

Dynamic Monitoring of a Railway Steel Bridge with MEMS Accelerometers: First Results on the Case Study of Portella

Alberico Sonnessa[1(✉)] and Mariano Macellari[2]

[1] Department of Civil, Environmental, Land, Construction and Chemistry (DICATECh), Politecnico di Bari, Via Orabona 4, 70125 Bari, Italy
alberico.sonnessa@poliba.it

[2] Trenitalia S.p.A. Transport Data Management, Piazza delle Croce Rossa, 1, 00161 Roma, Italy
m.macellari@trenitalia.it

Abstract. The continuous growth of demand on railway networks requires an efficient management strategy to guarantee proper levels of safety and service. Bridges are a fundamental asset for a railway infrastructure and their integrity must be ensured implementing feasible approaches aimed at avoiding critical occurrences (e.g. collapses) by quantifying infrastructure responses to standard and critical loads, in terms of vibration features and long-term displacements. The work proposes a geomatic monitoring configuration for a new steel railway bridge -the Portella bridge- located along the Roma–Napoli railway, showing the results obtained analyzing three weeks of continuous monitoring in different seasons. The system, based on triaxial MEMS accelerometers and envisaging additional sensors (GNSS receivers) in its developments, was installed at the end of the bridge construction as a lifetime tool, and is designed to highlight the effects of the train passage. Data analysis permitted to retrieve structure accelerations and vibration normal modes (natural frequencies). The findings of the investigations will be used to improve the Finite Element model of the bridge used in the design phase.

Keywords: Critical infrastructures · Railway bridges · Lifetime geomatic monitoring · MEMS accelerometers · Vibration normal modes analyses

1 Introduction

The increasing demand for mobility has significantly boosted the evolution of fast transport routes, as evidenced by the growth in the length of high-speed railways in Europe from 2.500 km to about 10.000 km in the period 2000–2022. The expansion of the Trans-European Transport Network (TEN-T), the planned network of roads, railways and airports infrastructure that is currently under development among the European Union countries [1], will make available further connections aimed at facilitating the mobility of goods and people.

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Along with this rapid evolution, the need of an effective management strategy arises, which is essential for ensuring a safe use of the infrastructures and avoiding critical events (i.e. large deformations, collapses), such as those that interested the transport networks during the last decades (e.g. the collapses of the road bridge in Albiano Magra - April 2020 and the Morandi bridge in August 2018 [2] and the I-35W Bridge in Minneapolis, USA in 2007 [3]). These occurrences evidenced how bridges are often the element of a transport infrastructure most likely to be affected by instability problems, caused by design flaws, aging or subsidence of the foundation soil. Among the transport networks, the railway infrastructure has constantly increased its importance, as a direct result of the growing demand for a faster and greener handling of passengers and goods. Rail bridges often work close to their operating limit in terms of axle load, maximum transfer speed, train frequency and length [4, 5]. This can speed up deterioration processes and decrease the safety level. The need to assess their health conditions (Structural Health Monitoring-SHM) requires overcoming the traditional approach based on testing in the pre-operating phase and monitoring if damages and/or risk conditions occur. A radical change of perspective must consider a monitoring system as the standard equipment of a structure over the entire life cycle. The importance of field measurement outcomes for quantifying vibration stresses in steel bridges is clearly evidenced in [6]. The structural health of these assets must be guaranteed implementing integrated real-time monitoring approaches, fulfilling both technological and budget constraints. Integrated survey and monitoring techniques [7–9] can play a key role in their safeguard, allowing a more accurate control of their behaviour and providing timely information to the infrastructure manager.

All these things considered, an effective monitoring configuration for rail bridges, based on the use of Micro Electro-Mechanical Systems (MEMS) accelerometers is hereafter presented, and preliminary results of the analyses on the data provided by MEMS sensors is discussed. This configuration has been designed to provide high-frequency measurements (acceleration and vibration frequencies) from accelerometers. MEMS accelerometers have proven to constitute an efficient and low-cost solution for SHM in civil engineering applications concerning the bridge control [10, 11], in multi-sensor systems aimed at monitoring the dynamic response of structures and gathering information such as natural vibrating frequencies [12], and as a tool for early warning in case of critical events (e.g. earthquakes) that may affect civil infrastructures [13]. A number of experiences on the use of MEMS aimed at analysing the effects of wind, earthquake and external loads, and evaluating their performances for damage detection purposes have been presented in [14–16]. This research shows the first results of the analysis conducted on the dataset acquired by a monitoring system based on triaxial accelerometers, installed on a steel bridge located along the high-speed Roma-Formia-Napoli railway (Italy). The monitoring system was installed to track its structural behaviour since the very beginning of the structure life, enabling to collect a bulk of information useful to timely detect possible anomalies. The test phase was aimed at highlighting the effects of the rail traffic on the infrastructure in terms of acceleration peaks and, mainly, vibration modes (natural frequencies) derived through Fourier analysis of accelerations. Section 2 presents the case study and the currently installed monitoring system; Sects. 3 and 4 illustrate the data analysis results, with respect to accelerations and vibration normal

modes (natural frequencies); Sect. 5 discusses the obtained results. Conclusions and planned evolution of the monitoring system are discussed in the last sections.

