RESIDUAL DISPLACEMENT OF PRESTRESSED CONCRETE PIERS UNDER SEISMIC LOADING

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1. Introduction

With increased interest in prestressed concrete piers after the Hyogoken Nambu earthquake, a number of experimental studies have been conducted recently. Lots of them have been related to the type of cross section and relative amount of prestressing steel to nonprestressing steel. In view of the results so far achieved, it has long been confirmed that prestressed concrete piers have great potential to replace conventionally reinforced concrete piers in the high seismicity area. However, the significance of tendon tensioning level have been under-estimated, relative to that of other parameters, or even left out of issue. On the top of that, the current residual displacement modification factor C_R for conventionally reinforced concrete piers is too conservative for prestressed concrete piers to extract its full potential out of given cross-section. From this point of view, that kind of effect should be clarified and should be taken into account for the design of prestressed concrete piers and for better representative hysteresis model to predict its inelastic response precisely. 6 types of cross-sections - having different flexural capacity ratio of prestressing steel to nonprestressing steel (hereafter, γ ratio), tendon tensioning level (hereafter, λ ratio) and details – have been tested to achieve optimized design proposals and better understanding for prestressed concrete piers.

2. Test specimens and loading program

As the geometric details of the specimens are illustrated in Fig. 1 and 2, the resistance of the prestressed concrete column to lateral loads is provided by post-tensioning steel in ducts grouted with cement paste, longitudinal reinforcements and concrete section confined with hoop reinforcements. The considered parameters were defined for symmetric cross-section as follows:

$$\gamma = A_P \sigma_{PY}/(A_P \sigma_{PY} + A_S \sigma_{SY}) \quad (1)$$
$$\lambda = \sigma_P/\sigma_{PY} \quad (2)$$

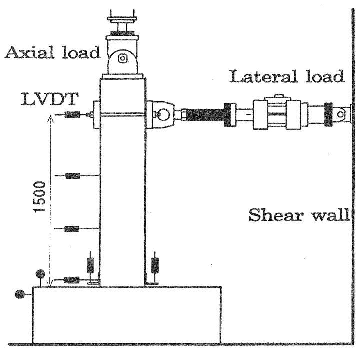
where, A_P: total area of prestressing steel, σ_{PY}: yield stress of prestressing steel, A_S: total area of longitudinal bar, σ_{SY}: yield stress in longitudinal bar, σ_P: stress in prestressing steel due to prestress

The details of the specimens are summarized in Table 1. SD345 bars were adopted for longitudinal and lateral reinforcement, and SWPR19(ϕ17.8) and SWPR7B(ϕ12.7) for prestressing steel, respectively. All the Type-S specimens have been designed to have the same flexural capacity and almost the same λ ratio in the case of prestressed concrete specimens but have different sectional details and reinforcement configuration, resulting in different γ ratio, varying from 0 to 0.87. The difference in cross-section between type-R2 and type-R1 is nonprestressing steel ratio by 1.92%. In order to find the proper relationship between the property of restoration and considered parameters λ ratio, tendon tensioning was introduced in the range of 0 to 75% of yield stress of tendon. All the specimens were subjected to lateral load reversals and constant axial load. Displacement control was adopted for lateral reversed cyclic loading with 1/200rad step after the yield of longitudinal bar and the loading lasted until the measured lateral load decreased by 80% of peak load. The constant axial load was applied as the probable dead load by conservative estimate.

Table 1 Details of specimens

Specimens (cross-section)	Concrete compressive strength [N/mm^2]	Reinforcement ratio [%]		Hoop ratio [%]	SP* (SA*) [N/mm^2]	γ ratio	λ ratio
		Nonprestressing steel	Prestressing steel				
S1-RH-00 (a)	61	5.08	0.00	0.53	0.00 (4.00)	0.00	*
S2-PH-P50 (b)	61	0.72	1.04	0.53	8.00 (4.00)	0.87	0.50
S3-PH-P50 (b)	61	0.72	1.04	0.53	8.00 (4.00)	0.87	0.50
S4-PH-P50 (c)	54	0.72	1.04	0.53	8.00 (4.00)	0.87	0.50
S5-PN-P52 (d)	53	0.71	0.49	0.53	4.00 (1.00)	0.76	0.52
R1-PH-P00 (e)	58	0.95	0.52	0.53	0.00 (1.00)	0.70	0.00
R1-PH-P25 (e)	57	0.95	0.52	0.53	2.10 (1.00)	0.70	0.25
R1-PH-P50 (e)	55	0.95	0.52	0.53	4.10 (1.00)	0.70	0.50
R1-PH-P75 (e)	57	0.95	0.52	0.53	6.20 (1.00)	0.70	0.75
R2-PH-P25 (f)	69	2.87	0.52	0.53	2.10 (1.00)	0.44	0.25
R2-PH-P75 (f)	65	2.87	0.52	0.53	6.20 (1.00)	0.44	0.75

