OPERATIONAL MODAL ANALYSIS
OF CURVED CABLE-STAYED BRIDGES

Carmelo Gentile, Dept. of Structural Engineering, Milan Politecnico, Italy
gentile@stru.polimi.it

Abstract
The paper is first focused on the re-analysis of the ambient vibration data of the curved
cable-stayed bridge located on the north-side of the Malpensa international airport (Milan, Italy) by using
the FDD and the SSI techniques; the modal estimates provided by these methods are compared,
taking also into account the estimates previously obtained by using the traditional spectral
techniques. Successively, the results of a recent ambient vibration survey of the new curved cable-
stayed bridge erected in Porto Marghera (Venice, Italy) are summarized.

1 Introduction
In the last decades, the efficient utilization of traditional structural materials and the introduction of
new materials, the innovation in the architectural concepts, the advances in the design procedures
and in the construction technologies induced an increasing popularity of cable-stayed bridges in the
bridge engineering community and a large number of these bridge was erected world wide. Almost
all existing cable-stayed bridges are straight and only few have a curved deck superstructures;
examples of known curved cable-stayed bridges are the Rhine bridge near Schaffhausen,
Switzerland (Ref. [7]), the Safti Link bridge in Singapore (Ref. [6]) and the twin curved cable-
stayed bridges of the Milan-Malpensa airport, Italy (Ref. [8]).

This paper examines the dynamic behaviour of two curved cable-stayed bridge.

The first investigated system was erected in 1997 the neighbourhood of the new air terminal of the
Malpensa airport. The special geometric layout of the bridge, its infrastructural role and the
existence of a twin structure (which was tested as well, Ref [8]) provided strong motivations for an
accurate assessment using dynamic testing. Since the experimental program included both
traffic–induced and free vibration measurements, the modal parameters estimated from the two
different excitations were compared so that the accuracy of ambient vibration survey is assessed. In
the paper, the ambient vibration data were re-analyzed by using the FDD (Ref. [4]) and the SSI
(Refs. [9]-[10]) techniques; the modal estimates provided by these methods are compared, taking
also into account the estimates previously obtained by using the traditional spectral techniques
(Ref. [3]). The experimental investigation was supplemented Furthermore, the study was completed
by the development of a 3D finite element model of the bridge and the observed modal parameters
were used to update some uncertain structural parameters of the model.

The second curved cable-stayed bridge was recently erected in Porto Marghera (Venice, Italy). The
special geometric layout of the deck and of the inclined tower have motivated: a) extensive studies
developed during the design and the construction phases aimed to adequately define static and
dynamic characteristics; b) the execution of extensive tests on the materials during the construction
phase; c) the execution of careful reception (static and dynamic) tests before the bridge opening; d)
the permanent installation of a monitoring system (based on fiber optic deformation sensors).

The dynamic tests involved ambient vibration survey of the bridge, including the measurements of
cable vibrations. Again, the FDD and the SSI methods were used to identify the modal characteristics of the bridge and the main results are summarized.

2 The curved cable-stayed bridge in Malpensa air terminal

2.1 Description of the bridge:

The access to the air terminal of the new Malpensa 2000 airport includes two curved cable-stayed bridges which are symmetrically placed in the north and south side of the air terminal, respectively. The two bridges curve with a radius of 100 m and 6% longitudinal slope so that the air terminal seems to be embraced by the bridges which, at the same time, provide a limit to the airport area. Thus, the curved layout of the bridges gives an unique appearance to the airport and architectural concerns played a determining role in the conceptual and executive design, entirely developed by the late F. Martinez y Cabrera. The construction was completed in 1997.

The schematic plan and elevation of the north-side bridge (which is the subject of the paper) and a general view are shown in Figs. 1(a) and 1(b), respectively. The bridge consists of a A-shaped concrete tower, double-plane cables and a concrete-girder deck. The curved deck has a centreline length of 140 m, with two equal side spans and 4 cables supporting each side span.

The deck is a five-cell box concrete girder which was cast in place and post-tensioned; the girder has 6% longitudinal slope and 4% transverse slope. The girder has a depth of 1.35 m and a total width of 11.75 m for two traffic lanes and two pedestrian walkways.

