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# Seismic Hazard of Viaduct Transportation Infrastructure

Wael Zatar

## Abstract

Prestressed concrete viaduct structures are used for the construction of many highways and railways. The objective of this study was to clarify the inelastic response behavior of partially prestressed concrete viaduct structures during severe earthquake excitations. A study that includes experimental and analytical phases was carried out. Small-scaled models were employed so as to represent actual viaduct structures. Specimens representing the PC girders of the viaduct structures were tested experimentally. The first technique was statically reversed cyclic loading test to study the inelastic response behavior of the PC girders and to obtain the hysteretic-load deformational characteristics. The sub-structured pseudo-dynamic testing technique was implemented as the second testing technique. During the sub-structured pseudo-dynamic test, the PC girder was tested experimentally, and the RC columns of the viaduct structure were simulated analytically. An amplified excitation of the 1995 Hyogo Ken Nanbu earthquake was used. Response analyses for the viaduct model were carried out. A comparison between the experimental results and results obtained from response analyses was made. An agreement between the experimental and analytical results was found. The study revealed that not only the RC columns but also the PC girders may undergo extensive damage during severe earthquake excitations.

**Keywords:** earthquake-resistant structures, viaduct structures, sub-structured pseudo-dynamic tests, statically reversed cyclic loading tests, partially prestressed concrete, dynamic analysis

## 1. Introduction

Viaduct structures and elevated bridges are becoming more common for railways and highways. During the past few decades, partially prestressed concrete has been used for the construction of viaduct structures. Earthquakes have a habit of identifying structural weakness and concentrating the damage at these locations. Elevated bridges and viaduct structures have little or no redundancy in structural systems, and failure of one structural element or connection is thus more likely to result in collapse [1]. Therefore, it is of a great importance to carefully understand the seismic response behavior of viaduct structures. Experimental investigations have been carried out in the past to study the deformation and cracking of partially prestressed concrete beams under static and cyclic fatigue loading [2]. 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. Moreover, few research studies have been carried out to study the effect of prestressing the reinforced concrete piers of highway bridges [3, 4].

On the other hand, because of the monolithic moment-resisting connection between the superstructure and the columns of the viaduct structures, less bending moments were expected in the bottom ends of the columns, and other plastic hinges at the tip of the columns may result to allow for some energy dissipation at these locations. Additionally, not only the columns but also the girders might have some damage. Yet not enough tests have been performed to study the inelastic response behavior of the partially prestressed concrete (hereafter known as PC) girders of the viaduct structures [5–7]. The objective of this study was to obtain the inelastic response behavior of such PC viaduct structures due to severe earthquake excitation.

A study that includes experimental and analytical phases was carried out. Specimens representing the PC girders of the viaduct structures were tested experimentally. Statically reversed cyclic loading and sub-structured pseudo-dynamic testing were conducted. The objective of the statically reversed cyclic loading test was to study the inelastic response behavior of the PC girders and to obtain the hysteretic-load deformational characteristics. During the sub-structured pseudo-dynamic test, the PC girder was tested experimentally, and the RC columns of the viaduct structure were simulated analytically. Response analyses for the viaduct model in terms of hysteretic moment-rotation curves and time histories were carried out. The plastic deformability expressed in terms of the ductility factor and the dissipated energy was examined. A comparison between the experimental results and results obtained from response analyses was made.

