PERFORMANCE OF DUCTILE HIGHWAY BRIDGE PIERS SUBJECTED TO BI-DIRECTIONAL EARTHQUAKE ATTACKS

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ABSTRACT

Circular reinforced concrete highway bridge piers, designed in accordance with the requirements of Caltrans, New Zealand and Japanese specifications, are experimentally investigated to assess their seismic performance. Pseudodynamic tests are performed on 30% scaled models of the prototype bridge piers. Each specimen is subjected to a sequence of three different earthquake ground motions scaled appropriately to represent: (i) the Design Basis Earthquake (DBE) with a 90 percent confidence; (ii) the Maximum Considered Event (MCE) with a 50 percent confidence; and (iii) the MCE with a 90 percent confidence. Test results show that when bridge piers are designed to the specifications of the three countries, satisfactory performance with only slight to moderate damage can be expected for DBE. For the MCE, severe damage without collapse is likely for the Caltrans and Japanese piers. However, the NZ pier may not be able to survive MCE motions with sufficient confidence to ensure the preservation of life-safety.

Introduction

Following recent major earthquakes such as the 1994 Northridge Earthquake and the 1995 Hyogoken-Nanbu earthquake which had severe impact on the serviceability of bridges in the surrounding areas, there has been a growing interest to compare seismic performance of bridge piers designed according to the codes of different countries. This is because both the loading requirements and structural detailing procedures vary considerably, even though the magnitude of hazard exposure is similar. As part of a cooperative four-country international project, Tanabe (1999) designed four bridge piers, in accordance with Caltrans, New Zealand, Japanese and European design standards. The main purpose of this international project was to identify differences in the cross-section dimensions and reinforcing details, to clarify the reasons for these differences, and to assess the seismic performance by computational means. This previous comparative research was restricted to uni-directional earthquake motions. Given that simultaneous bi-directional earthquake motions occur in reality, and computational predictions may differ from real response due to modelling simplifications, it is considered desirable to conduct an experimental investigation of bi-directional seismic response of bridge piers.

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In this study, the seismic performance of three highway bridge piers, designed and detailed according to the prevailing seismic design standards of California, New Zealand and Japan, are compared. For the experimental investigation, specimens representing 30% scaled models of these three bridge piers were constructed and tested using the "Pseudo-Dynamic Test" (referred to as PD test hereafter) method at the University of Canterbury. Bi-directional PD tests were carried out on these specimens using three earthquakes chosen based on the results of a rigorous Incremental Dynamic Analysis (IDA). Observed damage to the piers is assessed in terms of post-earthquake serviceability following the damage classification of HAZUS (Mander and Basoz, 1999). The observed seismic performance of each pier is then compared against each other.

