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Fragility curves for the typical multi-span simply supported bridges in northern Pakistan

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Abstract. Bridges are lifeline and integral components of transportation system that are susceptible to seismic actions, their vulnerability assessment is essential for seismic risk assessment and mitigation. The vulnerability assessment of bridges common in Pakistan is very important as it is seismically very active region and the available code for the seismic design of bridges is obsolete. This research presents seismic vulnerability assessment of three real case simply supported multi-span reinforced concrete bridges commonly found in northern Pakistan, having one, two and three bents with circular piers. The vulnerability assessment is carried through the non-linear dynamic time history analyses for the derivation of fragility curves. Finite element based numerical models of the bridges were developed in MIDAS CIVIL (2015) and analyzed through with non-linear dynamic and incremental dynamic analyses, using a suite of bridge-specific natural spectrum compatible ground motion records. Seismic responses of shear key, bearing pad, expansion joint and pier components of each bridges were recorded during analysis and retrieved for performance based analysis. Fragility curves were developed for the bearing pads, shear key, expansion joint and pier of the bridges that first reach ultimate limit state. Dynamic analysis and the derived fragility curves show that ultimate limit state of bearing pads, shear keys and expansion joints of the bridges exceed first, followed by the piers ultimate limit state for all the three bridges. Mean collapse capacities computed for all the components indicated that bearing pads, expansion joints, and shear keys exceed the ultimate limit state at lowest seismic intensities.

Keywords: concrete bridges; fragility curves; non-linear dynamic analysis; incremental dynamic analysis; Pakistan

1. Introduction

Damage of earthquakes worldwide has highlighted bridge as the most vulnerable component of the transportation system (Stefanidou and Kappos 2016). Failure of bridges during earthquakes causes failure of the transportation system and disrupts the relief measures after the earthquake. Bridges connect the infrastructure and their role is very much important in the overall impact assessment of earthquakes on the region (Neilson 2005). Therefore, their safety and stability is very crucial.

Pakistan is located at the junction of Indian, Eurasian and Arabian tectonic plates, which is seismically very active region. The Hindukush, Himalayans thrust fold belt are the major tectonic plate elements in the northern Pakistan which are very active and have potential to generate future large earthquakes. The Himalayan alone is capable to generate magnitude 8.0 or even greater earthquake (Bilham 2004).

The population of Pakistan is above 19 million (PESR 2015), so there exit high seismic risk to infrastructure and human lives. Keeping in mind the seismic risk pertaining to structures, it has been observed that very little effort is made to reduce the seismic risk related to bridges in Pakistan.

The poor performance of bridges in Pakistan has been observed during the recent past Kashmir (2005) earthquake in which many bridges were completely collapsed or severely damaged beyond repair (Ali *et al*. 2011). One of the major reasons for the poor performance of bridges was the substandard seismic design of bridges using old and non-regional codes and low quality of construction workmanship. Significant amount of research has been carried on the seismic vulnerability assessment of buildings (e.g., Ahmed *et al*. 2014, Ahmed *et al*. 2013, Ahmed *et al*. 2012a, b) however, very limited research can be found addressing vulnerability of bridges in Pakistan. Ali (2009) has recently conducted experimental work (quasi-static cyclic tests) on reduced scale single bridge piers and numerical analysis of the same for the computation of seismic response modification factor (R) and fragility curves.

Few other studies can be found dedicated to the seismic response assessment and retrofitting of bridge piers through quasi-static cyclic testing on reduced scale bridge piers (i.e., Iqbal *et al*. 2012, Khan *et al*. 2015, Saeed *et al*. 2015, Khan *et al*. 2015). However, all these studies are primarily based on the performance of individual bridge pier only. A comprehensive study dedicated to complete bridge structure analysis for the existing and newly constructed bridges are crucial to understand the seismic performance/vulnerability

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Table 1 Ground motion parameters suggested by the previous studies in northern Pakistan

S.No	Name of Cities	Peshawar	Islamabad	Muzaffarabad	Lahore
1	Building Code of Pakistan (1986)	MMI V-VI	MMI VII	MMI VII	MMI V-VI
2	Pakistan Metrological Department (PMD) Map (1999)	0.050-0.067 g	0.050-0.067 g	0.010-0.050 g	$\leq$0.067 g
3	Bhatia *et al*. (1999)	0.100-0.150 g	0.150-0.200g	0.200-0.250 g	-
4	Zhang *et al*. (1999)	0.166-0.244 g	0.244-0.326 g	0.244-0.326 g	0.166-0.244 g
6	MonaLisa *et al*. (2007)	0.150 g	0.150 g	0.130 g	-
7	Building Code of Pakistan (2007)	0.160-0.240 g	0.24-0.32 g	> 0.32 g	0.080-0.160 g
8	Rafi *et al*. (2012)	0.175 g	0.218 g	0.455 g	0.109 g
9	Hashash *et al*. (2012)	0.200-0.400 g	0.200-0.400 g	0.400-0.600 g	-
10	Zaman *et al*. (2012)	0.330-0.400 g	0.410-0.970 g	0.410-0.970 g	0.170-0.240 g

of bridge structure system as a whole (e.g., Molesh *et al*. 2016, Mirza *et al*. 2016, Tavares *et al*. 2012).

There are two main reasons for the present research study, dedicated to the seismic vulnerability assessment of bridges in Pakistan:

1. The existing ground motion seismic hazard map is not clearly defined to be used for the assessment and design of bridges.
2. Seismic design code for bridges is obsolete.

There are different versions of ground motion maps issued by different government departments and individual researchers from time to time. These maps define expected ground shaking in terms of Peak Ground Acceleration (PGA) and Modified Mercury Intensity (MMI) values. A comparison of all these maps at important cities is made in Table 1, which shows significant disparities.

The official code of practice for bridge design is known as the West Pakistan Bridge Code, 1967 (WPBC 1967). WPBC (1967) has been adopted from American Association of State Highway and Transport Officials (AASTHO), 1961, 11th Edition (Ali 2009). According to this code seismic forces are to be taken as 2-6% of the weight of structure depending upon the type of foundation system of structure. WPBC (1967) is rarely considered for the design of bridges and instead AASHTO- LRFD code is being followed for the design and the WPBC (1967) is only being considered for definition of live loads only.