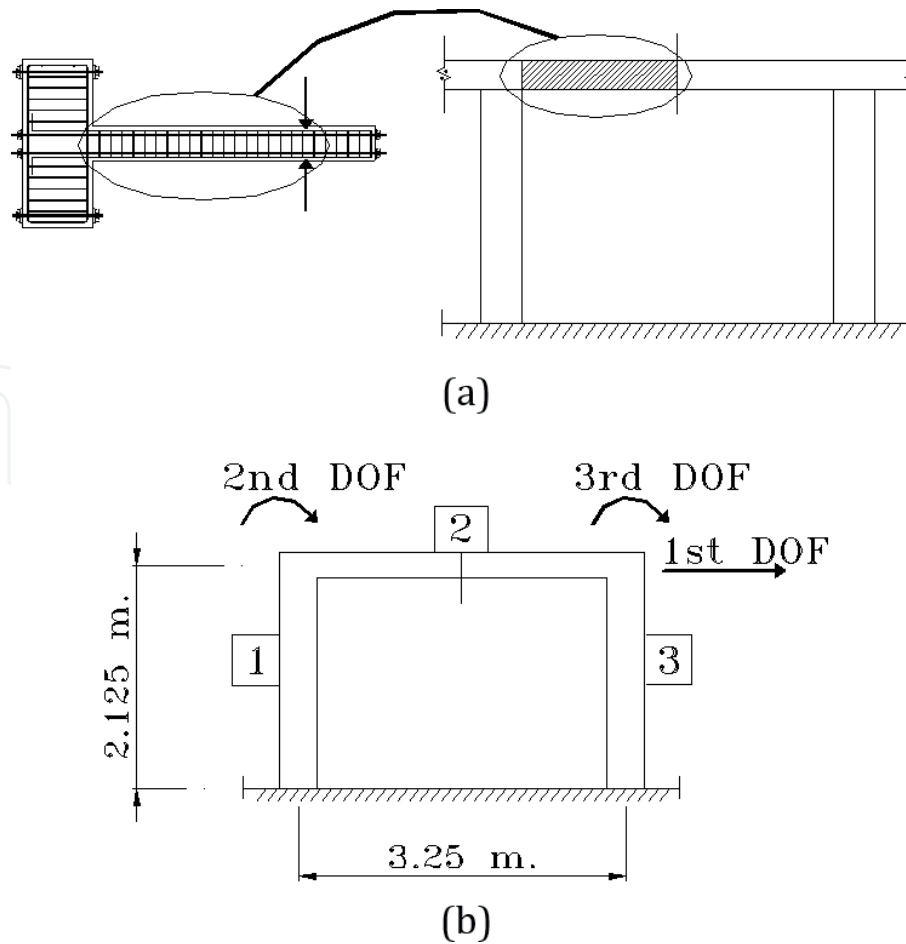
## **2. Outlines of tests**

### **2.1 Test specimens**

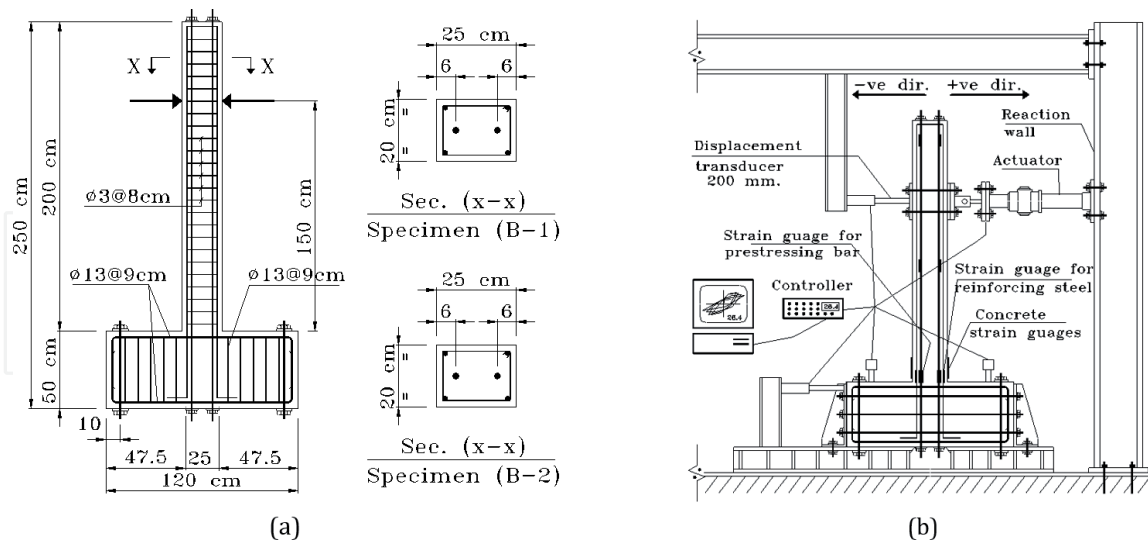
The viaduct model (**Figure 1**) was constructed at a 1/10 scale of a full-size viaduct structure. The PC girder of the viaduct structure was considered as the experimental substructure. It was reasonably assumed that the viaduct girder was symmetric with respect to the center of each bay. This assumption was made for simplicity and due to the difficulty of implementing members with different inflection points, and because of the linearly varying moment distribution. Consequently, the PC girder was assumed to be composed of two identical cantilever members satisfying compatibility and equilibrium conditions at the center. Only half of the PC girder was considered as the experimental member (**Figure 1a**). The model numbering scheme, dimensions, and degrees of freedom are shown in **Figure 1b**.

Two partially PC specimens representing the experimental PC girder members of the viaduct models and named B-1 and B-2 were tested. The specimens have the same dimensions, reinforcing bars, and prestressing tendons arrangement. Specimen B-1 was tested using a statically reversed cyclically loading, while specimen B-2 was tested using a sub-structured pseudo-dynamic test. The upper part of each specimen (**Figure 2**) represents the PC girder part. The PC girder part was placed monolithically with a lower part. The lower part represents the moment-resisting connection and the upper part of the reinforced concrete column of the viaduct model. The lower part has sufficient rigidity to allow the observation of the damage of the PC girders during testing.

The PC girder part has a depth of 25 cm, a width of 20 cm, and a length of 200 cm. The lower part of the specimen has a depth of 50 cm, a width of 50 cm, and a length of 120 cm. The girder part has two reinforcing bars with 13 mm diameter at each side of the section. The girder part has one D11 mm prestressing



**Figure 1.** *Experimental test specimen and model used during the sub-structured pseudo-dynamic testing: (a) Experimental test specimen; (b) model used in the sub-structured pseudo-dynamic testing.*



**Figure 2.**  
*Test specimens and loading setup: (a) test specimens; (b) loading setup.*

tendon at each side of the cross section (**Figure 2**). The mechanical prestressing ratio of the specimens is 0.55. The design philosophy implicitly requires that shear failure be prevented or delayed so that the member under consideration may dissipate, by flexure, energy larger than required for the applied earthquake. Therefore, relatively close-spaced transverse hoops were arranged for the entire length of the girder part. The rectangular hoops were 3 mm in diameter and were spaced at 8 cm.

The specimens were fixed to a testing floor by the use of side supports, pre-stressed rods, and high-strength bolts. The loading was applied through an actuator that was fixed at a height of 150 cm from the bottom end of the PC girder of each specimen (**Figure 2**). The corresponding a/d ratio is 6.8. The average compressive cylindrical concrete strength is  $400 \text{ kgf/cm}^2$ . The yield strength of the reinforcing bars is  $3400 \text{ kgf/cm}^2$ , and the yield strength of the prestressing tendons is  $12,200 \text{ kgf/cm}^2$ . Details of the specimens are shown in **Figure 2**.

## 2.2 Statically reversed cyclic loading testing

Statically reversed cyclic loading test was carried out for specimen B-1. The objective of conducting this test was to clarify the load-displacement characteristics of the PC girders. The specimen was tested using the setup shown in **Figure 2b**. The setup consisted of the specimen, actuator, reaction wall, testing floor, data loggers, computer for data acquisition, and displacement measuring devices. The yield displacement was the measured displacement corresponding to the recorded yield load. The imposed displacements to the specimen through the actuator were multiples of the prestressing tendons yielding displacement. Ten repetitions of each cycle were considered. Typically, ten repetitions cannot be attained during a real severe earthquake, but they were planned to fully clarify the load-displacement characteristics. **Figure 3** shows the input displacements that were applied to specimen B-1.

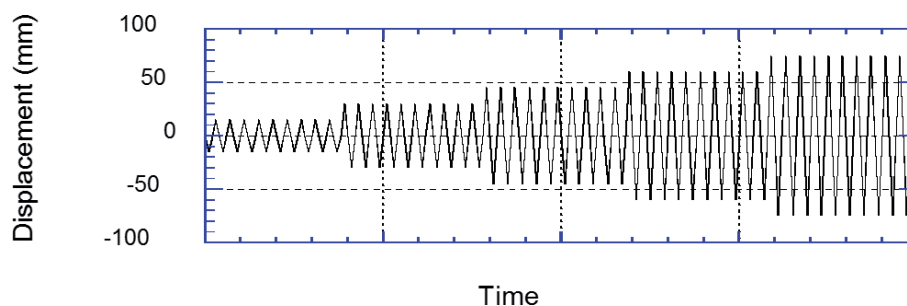
## 2.3 Sub-structured pseudo-dynamic testing

### 2.3.1 Structural model

Many numerical and experimental studies have been carried out to clarify the inelastic behavior of RC columns. However, very few experimental studies have been carried out to date on the response behavior of the full structures in which few members may undergo extensive inelastic deformations. The inelastic deformations of the few members may significantly affect the overall response behavior and the structure integrity of the full structure. The unavailability of test records for the full viaduct structures can be attributed to the high cost and scale of conducting the associated large tests.

Sub-structured 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 the sub-structuring concept, it is possible to test only the critical member effect on the inelastic seismic response of the whole structure.

