Identification and condition assessment of the "*Villa Passo*" reinforced concrete arch bridge

R. Alaggio, F. Benedettini and V. Gattulli
Università di L'Aquila, Dipartimento Ingegneria delle Strutture, delle Acque e del Terreno, L'Aquila, Italy

ABSTRACT: Structural reliability and safety evaluation of existing bridges are crucial concerns of managers of road and railway systems. The paper discusses a four-steps procedure able to evaluate the structural safety of existing reinforced concrete bridges. Recursive ambient vibrations of the bridge were measured. Enhanced Frequency Domain Decomposition and Stochastic Subspace Identification were applied to identify the modal properties of the structural system. The identified modal quantities were used to update the material property parameters of a finite element model. Two different non linear limit analyses were performed on the calibrated model: the first to evaluate structural redundancy and resistance to vertical load uniformly distributed on the superstructure, the second to evaluate structural performances with respect to lateral seismic-type loads. Obtained resistances were eventually compared with the internal stresses induced by regular traffic loads and the maximum expected earthquake, obtaining synthetic measures of the bridge structural safety.

1 INTRODUCTION

Bridges are in most of the cases rated and monitored through inspections, largely only at visual level. Visual inspections techniques are able to evaluate the overall efforts needed for maintenance more than to assess the structural condition. Indeed, several cases are reported in literature in which modal flexibility, unit load deflections and derivatives indices are sufficiently sensitive to damage that may elude visual inspection (Aktan et al. 1997a, Beolchini and Vestroni, 1997). Consequently, a reliable condition assessment of civil infrastructure should be pursued through the methodology of structural identification. On this respect, recent research studies have followed the idea to define a complete framework to asses infrastructure condition through structural identification methodology that contains: experimental and analytical arts, information technology, decision making arts and non-technical issues (Aktan et al. 1997a, Aktan et al. 1997b), even integrated with structural control strategy (Gattulli and Romeo, 2000). It is evident the needs to use all the information available on the infrastructure (visual inspections, measurements, modal analysis, instrumented monitoring, bridge database, etc…) to making the correct decision with respect the reliability evaluation for service and safety and the overall bridge management. On this scenario, the presented paper is mainly focused on the treatment of data available from recursive modal testing and from a 3D finite element model calibrated on the available measured quantities. The used methodology constitutes a possible approach to build up a more general framework to assess bridge condition through the fusion of different mechanism of evaluation as visual inspection procedures (Gattulli and Chiaramonte, 2005) and reliability evaluations (Stewart, 2001).
The studied bridge arch was monitored with 10 accelerometers positioned on 20 different locations and was tested under an ambient excitation (vehicular traffic). The Enhanced Frequency Domain Decomposition (EFDD) was used to identify both the frequencies and the modal

shapes. The method is frequency-domain based and makes use of the singular value decomposition of the power spectral density matrix (Brincker et al., 2000). These results were compared with the ones obtained with completely different procedures (e.g. the Stochastic Subspace Identification (SSI)). In order to assess the actual structural bridge safety, the results of modal testing were used to build up a finite element model. These investigations furnish a basis for future more refined models usable to follow the process of degradation.

2 AMBIENT VIBRATION TESTING AND STRUCTURAL IDENTIFICATION

A complete structural identification methodology should address coherently different perspective that can be summarized through four main arts (Aktan et al. 1997b): experimental, analytical, information and decision-making. Different excellent reviews of the state of the art on structural identification have been defined (Kozin and Natke, 1986; Natke and Yao, 1988; Ghanem and Shinozuka, 1995) although mainly devoted to the identification techniques.
The presented discussion is based on results extract from field data, obtained during an extensive campaign of on-site testing conducted recently on 50 bridges in Italy. Due to the large number of structures, in-operation modal analysis has been adopted to obtain the dynamical response. The procedure is a model-based system identification techniques adapted to use output-only data, hence furnishing an estimate of the modal properties of structures in-operational conditions excited by ambient noise (traffic, wind, etc.). One of the drawbacks of the adopted procedure is that the modal participation factors are not determined, since the ambient forces exciting the structure are not known. Consequently, the estimated mode shapes are not correctly scaled. On this respect, the use of additional known masses during the identification procedure has been demonstrated helpful (Parloo et al. 2002; Benedettini et al. 2005). In-operational modal analysis for the bridges has been conducted with both frequency (Brincker et al., 2000) and time domain based methods (Brincker and Anderseen, 1999; Overschee and De Moor, 1996) with a general good agreement; for this reason, in the following, only the results pertaining the frequency domain based technique results are considered. The procedure permits modal determination making use of accelerometer measures of the bridge structural response under traffic-induced vibrations.