Simply supported I-shaped girder bridges are very common in Pakistan. In this study seismic performance of the common simply supported Reinforced Concrete bridges found in northern Pakistan have been evaluated and discussed. Three real bridges are selected as representative bridges. Non-linear dynamic analysis using real earthquake records is carried to assess seismic performance of these bridges.

The non-linear dynamic analysis could not capture response at the specified level. Bridges under the study have different components which have the ultimate limit state different from one another. Determining the ultimate limit state of bridge components is a significant in the performance assessment of high bridge (Avsar and Yakut 2012). In order to know the ultimate limit state for each of the component and damage pattern of bridge structure, incremental dynamic analysis is also carried out.

Incremental dynamic analysis (Vamvatsikos and Cornell 2001) is carried out to derive fragility curves for different components of the case study bridges which are important tools to assess the level of safety of bridges probabilistically in case of seismic event. Additionally, fragility curves help in making rational decision in bridge cost-benefit analysis whether the bridge should be replaced or retrofitted. The results of the analysis have been used to derive the fragility curves for different components of the selected bridges.

2. Properties of bridges

Common type of bridges found in northern Pakistan are the multi-span simply supported bridges. The superstructure of these bridges consists of pre-stressed RC I-shaped girders and concrete deck slab. The girders are pre-stressed members placed on elastomeric bearing pads and shear keys are constructed between the girders to provide restraints in transverse direction of the bridge. The superstructure has expansion joints at abutments and intermediate bents. Three real Reinforced Concrete (RC) bridges are randomly selected to carry out the seismic performance assessment. The selected bridges have circular piers and simply supported superstructures. The superstructure of these bridges consists of piers of circular cross-sections. The bridges have a single, two and three piers bents. The cap beams cross sections are rectangular or tapered rectangular. I-shaped girders have high compressive concrete strength of 5000 pounds per square inch (psi) and all other components have 4000 psi concrete strength material.

2.1 Multi-span simply supported Mingora Bridge

Mingora Bridge located in Mingora district of Khyber Pakhtunkhwa province, north Pakistan, is a three span simply supported bridge having two multiple columns typical bents. The bent consist of a solid circular column and a rectangular cap beam. The abutment consists of a RC wall supported on the pile cap foundation and side wing walls. Abutment wall has seating arrangements for bearing pads. The bridge has I-shaped post-tensioned girders. I-shaped girders are supported at each end on rectangular bearing pads at a constant spacing of 2900 mm. These bearing pad rest on the cap beam and on the abutments. Each span has three rectangular diaphragms in the superstructure, at start, middle and end of each span. It

Table 2 Details of the bridges

S.No	Bridge	Spans	Span Length (mm)	Pier Diameter (mm)	Pier Height (mm)
1	Mingora Bridge	3	16100	1200	4450
2	Chakrisar Bridge	4	20000	1200	5870
3	Badeen Bridge	4	21025	1500	5580

connects the four girders and each girder has 1200 mm depth. The geometrical dimensions of members are all similar throughout the bridge. The reinforcement of each column and cap beam is identical throughout the bridge. The piers have pier shafts foundations 18 meters (m) inside the ground. The abutments have pile cap foundations supported on different piles. Each pier is 1200 mm in diameter and reinforcement is 1.47% of gross cross-sectional column area. The cap beam has a 1200 mm×2070 mm rectangular section. The superstructure is discontinuous, separated by expansion joints. Both deck slab and girders at bents are separated from each other by 25 mm expansion joints. The same gap exists between girders and abutment wall. The bearing pads are placed on concrete pads and are fixed to the pad with the help of epoxy grout. There are shear keys built alongside the bearing pads between the girders to prevent lateral movement of the girders. These shear keys are built on the top of each column bent and at the abutments. There is a gap of 10 mm filled by polystyrene between the shear keys and the girders. The geometrical dimensions of the shear keys provided at the bents are different from those provided at the abutments. As for the material properties, the girders have 5000 psi (34.41 MPa) cylindrical strength concrete, the piers, cap beam and piles, deck slab have 4000 psi (27.53 MPa) concrete while curb stones and sideways are made of 3000 psi (20.53 MPa) concrete. The bearings are laminated elastomeric bearings and the internal steel plates in laminated pads are rolled mild steel according to ASTM A-283. The bearing pad has a section 400×500×46 mm. The information about the design code is not available for the bridge.

2.2 Multi-pan simply supported Chakrisar Bridge

The Chakrisar Bridge is located in district Shangla of the Khyber Pakhtunkhwa province. It is a four span simply supported bridge. It has been designed according to WPHB 1967 and AASHTO-LRFD 2012. The Peak Ground Acceleration (PGA) design value used in the seismic design is 0.29 g that is mentioned on the drawing of the bridge. All the material properties of this bridge are similar to the Mingora Bridge. Each span of the bridge is 20,000 mm long and the superstructure consists of a deck slab supported on I-shaped pre-stressed girders. Three girders are supported on laminated elastomeric bearing pads. It has two end abutments and two intermediate bents. Each bent has two circular solid piers and a rectangular cap beam and every bent has a pile cap foundation. The elastomeric bearing pads are rectangular 400×375×54 mm laminated

