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Direct tension test and tensile strain capacity of concrete at early age

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Abstract

The tensile strain capacity of concrete under uniaxial tension is investigated using the direct tension test method. The adopted method of testing improves the weak bond strength between the embedded bar and concrete and reduces the stress concentration at the end of the embedded bar. The method has overcome the difficulties in centralizing and aligning the two embedded bars in the specimens. Seven mixes of concrete were designed to study the effects of age, compressive strength and mineral admixture on the tensile strain capacity. The investigation shows that the tensile strain capacity of concrete is a relatively independent parameter. The average tensile strains at failure and at 90% failure load are 120 and 100 με, respectively. The corresponding characteristic tensile strain values at failure and at 90% failure load are 86 and 78 με, respectively.

Keywords: Compressive strength; Fly ash; Granulated blast furnace slag; Portland cement; Tensile property

1. Introduction

The tensile strength and tensile strain capacity of concrete are widely used for evaluating the occurrence of cracks in concrete members. The tensile strain capacity is defined as the maximum tensile strain that concrete can withstand without a continuous crack forming. Based on the tensile strain capacity rather than the tensile strength, it is more convenient and simpler to evaluate cracking where the forces can be expressed in terms of linear or volumetric changes. The tensile strain capacity was mainly evaluated from modulus of rupture test [1], which was adopted by the Corps of Engineers as a measure of the capability of concrete to resist tensile strains [2,3]. Strains on the outer surface of the test specimens under tension were measured directly. Thomas et al. [4] used similar method to study the early age tensile strain capacity of concrete blended with fly ash. Xie and Liu [5] determined tensile strain capacity from specimens of large size of dimension $450 \times 450 \times 3600$ mm^3 subjected to the direct tension load. Hunt [6] measured tensile strain capacity from fully restrained concrete prisms in which temperature differentials were induced to cause cracks in concrete prisms. Houghton [3] proposed that the estimated tensile strain capacity is evaluated from the modulus of rupture divided by the modulus of elasticity. Liu and McDonald [7] developed an approximate method for estimating tensile strain capacity of concrete using the compressive strength and modulus of elasticity. The variation of tensile properties of concrete under various states of stress was presented by Wee et al. [8]. The tensile strain capacity of concrete using granite as coarse aggregate was stipulated by BS8110:1985 [9], PSA Specialist Services [10], Bamforth [11] and Bamforth and Price [12] to be 80 με, while ACI 207:1989 [1] and Harrison [13] proposed the values of 86 and 90 με, respectively.

The direct tension test of concrete, although of considerable theoretical and practical interest, has been given little attention by investigators. The indirect tension test methods are widely used for studying the tensile properties of concrete due to their simplicity. However, the direct tension test provides results, which are more rational and reliable. The tensile properties are imperative for the study of concrete structures under the state of tensile stresses, such

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as those due to temperature differentials and drying shrinkage. This paper presents the tensile strain capacity of concrete from direct tension test.

2. Experimental investigation

2.1. Materials and mix proportions

The cementitious materials used in the present study are ordinary Portland cement (OPC), ground granulated blast furnace slag (GGBFS) and pulverized fuel ash (PFA) complying with BS12:1978 [14], BS6699:1992 [15] and BS3892:1997 [16], respectively. The chemical compositions and physical properties of the materials are given in Table 1.

The fine aggregate is silica sand with a density of 2.60 g/cm^3 and fineness modulus of 2.59. The coarse aggregate of 20 mm maximum size consists of solid crushed granite with a density of 2.65 g/cm^3 and a fineness modulus of 6.67. The coarse aggregate was properly washed and separated into two categories, i.e., 5–10 and 10–20 mm. They were then proportionally mixed at the ratio of 1:3 and maintained in a saturated surface-dry (SSD) condition prior to the mixing of concrete. The fine and coarse aggregates comply with ASTM C33:1997 [17]. Seven mixes were designed for the present study and the mix proportions tabulated in Table 2. The replacement percentages by mass are 30% of PFA and 70% of GGBFS for PFA and GGBFS concretes, respectively. The slump test was carried out in accordance with BS1881:1978 [18].

