International Journal of Civil and Structural Engineering– IJCSE
Volume 1 : Issue 3 [ISSN 2372 – 3971]
Publication Date : 30 September, 2014

Response of a curved bridge with elastomer-based and friction-based bearings

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Abstract— **Horizontally curved bridges are important in modern highway systems and those are the most viable option at complicated interchanges or river crossings where geometric restrictions and constraints of limited site space make extremely complicated the adoption of standard straight superstructures. It has been observed that in past earthquakes, most of the damages of the bridges occurred due to the failure of the bearings and substructure. Selection of isolation bearings for a curved bridge is a demanding task because of the complexity involved in curved bridges than straight bridges. In the present study, the responses of an isolated curved continuous bridge for seismic loading conditions are determined. Two types of isolation bearings viz. one elastomer-based bearing and other friction-based bearing have been considered and the efficacy of both has been investigated.**

Keywords—Curved bridge, bearing, isolation, seismic

I. Introduction

The use of horizontally curved bridges is very important nowadays, especially to avoid a congested traffic and also to solve the limited space requirement in urban traffic conditions. The only problem with these types of bridges is the significant amount of torsion which makes it difficult for design. From the study of damages caused by past earthquakes, it has been found that the performance of bridges is generally governed by the performance of bearings and substructure. Efficacy of isolation bearings, especially in case of curved bridges, is important and selecting a proper solation bearing is also a demanding task as the performance of a particular type of bearing is affected by age, temperature, scragging, velocity, travel, contamination and level of ground movement. This paper presents a numerical study of the seismic response of a three-span continuous curved bridge with two types of isolation bearings viz. elastomer-based (Lead Rubber Bearing (LRB)) and friction-based bearing (Friction Pendulum System (FPS)). Imperial Valley earthquake ground motion (1940) has been considered in the study and relative performance and efficacy of the two isolation bearings w.r.to the selected earthquake ground motion has been determined in the study.

II. Isolation Bearings

Various types of isolation bearings are available. However, the present study is limited to a comparative assessment of the seismic performance of the two types of isolation bearings viz. elastomer-based (Lead Rubber Bearing (LRB)) and friction-based bearing (Friction Pendulum System (FPS)).

In case of Lead–rubber bearings (LRBs) (Abrahamson and Mitchell, 2003; Turkington et al., 1989), the rubber provides lateral flexibility to lengthen the period of the structure, and a lead core dissipates energy during cyclic movement due to earthquakes.Friction pendulum system (FPS) is a sliding-based seismic isolator (Dicleli, 2002; Ingham, 2003; Mokha et al., 1991; Wang et al., 1998) with a restoring mechanism. The FPS provides resistance to service load by friction. Once the coefficient of friction is overcome, an articulated slider moves over a spherical surface, which causes the supported mass to rise and provides the restoring force for the system. Friction between the articulated slider and the spherical surface generates damping. The Coulomb damping generated through sliding friction provides energy dissipation in the bearings.
The choice of bearing type in a particular situation is influenced by the cost of the bearing. According to an evaluation (Drozdov et al., 2007) of FPS bearings, LRBs and bearings containing rubber with high damping capability, for the same evels of structural displacement, the FPS bearings were found to be the cheapest.

III. Modelling and Analysis

A continuous single-chamber box girder curved bridge has been considered (Yu et al., 2008). The total length of the curved bridge is 165 m with two end span of 20 m and five intermediate spans of 25 m. The radius of curvature of the bridge is 150 m. The cross-sectional area of the box-girder is 3.1 m^2. The longitudinal moment of inertia and transverse moment of inertia of the box-girder are 0.60 m^4 and 16.58 m^4, respectively. The pier has a solid circular section with corss-sectional area of 1.7671 m^2 and moment of inertia of 0.2485 m^4. The height of the pier is 11 m. The piers are resting on rocky strata.

The structure has been modelled using the SAP2000 non-

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The present study is part of a research project sponsored by SERB, New Delhi. Thanks to SERB for their support.

linear software. The superstructure and the piers have been modelled using beam elements with mass lumped at discrete points. Since the piers are resting on rock, these have been modelled as fixed at the base. The abutments have been assumed to be rigid. The isolation bearings have been modelled as link elements. In non-isolated case, the bridge is supported by roller bearings at the abutment and fixed at the base of piers. In isolated case, isolation bearings are used both at abutments as well as between the deck and pier.