The cast-in-place concrete tower is 36.87 m high and consists of two concrete legs (3.00 wide in the longitudinal direction of the bridge), a lower concrete cross-beam supporting the deck and an upper steel strut connecting the upper legs and providing the anchorage for the stay cables.

![Figure 1](image.png)

Figure 1. (a) Plan, elevation and cross-section of the curved cable-stayed bridge at the Malpensa airport (dimensions in m); (b) View of the bridge; (c) Instrumented points in the dynamic tests
2.2 Full-scale dynamic testing:

The responses of the bridge were measured at selected points using PCB (model 393C) accelerometers; a schematic diagram of the sensor layout is shown in Fig. 1(c). The experimental program included both traffic-induced and free vibration measurements and was conducted over a period of three days; the data were sampled at a rate of 200 Hz to provide good wave-form definition.

Ambient vibration response was acquired in 48 minute records per channel per test configuration. Since the bridge was tested immediately before its opening, the traffic excitation was achieved by means of: (a) two-axle trucks with 340 kN gross weight each, crossing the bridge with symmetric and eccentric passages and velocities in the range of 10 to 40 km/h; (b) various other vehicles of different weight and suspension. During the tests, traffic had been directed generally on both the lanes and different traffic flow could be observed at various times when data was being acquired. Specifically, during a short time period the traffic consisted of only one or two 340 kN trucks while during most of tests other trucks or vehicles were simultaneously crossing the bridge.

In order to obtain the free-decay vibration response of the bridge, an impulsive force was applied on the deck; the impulsive vertical force was generated simply by letting the front wheels of a truck "fall down" from the top of a standardised plank. The plank was placed in correspondence of the transverse beams of the deck where the stays are anchored; for each transverse beam the impulsive force was applied in centred and eccentric position so that eight different sets of data were recorded. In each free vibration test, the data were recorded for a duration of about 1 minute.

2.3 Data processing and modal identification procedures:

Ambient Vibration Data

The extraction of modal parameters from ambient vibration data was carried out by using the traditional spectral analysis or Peak Picking (PP) method (Ref. [3]) and two complementary output-only techniques implemented in the ARTeMIS software (Ref. [2]): the Frequency Domain Decomposition (FDD, Ref. [4]) in the frequency domain and the data driven Stochastic Subspace Identification (SSI, Ref. [9]-[10]) in the time domain.

In the application of the PP and the FDD techniques, the auto and cross spectra were evaluated using a frequency resolution of about 0.0244 Hz.

In the application of the SSI method, the data was fitted by stochastic subspace models of order between 2 and 100; inspection of the stabilization diagrams highlights that the observed dynamic behaviour is well represented using model orders between 40 and 60.

Free Vibration Data

After preliminary record analysis using FFT, the evaluation of modal parameters (resonant frequencies, damping ratios and modal displacements) from free-damped accelerations was performed by using a multi-degrees of freedom curve-fitting procedure in the time domain with the methodology described in Ref. [7].

Mode Shapes Correlation

Once the modal identification phase was completed, the sets of mode shapes resulting from ambient and free vibration tests were compared by using the Modal Assurance Criterion (MAC, Ref. [1]).

To correlate the results of finite element analysis and operational modal analysis, in addition to the MAC, the Normalized Modal Difference (NMD, Ref. [11]) was also used. The NMD correlation technique is more discriminating for well correlated mode shapes (as it happens in the present applications) and is related to the MAC as follows:
\[
NMD(\phi_{A,k}, \phi_{B,j}) = \sqrt{1 - MAC(\phi_{A,k}, \phi_{B,j})} \tag{1}
\]

In practice, the \(NMD\) is a close estimate of the average difference between the components of the two vectors \(\phi_{A,k}, \phi_{B,j}\); for example, a \(MAC\) of 0.950 implies a \(NMD\) of 0.2294, meaning that the components of vectors \(\phi_{A,k}\) and \(\phi_{B,j}\) differ on average of 22.94\%. The \(NMD\) is much more sensitive to mode shape differences than the \(MAC\) and hence is used to better highlight the differences between highly correlated mode shapes.

