Introduction. This paper presents the results of an experimental and theoretical investigation on the Pietratagliata cable-stayed bridge (Udine, Italy). Based on ambient vibration and local dynamic experimental results carried out in order to estimate the dynamic characteristics of the lower vibration modes of the bridge and the axial forces on the suspending cables, the structural identification is performed by means of a geometrically detailed, computationally expensive but refined Finite Element (FE) model, being the optimized configuration representative of an increasingly accurate manual tuning and model updating procedure. Based on the rather close agreement between EMA and FE results, the same optimized FE model (FE-OPT, in the following) is then used to investigate the sensitivity of the bridge dynamic properties to damage in the suspending cables.

The Pietratagliata bridge. The construction of the bridge was completed in 2007 and the infrastructure was opened to traffic in 2008. The bridge consists of a steel-concrete composite deck simply supported at the ends, a system of double-plane cables supporting the deck, and an inclined steel tower, see Fig. 1a.

The total length of the deck is 67 m, while the bridge width is 11.10 m, including two lanes and two lateral footways. The deck structure consists of ‘Predalles’ concrete panels and a reinforced concrete (RC) slab supported by two lateral steel girders and a longitudinal central beam. The lateral longitudinal and transverse girders have double-T cross-section, 1.27 m and 1.20 m height respectively, while for the central longitudinal girder a HEB500 type cross-section is used. The interaction between the RC slab and the upper flange of the longitudinal girders consists of welded steel stud connectors.

The bridge deck is supported on a RC pier on the National Route (NR) n.13 side and on a cast-in-place RC foundation block on the Pietratagliata side, see Figs. 1a and 1b. On the NR n.13 side, two unidirectional bearing supports are used to sustain the lateral girders. On the Pietratagliata side, conversely, the lateral girders are restrained by means of spherical hinges. Three groups of forestays on the upstream and downstream side of the bridge provide additional support to the deck. Each group of cables consists of four Dywidag bars which are connected to the main girders by means of special metal devices (see the detail of Fig.1c). Further backstays connect the steel tower to a RC foundation block. The tower consists of two inclined columns, having thin-walled circular cross-section 1.10 m in diameter (thickness 20 mm). The connection between the inclined columns is given by two additional thin-walled tubes, 0.50 m in diameter (thickness 15 mm). Special steel restraints are realized at the base of the steel tower, in order to reproduce the effect of spherical hinges.

Dynamic testing programme and methods. Dynamic testing was carried out in 2009. Both ambient vibration testing and local dynamic measurements on the cable system were carried out.

Ambient vibration testing. Among the tools currently available for structural investigations, dynamic techniques play an important role for several motivations. Based on the measurement of the response of a given structural system, dynamic techniques allow to identify the main parameters governing the dynamic behavior. Ambient measurements do not interfere with the normal service of the structure, and can also be repeated. Ambient vibration testing methods, moreover, are particularly suitable for flexible systems like suspension or cable-stayed bridges (Caetano and Cunha, 2011; Benedettini and Gentile, 2011; etc.), since the most significant modes of vibration in the low range of frequencies are excited with sufficient energy by the environmental actions only, hence a large number of normal modes can be identified from
ambient vibration survey of these bridges. In this research project, ambient vibration testing was adopted to determine the dynamic characteristics of the bridge (with five-six cars per hour, approximately, the typical traffic entity). This working assumption for the experimental tests – strictly required by the Municipality of Pietratagliata – enabled the use of additional excitation sources and resulted in additional difficulty for the experimental analysis and interpretation of test measurements (Alaggio et al., 2015). The instrumentation chain was based on a 16-channels data acquisition system, connected to a remote PC. The sensors consisted in 16 Sprengnether mono-axial servo-accelerometers operating in the frequency range 0-25 Hz. Each sensor was provided with a pre-amplifier having variable gain controlled by the remote computer. The signal was transmitted in differential modality to the acquisition system, where it was converted in single-ended modality to be filtered and passed to a 16-bit A/D converter, before being stored
in different formats. Based on the dynamic estimations obtained from preliminary FE studies carried out on a simplified numerical model of the bridge (Alaggio et al., 2015), the instruments were located at 16 selected points, according to two different setups (Fig. 2a). Time acquisition during tests was set equal to 45’, corresponding to about 1600 times the expected vibration period of the fundamental mode of the bridge. The sampling rate used during acquisition was 400 Hz. During post-processing analysis, the signal data were further decimated in time by a factor 10, giving a baseband for the analysis ranging till to 20 Hz. Natural frequencies, damping ratios and mode shape components were estimated by means of the Enhanced frequency Domain Decomposition (EFDD) technique available in the commercial software ARTeMIS (ARTeMIS Extractor Software, 2002). Several repeated identifications were performed separately, either on the same data set or on partial data using different baseband and different sets of data, so that the estimation of the detected vibration modes could be optimized. In the case of the damping factors, obtained from the experimental power spectral density measurements, the

