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Flutter and Buffeting Response of A Long Span Cable Stayed Bridges

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Abstract: **Aerodynamic stability of a proposed Cable Stayed Pre-stressed Concrete Bridge of span 480 m under wind loads have been studied. Flutter and buffeting responses due to wind loads was investigated on a sectional mode to the scale of 1:200. The model was tested in a wind tunnel for two values of damping (0.03 and 0.06) with different combinations of live loads at the ratio of N_θ / N_z =1.2. The model exhibited coupled vertical and torsional oscillations in wind. In addition, another uncoupled mode in the form of rolling oscillation about the longitudinal axis of the tunnel was also consistently observed. This type of oscillation has not been reported in the literature and is believed to be due to the overtone flexural oscillation of the main span of the bridge. After trying out several curative measures, it was found that provision of small holes in the bottom of the deck, controlled the vertical and rolling oscillations. The test results were compared with the theoretical (design) values and conclusions drawn for predicting flutter and buffeting responses due to wind loads. Fatigue tests were also conducted on ' Ganga Bridge ' (Haridwar, U.P, India) and suitable remedial measures were suggested to increase the life of the bridge.**

Key words— **Long-span bridges, Flutter, Buffeting, Aerodynamic selection, Preliminary design stage**

I. INTRODUCTION

Of the several bridges existing all over the world, built of different material or techniques developed, cable stayed bridges stand out as the most recent technological development. Stromsund Bridge was the first cable stayed highway bridge constructed in Sweden in 1955 with a central span of 183 m. Subsequently, a number of cable stayed bridges were constructed world over in many countries. Cable stayed bridges are considered to be the most suitable system for the medium long spans in the range of 100 m to 300 m. However, there has been a continuous endeavor to this span limitation. Tatara bridge (Japan) with a world record span of 890 m opened up the vision for researchers to study the adoption of cable stayed systems with spans exceeding 1000 m which has been hitherto suspension system. In India too, after the completion of Vidyasagar Sethu Bridge (also known as Second Hooghly Bridge) at Calcutta, which was the world’s longest (457.2 m) cable stayed bridge until 1992, cable stayed system found an appropriate place with wider adoption in the years to follow.

The bridge structure requires to be designed for static as well as for dynamic wind effects. Static wind loads are derived from an assumption of a steady uniform wind with lift, drag and moment forces. There have been many instances of bending and torsional oscillations of such bridges even at moderate speeds. The most spectacular case has been that of the original Tacoma Narrows Bridge which finally failed in a torsional mode of oscillation at a wind speed of 67 kmph(ASCE, 1948).Since the Tacoma Narrows bridge collapse in 1940, wind engineering researchers made great efforts to understand the aeroelastic phenomena associated with long span bridges; namely vortex shedding, galloping, divergence, *flutter*, and *buffeting* response. However, in particular, flutter instability and buffeting response of the Cable stayed bridge decks are important to obtain the aerodynamic stability and can be checked by conducting the wind tunnel tests, which are more accurate.

Flutter is an oscillatory instability induced in the bridge deck at a particular critical wind velocity leading to an exponentially growing response. One or more modes may influence this instability leading to failure due to excessive deflections and stresses. Flutter is the aeroelastic instability, which originates from the mutual interaction of elastic, inertial, damping and self-excited aerodynamic forces. It causes the bridge to oscillate in a divergent and destructive manner at the same critical wind velocity. Buffeting is the random response of a structure due to turbulence in the oncoming flow, or due to signature or self-induced turbulence. Buffeting response does not generally lead to catastrophic failures but is important from serviceability consideration. Fatigue is a process of progressive permanent internal structural change in a material subjected to repetitive stresses. These changes result in progressive growth of cracks and fracture. Fatigue is often described as *‘fatigue life’*, which essentially represents the number of cycles required to cause failure in the material under a given repetitive stress.

