

Effects of concrete strength on structural behavior of holed-incrementally prestressed concrete (H-IPC) girder

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(Received October 7, 2014, Revised June 22, 2015, Accepted June 25, 2015)

Abstract. Holed-Incrementally Prestressed Concrete (H-IPC) girders are designed using the following new design concepts. At first, web openings reduce the self-weight of the girder, and also diffuse prestressing tendon anchorages. The reduced end anchoring forces decrease the web thickness of the end sections. Additionally, precast technology help to improve the quality of concrete and to reduce the construction period at the site. For experimentally verification, two 50 m full-scale H-IPC girders are manufactured with different concrete strength of 55 MPa and 80 MPa. The safety, stiffness, ductility, serviceability and crack development of H-IPC girder are measured and compared with each other for different strengths. Both girders show enough strength to carry live load and good stiffness to satisfy the design criteria. The experimental result shows the advantages of using high strength concrete and adopting precast girder. The test data can be used as a criterion for safety control and maintenance of the H-IPC girder.

Keywords: H-IPC; holed web; incrementally prestressed concrete girder; precast girder; opening

1. Introduction

In review of the trend for bridge construction in the world, the construction rate of prestressed concrete (PSC) bridges has been steadily increased. Even in Korea, PSC bridges have been the dominant bridge type due to the outstanding economic advantages, accounting 90% of the constructed bridges in 1990. However, they were used for mid and short span below 50 m. (Castrodale 2004) For long span bridges above 50 m, steel box girders or concrete box girders are mostly used in 50-70 m span bridges. In case the girder height has to be lowered, preflex girders have been restrictively used. When longer spans are required, high cost bridges including PSC box girder bridges, extradosed bridges, arch bridges, and truss bridges are used. If the span range of the PSC I-type girder could be extended to 50-70 m, it can replace conventional high cost bridge types, saving much of the cost for materials and construction.

In addition, the precast method has been widely used for new installations, replacements, and

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repairs of old bridges. If I-type PSC girders could be manufactured in segments at a factory and assembled in the field, it can secure the quality of concrete as well as a long span. Also, this can decrease construction time, save indirect cost, and prevent increase of social cost due to the blocking of transportation flow. The research of the segmentation in the literature was focused on segmental box girders on which technology has been accumulated. Examples of segmental construction with I-type PSC girders can be seen. (Janssen and Spaans 1994)

Conventional PSC I-type girders have not been fully applied in the design of concrete girders longer than 50 m in spite of the structural efficiency and economic advantage. Because as the span becomes longer, girder depth and required cross sectional area increase for the introduction of huge prestress, resulting in an abnormal increase of bending moment and difficulty in the quality control and installation for field production of girders.

In this paper, the new design concept for H-IPC girders improving conventional PSC girders is to introduce multiple web openings and to apply multi-stage prestressing by distributed anchorage devices in the openings. By introducing web openings, the total weight of the girder is reduced and the installation of anchorage devices in the multiple web openings transferred from the end enables effective multistage prestressing, which can reduce compressive forces concentrated at the support end. The H-IPC can also be segmented for precast which enables excellent quality control of girders. Field assembles after the manufacturing of precast in the factory can minimize the field work. To verify the practical usefulness of H-IPC girders, two full size specimens were manufactured using concrete strength of 55 MPa and 80 MPa. Static loading tests were conducted to investigate the structural behavior of girders according to concrete strength.

2. Design concept

2.1 Introduction of web openings

Various studies (Thompson and Pessiki 2006) have been conducted on the members with web openings. It was reported that precast concrete members can introduce large openings while satisfying strength and serviceability requirements.

The shear force in web openings is distributed upward and downward, and the distributed amount is determined by the strength and area of cross section. If each chord is properly designed for the shear, tension, and compressive failure, the failure occurs due to the formation of a hinge at the end of chord member. This failure mechanism is similar to that of a reinforced concrete beam (Mansur 1998 and Mansur *et al.* 2006). However, more attention is necessary because cracks could be occurred around web openings of prestressed beams. To control the cracks, sufficient amount of rebars should be arranged around the openings. It has been reported that the serviceability such as load-bearing capacity or deformation is not much affected if rebars are properly reinforced at the opening areas (Abdalla and Kennedy 1995).

As shown in Fig. 1, the diameter of the opening of H-IPC is 1 m. The decrease effect of the flexural strength due to web openings is significantly less than it appears to be. The moment of inertia of the cross section with an opening is approximately 94% of the case without the opening. When the slab and girder perform composite action, the cross sectional area of the slab is added and this value increased to approximately 96% (Table 1). The reason why the effect of an opening on the flexural strength is very small is that the area of the web opening is close to the neutral axis of the girder, while the flanges of the girder occupy much part in cross sectional areas, and are

located far away from the neutral axis. So, the decreased value of the moment of inertia due to the opening is insignificant.

