

LABORATORY TESTS OF MASONRY BRIDGE PIERS UNDER DYNAMIC LOADING

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INTRODUCTION

In almost all the seismic countries, the earthquake resistant design of bridge structure is based on the so-called seismic coefficient method.⁽¹⁾ The values of the seismic coefficients are based on the seismicity of the region and are rarely related to dynamic characteristics of bridge substructures. The actual forces acting on the bridge should depend upon, besides the seismicity of the region, the ground conditions, mass distribution, stiffness and damping characteristics of the various structural elements forming the bridge, and should vary along the height of the structure depending upon the mode shapes. An attempt has been made in this paper to study the dynamic characteristics and behaviour of masonry piers, which are commonly used in India, under free and forced vibrations with the help of a geometrically similar model of a recently constructed reinforced concrete balanced cantilever bridge supported on masonry piers and founded on rock.

DETAILS OF THE MODEL

The important dimensions of the prototype bridge are: Length of the main span 135 ft, length of cantilever span 22.5 ft, length of suspended span 90 ft, height of the pier above foundation level 22.5 ft, total weight of main span with overhangs 2065 kips, and total weight of suspended span=445 kips.

For making the model of the bridge, a scale factor of 15 was used. In the model, super structure consisted of one of the main spans with overhangs on both sides. Its details are given in Fig. 1.⁽²⁾ To avoid damage during handling and vibration tests, the model superstructure was made of steel with length, width and depth reduced in the above scale ratio. The thicknesses of the steel plates used in the model were selected so as to make the weight of the model equal to $1/n^3$ times the weight of the prototype. To account for the weight of the suspended span, extra weights were provided at the ends of the cantilevers in the model.

In order to study the effect of mass and stiffness of the pier on its response under vibration condition three different sets of piers were constructed in brick masonry as shown in Fig. 2. The height and the width and length at the top of the piers were obtained by reducing the corresponding dimensions of the prototype to scale but three tapers of faces were adopted resulting in different thicknesses and lengths below the top. The piers were fixed, through a 2 in. thick concrete base to a steel base plate which could be used for lifting the piers and also bolting down to a rigid foundation or floor of the shaking table. In the actual bridge the piers rested on rock and were anchored into it. The base plate details are shown in Fig. 3.

In the prototype bridge, rocker and roller type bearings were used on alternate piers. The details of rocker and roller bearings adopted in the model tests are shown in Fig. 4.

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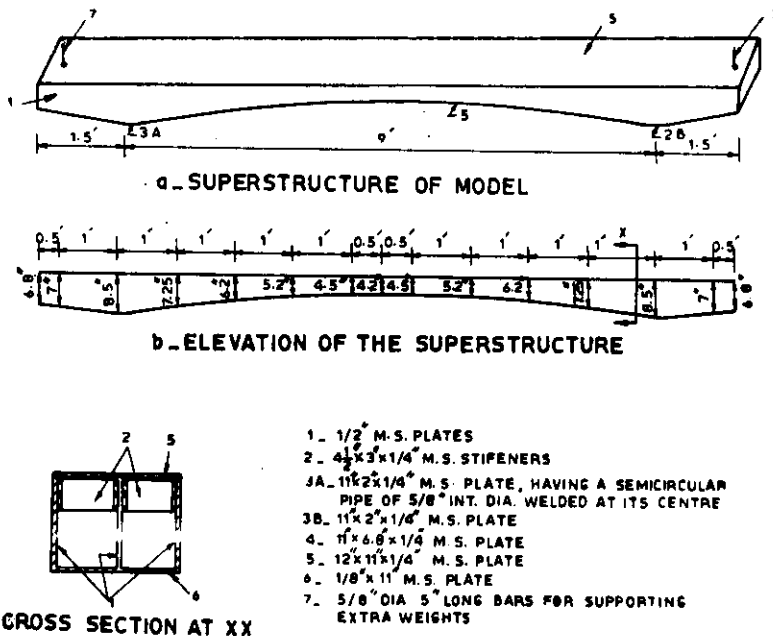


