Technical Note

A Coaxial Cross Section Bridge Pier

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ABSTRACT

In this study, a new structural system for either retrofitting or design of new structures has been presented that provides more resistance for lateral loading conditions in comparison with the conventional systems. This new structural system can be utilized for a wide range of steel or concrete infrastructure systems from bridges to jackets as offshore structures. In this paper, structural performance of the new system is compared with conventional system for bridges. The structural response of piers in long- and medium-span bridges has been studied. A comparative study is carried out through static pushover analysis of four medium-span bridge piers and reveals the new system has a higher load-carrying capacity compared with the conventional system, whilst no significant changes are observed for period-based ductility. A probabilistic analysis of the structural collapse is carried out through incremental dynamic analysis (IDA). The results from IDA analyses show higher seismic safety for the new system compared to the conventional system. Besides, a time history analyses for far-field earthquake ground motions to evaluate structural response of a long span bridge was conducted. The results indicate that the stiffness degradation observed in the conventional system caused more damage than the stiffness degradation observed in the new system.

Keywords:
Far-field ground motion; Incremental dynamic analyses; New structural system; Structural collapse; Probabilistic analysis; Reliability index

1. Introduction

Continuous service of infrastructure is one of the main concerns in the design codes. Nevertheless, structural safety is threatened by accidental loads. Amongst the structures, bridges are vulnerable to the accidental loads, and earthquake loading is one of the major accidental loads causing collapses in bridges. These types of loads are random and difficult to predict, and there is possibility of much higher forces compared with the capacity of the structure. Considering the random characteristics of these types of loads, many studies have been performed using risk analysis techniques for better estimation of the probable loads to improve the reliability of the design procedure [1].

In this paper, a new structural system is introduced that provides an alternate path for carrying the vertical and horizontal loads by bridge piers to avoid collapse of the structure due to accidental loads. This new structural system consists of three parts: internal, middle and external parts. A general view of the proposed system is shown in Figure (1). While the external part absorbs exerted dynamic force in case of lateral force or ground motion as a result of its higher moment of inertia, the internal part

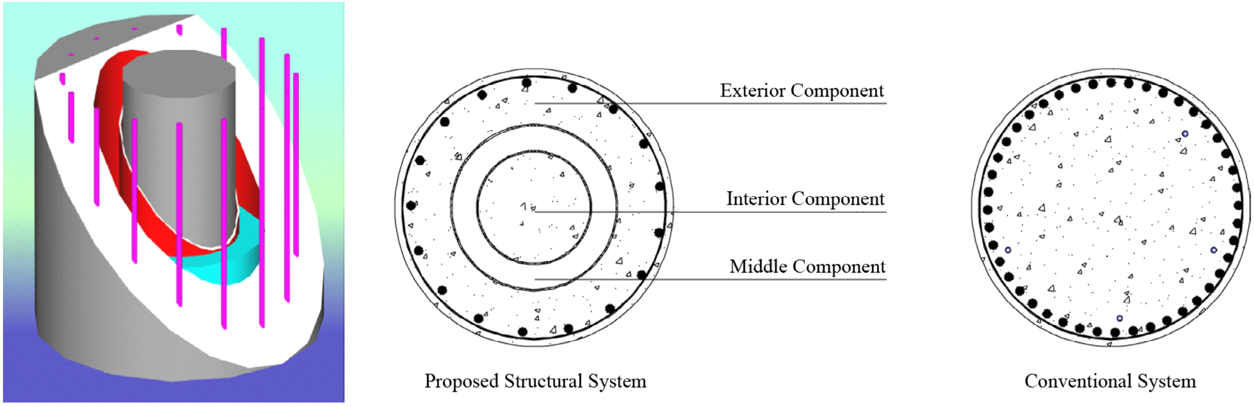


Figure 1. General view of the new and conventional system.

provides stability to the system for the vertical loads by providing enough cross-sectional capacity.

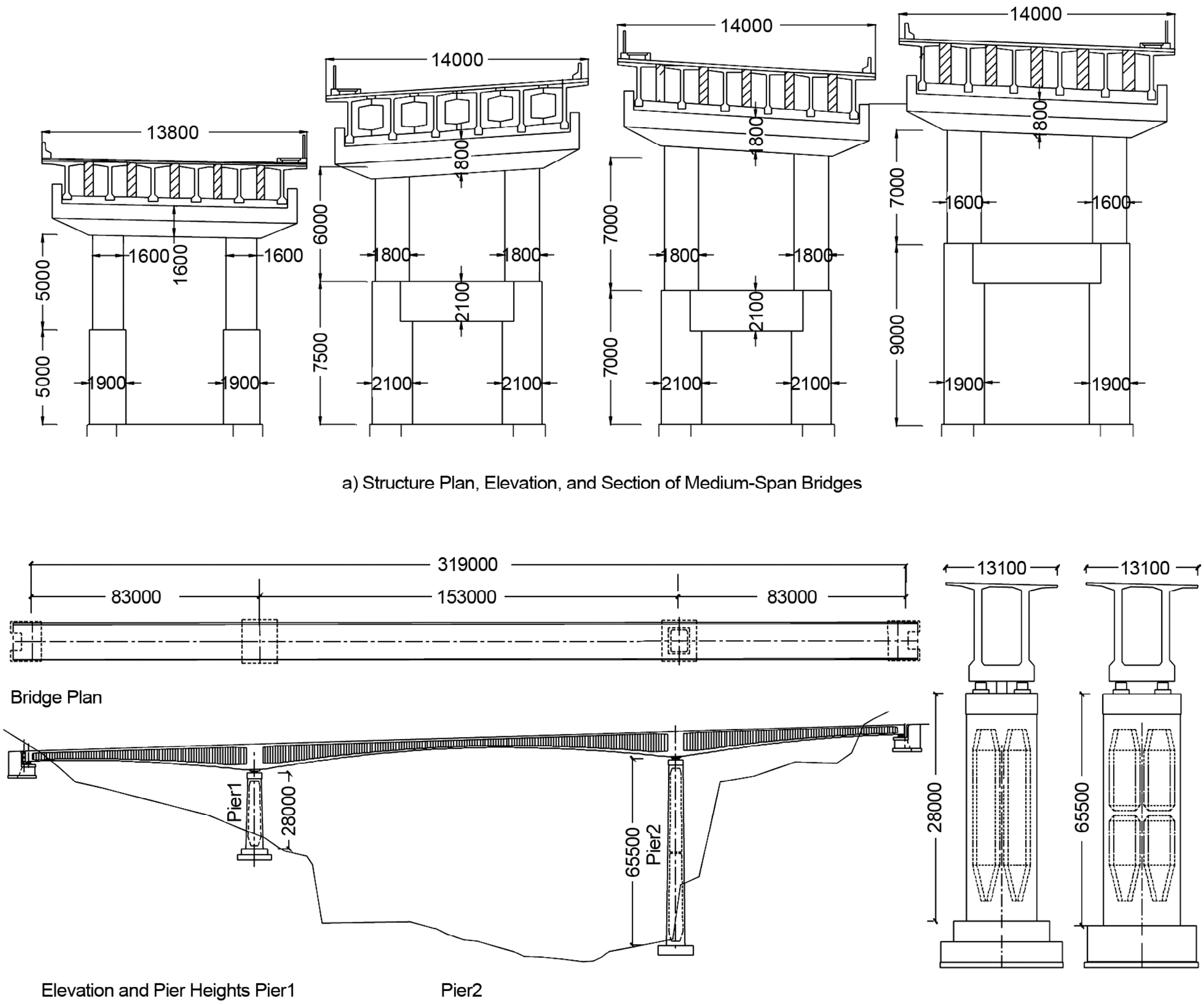
By absorbing a significant amount of the produced energy through the external part and introducing a time lag between the external and internal responses to the dynamic loading, this technique causes the energy of the intense loading to be absorbed mainly by the external part through elastic and, more importantly, plastic deformation. The middle part creates a lag between the internal and external parts. After dissipation of the accidental loads, the internal part maintains the stability of the system for existing vertical loads [2].

