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DAMAGE POTENTIAL OF RC BRIDGE PIERS DUE TO MULTIPLE EARTHQUAKES

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SUMMARY

Analytical study is discussed for the cumulative damage evaluation of reinforced concrete bridge piers under single and multiple earthquakes in the present paper. Analytical results show that the ductility demand under multiple earthquakes is in general higher than that under single earthquake. In addition, hysteretic response under multiple earthquakes demonstrates that once the first cycle of maximum displacement occurs, reloading stiffness is reduced in conjunction with a number cycles and thus more damage is expected. It is however noteworthy that the difference in the response between single and multiple earthquakes varies due to the applied back-to-back input motions. This suggests that the maximum displacement ductility demand imposed on the bridge piers depends on the applied ground motion characteristics and combinations. Also investigated is the effect of multiple earthquakes on the response with shear. Comparisons of the hysteretic response without and with shear demonstrate that both strength and stiffness degradation are pronounced in the response with shear.

INTRODUCTION

Turkey experienced two successive major earthquakes with three months interval on the North Anatolian Fault system in 1999. The earthquake of Kocaeli on 17 August 1999 caused severe damage on many of reinforced concrete structures. Following the event (approximately three months later), on the evening of 12 November 1999, another earthquake stroke the area of Duzce and Bolu in the northwestern part of Turkey, which is a nearby region of Kocaeli (Izmit) and caused heavy damage to major highway viaducts [1]. This implies that the survived lifeline structures, such as bridges under the former event may have damage or collapse potential under the latter event due to their possible stiffness degradation as well as the reduction in load carrying capacity. It is therefore important to examine the issue of cumulative damage effect due to successive back-to-back earthquakes.

Whereas most of damage analyses for such structures have been conducted in terms of single ground motion of all components, very few studies have been reported in the literature with regard to the seismic response analyses of such structures subjected to multiple earthquake motions of all components. Study by Aschheim and black [2] was among those very few studies that took into account the effect of prior earthquake damage. However, this study was somewhat limited since the prior damage was modeled as a

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reduction in initial stiffness under the assumption that residual displacements are negligible. The study was also restricted to cases in which prior demands are less than those that would result if the structure were initially undamaged.

The aim of this paper is to investigate the effect of prior earthquake damage on the maximum displacement response and ductility demand. For this purpose, an analytical study is discussed for the inelastic seismic response of reinforced concrete bridge piers subjected to successive back-to-back earthquakes. In the analysis, brief aspects of shear deformation are also taken into account.

PROGRAM ZEUSNL

ZeusNL has been developed for the nonlinear analysis of 2- and 3-dimensional steel, reinforced concrete and composite structures. The program is based on the fiber element section analysis taking into account the effects of both geometric and material nonlinearities and has a full graphical user interface facility. ZeusNL provides a series of solution strategies for nonlinear analysis and is described hereafter.

In static analysis, both force and displacement loading can be applied with independent values or constrained to vary in proportional ratios. In addition, displacement and acceleration time-histories can be applied at the supports. The solution procedure can be either full or modified Newton-Raphson method. Automatic load-step reduction is employed to provide an optimum efficiency and convergence criteria can be defined in terms of either displacement or force. For dynamic analysis, the Lanczos algorithm is used for eigenvalue analysis to obtain the required natural frequencies and mode shapes. Time-history analysis is performed through numerically integrating the equation of motion. Equilibrium is ensured for each time step using the same iterative strategies as employed in static analysis.

Also availed of are a variety of cross-section types over which a number of monitoring areas are divided in order to account accurately for the inelastic response of structural members. A detailed description of all available elements, cross-section types and material models can be found in the reference [3].

