Analysis on Damage and Mechanical Properties of Ballastless Track in a Tunnel after a Fire

Hujun Ma 1, Wei Chen 1,2, *, Xiang Li 3, Qingyuan Xu 1,2, Ping Lou 1,2 and Chencai Tong 4

1 School of Civil Engineering, Central South University, Changsha 410075, China
2 National Engineering Research Center of High-Speed Railway Construction Technology, Central South University, Changsha 410075, China
3 School of Civil Engineering, Sun Yat-sen University, Zhuhai 519082, China
4 T.Y.Lin International Engineering Consulting (China) Co., Ltd., Chongqing 401121, China
* Correspondence: chenwei.csu@foxmail.com

Abstract: In order to explore the damage and mechanical properties of ballastless track after a fire, the uniaxial compressive strength, shear strength, peak strain, and elastic modulus changes due to temperature were obtained through uniaxial compressive and shear tests of concrete after exposure to high temperatures. The test results showed that with increases in temperature, the uniaxial compressive strength, shear strength, and elastic modulus of concrete all presented a decreasing trend, while the peak strain had an increasing trend. Then, based on the classical damage theory model and the strength probability distribution function of concrete micro-units, the high-temperature damage constitutive equation for concrete was established, and the compressive stress–strain curve of concrete after exposure to high temperature was reproduced. Finally, using the CFD numerical simulation software, the temperature field of a ballastless track structure in a tunnel during a fire was obtained, and the temperatures at different positions of ballastless track bed were acquired. Combined with the high-temperature damage constitutive equation for concrete deduced from tests and theoretical analysis, the strength and damage values of the ballastless track bed at different positions after a tunnel fire were obtained.

Keywords: uniaxial compressive strength; shear strength; high-temperature damage constitutive equation for concrete; temperature field; damage of ballastless track

1. Introduction

Concrete is one of the most important building materials, and its mechanical properties are an important factor in determining its service performance. After exposure to a high-temperature environment, the mechanical properties of concrete are very different from those at normal temperatures [1–4]. The mechanical properties of concrete after exposure to high temperatures have a great influence on the performance evaluation and reinforcement of structures after a fire [5–8]. Many scholars have studied the compressive strength, shear strength, elastic modulus, damage and so on of concrete after exposure to high temperatures. Toric et al. [9] carried out a test by heating high-strength concrete samples to a high target temperature; the test analyzed compressive strength, tensile strength, and the tangent and secant elastic modulus. Knobloch et al. [11] investigated the uniaxial performance of concrete at elevated temperatures under cyclic compressive loading, and confirmed the suitability of the damage-plasticity modelling concept for concrete under uniaxial compressive stress at elevated temperatures. Aziz et al. [12] studied the high-temperature performance of a low-strength concrete specimen, and it was found that the ductility and residual Poisson’s ratio of the concrete increased with the increase in heating temperatures.
temperature after heating at 700 °C; the elastic modulus of the concrete decreased sharply after heating at 300 °C; and the concrete lost more than 80% of its stiffness after exposure to 600 °C. Le et al. [13] established a simple formula for the stress–strain-temperature relationship of concrete, which was consistent with the stress–strain curve of concrete in Eurocode 2 used for the heating phase. Nuaklong et al. [14] studied the confounding effect of multi-walled carbon nanotubes (MWCNTs) and polypropylene (PP) fibers on the mechanical and fire resistance of Portland cement mortar, and the strength of the mortar was measured at different temperatures below 1000 °C. Sukontasukkul et al. [15] studied various effects of high temperature on the mechanical properties of fiber-reinforced concrete (FRC), and conducted flexural toughness tests on two types of concrete (plain concrete and fiber-reinforced concrete) according to ASTM C1018. Sukontasukkul et al. [16] directly determined the damage to concrete under static and impact compressive loads using damage mechanics theory. Two methods based on the variation of (1) elastic modulus (E) and (2) strain rate (ε) were used.

In recent years, tunnel fire accidents have occurred frequently, often causing serious casualties and property losses. Tunnel fire research [17–21] has become an important issue in the field of fire research. Many scholars have simulated tunnel fires through different tests, models, and different software such as FDS and FLUENT under the influence of factors such as fire scale, fire environment, and tunnel types to obtain the structural temperature field, smoke flow rules and so on in the tunnel. Qu et al. [22] and Lu et al. [23] used the conservation law, the theories of high-temperature gas dynamics, and fluid mechanics to set up the physical and mathematical turbulence flow field model and simulated a tunnel fire and its smoke flow. Lu et al. [24] studied the temperature distribution in curved tunnels using a numerical simulation method, and the results showed that the temperature was slightly higher for a concave wall than a convex wall for the distant positions from the fire source, which was different from that in a straight tunnel. Li et al. [25] carried out 1:10 reduced-scale tests for branched tunnels to investigate the longitudinal fire location effects on thermal smoke temperature distribution beneath the ceiling. Qu et al. [26] established physical and mathematical turbulence flow field models to numerically simulate railway tunnel fire and smoke flow; the results showed that the temperature distribution of laminar flame, smoke concentration, and flow velocity can be represented by fully developed smoke flow in the downwind direction. Sukontasukkul et al. [27] studied the effect of fire on the flexural properties and residual strength of ordinary concrete and fiber-reinforced concrete and tested the bending load of the sintered sample to determine its toughness and residual strength.