II. LITERATURE REVIEW

V. Numes, J.W.C and Person A.J., (1979) investigated the vibrational behaviour of cable-stayed bridge under wind loads in a two-dimensional model and wind tunnel experiments

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were carried out to establish the structural stability of the bridge and to determine resonance vibration due to vortex shedding (1979). Aerodynamic response of a bridge is affected by mean wind direction as well as turbulence. Bridge buffeting in yawed wind was investigated by Tanaka et al (1993) and Kimura et al (1992) and vortex excited oscillation on 2D and 3D models of rectangular section were investigated by Utsunomiya et al (1993). Several investigations [e.g; Miyata et al,(1994), Chen (1994), Tanaka et al (1993), Namini et al,(1992), Jones and Scanlan (1991), Bucher and Lin (1988,1989) and Lin and Yang(1983)] identified the problems of multimode response of long span cable bridges to wind excitation. Analysis of flutter and buffeting can be done in two ways using time domain methods of Bucher and Lin(1988,1989), or frequency domain methods of Tanaka et al (1993), Jones and Scanlan(1991), Scanlan and Jones and Lin and Yang (1983). Studies by Tanaka et al (1993), and Bucher and Lin (1988, 1989) proposed solutions to the multimode flutter and the multimode buffeting problems .Recently, multimode flutter and buffeting analyses were developed by Jain et al, (1996) based on frequency-domain methods and incorporated the theory of Scanlan and Jones (1990), taking into account the fully coupled aeroelastic and aerodynamic response of long span bridges to wind excitation. However, the extent of this coupling was not significant for the span lengths considered.

III. ANALYTICAL METHODS

Flutter

The frequencies of oscillation of the proposed bridge are identified and have been analysed based on the following assumptions:

(i) All spans have identical mode shapes during vibration.
(ii) The interference of the piers is neglected.
(iii) Shear deformation and rotatory inertia effects are negligible.

Since all the spans of the bridge are of the same length, one typical span of the bridge is considered for analysis. Rayleigh method has been used to obtain the frequencies of the bridge. The co-ordinates for a typical bridge span are selected and the mode shapes assumed for the first two modes of vibration are depicted.

For each segment, an assumed deflection profile of the form is

$$y(x) = A_1 \cos kx + B_2 \sin kx + Cx + D \qquad (1)$$

The various constants A, B, C, D are selected so as to satisfy the conditions at the joints. The torsional modes have again been analysed using the energy method. It is assumed that the piers and towers do not deform when the deck undergoes twisting deformation about a longitudinal axis. Under this assumption, it is adequate to consider strain and kinetic energies in the deck and in the cable systems. Let θ_0 (x) represent the amplitude of rotation at any point in the deck. Then, maximum strain energy in the deck is

$$V_s = \frac{GJ}{2}\int_0^l \left(\frac{d_{\theta_0}}{dx}\right)^2 dx \qquad (2)$$

where GJ represents the torsional rigidity of the deck, l = half the span of the main deck,

Maximum Strain energy in the cables is

$$V = 2\,\frac{1}{2}\sum_{k=1}^{3}\frac{A_k E_s}{I_k}\cos^2\theta_k(\theta_{0k}s)^2 \qquad (3)$$

Where 2s the distance between two parallel sets of cables on the other side of the deck and

θ_{ok} the amplitude of rotation at the k^{th} cable.

Total maximum strain energy V = V_s + V_c

Maximum kinetic energy in the deck:

$$T = \frac{\gamma I_\theta p^2}{2}\int_0^l \theta_0^2 dx \qquad (4)$$

$$\theta_0(x) = a_1 \sin\left(\frac{\pi x}{2l}\right) + a_3 \sin\left(\frac{3\pi x}{2l}\right) \qquad (5)$$

shear centre and γ is the mass of deck material per unit volume. The torsional frequencies can now be obtained from the condition δ(V-T) = 0. The shape function θ_0 (x) is now expressed as,

This expression satisfies the condition that the rotation of the cross section is zero at the supports. By means of the Rayleigh-Ritz procedure, the frequencies turn out to be:

First symmetric mode : 1.19 H_z
Second symmetric mode : 1.69 H_z

Cable stays: High fatigue resistant DINA (Brand name of M/s BBR product) cables are considered in this bridge. The length of the cables varies from 37 m to 113 m and the number of 7 mm dia. HT wires in each stay cable varies from 96 to 264. The ultimate tensile strength (UTS) of HT wires is taken up to 1570N/mm 2 and the required fatigue stress is 180 N/mm 2 with an upper limit of 0.45 UTS. These cables are designed to carry ultimate tensile forces in the range of 5,800 kN to 15,950kN. Frequency of cables was calculated and given in

$$U_{cr} = \frac{bN_\theta}{K\sqrt{x}} = 33.205\ m/s$$

$$or \qquad U_{cr} = 119.5\ kmph.$$

$$E = \mu r_\alpha^2\left[l - \left(\frac{W_\alpha}{W}\right)^2 (l + ig_\alpha)\right] - \frac{l}{2}\left(\frac{1}{2} + a_h\right) + M_\alpha$$

$$- L_\alpha\left(\frac{1}{2} + a_h\right) + L_h\left(\frac{l}{2} + a_h\right)^2$$

Table.1.

Cable No.	Dia (mm)	Length (m)	Tension (kN)	Frequency (Hz)	Velocity (m/s)
1.	76	55.16	1890	2.50	2.100
2.	78	57.73	1970	2.20	1.900
3	82	61.03	2153	1.95	1.750
4.	85	65.27	2370	1.80	1.600
5.	90	69.30	2630	1.70	1.400
6.	96	76.23	3140	1.55	1.250

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7.	100	85.43	3405	1.25	1.150
8.	105	96.05	3670	1.10	1.020
9.	107	113.5	3935	1.05	0.905

The two-dimensional equations of motion for an airfoil are

$$A\frac{h}{b} + B\alpha + C\beta = 0$$

$$D\frac{h}{b} + E\alpha + F\beta = 0$$

$$G\frac{h}{b} + H\alpha + I\beta = 0 \qquad (6)$$

In the present case, torsion - bending flutter of the bridge was considered for the two dimensional section. $\begin{vmatrix} A & B \\ D & E \end{vmatrix} = 0,$ The flutter determinant was solved by Theodorsen's method

Where,

$$A = \mu\left[l - \left(\frac{N_\theta}{N}\right)^2 \left(\frac{N_Z}{N_\vartheta}\right)^2 \left(l + ig_h\right) \right] + L_n$$

$$B = \mu X_\alpha + L_\alpha - L_\eta \left(\frac{l}{2} + a_h\right)$$

$$D = \mu X_\alpha + \frac{l}{2} - L_n \left(\frac{l}{2} + a_h\right)$$

$$\left(\frac{N_\theta}{N}\right)^2 = X$$

Let real and imaginary roots were plotted Vs l /k. It was seen that the real and imaginary equations were both satisfied at the intersection of the curves when l/k=6.05 and $\sqrt{x}$ = 1.1425

Hence, the corresponding critical flutter speed

$$U_{cr} = \frac{bN_\theta}{K\sqrt{x}} = 33.205\ m/s$$

$$or \qquad U_{cr} = 119.5\ kmph.$$

Buffeting

Equations of motion for a two-dimensional section of the bridge under buffeting excitation may be generalized from eqs. (7) And (8) by adding time-dependent buffeting lift and moment per unit span, respectively, as follows:

$$L(t) = L(s) = \frac{1}{2}\rho\bar{v}^2(2B)c_L(s) \qquad (7)$$

$$M(t) = M(s) = \frac{1}{2}\rho\bar{v}^2(2B^2)c_M(s) \qquad (8)$$

where c_L (s) and c_M (s) are time-dependent lift and moment coefficients and v is the mean wind velocity.