2.4 Test results and dynamic behaviour of the bridge:

The results of the operational modal analysis in terms of natural frequencies can be summarized through the plots of Figs. 2(a) and 2(b); Fig. 2(a) shows the average of the first 4 normalized Singular Values of the spectral matrices of all data sets (FDD) while Fig. 2(b) shows the modes selected from the stabilization diagrams of each data set (SSI). The inspection of Figs. 2(a) and 2(b), and the application of the PP method (Ref. [8]) as well, yields to the identification of 11 vibration modes in the frequency range of 0–10 Hz. Free vibration tests enabled the identification of 8 modes of vibration.

Owing to the curvature of the superstructure all mode shapes are in principle three-dimensional. The degree of coupling between vertical and transverse motion of the deck at each mode was investigated simply by computing the ratio \(m_{cr}\) (Ref. [13]) of the maximum transverse amplitude and the maximum vertical amplitude. Since the ratio \(m_{cr}\) does not exceed 0.20 for all vibration modes, it can be concluded that, although slight coupling in the transversal direction was always present, the vertical components (either bending or torsion) strongly dominate the identified modal behaviour. A selected number of mode shapes is shown in Fig. 5.

(a)

(b)

Figure 2. (a) FDD: Average of normalized Singular Values (SV) of the spectral matrix of all data sets; (b) SSI: Selected modes and links across data sets
1. **Bending-dominant modes (DV⁺).** These modes are dominated by the vertical bending of the bridge deck, with anti-symmetric deck modes generally involving a greater longitudinal participation of the tower than the symmetric ones;

2. **Torsion-dominant modes (DV⁻).** These modes are dominated by the torsion behaviour of the deck and are usually coupled with longitudinal motion of the pylon and slight transverse displacement of the deck. DV⁻ modes occur in couples of closely spaced natural frequencies located at 3.69, 3.81 Hz and 7.30, 7.47 Hz.

Table 1 summarises the identified modes, their classification and the comparison of the natural frequencies and modal damping ratios estimated from free and ambient vibration methods. The natural frequencies estimated by both methods are almost coincident and a similar correspondence is found for mode shapes, with MAC values (not shown in the Table) generally in the range of 0.98-0.99. On the contrary, the damping ratios estimated by the SSF techniques exhibit significant standard deviation and are generally quite different from the free vibration estimates.

**Table 1. Modal parameters identified from free vibration test (FVT) and ambient vibration test (AVT)**

<table>
<thead>
<tr>
<th>Mode Identifier</th>
<th>FVT</th>
<th>AVT</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$f$ (Hz)</td>
<td>$\zeta$ (%)</td>
</tr>
<tr>
<td>DV1⁺</td>
<td>0.786</td>
<td>0.66</td>
</tr>
<tr>
<td>DV2⁺</td>
<td>1.224</td>
<td>0.46</td>
</tr>
<tr>
<td>DV3⁺</td>
<td>2.328</td>
<td>1.06</td>
</tr>
<tr>
<td>DV4⁺</td>
<td>2.815</td>
<td>0.69</td>
</tr>
<tr>
<td>DV1⁻</td>
<td>3.703</td>
<td>0.53</td>
</tr>
<tr>
<td>DV2⁻</td>
<td>3.814</td>
<td>0.58</td>
</tr>
<tr>
<td>DV5⁺</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>DV6⁺</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>DV3⁻</td>
<td>7.332</td>
<td>0.61</td>
</tr>
<tr>
<td>DV4⁻</td>
<td>7.465</td>
<td>0.80</td>
</tr>
<tr>
<td>DV7⁻</td>
<td>–</td>
<td>–</td>
</tr>
</tbody>
</table>

2.5  **Finite element model for dynamic analysis:**

The experimental investigation was complemented by the development of a 3D finite element model (Fig. 3), based on as-built drawings of the bridge and accurate in-situ geometrical survey. The model was formulated using the following assumptions: (a) the box-type deck was modelled by using 904 four-noded shell elements; (b) rigid links were offset from the deck to accommodate lumped masses and correctly model the cable attachments to the deck; (c) the tower was modelled using 30 3D beam elements while linear elastic truss elements were used to represent the cable-stays; (d) the tower footing was fixed (no soil springs); (e) the possibility of free sliding in the direction of the curved centreline at the abutments was simulated by means of radial pinned-pinned rod elements. Furthermore, the effect of geometric non-linearity was investigated but found to be negligible for the dynamics of this bridge, as it happened in other theoretical and experimental studies in the literature (Ref. [5], [7], [13]).

