論文 Influence of Strength Ratio on Behavior of Partially Prestressed Concrete Bridge Piers

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ABSTRACT: In order to clarify the inelastic response behavior and the interaction between ductility and shear strength of partially prestressed concrete (hereafter known as PPC) bridge piers, specimens with low strength ratios were tested under statically reversed cyclic loading. It was clarified how concrete shear strength decreases as a result of increasing ductility. The applicability of the model by Priestley et al. for RC columns was examined for PPC bridge piers. To account for the differences between the results obtained experimentally and by Priestley's et al. model, some modifications were proposed.
KEYWORDS: Earthquake resistant structures; bridge piers; partially prestressed concrete; cyclic loading tests; shear resistance; ductility

1. INTRODUCTION

Satisfactory seismic response of concrete structures requires that brittle failure modes should be inhibited. Since it is common practice to rely on ductile inelastic flexural response of plastic hinges to reduce the strength requirements for structures responding to strong seismic attack, it is necessary to inhibit shear failure by ensuring that the shear strength exceeds the shear corresponding to maximum feasible flexural strength [1]. In other words, in order to prevent occurrence of such failure of piers, the strength ratios should be more than unity. Nevertheless, It is generally known that for RC members, although the strength ratios may exceed unity, the members can still fail in shear because of the effect of cyclic load in decreasing the shear strength.

On the other hand, prestressed concrete is known for its ability to increase the shear strength. The use of PPC piers was previously proposed to obtain low residual displacements after an earthquake [2, 3, 4 and 5]. Yet it is not understood what is the optimum economical strength ratio that can ensure that shear failure will not occur for PPC piers when it is subjected to earthquake loads. Therefore, the objectives of this study were to quantitevely clarify how much increase in shear strength can be attained as well as to clarify how the strength ratio can influence the inelastic response and failure mode of PPC piers. Concrete shear strengths given in codes are almost assumed constant. Conversely, when a bridge pier is exposed to an earthquake excitation, it is shown that concrete shear strength contribution is generally variable as it usually degrades with increasing displacement ductility until it becomes negligible at high ductility levels. Therefore, it was proposed herein to study the interaction of shear strength and ductility for the previously studied PPC piers. Applicability of methods previously developed for RC bridge piers [1, 6, 7 and 8] were examined. Finally, modifications based on experimental data were proposed to predict the interaction of shear and displacement ductility of PPC piers with low strength ratios.

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2. EXPERIMENTAL PROGRAM

2.1 SPECIMEN VARIABLES AND TESTING SETUP

Since the objectives of this study were to identify the inelastic response behavior of PPC bridge piers and to study the effect of having low strength ratios on the behavior of the specimens, four specimens with low strength ratios were tested. The strength ratios are defined herein as the ratios of shear capacities to flexural capacities. The shear capacities are calculated based on ACI Code 318 requirements. Strength ratios of the specimens ranged from 1.07 up to 1.18 to study their effect on the resulting failure mode. The strength ratios were obtained through employing different shear resistance values resulted from changing the shear reinforcement and the prestressing level of the specimens. The 1st specimen (named E-1) was a RC control specimen while the other three specimens were PPC specimens. Nominal compressive strength of concrete was 35 MPa. The mechanical prestressing ratio (λ), which is defined as the contribution of the prestressing tendons to overall capacity of the cross section [9] ranged from 0.00 for the RC control specimen to 0.37 for the PPC specimens. Specimens were tested under statically reversed cyclic loading. Variables of test specimens are shown in **Table 1** while details of the specimens are shown in **Fig. 1**.

An axial stress level of 1 MPa was imposed to the specimens. Specimens were mounted vertically and were loaded horizontally through a loading actuator. The experimental loading setup is shown in **Fig. 2** while full description and instrumentation can be found elsewhere [2, 3 and 5]. Predetermined increasing displacement amplitudes were fed to the specimens during testing.

