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EXPERIMENTAL ANALYSIS OF COUPLED CABLE-DECK MOTIONS IN CABLE-STAYED BRIDGES

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ABSTRACT

A brief description of an experimental investigation involving the study of the dynamic interaction between cables and the deck/towers system on the physical model of a cable-stayed bridge is presented. The study shows a clear coupling between cable and deck/tower motions and includes an analysis of this effect in terms of the response to seismic action.

KEYWORDS

Cable-stayed bridges; physical models; modal analysis; shaking table.

1. INTRODUCTION

Cable-stayed bridges are very flexible structures that frequently experience vibration problems caused by environmental factors, such as wind, earthquakes, traffic, rain, etc.. It is thus essential to understand and predict realistically their structural response to these forms of loading. Accordingly, many efforts have been made in order to develop accurate methodologies for the analysis of the static and dynamic behaviour of this type of bridges, which may reveal a significant non-linear behaviour. In this context, it has been recently emphasized (Abdel-Ghaffar and Khalifa, 1991), using a numerical approach, the importance of the complex vibrations of the stay cables, which seem to be strongly coupled with the bridge deck and tower motions, although they have been usually overlooked or treated independently in the study of cable-stayed bridges. By discretizing each cable into small finite elements, there results new and numerous complex pure cable vibration modes, whose analytical prediction would be impossible using the linearized natural frequency expressions for the individual inclined cables. Furthermore, this model also provides coupled deck-cable motions involving bending and torsional motions of the deck as well as vertical and swinging cable motions, that cannot be predicted using traditional finite element models, and that may have a significant effect on the participation factors of any earthquake response calculation.

The objective of this paper is to describe some results of an investigation conducted at the Earthquake Engineering Research Centre of the University of Bristol (U.K.) in collaboration with the University of Porto (Portugal), with the aim of experimentally identifying the existence and importance of the interaction between the cable stays and the deck-tower subsystem. This study has been performed on the physical model of a cable-stayed bridge (Jindo Bridge, in South Korea), whose characteristics of stiffness and mass have been conveniently scaled. The main dynamic parameters of the bridge were then identified on the basis of standard experimental modal analysis techniques, using either an electodynamic shaker or a shaking table, and the response to artificial accelerograms used as input on the shaking table has been measured. A comparison between experimental and numerical results has been accomplished.

2. THE JINDO BRIDGE PROTOTYPE

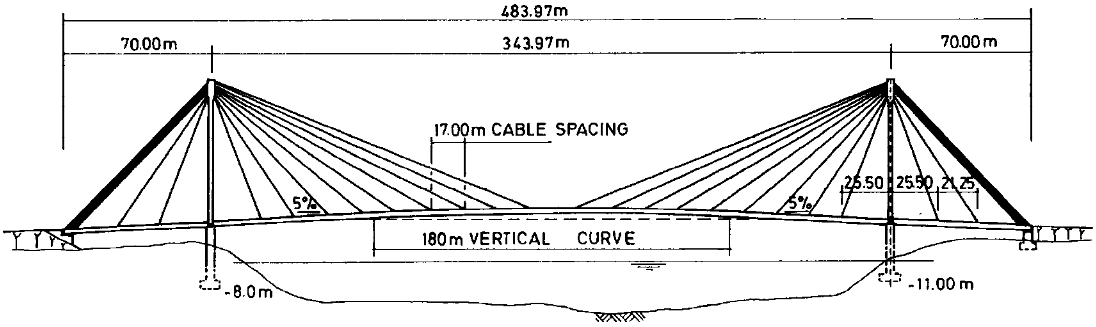

Fig. 1. General dimensions of Jindo Bridge.

The Jindo Bridge, designed by Rendel Palmer & Tritton and built in South Korea (Tappin and Clark, 1985), is a three span steel cable-stayed bridge with a continuous stiffening girder, having a total length of 484 m. The main span is 344 m and the side spans are 70 m, according to the scheme of Figure 1. The boundary conditions of the main girder are achieved by two rocker supports, at the ends, one pinned support in one of the towers and a roller in the other tower. The stays are arranged in a form of a fan converging at the top of each A-frame tower. Each frame carries 24 stays. The cables constitute locked coil ropes.