Fig. 2 – Dynamic experiments. a) instrumental layout; b) overview and details of the FE-OPT model (ABAQUS/Standard); c) 3D view of the identified modal shapes (ABAQUS/Standard).
values were calculated based on the identification procedure described in (Brincker et al., 2011), that is by working on the inverse Fourier transform of the fully or SDOF auto-spectral density functions.

**Dynamic estimation of the axial force on the cables.** Further local ambient vibration measurements were carried out on all the supporting cables, with the goal of estimating the axial force acting on them. Dynamic tests were performed by collecting the transverse time-history acceleration of each cable on the vertical plane, at measurement points located at the thirds of each cable length, approximately. Time series of 1200 s were recorded in each experiment. All the cable natural frequencies were identified by computing the auto-spectrum of the acquired acceleration signals. Time series were low-pass filtered and decimated before computing the auto-spectrum via the modified periodgram method (Welch, 1967), for a resulting frequency resolution of about 5/100 Hz. The first six cable frequencies were identified by computing the auto–spectrum of the acquired acceleration signals. Each cable was modelled as a uniform pinned-pinned Euler–Bernoulli straight beam, having (known) mass, density and bending stiffness, subjected to an unknown positive axial force. The axial force on each cable was estimated by means of a variational method. In particular, the optimal value of the axial force was determined so as to minimize the difference between a selected number of theoretical and experimental frequency values.

**FE modelling approach an solving technique.** A geometrically refined FE-model of the Pietratagliata Bridge was implemented by means of the ABAQUS/Standard computer package (Simulia).

**Geometry and materials.** Careful consideration was given to the geometrical description of the bridge components (e.g., deck, pylon, cables and pier), as well as to their reciprocal interaction, since of primary importance for the accuracy of the predicted mode shapes and frequencies of the bridge as a full structural system. 4-node stress/ displacement shell elements (S4R type) were used for the description of the bridge deck (82,000 elements) and the steel tower (29,000 elements). A free meshing technique was used, with average size of these elements equal to 0.15 m and 0.08 m for the deck and the pylon, respectively. Based on the technical drawings of the bridge, the nominal thickness was then assigned to these shell elements. In the case of the deck, the structural interaction between the concrete slab and the longitudinal girders (e.g., where steel stud connectors are used) was described by means of tie constraints able to account for a rigid connection between the corresponding DOFs, along the bridge length. Beam elements (B31 type) with nominal geometrical properties were used for the double-L shaped metal bracings. Their self-weight was described in the form of additional lumped masses at the beams ends. Further lumped masses distributed on the concrete slab of the deck were also used to take into account the self-weight of the asphalt layer and the lateral footways. The steel cables consisted in beam elements (B31 type) with nominal cross-sectional area (63.5 mm in diameter) and overall length derived from technical drawings. Lump masses representative of half the self-weight of the cables were applied at the ends of each beam element. The cables were then connected to the steel tower and to the deck respectively by means of join connectors able to restrain possible relative displacements between the interested nodes. Careful consideration was paid to the geometrical description of the metal supports and devices (Fig. 2b, details A and B), so that local deformations and improper effects could be avoided. The so described deck and pylon were then properly restrained. In the case of the pylon (Fig. 2b, detail C), the metal devices at its base consisted in two inclined steel plates (80 mm in thickness) opportune constrained, so that the typical base support could behave as a spherical hinge with respect to a local reference system. An analogous modelling approach was used for the description of the deck restraints of the longitudinal lateral girders on the RC abutment on the Pietratagliata side, see detail D in Fig. 2b. The RC pier on the NR n.13 side was finally modelled by means of 3D solid finite elements. Mesh size refinement required by the geometrical features of the pier
(Fig. 2b, box) resulted in 48,000 solid elements with average length 0.2 m. Possible soil-to-pier interaction was fully neglected, and the pier was rigidly restrained at its base. The mechanical interaction between the bridge deck and the RC pier (Fig. 2b, detail E), e.g. the unidirectional supporting devices, were described by means of *slot* connectors able to provide null relative displacements along the transverse and vertical bridge directions, between the connected nodes. At the same time, longitudinal displacements and relative rotations between the interested nodes were kept unrestrained. Globally, the so implemented FE model resulted in 700,000 DOFs and 160,000 elements.