Seismic loading has been considered for Imperial Valley earthquake ground motion (1940) with two horizontal orthogonal ground motion components (Table I and Fig. 1a and Fig. 1b). The nonlinear dynamic time history of the bridge has been performed for four cases of ground motions viz. (Case 1) PGA 0.31g in global longitudinal (X) direction and PGA 0.21 g in Global transverse (Y) direction; (Case 2) PGA 0.21g in global X direction and PGA 0.31 g in Global Y direction; (Case 3) PGA 0.31g with 45^0 angle w.r.to global X direction and PGA 0.21 g with 45^0 angle w.r.to in Global Y direction; and (Case 4) PGA 0.21g with 45^0 angle w.r.to global X direction and PGA 0.31 g with 45^0 angle w.r.to in global Y direction.

TABLE I. GROUND MOTIONS CONSIDERED

Record	Event	Mag.	Station	Orientation	PGA(*g*)	Distance-to-fault (km)
1	Imperial Valley (1940)	7.0	117 El Centro Array #9	IMPVALL/I-ELC180	0.313	8.3
2	Imperial Valley (1940)	7.0	117 El Centro Array #9	IMPVALL/I-ELC270	0.215	8.3

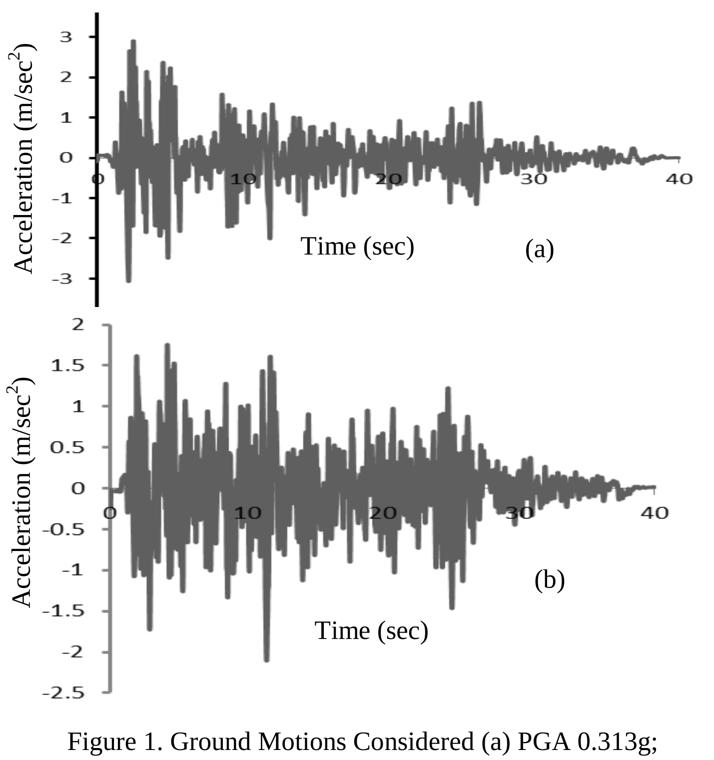

Figure 1. Ground Motions Considered (a) PGA 0.313g; (b) PGA 0.215g.

IV. Design of Isolation Bearings

The design of the isolation bearings was done according to different criteria provided in various codes (AASHTO, 999; IRC, 1987) and the literature (Dolce et al., 2007; Priestley et al., 1996). Three parameters are important for the design of the isolation bearings: time period of the isolated structure; the damping ratio of the isolation system; and the level of ground motion. In the present study, the isolation bearings have been designed for the Imperial Valley (1940) earthquake ground motion and the performance of different bearings has been compared.

V. Results and Discussions

The response of the curved bridge with the LRB and FPS bearings has been determined for four different cases as stated above in section III. The natural period of the isolated structure and damping of the isolation device are the most important parameters affecting the response of the structure. These in turn depend on size and design of the isolation bearings. In the present paper, the isolation time period and damping ratio of the isolation bearings have been considered as 1.5 sec and 20%.

Table II-XX show the results of the analyses of bridge response to bearing design parameters. The peak responses considered in the study are the maximum deck displacement, the maximum pier displacement, the maximum column shear force and the maximum column bending moment. It has been observed that the performance of FPS bearing is good as it results maxm. deck displ. 47-60% less than LRB bearing in longitudinal (x) direction and 7-34% less tha LRB bearing in transverse (y) direction.