2 Case Study – The Portella Bridge and Its Monitoring System

The Portella steel bridge is located at km 102 + 092 of the Roma-Formia-Napoli railway, between the Priverno-Fossanova and Monte S. Biagio stations. The bridge, with a span of 28 m, consists of two reticular beams placed longitudinally at a distance of 10,30 m, with a height ranging from 4 m to 5,50 m (Fig. 1). The deck is formed by a series of transverse steel beams, an overlying steel sheet extended to the entire perimeter, and a concrete slab supporting the ballast and the two tracks. The steel bridge rests on reinforced concrete shoulders by mean of reverse-shell teflon-steel supports [17].

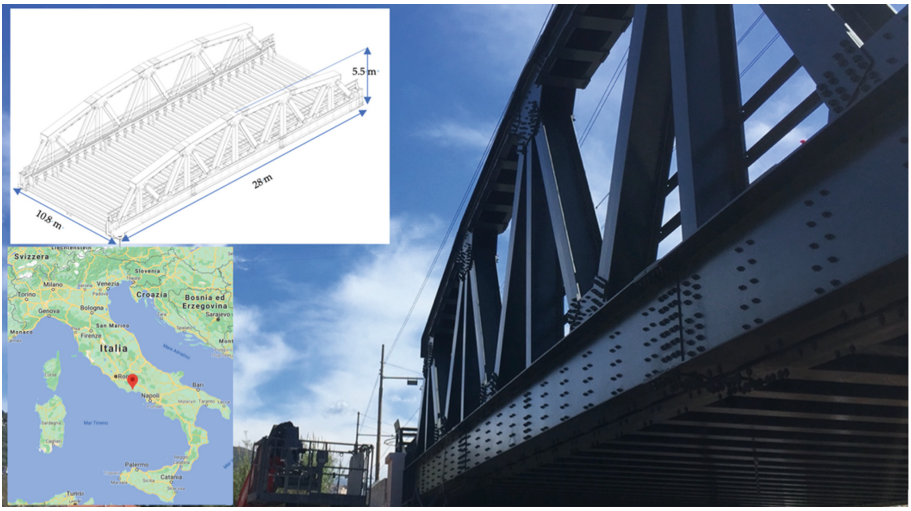


Fig. 1. The Portella bridge

The bridge crosses the Portella canal. Current line speed is 180 km/h, but the bridge is designed for allowing a maximum operative speed of 200 km/h.

The scheme of the implemented monitoring system encompasses the joint use of ten Kistler Type 8396A/2D0 high-sensitivity/low noise accelerometer (Fig. 2) simultaneously measuring acceleration on the X, Y and Z axes (A1D to A5D along the north-south track, A1P to A5P the south-north track) have been installed according to the layout in Fig. 3.

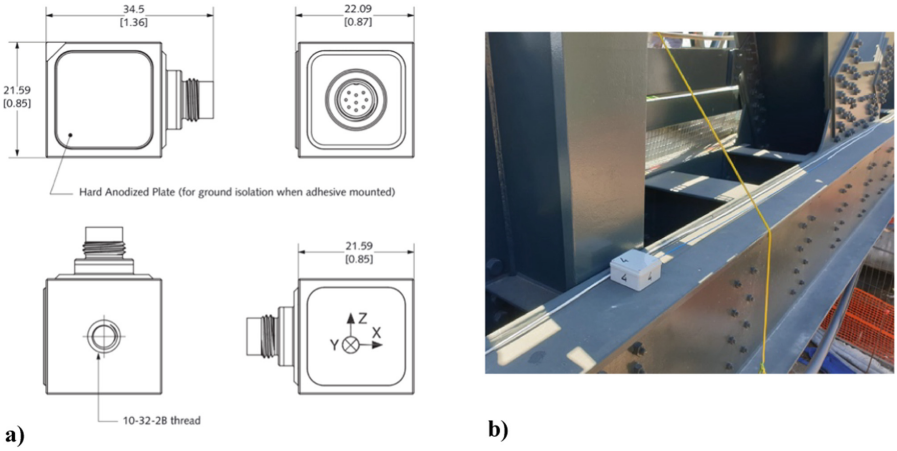


Fig. 2. (a) Kistler accelerometer. (b) One of the sensors located on the metal structure

