

Validation analyses of integrated procedures for evaluation of stability, buffeting response and wind loads on the Messina Bridge

A. Zasso^a, G. Diana^a, L. Rosa^a, A. Pagani^a, S. Stoyanoff^b, P.O. Dallaire^b,
E. Vullo^c, D. Khazem^d, K. Serzan^d

^a*Politecnico di Milano, Dipartimento di Meccanica, Via G. La Masa 1, Milano, Italy*

^b*RWDI, 109 bd. de Bromont, Bromont, Québec J2L 2K7, Canada*

^c*Stretto di Messina*

^d*Parsons, 100 Broadway, New York, NY 10005, USA*

ABSTRACT: As a part of Messina Strait Bridge independent checking, numerical analysis was carried out to evaluate the bridge aerodynamic response and associated wind loads. The topic of this paper is a comparative presentation of the results obtained by RWDI and Politecnico di Milano (POLIMI) through independent theoretical and numerical methods, on the basis of an identical scheme of the structure and the same aerodynamic and wind turbulence properties.

KEYWORDS: Suspended bridge, Flutter, Stability analysis, Buffeting analysis, Wind Loads, Messina Strait Bridge

1 INTRODUCTION

The framework of the analysis consists in drawings provided by the bridge contractor Eurolink designers COWI, and mass distribution and structural model (eigenvalues and eigenvectors) as provided by Parsons Transportation Group (PTG), the Project Management Consultants for the Stretto di Messina Project. The behavior of the bridge under strong winds was predicted numerically based on experimental data from sectional model wind tunnel tests, carried out on a 1:45 scaled deck sectional model by Politecnico di Milano (POLIMI) and on a 1:100 scaled sectional and aeroelastic models of the tower by RWDI. Aerodynamic static coefficients of the twin main cables were supplied by the high Reynolds number sectional model tests performed on behalf of the bridge contractor Eurolink [Schewe and Jacobs, 2010].

The generalized coordinate numerical approach was applied allowing comparative verifications through independent analysis by RWDI and POLIMI on stability, buffeting and wind loads using similar, yet completely independent, numerical approaches and procedures.

2 THEORETICAL FRAMEWORK OF STABILITY AND RESPONSE ANALYSIS

Analyses were performed using modal approach, solving the bridge dynamics equations of equilibrium in generalized coordinates through the state space matrix of the first 100 modes.

The aerodynamic stability of the structure was investigated using a numerical recursive method for taking into account the dependence of the reduced velocity in both the mean wind speed and the frequency of each mode. The aeroelastic terms were introduced by experimental coefficients (Flutter Derivatives).

The adopted numerical method allows for obtaining system eigenvalues as a function of the actual mean wind speed and the actual angle of attack (depending on the bridge average static configuration that is a non-linear function of the mean wind speed itself). Finally, the method

applies an innovative stability criterion focusing on the residual damping parameter, instead the traditional "flutter speed limit" criterion.

To describe the bridge response to turbulent winds, a time domain analysis was performed, based on time-space wind distributions in agreement with the local wind statistical parameters. Aeroelasticity effects were introduced with experimental Flutter Derivatives and buffeting loads were generated based on the quasi-static theory corrected with experimental aerodynamic admittances.

Based on the above theoretical background, independent solution methods were developed and implemented by POLIMI and RWDI producing two sets of "wind loads" that would cover fully the envelope of the expected time domain responses of the bridge to turbulent winds at the required limit states for structural design.

2.1 Stability Analysis

Using the described above theoretical framework, the stability analysis has been performed and its results compared to S.d.M. specifications. The stability of all important modes under wind action was verified.

The eigenvalues analysis takes into account the wind-structure aeroelastic effects using the Flutter Derivatives in order to simulate the state dependent forces and moments, accounting for the dependence of the local angle of attack and the frequency of every mode being analyzed. This procedure allows examining the evolution of the total modal damping (structural plus aerodynamic) as a function of the mean wind speed.

