Analysis of the damage caused by a fire in the formwork of the bottom slab of a liquefied natural gas storage tank

Análisis de los daños provocados por un incendio del encofrado de la losa de fondo de un depósito de gas natural licuado

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ABSTRACT

This paper presents the results of the study carried out on the structure of the bottom slab of a liquefied natural gas tank (circular slab 89 m in diameter and 1.20 m thick, raised 2 m above ground level, and supported by circular piles 1.20 m in diameter), in which a fire in the formwork affected both the slab and the piles. Concrete in these elements, apart from having a very high moisture content at the time of the fire, was also a very compact material - it contained 50% blast furnace slag, GGBS - which, together with the compressions generated on the surface of the slab and the supports by the effect of the fire, caused significant damage in some areas which, in any case, could be repaired using conventional techniques.

RESUMEN

En la presente comunicación se presentan los resultados del estudio realizado en la estructura de la losa de fondo de un depósito de gas natural licuado (losa circular de 89 m de diámetro y 1.20 m de espesor, elevada 2 m respecto del nivel de suelo, y apoyada en pilas-pilote de sección circular de 1.20 m de diámetro), en la que un fuego en el encofrado afectó tanto a esta como al tramo descubierto de las pilas-pilote. El hormigón de dichos elementos, aparte de conservar contenidos de humedad muy elevados cuando se produjo el incendio, era además un material muy compacto -contenía un 50 % de escorias de alto horno, GGBS-, lo que unido a las compresiones generadas en la superficie de la losa y de los soportes por efecto del fuego produjo daños significativos en algunas zonas que, en todo caso, pudieron ser reparados con técnicas convencionales.

KEYWORDS: fire, forensic engineering, liquefied gas storage tank.

PALABRAS CLAVE: fuego, incendio, patología, depósito gas licuado.

1. Brief LNG tank description

The tank affected by the fire was a full-containment tank, with an individual capacity of 180,000 m^3. This means that under normal conditions the liquefied gas is contained within an inner steel tank with an open roof. Full containment is provided by an outer concrete wall, base slab and domed roof (figure 1).

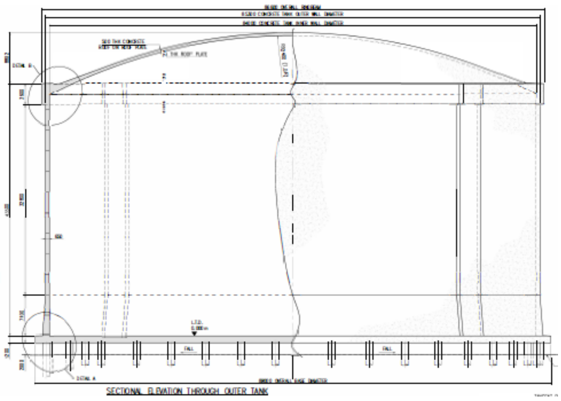

Figure 1. Sketch of the tank

The base slab is elevated 2 m and its thickness is 1.20 m, resting on 436 concrete piles. The piles are between 56 and 61 m long and have a diameter of 1.20 m.

Regarding the materials, reinforcement is grade 500B, concrete grade for the piles is M 40 and for the base slab and the columns is M 50. Other characteristics of the concrete are its low water/cement ratio, the use of basaltic aggregate and the fact that it uses 50% GGBS (Ground Granulated Blast-furnace Slag).

The concrete cover of the reinforcement is 70 mm for the columns and 50 mm for the base slab.

2. The fire

Judging from the emergence of the smoke from under the base slab, the fire initially localised in an area near the perimeter, probably arson. It intensified and the affected area grew, though the action of the firemen. The fired essentially ended after about 7 hours.

The fire consumed primarily the timber formwork under the base slab (figure 2).

