Seismic Performance Of Existing Residential Buildings

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SUMMARY:
One of the biggest challenges for structural engineering in Argentina is the evaluation of the seismic safety of existing buildings. This problem is particularly important when it comes to residential buildings, due to the high density of people and socio-economic consequences that would entail the partial or total collapse of the same, after the occurrence of strong earthquakes.
Based on the performance design, this paper presents an approach for the study of the seismic vulnerability of existing mid-rise buildings.
An initial qualitative assessment is used to determine structural abnormalities and calibrate the analytical model of the structure. The subsequent analytical evaluation uses a nonlinear incremental static analysis to verify if the analyzed structure achieves performance targets selected.
This study evaluated two apartment complexes built with obsolete regulations, both located in the city of San Juan, Argentina.
The results clearly indicate the desirability of using the proposed methodology for evaluating existing buildings designed with obsolete codes and also allowed to estimate the seismic performance of structural typologies most frequently used: frames and walls of reinforced concrete.

Keywords: seismic vulnerability, existing residential buildings, performance design.

1. INTRODUCCION

The evaluation of the seismic safety of existing buildings is a pending task for structural engineering in Argentina. Learn about the seismic performance for different levels of dynamic excitation that can be applied to structures during his life it is essential to assess the possibility of rehabilitation, to study the level of expected damage and its socio-economic consequences and to establish new guidelines for structural design. While it is expected that the buildings designed and constructed with the current regulations (INPRES CIRSOC 103) can withstand a destructive earthquake without collapsing, its behavior during earthquakes of lesser intensity, in relation to the performance of other states limits, as damage control and service, cannot be predicted with reasonable certainty.
On the other hand, the buildings designed and constructed with obsolete regulations, before the decade of the 80's, have a high probability of collapse in the face severe earthquakes. This is particularly important in the case of residential buildings due to the high population density and the social and economic impact that would be partial or total collapse of the building after the occurrence of a severe earthquake.
In this country there is no regulation or guideline for assessing the seismic performance of existing buildings. This regulation INPRES CIRSOC 103 sets out the conditions that must meet a new construction, in order to permit the design by elastic analysis, following the approach of force-resistance. The inelastic incursion is ensured by means of the ductility of the structure components. These conditions are not normally present in the constructions made with previous standards.
For these reasons, the evaluation methods in general should be based on displacements, recognizing the inelastic behavior of structures and under the considerations of the design for performance.

2. BUILDINGS EVALUATED

We was evaluated the seismic performance of existing residential buildings. We considered two structural types, which are the most common in mid-rise buildings in the Province of San Juan.
A secondary objective is to determine the advantages and disadvantages of the different types.
In San Juan structural typologies frequently used for residential buildings: masonry walls, reinforced concrete frames and reinforced concrete walls.
Statutorily the masonry walls can be used only for low-rise buildings, under 12 m, which is one reason why this typology was not studied in this work.
In view of the foregoing were elected two residential buildings, the first of them is a tower belonging to the San Martin neighborhood and the other the consortium 9 de Julio, both built with obsolete regulations (CONCAR 70). Have eight and nine levels respectively (the height is greater than 25 m) and different structural types, walls of reinforced concrete walls in the case of the tower San Martin and reinforced concrete frames in the tower of the consortium 9 de Julio.

2.1. San Martín neighbourhood

This is an urban neighborhood that has a school, a police station, shops and recreational areas. The apartments are located in two types of formations: towers of eight and three levels, Figure 2. Groups of two towers of eight levels are linked to different levels by horizontal circulations that also allow access from the elevator (central position). These bridges are structurally isolated seismically of the towers, so that each tower can be seen as an independent structure, Figure 3. Of the visual inspection and analysis of graphic documentation (drawings of structures, forms of structural elements and memories of calculation) could not check the disruption of some walls on the third floor, a similar situation occurs in some parts of the ground floor, Figure 4. While the quality of the materials used is good and the dimension of the elements is generous, there were some structural defects that are characteristic of obsolete rules, including:

- The amount of longitudinal reinforcement in some components is less than the minimum.
- The diameter and spacing of transverse reinforcement is inadequate and there is no further densification in critical areas.
- The anchorage length of longitudinal reinforcing bars in beams and columns in critical sections is limited.
- The length and location of splices of longitudinal reinforcement bars in critical sections is inadequate.
- Using bar bent to absorb shear stresses.

