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*Original Article*

# Influence of Exposure to High Temperatures on the Mechanical and Physical Properties of Reinforced Concrete

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***Abstract*** *- Reinforced concrete remains one of the most widely used materials in the construction sector. However, over time, its actual service life frequently falls shorter than its nominal projected lifespan, due to exposure to high temperatures, which is a significant factor in this deterioration. Under such conditions, individual components, specifically the cementing material and the steel reinforcement, get altered, and this results in distinct behavioral changes. Consequently, this study evaluates the effect of temperature on the structural behavior of reinforced concrete subjected to temperatures of 500ºC, 700ºC, and 800ºC and exposure periods of 3 and 6 hours. For the attainment of this paper's objective, compressive strength and splitting tensile strength tests were conducted. Furthermore, the results indicated that, across all of the samples, compressive strength of the materials was reduced by 75%, while their tensile strength demonstrated a 50% loss. Additionally, A weight variation of approximately 2% was also observed, though no significant dimensional changes occurred. It is concluded that putting concrete into high-temperature exposure severely impacts the cement-steel interaction, ending up altering internal properties and leading to progressive damage. It would be advisable to expand research aimed at mitigating the observed deterioration in order to develop tools to anticipate the behavior of structures exposed to extreme thermal conditions, such as those generated during fires.*

***Keywords*** *- Reinforced concrete, Temperature, High temperatures.*

## 1. Introduction

The progressive deterioration changes the elastoplastic behavior as a whole, which causes its analysis to be more complex. [3]. This consequence also alters the elasticity modulus and deformations in different local sections, which triggers the residual stresses in the Steel and the ductility behavior in the concrete to be compromised. There are cases in which the material can even detach from the structure, turning it into a weaker structure that can cause fragile failures due to the accumulation of internal pressures when interacting with the reinforcement steel [4]. There are studies in Turkey about the effect of this phenomenon, the gradient of the increasing temperature and the time of exposure causes negative effects that are inversely proportional to the material's properties, like the reduction in the compressive yield strength and the progressive increment of deformations [5]. Additionally, the reinforcement Steel presents unitary flaws that depend on the face that is exposed. When receiving a variation in the temperature degrees, the original properties of the Steel are altered and present residual stress that is scattered in each section [6]. There were experimental tests done in China, where it was found that the yield capacity in the Steel and ultimate tensile strength get reduced when they are constrained under high temperatures, this produces a notable deformation that can reach up to two times its original configuration [7]. A particular and significant case was evidenced when concrete was put under 600°C, which triggered a loss of tension in the reinforced steel, while when the reinforced steel was put under 900°C, the compressive strength resistance was affected by around 10% [8, 9]. Another problem that represents the growth of temperature is the CO2 impregnation in the material, which favors carbonation, low electrical resistivity associated with permeability, and the process of corrosion of the reinforcement in the long term; additionally, this material becomes a conduit material for aggressive agents [https://www.redalyc.org/pdf/1816/181623057003.pdf]. The module of elasticity, that is a factor of the constitutive curve, can be improved through the employment of flying ashes along with fine aggregate in temperatures that are superior to 150°C. Nonetheless, when the reinforced concrete is exposed to higher temperatures over 450°C, it can be seen that its properties are affected [10].

The alteration triggered in the concrete due to the exposure to these high temperatures is associated with the spalling phenomenon, depending on its level of severity. Nevertheless, one way of protecting it is by utilizing polypropylene microfibers, which are characterized as both an effective and economic solution that works as a vapor pressure relief for the material [11]. The phenomenon of thermal deterioration in concrete is particularly prevalent in tropical regions due to the climate and because the heat sources are high and constant, as a consequence, the structural integrity of concrete can be compromised. This problem has aroused greater interest in the research community, and now they seek to create models that are capable of not only anticipating damage in structures when exposed to heat, but also designing methods that help reduce its effects. Research in the fire resistance sphere has revealed that microstructural alterations of heated concrete are especially significant when recycled aggregates are used, although the specific influence of these materials is not fully defined yet [12].

The present study addresses this knowledge gap and proposes a comprehensive approach that evaluates the influence of exposure to high temperatures on the mechanical properties of reinforced concrete. An interaction that modifies the different sectors of an element is depicted in Figure 1, where the main purpose was to examine the effect of the concentration of high temperatures on the structural behavior. This means, in relation to the mechanical properties of the reinforced concrete, as well as the mechanical variation of each material, like the cementitious matrix, the steel, and the material as a whole. The quantitative relationships for each temperature exposition, exposition time, and continuous deterioration were established.