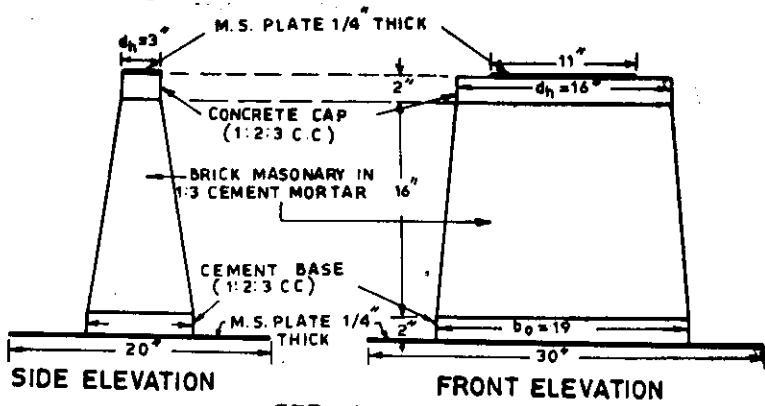
Fig. 1. Details of Model Superstructure

TESTS

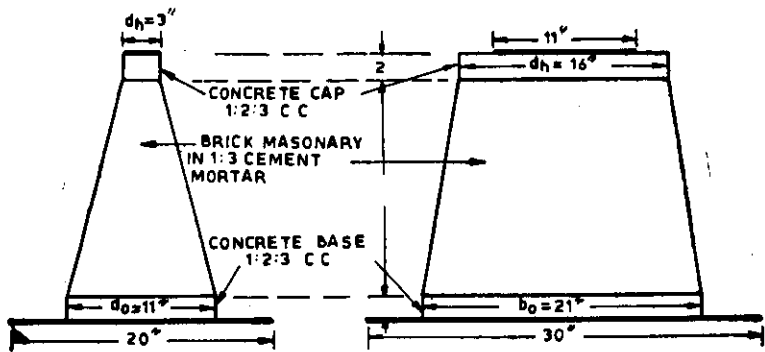
The following tests were carried out on the three pairs of piers :

(a) For determining the elastic modulus E of brick masonry in 1 : 3 cement sand mortar used in the piers, the piers were statically loaded as vertical cantilevers and load-deflection was determined. Horizontal pull was applied near the top of the pier and deflection measured 4 in. below it. The load was kept small so that piers did not crack at this stage of testing. Figure 5 shows the load deflection curves for the three types of piers giving an average value of E equal to 0.214×10^6 psi.

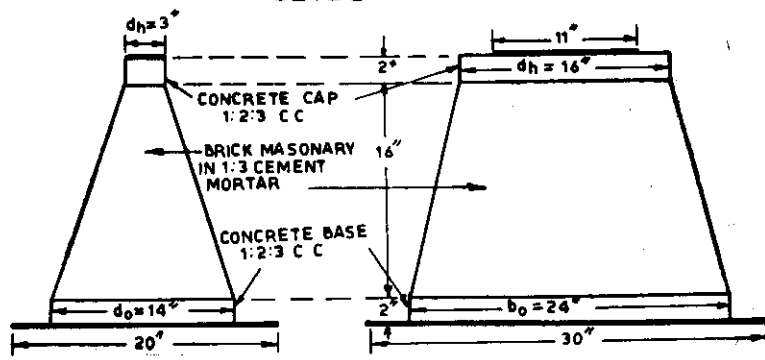
(b) Free vibrations of piers were carried out without the super-structure resting on them and also when superstructure was present. For this purpose, horizontal pull was applied near the top of the pier which was suddenly released by a clutch arrangement so that structure was left vibrating freely. The testing arrangement is shown in Fig. 6. When the superstructure was placed on the piers, rocker and roller piers were pulled separately. Vibrations were picked up by using accelerometers, amplified and recorded on direct ink writing oscillograph. Table 1 gives the frequencies of two rocker piers for various magnitudes of pulls applied giving the effect of stress level on the natural frequency. It is seen that as the applied pull increases, the natural frequency decreases. Although the difference is small, it is consistently so and should not be taken as statistical fluctuation of measurement since in the unloaded state, the frequency was again seen to be equal to the original value. Thus a slight softening characteristic of the stiffness of the piers is indicated. Table II gives the observed natural frequencies of the roller and rocker piers under the same pull but with and without the superstructure. The observed damping values are also given.



SET_1



SET_2



SET_3

SCALE 1"=10'