The span-wise integrals, namely C_L(s), C_M(s):

$$C_L(s) = \frac{1}{L_m}\int_0^{L_m} c_L(x,s)dx \qquad (9)$$

$$C_M(s) = \frac{1}{L_m}\int_0^{L_m} c_M(x,s)\,dx \qquad (10)$$

The equation for single-degree vertical motion with $\xi = \frac{h}{B} is$

$$\xi'' + 2\zeta_\zeta K_\zeta \xi' K_\xi^2 \xi = \frac{\rho B^2}{m}[KH_1^*]\xi' + C_L(s)] \qquad (11)$$

Or in adjusted notation

$$\xi'' + 2\gamma_1 K_\xi \xi' K_\xi^2 \xi = \frac{\rho B^2}{m} C_L(s) \qquad (12)$$

Where $\gamma_1 = \zeta_\xi - \frac{\rho B^2}{m}\frac{K}{K_\xi}H_1^*(K)$ (13)

C_L(s) is stationary random, of power spectral density (K), and the definition is introduced:

$$T(iK) = \frac{1}{K_\xi^2 - K^2 + 2i\gamma_1 K_\xi K} = \frac{1}{K_\xi^2[1 - \left\{\frac{\omega}{\omega_\xi}\right\}^2 + 2i\gamma_1\frac{\omega}{\omega_\xi}]} \qquad (14)$$

Then the power spectral density of ξ is given by

$$S_\xi(K) = |T(iK)|^2 \left[\frac{\rho B^2}{m}\right]^2 S_{C_L}(K) \qquad (15)$$

Since $|T(iK)|^2$ is experimentally obtainable from the ratio S_ξ / S_{C_L}, it is then possible to solve this for the value of γ_1, and hence H_1^*(K) as in eq. (11).

An alternative approach, if C_L(s) is not steady but a transient (decaying or divergent) one, is to employ Fourier transforms $\bar{\xi}$ $\bar{C}_L$ respectively of a selected portion of the motion ξ and input C_L(s), hence:

$$T(iK) = \bar{\xi}/\bar{C}_L$$

The non-dimensional dynamic equation for span wise section x of the full bridge is

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$$\theta^{''}+2\,\xi_A K_A\;\theta + {K_A}^2\theta$$

$$= \rho B4 \,/\; I^{s}\;[\,KA_2\;{}^{*\Theta} + KA_3\;{}^{*\Theta} + C_M(x,s)] \quad (16)$$

where all K parameters are based on mean wind velocity v, c_M (x,s) the local randomly varying moment coefficient .

$$\theta = \sum_{i=1}^{N} \Psi_1(X)\eta_i(s)$$

Unlike the flutter phenomenon, where the entire system acts concertedly at a single value of K = K_c, gust response occurs in several simultaneous modes

ψi (x) each of which having a specifiable value K = K_i.

Assuming that $\int_{Span} \psi i^{2}(x)\, dx = I\; I_s$

The equation pertaining to mode i of the bridge becomes.

$$\eta_i^{''} + 2\;\zeta_i\;K_i\;\eta_i^{'} + k_i^2\eta_i = \frac{\rho\,B^4}{I}\Big[C_{\psi\psi}\;K_i\;A_2^*\;(K_i)\;\;\eta_i^{'} + C_{\psi\psi}\;K\;A_3^*\;(K_i)\;\;\eta_i + \int_{Span} c_M(x,s)\psi_i(x)dx\Big] \quad (17)$$

The problem then devolves into determining the random force in eqn. (16) and its consequences and becomes.

$$\eta_i^{''} + 2\,\gamma_i\,K_i\,\eta_i^{'} + K_i^2\,\eta_i^{'} = \frac{\rho B^4}{I}\,L_o \sum_{j=1}^{N_B}\,(C_M(s))_j\,\psi_i\,(x_j) \quad (18)$$

The spectrum of the amplitude of response η_i for the i[th] mode ψ_i (x) is then given by

where,

$$S_{\eta_i}(K) = \sum_{j=1}^{N_B}\sum_{r=1}^{N_B}\beta_i^*(x_q,K)\;\;\beta_i\,(x_r,K) = S_{C_M}\;(q,r;K) \quad (19)$$

$$\beta_i\,(x_q,k) = \left[\frac{\rho B^4 L_o}{I}\right]\psi_i\,(x_q)\,/\,(K_i^2 - K^2 + 2i\,\gamma_i\,K_i\,K) \quad (20)$$

and β* (x_q, K) is its complex conjugate.

