EFFECT OF NON-CONVENTIONAL BEARINGS ON THE EARTHQUAKE RESPONSE OF BRIDGES

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SYNOPSIS

The rocker-roller arrangement conventionally used for supporting bridge spans imposes large inertia force on the superstructure horizontally due to the rigid link between superstructure and substructure at the rocker bearing. This force acts eccentrically on the superstructure and may produce in it excessive vertical accelerations. Possibility of reduction in the inertia force has been investigated by providing both end bearings of sliding or rolling type together with external dampers and springs. A simple mathematical model has been developed for girder bridges. The results show that substantial reduction in inertia loads is possible by a suitable combination of rollers and dampers.

INTRODUCTION

Multispan beam and truss bridges with simply supported spans upto about 120 m are most commonly built in India. Conventionally the spans are supported through rocker and roller bearings at the two ends except for small spans upto about 20 m which often have sliding bearings at both ends. The behaviour of such bridges during earthquakes has indicated large forces on the rocker bearings and excessive rolling on the roller bearings causing damage to bearings, superstructure and substructure. Large vertical accelerations in the superstructure are also indicated from such behaviour as well as from observations on laboratory model tests even when the predominent base motion is horizontal, possibly due to the eccentricity of the large horizontal, force at the rocker bearing.

The aim of this paper is to investigate the extent to which the large horizontal inertia force can be reduced by replacing the rocker bearing by a roller bearing, and adding external spring and damper devices to prevent excessive rolling. A methematical model h-as been developed for girder bridges with various types of bearings and the results compared to see the effectiveness of unconventional bearings.

MATHEMATICAL MODEL OF THE BRIDGE

A suitable model of the bridge for the earthquake motion applied in the longitudinal direction is shown in Fig 1a and the forces acting on various components are marked in Fig 1b. A single span of a multispan bridge has been considered and the effect of adjoining spans has been accounted by substituting the effective masses contributed by them. Such an assumption is valid due to the weak coupling that exists at the roller bearings. Same assumption has been made for sliding bearings as well, although the values of effective masses are different. The pier-foundation systems at the two ends are represented by single degree mass-spring-dashpot systems of unequal magnitudes, assuming that their higher modes have periods much smaller than 0.1 sec and consequently insignificant

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effect on the response. The superstructure is substituted by an equivalent beam of uniform section. The earthquake force transmitted to the girders through the bearings being eccentric, the motion of girders has been considered in the vertical plane and described by its first symetric and first antisymmetric modes of vibration. The rocker bearing is represented by a hinge and the roller or sliding bearing by Coulomb resistance. Axial deformations have been neglected. The equations of motion are coupled and their solution is non-linear.

Equations of Motion for the Girder

Moments at the ends of the girder are caused due to eccentricity of the horizontal forces. These end moments are replaced by combination of a symmetric component M_1 and an antisymmetric component M_2 at each instant of time. The mode shapes are assumed as the static deflection curves for these components.

Expressions for deflections, mode shapes, frequencies and equivalent quantities are given in table 1 for the two modes considered. Displacement responses x_3 and x_4 have been defined at the half-span and quarter-span points respectively in the two modes. The equivalent masses, stiffnesses and forces are obtained with respect to these points. Equations of motion for the two modes are then obtained as

$$x_3 + 2p_3 \zeta_3 x_3 + p^2 x_3 = 12M_1/m_oL \quad \ldots (1)$$

$$x_4 + 2p_4 \zeta_4 x_4 + p_4^2 x_4 = 23.625\, M_2/m_oL \quad \ldots (2)$$

where $m_o = mL$, $\zeta_3 = c_3/2m_ep_3$, $\zeta_4 = c_4/2m_ep_4$

Vertical reactions at the bearing are

$$R_1 = \tfrac{1}{2} m_og - \frac{32EIx_3}{L^3} + \frac{2M_2}{L} + \frac{268.8EIx_4}{L^3} \quad \ldots (3)$$

$$R_2 = \tfrac{1}{2} m_og - \frac{32EI}{L^3}x_3 - \frac{2M_2}{L} + \frac{268.8EIx_4}{L^3} \quad \ldots (4)$$

