論文 Dynamic Response Behavior of Prestressed Concrete Viaduct under Severe Earthquake

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ABSTRACT: In order to clarify inelastic behavior of prestressed concrete (hereafter PC) girders of a viaduct under severe earthquake, four small scale PC members were designed to represent actual viaduct structures, Three specimens were tested under statically reversed loading, while the 4th specimen was tested using a substructure pseudo-dynamic test. The inelastic response analysis of the 4th specimen based on one component model for the analytical modeling of members was conducted during the pseudo-dynamic test.
KEYWORDS: Earthquake resistant structures; viaduct structures; prestressed concrete; elasto-plastic behavior; substructure pseudo-dynamic test; dynamic analysis.

1. INTRODUCTION

Because of the importance of viaduct structures and elevated bridges in the construction of highways and railways, especially in Japan, various loading tests have been carried out to study the inelastic behavior of reinforced concrete bridge piers subjected to severe ground motions. Since the girders of these bridges are hinged to the piers, only the piers are affected during earthquakes. On the other hand, because of the fixation between girders and piers in viaduct structures, both the girders and the piers are affected. Yet not enough tests have been performed to study the inelastic behavior of partially PC girders of viaduct structures. The objective of this study is to obtain the inelastic response behavior of such partially PC viaduct structures.

In this study, 4 cantilever PC members were tested experimentally in which 3 specimens of PC girders were subjected to static cyclic loading imposed by an actuator. The significant difference among them was the amount and arrangement of prestressing tendons and longitudinal reinforcement. Substructure pseudo-dynamic test using an amplified excitation of 1995 HyogoKen Nanbu earthquake was carried out for another PC member. Cyclic loading was then applied to the same specimen till failure. The complete structure, from which the 4th specimen was selected, was treated analytically during the pseudo-dynamic test using one component model for the inelastic member model and a reinforced concrete hysteretic model, proposed by Takeda [4], for the piers of the viaduct structure. Experimental results expressed in terms of hysteretic load-deformation behavior and time histories were conducted. The plastic deformability expressed in terms of ductility factor was also examined.

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2. STRUCTURAL MODEL, SPECIMENS AND EXPERIMENTAL PROCEDURES

2.1. Test specimens

The PC girders of a viaduct structure are taken in this study as the experimental substructure where they might undergo extensive plastic deformations when a severe earthquake occurs. For simplicity and due to the difficulty of implementing members with varying inflection points, it was assumed that the viaduct girder is symmetric with respect to the centerline of each bay and thus only half of one girder bay was taken in this study as the experimental specimen (Fig. 1). This simplification would affect the accuracy of the total response. The considered viaduct shown in Fig. 8 had a time period of 0.25 sec.

Four partially PC specimens were tested in this study. They had the same dimensions but they differed in the amount of prestressing tendons and reinforcing bars for the purpose of studying the influence of ductility on the resulting response behavior. Details of specimens are shown in Fig. 2 and in Table 1. The concrete compressive strength is about 400 kgf/cm^2, yielding of reinforcement is 3600 kgf/cm^2 for D13 and 3400 kgf/cm^2 for D6 while the yielding of prestressing tendons is 10500 kgf/cm^2 for D17 and 12200 kgf/cm^2 for D11. All specimens were designed so that the shear capacity is higher than the flexural one. The ratio of the maximum allowable shearing force to the maximum allowable bending force was 1.86 for specimen (A-1), 2.03 for specimens (B-1), (B-2) and 2.52 for specimen (B-3). All specimens were tested using the same setup shown in Fig. 3. The loading point for all specimens was fixed at a height of 150cm from the face of the PC member. The bottom parts of specimens were rigid enough to represent the actual case of piers for a viaduct structure as well as to enable the observation of the damage that can occur to the girder during a real earthquake excitation. All specimens were fixed to the testing floor. The yielding displacement considered in this study is the displacement corresponding to yielding load of the reinforcing bars. Specimens (A-1), (B-1) and (B-2) were cyclically tested. The repetition of each cycle was 10 times. The interval for the applied displacements was