SP* : Stress in concrete due to prestress, SA* : Stress in concrete due to axial load



(unit of dimensions: [mm])

Fig. 1 Test setup

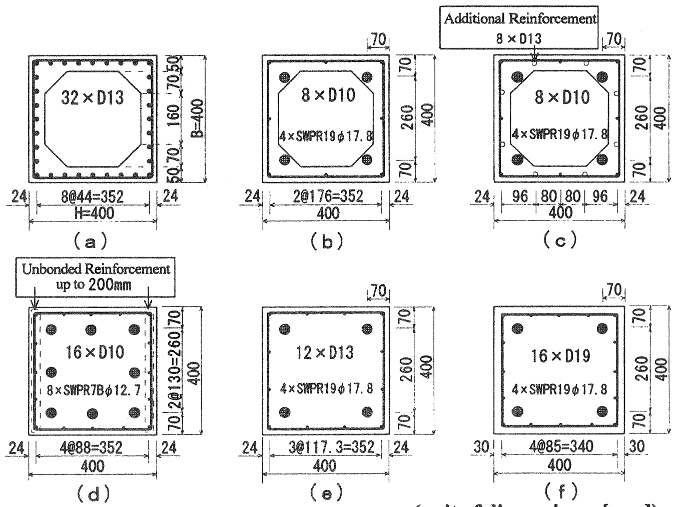


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Fig. 2 Cross-section of specimens

3. Observed behaviors and test results

3.1 Test results of Type-S

The experimental results were summarized in Table 2. While the yield displacement has been defined as the intersection of the secant stiffness through first yield with Pu, calculated by trial and error method to have same energy with the original load-displacement relationship to ultimate displacement, which is defined as displacement corresponding to 20% degradation from peak strength.

All the specimens, except S2-PH-P50, have been proved to have enough displacement ductility factors under reversed cyclic loading. Especially the hollow prestressed concrete specimen S3-PH-P50, whose cross-section is filled with concrete up to the height of 400mm from the footing, showed exceptional displacement ductility capacity. On the contrary, S2-PH-P50, having its cross-section change at the height of 200mm from the footing, showed quite brittle

