DE GRUYTER OPEN

DOI: 10.1515/ace-2015-0013

ARCHIVES OF CIVIL ENGINEERING
Vol. LXI ISSUE 2 2015



# UNIAXIAL DYNAMIC MECHANICAL PROPERTIES OF TUNNEL LINING CONCRETE UNDER MODERATE-LOW STRAIN RATE AFTER HIGH TEMPERATURE

**L. X. XIONG**[1]

To investigate the mechanical properties of tunnel lining concrete under different moderate-low strain rates after high temperatures, uniaxial compression tests in association with ultrasonic tests were performed. Test results show that the ultrasonic wave velocity and mass loss of concrete specimen begin to sharply drop after high temperatures of 600 ℃ and 400 ℃, respectively, at the strain rates of 10-5s-1 to 10-2s-1. The compressive strength and elastic modulus of specimen increase with increasing strain rate after the same temperature, but it is difficult to obtain an evident change law of peak strain with increasing strain rate. The compressive strength of concrete specimen decreases first, and then increases, but decreases again in the temperatures ranging from room temperature to 800 C at the strain rates of 10-5s-1 to 10-2s-1. It can be observed that the strain-rate sensitivity of compressive strength of specimen increases with increasing temperature. In addition, the peak strain also increases but the elastic modulus decreases substantially with increasing temperature under the same strain rate.

*Keywords*: concrete, high temperature, strain rate, mechanical properties

## 1. INTRODUCTION

Fire resistance is one of the main issues that must be considered in tunnel design. A number of tunnel fire accidents have shown that fire can result in extensive and severe damages to concrete tunnel linings (Schrefler et al. [1]).

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These damages involve mechanical properties degradation, thickness reduction of linings due to spalling, and generation of local stresses due to no uniform thermal strains, which can severely reduce the concrete lining safety, threaten future safe operation and even cause the tunnel lining collapse (Yan *et al*. [2]). Therefore, the study on the mechanical damage behaviour of lining concrete in fire has both theoretical and practical significance on improvement of tunnel lining's fire safety.

Many researchers have conducted tests on the post-fire mechanical properties of concrete. Chan *et al*. [3] carried out an experiment which studied the mechanical properties and pore structure of high-performance concrete and normal-strength concrete after exposure to high temperatures. Poon *et al*. [4] compared the strength and durability performance of normal-strength and high-strength pozzolanic concretes with silica fume, fly ash, and blast furnace slag at elevated temperatures of up to 800℃. Li *et al*. [5] investigated the mechanical properties of normal-strength concrete and high-strength concrete after high temperatures. Xiao & Konig [6] summarized the states-of-the-art studies on the mechanical behaviors of concrete at high temperature in China. Chang *et al*. [7] performed an experimental investigation on the complete compressive stress–strain relationship for concrete after heating to temperatures of 100-800 ℃. Husem [8] examined the variation of compressive and flexural strengths of ordinary and high-performance micro-concrete at high temperatures. Chen *et al*. [9] investigated the effects of curing age, type of cooling and the maximum temperature on the relative recovered strengths of concrete after exposure to high temperatures. He & Song [10] performed multiaxial tensile-compressive tests on cubic specimens of plain high-performance concrete (HPC) at various different stress ratios after the tested specimens were exposed to elevated temperatures. Tai *et al*. [11] presented a series of experiments which investigated the mechanical properties of RPCs after exposure to high temperatures. Zheng *et al*. [12] performed an experimental study on the complete compressive stress–strain relationships for reactive powder concrete (RPC) with various steel fiber contents after exposure to 20-900 ℃, Recently, Luigi *et al*. [13] described the consequences of progressive damage in architectural high performance concrete when exposed to different heat treatments. Marques *et al*. [14] investigated the effects of elevated temperatures on the residual mechanical performance of concrete produced with recycled rubber aggregate. More recently, Tahir [15] investigated the influence of the specimen size on the behaviour of engineered cementations composite specimens exposed to elevated temperatures. The above literature survey shows that, despite the large number of publications on the mechanical properties of concrete during or after its exposure to elevated temperatures, little research work has been focused on the dynamic mechanical properties of concrete during or after its exposure to elevated temperatures.