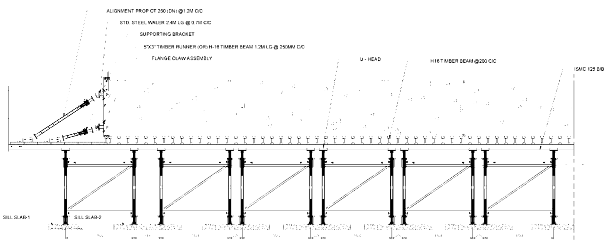
Figure 2. Formwork arrangement

It is worth mentioning, because of its impact on propagation, that the wind blew consistently from the northwest throughout the event, with wind speeds in the order of 10-20 km/h.

From the cameras around the tank the fire seems to have started in an area where the last concrete had been poured, which had already been de-shuttered though the support towers had not been removed yet. This would require the fire to start on combustible materials left over under the slab.

Then, the fire propagated essentially upwind. In the end, it extended over the complete area under the slab.

The amount of damage caused in the concrete of both columns and slab, which is described in the following section, can be approximately correlated with the temperatures that developed at each location, which are the result of two factors:

- The inventory of locally available combustible materials. The timber of the formwork seems to have been the main combustible material. Thus, the areas already de-shuttered generally suffered much less damage.
- The proximity to the perimeter, which allows for more efficient cooling than the central areas under the slab. Hence, perimetral areas suffered less than the inner ones.

3. Visible damages caused by the fire and preliminary assessment

3.1. Loss of reinforcement cover

Almost half of the surface of the slab was found to exhibit varying degree of spalling, leaving some bars without cover (figures 3 to 7); at few locations, the bars also suffered permanent deformations. Locally, the loss of concrete also exposed the lower part of some of the transverse reinforcement. The upper region of some of the columns also underwent similar effects.

The immediate cause of this damage was a combination of explosive and progressive surface spalling processes during the heating phase, induced by temperatures of around 200-300 ºC ([1], [2]). The combination of the two processes is evinced in figure 6 by the two different colours left on the concrete surface following spalling.

The first process is expected to be caused by the explosive evaporation of interstitial water, particularly in a relatively young concrete. On the other hand, progressive spalling is induced by the splitting and cracking of the concrete cover and the loss of bonding during the cooling phase ([1], [3]).

Locally, depths of up to 15 cm of concrete had fallen off. In these cases, permanent deformations of the bars were observed (figure 4). It is likely that those locations were exposed to higher temperatures or for a longer time. These effects, in combination with the compression forces generated by the thermal gradient across the section and the thermal expansion of the bars, would cause the damage observed. This is commonly described as falling off or sloughing off. In those areas, most of the first layer of the reinforcement had fallen (figure 3).

In most locations where spalling occurred, there were no significant soot stains. This observation helps to limit the effects: when the explosive spalling occurs, the loss of the outer concrete exposes deeper layers to the fire, which eventually lose their properties and fall off. The heating of the inner layers, once the outer layer is shed, is likely to be low, because it occurs during the final stages of the fire and the cooling phase, when the heat flow applied is significantly lower.

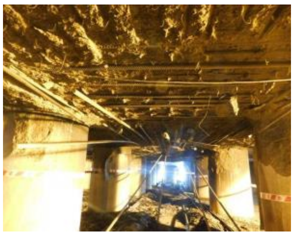

Figure 3. Slab. Concrete cover spalled, 3 rebar layers exposed

Figure 4. Slab. Concrete cover spalled, 1 rebar layer exposed

Figure 5. Columns. Concrete cover spalled, long. rebars exposed

Figure 6. Progressive spalling. Notice the different concrete colours

Figure 7. Splitting. Loss of bond between concrete and rebar

3.2. Surface spalling

Many zones of the slab and many of the columns exhibit local spalling in the concrete cover, although the thickness of concrete lost is generally not sufficient to expose the reinforcement (figure 8).

The loss of the concrete cover is due to the explosive evaporation of interstitial water, as already mentioned earlier. The spalled area might present some soot stains, as this spalling occurs in the initial stages of the fire. The site observations indicate that this was not significant, thus the fire in these areas was not very intense.