The PC girder of the viaduct structure was considered as the experimental sub-structure. The PC girder was assumed to be composed of two identical cantilever



**Figure 3.**  
*Input displacements applied to specimen B-1 during the statically reversed cyclic loading test.*



members satisfying compatibility and equilibrium conditions at the center, and thus having only half of the girder as the experimental member (**Figure 1a**).

### 2.3.2 Experimental procedures

The sub-structured pseudo-dynamic testing technique was used for testing specimen B-2 of the viaduct model shown in **Figure 1b**. The load was applied quasi-statically during the test, and the dynamic effects were simulated numerically [8]. An analytical inelastic mechanical model and its restoring force-displacement model were used for all the RC members of the viaduct structure except for the PC girder [9]. The restoring force for the PC girder was measured directly from the loading test system [10].

One component model [11] was employed for the inelastic member model. The one component model consists of a linearly elastic member with two equivalent nonlinear springs at the member ends (**Figure 4a**). The rotational deformation of the member due to the bending moment was expressed as the sum of the flexural deformation of the linear elastic member and the rotational deformation of the two equivalent nonlinear springs. The spring constants are known as KPA and KPB (**Figure 4a**) and are determined using Otani's method [12]. The inelastic moment-rotation relationship of the spring was calculated by means of the ordinary flexural theory based on the assumption that the point of contra flexure was located at the center of each member. Furthermore, the rotations due to bond slip of the reinforcing bars as well as the prestressing tendons from the connecting joint were taken into consideration using Ohta's method [13] for all the members of the viaduct model.

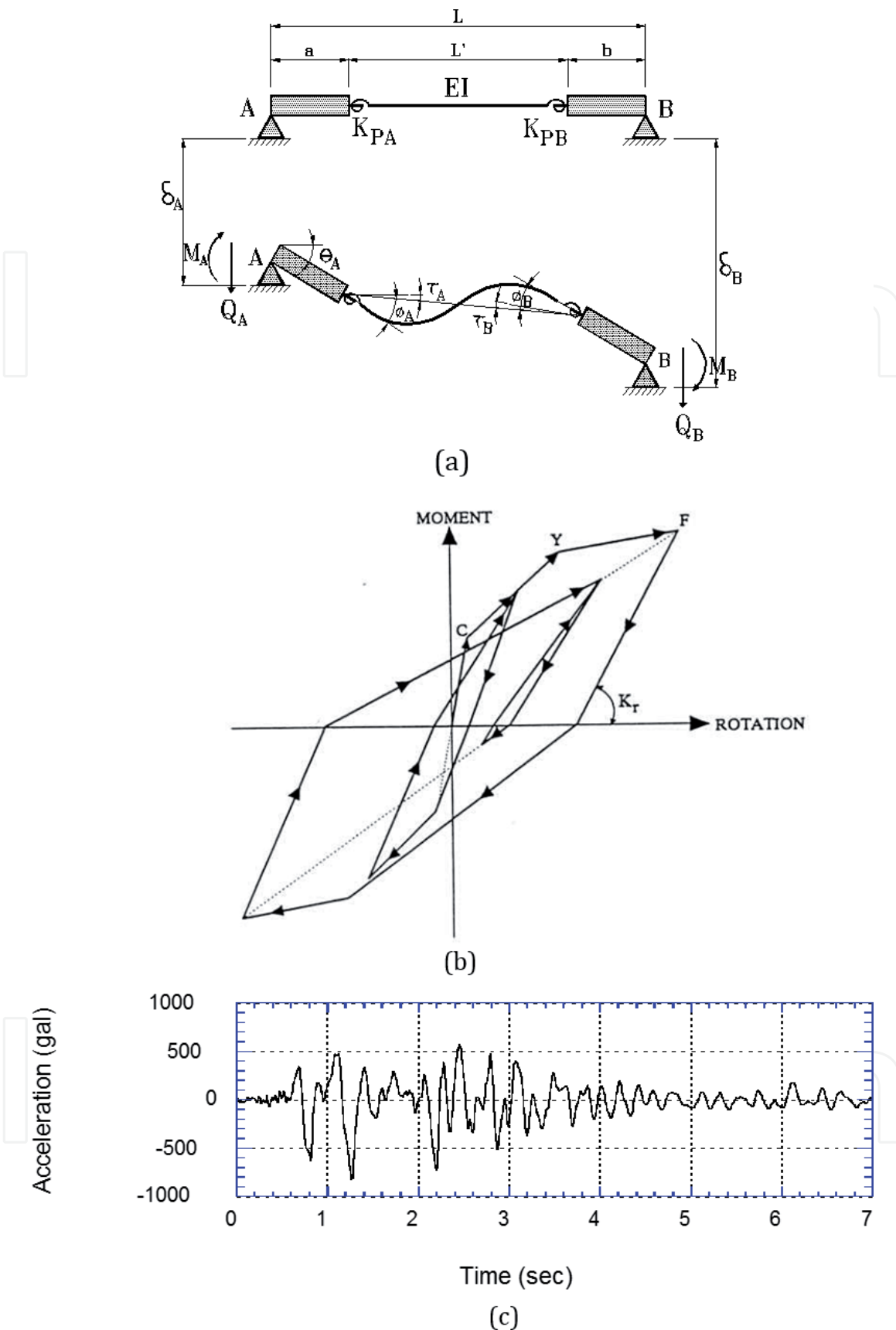
Takeda's et al. trilinear model [14] was used as the hysteretic restoring force model for the RC members (**Figure 4b**). Takeda's et al. model includes the characteristic behavior of concrete cracking, yielding, and strain hardening of the main reinforcement. Takeda's et al. model is a realistic and conceptual model that recognizes the continually degrading stiffness due to bond slip, shear cracks, and energy absorption characteristics of the structure during an earthquake excitation. The stiffness of Takeda's model during unloading ( $K_r$ ) was defined by Eq. (1):

$$K_r = (M_c + M_y) / (\theta_c + \theta_y) |\theta_y / \theta_m|^\alpha \quad (1)$$

where  $\alpha$  was the unloading stiffness parameter that was considered equal to 0.4 for the RC columns. The earthquake excitation during the sub-structured pseudo-dynamic test was the modified Hyogo-Ken Nanbu 1995 earthquake excitation (NS direction). The Hyogo-Ken Nanbu earthquake excitation was selected to represent a near-field excitation. The time scale was amplified to half the original time scale that was recorded during the original Hyogo-Ken Nanbu excitation. The maximum ground acceleration that was considered during the sub-structured pseudo-dynamic test was kept as the original acceleration (818 gal) that was recorded during the original excitation [15, 16] (**Figure 4c**).

The so-called mixed (explicit-implicit) integration method that was originally developed for finite elements analysis was found to be suitable for the sub-structured pseudo-dynamic test [10]. However, Nakashima et al. [17] found out that for the sub-structured pseudo-dynamic test, the constitutive operator splitting (OS) method is the most effective method in terms of both stability and accuracy. Consequently, the OS method was implemented in this study for the numerical integration of the equation of motion. The integration time interval was 0.0005 second, and the earthquake time interval was 0.005 second.