2. General Description of the Bridges for the Case Study

Four Typical piers from 17 bridges of a freeway that connects the capital to the northern borders, with medium span length of 30 and 40 meters and a long span bridge with the length of 320 meter with two piers are considered for com-paring the structural response of the new and conventional systems. The bridges consisting of two passing lanes and the superstructure include a 25-centimetre-thick reinforced concrete slab and six T-shaped pre-stressed beams. The substructure of the medium span bridges includes a pier cap sup-porting the beams and two circular columns. The heights of piers are 10 m (Pier1 Case), 13.5 m (Pier 2 Case), 14 m (Pier 3 Case) and 16 m (Pier4 Case). Span length of the bridges for piers 1 and 4 is 30 m, and it is 40 m for piers 2 and 3. Except pier 1, the columns for piers 2, 3 and 4 are divided by a beam that is elevated 7.5, 7 and 9 m from the top of the foundation. At each pier location, one span is free to slide longitudinally, while the other span is fixed along the axis of the structure. The geometry of piers 1 to 4 is shown in Figure (2a). The second model is a 319-metre long continuous bridge, which is named BR-02. This is a pre-stressed concrete bridge with three spans, including two 83-metre side spans and a 153-metre span in the middle. The superstructure of the bridge includes three passing lanes of an elevated 13.1-metre wide viaduct that is supported by single-column bents. The cellular deck of the bridge comprises pre-stressed concrete slabs at the top and bottom and corrugated steel plates for the side walls. This bridge is one of the longest bridges in Tehran, connecting the capital to the northern borders. The geometry of the bridge No. 2 is shown in Figure (2b).

3. Analytical Model for Bridges

The finite element software ZEUS-NL [3] developed at the Mid-America Earthquake Centre is utilised to perform the nonlinear analyses. Two pieces of software, SAP 2000 [4] and Section Builder [5], are used for designing the structural elements for the new system. The bridge is designed to carry conventional loads and also for a seismic zone IV according to AASHTO [6], with a design acceleration coefficient of 0.35 g. in addition to using AASHTO as a design code for bridges, ANSI/AISC 360-10 [7] is also considered for designing the piers of the new system, which consists of steel and concrete composite sections. The internal part is designed just to carry the dead load. Load combinations are considered for designing the pier with the new system according to AASHTO for the combined action of internal and external parts.



a) Structure Plan, Elevation, and Section of Medium-Span Bridges

b) Structure Plane, Elevation, and Section of the BR-02 Bridge

Figure 2. Geometry of the bridges (Dimensions in millimetre).

A two-dimensional model of the piers is assembled for dynamic analyses using ZEUS-NL. Two material models of Stl2 (Ramberg-Osgood model with a Masing-type hysteresis curve) and Con2 (Uniaxial constant confinement concrete model) from the material library of ZEUS-NL are considered for the nonlinear behaviour. Con2 and Stl2 are utilised for concrete and steel elements, respectively. A nonlinear element with the name of cubic (3D cubic elasto-plastic beam-column element) from the element library of ZEUS-NL is considered for modelling of the structural elements. Several analyses are conducted to compare the analytical results of ZEUS-NL with some experimental tests that had been carried out by PEER (Pacific Earthquake Engineering Research Centre) for the PEER Structural Performance Database [8]. A comparison of the results from ZEUS-NL with experimental tests reported by PEER shows that utilisation of the two material models of Stl2 and Con2 leads to reasonable compatibility with the test results. It was required to adopt marginal changes in the confinement factor of the Con2 model rather than the default values in the software.

The confinement factor of 1.27 is considered for the concrete elements in conventional system and the factor of 2 and 1 are considered for internal part, which is fully confined and external part of the new system. Both of the internal and external parts of the new system are categorized as in-fill and out-filled composite elements in concrete and steel design codes including AASHTO. The internal and external parts of the new system are designed according to AASHTO; therefore, considering the confinement factor of 1 for external part is a conservative selection due to the fact that the element is designed based on the requirements of the design codes. Cross sections of the piers for the new and conventional systems are shown in Figure (3).