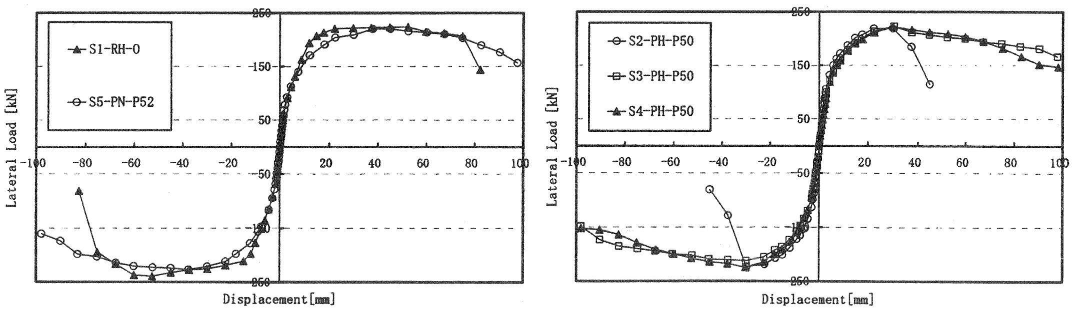


Fig. 3 Comparison of lateral load-displacement envelopes

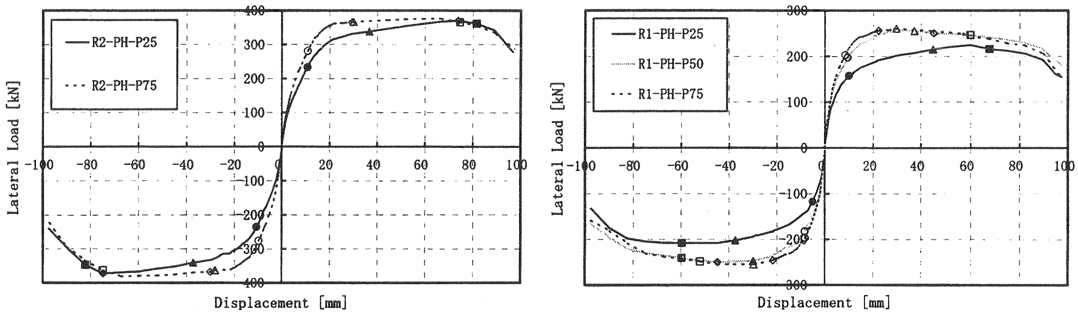
Table 2　Summary of experimental results

Specimens	Yield Displacement [mm]	Applied maximum load [kN]	Ultimate displacement [mm]	Displacement ductility
S1-RH-0	13.6 (-13.8)	225 (-239)	78.2 (-75.4)	5.8 (5.5)
S2-PH-P50	9.5 (-9.8)	219 (-224)	38.7 (-33.4)	4.1 (3.4)
S3-PH-P50	9.8 (-10.1)	224 (-212)	91.6 (-91.4)	9.3 (9.0)
S4-PH-P50	11.7 (-11.4)	223 (-226)	77.4 (-74.3)	6.6 (6.5)
S5-PH-P52	11.0 (-10.9)	220 (-228)	90.1 (-88.1)	8.2 (8.1)
R1-PH-P0	*	160 (-186)	*	*
R1-PH-P25	11.0 (-11.0)	226 (-216)	91.7 (-91.4)	8.3 (8.3)
R1-PH-P50	12.1 (-11.5)	257 (-249)	91.8 (-88.7)	7.6 (7.7)
R1-PH-P75	10.7 (-10.8)	261 (-254)	88.4 (-83.5)	8.3 (7.8)
R2-PH-P25	17.0 (-17.1)	372 (-375)	94.5 (-89.7)	5.6 (5.2)
R2-PH-P75	14.5 (-13.6)	378 (-383)	94.8 (-87.7)	6.6 (6.5)

failure, which is attributed to the cross-section change in plastic hinge region. Meanwhile, as far as the equivalent flexural stiffness is concerned, in spite of higher EI_{gross} value of S5-PH-P52 compared to the other hollow prestressed concrete specimens, S5-PH-P52 has failed to show its superiority under loading over the other hollow prestressed concrete specimens, since its lateral displacement was almost identical under the same external moment with the hollow prestressed concrete specimens from the middle of loading stage. Therefore, considering the equivalent flexural stiffness, hollow prestressed concrete columns were proved more effective than solid prestressed concrete column to resist external load.

3.2 Test results of Type-R1 and Type-R2

All the prestressed concrete specimens tested were designed to have greater shear strength than flexural strength and developed their full flexural strength except R1-PH-P0 and R1-PH-P25. The test result of R1-PH-P25 and R1-PH-P0 showed a little lower peak load than that of Analytical one in comparison. Such result was partially attributed to the fact that buckling of longitudinal bar occur prior to yield of prestressing steel due to its lower tendon tensioning level. On the contrary, in the case of R2-PH-P25, having identical tendon tensioning level with R1-PH-P25 and different nonprestressing steel ratio by 1.92%, extract its full flexural strength. While all the specimens were instrumented for the stress-strain curve of reinforcements with strain gage at a height of 20mm and 400mm from the footing. From the observed results, it was obvious that prestressing steel in R1-PH-P25 specimen didn't reach its yield stress, leading R1-PH-P25 to fail to show its full flexural capacity. On the other hand, in the other specimens having λ ratio higher than 0.25, prestressing steel reach its yield stress enabling the specimens to pull out their full flexural capacity. Therefore prestressing steel should be prestressed to yield prior to the buckling of longitudinal reinforcement to extract its full flexural capacity.