Experimental Details

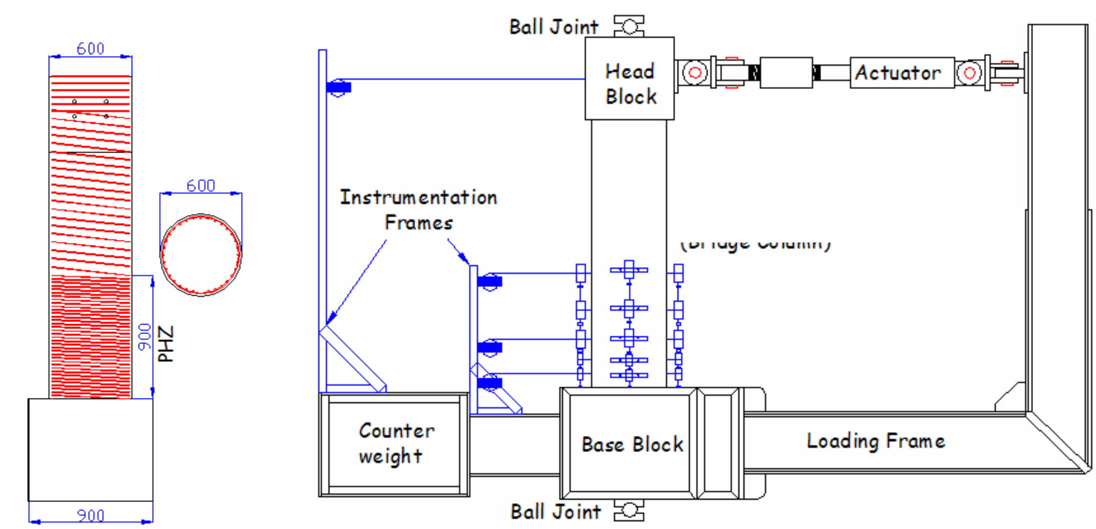
Square reinforced concrete bridge piers designed previously (Tanabe 1999) for the same level of seismic hazard using design standards of California (Caltrans 1999), New Zealand (NZS3101-1995) and Japan (JSCE 1995) were adopted as the initial basis of this study. The prototype bridge design of Tanabe (1999) consists of a common 40 m span superstructure (deck) supported on a 7 m high pier. Although the DBE with 10% probability of occurrence in 50 years (i.e. return period of 475 years) may correspond to different intensities in different countries, a common earthquake having peak ground acceleration (PGA) of 0.4g was adopted to represent the DBE in each country for designing these piers. In this study, the cross-section of the three square piers was changed to circular, without violating the design recommendations of the corresponding standards. The properties of the three prototype circular bridge piers designed and detailed using these three different design standards are tabulated in Table 1. In order to carry out the experimental investigation, reduced scale models detailed at 30 percent of full scale were prepared for all three prototype piers. A typical scaled model for the experiment is illustrated in Fig. 1(a), with the geometrical details and design parameters of the three specimens listed in Table 1.

The longitudinal and transverse reinforcement ratios provided in the scaled specimens were aimed at keeping the same proportional force capacities as in the prototypes. Each specimen was constructed in three phases: (i) the rectangular base block; (ii) the circular column; and (iii) the rectangular head block. The concrete was poured separately for each part. Cardboard tube formwork was placed and held securely over the tied reinforcing cages, then the concrete was cast. The specimens were tested well after 28 days of curing.

Fig. 1(b) presents an East-West (EW) elevation view from the north direction of a specimen set up in the test rig. As the loading applied to the specimen in the PD test was bi-directional, a similar view existed in the North-South (NS) direction as well. A constant axial force of 630 kN was applied via ball joints attached to the top and bottom plattens and the specimens. L-shaped loading frames and counterweight baskets were attached to the base block of the specimen in each direction. These were connected by 30 mm diameter high-strength threaded bars. Lateral loads were applied in both EW and NS directions via 800 kN hydraulic actuators that were connected to the specimen head block and L-shaped load frames via universal joints. In each lateral loading direction, a 1000 kN capacity load cell was installed in-series with the actuator. The lateral displacement profiles of the specimens were measured using four rotary potentiometers at different locations along the height of the specimens. In addition, curvature/rotation in the plastic hinge regions were measured with potentiometers attached to four sides of the specimens at several locations.

Table 1 Dimensions of prototypes and test specimens and material properties.