Also for horizontal equilibrium of the span

$$H_1 + H_2 = m_ox_2 + c_o(x_2 - x_1) + k_o(x_2 - x_1) \quad \ldots (5)$$

Equations of Motion for Pier-Foundation System

The equivalent mass of the pier-foundation system is lumped at the pier top level. Effective mass of the adjacent span is also added to it. A distribution of the superstructure mass in the ratio of 20% and 80% on expansion and fixed bearings respectively[1] has been suggested, so that if both adjacent spans have equal masses m_o, the lumped masses are obtained as $m'_1 = m_1 + 0.8\, m_o$, $m'_2 = m_2 + 0.2\, m_o$. For both end roller bearings and both end sliding bearings the effective lumped masses have been assumed as 20% and 40% respectively of the superstructure mass. Equations of motion of the pier-foundation systems are:

$$m'_1\ddot{x}_1 + c_1(\dot{x}_1-\dot{y}) + k_1(x_1-y) + c_o(\dot{x}_1-\dot{x}_2) + k_o(x_1-x_2) + H_1=0 \qquad \ldots (6)$$

$$m'_2\ddot{x}_2 + c_2(\dot{x}_2-\dot{y}) + k_2(x_2-y) + H_2 = 0 \qquad \ldots (7)$$

where $y(t)$ is the ground motion applied identically without phase shift to the two foundations. When rolling or sliding occurs,

$$H_1 = \mu R_1 \text{ sign } (\dot{x}_1 - \dot{x}_2) \qquad \ldots (8)$$

Numerical Solution of the Equations of Motion

The differential equations of motion have been integrated using fourth order Runge-Kutta method. A time interval of integration of 0.002 sec has been used which is approximately equal to one-fortieth of the time-period of vibration of the antisymmetric mode of vibration of the girder having the smallest of the periods of various elements.

BRIDGE AND EARTHQUAKE DATA

Data on an actual railway bridge in India was taken and some parameters were then varied to study their influence on the response. A general elevation of spans 3 and 13 of the truss bridge analysed is shown in Fig 2. The steel trusses rest on roller and rocker bearings consecutively. A pier consists of two identical circular sections connected by a wall. Main dimensions of the bridge are: span = 87.8m, pier height = 9.13m, well height = 33 m, dead weight of girders = 692 t, weight of track = 62 t and design load = 752t. The foundation soil is clay of varying composition from soft clay to firm sandy clay.

Soil-Foundation Stiffness - Stiffnesses of the soil-foundation-pier system were determined experimentally as well as theoretically. Static lateral load deflection tests and free horizontal vibration tests were conducted in the field. Bore log data being available, a suitable coefficient of subgrade reaction was assumed for each soil layer and the theoretical stiffnesses obtained. The experimental and theoretical values differed considerably[2]. The analysis has been carried out for the following five sets of stiffnesses.

(A) Experimental values, (B) Theoretical values, (C) Geometrical Mean of the above two values (D) Mutually interchanged experimental values of the two piers of a span, and (E) Arbitrary stiffnesses such that the ratio of stiffness for the pier having roller bearing to that having rocker bearing is 2.0. Numerical values of stiffnesses for spans 3 and 13 are given in table 2.

Ground Motion - Two strong motion records of earthquakes used in the analysis are i) modified accelerogram of May 18, 1940, El Centro, California, N-S component, original time co-ordinates reduced to 2/3 and acceleration co-ordinates to 0.556 and ii) modified accelerogram of Dec. 11, 1967, Koyna, India, Longitudinal component, time co-ordinates elongated to 5/3 and acceleration co-ordinates reduced to 0.50. The two accelerograms were normalised to have the same average zero-crossings and the same spectral intensity.

Bearing Types - Three types of bearings have been considered, i) Conventional fixed and roller bearings, ii) Roller bearings at both ends, iii) Sliding bearings at both ends. In case of roller bearings at both ends, an external spring is provided at one end to restrain the girders to the central position. Minimum possible stiffness of the spring is naturally used. Limiting the spring deformation to 2.5 cm a value of 10 t/cm was obtained for the spring stiffness, k_o to over-come the rolling friction. The spring is provided only at one end so that it is not stressed under changes in span length due to temperature variations, but the dampers are used at both the ends. Same spring stiffness has also been used for the case of 'both end sliding bearings'. The external spring was not used with the conventional bearings.

Effective pier masses (m_1, m_2) - The effective mass of each pier, idealised as a single degree of freedom system, has been obtained from the known stiffnesses and frequencies of the foundation-pier systems given in table 3. The masses thus determined are 185 t-s^2/m for pier 2 and 266 t-s^2/m for pier 13 of the bridge. The effective masses m_1 and m_2 of both the piers of a span are assumed to be equal.

Stiffness of Girder - The equivalent EI/L^3 for the girder is calculated by equating the dead load deflection at mid-span to that of a uniform beam of the same total weight. The value obtained is 4t/cm.

Geometrical Parameters of the Truss - Referring to Fig 1b, the significant eccentricities are : $e = 6.45$ m, $e' = 0.75$ m.