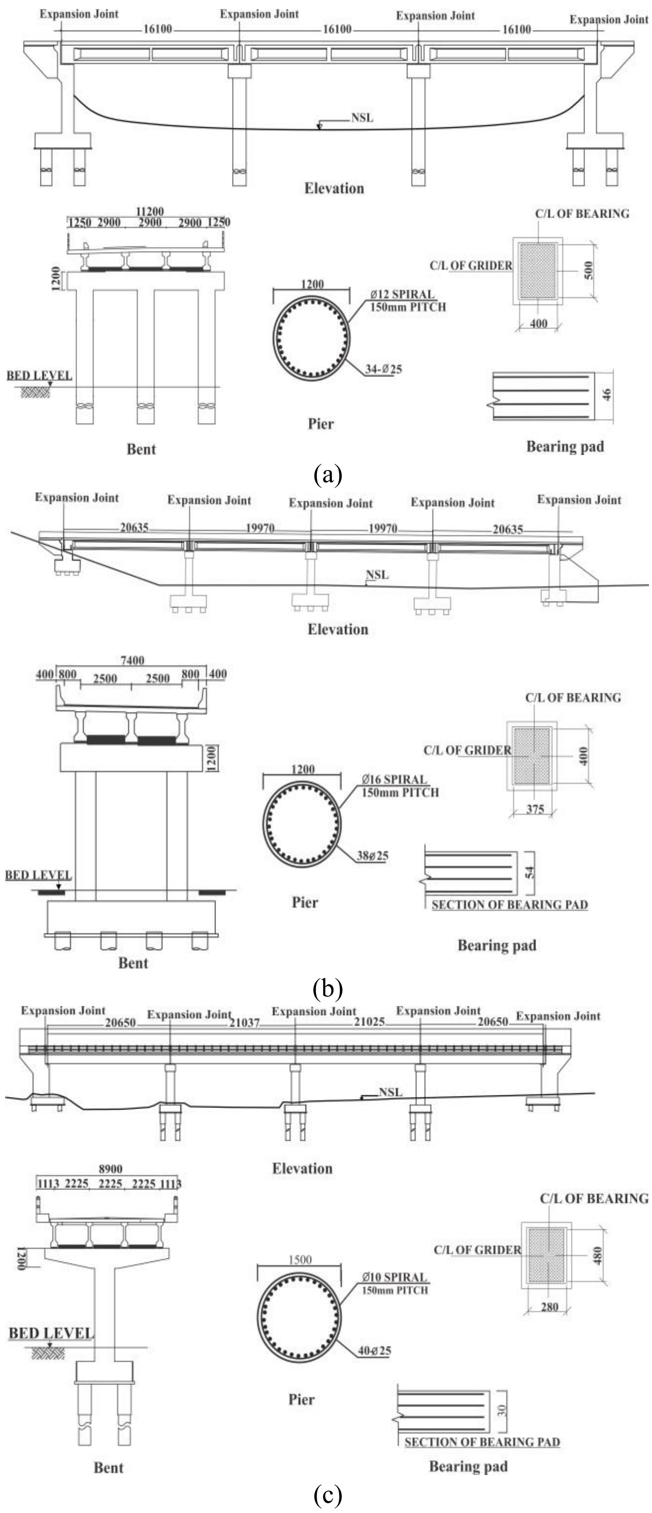


Fig. 1 Details of the bridges for (a) Mingora Bridge (b) Chakrisar Bridge (c) Badeen Bridge

elastomeric bearing pads. Thickness of bearing pad is 54 mm of which 16 mm consists of steel plates. Shear keys are constructed between the girders to prevent lateral movements. Three diaphragms are constructed in each span in the transverse direction of the bridge. The pier has a diameter 1200 mm. The bent heights are slightly different from each other i.e., 5790, 5580 and 5870 mm. There are three girders in each span of bridge; spaced 2500 mm and 7400 mm is the width of the bridge. Expansion joints of 30 mm are provided in the superstructure at each bent and at abutments. The reinforcement in each pier is 1.6% of the gross area.

2.3 Multi-span simply supported Badeen Bridge

The Badeen Bridge is located in district Lower Dir of Khyber Pakhtunkhwa province of Pakistan. It is single circular pier, bridge having four spans, two spans are 21025 mm long and two spans have 20625 mm. The total span length of this bridge is 83375 mm. The width of this bridge is 8900 mm. The Bridge is supported on three single solid circular piers and two end abutments. In each span, four girders are placed. Superstructure is discontinuous and simply supported both at the bents and abutments. Expansion joint of 25 mm is provided in the superstructure.

Design code followed for design of this bridge is not mentioned, it is assumed WPHB, 1967 and AASTHO LRFD design specifications have been followed for its design. Information about geometric properties and material properties for all of its components are available from the drawings. Each bent has one 1500 mm diameter circular pier and a tapered cap beam and a pile cap foundation, which has a four 750 mm diameter piles. Shear keys are placed between the girders to prevent lateral movement. Girders and deck slab are connected by three diaphragms in each span in the transverse direction, at the center, at the start and at the end of each span. The geometric and reinforcement details of the bents and piles are typical for the whole bridge. Piles are 18000 mm deep in the soil. All the material properties of different components are the same as other bridges (Mingora and Chakrisar Bridges). The reinforcement in the piers is 1.1% of the gross cross-sectional area of the piers. Girders are supported on rectangular laminated 280×480×45 mm elastomeric bearing pads. The geometrical parameters of the bridges are recorded in Table 2. The details about the three bridges are given in Fig. 1.

3. Analytical models

Bridge response in orthogonal directions and variation of the axial load in column bents is captured more accurately in Three Dimensional (3D) model that enables to correctly evaluate capacity and ductility of the bridge (Aviram *et al.* 2008). The 3D model also captures the mass distribution and geometrical characteristics of the bridge. Therefore, the 3D models of the bridges are developed to capture the global response of the entire bridge as whole and local response of the important individual bridge members. The finite models of the bridges are generated in MIDAS Civil (2015) which will be here in after referred as MIDAS.

Beam-column (frame) elements are used to model the pier, cap-beam and girders of the bridges. Plate elements are used to model the deck slab. The connection elements between the superstructure and substructure i.e., bearing pads and shear keys are modelled as non-linear elements

The expansion joints, abutments and the piers are also modelled as non-linear elements. Bearing pads, abutments are modelled with non-linear spring elements.

Expansion joints are modelled with the Gap elements in the MIDAS. Superstructure is modelled linear elastic components. Bearing pads are modelled as non-linear springs following elastic-plastic force displacement constitutive law. They are assumed to fail either in shear at 1.2-1.8 times G (i.e., G is shear modulus of rubber) according to Tortolini *et al.* (2011) or when the dynamic frictional force between neoprene and contact surface is reached assuming resulting sliding failure

The sliding friction failure is assumed for the bearing pad; once the friction force between the girders and bearing pads is exceeded, the bearing pads sliding failure is considered. The frictional force response is modelled following seismic design criteria of California State Department of Transportation, 2006 (CALTRANS 2006) recommendations. According to CALTRANS (2006) the co-efficient is taken to be 0.40 times of the dead load acting over the bearing pad.