For many concrete structures such as tunnels and buildings, it is expected that they could continue to be in service after a fire. These structures may also encounter seismic loads during the follow-up service lifetime. Thus, it is necessary to study the dynamic mechanical properties of concrete when or after they are exposed to elevated temperatures.

Currently, there are few researchers who studied the dynamic mechanical properties of concrete after high temperature exposure.. Li *et al*. [16] carried out quasi-static and impact loading experiments on concrete before and after exposure to a high temperature. Huo *et al*. [17] carried out impact tests using a SHPB bar to experimentally study the dynamic behaviours of concrete after exposure to high temperatures. Su *et al*. [18] studied the dynamic compressive mechanical properties of concrete at elevated temperatures using a self-designed high temperature SHPB apparatus. The strain rates used in these tests weremuch higher, normally about $30s^{-1}$ to $200s^{-1}$. These tests mainly examined the dynamic mechanical properties of concrete under impact or blast loading. However, for seismic loading the strain rate is usually between $10^{-3}s^{-1}$ and $10^{-2}s^{-1}$ for studying the dynamic performance of concrete (Shi *et al*. [19]).

In this study, ultrasonic test and uniaxial compression test were performed to investigate the mechanical properties of concrete under different moderate strain rates after high temperature exposure. The strain rates adopted in the tests were between $10^{-5}s^{-1}$ to $10^{-2}s^{-1}$.

## 2. EXPERIMENTAL PROGRAM

### 2.1. SPECIMEN PREPARATION

The compressive strength grade of concrete specimens used is C40. The mean value of compressive strength of C40 concrete after curing for 28 days is 42.5MPa. The size of concrete specimens is 100 ×100 ×100 $mm^3$. A total of 64 specimens were modelled and tested.

No. 425 cement employed in this test was normal cement produced by the China Building Materials Academy. The ISO standard sand was used in accordance with the Chinese National Standard, and its particle size ranges from 0.5 mm to 1.0 mm. The particle size of coarse aggregate ranges from 5 mm to 20 mm. Table 1 shows the mixtures of the concrete specimens.

Table 1. Mixtures of concrete specimens

| Cement ($kg/m^3$) | Water ($kg/m^3$) | Sand ($kg/m^3$) | Coarse aggregate ($kg/m^3$) |
|---|---|---|---|
| 382 | 141.34 | 637.94 | 958.82 |

### 2.2. Tables test procedure

The ultrasonic wave velocities of specimens were measured using ultrasonic detector after being properly cured for 28 days. Then these specimens were elevated to the peak temperatures of 200, 400, 600, and 800℃ at the heating rate of 20℃/min, respectively. After the peak temperature was reached, it was maintained for another 2 h; then the specimens were cooled down to room temperature in the furnace and then taken out for testing.

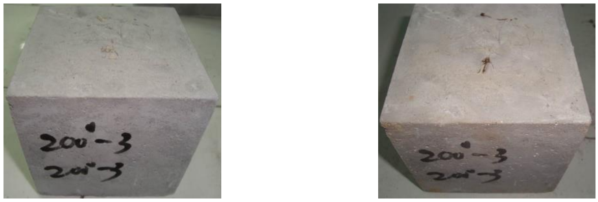
The ultrasonic wave velocities of these specimens were measured after 7 days, and then uniaxial compression tests were performed using WAW-600C universal testing machine. The maximum loading rate of the testing machine is 60 mm/min, and the maximum compression load is 600 kN Four strain rates were used in the uniaxial compression tests, which are $10^{-2}s^{-1}$, $10^{-3}s^{-1}$, $10^{-4}s^{-1}$ and $10^{-5}s^{-1}$.

Three specimens were tested for each temperature and strain rate. The results of compressive strength, peak strain and elastic modulus are the average values of the three tested specimens, which will be discussed in detail in following section.

## 3. Test results and analysis

### 3.1. The change of appearance characteristics of specimens

The changes of appearance characteristics of specimens after 200, 400, 600, and 800℃ are shown in Figs. 1-4.



