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COMPRESSIVE STRENGTH AND DEFORMATION OF SELF-COMPACTING CONCRETE AT HIGH TEMPERATURE

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Abstract

The main research focuses on the hot properties of self-compacting concrete at high temperature. The total strain of concrete at high temperature consists of four strain components that are, instantaneous stress-induced strain, free thermal strain, transient strain and creep. The instantaneous stress-induced strain can be obtained by the compressive stress-strain curve under stabilized temperature field. It is very important to research the stress-strain relationship at high temperature for establishing the constitutive model and stress analysis of self-compacting concrete structure.

The compressive strength and deformation properties of normal strength self-compacting concrete with addition of polypropylene fiber, high strength self-compacting concrete under different temperature were studied in this paper. The relationship between the compressive strength and temperature of self-compacting concrete was obtained, and the comparison with the normal SCC and high-strength concrete was done. The relationship between stress-strain curve and temperature were studied in this paper. The functions of compressive strength, strain at peak stress, elastic modulus and strain-stress relationship were brought forward. The instantaneous stress-induced strain can be obtained by the strain-stress function, which offers a basis for the establishment of constitutive model at high temperature.

1. INTRODUCTION

Self-compacting concrete (SCC) is a type of concrete developed throughout the last 15 years. It is widely used in different applications ranging from housing to large infrastructures, such as bridges and tunnels, because it can spread into place under its own weight and fill restricted sections without the need for mechanical consolidation. As such, it improves the work environment, reduces the manpower need for casting, and increases the speed of construction and the quality of cast structures [1]. SCC is usually considered as a special type of high-performance concrete (HPC) produced with higher amounts of filler materials and lower water/binder ratios as compared with other concretes. Thus, porosity of SCC is usually reduced and the material is characterized from high diffusion resistance. This fact is responsible for the superior durability usually observed in SCC [2].

As the use of self-compacting concrete becomes common, the risk of exposing it to fire increases. There are only a few investigations published on the behaviour of SCC in fire. It has been shown that the risk of SCC fire spalling may be greater than HPC concrete due to the difference material composition. This different composition is mainly due to the higher fines content and possibly lower permeability. More research is needed in the area to determine in which cases SCC is behaving normally and in which cases there are problems for concern. It is also necessary to do more work in correlating laboratory testing with real field performance.

In order to understand and eventually predict the performance of SCC structural members, the material properties that determine the behaviour of the member at elevated temperatures must be known. The behaviour of a structural member exposed to fire is dependent, in part, on the thermal and mechanical properties of the material of which the member is composed. One of the basic mechanical properties that are required for prediction of structural performance under fire conditions is the stress-strain relationship.

2. EXPERIMENTAL PROGRAMME

2.1 Materials and specimens

The specimens were heated without restraint and loaded to failure at high temperature, when the steady-state temperature is reached within the specimen. There are two types of concrete specimens used in tests, including normal strength self-compacting concrete with the addition of polypropylene fiber (PPF), and high strength self-compacting concrete with the addition of polypropylene fiber (PPF). The size of cylinder is 150 mm diameter, 300 mm height. In all batches, the specimens are cast with normal Portland cement, natural river sand, crushed limestone with 15 mm maximum size. In order to improve workability, superplasticizer (Glenium) and limestone power are used in three batches of concrete mix. The mix proportions and mean compressive cube strengths for 90 days are included in table 1. Polypropylene fibers were used in SCCPPF batch of concrete mix. The fiber was 12 mm in length with an 18 diameter.

The specimens are de-molded one day after casting, and then cured in a moist environment at 20°C, 95% relative humidity for a period of 28 days. To reduce the difference of the water content between specimens arising from a long test period, all specimens are tested after 90 days.

2.2 Testing equipment and procedure

The testing equipment consists of three systems: the temperature control system, the loading system, the measuring and data acquisition system. It is a closed loop servo-controlled hydraulic testing machine equipped with an electric furnace. Figure 1 shows the test apparatus-loading frame with furnace and data acquisition system.