Table 1: Variables of test specimens

Spec. No.	Variables of PC Pier Specimens											
	Strength Ratio	Mech. Prestressing Ratio	Cross Sec.		Reinforcing Bars & PC Tendons in Cross Section				Shear Reinforcement		Normal Stress *(MPa)*	
			Dim.	a/d	Rein.	%	PC	%	Ties	$A_{sh}/b.s$	Axial	PC
E-1	1.18	0.00	30*30	4.20	12D16	2.68	x	x	D6@7cm	0.27	1.0	0.0
E-2	1.07	0.30	30*30	4.20	10D16	2.22	2D13	0.30	D6@7cm	0.27	1.0	1.5
E-3	1.14	0.21	30*30	4.20	12D16	2.68	2D13	0.30	D6@5.5cm	0.34	1.0	1.5
E-4	1.11	0.37	30*30	4.20	10D16	2.22	2D17	0.50	D6@5.5cm	0.34	1.0	3.0

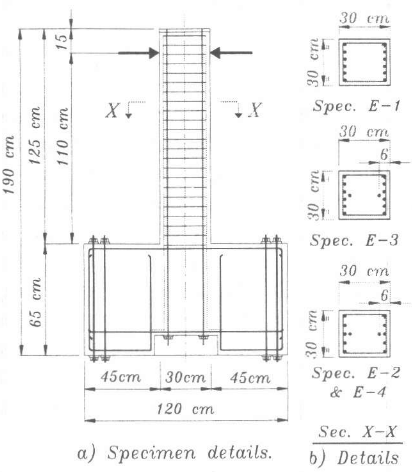


Fig. 1: Test specimens

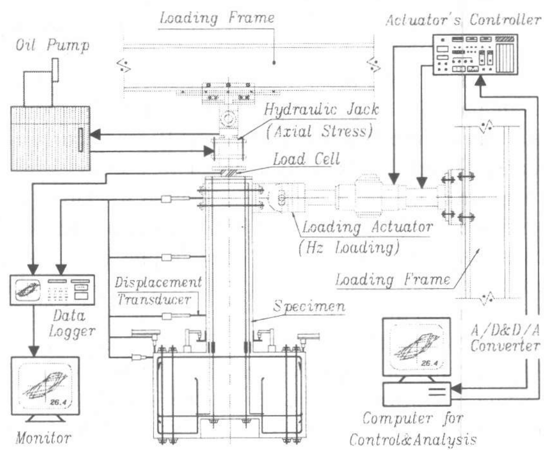


Fig. 2: Experimental loading setup

3. TEST RESULTS

The inelastic behavior of each specimen in terms of load-displacement relationship was obtained. As expected, the load carrying capacities of specimens increased by increasing the mechanical prestressing ratios through addition of prestressing tendons to the control specimen.

Fig. 3 shows the load-displacement curve of the control specimen E-1 with a strength ratio of 1.18. It was found that pinching of the hysteretic behavior was noticeable due to shear. Also, strength degradation was pronounced at a displacement of 52 mm (drift angle (δ) = 0.047) due to a final shear failure mode after flexural yielding of the reinforcing bars. For ductility considerations of PPC specimens and since there are no clear yielding points, the yielding point was assumed to be the intersecting point of extrapolating the stiffness after cracking and stiffness after yielding. **Fig. 4** shows the load-displacement curve of the PPC specimen E-2 with a mechanical prestressing ratio of 0.3 and a strength ratio of 1.07. It was found that although the strength ratio was less than that of specimen E-1, no pinching was observed. Strength degradation commences at a higher displacement amplitude of that of specimen E-1 followed by a final shear failure mode after flexural yielding of the prestressing tendons. **Fig. 5** shows the load-displacement curve of the PPC specimen E-3 with a mechanical prestressing ratio of 0.21 and a strength ratio of 1.14. No pinching was encountered. Strength degradation leading to a shear failure after flexural yielding commenced at a displacement of 70 mm (δ = 0.064). **Fig. 6** shows the load-displacement curve of the PPC specimen E-4 with the highest mechanical prestressing ratio of 0.37 and with a strength ratio of 1.11. Almost, no pinching was observed. Strength degradation after flexural yielding of the prestressing tendons was observed to start at a displacement amplitude of 70 mm (δ = 0.064) followed by a final shear failure.

It can be observed that slight decreases in the strength ratios of the PPC specimens than that of the control RC specimen did not necessarily lead to shear failures at displacement amplitudes lower than that of the RC specimen since the overall shear capacity was enhanced by prestressing.