X axis is oriented along the tracks direction, Z axis along the gravity direction and Y (across the tracks direction) completes the cartesian reference system. A1P/D and A5P/D lie on the concrete shoulders of the structure, whereas A2, 3, 4 P/D are placed on the metal bridge. The sensors are nominally characterized by a $\pm 2g$ range measurement and a frequency response ranging from 0 to 1150 Hz. The capability of better detecting the natural vibration frequencies has been the main criterion for identifying the proper sensors to be used in the system. Each accelerometer records data at a sample rate of 2000 Hz and stores the information in a proprietary format containing the acceleration value and the corresponding time epoch. Two photoelectric triggers (TRG-P and TRG-D), located approximately near A1-D and A5-P accelerometers, automatically detect the train approaching, and give the start to the beginning of the recordings. The dataset has been investigated using a code developed on purpose.

3 Data Analysis – Accelerations

A first statistical analysis has been developed on acceleration signals, directly linked to the stresses suffered by the structure, with the aim of evaluating their weekly peaks and average maximum. The acceleration values have been examined for each accelerometer to evaluate the actual stress impacting the bridge, to be compared with the design values. Data were recorded in three weeks, in different weather conditions have been considered.

- week 1 (W1), 28th April - 4th May
- week 2 (W2), 28th July - 3rd August
- week 3 (W3), 27th October - 2rd November

3.1 Analysis of Peak Accelerations

For each accelerometer, a cumulative analysis has been performed by estimating the peak accelerations on the X, Y and Z axes over the whole period W1-W3 (Table 1, Fig. 4. For

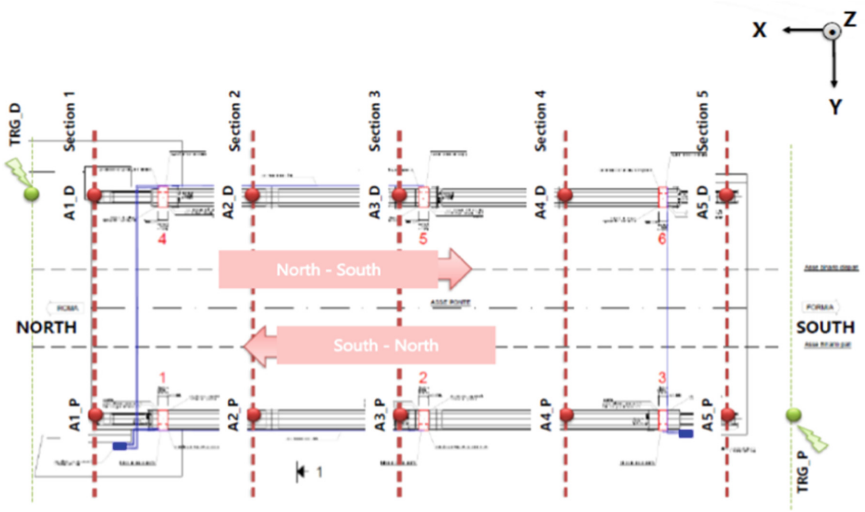


Fig. 3. Monitoring layout

each axis, the average values of the peaks are reported with relative standard deviation, and the six values (X min/max, Y min/max, Z min/max) representing the maximum accelerations affecting the structure along the three directions, are highlighted (Fig. 4).

The peak accelerations analysis evidenced that the Y and Z peaks measured by the accelerometers located on the bridge span (A2, A3, A4 D/P) are likely not fully indicative of the true peaks suffered by the structure, due to the limitations in the sensors measuring range. The example accelerogram of Fig. 5a highlights how the maximum values measured along the Y axis and the positive part of the Z axis correspond to the full scale of the instrumentation (indicated in the boxes) (Table 2).

The red squares show the reaching of the saturation of the measures for a high number of epochs in the Y (positive and negative component) and Z (positive component) directions, and for a limited number of epochs relative to the negative component in the Z direction (green square).

Therefore, the actual peak accelerations probably reach (at least) the value measured on the negative part of the Z axis, which, after being referred to the median values, is equal, in absolute value, to 33.5 m/s^2 (Fig. 5b). Since the main goal of the project was the detection of vibration normal modes, at this stage the replacement of bridge span accelerometers is not scheduled.