A three-dimensional model of the BR-02 Bridge is assembled for dynamic analyses using SAP2000 and ZEUS-NL. Nonlinear-layered Shell elements are utilised in the SAP2000 model for the piers and deck. Heights of the pier are 28 and 65 metres for Piers 1 and 2, respectively. The cross section of the piers is reinforced-concrete twin hollow boxes. A shell element with a nonlinear in-plane and out-of-plane element component behaviour is chosen for reinforced concrete pier walls. Five layers for concrete and two equivalent rebar layers for reinforcement are adopted for the layered shell element. The cross section of the piers for both the new and conventional system is shown in Figure (4a). Joints at the bearings are restrained to provide the required boundary conditions. The deck is seated on fixed and movable bearings. The bearings are movable for the abutments and Pier 1, whilst the other pier (Pier 2) has a fixed bearing, as shown in Figure (4b). The joints at the pier foundation are restrained to provide fixed supports as they are considered to be located on competent rock.

In the ZEUS-NL model, each structural member is assembled using elasto-plastic elements (3D cubic elasto-plastic beam-column). The deck sections are modelled using an equivalent hollow rectangular section with the same moment of inertia for the two main axes of the transversal sections and the same cross-sectional area.

A reinforced concrete hollow rectangular section and a reinforced concrete flexural wall section are utilised for the piers. Distributed mass elements to represent dead and moving loads are considered for the deck and pier members. Several 3D joint elements are utilised at the pier-deck connections, with sliding bearings, which allow rotation and longitudinal motion but are restrained transversely for the movable bearing at Pier 1, while all translation movements are restrained at Pier 2. Bearings at the two ends of the bridge are considered to provide moveable conditions longitudinally and are restrained for transversal movement. Some simplifications were considered in the ZEUS-NL model. The curved shape of the superstructure through the length of the bridge is divided and changed to the equivalent rectangular segments for each 5 metre length of the deck.

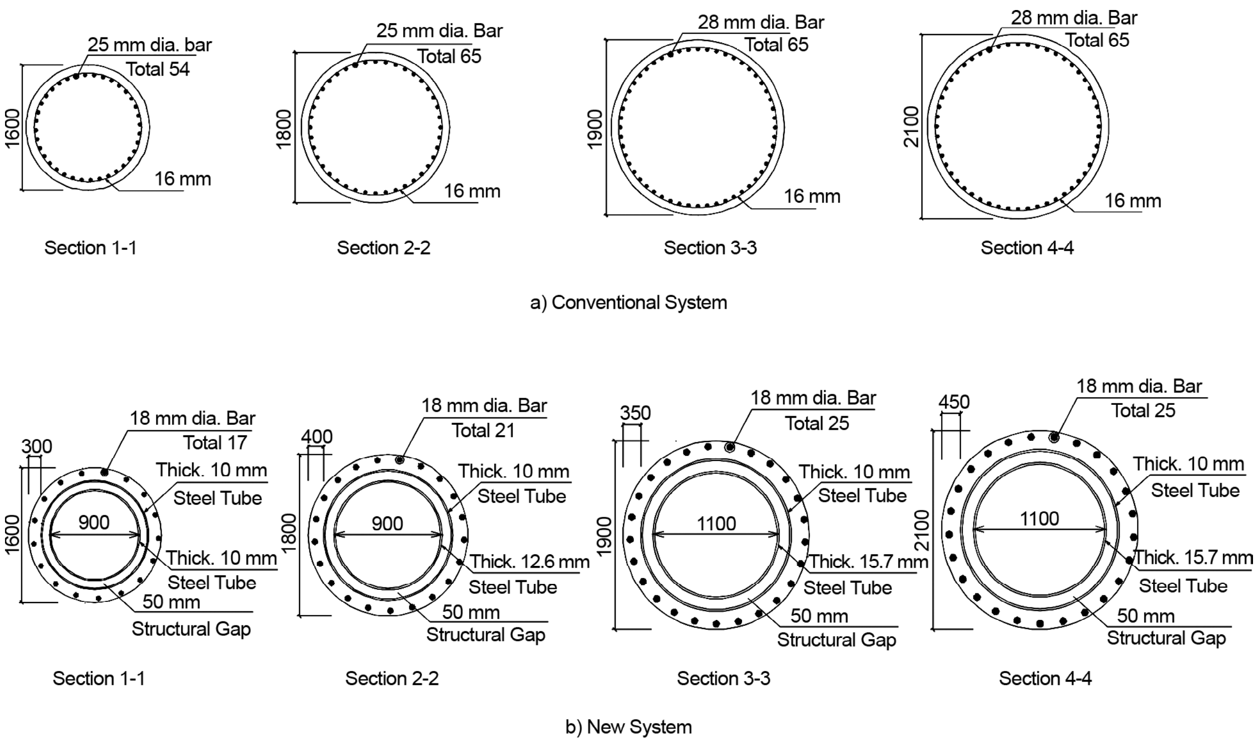


Figure 3. Cross section of the piers (Dimensions in millimetre).