Pier displ. of the bridge has been drastically reduced than non-isolated bridge as has been observed for most of the cases. LRB bearing resulted in upto 89% reduction in pier displ. and FPS bearing resulted in upto 69% reduction in pier displ.

Pier shear force and pier bending moment are also drastically reduced than non-isolated bridge. LRB bearing resulted in upto 87-89% reduction in pier shear force and pier bending moment and FPS bearing resulted in upto 72-74% reduction in pier shear force and pier bending moment.

In some of the cases, the pier displ, pier shear force and pier bending moment are seems to be more than that for non-isolated case, but that is within the acceptable limit. Also, if the resultant (square root of mam. of the response of x and y directions) of the pier displ., pier shear force and pier bending moment are compared w.r.to the same with non-isolated cases, it would have been observed that the resultant responses are much lesser as compared to the same for non-isolated cases. This is because, the considered bridge is a curved bridge and the resultant of the response of x and y directions can be more effective from design consideration of bridge.

TABLE II. MAXM. DECK DISPL OF CURVED BRIDGE

Loading	Deck Displ. X (m)			Deck Displ. Y (m)		
	Non-isolated	Isolated (LRB)	Isolated (FPS)	Non-isolated	Isolated (LRB)	Isolated (FPS)
Case 1	0.043	0.058	0.023	0.063	0.062	0.053
Case 2	0.026	0.047	0.025	0.073	0.068	0.045
Case 3	0.038	0.063	0.027	0.043	0.043	0.040
Case 4	0.037	0.063	0.027	0.043	0.043	0.040

TABLE III. MAXM. PIER(1)* DISPL. OF CURVED BRIDGE

Loading	Pier 1 Displ. X (m)			Pier 1 Displ. Y (m)		
	Non-isolated	Isolated (LRB)	Isolated (FPS)	Non-isolated	Isolated (LRB)	Isolated (FPS)
Case 1	0.039	0.007	0.014	0.011	0.012	0.020
Case 2	0.034	0.007	0.012	0.014	0.013	0.019
Case 3	0.032	0.006	0.016	0.009	0.010	0.019
Case 4	0.042	0.007	0.013	0.007	0.010	0.020

TABLE IV. MAXM. PIER(2) DISPL. OF CURVED BRIDGE

Loading	Pier 2 Displ. X (m)			Pier 2 Displ. Y (m)		
	Non-isolated	Isolated (LRB)	Isolated (FPS)	Non-isolated	Isolated (LRB)	Isolated (FPS)
Case 1	0.043	0.007	0.015	0.038	0.016	0.027
Case 2	0.029	0.006	0.012	0.046	0.017	0.026
Case 3	0.036	0.006	0.015	0.029	0.013	0.025
Case 4	0.039	0.007	0.013	0.026	0.013	0.025

TABLE V. MAXM. PIER(3) DISPL. OF CURVED BRIDGE

Loading	Pier 3 Displ. X (m)			Pier 3 Displ. Y (m)		
	Non-isolated	Isolated (LRB)	Isolated (FPS)	Non-isolated	Isolated (LRB)	Isolated (FPS)
Case 1	0.043	0.006	0.015	0.060	0.017	0.033
Case 2	0.026	0.005	0.011	0.069	0.018	0.032
Case 3	0.038	0.006	0.015	0.041	0.014	0.029
Case 4	0.038	0.006	0.014	0.041	0.013	0.029

TABLE VI. MAXM. PIER(4) DISPL. OF CURVED BRIDGE

Loading	Pier 4 Displ. X (m)			Pier 4 Displ. Y (m)		
	Non-isolated	Isolated (LRB)	Isolated (FPS)	Non-isolated	Isolated (LRB)	Isolated (FPS)
Case 1	0.043	0.005	0.014	0.059	0.017	0.032
Case 2	0.026	0.005	0.010	0.069	0.015	0.031
Case 3	0.038	0.006	0.014	0.041	0.014	0.029
Case 4	0.037	0.006	0.015	0.041	0.014	0.029