By the way, the arrangement of PC strands can be optimized suitable to the distribution of the bending moment by dispersing anchorage devices inside the web opening parts. This application can reduce the cross sectional area at the support end, so it can solve the disadvantage of the conventional method which must increase the cross sectional area near the support in order to endure excessive compressive force from all anchorage devices placed at the end.

2.2 Multi-stage prestressing

Prestressing by the post tension method according to the stress state of each loading step can be used to extend the bridge span and decrease the height of the girder compared to conventional design methods. Because the design of the girder by the multistage prestressing method requires separate prestressing for each loading step, all the loading steps and the corresponding stress changes by each loading step should be understood (Han *et al.* 2003).

The left figure of Fig. 2 shows the stages from assembling the segments of an H-IPC girder to the reinforcement with regard to damages that occur under service conditions. The right figure shows the stress changes of the girder that occur at the different stages. A segmental H-IPC girder is manufactured in segments, assembled and built at the site. The first stage is the assembly of the H-IPC girder on the building table. At this stage, because the load is not supported by the girder, it remains in a state of no stress.

It can be seen in Fig. 2(b) that the first prestressing and grouting are conducted on the ends of the girder after assembly. When the girder installed on the bridge pier, the dead load of the girder becomes active. The slab concrete is placed after installation and only acts as a load; it does not subject any external load because it is not composite part of the girder immediately after it is placed. After the curing of the slab, the slab and girder form a composite and begin to subject an external load. At this time, the second prestressing is conducted in the hollows as seen in Fig. 2(e). Figs. 2(a)-2(d) show the loading stages of the non-composite sections, and Figs. 2(e)-2(i) show the loading stages of the composite sections. As a result, the stress due to the maximum bending moment at the longitudinal center span remains within the range of the allowed stress under service loading conditions.

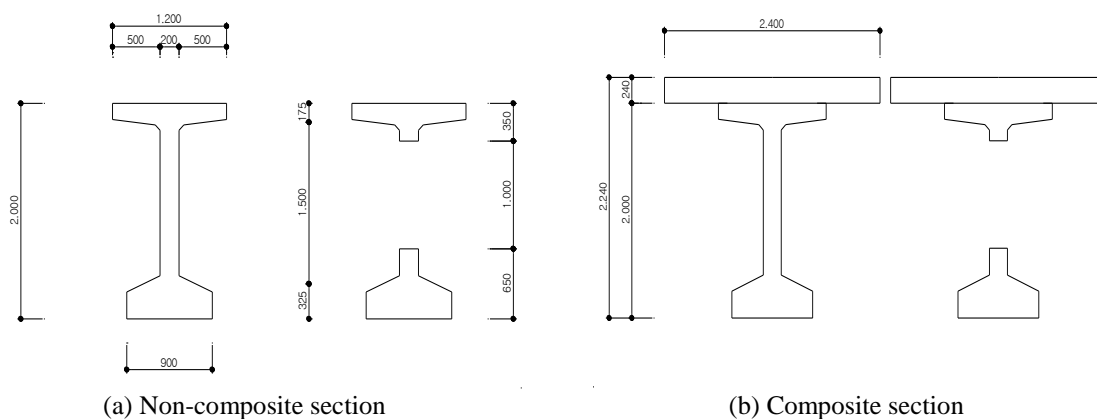


Fig. 1 Cross Section of a specimen

Table 1 Comparison of the cross section

Section		Area (A, mm ²)	Moment of inertia (I, mm ⁴)	Relative rate of I (%)
Non-composite	General	0.81×10^6	4.42×10^{11}	100
	Opening	0.61×10^6	4.17×10^{11}	94
Composite	General	1.38×10^6	8.92×10^{11}	100
	Opening	1.18×10^6	8.54×10^{11}	96

