Experimental and numerical studies on the control of horizontal cracking at the ends of hollow-type pretensioned girders

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Abstract

Recent bridge designs have created several efficient pretensioned prestressed concrete girder sections with high levels of prestress. Among them, hollow-type sections are of more consideration in Japan that exhibit good performance in practice. The transfer of large stresses from strands to concrete causes these sections to undergo horizontal cracking at the ends of the girders. To avoid such problem, this paper presents the implementation of various methods to the ends of two specimens prepared physically. Horizontal end cracks could not be fully eliminated by either of the methods in the selected type girder, but the method of enhancing end-zone reinforcement alongside strand-debonding could considerably decrease the principal stress that resulted in minimizing the cracks to a negligible level. To validate the experimental program, numerical models of the girder with identical parameters were simulated using the Finite Element Analysis software. Results obtained numerically well-matched the experimental ones to the extent of showing even the cracks pattern and vicinity.

Keywords Horizontal cracks · Prestressed concrete · Hollow pretensioned girders · MIDAS FEA

1 Introduction

In civil engineering field, pretensioned Prestressed Concrete (PC) girders receive great attention and popularity over other superstructure members due to their ability to efficiently span longer distances due to higher load carrying capacity [1]. As a result, a huge number of prestressed concrete girders are used every year for constructing bridge structures [2]. Prestressed members are highly recommended in view of cost-effective casting in plants under higher quality control, and their durability as low maintenance structures [1]. Although, it is well-known that prestressed concrete functions well for large spans, field observations and experimental studies showed that, some types of these prestress elements exhibit horizontal end cracking due to large prestressing forces [3]. The

cracks develop during the girder fabrication [4] and appear right after the prestress release process and they become noticeable at the time of lifting the girders from the prestressing bed [5]. Bridges with such type of cracked girders are of much more concern compared to others because the bridge supports are corrosion susceptible locations due to the deck expansion joints where the girder ends are located [6]. In the exposed situation, such cracks tend to get wider as the embedded steel gets corroded. [7, 8]. This may include girder durability problems, corrosion of strands leading to adverse effects on the structural capacity [9], and even the problem of the catastrophic collapse of the structure, which may result in loss of human lives and high expenditure for maintenance or replacement [10]. Observations showed that, girders having cracks resulted during manufacturing stage may cause an

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increase of maintenance cost throughout the life cycle of the bridge. Moreover, these cracks discourage engineers to use deeper sections or it may result in the total rejection of the utilization of prefabricated girders in constructions [2, 11]. The engineers desire the girders to be crack free at the time of erection at a bridge site [12]. However, the smaller cracks can be sealed and may be acceptable, as long as they don't pose durability concerns [13].

Several earlier studies [3] showed that the main cause of horizontal end cracking is the transferring of prestressing forces from tendons to the surrounding concrete that results in stress concentration at the end-zone of the element and the control of these stresses is quite a challengeable task [3, 5]. The transfer of prestressing forces from strands to concrete at the end-zone of PC-girders result in a complex stress state that can lead to concrete cracking [14]. Prestressing force creates compressive stresses that propagate into the member in a curved pattern until a linear stress distribution result [15], as shown in Fig. 1. The prestress force is introduced gradually by bonding between steel and concrete over a distance known as the transfer length as shown in Fig. 1. Transfer length is generally defined as the bond length required to fully transfer the effective prestress from strands to the concrete [16–19]. There are various factors that can have influence

in the transfer length, such as the type, number and diameter of prestressing tendons, effective prestress [20–22], concrete strength at the prestressing time [23–25], concrete cover [22, 25], strand releasing method, surface condition and many more [20, 26]. Due to the bond effects, compressive stresses radiate from strands into concrete, therefore, a zone of compressive stresses develops acting radially towards the strands, the dispersion of these compressive stresses causes tensile stresses to develop normal to the direction of prestressing forces [27, 28]. If the magnitude of these vertical tensile stresses (principal stresses) is more than the tensile strength of concrete, horizontal end cracks develop.