These studies have not only greatly deepened the understanding of the damage and mechanical properties of concrete after exposure to high temperatures but have also achieved fruitful work on smoke flow and temperature fields of tunnel fires. However, there is still a lack of targeted research on the temperature field distribution of the ballastless track and the damage and mechanical properties of the concrete track bed in a tunnel after a fire. Therefore, this paper carried out tests of the mechanical properties of concrete after exposure to high temperatures. Starting from the classical damage theory model and the strength probability distribution function of concrete micro-units, the high-temperature damage constitutive equation for concrete was established. Then, the temperature field of the ballastless track structure in a tunnel during a fire was simulated using the computational fluid dynamics (CFD) numerical software FLUENT, and finally the damage and mechanical properties of a ballastless track structure in a tunnel after a fire were obtained.

2. Uniaxial and Shear Tests

2.1. Test Preparation

The materials and mix properties used in preparing the concrete related to the experimental part of this paper were as shown in Table 1 [28].
2. Uniaxial and Shear Tests

2.1. Test Preparation

The materials and mix properties used in preparing the concrete related to the experiment were tested. In order to study the influence on the mechanical properties of concrete after exposure to high temperatures, uniaxial compressive and shear tests were carried out on the heated specimens. Compressive strength and shear strength are important indicators to measure the mechanical properties of concrete, which are greatly affected by temperature. In order to study the influence on the mechanical properties of concrete after exposure to high temperatures, uniaxial compressive and shear tests were carried out on the heated specimens.

Table 1. Mix proportions of concrete.

| Mixtures                        | Dosage |
|--------------------------------|--------|
| Cement (kg/m³)                  | 227    |
| Water (kg/m³)                   | 149    |
| Fine sand (kg/m³)               | 795    |
| Coarse aggregate (kg/m³)        | 1098   |
| Limestone powder (kg/m³)        | 68     |
| Slag (kg/m³)                    | 44     |
| Admixture (kg/m³)               | 6.4    |

The concrete was made into a cube specimen of 150 mm × 150 mm × 150 mm and cured in a standard curing room for 28 days after demolding. Before the mechanical properties were tested, a resistive heating furnace (model: KSY-12-T), produced by Changsha Changcheng Electric Furnace Factory, was used to heat the concrete specimens [29], as shown in Figure 1. During heating, all specimens were heated to a predetermined high temperature (100 °C, 200 °C, 400 °C, 600 °C, or 800 °C) in the heating furnace at a rate of 5 °C/min, and then the temperature was maintained for 2 h to make sure the specimens were heated evenly. Next, the high-temperature specimens were cooled by air cooling, that is, the specimens were placed in a room-temperature environment until the temperature dropped to room temperature. During the heating process, the thermometer reading inside the heating furnace was read and recorded every 60 s, and the temperature change of the specimen during the heating process was obtained as shown in Figure 1b. Finally, the mechanical properties such as uniaxial compressive strength and shear strength were tested.

Figure 1. Heating process: (a) the resistive heating furnace (model: KSY-12-T); (b) heating process.

2.2. Test Method

Compressive strength and shear strength are important indicators to measure the mechanical properties of concrete, which are greatly affected by temperature. In order to study the influence on the mechanical properties of concrete after exposure to high temperatures, uniaxial compressive and shear tests were carried out on the heated specimens at 20 °C, 100 °C, 200 °C, 400 °C, 600 °C, and 800 °C. The compression test used a
In addition, the peak stress and peak strain of specimens are shown in Table 2. It can be seen from Figure 3 that with the increase in temperature, the shape of the curve transitions from sharp to flat, indicating that the stress of the concrete gradually decreases while the strain gradually increases, resulting in damage inside the specimen, and high temperature will cause great damage to concrete.

3. Test Results and Analysis
3.1. Stress–Strain Curve of the Uniaxial Compressive Test

In order to obtain a more accurate stress–strain curve, during the experiment the instrument could automatically record the force and displacement, and then divide the force by the loading area to obtain the stress and divide the displacement by the height to obtain the strain. The converted curve is the stress–strain curve, and the stress–strain curves of concrete specimens after different heated temperatures are shown in Figure 3. In addition, the peak stress and peak strain of specimens are shown in Table 2. It can be seen from Figure 3 and Table 2 that with the increase in temperature, the shape of the curve transitions from sharp to flat, indicating that the stress of the concrete gradually decreases while the strain gradually increases, resulting in damage inside the specimen, and high temperature will cause great damage to concrete.

![Figure 2](image-url)  
Figure 2. Photos of test detail: (a) location of strain gauges; (b) location of LVDT; (c) shear test; (d) failure specimens.