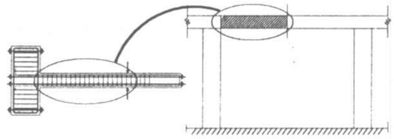

Fig. 1: Test specimens

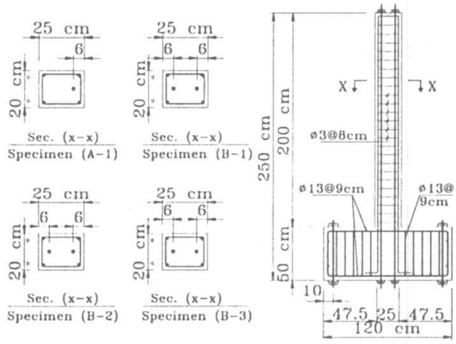

Fig. 2 : Test specimens

Table 1 :
Reinforcement provided to specimens

Specimen No.	Prestressed		Reinforcement	
	R.S. *	L.S. **	R.S. *	L.S. **
A-1	D17	--	2D6	2D6
B-1	D11	D17	2D13	2D6
B-2	D17	D17	2D6	2D6
B-3	D11	D11	2D13	2D13

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

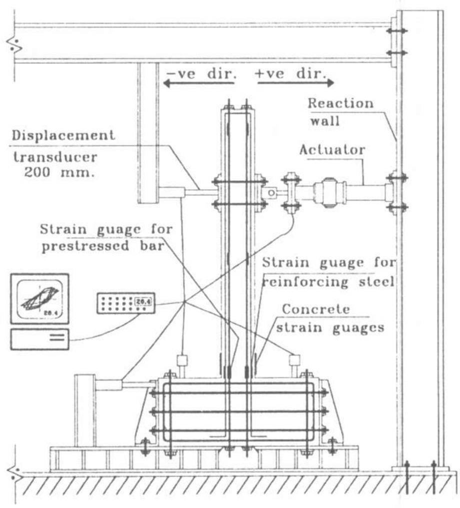

Fig. 3 : Loading setup

multiples of the reinforcing bars yielding displacement. Specimen (B-3) was tested using substructure pseudo-dynamic test.

2.2. Pseudo-dynamic test for specimen (B-3) of the 4th viaduct structure

In order to obtain inelastic response behavior for the above-mentioned viaduct structure shown in Fig. 8, Substructure pseudo-dynamic testing technique [2] was used in which load was applied quasi-statically during the test and the dynamic effects are simulated numerically. Analytical inelastic mechanical model and its restoring force-displacement model were used for all members in the structure except for the PC girder [2] where its restoring force was measured directly from the loading test system. One component model proposed by Giberson [1] was used for the inelastic member model in which the spring stiffness was determined using Otani's method [3]. Takeda's tri-linear model [4] that includes the characteristic behavior of concrete cracking, yielding and strain hardening of steel was used for the RC piers. Such a realistic conceptual model recognizes the continually degrading stiffness due to bond slip, shear cracks and energy absorption characteristics of the structure during load application. Takeda's inelastic mechanical model for PC was used for the girder of the viaduct model. The earthquake excitation used was the modified Kobe earthquake (NS 1995) where the time scale was magnified as half the original one while the maximum ground acceleration was 818 gal (Fig. 9) to allow the observation of the response. The time interval was taken 0.005 sec. After the completion of the substructure pseudo-dynamic test, a cyclic test was performed on the same specimen until failure in order to obtain the ultimate limit state. The used system in the substructure pseudo-dynamic test consists of the specimen, loading actuator, personal computer analyzes the inelastic earthquake response of the viaduct structure and controls the input/output data, measuring devices and another personal computer that controls the output data. The procedures of the test are as follows:

1- The displacement of the girder is calculated analytically by the program.
2- By means of a D/A converter, the calculated displacement is converted from digital value into analog one that can be applied to the specimen through the actuator.
3- Immediately after the actuator gives the required displacement to the specimen, the restoring force is measured using the displacement transducer fixed to the specimen at the same level of the actuator (Fig. 3), The computer records this restoring force after converting the data from analog to digital through an A/D converter.
4- The previous restoring force is used in the next step.
5- The previous steps (step 1-4) are repeated for the entire duration of the input earthquake.

3. TEST RESULTS

The hysteresis loops for all specimens indicated stiffness degradation, bauschinger effect for both the unloading and reloading and pinching of hysteretic load-deformation curves. Hair cracks were initiated during loading of specimens in the form of shear cracks in both the two directions of loading for all specimens. With the application of larger loads, the number and width of these cracks increased. Cracks opening and closure was remarkably noticed during loading in the two directions for all specimens. Cover spalling and buckling of longitudinal steel bars was also noticed in addition to cutting of some steel stirrups near the critical sections for shear of the specimens. The inelastic response behavior of PC girder changed during conducting the tests because of shear cracks occurred at critical sections. As a consequence, the load carrying capacity decreased.