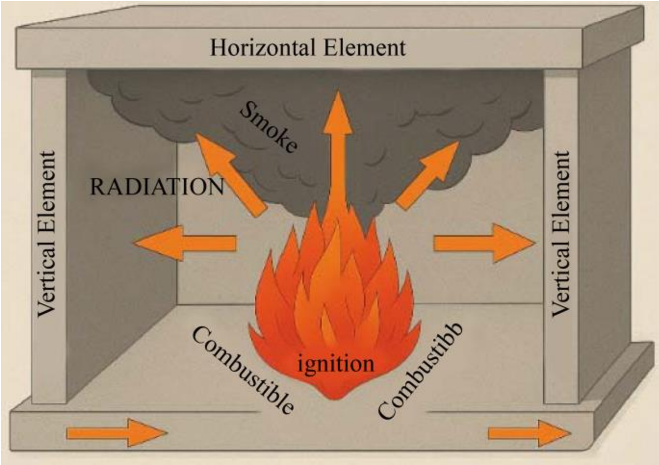


**Fig. 1 Fire spread pattern and its effects on structural elements**

## 2. Materials and Methods

### *2.1. Cements*

Cement can be defined as a material with both cohesive and adhesive properties, which give it the ability to bind mineral fragments together to form a compact whole, as shown in Table 1. Different types of Portland cement are manufactured to achieve certain physical and chemical properties for special objects. According to the standard, they can be classified as standard, and their manufacture is regulated by specific requirements. Type I Portland cement for general use was employed, in accordance with NTP 334.009:2019, equivalent to ASTM C150. This type of cement is suitable for conventional structures and does not have special properties against sulfates or require control of hydration heat.

**Table 1. Main compounds of portland cement**

| Compound name | Oxide composition | Abbreviation |
|---|---|---|
| Tricalcium silicate | 3CaO.SiO2 | C3S |
| Dicalcium silicate | 2CaO.SiO2 | C2S |
| Tricalcium aluminate | 3CaO.Al2O3 | C3A |
| Tetracalcium aluminoferrite | 4CaO.Al2O3.Fe2O3 | C4AF |
| Compound name | Oxide composition | Abbreviation |

### *2.2. Reinforcing Steel*

Steel, in its chemical definition, is an alloy of iron with a carbon content of up to 2%. Its ductility, weldability, and strength depend on this chemical composition. Standard E.060 establishes the use of two types of steel in the construction industry, smooth and corrugated, depending on the case. Reinforced concrete structures generally use corrugated steel with a yield strength of fy = 4200 kg/cm², which is the maximum stress that the steel can withstand without permanent deformation.

For cubic specimens, 3/8" diameter corrugated steel bars with a specified strength of fy = 4200 kg/cm² were used, in accordance with NTP 341.031:2013 (ASTM A615/A615M). These bars were embedded longitudinally to evaluate the physical effects after thermal exposure, such as mass loss, section reduction, and deformation.

### *2.3. Fire*

The fire originates from a chemical reaction that occurs with any object exposed to fire, through parameters such as fuel, oxygen, and heat. Each of these parameters has a starting point at which the act occurs as the fire grows, and new routes may open up that cause it to increase and consume everything in its path. There are three phases of fire: Initial phase: the oxygen content in the air has not been significantly reduced, and the fire produces water vapor, carbon monoxide, carbon dioxide, and perhaps a small amount of sulfur dioxide, among other gases. Free Combustion Phase: In this phase, oxygen-rich air is drawn into the flames as convection carries heat to the upper regions of confined areas. This is where temperatures in the upper regions can exceed 700°C. Latent Phase: In this phase, the flames may cease to exist if the confined area is sufficiently closed off. From this point on, combustion is reduced to incandescent embers. According to NTP 350.021 2012 Classification of fires and their graphic representation, fires are classified according to Table 2.

**Table 2. Fire classification**

| Class | Classification of fire |
|---|---|
| Class A | It is the fire produced by the combustion of common solid combustible materials such as paper, wood, fabric, straw, rubber, and some types of plastics. Its main characteristic is that it can form embers and residues. |
| Class B | It is the fire produced by the combustion of flammable liquids, liquid fuels, petroleum and its derivatives, oils, tars, oil-based paints, lacquers, solvents, and flammable gases. |
| Class C | It is fire produced in energized electrical circuit equipment or systems, that is, with the effective presence of electricity. |
| Class D | It is the fire produced by combustible metals, such as magnesium, titanium, zirconium, and their alloys; sodium, lithium, potassium, metallic, and others. |
| Class K | It is the fire produced in cooking appliances that involve a combustible medium used for cooking (animal or vegetable oils and fats). |

The designation for the sample batches consisted of elements subjected to different values of hours and different values of temperatures, according to Table 3.