SHEAR MODELING

The modified compression field theory (referred to as MCFT hereafter) developed by Vecchio and Collins [4] was adopted and modified for the derivation of shear primary curve of a reinforced concrete member under monotonic loading. The modifications were related to the constitutive relationship of concrete in compression. The stress-strain relationship for concrete in compression utilized in the MCFT did not seem to provide an effective confinement to the core concrete surrounded by transverse reinforcement and thus the model of Mander et al. [5] was invoked in order to take into account the confinement effect. To describe the loading, unloading and reloading response of hysteretic shear behavior, a set of hysteresis rules proposed by Ozcebe and Saatcioglu [6] was elaborated and developed further for axial force variation. Then, the hysteretic shear-axial interaction model was developed and implemented in a finite element program ZeusNL [3]. This subsequently allows flexure-shear-axial interaction response to be simulated in terms of a lumped hinge representation. A detailed computational procedure regarding the determination of primary curve and hysteretic shear-axial interaction model is described in the reference [7].

Verification of the model has been carried out for reinforced concrete columns subjected to different levels of constant axial force. This is mainly due to the fact that very few reinforced concrete column tests under varying axial force are available. Fourteen full-scale tests were conducted by Saatcioglu and Ozcebe [8] to assess the response of reinforced concrete columns under slowly applied lateral load reversals. The cross-sections of the columns were square of 350 mm and the column height was 900 mm.

As a representative case, U6 among the specimens is selected for the verification of the present development. Material properties of the specimen U6 are given in table 1.

Table 1 Material properties of U6 specimen

Specimen	Concrete Strength (MPa)	Axial force (kN)	Longitudinal reinforcement	Transverse reinforcement	
			Yield strength (MPa)	Yield strength (MPa)	Percent
U6	37.3	600	437	425	1.95

Figure 1 shows the comparison of hysteretic response between experiment and analysis with shear. In general, relatively good correlation is observed in the overall inelastic response. Shown in figure 2 is the comparison of hysteretic shear force-shear displacement response between experiment and analysis with shear. Although predicted shear displacement is slightly smaller than that of experiment in the positive loading direction, relatively good agreement is achieved in terms of both strength and stiffness. In particular, effect of pinching due to shear is well presented in the predicted response. It is thus encouraging that the current development can identify correctly the contribution of deformation mechanisms.

