Structural Health Monitoring of civil structures through FEM high-fidelity modelling.

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Abstract. FEM high-fidelity modeling is a useful method for evaluating the dynamic behavior of civil structures and their state of damage through the processing of real-time vibration data in the frame of the general Structural Health Monitoring. The development of reliable digital twin involves the use of high-fidelity models, which must be validated with experimental data. In this study, a prototype of a bridge, composed by a frame structure of steel beams and a slab made in concrete and asphalt, was considered. Two of the main beams have a bolted connection that have been used to simulate a local damage of the structure. FEM models have been validated with experimental data, thus they are able to evaluate both static and dynamic behavior of the bridge and its state of health, referred to benchmark structure. The bridge was degraded considering the breakage of one or more plates of the bolted connection, or an accidental overload capable of creating plastic deformations. The proposed models identify damage of the structure through two different methods: shift of natural frequencies, and evaluation of frequency response of the structure (FRF) at the sensor nodes. The second method is more suitable since the variations of observed variables is more consistent. Therefore, with these models, it is possible to carry out monitoring operations for the predictive maintenance of the structure and defining the limits that the observed variables can undergo in order to still consider the structure suitable for the operating loads.

1. Introduction
It is well known that unexpected damage of structure, mechanical systems or plant components can origin relevant safety risk or loss of money. This can be avoided by means of a smart maintenance based on continuous and accurate monitoring during the operating life coupled with models able to identify the damaging process and to predict its time progress [1][2]. In the last decades, Structure Health Monitoring (SHM) has attracted the interest of researchers and many papers and books on this topic are available in the scientific literature [3][4].

Generally, a robust SHM system should have some requirements that can be summarized as follows: able to identify very early the damage and locate where it is occurring. In addition, it is necessary to estimate its severity, predict the remaining useful life basing on objective procedures without involving the engineering judgement of the user. This last feature is the prognostic part of SHM [5]. All these features must be automated in order to manage lot of data collected on field. In recent years, thanks to the possibility of automatically handling large amounts of data at an acceptable cost, the creation and use of a digital twin of the structure under observation has been widely used in many fields of application, especially considering how complex can be the physical phenomena involved [6][7][8].
Without any doubt the heart of the Structural Health Monitoring is the selection of the methods to identify the behavior of the structure and its variation that can indicate a possible damage [9][8]. There are two strategies for structures monitoring: static monitoring, in which some parameters are detected in a long time, and dynamic monitoring oriented towards the evaluation of dynamic parameters, like natural frequencies, damping coefficients and modal shape. Consequently to this selection, the sensors, the acquisition chain, the managing software can be stated. The modern experimental technology offers several methods to investigate a structure as visual automated inspection, acoustic emission, static deformation, vibration behavior [10]. All the experimental data can compared with those extracted by a numerical model (the digital twin) in the same operative condition with the aim to identify and locate the damaging process.

Relating to the dynamic behavior of a structure, it is important to note that large-scale civil or mechanical structures are normally stressed by various loads time varying that can cause vibration as earthquakes, wind, temperature, or human-made excitation. On the other end both the sensors and numerical simulation methods are widely used so that this kind of monitoring is the main system for SHM. Several types of approaches are now available to monitor how a structure is changing [9][10] as frequency [11], mode shapes and frequency response functions [12], flexibility and compliance, damping and so on [9].

In this paper the change of frequency and of frequency response is used to identify the damage in a structural part of a prototype of bridge. In general, bridges are infrastructure very relevant [13] as they connect different regions and originate social and economic benefits. The health monitoring of these is essential to maintain properly adapting to changing conditions of use, to climate-driven or environmental events [14][15].

In this study a prototype of bridge built in the frame of the research project SHAPE was used to develop a digital twin FEM based, in order to define some type of damaging process and to evaluate the change in the dynamic behavior as frequency response function. In particular, as the prototype has in two of the main beams a bolted connection, on these elements has been simulated a local damage both of the bolts and of the connecting plates. As first step of the investigation, the FEM model was tuned basing on the natural modes measured on the prototype using, in the part where the damage is located, a high degree of definition. This allowed not only to study accurately how the dynamic response change due to the damage but also to reduce the computational times and costs allowing in this way to simulate several damaging scenarios.