![Figure 3](image-url)  
Figure 3. Stress–strain curve of concrete during the uniaxial compressive test after different heated temperatures.
### Table 2. Peak stress and peak strain of the uniaxial compressive test after different heated temperatures.

| Temperature/°C | 20  | 100 | 200 | 400 | 600 | 800 |
|----------------|-----|-----|-----|-----|-----|-----|
| Peak stress/MPa | 36.15 | 23.86 | 22.83 | 18.19 | 10.45 | 7.34 |
| Peak strain /10^{-3} | 1.63 | 2.35 | 3.46 | 5.27 | 7.46 | 9.45 |

#### 3.2. Elastic Modulus

In this paper, the stress–strain curves after different heated temperatures were normalized without dimensionality [30], that is, the strain on the abscissa was divided by the peak strain, the stress on the ordinate was divided by the peak stress, and then the full stress–strain curve was drawn as shown in Figure 4. It can be seen from Figure 4 that the full stress–strain curves after different heated temperatures basically overlap in the ascending segment before $\varepsilon / \varepsilon_m$ equals 1, while the descending segment after $\varepsilon / \varepsilon_m$ equals 1 has greater discreteness and has varying degrees of smoothness.

At the same time, in order to quantitatively describe the relationship between elastic modulus and temperature, on the basis of obtaining the full stress–strain curve, stress and the corresponding strain at the ordinate of 0.4 on the full stress–strain curve were taken. Finally, the elastic modulus of the concrete specimens after different heated temperatures were obtained by dividing the strain by the stress obtained through the above method [31], as shown in Table 3. Linear, polynomial, and exponential fitting methods were used to fit the relationship between the elastic modulus and the temperature, and the results are shown in Figure 5. It can be seen that the elastic modulus of the concrete decreases exponentially after high-temperature heating and cooling. During fitting, the larger the R-square was, the higher the fitting accuracy was. By comparing and analyzing the R-square of various fitting methods, it can be concluded that the exponential fitting result is more accurate, and the fitting formula is shown as follows:

$$E_T = 3.0066 \times e^{-\frac{T}{209.2671}} + 0.35259$$  (1)
There are two limitations on the use of this formula: first, the type of concrete was ordinary concrete, and the concrete strength grade was C35; second, the test type was a uniaxial compression test.

Table 3. Elastic modulus of concrete after different heated temperatures.

| Temperature/°C | 20 | 100 | 200 | 400 | 600 | 800 |
|----------------|----|-----|-----|-----|-----|-----|
| Elastic modulus $E_T \times 10^4$/MPa | 3.23 | 1.84 | 1.73 | 0.93 | 0.60 | 0.26 |

Figure 5. Fitting curve of temperature and elastic modulus: (a) linear fitting; (b) polynomial fitting; (c) exponential fitting.

3.3. Relationship between Uniaxial Compressive Strength and Temperature

Table 4 lists the uniaxial compressive strength and the reduction rate of uniaxial compressive strength relative to 20 °C at different heated temperatures. It can be seen that with the increases in the temperature, the uniaxial compressive strength becomes lower and lower, while the compressive strength reduction rate becomes higher and higher. In order to characterize the correlation between uniaxial compressive strength and temperature, linear, polynomial, and exponential fitting methods were used to fit it, and the results are shown in Figure 6. When exponential fitting was adopted, the R-square was the largest and the fitting result was more accurate. The fitting formula is shown as follows:

$$f_c(T) = 31.08989 \times e^{-0.23609} + 3.99768$$

There are two limitations on the use of this formula: first, the type of concrete was ordinary concrete, and the concrete strength grade was C35; second, the test type was a uniaxial compression test.

Table 4. Uniaxial compressive strength and the reduction rate of uniaxial compressive strength.

| Temperature/°C | 20 | 100 | 200 | 400 | 600 | 800 |
|----------------|----|-----|-----|-----|-----|-----|
| Uniaxial compressive strength $f_c(T)$/MPa | 36.15 | 23.86 | 22.83 | 18.19 | 10.45 | 7.34 |
| The reduction rate of uniaxial compressive strength/% | 0.00 | 34.00 | 36.85 | 49.68 | 71.09 | 79.70 |
This paper uses two ways to evaluate the damage of concrete:

1. Damage calculation using “residual strength” thermal damage model

Luo et al. [32] studied the compressive strength of concrete after exposure to high temperatures and proposed the “residual strength” thermal damage model, as shown in Formula (3). In combination with Formula (2), the “residual strength” thermal damage model can be acquired.

\[
D_1(T) = 1 - \frac{f_c(T)}{f_{c0}} 
\]

\[
D_1(T) = 1 - \frac{31.08989 \times e^{\frac{-T}{36.15}} + 3.97968}{36.15} 
\]

where \(f_{c0}\) is the uniaxial compressive strength of concrete at room temperature, \(f_{c0} = 36.15\) MPa, and \(D_1(T)\) is the damage value. The damage value \(D_1(T)\) calculated by Formula (4) is shown in Table 4.