TABLE VII. MAXM. PIER(5) DISPL. OF CURVED BRIDGE

Loading	Pier 5 Displ. X (m)			Pier 5 Displ. Y (m)		
	Non-isolated	Isolated (LRB)	Isolated (FPS)	Non-isolated	Isolated (LRB)	Isolated (FPS)
Case 1	0.045	0.005	0.014	0.037	0.015	0.025
Case 2	0.027	0.005	0.010	0.046	0.015	0.023
Case 3	0.039	0.007	0.013	0.026	0.013	0.025
Case 4	0.036	0.006	0.016	0.029	0.013	0.025

TABLE VIII. MAXM. PIER(6) DISPL. OF CURVED BRIDGE

Loading	Pier 6 Displ. X (m)			Pier 6 Displ. Y (m)		
	Non-isolated	Isolated (LRB)	Isolated (FPS)	Non-isolated	Isolated (LRB)	Isolated (FPS)
Case 1	0.047	0.006	0.016	0.011	0.011	0.017
Case 2	0.031	0.005	0.010	0.015	0.012	0.017
Case 3	0.042	0.007	0.013	0.007	0.010	0.020
Case 4	0.032	0.006	0.016	0.009	0.010	0.019

TABLE IX. MAXM. PIER(1) SHEAR FORCE OF CURVED BRIDGE

Loading	Pier 1 Shear Force X (kN)			Pier 1 Shear Force Y (kN)		
	Non-isolated	Isolated (LRB)	Isolated (FPS)	Non-isolated	Isolated (LRB)	Isolated (FPS)
Case 1	1577	262	488	192	218	335
Case 2	1519	222	400	314	249	319
Case 3	1352	284	577	212	212	336
Case 4	1855	276	525	208	201	326

TABLE X. MAXM. PIER(2) SHEAR FORCE OF CURVED BRIDGE

Loading	Pier 2 Shear Force X (kN)			Pier 2 Shear Force Y (kN)		
	Non-isolated	Isolated (LRB)	Isolated (FPS)	Non-isolated	Isolated (LRB)	Isolated (FPS)
Case 1	1806	268	528	671	211	290
Case 2	1450	223	433	811	238	290
Case 3	1515	283	586	481	203	293
Case 4	1874	277	549	510	191	275

TABLE XI. MAXM. PIER(3) SHEAR FORCE OF CURVED BRIDGE

Loading	Pier 3 Shear Force X (kN)			Pier 3 Shear Force Y (kN)		
	Non-isolated	Isolated (LRB)	Isolated (FPS)	Non-isolated	Isolated (LRB)	Isolated (FPS)
Case 1	1923	269	574	1048	208	294
Case 2	1269	223	462	1154	231	325
Case 3	1660	280	582	707	199	285
Case 4	1759	279	568	683	194	280

*Pier Numbering is 1 to 6 from staring after left abutment and ending before right abutment.

International Journal of Civil and Structural Engineering– IJCSE
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TABLE XII. MAXM. PIER(4) SHEAR FORCE OF CURVED BRIDGE

Loading	Pier 4 Shear Force $_X$ (kN)			Pier 4 Shear Force $_Y$ (kN)		
	Non-isolated	Isolated (LRB)	Isolated (FPS)	Non-isolated	Isolated (LRB)	Isolated (FPS)
Case 1	2014	270	609	1061	205	297
Case 2	1188	224	482	1143	232	329
Case 3	1767	279	569	685	195	280
Case 4	1652	281	582	705	198	285

TABLE XIII. MAXM. PIER(5) SHEAR FORCE OF CURVED BRIDGE

Loading	Pier 5 Shear Force $_X$ (kN)			Pier 5 Shear Force $_Y$ (kN)		
	Non-isolated	Isolated (LRB)	Isolated (FPS)	Non-isolated	Isolated (LRB)	Isolated (FPS)
Case 1	2096	270	631	679	204	299
Case 2	1307	224	492	794	232	291
Case 3	1875	277	550	490	192	275
Case 4	1517	283	586	470	203	291

TABLE XIV. MAXM. PIER(6) SHEAR FORCE OF CURVED BRIDGE

Loading	Pier 6 Shear Force $_X$ (kN)			Pier 6 Shear Force $_Y$ (kN)		
	Non-isolated	Isolated (LRB)	Isolated (FPS)	Non-isolated	Isolated (LRB)	Isolated (FPS)
Case 1	2028	266	629	276	209	292
Case 2	1331	224	485	310	244	320
Case 3	1847	275	526	234	200	325
Case 4	1348	284	576	231	212	277