2.2. Specimen preparation

Concrete was evenly placed into the specially designed tensile moulds of dimension $100 \times 100 \times 500$ mm^3. The specimens were demoulded about 24 h after casting and cured in an environmentally controlled room with temperature maintained at 27 ± 2 °C and relative humidity higher than 90% [19]. Two electrical resistance strain gages of 120 mm in length and gage factor of $2.09 \pm 1\%$ were glued on two opposite side faces in the middle section of the specimen. Twenty tensile specimens of $100 \times 100 \times 500$ mm^3 were cast for each of the following OPC1, OPC2, OPC3, OPC4 and OPC5 concretes. Fifteen of the 20 specimens were tested at different ages of 1, 3, 7, 14 and 28 days at a strain rate of 5 με/min, three at each age. The remaining five specimens were tested at 28 days at different strain rates. Three were tested at a strain rate of 1 με/min and two at 30 με/min. Nine tensile specimens were cast for PFA and GGBFS concretes and tested at 7, 14 and 28 days at a strain rate of 5 με/min, three at each age. Compressive strength was assessed through the crushing of three 100-mm cubes at each age.

Table 2
Mix proportion (kg/m^3)

Mix[a]	w/c	Water	OPC	PFA or GGBFS	Sand	Coarse aggregate	Slump[a] (mm)
OPC1	0.30	165	550	0	755	920	40[b]
OPC2	0.40	180	450	0	775	950	35
OPC3	0.50	180	360	0	810	990	35
OPC4	0.60	180	300	0	835	1020	30
OPC5	0.50	180	360	0	810	990	30
PFA	0.50	180	250	110	795	975	30
GGBFS	0.50	180	110	250	805	985	25

[a] Slump test complied with BS 1881:1978: Part 102 [17].
[b] The ASTM type F superplasticizer (powder) was used in OPC1 with a dosage of 0.5% of cement content.

2.3. Test procedure

The following modes of gripping the specimens for direct tension test have been adopted by many researchers. They are by means of (1) rings on truncated cones [20], (2) embedded steel bars [5], (3) gluing [21] and (4) lateral gripping [22]. Each method suffers different setbacks. All four methods encountered problems associated with load eccentricity and nonuniformity of stress and strain. Poor adherence between steel plates and moist concrete was experienced from gluing method. Stress concentration causing fracture at the ends of the specimens was observed when Methods 1, 2 and 4 were adopted. Inadequate guidelines to ensure good bond and central location of the embedded bars in the specimens were the main drawback of Method 2. In the present study, the following modifications were introduced to mitigate the problems inherent in the existing embedded bar method.

(i) A two-piece mould, as shown in Fig. 1, is introduced in lieu of the existing three- or five-piece mould. The lower number of pieces implies a lower number of degree of freedom to adjust resulting in ease of and

Table 1
Chemical compositions and physical properties of OPC, GGBFS and PFA

Material	C_aO (%)	S_iO_2 (%)	Al_2O_3 (%)	$F_{e2}O_3$ (%)	M_gO (%)	SO_3 (%)	$N_{a2}O$ (%)	K_2O (%)	Fineness (cm^2/g)	Specific gravity
OPC	63.80	21.20	5.50	3.00	2.00	2.00			3150	3.15
GGBFS	42.90	32.50	13.80	0.20	5.80	–	–	–	4500[a]	2.90
PFA	3.37	49.30	28.30	13.01	–	–	0.21	0.76	91.6%[b]	2.41

[a] Using the Blaine method.
[b] Percentage passing of 45 μm sieve.

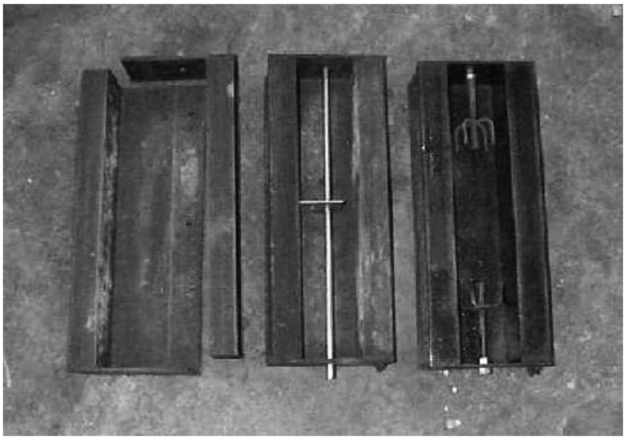

Fig. 1. Mould for direct tension specimens.

more accurate fabrication. The two-piece mould can be conveniently adjusted to align and centralize the two embedded bars by means of the standard rod and plate method. The two holes accommodating the two embedded bars were carefully drilled to ensure that after the assembly, their centers are exactly at the geometric centers of the two end plates.