These irregularities suggest that in these sectors, at least, the structural system will not have a suitable behavior and can compromise the overall behavior of the structural system.

Figure 2. View of the neighborhood "San Martín".

Figure 3. View of two towers and horizontal linkages.

Figure 4. Discontinuity walls in the first and fourth floor.

2.2. Consortium 9 de Julio

The consortium 9 de Julio is a housing complex that consists of four towers on nine floors each one. In this case, the seismic structure consists of reinforced concrete frames in the two main directions of the building. These towers are vertical irregularity. This irregularity is manifested by the presence of a ground floor free while masonry has been used to close the openings of the perimeter frames in the rest of the levels. The balconies on each floor were built using slabs cantilever, dibujo5. The visual inspection and analysis of the graphic documentation provides the following information:

- Vertical irregularities caused by partial disruption of the masonry at the ground floor.
- The balconies in many cases have been closed completely incorporating considerable weight to the structure of the building and causing displacement of the mass center of each level.
- Sanitary maintenance free cause moisture problems in walls and gates

The marked irregularity of stiffness in height (ground floor flexible) indicates that perhaps this is a factor that may compromise the overall behavior of the structural system.

Figure 5. Views of a tower of the consortium on July 9

3. METODOLOGY

3.1. Performance objectives

Performance objectives are an expression of the desired behavior of the building. They are the result of the relationship between level of performance previously established, the probability of occurrence of an earthquake and the fate of the building.
FEMA 356 "Pre-standard and Commentary for the Seismic Rehabilitations of Buildings" fixed the performance objectives to serve as a basis for a design that will determine the extent of rehabilitation, the cost, the feasibility of the project, both as the benefit of obtaining at the appropriate time a greater security, a reduction in the likelihood of damage and interruption of its services in future seismic events.
This regulation defines four performance levels: Operational, Immediate Occupancy, Life Safety and Collapse Prevention; and four levels of design earthquakes: Frequent, Occasional, Rare and Very Rare. They are the result of the possible combinations, the different performance objectives.
In accordance with the use and occupation, the buildings are grouped as follows: Safety Critical Facilities, Essential Facilities and Basic Facilities. Due to the fact that the two buildings are residential, the seismic performance objectives that need to be evaluated are the a, f, k and p, Table 3.1.
It could be said that the performance level "a" (Permanent Operation), the building works in the elastic range and does not damage is expected in the non-structural elements. The building continues to function after a frequent earthquake.
For the level "f" (Immediate Occupancy), are expected moderate damage. It is most likely that the service is temporarily suspended or for a small period of time to make the adjustments and revisions in all systems of the building. This would be related to an occasional earthquake.
For the level "k", (Life Safety), expected structural and non-structural damage. But the damage to the structure are economically repairable. The building still has a considerable structural reserve and is therefore far from collapse. This condition is necessary for a rare earthquake.
Finally, the level "p", it is expected that the structure did not collapse or crashing, but perhaps the damage are significant and in some cases it may be your repair economically unviable. This condition is expected to occur for an earthquake very rare.

Table 3.1. Performance objectives, FEMA 356.

Performance levels / Design earthquake	Operational 1-A	Immediate Occupancy 1-B	Life Safety 3-C	Collapse Prevention 5-E
Frequent (50% in 50 years)	a	b	c	d
Occasional (20% in 50 years)	e	f	g	h
Rare (10% in 50 years)	i	j	k	l
Very Rare (2% in 50 years)	m	n	o	p
e+j+o= Essential Facilities a+f+k+p= Basic Facilities				

3.2. Numerical modeling

Both buildings were modeled in three dimensions with six degrees of freedom assets, using the ETABS program, in its nonlinear version 9.0 by performing incremental non-linear static analysis (pushover), Figure 6.

To examine the behavior of the non-linear materials, were assigned to the beams, columns and walls of reinforced concrete plastic hinges defined according to the predominant form in which each element works.

Therefore the beams and the concrete walls are have flexural hinge, while the columns are plastic hinges of interaction between normal loading and bending moment in both directions of analysis.

For plastic hinges of beams and considering the excessive separation of stirrups and lack of densification in the critical areas of the Table 6-7 of FEMA 356 were extracted parameters that define the behavior of load / deformation of the element.

