CYCLIC LOADING TEST FOR EXISTING PRECAST PRESTRESSED CONCRETE BRIDGE COLUMN

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Abstract

This paper presents a research on the non-linear behavior of the existing precast segmental concrete bridge column with prestressing by high strength steel bars longitudinally. Precast segmental concrete bridge columns are effective to shorten the construction period at the site because of no need of formwork, placement and curing of concrete. Thus, they are expected to be applied for construction of overpass crossings in urban areas to minimize the effect on existing traffic.

In Japan, precast segmental PC columns with high strength steel bars were employed before the 1995 Hyogoken-Nanbu Earthquake. These columns were not designed considering the non-linear behavior and failure mechanism for a large earthquake. The limit states for these columns were investigated based on the experimental studies.

Introduction

The precast segmental concrete bridge columns would be a suitable structure for accelerated bridge construction because the construction period at the site can be shortened due to no need of formwork, placement and curing of concrete to construct bridge substructures. Additionally high quality of the concrete members would be ensured because the concrete segments are fabricated at factories.

The precast segmental PC columns with using high strength steel bars longitudinally are one of these precast segmental bridge columns designed and constructed before Hyogoken-Nanbu Earthquake occurred in 1995. Figure 1 shows the outline of these columns. The precast segments are produced at factory and transported to construction site. These segments are piled up at the site and connected each other through the steel bars, to be a column. Figure 2 shows the structural details of these columns. Each segment is post-tensioned by high strength steel bars, to integrate with column structure. After post-tensioning, the following segment is piled up on the lower segment and the high strength steel bars are installed into the section through the ducts. These bars are coupled with the lower high strength steel bars and grout is injected to the duct to be bonded. Figure 3 shows the detail of the segment connection. These processes are repeated up to the column height.

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However, the seismic design method for a large earthquake has not yet been developed because the failure mechanism, the seismic strength and ductility performance of the segmental PC columns have not yet been fully investigated. Therefore it is unable to clarify the necessary of seismic reinforcement for these columns.

In this study, a cyclic loading test for a scaled-model of the existing precast prestressed concrete bridge column was conducted to study the non-linear behavior and failure mode with the damage observation of high strength steel bars and cover concrete.

Outline of Experiment

(1) Specimen

A specimen of the precast segmental PC column is shown in Figure 4. The specimen was designed based on the existing segmental PC columns designed and constructed in Japan. The specimen is a 1/3 scale model and consists of 5 precast ring segments. The dimension of the section is 1000mm and thickness of the wall is 100mm. Photo 1 shows the test setup. The specimen was laid horizontally and the footing was mounted to the reaction wall thorough the base concrete. A lateral load was provided by a large stroke actuator at 4350 mm height from the footing, and a vertical load was applied at the top of column by an axial loading apparatus.

Table 1 shows the specimen details. 32 of 11mm diameter high strength steel bars were used as the longitudinal reinforcement. Spiral hoop reinforcement of 4.5mm diameter steel wire was used and placed at 43mm pitch.

The effective prestress of longitudinal steel bars was 8.1N/mm^2. A constant axial load of 534kN was applied for the specimen in order to provide 1.9N/mm^2 as the dead load of a superstructure. The total axial stress at the bottom of column was 10.0N/mm^2. This total axial stress was relatively larger than the common axial stress of existing segmental PC columns.

Table 2 shows the material properties when the specimen was tested. The average strength of concrete was 62.7N/mm^2. The yield strength of the longitudinal high strength steel bars was 1225 N/mm^2. The tensile strength of these bars was 1282 N/mm^2. The tensile strength of the reinforcing spiral steel wire was 590N/mm^2.

(2) Instrumentation

Lateral force was measured by the load sell of the actuator. The lateral displacements were measured at the loading point and each segment joint points by laser displacement sensors. The vertical displacements were measured at the segment joint point between segment No.1 and No.2 to measure the curvature. Strain gauges were employed for the measurement of longitudinal PC steel bars strains and concrete strains around each segment joint.

A CCD camera was set inside the column to observe the inside damage during the cyclic loading test. The severe damage such as spalling of cover concrete can be checked.

