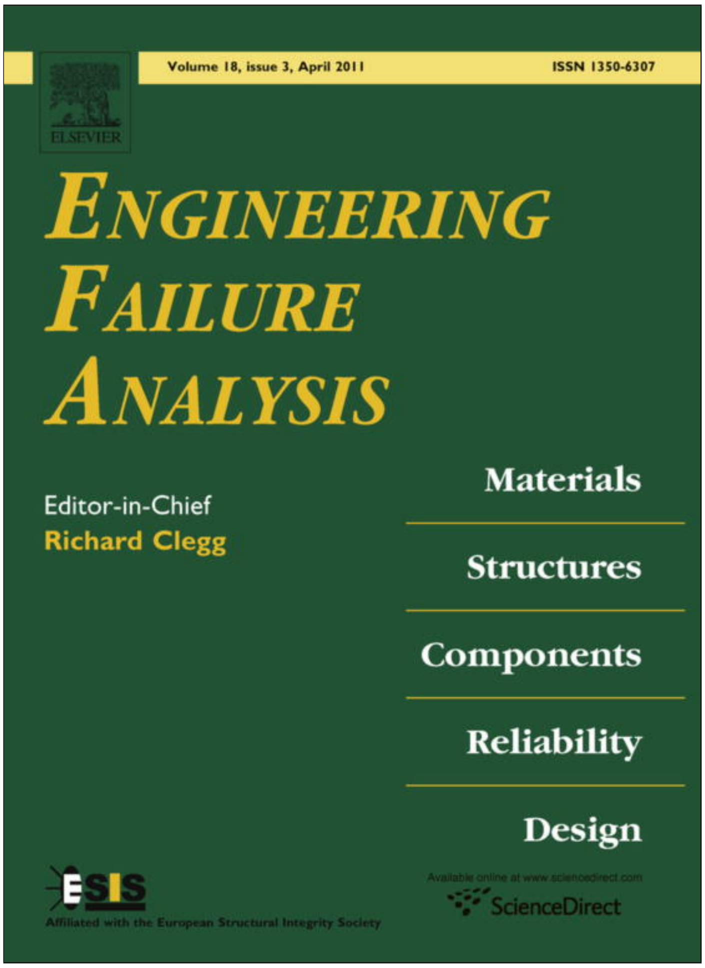
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The progressive failure of 15 balconies and the engineering techniques for their reconstruction

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ABSTRACT

Despite the fact that structural failures are not desirable, it's worth to observe that it always provides relevant information for enhance design and building techniques. Additionally, failure investigations can provide a clear understanding of the involved technical problems, avoiding known mistakes to be produced again. However, the causes of many structural failures are not registered in the technical literature, in a way that some unsubstantiated information produced by the media is usually prominent. The lack of technical reports concerning structural failures gives rise to the repetition of very basic errors, which may inclusive produce progressive failures. In this context, the present paper aims at providing the main reasons for the progressive failure of 15 reinforced concrete balconies of a residential building in Maringá, Paraná state, Brazil. The engineering techniques used for demolishing and reconstructing the balconies of the mentioned building are also discussed. Finally, it is important to highlight that the activities described in this article have contributed in a very positive way for recovering the right of property, the psychological health and the feeling of security of the residents of this damaged building.

1. Introduction

Brazil has been presenting an increasing number of victims related to structural failures, even not facing major natural actions like tornados as well as human actions like terrorism. Some reinforced concrete failures in Brazil, for example, are fully described in [10,11,21]. Elsewhere around the world this undesirable situation is also present, as can be seen in [7,9,14–17,23].

It is also not difficult to find structural accidents involving non-housing structures made of reinforced or prestressed concrete structures. Refs. [8,13,18,20,22], for example, have published some findings regarding failures related to non-housing structures.

Apparently, many accidents are consequence of misconduct and negligence. Maybe for that reason is just possible to find a few records concerning structural failures in the literature. However, the well-dressed divulgation of some accidents, just focusing on the scientific reasons of the problem, could contribute to the enhancement of the structural codes, as well as for the improvement and learning of the freshmen engineers. Additionally, the technical description of structural accidents could contribute at least for the awareness of experienced engineers on the need of keeping the proper conduct of the profession.

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When the good practice is reserved to the second plane and some external agents act in a systematical way, a very propitious environment is created for the occurrence of structural failures. This combination was exactly the scenario for a progressive failure occurred in Maringá, a city of 300.000 inhabitants located in the Paraná State, Brazil. At the dawn of October 26th 2008, the residents of a housing building were surprised by the progressive collapse of one canopy and 15 reinforced concrete balconies. The progressive failure occurred after heavy rains and what could be a tragedy without precedents was transformed in enormous material damage and some psychological problems for the owners of the apartments.

In an effort of preserving the involved parts, the present paper aims at presenting the main causes of the mentioned failure, focusing only in information which can contribute for the enhancement of the construction and design techniques. In this way, the strategies used for demolishing the remaining balconies as well as the procedures used for reconstructing the failed/demolished balconies are presented. As a matter of ethics, the name of the building, its location and the professionals involved in the construction and design will be omitted in this paper.

2. Architectural and structural drawings

Located in a prime area of Maringá, the damaged housing building has a terrace, a subsoil garage and 15 levels with two apartments by floor. Fig. 1 presents the building frontage, where is possible to detect the progressive failure occurred in the right side of the construction.

As can be seen in Fig. 1, there were two identical columns of suspended balconies in the principal façade of the building. This balconies had trapezoidal format with an approximated area of 2.00 m^2 as shown in Fig. 2a. The architectural project had specifications for a small pot plant at the corner of the balcony as well as a glass grid. However, there was no specification about the coating that should be used in the balconies floors.

The positive and negative reinforcements used for the cantilevered slabs of the balconies are shown in Fig. 2a. The analysis of the structural drawings revealed that the slabs designed for the balconies should have a thickness of 10 cm, concrete cover of 1.5 cm, concrete with characteristic compressive strength of 15 MPa and steel bars with yielding characteristic strength of 600 MPa.

For the canopies of the 15th pavement were also used cantilevered slabs which were linked to the adjacent slabs of the terrace (L24), as shown in Fig. 2b. The cantilevered canopy slabs were specified to have a thickness of 7 cm, concrete cover of 1.0 cm, concrete with characteristic compressive strength of 15 MPa and steel bars with yielding characteristic strength of 600 MPa.