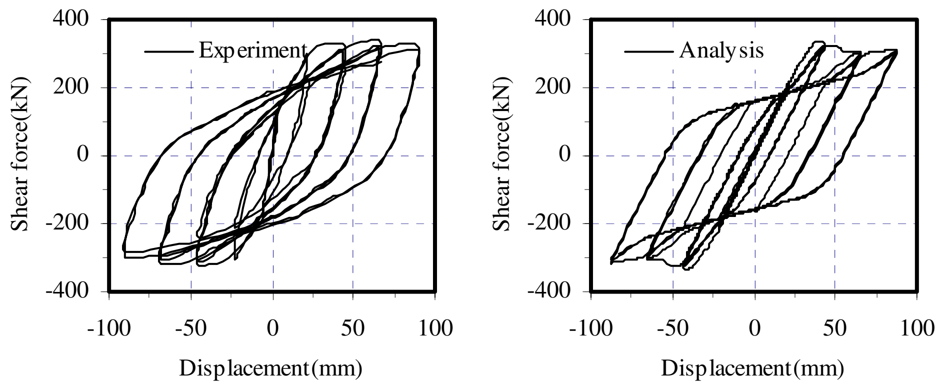


Figure 1 Comparison of hysteretic response for U6 specimen

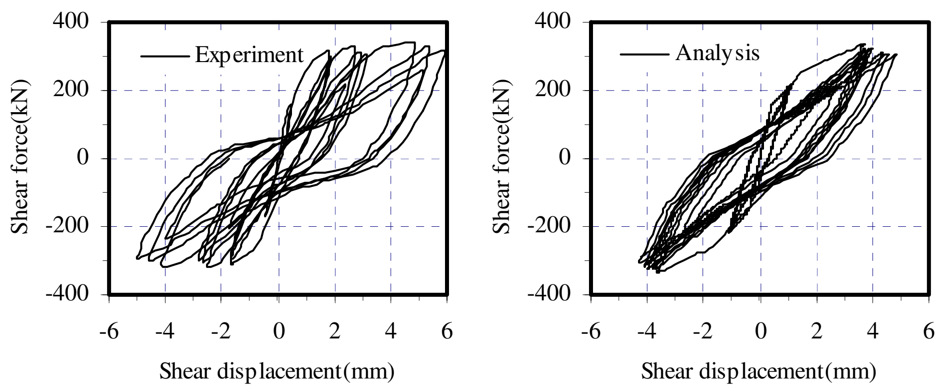


Figure 2 Comparison of hysteretic shear response for U6 specimen

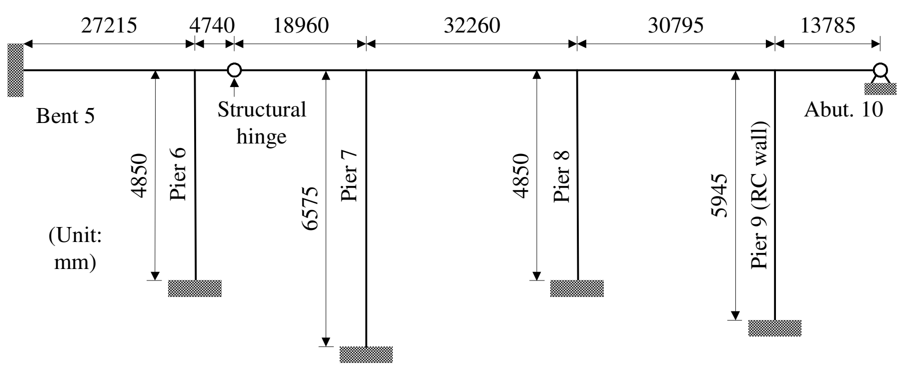
INELASTIC RESPONSE OF A RC BRIDGE UNDER MULTIPLE EARTHQUAKES

In order to investigate the effect of multiple earthquakes on the inelastic response of bridge piers, a reinforced concrete bridge structure, severely damaged by the Northridge earthquake of 17 January 1994 is selected. The bridge under consideration is a ramp structure (Collector Distributor 36) which continues on a line close to that of the main freeway, La Cienega-Venice Boulevard sector of the I-10 [7]. It is appreciated that the selected bridge structure is not as conclusive as a damaged bridge structure under the two successive Turkey earthquakes. However, since very few literatures are reported regarding the damaged reinforced concrete bridge structure under the two successive earthquakes, the present study can be considered as a benchmark for comparative analyses.

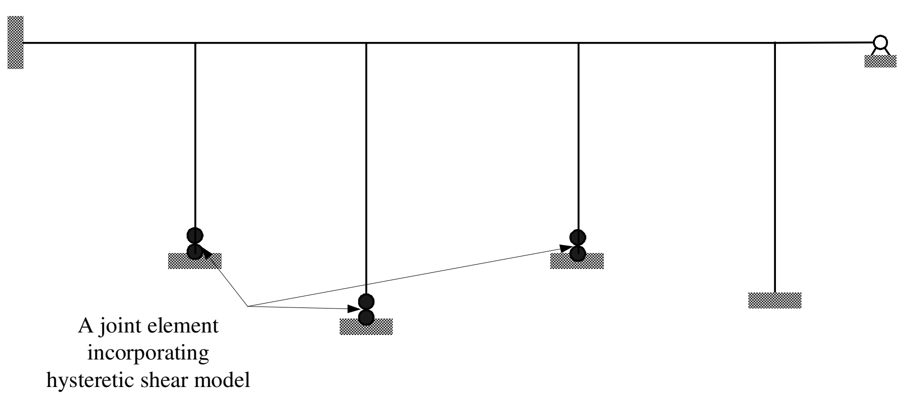
Analytical models of the RC bridge

The deck of the ramp structure consists of a three-celled box girder and is carried over the multi-column bent 5, then over three single column bents 6, 7 and 8, and over the pier wall of bent 9. The columns of all bents consist of 1219 mm diameter reinforced concrete circular sections. Column longitudinal reinforcement is identical for piers 6, 7 and 8, while less longitudinal reinforcement is employed in the columns of bent 5. A detailed description regarding the bridge structure is described in the reference [7].