2. Damage calculation using elastic modulus

Jiang et al. [33] determined the damage value \(D_2(T)\) of concrete structure based on elastic modulus, as shown in Formula (5). In combination with Formula (1), the damage value \(D_2(T)\) can be obtained.

\[
D_2(T) = 1 - \frac{E_T}{E_0} 
\]

\[
D_2(T) = 1 - \frac{3.0066 \times e^{\frac{-T}{3.23}} + 0.35259}{3.23} 
\]

where \(E_T\) is the elastic modulus of concrete after high-temperature heating, and \(E_0\) is the initial modulus of concrete at room temperature, \(E_0 = 3.23 \times 10^4\) MPa.

Similarly, the damage value calculated by Formula (6) is shown in Table 5. The results show that the damage to concrete increases nonlinearly with the increases in heating temperature.

Table 5. Damage values \(D_1(T)\) and \(D_2(T)\).

| Temperature/°C | 20  | 100 | 200 | 400 | 600 | 800 |
|----------------|-----|-----|-----|-----|-----|-----|
| Damage value \(D_1(T)\) | 0.0717 | 0.2210 | 0.3700 | 0.5757 | 0.6999 | 0.7749 |
| Damage value \(D_2(T)\) | 0.0448 | 0.3136 | 0.5329 | 0.7532 | 0.8379 | 0.8705 |
3.5. High-Temperature Damage Constitutive Equation for Concrete

The damage to concrete is caused by the initiation, expansion, and connection of its internal cracks; both temperature loads and external forces may cause the generation of internal cracks, which constitute damage. Based on the classical damage theory model and the strength probability distribution function of concrete micro-units, an equation describing the whole process of concrete compressive damage at high temperature is obtained in this paper.

Taking the representative concrete volume unit used in the research and based on Lemaitre’s hypothesis of “equivalent strain”, the Formula (7) based on concrete damage mechanics can be obtained [34]:

\[
\sigma = E_0 (1 - D) \varepsilon
\]  

where \( \sigma \) is the nominal stress, \( \varepsilon \) is the nominal strain, \( D \) is the damage value, and \( E_0 = 3.23 \times 10^4 \) MPa.

After the concrete was heated to high temperature, its mechanical properties weakened, and the stress–strain curves transitioned from sharp to flat, indicating that peak stress, uniaxial compressive strength, and elastic modulus decreased while peak strain increased. For this reason, the temperature-softening factor \( K_T \) is introduced as shown in Formula (8):

\[
K_T = \frac{f_c(T)}{f_c^0}
\]

where \( K_T \) is the temperature softening factor; \( f_c(T) \) is the uniaxial compressive strength of concrete at high temperature, as shown in Formula (2); \( f_c^0 \) is the uniaxial compressive strength of concrete at room temperature, and \( f_c^0 = 36.15 \) MPa.

Therefore, when the influence of temperature is considered, the high-temperature damage constitutive equation for concrete can be described as follows [34]:

\[
\sigma = E_0 (1 - D) \varepsilon K_T
\]

In the loading process of the uniaxial compressive test, it was assumed that the strength of each micro-unit in concrete followed the probability distribution of \( \varphi(\varepsilon) \), and the relationship between \( \varphi(\varepsilon) \) and damage value \( D \) is shown in Formula (10) [34]:

\[
\varphi(\varepsilon) = \frac{dD}{d\varepsilon}
\]

The damage process of concrete can be divided into yield damage and fracture damage. When describing the damage evolution of concrete, the probability density of damage and fracture obey a three-parameter Weibull distribution, and the probability density of yield damage and fracture damage obedience is consistent. The formula is listed as follows [34]:

\[
\varphi(\varepsilon) = \frac{\beta}{\alpha} (\varepsilon - \gamma)^{\beta-1} \exp \left[ -\left( \frac{\varepsilon - \gamma}{\alpha} \right)^{\beta} \right]
\]

where \( \alpha \) is the scale parameter, \( \beta \) is the shape parameter, and \( \gamma \) is the position parameter.

Combining Formulas (10) and (11), the expression of damage value \( D \) is shown in the following [34]:

\[
D = \int_0^\varepsilon \varphi(x)dx = \frac{\beta}{\alpha} \int_0^\varepsilon (x - \gamma)^{\beta-1} \exp \left[ -\left( \frac{x - \gamma}{\alpha} \right)^{\beta} \right] dx = -\exp \left[ -\left( \frac{\varepsilon - \gamma}{\alpha} \right)^{\beta} \right] \bigg|_0^\varepsilon
\]

\[
= 1 - \exp \left[ -\left( \frac{\varepsilon - \gamma}{\alpha} \right)^{\beta} \right]
\]

\[
1 - D = \exp \left[ -\left( \frac{\varepsilon - \gamma}{\alpha} \right)^{\beta} \right]
\]

In addition, in order to improve the accuracy of fitting, the corresponding coefficient \( A \) is added before the right side of the equal sign of Formula (13), so Formula (14) can be obtained by combining Formulas (2), (8), (9), and (13):

\[
\sigma = E_0 \times \varepsilon \times A \times \exp \left[ -\left( \frac{x - \gamma}{\alpha} \right)^\beta \right] \times \frac{31.08989 \times e^{-\frac{T}{36.15}} + 3.99768}{1} (14)
\]

Through the regression and analysis of the test data, the values of \( A, \gamma, \alpha, \) and \( \beta \) are listed in Table 6.