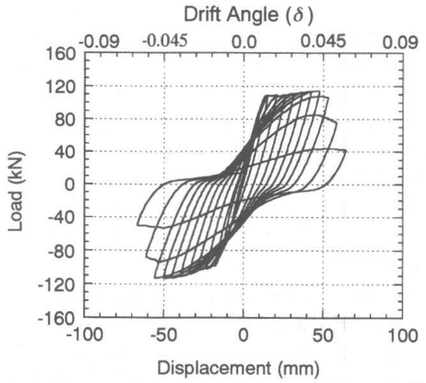


Fig. 3: Load-displacement curve of spec. E-1

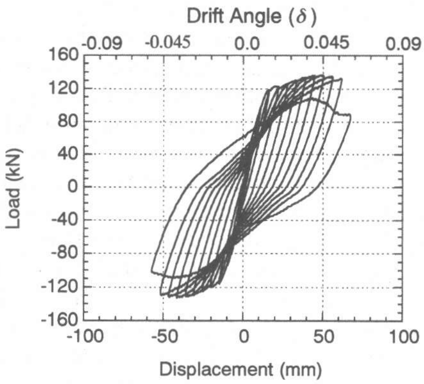


Fig. 4: Load-displacement curve of spec. E-2

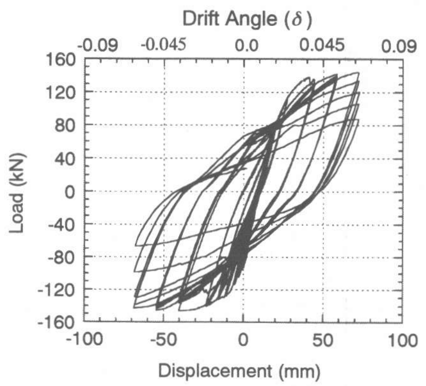


Fig. 5: Load-displacement curve of spec. E-3

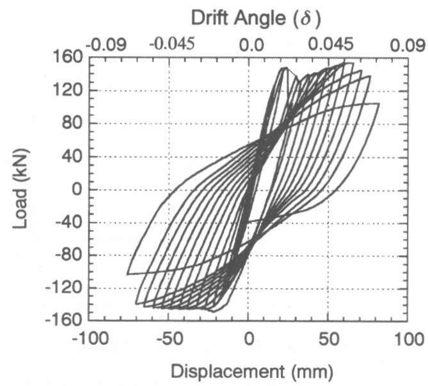


Fig. 6: Load-displacement curve of spec. E-4

4. DUCTILITY AND SHEAR STRENGTH INTERACTION

Examination of bridge columns in earthquakes enables a clear distinction to be made between brittle modes of failure and ductile shear failure, where a degree of ductility develops hinges before shear failure occurs. Therefore, it is necessary to inhibit shear modes of failure by increasing the strength ratio [1]. This is acknowledged in the conceptual model for shear strength proposed by the *Applied Technology Council* (*ATC-Seismic Design 1981*). In this model the shear strength is assumed to decrease in a linear fashion as the member displacement ductility increases. If the shear force corresponding to flexural strength is less than the residual shear strength, ductile flexural response is ensured. If it is greater than the initial shear strength, a brittle shear failure results. If the shear force is between the initial and the residual shear strength, the shear failure occurs at ductility corresponding to the intersection of the strength and force-deformation characteristics. Although this behavior is reasonably well accepted, it has not found its way into concrete design codes, except in very few codes. Codes such as the *ACI-318-89* and the *New Zealand Concrete Code* simply assume that the concrete shear strength can be ignored when $P < 0.05\, f'_c A_g$ and $P < 0.1\, f'_c A_g$, respectively. Neither of them includes an explicit relationship between ductility and shear strength [10]. The AIJ *recommendations* proposed that for a ductile member, the permissible diagonal compressive stress is progressively reduced as the plastic rotation increases which is somehow similar to the *ATC model*. Nevertheless there are two deficiencies in the *AIJ recommendations* (Design 1988). The first one is that its formulation implies that axial loads do not influence the ultimate shear strength while the other one is that these recommendations were proposed only for rectangular sections.