The general layout of the ramp structure is shown in figure 3(a). In this model, five cubic inelastic elements are employed in the piers with shorter elements at the base and top of the piers and longer elements toward the center. This arrangement allows plastic hinges to be captured accurately. The bridge deck is assumed to be fully restrained at its intersection with bent 5. This reflects the relative dimensions of the ramp on either side of this point, which suggests that a relatively insignificant amount of transverse deformation would occur to the west of bent 5.



(a) Structural model



(b) Structural model with shear

Figure 3 Analytical models of the ramp structure

To account for the effect of shear, each pier of the ramp structure is modeled by a combination of cubic inelastic elements with a single joint element incorporating the developed hysteretic shear-axial interaction model at the bottom of the piers. Hence flexure-shear-axial interaction behavior is simulated. The remaining parameters are kept as for the case of the structural model. The graphical representation of this model is illustrated in figure 3(b).

Input ground motions

Three sets of accelerograms are selected for comparative time-history analyses. The first set of accelerograms is Kocaeli (K) and Duzce (D) earthquakes, recorded at the same station Duzce under the two successive earthquakes, Kocaeli of August 1999 and Duzce of November 1999 respectively. The second set is Imperial Valley (I) and Morgan Hill (M) earthquakes, recorded at Bonds Corner under Imperial Valley earthquake of October 1979 and at Coyote Lake Dam under Morgan Hill earthquake of April 1984 respectively. The last set is Loma Prieta (L) and Northridge (N) earthquakes, recorded at Hollister City Hall under Loma Prieta earthquake of October 1989 and at Arleta-Nordhoff Fire Station under Northridge earthquake of January 1994. Whereas the first set represents the effect of damage accumulation on the structure due to their successive nature in the same area, other sets can be considered as a benchmark for exploratory analyses and thus the characteristic period contents of the earthquake ground motions have not been taken into account. The peak ground accelerations of all components for the three sets are given in table 2.

Table 2 Peak ground accelerations for the selected records

Set	Station (Earthquake)	Peak ground acceleration (g)		
		Longitudinal	Transverse	Vertical
Set 1	Duzce (K) (Kocaeli earthquake)	0.361	0.310	0.205
	Duzce (D) (Duzce-Bolu earthquake)	0.514	0.377	0.345
Set 2	Bonds Corner (I) (Imperial Valley earthquake)	0.588	0.775	0.425
	Coyote Lake Dam (M) (Morgan Hill earthquake)	0.711	1.298	0.388
Set 3	Hollister City Hall (L) (Loma Prieta earthquake)	0.215	0.247	0.216
	Arleta-Nordhoff Fire Station (N) (Northridge earthquake)	0.308	0.344	0.552

Static response

Static analysis of each pier was carried out using the MCFT for preliminary evaluation of the pier capacities. Gravity loads assessed from the cross-sectional areas of the box girder and the piers were applied as an axial force at the top of each pier. Figure 4 shows shear force-displacement response of each pier in the transverse direction. Comparison of static response of the piers shows that the response of piers 6 and 8 are nearly identical but slightly higher shear force capacity for pier 8. The response of pier 7, lying closest to the center of the bridge exhibits a lower stiffness than that of piers 6 and 8, and experiences the greatest displacement. This can be attributed to different heights between the piers, being the tallest for pier 7. Static response parameters are summarized in table 3. As observed, pier 8 carries a greater axial force due to adjacent long spans. Also indicated is the yield displacement, being identical (32 mm) for piers 6 and 8 and much greater (62 mm) for pier 7.

