Earthquake damage of prestressed concrete viaduct structures

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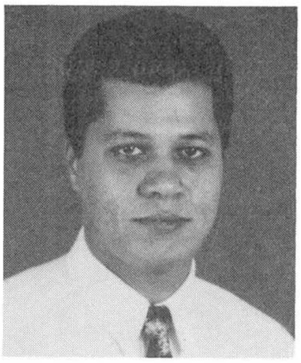

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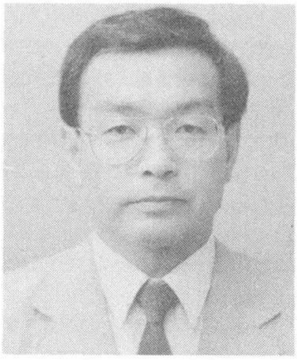

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Summary

In order to clarify the inelastic response behavior of prestressed concrete (hereafter PC) viaduct structures under severe earthquake excitations, experimental and analytical studies were conducted. Small-scaled models were designed so as to represent actual viaduct structures. Specimens representing PC girders were made and tested experimentally. The experimental program consisted of reversed cyclic loading tests and substructured pseudo-dynamic test in which the PC girder was tested experimentally while the RC piers of the viaduct model were simulated analytically. Response analyses were also conducted and a comparison between the experimental and analytical results was performed. It was clarified that not only the RC piers but also the PC girders are subjected to inelastic deformations and may undergo extensive damage due to earthquake excitation.

1. Introduction

Viaduct structures and elevated bridges are becoming more common for highways and railways. A common type of both the viaduct structures and elevated bridges generally consists of RC piers and PC girders. Various loading tests have been carried out to study the inelastic response behavior of the elevated bridges when subjected to ground motions. Since the girders of these bridges are generally hinged to the piers, only the piers are subjected to earthquake forces. On the other hand, because of the monolithic moment-resisting connection between the superstructure and the piers in the viaduct structures, less response can be observed in the piers bottom ends and another plastic hinges at the tip of the piers can be formed allowing for some energy absorption at these locations [1]. Additionally, not only the piers but also the girders might have some damage. Yet not enough tests have been performed to study either the inelastic response behavior of such PC girders or the complete viaduct structures in which some members may undergo extensive inelastic deformation and thus significantly affect the total response behavior and their integrity. The objective of this study is to obtain the inelastic response behavior of such PC viaduct structures under severe earthquake. In the current study, experimental and analytical studies were conducted. Specimens representing PC girders were tested under statically reversed cyclic loading to obtain the hysteretic load-displacement behavior for three specimens while one specimen was tested using a substructured pseudo-dynamic test in which a modified excitation of the Hyogo-Ken Nanbu 1995 earthquake was used. Takeda's tri-linear model was used for the RC elements. One component model was employed for the inelastic member model during the analytical study.

2. Outlines of tests

2.1. Test specimens

Four partially PC members representing the PC girders of the viaduct structures were tested (Fig. 1). All details of the specimens are shown in Fig. 2. The specimens have the same dimensions but the significant differences were the amount and arrangement of the prestressing tendons and the reinforcing bars. Specimens (A-1), (A-2) and (A-3) were tested under statically reversed cyclic loading while specimen (B-1) was tested by a substructured pseudo-dynamic test. The upper parts of the specimens that represent bonded PC girders were placed monolithically with lower parts that represent reinforced concrete piers of the viaduct models as can be shown in Fig. 2. The design philosophy implicitly requires that shear failure be prevented or delayed so that the member under consideration may fail in flexure. The details of specimens are shown in Table 1. The compressive strength of concrete is about 400 kgf/cm^2. Yielding stresses of the reinforcing bars are 3600 kgf/cm^2 and 3400 kgf/cm^2 for D13 and D6 respectively while the yielding stresses of the prestreesing tendons are 10500 kgf/cm^2 and 12200 kgf/cm^2 for tendons D17 and D11 respectively. The specimens were fixed on a testing floor. The load was applied to the specimen at a height of 150 cm from the bottom end of the PC girder (Fig. 2).