Table 6. Parameters for high-temperature damage constitutive equation for concrete.

| Temperature/°C | A     | \( \gamma \) | \( \alpha \) | \( \beta \) |
|----------------|-------|-------------|-------------|---------|
| 20             | 0.0008687 | 1.258       | 0.745       | 2       |
| 100            | 0.0005413 | 1.641       | 1.015       | 2       |
| 200            | 0.0003475 | 2.973       | 1.810       | 2       |
| 400            | 0.0003178 | 3.701       | 2.092       | 2       |
| 600            | 0.0001827 | 5.668       | 4.242       | 2       |
| 800            | 0.0001223 | 8.011       | 5.920       | 2       |

It can be seen from Table 6 that the value of \( A \) gradually decreases with the increase in temperature, while the values of \( \gamma \) and \( \alpha \) gradually increase with the increase in temperature. In order to get the high-temperature damage constitutive equation for concrete at different heated temperatures besides 20 °C, 100 °C, 200 °C, 400 °C, 600 °C, and 800 °C, Formula (15) is obtained after fitting the relationship of values \( A, \gamma, \alpha \) and temperature.

\[
\begin{align*}
A &= 0.00828 \times T^{-0.02738} - 0.00676 \\
\gamma &= 0.00142 \times T^{1.26} + 1.318 \\
\alpha &= 0.00014 \times T^{1.568} + 0.8194
\end{align*} (15)
\]

Figure 7 shows the test results and the theoretical results from the high-temperature damage constitutive equation for concrete. It can be seen that the test data are close to the theoretical data. In the process of fitting by MATLAB, when the temperatures are 20 °C, 100 °C, 200 °C, 400 °C, 600 °C, and 800 °C, its R-square reaches 0.9851, 0.9930, 0.9914, 0.9945, 0.9955, and 0.9862 respectively. To sum up, the theoretical data calculated from the high-temperature damage constitutive equation is very close to the test data, which means the high-temperature damage constitutive equation can be used for reference in subsequent analysis.

3.6. Shear Strength

Shear strength is the basic mechanical property of concrete. In this paper, a shear test was carried out on concrete after being heated to a high temperature, and the test results are shown in Table 7.

Table 7. Shear strength of concrete after different heated temperatures.

| Temperature/°C | 20   | 100  | 200  | 400  | 600  | 800  |
|----------------|------|------|------|------|------|------|
| Shear strength/MPa | 4.36 | 4.25 | 4.79 | 2.80 | 2.01 | 1.63 |

In addition, in order to express the relationship between shear strength and the temperature of the concrete, MATLAB was used to fit the data. The fitting Formula (16) was obtained, and the fitting results are shown in Figure 8.

\[
\tau(T) = 7.184e^{-0.00203T} - 3.153e^{-0.00917T} (16)
\]
It can be seen from Figure 8 that the shear strength of concrete decreases with the increase in temperature, mainly because the hydration of cement in concrete has been basically completed during the 28-day curing period. With the increase in temperature, concrete gradually dehydrates. Due to the different thermal expansion performances of the concrete's internal composition, thermal stress is generated inside the concrete and can lead to cracks, so the shear strength of concrete decreases gradually.
4. Temperature Field and Mechanical Properties of a Ballastless Track Structure in a Tunnel during a Fire

During a fire, different positions of a ballastless track structure in a tunnel will have different temperatures. In order to obtain the temperature field, the CFD numerical simulation software FLUENT was used.

4.1. The Geometric Dimensions of the Model

A numerical model was built. The total length of the tunnel was 100 m, the thickness and the inner diameter of the tunnel lining were 0.3 m and 4.9 m, respectively, and there were 7 carriages, each of which was $12 \times 3 \times 4$ m$^3$ in size, and the carriages were 1 m apart [35]. The substructure of this model consists of steel rail and a cast-in-place C35 concrete track bed. The cross-section width of the track bed was 2.8 m and the thickness was 0.3 m [36]. Figure 9 shows screenshots of the model, including the cross section, overall view, and carriage layout. In the model, the X, Y, and Z axes represents the width, thickness, and longitudinal direction of the bed, respectively.

![Screenshot of the model: (a) cross section of the model; (b) overall view of the model; (c) carriage layout of the model.](image)

4.2. Initial and Boundary Conditions of the Model

In this model, the grid size was $0.1 \times 0.1 \times 0.1$ m within 20 m before and after the fire source, and the grid size was $0.4 \times 0.1 \times 0.1$ m at other locations. It was assumed that in the initial state before the fire, the temperature of each element inside and outside the model was the same and equal to the temperature of the external environment, and the initial air pressure was a standard atmospheric pressure. In the model, the inlet boundary condition was the velocity inlet boundary, the outlet boundary condition was the pressure outlet boundary, the wind speed was set as 2 m/s, the heated boundary had heat convection and heat radiation, and the non-heated boundary only had heat radiation. In addition, the density of the concrete material was 2400 kg/m$^3$, the specific heat capacity was 1100 J/(kg·K), and the thermal conductivity was 1.2 W/(m·K) [37], and this paper only carried out a simulation calculation for a designed fire of 15 MW [38]. It was assumed that the air inlet and outlet were respectively in the left and the right end of the tunnel, and the fire location was assumed to be in the third carriage from left to right.