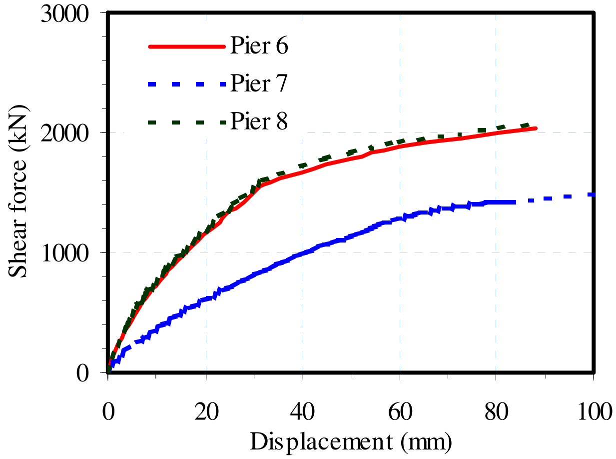


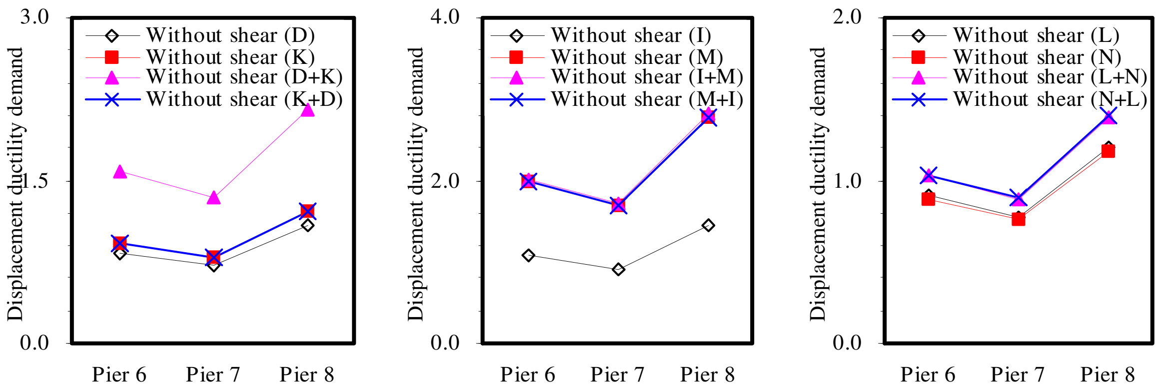
Figure 4 Static shear force-displacement response

Table 3 Static response parameters

Pier	Axial force (kN)	Shear force (kN)	Yield displacement (mm)
6	2395	2030	32
7	2830	1645	62
8	3180	2065	32

Time-history response without shear

Inelastic time-history analyses were carried out for the model shown in figure 3(a). The model was subjected to single and multiple earthquakes of all components. Figure 5 shows comparisons of displacement ductility demand of the piers in the transverse direction subjected to each set of earthquake record.



(a) Set 1 (b) Set 2 (c) Set 3

Figure 5 Comparison of transverse displacement ductility demand for each set of record