(ii) The weak bond between the embedded bar and concrete and the high stress concentration at the end of the embedded bar are two problems associated with this test method. The main embedded bar is a screw bar of 12-mm diameter and 150 mm in length. The bars were embedded with sufficient embedded length of 125 mm in concrete specimen leaving about 25 mm for connection between specimen and testing machine. The bond strength of the embedded bars is enhanced further through the introduction of four additional 6-mm diameter bars of about 50 mm in length welded evenly to the main bars as shown in Fig. 1. These bars also help reducing stress concentration at the end of the embedded bars mitigating fracture in concrete specimen.

(iii) A specially designed tensile joint was used to connect between the testing machine and specimen in order to eliminate any bending moment during testing. The tensile specimen was connected with the tensile joint and subsequently attached to the Instron Testing Machine (Fig. 2). The constant strain rate was achieved through the stroke speed adjustment and maintained in the linear portion of the stress–strain curve.

Fig. 2. Direct tension test using Instron testing machine.

3. Results and discussion

3.1. Direct tension test

Typical stress–strain curves recorded from direct tension tests adopted in this study are shown in Fig. 3. The two tensile strain values monitored on the two opposite faces of each specimen are very close to each other up to about 70–80% of failure load. This demonstrates that the modified test method adopted in the present study is effective in minimizing the eccentricity of the load. Since concrete is a nonhomogeneous material, the two curves deviate at higher stress level depending on the stress concentration at the tips of the flaws or crack pattern existing in concrete specimen. In the present study, stress concentration at the ends of the embedded bars is substantially reduced. A total of 117 tensile specimens were tested successfully with more than 90% of the specimens fractured in the middle of the 200-mm section. Fig. 4 shows some typical tensile specimens after testing.

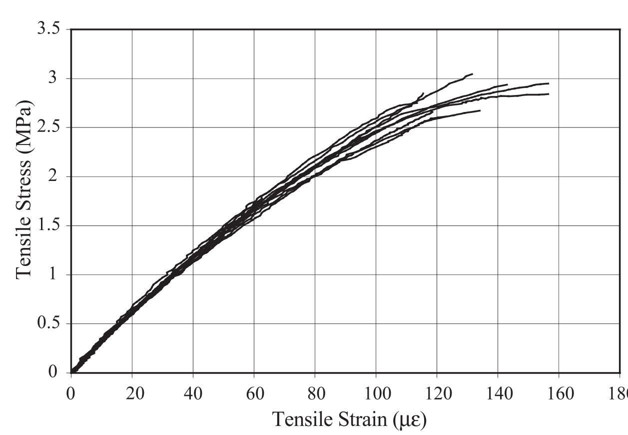

Fig. 3. Stress–strain curves of concrete in tension.

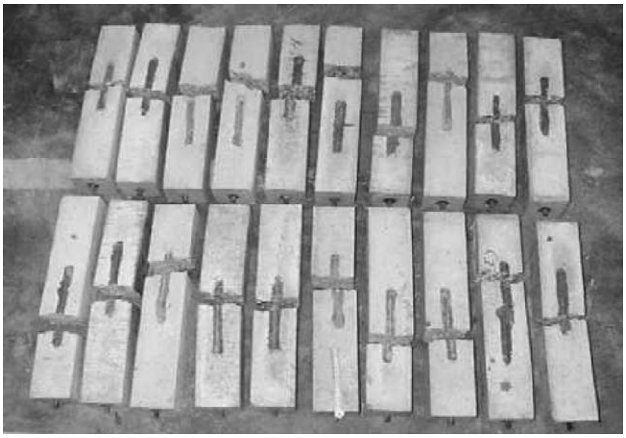

Fig. 4. Specimens after direct tension test.