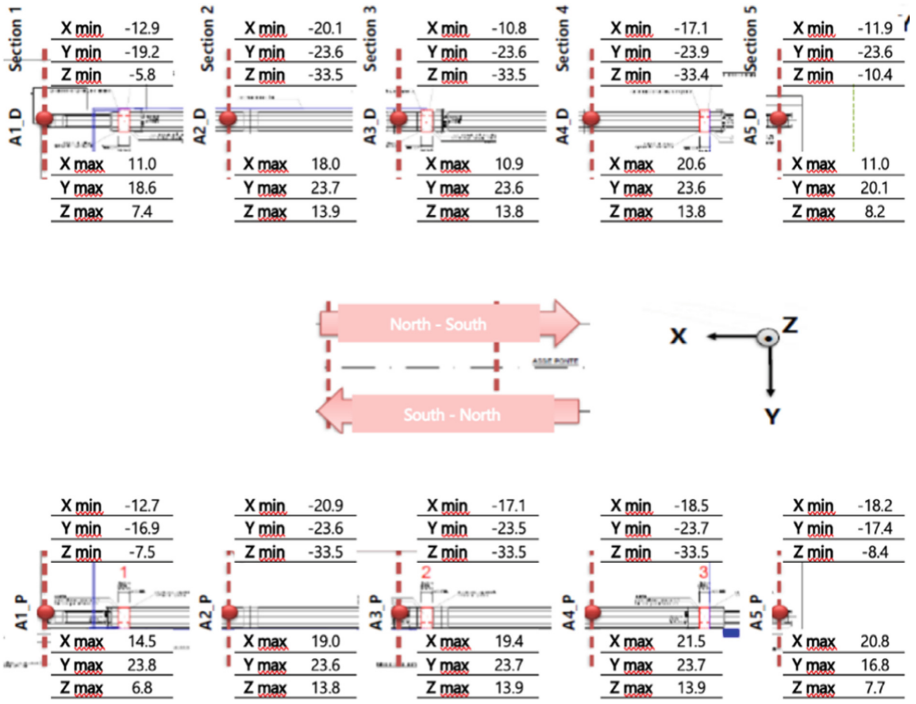


Fig. 4. Synoptic view of the peak acceleration statistics (m/s^2)

3.2 Analysis of Weekly Average Accelerations

In order to characterize the average maximum (in absolute value) stress, min/max acceleration suffered by the structure for each train passage were recorded and averaged on the whole period W1-W3. It appears that, in the average, highest absolute values (up to approximately 13 m/s^2) are related to the Y component, transversal to tracks, followed by those related to Z component. Statistics in (Table 4 in Table 2, and Fig. 7 in Fig. 6) summarize the analysis of min/max accelerations performed on the dataset over the W1-W3 period. This has been developed to characterize the accelerometer belonging to the monitoring system using 60 values (2 for each accelerometer, for each direction), namely mean and standard deviation of min and max values for all components. The six values (X min/max, Y min/max, Z min/max) which represent the maximum average accelerations to which the structure is subjected to, along the three directions are also highlighted.

Table 1. Overall peak acceleration (m/s^2) statistics on the whole period (W1-W3).

		Average	St.dev.	Peak value			Average	St.dev.	Peak value
A1-D	**X min**	-7.3	1.6	-12.9	**A1-P**	**X min**	-9.6	1.5	-12.7
	Y min	-11.0	2.3	-19.2		**Y min**	-11.3	2.6	-16.9
	Z min	-3.3	0.9	-5.8		**Z min**	-4.8	1.1	-7.5
	X max	7.0	1.3	11.0		**X max**	8.7	2.2	14.5
	Y max	11.6	2.3	18.6		**Y max**	16.6	3.4	23.8
	Z max	4.1	1.1	7.4		**Z max**	4.4	0.9	6.8
A2-D	**X min**	-15.0	2.5	-20.1	**A2-P**	**X min**	-16.1	2.5	**-20.9**
	Y min	-23.6	0.0	-23.6		**Y min**	-23.6	0.0	-23.6
	Z min	-33.5	0.1	**-33.5**		**Z min**	-33.5	0.0	-33.5
	X max	13.1	2.4	18.0		**X max**	14.6	2.2	19.0
	Y max	23.7	0.0	23.7		**Y max**	23.6	0.0	23.6
	Z max	13.8	0.0	13.9		**Z max**	13.8	0.0	13.8
A3-D	**X min**	-7.8	1.7	-10.8	**A3-P**	**X min**	-13.3	2.1	-17.1
	Y min	-23.6	0.1	-23.6		**Y min**	-23.5	0.0	-23.5
	Z min	-31.1	2.4	-33.5		**Z min**	-33.5	0.0	-33.5
	X max	7.7	1.8	10.9		**X max**	13.0	2.3	19.4
	Y max	23.6	0.0	23.6		**Y max**	23.7	0.0	**23.7**
	Z max	13.8	0.0	13.8		**Z max**	13.9	0.0	13.9
A4-D	**X min**	-13.1	2.1	-17.1	**A4-P**	**X min**	-13.5	2.3	-18.5
	Y min	-23.7	0.1	**-23.9**		**Y min**	-23.6	0.0	-23.7
	Z min	-33.0	0.7	-33.4		**Z min**	-30.9	2.0	-33.5
	X max	14.9	2.6	20.6		**X max**	16.2	2.6	**21.5**
	Y max	23.5	0.0	23.6		**Y max**	23.7	0.0	23.7
	Z max	13.8	0.0	13.8		**Z max**	13.9	0.0	**13.9**
A5-D	**X min**	-8.5	1.5	-11.9	**A5-P**	**X min**	-10.6	2.9	-18.2
	Y min	-18.4	3.4	-23.6		**Y min**	-11.7	2.6	-17.4
	Z min	-4.9	1.5	-10.4		**Z min**	-4.4	1.5	-8.4
	X max	7.2	1.4	11.0		**X max**	10.9	3.3	20.8
	Y max	12.8	2.6	20.1		**Y max**	11.9	2.3	16.8
	Z max	5.4	1.2	8.2		**Z max**	4.4	1.1	7.7