The bearing pads are assigned three translational stiffness values and zero rotational stiffness. Horizontal stiffness values of the bearings are computed using the relationship proposed by Yazdani *et al.* (1996) given by Eq. (1).

$$K_H = {G \times A_b}/{h} \tag{1}$$

Where, G is the shear modulus of rubber (Neoprene), A_b represents area subjected to shear and h is the total thickness of neoprene excluding the steel plate thickness in the bearing. A_b is taken as the plan area of the bearing pad. In the vertical direction, a high stiffness value is assumed i.e., 100 times the horizontal stiffness expressed by Eq. (2).

$$K_V = 100 \times K_H \tag{2}$$

The shear modulus of neoprene is assumed to be 1 Mega Pascal (MPa). Mathematically, shear stress and shear failure force in bearing pads is given by Eqs. (3)-(4)

$$\tau = 1.2 - 1.8 \times G \tag{3}$$

$$V_F = \tau \times A_b \tag{4}$$

Where A_b is the area of the bearing subjected to shear V_F. The properties of bearing pads are reported in Table 3. Shear keys are modelled using criteria of sliding shear

Table 3 Bearing pads properties

S.No	Bridge Name	Bearing pad Area (Ab, mm^2)	Thickness of the bearing pad (ht, mm)	Neoprene thickness (h, mm)	Horizontal stiffness (K_H, kN/m)	Failure force (V_F, kN)
1	Mingora Bridge	400×550	46	38	5789	126
2	Badeen Bridge	280×480	45	30	4480	101
3	Chakrisar Bridge	375×400	54	38	3947	162

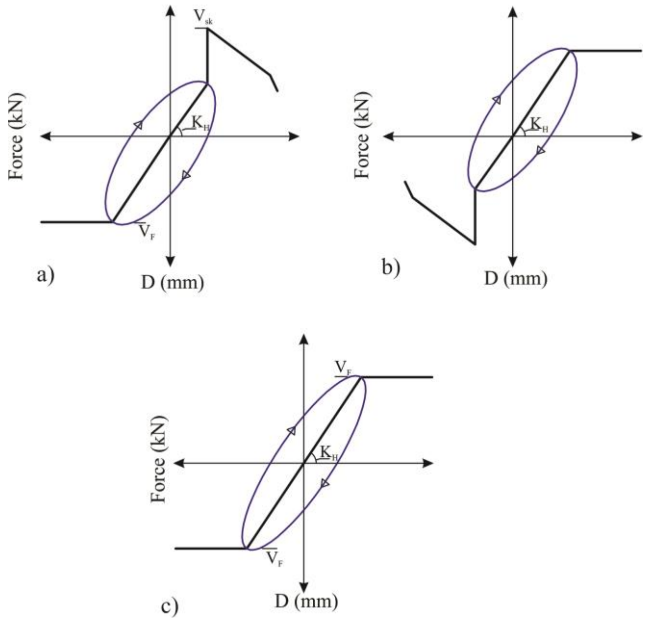


Fig. 2 Analytical laws for bearing pads (a)-(b) transverse (c) longitudinal directions

failure proposed by Prestiely *et al*. (1996) given in Eq. (5).

$$V_{sk} = \phi_s \times \mu \times f_Y \tag{5}$$

Where, φ_s is reduction factor value equal to 0.85 and μ is the friction co-efficient, A_s is the total area of steel crossing the critical section and f_Y is the yield stress of the reinforcement.

Bearing pads behave elastic perfectly plastic in the longitudinal direction of bridge and multi-linear in the transverse direction of bridge. Expansion joints are modelled with the Gap element. High value of stiffness is assigned to the Gap elements. The stiffness value is assumed one hundred times stiffness of bearing pad in horizontal direction of the bridge.

Abutments are modelled according to the recommendations of CALTRANS (2006) having elastic perfectly plastic behavior. The passive pressure (P_{bw}) expressed as Eq. (6) that develops behind the abutment is proposed by CALTRANS equation 7.44. A maximum value is suggested by CALTRANS (2006) and it is proportioned to height of abutment. The maximum pressure is 5 ksf (239 kPa) expressed in Eq. (7). The initial stiffness was obtained from force deflection tests of large scale abutment at University of California, Davis and taken as 20 kip/in /ft or 11.5 kN / mm/ m proportioned to the height of the back wall following CALTRANS recommendations.

$$P_{bw} = A_e \times 239(kPa) \times \frac{h_{bw}}{1.7} \tag{6}$$

$$K_{abut} = K_i \times w \times \frac{h}{1.7} \tag{7}$$

Where, w is the width of the back wall/ diaphragm respectively for seat and diaphragm abutments. K_i is the initial stiffness value. The typical analytical model adopted for bridges is shown in Fig. 3.

A shear spring has been considered coupled to piers at the bottom following properties proposed in FEMA 356. The results of this combination are observed to be the similar to that of shear force results in piers modelled without the shear spring in the MIDAS.

4. Input ground motion

Non-linear dynamic analysis and incremental dynamic are carried out to estimate response of the bridges. Ground motion map corresponding to 10 % of probability in 50 years by Waseem (2016) is used. The ground motion values suggested by Waseem (2016) are for flat rock sites having shear wave velocity of 800 m/s. Acceleration time histories records of earthquakes for north Pakistan do not exist in abundance or in sufficient, therefore real acceleration time histories records are obtained from Pacific Earthquake Engineering Center Databank and used to perform the analyses. The time histories are matched to 5% damped response spectra for 475 years return period at the bridge site representing the hazard. Eurocode 8 (EC 8) response spectrum for 5% damping for type B soil is used for scaling of the time histories. The matching is done between 0.2T-2T periods range for each bridge in SeismoMatch (2014).