△: concrete compressive failure, ○: yield of longitudinal bar, □: buckling of longitudinal bar, ◇: yield of prestressing steel

Fig. 4 Observed phenomena on load-displacement envelope

In Fig. 4, the observed phenomena were marked on the load-displacement relationship. From Fig. 4, it have been verified again that prestressed concrete piers didn't show abrupt decrease in load carrying capacity even after yield of prestressing steel. Comparing the load-displacement envelopes can make an overall view of the results for Type-R1 and Type-R2 specimens. λ ratio, as expected, put an notable influence on the ascending branch of the load-displacement relationships of prestressed concrete columns. As far as λ ratio remain between 0.5 and 0.75, regardless of the type of specimen, the specimens demonstrated their full flexural strength resulting in nearly identical peak load with the analytical one. In residual displacement, as it have long been confirmed, residual displacement have been governed mostly by γ ratio.

4. Hysteresis model and residual displacement modification factor

4.1 Hysteresis model

The basic procedure of hysteresis model that has its origin in the previously proposed model, which decreases rigidity according to response ductility factor after yield of member, have been retouched and revised for prestressed concrete column as well as conventionally reinforced concrete column. The outer envelope curve, computed through the fiber analysis, was reduced into a few straight lines taking into account onset of crack, yield of longitudinal bar and member, ultimate load and failure. The following expressions have been derived to adequately represent the hysteretic behavior of prestressed concrete column at unloading and reloading.

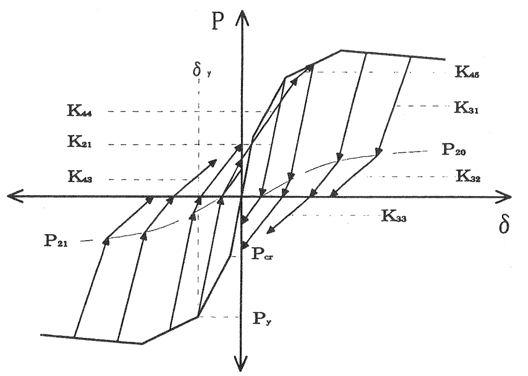


Fig. 5 Schematic expression of hysteresis model

$$K_{3j} = (1.072\times P_y/\delta_y)/(|\delta|_{max}/\delta_y)^{AL} \quad (3)$$

$$K_{4j} = (|P|_{max} - P_0)/(|\delta|_{max}\times G) \quad (4)$$

$$P_{2m} = GRAD\times |LL|_{max}\times \sqrt{(|\delta|_{max}/\delta_y \text{ -1})} \quad (5)$$

$$AL = (\alpha\times LN(\lambda) + 0.65)\times(\gamma - 0.25) + 0.3 \quad (6)$$

$$GRAD = 0.315\times(\gamma - 0.3)\geq 0 \quad (7)$$

where, P_{cr}: cracking load, P_y: yield load, δ_y: yield displacement, $|\delta|_{max}$: observed maximum displacement thus far, $|P|_{max}$: observed maximum load thus far, P_o: load corresponding to zero displacement, $|LL|_{max}$: observed max. load at each hysteresis loop

From the test results, the above expressions for AL and GRAD were found as in Fig. 6 and Fig. 7, which were conveniently approximated into the form of polynomial. AL was found under the influence of λ and γ ratio, while GRAD was governed mostly by γ ratio.

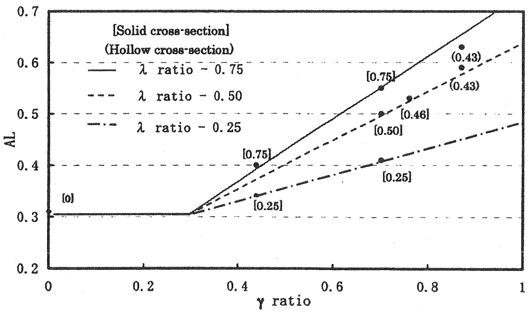


Fig. 6 AL according to γ ratio and λ ratio

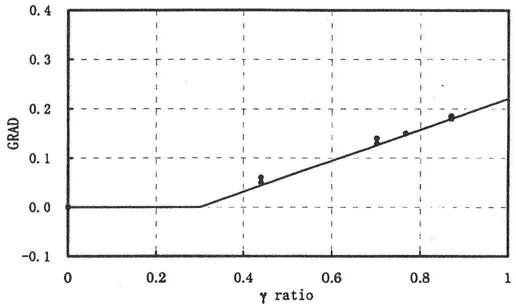


Fig. 7 GRAD according to γ ratio

4.2 Residual displacement modification factor C_R

Reinforced bridge piers should be seismically designed to satisfy the following Eq. (8) and the residual displacement δ_R after an earthquake shall be evaluated by the Eq. (9).