3.3. Cracking at the bottom of the slab

Around some areas that underwent spalling, the concrete was found to exhibit local "alligator" cracks on the soot-stained surface (figure 9). These cracks arise because of the differential cooling between the surface and the inner regions of the concrete. Nevertheless, in most cases these cracks appeared to be a surface feature that did not affect the inner regions of the concrete.

Figure 8. Columns. Spalling shallower than 25 mm

Figure 9. No spalling but cracks visible on the surface

In some columns, around areas that suffered spalling, the surface of the concrete presented a few cracks, indicating a region where the cover was affected but had not fallen off.

As could be expected, given the thickness of the base slab, no cracks were observed on its upper surface.

4. Experimental campaign

4.1. Research approach

As a consequence of the fire, it became necessary to assess the resulting consequences on the strength and durability of the concrete slab and the columns, as well as the remedial measures that may be required.

The behaviour of the concrete during the fire was conditioned by its relatively young age, less than two months in most areas, and by the high humidity due to the large dimensions of the elements affected (the slab was 1.20 m thick, and the columns had a diameter of 1.20 m). It was also influenced by its low water/cement ratio and the 50% GGBS content.

Because of these characteristics, an experimental study was designed pursuing the following objectives:

- Comparison of the concrete properties at different depths.
- Characterization of the unaffected concrete after exposure to high temperatures of 100, 300, 500, and 700 °C.
- Evaluation of the depth to which the concrete was affected by the fire.
- Investigation of the susceptibility to spalling under fire, to assist in the interpretation of the fire effects.
- Estimation of the maximum temperature reached by the remaining concrete during the fire.

For these purposes, the experimental analysis includes the following tests:

- Inspection of the cores and samples
- Compressive strength and elastic modulus after heating on samples with drying treatment (to prevent spalling)
- Mechanical tests on samples without drying treatment
- Ultrasonic pulse velocity (UPV)
- Determination of pH by phenolphthalein
- Tests on reinforcing bars

4.2. Main results

4.2.1. Results of tests on samples with drying treatment

Inspection of the cores and samples

No significant anomalies were visible, excluding air voids evinced in cores extracted from the columns, which are due to the execution process.

In some cases, the cores were apparently broken because of the presence of cracks, both in the columns (parallel to the external surface: figure 10) or in the slab (vertical), which apparently occurred due to hygrothermical behavior during execution.

Regarding possible colour variations, none of the cores received showed significant changes along their length. Nevertheless, after heating in the furnace, the core showed a slight change of colour, from grey to pink, more significant in those heated at 500 °C (figure 11).

Figure 10. View of some cores extracted from columns

Figure 11. Temperature effects on colour: 100 °C (left), 500 °C (right)

Compressive strength and elastic modulus after heating

As an example, figure 12 illustrates the results obtained in compressive strength and elastic modulus determinations of samples from the columns.

As can be observed, there is significant dispersion for each temperature. There is also a significant dispersion in concrete resistance between the outer sample (no. 1) and the closest inner sample (no. 2) for 100 °C treatment. It is to be noted that the strength of the columns was, in every test, under the characteristic value (50 MPa). This did not occur in cores obtained from the slab. This difference will be discussed later.

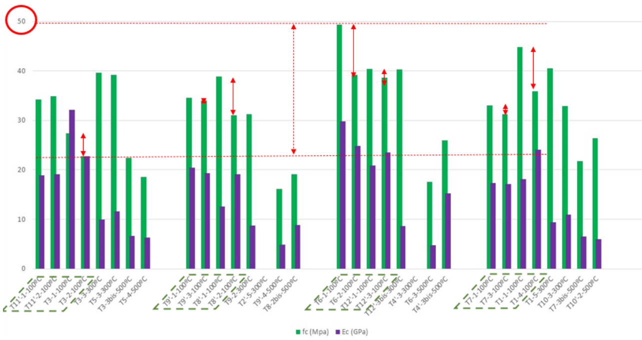

Figure 12. Columns. Mechanical properties of concrete. Each group corresponds to different level of damage

It could also be observed that, as expected, the elastic modulus reduces faster (to 40% at 300 °C and 30% at 500 °C, compared to 100 °C) than the strength, which only drops after treatment at 500 °C (to about 40%). Figures 13 and 14.

