Challenges of Active Seismic Control of Cable-stayed bridge: A case study

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SUMMARY:
The paper shows challenges in active control strategy in view of the discrepancies observed between the FE model of the Benchmark Control problem for cable stayed bridge originally proposed by Dyke *et. al (2000)* and the one updated in the present study using system-identification data. It has been demonstrated that the updated model is strikingly different from the original model in terms of the dynamic properties. The design of an active controller is based on States Space approach and the *states* depend on the numerical model of the structure. The challenges of the active control regime have been discussed keeping in view this theoretical background of design of active controller. It is concluded that though there cannot be a direct correlation between model updating and resulting control efficacy, the updated model will represent a control strategy which is closer to the reality.

Keywords: Active Controller, States Space, Modal identification, Model updating

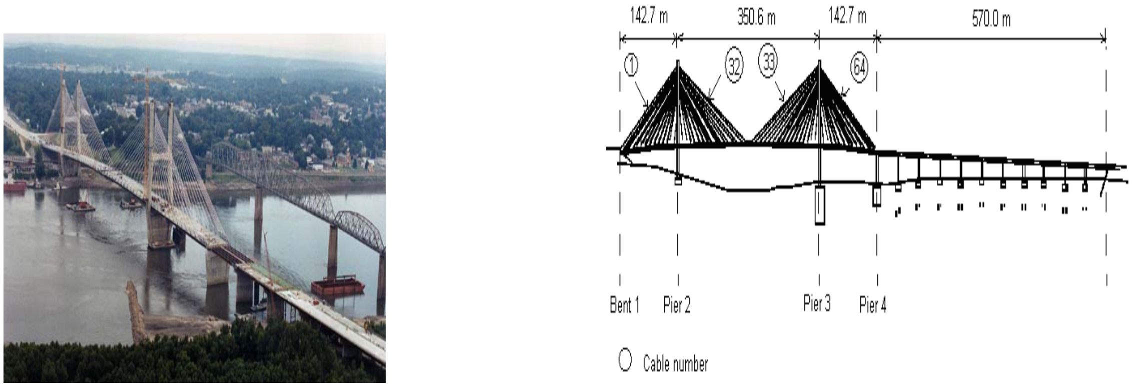
1. INTRODUCTION

Cable-stayed bridges have gained popularity in the category of long span bridges over the last three decades due to improved structural performance and aesthetic appeal in comparison to suspension bridges. It is appreciated that development of a numerical model, which simulates natural frequencies and mode shapes of the structure, is an important step for computation of dynamic responses of a cable-stayed bridge. However, building a numerical model to represent dynamic characteristics of a cable stayed bridge is rather difficult as this flexible structure exhibits complex behavior with the flexural, lateral and torsional motions being very often coupled. As a consequence, structural characteristics and responses predicted by idealized finite element (FE) models have discrepancies and errors as compared to those obtained from the actual measurements. Lack of proper idealization of the complex structure is regarded as one of the main causes of such discrepancies in a numerical model for a cable-stayed bridge. Modal identification through full-scale testing is the most reliable method to determine the true dynamics properties (e.g. natural frequencies, damping ratios and mode shapes) of a structure. This serves as a basis for validating and/or updating an analytical model of a structure so that the model represents the actual structural properties and the boundary conditions.

The protection of cable stayed bridges against seismic excitation is an active area of research. Active control system can offer the advantage of being able to dynamically modify the response of a structure in order to increase its safety and reliability. An active control strategy serves as a benchmark for evolving other control strategies. It also serves as a guide and reduces the number of iterations in building a real system for control implementation. In an active control strategy the control effort is dictated by the states of the numerical model and hence accuracy in modeling of the structure is of paramount importance. Thus, model updating would play an important role in evolving a more realistic active control strategy. The present work primarily addresses challenges of active control design in view of complexities in model updating. A real life cable stayed bridges have been considered for the study.

2. DESCRIPTION OF THE BRIDGE

The cable-stayed bridge used for this study is the Bill Emerson Memorial Bridge in Missouri, USA shown in Fig.2.1. Dyke *et. al.* (2000) formulated the benchmark control problem on this bridge with the design of an active controller against seismic excitation. The structural details of the bridge have been given in detail by Dyke *et. al.* (2000) and Caicedo (2003). The original and the line diagram of the bridge is shown in Fig. 2.1.