Table 1: Design of Mix Proportion (Unit: kg/m^3)

	SCCPPF1-C30	SCCPPF1-C60
Cement	310	400
Water	199	165
Sand	853	853
Coarse aggregate 4-8 mm	300	300
Coarse aggregate 8-15 mm	400	400
Limestone power	133	200
Superplasticizer (liter)	5.95	5.55
PP fibre	1	1
Water/cement ratio	0.64	0.41
Water/power ratio	0.45	0.28

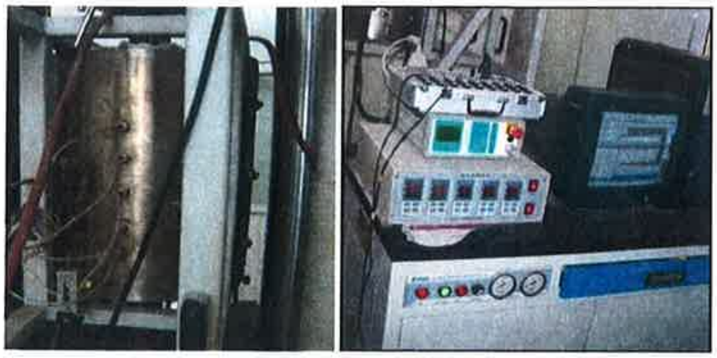
The deformations of concrete specimens were determined through measuring the displacements of the ends of the specimens, which include the displacement of steel-based alloy platens and specimens. The deformations were recorded by the two LVDTs on the steel support at the bottom of the platens. In order to obtain the deformations of the specimens, the deformations of the platens were tested separately. Because most parts of the compressive platens are at room temperature, the strength loss and creep are so small that they can be omitted. Only the thermal expansion and elastic deformation under elevated temperature and load are considered in this paper. Displacements of concrete specimens can be determined by subtracting that of the platens from the test results.

The temperature in the test specimens are measured in two points by chromel-nisiloy thermocouples and recorded continuously. One is situated at the mid section of specimen, the other is placed about 38mm from surface in the mid section.

For each type of concrete and target test temperature, two specimens were tested and the average of these results was used as the final result. The target test temperatures were determined to be 20□ (room temperature), 200□, 400□, 600□ and 800□, respectively. All tests were conducted under hot conditions and no residual strength tests were carried out.

The cylindrical specimens were heated, without any load, at a constant heating rate of 5°C/min in the furnace chamber until the furnace temperature reaches the target furnace and maintained for 6 h to attain a thermal steady condition. The specimens were then loaded to fail.

For generating data in the descending part of the stress-strain curve of concrete, a strain control technique was adopted. After the specimen reached the steady state, it was loaded under uniaxial compression until the specimen could no longer sustain the axial load.



(a) Loading frame with furnace (b) Data acquisition system

Figure 1: Furnace and data acquisition system

3. EXPERIMENTAL RESULTS AND DISCUSSION

3.1 Failure Mode

For SCCPPF1-C30 specimens, there was no difference in failure modes observed from room temperature to 200°C. When specimens heated up to 400°C, there was the obvious through crack along the longitudinal direction. With increase in temperature, the failure mode became irregular above 600°C. The central section of specimens became thinner, and the specimens failed in the surface of aggregate and cement paste or the cement paste. When specimens were heated up to 800°C, the cracks extended with increase in load. There was no obvious main crack on the surface. The total structure became loose compared to that of room temperature. The specimens showed slight ductile failure pattern when heated up to 200°C. Concrete could undergo larger strain after peak strength. The stress-strain could be obtained easily.

SCCPPF1-C60 specimens showed a very brittle type of failure at temperatures less than 400°C. It was impossible to define the complete stress-strain curves, especially in the descending portion. The specimens failed soon after reaching their peak strength. The failure surfaces are neat surfaces and pass through the broken aggregate. The end portion of the failed specimens resembled double–cone pattern (at the top and bottom). When exposed to a temperature of 600°C and 800°C, concrete specimens exhibited a gradual failure and complete stress-strain curves could be defined. These specimens failed in an irregular pattern.

Since HSC is a brittle material, it fails soon after crack propagation is initiated. If the number of voids is increased, the propagation of cracks occurs more gradually, resulting in less brittle failure. When temperature increases, the pore water is removed and more voids appear. The ductility of specimens increased with increased temperatures.

3.2 Compressive strength

The relative compressive strength of high strength SCCPPF and normal strength SCCPPF specimens at high temperature is shown in Figure 2. The relative strength is defined as the ratio of the compressive strength at a specified temperature to the compressive strength at room temperature.