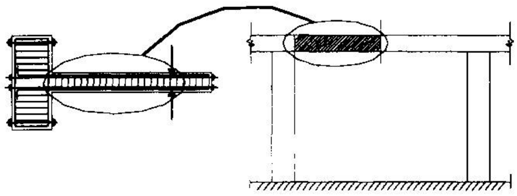

Fig. 1: Experimental test specimens

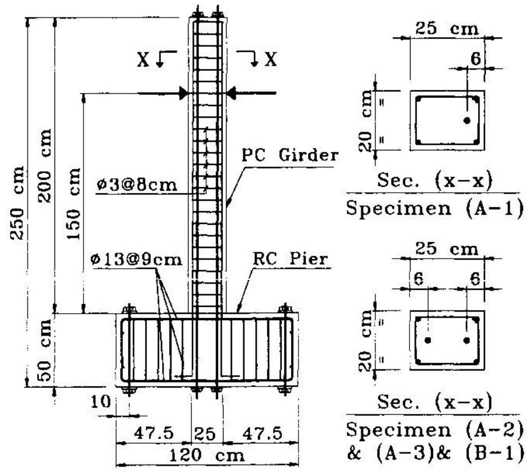

Fig. 2: Test specimens

Table 1: Details of test specimens

Specimen No.	Prestressed		Reinforcement	
	R.S. *	L.S. **	R.S. *	L.S. **
A-1	ø17	--	2D6	2D6
A-2	ø11	ø17	2D13	2D6
A-3	ø17	ø17	2D6	2D6
B-1	ø11	ø11	2D13	2D13

* R.S. : Right side of specimen at the test setup
** L.S. : left side of specimen at the test setup

2.2. Statically reversed cyclic loading tests.

Statically reversed cyclic loading tests were carried out for specimen (A-1), (A-2) and (A-3). The objective of conducting these tests is to clarify the load-displacement characteristics of the PC girders. The specimens were tested using the setup shown in Fig. 3. The repetition of each cycle was 10 times. The applied displacements imposed to the specimens through the actuator were multiples of the prestressing tendons yielding displacements. The yielding displacements considered in the current study are the measured displacements corresponding to attaining the yielding loads.

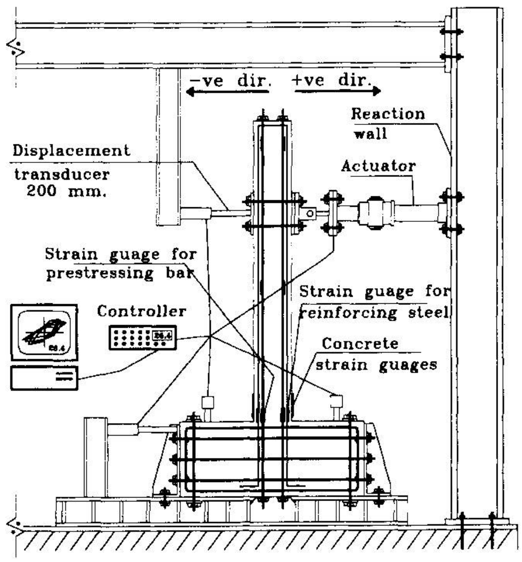

Fig. 3: Experimental Loading setup

2.3. Substructured pseudo-dynamic test

2.3.1. Structural model

Substructured pseudo-dynamic test is a computer-controlled experimental technique in which direct numerical time integration is used to solve the equation of motion. By incorporating substructuring concept, it is possible to test only the critical member effects on seismic response of the whole structure. In the current study, the considered viaduct model shown in Fig. 1 has a 1/10 scale of the real one. The PC girder of the viaduct structure is considered as the experimental substructure. It was assumed that the viaduct girder is symmetric with respect to the center. Consquently, the PC girder has two identical cantilever members satisfying compatibility and equilibrium conditions at the center as can be seen in Fig. 1. The used model numbering scheme, dimensions and degrees of freedom are shown in Fig. 4.