(a) The bridge (b) Line diagram of bridge[Dyke *et. al.* (2003)]

Fig. 2.1 Bill Emerson Memorial Bridge, Missouri, USA

3. FINITE ELEMENT MODEL

The FE model of Bill Emerson Memorial Bridge, Missouri, USA was developed for benchmark control problem for seismic response of cable stayed bridge [Dyke *et. al.* (2000]. The model was subsequently modified and transferred to Matlab® environment and was used for structural health-monitoring study by Caicedo (2003). The present model has been adopted from the study of Caicedo (2003). The finite element model has 573 nodes, 418 rigid links, 156 beam elements, 198 nodal mass elements and 128 cable elements. This has been illustrated in Fig. 3.1.

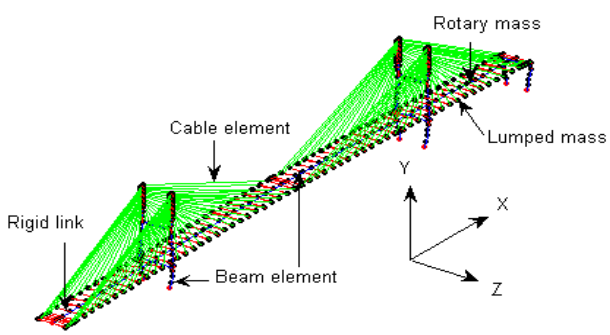


Fig. 3.1 Finite element model

4. ACTIVE CONTROL

The block diagram for the active control system followed in the benchmark problem has been illustrated in Fig. 4.1. The control regime is governed by an LQG system, where the states are derived from the nonlinear static deformed configuration of the bridge.

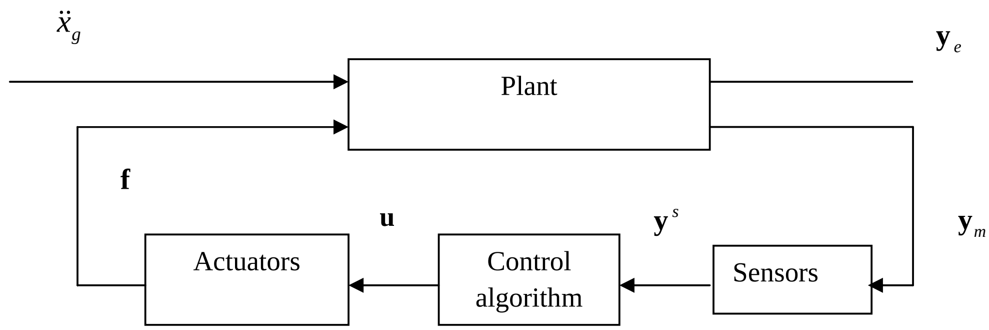


Fig. 4.1 Block diagram representation of active control system

This plant corresponding to the FE model, has been excited by the earthquake excitation $\ddot{x}_g$. The evaluation parameters $\mathbf{y}_e$ have been calculated. The measurements $\mathbf{y}_m$ have been fed to the sensors. The sensor output data has been given as input to the discrete controller as $\mathbf{y}^s$. The controller signal **u** has then been converted to an actuator signal **f**, which acts as input to the plant.

The sensors have been defined to measure the outputs of the control evaluation model as,

$$\dot{\mathbf{x}}^s = \mathbf{g}_1\left(\mathbf{x}^s, \mathbf{y}_m, t\right) \tag{4.1}$$

$$\mathbf{y}^s = \mathbf{g}_2\left(\mathbf{x}^s, \mathbf{y}_m, t\right) \tag{4.2}$$

where, $\mathbf{x}^s$ is the continuous-time state vector of the sensors measured in Volts. The discrete-time control algorithm takes the form:

$$\mathbf{x}_{k+1}^c = \mathbf{g}_3\left(\mathbf{x}_k^c, \mathbf{y}_k^s, k\right) \tag{4.3}$$

$$\mathbf{u}_k = \mathbf{g}_4\left(\mathbf{x}_k^c, \mathbf{y}_k^s, k\right) \tag{4.4}$$