The loss of strength resulting from exposure to high temperature was smaller for high strength SCCPPF specimen compared to that suffered by normal strength SCCPPF specimen. This difference is particularly noticeable in the lower temperature range of 200°C to 400°C.

For the high strength SCCPPF specimen, the loss in strength was 18 percent of the room temperature strength, the same for normal strength SCCPPF was of the order 10 percent. After the initial loss of strength, the high strength SCCPPF began to recover its strength at 400°C and reached a value which was 5 percent above the strength at 200°C. The strength of normal strength SCCPPF decreased sharply in this temperature range of 200°C to 400°C. At 400°C, the strength was only 50 percent of that at room temperature. The strength gain phase didn't occur during the heating. There was a more cement paste content for high strength SCCPPF specimen, the effect of the general stiffening of the cement gel or the increase in surface forces between gel particles on strength recovery was more obvious. Since the effect that aggregate expands is strengthened for normal strength SCCPPF specimen having a lower cement-aggregate ratio, there is probably more internal stress setup and thus a larger reduction in strength.

In temperature range from 400°C to 600°C, the percent drop rate in strength for both the normal strength and high strength SCCPPF specimen was alike. The dehydration of cement paste resulted in its gradual deterioration. The normal strength concrete had a larger strength loss. At 600°C, the compressive strength of normal strength was 34 percent of initial strength.

Above 800°C, the strength was similar for both types of SCCPPF specimens, and it was 30 percent of initial strength at room temperature. The decomposition of calcium carbonate at about 780°C caused the loss strength in concrete.

The critical temperature of strength loss is 200°C for normal strength SCCPPF, and 400°C for high strength SCCPPF. After the critical temperature, the loss in strength increased.

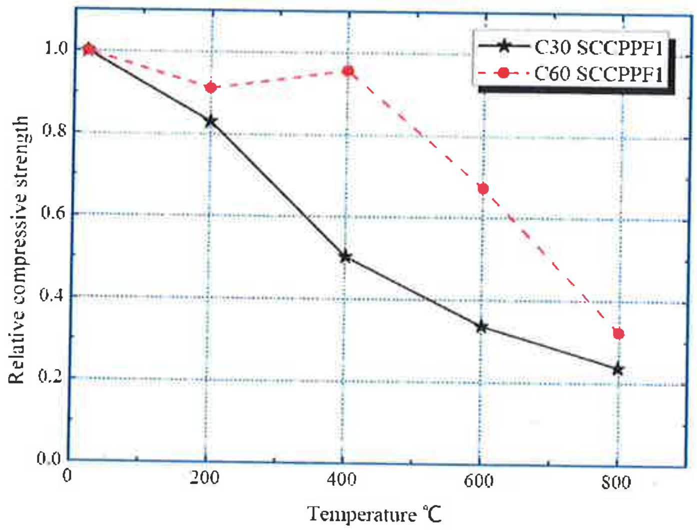


Figure 2: Relative compressive strength of SCCPPF1-C60 and SCCPPF1-C30

3.3 Strain at peak stress

For both two types of SCCPPF the strains at peak stress were almost the same up to about 600°C, as shown in Figure 3. The critical temperatures for both types of concrete were 400°C. From room temperature to 400°C, the strain at peak strain increased slightly for both types concrete. Above 600°C, the strain at peak strain increased significantly for high strength SCCPPF specimen. At 800°C, it was three to five times that of room temperature.

