論文 Dynamic Response Behavior of Prestressed Concrete Piers under Severe Earthquake

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ABSTRACT: In order to clarify inelastic dynamic behavior of prestressed concrete (hereafter PC) piers under severe earthquakes, experimental and analytical studies were conducted. Three specimens representing such PC piers were tested using a pseudo-dynamic test in which amplified excitations of the Hyogo-Ken Nanbu 1995 earthquake were used. Inelastic response analyses for the specimens were carried out. A comparison between the experimental and analytical results was also conducted. The study revealed that the use of PC piers has a tendency to reduce the residual displacements after the earthquake excitations.
KEYWORDS: Earthquake resistant structures; bridges; prestressed concrete; piers; pseudo-dynamic test; dynamic analysis.

1. INTRODUCTION

A common type of highway concrete bridges consist of PC or RC girders and reinforced concrete piers [1]. The benefits of using the reinforced concrete piers are to obtain high energy dissipation characteristics and high values of ductility factor during earthquake excitations. In spite of their energy absorption capacities, some bridge piers suffered from severe damage during the Hyogo-ken Nanbu 1995 earthquake. Additionally, high residual displacement values [2] were observed for the same piers after the earthquake. On the other hand, the usefulness of using the prestressed girders are to obtain less dead load and achieve long spans while they have less energy absorption characteristics and ductility factor. As a consequence, a new technique is being examined in the current study in which partially prestressed piers [3] were implemented in such a way to make a compromise between the merits and disadvantages of both the RC and the PC.

Three specimens representing such PC piers were examined using pseudo-dynamic test in which amplified excitations of 1995 Hyogo-Ken Nanbu earthquake (NS direction) were used. The first specimen is a control RC specimen while the other two specimens are PC specimens. No grouting was used for the second specimen while grouting was used for the last specimen. Experimental results in terms of hysteretic load-deformation behavior and time histories were obtained. The plastic deformability in terms of ductility factor was also examined. The objective of the current study is to reveal the merits and disadvantages of partially prestressed concrete piers. Response analyses, based on Takeda's model for RC and modified Takeda's model for PC, were also conducted for the same specimens in order to check the available analytical hysteretic models. A comparison between experimental and analytical results was also conducted.

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2. SPECIMENS AND EXPERIMENTAL PROCEDURES

2.1. Test specimens

Three partially PC specimens were tested in this study. The main difference between specimens (S-1) and (S-2) is the usage of ungrouted prestressing tendons. Specimen (E-1) has grouted prestresssing tendons but with a lesser ratio than (S-2) and also has a smaller ratio of shear reinforcement to clarify the influence of strength ratio on the resulting failure mode. Details of specimens are shown in Fig. 1 and in Table 1. Concrete compressive strength is about 36 N/mm^2, yielding stresses of reinforcement are 401 N/mm^2 for D13 and 411 N/mm^2 for D16 while the yielding stresses of prestressing tendons are 1421 N/mm^2 for SBPR12.7 and 1315 N/mm^2 for D13. All specimens were tested using the same setup shown in Fig. 2. The bottom parts of specimens were rigid enough to represent footings for these PC piers. All specimens were fixed to the testing floor. The yielding displacements considered in this study are the displacements corresponding to yielding loads of the reinforcing bars.

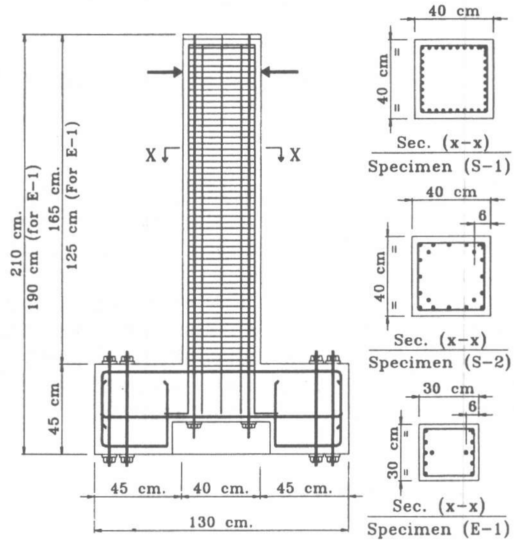

Fig. 1: Details of test specimens

2.2. Experimental procedures

In order to obtain inelastic response behavior for the above-mentioned PC piers, a pseudo-dynamic testing technique [4] was used in which load was applied quasi-statically during the test and the restoring force was measured directly from the loading test system. The used ground acceleration was the modified Hyogo-Ken Nanbu 1995 (NS direction) earthquake. For specimens (S-1) and (S-2), the time scale was kept the same as the original one while the maximum acceleration was considered as 563 gal and 474 gal respectively. The time