3. Description of the progressive failure

The housing building described in the present paper had its beginning in 1987. In that period the construction presented some initial problems, which in turn led to a judicial fight. The building had its construction interrupted for many years and just around 1996 the construction was released for being finished.

Based on some testimonials, the interruption of the mentioned building has occurred based on the doubtable procedures adopted for the design and construction of the sheet-pile walls in the subsoil, where the garage of the building was located. The failure of the sheet-pile wall has given rise to a series of structural problems in some neighbor buildings.

Fig. 1. Progressive failure of balconies of a housing building in Maringá (Brazil).

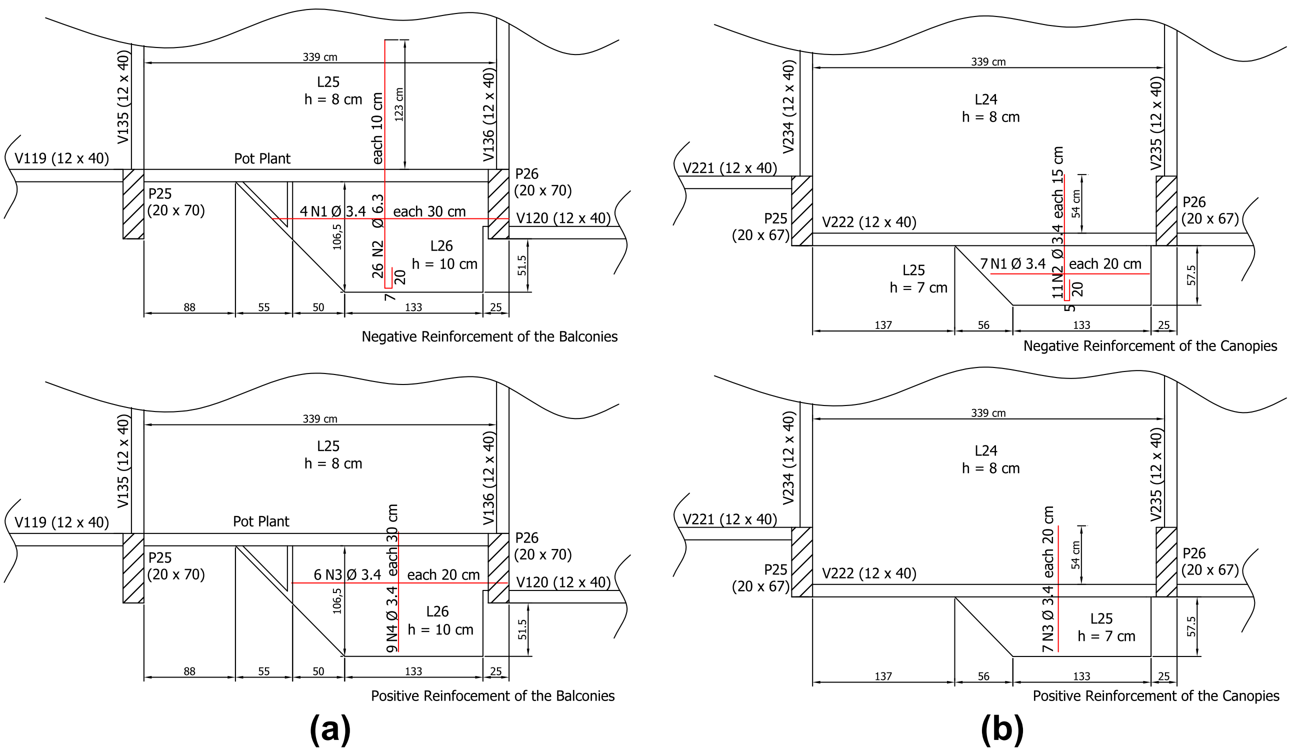

Fig. 2. Dimensions and reinforcements of (a) balconies and (b) canopies.

A eight-floor housing building, for example, has presented differential settlements of about 18 cm and a major non-alignment between the top and the bottom of the construction of about 35 cm. Grigoli [12] describes that this building had to pass by a general recuperation of the vertical and horizontal alignments by using mechanical jacks. Besides that, the foundation floor, firstly executed with reinforced concrete piles, needed to be substituted by large diameter bored piles and equilibrium beams strategically located in order to recover the global stability of the building.

Grigoli [12] described that the verified damages occurred in May 24th 1987 after heavy rains. The free-earth sheet-piles cantilevered walls were made with circular reinforced concrete elements and were not capable of supporting the induced pressures. The damages caused in the mentioned eight-floor housing building was so intense that the building needed to be leaned with struts, taking into account the possibility of global failure based on the excessive settlements observed.

Approximately 12 years after recording this first incident in the vicinity, the building in question has passed by another damage situation, once again without presenting victims. At the dawn of October 26th 2008, occurred the progressive collapse of one canopy and 15 reinforced concrete balconies throughout the frontage of the building, as previously shown in Fig. 1.

At the time of the accident, many residents were surprised by a strong noise. In a naturally way, some residents ran to their balconies in order to find some answers for that low noise heard. Fortunately, before opening the windows of the balconies, the residents were able to realize that was exactly from there the origin of that uncommon noise. It looks a little bit fantastic, but some residents could be failed from their apartments, simply by an instinctive reaction of curiosity and fear.

The progressive failure has produced the collapse of one slab situated above the garage in the subsoil, as illustrated in Fig. 3a. As can be seen, there was a trend of deviation of the free-falling debris to the right side of the balconies columns, taking as reference the terrace of the building. This deviation of the debris was probably due to the high stiffness provided in the connection between the slabs of the balconies and the columns P26, as can be seen in Fig. 2a. The progressive failure has given rise to the failure of a slab above the subsoil-garage in the building (Fig. 3b), as well as some beams (Fig. 3c) and columns (Fig. 3d).

Taking into account the possibility of new failures, the remaining canopy above the 15th balcony was shored using wood struts and the debris accumulated in front of the main entrance of the building was immediately removed. Despite the fact that all providences were taken focusing on the preservation of human life, one should realize that some important steps, recommended by Peraza and Cuoco [19], for example, were not followed by the authorities taking control of the damaged site.