Two percent damping was assumed for each mode of the modal damping until the member under consideration experience a rotation angle equal to the yield



**Figure 4.** One component model, Takeda's hysteretic restoring force model, and input ground excitation: (a) One component model, (b) Takeda's hysteretic restoring force model; (c) input ground excitation (Hyogo-Ken Nanbu Earthquake, 1995, Kobe city, NS direction).

rotation angle. After reaching the yield rotation angle, the damping was assumed to become zero due to the fact that only the hysteretic damping is dominant after the displacement reaches the yield displacement. The system that was used in the

sub-structured pseudo-dynamic test consists of the specimen, loading actuator, reaction wall, data loggers, personal computer for analyzing the inelastic response of the viaduct model and for controlling the input/output data, measuring devices, another personal computer for data acquisition, digital/analog (D/A) converter, and analog/digital (A/D) converter. The test procedures were as follows:

1. The displacement of the girder at the first step was calculated analytically by the response analysis program that was based on Takeda's trilinear model.
2. By means of the digital/analog converter, the calculated displacement was converted from a digital value into an analog value that can be applied to the specimen through the actuator.
3. Immediately after the actuator applies the required displacement to the specimen, the restoring force was directly measured from the loading system. The computer records this restoring force after converting the data from analog to digital through the A/D converter.
4. The previous restoring force was used for the calculation of the displacement in the next step.
5. The previous steps (steps 1–4) were repeated for the entire duration of the input excitation.

### 3. Test results

#### 3.1 Statically reversed cyclic loading test

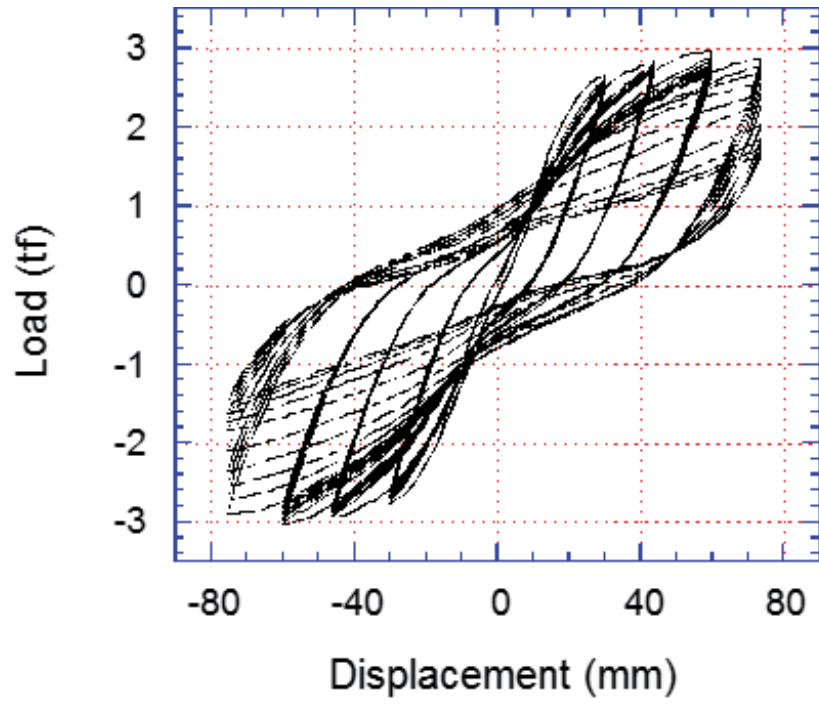
The input cyclic wave, shown in **Figure 3**, was employed during the statically reversed cyclic loading testing of specimen B-1. **Figure 5a** shows the load-displacement curve for specimen B-1. The test was continued, after reaching the ultimate load, till a decrease of the load to 80% of the ultimate load was noticed. The 80% is a common acceptance criterion stipulated in the New Zealand standards [18] and has been adopted by many prominent researchers [1].

The maximum displacement, in the two directions of loading, was about five times the yielding displacement of the prestressing tendons. The skeleton (backbone) curve for the specimen was experimentally obtained and shown in **Figure 5b**. The anticipated bond slip of the reinforcement and prestressing tendons was considered while predicting the analytical skeleton curve. A good agreement between the analytical and the experimental skeleton curves was found (**Figure 5b**).

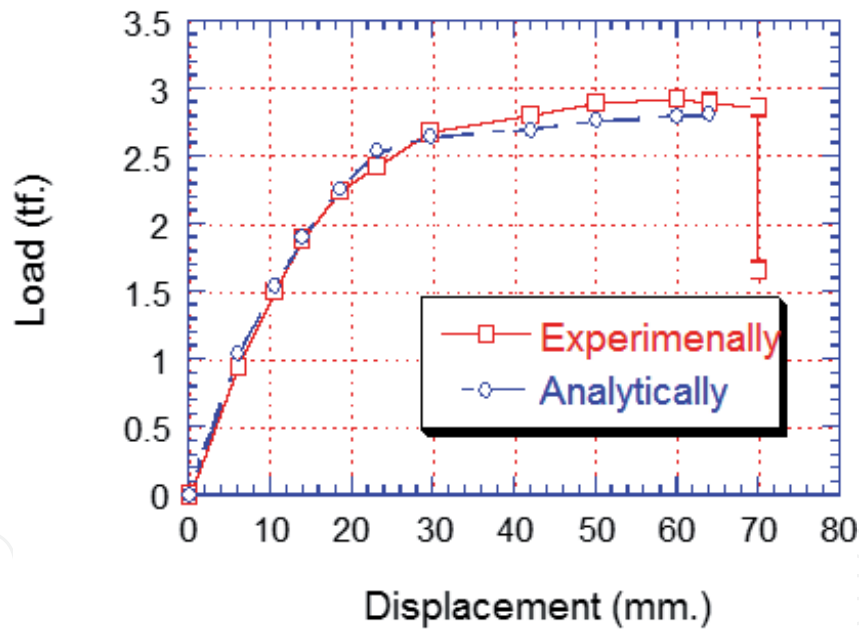
The flexural cracks were opened and closed, while almost no shear cracks were observed during the test. The hysteretic loops shown in **Figure 5a** show stiffness degradation and a change in stiffness during reloading which is known as pinching [19]. The pinching can be attributed to opening and closing of the cracks during the cyclic loading. Shear, which is generally responsible for the pinching of the load-deformation curve, was not the cause of the pinching.

Prestressed concrete members usually show marked elastic recovery even after considerable inelastic deformations, and thus leading to the occurrence of the pinching of the hysteretic loops. Energy dissipation capacities of the prestressed concrete members were less than those of reinforced concrete members because of the elastic recovery after considerable inelastic deformations.