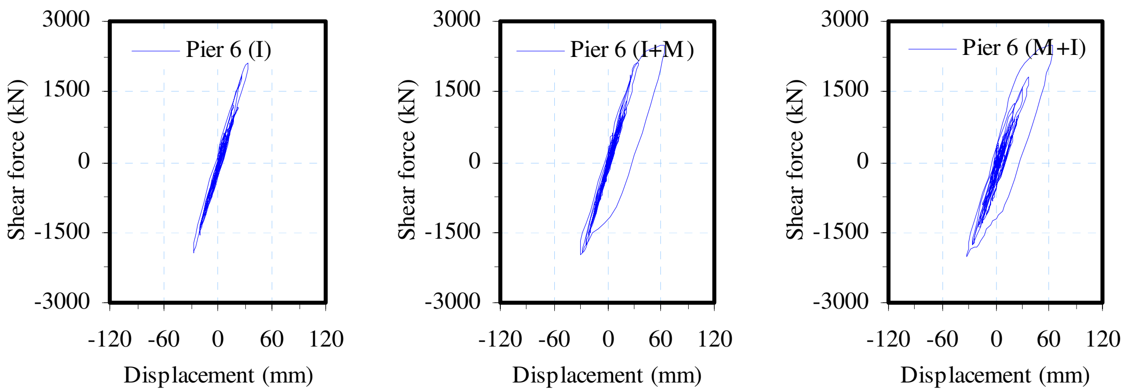
Figure 5(a) shows that while the ductility demand under K+D is not significantly increased in comparison with that under the two single earthquake, the ductility demand under D+K is considerably increased (up to 80% higher). This suggests that the maximum displacement ductility demand imposed on the piers depends on the applied ground motion characteristics and combinations, and that even the small intensity of successive earthquake may tend to cause increased displacement ductility demand due to a prior earthquake damage. Shown in figure 5(b) is the comparison under the set 2 record. In general, the response under multiple earthquakes shows higher displacement ductility demand than that under single earthquake (being approximately 90% increase). Whereas the same trend of displacement ductility demand increase is observed in figure 5(c), the discrepancy of response between single and multiple earthquakes is rather insignificant. In all, the response under multiple earthquakes demonstrates that the piers experience increased displacement and thus more damage is expected to be accumulated in the piers. The transverse displacement ductility demand under each set of record is summarized in table 4.

Table 4 Transverse displacement ductility demand

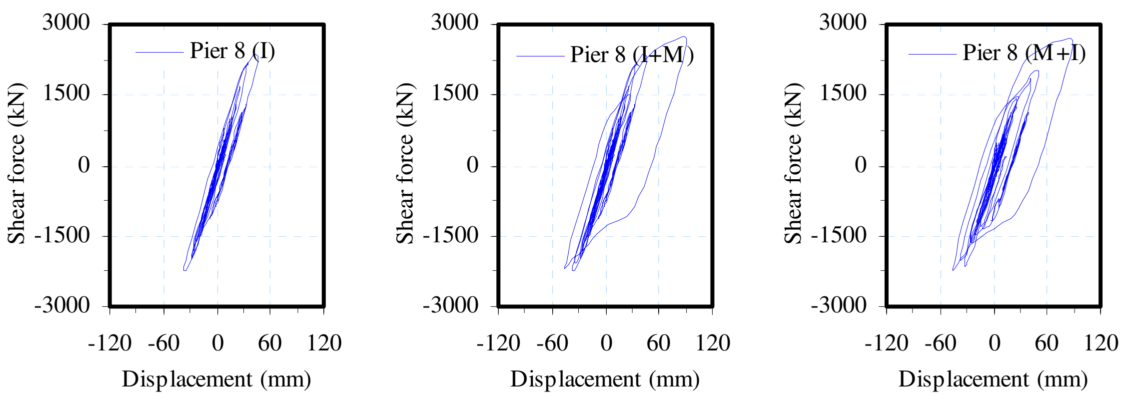
Pier	Input motion set 1				Input motion set 2				Input motion set 3			
	D	K	D+K	K+D	I	M	I+M	M+I	L	N	L+N	N+L
6	0.83	0.92	1.58	0.92	1.07	1.98	2.01	1.98	0.91	0.88	1.03	1.03
7	0.71	0.78	1.35	0.78	0.91	1.70	1.73	1.70	0.77	0.76	0.88	0.89
8	1.10	1.22	2.16	1.22	1.46	2.76	2.81	2.76	1.20	1.17	1.40	1.40

As observed in the table 4, the displacement ductility demand is considerably affected by the applied ground motion combinations. This implies that period may be elongated due to the stiffness degradation under a prior earthquake and thus resonance may occur under the successive earthquake, leading to an increased displacement ductility demand. This is assessed further by the hysteretic response of the piers.

In order to investigate the effect of multiple earthquakes on the stiffness degradation of piers, comparison of hysteretic response is made. Figure 6 shows comparisons of transverse hysteretic response of piers for the cases between single and multiple earthquakes subjected to set 2 record.