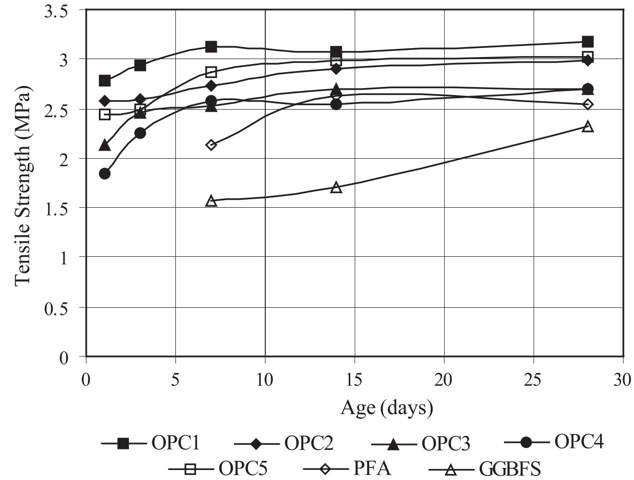

Fig. 6. Relationship between tensile strength and age of concrete.

3.2. Tensile and compressive strengths

Figs. 5 and 6 show the relationship between the compressive and tensile strengths against the age of the specimen, respectively. As expected, concretes with low w/c ratios gain strength faster than those with higher w/c ratios. For the same w/c ratio, the use of large particles reduces the specific area of aggregate and hence the lower bond strength resulting in the reduction of concrete strength [23]. This is illustrated in Figs. 5 and 6 where both compressive and tensile strengths of OPC5 concrete are greater than those of OPC3. Both have similar mix proportion but the former contains smaller maximum size of coarse aggregate (10 mm) than that of the latter (20 mm).

Compressive strengths of both PFA and GGBFS concretes at early age are lower than those of OPC3 concrete but caught up with the latter after 14 days as depicted in Fig. 5. Similar trend was also observed for the development of tensile strength of PFA concrete (Fig. 6). GGBFS concrete, however, provided lower tensile strength than OPC3 concrete at 7 and 14 days as shown in Fig. 6, though the development of its compressive strength was comparable to that of PFA concrete (Fig. 5).

Fig. 7 demonstrates the relationship between tensile to compressive strength ratio and age of concrete. The tensile to compressive strength ratio decreases as concrete matures. This implies that the gain on tensile strength is smaller than the increase in compressive strength. The tensile to compressive strength ratio varies from 0.04 to 0.10.

The relationship between tensile to compressive strength ratio and compressive strength of OPC concrete is depicted in Fig. 8. The tensile to compressive strength ratio is higher at lower compressive strength and decreases as the compressive strength of OPC concrete rises. Different mix proportions and ages of OPC concrete do not seem to affect this relationship provided that the compressive strength does not vary. PFA concrete shows similar relationship but GGBFS mixture produces concrete with

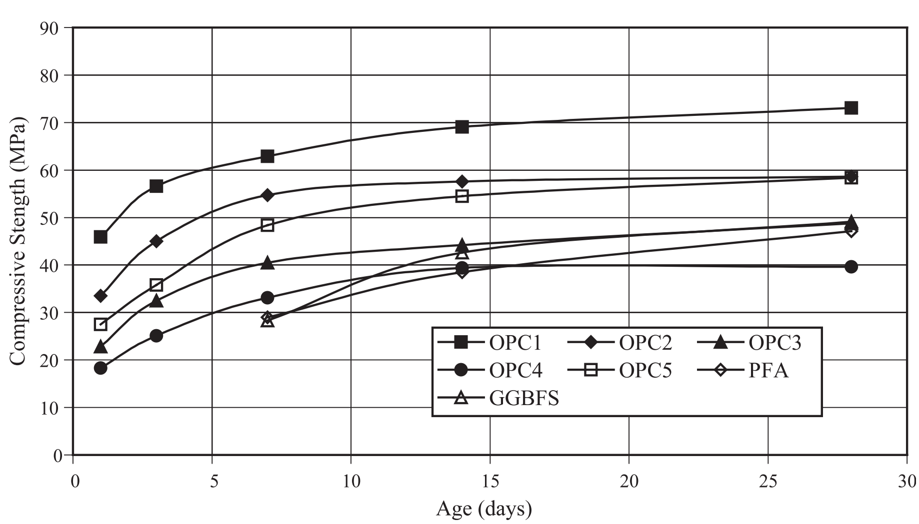

Fig. 5. Variation of compressive strength with age of concrete.