2.1 Output-only identification procedure

The identification using output-only data has received an increasing attention due to the possibility of estimating modal models for in-operational conditions structures excited by ambient noise (traffic, wind, etc.). In the present study, the Enhanced Frequency Domain Decomposition, EFDD, has been adopted as a useful procedure to identify the bridge modal properties (Brincker et al., 2000). The EFDD procedure pursues the objective to identify a certain number of simple sdof systems from a set of measured data. It is assumed that each sdof oscillator represents a modal component of the structure. The identification technique makes use of the singular value decomposition of the spectral density matrix containing the Fourier transform of auto and cross correlations of acquired data followed by a backword trasform in the time-domain. The data acquired during the testing are processed through digital treatment of the signal (DFT, overlapping averaging, windowing to avoid leakage, etc.) in order to determine Gyy(f) (m×m), the Power Spectral Density (PSD) matrix of frequency functions of the m system measurements. The EFDD procedure provides the Singular Value Decomposition (SVD) of Gyy(f) PSD-matrix according to:

$$\mathbf{G}_{yy}(f) = \mathbf{USU}^H \tag{1}$$

where S is a the diagonal matrix containing in each diagonal term a singular value of Gxx(f) and U is the matrix containing the corresponding singular vectors. Plotting every singular value as function of the frequency makes possible to select portion of the frequency interval where each singular value posses a peak (peak-picking). The selection of the opportune frequency range is done by means of the Modal Assurance Criterion (MAC) to assure that the singular vector associated to frequency close to the resonant frequency (peak-frequency) are close enough. The use of such criterion permits to treat also cases where two or more modal frequencies are very close. The identification of the damping and the frequency associated to a selected peak, is pursued through the inverse discrete Fourier transform (IDFT), in order to obtain the autocorrelation function. On the autocorrelation function R, a window is used to select a certain number of data used for damping evaluation through logarithm decrement as

$$\delta = \frac{2}{k} \ln \frac{r_0}{|r_k|} \qquad \varsigma = \frac{\delta}{\sqrt{\delta^2 + 4\pi^2}} \qquad f = \frac{f_d}{\sqrt{1-\varsigma^2}} \tag{2}$$

where r_0 is the initial value of R and r_k is the value at the k-point; the relation for the damping is straightforward. The frequency is determined by a linear regression on the interval measured on the crossing of R on the time-axis. Frequency and damping evaluated from any data set are averaged in order to furnish final identified values. Modal shapes, are well estimated by singular vectors corresponding to the peak-values of the singular values, however a weighted average of all singular vectors belonging to the selected set around the peak is pursued. The used weighting factors are related directly to the respective singular vectors.

2.2 Finite element model updating

Many issues should be addressed in the construction of a reliable FE model, among these the main aspects can be grouped in convergence and sensitivity analyses. Finite element convergence analysis is one of the main subject of numerical modelling, however the implication related to the comparison of this error source and the others involved in the structural identification problem, have been only addressed for simple system (Chen and Ewins, 2000). A common understanding is that before starting a FE model updating procedure a convergence analysis should be performed (Aktan et al. 1997b; Chen and Ewins, 2000; Brownjohn et al., 2001). The parameter influencing the final results should be grouped in: (1) material properties, (2) boundary and continuity conditions, and (3) geometric properties. On this respect eigenvalue analysis measuring the impact of parameter variations upon the frequency characteristics by incrementally changing one selected parameter can be pursued neglecting any cross sensitivity. FE model updating involves global issues such the selection between modal or physical models, the formulation of a suitable objective functions, constraint conditions and optimisation criteria (Capecchi and Vestroni, 1993).