4.3. Temperature Field of a Ballastless Track Structure in a Tunnel during a Fire

The temperature cloud of the tunnel vault along the longitudinal direction is shown in Figure 10, and the comparison of the temperature range of the tunnel vault along the longitudinal direction between the reference [38] and the model in this paper is shown in Figure 11. It can be seen from Figure 11 that the temperature distribution of the tunnel vault along the longitudinal direction in the model was similar to the result from the reference [38]. Moreover, the temperature was the highest in the burning compartment, and the temperature near the two ends of the tunnel hole was lower. As for the specific
value of the tunnel vault temperature, the maximum temperature value simulated in this paper and in the reference [38] was 275.54 °C and 255.58 °C, respectively, both of which are similar, so the correctness of the model in this paper is verified.

![Temperature cloud of the tunnel vault along the longitudinal direction.](image1)

**Figure 10.** Temperature cloud of the tunnel vault along the longitudinal direction.

![Comparison of the temperature range in the tunnel vault along the longitudinal direction between the reference [38] and the model in this paper.](image2)

**Figure 11.** Comparison of the temperature range in the tunnel vault along the longitudinal direction between the reference [38] and the model in this paper.

The overall temperature of the concrete track bed is shown in Figure 12. As can be seen from Figure 12, the temperature was the highest in the burning compartment, which was about 463.97 °C, and the lower temperatures were near the two ends of the tunnel hole; the temperature stratification is obvious in different locations.
In addition, the overall temperature field of the cast-in-place C35 concrete track bed was symmetrical with respect to the X axis (width) of the track bed, so half of it was taken for study in the following. The temperature distribution at different positions in the track bed are shown in Figure 13. There is little difference in the temperature along the depth of the cast-in-place C35 concrete track bed. The highest temperatures at the positions of surface, middle, and bottom were 475.73 °C, 474.79 °C, and 472.63 °C, respectively, and the lowest temperatures were 39.24 °C, 39.23 °C, and 39.22 °C, respectively. However, the temperature variations were obvious along the width and longitudinal direction.

![Temperature Distribution](image)

**Figure 12.** Overall temperature of the concrete track bed.

In addition, the overall temperature field of the cast-in-place C35 concrete track bed was symmetrical with respect to the X axis (width) of the track bed, so half of it was taken for study in the following. The temperature distribution at different positions in the track bed are shown in Figure 13. There is little difference in the temperature along the depth of the cast-in-place C35 concrete track bed. The highest temperatures at the positions of surface, middle, and bottom were 475.73 °C, 474.79 °C, and 472.63 °C, respectively, and the lowest temperatures were 39.24 °C, 39.23 °C, and 39.22 °C, respectively. However, the temperature variations were obvious along the width and longitudinal direction.

![Temperature at Different Positions](image)

**Figure 13.** Temperature at different positions in the track bed: (a) surface; (b) middle; (c) bottom.

4.4. Mechanical Properties of a Ballastless Track Structure at Different Positions after a Tunnel Fire

Combined with Figure 13 and the lab test results, the mechanical properties at different positions of the track bed are plotted in Figures 14–16.

It can be seen from Figures 14–16 that the mechanical properties of cast-in-place C35 concrete track bed are also different at different positions due to temperature variations. The compressive strength, shear strength, and elastic modulus of the track bed along the longitudinal direction show a V-shaped distribution, while the damage value shows an inverted V-shaped distribution. At the surface, middle, and bottom of the track bed, the smallest compressive strength was 13.37 MPa, 13.39 MPa, and 13.44 MPa, respectively; the smallest shear strength was 2.69 MPa, 2.70 MPa, and 2.71 MPa, respectively; and the smallest elastic modulus was 0.66 MPa, 0.66 MPa, and 0.67 MPa, respectively. The
biggest damage value $D_1(T)$ and $D_2(T)$ at the surface, middle, and bottom of the track bed were 0.63 and 0.79, respectively. The higher the temperature, the lower the uniaxial compressive strength, shear strength, and elastic modulus, while the greater the damage value. Therefore, according to the high-temperature damage constitutive equation for concrete and the temperature field of the track bed, the strength and damage of the track bed at different positions after a tunnel fire can be accurately obtained, which can provide a theoretical basis for post-disaster assessment and reinforcement of the track structure.

Figure 14. Uniaxial compressive strength and shear strength at different positions in the track bed: (a) surface; (b) middle; (c) bottom.

Figure 15. Elastic modulus at different positions in the track bed: (a) surface; (b) middle; (c) bottom.

