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Technical Development of Highly Durable Pretension PC Girder

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ABSTRACT

This study presents a full scale testing of four PC girders to compare their mechanical behaviour. Two girders were made by normal concrete and others were made by fly ash concrete. The girders were reinforced with normal and epoxy coated steel strands, where epoxy coated steel strands had an external epoxy coating on each wire for preventing the corrosion from chloride ions. Fly ash increases workability and durability, and reduces permeability. Also, fly ash is an environmentally friendly aggregate that exceeds performance specifications and contributes to Leadership in Energy and Environmental Design points.

Test concrete blocks with normal cement, high early strength cement and fly ash were made to investigate the performance of concrete, where temperature history and concrete strength were studied under steam curing. The results were used to design the concrete mixture for PC girders.

Results of bending test ensure higher load bearing capacity for all PC girders.

Keywords. PC Girder, Bond Performance, Load Bearing Capacity, Fly Ash Concrete, Epoxy Coated Steel Strand

INTRODUCTION

Benefits of using fly ash in concrete are temperature crack control, mitigation of alkali-silica reaction, durability and long-term strength, and the use of fly ash helps the environmental preservation by reducing greenhouse gases emissions associated with the manufacture of Portland cement. Effective utilization of fly ash concrete has become a significant trend internationally.

Japan already issued the guidelines for design and construction of fly ash concrete (JSCE, 2009). However, fly ash concrete is not widely used yet. The importance of fly ash concrete is reflected on the project of Irabu Island Bridge which is the longest bridge in Okinawa Prefecture, Japan. The bridge is using a large amount of fly ash Class II specified by JIS

A6201(JIS, 1991) at abutments and piers to improve durability. But, there is no precedent applying fly ash concrete to PC superstructure, where high strength at early stage is required.
This technical development deals with the use of fly ash concrete on PC girders, where the mixture proportion of fly ash concrete is designed to reduce the hydration heat of cement and ensure the early concrete strength. Fly ash Class II specified by JIS A6201(JIS, 1991) has been used as a replacement of cement and fine aggregate. Fly ash concrete is designed to secure the specific design strength and the controlled concrete temperature at prestressing of epoxy coated steel strands.
Also, this study intends to clarify the effects of steam curing on fly ash concrete by applying different curing factors. There are already works on the steam curing of PC girders, but the effects of steam curing on temperature history and early strength of fly ash concrete are not clear yet.
Fly ash concrete and epoxy coated steel strands (PWRC, 2010) are used to improve durability of PC girders. Epoxy coated steel strands have an external epoxy coating on each individual wire to protect against chloride ions. Then, the proposed PC girder is considered the most appropriate type of structure to ensure high durability at salt damage areas. However, there is a concern about degradation of bond performance of epoxy coated steel strands due to the high hydration heat of cement. Furthermore, there are very few studies on the PC girder with epoxy coated steel strands. Due to these reasons, this type of PC girder is not listed in the widely used Japanese construction handbook (JPCCA, 2005).
This study intends to provide the data on bond performance and the load bearing capacity of PC girder by bending test to failure.