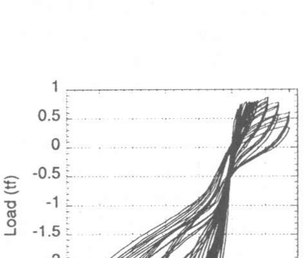

Fig. 4: Load-displacement curve for specimen (A-1)

Fig. 4 shows the load-displacement curve for specimen (A-1). The test was continued, after reaching the maximum load, till the load decreased to become about 80% of the maximum load. For the left side of the load-displacement curve, the reached displacement is about 11 times the yielding displacement of reinforcing bars and about 3 times the yielding displacement of the prestressed tendon. On the right side of the load-displacement curve, the reached displacement is about 13 times the yielding displacement of reinforcing bars. The hysteresis loop shows that the deformational capacity is different in the two directions of load application because of the unsymmetric arrangement of prestressed tendons.

Fig. 5 shows the load-displacement curve for specimen (B-1). After the maximum load, the test was performed as can be shown in the left side of the load-displacement curve till the load decreased to become about 80% of the maximum load after which the test was stopped in this direction. The displacement reached about 4 times the yielding displacement of the prestressed tendon. At this stage of the test, the prestressed tendon D17 that resisted the +ve direction of loading did not reach yielding. The test was continued as shown in the right side of the load-displacement curve till the displacement reached about 8.5 times the yielding displacement of the reinforcing bars. It can be seen from the hysteresis loop that the skeleton curve for the right side of the load-displacement curve can be approximated by a skeleton curve for prestressed concrete while the skeleton curve for the left side of this curve can be approximated by the tri-linear model of reinforced concrete. The last observation can be attributed to the relative ratio of prestressed tendons to reinforcing bars in the specimen. Also because of the unsymmetry of the cross section, the ultimate load is different in the two directions.

Fig. 6 shows the load-displacement curve for specimen (B-2), the test was performed after yielding of the prestressed tendons till the displacement reached about 2.5 times the yielding displacement of the prestressed tendon. The skeleton curve for both directions of loading can be approximated by a skeleton curve for prestressed concrete because the resistance of the x-sec. is dependent mainly on the prestressed tendons rather than the reinforcing bars.

Fig. 10 shows the actuator load time history for the substructure pseudo-dynamic test of specimen (B-3). The resulting hysteresis loops is shown in Fig. 12-C & 12-D show also pinching of the hysteresis loops. Fig. 12-A through 12-E show the hysteresis loop for each member of the viaduct in Fig. 8 from which it can be seen that not only the piers but also the PC girders undergo extensive damage during the earthquake excitation. The time history for the 1st DOF in Fig. 11 shows that the time and direction of the max. acceleration is consistent with the time of the max. input ground acceleration. After the completion of the pseudo-dynamic test, cyclic loading for the same specimen was conducted. Fig. 7 shows the resulting load-displacement

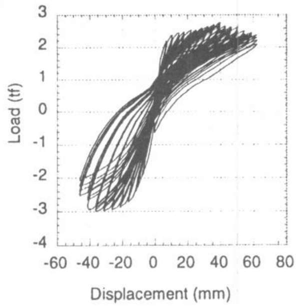

Fig. 5: Load-displacement curve for specimen (B-1)

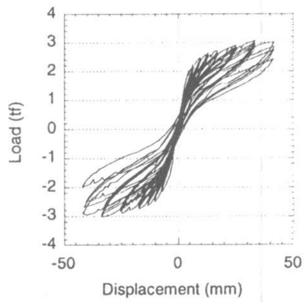

Fig. 6: Load-displacement curve for specimen (B-2)

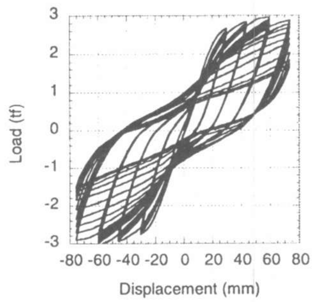

Fig. 7: Load displacement curve for cyclic loading of specimen (B-3)

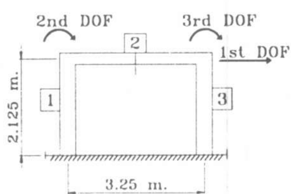

Fig. 8 : Model used in pseudo-dynamic test.