Horizontal cracks are observed at the ends of various shapes girders, such as box girders, I-shaped girders, hollow slabs, and tee beams [29]. Several studies have been conducted to control horizontal cracking at the ends of above-mentioned type girders. However, there are just few researches on hollow-type girders whereas they are very vulnerable to the development of horizontal end cracks due to prestressing forces. Actual horizontal cracks at the end of hollow girder are shown in Fig. 2.

Hollow girders are valuable alternative PC-girders for bridge construction as they have extremely light weight and consume limited amount of material [31]. Although these girders are hollow along the entire length, nonetheless they have a good load bearing capacity to resist service loads.

The purpose of this study was to eliminate or minimize horizontal cracking at the ends of pretensioned hollow-type PC-girders. Several sections were selected from JIS (Japan Industrial Standards) BS-sections (BS represents B-live load slab girder and the number illustrates span length and the number of strands of the girder) for the experiments [32]. Among them, BS22 type was known to be quite vulnerable to horizontal end cracks. Considering its maximum height of (900 mm) and the number of PC-strands, the girder needs a large amount of prestressing forces that may cause horizontal cracks at the ends of the girder. MIDAS FEA was adopted as the Finite Element Analysis software for simulating the girder's models. First, two

Fig. 1 Bond stress distribution at the strand end [30]

Fig. 2 Horizontal cracks at the end of pretensioned hollow PC-girder

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Fig. 4 Reinforcement details of BS22 girder [32] (Unit: mm)

Fig. 5 Debonded strands pattern in Specimen-A (● Debonded Strand, Units: mm)

2.3 Design procedure of end-zone reinforcement

End-zone reinforcement was used in the both ends of Specimen-B. According to the design procedure [35], steel area was calculated from the following equation,

$$A_s = 0.4 \times A_{ps} \quad (1)$$

where A_{ps} is the total area of prestressing steel, which is equal to 3051.4 mm² for BS22 girder. The total of steel area was calculated from Eq. (1) as 1220.56 mm². D10 or 10 mm steel bars were used as end-zone reinforcements, cross sectional area of a bar is 78.5 mm², and the number of stirrups to be placed at the end of girder was obtained as 16 bars. At least 50% of obtained steel area should be placed within a distance ($h/8$) from the girder end [35].

However, it is hard to place 8 stirrups within 112.5 mm length. Therefore, the designed steel bars were arranged as a mesh and it is referred as the end-mesh. The end-mesh was placed at a distance of 40 mm from the girder end as shown in Fig. 7.

2.4 Instrumentation

Five 100 mm long (KM-100) type strain-gauges were embedded at each end of Specimen-A and at the debonded end of Specimen-B as shown in Figs. 8 and 9. Considering high cost of strain-gauges and budget limitations, the gauges were not embedded at the fully bonded end of Specimen-B.

Prestress transfer procedure was implemented 15 h after concrete casting. For detensioning, stresses were

Fig. 6 Debonded strands pattern and used end-mesh in Specimen-B (Units: mm)

Fig. 7 End-zone reinforcement mesh made of D10 bars

Fig. 8 Embedded strain-gauges in Specimen-A and B (O Embedded gauge, Units: mm)

gradually decreased on PC-strands through hydraulic jack until all strands were completely detensioned in both specimens. Pretensioned and detensioned strands of specimens are shown in Figs. 10 and 11, respectively.

Two displacement-meters were embedded on two fully bonded strands in the second horizontal row at the fully bonded end of each specimen (Fig. 11) for recording the PC-strand displacement during the detensioning period.