where, $\mathbf{x}_k^c$ is the discrete-time state vector at each sampling time $t = kT$, $\mathbf{u}_k$ is the discrete time control command and $\mathbf{y}_k^s$ is the discrete-time input vector from the sensors. In may be mentioned that the analog signal from the sensors, $\mathbf{y}_m$ have been discretized in time and quantized through analog-to-digital (A/D) converter as $\mathbf{y}_k^s$. The actuator-structure interaction has been neglected in this study. The interfacing of the actuators with the bridge model has been done through,

$$\mathbf{f} = \mathbf{g}_5\left(\mathbf{u}_k, t\right) \tag{4.5}$$

$$\mathbf{y}_f = \mathbf{g}_6\left(\mathbf{u}_k, t\right) \tag{4.6}$$

where $\mathbf{f}$ is the continuous-time force output (kN) of the actuators applied to the structure and $\mathbf{y}_f$ is the continuous-time output vector from the control device output model, comprising forces produced by individual control devices, device stroke, device acceleration and has been used for evaluation of control strategy.

5. MODEL UPDATING

The studies by Song *et. al.* (2006) and Caicedo *et. al.* (2006) established limitations of the FEM model in simulating the experimentally observed modal data. This necessitates the updating of the FEM model.

5.1 System identification

The bridge was instrumented extensively for collection of ambient vibration data [Celebi (2006)] with a total of 66 accelerometers located suitably over the structure and the surrounding soil. System identification is conducted using Subspace Identification (SSI) Method using 66 accelerometers. The flowchart in Fig. 5.1 compares SSI method from that of classical identification method.

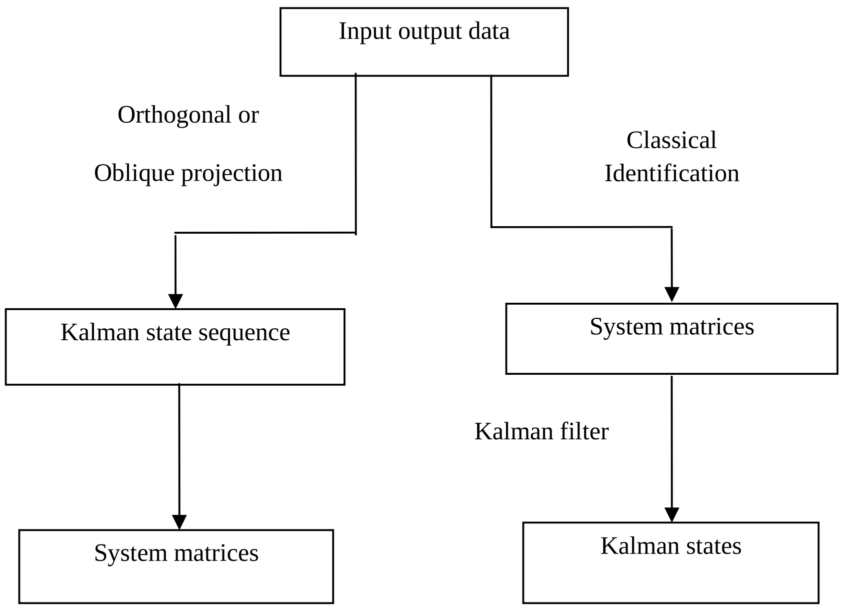


Fig. 5.1 Comparison between subspace method and classical method

Out of the total of 66 accelerometers, evenly distributed in the superstructure, substructure and surrounding soil, 23 are vertical sensors, 11 are sensors in the longitudinal direction of the bridge and remaining 22 are oriented in the transverse direction. Some key locations have been shown in the Fig. 5.2. Typical sensor-signals at two key locations have been shown in Fig. 5.3. These signals have been subjected to analysis for Power Spectral Density (PSD). Two typical results have been illustrated in Fig. 5.4. The peaks of the PSD curves indicate the predominant modal frequencies. The data from the sensors are subjected to Sub-Space Identification Process and modal characteristics have been shown in Fig. 5.5