An innovative aspect of this independent check is that for the first time, according to S.d.M. specifications, in order to evaluate the stability performances of a suspended bridge, the key-factor of the analysis is the residual damping of each modes instead of the usual value of critical flutter velocity, corresponding to the lowest value of mean wind speed that causes the vanishing of the residual damping. The residual or total damping, also called "stability index", is defined as $\alpha_i/\omega_i = f(V^*)$ where α_i and ω_i are the real and imaginary part of eigenvalues related to the i^{th} mode with frequency f_i being a function of the reduced velocity $V^* = V/(f_i B)$ (B is the deck width). It comprises structural and aerodynamic damping at given wind speed. Due to the relatively low level of the damping of each mode, this residual damping is the linearization of the theoretical modal damping without losing accuracy.

The residual damping is obtained solving the "Eigenvalues problem" of the coupled mechanic-aerodynamic system:

$$[m_s^*]\ddot{q} + [r_s^*]\dot{q} + [k_s^*]q = \underline{F}_{Aero} = [\Phi]^T \underline{F}_{Aero}(V, V^*, \eta) \quad (1)$$

where $[m_s^*]$, $[r_s^*]$ and $[k_s^*]$ are the structural mass, damping and stiffness generalized matrixes. Structural damping ζ_s is set equal to 0.3% of critical for all deck and towers modes while for the main cables pure modes without deck motions, $\zeta_s = 0.03\%$ representing the expected values for this structure and modes under consideration. The vector q and the matrix $[\Phi]$ contain respectively the modal coordinates and the modal vectors; $\underline{F}_{Aero}(V, V^*, \eta)$ expresses the aeroelastic forces as a function of mean wind speed V (equal for each mode) and of the reduced velocity V^* that changes for each modes and the angle of attach η (due to mean wind static loads on the structure obtained by a non-linear iterative static procedure) that changes for each section.

The aerodynamic loads of each section were expressed using the Flutter Derivatives in the POLIMI convention[Zasso, 1996]. POLIMI used a modified formulation which includes the definition of an aerodynamic inertial matrix:

$$\begin{Bmatrix} F_y / qBL \\ F_z / qBL \\ F_\theta / qB^2L \end{Bmatrix}_{\substack{\text{Deck section} \\ \text{Local}}} = [M^*_{aero}] \begin{Bmatrix} \ddot{y} \\ \ddot{z} \\ \ddot{\vartheta} \end{Bmatrix}_{\substack{\text{Deck section} \\ \text{Local}}} + [R^*_{aero}] \begin{Bmatrix} \dot{y} \\ \dot{z} \\ \dot{\vartheta} \end{Bmatrix}_{\substack{\text{Deck section} \\ \text{Local}}} + [K^*_{aero}] \begin{Bmatrix} y \\ z \\ \vartheta \end{Bmatrix}_{\substack{\text{Deck section} \\ \text{Local}}} \quad (2)$$

where the aerodynamic matrixes are:

$$\begin{aligned} [M^*_{aero}] &= - \begin{bmatrix} p_6^* \frac{\pi B}{2V^2} & p_4^* \frac{\pi B}{2V^2} & 0 \\ h_6^* \frac{\pi B}{2V^2} & h_4^* \frac{\pi B}{2V^2} & 0 \\ a_6^* \frac{\pi B}{2V^2} & a_4^* \frac{\pi B}{2V^2} & 0 \end{bmatrix} & [R^*_{aero}] &= - \begin{bmatrix} -p_5^* \frac{1}{V} & -p_1^* \frac{1}{V} & -p_2^* \frac{B}{V} \\ -h_5^* \frac{1}{V} & -h_1^* \frac{1}{V} & -h_2^* \frac{B}{V} \\ -a_5^* \frac{B}{V} & -a_1^* \frac{B}{V} & -a_2^* \frac{B^2}{V} \end{bmatrix} \\ [K^*_{aero}] &= - \begin{bmatrix} 0 & 0 & p_3^* \\ 0 & 0 & h_3^* \\ 0 & 0 & a_3^* B \end{bmatrix} \end{aligned} \quad (3)$$

To consider the Flutter Derivatives not identified through Wind Tunnel tests ($a_{5,6}^*$; $h_{5,6}^*$; $p_{5,6}^*$) the quasi-static approximation using the static aerodynamic coefficients was adopted:

$$\begin{aligned} \{a^*, h^*, p^*\}_6 &= 0 \\ \begin{cases} p_5^* = 2C_D \\ h_5^* = 2C_L \\ a_5^* = 2C_M \end{cases} \end{aligned} \quad (4)$$

where C_D , C_L and C_M are respectively the drag, lift and moment aerostatic coefficients of the bridge deck section.