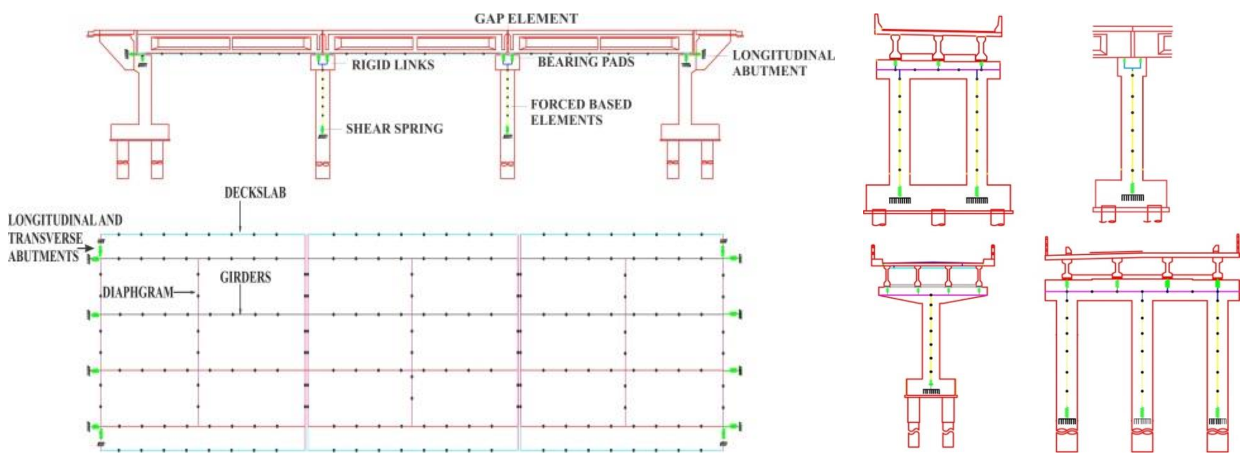
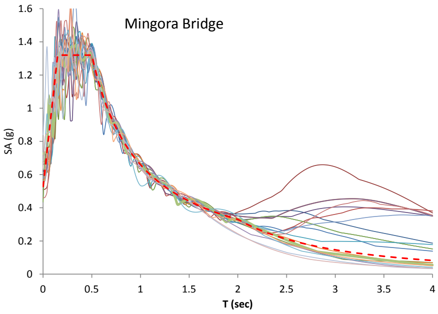


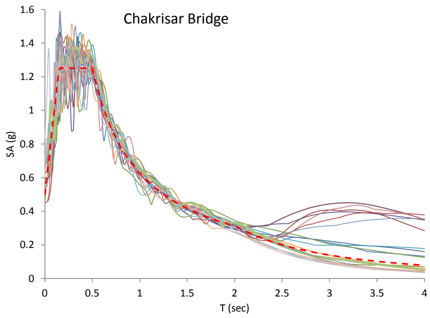
Fig. 3 Typical finite element model of bridge considered in the analysis

Table 4 Earthquake records used in analysis

S.No	Time History	Earthquake	Station	M_w	Distance (Km)
1	TH1	Imperial Valley, 1979	El Centro Array # 4	6.53	7.05
2	TH2	Imperial Valley, 1979	El Centro Array # 5	6.53	3.95
3	TH3	Imperial Valley, 1979	El Centro Differential Array	6.53	5.09
4	TH4	Imperial Valley, 1979	El Centro Array # 8	6.53	3.86
5	TH5	Imperial Valley, 1979	Hotville Post Office	6.53	7.50
6	TH6	Chuestsu-oki, 2007	Joestsa Kakizakika	6.80	11.94
7	TH7	San Fernando,1971	Pacoima Dam	6.61	1.81
8	TH8	Duze, 1999	Bolu	7.14	12.02
9	TH9	Imperial Valley, 1979	El Centro Meloland Geot. Array	6.53	0.07
10	TH10	Imperial Valley, 1979	El Centro Array # 6	6.53	1.35



(a)



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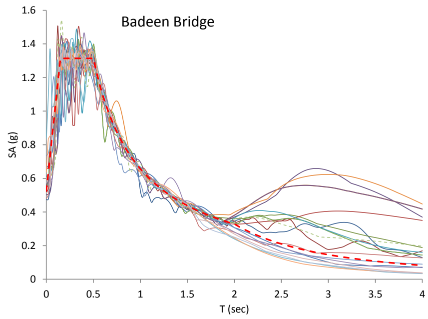

(d)

Fig. 4 Time history records for (a) Mingora Bridge (b) Chakrisar Bridge (c) Badeen Bridge

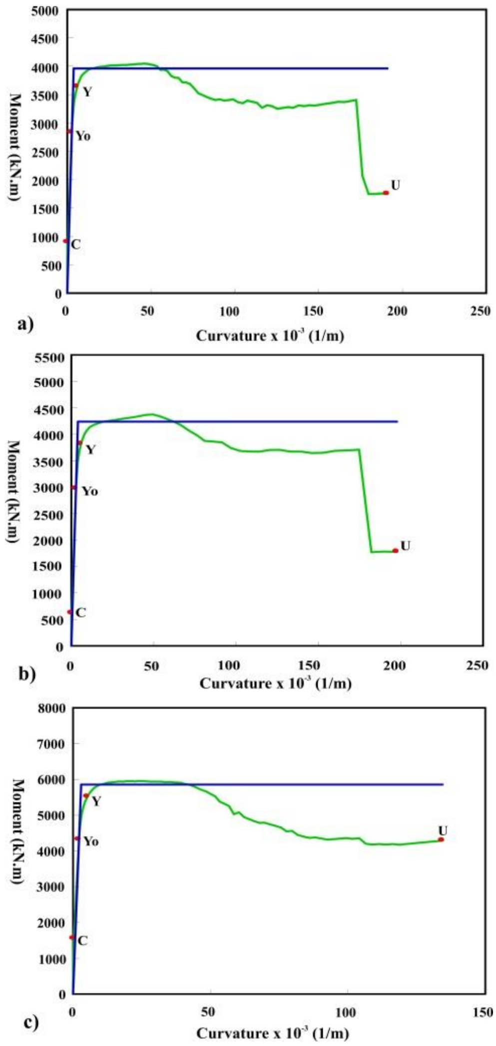

Fig. 5 Moment curvature plots (a) Mingora Bridge (b) Chakrisar Bridge (c) Badeen Bridge

The site response spectra and the average response spectrum due to the time histories is computed and shown in the Fig. 4. The details of the earthquake time histories are given in the Table 4. The moment magnitude of the selected time histories is 6.53-7.14 recorded between 0-12 Km distances. Non-linear dynamic analysis is performed using these ten time histories suite.

