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Strengthening of Prestressed Concrete Girders Having Sheath Partially Filled with Grout

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ABSTRACT

The inside of the sheaths for post-tensioned prestressed concrete (PC) beams should be filled with grout to protect the PC tendons from corrosion. The grout, however, does not completely fill inside the sheath for various reasons. As a result, cases have been reported of corrosion and ruptured tendons. The objective is to evaluate the bond between tendon and grout inside the sheath, and the flexural load capacity of incompletely-grouted PC girders with broken tendons. A series of PC tendon rupture tests were carried out by using uniaxially prestressed specimens. The bond characteristics of the PC tendon were assumed to be a function of the PC-tendon type and the grout strength, which was useful for calculating the flexural load capacity for incompletely-grouted PC girders with broken tendons. The paper also proposes the strengthening method for the PC beam by using steel plates.

Keywords. Post-tensioned PC, Broken PC-tendon, Bond characteristics, Grout, Partially reinforcing method

INTRODUCTION

More than 10,000 such post-tensioned PC girders have been used in constructions for Japanese railways. The inside of the sheaths for such PC girders should be filled with grout to protect the PC tendons from corrosion. The grout, however, does not completely fill inside the sheath for various reasons. As a result, cases have been reported of corrosion and ruptured tendons. (Coronelli, et al. 2009 and Ishibashi, 2003) PC tendons in post-tensioned PC girders are anchored at both ends in order to provide the necessary stress. Corrosion of these tendons drastically changes the mechanical performance of PC girders and reduces the serviceability and safety of the structure. Therefore, the mechanical performance of incompletely-grouted PC girders with broken tendons should be evaluated in order to carry out the appropriate repairs and reinforcement work.

If the sheath is partially filled with grout, stress transfers from the tendons to the concrete because of adhesion between the tendons and the grout after the tendon snaps. (Watanabe, et al. 2011) This means that the bond between grout and tendons as well as the residual stress of the PC girder is a key in reasonable evaluation of the PC girder's mechanical performance. There are several reports on pull-out tests involving PC tendons from concrete if reference is made to tests performed on reinforcement bars. The method, however, cannot be utilized to duplicate the strain conditions of a broken tendon.

The objective of this paper is to evaluate the flexural load capacity of incompletely-grouted PC girders with broken tendons, and propose the strengthen method for such a PC beam. To meet this goal, the bond characteristics between the PC tendon and the grout inside the sheath were assumed to be a function of the PC-tendon type and the grout strength, which was useful for calculating the flexural load capacity for incompletely-grouted PC girders with broken tendons. The paper also proposes the strengthening method for the PC beam by using steel plates.

CALCULATION OF PRE-STRESS DECREMENT OF PC BEAMS BY CUTTING TENDON

Since the post tension PC beam had introduced pre-stress power into PC beam by ensuring the anchorage of main steel tendon, once PC main tendon was failed, the applied stress by the Tendon will be difficult, and the magnitude of pre-stress of PC beam will be decreased. The pre-stress of PC beam, however, sometimes remains if grout partially exists inside a sheath. There is bond between tendon and grout, and the stress of Tendon can transfer to concrete. In this case, pre-stress reduced area can be identified as the range of PC beam where non-grouting section and the vicinity of the section even being grouted.