With these data, plastic hinge of beams and the walls, flexion were defined as FH1 and the corresponding to the columns of type PMM, as FH2.

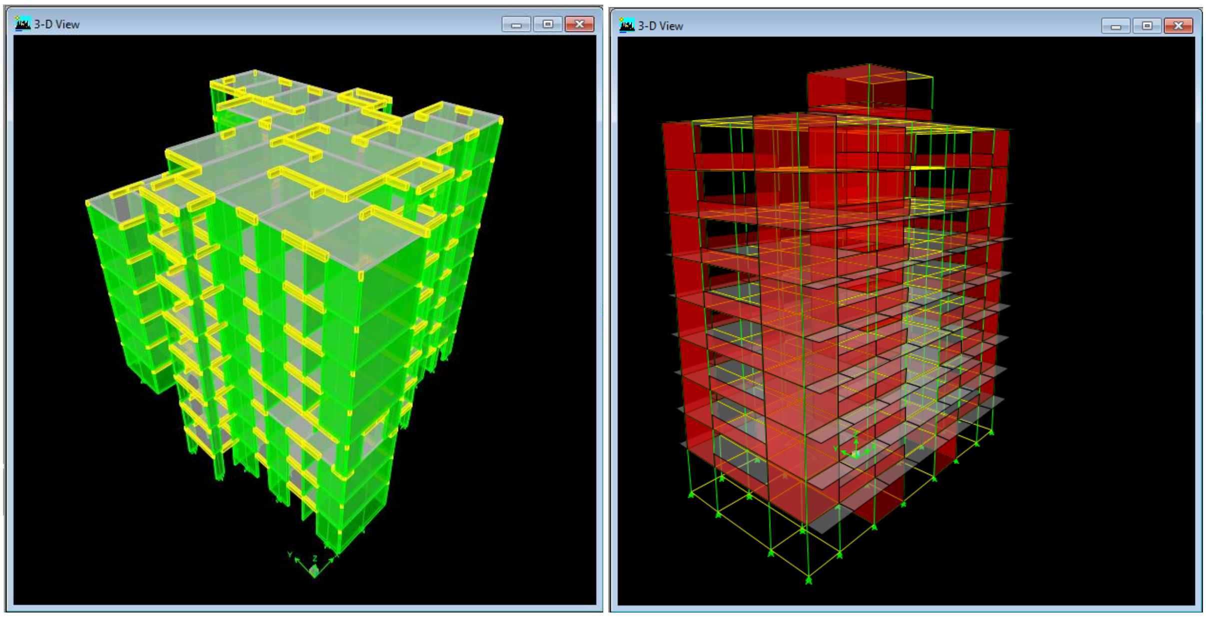

Figure 6. Models of the towers analyzed, San Martin tower (left) and July 9 tower (right).

3.3. Definition of loads

It is assigned to two types of charges, those that his characteristic can be considered as permanent and other temporary or accidental.

The permanent loads were divided in: Dead Loads, Super Dead Loads and Live Loads.

Table 3.2 shows the total values for all the loads of both buildings.

Table 3.2. Total permanent loads.

Types of loads	San Martín	9 de Julio
	[t]	
Dead Load	2606.7	2295.0
Super Dead Load	77.5	799.5
Live Loads	570.4	632.0
Total	3254.5	3726.5

The accidental loads due to the earthquake were applied in the two main directions for each building. The vertical distribution adopted for the seismic force results in an inverted triangle. Equation 1 calculates the seismic force side (F_i) that corresponds to a level "i" considered, where m_i is the mass of that level.

$$F_i = \frac{m_i \times h_i}{\sum m_i \times h_i} \qquad (1)$$

3.4. Nonlinear static analysis - Pushover

Of the various methods available for the evaluation and seismic design of structures, the methods of nonlinear static analysis are the most commonly used in practice, due to the simplicity of its application without losing consistency of the results. Through them we can estimate the inelastic displacement respondent for earthquakes defined in each of the pre-defined goals. The estimate of the maximum inelastic displacement defendant in a structure of several degrees of freedom is performed from the maximum demand of displacement of a equivalent system of one degree of freedom. The general procedure of the methods of nonlinear static analysis is to transform the actual structure of various degrees of freedom, in a system equivalent of a degree of freedom and represent earthquakes through the use of response spectra, which leads to the estimation of the demand for global offset.
The accuracy of the results of the methods of nonlinear static analysis focuses primarily on the accuracy of data entered into the software. Hence the importance of the preliminary study and interpretation of the technical documentation of buildings.