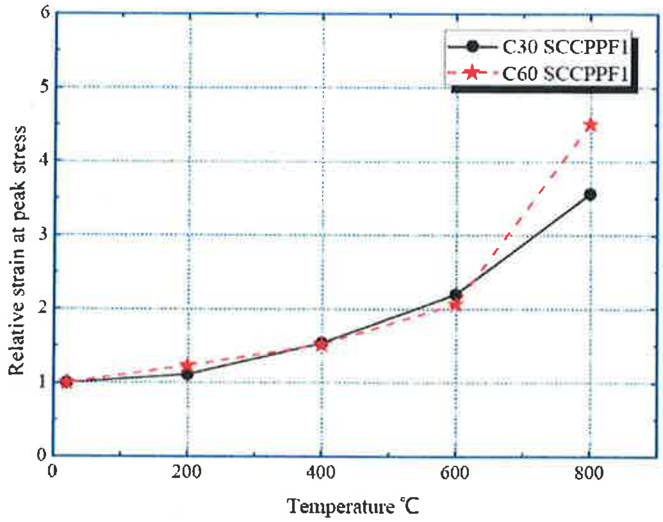


Figure 3: Relative strain at peak stress of SCCPPF1-C60 and SCCPPF1-C30

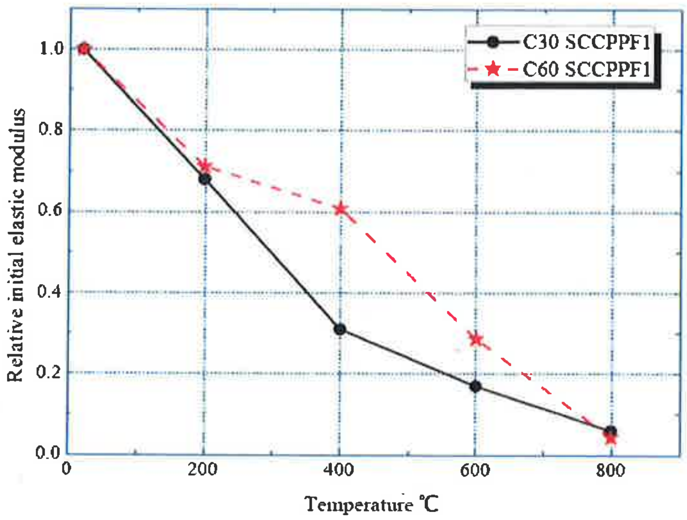


Figure 4: Relative elastic modulus of SCCPPF1-C60 and SCCPPF1-C30

3.4 Initial elastic modulus

The initial elastic modulus is defined by the ratio of the stress value at 40% of the ultimate stress at high temperature and the corresponding strain. The relative initial elastic modulus defined as the ratio of elastic modulus at a specified temperature to that at room temperature, is shown in Figure 4.

The changes of initial elastic modulus with temperature were characterized by a monotonic reduction with temperature. The effect of temperature on the modulus of elastic modulus of both high strength and normal strength self-compacting concrete was very similar up to 200°C. From 200°C to 400°C, the loss in elastic modulus of normal strength SCCPPF specimen was larger than that of high strength SCCPPF. The difference in loss of elastic modulus was narrowed with increase in temperature. At 800°C, the elastic modulus's of both types concrete were 8 percent of room temperature value.

The degradation of elastic modulus is mainly caused by the loss in stiffness of gel. The structure of gel became loosening at high temperature due to the chemical transformation.

3.5 Instantaneous stress-strain curve

The instantaneous stress-strain curves under different temperatures are shown in Figure 5 and Figure 6. It is very clear that for the ascending phase of the curves, the slope gradually becomes lower with an increase in temperature. Hence, the ascending phase of the curves at higher temperatures is always below that at lower temperatures.

The ascending branches of the curves comprise essentially two components: an elastic recoverable strain component, which is temperature dependent and is affected principally by the temperature and load level during initial heating to the test temperature; and an irrecoverable plastic strain component. For high strength SCC, at room temperature, the ascending branch remained linear till the stress reached about 60% of the peak strength indicating that the irrecoverable components of strain were small. From room temperature to 200°C, an increase in temperature slightly changed the slope of the curves and the peak strength. The specimens failed in a brittle way and the curves had short descending branches. Above 400°C the strains corresponding to peak stress increased considerably. A progressive softening after peak strength and a ductile failure was observed at 600°C and 800°C.

The descending phase of the curves behaves differently from the ascending phase. This is because the rate of decrease of stress at the higher temperature is slower than that at the lower temperature. Thus, the slope of the descending phase of the curves at higher temperature was lower than that at lower temperature. The entire stress-strain curve becomes smoother and flatter with an increase of temperature.

In summary, these results suggest that the normal strength SCCPPF exhibits better ductility characteristics than high strength SCCPPF at elevated temperatures. The high strength materials are brittle and possess a small descending branch except when the concrete is heated at 600-800°C thus indicating a softening of the material at those temperatures.