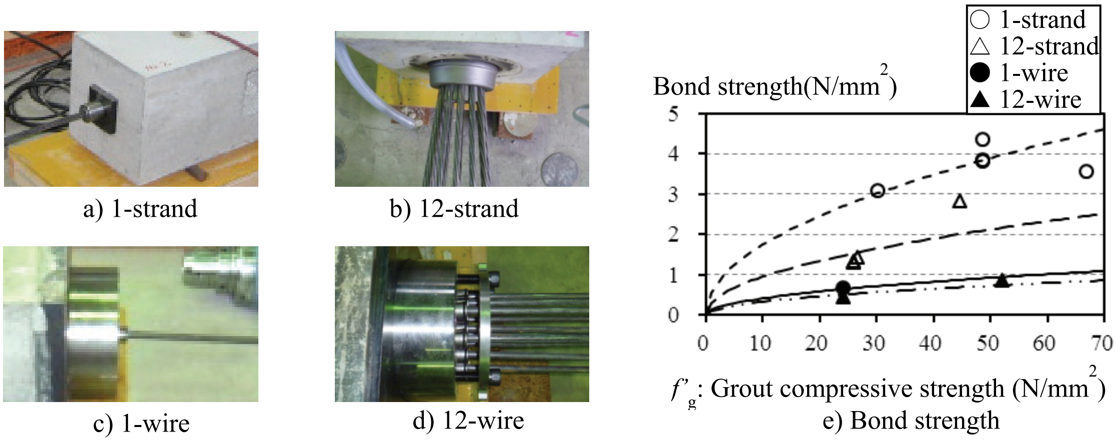
In order to predict the length of the pre-stress reduction range, cutting test of Tendon has been conducted. Figure 1 plots the bond strength (τ_{max}) by each strand or bar as a function of compressive strength of grout. (f'_g) The relationship was formulated as Eq.(1):

$$\tau_{s1} = \alpha\sqrt{f'_g} \tag{1}$$

where, $\alpha = 0.55$ at 1-strand, = 0.30 at 12-strand, = 0.13 at 1-wire, and =0.10 at 12-wire.

The length of pre-stress reduction range in the grout filling section (x_g) is calculated as the force by accumulating τ_{max} in the length (x_g) being balanced with tendon force. Therefore, the pre-stress reduction range (length: x) including the grout unfilled up section becomes reckonable by Eq. (2).

$$x = x_{ng} + x_g = x_{ng} + P/(\tau_{max} * l) \tag{2}$$



a) 1-strand

b) 12-strand

c) 1-wire

d) 12-wire

e) Bond strength

Figure 1. Bond strength by type of strand

where, P: force of Tendon (N), τ_{max}: bond strength between tendon and grout (N), l: peripheral length of tendon (mm), and f'_g: compressive strength of grout (N/mm^2).

EVALUATION OF LOAD BEARING CAPACITY OF PC BEAMS WITH BROKEN TENDONS

Experiment Outline

The method of presuming the mechanical performance of PC beam having some broken PC tendons was verified based on the experimental result of the actual size specimen. Figure 2 indicates the one-side of the specimen, which was a post tension-type PC beam having 13.7-m length between supports and six tendons (No.1-No.6). One tendon consisted of five PC strands. In order to reproduce being grout un-filled up of a sheath, the bond between tendon and grout was removed by setting a vinyl-chloride pipe and grease in the range of one fourth of beam span from right support.

Two loading-points were located at the middle of span. During the loading test, after load of 800-kN was applied, unloaded then one tendon was cut by grinder and reloaded 800-kN. The cycle had been conducted till the specimen being failed. The name of the tendon was designated as No.1, No.2, to No.6 from top of the cross section. The cutting order was set to No.6, No.4, No.3, No.2, and No.5 which had been arranged from the bottom of cross-section.

Experimental Results

Figure 3(a) shows the variation of pre-stress of PC beam by cutting tendons. The stress release rate was corresponding to the strain reduction of PC beam after cutting divided by strain at the time of examination. Figure 3(a) also indicates the length of stress decreasing section calculated by Eqs. (1)(2). The calculated value is mostly in agreement with the length section measured during the experiment.

Figure 3(b) indicates the load-displacement (at middle of shear-span) relationship until the end of an experiment. Figure 3(c) indicates the flexural moment diagram in the one-side of the span. The resistant moment capacity at the time of the lower fiber stress reached flexural strength of specimen by cutting tendons No.6 and No.4 is expressed as M_{cr}, calculating according to RTRI 2004. On the other hands, M_k is the acting moment by loading of 800-kN.