3.5. Determination of Performance Point

Performance Point is the point where the capacity of the structure and the demand from the earthquake used for the analysis intersect. In order to verify silos buildings analyzed are able to achieve a performance point we proceeded to the use of the program ETABS. 9.0. The software used for determining the performance point of the procedure B of Method ATC 40.

3.5.1.Definition of earthquakes for each level of performance
We introduce the characteristic parameters of the response spectra corresponding to 5% damping for Earthquakes Very Rare, Rare and Occasional, Ca (peak ground acceleration) and Cv (spectral acceleration for a period of 1 s) obtained previously for soil type C.
The spectra of demand for other levels of damping are created from the spectrum of 5% using the reduction factors described by the ATC 40 and FEMA 356. The ATC-40 provides reduction factors based on the effective damping βeff to areas controlled by the acceleration (SRA) and the area controlled by the velocity (SRV), these factors are applied to linear elastic design spectrum with 5% buffer or spectrum demand. The reduction factors were calculated using Equations 2 and 3.

$$SR_A = \frac{1}{B_S} = \frac{3.21 - 0.681 \times \ln(\beta eff)}{2.12} \qquad (2)$$

$$SR_V = \frac{1}{B_L} = \frac{2.31 - 0.41 \times \ln(\beta eff)}{1.65} \qquad (3)$$

When the structure enters the inelastic field, the energy dissipated by hysteresis resembles an equivalent viscous damping. This can be represented as a combination of viscous damping (β_0) inherent in the structure and hysteretic damping (β_{eq}) associated with the inner area of the bonds that form when you plot the shear stress at the base of the building vs. the displacement of the structure (ATC, 1996), Equation 4.

$$\beta eq = \beta_0 + 0.05 \qquad (4)$$

This equation is realistic for ductile structure with good detail and subject to shocks of short duration, for other configurations lead to overestimates of viscous damping. To correct the imperfection of the hysteresis loops in structures that do not meet the characteristics mention introducing a modification factor (k). Equation 5 was used to calculate the effective damping (βeff).

$$SR_A = \frac{63.7 \times k \times (ay \times dm - dy \times am)}{am \times dm} + 5 \qquad (5)$$

In the case of San Martin tower, the seismic behavior of the structure responds to the type C, with severe degradation in the hysteresis loops, accounting for the minimum values of equivalent damping, with K = 1/3.

With these data, the program draws a family of demand spectra for each selected verification earthquake; every curve of the family has a different effective damping.

3.5.2. Capacity-Demand spectrum

The procedure applied for the determination of the performance point is the same for both buildings analyzed. Although then discussed and compared the results of both structural types are listed below by way of example only results corresponding to the tower San Martin.

Three analyzes were performed type nonlinear pushover. In the first one (Push1) gravity loads were applied incrementally with force control, Equation 6.

$$Push1 = Dead\ load + Super\ dead\ load + 0.5 \times Live\ load \qquad (6)$$

The two following pushovers (Push2 and Push3) correspond to the lateral load patterns previously defined and applied in the X and Y direction respectively.

It was modeled inelastic behavior of materials first, considering only flexural hinges, defined as above. These plastic hinges are characterized as having a ductile behavior, even where structural design corresponding to obsolete regulations where the capacity of the plastic hinge is less than in the case of actual seismic-resistant design. The analysis is performed first for the Rare earthquake to check the response and then as the results continue with the remaining earthquakes.

The demand spectra for a given earthquake verification and for different damping levels are automatically determined from the spectrum corresponding to 5% damping.

The demand spectrum for the Rare earthquake and soil type C (INPRES-CIRSOC) shows an acceleration anchor (Ca = 0.35) and acceleration for a period of 1s. (Cv = 0.455). For this earthquake, in both directions of analysis, it reaches a point of performance although this situation is developed at the end of the capacity curve, leaving the remaining structure unable to reach a point of intersection for situations more demanding as it would be if imposed by the quake very rare.

When shear hinges were used, keeping all other parameters unchanged, could not reach a yield point in any direction of analysis, namely that the capacity of the structure is not sufficient demand for rare earthquake.