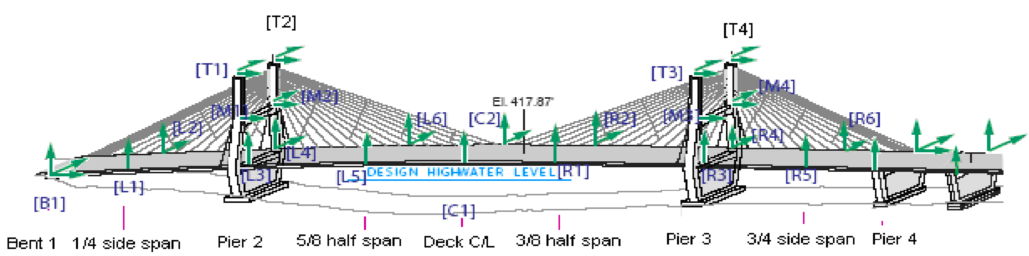
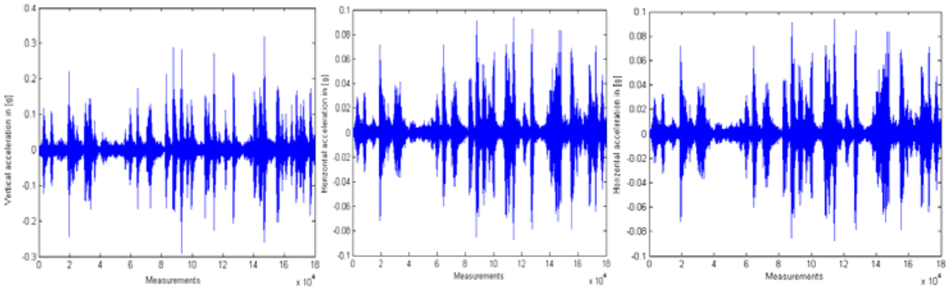
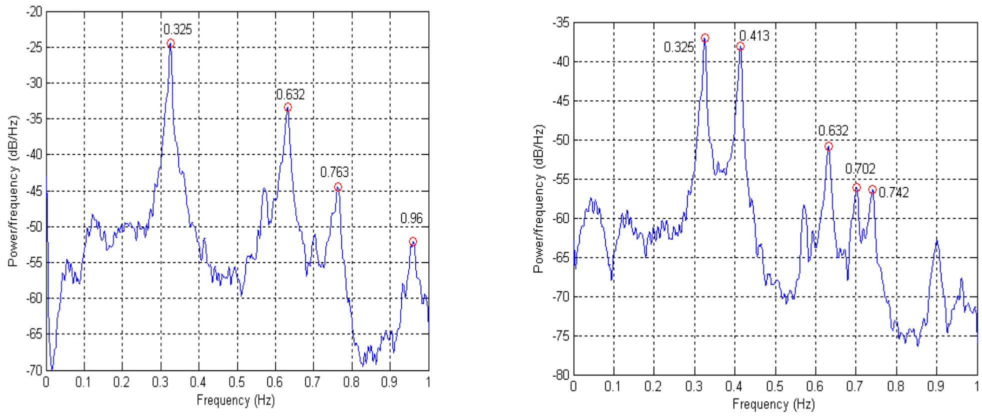