2. SHM of civil structures: bridge example.

The laboratory structure investigated is a bridge 1:4 scale using an orthotropic steel-concrete deck and two concrete piles Figure 1. The deck consists of five profiles IPE 300 for the main structure made of steel S275, while three of them were used as main beam, the other two were used as head transverse beam Figure 2. The in-plane stiffness was increased by using a series of coupled UPN 140 profiles as a transverse beam and “L” shaped profile as cross-bracing. This small steel grillage works together with a composite slab made of a steel corrugated sheet and covered with concrete. The steel deck rests on a rubber layer of 15 mm placed below each beam, each support rest on the two lateral abutments made of concrete.

Figure 1. View of the steel-concrete bridge.
As above noted, two of the main beams have a bolted connection where the simulated damage has been located. The bolted joint is placed at one third of the length (Figure 2).

![Bolted connection at one third of the length.](image)

**Figure 2. Bolted connection at one third of the length.**

Experimental dynamic investigation on the bridge was performed extracting the mechanical parameters of concrete and the stiffness of supports [16]. Table 1 shows the results obtained for the dynamic investigation executed placing accelerometers on the deck and on the beams. As excitation source a hammer below the beam was used in different positions.

| Order | Type            | Frequency [Hz] |
|-------|-----------------|----------------|
| 1     | 1° Flexural     | 17.50          |
| 2     | 1° Torsional    | 20.71          |
| 3     | 2° Flexural     | 56.78          |
| 4     | 2° Torsional    | 59.63          |
| 5     | 1° Plate        | 64.42          |

**Table 1. Results of the dynamic investigation.**

3. **Simplified model identification and calibration.**
Basing on the data above reported, the FE model for the code NASTRAN NX [17] equipped with pre-post processor FEMAP was implemented. The model is composed by shell elements (equal to 76 mm or 3 inches) reproducing the slab of the deck, truss elements modelling the cross bracing, and finally beam elements are used to model the main and transverse beam. During the modelling phase, the rubber layer below each beam has been modelled by replacing the rigid restraints with three elastic springs along the three main direction. Starting with the initial values of the density, the elastic modulus of each material and preliminary values of the springs’ stiffness, these parameters have been updated to better fit the experimental frequencies.

![Simplified FEM model.](image)

**Figure 3. Simplified FEM model.**
Table 2 shows the values of mechanical properties of the model, while Table 3 shows the properties of the restraint springs for the FEM.

### Table 2. Mechanical properties assigned to the steel bridge.

| Material       | Thickness [m] | Elastic modulus [MPa] | Density [kg/m³] |
|----------------|---------------|-----------------------|-----------------|
| Steel S275     | -             | 210000                | 7860            |
| Concrete       | 0.073         | 30000                 | 2450            |
| Asphalt        | 0.03          | 6000                  | 1800            |

### Table 3. Properties of restraint springs.

| Spring direction | Direction     | Elastic modulus [kN/m] |
|------------------|---------------|-----------------------|
| X                | Deck axis     | 1*10⁴                 |
| Y                | Vertical      | 1*10⁵                 |
| Z                | Transversal   | 1*10⁶                 |

After identification of material properties, a comparison between the natural frequencies of FE model and experimental ones has been carried out. Relative error between the two values has been computed and collected in Table 4.

### Table 4. Comparison between experimental results and numerical results.

| Order | Type         | Experimental [Hz] | FEM [Hz] | Err % |
|-------|--------------|-------------------|----------|-------|
| 1     | 1° Flexural  | 17.50             | 18.26    | 4.34  |
| 2     | 1° Torsional | 20.71             | 19.76    | 4.59  |
| 3     | 2° Flexural  | 56.78             | 55.37    | 2.48  |
| 4     | 2° Torsional | 59.63             | 58.35    | 2.15  |
| 5     | 1° Plate     | 64.42             | 64.41    | 0.02  |

Figure 4 shows the mode shapes for the bridge. They are in good agreement with those identified experimentally by using Frequency Domain Decomposition procedure.