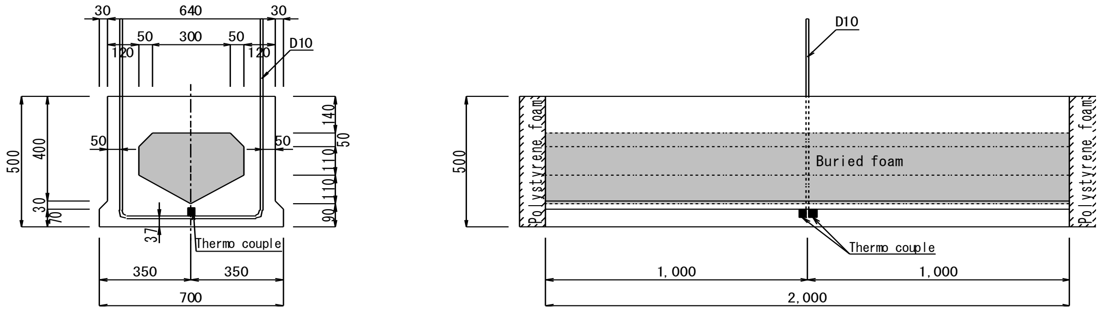
TEMPERATURE HISTORY AND STRENGTH PERFORMANCE OF FLY ASH CONCRETE

Concrete Block and Compression Test. Four concrete blocks (NC-1, NC-2, NFC-1, HFC) were prepared to study their temperature history and compressive strength behaviour for different curing temperature-time variables. The section of concrete block is shown in Figure 1, which is similar to the section of PC girder mentioned later. Concrete mixture proportions are shown in Table 1. Two concrete blocks (NC-1, NC-2) were made by normal concrete mix (NC) and the third block (NFC-1) was made by normal fly ash concrete mix NFC-1. And the fourth block (HFC) was made by fly ash concrete with high early strength Portland cement mix HFC.

Lateral and bottom sides of the block were confined by metal formwork and the ends were restrained by polystyrene foam plates. Concrete temperature was measured at the center of span and at 50 mm from bottom. Also, steam temperature was measured at the center of span under curing sheet.

Standard 100x200 mm lab concrete specimens were obtained for every concrete mixture, where lightweight mould frames were used. The specimens were protected with polystyrene foam at lateral and bottom sides and they were cured together with the concrete blocks. This curing was called as the simplified insulated curing method.

Concrete for all mixtures was designed for a 50 MPa compressive strength. Concrete mixes were based on normal Portland cement, high early strength Portland cement and fly ash. Crushed limestone was used as coarse aggregate, and mix sand of crushed limestone and sea sand was used as fine aggregate.
Mixture proportion of normal fly ash concrete NFC-1 in Table 1 considered the following factors such as early concrete strength at prestressing, controlled temperature, durability and



(a) Section **(b) Elevation**

Figure 1. Details of cross section and elevation (Dimension: mm)

Table 1. Concrete Mixture Proportions

Designa-tion	W/(C+FA) (%)	s/a	Unit weight(kg/m^3)					
		(%)	W	C+FA1+FA2	S1	S2	G	A
NC	32.5	42.7	156	480	231	523	1048	4.80
NFC※	30.9	40.0	156	392+86+25	207	469	1048	4.80
NFC-1	30.9	41.1	156	418+62+25	215	492	1048	4.80
HFC	30.9	40.0	156	394+86+25	207	469	1048	4.85

Where W: water, C: cement, FA1 and FA2: unit weight of fly ash replacing to cement and fine aggregate, S: fine aggregate (S1: crushed limestone S2: sea sands) G: coarse aggregate, A: receding agent. W/(C+FA): water/binder ratio, s/a: fine aggregate ratio, Maximum size of coarse aggregate: 20mm, Slump: 120mm, Air: 1.5%.
NFC※ :Used for PC girders mentioned later

perspective environmental protection. Thirteen per cent of normal Portland cement was replaced by fly ash similarly to the fly ash cement type B specified by JIS R5213 (JIS, 2009). Also, 25 kg/m^3 of fine aggregate was replaced by fly ash to improve workability. High early strength Portland cement was used for fly ash concrete HFC, where 18 % of the cement was replaced by fly ash.

The concrete blocks, NC-1 and NC-2 were cured under the temperatures at 50 ^oC and 65 ^oC for a period of six hours, respectively. Similarly, the concrete blocks, HFC and NFC-1 were cured at 50 ^oC and 65 ^oC for a period of six hours, respectively. The blocks were covered by sheets during steam curing, and after a period of six hours, heating source was shut down to allow the concrete blocks to cool down.

Concrete specimens were cured under the same curing condition as the concrete blocks. The specimens, NC-2 and NFC-1 were removed from curing at 17.5 hours of concrete age. In the same way, the specimens, NC-1 and HFC were removed from curing at 19.5 hours of concrete age. Compressive tests were performed at 18 or 20 hours of concrete age. These test ages were determined according to the prestressing of PC girders mentioned later. Elastic modulus was calculated by measured strains and calculated stresses.

Test Results and Discussions.