5. Non-linear dynamic time history analysis

Non-linear dynamic time history analysis using the selected ten accelerograms suite and the analytical model of bridges is performed in MIDAS. Eigen value analysis is carried out to find the fundamental periods of vibrations. Mingora Bridge has fundamental period of 0.53 secs, while Chakrisar and Badeen Bridges have 0.62 and 0.59 secs respectively. Moment curvature analysis for piers having maximum axial dead load is carried out to determine the yield moment and determine the yield and ultimate curvature for the three bridges. The moment curvature plots of the central bent pier are also recorded and are shown in the Fig. 5. In Eq. (8)-Eq. (10) these parameters are reported for Mingora, Chakrisar and Badeen Bridges respectively.

$$M_y = 3959.5kN - m; \phi_y = 0.0038(1/m); \quad \phi_u = 0.190(1/m); P = 1345kN \tag{8}$$

$$M_y = 4238.5kN - m; \quad \phi_y = 0.0040(1/m); \quad \phi_u = 0.192(1/m); P = 1173.1kN \tag{9}$$

$$M_y = 5853.9kN - m; \quad \phi_y = 0.0029(1/m); \quad \phi_u = 0.1344(1/m); P = 2492kN \tag{10}$$

The time histories are applied in the horizontal orthogonal directions. Newmark's constant acceleration algorithm which is insensitive to the time step is used in the analysis to capture the response of the bridges. A constant Rayleigh damping value of 2 % is assumed proportional to the first two fundamental modes is used. Conversion energy based criteria is used for the solution.

The results of non-linear dynamic analysis for the three bridges are recorded for bearing pads, shear keys, and piers. The shear capacity is computed using ACI 318-2005 given by Eq. (11).

$$\begin{aligned} V_n &= V_c + V_s \\ V_c &= 0.166 \times [1 + \frac{P}{13.8 * A_g}] \times \sqrt{f_c} \times bd(MPa) \\ V_s &= A_v \times f_Y \times d/s \end{aligned} \tag{11}$$

In the above equations V_c is shear capacity component provided by concrete, V_s is the shear capacity of shear reinforcement, P is the axial load, A_g is the gross area of the section, b is the section width, f_y is yield stress of the spirals or hoops, d is the effective depth of the section and s is the spacing of spirals or hoops in columns. The results for the Mingora Bridge are shown in Table 5, Chakrisar Bridge in Table 6 and Badeen Bridge in Table 7.

In the case of three bent bridge (Mingora Bridge) failure of bearing pads, shear keys, expansion joints measured as pounding phenomena and failure of piers is observed. All the bearing pads and shear keys are failing (i.e., 100%) in each analysis case of non-linear dynamic analysis while failure of all piers is observed (i.e., 100%) in the seven cases only. This type of bridge is expected to have failure of bearing pads, shear keys and to develop pounding phenomena and failure of piers resulting in collapse of the bridge in the event of the ground motions. As the results suggest the failure of bridge due to shear in pier will be very abrupt. The Chakrisar Bridge response obtained through non-linear dynamic analysis suggests that bearing pads, shear keys and closure of the expansion joints (pounding phenomena) and failure of piers in shear is expected. Failure of all the bearing pads, shear keys and the pounding phenomena is observed (i.e., 100%) in each case of non-linear analysis while failure of all piers is observed in the eight cases of the analysis only. These types of bridges are also expected to collapse. The response of Chakrisar Bridge and Mingora Bridge is identical.

In case of single pier bent bridge (Badeen Bridge) pounding phenomena is observed in every time history analysis. Bearing pads and shear keys failures are also observed in each case but not all the bearing pads and shear keys reach to failure. The piers failure of this bridge is controlled by the drift (flexure). The maximum drift

Table 5 Components of Mingora Bridge reaching to ultimate limit state in non-linear time history analysis

S.No	Description	TH1	TH2	TH3	TH4	TH5	TH6	TH7	TH8	TH9	TH10
1	Bearing pads	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
2	Shear keys	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
3	Expansion joints	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
4	Piers(Shear failure)	100 %	100 %	100 %	66 %	84%	100 %	100 %	50%	100 %	100 %

Table 6 Components of Chakrisar Bridge reaching to ultimate limit state in non-linear time history analysis

S.No	Description	TH1	TH2	TH3	TH4	TH5	TH6	TH7	TH8	TH9	TH10
1	Bearing pads	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
2	Shear keys	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
3	Expansion joints	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
4	Piers (Shear failure)	100 %	100 %	100 %	66 %	84%	100 %	100 %	50%	100 %	100 %

Table 7 Components of Badeen Bridge reaching to ultimate limit state in non-linear time history analysis

S.No	Description	TH1	TH2	TH3	TH4	TH5	TH6	TH7	TH8	TH9	TH10
1	Bearing pads	88%	97%	100%	88%	97%	97%	88%	88%	85%	72%
2	Shear keys	50%	40%	56%	47%	47%	22%	47%	06%	41%	28%
3	Expansion joints	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
4	Piers(max drift)	3.2%	2.5 %	2.0 %	2.0 %	4.5%	3.7 %	3.7%	2.2%	2.1 %	2.2 %

observed in piers is 4.51%. The results of time history analysis are reported in Table 7.

6. Fragility curves

Fragility curves are important tools that can be used in both pre-event and post-event earthquake phase for probabilistic based performance assessment of bridges. These curves are derived either empirically or analytically. Empirical curves are developed from the reported damages to bridges (Choi *et al*. 2004).

Fragility curves express the conditional probability of reaching or exceeding a given limit state for a given ground motions developed using incremental dynamic analyses based on a suite of ground motions. They serve as tool for seismic vulnerability assessment and decision making for the structures. The fragility curves are derived for a given limit state considering a standard cumulative density function based on the logarithmic difference of intensity and at threshold intensity for the given limit state with certain standard deviation (Ahmad *et al*. 2014).

Analytical fragility curves are derived in this study and the fragility curve formulation used in derivation is given by Kircher *et al*. (1997) shown in Eq. (12).

$$P_f = (D \geq d_{LS} / IM = im) = \phi.((\frac{1}{\beta}) \times \ln(\frac{IM}{im_{LS}})) \tag{12}$$

Where, P_f is the probability of reaching or exceeding a given limit state d_{LS}. $\emptyset$ is representing the standard normal distribution; β is the total uncertainty in the function related to uncertainties in input ground motions, material properties, bridge response, damage state etc. (Cardone *et al*. 2011) and im_{LS} is the intensity measure (mean collapse capacity).