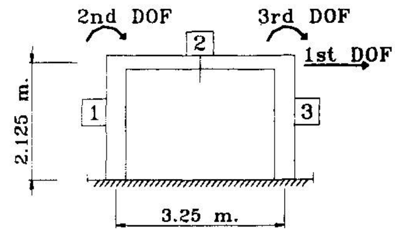

Fig. 4: Model used in the sub-structured pseudo-dynamic test

2.3.2. Experimental procedures

For testing the experimental member in specimen (B-1) of the viaduct model shown in Fig. 1, a substructured pseudo-dynamic testing technique was employed in which the load was applied quasi-statically during the test and the dynamic effects were simulated numerically [2]. Analytical inelastic mechanical model and its restoring force-displacement model were used for all members in the structure except the PC girder [3] whose restoring force was measured directly from the test [4]. Takeda's tri-linear model [5] was used for the RC members. Such a realistic conceptual model recognizes the continually degrading stiffness due to bond slip, shear cracks and energy absorption characteristics during earthquake excitation. The earthquake used for the test was the modified Hyogo-Ken Nanbu 1995 earthquake (NS direction). The time scale was amplified as half the original one while the maximum ground acceleration was 818 gal [6]. Fig. 5 shows the used input ground acceleration. Since the constitutive operator splitting (OS) method was found to be the most effective one in terms of both stability and accuracy [7], it was implemented in this study for integration of equation of motion numerically. The integration time interval was taken as 0.0005 second while the earthquake time interval was taken as 0.005 second.

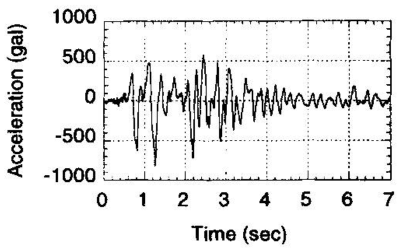

Fig. 5: Input ground acc. for the substructured pseudo dynamic test

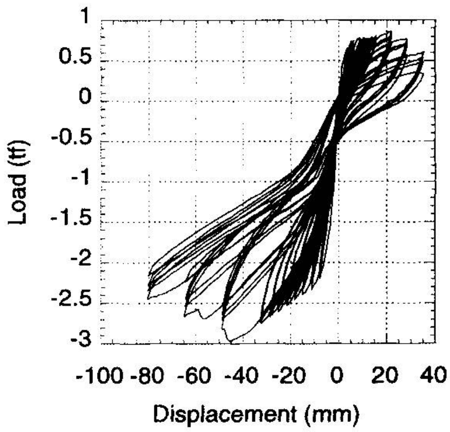

Fig. 6: Load-displacement curve for specimen (A-1)

3. Test results

3.1. Statically reversed cyclic loading tests

The hysteresis loops for all specimens indicated stiffness degradation, Bauschinger effect for both the unloading and reloading and also showed pinching of hysteretic load-deformation curves. Cover spalling and buckling of longitudinal steel bars were also noticed. The inelastic response behavior of the PC girders changed, during the tests, resulting in a decrease in the load carrying capacity. Therefore, adequate ductility without decrease of the load carrying capacity should be maintained to satisfy the requirements of seismic resistant structures.
Fig. 6 shows the load-displacement curve for the specimen (A-1). For the left side of the load- displacement curve, the maximum displacement was about 3 times the yielding displacement of the prestressing tendon. On the right side of the load-displacement curve, the reached displacement was about 13 times the yielding displacement of the reinforcing bars. The hysteresis loops show that the

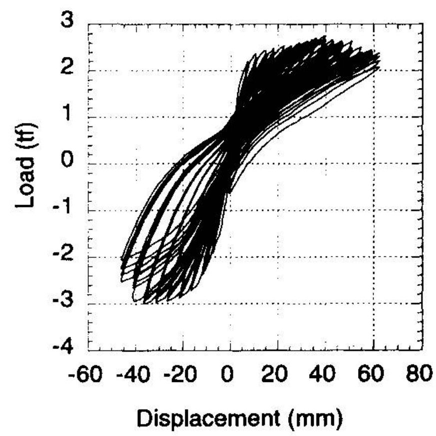

Fig. 7: Load-displacement curve for specimen (A-2)

deformational capacity is different in the two directions because of the unsymmetric arrangement of the prestressing tendons. Fig. 7 shows the load-displacement curve for the specimen (A-2). The displacement reached about 4 times the yielding displacement of the prestressing tendon in the left side of the curve, while it reached about 8.5 times the yielding displacement of the reinforcing bars in the right side. It can be seen, from Fig. 7, that the skeleton curve in the right side can be approximated by a skeleton curve for prestressed concrete while the skeleton curve for the left side can be approximated by a tri-linear model for reinforced concrete. The last observation can be attributed to the relative ratio of prestressing tendons to reinforcing bars in the specimen. Also because of the unsymmetry of the cross section, the ultimate load was different in the two directions. Fig. 8 shows the load- displacement curve for the specimen (A-3), the test was performed till the displacement reached about 2.5 times the yielding displacement of the prestressing tendon. The skeleton curve for both directions of loading can be approximated by a skeleton curve for prestressed concrete because the resistance of the cross section was mainly dependent on the prestressing tendons rather than the reinforcing bars.