3. THE PHYSICAL MODEL OF JINDO BRIDGE

The physical model of Jindo Bridge, built at the Earthquake Engineering Research Centre of the University of Bristol can be described as a distorted small scale model with artificial mass simulation. This model was designed by Garevski (Garevski, 1990) and has recently been modified by the authors (Caetano and Cunha, 1995).

The total length of the bridge model is 3.227 m, according to the linear scale factor $S_L = 150$. The choice of such a small scale, conditioned by the dimensions of the shaking table, made it almost impossible to maintain all the proportions of the prototype. So, the box cross-section of the girder and tower legs have been substituted by solid rectangular cross-sections with correctly scaled bending stiffness (but not axial stiffness). These components, constructed in an aluminium alloy, have the following dimensions: Girder, $25.4x7.92mm^2$, Tower legs, $9.2x8.4mm^2$. The stay cables have been simulated with piano wires. The axial stiffness has been correctly scaled and the only difference to the prototype lies on the back stays, that are made of 6 cables in the real structure, and constituted by one piano wire with equivalent scaled area in the model. To compensate for the dead weight of such a light model, additional masses have been designed. Small steel plates have been attached to the towers and girders by two bolts, without any further contact. A general description of the overall bridge can be found in Figure 2, while a more detailed characterisation of the model is presented in (Garevski, 1990).

Two different types of masses were used to compensate for the dead weight of the cables. For the back stays, small lead spheres were glued to the wire. For the other cables, smaller masses made of a zinc alloy were used. The set of final tensions on the cables from the bridge prototype was scaled according to the similitude theory. The wires from the model were prestressed by means of two bolts. The tensions on each cable were tuned indirectly by means of a magnetic sensor and a Fourier analyzer. The process consisted of moving the sensor to the cable and manually plucking the cable. The signal generated by the sensor, proportional to the velocity of the cable, was then processed in the frequency domain. The first two or three harmonics frequencies have been compared to the frequencies of a single prestressed cable supported at the ends (Irvine, 1978) and related to the corresponding installed tension.

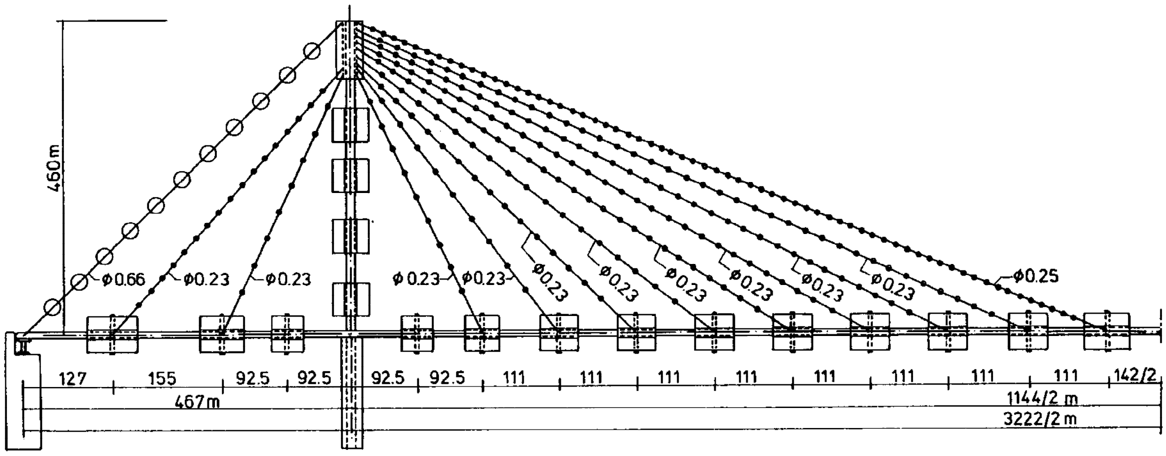

Fig. 2. Overall characteristics and dimensions of the Jindo Bridge physical model.