Since the sum of all modal responses constitutes the total responseθ, it is then necessary to assess S_θ (x, K) of θ (x) for all modal contributions at the given span-wise point x, to take into account all modal and cross-modal contributions. But it is likely for this fairly lowly-damped case that a good estimate can be made by taking the sum as squared modal contributions

$$S_\alpha\;(x,\,K) = \sum_{i=1}^{N} S_{\eta_i}\;(K)\;\;\psi_i^2\;(x) \quad (21)$$

which then constitutes the result for the spectrum of response. Other details on estimated maximum of response, number of deflection excursions contributing to fatigue, etc. can be pursued by known techniques. From the equations 7 to 21, it is possible to predict the buffeting response of a bridge analytically.

IV. EXPERIMENTAL INVESTIGATIONS

A model was built to a scale of 1:200 representing a 240 m length of the bridge span. Well-seasoned teakwood was used to fabricate the model. Various components of the model, such as the main girder, longitudinal beam, cross beam, deck slab, foot path, hand railing, camber and fillet were all fabricated separately and carefully assembled to get the replica of the prototype. Two designs for the hand railings, one with an ornamental design and the other, a plain design fabricated using angle iron, are investigated. The important physical properties and dimensions of the model and were shown in Table 2. The ratio of flexural to torsional frequency (N_θ/ N_z) equals to 1.2 is maintained by adjusting the frequency of oscillation of the model with some limitations. The frequency and amplitude of oscillations were measured by means of accelerometers with preamplifier. The model frequencies and its damping were measured by giving an impulse and also by using an electro-dynamic shaker.

Damping was measured by giving an impulse disturbance to the model and recording the decaying signal to arrive at the logarithmic decrement. Freeman, Fox and Partners (designers of the bridge) had specified a damping value δ_s of 0.06 in both bending and torsion and a value close to this was obtained in flexure by adjusting the size of damper disc. The location of the damper disc on the longitudinal bar was then adjusted to obtain a torsional damping of nearly 0.06. A number of tests were conducted even at this lower value of damping i.e 0.03, as some bridges are known to possess such low values of damping.

Measurements were made for vertical and torsional oscillations with and without live loads for the damping values of 0.06 and 0.03 at positive and negative angles of attack ranging from 0 to 7.5 degrees at intervals of 2.5 degrees. In the present investigation, a distinct single degree of freedom oscillation in the rolling mode has also been consistently observed. In configurations where torsional oscillation occurred, the rolling mode of instability was seen to occur almost immediately. The rolling mode of instability seen in sectional model tests may be described as due to the overtone flexural oscillation of the main span of the bridge.

Table. 2

S No	Property	Scale Ratio	Model Value	Full Scale Value
1	Width	1:100	0.11m	10.54m
2	Effective area of cross section	$1{:}100^2$	$5.86\text{X}10^{-4}$ m^2	5.865 m^2
3	Location of the Neutral Axis below	1:100	0.5135 cm	51.35 cm

	top of the deck			
4	Shear center above top of the deck	1:100	0.874 cm	87.4 cm
5	Weight per unit length	$1:100^2$	2.5 kg/m	25040 kg/m
6	Mass moment of inertia per unit length	$1:100^4$	0.0025 kgm^2/m	253600kg m^2/m
7	Frequency of Oscillation: a) Vertical motion b) Torsional	 6:1 6:1	 6.0 Hz 7.2 Hz	 1.00 Hz 1.19 Hz
8	N_θ/N_z	1:1	1.2	1.2