Fig. 9 Strain-gauge locations and orientations marked on the side view (Unit: mm)

Fig. 10 Pretensioned PC-strands of both specimens on one prestressing bed

Fig. 11 Embedded displacement-meters and detensioned PC strands

2.5 Concrete compressive strength testing

Three concrete cylinders of 10 cm in diameter and 20 cm of height were prepared at the time of concrete casting for evaluating the concrete compressive strength. The cylinders were cured by steam curing together with girder specimens for 15 h (Fig. 12). Compressive test was carried out according to the procedure of JIS A 1108 [36] in the laboratory (Fig. 13).

2.6 Experimental results

2.6.1 Concrete compressive strength

The compressive strength and young modulus of concrete acquired from the laboratory testing are given in Table 1. The purpose of this testing was to confirm whether the concrete has attained the required strength for the pre-stress application or not. Based on JIS A 5373 [32], the concrete compressive strength at the time of prestress application process has to be 35 N/mm² or more. The fact is that the girders and cylindrical samples should be steam cured simultaneously under identical conditions. Since the cylinders are smaller in dimensions than the girders, thus the girder could not attain the same strength as the cylinders achieve in the selected period of steam curing. Table 1 illustrates the compressive strength values of cylinders higher than 35 N/mm² which confirmed the required strength of the girder for prestressing application.

2.6.2 Strain readings

Strain readings right after the strand releasing process were obtained from the embedded gauges, these strain values were transformed to their corresponding effective

Fig. 12 Steam curing of girder specimens and concrete cylinders

Fig. 13 Compression test of concrete cylinder in the laboratory

stresses utilizing Hook's law (Eq. 2). Measured strains and their corresponding effective stresses at the debonded ends of Specimen-A and B are given in Tables 2 and 3 respectively. Acquired strains and their corresponding effective stresses at the both ends of Specimen-A and at the debonded end of Specimen-B are shown in Figs. 14 and 15. The stresses and strains at the fully bonded end of Specimen-B are not illustrated because of not embedding strain-gauges at that end. Figures 14 and 15 show that, the effective stress and strain distribution within the transfer length is in a curved pattern and satisfy the standard stress distribution of Fig. 1.

Effective stresses are the compression stresses transferred from PC-strands to concrete at the time of strands releasing. The design value of effective stress (σ_{ct}) at the centreline of the lower most fibre of the girder concrete was calculated by the Eq. (3) [32],

$$\sigma = E \times \epsilon \quad (2)$$

$$\sigma_{ct} = \sigma_{pt} \times \sum A_p \times \left(\frac{1}{A'} + \frac{Y_e}{Z} \right) \quad (3)$$

In Eq. (2), E shows the concrete modulus of elasticity, ϵ and σ are the acquired strain and its corresponding effective stress, respectively. In Eq. (3), σ_{pt} is the prestress amount which exists in the strands just after the tensioning, $\sum A_p$ is

Table 1 Concrete compressive strengths acquired from laboratory testing

Cylinder no.	Compressive strength (N/mm ²)	Mean (N/mm ²)	Young's modulus (N/mm ²)	Mean (N/mm ²)
1	45.6	46.3	31,284.9	31,180.1
2	45.7		30,798.7	
3	47.7		31,456.7	

Table 2 Measured strains and computed stresses in Specimen-A

Gauge number	1	2	3	4	5
Gauge distance from the end (mm)	450	600	750	900	1050
Measured strain (μs)	517.92	576.85	610.05	618.35	574.36
Effective stress (N/mm^2)	16.1	18.0	19.0	19.3	17.9

Table 3 Measured strains and computed stresses in Specimen-B

Gauge number	1	2	3	4	5
Gauge distance from the end (mm)	450	600	750	900	1050
Measured strain (μs)	475.59	555.27	605.07	590.13	577.68
Effective stress (N/mm^2)	14.8	17.3	18.9	18.4	18.0

Fig. 14 Effective strains measured through strain-gauges in Specimen-A and B **Fig. 15** Effective stresses calculated from measured strains in the concrete of Specimen-A and B

the cross-sectional area of all PC-strands, A' is the cross-sectional area of concrete, Y_e is the distance from the lower most fibre to neutral axis of the transformed section, Z is

the section modulus of the lower most fibre. The design value of effective stresses at the centerline of lower most

fibre of the girder concrete was calculated from the Eq. (3) as 20.505 N/mm².