The same procedure is adopted for the main cables using the static coefficients provided by the dedicated wind tunnel tests [Schewe and Jacobs, 2010], and for the hangers considering only the drag contribution adopting a C_D derived from literature [Zdravkovich, 1997].

	C_{D0}	K_{D0}	C_{L0}	K_{L0}
Main Cables	0,52	0	0	-2.6767
Hangers	0,6	0	0	0

Table 2-1 Main Cables and hanger aerodynamic coefficients

The quasi-static approximation was also used for the two towers. Since their influence was judged small on stability estimation, the towers were included only for buffeting analysis in the POLIMI model. Given the same numerical model and solver was applied by RWDI, towers were included both into flutter stability and buffeting analyses.

Stability assessment was done using a modal approach on the basis of modal properties of the bridge provided by Parsons. The first mode becoming unstable is the first torsional anti-symmetric one (#4 - $f = 0.080$ Hz); Table 2-2 reports a summary of the evolution of the residual damping at different angle of attach and at different mean wind speed value. Figure 2-1 shows the behavior of the first torsional anti-symmetric mode and the corresponding vertical and lateral modes those are directly involved in flutter. Results are presented considering or disregarding the main cables effect.

η [deg]	V_n [m/s]		Wind speed[m/s]											
			10		44		47		54		60		75	
-4	∞	∞	1,25	1,36	2,86	3,20	3,04	3,41	3,46	3,87	3,79	4,25	4,39	4,98
-2	91	98	0,98	1,07	3,12	3,46	3,24	3,61	3,40	3,83	3,34	3,82	2,22	2,82
0	83	87	0,71	0,78	2,73	3,08	2,79	3,17	2,79	3,22	2,58	3,07	1,06	1,65
2	88	93	0,55	0,60	2,90	2,94	3,02	3,15	3,17	3,65	3,12	4,00	1,97	4,53
4	83	88	0,47	0,55	2,70	3,04	2,81	3,17	2,90	3,31	2,71	3,17	1,15	1,70

Table 2-2 Stability limit and residual damping of the 1st torsional mode with and without (gray cells) the main cables

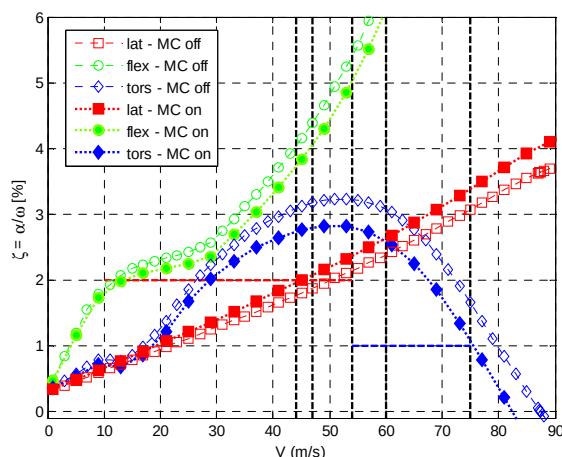


Figure 2-1 Residual damping of 1st lateral, vertical and torsional modes as functions of mean wind speed. Results of simulation with and without main cables. Assumed angle of wind attack 0°.

The project requests for aerodynamic specifications stability were satisfied for all fundamental modes controlling the deck stability at angle of attack $\eta=0^\circ$ as showed in Figure 2-2 that collects the evolution of all modes as requested by the project specification (modes: from 1 to 4 torsional asymmetric). The torsional instability is overcome at higher wind speed due to the regular positive aerodynamic damping shown by the asymptotic trend of the a_2^* Flutter Derivatives at higher reduced velocity.

Due to negative lift coefficient of the Main Cables in twin configuration (as reported in Table 2-1) some higher cable modes show quite low damping. This problem must be addressed in the “Esecutivo” phase but it is easily resolvable.

Based on the same set of full data and theoretical framework, RWDI applied its procedure called “3D Flutter Analysis” [Stoyanoff, 1993] modified to include POLIMI’s derivatives formulation. On the selected trial case at zero degree, mean angle of wind attack, almost identical residual damping traces (Figure 2-1) were predicted and critical flutter speeds within 2.5 m/sec.