(3) Loading Condition

The cyclic loading test was conducted to investigate the nonlinear behavior of precast segmental PC column after the longitudinal high strength steel bars yielded. A preliminary reference displacement δ_c which is 2.5mm was determined as a half of the displacement when the outside cover concrete calculationally cracks. The lateral displacement was increased stepwisely ($\pm 1\delta_c$, $\pm 2\delta_c$, $\pm 3\delta_c \cdots$). The number of cyclic loading in each step was one until the longitudinal PC bar at the edge yield. However the longitudinal PC bar didn't yield when the lateral displacement exceed the calculational yield displacement. Therefore $10\delta_c$ was determined as the main reference displacement δ which is 25mm. The lateral displacement was increased each a half of δ ($\pm 1.0\delta$, $\pm 1.5\delta$, $\pm 2.0\delta$ $\cdots\cdots$). The number of cyclic loading in each step was three.

Experimental results

Figure 5 shows the lateral force - lateral displacement relationship. The lateral displacement was modified by removing the displacement caused by strain penetration of longitudinal PC bar into the footing. The spalling of outside and inside cover concrete was occurred when the displacement was 2.0δ as shown in Figure 6. The deterioration of lateral force was observed in the loading step of 2.5δ. The lateral force in the loading step of 3.0δ decreased with each loading reputation. The observed spalling of outside and inside cover concrete progressed. However, the longitudinal PC bar didn't yield. The observed spalling of outside and inside cover concrete and the deterioration of lateral force progressed and the some spiral hoops were ruptured in the loading step of 3.0δ as shown in Figure 7. The cover concrete was crushed around the segment joint between No.1 and No.2 and the many more spiral hoops were ruptured in the loading step of 3.5δ as shown in Figure 8. The axial load was constantly kept before the cover concrete was crushed. Figure 9 shows the strain where is measured at longitudinal PC bar No.1. The initial value of strain is 3770μ before testing. The yield strain of PC bars is 6135μ. The longitudinal PC bars didn't yield during testing. The PC bar's strain at upper of segment No.2 is smaller than the lower one before the lateral displacement is 1.0δ. However, the both strain values are similar after the lateral displacement is 1.0δ. It was highly possible that the PC bars were not bonded with grout, because the yield of PC bars didn't occurred and the strain spread upward in segment No.2.

The failure mode of this specimen was not yielding of PC bars but compressive failure of cover concrete. The failure mode is not desirable because the columns would lost the axial loading capacity. This should be considered when the design limit state of existing precast prestressed concrete bridge columns.

However, if the PC bars would be bonded, the yielding of PC bars would occur before the compressive failure of cover concrete. It was not designed to bond that the PC bars of these columns under a large earthquake. It is necessary to clarify the bond characteristic of the grout with PC bars in order to estimate the non-linear behavior of these columns.

Conclusion

Cyclic loading test for existing precast prestressed concrete bridge column was conducted to investigate the failure mode of these columns. The results from the test are concluded as follows.

1) It is found that the failure of these columns is caused by significant crush of concrete and the PC bars didn’t yield at the event of failure. Since the existing precast PC segmental columns were designed with the hollow section in the plastic hinge region, this type of failure mode is undesirable in terms of the vertical support capacity for the bridge column.

2) It was considered that the PC bars didn’t become bonded with grout as the lateral displacement increased. It is considered that the non-linear behavior and failure mode of these columns are influenced with the bond characteristic of the grout with PC bars. It is necessary to clarify the bond characteristic of the grout with PC bars in order to estimate the non-linear behavior of these columns.