4. NUMERICAL EVALUATION OF MODAL PARAMETERS OF THE MODEL

In order to make a comparison between the modal parameters obtained experimentally and the corresponding parameters of a theoretical model, two preliminary 2-D finite element models of the Jindo Bridge physical model were developed. These were designated (Abdel-Ghaffar and Khalifa, 1991) as OECS (One-Element-Cable-System) and MECS (Multi-Element-Cable-System) models. The OECS model discretized the deck and towers into 76 beam elements and idealised each stay cable as a simple truss element with an equivalent Young's modulus. The MECS model adopted the same discretization of the deck and towers into 76 beam elements and each stay cable was discretized into 12 truss elements. The total of 100 elements (76 beam elements, 24 truss elements) was used for the OECS model and 364 (76 beam elements, 288 truss elements) for the MECS. Using these models, dynamic analyses were performed based on the knowledge of the tangent stiffness matrices obtained under a previous application of the static loads in a non-linear geometric calculation, using the commercial software package SOLVIA. The lowest 20 natural frequencies in the range 0-100 Hz and the corresponding modal shapes have been evaluated for the OECS. Table 1 summarises the values of these natural frequencies and the type of the corresponding modal shape (SYM- modal configuration of the girder and towers symmetric with respect to the symmetry axis of the bridge; ASM- modal configuration of the girder and towers anti-symmetric; TWL- modal configuration involving essentially the movement of the left tower; TWR- modal configuration involving essentially the movement of the right tower; AXG- modal configuration involving essentially the axial deformation of the girder). For the MECS, the lowest 60 natural frequencies in the range 0-30 Hz and the corresponding modal shapes were calculated. Table 2 summarises the values of some of these natural frequencies.

It should be emphasised that the behaviour of cable-stayed bridges is clearly three-dimensional, and so a valid numerical analysis requires the use of a 3-D FEM modelling. This is in fact the approach the authors intend to adopt. The present 2-D formulation, has been used at this stage in order to prepare the experimental tests and obtain preliminary conclusions.

The analysis of tables 1 and 2 and the observation of the modal shapes (Caetano and Cunha, 1995) led to the conclusion that the FEM analysis based on the discretization of the cables into several truss elements (MECS) produced many new modes of vibration. Most of the new modes simply involved local vibrations of the cables and so were not important in terms of the contribution to the global response of the bridge. Others of these new modes, however, involved simultaneous motion of the cables and of the deck and towers. In most cases, several of these modes of vibration had similar natural frequencies and mode shapes for the deck/towers system, but different movements of the cables, the relative order of magnitude of the motion also varying. The importance of the coupling can be quantified by a factor obtained from the ratio between the maximum displacement that results for the group of cables and the corresponding value for the deck/towers system (Table 2). In the present analysis, for the set of 60 natural frequencies, this factor varies from 1.3 to 566.9. Small ratios denote a very clear interaction between the cables and the girder/towers movement. High ratio values are typical of modal shapes that involve essentially vibration of one or more cables, here designated as local modes.

Table 1. Calculated natural frequencies of Jindo Bridge model (2-D OECS numerical model)

Mode number	Natural frequency (Hz)	Type of Mode	Mode number	Natural frequency (Hz)	Type of Mode
1	6.21	1st SYM	11	41.36	1st TWR
2	9.16	1st ASM	12	48.50	5th ASM
3	13.68	2nd SYM	13	59.37	6th SYM
4	18.14	2nd ASM	14	63.04	2nd TWL
5	22.66	3rd SYM	15	64.18	2nd TWR
6	26.13	3rd ASM	16	66.77	6th ASM
7	28.79	4th SYM	17	71.85	7th SYM
8	32.68	4th ASM	18	76.65	7th ASM
9	40.19	5th SYM	19	90.04	8th SYM
10	41.13	1st TWL	20	99.29	1st AXG

Table 2. Calculated natural frequencies of Jindo Bridge Model (2-D numerical model, MECS)