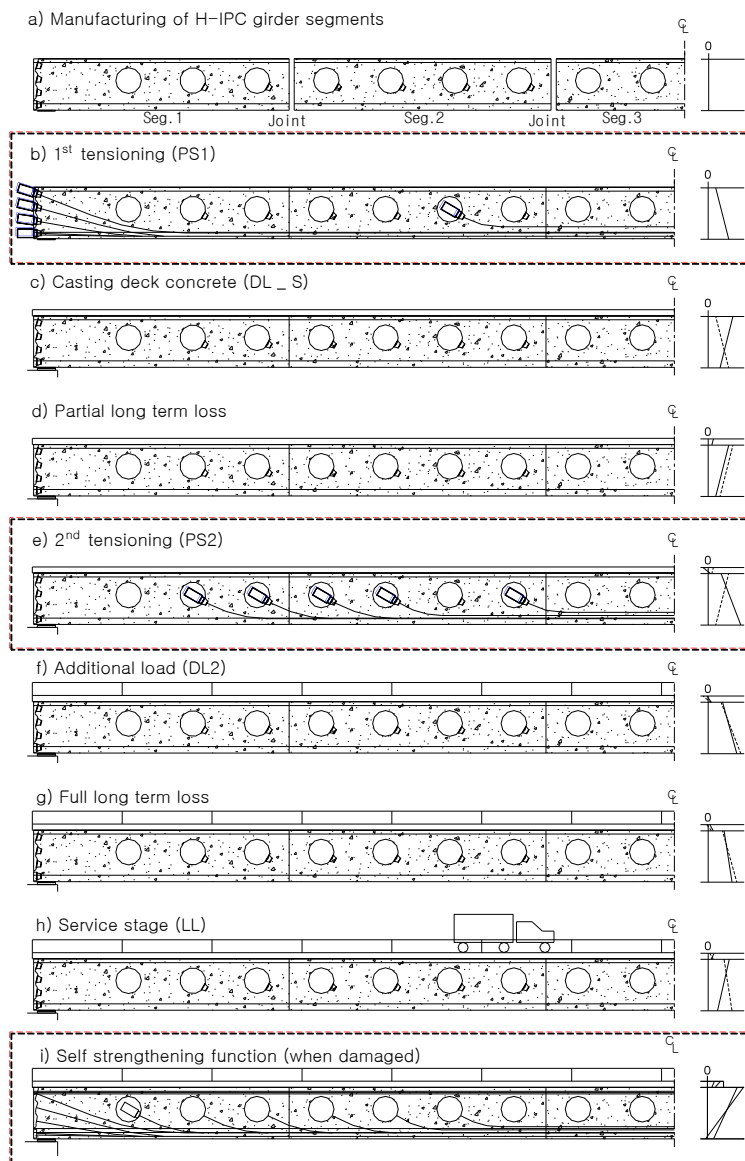


Fig. 2 Construction procedure of segmental H-IPC girder bridge

2.3 Segmentation

In the case of H-IPC segmental girders, the connection method to join the segments should be considered. A shear key was installed at the connection of segmental joint to effectively transfer shear and compressive force between members. Experimental results for evaluating the performance of shear key showed that multiple shear keys have excellent load resistance at the joint and effectively transfer shear force and compressive force of segments. Also, they have sufficient load-bearing capacities and less displacement compared to other shear keys, and do not require rebar reinforcement (Han *et al.* 2009, 2010, 2014).

In the meantime, a wet connecting method with epoxy was used at the joint. There is no significant increase of load-bearing capacity by the use of epoxy, but the epoxy makes it easier to assemble girders because epoxy acts as lubricant on the surface during the joint connection. In addition, it fills the gaps that can occur at the joint and prevents corrosion of tendons by blocking water permeation into the sheath.

A verification test conducted for the manufactured 5 segmental and the monolithic type also showed that there is a minimal difference of structural behavior between the segmental type and monolithic type. (Han *et al.* 2010, 2011)

Table 2 Material properties

Item	T55		T80			
	Girder	Slab	Girder	Slab		
Concrete	Design strength(MPa)	55	27	80	27	
	Elastic modulus (MPa)	33,800	25,300	37,800	25,300	
	f_{ca} (MPa)	Non-Composite	26.4	-	38.4	-
		Composite	24.8	10.8	36.0	10.8
	f_{ta} (MPa)	Non-Composite	-1.7	-	-2.0	-
		Composite	-3.7	-2.6	-4.5	-2.6
Rebar	Grade	SD 400				
	Yield strength (MPa)	400				
	Elastic modulus (MPa)	200,000				
PC Strand	Type	SWPC 7B				
	Diameter(mm)	15.2				
	Yield strength (MPa)	1,860				
	Elastic modulus (MPa)	200,000				

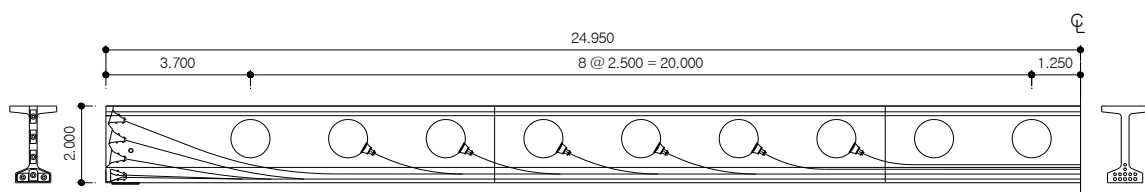


Fig. 3 Specimen configuration (left half of a girder)