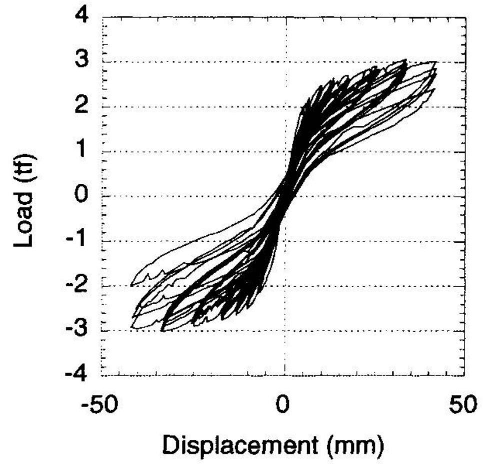

Fig. 8: Load-displacement curve for specimen (A-3)

3.2. Substructured pseudo-dynamic test

The model used for the substructured pseudo-dynamic (PSD) test is shown in Fig. 4. The resulting hysteresis loops for the left end of the PC girder is shown in Fig. 9-A. Both the Bauschinger effect and pinching of the hysterestic loops are clear in the figure. The figure also shows that a considerable damage occurred to the PC girder during the earthquake excitation. From other test results, it was noticed that not much energy was dissipated in the plastic hinge formed at the top of the RC pier. Fig. 9-B shows the moment-rotation curve at the bottom end of the pier. It can be noticed from the curve that a considerable damage occurred during the earthquake excitation. A comparison between the hysteresis curves shown in Fig. 9-A and 9-B shows that not only the RC piers but also the PC girders may undergo extensive damage during earthquake excitation. The time history of the response acceleration in Fig. 10 shows that the maximum obtained accleration was about 12.2 m/sec^2. The time and direction of this maximum acceleration are consistent with the time and direction of the maximum input ground acceleration. The time history of the response displacement in Fig. 12 shows that the maximum displacement reached about 8.5 cm.

4. Analytical results

The last viaduct model was studied analytically. One component model proposed by Giberson [8] was employed for the inelastic member model. The inelastic moment-rotation relationships of the springs were calculated by means of ordinary flexural theory. Furthermore, the rotation due to bond-slip of the reinforcing bars and the prestressing tendons from the connecting joints was taken into consideration using Ohta's method [9]. Takeda's tri-linear model was used for the RC piers. A value of 2% modal damping was assumed for all modes until one member has a rotation angle equals to the yielding rotation angle. The damping was then considered equals to zero because only the hysteretic damping was dominant after the yielding displacement is reached.
Fig. 9-C and 9-D show that the analytical results agreed well with the experimental ones in terms of energy absorption, damage extent and ductility factor. The analytical acceleration and displacement time histories in Fig. 11 and 13 also showed good agreement with the experimental ones in terms of the maximum values, corresponding time and the over all time histories.

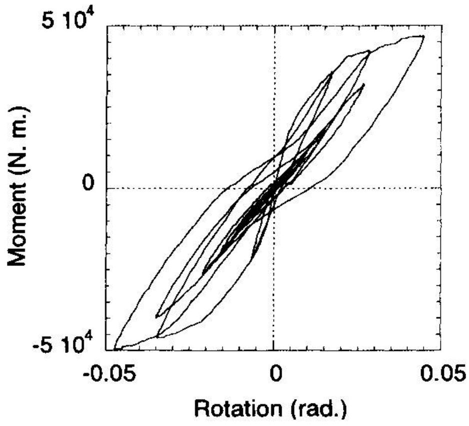

Fig. (9-A): Left end of PC girder (Experimentally)

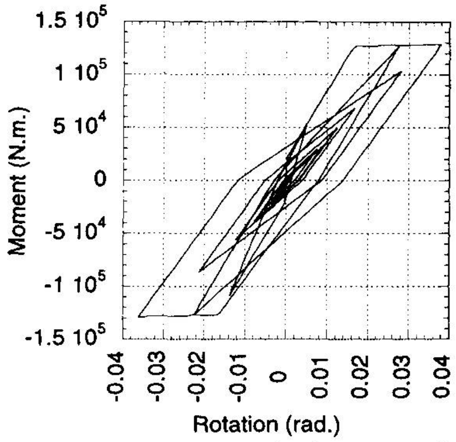

Fig. (9-B): Bottom end of RC pier during the substructured pseudo-dynamic test.