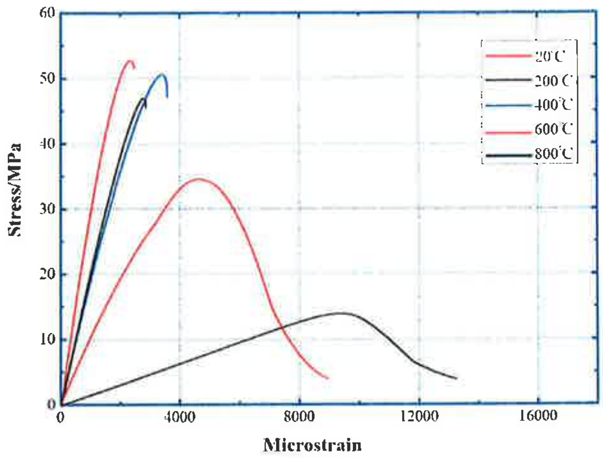


Figure 5: Stress- strain curves of SCCPPF1-C60

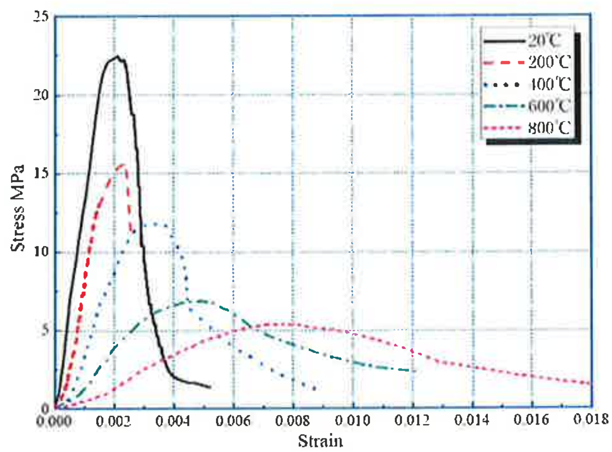


Figure 6: Stress-strain curve of SCCPPF1-C30

3.6 Instantaneous stress-related strain

The instantaneous stress-related strain is defined the compressive strain when the instantaneous stress applied at a specific temperature. It can be obtained by the stress-strain curves of concrete at different temperatures.

There are many stress-strain equations at room temperature, such as polynomial expression, exponential expression, trigonometric function and rational fraction. The equations at high temperature, there are polynomial expression [3, 4], exponential expression [5].

The stress-strain curves of high strength and normal strength self-compacting concrete exhibit a rather complicated relationship with temperature, non-linear over most of the range. However, some conclusions may be drawn by imposing the following transformation on the data. For each temperature, let be the maximum stress f_{cT} (the peak of the stress-strain curve) and let the strain at this point be ε_{pT}. We plotted $y=\sigma/f_{cT}$ against $x=\varepsilon/\varepsilon_{pT}$ for each temperature and obtain Figure 7 and Figure 8. It is clear that the effect of this transformation is to bring all the curves for different temperatures together so that they lie along the same course [6]. It implies that the stress-strain curves for high temperature can be derived entirely from the stress-strain curves measured at room temperature together with the variation of compressive strength of SCC with temperature, strain at peak stress [6].

Based on the comparison of existing models under unstressed condition [3, 6,7] there is an advantage in the model of Guo because significance of parameters is obvious and simple. The stress-strain is shown in Equation 1.

$$y=\begin{cases} ax+(3-2a)x^2+(a-2)x^3 & 0\le x\le 1 \\ \dfrac{x}{b(x-1)^2+x} & x\ge 1 \end{cases} \tag{1}$$

Where, $x=\varepsilon/\varepsilon_{pT}$, $y=\sigma/f_{cT}$, f_{cT}, ε_{pT} is peak strength and strain at peak strength at high temperature respectively, σ, ε is stress and instantaneous stress-related strain at high temperature.

Based on the experimental results of self-compacting concrete, the parameters are shown in Equation 2 to Equation 5. The comparison between test result and the above function at high temperature is shown in Figure 9.