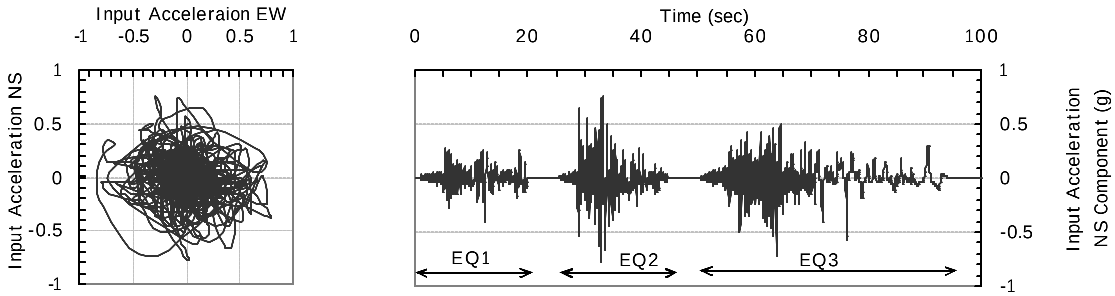
		Code	Unit	Caltrans	New Zealand	Japan
Prototype details	Diameter	D	mm	2000	1700	2000
	Plastic hinge length	PHZ	mm	3000	1700	4000
	Weight of superstructure	P	kN	7000	7000	7000
	$P/A_g f'_c$			0.11	0.15	0.11
	Longitudinal reinforcing bars			32-D41	28-D32	28-D51
	Longitudinal steel volume	ρ_t	%	1.34	0.99	1.82
	Diameter and pitch of Spiral in PHZ			R20@85	R20@170	R20@115
	Spiral steel volume	ρ_s	%	0.78	0.49	0.61
	Concrete strength	f'_c		25	25	25
	Longitudinal bars: yield strength	f_y		500	500	500
Details of test specimens	Diameter	D	mm	600	500	600
	Gravity Load	P	kN	630	630	630
	Longitudinal reinforcing bars			32-D12	24-D10	24-D16
	Longitudinal steel volume	ρ_t	%	1.28	0.96	1.71
	Length of plastic hinge zone	PHZ	mm	900	500	1200
	Spirals in PHZ			R6@25	R6@50	R6@35
	Spiral steel volume	ρ_s	%	0.83	0.51	0.60
	Concrete measured strength	f'_c	MPa	40.7	41.2	38.5
	Longitudinal steel: yield strength	f_y	MPa	528	539	517
	Spiral steel: yield strength	f_y	MPa	461	461	461

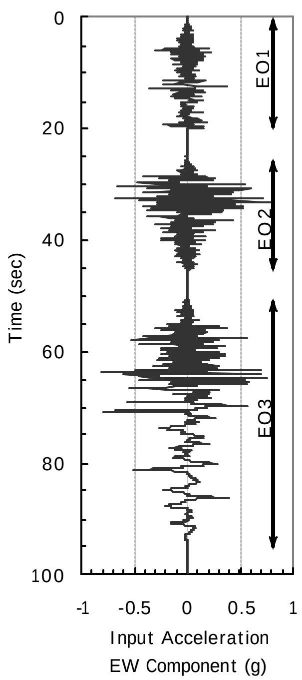


(a) (b)

Figure 1 The 30% scaled specimens for the PD tests and the test setup.

Takanashi et al. (1975) developed the PD test method for experimentally assessing the seismic performance of critical elements under real earthquakes using real earthquake ground motion as input. The PD test method consists of two parts. First, the structure is represented “virtually” as a computational model, for which the equations of motion are formulated and analysed in a normal fashion. Next, the tangential stiffness of the structure is measured physically at each time-step increment and an updated value is used in subsequent computational modeling. As the PD test is conducted at a much slower rate than in real time, the inertial effect on the physical test specimen does not exist, but needs to be accounted for computationally.





No	EQ1		EQ2		EQ3	
Class	DBE		MCE CI=50%		MCE CI=90%	
[1]PGA (g)	0.376	0.400	0.800	0.787	0.800	0.700
[2]PGA (g)	0.254	0.270	0.244	0.240	0.207	0.181
[3]Component	EW	NS	EW	NS	EW	NS
[4]ϕ (degree)	282	012	270	360	360	90
Event	Imperial Valley		Loma Prieta		Superstition Hills	
year	1979		1989		1987	
Station	Chihuahua		Anderson Dam		Wildlife Liquefaction Array	
[5]Magnitude	6.5		6.9		6.7	
[6]R (km)	28.7		21.4		24.4	

[1]Peak Ground Acceleration for the PD test, [2] Original Peak Ground Acceleration, [3]Applied component to the test, [4]original angle measured, [5]Moment Magnitudes and [6]Closest Distances to Fault Rupture, Source: PEER Strong Motion Database, http://peer.berkeley.edu/smcat/

Figure 2 Earthquake records used in the PD tests.