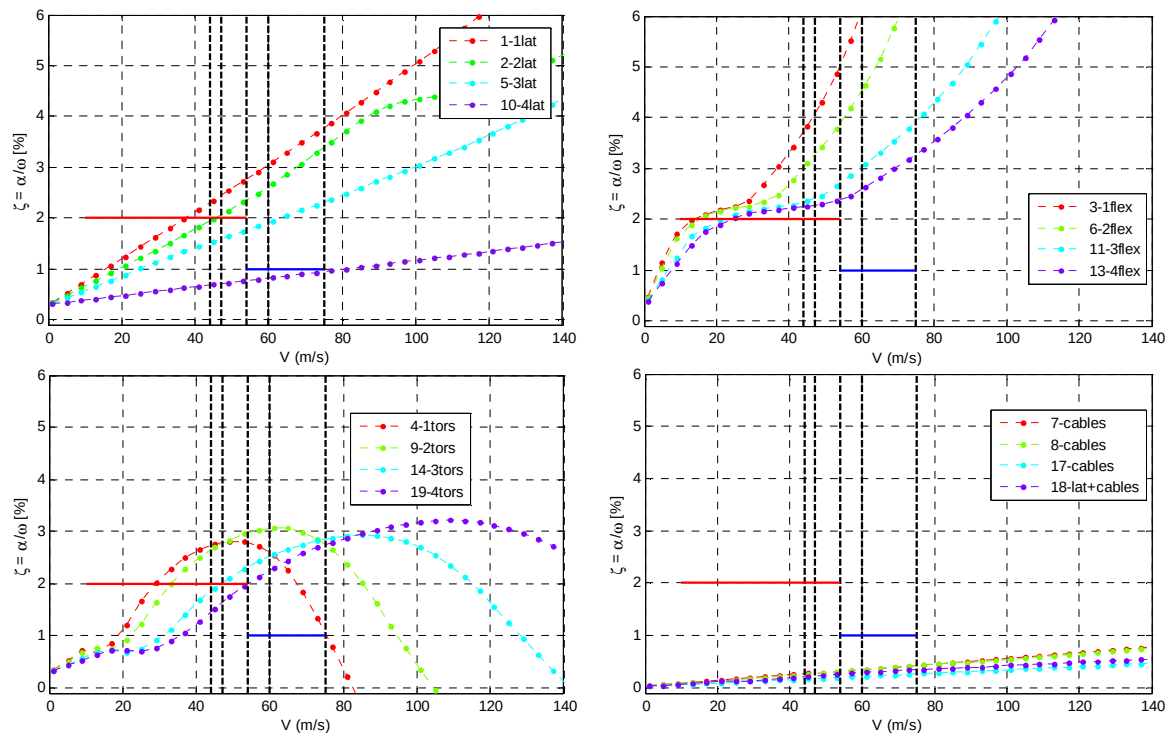


Figure 2-2. Instability coefficient vs. mean wind speed – Main cables + hangers ON – Modes by category.

2.2 BUFFETING ANALYSIS

Buffeting analysis has been performed with evaluation of displacements and accelerations distribution of the structure. The time histories of the bridge displacements and accelerations, together with the mean and turbulent wind components actions allow evaluation of the overall reference wind loads on the structure due to combined static and dynamic effects. The analysis has been performed using a linear integration analysis in time domain. RWDI's response solver uses a highly efficient "exact" integration method (Stoyanoff, 2001).

The buffeting analysis procedure entails the following steps:

- Choose a "wind grid" which covers the structure.
- Interpolate the bridge structure and the wind grid to locate the wind loads points of application.
- Generate wind time histories at all nodes get at previous step according with S.d.M. target turbulence properties
- Using these wind time histories, the Drag, Lift and Moment loads are expressed in the local frame of reference through the aerodynamic admittance functions define for each section and time step. As done for the Flutter Derivatives, the quasi-static approximation was used to extrapolate the aerodynamic admittance functions coefficients and cover for drag which was not experimentally identified. This simplification conserves the dependence from the angle of attach but obviously disregards the one to the reduced velocity.
- Integrate the equations of motion with buffeting loads obtained in the previous step.