The lack of preparation of the authorities, which were the first professionals to arrive at the site, could give rise to the disappearance of import perishable proofs. Fortunately, the mistakes committed on preserving this damage building were so clear that even the mentioned lack of procedure did not affect the investigation and the main conclusions about this progressive failure.

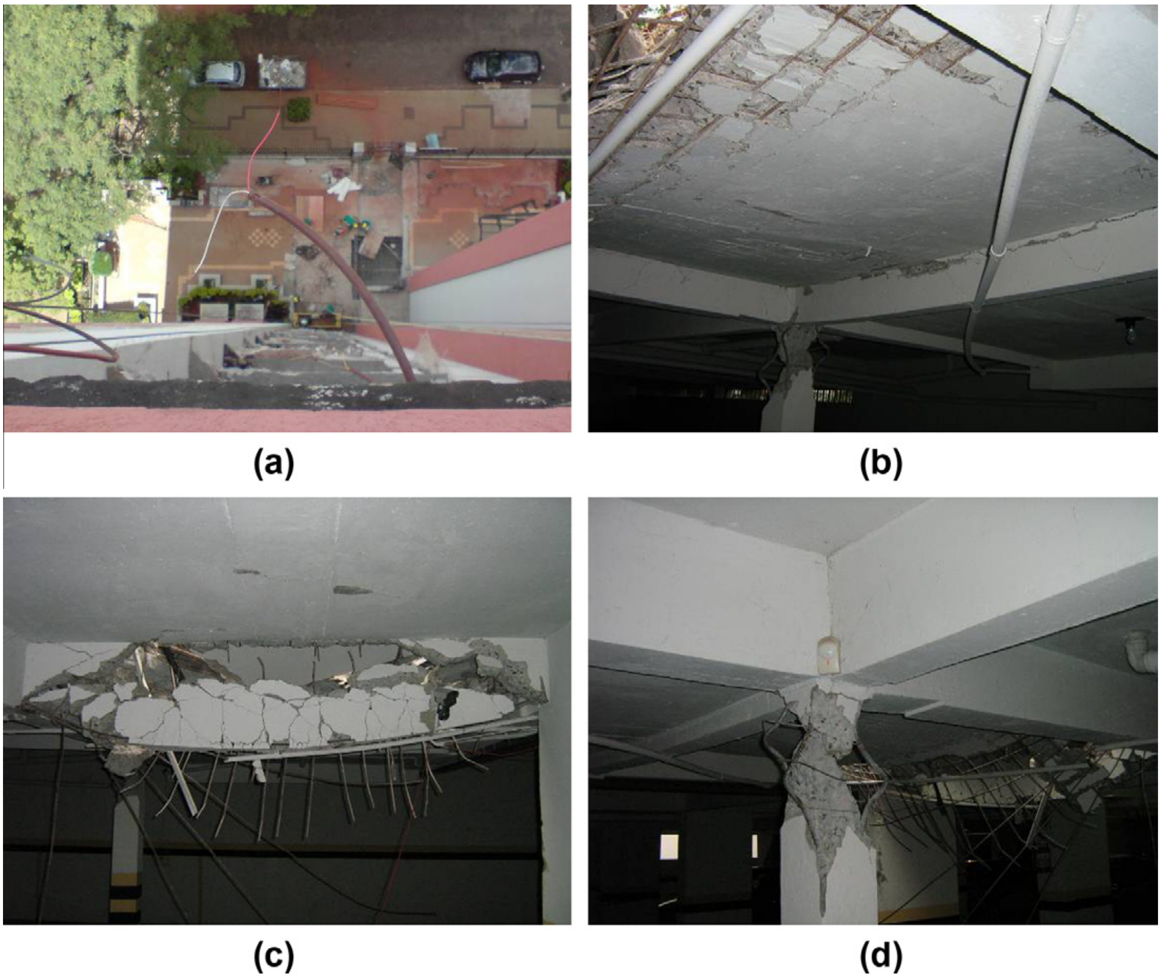

Fig. 3. (a) Lateral deviation of the debris and subsequent (b) failure of a slabs, beams (c) and columns located at the subsoil garage (d).

By another hand, this unexpected failure suggested to the authorities (Fire Department and Civil Defense) the strong necessity of keeping structural and material science engineers duly prepared to help in similar critical situations. In this way, at the same time that is possible to save/preserve lives is also possible to obtain a more rational and safe procedure for intervention in damaged structures. Maringá is now prepared for critical situations involving structural failures and the authors have been contributing as permanent volunteers along with Civil Defense.

4. Main reasons for the progressive failure

It is an incontestable fact that this progressive failure was generated by the free-falling of the cantilevered canopy situated above the 15th balcony. The fragile collapse of the mentioned cantilevered slab generated the sequential failure of 15 balconies, a slab, some beams and columns located at the garage floor (subsoil pavement).

The dead load plus the live load acting on the failed canopy were estimated in about 3.24 kN/m^2, based on the structural and architectural drawings. Taking into account the loading and the clear span of the canopy, a negative reinforcement of about 0.25 cm^2/m would be sufficient for the equilibrium, based on a maximum negative bending moment of 0.535 kN m/m.

As can be realized, the negative reinforcement specified by the structural engineer in the structural drawing, i.e., bars of 3.4 mm of diameter spaced each 15 cm ($A_s \cong 0.54$ cm^2/m as shown in Fig. 2b), is practically twice times the negative reinforcement calculated based on the features presented on the architectural drawing. Also, the amount of negative reinforcement specified by the structural engineer satisfies the minimum reinforced recommend by the Brazilian structural code in that time, i.e., [1]. Based on a cracking moment 1.22 kN m/m for the canopy, the mentioned code would lead to a negative reinforcement of 0.58 cm^2/m, practically the same reinforcement specified by the structural engineer.

Despite the fact that the structural engineer has recommended a negative reinforcement adjusted to the Brazilian structural code in that time, one should realize that it may be considered an inadequate percentage of steel, taking into account the uncertainties usually seen in reinforced concrete structures. Furthermore, the adoption of a very small diameter for the main steel bars was not a smart decision, taking into account the frequent humidity problems faced by canopies and balconies. The Brazilian structural code [1], had no advice about a minimum diameter to be used in this kind of structures, however it looks like consensual that small bars are more susceptible to corrosion problems.