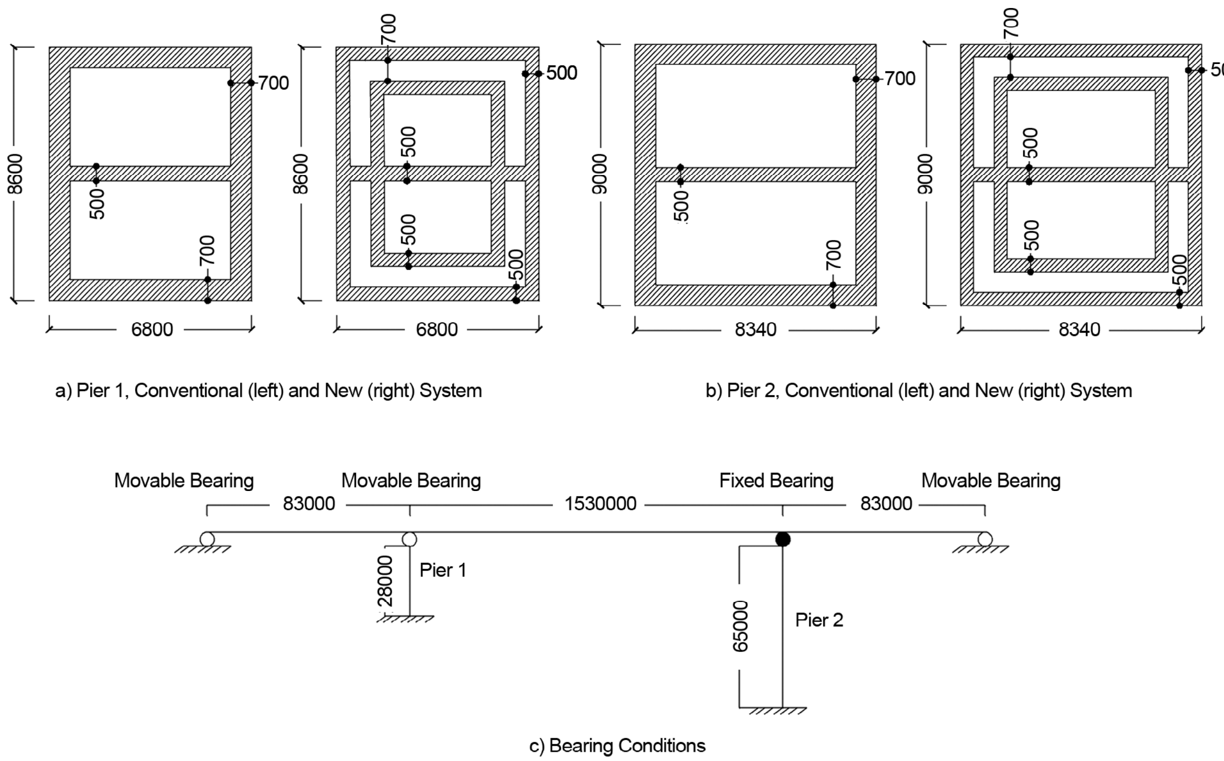


Figure 4. Bearing conditions and cross section of the piers for the BR-02 Bridge (Dimensions in millimetre).

4. Analytical Investigation

To evaluate the structural behaviour of the new and conventional systems, pushover and time history analyses are carried out. Incremental dynamic analyses [9] and static pushover analyses are conducted for assessment of the seismic collapse safety of the piers with medium span bridges. Three earthquake ground motions [10] are taken into account to evaluate the structural response to seismic conditions in Br-02 as a long span bridge. Global performance criteria for failure are adopted as a drift of 3% and a degradation of lateral resistance of more than 10% [11].

5. Static Push-Over Analyses

Static pushover analyses are performed to investigate the general load-deflection relationship and load-carrying capacity for the piers of the medium and long span bridges. Pushover analysis is conducted by applying the load at top of the piers. The structural response is presented in Figure (5).

Generally, the results show a much higher capacity for the new structural system in comparison with the conventional system. The results for the new system show almost twice as much capacity as the conventional system. The nonlinear static pushover analyses are used to quantify the maximum base shear Vmax and the ultimate top displacement δu, which are then used to compare archetype over strength and period-based ductility. The over-strength factor, which is defined as the ratio of the maximum base shear resistance to the design base shear, is almost twice for the new system in comparison with the conventional system. The period-based ductility is defined as the ratio of ultimate top displacement to the effective yield top displacement. The results reveal that whilst significant increase is observed for the base shear in the new system, which is around 200%, there is marginal difference for the ductility ranges from 5% to 17%. A comparison of the results for the ductility based period and the over-strength factor is shown in Tables (1) and (2).

6. Incremental Dynamic Analyses (IDA)

For evaluating the likelihood of earthquake-induced collapse in the piers, performance-based engineering methods are applied. This method relates the structural response to the ground motion intensity through probabilistic assessment of the results from nonlinear dynamic analyses. Structural safety is evaluated