**Table 3. Exposure conditions in concrete**

| Batch | Temperature | Time (Hrs.) | Condition nomenclature |
|---|---|---|---|
| 1 | - | - | CP |
| 2 | 500 | 3 | C_T500-3H |
| 3 | 500 | 6 | C_T500-6H |
| 4 | 700 | 3 | C_T700-3H |
| 5 | 700 | 6 | C_T700-6H |
| 6 | 800 | 3 | C_T800-3H |
| 7 | 800 | 6 | C_T800-6H |

**Table 4. Concrete cylindrical specimens**

| Group | N° Specimen |
|---|---|
| DB-14D | 6 |
| DB-28D | 6 |
| D1-500T-3H | 6 |
| D2-500T-6H | 6 |
| D1-700T-3H | 6 |
| D2-700T-6H | 6 |
| D1-800T-3H | 6 |
| D2-800T-6H | 6 |
| DB: base design, D: days, T: temperature, H: hours | |

### *2.4. Dosing of Specimens*

Forty-eight cylindrical specimens measuring 4 inches in diameter and 8 inches in height were prepared, with 24 samples distributed for compression testing and 24 for indirect tensile testing, as shown in Table 3. In addition, 18 cubes measuring 10 × 10 × 10 cm with embedded steel were prepared for the characterization of the reinforcement, as shown in Table 5.

**Table 5. Steels used in cubic specimens**

| Sample N° | Large (mm) | Ø (mm) | Weight (g) |
|---|---|---|---|
| S1 | 51.37 | 12.30 | 49.41 |
| S2 | 49.92 | 12.12 | 48.45 |
| S3 | 51.93 | 12.19 | 50.30 |
| S4 | 48.29 | 12.10 | 46.54 |
| S5 | 51.51 | 12.86 | 50.79 |
| S6 | 47.97 | 11.92 | 46.94 |
| S7 | 51.62 | 11.97 | 50.52 |
| S8 | 51.81 | 11.94 | 51.07 |
| S9 | 50.62 | 12.13 | 48.35 |
| S10 | 48.88 | 12.01 | 47.58 |
| S11 | 49.92 | 12.24 | 48.78 |
| S12 | 52.48 | 11.79 | 50.17 |
| S13 | 48.94 | 11.95 | 47.34 |
| S14 | 50.41 | 11.90 | 49.56 |
| S15 | 51.65 | 12.24 | 49.52 |
| S16 | 49.02 | 11.44 | 46.89 |
| S17 | 51.59 | 11.87 | 48.89 |
| S18 | 52.18 | 12.44 | 49.40 |
| S: Sample | | | |

All specimens were cured for 28 days under laboratory conditions, in a humid environment, in accordance with ASTM C192 [13]. Additionally, Figure 2 shows some samples of concrete cubes, to subsequently test them and quantify the physical characteristics of steel. After thermal exposure, the concrete was manually removed to expose the bars of the samples indicated in Table 5, which were measured for weight, length, and diameter.

**Fig. 2 Steel cube specimens**

### *2.5. Compression Strength Test*

It was applied to 4" × 8" cylindrical specimens without reinforcement, in accordance with Peruvian Technical Standard NTP 339.034:2017. The test specimens were placed centrally in a controlled-load hydraulic press, applying an increasing axial force until failure. Compressive strength was calculated by dividing the maximum load supported by the cross-sectional area of the cylinder. In this case, the procedure to be followed for breaking specimens after exposure to different fire conditions was as follows: the specimen was placed in position, and any impurities and other debris were cleaned from the lower and upper cross sections. Subsequently, a loading rate in the range of 0.25 ± 0.05 MPa/s (35±7psi/s) was applied. In this case, the selected rate was maintained, at least during the second half of the test cycle. The maximum load supported during the test process was recorded, and the failure pattern was also noted.

This test made it possible to quantify the reduction in the structural capacity of concrete after exposure to 500°C, 700°C, and 800°C for 3 and 6 hours, compared to the control group without thermal exposure.

### *2.6. Indirect Tensile Test*

The same cylinders were subjected to the indirect tensile test, also known as the Brazilian test, in accordance with NTP 339.078:2017. The load was applied diametrically on the cylinder's generatrix until tensile fracture.

In this case, the procedure to be followed for the test was carried out after the specimen had been subjected to different fire exposure conditions for 24 hours. To do this, the following steps were taken: First, the specimen was placed, then impurities and other substances were cleaned from the specimen, a loading speed in the range of 0.7MPa/min and 1.4MPa/min was applied until the specimen failed due to the stress produced, and finally, the maximum load supported during the test process was recorded, and the type of failure was noted.

This method allows the tensile strength of concrete to be determined, a critical property in structures exposed to fire due to the increase in tensile stresses caused by thermal gradients and loss of internal cohesion.

## 3. Results

### *3.1. Compression Tests*

The analysis of the compressive strength results shown in Figure 3 shows the averages of each of the three samples of each specimen, in which there are significant variations. The DB-28D sample recorded the highest value at 30.71 MPa, followed by DB-14D (26.63 MPa) and D1-500T-3H (26.12 MPa), confirming the effectiveness of conventional curing in developing strength. On the other hand, samples exposed to high temperatures for prolonged periods, such as D2-800T-6H (7.51 MPa) and D2-700T-6H (9.67 MPa), had the lowest values, suggesting a degradation in the microstructure of the material.