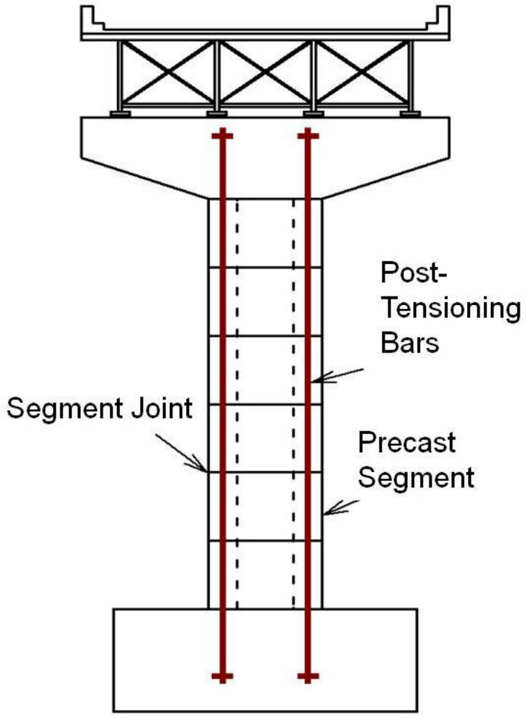


Figure 1 Illustration of Precast Segmental Concrete Columns

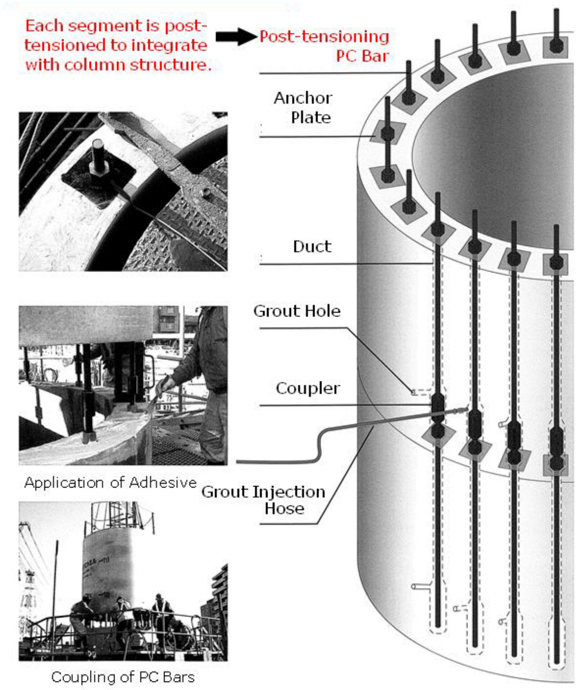


Figure 2 Structural details of precast segmental PC column

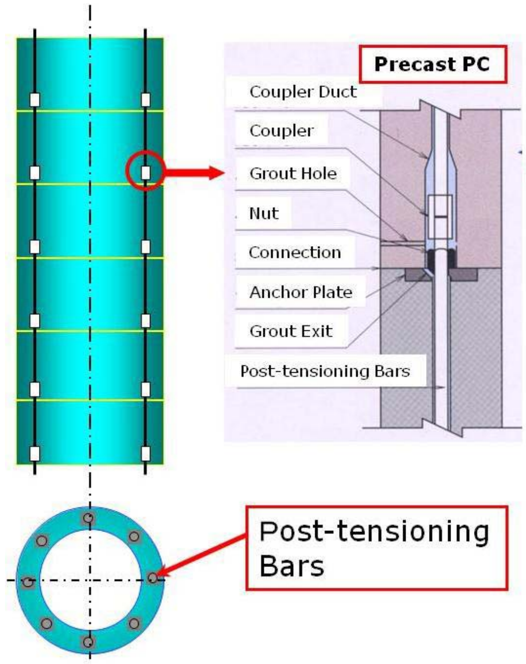


Figure 3 Detail of segment connection of precast segmental PC column

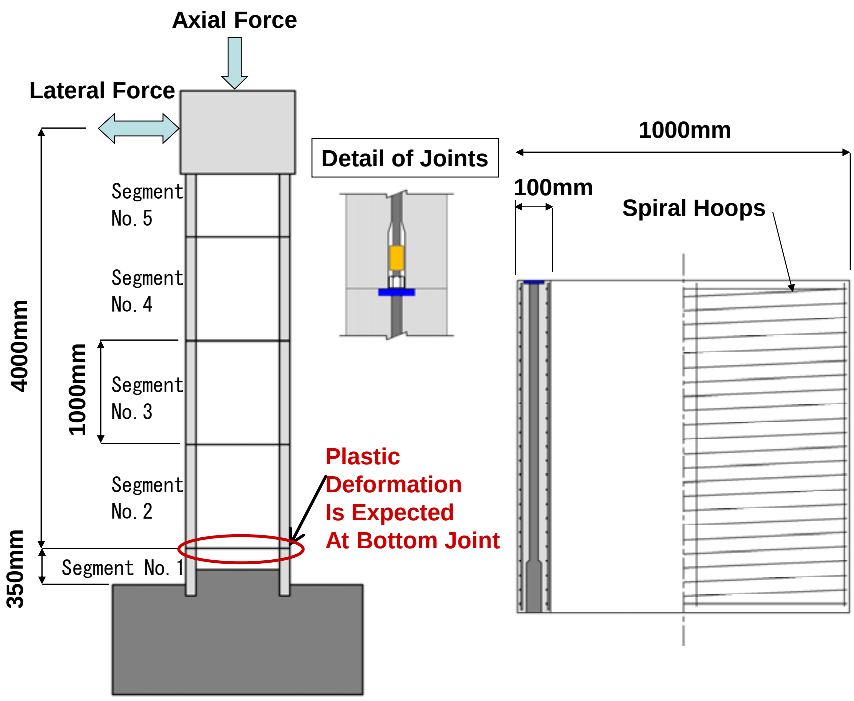


Figure 4 Specimen of precast segmental PC column

Photo 1 Test setup

Table 1 Specimen details

Height(mm)		4350
Circular cross-section dimension(mm)		1000
Long. bar	type	SBPR1080/1230
	diameter(mm)	11
Hoop	type	SWM-B
	diameter(mm)	4.5
Axial stress (N/mm^2)	Effective prestress	8.1
	Dead load of superstructure	1.9
	Total	10.0

Table 2 Material properties

(a) Concrete properties

Material		Compression Strength (N/mm^2)	Young Modulus (N/mm^2)
Concrete	Segment No.1	61.5	34416
	Segment No.2	59.3	35234
	Segment No.3	62.2	35109
	Segment No.4	61.5	37933
	Segment No.5	70.7	37428
	Footing	60.7	35497
	Average	62.7	35936
Grout		55.1	-

(b) Steel properties

Material	Yield Strength (N/mm^2)	Tensile Strength (N/mm^2)	Young Modulus (N/mm^2)
SBPR1080/1230 ϕ11	1225	1282	199666
SWM-B ϕ4.5	-	590	-