(a) Pier 6



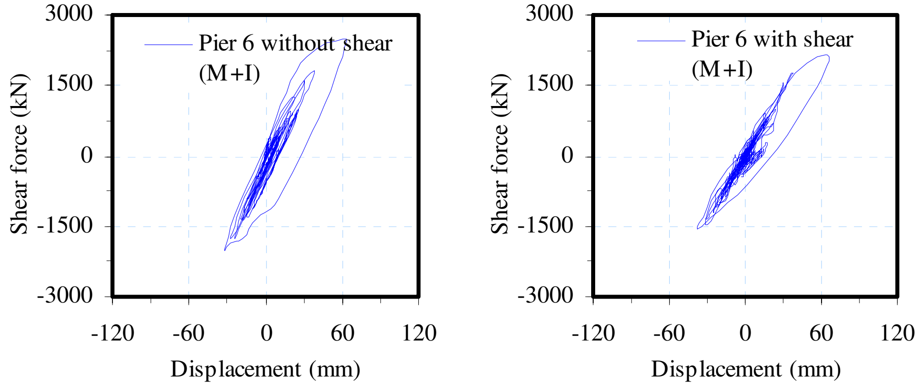
(b) Pier 8

Figure 6 Transverse hysteretic response of piers 6 and 8 under set 2 record

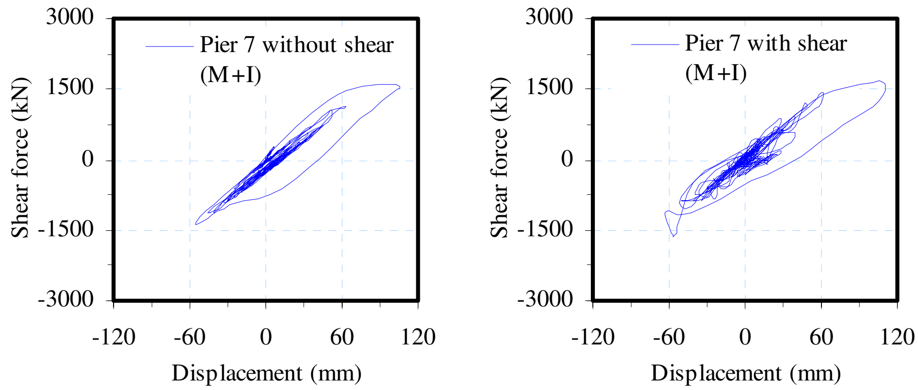
In general, the response under multiple earthquakes experiences a large number of cycles as well as low cycles. In addition, reloading stiffness is reduced in the response under multiple earthquakes once a large cycle of loading is experienced, particularly for the response under M+I. This suggests that more damage is expected to occur in the piers subjected to multiple earthquakes due to their stiffness degradation in conjunction with a number of cycles. Hence the best measure of damage evaluation for the bridge piers should include both the maximum displacement and the stiffness.

Time-history response with shear

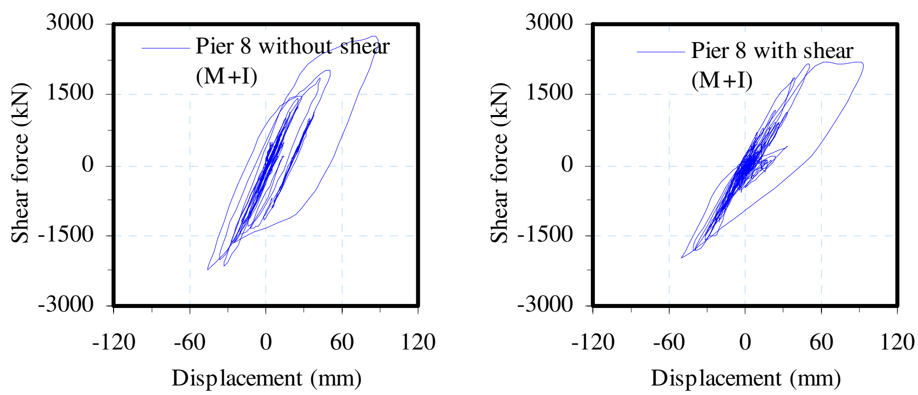
Further investigation is carried out for the effect of multiple earthquake on the response with shear. For this purpose, use is made of the MCFT to define the input parameters of the new shear representation. Each pier was analyzed for several levels of constant axial force. These levels were 10%, 20% and 30% of compressive axial force capacity, zero axial force, and 10% and 30% of the tensile axial force capacity. In general, the exceedence of 15% of compressive axial force capacity is not common for reinforced concrete bridge piers [9]. Therefore the selection of the above axial force range can be considered as an upper-bound.