Investigations conducted at the damaged building, more specifically in the reminiscent canopy, revealed the occurrence of significant changes in the dimensions of the structure without the updating of the negative reinforcement. As can be seen in Fig. 2b, the canopy was first specified in the structural and architectural drawings for having a span of 57.5 cm as well as a length of 189 cm. At the site was possible to verify a reminiscent canopy with 94 cm of span and length of 213 cm.

Moreover, the cantilevered slabs used in the canopies were specified to have thickness of 7.0 cm, and regardless the omission in the architectural project about the coating material type, it was expected just a small thickness for the regularization, coating and waterproofing of the slab. For the failed canopy was realized a thickness of about 15.45 cm, being 10.5 cm for the reinforced concrete slab. For the reminiscent canopy was realized a thickness of 18.15 cm, being 11.75 cm for the reinforcement concrete slab.

The exaggerated thickness observed for the canopies is result of frustrated attempts aiming to solve humidity problems. As can be seen in the face of the anchorage of the failed canopy (Fig. 4a), there are at least two layers of waterproofing materials as well as exaggerated thickness of mortar for the passage of a run-off water pipeline (Fig. 4c).

It was still possible to realize the downgrade of the negative reinforcement as well as the lack of uniformity in the spacing between longitudinal bars. For the steel bars located in the reinforced concrete balconies was verified a medium spacing of 10.85 cm when it should be 10 cm. For the steel bars located in the reinforced concrete canopies was verified a medium spacing of 16.67 cm when it should be 15 cm.

Based on a careful analysis, it was possible to raise the hypothesis that the referred run-off water pipeline was object of previous interventions. The hypothesis of leakage was later confirmed by simple tests using phenolphthalein, which revealed the strong presence of humidity in the failed canopy, as show in Fig. 4c. Furthermore, it was realized that the pipeline sit-

Fig. 4. (a) Details of the region of detachment of the canopy, (b) non-forecasted loadings acting on the reminiscent canopy, (c) simple test using phenolphthalein for showing a propitious region for corrosion, and (d) decreasing of the main negative bar in the failed canopy indicating corrosion.

uated above the canopy (Fig. 5b) had an inadequate diameter, enabling in this way the accumulation of water above the canopy, i.e., introducing a non-forecasted extra loading.

In a general way, the agent which has unleashed the corrosion of the main longitudinal bars (negative reinforcement) was the infiltration generated by the rupture of a PVC link connecting the strainer of the canopy to the hydraulic system on the frontage of the building. The region affected in the hydraulic system was above the reinforced concrete canopy and under the waterproofing system as can be seen in Fig. 4c. Taking into account this undesirable infiltration was possible to realize that the main longitudinal steel bars (negative reinforcement) were suffering an advanced process of corrosion, based on the reduction of the initial diameter (Fig. 4d). For the balconies was verified a medium diameter of 5.8 mm for the negative reinforcement when it should be 6.3 mm. For the canopies was verified a medium diameter of 3.0 mm when it should be 3.4 mm.

It is important to highlight that the position of the run-off water pipeline also contributed for accelerating the process of deterioration of the negative steel bars. This tabulation was located above the most demanded tension region of the structural system, i.e., immediately above the negative reinforcement of the canopy. It is also important to observe that was very difficult to detect this infiltration being under the referred canopy. Only the occurrence of continuous rain could saturated the concrete matrix in a way that could be possible identify the mentioned problem, taking in this way a maintain procedure.

Additionally, as can be seen in Fig. 2a, the balconies have been specified in the architectural and structural drawings for having a span of 106.5 cm and a length of 233 cm. In the failed balconies as well as in the reminiscent balconies was possible to verify a span of 150 cm and a length of 254 cm. It should be observed that even with this modification the longitudinal main reinforcement (negative reinforcement) was not updated.

For the balconies, regardless the absence of the specified pot plants, was possible to verify excessive layers of mortar, filler and coating. For the failed balconies was possible to verify a medium thickness of 20.74 cm, being 9.24 cm the medium thickness for the reinforced concrete slab. For the reminiscent balconies was realized a medium thickness of 19.75 cm, being 10.27 cm the medium thickness of the reinforced concrete slabs. As can be seen in Fig 2b, the specified thickness was 10 cm.

Fig. 5a illustrates the exaggerated layers of materials above the balconies slabs while Fig. 5b illustrates the irregularity concerning the spacing between the negative bars as well as its downgrade. As can be seen, the lack of control in the process of execution of this building generated a considerable increasing of loading above the canopies and balconies. It is incredible how this structure was able to carry so many non-forecasted loads for so many years.

Taking into account the free fall of the canopy from the 15th floor, and considering that the balconies were also working in their limit of strength, it is not surprising that this progressive failure had happened, even the strength used for the materials were superior than the strengths specified in the projects. Experimental tests revealed a medium compressive strength of 30 MPa for the concrete, medium yielding strength of 883 MPa for the 3.4 mm diameter steel bars and medium yielding strength 694 MPa for the 6.3 mm diameter bars.

The high strength used for steel and concrete may explain the equilibrium of a so critical structure for so many years. Additionally, the present case reveals that even for free-standing structures, where the redistribution of stress is more limited, it is possible to obtain considerable values of strength, based on the consideration of load factors and reduction strength coefficients when designing using the Ultimate Limit State philosophy.

5. Demolition processes of the reminiscent balconies

After concluding that the reminiscent balconies in the left row of the principal facade should be demolished, the necessary procedures in order to withdraw the free-stand slabs in a safe way have been initiated. Initially, metal guides were

(a) (b)

Fig. 5. (a) Excessive layers of mortar, filler and coating and (b) detail of the downgrade of the negative reinforcement and lack of uniformity for the spacing between bars.

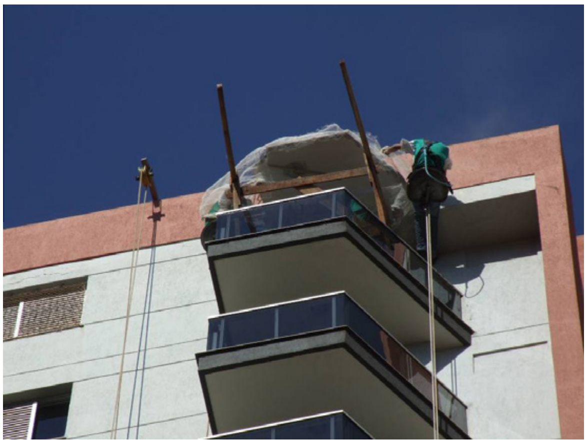

Fig. 6. Metal guides installed in the coverage of the building in order to suspend workers using safety belts connected to steel cables.

installed in the coverage of the building, as shown in Fig. 4a. Using the metal guides the workers could be suspended using safety belts connected to steel cables, as shown in Fig. 6.