Figure 8 shows the end distribution of the plastic hinges reaching San Martin tower, modeled flexural and shear hinges, while Figure 9 shows the Capacity-Demand spectrum for the rare earthquake in x direction of analysis.

For the occasional earthquake (Ca = 0.153 and Cv = 0.199) with flexural and shear hinge is reached the point of performance in both directions. The displacement demand is almost a third of the available capacity.

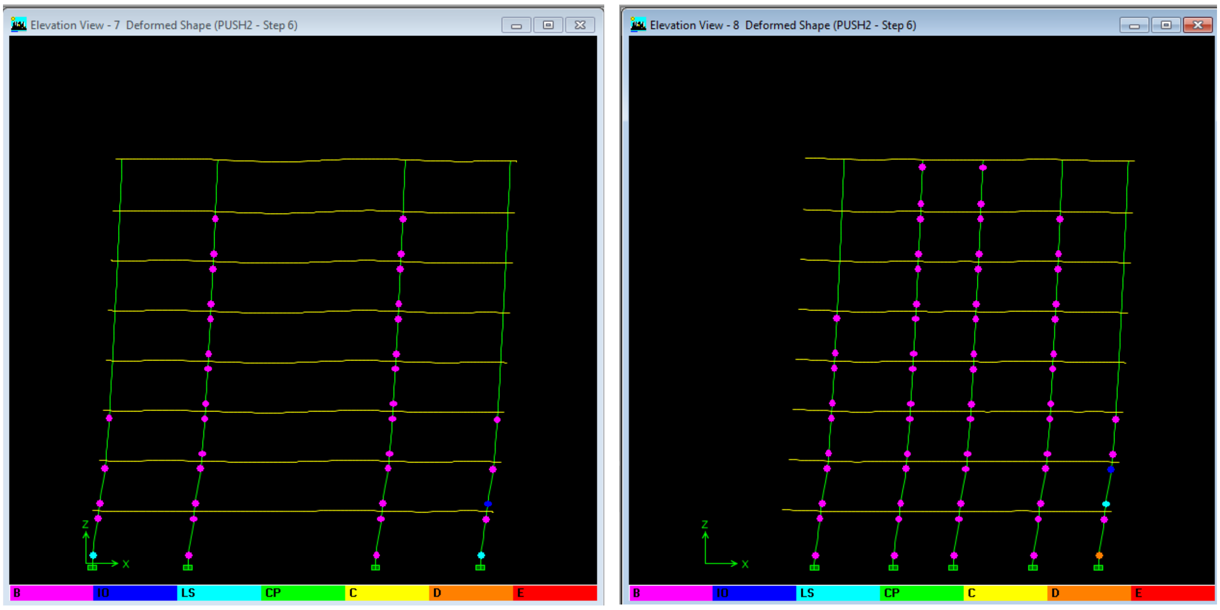

Figure 8. Distribution of the plastic hinges, San Martin tower.