3 RELIABILITY EVALUATION OF A BRIDGE STRUCTURE

The reliability of a single bridge component, can be evaluated through its probability of failure Pf, as defined:

$$P_f = P[R - S < 0] \tag{3}$$

where R, S are respectively the last resistance of the element and the internal stresses due to loads. R and S in the Eq. (4) are independent random variables, having a certain level of uncertainties that can be reduced, but not eliminated, with some investigations in site. The safety of a structure is generally expressed through the index of reliability β. If R and S are standard normally distributed (zero mean and unit standard deviations), β can be expressed as

$$\beta = \Phi^{-1}\left(1 - P_f\right) \tag{4}$$

where Φ^{-1} is the inverse function of the normal standard distribution function.

3.1 Reliability of a single component of the structure

It is assumed that the resistance R and the stress S to which the element is submitted, are random variable with lognormal distribution. Therefore, the reliability index can be determined through (Liu et al., 2001)

$$\beta_{member} = \ln\left(\overline{R} / \overline{S}\right) / \sqrt{\sigma_R^2 + \sigma_S^2} \quad (5)$$

where $\overline{R}$ and $\overline{S}$ are the mean of the resistance and the stress while σR and σS are respectively the standard deviations of R and S.

3.2 Reliability of the substructure (superstructure)

The reliability of a bridge substructure is evaluated in a conceptual equivalent way to the case of a single element of it. Assuming again that the loads Fu and Fw are random variables following lognormal distributions, the reliability index of the substructure against the limit state can be defined as

$$\beta = \ln\left(\overline{F_u} / \overline{F_W}\right) / \sqrt{\sigma_{Fu}^2 + \sigma_{Fw}^2} \quad (6)$$

where $\overline{F_u}$ represent the mean of the load causing the failure of the substructure (superstructure) while $\overline{F_w}$ is the mean of the maximum expected external load occurring during the life of the structure and σFu and σFw are respectively the standard deviations of Fu and Fw. In redundant structures βult is always greater than βmember; the difference is due to the structural reserve of carrying loads after the failure of the first element. In other words the difference Δβ constitutes a direct measure of the structural redundancy and it can be calculated by

$$\Delta\beta = \ln\left(\overline{F_{ult}} / \overline{F_{member}}\right) / \sqrt{\sigma_{Fult}^2 + \sigma_{Fmember}^2} \quad (7)$$

3.3 Redundancy of bridge structures

Redundancy is the capability of a bridge structural system to carry loads after damage or failure of one or more of its members. It can be evaluated through direct redundancy analysis executed by a finite element model of the structure. Different analysis can be conducted depending on the directions of the prevailing loads.
Failure of a member of the structure will be recognizable by the formation of a series of plastic hinges, while the failure of the whole superstructure will be identified from the formation of the number of plastic hinges that determines a collapse mechanism. The ratio between forces causing the failure of the entire system and the force causing the failure of one structural member is defined as the system reserve ratio for the ultimate capacity. It constitutes a direct measure of the structural redundancy.

4 VILLA PASSO ARCH BRIDGE: A CASE STUDY

The Villa Passo bridge is a concrete arch bridge constructed on 1917 to overpass the deep valley of Salinello river (Fig. 1). On 1999 the bridge has been fully retrofitted with some main modification in the structural system. After that, starting on 2003, the bridge has been visual inspected and dynamically tested two times within a time interval of one year. Consequently, the campaign of modal testing had the double aim to evaluate the retrofitting effects and to evaluate structural degradations.

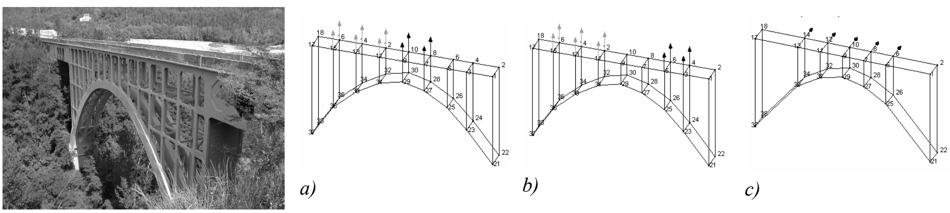

Figure 1: General view of Villa Passo Bridge and accelerometers lay-outs.