(a) before high temperature (b) after high temperature

Fig.1. The change of appearance characteristics of specimen after 200℃

(a) before high temperature (b) after high temperature

Fig.2. The change of appearance characteristics of specimen after 400℃

(a) before high temperature (b) after high temperature

Fig.3. The change of appearance characteristics of specimen after 600℃

(a) before high temperature (b) after high temperature

Fig.4. The change of appearance characteristics of specimen after 800℃

The number of specimens which experienced high temperature, and the number of specimens which had surface cracks after each temperature are presented in Table 2.

Table 2. The number of samples before high temperature and the number of samples which had surface cracks after high temperature

| Temperature(℃) | Number of samples before high temperature | Number of samples having surface cracks after high temperature |
|---|---|---|
| 200 | 12 | 0 |
| 400 | 12 | 1 |
| 600 | 12 | 6 |
| 800 | 16 | 16 |

The specimens after 200 ℃ have no apparent cracks; only one of the specimens after 400 ℃ has apparent cracks; about half of the specimens after 600 ℃ have apparent cracks; all the specimens after 800 ℃ have apparent cracks.

### 3.2. THE CHANGE OF ULTRASONIC WAVE VELOCITIES OF SPECIMENS

The distribution of ultrasonic wave velocities of specimens before being exposed to high temperatures is listed in Table 3.

Table 3. The distribution of ultrasonic wave velocities of specimens before high temperature

| Ultrasonic wave velocity range (m/s) | Number of specimen |
|---|---|
| 4700-4800 | 5 |
| 4800-4900 | 21 |
| 4900-5000 | 27 |
| 5000-5100 | 10 |
| 5200-5300 | 1 |

There were 64 specimens before high temperature in total, and the number of specimens with ultrasonic wave velocities ranging from about 4800 m/s to 5000 m/s accounted for 75% of the total samples. Therefore, it can be noted that minor difference in the specimens is observed.

In contrast, the ultrasonic wave velocities of the specimens before and after heating is shown in Fig. 5.

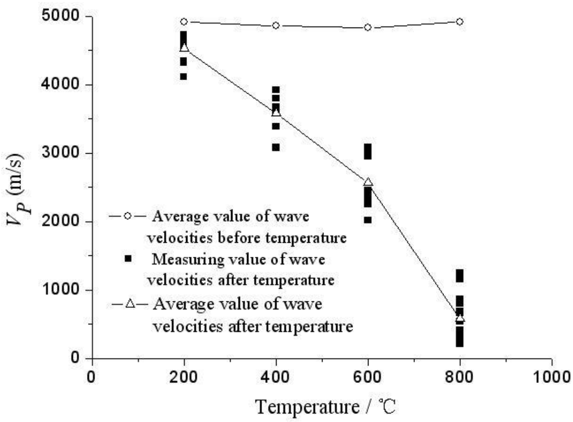



Fig. 5. The contrast of ultrasonic wave velocities of specimens before and after high temperature

It can be seen from the figure that, from room temperature to 200℃, the variation of the ultrasonic wave velocity of specimen is not evident. From 200 ℃ to 400 ℃, the decreasing extent of the ultrasonic wave velocity of specimen is similar to that from 400℃ to 600℃. While, the decreasing extent of the ultrasonic wave velocity of specimen from 600 ℃ to 800 ℃ is the greatest.

The damage index of concrete exposed to high temperature can be given as the function of ultrasonic wave velocity:

$$D = 1 - \left(1 - \frac{V_{PT}}{V_P}\right)^2 \tag{3.1}$$

Where $D$ is the damage index, $V_P$ is the initial ultrasonic wave velocity of specimen before being exposed to high temperature, and $v_t$ is the ultrasonic wave velocity of specimen after being exposed to high temperature.

The variation of damage index associated with temperature is shown in Fig. 6.

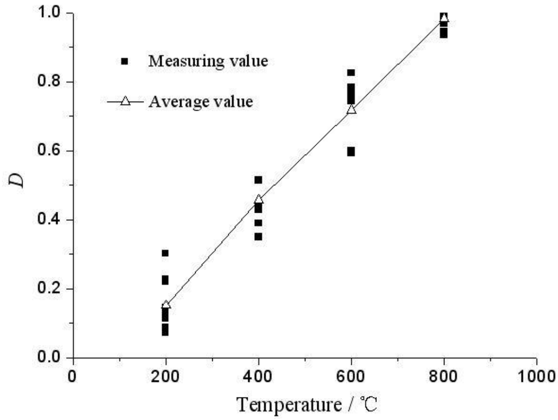



Fig. 6. The evolution of damage index

It can be seen from Fig.6 that, the damage index gradually increases linearly with increasing temperature. The values of damage index after being exposed to high temperatures of 200, 400, 600, and 800℃ are 0.15127, 0.45675, 0.71802 and 0.98412, respectively.