2.6.3 PC-strand displacement

Strand displacements for fully bonded strands in both specimens were recorded during the de-tensioning period. Two displacement-meters were mounted on two fully bonded strands of specimen-A and two other displacement measurements were embedded on two fully bonded strands of specimen-B. The results that demonstrates load–displacement relationship is shown in Fig. 16.

2.6.4 Inspection of horizontal cracking at the ends of specimens

Horizontal end cracking was monitored at the time of prestress releasing and after the completion of the de-tensioning process. The cracks started appearing during the detensioning process at the ends of Specimen-A prior the Specimen-B. Horizontal cracks occurred due to the transfer of prestressing forces from tendons to the concrete and the cracks began with quite small in size and then widened gradually as the transfer forces increased. Figures 17 and 18 show the ends of both specimens where the cracks occurred.

3 Numerical analysis

Finite element models were created for BS22 girder with the consideration of actual dimensions using MIDAS FEA software. Four specific cases were analyzed to simulate measured responses of the specimens during experiments.

Fig. 16 Relationship between fully bonded strand displacement and released prestress load

In Case #1 and #2, principal stress condition at the both ends of Specimen-A were analyzed and in Case #3 and #4, two models were analyzed to simulate the stress state at the ends of Specimen-B. Non-linear analysis was performed to analyze the concrete material with the consideration of its non-linear behavior. In the analysis, the main purpose was to check the principal stress distribution along the girder height at the ends of the girder upon the application of prestressing forces to the concrete. Principal stresses were measured at the vertical edges of the girder end in this study.

3.1 Model properties

As the geometry and loading conditions of the girder are symmetric, thus the model size was reduced to one-fourth of the full model. Material properties of both experimental specimens and FEA models were the same. In this modeling, the smeared crack approach was selected in which the stresses and strains of concrete and steel are evaluated by average or smeared values crossing several cracks. The Total Strain Crack Model (Rotating Crack Model) was utilized to model the concrete behaviour. Hordijk and Parabolic functions were used to model the concrete in tension and compression, respectively. The Von Mises model was used to model PC-strands and Steel reinforcement bars. Bond stress and threshold value of slippage for PC-strands were assumed to be 5.0 N/mm² and 0.2 mm respectively, and constitutive laws of materials used in the analysis are shown in Fig. 19.

According to JIS A 5373 [32], the design value of concrete compressive strength at the time of prestress application is 35 N/mm². Generally, it is assumed that the desired experimental results can be obtained from FEA analysis by selecting the design value (35 N/mm²) of JIS A 5373 for concrete compressive strength. Therefore, the design value (35 N/mm²) was used as the concrete compressive strength at the prestressing time in the numerical analysis. Other material properties of concrete were obtained from the utilization of the compressive strength from the following equations [37] and are given in Table 4.

$$\text{Tensile strength: } f_t = 0.23 \times f_c^{\frac{2}{3}} \quad (4)$$

$$\text{Tensile fracture energy: } G_t = 10 \times d_{max}^{\frac{1}{3}} \times f_c^{\frac{1}{3}} \quad (5)$$

$$\text{Compressive fracture energy : } G_c = 8.8 \times f_c^{\frac{1}{2}} \quad (6)$$

For the nonlinear analysis, three dimensional hexahedral elements were used for modelling of concrete, loading plate, and supports. Hexahedral elements are one of the