Concerning the characterization of materials, both concrete and steel were assumed to behave linear elastically, with isotropic mechanical properties derived from technical drawings and small samples. For the concrete of the deck slab, specifically, experimental test on cylindrical cores provided an average Young’s modulus equal to $E_c = 42$ GPa. A weight per unit volume $\rho_c = 25$ kN/m$^3$ was assumed for the RC structural members with $\nu_c = 0.3$ the Poisson’s ratio. The Young’s modulus, weight for unit volume and Poisson’s ratio of steel were assumed equal to $E_s = 206$ GPa, $\rho_s = 78.5$ kN/m$^3$ and $\nu_s = 0.3$, respectively.

**Solving method.** A static incremental, nonlinear analysis under the effects of the bridge self-weight and dead loads (e.g., footways and asphalt layer) was preliminary carried out on the FE model (Step I), in order to determine the equilibrium reference configuration. In the subsequent step (Step II), the first 20 analytical vibration modes were predicted by means of linear modal analysis around the reference configuration derived in Step I.

**Interpretation of dynamic test results and EMA-FE correlation.** The high modeling and computational cost of the FE model, geometry refinement of the bridge components, as well as their reciprocal interaction, generally resulted in dynamic estimations in rather close agreement with test measurements.

**Vibration frequencies and mode shapes.** Tab.1 proposes a comparison of EMA and FE natural frequencies, with the corresponding MAC values. The FE mode shapes associated to the identified vibration frequencies are collected in Fig. 2c.

The primary effect of the accurate FE model, based on the interpretation of dynamic results, was represented by the prediction of a fundamental vibration mode of the bridge not detected in a preliminary experimental interpretation of ambient vibration test data (‘EMA 0’). The corresponding mode shape is characterized by torsional motion of the deck and large deformation of the steel tower (Fig. 2c). While the singular value curves of the spectral density matrix did not show the presence of this first torsional mode, probably since the corresponding natural frequency is very close to the fundamental flexural one (EMA 1), in a subsequent phase the modal parameters were separately estimated in the frequency domain, for the half-sum and half-difference of the recorded time histories. The advantage of this approach, based on the vertical oscillations of two control points located on the opposite sides of the same transverse deck cross-section, is that if a vibration mode is mainly flexural, the measured amplitude oscillation at the selected pair of control points are similar, and their difference is small. The half-sum of time histories, consequently, magnifies the presence of vertical bending modes and hides the peaks corresponding to torsional ones. Conversely, if a mode is mainly torsional, the vertical modal components at the same control points are similar in amplitude, but have opposite sign, so their sum is small. The half-difference of the corresponding time histories, as a result, automatically excludes the peaks associated to bending modes.

Apart from the EMA 0 mode, a general close agreement was found for the identified EMA and FE modes.

The numerical simulations carried out on the FE-OPT model, in particular, highlighted the importance of refined geometrical description of few, but crucial, bridge components, and specifically the proper geometrical and mechanical characterization of the bridge supports (details C and D of Fig. 2b), as well as the stays-to-deck and stays-to-pylon connections (details A and B of Fig. 2b). On the other hand, the progressive increase of the modelling complexity
required the solution of a series of additional uncertainties and numerical instabilities. The occurrence of local deformations at the stays-girders connection systems, in particular, was fully prevented by taking into account the nominal geometry of the steel supporting device, including in it all the stiffening elements. Concerning the restraint supports, a preliminary assessment of the same modelling approach was carried out on small FE models representative of the bridge and tower restraints only. The optimized restraining devices were successively implemented in the full FE model of the bridge. The frequency of the first vibration mode of the bridge highlighted in fact a marked sensitivity to the deck and pylon base restraints, with variations in the estimated frequencies up to 25% the optimal value (Tab. 1). The presence of the RC pier, involving an asymmetry in the overall geometry, also highlighted the presence of vibration modes pairs (e.g., bending modes of the deck), characterized by comparable in-phase or out-of-phase motion of the deck and bending deformation of the RC pier, corresponding to almost identical natural frequencies. In all these circumstances, the correlation between EMA and FE modes was based on minimization of natural frequency discrepancy values and MAC factor.

In some other circumstances, despite an optimal correlation between EMA and FE frequencies, rather scarce MAC values were found. This is the case of higher vibration modes (e.g., EMA mode 5 in Tab. 1) characterized by significant motion of the deck coupled with large deformation of the steel tower. Due to few available experimental measurements, modal correlation was undergone in this case by taking into account not only the natural frequency and the calculated MAC value, but also an additional visual correlation.