Concrete Temperature History. Temperature histories of the concrete blocks and the specimens, NC-1 and HFC are presented in Figure 2. This figure shows ambient temperature variation, concrete temperatures and curing steam temperature with respect to age after concrete mixing. The concrete blocks, NC-1 and HFC raise their concrete temperature similarly to curing temperature, and reach 64 ^{o}C and 62 ^{o}C, respectively, where the block with fly ash concrete, HFC presents slightly lower temperature, but it reaches the maximum temperature slightly faster than normal concrete block NC-1. It should be noted that both HFC and NC-1 present relatively low temperatures due to the low curing temperature of 50 ^{o}C, and these low concrete temperatures contribute greatly to the bond performance of epoxy coated steel strands for PC girders mentioned later. The concrete blocks have the same behaviour in cooling down.
Comparing to the temperature histories of the concrete blocks, the specimens, NC-1 and HFC raise their temperatures slowly, and reach the same maximum temperature as the concrete blocks.

Temperature histories of the concrete blocks and the specimens, NC-2 and NFC-1 are presented in Figure 3. The concrete block NC-2 raises the temperature similarly to the curing

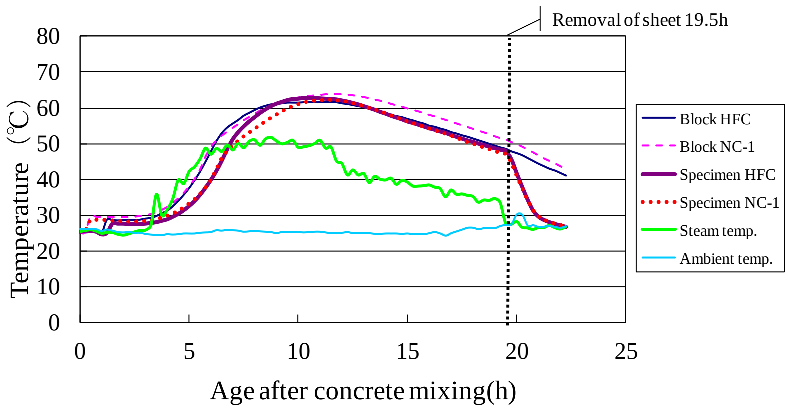


Figure 2. Temperature histories of NC-1 and HFC cured at 50 ^{o}C

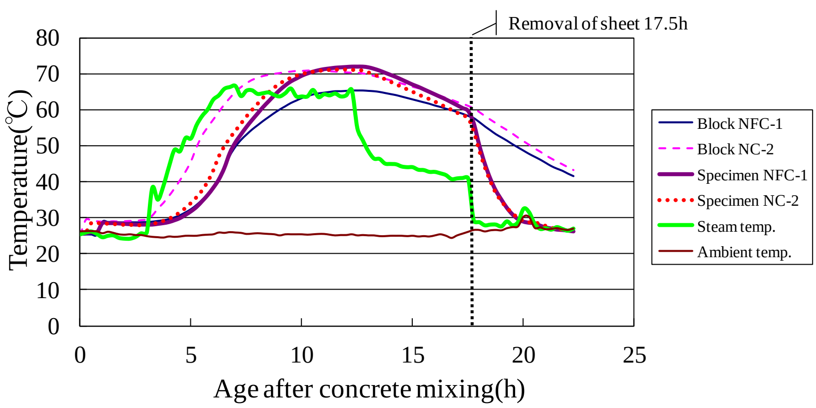


Figure 3. Temperature histories of NC-2 and NFC-1 cured at 65 ^{o}C

temperature and reaches the maximum temperature of 70 ^{o}C. This block keeps higher temperature for a longer period of time. The concrete block NFC-1 raises the temperature more slowly than NC-2, and it reaches the maximum temperature of 65 ^{o}C. This figure shows clearly the temperature differences between NC-2 and NFC-1, and the block with fly ash concrete, NFC-1 presents lower controlled temperature.

The specimens, NC-2 and NFC-1 show similar curing history and the same maximum temperature of 70 ^{o}C. However, these specimens present different temperature histories from the concrete blocks, and more tests are needed to clarify this difference.

Compressive Strength and Elastic Modulus. Table 2 shows the mean values of compressive strengths and elastic modulus of the specimens. Results of all compressive strength satisfy the design strength of 35 MPa required for the JIS type girder at prestressing. The specimens, NC-2 and NFC-1 cured at 65 ^{o}C for a period of 6 hours show 40.4 MPa and 37.5 MPa, respectively at 18.0 hours of concrete age, where the specimen with fly ash concrete, NFC-1 presents a low strength. The specimens, NC-1 and HFC cured at 50 ^{o}C for a period of 6 hours show 38.9 MPa and 41.0 MPa, respectively at 20.0 hours of concrete age, where the specimen, HFC replacing 18 % of high early strength Portland cement by fly ash shows a high strength.