(a) Placing of rebars and ducts



(b) Casting girder concrete



(c) Transportation of segments



(d) First prestressing and assembly of segments



(e) Erection of girder



(f) Placing of slab rebars



(g) Casting deck slab



(h) Second prestressing

Fig. 4 Construction procedure of segmental H-IPC girder bridge

3. H-IPC girder design and loading test

3.1 Design and manufacturing of Specimen

Recently many full-scale experiments have been performed to investigate precisely structural behavior of concrete (Smith *et al.* 2014, Laskar *et al.* 2010, Hsu *et al.* 2010 and Garber *et al.* 2014). In order to verify the structural behavior and the effect of concrete strength in H-IPC girders, full size girders were manufactured and static loading tests were conducted. Test specimens were constructed in 5 segments of a 10 m unit with the concrete strength of 55 MPa and 80 MPa. The span of the segmental H-IPC girder was 50 m and it was designed for a DL-24 load, similar to AASHTO truck load. The girder height was 2 m and the height to span ratio was 1/25. A web thickness of 200 mm was applied to meet the Korea design criteria considering the diameter of the sheath, covering depth of the concrete, and diameter of the shearing bar. By adopting an upper flange width of 1,200 mm, both the bending stress and the resistance to lateral buckling during transportation and construction were increased. A width of 900 mm and depth of 250 mm was applied to the flange to increase the bending stiffness and ensure sufficient space for the arrangement of the tendons. Material properties used in test specimen are shown in Table 2.

As shown in Fig. 3, there are 18 openings equally spaced in the web. There is no opening near girder ends where prestressing tendons are closely spaced and stress concentration in concrete is very high. There are 12 anchorage devices on half span of the girder; 6 of them are anchored at support end, and the other 6 are placed multiple inside of web openings.

As shown in Fig. 4, the manufacturing procedure for the test specimen is similar to that of general PSC with the exception that the second prestressing is added and web opening of the segmentation. The procedure for the manufacturing of the test specimen is as follows: a) placing of rebars and ducts, b) casting girder concrete, c) transportation of segments, d) first prestressing and assembly of segments, e) erection of girder, f) placing of slab rebars, g) casting deck slab, h) second prestressing.

3.2 Loading test

A loading test was conducted on the simply supported segmental H-IPC girder as shown in Fig. 5. The two-point loading was applied at distances of 2.5 m on either from the center of the span and used two hydraulic jacks of capacity 2,000 kN each.

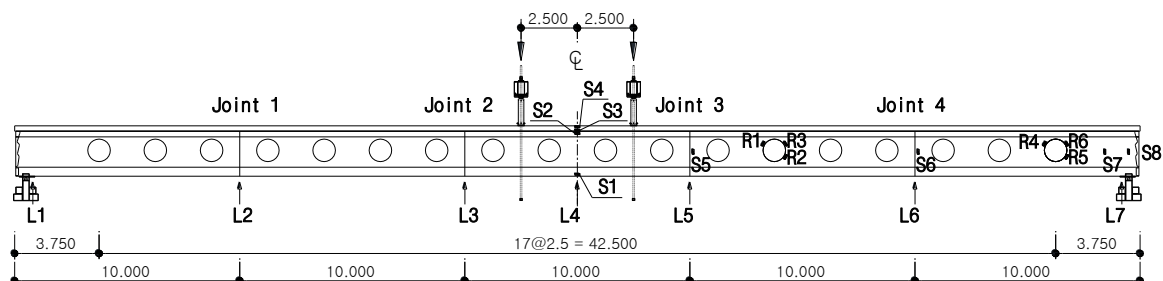


Fig. 5 Loading and measuring plan

In order to trace the structural behavior during the loading test, many strain gages were installed on the rebars of the tensile and compressive sides at the center span (S1-4), segmental joints (S5-S6), literally critical section (S7), near end (S8) and around the openings (R1-6). Also, displacement measuring devices (LVDT) were installed at the center span, segmental joints and each ends to measure the vertical deflection.

4. Experimental result

4.1 Load-deflection

The designed and measured load-deflection relations of the segmental H-IPC girder are compared in Table 3. When the live load (543 kN) including an additional dead load acts on T55 and T80 specimens, there was little difference of deflection between the measured and designed values. The designed flexural cracking loads were higher than the measured values by 29% in a T55 specimen, and 33% in a T80 specimen.

