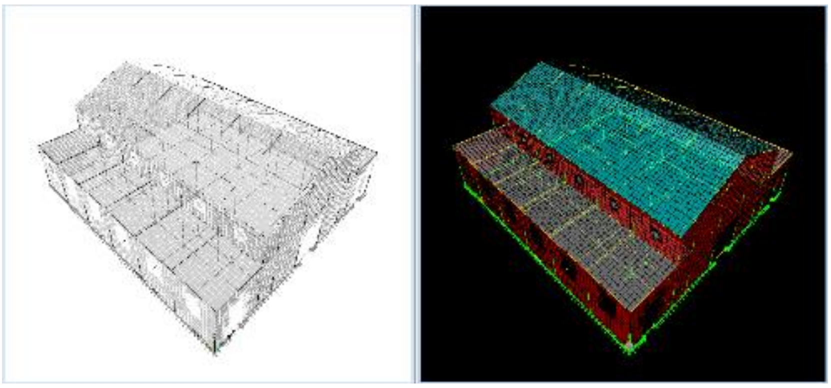

Figure 12: Model of the building under study

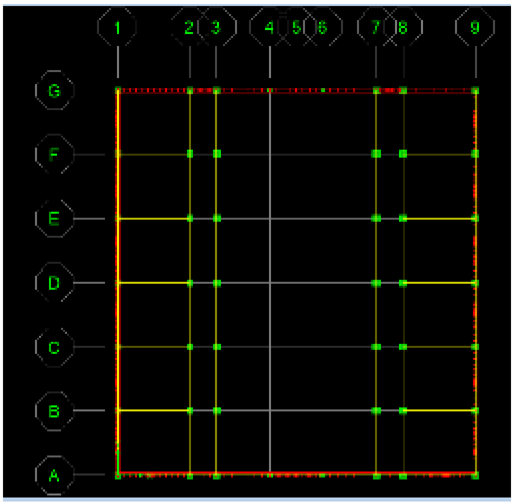

Figure 13: Structural lines

Regarding the reinforced concrete elements, a H-20 concrete type (CIRSOC 201, 2005) has been considered, resulting in its modulus of elasticity E = 21,000 Mpa.

4.2 Demand in structural elements versus capacity.

The determination of the seismic coefficient was made according to the prescriptions established in the INPRES - CIRSOC 103 - Part I (2018) standard. Then the shear base demand was calculated. Table 4 presents the seismic demand on each of the structural planes considered in the modeling.

Tabla 4 – Seismic demand on structural elements

Elemento	Dirección	eje	Demanda [kN]
Muro	X	A	2 740
Pórtico		B	52
Pórtico		C	62
Pórtico		D	66
Pórtico		E	62
Pórtico		F	50
Muro		G	2 537
Muro	Y	1	2 656
Pórtico		2	84
Pórtico		3	86
Pórtico		7	86
Pórtico		8	84
Muro		9	2 656

In accordance with INPRES – CIRSOC 103 – Part III (2018), the shear design resistance of the walls was determined. Table 5 shows the resistances obtained. To determine the design strength of the structural planes, the shear capacities of each column (determined according to CIRSOC 201, 2005) were considered and values were compared with the base shear determined by nonlinear static analysis. The design resistance values considered for these structural planes are shown in Table 6.

Table 5 – Design shear strength of walls

Dirección	eje	f'v [Mpa]	fo [Mpa]	Ag [mm^2]	Vn [kN]	ϕ	Vd [kN]	Vmuros [kN]
X	A	0.190	0.088	11.3E+06	2545	0.80	2036	3592
	G	0.190	0.097	8.5E+06	1945		1556	
Y	1	0.190	0.022	9.1E+06	1809		1447	2894
	9	0.190	0.022	9.1E+06	1809		1447	

Table 6 – Shear design strength of frame columns.

Dirección	eje	Vd [kN]	Vpórticos [kN]
X	B	221	1105
	C	221	
	D	221	
	E	221	
	F	221	
Y	2	291	1198
	3	308	
	7	308	
	8	291	

4.2 Safety index r.

Table 7 shows the security index in each direction of analysis.

Table 7– Safety index in each of the directions.

Dirección	Vmuros [kN]	Vpórticos [kN]	Capacidad [kN]	Demanda [kN]	Índice de seguridad r [%]
X	3592	1105	4697	5500	85
Y	2894	1198	4092	5500	74

5. Intervention methodologies

5.1 Masonry walls

All the masonry walls of the original work will be transformed into reinforced masonry walls with distributed reinforcement and shotcrete, Figure 13; constituting this structural typology as the main one against the seismic demand.