(a)

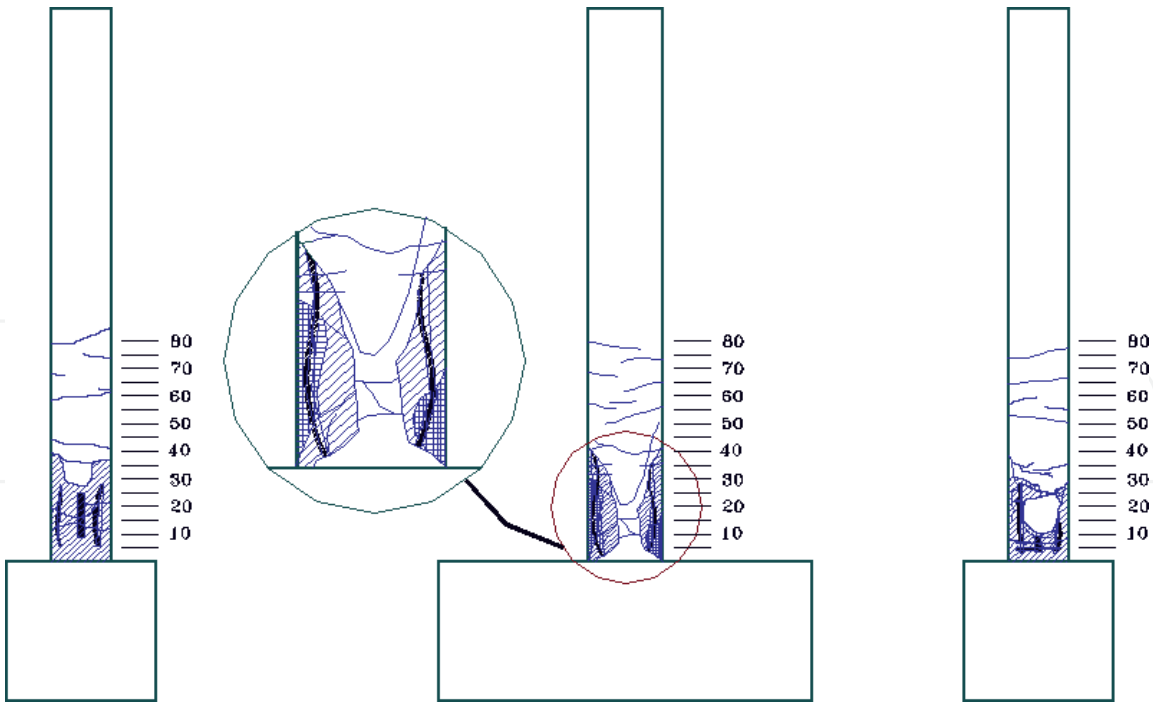


(b)

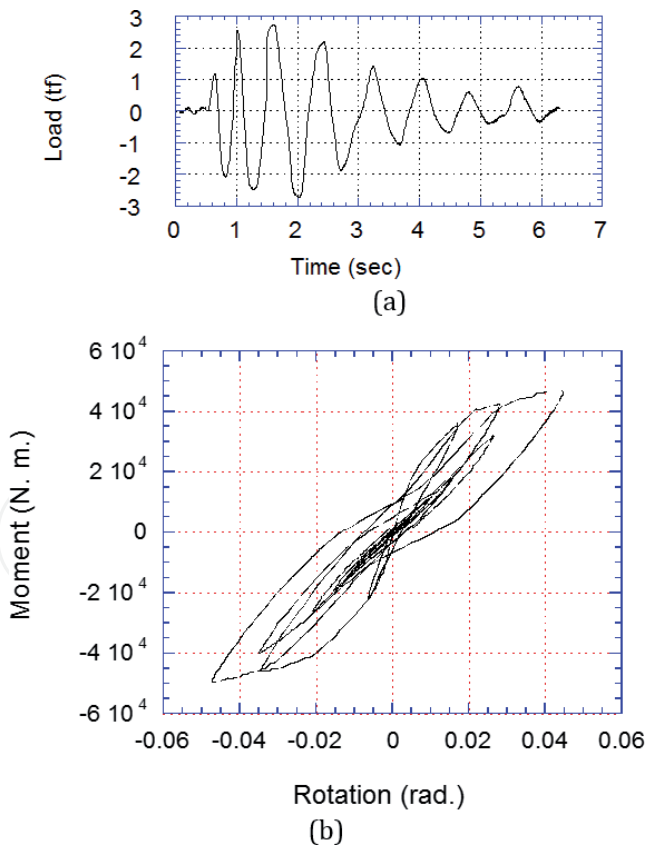
**Figure 5.** Hysteretic load-displacement and backbone curves for specimen B-1 during the statically reversed cyclic loading test: (a) Hysteretic load-displacement curve; (b) experimental and analytical backbone curves.

Specimen B-1 was a partially prestressed concrete specimen, and therefore the pinching was not significant. Consequently, a higher energy dissipation capacity than that of a fully prestressed concrete member was attained. The hysteretic load-displacement curve (**Figure 5a**) shows a stable behavior with a comparatively minor strength enhancement.

At early stages of loading and until a displacement of three times the yield displacement of the PC tendons, the residual tensile forces in the PC tendons were adequate to close previously opened cracks. At a displacement equal to four times the yielding displacement of the PC tendon, the concrete compression strains in the plastic hinge