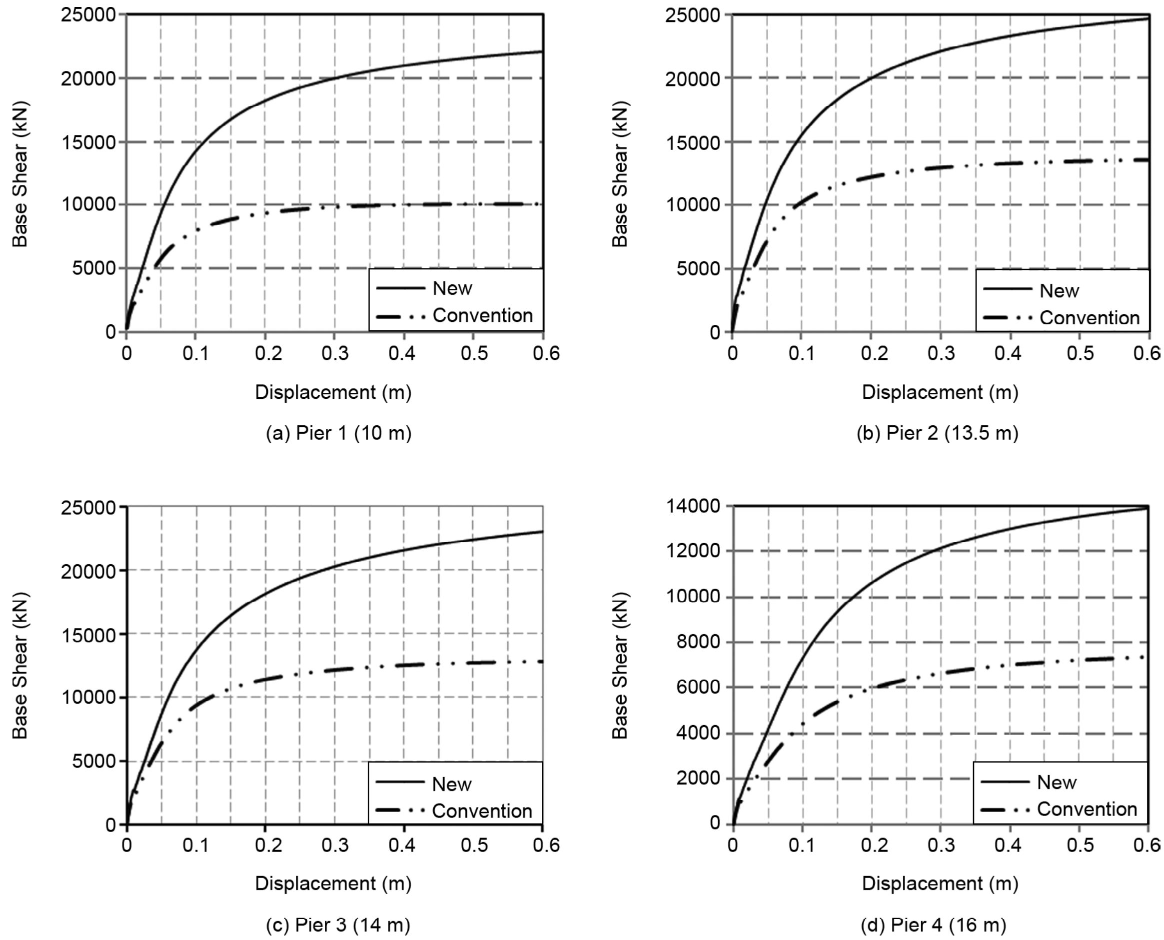


Figure 5. Pushover Analyses result.

Table 1. Period-based ductility for the new and conventional system.

Span Length (m)	40	35	35	30
Pier Height (cm)	1000	1350	1400	1600
Maximum Base Shear for the new system (kN)	21000	22500	23000	14000
Maximum Base Shear for the conventional system (kN)	10000	14000	12500	6700
$\delta_{y,eff}$ for the new system (cm)	9	9.5	10	16
$\delta_{y,eff}$ for the conventional system (cm)	7.5	9	9.5	14
Period-Based Ductility for the new system	3.3	4.2	4.2	3.0
Period-Based Ductility for the conventional system	4.0	4.4	4.4	3.4

Table 2. Results of the collapse performance assessment.

Pier Height (m)	Structural System	Over-Strength Factor	Fundamental Period (sec)	Median Collapse Capacity	Ground Motion Intensity (g)	Collapse Margin Ratio
10 (Pier-1)	New	4.02	0.24	2.61	1.09	2.39
	Convention	2.02	0.27	1.65	1.01	1.63
13.5 (Pier-2)	New	4.12	0.24	2.69	1.09	2.47
	Convention	2.55	0.25	1.97	1.06	1.86
14 (Pier-3)	New	4.27	0.26	2.59	1.03	2.51
	Convention	2.59	0.27	2.41	1.01	2.39
16 (Pier-4)	New	3.18	0.36	2.31	0.83	2.78
	Convention	1.88	0.39	1.79	0.79	2.27

through collapse performance assessment. An incremental dynamic analysis (IDA) is conducted for collapse assessment of the piers for medium span bridges. The piers are subjected to analysis under multiple ground motions that are scaled to increasing intensities. The ground motion set that is utilised for performing nonlinear analyses is the far-field ground motion set used in FEMA P-695 [12]. The far-field record set includes twenty-two records (44 individual horizontal components) from large-magnitude earthquakes (magnitude of 6.5-7.6), and the record selection criteria for this ground motion set are documented in Haseltone and Deierlein [13]. Ground motion records are selected and anchored to specific ground motion intensity such that the median spectral acceleration of the record set matches the spectral acceleration at the first mode period of each pier that is being analysed.

Nonlinear response history analyses are conducted under the factored gravity load combination in FEMA P-695. Horizontal components of ground motions are applied to the piers by using the IDA approach. Individual ground motions are scaled to increasing intensities until the structure reaches a collapse point, which is considered dynamic instability. Collapse under each ground motion is judged to occur from the dynamic analysis results as evidenced by excessive lateral displacements (sideways collapse) and loss of strength, which are a drift of 3% and a degradation of lateral resistance of more than 10%. Sample results from an incremental dynamic analysis for pier 4 (16 metre height) are depicted in Figures (6a) and (6b) for the new and conventional systems.

A collapse fragility function can be defined through a cumulative distribution function (CDF) by using data from IDA results. The fragility curve relates the ground motion intensity to the probability of collapse. Figures (6c) and (6d) present the fragility curve, which is obtained by fitting a lognormal distribution through the collapse data points. The two parameters of median collapse capacity and