After start of loading and cutting No.6 and No.4, the flexural crack was observed visually at the middle of shear span. Figure 3(c) indicates that $M_k > M_{cr}$ was obtained at the middle of shear span when 2-tendon were cut. The location of cracking and amount of cut tendons was

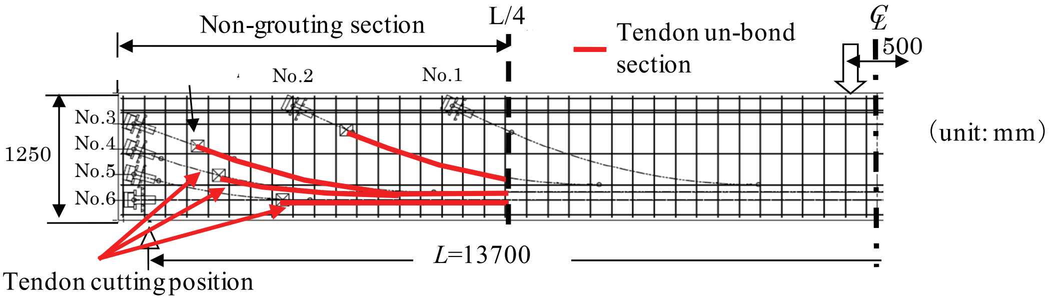
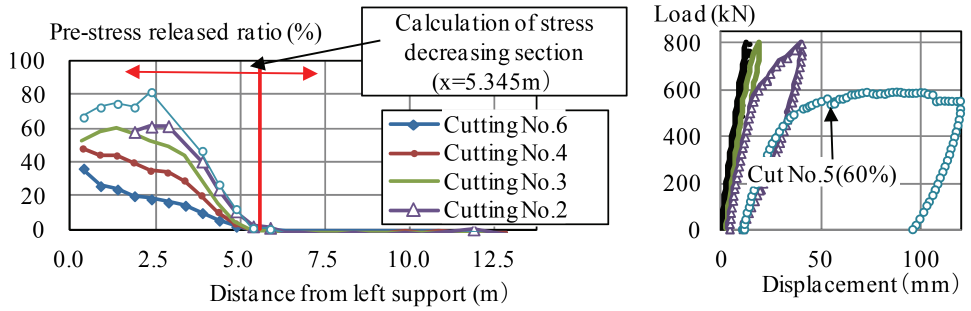


Figure 2. Variation of applied stress by cutting tendons

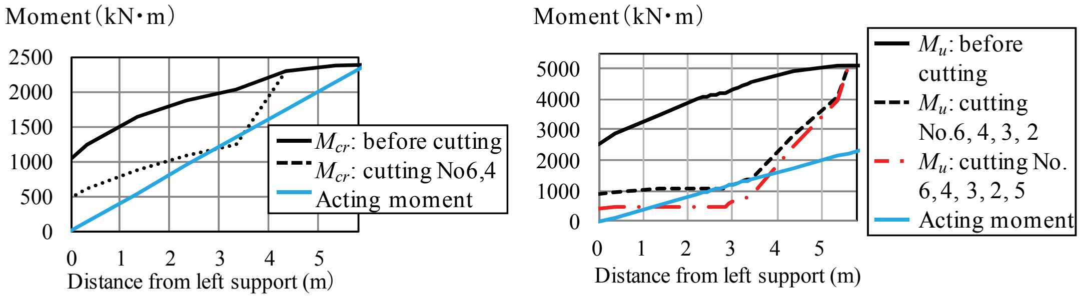
good agreement with the result of the experiment. This means that cracking load and crack location PC beam with broken tendons can be predicted by using residual stress. In addition, even the crack occurred, the load-displacement relationship in Figure 3(b) was not bent at all, therefore, the stiffness of the PC beam sustained its stiffness at beginning.