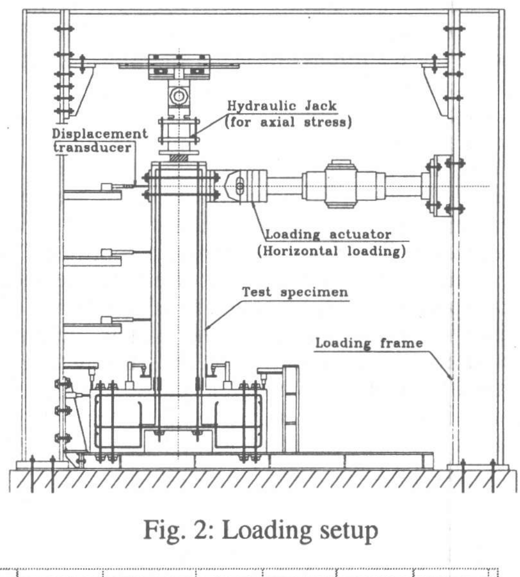

Fig. 2: Loading setup

Table 1: Details of specimens:

Specimen	Cross	a/d	Reinforcing bars		Prestressing tendons		Shear reinforcement		Natural	Flexural	Shear	strength
No.	Section	Ratio	Rein.	(As/bd) %	Tendons	(Aps/bd) %	Hoops	(Ash/bs) %	Period (sec.)	Capacity (tf.m)	Capacity (tf.m)	Ratio
S-1	40*40	4.00	32D13	2.65	-----	-----	D6@3 cm	0.47	0.30	31.2	46.88	1.5
S-2	40*40	4.00	16D10	0.79	8D12.7	0.63	D6@3 cm	0.47	0.30	27.82	48.75	1.75
E-1	30*30	4.10	10D16	2.22	2D13	0.3	D6@7 cm	0.27	0.23	13.9	15.4	1.11

interval was taken as 0.01 sec. For specimen (E-1), the time scale was reduced as half the original one while the maximum ground acceleration was maintained as 818 gal (Fig. 3). The time interval was taken 0.005 sec. An axial stress value of 1 Mpa was applied at the top of pier specimens. The used system consisted of the specimen, loading actuator, loading jack, loading cell, displacement transducers, data logger, personal computer that analyzes the inelastic earthquake response and controls the input data, measuring devices and another personal computer that controls the output data.

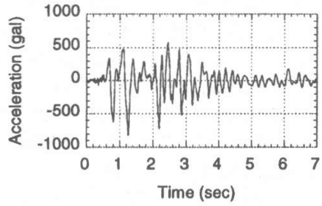

Fig. 3: Input earthquake ground acceleration for specimen (E-1)

3. TEST RESULTS

Fig. 4 shows the load-displacement curve for specimen (S-1) obtained from the pseudo-dynamic test. Both the bauschinger effect and stiffness degradation can be observed. The maximum displacement reached about 5.4 times the yielding displacement in the left side of the curve while it was about 2.1 times the yielding displacement in the right side of the curve, showing that the deformations occurred due to the earthquake excitation were drifting in the negative direction of loading. It can be observed from the figure that high energy was dissipated during the test. The maximum attained acceleration during the test was about -7 m/sec^2. Fig. 5 shows the displacement time history obtained during the test. The maximum attained displacement was about -0.11 m that occurred at the peak in the negative excursion after which the displacement time history fluctuates around a negative value. At the end of the test a residual displacement of about -2.5 cm was observed.

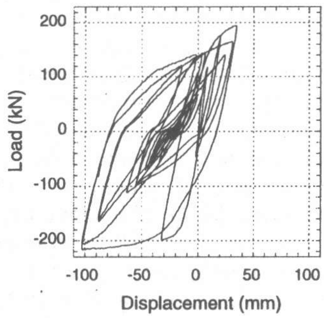

Fig.4: Observed load displacement curve for specimen (S-1)

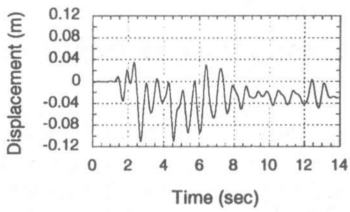

Fig. 5: Displacement time history for Specimen (S-1)

Fig. 6 shows the load-displacement curve for specimen (S-2). It can be observed that the bauschinger effect was dominant after unloading. Stiffness degradation during unloading was clear in both directions of loading. Also, a marked change in slope during reloading known as pinching [5] was clear. Pinching can be attributed to the fact that prestressed members usually show marked elastic recovery even after considerable inelastic deformations. Energy absorption was lesser than a comparable RC specimen (S-1) due to such hysteresis curve with marked pinching. Flexural crack widths were lesser than of specimen (S-1) during the test. The residual tensile forces in the PC tendons enabled to close previously opened cracks. Cover spalling was observed during testing specimens (S-1) and (S-2). Because of the existence of closely spaced transverse hoops, crushing was delayed inside the concrete core and buckling occurred only between two successive hoops in the plastic hinge locations. The maximum obtained acceleration during the test was about -7.5 m/sec^2. Fig. 7 shows the experimentally obtained displacement time history in which the maximum displacement was about -0.075 m. Although the difference between the maximum negative amplitude and the