![Mode shapes](image.png)
4. Static and dynamic behaviour of bolted joint.
The second step of the investigation was to study the mechanical behaviour of the steel beam with bolted connection. Figure 5 shows the beam section geometry. Both beam and plates of connection are modelled with solid elements, while bolts are modelled with beam elements. RBE2 rigid elements, available in NASTRAN NX, simulate the presence of nut and lock nut. Bolt preloads are evaluated starting from tightening torque of 281 Nm, according to the values suggested by EUROCODE. Contacts between plates and beam are modelled as frictional contacts. In this case, the continuity between the two parts is linked to the effect of bolts tightening. The friction coefficient is set to $\mu = 0.4$.

$$
\begin{align*}
&h = 300 \text{ mm}; \quad b = 150 \text{ mm}; \quad e = 10.7 \text{ mm}; \quad a = 7.1 \text{ mm}; \quad A = 5188 \text{ mm}^2 \\
&I_{xx} = 7.996 \cdot 10^7 \text{ mm}^4; \quad I_{yy} = 6.027 \cdot 10^6 \text{ mm}^4; \quad J_t = 1.557 \cdot 10^5 \text{ mm}^4 \\
&W_{xx} = 5.331 \cdot 10^5 \text{ mm}^3; \quad W_{yy} = 8.036 \cdot 10^4 \text{ mm}^3; \\
&W_t = 1.456 \cdot 10^4 \text{ mm}
\end{align*}
$$

Figure 5. Beam section geometry.

4.1. Prestressed Modal analysis.
It is customary to perform modal analysis on mechanical systems without due regards to their stress state. This approach is of course well accepted in general but can prove inadequate when dealing with cases like beam with a bolted connection. It is believed that the stress stiffening can change the response frequencies of a system which impacts both modal and transient dynamic responses of the system. This is explained by the fact that the stress state would influence the values of the stiffness matrix [18]. To takes into account of bolt preloads, preliminary nonlinear static analysis is performed for calculating differential stiffness to be included in the normal mode solution.

Figure 6. FE mesh.

Figure 7. Prestressed modal analysis.

Prestressed modal analysis is performed using SOL401 solution in NASTRAN NX. SOL 401 - NLSTEP. SOL 401 is a multistep, structural solution which supports a combination of static (linear or
nonlinear) subcases and modal (real eigenvalue) subcases. The primary solution operations are time increments, iterations with convergence tests for acceptable equilibrium error, and stiffness matrix updates. The iterative process is based on variations of Newton's method. The stiffness matrix updates are performed to improve the computational efficiency. SOL401 consider the internal force computation as follows [5]:

\[ F = \int_V \bar{B}^T \sigma dV \]  

(1)

the element matrix \( \bar{B} \) is defined from the strain definition as

\[ d\varepsilon = \bar{B} du \]  

(2)

in which \( \bar{B} \) could be divided into two parts (linear and nonlinear), i.e.

\[ \bar{B} = B_L + B_N \]  

(3)

upon differentiation of Eq.(1), we have

\[ dF = \int_V \bar{B}^T d\sigma dV + \int_V d\bar{B}^T \sigma dV \]  

(4)

\( \sigma \) represents stresses. Substituting Eq(3) and

\[ d\sigma = D d\varepsilon = D \bar{B} du \]  

(5)

Eq(4) becomes

\[ dF = [K_L + K_R + K_\sigma] du \]  

(6)

with

\[ K_L = \int_V B_L^T DB_L dV \]  

(7)

\[ K_R = \int_V B_L^T DB_N + B_N^T DB_N + B_N^T DB_L dV \]  

(8)

\[ K_\sigma du = \int_V dB_N^T \sigma dV \]  

(9)

in which \( K_L \) represents the usual linear stiffness matrix, \( K_R \) a stiffness due to large rotation, and \( K_\sigma \) a geometric stiffness dependent on the initial stress level.

Figure 8. Contact pressure on plates.
Figure 8 shows contact pressure on plates caused by bolt preloads. Beam is simply supported at both ends. Table 5 shows the results of the modal analysis in different grip conditions of bolted joint:

- C0: design conditions.
- C1: joint with one free beam wing.
- C2: joint with free beam wings.
- C3: joint without core plates.
- C4: joint without core plates and one free beam wing.

|    | C0 [Hz] | C1 [Hz] | C2 [Hz] | C3 [Hz] | C4 [Hz] |
|----|---------|---------|---------|---------|---------|
| 1  | 26.24   | 24.75   | 22.35   | 26.60   | 4.59    |
| 2  | 102.06  | 98.82   | 95.08   | 101.51  | 78.28   |
| 3  | 206.92* | 199.70* | 195.39  | 204.15* | 155.87  |
| 4  | 219.25  | 219.02  | 216.89  | 207.49  | 197.27  |
| 5  | 343.42  | -       | 348.26  | 353.33  | 314.04  |

Later, a comparison of FE results with theoretical solution of a beam with a concentrated mass representing the bolted joint (Figure 8) is reported.