Fig. 17 Horizontal cracking at the ends of Specimens-A

convenient types of solid elements and these elements are quite practical to model in arbitrary shapes when an arbitrary mesh is necessary. Steel reinforcement bars were created as “Embedded Reinforcement Bar Element”. Rather than defining reinforcements with distinct finite elements, the concept of embedded reinforcements can be used. In this concept, the stiffness of the reinforcements is added to the stiffness of the continuum elements in which the reinforcements are located. PC-strands were created as 2-node linear three-dimensional truss elements. Truss elements can be used for both static (linear & nonlinear) and dynamic analyses. A truss element transmits only axial forces and may be combined with tension-only/compression-only functions [38]. Compressive forces were applied to truss elements as prestressing forces. The mesh size was selected as 50 mm for the models and the generated mesh is shown in Fig. 20. End-zone reinforcement was used at the both ends of Specimen-B. Figure 21 shows half model of the end-mesh placed at a distance of 40 mm from the end of Specimen-B model.

In Fig. 19, f_c shows concrete compressive strength, f_t is tensile strength of concrete, G_{fc} represents compressive fracture energy, G_{ft} is tensile fracture energy, ϵ_{cu} denotes the ultimate strain of concrete in compression, ϵ_{tu} is the ultimate strain of concrete in tension, h is the characteristic element length, d_{max} is maximum size of the coarse aggregate, and f_{ud} is the design tensile strength of PC-strands. The values chosen for h and d_{max} in this study are 40.34 mm and 20 mm, respectively.

3.2 Numerical results

3.2.1 Principal stresses

Four numerical cases were analyzed for simulating both specimens end conditions. In Case #1, all PC-strands were fully bonded and the stress contours and principal stresses in the concrete of the girder end-zone were as shown in Fig. 22. In Case #2, four strands were debonded in the first horizontal row and principal stresses at the end of model

(a) Specimen-B (Fully Bonded Strands + End-Mesh)

A line drawn on the crack for visibility

(b) Specimen-B (Debonded Strands + End-Mesh)

A line drawn on the crack for visibility

Fig. 18 Horizontal cracking at the ends of Specimen-B

are shown in Fig. 23. In Case #3, all PC-strands were fully bonded and one end-mesh (Fig. 21) was placed in a distance of 40 mm from the girder end, principal stresses of Case #3 are shown in Fig. 24. In Case #4, four stands were debonded in the first horizontal row and one end-mesh was placed 40 mm distant from the girder end. Principal stresses for Case #4 are shown in Fig. 25. The principal stresses in Figs. 22, 23, 24 and 25 were found to be greater than the tensile strength of concrete in some portions of the height, therefore, horizontal cracks were likely to occur in all cases. However, the magnitude of principal stresses in Case #4 was very close to the tensile strength of concrete, which shows that the cracks would be narrow and short for Case #4 as observed during the experiments (Fig. 18b).

3.2.2 Stresses in reinforcement bars

The steel bars assisting in carrying loads after cracking occurs, thus determining stresses or strains in the reinforcement bars were essential to evaluate the efficiency of the bars. The stresses in reinforcement bars of the one-fourth portion of the girder are shown in Fig. 26.

4 Discussion

4.1 Comparison between experimentally developed cracks

At the ends of specimens, the width, length and location of horizontal cracks were measured with crack-scale

Fig. 19 Constitutive laws

Table 4 Concrete material properties of BS22 girder model

Analysis type	Material properties				
	Compressive strength (N/mm ²)	Tensile strength (N/mm ²)	Modulus of elasticity (GPa)	Compressive fracture energy (N/mm)	Tensile fracture energy (N/mm)
Pre-stressing	35	2.46	29.5	52.1	0.1

measurement device and their geometries are drawn and shown in Figs. 27, 28, 29 and 30.

Figure 27 shows the widths of horizontal cracks at the end of Specimen-A with fully bonded strands in the range of 0.3–0.35 mm. As shown in Fig. 28, the width of cracks at the end of Specimen-A with four debonded strands is 0.1 mm and this crack width is very small compared with the other end. As seen in Fig. 29, crack width is 0.06 mm at the end of Specimen-B with fully bonded strands and an end-mesh, which was placed 40 mm distant from the end of the girder. Figure 30 shows horizontal cracks with 0.04 mm width occurred at the end of Specimen-B with four debonded strands and one end-mesh, and this width size of cracking is very small compared to the opposite end.