After cutting three tendons (No.6, No.4, No.3), the decrement of stiffness was observed when the diagonal crack between middle of span and that of shear span. Figure 3(d) indicates the flexural moment diagram at failure of PC beam. The calculation of flexural resistance involves the effect of broken tendons: the maximum stress (σ_{pn}) carried by broken tendon of PC beam is corresponding to the stress at slipping of tendon calculated by τ_{max} as Figure 4. According to Figure 3(d), additionally cutting No.2 and 60% of No.5 made the ratio between the acting moment and the flexural moment capacity exceed 1.0. The crack generated around the middle of shear span reached upper fiber of the PC beam. The load decreased with the



(a) Pre-stress decrease by tendon cutting

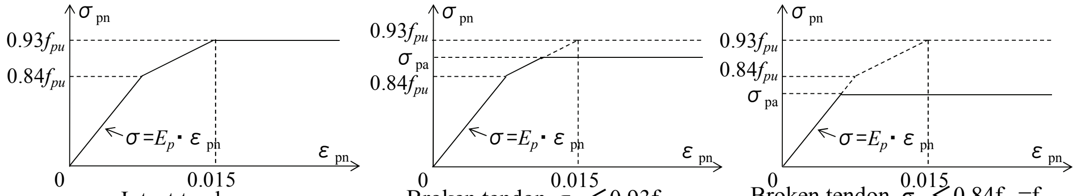
(b) Load-displacement



(c) Moment diagram at cracking (M_{cr})

(d) Moment diagram at failure (M_u)

Figure 3. Variation of applied stress by cutting tendons



Intact tendon

Broken tendon, $\sigma_{pa} < 0.93f_{pu}$

Broken tendon, $\sigma_{pa} < 0.84f_{pu}=f_{py}$

where, σ_{pa}: stress of tendon at the tendon starts slipping (N/mm^2) $=\tau_{max} \times f \times \pi \times L_a/A_p$,
L_a: distance between the cross-section and tendon broken position (mm)

Figure 4. Stress-strain of tendon depended on the bond strength

fracture of concrete at upper flange.

In order to maintain PC beams, the corrosion of tendons should be specified at an early stage. As mentioned above, there seems to be margin before the capacity of the PC beam reaches the limit of its safety, even flexural cracks observed, however, the margin cannot be quantified because the lack of information for corrosion development of tendons. Even we need to keep studying, to detect the crack opening at the time of train running (when active load applied) will be one of the effective method to detect the broken of tendons when judgment of tendon corrosion by nondestructive-test method is difficult.

DEVELOPMENT OF PARTIAL REINFORCEMENT METHOD BY STEEL PLATES

Outline of Method

Since stress remains partially as described when tendons are broken at the grout unfilled up section of PC beam, PC beam can be economically strengthening by reinforcing only the section where stress decreased. According to this result, “partial reinforcement method by steel plates” which provides steel plates only for the stress decreasing section was developed. The proposed method has an advantage which focuses on the required section in the narrow bottom space of the PC beam, and can reinforce it simply compared with the outside-cable reinforcement. In particular, this kind of reinforcement method requires good attachment of a steel-plate to the PC beam. Authors decided to use a bottom-wide type bolt for anchoring steel plates to bottom of the PC beam, which will enable being the embedded length shorter.

Loading Test of Bottom-wide Type Bolt

In order to examine the design method of the a bottom-wide type bolt, the loading test of the bolt embedded in concrete was carried out as Figure 5. The application of a bottom-wide type bolt on steel plate attachment bottom PC beam requires the bolt embedded inside the concrete to resist the horizontal force. According to the requirement, the load applied perpendicular to the direction of the embedded bolt. The experimental results indicate that fracture occurred at the bolt itself or concrete near the bolt. The experimental results implied the failure type an anchor was varied by the compressive strength of concrete and the embedded length of bolt. According to Figure 6, the load-carrying capacity of both failure types can be calculated by Eqs. (3)(4) as:

$$q_1 = 26.2 \cdot \sqrt{f'_{\mathrm{cd}}} \cdot l_{\mathrm{e}}^{1.5} / \gamma_{b1} \tag{3}$$

$$q_2 = 1/\sqrt{3} \cdot f_{\mathrm{ud}} \cdot A_{\mathrm{s}} / \gamma_{\mathrm{b2}} \tag{4}$$

where, q_1: the shear strength determined by the corn-type failure of concrete (N), l_e: effective embedded length of anchor bolt (mm), f'_{cd}: the design compressive strength of concrete (N/mm^2), r_{b1} = 1.3, q_2: the shear strength determined by fracture of anchor bolt (N), f_{ud}: tensile strength of anchor bolt (N/mm^2), A_s: nominal cross-section area of the bolt (mm^2), r_{b2} = 1.2.