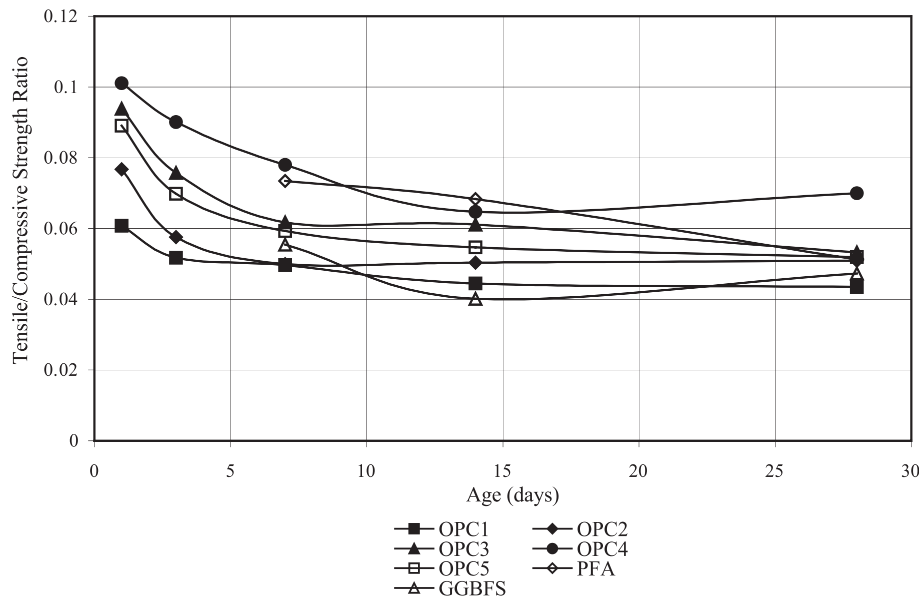

Fig. 7. Tensile/compressive strength ratio and age of concrete.

lower tensile strength and hence smaller value of tensile to compressive strength ratios compared to OPC concrete as demonstrated in Fig. 8. The finding is consistent with the statement stipulated earlier by Wee et al. [24] that the flexural strength of concrete with GGBFS is lower than that of OPC concrete of the same compressive strength.

3.3. Tensile strain

The relationship between tensile strain at 90% failure load and age of concrete is shown in Fig. 9. The tensile strain capacity does not vary significantly with either mix proportions or curing ages of concrete. If the linear relationship is assumed between tensile stress and tensile strain, the value of tensile strain (ε_{ten}) at maximum load is proportional to the ratio of tensile strength (f_t') to modulus of elasticity in tension (E_{ten}), as stipulated in Eq. (1).

$$\varepsilon_{\text{ten}} \propto \frac{f_t'}{E_{\text{ten}}} \qquad (1)$$

Since f_t' and E_{ten} are both similarly affected by age of concrete though not in the same proportion, any errors in the prediction of the ratio of f_t' to E_{ten} based on changing maturity would be largely self-canceling. It can be concluded that the ratio of the tensile strength to modulus of elasticity under tension or tensile strain capacity does not depend significantly on strength, mix proportion and age of concrete. The different maximum aggregate size and the replacement with mineral admixture (PFA and GGBFS) do not seem to affect the tensile strain capacity of concrete either (Fig. 9).

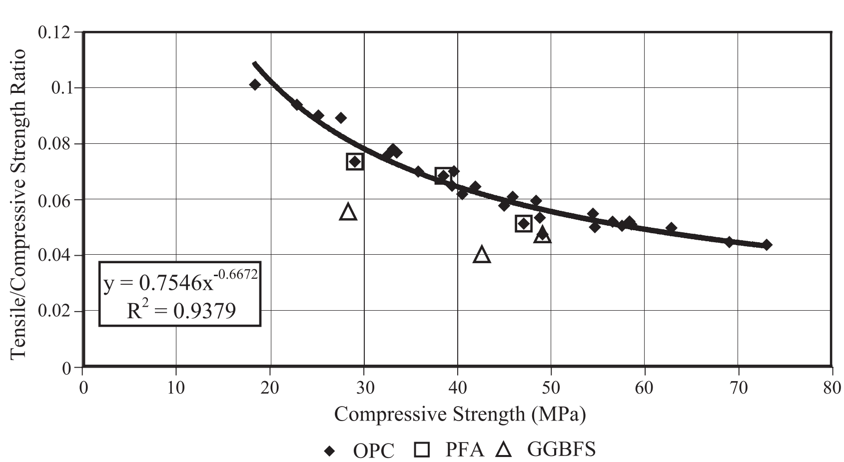

Fig. 8. Variation of tensile/compressive strength ratio with compressive strength of OPC concrete.