Static force and moment coefficients on the towers were included based on the 1:100 sectional model test undertaken by RWDI (Stoyanoff et al, 2012).

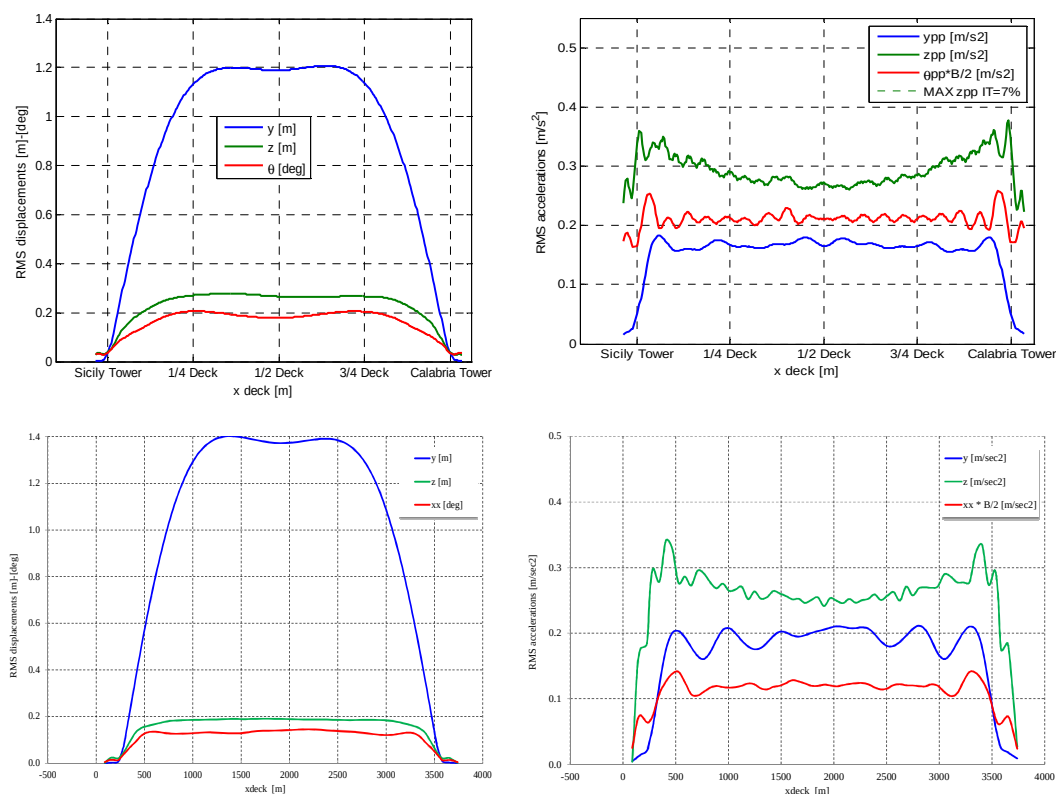


Table 2-3 Comparison between POLIMI (up)/RWDI (down) - rms displacements (left) and acceleration (right) of the deck in physical coordinates.

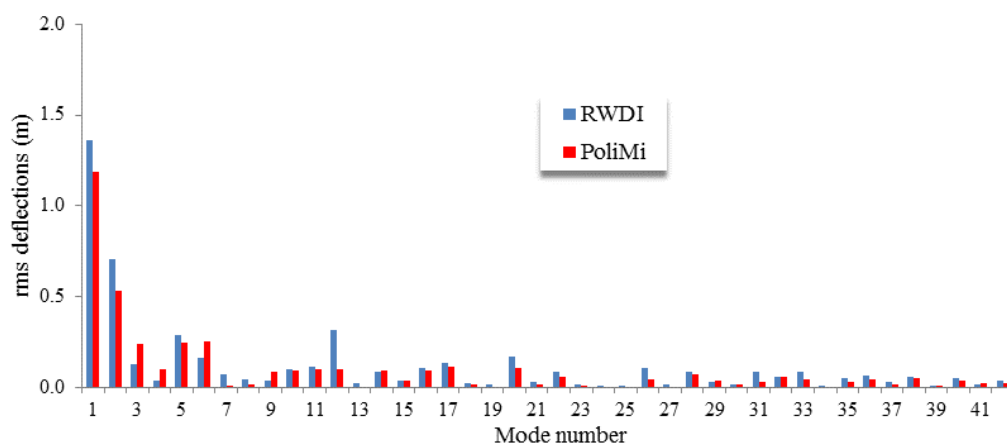


Table 2-4 Comparison between POLIMI /RWDI –modal rms. displacements for the first 50 modes