Fig. 2. Details of Piers

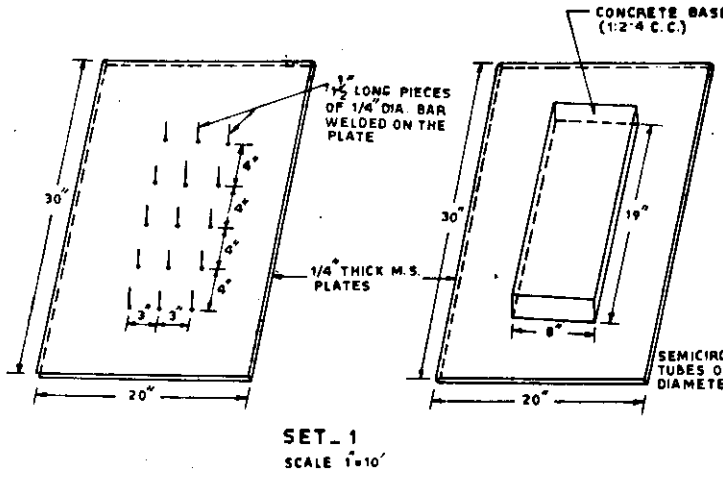
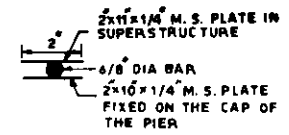
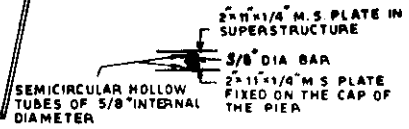


Fig. 3. Details of Base Plates



ROLLER BEARING



ROCKER BEARING

SCALE 1"=4"

Fig. 4. Details of Bearing

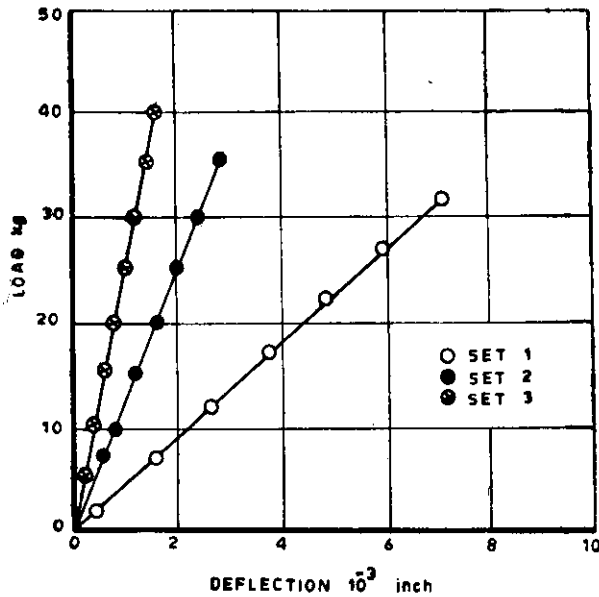


Fig. 5. Load vs. Deflection of Piers without Superstructure

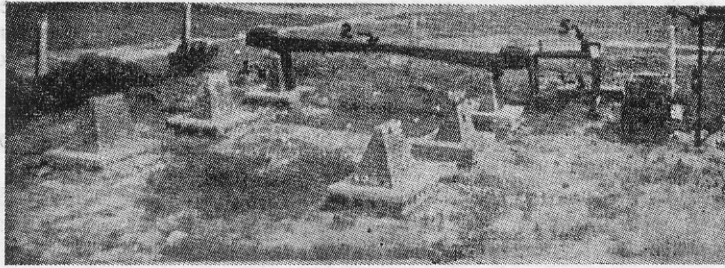


Fig. 6. Testing Arrangements for Free Vibration of Bridge.

1. Pier, 2. Superstructure, 3. Clutch, 4. Reaction Frame, 5. Pen Recorder.

TABLE I
FREQUENCIES OF MASONRY PIERS AT
ROCKER END

Horizontal Pull before Release kg	Frequency of Pier at Rocker End. Set 1, c/s	Frequency of Pier at Rocker End, Set 2, c/s
20	20.67	24.90
30	22.10	24.30
40	21.83	23.85
50	21.60	24.02
60	21.54	23.85
70	21.40	23.60
80	21.36	23.55
90	21.26	23.50

TABLE II
FREE VIBRATION TEST RESULTS

Pier Set No.	Without Superstructure		When superstructure was placed on piers			
	Natural Frequency c/s	Damping % of critical	Pier with Roller Bearing		Pier with Rocker Bearing	
			Frequency	Damping % of critical	Frequency c/s	Damping % of critical
1.	56.8	3.4	41.66	3.35	21.80	3.40
2.	66.8	3.5	50.00	3.50	24.08	3.45
3.	100.0	3.5	62.50	3.40	29.40	3.50

(c) Forced vibration tests of the bridge model with the first set of piers were carried out on a 16.5 ft x 10 ft. shock-type vibration table. The table platform can be shaken with different amplitudes by giving different impulses by means of a striking pendulum which can be raised to different heights before its free fall. The raising of the pendulum is measured by its inclination with the vertical in degrees. Photo shows the set up used.