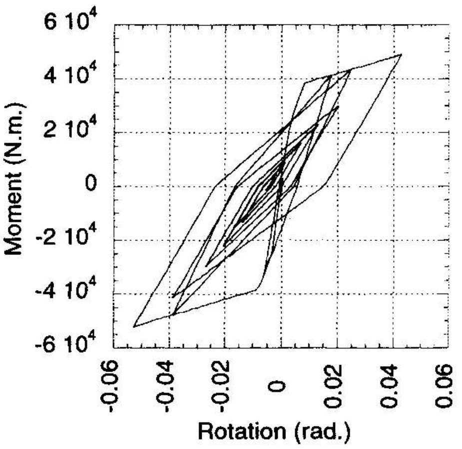

Fig. (9-C): Left end of PC girder (Analytically)

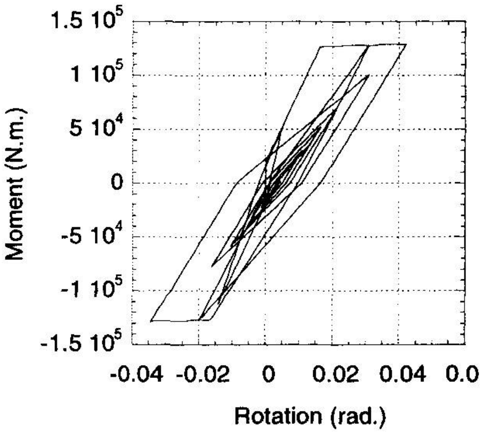

Fig. (9-D): Bottom end of RC pier (Analytically)

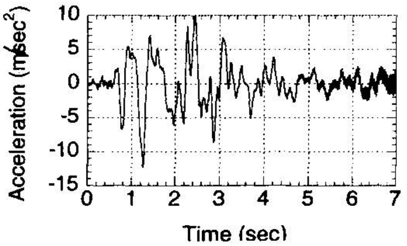

Fig. 10: Acceleration-time history of the substructured PSD test (Experimentally)

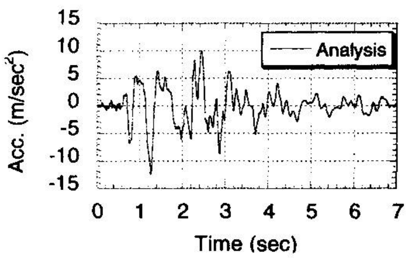

Fig. 11: Acceleration-time history of the substructured PSD test (Analytically)

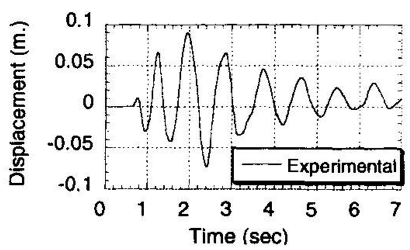

Fig. 12: Displacement-time history of the substructured PSD test (Experimentally)

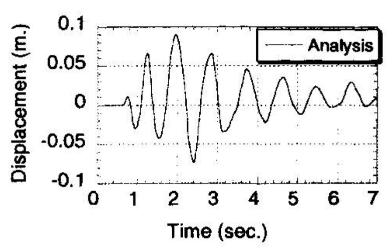

Fig. 13: Displacement-time history of the substructured PSD test (Analytically)

5. Conclusions

In order to clarify the inelastic response behavior of partially prestressed concrete girders of a viaduct structure under severe earthquake, small-scaled specimens representing half of a girder bay of a viaduct structure were tested under statically reversed cyclic loading tests and by a substructured pseudo-dynamic method. Analytical investigation for the same viaduct model was also carried out. From the results, it can be concluded that:

1. Not only the RC piers but also the PC girders are subjected to inelastic deformation that may cause a considerable damage during real earthquake excitations. As a consequence, adequate care should be given to the PC girders design to satisfy the requirements of a seismic resistant structure.
2. The inelastic response behavior of the PC girder of a viaduct structure can be remarkably changed. Consequently, the load carrying capacity decreases. Therefore, adequate ductility without decrease of the load carrying capacity should be maintained in order to ensure a seismic resistant viaduct girder.
3. A good agreement between both the experimental and analytical results was obtained in the resulting time histories, hysteresis curves and dissipated energy during earthquake excitation.
4. Further analytical response analyses have to be carried out in order to accurately identify the significant parameters of the PC girder that influence the overall response behavior.

Acknowledgment

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