(a) Pier 6



(b) Pier 7



(c) Pier 8

Figure 7 Comparison of transverse hysteretic response without and with shear under set 2 record

Shown in figure 7 is the comparison of transverse hysteretic response for the cases without and with shear subjected to set 2 of multiple earthquakes, M+I. As observed, the response with shear experiences a greater displacement and shows pronounced strength and stiffness degradation. It is also important to note that whereas the response without shear shows relatively stable hysteresis loops, that with shear exhibits a considerable fluctuation. This indicates that the amount of energy absorbed and dissipated by piers can be affected by the presence of shear. Therefore, the damage evaluation of the piers without shear may prejudice the stability assessment. The maximum transverse response parameters regarding displacement components are summarized in table 5.

Table 5 Maximum transverse displacement components (unit: mm)

Input motion		Pier 6			Pier 7			Pier 8		
		Total	Flexure	Shear	Total	Flexure	Shear	Total	Flexure	Shear
Set 1	D	33.5	20.9	12.6	53.5	41.7	11.8	42.8	25.6	17.2
	K	36.4	20.3	16.1	57.4	40.2	17.2	44.5	28.7	15.8
	D+K	56.4	35.9	20.5	92.8	69.7	23.1	77.4	40.3	37.1
	K+D	36.5	20.0	16.5	57.6	39.4	18.2	44.6	28.6	16.0
Set 2	I	37.3	25.8	11.5	60.0	49.6	10.4	49.8	31.7	18.1
	M	69.5	37.2	32.3	112.2	68.4	43.8	93.2	39.8	53.4
	I+M	66.4	37.2	29.2	109.3	73.0	36.3	93.0	42.7	50.3
	M+I	69.5	37.2	32.3	112.2	68.4	43.8	93.2	39.8	53.4
Set 3	L	36.5	19.7	16.8	57.5	40.9	16.6	45.0	28.4	16.6
	N	31.9	22.4	9.50	51.6	42.9	8.70	42.1	28.0	14.1
	L+N	40.3	21.7	18.6	65.0	45.0	20.0	51.1	31.6	19.5
	N+L	37.6	25.7	11.9	61.2	50.6	10.6	49.8	32.4	17.4

As observed in the table 5, the breakdown of total displacement into its component shows that shear displacement reaches up to significant level. The contribution of shear displacement to total displacement reached up to 45% and 57% for piers 6 and 8, and a lower value of 39% for pier 7.

CONCLUSIONS

Analytical study has been undertaken for the cumulative damage evaluation of reinforced concrete bridge piers under single and multiple earthquakes. The analytical results show that the maximum displacement ductility demand under multiple earthquakes is greater than that under single earthquake. In addition, hysteretic response under multiple earthquakes demonstrates that once the first cycle of maximum displacement is attained, reloading stiffness is reduced with a number of cycles and thus more damage is expected to occur. It is however noteworthy that the difference in the response between single and multiple earthquakes varies by the applied ground motions, which suggests that seismic damage potential of piers may be affected by the applied ground motion characteristics and combinations. Also evaluated is the effect of multiple earthquakes on the pier response with shear. Comparative studies between the cases without and with shear indicate that both strength and stiffness degradation are pronounced in the response with shear. It is thus recommended that a prior damage effect should be taken into account for the performance and stability assessment of bridge piers experienced a prior earthquake.

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