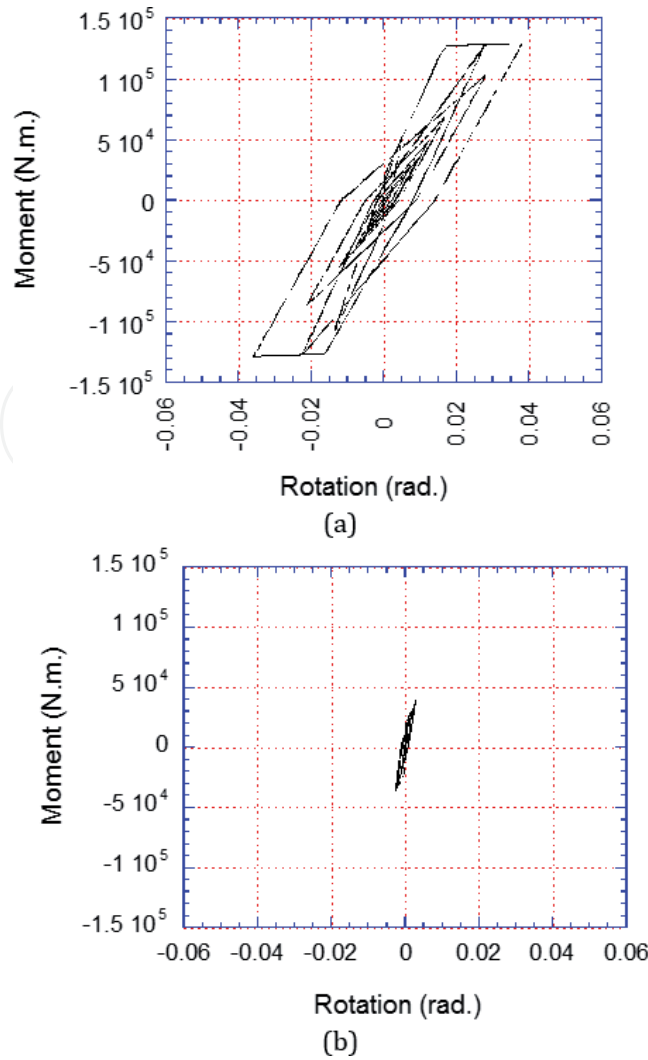


**Figure 6.**  
*Cracking pattern of specimen B-1 at the end of the statically reversed cyclic loading test (heights are given in centimeters).*



**Figure 7.**  
*Load-time history of the actuator and experimental hysteretic moment-rotation curve of the left end of the PC girder: (a) Load-time history of the actuator; (b) experimental hysteretic moment-rotation curve of the left end of the PC girder.*

region exceeded the unconfined compression strain capacity, and concrete cover spalling was noticeable. Because of the existence of relatively close-spaced transverse hoops, crushing was delayed inside the concrete core as they act to restrain the lateral



**Figure 8.**

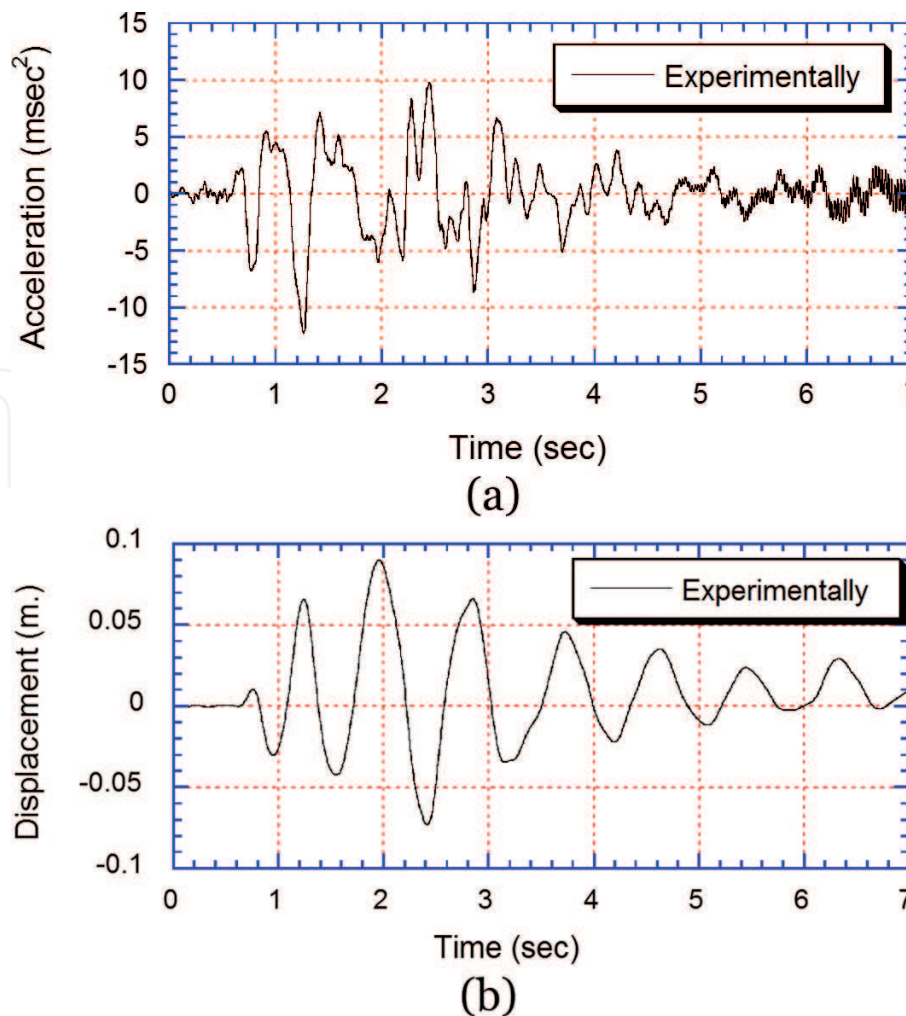
*Experimental hysteretic moment-rotation curves of the bottom and top ends of the RC column during the sub-structured pseudo-dynamic testing: (a) Experimental hysteretic moment-rotation curve of the bottom end of the RC column; (b) experimental hysteretic moment-rotation curve of the top end of the RC column.*

compression of the concrete that accompanies the onset of crushing, thus maintaining the integrity of the concrete core. It was not until a displacement of five times the yield displacement when the crushing began to penetrate inside the core concrete due to the large number of repetitions of the cycles. Additionally, the reinforcing bars experienced large increase in the tensile strains and buckling after cover spalling in the plastic hinge region. The cracking pattern of specimen B-1 after the test is shown in **Figure 6**.

### 3.2 Sub-structured pseudo-dynamic test

The used time history of the actuator load during the test is shown in **Figure 7a**. The resulting hysteretic moment-rotation curve for the left end of the PC girder is shown in **Figure 7b**. Pinching of the hysteretic loops is clear in **Figure 7b**. A maximum rotation angle of 0.045 rad. was observed and, the figure also indicates a considerable damage of the PC girder due to the input excitation.

**Figure 8a** shows the hysteretic moment-rotation curve of the bottom end of the left column of the viaduct model. It can be noticed from the curve that a considerable damage occurred during the input excitation. A maximum rotation of 0.036 rad. was observed. **Figure 8b** shows the hysteretic moment-curvature curve of the top end of the left column of the viaduct model. It can be observed from the curve that limited energy was dissipated in the plastic hinge that was expected



**Figure 9.**  
*Experimental acceleration time history and displacement time history during the sub-structured pseudo-dynamic testing: (a) Acceleration time history; (b) displacement time history.*

to form at the top of the left column of the viaduct model. Similar results were obtained for the bottom and top ends of the right column of the viaduct model. A comparison between the hysteretic moment-rotation curves in **Figures 7b** and **8a** shows that not only the reinforced concrete column but also the PC girder may undergo extensive damage during an earthquake excitation. As a consequence, adequate care should be given to the PC girder design to satisfy the requirements of a seismic-resistant structure.