4.1 MODEL BY PREISTLEY ET AL.

Recently, considerable experimental research for RC columns, particularly by Ang et al. [1] and Wong et al. [6] has been directed towards a better definition of the shear strength/ductility relationship. Additional results from Priestley et al. [7, 8] have supplemented this data. Furthermore, Priestley and Benzoni [11] refined the models by Ang et al. and Wong et al. His model was shown to be accurate enough to a full range of experimental database. Priestley et al implemented further experimental data by Mattock and Wang, Jirsa and Woodward, Xiao et al. and an extensive Japanese database by Watanabe and Ichinose to verify the superior accuracy of his proposed model over the others [10]. Additionally, Priestley and Benzoni [11] and Xiao et al. [12] presented modifications of the original model to account for special cases of RC columns.

In the approach by Priestley et al. for RC columns, the components of the overall shear resistance were separated in the form of steel truss component (V_s), axial force component (V_p) and concrete component (V_c). The magnitude of V_s depends on the transverse reinforcement content. The V_p component depends on the column aspect ratio. The concrete component $(V_c) = k\sqrt{f'_c}\,A_e$ where k depends on the level of ductility (**Fig. 7**) and A_e is the effective shear area = $0.8\, A_g$.

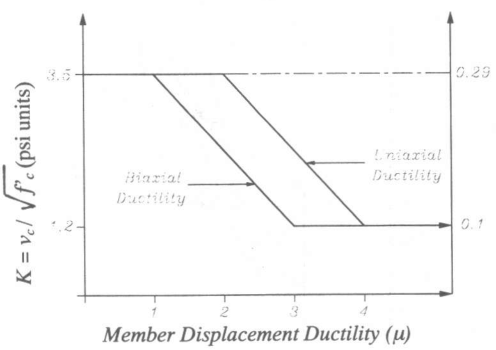
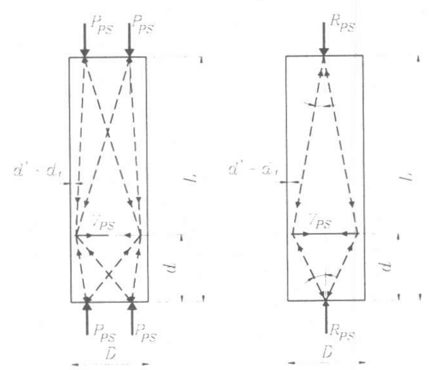


Fig. 7: Model for concrete shear strength versus ductility by Priestley et al.



a) Prestressing forces *b) Resultant*

Fig. 8: A truss model for prestress component

4.2 A PROPOSED MODEL FOR PPC PIERS WITH LOW STRENGTH RATIOS

Since the model by Priestley et al. was proposed for RC columns and not for PPC piers, the effect of the prestressing tendons in increasing the shear strength was not accounted for. Therefore, there was a necessity to have a model for such contribution as proposed herein. It was considered that the column axial stress due to prestressing (ΣP_{PS}) could enhance the shear strength by an arch action that forms inclined struts (**Fig. 8**). The critical section was at a distance = d from the footing surface. Individual prestressing tendons could participate in the overall shear strength through formation of individual compression struts. A simple calculation method was suggested through identification of the resultant (R_{PS}) magnitude and point of application of the external applied prestressing forces calculated from static and solving the equilibrium truss mechanism in **Fig. 8(b)**.

4.3 JUSTIFICATION OF METHODOLOGY

During testing of specimens, strains in the reinforcing ties were measured and the associated stresses were obtained. Using these measured stresses and the determined inclination angle of the major shear crack, the V_s component could be experimentally obtained at different loading stages. Calculating V_p and V_s and then subtracting from the experimental lateral loads at each loading stage, the V_c component could be calculated. **Fig. 9** shows the experimental components of shear resistance of the specimens. From this V_c component, the k factors could be calculated and could be plotted versus ductility. A comparison between these results and those of Priestley's et al. approach for RC columns (**Fig. 10**) shows some differences at almost all ductility levels. similar findings were observed by Priestley and Benzoni [11] and Xiao and Marirossyan [12] where it was found that the upper limits which identify Priestley's model are conservative especially at low displacement ductility factors.