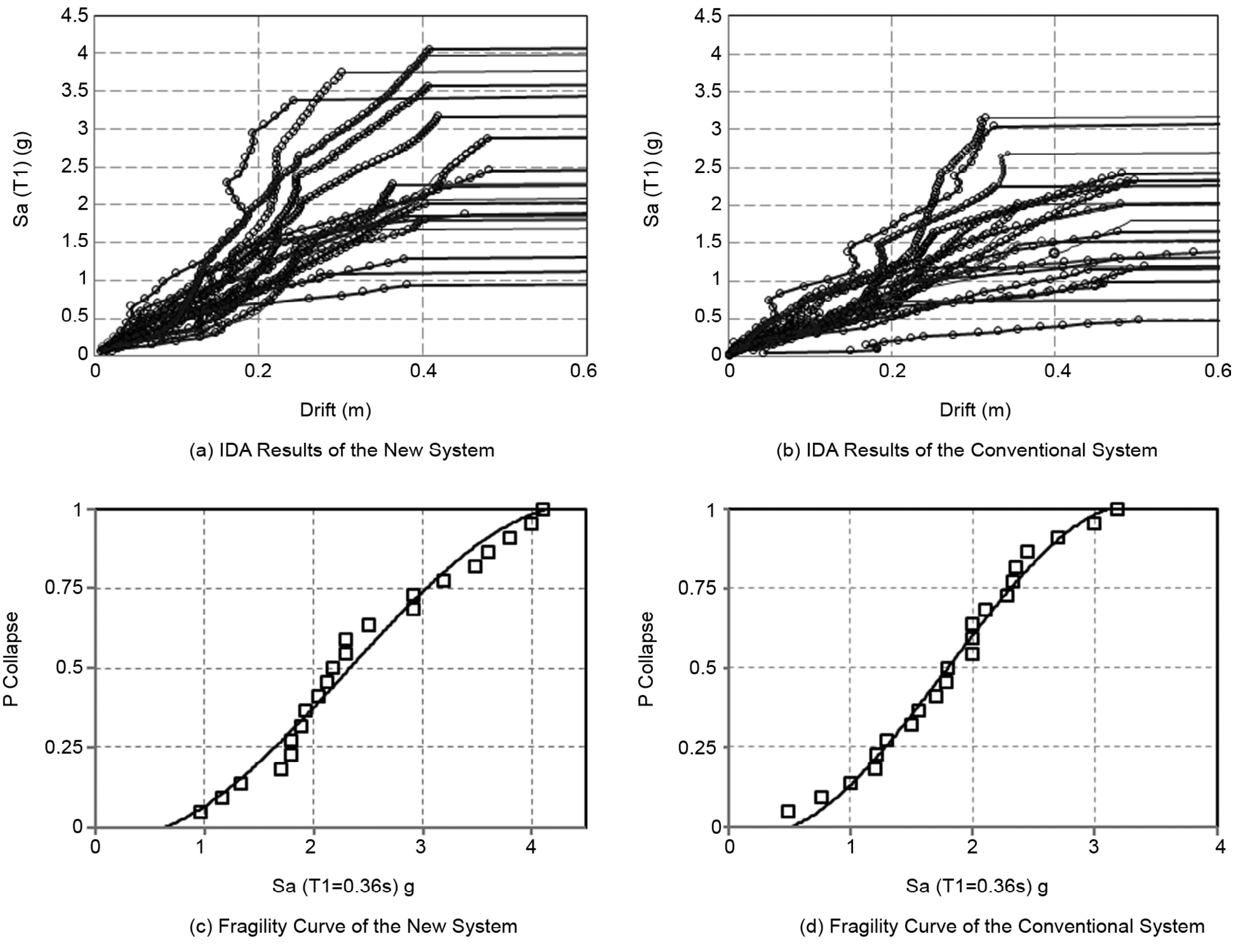


Figure 6. Results of incremental dynamic analyses and fragility curves for pier 4 (16 M).

collapse margin ratio are computed from analysis results. The median collapse capacity is computed as the spectral intensity when half of the ground motions cause the structure to collapse. Uncertainty effects are neglected in this study, as the main purpose is to compare the structural behaviour of the two systems. The ratio between the median collapse intensity and the ground-motion intensity, with a 2% chance of exceeding in 50 years, is the collapse margin ratio. The ground-motion intensity is taken directly from the response spectrum (AASHTO). The results for the seismic collapse assessment of the piers are depicted in Table (2).

An assessment of the results reveals that both of the key metrics, the median collapse capacity and the collapse margin ratio, for the new system are larger than the conventional system within a reasonable limit that provides a higher seismic safety for the new system.

7. Nonlinear Dynamic Time History

The structural response of the BR-02 model for the new and conventional system is compared through nonlinear dynamic analyses. The input motions employed in the following dynamic analyses are an acceleration time history of Imperial Valley (El Centro), Northridge and Kobe, selected from PEER strong ground motion database [14]. The corresponding peak ground accelerations for transverse, longitudinal and vertical components are shown in Table (3). Because most of the strong motion occurred during the first 20 seconds of each ground motion, just this part is considered in the analysis.

An eigenvalue analysis is carried out to evaluate the dynamic characteristics of the new and conventional system in the Zeus-NL and SAP 2000 models. A comparison of the periods for the BR-02 Bridge is shown in Table (4). The results for the first five modes of vibration reveal that the figures for SAP 2000 are higher than those for ZEUS-NL with marginal differences. The first mode is the predominant one in the longitudinal direction. Except for the 3rd mode in SAP2000, the period of the conventional system is higher than the new system, which is also the same for ZEUS-NL. The mode shapes of the first three modes, which are the same for both of the systems, are shown in Figure (7) for SAP 2000 and ZEUS-NL. The first and second modes are in the longitudinal and transversal direction, respectively. As a result, the new system has a higher stiffness in the horizontal direction. The third mode is in the vertical direction.

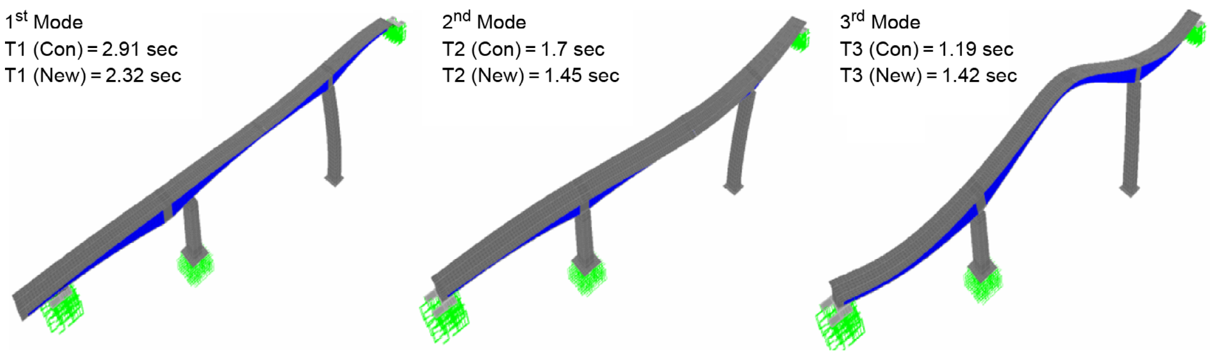
The BR-02 Bridge is analysed to evaluate the dynamic response of tall piers. Figure (8) depicts comparisons of the longitudinal displacement for pier 2 (65 metres in height) for El Centro and Northridge ground motions. In general, the new system shows lower deflection compared with the conventional system with regard to the total displacement. The deformation of the pier with the conventional system is almost twice the value of the new system for El Centro and Northridge ground motions. The structural response for the El Centro ground motion shows period elongation after 10 seconds.