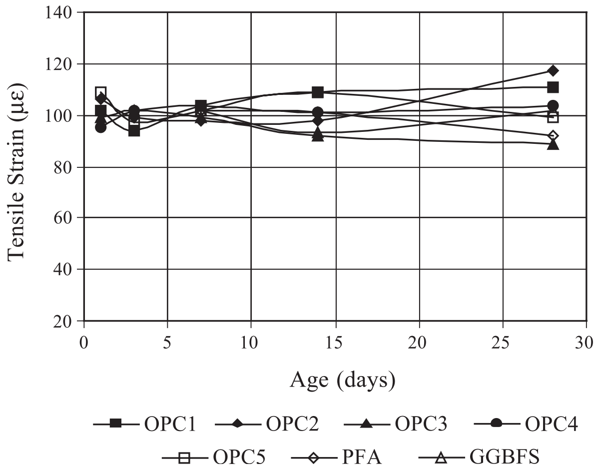

Fig. 9. Relationship between tensile strain at 90% failure load and age of concrete.

Fig. 10 depicts the relationship between the tensile strain capacity and the compressive strength. Both tensile strains at failure and at 90% failure load are included in the figure. Though high compressive strength concrete is usually associated with low compressive strain capacity, both tensile strains at failure and at 90% failure load do not vary significantly with concrete strength. The difference can be explained through the fracture behaviour of concrete. Considering a concrete specimen under uniaxial tension, the randomly distributed precracks are pulled apart when a tensile load is applied on the specimen. The initiation and growth of every new crack will reduce the load carrying area. This reduction in area further causes an increase in the stress concentration at critical crack tips. The magnitude of the stress concentration at the crack tips and the crack arrest capability dictate the continuation of the crack. Under tension, only a few cracks need to be developed to cause the tensile failure.

Since tensile strain is a relatively independent parameter, statistical methods can be adopted to analyze the whole set of the tensile strain data. The average and standard deviation calculated from experimental results are 120 and 18 με for tensile strain at failure and 100 and 12 με for tensile strain at 90% failure load, respectively. The characteristic tensile strain value can be defined as the lower limit of the one-sided statistical tolerance interval at which there is a 90% probability that 95% of the values are at or above this lower limit. The values can be obtained from

$$\varepsilon_k = \varepsilon_{av} - ks \qquad (2)$$

where ε_k, ε_{av} and s are the characteristic, average and standard deviation values of tensile strain, respectively. The statistic coefficient, k, is the function of the number of the test results. In this study, 117 tensile specimens were tested leading to the coefficient value of 1.84. Therefore, the characteristic tensile strain values at failure and at 90% failure load are 86 and 78 με, respectively. The magnitudes of the characteristic tensile strains obtained are very close to the tensile strain capacity stipulated by ACI 207:1989 [1], BS8110:1985 [9], PSA Specialist Services [10], Bamforth [11], Bamforth and Price [12] and Harrison [13].

3.4. Modulus of elasticity

Modulus of elasticity in tension is estimated from the slope of the tensile stress–strain curves. In the linearly elastic regime, i.e., from 0–90% failure load, all values of regression coefficient are greater than .98.

The relationship between modulus of elasticity under tension test and compressive strength is shown in Fig. 11. The relationship between modulus of elasticity under tension and compressive strength of PFA concrete is similar to that of OPC concrete, but GGBFS concrete shows lower values of modulus of elasticity under tension. The low values of modulus of elasticity of GGBFS concrete result in the comparable tensile strain capacities of GGBFS and OPC concretes as depicted in Fig. 9.