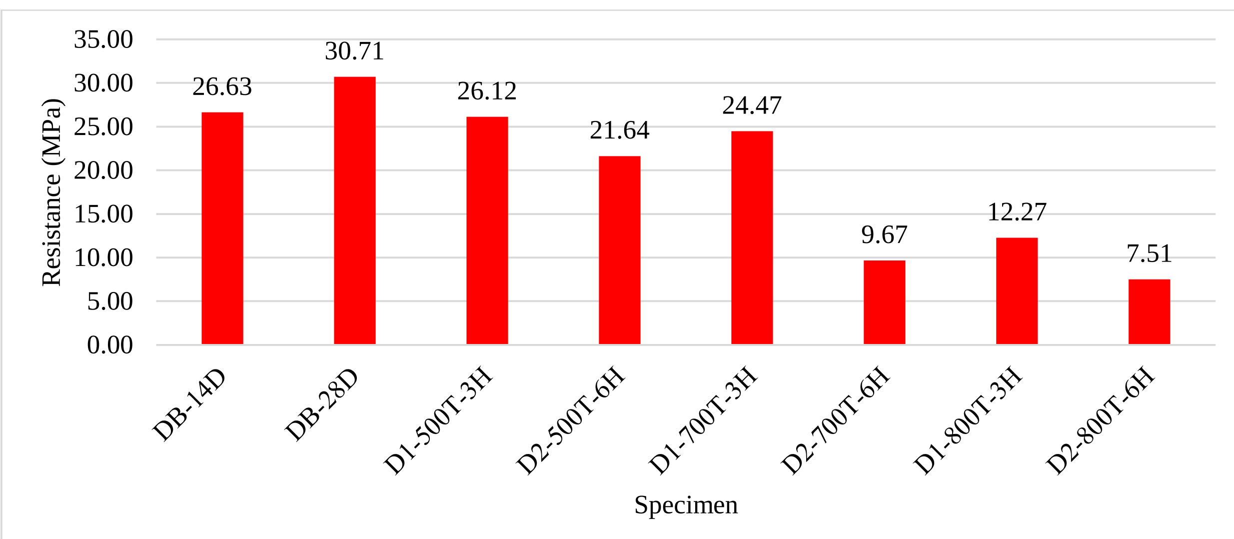



**Fig. 3 Trend in compressive strength of specimens**

Figure 4 shows the percentage reduction in compressive strength as a function of the heat treatment applied, taking as an initial reference 100% corresponding to the base design sample at 28 days (DB-28D). This unified representation shows the trends of the samples subjected to D1-500T-3H, D2-500T-6H, D1-700T-3H, D2-700T-6H, D1-800T-3H, D2-800T-6H, whose linear regression lines indicate a progressive decrease in strength with increasing thermal exposure time. Specimen D2-500T-6H shows a reduction of up to 70.47%, with a coefficient of determination =1.0000, implying a perfect linear fit. In the case of specimen D2-700T-6H, strength is reduced by up to 31.48% with =0.9476, while specimen 800T achieves the greatest loss with only 24.47% of strength remaining and 0.8960. These results confirm that the severity of the heat treatment (higher temperature and time) has a direct influence on the mechanical degradation of the material, with D2-type mixtures being the most affected, as evidenced by the downward trend of each curve, with 100% corresponding to the base design sample at 28 days (DB-28D) as the initial reference. The linear regression for different temperature states (500T-700T-800T) is displayed in Figure 4, as well as the exposure time (3 hours – 6 hours) of the examined samples. It can also be observed that the regression lines have a steady decreasing tendency that represents the concrete strength based on the exposure. Among the samples, the ones that are relevant to the D2-500T-6H; this one represents a reduction ratio of up to 70.5% that reflects a statistical coefficient of $R^2$=1.0, giving a relatively exact fit. However, the simple D2-700t-6H exhibits another reduction of 31.5% with an $R^2$=0.95. With regards to the example D2-800T-6H, this one has the most significant decay of up to 24% and an $R^2$=0.90. This result depicts a sensible behavior to exposure time and temperature rise, contrasting that the D2 samples reflected a greater sensitivity.

The ANOVA statistical evaluation was executed in order to verify the variations of each group of samples depending on the temperature (T:500 ±30º, T:700 ±30º, T:800 ±30º) and the exposure duration (3 hours, 6 hours). Thus, a significance level of 0.05 was utilized, obtaining the results shown in Tables 6 and 7.