In any case, the behaviour after furnace treatment differs slightly from expectations, as deterioration tends to occur at temperatures above 400 °C, retaining its structural properties up to 600 °C ([1], [2]). The 40 % drop of strength at 500 °C is higher than expected.

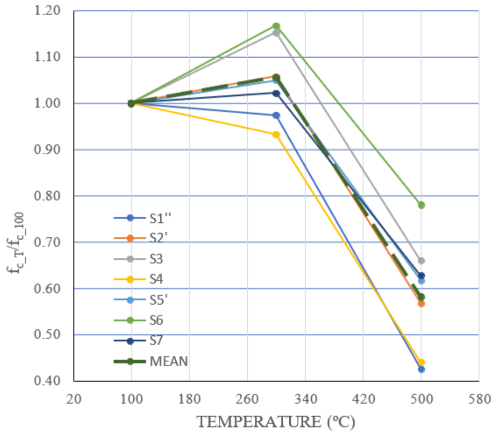

Figure 13. Slab. Concrete strength. Furnace treatment

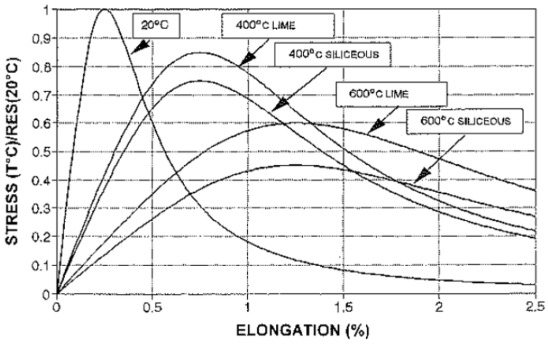

Figure 14. Usual concrete behaviour with temperature [1]

In relation to the above, it should be noted that high temperature behaviour has usually been studied for concrete made with Portland cement and siliceous or lime aggregates. There are fewer references for concrete using basaltic aggregate and 50% GGBS. Recent investigations ([4], [5]) suggest that GGBS seems to improve the resistance to fire conditions: when the exposure to fire temperature increased from 200 to 400 °C, the compressive strength increased for concrete with OPC/GBBS proportions of 70/30 and 50/50 -as concrete from the tank-, but with 30/70 the opposite was observed. Any case, concrete suffer a strong deterioration after 400 °C, as observed in the test (40% strength after treatment at 500 °C).

Ultrasonic pulse velocity (UPV)

Figure 15 illustrates the results obtained in the test in samples from the columns and the slab. The dispersion in the results of the columns ($\sigma/\mu \approx 0.1$) is the usual in this kind of tests.

As expected, the UPV is notably reduced at 300 ºC (around 30 %) and 500 ºC (around 65 %), consistent with the reduction in elastic modulus shown earlier.

Regarding the external cores, the reduction of the UPV at the exposed faces is quite moderate in practically all cases, particularly when results are compared with those at 300ºC. There is no clear relation between the reduction and the damage level.

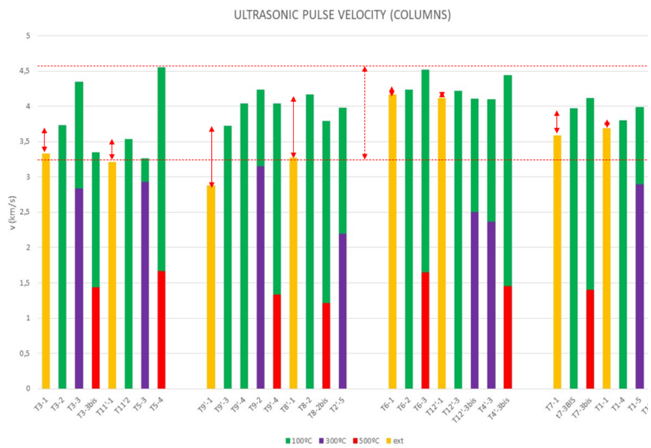

Figure 15. Columns. UPV. Each group corresponds to different level of damage

4.2.2. *Results of tests on samples without drying treatment*

Mechanical tests

After furnace treatment at 300 ºC, no spalling was observed in samples, possibly because of their small size (compared with that normally used to check this effect) and the rate of heating (high compared to RILEM specifications, but very low compared with that in real cellulosic fires).