Table 2. Compressive Strength (fc) and Elastic Modulus (E)

Designation	Curing age			
	18 hours		20 hours	
	fc (MPa)	E (GPa)	fc (MPa)	E (GPa)
NC-1			38.9	30.0
NC-2	40.4	31.0		
NFC-1	37.5	29.0		
HFC			41.0	32.0

LOAD BEARING CAPACITY OF PC GIRDERS

Construction of PC Girders. Four JIS type PC girders (BS12-NN, BS12-NFA, BS12-EN, BS12-EHFA) with 12 strands were prepared for this test. Typical details of the girders are shown in Figure 4. Regular hollow section is referred to the slab bridge girder for B-type live load (JPCCA, 2005). Total length of the girders is 12.5 m, and the girders have a bond control length of 1.0 m at both ends. Anchoring length of 65 diameters is applied to the girders in compliance with Japanese Bridge Specification (JRA, 2012) and PCI Committee Guideline (PCI, 1993).

The girders were made by three different concrete mixtures; NC, NFC and HFC with weight proportions as shown in Table 1, and two different prestressing strands ; normal steel strands (N strands) and epoxy coated steel strands (E strands). Girder BS12-NN was made by normal concrete NC and reinforced with N strands. Girder BS12-NFA was made by fly ash concrete NFC and reinforced with N strands. Girder BS12-EN was made by normal concrete NC and reinforced with E strands. Girder BS12-EHFA was made by fly ash concrete HFC and reinforced with E strands. It should be noted that fly ash concrete NFC was modified by replacing 18 % of normal Portland cement by fly ash.

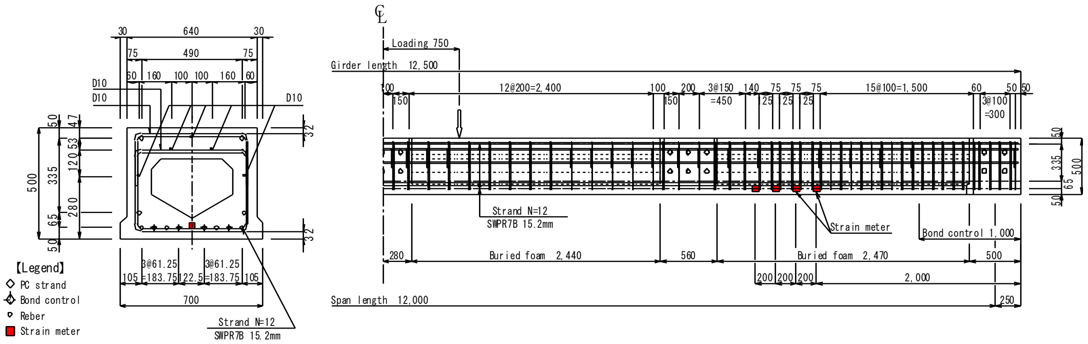


Figure 4. Details of Girder BS12-EN (Dimension: mm)

Curing temperature of Girder BS12-NN and Girder BS12-NFA was set to 65 ^{o}C for six hours, and the prestressing was induced at 18.0 hours of concrete age. Curing temperature of Girder BS12-EN and Girder BS12-EHFA was set to 50 ^{o}C for six hours and the prestressing was induced at 20.0 hours of concrete age.

Specifications for N strands and E strands are based on SWPR7BL and SWPR7BN specified by JIS G3536 (JIS, 2008), respectively. Mechanical properties of E strands are exactly same as N strands (PWRC, 2010). However, these two steel strands have different relaxation ratios: N strands have 2.5 % under steam curing and 1.5 % under normal curing, and E strands have 7 % under steam curing and 5 % under normal curing.

Concrete strength of 50 MPa and elastic modulus of 33 GPa were used for designing all girders. At prestressing, concrete strength of 35 MPa (JPCCA, 2005) and elastic modulus of 30 GPa were applied to three girders, BS12-NN, BS12-EN and BS12-EHFA. But, concrete strength of 30 MPa was applied to Girder BS12-NFA (JRA, 2012). This girder was made by fly ash concrete mixture NFC, where 18 % of normal Portland cement was replaced by fly ash, and high strength at early stage was not expected for this girder as the others.