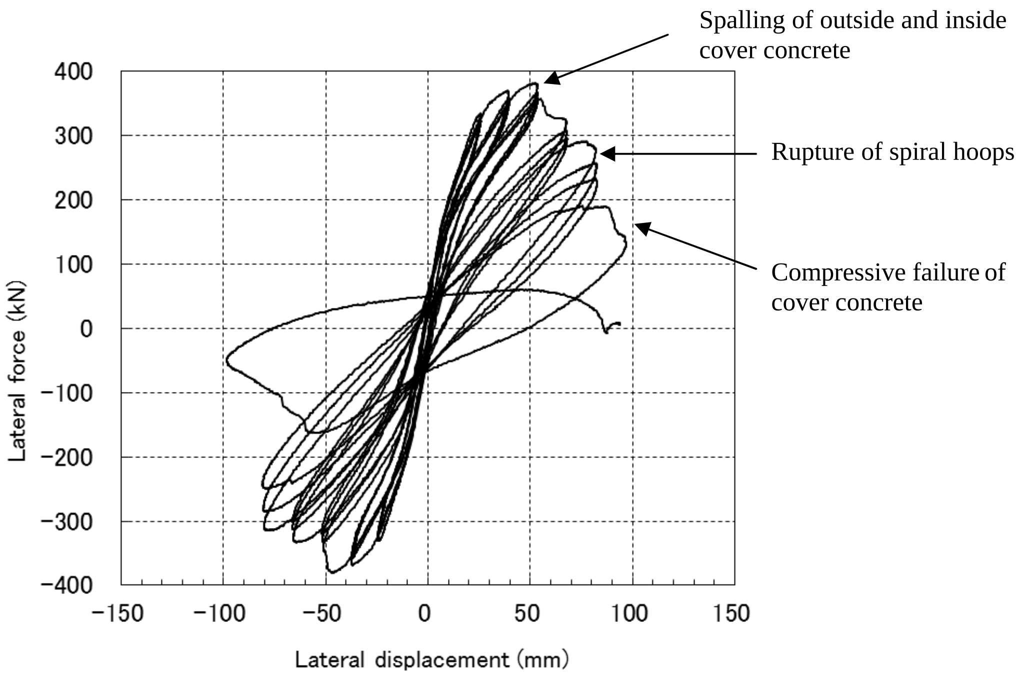
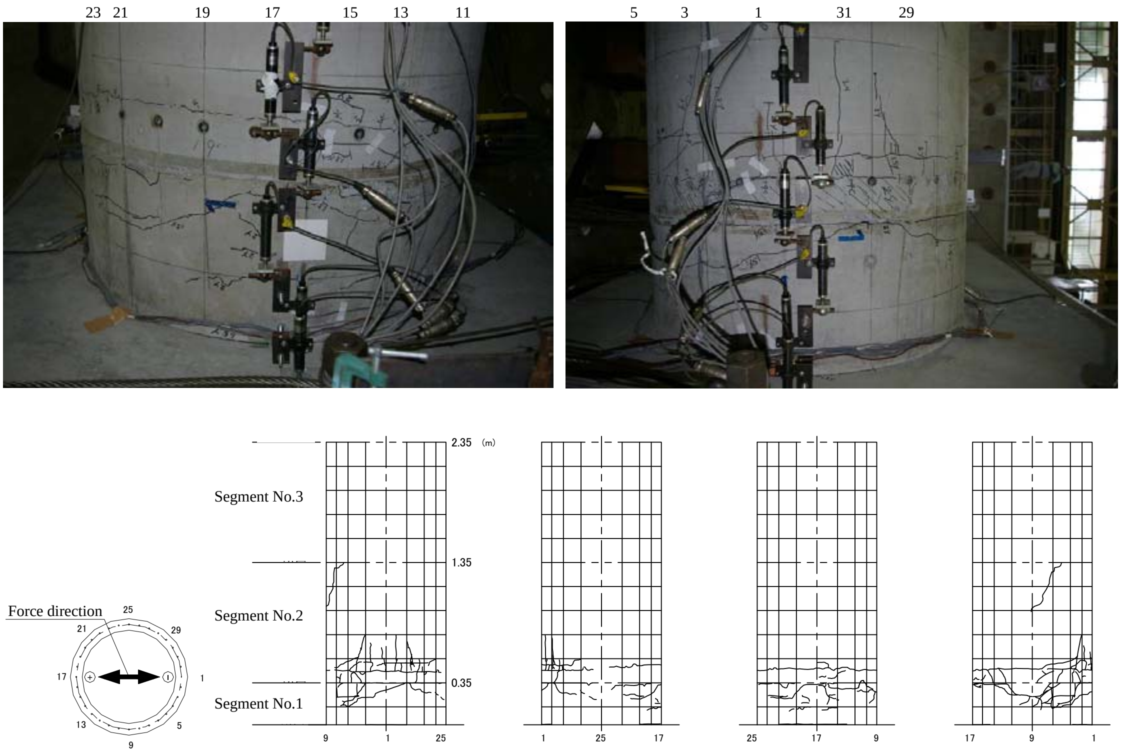