The measured deflection of T80 was 11% less than that of T55 subjected to the live load, and 12% under the cracking load. So it can be seen that the deflection decrease rate of T80 to T55 is almost same under the live load and cracking load conditions.

Fig. 6 depicts the load-deflection relation at the center span of the specimen. The load magnitude at the vertical axis is the sum of two vertical loads shown in Fig. 6. The overall flexural strength of T55 is a little lower than that of T80 specimen. The T55 girder shows a larger deflection up to 5-9%, comparing to T80 under flexural cracking load. However, the deflection difference between T55 and T80 is 27 mm (11%) under 2,000 kN and 73 mm (19%) under 2,500 kN. So, it shows that the load carrying capacity of T80 is better than T55, especially under the high load level exceeding flexural cracking load.

The Korea Highway Bridge Design Specification (KHBDS) requires that the deflection due to live plus impact load should not exceed 1/800 of the span for simple or continuous bridges, which is 61 mm for the current test girder. The measured deflection of T55 and T80, 50 mm for both girders, under design live load of 543 kN including an additional dead load well satisfied the specified criterion.

Table 3 Load-deflection comparison of T55 and T80

Specimens		T55				T80			
		Design		Measurement		Design		Measurement	
		Load (kN)	Deflection (mm)	Load (kN)	Deflection (mm)	Load (kN)	Deflection (mm)	Load (kN)	Deflection (mm)
Live load	LL	543	45.6	543	45.5	543	40.8	543	40.5
Load of deflection limit	P _{df}	939	78.9	866	78.9	1,046	78.6	946	78.6
Flexural cracking load	P _{cr}	1,810	152.1	1,440	157.2	1,861	139.8	1,500	137.5

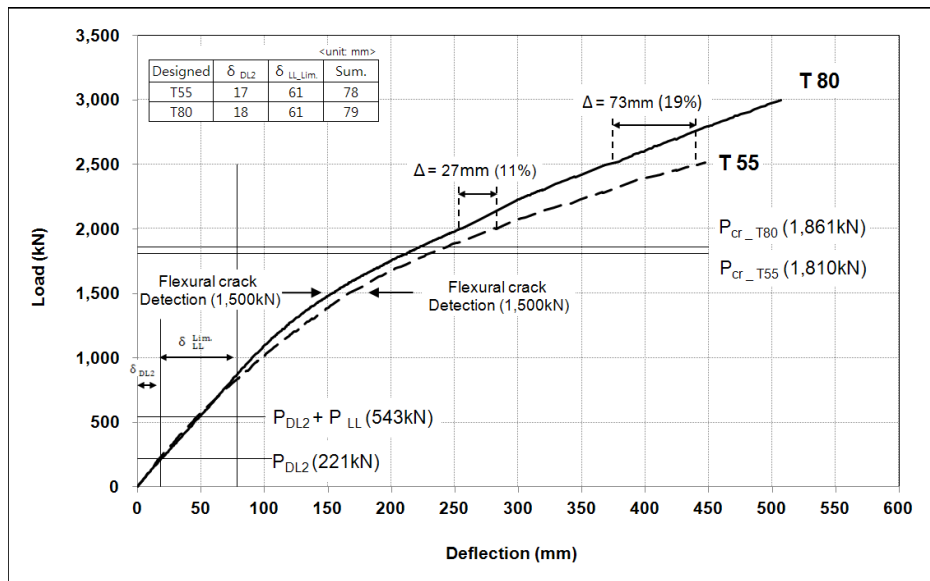


Fig. 6 Load-deflection relation

4.2 Load-strain

Fig. 7 depicts the load-strain curve of the main rebar at the bottom of center span, S1 shown in Fig. 5. It shows that the load-strain relation keeps almost linear for reaching the live load, but the strain of the bottom flange increases nonlinearly above 1,500 kN. Therefore, it could be difficult to find any problem in the behavior of the girder under the service load condition.

Fig. 8 shows the strain of vertical rebars located S5, S6, S7 and S8 shown in Fig. 5. It can be seen that the shear strains at the live load 543 kN are less than 50 micro-epsilon. Also, the shear cracks at the web opening for T55 and T80 are initiated at the load 800 kN and 1000 kN, respectively. The shear strains at S2 and S3 of T55 are abruptly increase after the initial shear crack is occurred. However, the shear strain of T80 does not rapidly increase until 1400 kN, which is 40% higher than the initial shear cracking load.

Fig. 9 indicates the longitudinal strains measured at different heights of the cross section. As the load increases, the strains in the slab and top flange are indicated as negative, which means compression, and the strains in the bottom flange are indicated as positive, which means tension. Since the strains are measured starting with loading test, they don't include the strains due to the first and second prestressing and the self-weight of slabs and girders.