A preliminary dynamic analysis was performed to check the similarity between experimental and theoretical modal parameters. The design data suggested for the concrete Young’s modulus of the deck ($E_d$) and the tower ($E_t$) a base value of 35.0 GPa; furthermore, engineering judgement suggested a Poisson’s ratio of 0.15 for the concrete of both tower and deck.
The modal parameters of the base model are compared with the actual ones in columns (3)-(6) of Table 2 through the frequency discrepancy \( D_f \), the MAC and the NMD; the comparison shows imperfect correlation, with \( D_f \) ranging up to about 10% and an average NMD of 15.21%. However, the correspondence of theoretical and experimental behaviour, notwithstanding its roughness, seems to provide a sufficient verification of the model main assumptions.

The experimental data were then used to adjust some structural parameters of the base model. The updating involved the parameters \( E_D \), \( E_T \). Since the corresponding theoretical and experimental mode shapes were quite similar in the investigated variation range of the parameters, the model updating was first carried out to improve the correlation between analytical and identified frequencies only. Thus, the optimal estimates of \( E_D \) and \( E_T \) were first defined to be the values which minimise the following:

\[
J_1 = \frac{100}{N_m} \sum_{i=1}^{N_m} \left| \frac{f_i^M - f_i^C}{f_i^M} \right|
\]

being \( N_m \) the number of identified modes and \( f_i^M \), \( f_i^C \) the \( i \)-th measured and computed natural frequency, respectively. Successively, the optimisation procedure was repeated by including in the objective function (2) a measure of the mode shape difference through the NMD and the following function was minimised:

<table>
<thead>
<tr>
<th>Mode Identifier</th>
<th>Exp. ((E_D = E_T = 35.0 \text{ GPa}))</th>
<th>Base Model</th>
<th>Updated Model ((E_D = 30.0 \text{ GPa}, E_T = 40.0 \text{ GPa}))</th>
</tr>
</thead>
<tbody>
<tr>
<td>DV1(^+)</td>
<td>(f_{\text{EXP}}) (Hz)</td>
<td>(f_{\text{EM}}) (Hz)</td>
<td>(D_f) (%)</td>
</tr>
<tr>
<td>DV2(^+)</td>
<td>0.781</td>
<td>0.761</td>
<td>2.56</td>
</tr>
<tr>
<td>DV3(^+)</td>
<td>1.221</td>
<td>1.306</td>
<td>6.96</td>
</tr>
<tr>
<td>DV4(^+)</td>
<td>2.344</td>
<td>2.369</td>
<td>1.07</td>
</tr>
<tr>
<td>DV1(^-)</td>
<td>2.808</td>
<td>2.972</td>
<td>5.84</td>
</tr>
<tr>
<td>DV2(^-)</td>
<td>3.687</td>
<td>4.053</td>
<td>9.93</td>
</tr>
<tr>
<td>DV3(^-)</td>
<td>3.809</td>
<td>4.107</td>
<td>7.82</td>
</tr>
<tr>
<td>DV5(^+)</td>
<td>4.541</td>
<td>4.969</td>
<td>9.43</td>
</tr>
<tr>
<td>DV6(^+)</td>
<td>5.371</td>
<td>5.627</td>
<td>4.77</td>
</tr>
<tr>
<td>DV3(^-)</td>
<td>7.300</td>
<td>7.857</td>
<td>7.63</td>
</tr>
<tr>
<td>DV4(^-)</td>
<td>7.471</td>
<td>7.995</td>
<td>6.48</td>
</tr>
<tr>
<td>DV7(^+)</td>
<td>8.789</td>
<td>8.829</td>
<td>0.46</td>
</tr>
</tbody>
</table>
\[
J_2 = \frac{100}{N_m} \sum_{i=1}^{N_m} \left[ \frac{f_{i,M} - f_{i,C}}{f_{i,M}} \right] + \text{NormDiff}\left( \phi^M, \phi^C \right)
\]

being \( \phi^M \), \( \phi^C \) the \( i \)-th measured and computed mode shape, respectively. The minimisation of error functions (2) and (3) (Fig. 4) provided practically the same values of \( E_D \) and \( E_T \):

\[
E_D = 30.0 \text{ Gpa} \quad \quad \quad \quad E_T = 40.0 \text{ GPa}
\]

The modal parameters of the updated model are compared with the experimental data in columns (8)-(10) of Tables 2. Specifically, the updated model exhibits slightly lower frequencies than the measured ones (with the maximum discrepancy being lower than 5\%) and the good match between measured and computed dynamic properties is confirmed by the \( MAC \), with values ranging from 0.928 to 0.998. As a further example, Fig. 5 compares the predicted mode shapes with the measured ones for a selected number of modes.