For high strength self-compacting concrete:

$$a=-0.0784*e^{\frac{\theta}{313.06}}+1.180 \tag{2}$$

$$b=0.05983+0.20272e^{\frac{\theta}{163.5371}} \tag{3}$$

For normal strength self-compacting concrete:

$$a=0.00863*e^{0.003113\theta}-0.00983 \tag{4}$$

$$b=345.33e^{-0.000005\theta}-328.01488 \tag{5}$$

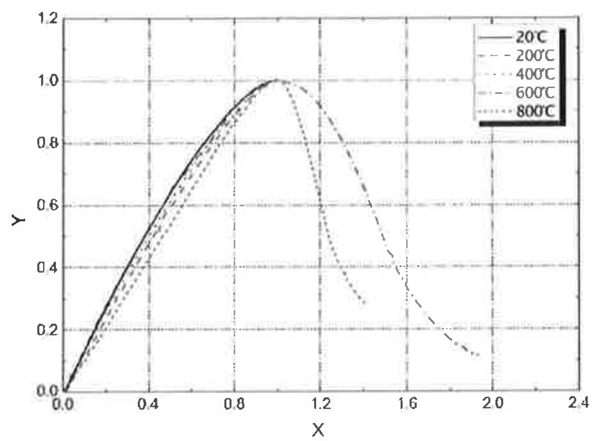


Figure 7: Normalized stress-strain curves for SCCPPF1-C60

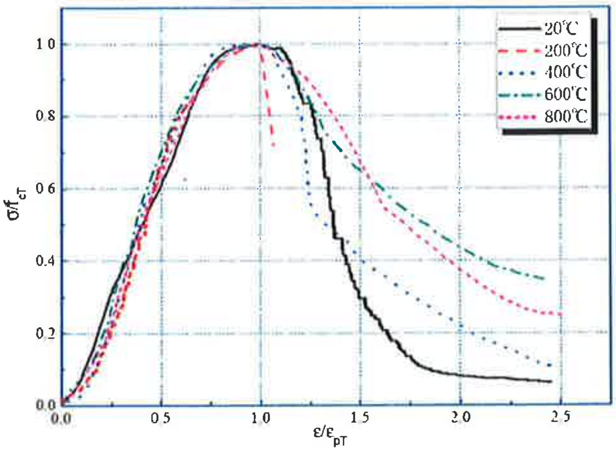


Figure 8: Normalized stress-strain curves for SCCPPF1-C30

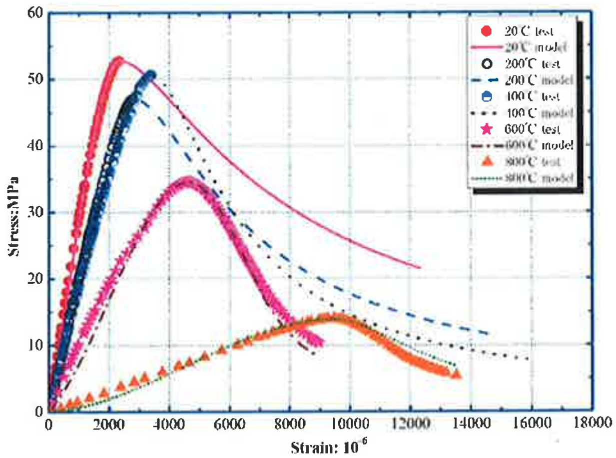


Figure 9: Comparison of test and calculation stress-strain curves for SCCPPF1-C60

4. CONCLUSIONS

Based on the experimental results of unstressed tests for different strength self-compacting concrete with addition of polypropylene fiber, the following conclusions can be drawn:

– The failure mode of cylinders is characterized by longitudinal splitting through both the mortar and the aggregate particles above 400°C. The post-failure behaviour after peak-strength was characterized by a progressive softening with increasing temperature, especially above 400°C for three types of self-compacting concrete.
– The critical temperature is 400°C for the mechanical properties of high strength SCC. The compressive strength and modulus decreased very slowly below 400°C and dropped sharply above 400°C. For normal strength self-compacting concrete, strength and stiffness decreased above 200°C.

- Strength grades have an effect on the strength loss of concrete, especially in the temperature range below 400°C. For normal strength self-compacting concrete specimen was greater than the loss of strength for high strength self-compacting concrete specimen. This difference is narrowed in the permanent strength loss stage.
- The instantaneous stress-strain curves became smoother with the increase of temperature. Strength grades have an effect on the descending branch of stress-strain curve of self-compacting concrete at different temperatures. The steeper descending branch occurred for high strength self-compacting concrete.

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