Natural frequencies of a beam without joint can be obtained from Eq. (10):

\[ f_i = \frac{K_i^2}{2\pi^2} \sqrt{\frac{EI}{\rho A}} = 28.11 \text{ Hz} \]

(10)

where \( E = 210 \text{ GPa} \), is the Young modulus of material; \( I_{xx} = 7.996 \times 10^{-5} \text{ m}^4 \), represent moment of inertia of the section in the vertical deformation plane; \( A \) is the section area; density of material \( \rho = 7800 \text{ kg/m}^3 \); \( L \) the distance between supports. \( K_i \) values depends on boundary condition and frequency order. Considering a simply supported beam and the first natural frequencies \( K = \pi \). The bolted joint, including plates and bolts, weigh 17 kg. The application of superposition principle using Dunkerlay method allow the evaluation of the first natural frequency.

\[ \frac{1}{f_{tot}} = \sqrt{\left(\frac{1}{f_i}\right)^2 + \left(\frac{1}{f_m}\right)^2} = 26.53 \text{ Hz} \]

(11)

Percentage error between numerical and theoretical is 1.09%. FE model is validated also considering axial deformation.
\[ f = \frac{1}{4L} \sqrt{\frac{k}{\delta}} = 216.20 \text{ Hz} \]  

Compared with FE result of 206.90 Hz, relative error is around 4.30%. The influence of different joint configurations on the dynamic behavior of the structure are shown in Table 5. The major contribution to the strength of the joint is given by the connections on the wings as they are more distant from the neutral axis (C1, C4).

4.2. Nonlinear static analysis.
Nonlinear static analysis is performed to study the mechanical performance of bolted joint. The applied loads are:
- Gravity load.
- Overload which causes yielding in the midsection of the beam.

For each step of the applied load the following quantities are calculated:
- Maximum deflection.
- Wingspan on the traction side of beam section.
- Bolt preload.

Figure 10 shows these quantities function of applied load.

**Figure 10.** (a) Maximum deflection, (b) Relative displacement traction plates, (c) Wingspan.
No regular trends are linked to friction and local deformability. Friction between plates involves a variation in bolt preloads and a relaxation of the connection. This effect shows local deformability of the joint plates (Figure 11).

Figure 11. Deformability of the joint plates.

Damage of bolted connection is simulated removing lower plate of the joint (Figure 14).
Another nonlinear static analysis is performed in this configuration. The damaged joint has a lower stiffness, as expected. In this case, maximum deflection of the structure is at the joint (Figure 15).

5. FEM model: Hybrid formulation.
Next step of this study concerns the development of a FEM hybrid model, where one of the main beams and its bolted joint are modelled with solid elements. In this way, the actual behaviour of bolted joint and its influence on the dynamic behaviour of the bridge is considered. Figure 16 shows the mesh used in this model. Glued contacts are set between solid beam and surrounding structure, in order to guarantee structural continuity.
Dynamic analysis is performed in the same manner as previous analysis of steel beam with bolted joint shown in Chapter 4. Preliminary nonlinear static analysis takes into account of bolt preloads and its influence on the next normal mode solution. Table 6 shows the results of modal analysis.

| Order Type | Experimental [Hz] | FEM [Hz] | FEM hybrid [Hz] |
|------------|-------------------|----------|-----------------|
| 1          | 1° Flexural       | 17.50    | 18.26           | 16.89           |
| 2          | 1° Torsional      | 20.71    | 19.76           | 18.63           |
| 3          | 2° Flexural       | 56.78    | 55.37           | 53.14           |
| 4          | 2° Torsional      | 59.63    | 58.35           | 54.47           |
| 5          | 1° Plate          | 64.42    | 64.41           | 59.97           |

The results are in good agreement with experimental one. Relative errors are lower than 10%. Later, bridge is degraded considering the breakage of one or more plates of the bolted connection. Damage is identified through two different ways: shift of natural frequencies, and evaluation of frequency response of the structure (FRF) at the sensor nodes.