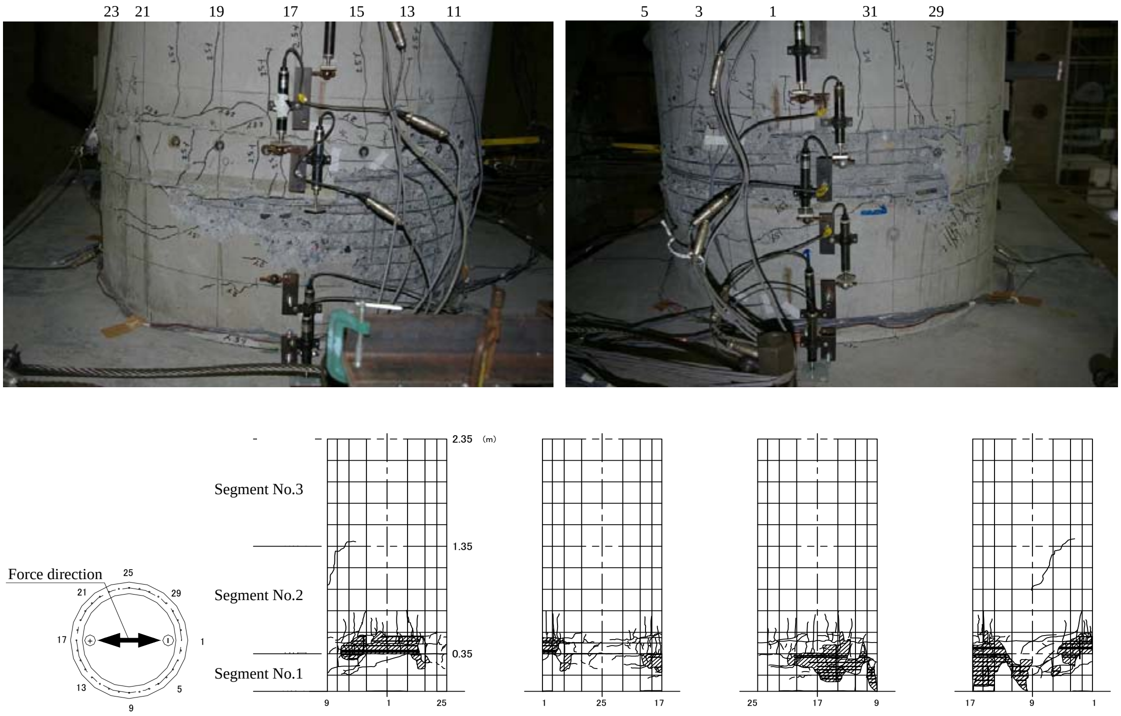
Figure 5 Lateral force vs lateral displacement