Concrete specimens cured under the simplified insulated curing were used for measurement of compressive strength and calculation of elastic modulus at prestressing. Specimens, BS12-NN and BS12-NFA were tested at 18 hours of concrete age, and Specimens, BS12-EN and BS12-EHFA were tested at 20 hours of concrete age. Compressive strength and elastic modulus of the specimens for four girders were obtained as follows; BS12-NN: 38.2 MPa and 29 GPa; BS12-NFA: 32.7 MPa and 29 GPa; BS12-EN: 37.5 MPa and 30 GPa; and BS12-EHFA: 41.1 MPa and 32 GPa.

Bending test was performed at 20 days of concrete age and the specimens were also tested on the same day. Compressive strength and elastic modulus of the specimens for four girders were obtained as follows; BS12-NN: 51.8 MPa and 33 GPa; BS12-NFA: 50.1 MPa and 35 GPa; BS12-EN: 51.0 MPa and 35 GPa, and BS12-EHFA: 55.6 MPa and 36 GPa.

Important factors for Girder BS12-EN and Girder BS12-EHFA with E strands are concrete temperature and compressive strength at prestressing. The temperature shall be below 66 ^{o}C as specified by ASTM A882 (ASTM, 2004) and the compressive strength over 30 MPa is specified by Japan Road Association (JRA, 2012). Due to these specifications, the girders with E strands require a lower curing temperature and a longer curing time than the other girders with N strands. Referring to the test results of the concrete blocks, NC-1 and HFC, the prestressing was induced to Girder BS12-EN and Girder BS12-EHFA at 20 hours of concrete age.

Test Method. Concrete temperature at prestressing was measured using thermo couples, where thermo couples were set adjacent to lower strands at the center of span. Concrete temperature histories of the girders were referred to the results of the concrete blocks mentioned above.

Buried type strain meters were set in concrete to measure the induced strains at prestressing, where the strains in concrete were determined by the strain variations just after prestressing. Strains of PC girders induced at prestressing have been studied by the authors (Oshiro et al., 2011). This study is also studying the strains induced at prestressing near the anchoring length shown in Figure 4, and strain distributions are carefully examined. Four strands out of 12 are bond controlled up to 1.0 m from the edges, and this length is called as the bond control length. The location at 1.0 m is considered the first anchoring length. Then, the location at 2.0 m, where full strains of 12 strands are expected, is considered the second anchoring length. The strains at 3 or 4 points from 2.0 m are measured at 200 mm apart.

Load bearing capacity of the test girders was obtained by the bending test specified by JIS A5373 (JIS, 2010). Loading position is shown in Figure 4, where loads are applied at two points. Four strain gauges (polyester base, measuring length 60 mm) were placed on each upper and lower concrete surfaces at the center of span and concrete strains were measured corresponding to applied loads. Also, the deflections at 1/4 span and at the center of span were measured using displacement meters.

Test Results and Discussions.

Concrete Temperature at Prestressing. Girders, BS12-EN and BS12-EHFA had the same curing process and same concrete as the concrete blocks, NC-1 and HFC mentioned above. As the concrete block, NC-1 presented the maximum temperature of 64 ^{o}C, the two girders were considered not to be subjected to higher than this temperature.
At prestressing, concrete temperatures of Girder BS12-EN and BS12-EHFA were measured, and the girders showed the same temperature of 51 ^{o}C, which was lower than the controlled temperature of 66 ^{o}C in compliance with ASTM A882.

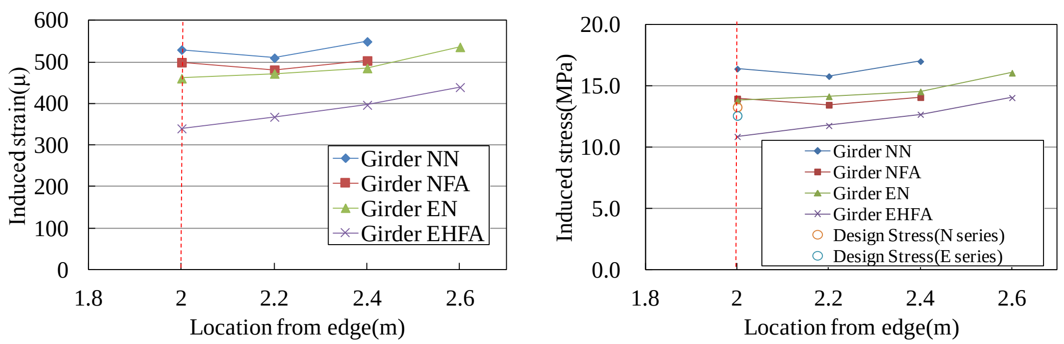
Girders, BS12-NN and BS12-NFA had the same curing process and same concrete as the concrete blocks, NC-2 and NFC-1. These girders showed the temperature of 62 ^{o}C at prestressing. The temperature requirement mentioned above has not been applied to these girders with normal steel strands.