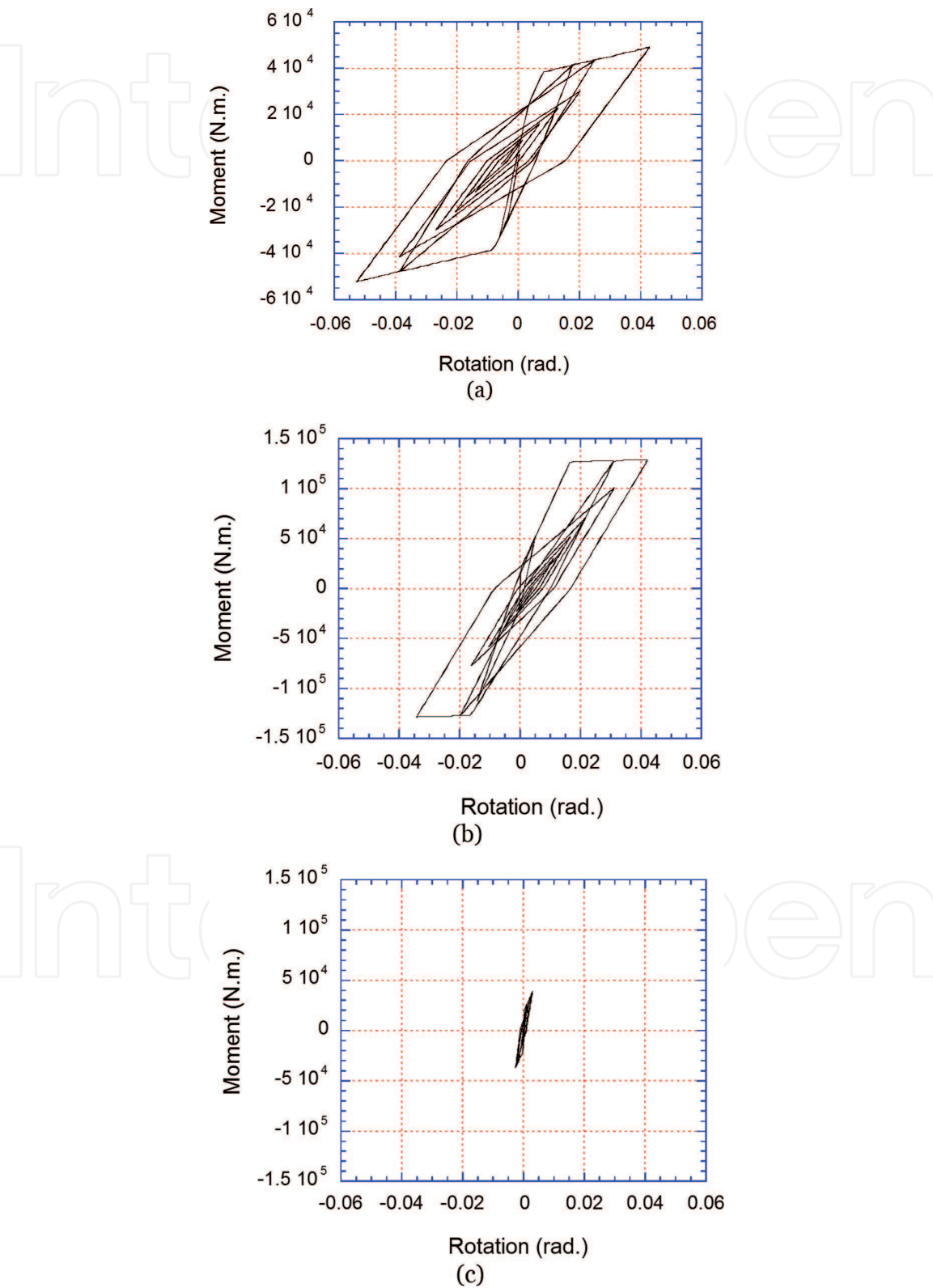
The time history of the response acceleration (**Figure 9a**) shows that the maximum observed acceleration was  $12.2 \text{ m/sec}^2$  that occurred at a time equal to 1.25 second. The time and direction of the maximum acceleration were consistent with the time and direction of the maximum input ground acceleration (**Figure 4c**). The time history of the response displacement (**Figure 9b**) shows that the maximum displacement was 8.5 cm, which occurred at a time equal to 1.95 second.

#### 4. Response analysis results

The results that were obtained from the reversed cyclic loading tests and the sub-structured pseudo-dynamic tests for the tested viaduct models show that not only the RC piers but also the PC girders may be damaged during earthquake excitations. This conclusion cannot be generalized without investigating to what extent changes in the viaduct model can influence the resulting response behavior and ductility

factor. A parametric study that includes parameters such as the yielding ratio ( $P_y/mg$ ), the elastic natural period, and the strength ratio between the PC girder and the RC columns is required to verify the conclusion.

The accuracy of any parametric study is dependent on the accuracy of the available analytical hysteretic restoring force models for prestressed and reinforced



**Figure 10.** Analytical hysteretic moment-rotation curve of the left end of the PC girder and the bottom and top ends of the RC column: (a) Analytical moment-rotation curve of the left end of the PC girder; (b) analytical moment-rotation curve of the bottom end of the RC column; (c) analytical moment-rotation curve of the top end of the RC column.