External hydraulic dampers - Hydraulic dampers have been provided between the girder and the pier at the roller and sliding ends. The damper constant c_o was varied from 0.10 to 4.0 t-s/cm, a range in which the design of hydraulic dampers is feasible.

RESULTS

The maximum values of the responses amongst all the foundation pier stiffness sets, excluding as well as including the arbitary stiffness case, are given in table 4 wherein the notations used are: ΔZ_1 , ΔZ_2 = rolling or sliding movement at the bearing, y_c = vertical mid-span displacement of the girder, H_2 = horizontal force at the rocker bearing pin, RD_2 = vertical dynamic reaction at rocker bearing.

Conventional Bearings - The maximum roller movement, ΔZ_1 is 1.6 cm for stiffness cases (A) to (D) and is double this value for stiffness case (E). For span 3 which is much stiffer than the span 13, the rolling movement is much smaller. It is also usually smaller than and sometimes nearly equal to Z_1 or Z_2. Thus the out of phase motion between the two piers for each of the spans 3 and 13 is very small. However, an upper limit on ΔZ could be taken equal to the absolute sum of the pier displacement responses Z_1 and Z_2 assuming them to be out of phase.

Introduction of damper reduces ΔZ but the reduction is insignificant for $c_0 \leqslant 1.0$ t-s/cm. The reduction is 12% to 33% for span 3 and 23% to 39% for span 13 for c_0 = 4.0 t-s/cm. Large vertical accelerations occur in girder exceeding 1.0 g in some cases. The values remain unaffected by introduction of dampers. The presence of these accelerations could indeed explain the phenomenon of jumping of girders seen in some past earthquakes.

Horizontal force H_2 on the fixed bearing has little change for $c_0 \leqslant$ 1.0 t-s/cm. But it reduces somewhat for c_0 = 4.0 t-s/cm, the reduction being more for greater values of the ratio k_1/k_2.

Roller bearings at both ends - Without external hydraulic dampers, the rolling motion, ΔZ at bearings may be quite large, but the hydraulic dampers considerably reduce the rolling motion. For span 13, with c_0 = 4 t-s/cm, ΔZ does not exceed 3.1 cm whereas with nominal dampers a value of 9.9 cm is obtained. For span 3 corresponding maximum ΔZ = 2.4 cm.

Horizontal force at the bearings is reduced by more than 80% of that developed in a conventional rocker bearing. Reduction in the dynamic vertical reaction at the bearings is more than 50%. There is also considerable reduction in the vertical acceleration response of the girders, the value generally being less than 0.15g.

Sliding bearings at both ends - A sliding friction coefficient equal to 0.25 was used for both end bearings. Sliding motion at the bearings is seen to be generally less than the rolling motion in case of conventional bearings. Dampers further reduce the movement to some extent depending upon the value of c_0.

The horizontal force on the bearing is also seen to be less than that found acting on the conventional rocker bearing. Vertical accelerations in girders is nearly the same as in conventional bearings, the value being more than 1.0g in some cases.

CONCLUSIONS

The results of the study presented here clearly explain the phenomenon of jumping of girders even during moderate earthquakes. They also indicate that use of roller bearings in parallel with appropriate hydraulic dampers at both ends and a soft spring at one end offers substantial advantages over the conventional set of rocker-roller bearings under earthquake conditions in the following aspects

1. The inertia load transferred from superstructure to the substructure can be reduced to as low a value as 20%.

2. The vertical acceleration of girder can be reduced to about 15% that in the case of conventional bearings.

It is also seen that the use of external dampers with conventional set of bearings or sliding bearings is not so useful due to insignificant reduction in the earthquake response of the superstructure or the substructure.

REFERENCES

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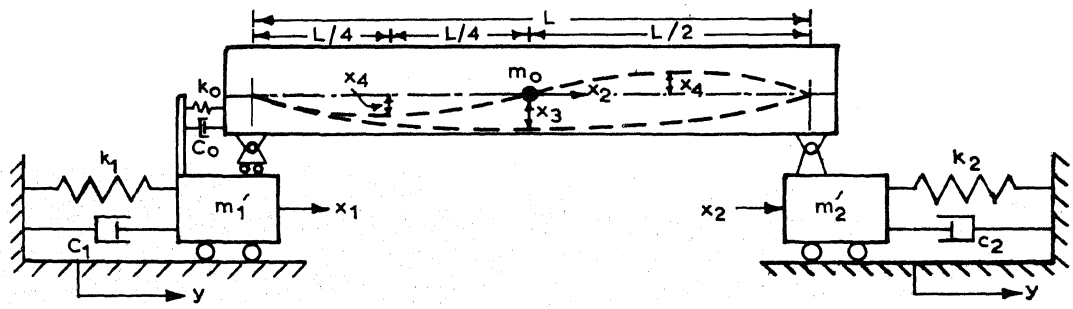