5.1. Shift of natural frequencies.
A modal analysis is performed, and the first twenty frequencies of the structure evaluated. Figure 17 shows the shift of each mode. Six different types of damage are considered:

1. Core plate.
2. Upper plate.
3. Lower plate
4. Upper plate + core plate.
5. Upper plate + lower plate.
6. Lower plate + core plate.

The greatest variation is observed in the absence of the bottom plate and core plates (Level 6). These frequencies are associated with modes shape in which the joint has high curvature.
Table 7. Structure natural frequencies for each damage level and relative variation respect to design condition.

| DAMAGE LEVEL | 0  | 1  | 2  | 3  | 4  | 5  | 6  | 7  | 8  | 9  | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 |
|--------------|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|
| FREQ (Hz)    |    |    |    |    |    |    |    |    |    |    |    |    |    |    |    |    |    |    |    |    |
| 0.150        | 64.972 | 64.972 | 64.972 | 64.972 | 64.972 | 64.972 | 64.972 | 64.972 | 64.972 | 64.972 | 64.972 | 64.972 | 64.972 | 64.972 | 64.972 | 64.972 | 64.972 | 64.972 | 64.972 |
| 150          | 64.972 | 64.972 | 64.972 | 64.972 | 64.972 | 64.972 | 64.972 | 64.972 | 64.972 | 64.972 | 64.972 | 64.972 | 64.972 | 64.972 | 64.972 | 64.972 | 64.972 | 64.972 | 64.972 |

5.2. FRF analysis.
Frequency response is the quantitative measure of the output spectrum of a system or device in response to a stimulus and is used to characterize the dynamics of the system. It is a measure of magnitude and phase of the output as a function of frequency, in comparison to the input. The frequency response diagrams were obtained using an input-output methodology named SISO (Single-Input-Single-Output).

Figure 18 shows load node at the centre of slab and the sensor node located at the joint. Frequency range is 0 – 150 Hz. Structural damping is set to 5%, usually used in civil structures.

Figure 18. Load node and sensor node location.

Figure 19 display frequency response functions (inertness) for six different types of damage.
Figure 19. Frequency response amplitude.

Table 8 show the shift of first peak of measured acceleration for different levels of damage scenarios. In this case, amplitude variations are more consistent than frequency variations, compared with the same damage configuration. Considering the absence of core plates, amplitude variation is around 2.7%, while frequency variation is only 0.2%. For this reason, observing amplitude variations allow a better identification of structural damage.

Table 8. Shift of FRF amplitude (1° peak).

|   | Δa %  | Δω %  |
|---|------|------|
| 1 | 2.72 | 0.22 |
| 2 | 0.35 | 0.06 |
| 3 | 17.57 | 2.73 |
| 4 | 2.31 | 0.16 |
| 5 | 17.57 | 2.63 |
| 6 | 26.42 | 21.07 |
FRF analysis is also performed considering a wind load in the horizontal direction.

As seen in Figure 20, frequency response functions for C1, C2 and C4 configurations have the peak amplitude at around 60 Hz, and its value is higher than the benchmark configuration; while for C3, C5, C6 amplitude at 60 Hz is lower than the undamaged structure but amplitude at around 100 Hz increases and for C6 configuration becomes the highest.

**Conclusions**

In this study, a digital twin model of a prototype bridge structure was built, with the aim of evaluating both static and dynamic behaviour. Two of the main beams of the bridge have a bolted connection used to simulate a local damage of the structure. The novel approach consists in modelling only one beam of the structure and its bolted connection with a high degree of definition. This allowed a reduction in computational times and costs and the possibility of simulating a damage to the joint considering its actual behaviour. Damage was identified through two different methods: shift of natural frequencies, and evaluation of frequency response of the structure (FRF) at the sensor nodes. Amplitude variations...
of FRF allow a better identification of structural damage since variations were more consistent. Therefore, with these models it is possible to carry out monitoring operations for the predictive maintenance of the structure and defining the limits that the observed variables can undergo in order to still consider the structure suitable for the operating loads.

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