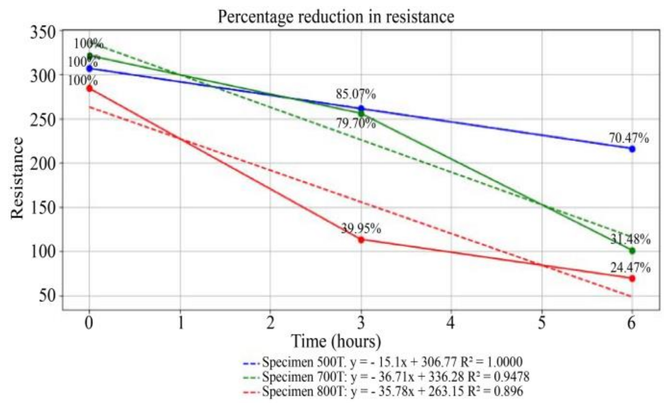



**Fig. 4 Resistencia a compression en funcion al tiempo de exposicion del incremento de temperatura**

**Table 6. ANOVA test on compressive strength for 3 hours**

| Condition | ∑ $X^2$ | GDL | ∑ $X^2$/GDL | F | Prob. | Fcrit. |
|---|---|---|---|---|---|---|
| Group | 343.90 | 2 | 171.95 | 8597.56 | 0.00 | 5.14 |
| In the group | 0.12 | 6 | 0.02 | | | |
| Total | 344.02 | 8 | | | | |

**Table 7. ANOVA test on compressive strength for 6 hours**

| Condition | ∑ $X^2$ | GDL | ∑ $X^2$/GDL | F | Prob. | Fcrit |
|---|---|---|---|---|---|---|
| Group | 347.65 | 2 | 173.82 | 14221.9 | 0.00 | 5.14 |
| In the group | 0.07 | 6 | 0.01 | | | |
| Total | 347.72 | 8 | | | | |

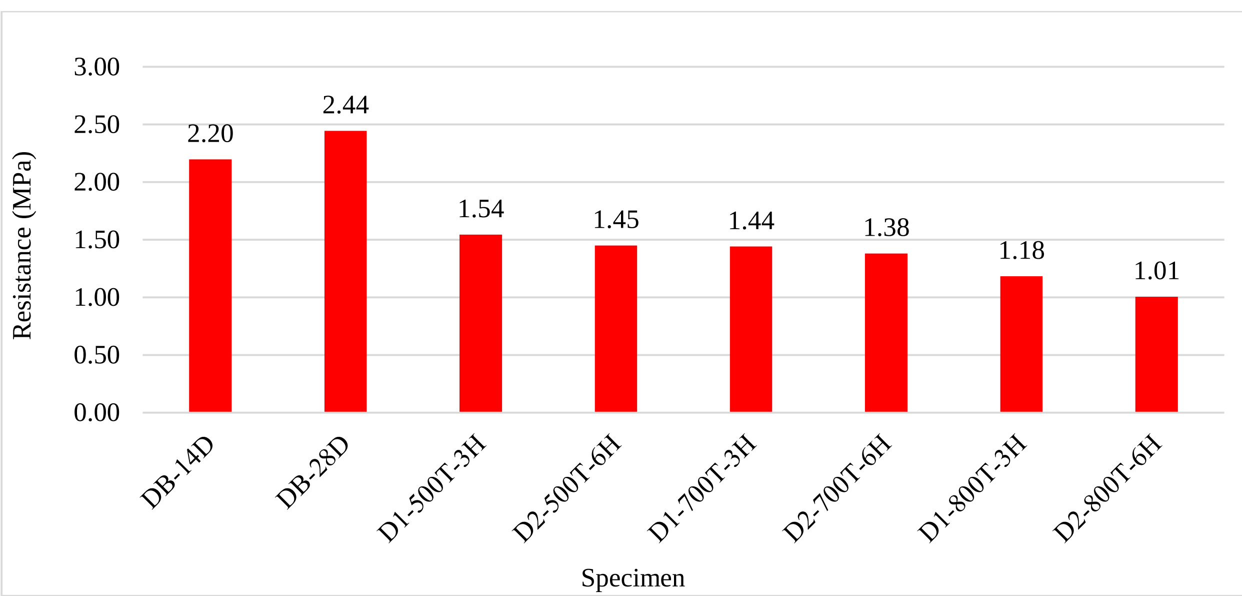



**Fig. 5 Tensile strength trend of the samples**

### *3.2. Indirect Tensile Test*

Figure 5 shows the results of the tensile strength for the samples that were subjected to temperatures from 500 Cº to 800 Cº, along with an exposure time of 3 and 6 hours. From them, the resistances were obtained for the control samples during 14 and 28 days, with a resistance of 2.44 MPa and 2.20 MPa, respectively. The prominent values in the loss of this resistance are shown in D2-800T-6H and D1-800T-3H, which resulted in 1.01MPa and 1.18 MPa. These results can be attributed to a clear deterioration of the reinforced concrete's internal components due to the alteration caused by the thermal effect. This allows us to highlight the importance of performing a proper curing process during the setting regime do t he mechanical components of the material can be preserved.