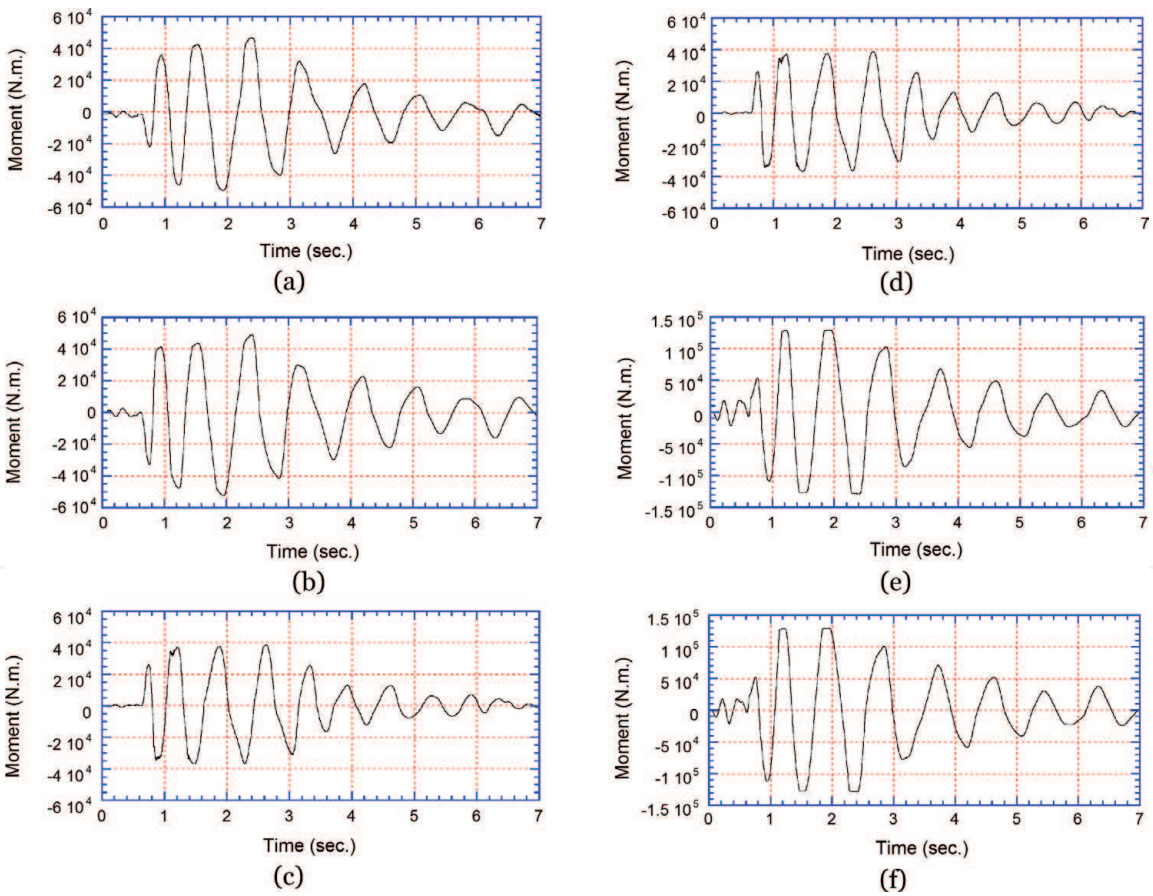
concrete members of the viaduct model. Therefore, response analyses were carried out for the same viaduct model that was tested using the sub-structured pseudo-dynamic test in the previous section. The response analysis results were compared with the experimental results of the sub-structured pseudo-dynamic test.

The one component model proposed by Giberson [11] was employed during the response analyses. Takeda's trilinear restoring force model was used for the RC columns, and the modified Takeda's model was used for the PC girders. The modified Takeda's model [7] accounts for the partial prestressing that was applied to the girders. Zatar et al. [20, 21] presented and verified the accuracy of another restoring force model for prestressed and partially prestressed members. The model by Zatar et al. incorporated modifications for Takeda's restoring force model.

**Figure 10a** shows the hysteretic moment-rotation curve analytically obtained for the left end of the PC girder. The maximum moment was  $-5.15 \times 10^{-4}$  Nm, and the corresponding rotation was  $-0.043$  rad.

**Figure 10b** and **c** shows the hysteretic moment-rotation curves for the bottom and the top ends of the RC column, respectively. Little energy was dissipated at the top end of the column. Conversely, considerable damage was observed at the plastic hinge that existed at the bottom end of the RC column. The maximum moment in the bottom end of the column was  $1.3 \times 10^5$  Nm, and the corresponding rotation was  $-0.042$  rad.

A comparison was made between the experimental and analytical hysteretic moment-rotation curves for the left end of the PC girder and for the bottom and



**Figure 11.**  
*Experimental versus analytical moment time history for the left end of the PC girder and the top and bottom ends of the RC column: (a) Experimental moment time history for the left end of the PC girder; (b) analytical moment time history for the left end of the PC girder; (c) experimental moment time history for top end of the RC column; (d) analytical moment time history for top end of the RC column; (e) experimental moment time history for the bottom end of the RC column; (f) analytical moment time history for the bottom end of the RC column.*

the top ends of the RC column, respectively (**Figures 7, 8, and 10**). The comparison included the observed damage, the hysteretic behavior, the maximum moment, and the associated rotation. An overall good agreement was found between the sub-structured pseudo-dynamic test and the response analysis results. The unloading stiffness of the hysteretic moment-rotation curve of the PC girder that was obtained from the response analyses was different from the unloading stiffness that was found during the sub-structured pseudo-dynamic test. However, the total dissipated energy that was obtained from the response analyses was found to be almost similar to the experimentally dissipated energy during the excitation.

The moment time history curves that were obtained from the sub-structured pseudo-dynamic test for the left end of the PC girder and the bottom and the top ends of the RC column are shown in **Figure 11a, c and e**, respectively. The corresponding moment time history curves that were obtained from the response analyses are shown in **Figure 11b, d and f**, respectively. The comparison between the experimental and analytical moment time histories shows good agreement, thus verifying the accuracy of the used analytical hysteretic restoring force models for both the prestressed and the reinforced concrete members of the viaduct model. Consequently, the restoring force models can be further employed in a parametric study that includes the yielding ratio ( $P_y/mg$ ), the elastic natural period, and the strength ratio between the PC girder and the RC columns. A parametric study that included these parameters is carried out in order to verify the study conclusions as well as to fully understand the response behavior of the viaduct structures during severe earthquake excitations. Because of space limitations, the results of the parametric study are not included in this paper. However, all the results can be found elsewhere [22].

## 5. Conclusions

The objective of this study was to clarify the inelastic response behavior of partially prestressed concrete girders of viaduct structures during severe earthquake excitations. A study that includes experimental and analytical phases was carried out. Small-scaled models were employed so as to represent actual viaduct structures. Specimens representing the PC girders of the viaducts were tested experimentally. Two testing techniques were employed in the experimental phase of the study. The first technique was a statically reversed cyclic loading test. The objective of the statically reversed cyclic loading test was to study the inelastic response behavior of the PC girders and to obtain the hysteretic-load deformational characteristics. The sub-structured pseudo-dynamic testing technique was implemented as the second testing technique. During the sub-structured pseudo-dynamic test, the PC girder was tested experimentally, and the RC columns of the viaduct structure were simulated analytically. Response analyses for the same viaduct model in terms of hysteretic moment-rotation curves and time histories were carried out. From the test results, it can be concluded that:

1. Not only the RC columns but also the PC girders are subjected to inelastic deformations that may cause a considerable damage during earthquake excitations. As a consequence, adequate care should be given to the PC girder design to satisfy the strength and ductility requirements of a seismic-resistant structure.
2. A comparison between the experimental and analytical results in terms of the resulting skeleton curves, time histories, hysteretic curves, and the dissipated energy was made. A good agreement between the experimental and analytical results was found. Therefore, the analytical model can be utilized

in further parametric studies that aim to fully clarify the response behavior of prestressed concrete viaduct structures.

3. A parametric study that is based on the calibrated hysteretic analytical restoring force model is conducted and shall result in having design guidelines for the partially prestressed concrete girders under earthquake excitations.

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