The PD tests conducted were bi-directional; i.e. seismic ground motions were applied in both EW and NS directions. The acceleration time-histories applied to the specimens are real ground motions in two mutually perpendicular directions recorded during recent earthquakes. Among a suite of 20 earthquake records (Vamvatsikos and Cornell, 2004), three different records were identified to represent the DBE with 90% confidence, and the MCE (2% probability of occurrence in 50 years; i.e. return period of 2475 years) with 50% and 90% confidences, respectively (Naoto 2006). Detail information on these three earthquake records, respectively designated as EQ1, EQ2 and EQ3, are given in Fig. 2. Note that the original ground acceleration history of these earthquake records have been scaled to obtain the PGA values corresponding to the seismic hazards; i.e. PGA = 0.4g for DBE and PGA = 0.8g for MCE in this study.

These three scaled ground motion records were applied in sequence in an increasing order of severity. This enabled a one-off PD test to be conducted on a single scaled model to obtain a comprehensive seismic performance assessment of the prototype pier. As shown in Fig. 2, the first 20 sec of the first two earthquake records was used for the input ground motion, since the main shock occurs in this range. The three records were connected together with 5 sec zero acceleration period between them to measure values such as residual drift.

Experimental Results

Caltrans Pier

Fig. 3 shows the PD test results of the Caltrans pier, showing (a) a plan view of the bi-directional orbit of the response drift; (b) load-displacement curves; (c) the complete time-history of the response drift; (d) photograph showing cover spalling at a drift of 3.7%; and (e) photograph at the end of the PD test showing bar buckling along with crushing of core concrete. The seismic performance of the Caltrans pier is described below in detail based on the damage events observed during each record and damage states assigned to the specimen after each record.

EQ1 with PGA = 0.4g (0-20 sec)

During the first record (EQ1 with PGA = 0.4g), flexural cracks appeared near the base of the pier with a spacing of approximately 200 mm. These cracks opened and closed alternately during the excitation, but finally closed at the end of the record. Calculating the strain profile from the externally measured curvatures, yielding of the first bar occurred at 5.6 sec when the drift was 0.24% and the lateral load was 94 kN. As commonly expected in circular piers with reinforcing bars distributed in the perimeter, the lateral load kept increasing steadily after first yielding. At the maximum response at 13.1 sec, the lateral load was 232 kN and the drift was 1.53%. At the end of the record EQ1 (i.e. 20 sec), the residual drift was 0.12% in the EW direction and close to zero in the NS direction. At this stage, the specimen had crossed the elastic response limit but did not have residual damage despite yielding of the longitudinal bars. According to the HAZUS damage classification (Mander and Basoz 1999), the damage state of the specimen after record EQ1 could be classified as DS2 (slight damage).

EQ2 with PGA = 0.8g (25-45 sec)

During the second record (EQ2 with PGA = 0.8g), more cracks emerged at the bottom of the specimen, especially in the lowermost one-diameter range (i.e. 600 mm). These cracks, which were spaced approximately at 50 mm apart from each other, closed and opened alternately during the excitation. Despite the two-fold increase in PGA, the maximum response during the record EQ2 was only slightly larger than that in the previous record EQ1. The maximum drift during EQ2 record was 1.95% at 36 sec when the lateral load reached 259 kN. At the end of the record (i.e. 45 sec), the residual drift was 0.13% in the EW direction. Concrete in the cross-section neither spalled nor crushed in this phase. Nevertheless, the flexural cracks did not close completely and a few closely spaced hairline cracks were visible at the end of the record EQ2. The physical condition of the specimen suggested that the damage state was still in the "slight damage" category (i.e. DS2).