$$\delta_R \leq \delta_{Ra} \quad (8)$$

$$\delta_R = C_R(\mu_R - 1)(1-\gamma^*)\delta_y \quad (9)$$

where, δ_{Ra}: allowable residual displacement, γ^*: ratio of the post-yield stiffness to yield stiffness (0 for reinforced concrete piers), μ_R: response ductility factor

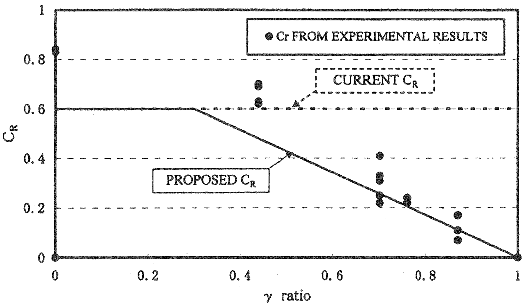


Fig. 8 Proposed displacement modification factor

Residual displacement is evaluated based on equation (9) composed of μ_R, γ^*, and C_R. 0.6 have been taken as the residual displacement modification factor C_R for conventionally reinforced concrete pier regardless of its reinforcement configuration. The residual displacement modification factor for prestressed concrete pier, however, is believed under influence of γ and λ ratio. The characteristic of prestressed concrete piers, having smaller residual displacement and greater restoring force, was found apparent from γ ratio 0.3 or more, as in Fig. 6 and Fig. 7. On the top of that, when λ ratio is 0.5 or more, the effect of tendon tensioning level of prestressing steel get relatively smaller on its behavior. Therefore C_R could be expressed conveniently as follows for prestressed concrete piers having λ ratio 0.5 or more.

$$C_R = 6(1-\gamma)/7 \leq 0.6 \quad (10)$$

The above Eq. (10) has been verified by comparing with the residual displacement modification factors computed from experimental results in Fig. 8. Allowable ductility factors of the specimens were taken as 4 for the computation of residual displacement modification factor C_R. In Fig. 8, it looks apparently that the error in the prediction for C_R gets increased as γ ratio decrease. That still could be justified by the following. In general, reinforced concrete pier test have been conducted with downscaled test units. Furthermore, when a test unit is subjected to reversed cyclic loading, the residual displacement of test unit might be overestimated compared to that of full-scale test unit due to its discrepancy in reinforcements between test units and real structures. In other words, the bar arrangement and diameter of test unit might be difficult to accurately duplicate that in real structures. Accordingly, even though flexural crack does not open wide in real structure, the flexural crack relatively open wide especially at the bottom part of test unit. And the flexural cracks remained open in the middle of loading stage even when the load was removed, which mean that the longitudinal reinforcement would suffer from cyclic compression and tension, resulting in greater residual displacement than in reality. On the other hand, flexural crack will be closed as γ ratio increase owing to increased restoring force. Therefore the above effect would get diminished for the test units having relatively higher γ and λ ratio. Judging from these points of view, the expression of residual displacement modification factor C_R in Eq. (10) could be justified applicable to the design of prestressed concrete piers when their λ ratio is 0.5 or more.

5. Conclusions

Prestressed concrete specimens with different γ ratio (flexural capacity ratio of prestressing steel to nonprestressing steel), as well as λ ratio (tendon tensioning level), have been experimentally studied in an effort to clarify the effect of considered parameters. Based on the analysis of test results and comparisons with theoretical predictions, the following conclusions were drawn.

1. In order to extract its full performance out of a given prestressed concrete cross-section, it would be recommendable to keep λ ratio relatively higher for prestressing steel, being able to yield prior to buckling of longitudinal reinforcement.
2. In general, γ ratio put a significant influence on the behaviors of prestressed concrete columns under reversed cyclic loading, especially on residual displacement and restoring force. Judging from the test results, γ ratio should remain 0.5 or more so as to maintain inherent merits, that is, desirable property of restoration and less residual displacement.
3. According to the analytical results, the peculiarity of prestressed concrete piers, having smaller residual displacement and greater restoring force, was found effective and obvious from γ ratio 0.3 or more. However, considering the test results, γ ratio should be 0.5 or more to keep its outstanding seismic performance.
4. In comparison of the experimental and analytical results, the proposed expression for displacement modification factor C_R is verified applicable to the design of prestressed concrete piers.

References

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