The piers were bolted down to the table platform and superstructure place on them. The model was set on the shaking table such that the vibration of the table imparted the vibrations to the model in the longitudinal direction of the superstructure. The testing arrangement is shown in Fig. 7. The impulses were given at 10° and 15° fall of the pendulum and the accelerations were recorded in the longitudinal direction at the underneath of the table platform: bottom, midheight and top of the roller and rocker piers and at the two ends of the super-structure. Besides these, vertical accelerations were measured at mid-span of the superstructure. During the longitudinal vibration of the table, the superstructure of the model was seen to jump on the bearings.

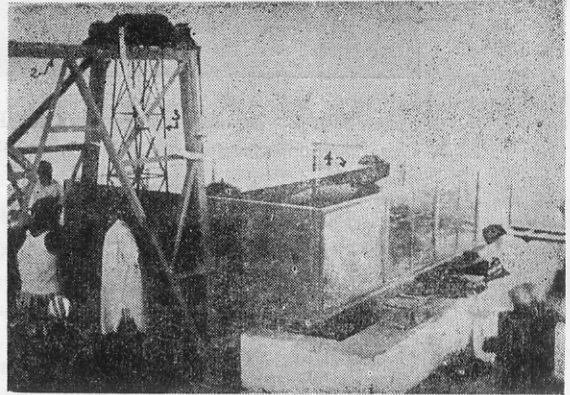


Fig. 7. Shake Table Test

1. Shake Table, 2. A-Frame, 3. Pendulum, 4. Bridge under Test.

The various observed maximum accelerations are given in Table III.

TABLE III
FORCED VIBRATION TEST RESULTS

Sl. No.	Degree of Impact	Maximum longitudinal acceleration in units of g								Maximum vertical accn. at centre of superstructure in units of g
		Roller End				Rocker End				
		B+	M	T	S	B	M	T	S	
1.	10°	0.61	0.915	1.195	0.255	0.583	0.30	0.277	0.189	0.510
2.	15°	2.01	2.100	2.33	0.289	2.22	1.78	1.050	0.277	0.899

+B — Base of Pier

M — Mid-height of Pier

T — Top of Pier

S — Ends of Superstructure

DISCUSSION OF TEST RESULTS

From the free vibration test results (Table II) it is seen that after placing the superstructure on piers, the natural frequency of both the roller and rocker piers are reduced but the reduction is much greater in the case of rocker pier (about 2/3 of its own frequency) than the roller pier (about 1/3 of its own frequency). Theoretically speaking, if the roller bearing was truly frictionless, there should have been no reduction in the frequency of the roller pier, since all the horizontal inertia of the mass of the superstructure should have been transmitted to the rocker bearing. It therefore appears that, on account of the friction in the roller bearing, part of the mass of the superstructure was *virtually* attached on the top of the roller pier.

It is also seen that the structural damping remained practically unaltered at an average value of 3.5%.

The acceleration pattern obtained at resonance in the forced vibration test is plotted in Fig. 8. From this it is seen that, whereas the horizontal acceleration shows an amplification from the base to the top of the roller pier, it is reduced from base to the top of

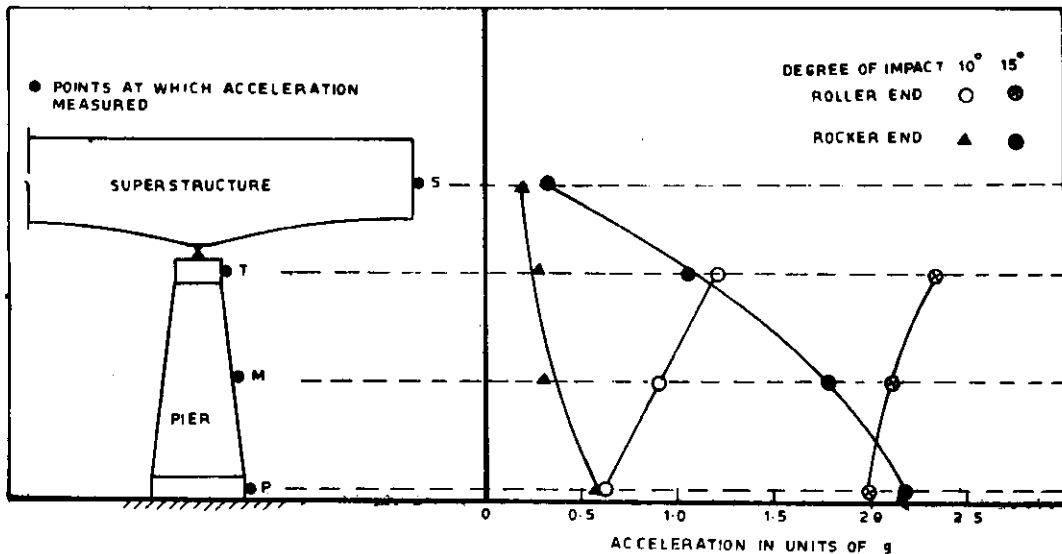


Fig. 8. Maximum Acceleration vs. Height along Pier at 10° and 15° Impact

the rocker pier. The acceleration is observed to be the least at both ends of the superstructure. This is a significant observation because of its far reaching consequences in the design of bridge sub-structures. However this result has yet to be proved by theory.