Basically, two workers were responsible for the total demolition of the reminiscent canopy and balconies of the damaged building, as shown in Fig. 7a. In order to avoid the possibility of fall of the cantilevered slabs due to the vibrations produced by the mechanical drill, the slabs have been arrested using steel cables, as shown in Fig. 7b. The slabs were demolished in a very gradual manner, in a way that very small fragments could be gathered in the balcony situated below the slab under intervention.

Just above the principal entrance of the building was necessary to apply a diamond saw, taking into account the presence of a very rigid structural configuration provided for a reinforced concrete plant cup, as show in Fig. 8a. Once the canopy and all the balconies have been removed, a suspended working platform has been installed, in order to help the reconstruction of new canopies and balconies, as shown in Fig. 8b.

The failed slab of the garage level (subsoil), where the debris has been accumulated, was reconstructed at the same time while the balconies were being removed. For that, the perimeter of the slab was shored and the affected regions were demolished and reconstructed, including some beams and a column. This procedure of recovering enabled the normal access to the main entrance of the construction, once the damaged elements were necessary for providing the link between the sidewalk and the building.

6. Proposals of reconstruction of the frontage

At first-time, the owners of the apartments decided to leave the frontage of the construction without the balconies, choosing a more simple intervention like a curtain glass covering the damaged area. This first decision was made taking into account that the recovering of the original structural system would lead to an extensive circulation of workers inside the apartments, in order to construct new cantilevered balconies. Also, the TV room of all apartments would need to be closed in order to develop the necessary activities, disturbing in this way the welfare of the families living in the building.

Fig. 7. (a) Free-stand slab being demolished using mechanical drill and (b) steel cables providing additional safety for the slab under demolition.

Fig. 8. (a) Reinforced concrete pot plant extracted using a diamond saw and (b) installation of a suspended working platform after removing all the reminiscent balconies.

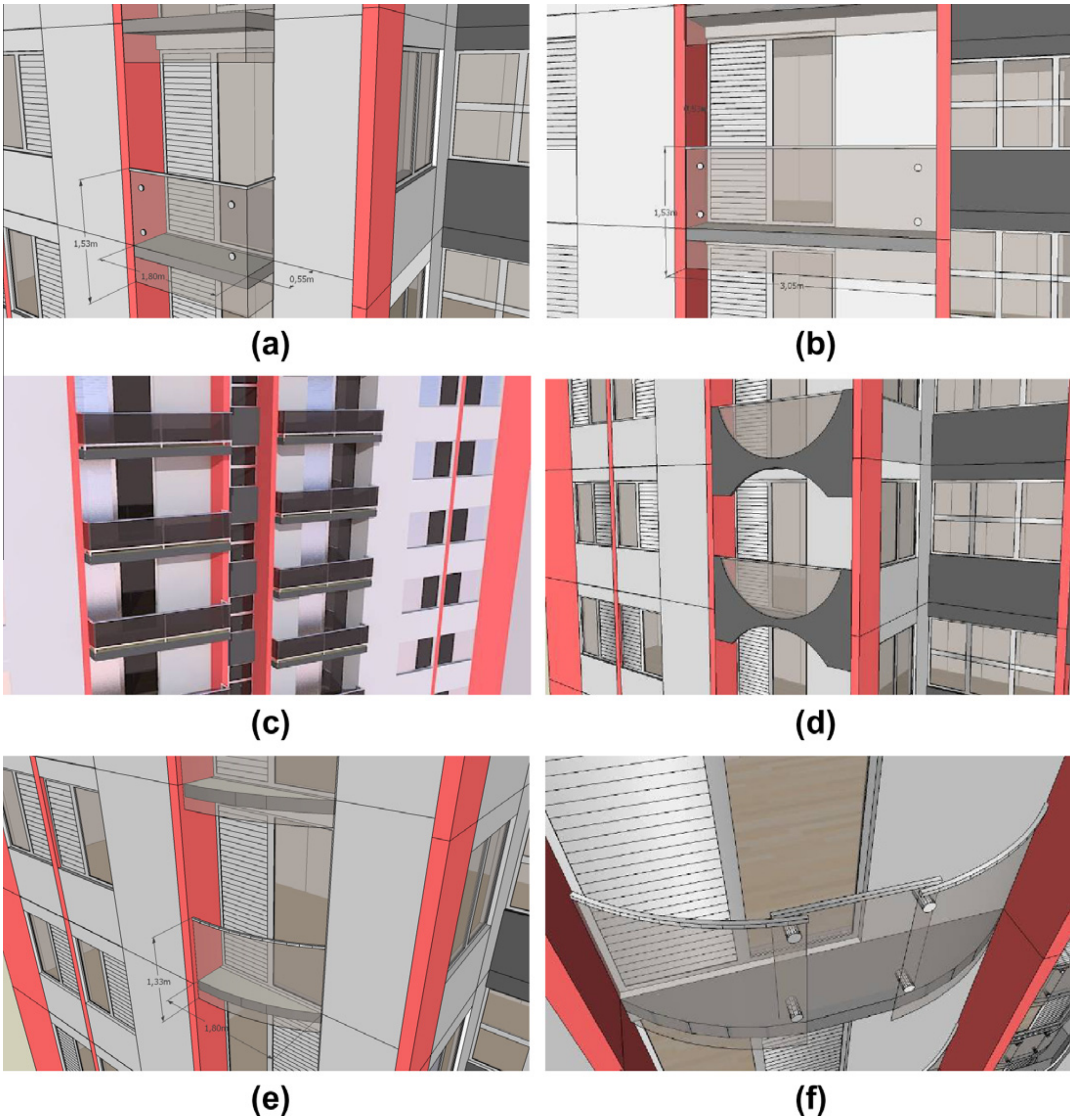

Fig. 9. Proposals for reconstructing the balconies of the building using (a) a reduced rectangular balcony, (b, c, d) extended rectangular balconies with different aesthetics effects, (e) a demilune balcony and (f) a semicircular balcony.