Determination of pH by phenolphthalein

In external pieces that could not be used for other tests, the phenolphthalein method showed that the carbonation depth did not reach 10 mm in any case. The same can be seen in the reference samples after breaking them in compression.

It must be noted that, using 50% of GBBS, the natural carbonation is somewhat higher than with 100% OPC, and part of the thickness measured could arise from this effect.

In any case, these tests indicated that the durability of the concrete in both the columns and the slab had not been affected by the fire, and it was considered that the application of a surface treatment was not necessary.

4.2.3. *Tests on reinforcing bars*

The bars barely reduced their properties. The yield stress $R_{0,2}$ = 500 MPa and the tensile strength R_m = 1.1× $R_{0.2}$ are both reached in all cases.

Even the ductility is essentially maintained: it reduced slightly in 2 out of 6 samples, with a ratio $R_m/R_{0.2}$ of 1.17 and 1.18, while the ratio remained close to 1.25 in the rest of the samples. The reductions may be due to a slight decarburization of the steel, but there is no information on the carbon content. In any case, as the ratio should be comprised between 1.15 and 1.35, the matter is really immaterial.

5. Numerical simulations

The numerical simulations included thermal calculations to determine the concrete thicknesses affected by the various temperature levels and mechanical calculations to establish the structural implications.

Regarding these calculations, the analysis of the consequences of an external fire requires considering two different aspects: thermal effects and mechanical response.

The thermal and mechanical problems are only weakly coupled, in the sense that, although the temperatures have a strong influence on the mechanical problem, the mechanical variables have negligible effects on the thermal problem. This allows dealing with the coupling in a sequential manner: the thermal analyses are conducted first, conducting a transient thermal analysis and then their results are fed to the mechanical analyses where the structural problem is addressed; the new demands arise from the thermally induced stresses.

Both types of calculations have been performed using the general-purpose finite element code Abaqus [9] in which temperature dependent properties were also used for the various materials involved.

The main results of the numerical simulations are presented below:

5.1. Surface temperatures developed

The first step in the calculations is to determine the temperatures reached by the outer concrete surfaces during the fire.

To estimate the magnitude of the heat released, it was assumed that the primary material available for the combustion was the timber from the formwork. Hence, this requires estimating the amount of timber and its calorific value.

Taking into account the thermal properties of the timber (obtained from [5]) and the shuttering configuration employed (about 24 kg of timber /m^2), the potential heat release that results was about 340 MJ/m^2.

Based on the existing data, the information provided by the security cameras, and the experience from other fires of similar type, at any given location the consumption of the available timber took about 1 hr, during which the heat release rate would have been around 100 kW/m^2. The fire, of course, would progress to the other areas under the base slab and, hence, the fire would exist at one or other location under the tank for a much longer period. But, at each location where the fire occurred, the combustion released 100 kW/m^2 in the course of 1 hr. This assumption was used for estimating the temperatures developed in the concrete.

With a rate of heat release of 100 kW/m^2, the radiation exchange with the concrete surface of the concrete reaches a steady-state situation in a matter of a few minutes. The steady-state surface temperature is about 900 ºC. In this situation, a reasonable approach is to impose a constant temperature at the surface of the concrete for evaluating the time evolution of temperatures within the concrete.

5.2. Concrete behaviour

The thermal and mechanical properties were mainly based on Eurocode 2 Part 1.2 "Structural Fire Design" [6]. The properties used were mean properties for the C40/50 designation.