Attention is also drawn to the fact that although the table motion was horizontal, the superstructure exhibited jumping on the bearings and substantial vertical accelerations were measured at mid-span as shown in Table III. This happens since the ground motion effect is transmitted from the pier to the superstructure eccentrically with reference to the centre of gravity of the deck giving rise to a moment which requires vertical acceleration of the deck for balancing it.

THEORETICAL ANALYSIS

In order to have a verification on the experimental results, the undamped natural frequencies and modes were calculated for the three types of piers using the value of modulus of elasticity as determined by the static load-deflection tests.

During vibrations of the rocker pier, the deflection of the superstructure will be the same as that of the top of rocker pier at any instant. Also, if there was no friction in the rollers, the inertia force due to the mass of superstructure would get transferred on the top of the rocker pier. In that case, the dynamic analysis of the rocker pier can be made on the assumption that the whole mass of the superstructure is attached at the top of the rocker pier. In case, there is some friction in the rollers, frictional force F will also act at the top of the rocker pier in the direction which will tend to oppose its motion. The actual value of frictional force F at any instant during vibration will be equal to the dynamic coefficient of friction multiplied by the vertical reaction at the roller end. The vertical reaction at the roller end will itself be a varying quantity depending not only on the longitudinal inertial force but also on the vertical acceleration of the superstructure. In order to simplify the analysis of the rocker pier, a reduced mass of the superstructure was considered effectively attached to the pier top.

On the other hand, during vibrations of the roller pier, the frictional force F in the rollers will tend to pull the superstructure in the same direction as the motion of roller pier. However the superstructure will be displaced to a much smaller extent than the top of roller pier. Therefore the free vibration of the roller pier will be considerably restrained reducing its frequency. Again as a simplifying assumption a portion of the mass of the superstructure may be considered to be attached to the top of roller pier for calculating its natural frequency.

Due to massiveness of masonry piers, both bending and shearing deformations are significant. Therefore the natural frequencies were calculated considering both types of deformations. The piers were assumed fixed at the base and rotatory inertia was neglected. The frequencies were computed numerically by Myklested-Prohl method (2, 4, 5) and are given in table 4 for several percentages of mass of superstructure assumed attached with the rocker and roller piers.

TABLE IV
THEORETICAL VALUES OF NATURAL FREQUENCIES OF PIERS

Pier Set No.	Natural Frequencies, c/s									
	Roller Pier					Rocker Pier				
	% of the mass of superstructure considered attached at top of pier					% of the mass of superstructure considered attached at top of superstructure				
	0%	10%	18%	20%	22%	100%	90%	82%	80%	78%
1.	112.20	49.81	38.69	36.95	35.40	17.19	18.12	18.87	19.20	19.40
2.	152.10	69.40	58.49	53.95	49.20	23.86	25.20	26.38	26.61	26.99
3.	189.53	88.65	68.89	65.61	63.01	30.61	32.21	34.00	34.08	34.21

The results show that while there is small change in the fundamental frequencies of the rocker pier by varying the percentage of the mass of superstructure, the frequencies of the pier under the roller bearing are very much reduced due to associating even a small fraction of the mass of the superstructure with the roller pier. Comparing the experimen-

tal values with the theoretical, it is found that they are in good agreement if about 20% of the mass of superstructure is considered to act at the top of the roller pier in this case.

CONCLUSIONS

The experimental investigations presented herein regarding free vibrations of a model bridge with masonry piers indicate that the presence of friction at the roller end greatly influences the natural frequency of the pier under the roller bearing. A simplified theoretical analysis shows that if a virtual distribution of the mass of the superstructure is assumed between the piers under rocker and roller bearings, the theoretical frequencies compare well with experimental values.

The test of the model bridge on the shock-table under forced vibration conditions shows that the accelerations increased from base of roller pier towards its top while they decreased in the case of rocker pier being the minimum at superstructure level. Besides the longitudinal accelerations, the superstructure was found to vibrate vertically and jump on the bearings.

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