The general good agreement between EMA and FE predictions obtained for the lower vibration modes of the Pietratagliata bridge, in conclusion, justified in fact the fundamental role of the sophisticated FE-OPT model, especially for future, possible diagnostic applications.

**Axial forces on the cables.** Further validation of the FE-OPT model was in fact provided by comparison of experimental and numerical axial forces on the cables.

The performed simulations highlighted in fact that the FE-model correctly estimates the effects of the applied permanent loads, hence suggesting the use of the same refined FE-model for further advanced sensitivity studies and diagnostic investigations. The close agreement was found especially in terms of total axial forces $T_{total}$ taken up by the cables system (Tab. 2).

Whatever a good global symmetry of the cable system supporting the deck of the bridge was found, the experimental measurements highlighted important discrepancies, within each group of cables, and suggested further detailed investigations. Tab. 2 summarizes in fact the average axial force $T_{group,EXP}$ for each group of cables, but also the discrepancy $\Delta_{Cn}$ ($n=1,..4$)

Tab. 1 - Experimental Modal Analysis (EMA) results and correlation with FE calculations. Mean value of natural frequency $f_r$ and damping ratio $\xi_r$, with corresponding maximum deviations ($r$= mode order). B = Bending; T = Torsional mode shapes. Frequency error: $\Delta = 100 \times (f_{r,(FE-OPT)} - f_{r,(EMA)})/f_{r,(EMA)}$.

<table>
<thead>
<tr>
<th></th>
<th>Description</th>
<th>$f_r$ [Hz]</th>
<th>$\xi_r$ [%]</th>
<th></th>
<th>Description</th>
<th>$f_r$ [Hz]</th>
<th>$\Delta$ [%]</th>
<th>MAC [%]</th>
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<td>0</td>
<td></td>
<td>1.619</td>
<td></td>
<td>1</td>
<td></td>
<td>1.599</td>
<td>1.2</td>
<td>98.5</td>
</tr>
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<td>1</td>
<td>1$^{rd}$ B</td>
<td>1.665</td>
<td>1.2 ± 0.5</td>
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<td>1.619</td>
<td>2.8</td>
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<tr>
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<td>2.691</td>
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<tr>
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<td>3.411</td>
<td>0.7 ± 0.2</td>
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<td>3.234</td>
<td>5.2</td>
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<tr>
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<td>0.4 ± 0.0</td>
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<tr>
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<td>3$^{rd}$ T</td>
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<td>0.9 ± 0.2</td>
<td>13</td>
<td></td>
<td>7.371</td>
<td>-0.5</td>
<td>78.4</td>
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</table>
between \( T_{\text{group},\text{EXP}} \) and the axial force measured on a single cable. It can be seen that the calculated deviation is negligible, e.g., 2–3% the average group value, for the four cables of group 1, both on the upstream (1U) and downstream side (1D). On the contrary, the axial forces in groups 2 and 3 show larger deviations from the corresponding average values, e.g., up to 16% and 11%, for groups 2D and 3U respectively. In this context, the marked difference of estimated axial forces in cables belonging to the same groups (e.g., 2D and 3U) should be considered as a symptom of potential anomaly of the suspension system of the bridge, thus requiring further extended investigations.

**Sensitivity of the bridge to damage in the cables.** During the year 2010, two cables belonging to the group 2U were separately interested by the collapse of the stays-to-deck connection detail (e.g., Fig. 1b). According to Tab. 2, the marked difference in identified axial forces on the cables belonging to groups 2 and 3 could be considered a symptom of a potential anomaly of the suspension system. Consequently, with the aim of investigating the sensitivity of the natural frequencies, vibration modes and axial forces on the stays with respect to a possible damage on the suspending system, an extensive numerical analysis was carried out by using the refined FE-OPT model as reference configuration for the undamaged bridge.

Among several parametric numerical simulations, six main damage scenarios were considered, as obtained by separately removing one or two cables from the groups 1U, 2U and 3U of cables. For all these scenarios, the corresponding FE-DAM models were derived by imposing a null cross-sectional area to the cable of interest for the simulation of damage. The same solving method taken into account for the reference undamaged model FE-OPT (e.g., Step I for the application of dead loads and Step II for the modal analysis of the bridge) was considered for each one of them. The main results of these additional numerical simulations are collected in Fig. 3, where the labels ‘1U-1’ and ‘1U-2’ in them denote the damage in one or in two cables belonging to the group 1U, respectively, and so on. A generally appreciable sensitivity of the bridge dynamic response to the induced damage was found, hence highlighting the usefulness of diagnostic investigations based on modal data.