EQ3 with PGA = 0.8g (50-95 sec)

The third and the final record (EQ3 with PGA = 0.8g) caused visibly severe damage to the specimen and the response was much larger than that due to the previous record (EQ2) despite having the same PGA. The record EQ3 induced a maximum drift of approximately 6% at about 71 sec, when the lateral load reached nearly 300 kN. During this phase, the cracks opened wider, and the cross-section deteriorated noticeably around the base of the specimen. As shown in Fig. 3 (d), the cover

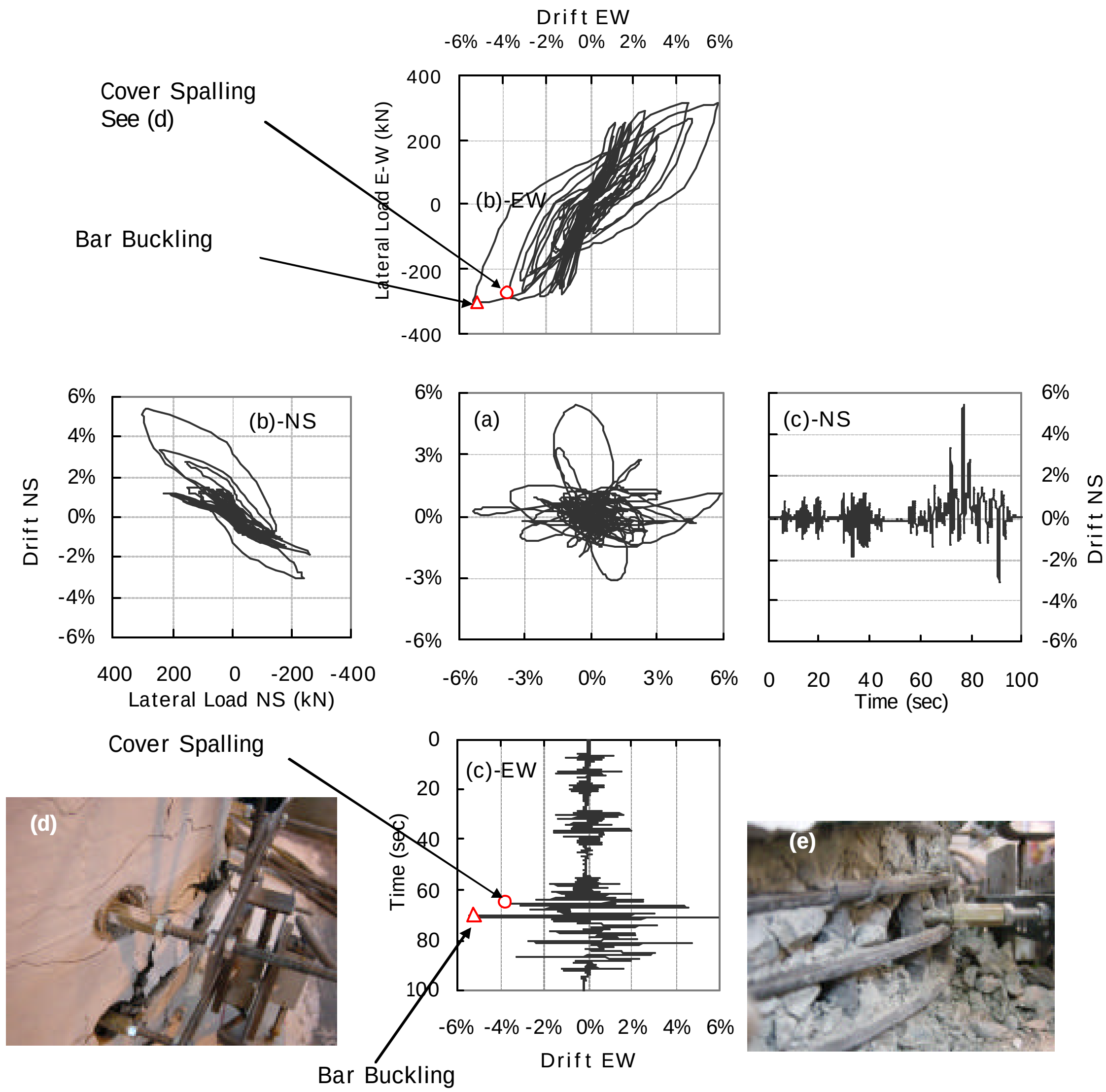


Figure 3 Experimental results of the Caltrans pier showing (a) Bi-directional drift orbit; (b) Load-displacement curve; (c) Time-history of the drift; (d) Photograph showing cover spalling at drift of 3.7%; and (e) Photograph at the end of the test.

concrete spalled off at 65 sec when the drift was about 3.7% and the longitudinal bars buckled at 70 sec when the specimen drifted 5.3% laterally. At the end of record EQ3, the residual drift was 0.2% in the EW direction. As the buckling of several longitudinal bars led to an irreparable condition, the damage state at the end of the test was identified as DS4; i.e. “severe damage” category in the HAZUS damage classification.