Fig. 5.2 Locations of sensors in the bridge



(a) mid-span (b) top of left pylon (c) top of right pylon

Fig. 5.3 Sensor signals at three key locations



(a) mid-span (b) top of left pylon

Fig. 5.4 Power Spectral Density of signals for two key locations

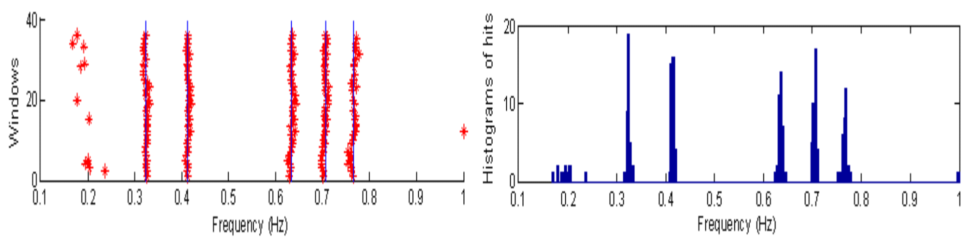


Fig. 5.5 Details of modal identification by SSI

5.2 Updating Parameters

Three different types of parameters have been considered for updating the finite element model: (i) mass of the deck, (ii) moment of inertia of the spine beam, and (iii) the rotational stiffness of the connection between the deck and the pylons. A total of 66 translational lumped masses have been used to model the deck. Assuming that the mass distribution is symmetric due to the symmetry of the bridge, this can be reduced to 33 parameters to optimize, which is still a large number. As it is unlikely to have a sudden change in the mass along the deck, the numbers of parameters has been reduced to 3 master masses at 3 locations along the deck. The other 30 masses have been calculated by fitting a spline between the 3 master masses. The support of the bridge at Bent 1, the support at Pier 2, and centre of the main span have been selected as the locations for the 3 master masses as shown in Fig. 5.6.

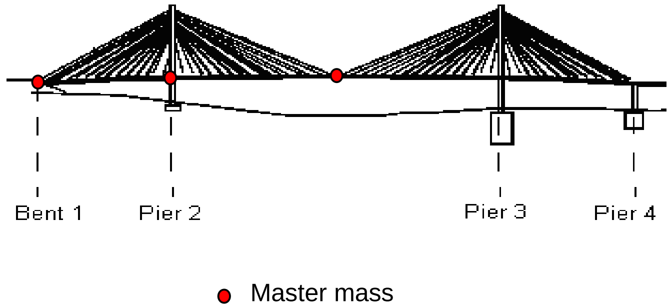


Fig. 5.6 Locations of master masses in the finite element modal

Two parameters have been used to update the connection between the deck and the supports. Using the symmetry of the bridge one parameter has been used for Bent 1 and Pier 4 and a second parameter has been used for the connection at the Pier 2 and Pier 3. The introduction of rotary stiffness to the connections has been done through spring element as illustrated in the Fig. 5.7 and Fig. 5.8. Only one parameter has been used to update the moment of inertia of the spine beam. Therefore, a total of six parameters to be updated have been utilized in this study.

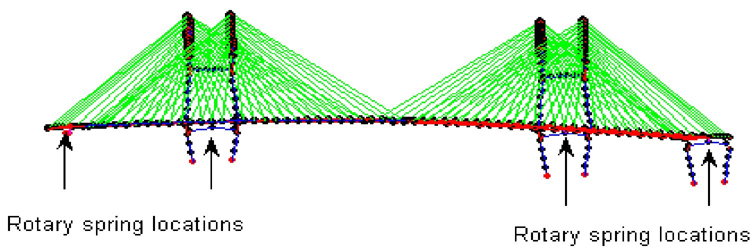


Fig. 5.7 Finite element model with added rotary springs

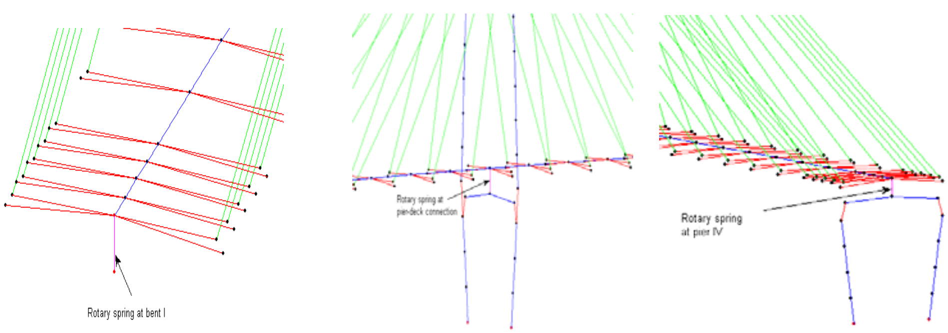


Fig 5.8 Spring connection in close view

The method used for model updating has three main parts: (i) find a local minima based on the base finite element model; (ii) obtain other solutions using HSJ; (iii) obtain a local minima close to the solutions obtained in the previous step. For all steps a trust-region for non-linear minimization algorithm available in the Matlab® optimization toolbox has been used to minimize an objective function. In the first step, the function to minimize is described by the equation,