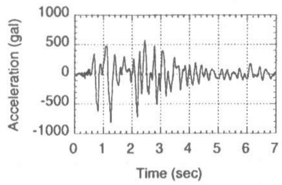

Fig. 9: Input ground acceleration for the 4th viaduct model

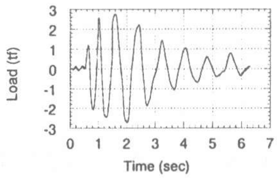

Fig. 10: Time history of actuator load for the 4th viaduct model

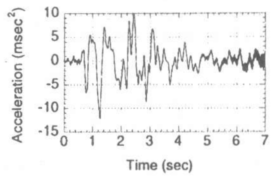

Fig. 11: Acceleration time history of the 4th viaduct model

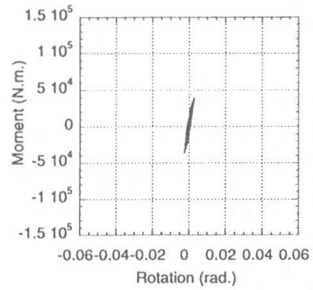

Fig. 12-A: Moment rotation curve for top end of member No. 1

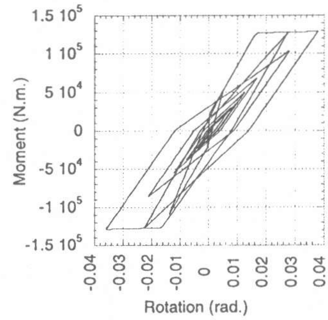

Fig. 12-B: Moment rotation curve for bottom end of member No 1

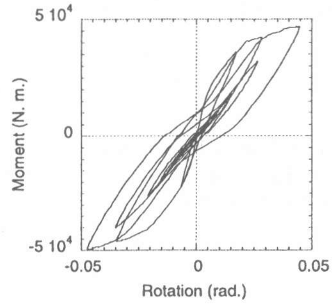

Fig. 12-C: Moment rotation curve for left point of member No 2

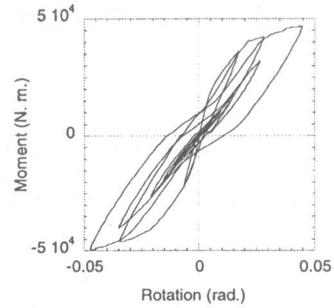

Fig. 12-D: Moment rotation curve for right point of member No 2

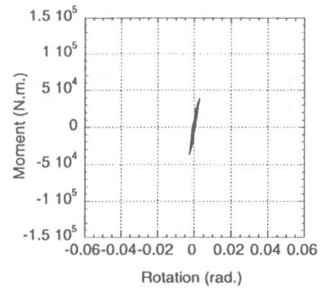

Fig. 12-E: Moment rotation curve for top end of member No. 3

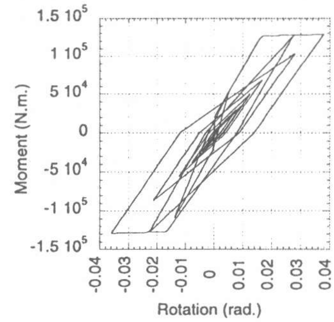

Fig. 12F: Moment rotation curve for bottom end of member No 3 curve. The test was continued till the displacement reached about 5 times the yielding displacement of the prestressed tendons.

4. CONCLUSIONS

In order to clarify the inelastic response behavior of partially prestressed concrete girders of a viaduct under severe earthquake, 4 small scale specimens representing half of a girder bay of a viaduct structure were made. The main difference among them was in the relative ratio of prestressing tendons to reinforcing bars. Cyclic loading tests were carried out for three specimens while the 4th specimen was tested using a substructure pseudo-dynamic test and then a cyclic loading test was conducted on the same specimen. For the substructure pseudo-dynamic test, one component model was used for the RC piers. From the test 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. As a consequence, adequate care should be given to the PC girder design to satisfy the requirements of a seismic resistant structure.

2) In general, since PC girder is designed mainly to resist dead and live loads, Prestressed tendons and steel bars are arranged unsymmetrically in the x-sec. Therefore, they can not resist the reversed loading resulted from earthquake excitation whereas one direction will suffer severe damage. Symmetrical specimens (B-2) and (B-3) can obtain the same load carrying capacity in the two directions rather than unsymmetrical specimens (A-1) and (B-1).

3) The inelastic response behavior of PC girder of a viaduct can be remarkably changed due to shear cracks. Consequently, the load carrying capacity decreases for all specimens. Therefore, adequate ductility without decrease of the load carrying capacity should be maintained in order to ensure a seismic resistant structure of the viaduct girders.

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