In the figure, the neutral axes of the cross section were located at 1,260 mm and 1,300 mm in T55 and T80 respectively, which are similar, before flexural cracks took place. However, the location of neutral axes starts to move upward at the load 1,600 kN and 1,700 kN, respectively. It can be seen that as flexural cracks occur, the flexural strength of the member reduces and the neutral axis moves upward.

Load that neutral axis begins to move is larger than 1,500 kN of initial cracking load. Accordingly, it is judged that the initial cracks occurring at center of the girder was not affected greatly on the bending strength until they proceed somewhat and have some effects on the flexural strength only after the crack width has increased up to a visually checkable level.

The cracks around the opening were aimed to be controlled by diagonally arranging rebars around the opening. Of them, for a segmental girder, its behavior was measured by attaching a strain gage to each diagonal rebar.

As shown in Fig. 10, R3 and R6 are the strain measured from the rebar arranged in parallel with shear cracks from the opening, which shows that compressive strain increases continuously. On the other hand, R1, R2, R4 and R5 are the strains in the rebars arranged vertically to the shear cracks, showing tensile deformation, so all the gages of T55 and T80 show an similar behavior until the load reaches about 800 kN and 1,500 kN, respectively and then show independent, irregular deformation thereafter. This is a point of time when diagonal rebars started being subjected to deformation by the initial shear cracks occurring around the opening.

4.3 Crack pattern

The initiation and development pattern of shear and flexural cracks of T55 and T80 girders are shown in Fig. 11. There are no big differences in crack patterns between T55 and T80 girders. The diagonally inclined cracks are founded between web openings, and the flexural cracks at the bottom of center with large bending moment. It is difficult to quantitatively compare crack creation and progress angles, but the crack creation angle at the end is a little bigger than the angle at the center span.

Except the middle segment subject to large bending moment and small shear forces, shear cracks are created nearly horizontally below the web openings. Therefore, it will be efficient in controlling these horizontal cracks to arrange rebars vertically below the web openings. Also, it can be seen that the web shear cracks are not disconnected at the segmental joints but are prolonged. This shows that the joint of the segmental girder behaves integrally.

Final shear cracks were observed to be close to failure of frame patterns. So, the design of rebars near the web openings needs to be different to the normal I-type girders with solid cross sections. It is judged that it will be proper to use a strut-tie analysis model for determining the detailed amount of rebars near web openings, so additional researches are necessary.

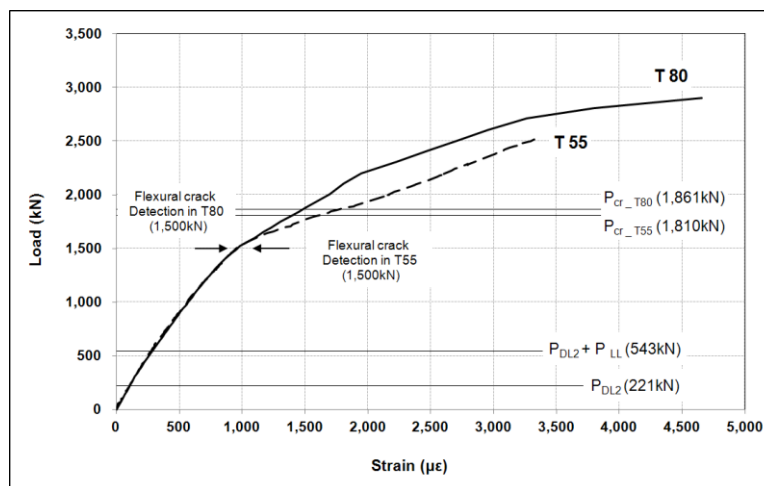


Fig. 7 Strain of the main rebar at bottom of center span(S1)