Concrete Strain at Prestressing. Figure 5(a) shows the concrete strains of four girders induced at prestressing. Girders with normal steel strands, BS12-NN and BS12-NFA have three measuring points at 2.0, 2.2 and 2.4 m from the edge of girders. At 2.0 m where full strains are expected, Girder BS12-NN and Girder BS12-NFA show the strains of 530μ and 500μ, respectively and at 2.0, 2.2 and 2.4 m, the girders show similar values. This is an indication of constant strains from 2.0 m to the center of girders.

Girders with epoxy coated steel strands, BS12-EN and BS12-EHFA have four measuring points. Girder BS12-EN shows the strain of 460μ at 2.0 m, which is smaller than the strains of Girder BS12-NN and Girder BS12-NFA mentioned above. This difference is considered as the result of lower bond performance of epoxy coated steel strands. The strains at 2.0, 2.2,

2.4 and 2.6 m are increased from 460μ to 540μ, indicating the increase of the bond performance further than 2.0 m.
Girder BS12-EHFA shows the smallest strain of 340μ at 2.0 m. Strains at 2.0, 2.2, 2.4 and 2.6 m are increased from 340μ to 440μ, indicating that higher strains than 440μ are expected at the points further than 2.6 m. The smaller strains shown above are considered as the result of lower bond performance of epoxy coated steel strands and fly ash concrete. This result indicates that a longer anchoring length is required to Girder BS12-EHFA.

Figure 5(b) shows the induced stresses at the same points as the strains. Stresses are calculated from the registered strains, elastic modulus and the linear Hook's Law equation. For Girder BS12-NN, using the strain of 530μ at 2.0 m and the elastic modulus of 29 GPa, a concrete stress of 15.4 MPa is calculated. Similarly, for Girder BS12-NFA, using the strain of 500μ and the elastic modulus of 29 GPa, a concrete stress of 14.5 MPa is calculated. At the same point, a design stress of 13.5 MPa is calculated. This result shows that the induced stresses are higher than the design stress.
Girder BS12-EN is reinforced with epoxy coated steel strands. For this girder, using the strain of 460μ at 2.0 m and the elastic modulus of 30 GPa, a concrete stress of 13.8 MPa is calculated. At the same point, a design stress of 12.6 MPa is calculated. This result also shows that the induced stress is higher than the design stress.
For Girder BS12-EHFA, using the strain of 340μ and the elastic modulus of 32 GPa, a concrete stress of 10.9 MPa is calculated at 2.0 m. This stress is 13.5 % lower than a design stress of 12.6 MPa. However, this stress complies with the allowable concrete stresses just after prestressing and with the required stresses under all external design loads. Referring to the stress distribution, the higher stresses than 10.9 MPa are expected at the points further than 2.0 m.



(a)Induced strains **(b)Induced stresses**

Figure 5. Induced strains and stresses versus measuring points

Bending Test. Bending test to failure was performed based on JIS A5373(JIS, 2010). Crack load is defined as the load causing the first crack, and failure load is the load causing cracks on upper concrete surface. Girder BS12-NN and Girder BS12-NFA present a design crack moment of 473kN・m and a design crack load of 137 kN. Girder BS12-EN and Girder BS12-EHFA present a design crack moment of 440 kN・m and a design crack load of 124 kN. Applying the design crack load of 137 kN, Girder BS12-NN and Girder BS12-NFA present a design deflection of 21.4 mm. Similarly, Girder BS12-EN and Girder BS12-EHFA present a design deflection of 19.4 mm. All girders present a design failure moment of 972 kN・m and a design failure load of 310 kN.