Probabilistic Non-linear Dynamic Reliability based method (NDRM) proposed by Ahmed *et al*. (2014), has been used for deriving fragility curves in the present study. The NDRM method is based on the incremental dynamic analysis procedure of Vamvatsikos and Cornell (2002) and the first order reliability method (FORM) approximation developed by Der Kiureghian, 2005 for the estimation of probability of exceedance of a given limit state of structure components. Incremental dynamic analysis consist of performing the non-linear dynamic analysis using suite of ground motion records scaled to different intensity levels to force the structure to collapse (Mander *et al*. 2006). The bridge site hazard consistent records used for non-linear dynamic analysis are used in the incremental dynamic analysis. Every record is scaled up and down by different factors and non-linear time history analysis was carried out to record different Engineering Demand Parameters (EDP's).

Engineering Demand Parameters and the selected Intensity Measuring (IM) parameter are plotted together to get the capacity curves.

The present study considers the maximum displacement of the bearing pads, shear keys and maximum drift (%) of the piers and maximum shear force in the piers as EDP's. Spectral acceleration corresponding to the first mode fundamental of bridges is selected as the ground motion intensity measuring parameter.

The demand and capacity are convoluted and this convolution is performed using classical reliability approach of first order reliability (FORM) approximations. If R is a random variable resistance and S is random variable load failure is of any element is considered for $R < S$. The probability of failure can be obtained by the following Eq. (13)

$$P_f = \int_0^{\infty}[1 - F_S(S/_r)](f(r)d_r \tag{13}$$

Where, $F_S\,(s/r)$ is the cumulative density function and $f(r)$ is the probability density function of R. The failure probability can be computed by Eq. (14)

$$P_f = \phi.(-\beta) \tag{14}$$

Where $\emptyset$ standard normal distribution is function and β is called reliability index. If the R and S can be considered log normally distributed. The value of β can be approximated by Eq. (15)

$$\beta = \frac{\lambda_R - \lambda_S}{\sqrt{\zeta_R{}^2 + \zeta_S{}^2}} \tag{15}$$

Where,

$$\lambda_R = \ln(\mu_R) - 0.5\zeta^2$$

$$\zeta = \sqrt{\ln(1 + \delta_R^2)}$$

$$\delta_{R=}\frac{\sigma_R}{\mu_R}$$

μ_R is the mean value of R and δ_R the standard deviation of R. The mean collapse capacities were computed using the above formulation.

In addition to the above procedure mean collapse capacities are also computed by the formulation in Eq. (16)

$$P_f(X \geq d_{LS}) = \phi \times (\frac{Ln(X) - \mu}{\sigma}) \tag{16}$$

Where,

$\emptyset$ = Standard normal or Lognormal distribution

X= Engineering Demand Parameter

$\mu = Median\ value\ of\ X$

σ = Standard deviation

The mean collapse capacities (50%) computed by the two methods came out to be similar. The curve fitting has been performed using mean collapse capacity values and β value equal to 0.6 for existing bridges based on the recommendations of similar studies (e.g., Dutta and Mander 1998, Basöz and Mander 1999, Kappos and Paraskeva 2008, Paraskeva and Kappos 2010). A lower value of β can also be assumed when the structural properties are accurately known for existing bridges (Cardone *et al*. 2011).

The intensity values (spectral acceleration corresponding to the fundamental mode of the bridge) corresponding to 50% and 100% capacities of the different components of the Mingora Bridge, Chakrisar and Badeen Bridge bridges are recorded at Table 8, Table 9 and Table 10.

The shear key and expansion joint reach failure first

Table 8 Intensity values for 50 and 100 % probability of failure (Mingora Bridge)

Mingora Bridge (Three Piers Bent Bridge)				
S.No	Description	*SA* (g) for P_f (50%)	*SA* (g) for P_f (100%)	Hazard at site (g)
1	Failure of the first bearing pad	0.220	1.030	1.2422
2	Failure of the first shear key	0.190	0.610	1.2422
3	Failure of first expansion joint	0.190	0.610	1.2422
4	Pier failure (Shear failure)	0.940	3.020	1.2422
5	Failure of all bearing pads	0.680	2.180	1.2422

Table 9 Intensity values for 50 and 100 % probability of failure (Chakrisar Bridge)

Chakrisar Bridge (Two Piers Bent bridge)				
S.No	Description	*SA* (g) for P_f (50%)	*SA* (g) for P_f (100%)	Hazard at site (g)
1	Failure of the first bearing pad	0.280	0.900	1.008
2	Failure of the first shear key	0.156	0.400	1.008
3	Failure of first expansion joint	0.270	1.260	1.008
4	Pier failure (Shear failure)	0.601	1.920	1.008
5	Failure of all bearing pads	0.419	1.691	1.008

Table 10 Intensity values for 50 and 100 % probability of failure (Badeen Bridge)

Chakrisar Bridge (Two Piers Bent bridge)				
S.No	Description	*SA* (g) for P_f (50%)	*SA* (g) for P_f (100%)	Hazard at site (g)
1	Failure of the first bearing pad	0.215	0.650	1.1250
2	Failure of the first shear key	0.380	1.210	1.1250
3	Failure of first expansion joint	0.137	0.554	1.1250
4	Pier drift (1.0%)	0.470	1.890	1.1250
5	Pier drift (1.5%)	0.700	2.830	1.1250

then the bearing pad followed by failure of all the bearing pads and finally shear failure of the pier in the Mingora Bridge. Mean collapse capacities of these components of the Mingora Bridge are less than expected ground motion spectral value of 1.242 g.

In the case of the Chakrisar Bridge shear key reaches ultimate limit state then expansion joint and bearing pads followed by all the bearing pads then at the end pier shear failure. The expected ground motion value (corresponding to fundamental mode) is 1.0 g.