Figure 6 illustrates the simple values for joint comparison of the reduction in tensile strength, using the 28-day control specimen as the baseline model relative to the other samples. As previously noted, this decreasing trend associated with increasing temperature and exposure time contributes to the progressive deterioration of the material's nominal service life, which translates to a reduction in its load-bearing capacity. The trend lines shown allow us to visualize the linear relationship between the treatment and the loss of strength, highlighting coefficients of determination $R^2$ of 0.8198, 0.7916, and 0.8404 for the treatments at 500 °C, 700 °C, and 800 °C, respectively. These values show an adequate correlation in each case, confirming that more severe heat treatments generate a greater decrease in the indirect tensile strength of the material.

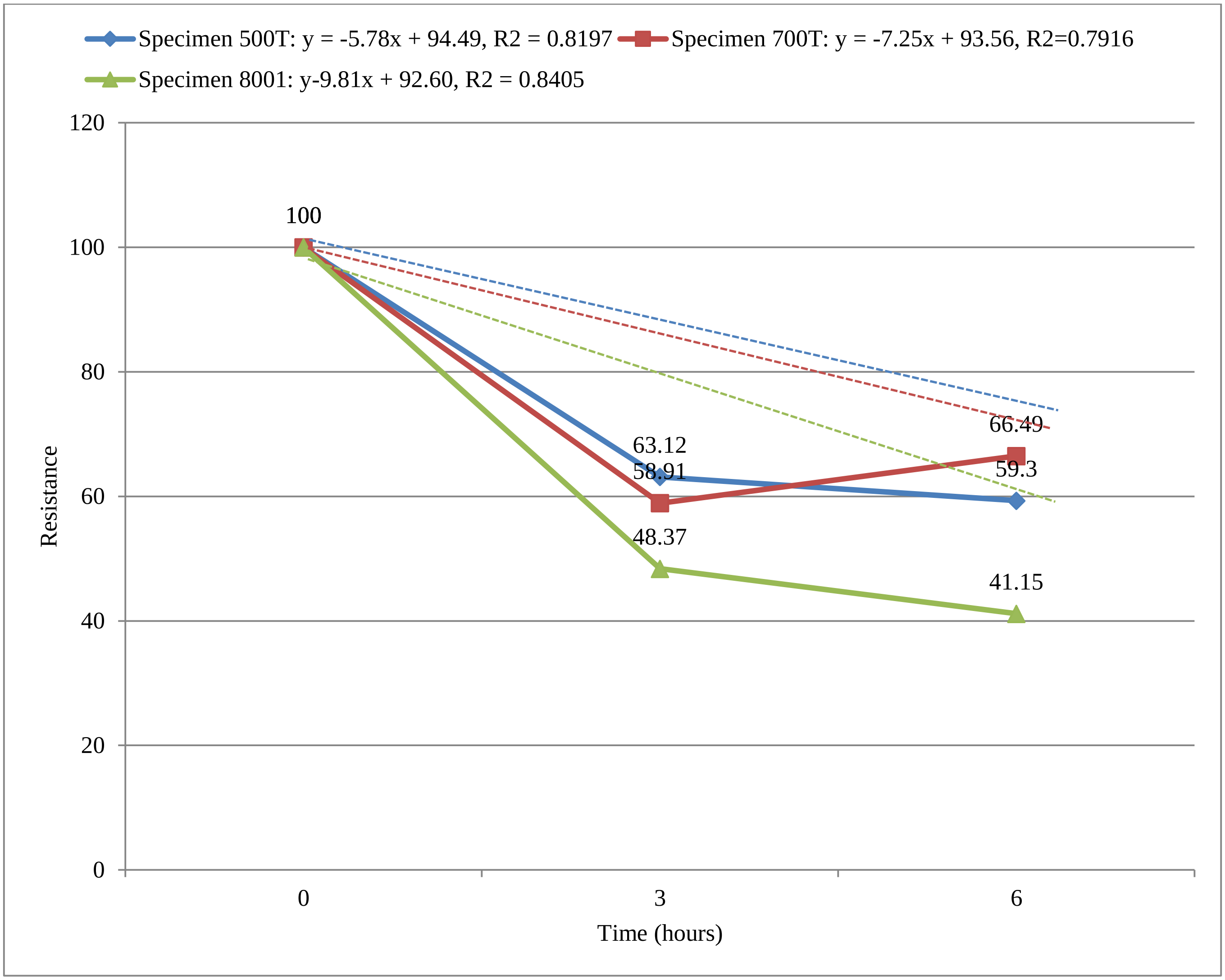



**Fig. 6 Linear regressions of the resistances in the tested samples**

In the same way as in the statistical analysis of the compression strength tests, the tensile strength tests were also executed, and the same conditions of temperature, exposure time, and the coefficient values were respected as shown in Tables 8 and 9.