![Comparison between the selected predicted and the measured mode shapes](image)

Figure 5. Comparison between the selected predicted and the measured mode shapes

# 3 The new curved cable-stayed bridge in Porto Marghera

## 3.1 Description of the bridge:

The cable-stayed bridge belongs to the new roadway viaduct that crosses the West Industrial Canal of the commercial harbour (Porto Marghera) of Venice. The viaduct (Figs. 6-7) generally curves with a radius of 175 m and includes six spans (42 m + 105 m + 126 m + 30 m + 42 m + 42 m) for a total length of 387 m. The two longer curved spans are suspended by cable-stays from an inclined tower. An elevation and a plan view of the viaduct are presented in Fig. 7.

The cable-stayed bridge consists of an inclined triangularly-shaped concrete tower, single-plane cables and a composite deck. The curved deck has a centreline length of 231 m, with two different side spans and 9 cables supporting each side span.
Figure 6. Views of the new curved cable-stayed bridge in Porto Marghera (Venice, Italy)

Figure 7. Plan and elevation of the investigated bridge and tower elevation (dimensions in cm)

Figure 8. Experimental set-ups for the bridge test
The cable-stayed bridge consists of an inclined triangularly-shaped concrete tower, single-plane cables and a composite deck. The curved deck has a centreline length of 231 m, with two different side spans and 9 cables supporting each side span. The cast-in-place inclined tower is 75 m high (Fig. 7) and was pre-stressed to reduce the eccentricity of the vertical dead load due to the deck curved layout and to the tower geometry itself. The cross-section of the bridge consists of three steel girders, 185 cm high; the outer girders are wide flange sections while the central one is of box section. The girders are framed by floor beams 5.00 m spaced. Girders and floor beams are all composite with a 30.0 cm concrete slab. The total width of the deck is 23.70 m for two traffic lanes and three pedestrian walkways.

3.2 Ambient vibration tests:

The ambient vibration tests were carried out using a 16-channel data acquisition system with 14 uniaxial WR 731A piezoelectric accelerometers. In the tests, accelerations were measured in 34 selected points of the deck while only one cross-section of the tower (up-rising the deck of about 15 m) was instrumented; in addition, accelerometers were placed also on all the 9 stays of the Mestre side of the bridge. The tests were performed in a total of 4 set-ups, as it is shown in Fig. 8. It should be noticed that the last set-up, shown in Fig. 8(d), included also the accelerometers on the cable-stays.

Since the tests were performed before the bridge was opened to the traffic, two different series of ambient vibration data were recorded for each set-up: in the first series, the excitation was only provided by the wind and the micro-tremors while in the second series the traffic excitation was achieved by means of two-axle trucks, crossing the bridge with symmetric and eccentric passages. In the following, these two different series of ambient vibration data will be referenced to as AV1 (micro-tremors and wind) and AV2 (simulated traffic).

For each channel and for each type of ambient excitation, the acceleration-time histories were recorded for 3000 s. The data, originally sampled at 400 Hz, was decimated 20 times and high-pass filtered to remove any offset and drift. After decimation, the number of samples in each record was of 60000 with a sampling interval of 0.05 s. Subsequently, data was processed in order to estimate the spectral matrix by using the modified periodogram method (Ref. [12]). In the present application, smoothing is performed by 1024-points Hanning-windowed periodograms that are transformed and averaged with 66.7% overlapping, so that a total number of 173 spectral averages was obtained. Since the re-sampled time interval is 0.05 s, the resulting frequency resolution is about 0.0195 Hz.

3.3 Dynamic characteristics of the bridge:

Global dynamic behaviour

The output-only modal identification was again carried out by using the FDD and the SSI techniques, available in the commercial program ARTeMIS (Ref. [2]) and the two sets of mode shapes resulting from the application of the two methods were compared using the MAC (Ref. [1]).