Figures 2-3 and 2-4 compare bridge response predicted by Polimi and RWDI at wind speed of 44 m/sec based on the same dynamic model and set of aerodynamic data. The observed differences were attributed to the different interpretations and simulations of the wind turbulence model and response analyses. These should outline the boundary of predictions for possible bridge responses.

3 WIND LOADS

3.1 Time domain approach (POLIMI)

The total loads on the bridge include stationary ones, due to the mean wind speed and time-averaged structure configuration and non-steady loads, caused by time dependent aeroelastic forces and by incoming turbulence fluctuations (buffeting forces). In response to these loads, the structure accelerations generate additional time dependent inertial forces. Performing the buffeting analysis, as describe in the previous paragraph, it is possible to calculate these loads at each simulation time step and for each section of the structure.

A common design practice is to express the effect of both steady and non-steady wind loads as equivalent wind loads (EWL), statically applied to the structure and taking into account also the dynamic effects of wind interaction. Different criterions were adopted in order to select these equivalent "scenarios" representative of critical loading.

A possible straightforward strategy is to select the loads combination determining the peak curvature values along the deck, main cable or tower sections, considering the direct link among structural curvatures and corresponding stresses in the structural components.

In order to calculate the structure time-space curvature distribution it's possible to take advantage of the modal approach due to the intrinsic separation of time and space domain functions, and writing the space partial derivative in terms of derivative of the eigenvectors as functions of the running space coordinate along the structure as follows:

$$\underline{x}(\zeta, t) = [\Phi(\zeta)] \underline{q}(t) \quad \rightarrow \quad \frac{\partial^2 \underline{x}}{\partial \zeta^2} = \left[\frac{\partial^2 \Phi}{\partial \zeta^2} \right] \underline{q}(t) \quad (5)$$

where $\underline{x}(\zeta, t)$ represent the time-space vector of the displacements, $[\Phi(\zeta)]$ the modal matrix of the eigenvectors and $\underline{q}(t)$ the vector of generalized coordinates. This vector of is a known function calculated from the buffeting response. It is possible now to select among the time-space curvature distributions, extreme values causing extreme stress conditions along the structure and to select those instantaneous configurations (and their associated equivalent static loads) as components of the relevant scenario of wind loading as required for structural design.

For instance, Figure 3-1 shows, for the mean wind speed $V=44\text{m/s}$, one of the selected scenario in which the deck curvature in the horizontal plane reached extreme values (close to the north-west tower). The figure shows also the other curvatures (vertical and torsional) and the associated displacements simultaneously reached by the structure. The same figure shows finally the corresponding distribution of equivalent wind loads determining the over described extreme configuration described (external, inertial and total loads). It is important noting that wind loads causing maximum curvature are not caused by loads local extremes on a section, rather to overall load distributions along the deck, main cables and towers.

Proposed method supplies the "true" instantaneous distributions of external and inertial forces and moments determining the extreme bending or torsion curvatures which represent the loading condition at a selected limit state. They are derived straightforward from the time domain response analysis. Further work is ongoing to establish a probabilistic interpretation of these results.