However, after analyzing the strength of some structural elements and some architectural possibilities using mixed structural systems in the region of the damaged balconies, it was possible to develop some proposals for the reconstruction of the failed/demolished balconies, as shown in Fig. 9.

After many meetings, the owners of the apartments have decided for the reconstruction of the balconies, following the proposal presented in Fig. 9c. This decision was mainly made based on the minimum disturbance that the reconstruction will lead, as well as the visible gain of area that this reconstruction will produce. Fig. 10a shows the original dimension of the failed/demolished balconies while Fig. 10b shows the proposal of reconstruction.

One should observe that besides the enlargement of the new balconies, the chosen proposal is also an excellent solution in terms of maintenance and conducts to a more economic solution when compared to the original design. It was possible through the use of a mixed structural system containing metal profiles and lattice girders beams.

7. The process of reconstruction of the balconies

After deciding for the proposal presented in Fig. 9c, the process of structural design of the new balcony was initiated. The first steps were mainly based on the following items: design of the metal profiles to be used as support for lattice girders, design of the bolts for holding the metal profiles on the reinforced concrete columns and design of the main reinforcement of the lattice girders beams.

The structural design was made taking into account a live loading of 3.0 kN/m^2 as well as the recommendation of the following codes [2–6]. The structural design conducted to the adoption of 100 mm $\times$ 100 mm $\times$ 6.34 mm steel angles made of steel A36 (characteristic yield strength of 250 MPa), which were bolted in the columns P25 and P26 (see Fig. 9), using four galvanized bolts with diameter of 16 mm.

The new balconies were constructed using three lattice girders beams supported on two laterals steel angles, which acted as permanent formwork for the new slabs. The new slabs have a thickness of 11 cm, being 4 cm due to a concrete top with characteristic compression strength of 25 MPa. Also was used a thickness of 7 cm due to a filling of expanded polystyrene (EPS), in order to reduce the dead loading of the new balconies.

The reinforcement of the two lattice girders beams has been constituted by 1 $\varnothing$ 6.3 mm for the negative reinforcement and by 2 $\varnothing$ 4.2 mm plus 2 $\varnothing$ 5.0 mm for the positive reinforcement. The lattice girder beam situated in the end of the free-standing balcony, was reinforced by 1 $\varnothing$ 6.3 mm for the negative reinforcement and 2 $\varnothing$ 4.2 plus 2 $\varnothing$ 6.3 mm for the positive reinforcement, taking into account the construction of a groundsill in the balconies.

After finishing the design of the structural elements to be used in the new balconies, the process of scarification of the regions of covering of the columns was conducted, in order to install the lateral steel angles, as shown in Fig. 11a. After this activity, the steel angles were bolted, as show in Fig. 11b, and the lattice girders beams were accommodate in order to form the formwork for the new slabs.

Fig. 11c shows a superior view of the structure while Fig. 11d shows an inferior view of the new balconies, where is possible to observe the steel angles, the line of bolts, the lattice girders beams and the expanded polystyrene (EPS) elements before casting the slabs. As can be seen in Fig. 11c, before the casting of the slabs with concrete, an orthogonal reinforcement mesh constituted by 8 mm bars spaced by 18 cm was positioned just above the expanded polystyrene elements.

In the border regions, i.e., in the contact regions between the reinforcement grid and the columns or adjacent slabs (TV room), there has been a concern about the anchoring of the steel bars of the mesh. That was possible by means of a drilling machine which promoted perforations of about 20 cm in the adjacent reinforced concrete elements. In this way, the reinforcement mesh was fit in the perforations and epoxy resin was provided in order to consolidate the anchorage.

Finally, the casting of the slabs using in situ concrete has taken place and a special attention was given for the curing period. One should observe that while the process of demolishing was from top to bottom, the process of reconstruction was from bottom to top. In that way, each new balcony could contribute for the shoring of the successive balconies to be constructed. The whole process of demolition and reconstruction of the balconies has taken about six months and no major

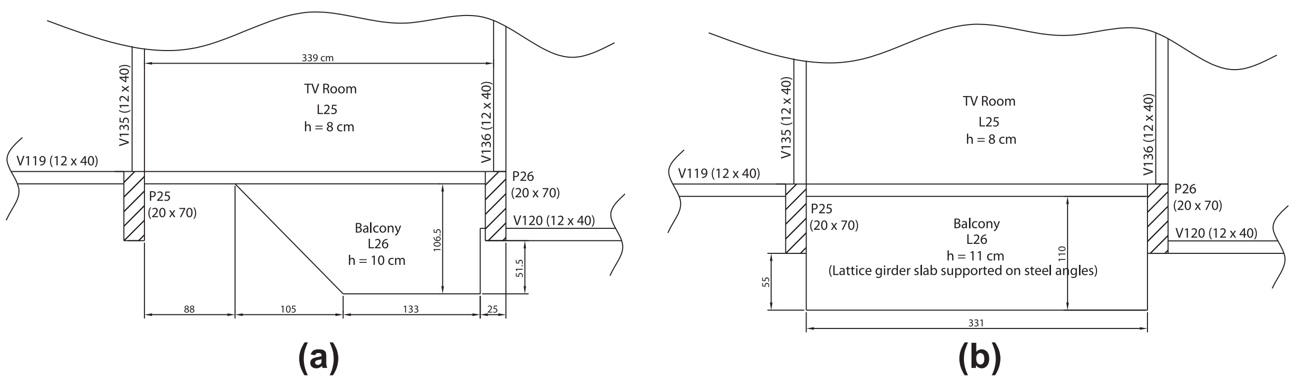

Fig. 10. (a) Original dimensions of the failed/demolished balconies and (b) dimensions of the approved proposal for the reconstruction of the balconies.