Figure 16. Damage value at different positions in the track bed: (a) surface; (b) middle; (c) bottom.
5. Conclusions

- Taking 20 °C as the benchmark, when the temperature reaches 100 °C, 200 °C, 400 °C, 600 °C, or 800 °C, the uniaxial compressive strength, elastic modulus, peak strain, and shear strength of concrete changes after the increase in temperature as follows: the uniaxial compressive strength decreases by 34.00%, 36.85%, 49.68%, 71.09%, or 79.70%, respectively; the elastic modulus decreases by 43.03%, 46.44%, 71.21%, 81.42%, or 91.95%, respectively; the peak strain increases by 44.17%, 112.27%, 223.31%, 357.67%, or 479.75%, respectively; and the shear strength decreases by 2.52%, increases by 9.86%, or decreases by 35.78%, 53.90%, or 62.61%, respectively.
- Based on the mechanical testing of concrete after exposure to high temperatures, the high-temperature damage constitutive equation for concrete was established, and the stress-strain curves of concrete under uniaxial compressive after exposure to high temperatures were accurately simulated.
- FLUENT was used to simulate the temperature field of the cast-in-place C35 concrete track bed when a fire occurs in a tunnel. The surface of the track bed near the fire carriage had the highest temperature. Combined with the high-temperature damage constitutive equation for concrete and the formula for damage fitting, the strength and damage values of the track bed slab at different positions after a tunnel fire can be obtained quickly and accurately, which can provide a theoretical basis for post-disaster assessment and reinforcement of the track structure.

Author Contributions: H.M.: Conceptualization, writing—original draft. W.C.: Conceptualization, writing—review and editing, resources. X.L.: Data curation, validation. Q.X.: Numerical simulation, data curation. P.L.: Formal analysis. C.T.: Methodology. All authors have read and agreed to the published version of the manuscript.

Funding: This work was supported by the Research Foundation of the Education Bureau of Hunan Province (19K100) and National Natural Science Foundation of China (51978673). The constructive comments by anonymous reviewers are appreciated.

Institutional Review Board Statement: Not applicable.

Informed Consent Statement: Not applicable.

Data Availability Statement: Not applicable.