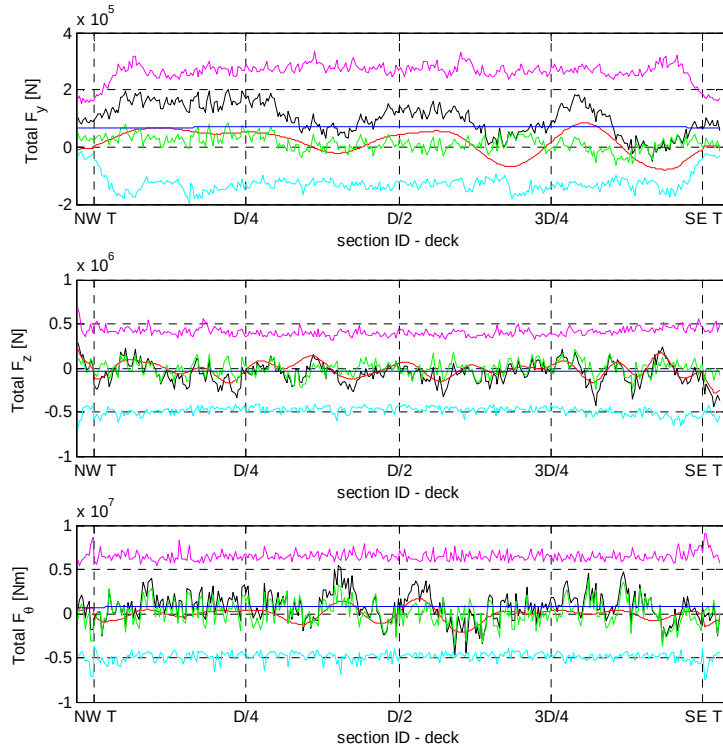
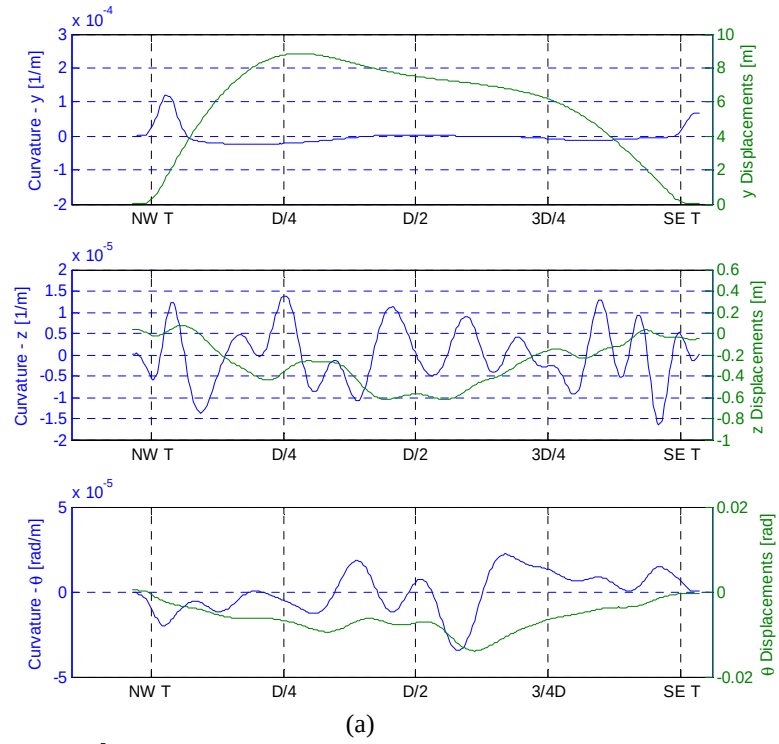


Figure 3-1 Snapshot of the scenario that represents the maximum curvature reached by the deck in the along wind direction. Also the other components (deflections, curvatures and loads in the crosswind direction and along the deck) are quoted. (a): curvature (blue), displacement (green). (b): total loads (black), inertial loads (red), buffet-ing loads (green), static loads (blue), max/min envelope loads on the total simulated time history(magenta/cyan).

3.2 Wind Loads: modal participation approach based on top-bin sorting (RWDI)

Wind loads were also developed by RWDI's independent procedure. The dynamic loads due to wind buffeting at various time instances tend to stress critically different bridge parts. They occur simultaneously with other static loads. It has been most convenient to designers to present wind loads as equivalent static loads. The dynamic wind loads are decomposed into various "snap-shot" equivalent-static load cases in order to cover likely critical conditions. Various combinations are developed with the load patterns being distributed lateral, vertical, longitudinal, and torsional loads.