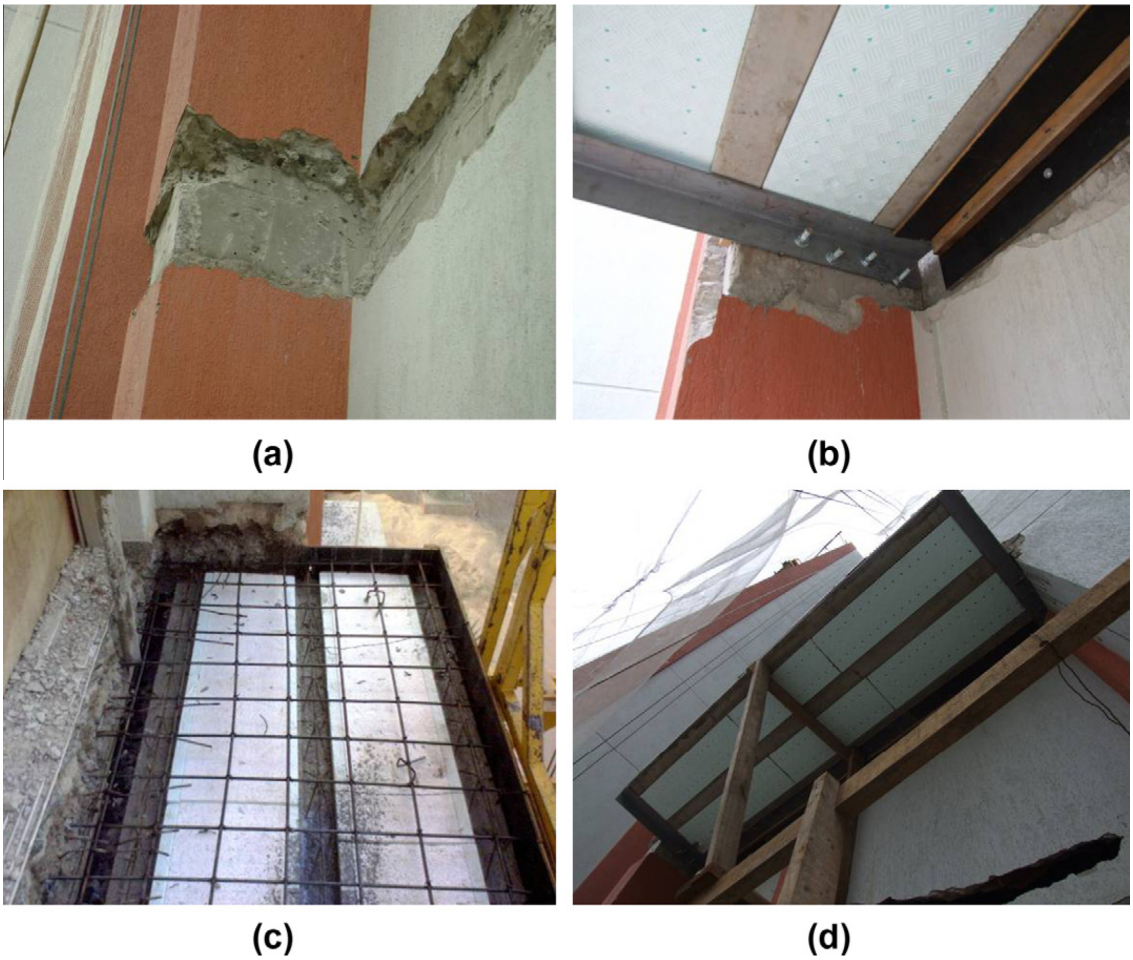

Fig. 11. (a) Scarification of the layers of mortar covering the columns, (b) details of the steel angles bolted with galvanized bolts, (c) superior view and (d) inferior view of the new balconies.

problems were identified during the activities. Fig. 12 shows the frontage of the damaged building with the reconstructed balconies and canopies.

8. Conclusions

A structural accident never takes place just for one reason, but for adding up multiple reasons in the most unfavorable conditions. In this way, relatively simple problems can unleash major problems which can lead to progressive failures. The structural failure described in the present paper is an uncontestable proof of this situation and could serve as alert to the engineers and to the society.

Basically, the progressive failure of the balconies was result of an inefficient hydraulic system (run-off water pipeline). With the rupture of the mentioned tabulation, the infiltration of water has occurred in the interior of the canopy, corroding in this way the main longitudinal reinforcement (negative reinforcement). To this effect were added the following complementary problems: downgrade of the main reinforcement in the canopies and balconies, geometric modification of canopies and balconies without updating the main reinforcement, excessive dead loads (mortar, filler and coating) non-forecasted above the canopies and balconies and heavy rains/winds occurred in the day of the collapse. Taking into account the insecurity generated for the debris, as well as by the features of the building, a recommendation for demolishing the reminiscent canopy and balconies in the façade of the construction was made.

At a first time, the owners of the apartments decided not to reconstruct the balconies, once they were dominated by the panic generated by the collapse. However, considering the high level of security demonstrated during the process of demolition of the reminiscent balconies, and taking into account the necessity of prevent the devaluation of the apartments, some proposals of interventions could be made, always avoiding the circulation of workers in the interior of the apartments.

In this way, was decided to reconstruct the failed/demolished balconies based on a mixed slab system, constituted by lattice girders beams supported on bolted steel angles. This solution introduced an enlargement of the area of the balconies and was also considered very practical and less expansive than the original reinforced concrete slabs. The present intervention has strongly contributed for recovering the property as well as the psychological comfort of the residents of the damaged building.

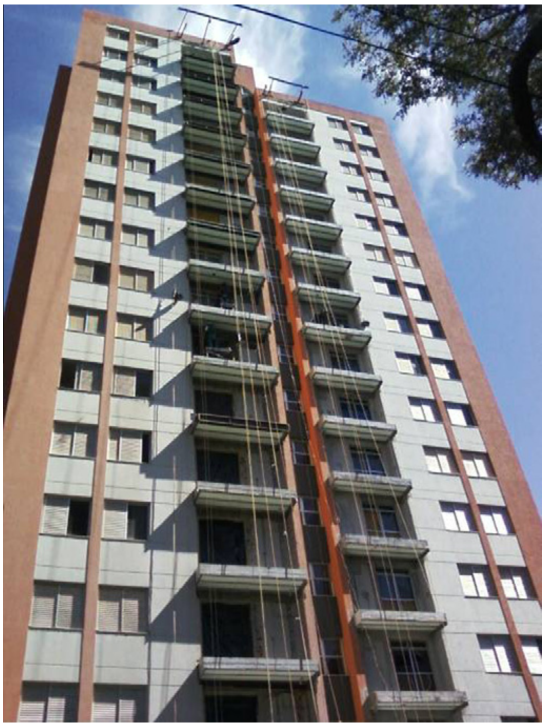

Fig. 12. Frontage of the damaged building after the reconstruction of the failed/demolished balconies.