Table 2. Average values of the minimum and maximum accelerations (m/s^2) - period W1-W3.

	W1-W3									
	A1-D	**A2-D**	**A3-D**	**A4-D**	**A5-D**	**A1-P**	**A2-P**	**A3-P**	**A4-P**	**A5-P**
Min	-1.8	-3.4	-1.9	-3.1	-2.3	-2.6	**-3.5**	-2.9	-3.0	-2.3
(Average)	-2.6	**-12.8**	-8.6	-11.0	-3.8	-2.5	-11.6	-12.1	-8.9	-2.6
X,Y,Z	-0.7	-11.2	-8.3	-9.9	-1.1	-1.1	-10.9	**-11.2**	-8.9	-0.9
Min	1.3	2.7	1.5	2.5	1.6	1.8	2.9	2.4	2.5	2.0
(St.dev.)	2.2	7.8	6.9	7.8	3.6	2.2	7.5	7.8	6.7	2.2
X,Y,Z	0.6	8.4	6.7	7.9	0.9	0.9	8.3	8.7	6.5	0.8
Max	1.5	3.2	1.8	3.2	1.7	1.9	3.3	2.9	**3.3**	2.4
(Average)	2.7	**12.8**	9.5	11.4	2.9	3.6	11.1	11.8	8.7	2.7
X,Y,Z	0.8	8.5	7.0	7.8	1.2	1.0	8.5	**8.6**	7.7	0.9
Max	1.3	2.5	1.5	2.8	1.5	1.7	2.6	2.4	3.0	2.0
(St.dev.)	2.2	7.9	7.2	7.7	2.6	3.2	7.7	7.9	6.5	2.3
X,Y,Z	0.8	4.4	4.3	4.4	1.0	0.9	4.4	4.6	4.3	0.8

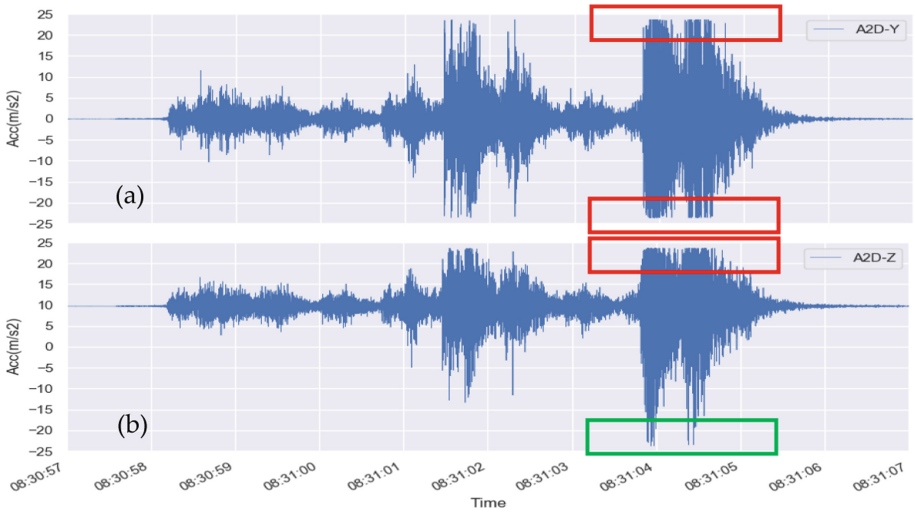
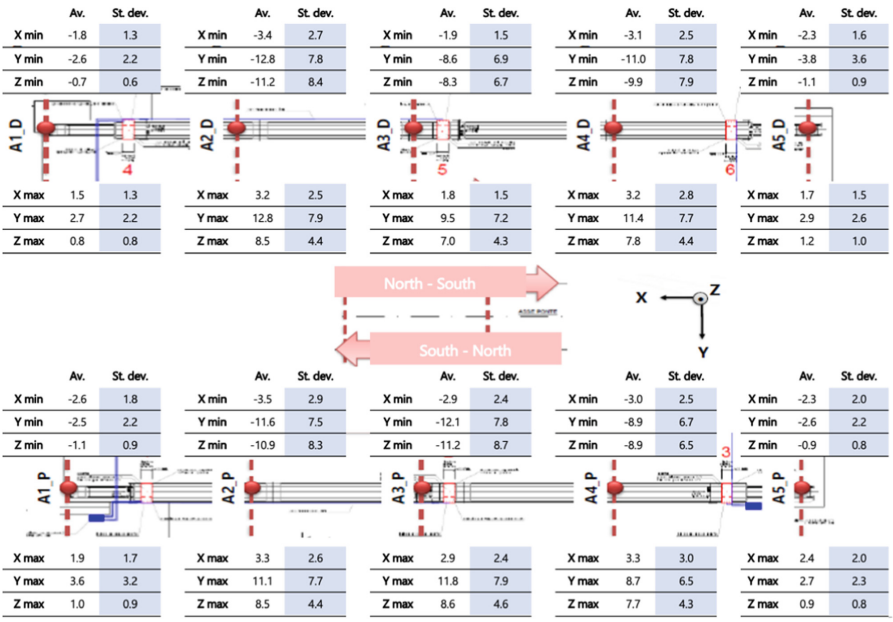