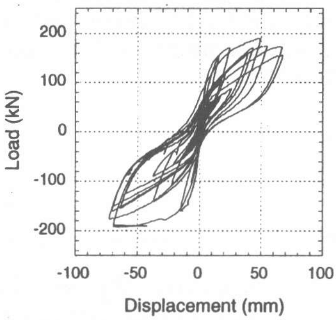

Fig.6: Observed load displacement curve for specimen (S-2)

following positive amplitude is higher than that of a comparable distance in Specimen (S-1), no shift of the response in the negative side was observed. Additionally, at the end of the test, the residual displacement was much smaller than that for specimen (S-1), and this can be considered as a great advantage of using such PC piers.

Fig. 8 shows the load-displacement curve for specimen (E-1). The displacement in the left side reached about 8 times the yielding displacement of the reinforcing bars while this value reached about 4.1 in the right side of the curve. It was clear from the curve that the specimen suffered from severe damage after the peak in the negative excursion of excitation at about 1.2 second, after which a permanent shift in the response was observed (Fig. 9). The residual displacement at the end of the test was about -2 cm.. This can be attributed to the fact that the specimen has a low ratio of prestressing tendons as compared to specimen (S-2). Consequently, the behavior is, more or less, similar to RC specimens. Also, because of the low strength ratio given to specimen (E-1), shear cracks were noticeable with marked widths. Spalling of concrete cover and buckling of all reinforcing bars were pronounced. Furthermore, extension of crushing inside the core concrete occurred and a final shear failure was observed.

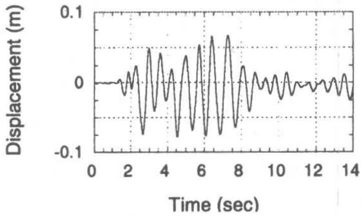

Fig. 7: Displacement time history for Specimen (S-2)

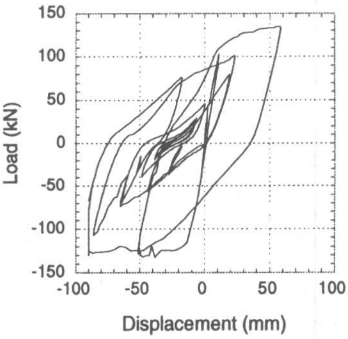

Fig.8: Observed load displacement curve for specimen (E-1)

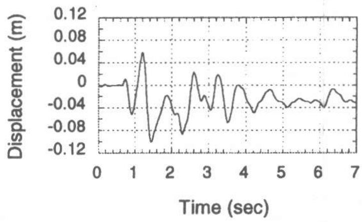

Fig. 9: Displacement time history for Specimen (E-1)

4. ANALYTICAL RESULTS

Response analyses was conducted for the three specimens. Takeda's tri-linear model [6] that includes the characteristic behavior of concrete cracking, yielding and strain hardening was used for the RC pier. Such a realistic conceptual model recognizes the continually degrading stiffness due to bond slip and energy absorption characteristics during load application. A modified Takeda's model [7] for PC which takes into consideration the effect of prestressing was used for the other PC specimens. The unloading stiffness $K\mu$, previously proposed for PC girders with grouted prestressing tendons where axial force was ignored [7], is defined based on the contribution of the prestressing tendons to the resulting flexural strength.

$$\lambda = (Aps.Fpy) / (As.Fsy+Aps.Fpy)$$

$$K\mu = (Mc+My) / (\theta c+\theta y) \, |\theta y/\theta m|^{\alpha}$$

where $\alpha = 0.5$ if $\lambda \leq 0.5$ $\qquad \alpha = \lambda$ if $\lambda \geq 0.5$

Aps = Area of prestressing tendons
Fpy = Yielding stress of prestressing tendons
As = Area of reinforcing bars
Fsy = Yielding stress of reinforcing bars
Mc = Cracking moment of the pier
My = Yielding moment of the pier
θc = Rotation angle of the pier at cracking
θy = Rotation angle of the pier at yielding
θm = Maximum rotation angle of the pier

Fig. 10 shows the analytical load-displacement curve for specimen (S-1). Although the unloading stiffness is slightly different from the experimental one, the total energy dissipated is of a relatively good accuracy. The analytical displacement time history shown in Fig. 11 declared that the residual displacement was about -2 cm. Although there was a smaller shift in the displacement time history after the negative peak acceleration, the overall response can well simulate the experimental results.