At the bending test, Girder BS12-NN and Girder BS12-NFA showed higher crack loads of 190 kN and 185 kN, respectively than a design load of 137 kN. Both girders showed the same failure load of 321 kN, which was higher than a design load of 310 kN.
Girder BS12-EN and Girder BS12-EHFA showed higher crack loads of 185 kN and 184 kN, respectively than a design load of 124 kN. Local breaking failure occurred on upper concrete surface at 315 kN and 318 kN, respectively which were higher than a design load of 310 kN.

Concrete strains on upper and lower surfaces of the four girders are shown in Figure 6, where the mean values of four measured strains are shown. The four girders present almost the same load-strain relation up to a load of 180 kN, and similar bending behaviours are observed up to failure. The result shows a higher elastic limit than design crack loads of 137 kN and 124 kN.

Deflections of the four girders are shown in Figure 7, where the girders show almost the same bending behaviour. At the design crack load, Girder BS12-NN and Girder BS12-NFA show deflections of 20.6 mm and 19.9 mm, respectively which are smaller than a design deflection of 21.4 mm. Also, Girder BS12-EN and Girder BS12-EHFA show deflections of 18.1 mm and 16.7 mm, respectively which are also smaller than a design deflection of 19.4 mm. At the design crack load, the deflections of the four girders are smaller than the design values, and the linear relation between load and deflection is kept up to 200 kN.

Test results show that the four girders maintain similar linear load-deflection and load-strain relations up to higher loads than the design crack loads. These linear relations present higher safety for cracks and also ensure the higher load bearing capacity of the girders.

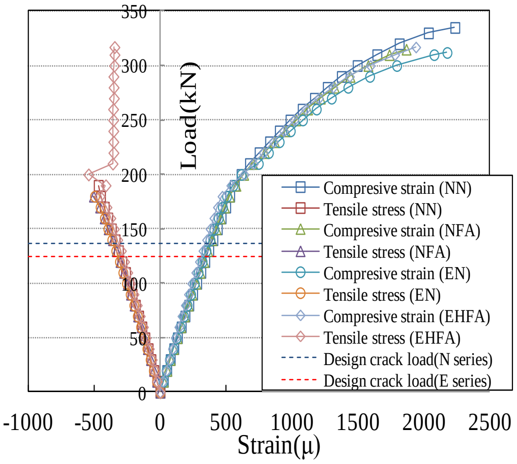


Figure 6. Loads versus strains

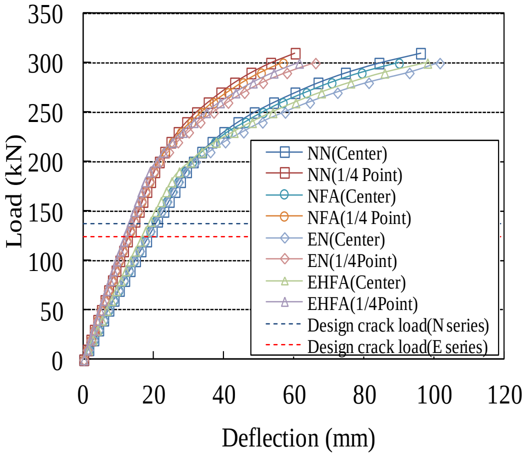


Figure 7. Loads versus deflections

CONCLUSIONS

This study presents the test results of temperature history and strength performance of the concrete blocks with normal and fly ash concrete under steam curing. Also, the strains induced at prestressing and the load bearing capacity of PC girders are presented. From the test results, the following conclusions are obtained.

(1) Fly ash concrete is able to control lower concrete temperature and ensures the specified compressive strength for PC girder under the specified steam curing.

(2) PC girders with normal and fly ash concrete reinforced with N strands and with normal concrete reinforced with E strands present higher induced stresses than the design stresses at the anchoring length.

(3) PC girder with fly ash concrete reinforced with E strands presents smaller induced stress than the design stress at the anchoring length. However, the stress complies with the allowable concrete stresses at prestressing and the required stresses for all external design loads.

(4) All girders maintain similar linear load-deflection and load-strain relations up to higher loads than the design crack loads, and these results ensure higher safety for cracks.

(5) All girders show higher crack loads and failure loads than the design loads, and the results ensure higher load bearing capacity for all girders.

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