FIG.1a_MATHEMATICAL MODEL SHOWING DEGREES OF FREEDOM

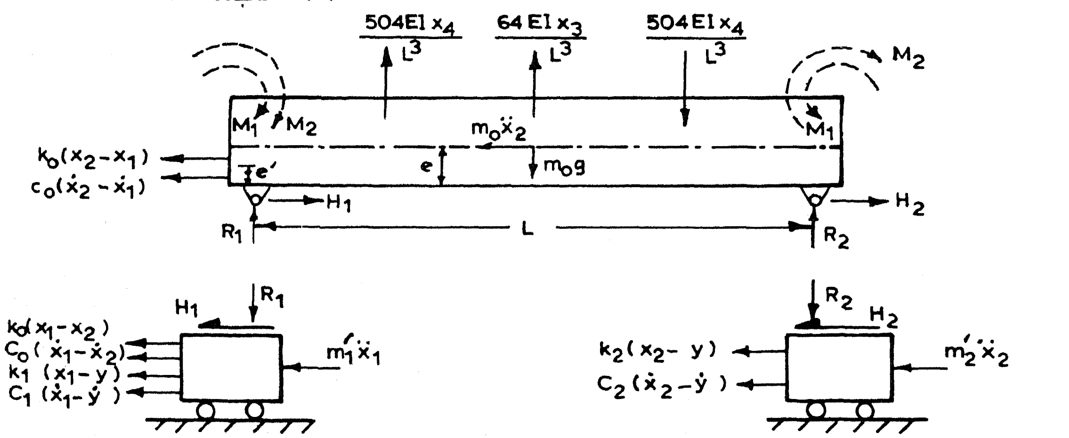

FIG. 1b_FREE BODY DIAGRAMS OF VARIOUS COMPONENTS

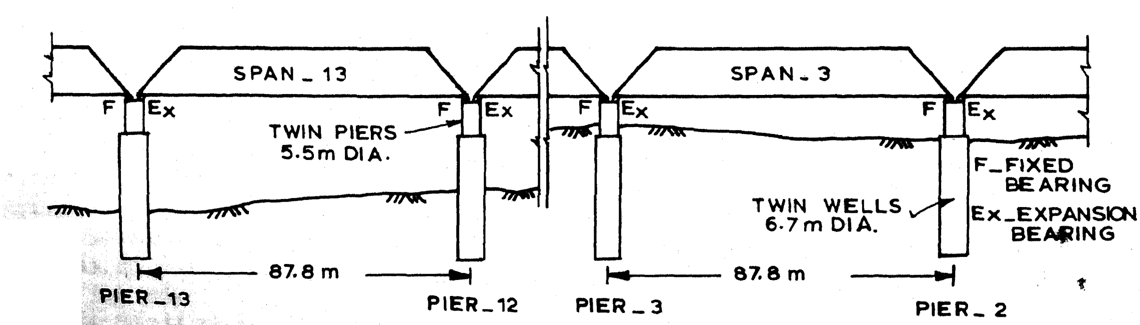

FIG. 2_GENERAL ELEVATION OF SPANS 3 AND 13

TABLE 1 - MODAL CHARACTERISTICS OF A SIMPLY SUPPORTED UNIFORM BEAM

Mode	First Mode (Symmetric)	Second Mode (Antisymmetric)
Static deflection	$x = \frac{M_1L^2}{2EI}(\frac{z}{L} - \frac{z^2}{L^2})$	$x = \frac{M_2L^2}{24EI}(\frac{2z}{L} - \frac{8z^3}{L^3})$
Origin for z	At end	At mid-span
Representative displacement	$x_3 = \frac{M_1L^2}{8EI}$ at $z = \frac{L}{2}$	$x_4 = \frac{M_2L^2}{64EI}$ at $z = \frac{L}{4}$
Mode shape	$\Phi = \frac{x}{x_3} = 4(\frac{z}{L} - \frac{z^2}{L^2})$	$\Phi = \frac{x}{x_4} = \frac{8}{3}(\frac{2z}{L} - \frac{8z^3}{L^3})$
Frequency	$p_3 = \sqrt{96EI/mL^4}$	$p_4 = \sqrt{1512EI/mL^4}$
Equivalent mass	$m_e = \int m\Phi^2 dz = 0.533mL$	$m_e = 0.542mL$
Equivalent Stiffness	$k_e = m_e p_3^2 = 51.2EI/L^3$	$k_e = m_e p_4^2 = 409.6EI/L^3$
Equivalent force	$F_e = k_e x_3 = 6.4M_1/L$	$F_e = k_e x_4 = 6.4M_2/L$
Inertia force	$I = \int_0^L m p_3^2 \Phi x_3 dz$ $= 64EI x_3/L^3$	$I = \int_0^{L/2} m p_4^2 \Phi x_4 dz$ $= 504EI x_4/L^3$
Distance of I from mid-span	$\bar{z} = 0$	$\bar{z} = \int_0^{L/2} \Phi z dz / \int_0^{L/2} \Phi dz$ $= 4L/15$
End Reaction	$R = \frac{I}{2} = 32EI x_3/L^3$	$R = \frac{2}{L}(M_2 + I\bar{z}) = \frac{2M_2}{L} + \frac{268.8EI x_4}{L^3}$

TABLE 2- FOUNDATION - PIER SYSTEM STIFFNESSES

Case	Span 3		Span 13	
	k_1 t/cm	k_2 t/cm	k_1 t/cm	k_2 t/cm
A	1770	2040	536	658
B	148	170	32	39
C	520	600	128	160
D	2040	1770	658	536
E	2000	1000	600	300

TABLE 3 - THEORETICAL AND EXPERIMENTAL STIFFNESSES AND FREQUENCIES

Pier No.	Stiffness (t/cm)		Frequency (c/s)	