## 3.3. THE CHANGE OF MASS LOSS

The mass loss of specimen can be defined as

(3.2) $$M_c = \frac{m_0 - m_T}{m_0} \times 100\%$$

Where $M_c$ is the mass loss, $m_0$ is the initial mass of specimen before exposed to high temperature, and mT is the mass of specimen after exposed to high temperature.
The evolution of mass loss with the temperature is shown in Fig. 7.

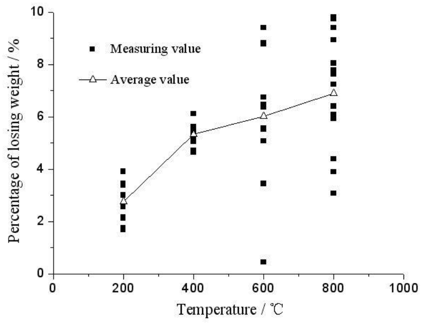
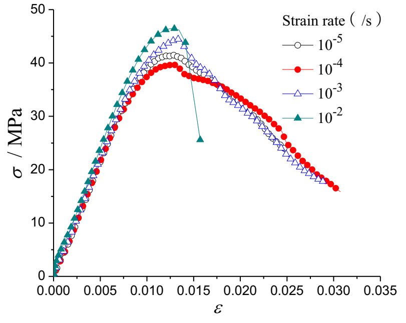



Fig. 7. The evolution of mass loss of concrete specimen with temperature

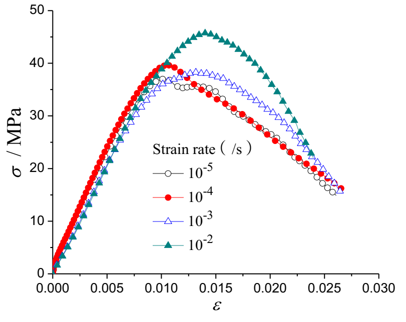
It can be seen from the figure that, from room temperature to 400℃, the mass loss increases rapidly with increasing temperature. In contrast, from 400℃ to 800℃, the mass loss increases slowly with increasing temperature.

### 3.4. THE VARIATION OF STRESS-STRAIN CURVES

The influence of strain rate on the stress-strain curve of the specimen for the same temperature is displayed in Fig. 8.

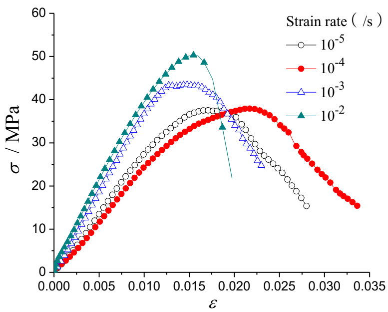



(a)Under room temperature

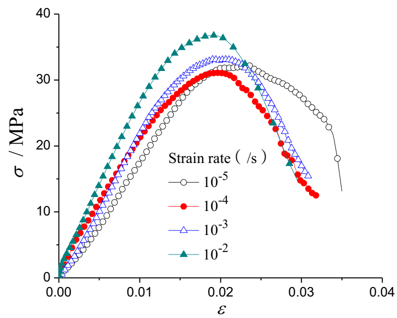



(b) Under 200℃

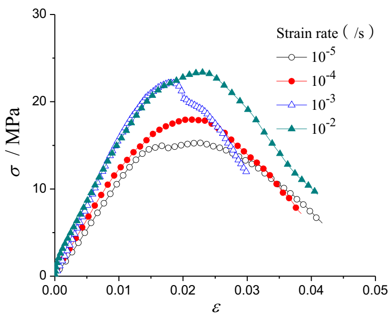
Fig. 8. The influence of strain rate on the stress-strain curve of specimen when the temperature is the same