As it has to be expected, several vibration modes were identified in the investigated frequency range from both the AV1 and AV2 data series. For example, Fig. 9 shows the average of normalized Singular Values of the spectral matrices of all data sets obtained from data series AV1. Inspection of Fig. 9 shows that 23 modes are reasonably well represented in the AV1 data series (while few peaks in the first Singular Value are lost or less clearly detected in the AV2 data series).

Fig. 10 shows a selected number of mode shapes identified by applying the FDD algorithm to AV1 data series. Table 3 summarizes the modal parameters identified from the same AV1 series by the FDD and SSI methods, the ratio of modes evaluated for both deck and tower and the mode classification. Furthermore, Table 3 compares the corresponding mode shapes obtained from the two different identification procedures through the frequency discrepancy $D_p = (f_{FDD} - f_{SS}) / f_{FDD}$ and the MAC.
Figure 9. *FDD*: Average of normalized Singular Values (SV) of the spectral matrix of AVI data sets

Figure 10. Selected vibration modes identified from AVI ambient vibration measurements

Fig. 10 shows a selected number of mode shapes identified by applying the *FDD* algorithm to AVI data series. Table 3 summarizes the modal parameters identified from the same AVI series by the FDD and SSI methods, the $m_{CB}$ ratios (evaluated for both deck and tower from the FDD estimate) and the mode classification. Furthermore, Table 3 compares the corresponding mode shapes and scaled modal vectors obtained from the two different identification procedures through the frequency discrepancy $D_f = \frac{|f_{FDD} - f_{SSI}|}{f_{FDD}}$ and the MAC.
### Table 3. Dynamic characteristics identified from AV1 data series (micro-tremors and wind)

<table>
<thead>
<tr>
<th>Mode Type</th>
<th>$f_{DB}$(Hz)</th>
<th>Deck $m_{CR}$</th>
<th>Tower $m_{CR}$</th>
<th>$f_{SS}$(Hz)</th>
<th>$\zeta_{SS}$ (%)</th>
<th>$D_T$ (%)</th>
<th>MAC</th>
</tr>
</thead>
<tbody>
<tr>
<td>DV1†</td>
<td>0.645</td>
<td>0.025 0.070 0.118 0.009</td>
<td>0.657 0.51 1.86 1.000</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DV2†</td>
<td>0.996</td>
<td>0.026 0.114 0.203 0.376</td>
<td>– – – –</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DV3†</td>
<td>1.133</td>
<td>0.016 0.072 0.019 0.084</td>
<td>1.129 0.65 0.35 0.990</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DV4†</td>
<td>1.191</td>
<td>0.014 0.060 0.013 0.058</td>
<td>1.197 1.08 0.50 0.997</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DV5†</td>
<td>1.445</td>
<td>0.018 0.035 0.006 0.001</td>
<td>1.466 0.64 1.45 0.991</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DV1‡</td>
<td>1.582</td>
<td>0.059 0.114 0.065 0.016</td>
<td>1.584 0.92 0.13 0.987</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DV2‡</td>
<td>1.699</td>
<td>0.025 0.078 0.080 0.009</td>
<td>1.715 1.31 0.94 0.907</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DV3‡</td>
<td>2.012</td>
<td>0.031 0.115 0.031 0.019</td>
<td>2.019 0.87 0.35 0.998</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DV6†</td>
<td>2.168</td>
<td>0.104 0.201 0.025 0.017</td>
<td>– – – –</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DV7†</td>
<td>2.637</td>
<td>0.036 0.070 0.028 0.026</td>
<td>2.638 1.25 0.04 0.993</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DV2‡</td>
<td>2.773</td>
<td>0.015 0.029 0.005 0.006</td>
<td>– – – –</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DV3‡</td>
<td>2.871</td>
<td>0.037 0.122 0.044 0.050</td>
<td>2.875 0.79 0.14 0.987</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DV5‡</td>
<td>3.320</td>
<td>0.024 0.104 0.003 0.007</td>
<td>3.326 0.97 0.18 0.992</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DV6‡</td>
<td>3.652</td>
<td>0.031 0.060 0.057 0.005</td>
<td>3.641 1.70 0.30 0.984</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DV9‡</td>
<td>4.180</td>
<td>0.013 0.026 0.065 0.023</td>
<td>4.168 1.00 0.29 0.977</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DV4†</td>
<td>5.176</td>
<td>0.023 0.045 0.013 0.006</td>
<td>5.186 0.71 0.19 0.992</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DV10‡</td>
<td>5.449</td>
<td>0.028 0.122 0.017 0.013</td>
<td>5.490 1.03 0.75 0.965</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DV11†</td>
<td>5.723</td>
<td>0.030 0.124 0.008 0.013</td>
<td>– – – –</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DV5†</td>
<td>5.859</td>
<td>0.007 0.029 0.005 0.006</td>
<td>5.851 0.55 0.14 0.967</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DV12‡</td>
<td>6.035</td>
<td>0.022 0.044 0.062 0.059</td>
<td>6.045 0.67 0.17 0.959</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DV6†</td>
<td>7.129</td>
<td>0.026 0.114 0.009 0.005</td>
<td>7.149 0.82 0.28 0.992</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DV7†</td>
<td>7.656</td>
<td>0.025 0.111 0.025 0.011</td>
<td>7.692 0.92 0.47 0.814</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DV13†</td>
<td>7.930</td>
<td>0.022 0.096 0.006 0.010</td>
<td>7.957 1.25 0.34 0.948</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Inspection of Table 3 clearly highlights that for the deck the ratio $m_{CR}$ is again less than 0.20 for all vibration modes meaning that, although slight coupling in the longitudinal and transverse direction is always present, the vertical components dominate the identified modal behaviour. Hence, the observed modes can be basically arranged as follows: (a) bending-dominate modes (DV†); (b) torsion-dominate modes (DV‡); (c) mixed modes (DV‡), where bending and torsion behaviour of the deck are simultaneously present and a significant transverse deformation of the cross-section is often detected (see e.g. the mode placed at 2.77 Hz in Fig. 10). The correlation values listed in Table 3 show a very good agreement between the FDD and the SSF techniques in terms of natural frequencies, with the frequency discrepancy being usually less than 1%. A similar correspondence is detected also for the estimates obtained from AV2 data series.