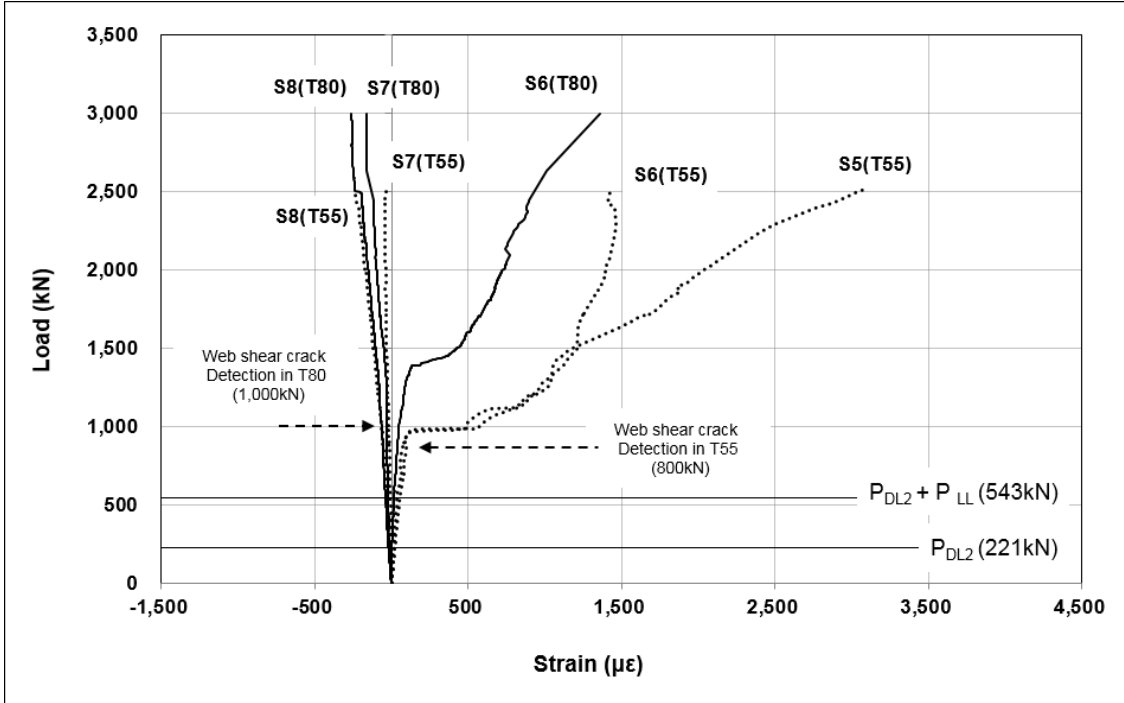


Fig. 8 Strains of vertical rebar

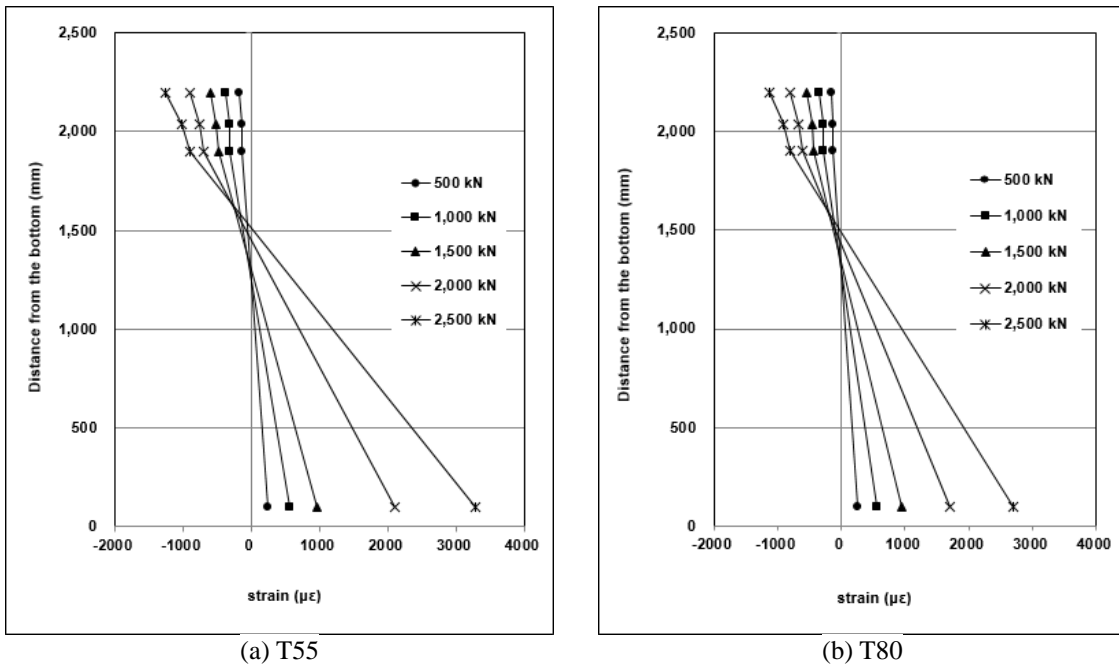
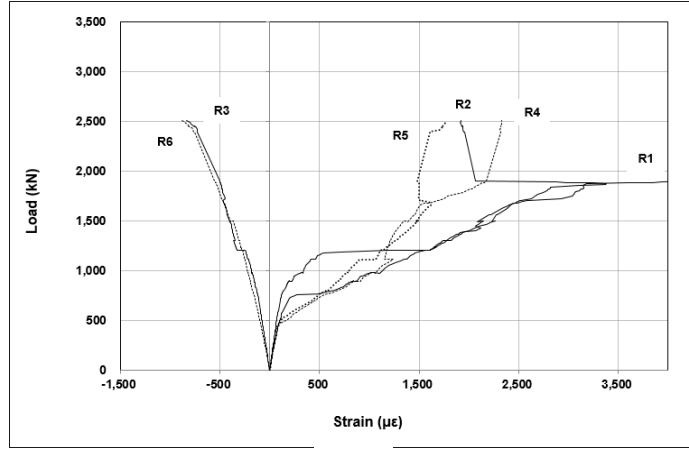
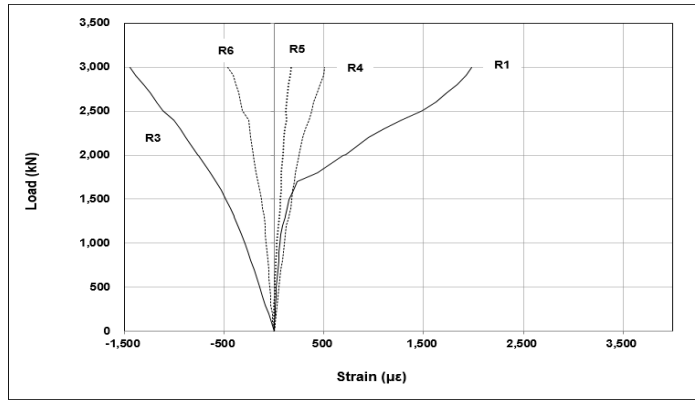