(c) Under 400℃




(d) Under 600℃

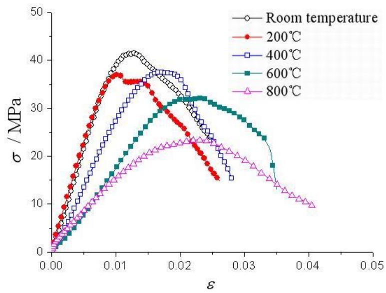



(e) Under 600℃

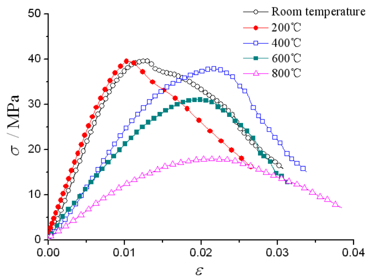
Fig. 8. The influence of strain rate on the stress-strain curve of specimen when the temperature is the same - continued

The peak stress of specimen increases substantially with increasing strain rate regardless of the temperature experienced according to Fig.8. The slopes of softening stages of stress-strain curves of not-heated specimen and specimens after high temperature of 200℃ and 400℃, under strain rate of $10^{-2}s^{-1}$ are higher than those under other strain rates. The slopes of softening stages of stress-strain curves of specimens after high temperature of 600℃ and 800℃ under strain rate of $10^{-2}s^{-1}$ are similar to those under other strain rates, and are relatively flat. This suggests that the ductility of the specimen is improved at the temperatures of 600℃ and 800℃.

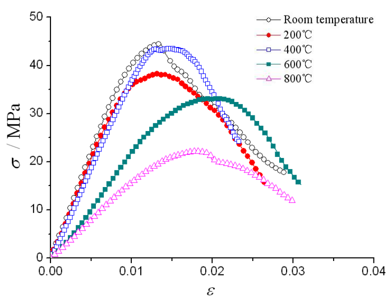
The influence of temperature on the stress-strain curve of the specimens when the strain rate is the same is shown in Fig. 9.



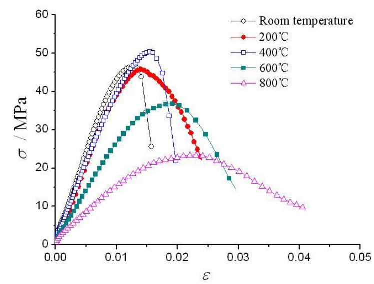

(a) Strain rate of $10^{-5}$ $s^{-1}$




(b) Strain rate of $10^{-4}$ $s^{-1}$




(c) Strain rate of $10^{-3}$ $s^{-1}$




(d) Strain rate of $10^{-2}$ $s^{-1}$

Fig. 9. The influence of temperature on the stress-strain curve of specimen when the strain rate is the same

## 3.5. THE EVOLUTION OF COMPRESSIVE STRENGTH

The evolution of the compressive strength is illustrated in Fig. 10.

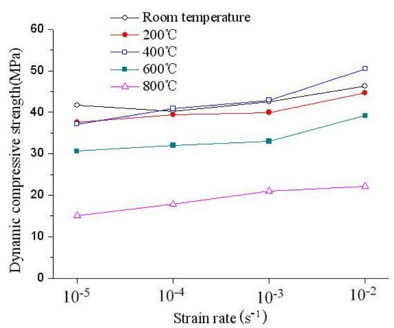



Fig. 10. The evolution of compressive strength

The compressive strength of the specimens decreases from room temperature to 200℃ under the same strain rate. However, the strength increases from 200℃ to 400℃. There is a decrease of the compressive strength to larger extent from 400℃ to 800℃.

The dynamic increase factor (DIF) for the compressive strength could be defined as:

$$DIF = f_{cd} / f_{cs} \tag{3.3}$$

where $DIF$ is the dynamic increase factor, $f_{cd}$ is the compressive strength of concrete specimen at current strain rate, and $f_{cs}$ is the compressive strength of concrete specimen at quasi-static strain rate. The quasi-static strain rate is taken as $10^{-5}s^{-1}$ in this study.