Cyclic loading was also applied to the embedded anchor. After 2-million times of load corresponding to car weight, even slight exfoliation of concrete at the interface of anchor bolt was observed, any cracking and displacement of anchor bolt was not observed. This made the shape of load-displacement relationship of the loading test obtained after the

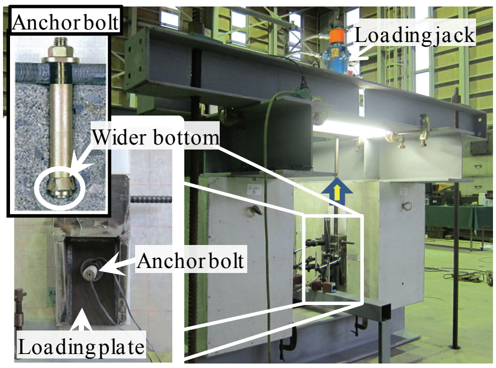


Figure 5. Loading test of anchor embedded in RC

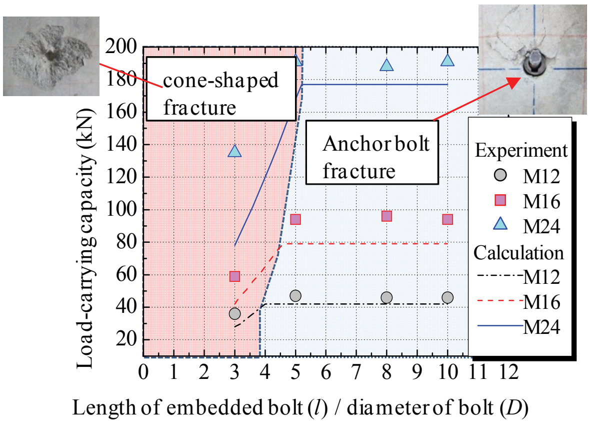


Figure 6. Load-carrying capacity functioned by length of embedded and diameter of bolt (f'_c=40 N/mm^2)

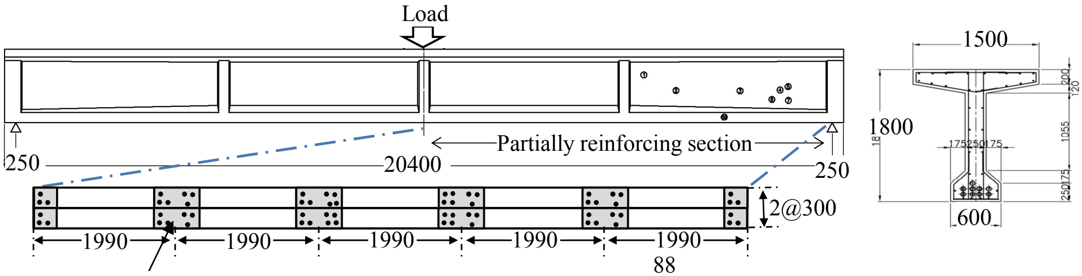
fatigue loading also be identical with original. This means the variation of anchorage capacity during the fatigue was not large, and required number and embedding length of the bottom-wide type bolt for fixing the steel plates can be calculated based on Eqs.(3)(4) for the design of partial reinforcement method by steel plates.