Table 3. Ground motion characteristics.

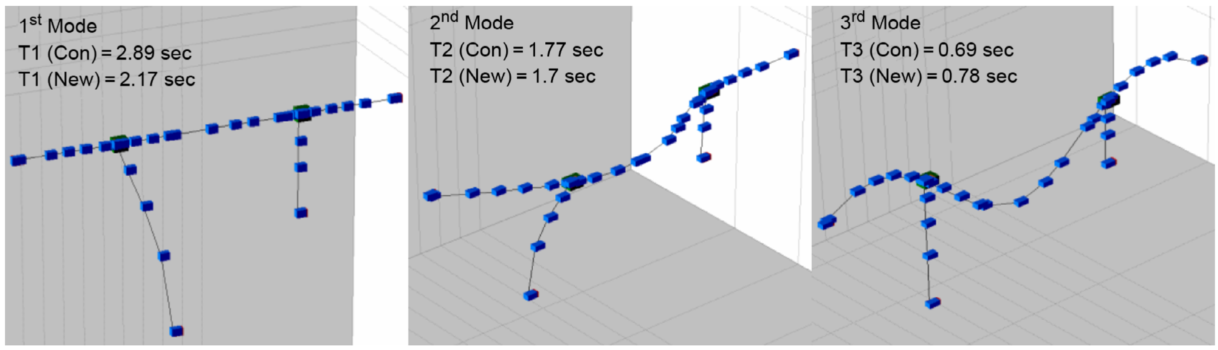
Record/Component	PGA (g) Up	PGA (g) Longitudinal	PGA (g) Transversal	Date	Magnitude
KOBE/KJM000	0.343	0.821	0.599	1995/01/16	M (6.9)
IMPVALL/H-BCR140	0.425	0.775	0.588	1979/10/15	M (6.5) Ml (6.6) Ms (6.9)
NORTHR/SYL360	0.535	0.843	0.604	1994/01/17	M (6.7) Ml (6.6) Ms (6.7)

Table 4. Periods of the first five modes of vibration for the new and conventional systems.

Period	SAP 2000		ZEUS-NL	
	New	Convention	New	Convention
T1 (Sec)	2.32	2.91	2.17	2.89
T2 (Sec)	1.45	1.7	1.7	1.77
T3 (Sec)	1.42	1.19	0.78	0.69
T4 (Sec)	0.79	0.94	0.58	0.67
T5 (Sec)	0.51	0.63	0.52	0.58

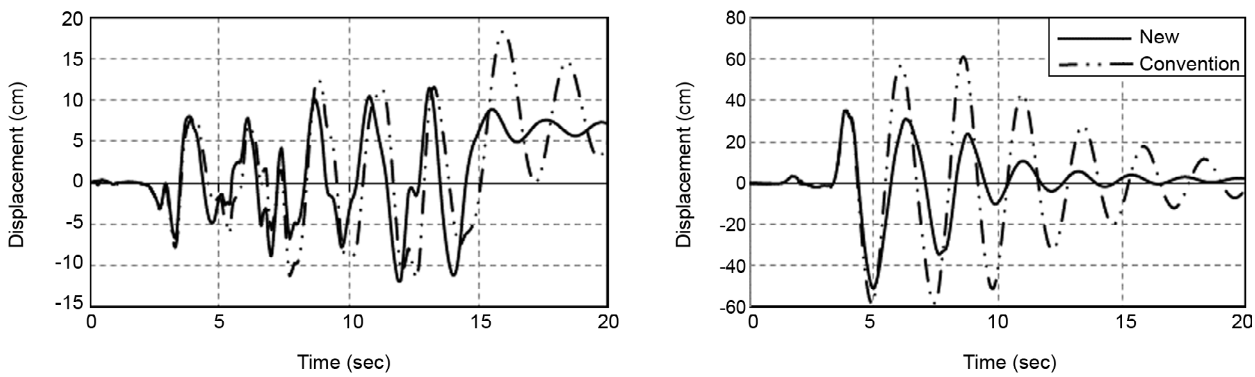


(a) Mode Shapes of the BR-02 in SAP 2000



(b) Mode Shapes of the BR-02 in ZEUS-NL

Figure 7. Mode shapes of the first three modes of vibration for BR-02 bridge.