V. RESULTS AND DISCUSSIONS

From the results of wind tunnel tests on the sectional model with various configurations, it is evident that corresponding full-scale values can be obtained by applying appropriate scale factors. With regard to the two flexural modes, the second flexural mode of 1.07 Hz was considered along with the torsional mode of 1.20 Hz to get the lowest value of N_θ/N_z for model simulation. However, since the oscillations of the model were uncoupled, oscillation in each of those modes had to be interpreted independently. The basic model, without any modifications, vibrates with a maximum amplitude of 1.61 mm and with a wind speed of 15.5 m/s, the value of δ_{zx} was 0.03 with angle iron hand railing. The central amplitude for the full-scale bridge turns out to be 64.4 mm. This amplitude in the full-scale structure occurring at a speed of 50 m/s is seen to be fairly large and remedial measures to reduce this amplitude would become inevitable. When the hand-railing is of the ornamental type, the model vibrates with more or less the same amplitude. The speed range for instability is also approximately the same. With the introduction of dashpots, the value of δ_{zx} is raised to 0.06 and the maximum amplitude of the model with angle iron hand railing is brought down by about 40 percent. The vibration of the model with ornamental hand-railing is insignificant by the introduction of dashpots.

It is seen that a positive angle of attack leads to slightly increased amplitude of oscillation, the speed range for instability being essentially unaffected. With a negative angle of attack the amplitude is reduced significantly and the speed for the inception of instability is also higher by about 40 percent. Maximum amplitude of 1.97 mm was observed for the model with ornamental hand railing when the truck convoys were moving in the same direction on the leeward side and at 0.25 m from the centre. For the angle iron hand railing, the maximum amplitude was 0.82 mm, when the truck convoy was moving in the same direction, very close to the centre, on the windward side. The ranges of wind speed for instability in vertical motion were not affected by the presence of tank or truck as live loads. It is the amplitude of oscillation, which tended to be much more pronounced.

The wind tunnel tests showed that the sectional model of the bridge oscillated in the torsional mode as the wind speed was increased beyond the range for instability in vertical motion. The sectional model with ornamental hand railing showed instability in torsion between wind speeds of 30 m/s to 35 m/s. The maximum amplitude observed was 0.9^0, when $^{\delta}\theta_s$ was 0.03. There was a slight reduction in the amplitude; the value of $^{\delta}\theta_s$ was raised to 0.06. With the introduction of the tank live load on the model with ornamental hand railing, the torsional amplitudes were pronounced. The maximum amplitude of 1.5^o occurred when the tank was on the windward side and 0.5 m from the centre. The smallest amplitude was 0.95^o and was realized with the tank on the leeward side and close to the centre. When the hand railing was of the angle iron type, it did not record in any measurable torsion. The maximum amplitude for this case was as small as 0.1^0 when the tank was located on the windward side at 0.5 m from the centre.

As the wind speed was increased beyond the range of torsional instability, it was found that the model was soon oscillating in the rolling mode. For the bare model with ornamental hand railing, the rolling oscillations started when the wind speed was 30 m/s and continued upto a wind speed of 40 m/s. The maximum rolling amplitude was 0.04°. With the introduction of dashpots, the maximum amplitude for both the types of hand railing was 0.02°. At a positive angle of attack of 10°, the rolling amplitude was 0.04° for the model with ornamental hand railing and dashpots. With the angle of attack at -10°, the amplitude came down to 0.02° and speed for the onset of instability raised by about 20 percent.

VI. CONCLUSIONS

The following conclusions are arrived at from the present study:

1).From the results of the flutter test on the bridge model, it is observed that the model showed oscillation mainly in the bending mode and relatively weaker in the torsional mode. The oscillation tended to be larger at positive angles of attack and smaller at negative angles of attack, without any significant effect on the critical speed of the wind.

2). The bridge deck is more susceptible to wind excited oscillations of high amplitudes under live loads. It is also observed that a rolling mode of instability occurs for long span cable stayed bridges. Further, coverage of the bottom of the deck proved to be the most effective modification in increasing the stability of the bridge in bending and rolling.

3). While recording buffeting response of the bridge, it is observed that the torsional instability of a bridge is not affected by moderate turbulence. It is also observed that the component of wind, normal to the deck, governs the torsional instability and buffeting behaviour for the yawed wind attack.

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