4.1 Dynamic spectral properties identified through EFDD

A modal identification procedure based on an ambient excitation has been applied to the bridge structure (length spans (meter): 21.6 - 22.0 - 60.4 - 20.0). The ambient excitation was given by the regular traffic on the bridge. The identification of the modal parameters was carried out using 10 accelerometers: 14 points of the bridge deck were monitored on 2 different setup to identify the vertical components while 5 points were used to identify the horizontal ones in the 3th setup. Time series corresponding to about 3000 first natural period of the structure were acquired at a sampling rate of 400 HZ.
The identification procedure is based on the Enhanced Frequency Domain Decomposition and was conducted using the software ARTeMIS (2002) issued by the Structural Vibration Solutions, Aalborg.
After identifying each mode in the frequency domain, going back in the time domain it is possible to estimate the relevant modal damping by means of the logarithmic decrement method applied to the auto-correlation function.

4.2 Dynamic spectral properties identified through SSI

To validate results obtained by using the EFDD, a time domain identification procedure (Stochastic Subspace Identification) has been applied to the same data acquired during the summer 2003. In this case a parametric model is fitted directly to the raw times series returned by the accelerometers.
The identification starts by fitting different order parametric models by using the time series returned from the accelerometers. After locating the right model order as the minimum order able to give a stable model repeated over the two time series corresponding to the two different setups, the location of the structural modes is obtained.
The results are in good agreement with those obtained working in frequency domain concerning both the natural frequencies and the damping ratios.

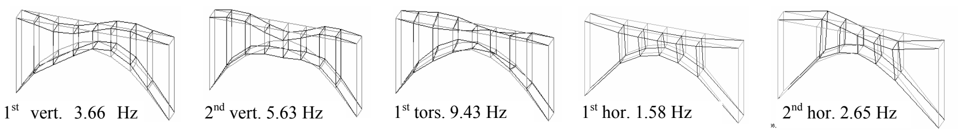

Figure 2: Measured frequency and mode shapes (case B 27.05.2003)

Table 1: Spectral properties obtained by EFDD: case A (07.08.2004), case B (27.05.2005)

Mode	1st vertical	2nd vertical	1st torsional	1st horizontal	2nd horizontal
$f\,(Hz)$ (case A)	3.67	5.64	-	-	-
ξ (%) (case A)	1.5	1.1	-	-	-
$f\,(Hz)$ (case B)	3.66	5.63	9.43	1.58	2.65
ξ (%) (case B)	1.6	1.1	0.8	1.7	1.5

Table 2: Compared spectral properties: estimated (*EFDD*) vs analytical (*FEM*).

Mode	*EFDD* $f\,(Hz)$	*FEM* $f\,(Hz)$	ε_e (%)	ε_c (%)	*MAC* (%)
1st vertical (case A)	3.67	3.69083	0.56	0.44	0.994
2nd vertical (case A)	5.64	5.54924	-1.63	0.69	0.981
1st vertical (case B)	3.66	3.69083	0.83	0.44	0.991
2nd vertical (case B)	5.63	5.54924	-1.45	0.69	0.973

4.3 Villa Passo Bridge FE model updated by modal testing

A reliable structural model of the arch bridge has been developed with particular care to the correct representation of the complex system geometry. The following choices has been made: the arch members, girders, floor beams, columns and bracing members are modeled by two-node 3D beam elements; a) four node shell elements were used to represent the deck slab, the stiffening diaphagrams of the outer walls and the second walls supporting the longer approaching girder, the two approaching box girders and the concrete barriers. The sensitivity analysis of the model frequencies to the mesh refinement was performed with a geometric increment of the number of elements, preserving the initial mesh and subdividing both the shell and the beam element in equal numbers. Four different models have been considered to derive the first two sensitivity of eigenfrequencies to the dimension of the elements. The obtained results are compared each other through the definition of the convergence frequency error as $\varepsilon_c=(f_i - f_{i+1})/f_i$, where i=1,..4 is the number of the considered *FE* model.