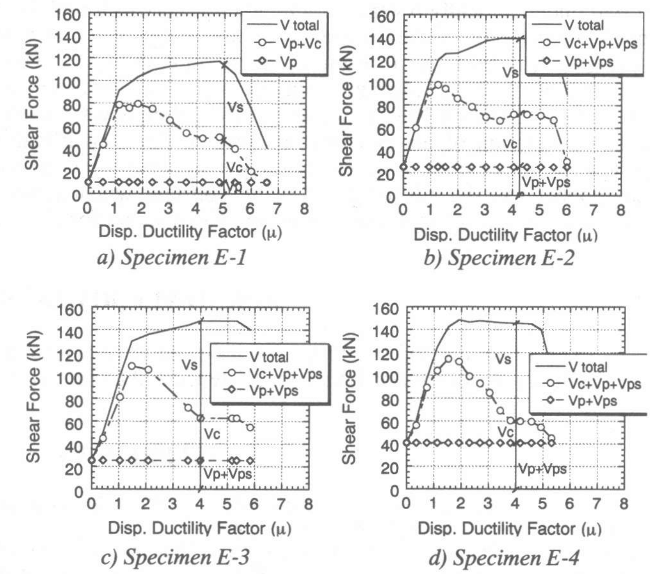


Fig. 9: Ductility versus shear components of specimens

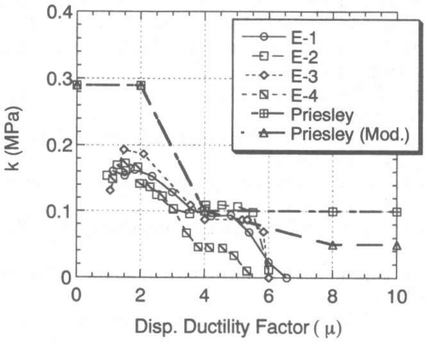


Fig. 10: Experimental and Priestley's et al. k values versus displacement ductility

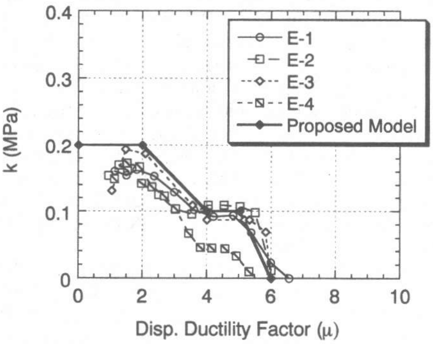


Fig. 11: Justification of the proposed k values versus displacement ductility

In order to have a better simulation of the obtained experimental k values than those obtained by Priestley's et al. approach, a modification in Priestley's et al. model is proposed herein to account for cases of PPC piers with low strength ratios. The proposed model assumes that a value of 0.2 is to be used as k value, if *MPa* units are used, up to a ductility of 2.0 followed by a gradual decrease in k when increasing the ductility up to 4. Then, a constant k of 0.1 can be used up to a ductility of 5.0 followed by a gradual decrease in k till it vanishes at a ductility of 6.0. It should be noted that, opposite to Priestley's model, k value vanishes in the proposed model because this model was intended for PPC piers with low strength ratios. It is expected that if the PPC piers have higher strength ratios, the k values might have values rather than zero for ductility more than 6. Nevertheless such expectation requires further experimental verification.

5. CONCLUSIONS

In order to clarify the inelastic response behavior and the interaction between ductility and shear strength of partially prestressed concrete bridge piers, specimens with low strength ratios were tested under statically reversed cyclic loading. It was found that the addition of prestressing tendons could increase the shear resistance of the bridge piers. It was also clarified how concrete shear strength decreases as a result of increasing ductility. The interaction between the concrete component of shear resistance and displacement ductility was clarified. The applicability of the interaction model by Priestley et al. for RC columns was examined for PPC bridge piers. To have a better simulation of the experimental results, some modifications to Priestley's et al. model were proposed. A separate axial prestressing component was added to Priestley's et al. model. Such component has the advantage of being constant when increasing the ductility. Furthermore, the model of concrete component was changed to account for PPC piers with low strength ratios.

ACKNOWLEDGMENTS

The authors would like to acknowledge the financial support of the Grant-In-Aid for Scientific Research of the Ministry of Education, Science and Culture in Japan.

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