Fig. 5. Example of accelerograms related to the Y (a) and Z (b) axes. (Color figure online)



	Av.	St. dev.
X min	-1.8	1.3
Y min	-2.6	2.2
Z min	-0.7	0.6
X max	1.5	1.3
Y max	2.7	2.2
Z max	0.8	0.8

	Av.	St. dev.
X min	-3.4	2.7
Y min	-12.8	7.8
Z min	-11.2	8.4
X max	3.2	2.5
Y max	12.8	7.9
Z max	8.5	4.4

	Av.	St. dev.
X min	-1.9	1.5
Y min	-8.6	6.9
Z min	-8.3	6.7
X max	1.8	1.5
Y max	9.5	7.2
Z max	7.0	4.3

	Av.	St. dev.
X min	-3.1	2.5
Y min	-11.0	7.8
Z min	-9.9	7.9
X max	3.2	2.8
Y max	11.4	7.7
Z max	7.8	4.4

	Av.	St. dev.
X min	-2.3	1.6
Y min	-3.8	3.6
Z min	-1.1	0.9
X max	1.7	1.5
Y max	2.9	2.6
Z max	1.2	1.0

	Av.	St. dev.
X min	-2.6	1.8
Y min	-2.5	2.2
Z min	-1.1	0.9
X max	1.9	1.7
Y max	3.6	3.2
Z max	1.0	0.9

	Av.	St. dev.
X min	-3.5	2.9
Y min	-11.6	7.5
Z min	-10.9	8.3
X max	3.3	2.6
Y max	11.1	7.7
Z max	8.5	4.4

	Av.	St. dev.
X min	-2.9	2.4
Y min	-12.1	7.8
Z min	-11.2	8.7
X max	2.9	2.4
Y max	11.8	7.9
Z max	8.6	4.6

	Av.	St. dev.
X min	-3.0	2.5
Y min	-8.9	6.7
Z min	-8.9	6.5
X max	3.3	3.0
Y max	8.7	6.5
Z max	7.7	4.3

	Av.	St. dev.
X min	-2.3	2.0
Y min	-2.6	2.2
Z min	-0.9	0.8
X max	2.4	2.0
Y max	2.7	2.3
Z max	0.9	0.8

Fig. 6. Average values of the min. and max. accelerations (m/s^2) and relative standard deviations associated with each accelerometer

4 Data Analysis – Vibration Normal Modes

A second analysis has been developed on acceleration signals, with the aim of assessing vibration normal modes (natural frequencies) by using standard FFT.

4.1 Frequency Dynamic Analysis

The dynamic analysis has been carried out to provide further indications on the structural behaviour of the Portella bridge, compared to those obtainable with purely static analyses, through the experimental determination of the vibration normal modes of the structure.

4.1.1 Analysis Mode

The structure vibration natural frequencies were obtained starting from the analysis of the accelerograms relating to each individual event (train crossing), processed using the FFT. The analysis of the accelerometric data for the calculation of natural frequencies is carried out on the final part of the signal, when the structure displays free oscillations after the passage of the train which constitutes the external forcing triggering the structure vibrations. Therefore, it is possible to identify the range of natural frequencies by processing the accelerogram tail. The workflow implemented (Fig. 7) therefore provides the acquisition of accelerations values, the analysis of the signal tail, highlighted in yellow, and the extraction of the relevant frequency data.