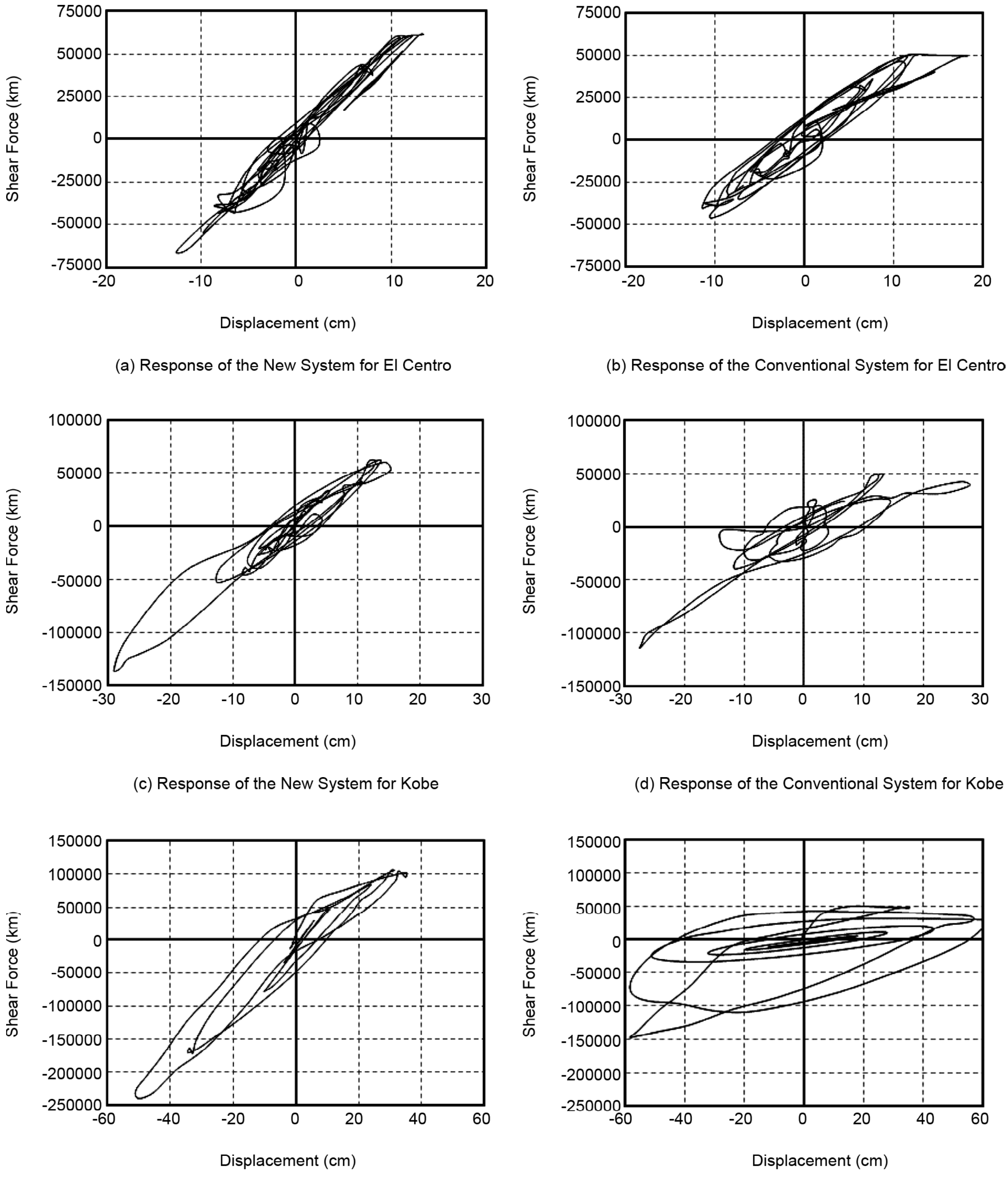
(a) Response for El Centro

(b) Response for Northridge

Figure 8. Longitudinal displacement response.

Further assessment is carried out in terms of the hysteretic response of both systems [15]. Figure (9) depicts the hysteretic response of pier 2 for the three ground motions. Generally, different energy absorption and dissipation capacity values are observed for both systems. The difference is especially significant for the Northridge and Kobe ground motions, in which a significant reduction in energy dissipation capacity and stiffness degradation is observed. This can be due to the large cycle of loading and pronounced pinching during reloading.

The displacement response of the conventional system is larger than that of the new system for all three ground motions. In addition to less deformation, a higher shear force capacity is observed for the new system. This reflects the fact that, response characteristics, particularly stiffness, are significantly affected in the new system and a more severe damage pattern is observed in the conventional system. As a result, comparing the observed damage pattern



(a) Response of the New System for El Centro

(b) Response of the Conventional System for El Centro

(c) Response of the New System for Kobe

(d) Response of the Conventional System for Kobe

(c) Response of the New System for Northridge

(d) Response of the Conventional System for Northridge

Figure 9. Hysteretic response of pier 2, BR-02 Bridge.

and the stiffness degradation for the two systems shows an improved seismic behaviour for the new system.

8. Conclusion

A new structural system is developed and implemented for bridge piers. To investigate the structural behaviour of the new system, two different types of bridges with medium and long spans are analysed and evaluated. A static pushover analysis is carried out to compare the ultimate capacity of the new system with the conventional system. Incremental dynamic analyses are conducted to evaluate the seismic collapse safety of the new and conventional systems for the medium spam bridges. A nonlinear time history analysis is also conducted to assess the seismic response of the new and conventional systems under three earthquake ground motions for a long span bridge.

In general, the structural response shows more stability for the new system in terms of strength and stiffness degradation within a reasonable limit compared to the conventional system. An evaluation of pushover analyses reveals that the ultimate capacity of the new system is higher than the conventional system. Over-strength factors for the new system are approximately two times larger than those of the conventional system. Comparison of the results from ductility-based period reveals that whilst significant increase for the base shear is observed in the new system, no significant difference is observed for the ductility. A probabilistic assessment of the safety indicates that the new system has a higher collapse safety than the conventional system and that piers built with the new system are able to withstand higher ground motion intensities. Both key metrics, the median collapse capacity and the collapse margin ratio, are larger for the new system compared with the conventional system. The hysteretic response resulting from shear-displacement of the model exhibits pronounced stiffness degradation for the conventional system. A significant degradation in strength is observed once the maximum cycle occurs, leading to a reduction in energy dissipation for the conventional system. Utilisation of the new system has a direct effect on the damage pattern of the piers by reducing the deformation and increasing the shear capacity.

The new system presented in this paper has the ability to be used for either the design of new bridges or the strengthening of existing bridges. The improved structural behaviour and the ability to monitor existing parts are the advantages of the proposed system. This monitoring can be performed via monitoring points located at the points on the external part with the lowest stress and strain level. The gap in the middle part will provide the required space for monitoring and maintenance purposes. This is especially critical for the strengthening of existing structures because in most usual retrofitting cases, the existing elements are covered with new materials in such a way that there is no possibility for monitoring the internal parts.

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