$$f(\mathbf{p}) = \sum_{i=1}^{n} \sigma_i \left[1 - \mathrm{MAC}\left(\phi_{id,i}, \phi_{fe,i}(\mathbf{p})\right)\right] + \frac{1}{1-\beta_i} \left\| \frac{\omega_{id,i} - \omega_{fe,i}(\mathbf{p})}{\omega_{id,i}} \right\| \tag{5.1}$$

where n is the number of identified modes, MAC($\phi_{id,i}$, $\phi_{fe,i}(\mathbf{p})$) is the modal assurance criteria between the ith identified mode shape ($\phi_{id,i}$) and the ith mode shape of the finite element model $\phi_{fe,i}(\mathbf{p})$, $\omega_{id,i}$ is the ith identified natural frequency, $\omega_{fe,i}(\mathbf{p})$ is the ith natural frequency of the finite element model, **p** is the vector of parameters to be minimized, η_i and β_i are the weighting factors for the ith natural frequency and mode shape. The values of σ_i and β_i have been determined using the identified modal parameters. These weights account for the deviations modal characteristics of a particular mode of the updated model from that of the corresponding identified mode. Constraints have been applied to the procedure to assure that the variation of the parameters is not higher than reasonable limits. Constraints have been used to allow a maximum variation of 5% in the masses and the moment of inertia. The stiffness of the connection between the deck and the pylon can change from 0 to 100% of the highest value of stiffness matrix. The FE model by Caicedo (2003) has been used as the initial point for the optimization.

The hypothesis in this study is that the results obtained from the first optimization will provide a solution of the problem; but this solution may not be unique and could be a local minimum. A process similar to HSJ is used to provide other solutions, which are different from the original solution. Here, a new objective function is then defined to obtain the (l+1) th solution as,

$$g(\mathbf{p}_{l+1}) = \sum_{j=1}^{l} \frac{\mathbf{p}_j \cdot \mathbf{p}_{l+1}}{\|\mathbf{p}_j\| \|\mathbf{p}_{l+1}\|} \tag{5.2}$$

where $\mathbf{p}_j$ is the jth solution, $(\cdot)$ denotes dot product and $\|.\|$ denotes the norm. The value of this function is high if the dot product between the current solution and previous solutions is close to one, while the value of the function is low if the new solution is perpendicular to any other solution found before. Given that orthogonal vectors do not always provide a good fit for the finite element model, constraints have been used to assure that the new solution has a similar performance to the first solution. The constraint used for this optimization is described by the equation,

$$f(\mathbf{p}_{l+1}) \leq \alpha\, f(\mathbf{p}_l) \tag{5.3}$$

where, $f(\mathbf{p}_l)$ is the value of the objective function shown in Eq. 5.1 for the first solution found, $f(\mathbf{p}_{l+1})$ is the value of Eq. 5.1 for the current solution and α is a constant. In this study α has been selected as 1.06 so as to discard the solutions that vary more than 6% in value of the cost function from that of the first minimum. The result from this second step provides solutions to the problem that are significantly different in the values for updating parameters but have similar values for cost function. The updated model has been selected from these solutions is the one that has the most realistic update for the selected parameters.

6 RESULTS AND ANALYSIS

The results of the optimization problem have been enumerated in this section.

6.1 Optimization solutions

The solutions of optimization (the second step of the procedure) have been presented in Table 2. A total of three optimization solutions with comparative values of cost functions have been listed in the table. Different optimized parameters have been listed in the table corresponding to each of these alternative solutions. The optimized variations for mass as a percentage of original mass has been shown for the master locations as in Fig. 5.7. Similarly optimized variation of moment of inertia of spine has been listed for different solutions. Thirdly, the optimum value of rotary stiffness at the bent and piers have been listed as a percentage of maximum stiffness of the FE model by Caicedo (2003). Out of these alternative solutions, the most logical one has been selected for updating the finite element model.