**Table 8. ANOVA test on tensile strength for 3 hours**

| Condition | ∑ $X^2$ | GDL | ∑ $X^2$/GDL | F | Prob. | Fcrit |
|---|---|---|---|---|---|---|
| Group | 199121.56 | 2 | 99560.78 | 62.70 | 0.00 | 5.14 |
| In the group | 9526.67 | 6 | 1587.78 | | | |
| Total | 208648.22 | 8 | | | | |

**Table 9. ANOVA test on tensile strength for 6 hours**

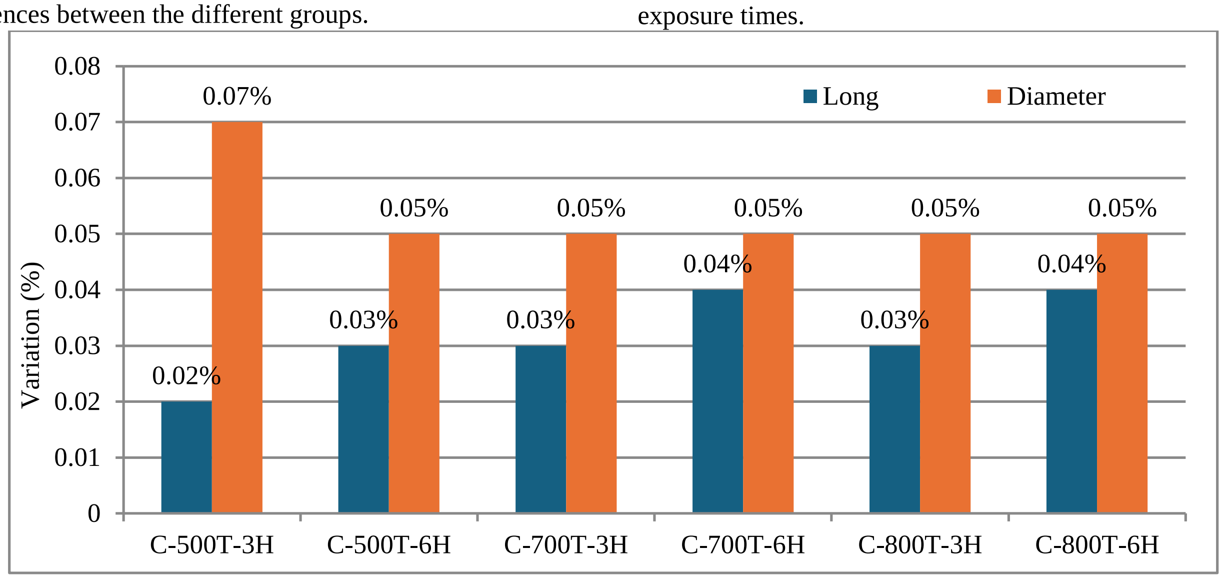
| Condition | ∑ $X^2$ | GDL | ∑ $X^2$/GDL | F | Prob. | Fcrit |
|---|---|---|---|---|---|---|
| Group | 328937.56 | 2 | 164468.78 | 107.34 | 0.00 | 5.14 |
| In the group | 9193.33 | 6 | 1532.22 | | | |
| Total | 338130.89 | 8 | | | | |

### *3.3. Testing the Cubic Sample*

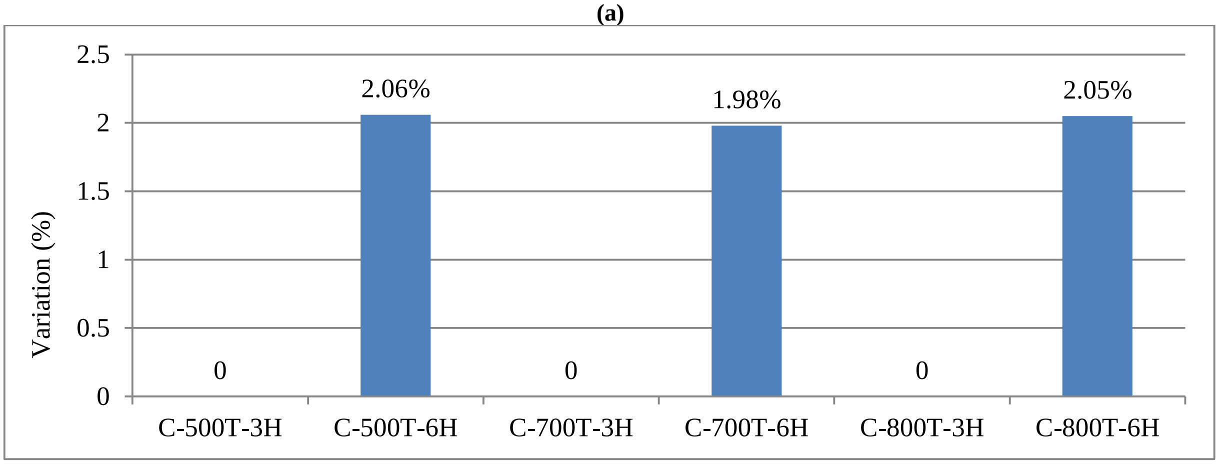
Figure 7(a) shows the comparison of the percentage variation in the length and diameter parameters for six types of samples. It can be seen that the variation in diameter is greater than that in length in all cases, being most noticeable in the concrete sample at 500°C for 3 hours (C-500T-3H), which has the greatest dispersion in diameter at around 0.07%. In contrast, the variations in length remain within a narrow range between 0.02% and 0.04%, with no significant differences between the different groups.