The relationship between the dynamic increase factor (DIF) and the current strain rate can be described by

$$DIF = a + b\log_{10}(\dot{\varepsilon} / \dot{\varepsilon}_0) \tag{3.4}$$

where $a$ and $b$ are the fitting parameters, and $\dot{\varepsilon}$ is the current strain rate. $\dot{\varepsilon}_0$ is the reference strain rate and can be set as $1s^{-1}$. The parameter $b$ is the slope of the fitting line, reflecting the sensitivity of compressive strength on the strain rate.

The relationships between the dynamic increase factor (DIF) and the stain rate of specimens are shown in Fig.11.

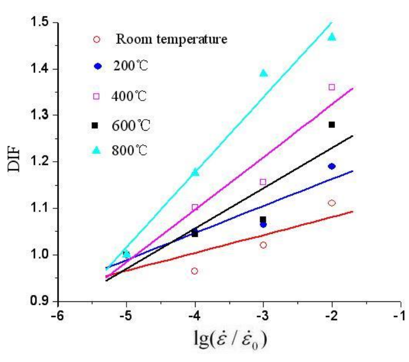



Fig. 11. Relation between strength increasing factor and stain-rate

The relations between DIF and strain rate can be described as Eq. (3.5)-(3.9).

(3.5) Room temperature: $DIF = 1.15883 + 0.0386\log_{10}(\dot{\varepsilon}/\dot{\varepsilon}_0)$

(3.6) T=200℃: $DIF = 1.27981 + 0.0582\log_{10}(\dot{\varepsilon}/\dot{\varepsilon}_0)$

(3.7) T=400℃: $DIF = 1.55005 + 0.1132\log_{10}(\dot{\varepsilon}/\dot{\varepsilon}_0)$

(3.8) T=600℃: $DIF = 1.40342 + 0.08679\log_{10}(\dot{\varepsilon}/\dot{\varepsilon}_0)$

(3.9) T=800℃: $DIF = 1.82428 + 0.16169\log_{10}(\dot{\varepsilon}/\dot{\varepsilon}_0)$

The value of $b$ increases substantially with increasing temperature. The increment of the value of $b$ is very evident from 600 ℃ to 800 ℃. The compressive strength is found to decrease with increasing temperature, thus the strain-rate sensitivity of compressive strength of specimen will increase with increasing temperature. This result is different from that attained by Li *et al*. [16].

### 3.6. THE EVOLUTION OF PEAK STRAIN

The evolution of compressive strength is shown in Fig. 12.

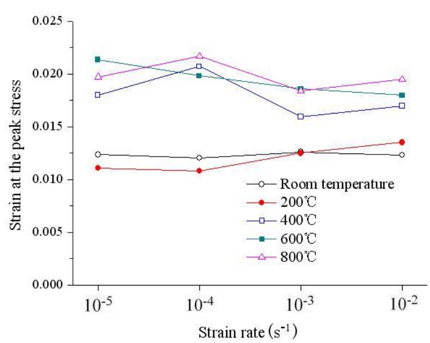



Fig. 12. The evolution of peak strain

There is slight change of peak strains of not-heated specimens and specimens after high temperature of 200℃ with increasing strain rate. The peak strain of specimens after high temperature of 400℃ decreases with increasing strain rate. The peak strains of specimens after high temperature of 600℃

and 800℃ first increase and then in general decrease with increasing strain rate. The peak strain of specimens tends to increase by increasing temperature under the same strain rate.

### 3.7. THE EVOLUTION OF ELASTIC MODULUS

The elastic modulus is calculated as a secant modulus in this study. The secant modulus is calculated from the origin to a defined point on the stress-strain curve, within 30% of the specimen's peak stress.
The evolution of elastic modulus is shown in Fig. 13.

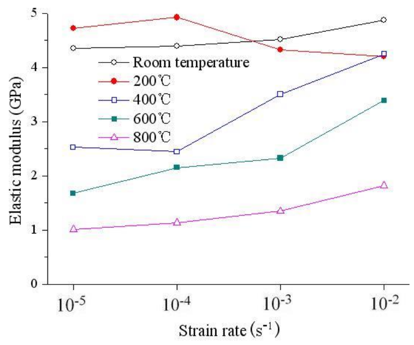



Fig. 13. The evolution of elastic modulus

The elastic modulus of specimens increases substantially with increasing strain rate after exposure to the same high temperature. However, the elastic modulus of specimens decreases substantially with increasing temperature under the same strain rate.