It is worth mentioning that the higher the specific heat, the slower the temperature progression. Besides, the moisture content is not a function only of the w/c ratio. It also depends, for example, on the relative humidity. In fact, there is a peak in the specific heat at around 115 ºC, caused by the latent heat of water evaporation. Since the concrete is relatively young, it is expected that to have a value of the moisture content of 5% and, consequently, the highest curve provided by Eurocode 2 was used, which corresponds to a moisture content of 3%. The rest of the thermal and mechanical properties do not depend directly on the moisture content.

5.3. Results

5.3.1. Columns

The maximum depth with temperatures higher than 200 ºC obtained was of 70 mm at 60 minutes, with a slight increase during cooling according to Figure 16.

In the mechanical model, it is assumed that the sections remain plane, allowing axial strains. The distribution of the stresses after 60 min fire are presented in Figure 17. It is worth clarifying that no softening was included in the tensile behaviour of the concrete, in order to assess the cracking pattern. According to the calculations, horizontal cracks are expected in the inner part of the column after 5 minutes.

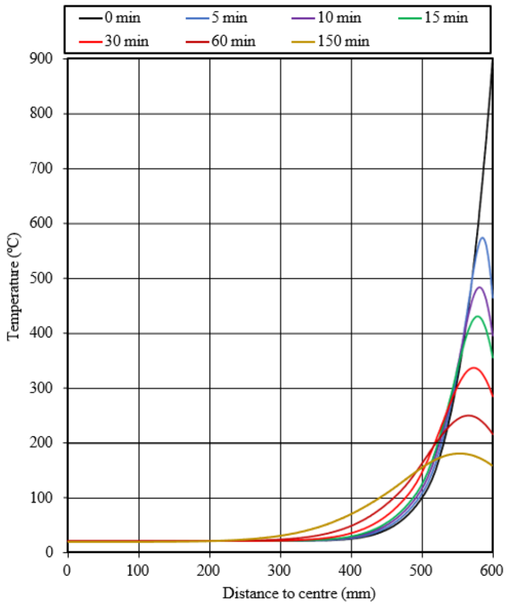

Figure 16. Columns. Temperature distribution after the fire event

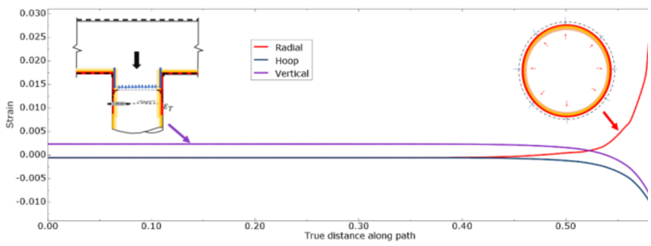

Figure 17. Columns. Stress distributions after 60 min fire

5.3.2. Slab

The maximum depth with temperatures higher than 200 ºC is 70 mm after 60 minutes, with a slight increase during cooling, very similar to what happened with columns. For this scenario, it is reasonable to assume that the slab region not thermally affected, together with the restraining action of the piles, prevents the global elongation of the slab. The distribution of the stresses at different times are presented in figure 18. According to the calculations, no cracks are expected beyond 100 mm near the bottom surface.

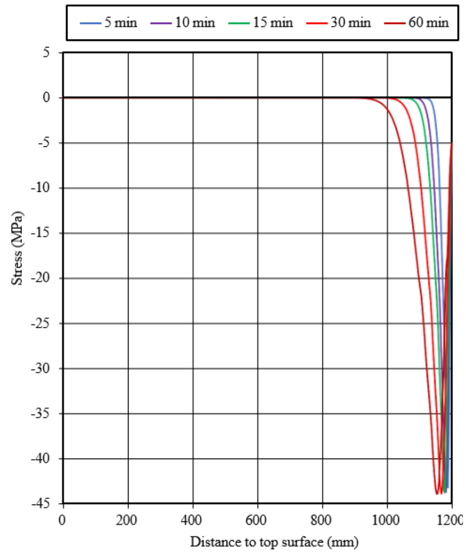

Figure 18. Slab. Stress distributions during the fire

6. Discussion

As a result of the studied carried out, the following can be stated:

- The most significant effect produced by the fire on the structure was the spalling of concrete caused by temperatures triggering the explosive evaporation of interstitial water. This effect was probably enhanced by the young age of the concrete, the dimensions of the elements affected, the low water/cement ratio, the use of 50 % slag, and the basaltic aggregate.
- The tests indicated that the remaining concrete retained its initial mechanical properties: the tests at 100 ºC performed on the outer samples and their neighbours hardly show any degradation. Serious deterioration would have arisen for temperatures of 500 ºC, but those temperatures were not reached by the remaining concrete, as indicated both by the tests and the colour variations. They might have been reached by the concrete

of the outer cover, but this fell off, as shown by the tests and by the light soot stains left in the exposed surfaces, previously protected by the outer layers that fell off.

- The only minor exception is the central part of the columns. Heating the outer layers of the columns entails their longitudinal compression (by thermal expansion) and associated tensions in the central region. This is likely to have caused some tensile cracking along horizontal planes, which would explain the difficulties experienced in getting continuous cores from the columns and the lower mechanical properties encountered.

  When the outer concrete is repaired, some horizontal cracks are likely to remain in the central region of the columns. Nevertheless, this is unlikely to affect the structural performance and is not deemed to have structural consequences, since the cracks will be compressed.

- The same mechanism does not occur in the slab, because of differences in geometry and because the presence of the columns helps to restrain the horizontal displacements.
- Progressive spalling left some reinforcing bars without cover. During the cooling phase, splitting due to cracking of the concrete cover over the bars and loss of bond could have also contributed to the loss of the concrete cover. The exposed reinforcing bars could have been damaged by the fire. The tests indicate that the steel properties have barely changed, which allows reusing the exposed bars; only those with visible deformations should be replaced.
- Though the concrete and steel properties are essentially unchanged, the bond could have been affected. This process starts at temperatures of around 200 ºC, with the bond decaying to 65% at 400 ºC, and totally at 600 ºC [8]. Based on the tests, it seems that 200 ºC were reached by the reinforcement in places where the concrete cover was not lost.
- Where spalling occurred and the bars were exposed, the surrounding concrete should be removed to guarantee bonding with the new concrete; if they are not exposed but they are close to the surface (say within 15 mm), this concrete should also be removed.
- Finally, no significant effects were triggered at the global or structural level; indeed, all the damages were of a local nature. The structural capacity of the different components involved was not affected significantly.

Because of the previous findings, the following recommendations were offered:

- Remove the outer 15 mm of concrete only in the areas where the reinforcement is exposed or almost exposed. The removal of concrete should expose the reinforcement to guarantee the quality of the bond with the shotcrete to be projected later.
- Restore the design dimensions of the slab and columns with shotcrete in all the areas where those dimensions have been reduced by the effects of the fire or the subsequent cleaning.

7. Conclusions

Fires in timber formwork are relatively frequent, often caused by accidental events during the construction stage.

This type of fire has some relevant characteristics:

- On the one hand, the formwork itself serves as a protective layer for the concrete, delaying its exposure to fire.
- On the other hand, the fire load due to the formwork is not usually significant and fully developed fires are rare; that said, the local temperatures and heating rates can be high due to the ventilation available.
- Finally, the concrete affected by the fire was still very young, and therefore had a high moisture content.

The above factors mean that the main damage to the concrete in this type of fire is explosive spalling due to evaporation of the interstitial water, which is usually very spectacular but is not normally of any significance. The temperatures reached inside the concrete are, on the other hand, low, and the fire does not affect its mechanical characteristics except very superficially, and the mechanical effects are normally negligible, as shown in the case of study.

Beyond the results of the study and the effect of a fire on young structures, the following aspects should be highlighted in relation to an investigation of this kind:

- The importance of inspection and interpretation of fire damage.
- The difficulties and precautions to be taken in the extraction and breaking of concrete test specimens, as well as in the interpretation of results.
- The usefulness of ultrasonic measurements as an index of the degree to which the structure has been affected by the fire, including an estimation of the thicknesses that may be affected.

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