New Zealand Pier

When judged from the longitudinal bar strain inferred by external instrumentation, yield occurred at 5.6 sec when the drift exceeded 0.3% eastwards. The lateral load when the pier yielded was 63.3 kN. During the first earthquake record (EQ1 with PGA = 0.4g) several horizontal cracks were observed approximately 150 mm apart, but these cracks closed after the record EQ1 finished. The maximum drift and lateral load measured were 1.65% at 13.83 sec and 159 kN at 6.24 sec, respectively. The residual drift was 0.17%. The damage state after the 20 sec EQ1 record was assessed as DS2 (slight damage), since the pier exceeded the yield limit and cracks appeared, but these cracks closed after the test and no spalling was apparent.

During the second earthquake record (EQ2 with PGA = 0.8g) new horizontal cracks emerged at about 50 mm spacing over the two-diameter range (approximately 1 m) from the bottom of the pier. The cracks were found to be more intensive than those formed during the record EQ1, but the residual crack width was still relatively small (not more than 0.2 mm). The cover concrete remained in the same condition and no spalling was confirmed. The maximum drift of about 2.5% occurred at 36.9 sec, and the residual drift at the end of the record EQ2 (after 45 sec) was 0.25%. The damage state after the record EQ2 was still assessed as DS2 (slight damage), the same as before the record.

The important damage events observed during the final record (EQ3 with PGA = 0.8g) were cover spalling, longitudinal bar buckling, and severe bar fractures, which resulted in a rapid strength degradation forcing the termination of the test. Cover concrete spalling and longitudinal bar buckling were observed for the first time at 63.7 sec with 2.5% drift and at 68.4 sec with 3.6% drift, respectively. Subsequently, the first bar fracture occurred at 71.7 sec with 6.0% drift. The major degradation of strength started at 74.5 sec when the top of the pier was at 6.5% drift. Thereafter, the lateral load showed 20% reduction (from 78.7 kN to 62.6 kN), while the drift of the pier increased 1.75% (from 6.53% to 8.27%). The strength degradation signalled a potential collapse of the pier and the test was terminated. It was clearly evident from the physical condition of the specimen after the test that the damage state category was DS5 (complete damage).

Japanese Pier

The Japanese pier yielded when the drift reached 0.2% at 5.6 sec. During the first record (EQ1 with PGA = 0.4g), two principal horizontal cracks formed, one at the bottom of the pier and the other 300 mm from the bottom; however these cracks closed after 20 sec when the record EQ1 finished. The maximum drift and the corresponding lateral load measured were 1.48% and 327 kN, respectively, at 13 sec. The residual drift after the record EQ1 was negligible (measured to be 0.05%). Yielding of longitudinal bars suggested that the pier was in "slight damage" category after the record EQ1, i.e. DS2 according to the HAZUS damage classification.

During the second record (EQ2 with PGA = 0.8g), maximum drift of 1.76% was measured when the lateral load was 355 kN at 30.2 sec. In this phase, more horizontal cracks appeared throughout the bottommost one-diameter region (i.e. 600 mm) and these cracks were spaced approximately at 100 mm apart. Although these cracks were more intensive than those during the record EQ1, no residual cracks were visible after the record EQ2 terminated. The residual drift was measured to be 0.11% at the end of this phase. The cover concrete remained intact and no spalling was observed. Correspondingly, the damage state inspected after the record EQ2 was assessed to be DS2 (slight damage).