Mode Number	Natural Frequency (Hz)	Ratio Truss/Beam Max. Displ.	Type of mode	Mode Number	Natural Frequency (Hz)	Ratio Truss/Beam Max. Displ.	Type of Mode
1	6.19	**1.97**	1st SYM	16	13.13	**32.6**	2nd SYM
2	6.90	276.7	LOCAL	17	13.20	134.6	LOCAL
3	6.91	285.4	LOCAL	18	13.33	**35.1**	2nd SYM
4	7.78	566.9	LOCAL	19	13.37	92.7	LOCAL
5	7.78	282.6	LOCAL	20	13.86	**3.4**	2nd SYM
6	8.35	254.3	LOCAL	21	14.90	493.4	LOCAL
7	8.36	325.0	LOCAL	22	14.91	497.2	LOCAL
8	9.04	**17.7**	1st ASM	23	15.18	137.4	LOCAL
9	9.17	**10.6**	1st ASM	24	15.20	101.1	LOCAL
10	9.60	161.4	LOCAL	25	16.30	111.8	LOCAL
11	9.65	**31.8**	1st ASM	26	16.33	176.7	LOCAL
12	11.62	162.9	LOCAL	27	16.34	144.0	LOCAL
13	11.65	225.4	LOCAL	28	16.36	239.1	LOCAL
14	11.86	**45.1**	2nd SYM	29	17.81	**48.8**	2nd ASM
15	11.89	**49.2**	1st ASM	30	17.85	75.1	LOCAL

5. MODAL SURVEY

Experimental setup

The identification of modal parameters from the physical model used an electrodynamic shaker. The procedure consisted of the application of vertical and transversal multi-sine excitation at the mid-span (node 31) and at the thirds of the span (left, node 24; right, node 38). The response was measured along the deck and towers using a piezoelectric accelerometer, and along some of the cables, either using a piezoelectric accelerometer (only on the back-stays) or a magnetic sensor. Force was measured by means of a force sensor, interposed between the shaker and the bridge model. The acquired time signals were used to obtain frequency response functions (FRF's). Modal parameters were extracted from the set of FRF's using a least squares frequency domain identification algorithm, MODHAN (Han and Wicks, 1989). Figure 3 illustrates an example of a measured/synthesized FRF, based on the mentioned identification algorithm.

Similar modal analysis tests have been performed on a 6-DOF shaking table, by application of a multi-sine base excitation (along the longitudinal, X, transversal, Y, and vertical, Z, directions) and by measurement of the structural response in terms of accelerations and displacements along the deck and towers. Good coherence function estimates have been found for the set of FRF's (obtained from the relation of the measured response to the measured acceleration on the shaking table) and, most important, consistency between data obtained from these two different forms of excitation seems to exist.

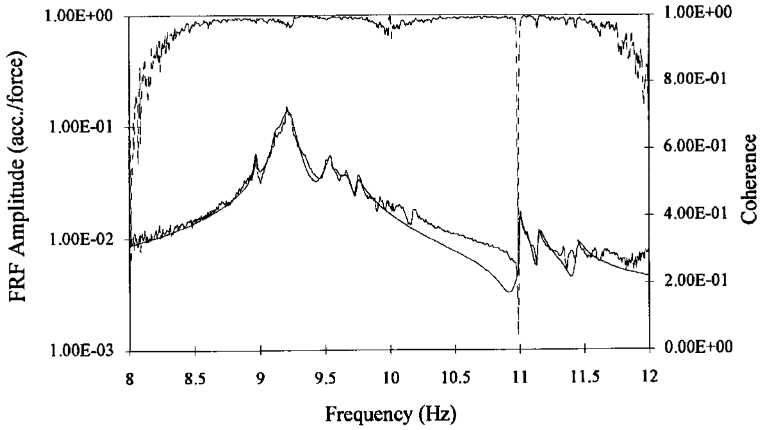

Fig. 3. FRF and Coherence measured / synthesised from node 38 Vert. (excitation) to node 24 Vert. (response).

Test results

Table 3 summarises some of the values of the natural frequencies and damping factors identified from the set of FRF estimates, as well as the corresponding calculated frequencies, based on the finite element models.