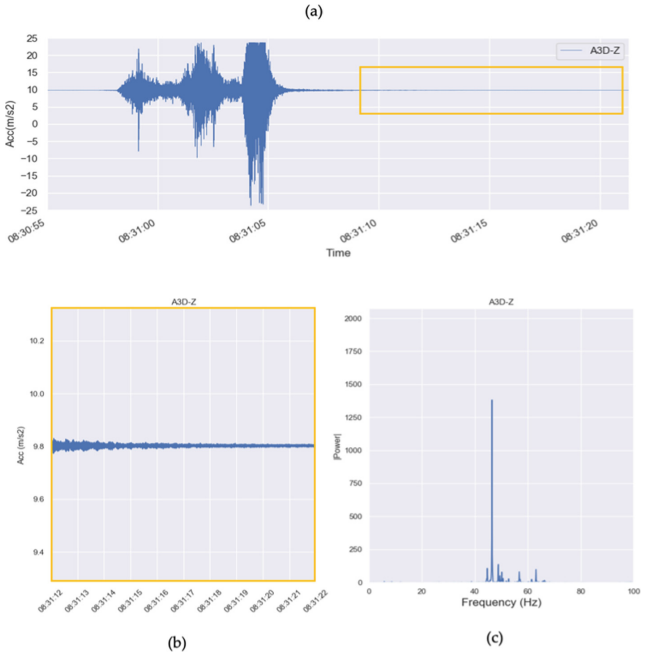


Fig. 7. Extraction of vibration frequencies, applied to the A3D-Z accelerogram

4.1.2 Frequency Calculation

Frequency analyses have been conducted on reference events, characterized by peak values in accelerations, and the peak frequency values related to each train passage investigated have been processed over the weeks W1-W3 (excluding the days 01.05.2019 and 31.07.2019, not available at the time of processing). Tables 3, 4 and 5 summarize, for the accelerometers located on the bridge span, the mean, median and standard deviation values of frequency obtained, coupled according to the transversal direction. In the cumulative analysis, the pairs of accelerometers A1D/P and A5D/P, placed on the shoulders of the bridge, were not considered as characterized, on average, by low accelerations and frequencies with not significant powers, as shown in the example in Fig. 8. The results highlight that:

- frequency distributions are almost equal for the corresponding transversal accelerometer pairs (A2D-A2P, A3D-A3P, A4D-A4P), so that the bridge behaviour appears homogeneous in transversal direction
- frequency distributions are very similar for corresponding (with respect to the bridge centerline) longitudinal accelerometer pairs (A2D-A4D, A2P-A4P), so that the bridge behaviour appears homogeneous in longitudinal direction
- frequency medians are higher for X and Z components (slightly less than 50 Hz, higher for A2D-P and A4D-P accelerometers) that for Y component (about 35 Hz for A2D-A2P and A4D-A4P accelerometers, and about 45 Hz for A3D-A3P and A4D-P)
- frequency standard deviations are within 15–20 Hz in all cases

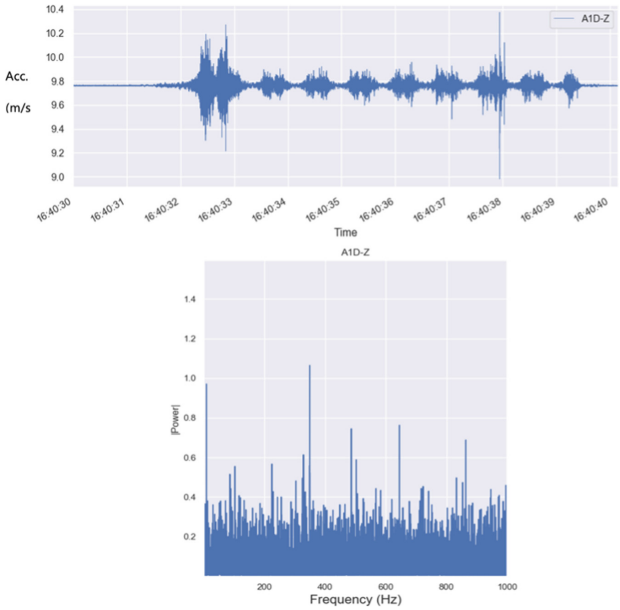


Fig. 8. Example of frequencies calculated for an accelerometer placed on a bridge shoulder

Table 3. Mean, median, and standard deviation of the natural frequency, computed on the X, Y and Z directions, related to the A2D and A2P accelerometers W3

	A2D-X	A2P-X	A2D-Y	A2P-Y	A2D-Z	A2P-Z
	Frequency (Hz)					
Mean	51,5	55,4	39,7	35,8	48,7	49,5
Median	48,5	49,2	35,0	35,0	48,7	48,9
St.dev	17,1	14,6	18,5	15,6	18,2	19,4