The extent of damage resulting from the final record (EQ3 with PGA = 0.8g) was restricted to cover concrete spalling, which occurred at 66 sec at a drift of 2.7%. The maximum response occurred at about 72 sec when the drift exceeded 4.2% and the lateral load at this stage was about 390 kN. At the end of the test, the residual drift was still 0.11%, same as that after the previous record EQ2. During this phase, neither buckling nor fracturing of the bars was noticed and the longitudinal bars remained intact. Even after the whole record finished, the core concrete was not damaged at all, and no cracks and gaps appeared around the longitudinal bars. Therefore, this bridge pier was apparently repairable simply by replacing the concrete at the area where the cover concrete peeled off. Accordingly, the damage state was assessed as DS3; i.e. "moderate damage" category in the HAZUS damage state classification (Mander and Basoz, 1999).

Comparison of the Three Piers' Performances

Responses of each of the three piers in the EW direction under each of the three successive records are arranged separately in Fig. 4 to give nine force-displacement hysteresis curves along with three response drift time-histories. Under the first record EQ1 with PGA = 0.4g (representing DBE), the load-displacement relationships in the first column of Fig. 4 show that all the bridge piers exhibited limited hysteresis response and only a minimal residual drift remained at the end of EQ1. The stiffness of the New Zealand pier was less than that of the other two due to its smaller diameter and lower lateral strength, and the maximum displacement response of the Caltrans pier was slightly smaller than that of the other two piers.

As shown in Fig. 4(d)-1, the drift time-histories of the three piers are similar before the first positive peak at approximately 6.3 sec, but the responses of the three piers differed afterwards. The Caltrans and Japanese piers moved almost together throughout the duration of the record EQ1, but the NZ pier showed a consistently larger response than the other two. The second record EQ2 with PGA = 0.8g also attracted a significantly larger response from the NZ pier compared to the other two. As suggested by the hysteresis loops in the second column of Fig. 4, Caltrans and Japanese piers dissipated less energy than the NZ pier did. Furthermore, the NZ pier had a 2.5% maximum drift, whereas the Japanese and Caltrans bridge piers responded within a 2% drift. As shown in Fig. 4(d)-2, the NZ pier swayed consistently more than the other two throughout the record EQ2, resulting in the largest residual drift among the three piers by the end of the record.

When subjected to the final and most severe earthquake record EQ3 with PGA = 0.8g, NZ pier failed completely before finishing the record, Caltrans pier was severely and irreparably damaged but could sustain the whole record without collapse, and the Japanese pier suffered only moderate damage and was in a repairable condition after the record. The first 14 sec of the record EQ3 showed similar response for all three piers. Then, the devastating part of the record hit the piers (see Fig. 4(d)-3). The NZ pier experienced 8% drift, which was the maximum drift observed in the whole test series. Soon, the longitudinal bars started to fracture and the lateral load started to reduce sharply (see Fig. 4(b)-3) forcing the termination of the test.

As can be seen in the third column of Fig. 4, the other two piers did not show any significant loss of lateral-load nor did they experience bar fracture. Hence, the tests with these two piers were continued for the whole duration of the record EQ3, which attracted 4.4% maximum drift from the Japanese pier and 6% maximum drift from the Caltrans pier.

Conclusions

Bi-directional PD tests were conducted on three RC piers designed according to the seismic design codes of Caltrans, NZ and Japan. Three earthquake records representing an upper-bound DBE, a median MCE and an upper-bound MCE were sequentially applied to 30% scaled physical models of these three prototype piers. Based on the experimental investigation reported herein, the following specific conclusions can be drawn.