The first solution is obtained using the model by Caicedo (2003) as the starting point. The differences in the three solutions have been appreciated from the tabular representation (Table 1). The first solution is based on decreasing the mass of the bridge, specially at the Pier 2 (Loc 2) and at the mid-span. The moment of inertia has also shown to be decreased marginally in this solution. In contrast, the second solution has suggested considerable increase in the rotational stiffness of the connection at the Bent 1 and Pier 4. The mass at the Pier 2 (Loc 2) is shown to be slightly lower than in the first solution while it suggests for only a marginal decrease in the inertia. On the other hand, the third solution lowers the moment of inertia of the spine beam significantly. In this solution the masses at the Bent 1 (Loc 1), Pier 2 (Loc 2) decreases uniformly while that in centre of the span decreases by the same amount as in the first solution. The solution further suggests for an increase in the rotary stiffness at the Bent 1. The values for the objective function $f(\mathbf{p})$ are also shown in the table and these are very similar for all the three solutions. These values are about 44% lower than the value of $f(\mathbf{p})$ for the original model by Caicedo (2003) which has been obtained as 1194.3. The second, third solutions have been observed to be within 6% of the first solution as specified by the parameter α in Eq. (5.3).

6.2 Modal characteristics of different solutions

Table 2 shows the first four natural frequencies of the model by Caicedo (2003) and those from the optimized models. The MAC values between the identified mode shapes and the numerical modes have also been represented in the table. It is clear that the optimization procedure has resulted in models which have greatly improved natural frequencies as compared to those of the FEM model by Caicedo (2003). The MAC values between identified and numerical modes were high in case of FEM model by Caicedo (2003). These get slightly reduced in case of the optimized models.

6.3 Selection of updated model

The first solution from Table 1 has been selected for updating the model. As indicated in the table, this solution is the most realistic solution with no unreasonable demand on parametric changes. The second solution has not has not been adopted in spite of having the lowest value for objective function as this solution puts an unreasonably high demand on the increase in rotary stiffness to the degree of 50.34%.

Table 1 Alternative solution from optimization

Solution	Mass (%)			Moment of Inertia (%)	Rotary Stiffness (%)		f(p) (Eq. 5.1)
	Loc1	Loc2	Mid-span		Bent 1 & Pier 4	Piers 2 &3	
1	-0.87	-5.00	-5.00	-2.05	0.02	0.00	494.8
2	-1.17	-4.70	-5.00	-0.18	50.34	0.00	493.8
3	-2.24	-2.03	-5.00	-4.95	8.68	0.00	498.4

Table 2 Modal characteristics for alternative solutions

Solution	ω_1	ω_2	ω_3	ω_4	MAC			
					$(\phi_{id,1},\phi_{fe,1})$	$(\phi_{id,2},\phi_{fe,2})$	$(\phi_{id,3},\phi_{fe,3})$	$(\phi_{id,3},\phi_{fe,4})$
Original FE model	0.291	0.392	0.608	0.674	0.955	0.947	0.968	0.973
1	0.307	0.413	0.635	0.703	0.956	0.944	0.960	0.967
2	0.308	0.413	0.635	0.703	0.956	0.943	0.957	0.966
3	0.307	0.413	0.635	0.702	0.955	0.942	0.957	0.965

6.4 Comparison of dynamic performance

The dynamic performance of the original FEM model as reported by [Dyke *et. al.* (2000)] has been presented in the Fig. 6.1