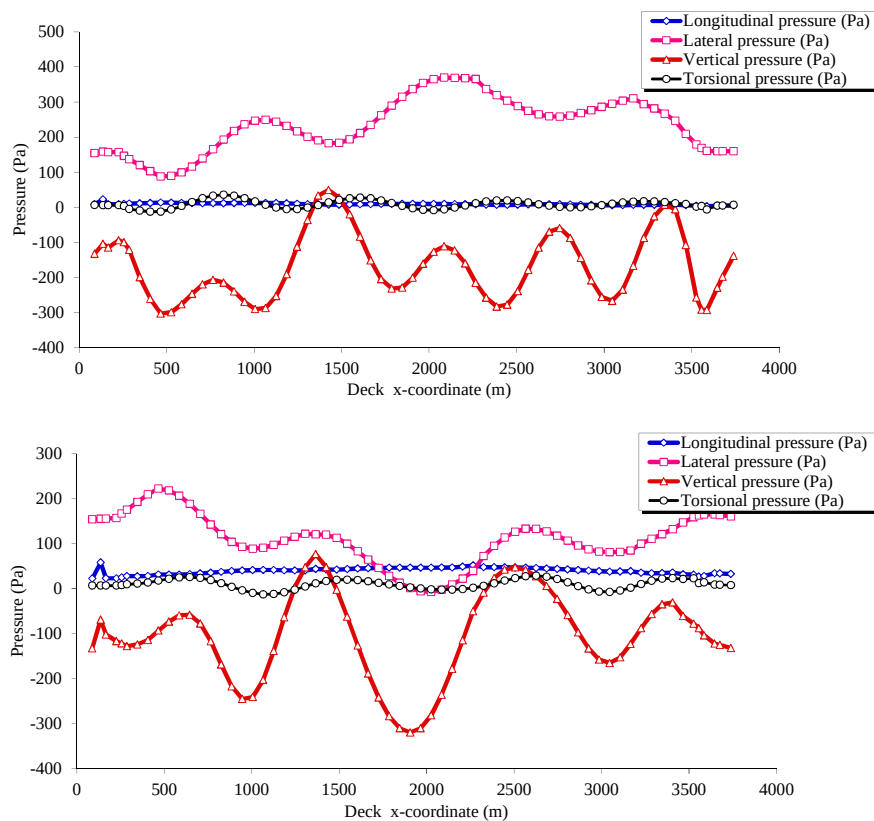


Figure 3.2 Equivalent static load distributions derived for the deck based on the modal participation factors identified from the top-bin sorting method. Longitudinal pressure (blue), lateral pressure (purple), vertical pressure (red) and torsional pressure (black) are given for the deck physical coordinates. UP: load distribution causing maximum lateral deflection. BOTTOM: load distribution causing maximum vertical negative deflection.

Whereas combining mean and direct wind gusts, and inertial loads is a straightforward procedure, the challenge is in defining the modal participation factors. One new technique can be applied where the time series of known responses at selected locations, e.g. on the deck near by the towers, are normalized and sorted. After normalization, the time histories of modal vectors will take values in the range of ± 1 (in most cases only one of the boundaries will reach unity). The response then is divided into bins where the top-bin (for example from 0.99 to 1.0) will provide several sets of participation factors for simultaneously occurring lateral, vertical and torsional responses. Since RWDI's method for the response integration is in fact an exact solution in generalized coordinates, the modal participation factors are then directly obtained as required for load combinations.

Considering deflection only, on a long-span bridge typically are selected $\frac{1}{4}$, $\frac{1}{2}$, and $\frac{3}{4}$ of the main span locations and time series of the overall response vectors produced. To examine differential responses, the cross-response vectors of $(\frac{1}{4}+\frac{1}{2}+\frac{3}{4})/3$ and $(\frac{1}{4}-\frac{3}{4})$ span locations are also produced. Accepting then as a sorting objective to identify time instances of maximum and minimum deflections in along, lateral, vertical, and torsion direction would lead to about 50 load cases. Examples of load distributions on the deck causing maximum lateral and vertical deflections are given in Figure 3.2.

4 CONCLUSIONS

Considering stability in strong winds, reasonably similar results were predicted by the two independent analyses, confirming a high reliability of the applied experimental-numerical methodology. Based on the same structural and aerodynamic parameters, comparing the response predictions to turbulent winds, however larger scatters were found between the two independent solutions. This confirms the expectation of wider uncertainties depending on the actual implementation of the established methodological framework of increased complexity.

Concerning the wind loads, there is an ongoing development in both methodologies including the implementation of probabilistic interpretations with a reference to structural fatigue.

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