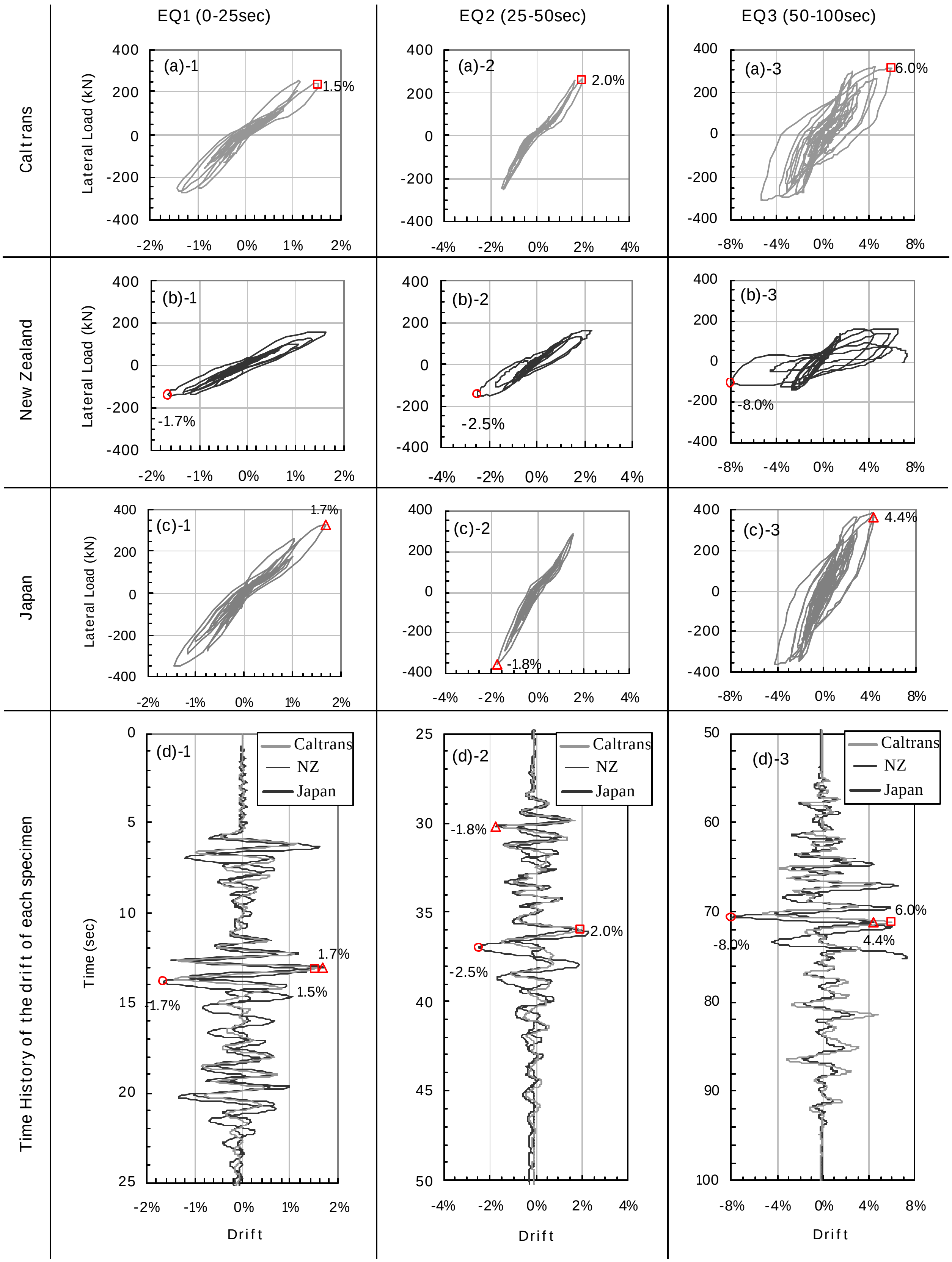


Figure 4 Comparison of the seismic performance of the three specimens.

1. All three bridge piers were only slightly damaged under the upper-bound (90^{th} percentile) DBE and the median MCE. For each country it could hence be said with high confidence that the bridge piers designed and detailed in accordance with existing specifications can be quickly restored after a DBE. It could also be said that the piers have at least 50% chance of surviving an MCE without severe damage and collapse.

2. The final earthquake record EQ3 with PGA = 0.8g representing the upper-bound (90^{th} percentile) MCE caused incipient collapse to the NZ bridge pier, inflicted severe and irreparable damage on the Caltrans pier, and induced moderate and reparable damage on the Japanese pier. Although designed for the same level of seismic hazard and under similar design conditions, Caltrans yielded a pier which was much stronger than the pier designed by NZ standard but slightly weaker than that designed by Japanese standard.

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