Table 3. Identified and calculated natural frequencies

Identified natural frequency (Hz)	OECS: Calculated natural frequency (Hz)	MECS: Calculated natural frequency (Hz)	Identified damping factor (%)
6.20 / 7.10	6.21	6.19	1.3 / 0.6
9.20 / 9.50 / 9.58 / 9.65	9.16	9.04 / 9.17 / 9.65 / 11.89	0.6 / 0.2 / 0.8 / 0.4
13.20 / 13.43 / 14.2 /14.41	13.68	11.86 / 13.13 / 13.33 / 13.86	0.2 / 1.2 / 0.2 / 0.5
18.97 / 19.05 / 19.30	18.14	17.81 / 18.29	0.5 / 0.3 / 0.6
24.20	22.66	22.81 / 23.30 / 23.34	0.4
31.99	26.13	26.23 / 26.70 / 26.73	0.9
29.35 / 30.18	28.79	28.86 / 29.63	1.1 / 1.4
37.38	32.68		
42.10	40.19		1.8

The observation of the FRF's and of this table show the existence of several peaks in the vicinity of the natural frequencies obtained from the OECS analysis. These peaks are closely spaced. The FRF zoom presented in Figure 3, for instance, lead to the identification of 6 modes of vibration, at the frequencies 8.97 Hz, 9.12 Hz, 9.20 Hz, 9.50 Hz, 9.63 Hz and 9.74 Hz. These frequencies compare well with the MECS analysis and are consistent with the identified frequencies from the FRF estimates. An attempt has been made in order to prove that the associated mode shapes involve similar configurations for the girder/towers and different movements of the cables, as has been reported for the numerical analysis. It consisted of collecting time series at several nodes of the girder and towers, measuring the response to a sinusoidal excitation applied vertically at node 38 with frequencies of 9.20 Hz and 9.50 Hz. From these series, components of the two mode shapes have been estimated. Figure 4, representing a plot of these components against the associated values for the 1st anti-symmetric mode obtained on the basis of the OECS analysis, shows the similarity of configurations of the mode shapes.

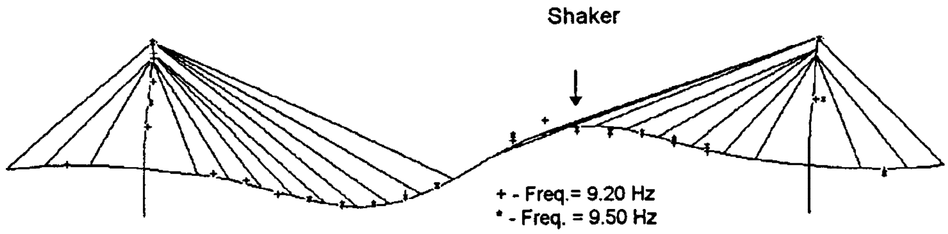

Fig. 4. Identified and calculated modal shapes (+- identified freq.= 9.20Hz; *- identified freq.= 9.50Hz)

With regard to the cables, it has been visually observed that a sinusoidal excitation applied with close frequencies induces different movements of the cables. Moreover, for the levels of vibration induced, the persistence of the excitation showed no evidence of inducing a modification of the configurations of vibration (either of the girder/towers or of the cables) into other configurations (for instance, the configuration of adjacent modes of vibration). It was also observed that repeating the application of some sinusoidal excitation at different times and with different initial conditions would not induce different movements of the cables. Table 4 reports the observed movement of the cables for different frequencies of excitation close to 9Hz. To distinguish the two opposed cables connected to the same tower, symbols F (front cable) and B (back cable) have been employed. The relative amplitude of the cable movement is represented by $i, \underline{i}$ or $\underline{\underline{i}}$, the first meaning that the amplitude of vibration of cable i is low and the last meaning this amplitude is quite high.

It is noteworthy that there is in fact a three-dimensional character of the vibration, as the 4 equal stay cables with similar tensions associated with each cable number (e.g.: 8B left, 8F left, 8B right, 8F right) behave differently. As a consequence, we can not expect a perfect agreement with the configurations of modal shapes obtained from the 2-D MECS analysis (Table 5). It is clear however from Tables 4 and 5 that the cables with major vibrations are the same.