Fig. 9 Strain of vertical section

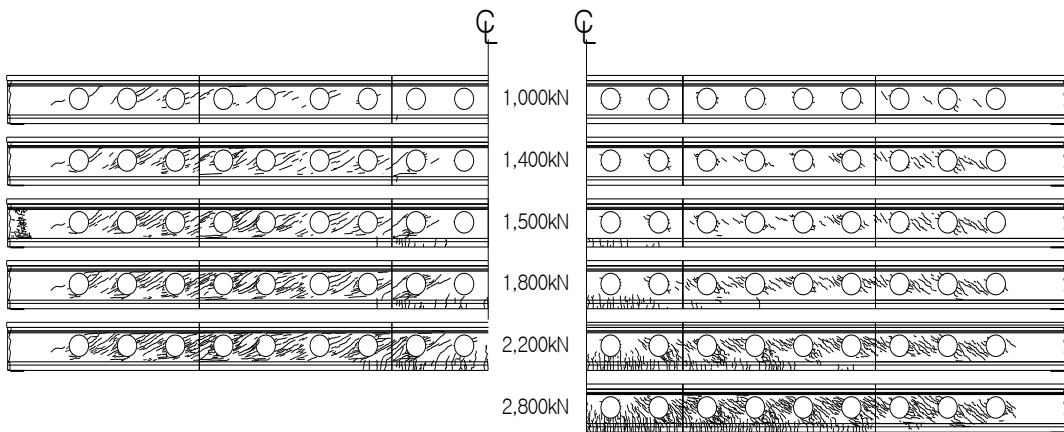


(a) T55



(b) T80

Fig. 10 Strain of rebar near opening



(a) T55

(b) T80

Fig. 11 Crack pattern

5. Conclusions

The design and experiment of Holed Incrementally Prestressed Concrete (H-IPC) were presented. Introducing the prestressed openings in the web, it is possible to extend span length longer than 50m. Several conclusions are made from design and experimental results.

- A new design concept of H-IPC girders is proposed a) pre-cast fabrication and quality control of girders are possible by girder segmentation, b) web openings are able to distribute the prestressing forces through multiple anchorages, c) efficient design of cross section is enabled by adopting a multi-stage prestressing method.
- Higher drops in the in-plane natural frequency are observed when the crack is located near the roots or corners of the frames.
- It is judged that H-IPC segmental girders are able to be applied to long span bridges over 50 m because reduction of flexural strength by web openings are very small, and disadvantage effects on girder segmentation are not big enough.
- Although both specimens of T55 and T80 satisfy the design criterion, it is desirable to use high-strength concrete under the consideration of design efficiency, crack control, opening reinforcement and segmentation.
- In order to delay the initiation of shear cracks and induce the flexural failure of girder, it could be a useful method to improve shear strength, reinforce the vertical rebars between openings, and arrange vertical rebars below web openings.

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