## 4. CONCLUSIONS

1. The surfaces of specimens after temperature of 600 ℃ begin to have lots of apparent cracks, and the ultrasonic wave velocity of specimens begins to decrease greatly after temperature of 600 ℃. The mass loss increases slowly with increasing temperature from 400℃ to 800℃.
2. The peak stress of specimens increases substantially with increasing strain rate regardless of the temperature experienced. The compressive strength of specimens decreases from room temperature to 200℃, increases from 200℃ to 400℃, and decreases again from 400℃ to 800℃

under the same strain rate. The strain-rate sensitivity of compressive strength of specimen increases with increasing temperature.

3. There is no fixed change law for peak strain of specimens with increasing strain rate after the same temperature. But the peak strain of specimens is found to increase with increasing temperature under the same strain rate.

4. The elastic modulus of specimens increases substantially with increasing strain rate after the same high temperature. However, the elastic modulus of specimens decreases substantially with increasing temperature under the same strain rate.

## 5. ACKNOWLEDGMENTS

This work was supported by the Natural Science Foundation of Zhejiang Province (Grant No. LY13E080021), the Ningbo Municipal Natural Science Foundation (Grant No.2011A610072), the Project(Grant No. XKL14D2063) Subject Program of Ningbo University, and the Open Project (Grant No. 2014ZDK009) of Guangxi Key Laboratory of Disaster Prevention and Structural Safety.

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*Received 18. 02. 2015*
*Revised 30. 04. 2015*

## LIST OF FIGURES AND TABLES:

## DYNAMICZNE WŁAŚCIWOŚCI MECHANICZNE BETONU DO OBUDOWY TUNELI W STANIE UMIARKOWANEGO - NISKIEGO NAPRĘŻENIA JEDNOOSIOWEGO PO ZASTOSOWANIU WYSOKICH TEMPERATUR

*Słowa kluczowe:* beton, wysoka temperatura, wskaźnik naprężenia, właściwości mechaniczne

**STRESZCZENIE:**

Odporność ogniowa to jedno z najistotniejszych zagadnień, jakie należy brać pod uwagę przy projektowaniu tunelu. Testowanie właściwości mechanicznych betonu po pożarze stało się, jak dotąd, przedmiotem wielu projektów badawczych. Jakkolwiek powstały liczne publikacje na temat właściwości mechanicznych betonu w trakcie oraz po poddaniu go działaniu wysokich temperatur, niewiele uwagi poświęcono jak dotąd dynamicznym właściwościom mechanicznym betonu w trakcie lub po takim oddziaływaniu.

W przypadku wielu konstrukcji betonowych takich, jak tunele czy budynki, oczekuje się, że po pożarze będą one nadal pełniły swoją funkcję. Konstrukcje te mogą być też narażone na obciążenia sejsmiczne w trakcie okresu użytkowania. Dlatego też konieczne jest studiowanie dynamicznych właściwości mechanicznych betonu w trakcie lub po ekspozycji na działanie wysokich temperatur.

Aby przeanalizować właściwości mechaniczne betonowych okładzin tunelowych w stanie zróżnicowanego naprężenia od umiarkowanego do niskiego po oddziaływaniu wysokich temperatur, przeprowadzono testy naprężenia jednoosiowego w połączeniu z badaniami ultradźwiękowymi.

Klasa wytrzymałości betonu na ściskanie w przypadku wykorzystanych próbek to C40. Próbki poddawano oddziaływaniu szczytowych temperatur, wynoszących 200, 400, 600 i 800°C.

W ramach testów naprężenia jednoosiowego zastosowano cztery wskaźniki naprężenia, tj.: 10-2s-1, 10-3s-1, 10-4s-1 oraz 10-5s-1.

Wyniki testów przeanalizowano w odniesieniu do następujących właściwości:

(1) zmiana charakterystycznych cech wyglądu próbek,

(2) zmiana prędkości fal ultradźwiękowych w próbkach,

(3) zmiana ubytku masy,

(4) zróżnicowanie krzywych naprężenia i odkształcenia,

(5) ewolucja wytrzymałości na ściskanie,

(6) ewolucja szczytowej wartości naprężenia,

(7) ewolucja modułu sprężystości.