Reinforcement Effect of the Partial Reinforcement Method by Steel Plate

In order to examine the effect of partial reinforcement method by steel plates, the post-tension type of PC beam was prepared and loaded. The dimension of the PC beam was decided according to PC beam with the I-shape cross-section which has been constructed comparatively much. Figure 7(a) described the size and dimension of PC beam used in this study. There was nine sheathes and tendons in one PC beam, however, grout unfilled up section was set with length of a quarter of beam span from a right support. After load of 1200kN was applied, PC beam was unloaded then 1-tendon was cut by grinder and reloaded till 1200kN. The loading-cycle had been conducted till the PC beam was failed.

On the other hands, one of two PC beams was reinforced with steel plate having 1990×250×9 mm at the right-side of one PC beam after generation of flexural cracks. The amount of the steel plate was decided to recover the flexural strength corresponding to the decrement of strength by cutting 1-tendon. A series of steel plates, of which yield strength were 339 N/mm^2, were connected by splice plates. Figures 7(b)(c) shows the view of construction and the arrangement of the anchor bolt. The steel plates were fixed by the bottom-wide type bolt with 24-mm in diameter. Eight anchor bolts were provided for a steel plate.

Figure 8 compared two load-displacement relationships measured at the quarter of beam span from right support. The displacement of specimen without reinforcement by steel plates increased drastically when six tendons (66%) were cut. After cutting 7th tendon, the specimen resulted in the end without reaching the load of 1200 kN. On the other hand, the displacement of the reinforced specimen with steel plates was not increased drastically even after six tendons were cut. The specimen resulted in the end after 7 tendons were cut with reaching the load of 1200kN. Figure 9 shows the failure of PC beam with partial



(a) Side and bottom of specimen

(b) Dimension of steel plate and location of anchors

(c) Construction of steel plate

Figure 7. Location of steel plate and anchor bolt for strengthening PC beam (unit: mm)

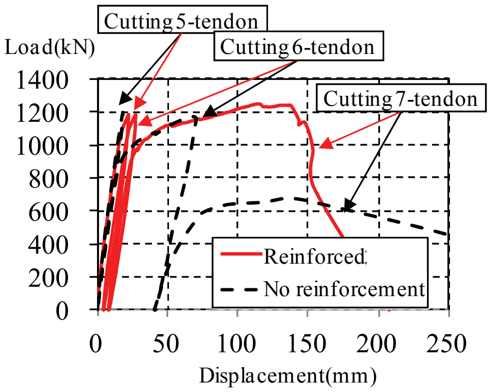


Figure 8. Effect of partially reinforcement by steel

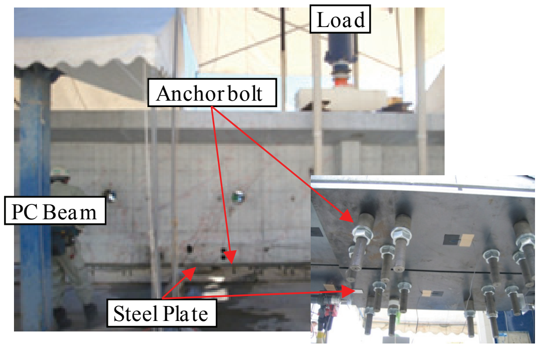


Figure 9. Failure of PC beam with partially reinforcement by steel plate

reinforcement by steel plate. Any damage was not observed at the anchor bolts used for fixing steel plates as well as concrete facing the bolt. The experimental result means PC beams, having steel plates fixed by using a bottom-wide type anchor, carried the assumed load enough.

CONCLUSIONS

This research has developed the method for calculating the load-bearing capacity of partially grouted post-tension type PC beams having broken tendons at cracking by train running. The method involves the bond strength between tendon and mortar calculated by proposed equations, and calculates the magnitude of residual stress in the PC beam. The method also engineers to make a reinforcing plan of PC beam efficiently. This study also developed the partial reinforcement method by steel plates attached to the bottom surface of PC beam by using bottom-wide type bolts. This construction method has advantages: such as simple enough reinforcing in narrow space. The effect of the method was verified by the experiment using full-scale PC beams.

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