Figure 7(b) shows the percentage variation in weight for the same sample groups. There is a marked difference between the groups, as three of them (C-500T-6H, C-700T-6H, and C-800T-6H) show a notably high variation (greater than 2%), while the remaining groups (C-500T-3H, C-700T-3H, and C-800T-3H) show no variation (0.00%).

This indicates more stable weight behavior for samples with shorter exposure times compared to those with longer exposure times.




**(a)**




**(b)**

**Fig. 7 (a) Percentage variation in length and diameter, (b) Percentage variation in weight.**

## 4. Discussion

From the results obtained in this paper, it can be stated that they show a decreasing trend in the compressive strength of reinforced concrete when it is exposed to increasing temperature and long exposure time. It was found [14] that concrete under temperatures from 400°C has an abrupt change in its structural behavior that can reach up to 60% when it is exposed to 800°C, which is directly related to loss of capillarity and surpasses the elastic deformation threshold. Moreover, it was perceived [15] that even when concrete is reinforced with vegetal fibers, it also causes mechanical losses that start from 500°C, which can easily be reduced up to 40% if a 200°C temperature increment is applied. These findings corroborate that a decline in the material behavior happens between 500°C and 800°C. Additionally, one of the main components of the cementing material is the hydrated silicates, and when these break when they come in touch with each other, the porosity increases drastically, turning into calcium oxide, this phenomenon contributes to damage caused by spalling [16].

The tensile behavior of concrete achieved through its interaction with steel is essential to guarantee its stability. Nevertheless, it was proven that this is very sensitive to high temperatures, and this is reflected by the progressive appearance of small fissures that can have long-term repercussions.

[https://revistas.uees.edu.ec/index.php/IRR/article/view/560/515].

Another study related to the tensile behavior of this material [17] argues that strength gets reduced and even surpasses 60% when it is subjected to foundry sand and temperatures around 600°C. Nonetheless, this can be improved by considering that the concrete composition must have pozzolanic ash because it tolerates temperatures of 300°C [18]. Whereas, Steel, as a complement of cementing material, is the most affected due to the high temperatures, despite the fact of employing heterogeneous mixtures.

Another fundamental parameter is present in the weight of the material, but it can be apparently related to dimensional changes. Nonetheless, this is not always the case because there are study cases in which, even though there was a loss of up to 14% of the weight material when subjected to temperatures of 800°C, the geometrical configuration remained almost negligible [19]. On the other side, this damage attributed to concrete is related to the structural risk, and this is a result of an internal decay of its components [15]. In this sense, it is expected to comprehend that in spite of the absence of dimensional changes in the material, this should not be interpreted as minor damage, but as a possible internal alteration in its composition.

## 5. Conclusion

The findings from the tests performed confirm that when reinforced concrete is subjected to gradual increments of temperature and exposure time, it produces a progressive and significant deterioration in its structural behavior. This circumstance prevents this material from guaranteeing its full load-bearing capacity for the service time for which it was projected. One of the predominant aspects was found in compressive strength when it was subjected to 800°C for 6 hours. It was found that the residual capacity tolerates 24.47% when compared to its unaltered design. Additionally, with regard to the tensile strength, a reduction of up to 1MPa, which corresponds to 50% of the relative control design, is observed. These anomalies were in the fissure appearance that propagated steadily as a product of the primary alterations, like the loss of silicates in the cementing material and the residual stresses from the thermal dilatation of the reinforcement steel, reflecting a deficit in the structural integrity.

Besides all these observations, a weight loss of over 2% was also noticed in spite of not accounting for the loss of its geometry, and this shows a sensitivity to the internal decay. On the other side, the exposure time resulted in one of the most important indicators for intensifying the thermal effect, in which the samples that had an exposure of 6 hours were the most unreliable. This situation suggests proposing preventive methods towards the development of thermal control in the concrete, as well as a challenge to systematize the predictive models based on damage indices that can have better control in the conservation of the material.

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