Table 4. Observation of cable movement during sinusoidal excitation

Frequency (Hz)	Cables connected to left tower	Cables connected to right tower
9.20	$\underline{8}F, \underline{\underline{8}}B, \underline{9}F, 9B, \underline{3}B$	$\underline{\underline{8}}B, \underline{9}F, \underline{\underline{9}}B, \underline{5}F, \underline{5}B$
9.25	$\underline{8}F, \underline{\underline{8}}B, 9F, 9B$	$\underline{8}F, \underline{\underline{8}}B, \underline{\underline{9}}F, \underline{\underline{9}}B, \underline{5}F, \underline{5}B$
9.50	$\underline{\underline{8}}F, 9F, 9B, 10F, 10B$	$\underline{8}F, \underline{\underline{8}}B, \underline{\underline{9}}F, \underline{\underline{9}}B, \underline{5}F, \underline{5}B, 12F, 12B$
9.55	$\underline{\underline{8}}F, \underline{9}B, 10F, 10B$	$8B, 9B, 9F, \underline{5}F$

Table 5. Description of cable movement using a 2D MECS analysis

Frequency (Hz)	Cables connected to left tower	Cables connected to right tower
9.04	8	8,9
9.17	8	8,9,11,12
9.65	8	8
11.89	1	1

With respect to the FRF's obtained from measurements on the cables, the analysis of peak frequencies has revealed not only the frequencies of individual vibration of the cables, but also the existence of other frequencies measured on the girder/towers related to global modes of vibration.

6. SEISMIC TESTS ON THE SHAKING TABLE

In the last phase of the work, a series of seismic tests was performed on the shaking table in order to evaluate the importance of the identified coupling in terms of the structural response. For that purpose, three different time histories of the ground acceleration (with about 30s duration for the prototype) were generated and scaled (scale factor for time $S_t = 1/\sqrt{150}$) based on three different target response spectra. The response was measured for an input motion in each direction X, Y and Z, and for the combinations XZ and XYZ, with about 5%g and 10%g peak values along the two horizontal directions, and about 3%g and 6%g in the vertical direction.

Using the accelerograms measured on the seismic platform as input motion, and admitting, in a first approach, that no structural damping is present in the system, the response of both OECS and MECS has been evaluated based on a direct integration algorithm (Newmark method). Tables 6 and 7 resume several absolute peak values that were obtained for two different time histories on some of the most significant nodes of the structure (mid-span, node 31Z; attachment of longest cable, node 33Z; third of span, node 38Z; top of left tower, node 69X; top of right tower, node 77X), as well as the corresponding values obtained from the response measurements.

Table 6. Peak response for input RRS1, 6%g (Z)

Node	Experimental		OECS		MECS	
	Accel. (m/s2)	Displ. (m)	Accel. (m/s2)	Displ. (m)	Accel. (m/s2)	Displ. (m)
31Z	7		7.7		5.9	
33Z		2.8E-3		3.7E-3		3.4E-3
38Z	3		4.9		4.2	
69X	0.8		1.3		1.1	
77X	0.8		1.4	1.7E-3	1.2	1.6E-3

Table 7. - Peak response for input RRS2, 6%g (Z)

Node	Experimental		OECS		MECS	
	Accel. (m/s2)	Displ. (m)	Accel. (m/s2)	Displ. (m)	Accel. (m/s2)	Displ. (m)
31Z	3.0		3.6		3.2	
33Z		1.3E-3		2.0E-3		1.9E-3
38Z	2.0		2.2		2.2	
69X	0.6		0.61		0.60	
77X	0.5		0.68		0.55	

The study of the numerical solutions obtained for the two FEM models shows a trend for the MECS to lead to smaller accelerations and displacements than the OECS, the relative decrease not being in general superior to 20 %. This fact is evidenced by Tables 6 and 7.

7. CONCLUSIONS

The main objective of this paper was to describe an experimental investigation recently developed by the authors at the Earthquake Engineering Research Centre of the University of Bristol, concerning the experimental study of the dynamic behaviour of the physical model of a cable-stayed bridge. Although the results presented here correspond to a preliminary phase of analysis of measured data and of numerical modelling, a few conclusions can be drawn:

1. Extensive measurements on the bridge model present good correlation with 2D numerical models, suggesting a good quality of the database created;
2. The modal survey confirmed the existence of interaction between the cables and the deck/towers and led to a deeper understanding of the dynamic behaviour of this complex system;
3. The numerical response to some artificially generated time histories seems to be consistent with experimental measurements;
4. A comparison between numerical responses obtained on the basis of the OECS and the MECS models shows a trend for the last model to produce smaller responses than the first model, thus suggesting that mass distribution along the cables contributes to a damping of the global system. Note however that these results have been obtained for a limited set of input motions and that no aditional damping has been included in the analysis, these aspects needing further investigation.

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