Fig. 12 and Fig. 13 show the analytical load-displacement curves for specimen (S-2) and (E-1) respectively. A considerable accuracy was obtained when comparing with the previous experimental results especially for specimen (E-1). The analytical displacement time history for specimen (S-2) obtained experimentally is shown in Fig. 14. The comparison between experimental and analytical load-displacement curves for specimen (S-2) shows a difference in the total energy dissipated. This shows that the model has to be further modified in case that higher prestressing ratios and/or ungrouted tendons are to be used. The analytical displacement time history for specimen (E-1), shown in Fig. 15, indicates that although a relatively smaller response and residual displacement were obtained, an overall good accuracy, when compared with the experimental results, can be observed.

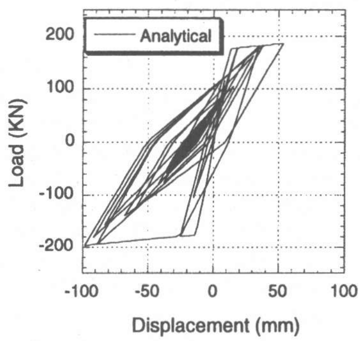

Fig.10: Predicted load displacement curve for specimen (S-1)

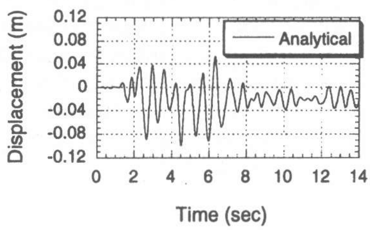

Fig. 11: Displacement time history for Specimen (S-1)

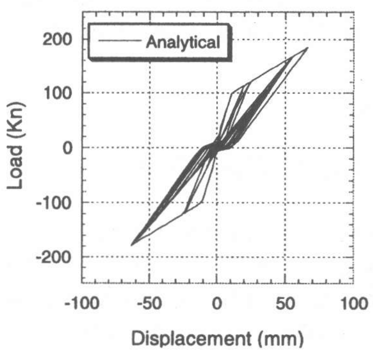

Fig.12: Predicted load displacement curve for specimen (S-2)

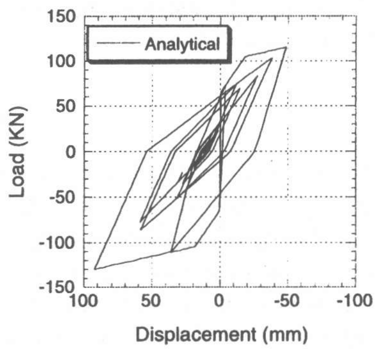

Fig.13: Predicted load displacement curve for specimen (E-1)

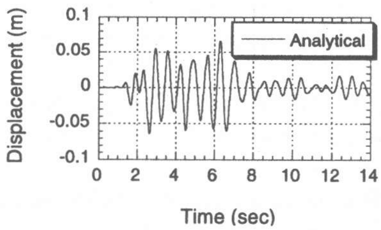

Fig. 14: Displacement time history for Specimen (S-2)

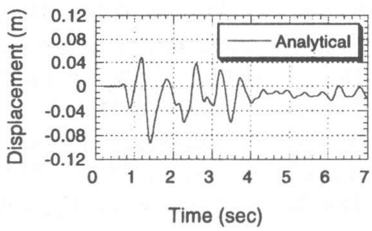

Fig. 15: Displacement time history for Specimen (E-1)

4. CONCLUSIONS

In order to clarify the inelastic response behavior of prestressed concrete piers under severe earthquake, three small scaled specimens were tested. The first specimen was RC control specimen while the other two were PC specimens. The specimens were tested using pseudo-dynamic testing technique. Response analyses were also conducted for the same specimens using Takeda's model for RC specimen and modified Takeda's model for PC specimens. A comparison between the experimental and analytical results was carried out. It can be concluded that the usage of PC piers has the following merits and disadvantages:

1. The usage of PC piers has the advantage of decreasing the residual displacement and the response, as compared to RC piers, when excited with the Hyogo-Ken Nanbu 1995 ground acceleration.
2. The residual cracking patterns, after earthquake excitation, of PC piers are better than that of ordinary RC piers.
3. A lower energy dissipation capacity, depending on the used relative ratio of prestressing tendons, is obtained. Consequently, the ratio of prestressing tendons, to be used, should be chosen in such a way that balances the merits and disadvantages.
4. A good agreement between the experimental and analytical results for the case of RC and PC piers with low ratio of grouted prestressing tendons was achieved.
5. For PC piers with a high ratio of ungrouted prestressing tendons, the analytical results can simulate the experimental results to some extent. A modification to the model has to be further developed.

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