In figure 3a) the convergence error ε_c is reported for the two first vertical frequency as function of the number of *dofs* involved in each model. The analysis permits to quantify the level of error involved in this type of analysis, mainly to the lumped mass assumption in the FE model construction (Chen and Ewins, 2000). The expected relative error is of the same order of the error evaluated by updating the material properties. (0.3% for f_{1v}, and 0.5% for f_{2v} are the error obtained by an enlargement of one order (10 ×) in the *dofs* model). A preliminary study has been conducted on the sensitivity of the first two vertical natural frequencies (f_{1v} , f_{2v}) to the modification of two selected material parameters (Young modulus *E*, material mass density *m*). The results have been used for the manual calibration of the FE model. The comparison between the selected FE model candidate and the modal testing data have been conducted evaluating the estimate relative error $\varepsilon_{ei}=(f_{fi} - f_{ei})/f_{fi}$ between the frequency evaluated by FEM f_{fi} and the estimated frequency f_{ei} by EFDD. The comparison has been also performed comparing the FEM modal shapes φ_{fi} and the estimated one φ_{ei} using the conventional MAC. The results are compared in Table 2 for both the campaign of testing conducted on the bridge.

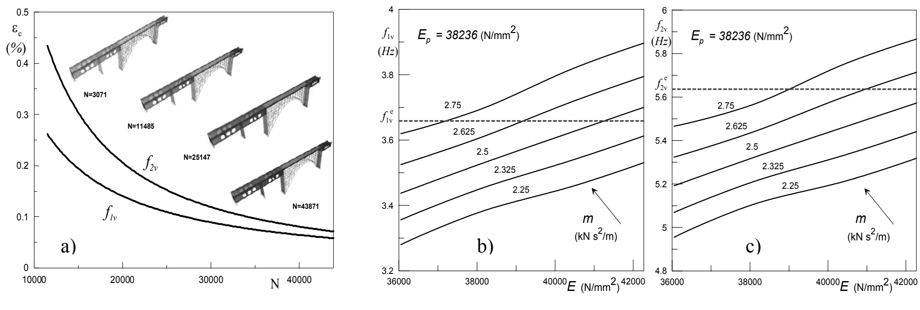

Figure 3: a) Finite element convergence analysis: frequency error vs N number of degree of freedom (1st and 2nd vertical natural frequencies); b), c) Frequency sensitivities to concrete Young modulus and mass density: first and second vertical modes respectively

Table 3: Reliability indices

		β_{member}
S. 1	*Bending*	2.93
	Shear	2.35
S.2	*Bending*	4.13
S.3	*Bending*	3.02
	Shear	2.35

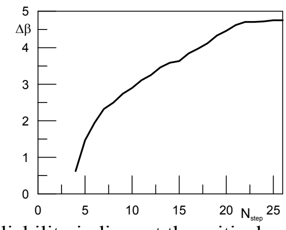

Figure 4: Reliability indices at the critical sections: effects of structural redundancy on reliability to vertical load

4.4 Nonlinear static limit analysis to vertical and horizontal loads

A linear analysis has been performed to determine the internal stresses due to vehicular loading in critical sections of the structure. Successively, positions of plastic hinges have been assigned to the critical sections of the beams belonging to the superstructure and a vertical load uniformly distributed has been increased at each step up to reach the failure condition. According with the method presented the structural reliability was evaluated by Eq.(7) where the standard deviations (σ_{member} = 0.13, σ_{ult} = 0.30) were assumed as suggested by analyses conducted by governative institutions (Liu et al., 2001). Table 3 shows the reliability indices of the superstructure, for bending or shear, at the beam supports (Sect. 1 and 3) and at the middle span (Sect. 2). Increasing the loading in the pushover analysis, several hinges reaches the yielding limits in both the superstructure and in the arch beams. The structural redundancy with respect to vertical loads is evaluated. The analysis are considered finished when the structure is not able to carry any increment of push loading and the increment $\Delta\beta$ in the reliability index reaches a stationary value.