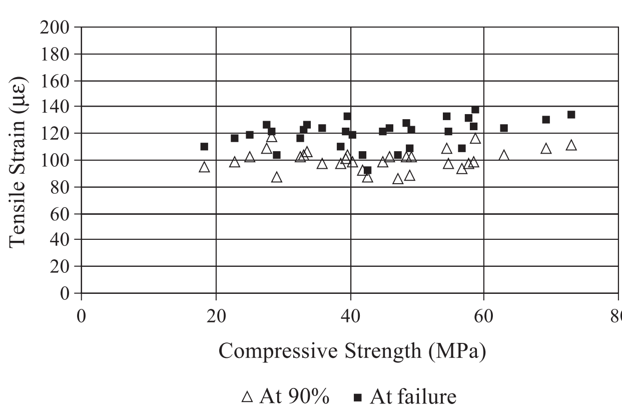

Fig. 10. Relationships of tensile strain at failure and at 90% failure load with compressive strength.

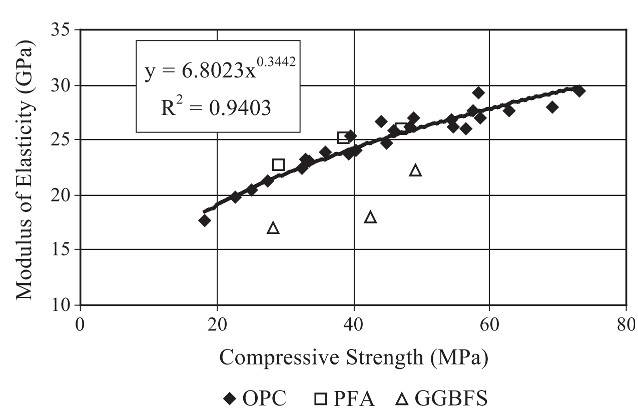

Fig. 11. Relationship between modulus of elasticity in tension and compressive strength.

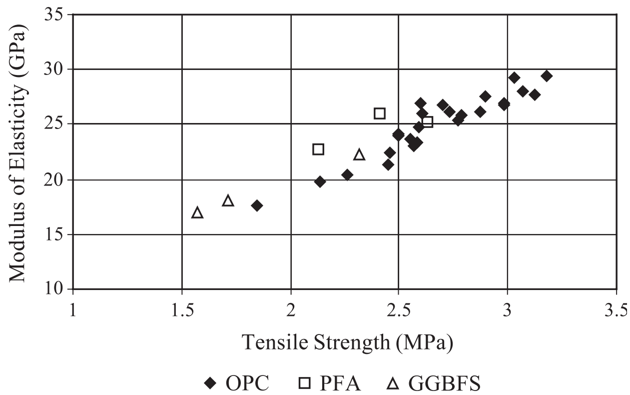

Fig. 12. Relationship between modulus of elasticity in tension and tensile strength.

Modulus of elasticity under tension has a good linear relationship with tensile strength as illustrated in Fig. 12. This linear relationship was also observed for GGBFS and PFA concretes.

4. Conclusion

Based on the mix proportions, cementitious materials used and the experimental method adopted in this study, the following conclusions can be made:

1. The proposed tension test procedure, which was designed for ease and accuracy of assembling, minimizes the eccentricity of the load and improves significantly the experimental results. This is evident from (i) the similar two tensile strain values monitored on the two opposite surfaces of each specimen loaded up to about 75% of failure load, (ii) more than 100 of 117 tensile specimens fractured in the middle section, which is desirable, and (iii) the values of standard deviation of results from the proposed tension test of 12–18 $\mu\varepsilon$ are significantly lower than those of other similar tests such as flexure test.
2. Tensile strength of concrete increases with curing age at the lower rate than compressive strength. The tensile to compressive strength ratio varies from 0.04 to 0.10. The values of the ratio decrease as concrete matures and the compressive strength rises. The tensile strength of OPC concrete has a good relationship with its compressive strength regardless of its age and mix proportion.
3. The mix proportion, curing age and concrete compressive strength do not seem to affect significantly on the tensile strains at failure and at 90% failure load. The tensile strain at failure or at 90% failure load can be considered to be relatively independent parameters. The average tensile strains at failure and at 90% failure loads are about 120 and 100 $\mu\varepsilon$, respectively, while their characteristic values are 86 and 78 $\mu\varepsilon$, respectively.
4. Modulus of elasticity under tension has a good linear relationship with tensile strength for all concrete mixtures, including PFA and GGBFS concretes. Though GGBFS concrete shows lower tensile strength, its tensile strain capacity is comparable to other concretes as its modulus of elasticity under tension is also lower than other concretes of the same compressive strength.

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