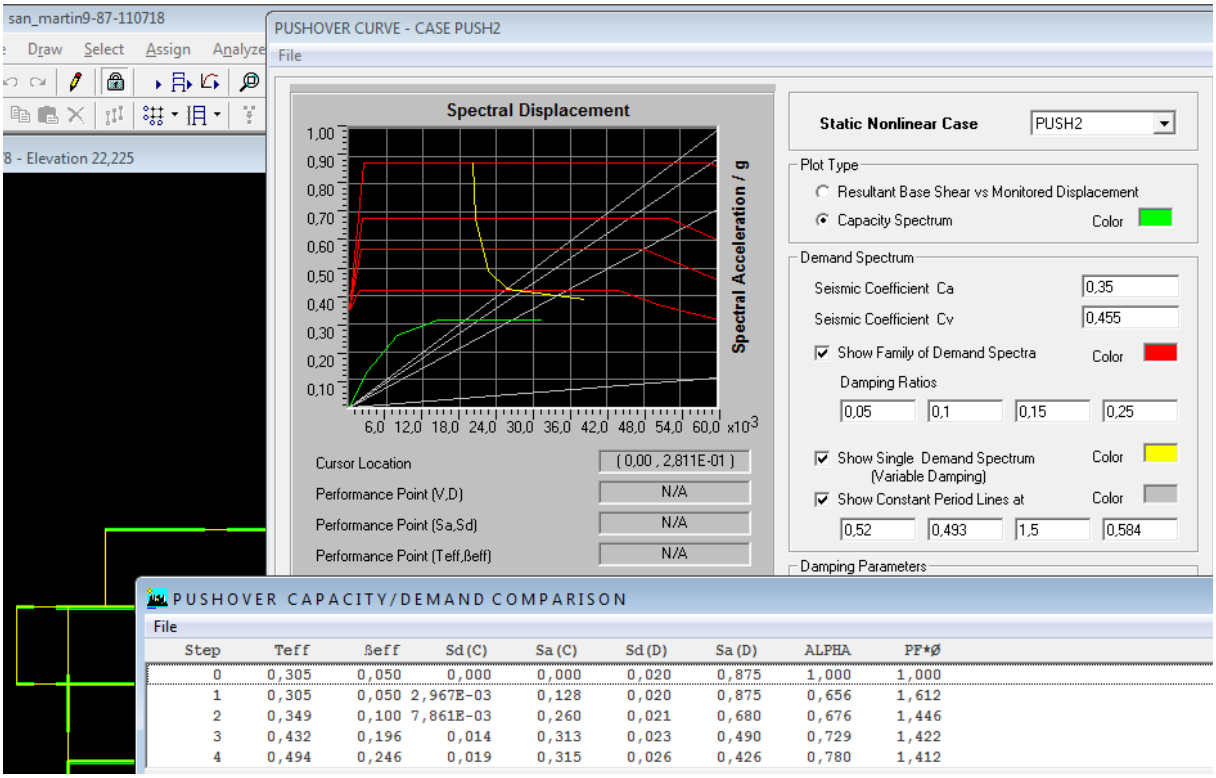

Figure 9. Capacity-Demand spectrum for the rare earthquake, x direction.

4. ANALYSIS OF RESULTS

4.1. San Martín tower

For the frequent and occasional earthquakes performance point is reached.
For the rare earthquake, considering only flexural hinges, the collapse occurs when a large number of

plastic hinges have passed the area of prevention of collapse. With flexural and shear hinges collapse occurs due to the formation of a soft floor mechanism. plastic hinges are formed in base and capital of several walls in a level.
For rare earthquake, more demanding, the building not reached the performance point.

4.2. 9 de Julio tower.

For the frequent and occasional earthquakes performance point is reached.
For the rare earthquake, only with flexural hinges, the point of performance is achieved with a translation of beams mechanism, some plastic hinges are worn out with a remaining capacity that is not enough to meet the demand of the earthquake very rare, whereas flexural and shear hinges, the collapse occurs due to the formation of a soft floor mechanism. Plastic hinges occur in base and capital of multiple columns on one level. For rare earthquake, more demanding, the building not reached the performance point.

5. CONCLUSIONS

The methodology used to assess the overall performance of a building, identify their strengths and weaknesses and design a rehabilitation accordingly. It is very important preliminary assessment which gives a first information of the deficiencies of the construction and avoids the more sophisticated calculations.
Both towers tested failed the Preliminary Assessment and Numerical Analysis confirmed their inability to meet established performance objectives. According to the methodology presented is sufficient that only one objective is not met to conclude that the building requires rehabilitation.
The reasons that limit their capacity to be the sum of several factors, among which mainly include:
The codes used at the time of construction of these buildings did not include specifications designed to increase the shear capacity in critical areas of structural elements, for example, densification of transverse reinforcement in the proximity of knots, and on the other hand considered lower seismic demands to the present.
Problems in architectural design, such as those found in the discontinuity walls to San Martin tower or flexible floor of the 9 de Julio tower.
Modifications made after the construction of buildings by the owners. The most important are related to the closing of balconies in the 9 de Julio Tower and the opening of internal walls to communicate environments in the San Martin tower.
As a plus in both cases studied showed good quality of materials and some structural redundancy of great importance to meet the deficits identified.
Only managed to achieve the performance objectives for Occasional earthquake, with good displacement reserves, this would imply a low level of structural and nonstructural damage.
For Rare earthquake associated with the Safety of Life, not reached the performance point, resulting in concentration of damage at the levels where there is discontinuity of rigidities (third level for the San Martin tower and ground floor for the 9 de Julio tower) which leads to total collapse of the structure.

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