**Stay-Cables behaviour**

By numbering the stay-cables of the Mestre side in descending order (i.e. the longer cable is referred to as “Stay-cable n. 1” while the shorter is referred to as “Stay-cable n. 9”), the following fundamental frequencies of the stays were identified from both AV1 and AV2 data series:

- Stay-cable n. 1 $f_1 = 0.957$ Hz
- Stay-cable n. 2 $f_1 = 1.094$ Hz
- Stay-cable n. 3 $f_1 = 1.230$ Hz
- Stay-cable n. 4 $f_1 = 1.270$ Hz
- Stay-cable n. 5 $f_1 = 1.348$ Hz
- Stay-cable n. 6 $f_1 = 1.484$ Hz
- Stay-cable n. 7 $f_1 = 1.582$ Hz
- Stay-cable n. 8 $f_1 = 1.133$ Hz
- Stay-cable n. 9 $f_1 = 1.230$ Hz

Beyond the traditional use of the above values for estimating the tension $T$ in the stay-cables, it should be noticed that the lower global modes of the bridge (i.e. 0.996 Hz, 1.133 Hz, 1.191 Hz, 1.445 Hz and 1.582 Hz) occur at stay cable frequencies or very close to stay-cable frequencies.
4 Conclusion
Experimental investigation of a two different curved cable-stayed bridges has been presented and discussed. The following main conclusions can be drawn:
1. Within the frequency range 0–10 Hz, the FDD and the SSI techniques provided the identification of 11 (Milan bridge) and more than 20 vibration modes (Venice bridge), respectively.
2. Notwithstanding the curved layout of the bridge superstructures, slight coupling between vertical and transverse vibration of the deck was observed at the low level of vibration that existed during the tests. For both bridges and for each vertical mode, either bending or torsion, the vertical component exceeds the transverse one by at least a factor of 5;
3. Few mixed bending+torsion modes were identified for the Porto Marghera bridge and such modes generally involve a significant transverse deformation of the deck cross-section;
4. some global modes of the Porto Marghera bridge were found to occur at (or very close to) the fundamental frequencies of the stay-cables.

5 References