In terms of natural frequencies, all the damaged models showed small reduction, compared to the reference undamaged values (see Fig. 3a). Negligible frequency sensitivity to damage was also generally noticed for the higher order modes, e.g., the EMA modes 5 and 6. Although in few cases only, large variations (up to 5%) were found in some cases.

Worth of interest is the effect of damage on the lower vibration modes of the bridge. Especially the introduction of damage in the 2U group manifested in fact a very high sensitivity of the FE mode shapes 1 and 2 (e.g., EMA modes 0 and 1 in the notation of Tab. 1 and Fig. 3). As expected, the progressive removal of a single or two cables in the groups 1 and 3 generally

<table>
<thead>
<tr>
<th>Group [-]</th>
<th>( T_{\text{group},\text{EXP}} ) [kN]</th>
<th>( \Delta_{c1} ) [%]</th>
<th>( \Delta_{c2} ) [%]</th>
<th>( \Delta_{c3} ) [%]</th>
<th>( \Delta_{c4} ) [%]</th>
<th>( T_{\text{group},\text{FE}} ) [kN]</th>
</tr>
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<td>1D</td>
<td>380.1</td>
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<tr>
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<td>1.6</td>
<td>2.8</td>
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<tr>
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<td>15.9</td>
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<td>529.6</td>
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<td>2.7</td>
<td>4.4</td>
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</tr>
<tr>
<td>3D</td>
<td>452.5</td>
<td>-5.7</td>
<td>1.6</td>
<td>-4.4</td>
<td>8.5</td>
<td>524.8</td>
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<tr>
<td>3U</td>
<td>460.6</td>
<td>5.8</td>
<td>-2.3</td>
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<td>-10.3</td>
<td>525.1</td>
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<td>11080.0</td>
</tr>
</tbody>
</table>
resulted in increasing mode shape discrepancy, with respect to the undamaged configuration. On the contrary, FE modes 1 and 2 subjected to damage in the central group 2 highlighted an apparent misleading modification of their modal shapes, with larger modifications deriving from the removal of one cable only, rather than two (Fig. 3b). The reason of this finding is strictly related to the vicinity of the natural frequencies of the two modes in the undamaged configuration. A detailed numerical study carried out by gradually introducing the damage in those cables, highlighted in fact a sort of mode hybridization for FE modes 1 and 2. In Fig. 3c, the so calculated natural frequencies in the 2U-1 and 2U-2 damage configurations are proposed as a function of the damage ratio $R_d = A_{cable,DAM} / A_{cable}$, where $A_{cable,DAM}$ and $A_{cable}$ denote the cross-section area of the damaged and undamaged state.

Finally, the effect of damage on the maximum axial forces on the cables was investigated. Some comparative results are collected in Fig. 3d, where the maximum variation of axial force on each group of cables, with respect to the average force values for the undamaged groups of stays, are proposed for the examined damage scenarios. Large sensitivity to damage was found especially when removing one or two cables in the group 2U (in the order of 15% and 32% for the scenarios ‘2U-1’ and ‘2U-2’ respectively), hence emphasizing the importance and usefulness of diagnostic investigations.
Conclusions. In the paper, dynamic characterization of the cable-stayed bridge of Pietratagliata was carried out using ambient vibration tests and FE analyses. A refined 3D FE model of the bridge was calibrated (FE-OPT), based on natural frequencies and modal shapes extracted from FRFs, as well as dynamic measurements of the axial forces on the cables. The optimized 3D model highlighted the importance of geometrical refinement and computationally expensive but accurate modelling assumptions, and resulted in good agreement with experimental predictions, hence suggesting its use as a baseline for further diagnostic investigations and monitoring programs. The sensitivity of the bridge dynamic parameters to damage in the cables, specifically, was properly assessed by means of extended numerical studies (FE-DAM). The main effects of damage on natural frequencies, mode shapes and axial force on the undamaged cables was emphasized.

Acknowledgements. The authors would like to commemorate the dear friend and colleague Prof. Francesco Benedettini (University of L’Aquila), a great scholar of Structural Dynamics and, specifically, of ambient vibration testing and operational modal analysis methods on bridges. This research was made possible thanks to the interest and the support of the Dipartimento della Protezione Civile of the Friuli Venezia Giulia. The authors would like to gratefully acknowledge the cooperation of Drs. G. Berlasso and C. Garlatti. The collaboration of Prof. Rocco Alaggio and Dr. Daniele Zulli (University of L’Aquila) during dynamic testing is gratefully appreciated.

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