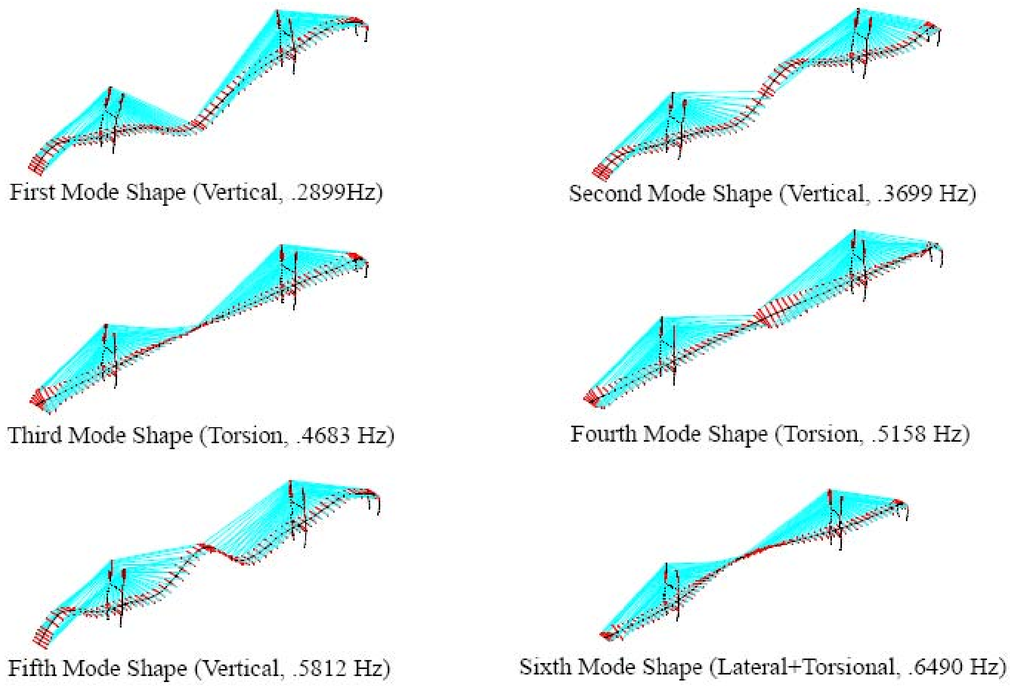


Fig. 6.1 Modal characteristics of original FEM model [Dyke *et. al.* (2000)]

The mode shapes from the updated model together with the identified modal ordinates at the sensor locations have been presented in Fig. 6.2. The black dots in the figure indicated the identified modal amplitudes corresponding to the sensor locations.

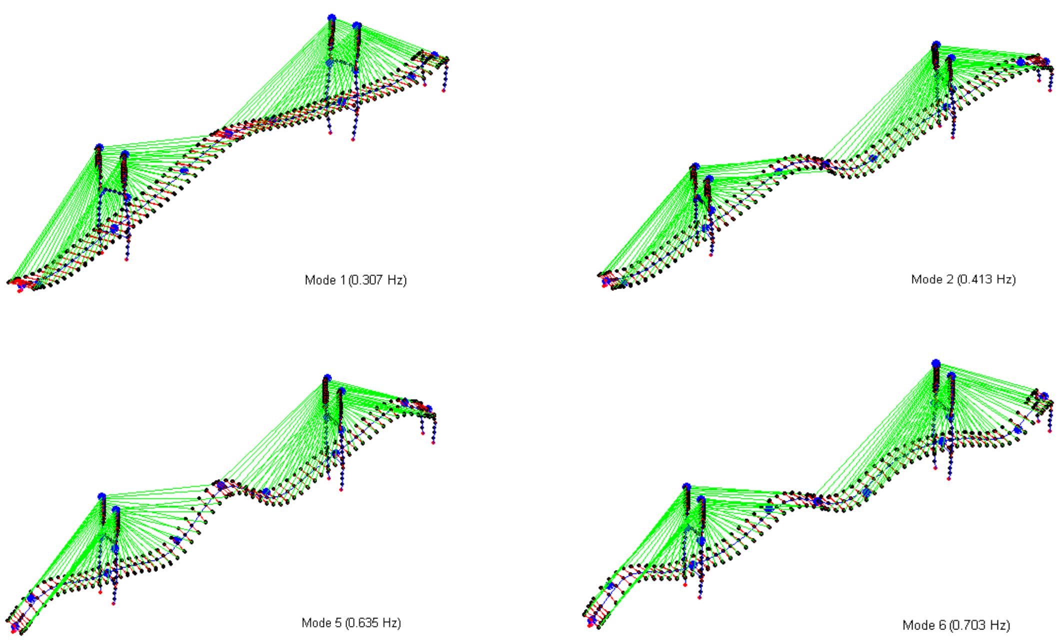


Fig. 6.2 Modes shown by updated FEM model

7 CONCLUSION

It has been observed that the dynamic performance showed by the updated model is quite different from the original FEM model. It is hence concluded that the model updating is an important issue that is to be addressed while devising any control strategy on a complex structure such as the cable-stayed bridge. However, the system identification being a complex issue, the same should be addressed properly. Unless this is done, active control cannot serve as a full-proof safety against seismic excitation of the structure. Thus the model updation and identification of modal parameter remains real challenge for the Engineers.

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