In the Badeen Bridge expansion joint first reach the ultimate limit state followed by the bearing pad, pier drift (1%), all the bearing pads and then followed by shear key, pier drift (1.5%), pier drift (2%) and pier drift (3%). Mean collapse capacities of these components are less than site

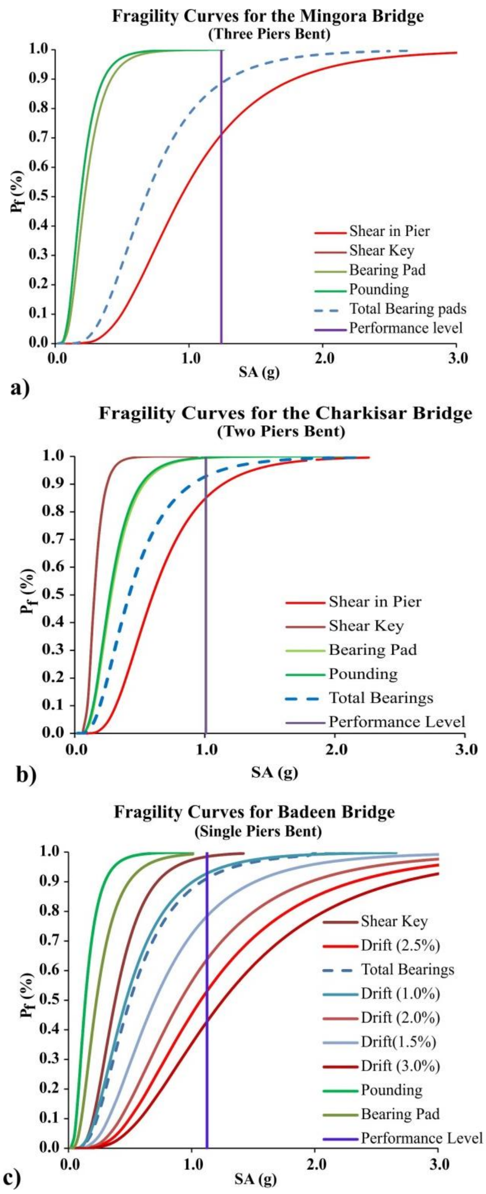

Fig. 6 Analytical fragility curves developed for (a) Mingora Bridge (b) Chakrisar Bridge (c) Badeen Bridge

hazard of 1.125 g except for 3% pier drift curve.

The fragility curves derived for different components of these bridges are shown in Fig. 6.

7. Conclusions

Seismic design code for bridges in Pakistan till date is WPBC (1967) which is obsolete. Ignorance of the seismic issues in the bridges constructed in Pakistan has made them seismically vulnerable that has to be addressed to reduce the seismic risk associated with the bridges. The WPBC (1967) code, the available design code has not been updated even after the Kashmir earthquake (2005) that brought a lot of casualties and damaged to a lot of infrastructure including bridges. Bridges are now a days being designed according different versions of AASTHO codes-and the design ground motion intensity is not specified in either AASTHO code or

WPBC (1967) for Pakistan. It has allowed the bridges to be designed for any published or arbitrary values of ground motion intensity values given in MMI or g units. After the revision of Building Code of Pakistan, bridges are probably being designed according to seismic hazard map of Building Code of Pakistan (2007). Absence of seismic design code and unified ground motion values makes bridges seismically vulnerable and require performance evaluation for expected ground motion in the region. In this study seismic performance of RC bridges is carried out using non-linear dynamic analysis and incremental dynamic analysis approaches.

The results of non-linear dynamic time history analysis carried out using suite of ten ground motion time histories suggest that the seismic response of the bridges is poor and inadequate making them vulnerable. It is observed that both local failure (e.g., bearing pads, shear keys, pounding phenomena) and global failure (i.e., piers) in the bridges. Global failure is governed by shear failure of the piers for multi pier bent bridges (i.e., Mingora and Chakrisar Bridges) or in other words global failure for these bridges is controlled by the shear in piers while the flexure action is controlling the global failure for the single pier bridge (Badeen Bridge).

Bearing pads, expansion joints and shear keys components of the selected bridges reach to their ultimate limit state (i.e., failure) in most of the cases of analyses and can be classified as the most vulnerable components of these bridges inviting attention. The expansion joints provided in the bridges have very inadequate width and are responsible for the pounding phenomena in the superstructure. The bearing pads are failing in slip rather than the shear ultimate state. They require proper design and evaluation. All the bearing pads are observed to reach the ultimate limit state in each bridge. Shear keys which provide lateral restraints are also one the key components of the bridges. Their failure may lead to overturning of the superstructure of the bridge. Failures of the shear keys have also been observed in bridges. Not all the shear keys reach to ultimate limit state and but some have seen to reach ultimate limit state.

The incremental dynamic analysis is carried to derive the fragility curves for the bearing pads, shear keys, expansion joints and piers components. The mean collapse capacity is computed for each of component and using the NDRM methodology fragility curves were derived.

Incremental dynamic analysis has been carried out to capture the response of the bridge and failure sequence of the components of bridge. The sequence of failure of bridges is clearly defined and indicated by the derived fragility curves. The bearings pads, shear keys and pounding phenomena reach to 100 % failure before the failure of the piers indicating the local failure before the global failure of bridge.

The need of seismic design code is very essential to mitigate the seismic vulnerability of the bridges. National code for bridges should be prepared and adopted immediately including the design ground motion values map of Pakistan. The results obtained by the analyses show that ultimate limit state of the case study bridges in the case of the seismic event. The non-linear dynamic analysis seismic analysis showed the following damages mechanisms in case study bridges: bearing pads, shear keys, and piers reach to the ultimate limit (failure). The pounding at the intermediate bays and at abutments is also a common mechanism in these bridges. All the bearing pads capacities are lost in Mingora bridge (three piers bent) and Chakrisar bridge (two piers bent), but in Badeen Bridge (single pier bent) not all the bearing pads reach failure. Some but not all shear key are lost in these bridges.

Fragility curves derived for different components of the selected bridge show bearing pads, shear keys and expansion joints have least mean collapse capacities for each bridge case. The piers of bridges are observed to experience damage after these three components of the bridge have damaged

These type of bridges are seismically very vulnerable and the analyses confirms it. Therefore, proper retrofit procedure should be designed and implemented for the bridges before a major disaster takes place. Moreover, a national seismic design code for bridges should be prepared and implemented with immediate effect as done for building in 2007.

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