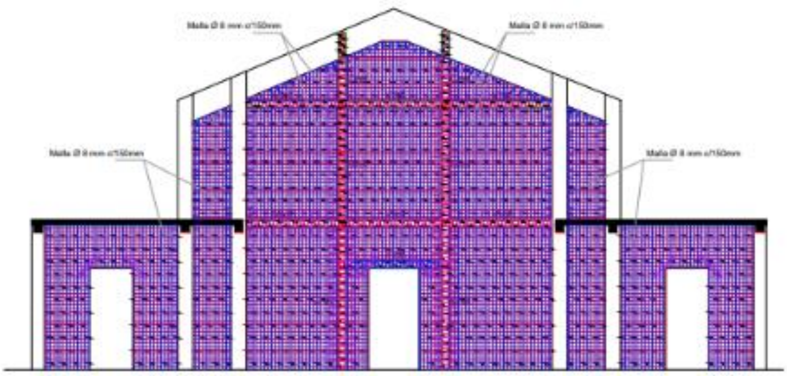

Figure 13: Shotcrete on facades

On the other hand, in order to ensure that the transverse walls of the façade and the altar have the necessary capacity to resist actions perpendicular to their plane, a grid system of reinforced concrete elements has been designed, Figure 14.

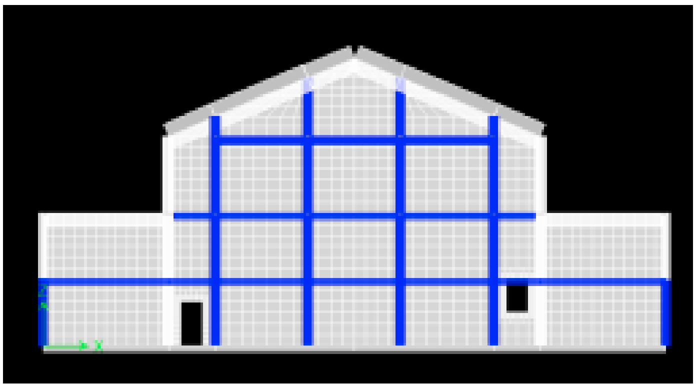

Figure 14: Reinforcement of façade walls when overturning out of plane.

5.2. Reinforced concrete frames

As the main seismic-resistant system of the new construction, masonry walls reinforced with distributed reinforcement and shotcrete have been considered, in both main directions. Therefore, the transverse and longitudinal frame system has been considered as a redundant structure. Even so, the increase in the confinement of the isolated columns at the ground floor level was foreseen, also with the technique of distributed reinforcement and shotcrete.

5.3 Slabs

For the reinforcement of joists in slabs with insufficient capacity, or in areas where there is a loss of reinforcement due to corrosion, carbon fiber plates will be used. In addition, a specific treatment will be applied to the joist areas where corrosion has affected the concrete and reinforcement; and ceramic elements will be removed when fractures or loose parts are detected..

6. Conclusions

In the present work, the integral evaluation of the performance before seismic actions of a structure located in an area of high seismic danger is presented. For this evaluation, it was essential to carry out a study of the pathologies that affected the construction, as well as the characterization of the constituent materials. These studies were carried out with appropriate devices in order to be able to infer the possible pathologies that affected the structure. The need to have characteristic values of the materials is extremely important, since these parameters are then used as input data for structural modeling. Finally and through the processing of all the experimental and numerical information obtained, it was possible to adopt the different rehabilitation strategies considered most convenient to correct each of the pathologies detected, thus achieving that the structure meets the demands imposed by the current standards, guaranteeing its structural safety.

Acknowledgment

To the Regional Center for Technological Developments for Construction, Seismology and Earthquake Engineering (CeReDeTeC) of the National Technological University - Mendoza Regional Faculty.

References

IRAM 1694 (1989). Portland cement concrete. Test method for the surface hardness of hardened concrete by determining the rebound number using the spring sclerometer.

IRAM 1683 (1990). Portland cement concrete. Method for determining the speed of ultrasonic pulses.

INPRES – CIRSOC 103 – Part I (2018). Argentine Regulation for Earthquake Resistant Constructions. Constructions in general.

INPRES – CIRSOC 103 – Part III (2018). Argentine Regulation for Earthquake Resistant Constructions. Masonry constructions.

CIRSOC 201 (2005). Argentine Regulation of Concrete Structures.