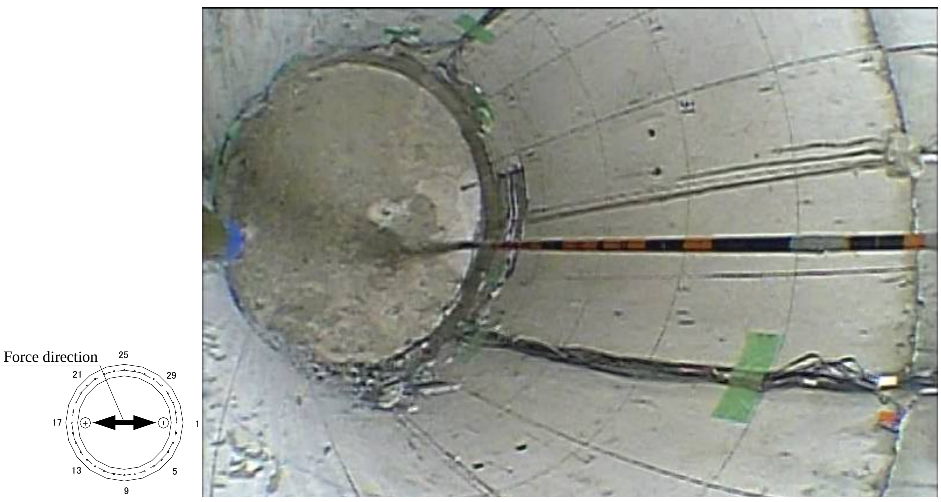
(a) Damage observed at outside face

(b) Damage observed at inside face

Figure 6 Observed damage in the displacement 2.0δ

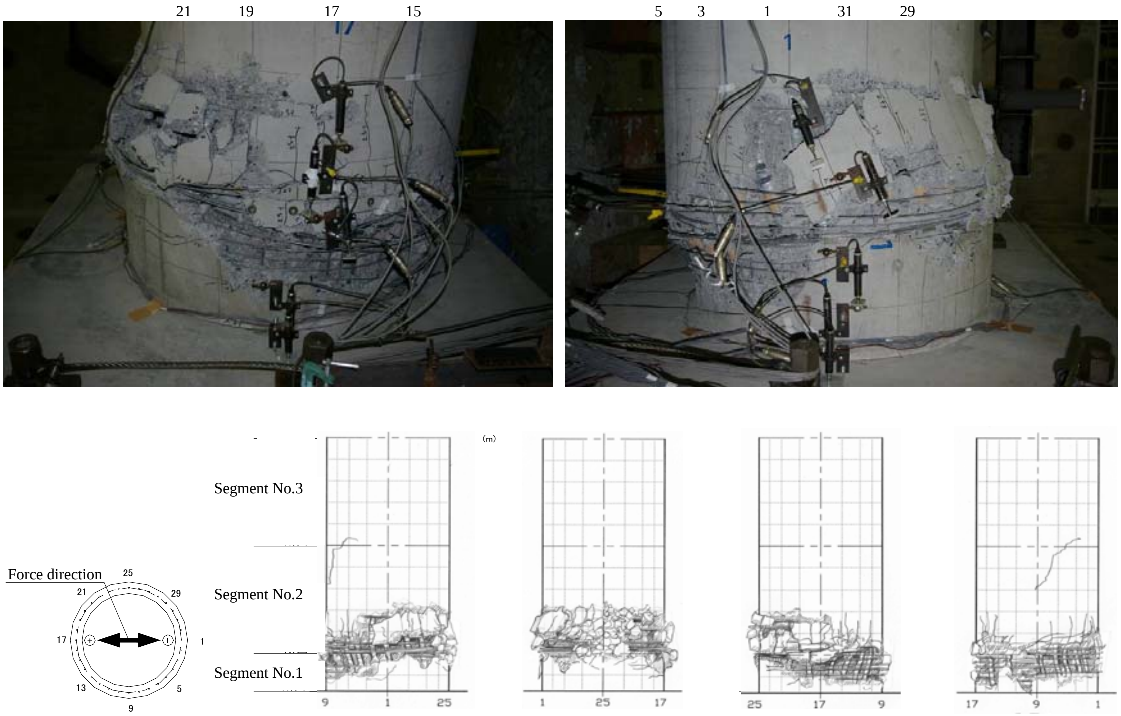


(a) Damage observed at outside face



(b) Damage observed at inside face

Figure 7 Observed damage in the displacement 3.0δ



(a) Damage observed at outside face

(b) Damage observed at inside face

Figure 8 Observed damage in the displacement 3.5δ

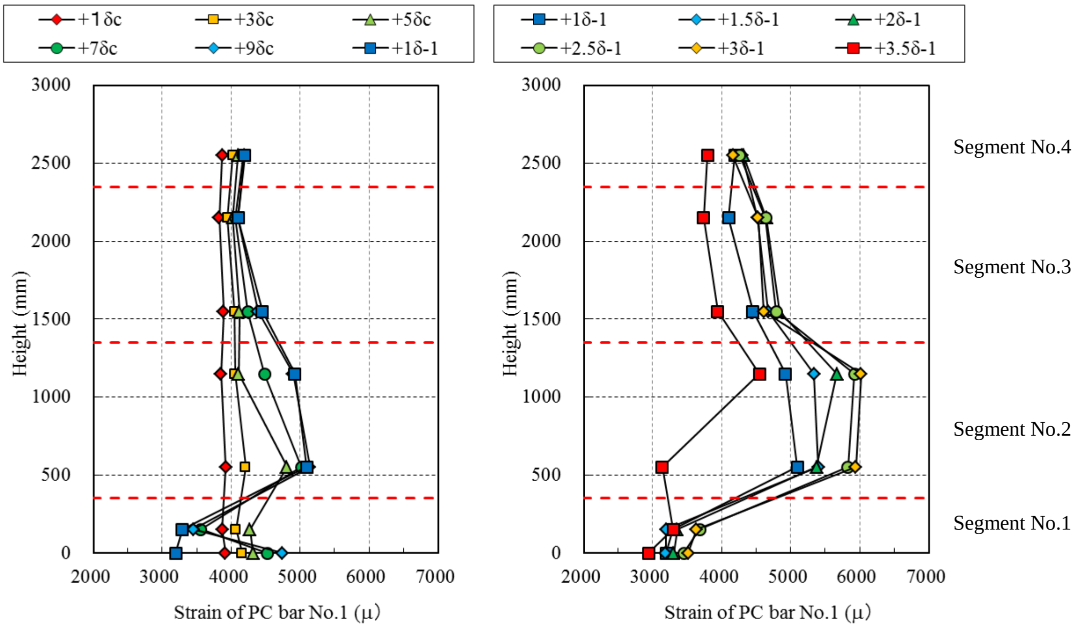


Figure 9 Strain of PC bar No.1