	Theoretical	Experimental	Theoretical	Experimental
2	142.8	2040	1.19	4.20
13	32.4	536	0.518	1.88

TABLE 4 - MAXIMUM EARTHQUAKE RESPONSES OVER A RANGE OF FOUNDATION-PIER STIFFNESSES

Span	C_o	k_1/k_2	Ground Motion	z_1, z_2 (cm)			$\Delta z_1, \Delta z_2$ (cm)			y_c/g			H_2 (t)			RD_2 (t)		
				R-F	R-R	S-S	R-F	R-R	S-S	R-F	R-R	S-S	R-F	R-R	S-S	R-F	R-R	S-S
SPAN 3	.10	(A) to (D)	El Centro	1.82	1.97	1.85	0.40	4.87	0.61	1.16	.036	0.47	300	17	192	284	19	150
			Keyna	2.69	2.54	2.64	0.35	5.19	1.53	1.76	.064	0.63	342	12	224	446	27	224
		(A) to (E)	El Centro	1.82	1.97	1.85	1.29	4.87	0.92	1.16	.071	0.86	344	17	192	284	25	218
			Keyna	2.69	2.54	2.64	1.33	5.19	1.53	1.76	.146	2.06	378	13	224	446	43	224
	4.0	(A) to (D)	El Centro	1.82	1.75	1.85	0.31	1.73	0.41	0.95	.052	0.47	300	18	162	248	42	140
			Keyna	2.69	2.37	2.64	0.29	2.41	0.80	1.76	.096	0.64	342	13	220	442	63	220
		(A) to (E)	El Centro	1.82	1.75	1.85	1.08	1.73	0.65	0.95	.289	1.06	300	18	210	248	83	262
			Keyna	2.69	2.37	2.64	1.15	2.41	0.84	1.76	.491	2.29	342	15	230	442	132	232
SPAN 13	.10	(A) to (D)	El Centro	7.65	7.46	7.75	0.97	9.89	0.50	0.20	.053	0.26	222	12	156	128	22	74
			Keyna	5.03	5.07	4.98	1.62	6.80	1.71	0.40	.044	0.35	330	16	172	206	20	172
		(A) to (E)	El Centro	7.65	7.46	7.75	2.06	9.89	1.36	0.20	.087	0.30	222	12	188	128	30	74
			Keyna	5.03	5.07	4.98	3.22	6.80	2.04	0.40	.058	0.64	330	16	200	206	24	200
	4.0	(A) to (D)	El Centro	7.64	7.95	7.95	0.72	2.28	0.39	0.20	.055	0.25	226	12	190	128	40	78
			Keyna	4.98	5.02	4.98	1.16	3.08	1.13	0.42	.070	0.34	340	16	260	216	78	260
		(A) to (E)	El Centro	7.64	7.95	7.75	1.56	2.28	0.98	0.20	.086	0.25	226	12	190	128	40	78
			Keyna	4.98	5.02	4.98	2.38	3.08	1.61	0.42	.143	0.67	340	16	260	216	78	260