TABLE XV. MAXM. PIER(1) BENDING MOMENT OF CURVED BRIDGE

Loading	Pier1 Bending Moment $_X$ (kN-m)			Pier1 Bending Moment $_Y$ (kN-m)		
	Non-isolated	Isolated (LRB)	Isolated (FPS)	Non-isolated	Isolated (LRB)	Isolated (FPS)
Case 1	1748	2005	3017	9592	1578	3039
Case 2	2374	2156	2949	8934	1432	2579
Case 3	1173	1692	2949	7989	1585	3347
Case 4	1277	1640	3011	10961	1681	2983

TABLE XVI. MAXM. PIER(2) BENDING MOMENT OF CURVED BRIDGE

Loading	Pier2 Bending Moment $_X$ (kN-m)			Pier2 Bending Moment $_Y$ (kN-m)		
	Non-isolated	Isolated (LRB)	Isolated (FPS)	Non-isolated	Isolated (LRB)	Isolated (FPS)
Case 1	6134	2341	3260	10768	1555	3203
Case 2	7645	2493	3248	8142	1386	2688
Case 3	4360	1937	3115	9098	1544	3315
Case 4	4141	1886	3118	10681	1642	3060

TABLE XVII. MAXM. PIER(3) BENDING MOMENT OF CURVED BRIDGE

Loading	Pier3 Bending Moment $_X$ (kN-m)			Pier3 Bending Moment $_Y$ (kN-m)		
	Non-isolated	Isolated (LRB)	Isolated (FPS)	Non-isolated	Isolated (LRB)	Isolated (FPS)
Case 1	9812	2466	3420	11278	1477	3286
Case 2	11277	2567	3518	7199	1312	2717
Case 3	6668	2004	3221	9739	1539	3245
Case 4	6617	1951	3214	10090	1581	3155

TABLE XVIII. MAXM. PIER(4) BENDING MOMENT OF CURVED BRIDGE

Loading	Pier4 Bending Moment $_X$ (kN-m)			Pier4 Bending Moment $_Y$ (kN-m)		
	Non-isolated	Isolated (LRB)	Isolated (FPS)	Non-isolated	Isolated (LRB)	Isolated (FPS)
Case 1	9755	2400	3397	11613	1415	3322
Case 2	11202	2499	3453	6841	1245	2721
Case 3	6629	1955	3217	10133	1580	3155
Case 4	6654	2002	3218	9696	1540	3246

TABLE XIX. MAXM. PIER(5) BENDING MOMENT OF CURVED BRIDGE

Loading	Pier5 Bending Moment $_X$ (kN-m)			Pier5 Bending Moment $_Y$ (kN-m)		
	Non-isolated	Isolated (LRB)	Isolated (FPS)	Non-isolated	Isolated (LRB)	Isolated (FPS)
Case 1	6171	2227	3148	12060	1393	3367
Case 2	7411	2338	3052	7439	1206	2729
Case 3	4130	1881	3123	10699	1641	3058
Case 4	4393	1938	3114	9090	1540	3315

TABLE XX. MAXM. PIER(6) BENDING MOMENT OF CURVED BRIDGE

Loading	Pier6 Bending Moment $_X$ (kN-m)			Pier6 Bending Moment $_Y$ (kN-m)		
	Non-isolated	Isolated (LRB)	Isolated (FPS)	Non-isolated	Isolated (LRB)	Isolated (FPS)
Case 1	1779	1900	2759	12076	1425	3421
Case 2	2269	2062	2847	7912	1236	2733
Case 3	1333	1645	3013	10929	1670	2992
Case 4	1202	1697	2957	7966	1589	3339

VI. Conclusions

The efficacy of an elastomer-based (LRB) and a friction-based FPS) isolation bearing for a curved bridge has been investigated. It has been found that the FPS system is more effective in reducing maxm. deck displ. of the bridge than LRB system. . Both of the isolation bearings are effective in reducing pier displ. and pier forces. However, LRB system is more effective than FPS system in this case. Based on the present study, a general conclusion on the selection of the isolation bearing cannot be drawn as that involves a lot more investigation. Within the scope of the present study, valid for the considered ground motion only, any of the above systems can be used depending on the availability of skill and cost considerations for local conditions.

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