As seen in Figs. 28, 29 and 30, cracks width for the last three cases (Case #2, #3 and #4) are within the acceptable range. Furthermore, these cracks do not result in the reduction of structural capacity of the girder so that there is no durability concern [34]. Horizontal cracks with a width of less than 0.3 mm do not affect structural capacity and durability of the girder [34]. For safety and maintenance purposes, cracks should be filled up with cement slurry to avoid the ingress of chlorides and other corrosive chemicals. Considering the aim of the study, the crack width and length at the girder end with four debonded strands and one end-mesh are so small compared to other cases. For this case, length of the cracks was reduced to 50 mm and 100 mm along the longitudinal axis of the girder. Therefore, Case #4 was found as the most efficient and recommended case to be adopted in application.

Fig. 20 Half cross section and mesh division of BS22 girder model (Unit: mm)

Fig. 21 Half model of end-mesh made from D10 bars

Fig. 22 Principal stresses at the end of girder with fully bonded strands

Fig. 23 Principal stresses at the end of BS22 girder with four debonded strands

Fig. 24 Principal stresses at the end of girder with placement of end-mesh

Fig. 25 Principal stresses at the end of girder with placement of end-mesh and four debonded strands

Fig. 26 Resultant stresses in the reinforcement bars

Fig. 27 Horizontal cracks at the fully bonded end of Specimen-A (Unit: mm)

Fig. 28 Horizontal cracks at the debonded end of Specimen-A (Unit: mm)

Fig. 29 Horizontal cracks at the fully bonded end with end-mesh in Specimen-B (Unit: mm)

Fig. 30 Horizontal cracks at the debonded end with end-mesh in Specimen-B (Unit: mm)

Fig. 31 Comparison of effective stresses computed from MIDAS FEA and experiment

4.2 Comparison of experimental and numerical results

A comparison of effective stresses obtained from experiment and numerical simulation is shown in Fig. 31. As noted from Fig. 31, effective stresses computed from MIDAS FEA are in good agreement with those measured in the experiment and the theoretical design value of concrete effective stresses obtained from Eq. (3).

5 Conclusion

Prestressing forces transfer from PC-strands to concrete, vertical principal stresses develop at the ends of pre-tensioned hollow PC-girders and horizontal cracking occurs right after the prestress application. The aim of this study was proposing solutions that could literally

control horizontal end cracking. For this ground, girder specimens were fabricated in the factory to meet the purpose of the study. The obtained experimental results were verified by numerically plotted results using MIDAS FEA software. The study founded that, in a typical hollow-type BS22 girder, horizontal cracks occur at the ends of the girder which is caused by the immediate development of large amount of principal stresses. By debonding specific number of strands in particular locations could directly decrease principal stresses, that could significantly reduce the number and size of horizontal end cracks. The most effective method for end-zone reinforcements to control these cracks at the girder ends is the placement of stirrups at the extreme ends of the girder. Arrangement of the designed rebars in a mesh pattern could be a good solution to place the required amount of end reinforcement at the very end of the girder.

Based on the findings of this study, only strand-debonding or placing end-zone reinforcement would not be able to eliminate horizontal cracking at the ends of BS22 girder. Combination of both, debonded strands and end-zone reinforcements are required in controlling the occurrence of horizontal cracks at the ends of the girder. This method could control the crack width and length to an acceptable range. Nevertheless, further research is needed to completely eliminate horizontal cracks at the ends of BS22 and similar type hollow girders by considering various parameters likewise; end-zone reinforcement diameter, the number, position and length of debonded strands and some others.

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Compliance with ethical standards

Conflict of interest All authors declare no conflict of interests.

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