It must be pointed out the great difficulties that exist in a procedure like that one conducted in here. The challenges are mainly due by the fact that all decisions must be in concordance with the collective opinion of a majority that is into a state of shock, originated by the fragile failure of part of their properties. Usually, this kind of experience of being witness of a failure produces insecurity, mistrust and stress and a strong effort in necessary in order to recovery the confidence of these people regarding engineering.

It is recommended that new updates of the Brazilian structural code [5], include in its future revisions some mechanisms in order to avoid progressive failure of free-standing structures. The imposition of a minimum amount of positive reinforcement in canopy and balcony structures could generate, for example, additional strength in critical situations. Also, the imposition of the use of constructive reinforcements could avoid the frequent downgrade of the main reinforcement (negative bars). Besides that, the imposition of a minimum diameter for the main reinforcement bars is also another simple recommendation which could avoid the fast propagation of corrosion process, enhancing in this way the durability and ductility features.

Despite the bad practice that has conduct to the progressive failure described in here, the present paper has presented a very effective and economical engineering solution in order to demolish and recover the balconies of the damage building described before. The described procedure has recovered the confidence regarding engineering and also restored the main characteristics desirable in a good design: security, economy and durability.

Finally, excessive crack widths, water infiltrations (stains), exaggerated deformations in the structural elements, doors and windows with difficulty for closing/opening and detachment of flagstones are always a clue for important problems affecting the structural system. These problems could be easily detected with an inspection/maintenance program, conducted at least one time each two years. The historical failures occurred in Brazil has signalized the strong necessity of impose several laws concerning the inspection of built constructions, in order to avoid or at least minimize new failures originated from simple problems.

References

[1] ASSOCIAÇÃO BRASILEIRA DE NORMAS TÉCNICAS. "NB1 – Projeto e Execução de Obras em Concreto Armado", Rio de Janeiro; 1978.
[2] ASSOCIAÇÃO BRASILEIRA DE NORMAS TÉCNICAS. "NBR12655: Concreto – Preparo, Controle e Recebimento – Procedimento", Rio de Janeiro; 2006.ASSOCIAÇÃO BRASILEIRA DE NORMAS TÉCNICAS. "NBR14762: Dimensionamento de Estruturas de Aço Constituídas por Perfis Formados a Frio - Procedimento ", Rio de Janeiro; 2001.
[3] ASSOCIAÇÃO BRASILEIRA DE NORMAS TÉCNICAS. "NBR14859-1: Lajé Pré-Fabricada – Requisitos – Parte 1: Lajes Unidirecionais", Rio de Janeiro; 2002.
[4] ASSOCIAÇÃO BRASILEIRA DE NORMAS TÉCNICAS. "NBR14931: Execução de Estrutura de Concreto - Procedimento", Rio de Janeiro; 2004.
[5] ASSOCIAÇÃO BRASILEIRA DE NORMAS TÉCNICAS. "NBR6118: Projeto de Estruturas de Concreto - Procedimento", Rio de Janeiro; 2003.
[6] ASSOCIAÇÃO BRASILEIRA DE NORMAS TÉCNICAS. "NBR6118: Projeto de Estruturas de Concreto - Procedimento", Rio de Janeiro; 2003.ASSOCIAÇÃO BRASILEIRA DE NORMAS TÉCNICAS. "NBR8681 – Ações e Segurança nas Estruturas - Procedimento", Rio de Janeiro; 2003.
[7] Buchhardt F, Magiera G, Matthees W. Structural investigation of the Berlin congress hall collapse. Concr Int 1984;6(3):63–8.
[8] Calderón PA, Adam JM, Payá-Zaforteza I. Failure Analysis and Remedial Measures Applied to a RC Water Tank. Eng Failure Anal 2009;16(5):1674–85.

[9] Carino NJ, Leyendecker EV, Fattal SG. Review of the skyline plaza collapse. Concr Int 1983;5(7):35–42.
[10] Cunha AJP, Lima NA, Souza VCM. Acidentes Estruturais na Construção Civil – Volume I. Editora Pini, São Paulo; 1996.
[11] Cunha JC. Palace II – A Implosão Velada da Engenharia. Editora Autêntica, Belo Horizonte; 1998.
[12] Grigoli AS. Recuperação, Renível e Reprumo de Edifício de Oito Pavimentos. In: 41° Congresso Brasileiro do Concreto, Salvador; 1999.
[13] Jannadia MO, Tahirb BM. A concrete pier: case history of failure and repair. Constr Build Mater 2000; 14:7–16.
[14] Kaltakci MY, Arslan MH, Korkmaz HH, Ozturk M. An investigation on failed or damaged reinforced concrete structures under their own-weight in Turkey. Eng Fail Anal 2007;14:962–9.
[15] Lew HS, Carino NJ, Fattal SG. Cause of the condominium collapse in cocoa beach, Florida. Concr Int 1982;4(8):64–73.
[16] Libby JR. Three chloride-related failures in concrete structures. Concr Int 1987;9(6):29–31.
[17] Minor JE. Lessons learned from failures of the building envelope in windstorms. J Archit Eng 2005;11(1):10–3.
[18] Nielsen TB. Collapse of danish prestressed tanks. Concr Int 1999;21(4):55–6.
[19] Peraza DB, Cuoco DA. The First Steps After a Failure. Forensic Structural Engineering Handbook, Edited by Robert Ratay, McGrawHill, New York, 2000.
[20] Russel HG, Rowe TJ. Collapse of ramp C. Concr Int 1985;7(12):32–7.
[21] Souza RA. Ruínas Recentes de Edifícios no Brasil. In: II Encontro Tecnológico da Engenharia Civil e Arquitetura de Maringá; 2001, Maringá. II Encontro Tecnológico da Engenharia Civil e Arquitetura de Maringá; 2001.
[22] Trebuña F, Simcák F, Bocko J. Failure analysis of storage tank. Eng Fail Anal 2009;16:26–38.
[23] Vecchio FJ, Collins MP. Investigating the collapse of a warehouse structure. Concr Int 1990;12(3):72–8.