Conflicts of Interest: The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

References

1. Husem, M. The effects of high temperature on compressive and flexural strengths of ordinary and high-performance concrete. Fire Saf. J. 2006, 41, 155–163. [CrossRef]
2. Gao, D.Y.; Zhao, L.P.; Chen, G. Flexural behavior of fiber and nanoparticle reinforced concrete at high temperatures. Fire Mater. 2018, 42, 725–740. [CrossRef]
3. Lee, J.; Xi, Y.P.; Willam, K. Properties of concrete after high temperature heating and cooling. Aci Mater. J. 2008, 105, 334–341.
4. Lee, J.S.; Xi, Y.; Willam, K. Strength and stiffness of concrete under heating and cooling treatments. In Proceedings of the Proceedings of the 6th International Conference on Fracture Mechanics for Concrete and Concrete Structures, Catania, Italy, 17–22 June 2007.
5. Memon, S.A.; Shan, S.F.A.; Khushnood, R.A. Durability of sustainable concrete subjected to elevated temperature—A review. Constr. Build. Mater. 2019, 199, 435–455. [CrossRef]
6. Rozsypalova, I.; Danek, P.; Simonova, H. A Pilot Study of Methods for Measuring the Residual Properties of Concrete Exposed to Elevated Temperatures. Mater. Tehnol. 2018, 52, 243–252. [CrossRef]
7. Wroblewska, J.; Kowalski, R. Assessing concrete strength in fire-damaged structures. Constr. Build. Mater. 2020, 254, 9.
8. Huo, J.S.; He, Y.M.; Xiao, L.P. Experimental study on dynamic behaviours of concrete after exposure to high temperatures up to 700 A degrees C. Mater. Struct. 2013, 46, 255–265. [CrossRef]
9. Toric, N.; Boko, I.; Peros, B. Degradation of mechanical properties of high-strength concrete after exposure to fire. Gradevinar 2011, 63, 1033–1041.
10. Xu, Z.S.; Xie, B.C.; Duan, X.W. Experimental Research on Fire Damage of Tunnel Lining Concrete Test Blocks. Prog. Saf. Sci. Technol. 2010, 8, 690–696.
11. Neuenschwander, M.; Knobloch, M.; Fontana, M. Suitability of the damage-plasticity modelling concept for concrete at elevated temperatures: Experimental validation with uniaxial cyclic compression tests. Cem. Concr. Res. 2016, 79, 57–75. [CrossRef]
12. Rafi, M.M.; Aziz, T.; Lodi, S.H. Effects of Elevated Temperatures on Residual Properties of Low-Strength Concrete. J. Test. Eval. 2019, 47, 3448–3469. [CrossRef]
13. Le, Q.X.; Torero, J.L.; Dao, V.T.N. Stress-strain-temperature relationship for concrete. Fire Saf. J. 2021, 120, 6. [CrossRef]
14. Sukontasukkul, P.; Boonchoo, N.; Jongvivatsakul, P. Hybrid effect of carbon nanotubes and polypropylene fibers on mechanical properties and fire resistance of cement mortar. Constr. Build. Mater. 2021, 275, 122189. [CrossRef]
15. Sukontasukkul, P.; Pomchiengpin, W.; Songpiriyakij, S. Post-crack (or post-peak) flexural response and toughness of fiber reinforced concrete after exposure to high temperature. Constr. Build. Mater. 2010, 24, 1967–1974. [CrossRef]
16. Sukontasukkul, P.; Nimityongskul, P.; Mindess, S. Effect of loading rate on damage of concrete. Cem. Concr. Res. 2004, 34, 2127–2134. [CrossRef]
17. Ji, J.; Tian, T.T.; Gao, Z.H. Numerical Investigation on the Influence of Length-Width Ratio of Fire Source on the Smoke Movement and Temperature Distribution in Tunnel Fires. Fire Technol. 2019, 55, 963–979. [CrossRef]
18. Liu, B.; Mao, J.; Xi, Y.H. Effects of altitude on smoke movement velocity and longitudinal temperature distribution in tunnel fires. Tunn. Undergr. Space Technol. 2021, 112, 10. [CrossRef]
19. Lu, G.D.; Qu, Z.M.; Li, J.X. Numerical Investigation on Tunnel Fire Smoke Flow. Procedia Eng. 2014, 71, 390–396. [CrossRef]
20. He, G.H.; Gong, Y.J.; Tang, H. Tunnel fire simulation based on FDS. In Proceedings of the 7th International Conference on Fluid Power Transmission and Control (ICFP 2009), Hangzhou, China, 7–10 April 2009.
21. Li, J.M.; Liu, X.; Fu, C.Y. Numerical Simulation on Fire Environment in Urban Traffic Tunnel. Prog. Saf. Sci. Technol. 2010, 8, 632–638.
22. Qu, Z.M.; Wang, X.L. 3-D Numerical Simulation of Smoke Flow during Tunnel Fire. In Proceedings of the 2009 International Conference on Measuring Technology and Mechatronics Automation, Zhangjiajie, China, 11–12 April 2009.
23. Lu, G.D.; Qu, Z.M.; Li, J.X. Numerical Investigation on Tunnel Fire Smoke Flow. Prog. Saf. Sci. Technol. 2008, 7, 653–657.
24. Lu, K.H.; Xia, K.H.; Shi, C.L. Investigation on the Tunnel Curvature Effect upon the Ceiling Temperature of Tunnel Fires: A Numerical Simulation. Fire Technol. 2021, 57, 2839–2858. [CrossRef]
25. Li, Z.; Luo, Y.; Gao, Y.; Yang, X.; Guo, H.; Zhang, Y. Experimental and Theoretical Studies of the Effects of Fire Location on the Smoke Temperature Distribution in a Branched Tunnel. Fire Technol. 2022, 58, 1265–1284. [CrossRef]
26. Qu, Z.M.; Ma, X.Y. Numerical Simulation of Smoke Flow and its Effect during Railway Tunnel Fire. In Key Engineering Materials; Trans Tech Publications Ltd.: Wollerau, Switzerland, 2010.
27. Sukontasukkul, P.; Jamnam, S.; Sappakittipakorn, M. Residual flexural behavior of fiber reinforced concrete after heating. Mater. Struct. 2018, 51, 98. [CrossRef]
28. Shen, Q.C.; Chen, W.; Liu, C.; Zou, W.J. The Tensile Strength and Damage Characteristic of Two Types of Concrete and Their Interface. Materials 2020, 13, 16. [CrossRef]
29. Li, J.B.; Li, X.; Wang, Y.; Yin, T.B. Influence of temperature shock on dynamic tensile mechanical properties of granite. Gold Sci. Technol. 2021, 29, 545–554.
30. Chen, Z.P.; Wang, H.H.; Chen, Y.L. Experimental study on mechanical properties of concrete after high temperature. Concrete 2015, 1, 13–17. (In Chinese)
31. Lv, T.Q.; Zhao, G.F.; Lin, Z.S. Experimental study on mechanical properties of statically placed concrete after high temperature. J. Build. Struct. 2004, 1, 63–70. (In Chinese)
32. Luo, Y.S.; Chen, C.; Tang, S.H. Testal study on compressive strength of concrete under high temperature. J. Nat. Sci. Xiangtan Univ. 2013, 35, 30–34+40. (In Chinese)
33. Jiang, L.H. Testal Study on Mechanical Properties of High Temperature Concrete; University of Shanghai for Science and Technology: Shanghai, China, 2011. (In Chinese)
34. Jia, B.; Yang, F.; Tao, J.L. High temperature mechanical properties and constitutive equations of concrete. Concrete 2014, 2, 25–28+32. (In Chinese)
35. Chen, C.K.; Kang, H. Numerical simulation of fire temperature field in heavy-duty railway tunnels under conditions of extremely rich combustibles. Fire Sci. 2013, 22, 24–30. (In Chinese)
36. Sun, X.P. Analysis of Cracking Mechanism of Ballast Slab Concrete; Central South University: Changsha, China, 2010. (In Chinese)
37. Wang, S.Y. Experimental and Numerical Simulation Research on Temperature Distribution of Tunnel Fire. In Proceedings of the 2011 Academic Annual Conference of China Occupational Safety and Health Association, Liuzhou, China, 19–20 October 2011; pp. 371–377. (In Chinese)