In order to perform a non-linear lateral limit analysis of the whole structure, a few modifications of the finite element model was necessary. Indeed, in order to assign discrete plastic hinges, the concrete walls forming the piers, previously modelled with shell elements, were modelled in these analyses with frame elements connected by rigid links with the secondary beams of the bridge. Soil-structure interaction was also modelled through horizontal and rotational springs. A Modal Pushover Analysis (MPA) was performed, in which the structure is controlled by a mode collapse shape unchanged after yielding. In the analysis, firstly, the structural weight were applied and a vertical nonlinear analysis was performed including both local (P-δ) and global (P-Δ) nonlinear geometrical effects. Then, the horizontal load was increased through the MPA and the global capacity curve (base shear versus top displacement) was obtained. To evaluate the structural reliability to seismic loads, bridge lateral resistance capacity was compared with the expected stresses induced by seismic force. The latter evaluation was performed according to Italian code spectrum. The amplitude of the first horizontal modal load was evaluated as $F_w = M\,S_d(T_1)$ where M, T_1 and $S_d(T_1)$ are modal mass, natural period and response acceleration. Assuming a design ground acceleration $a_g = 0.25g$ and B-type soil, the elastic spectrum was defined and through the structural factor $q = 3.5$. Then, the seismic load F_w was easily evaluated. The reliability index referred to first member failure was evaluated ($\sigma_{Fu} = 0.13$, $\sigma_{Fw} = 0.33$) where $\overline{F}_u$, determined through MPA, is the level of pushing force causing the first failure. The residual strength or redundancy $\Delta\beta$ with respect to lateral loads was estimated increasing the lateral forces and evaluating $\Delta\beta$. The numerical results are depicted on Fig.5 where the augment of reliability due to structural redundancy $\Delta\beta$ for each level of pushing force $\overline{F}_u$ (Nstep) is added to the reliability associated to the first member failure to obtain the global reliability index $\beta_{ult} = \beta_{member} + \Delta\beta$.

5. CONCLUSIONS

The paper summarizes initial efforts of a research devoted to the development of a structural identification methodology useful for bridge management and maintenance of large road, highway and railway infrastructure systems. The main goal of the study has been to determine spectral properties of in-operation bridges through output-only based modal identification techniques and to use this information to build up a reliable FE model of the bridge. The data fusion of information coming from visual inspections, recursive modal testing, FE model updating should be pursued in future works to assess the bridge condition in a reliable and useful procedure. A methodology for evaluation of structural safety of an existing concrete arch bridge has been pursued for the loading cases of vehicular traffic and expected earthquakes in site. The method utilizes as safety indicator the reliability factor β, the use of which permits to take into account the uncertainties related to the external loadings and the internal resistances with particular attention to the evaluation of the effects of the structural bridge redundancy. The study evidences that while the reliability related to the formation of the first plastic hinge is quite similar for vertical vehicular traffic and horizontal seismic loads, the vertical structural redundancy in substaining traffic loading is significantly less than the one related to horizontal seismic loads. The presented reliability analyses constitute a preliminary investigation useful to conceive a reliability based bridge management system.

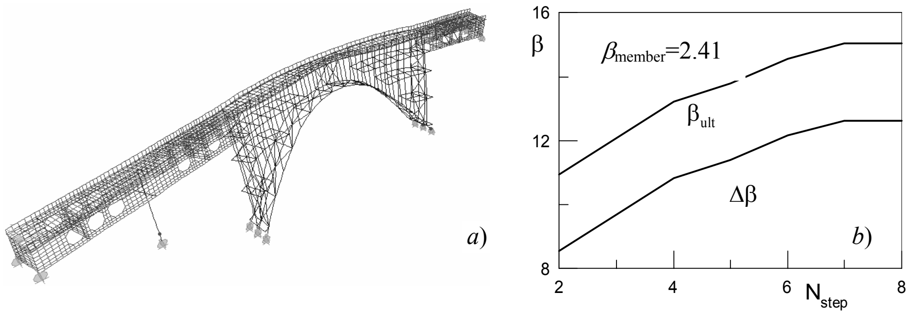

Figure 5: MPA results: a) last collapse scenario,
b) redundancy and global safety level of the structure vs MPA steps.

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