Table 4. Mean, median, and standard deviation of the natural frequency, computed on the X, Y and Z directions, related to the A3D and A3P accelerometers W3

	A3D-X	A3P-X	A3D-Y	A3P-Y	A3D-Z	A3P-Z
	Frequency (Hz)					
Mean	41,4	50,2	45,4	44,5	45,3	45,6
Median	49,5	49,8	45,1	43,3	46,3	46,2
St.dev	24,0	19,7	17,7	18,0	19,1	18,0

Table 5. Mean, median, and standard deviation of the natural frequency, computed on the X, Y and Z directions, related to the A4D and A4P accelerometers W3

	A4D-X	A4P-X	A4D-Y	A4P-Y	A4D-Z	A4P-Z
	Frequency (Hz)					
Mean	52,7	54,4	37,0	39,4	48,7	48,6
Median	50,0	53,4	35,0	35,0	49,7	48,9
St.dev	16,6	16,7	17,0	20,4	17,5	20,9

5 Discussion

The work describes the preliminary results obtained from the analysis of the measurements acquired by the monitoring system installed at the Portella metal railway bridge.

A comprehensive study has been performed on the data acquired by MEMS sensors, aimed at identifying the stress ranges, in terms of accelerations along the X, Y and Z axes, affecting the structure following the passage of the trains. The data analysis, conducted in three periods W1, W2 and W3, characterized by different environmental parameters, did not highlight any type of variations in the behaviour of the structure; the results obtained can therefore be considered indicative under conditions like those encountered during the events considered. It is stressed again that the events analysed are characterized by the highest acceleration values. However, as already highlighted in 3.1, the span peak values can only be inferred indirectly from the accelerometric data acquired along the Z axis, due to the limited measurement range of the instrumentation.

The accelerometric sensors also made it possible to derive the (supposed) natural frequencies, ranging in the interval 35–50 + 20 Hz for all components. The natural frequencies analysis highlighted a homogeneous transversal and longitudinal symmetric (with respect to the transversal centerline) behaviour of the bridge. Results have been compared with the outputs obtained from the Finite Element (FE) modelling of the bridge used in the design phase. The model issued natural frequencies ranging from 5 to about 10 Hz, namely one order of magnitude smaller than the experimental frequency. The reasons of these discrepancies are currently under examinations; a first hypothesis indicates the underestimation of the action of some structural constraints as a possible cause. However, a malfunctioning of the shield against electric power cannot be completely excluded. Following the findings of the investigation, an updated FE model will be developed in order to better match the results of the FE analyses and the measurement of the real behaviour of the structure. As evidenced in [18], acquired experimental dataset deliver comprehensive information regarding the behaviour of the bridge, useful for improving the initial FE model characteristics and better defining its boundary conditions.

The acquired data will allow to constantly monitor the structural integrity of the bridge and to promptly intervene if the parameters measured in real time should deviate from the typical parameters obtained from the analyses carried out, addressing the bridge maintenance process, which has a large impact on the whole life cycle cost.

6 Expected Developments

The monitoring system has provided data useful for the dynamic characterization of the investigated structure, or in the worst case, highlighted a bug in the insulation of the equipment from electric currents. Nevertheless, the system has been designed for harnessing the integration of different type of sensors and acquiring a huge amount of data.

In light of this, expected developments will encompass the full exploitation of the measurements acquired by MEMS accelerometers, by systematically processing the whole available database, in this way increasing the statistical base of analysed data and allowing a more accurate description of the structure, both in terms of vibrating natural frequencies and accelerations. The replacement (or the coupling) of the span accelerometers with different sensor units characterized by a wider measuring range could be necessary to directly measure the peak accelerations in the Y and Z directions, along which, as expected, the structure shows the greatest accelerations. Modal analyses from peak frequencies are being implemented, also taking into account the results obtained in [19], where the accuracy of MEMS accelerometers for these purposes has been experimentally assessed providing evidence of maximum expected frequency errors up to 5%. A comparison with the results obtained from an analogous monitoring system installed on concrete steel bridge is also planned.

Further improvements in the definition of the Portella bridge behaviour are expected from the evaluation of short and long-term displacements (or possible deformations) of the structure, with the availability of the monitoring system module based on GNSS sensors, as widely discussed in [20, 21]. The direct monitoring of displacements will provide the opportunity of using the original acceleration signal to retrieve velocity and displacements, starting from the experience conducted in [22], and the original approaches described in [23].

Finally, many monitoring systems have been installed on structures all over the world. Since is not easy to find detailed reports containing the achieved results, authors duty will be the reporting back on the operation, use, and success (or failure) of installed systems in future years, as intended in [24]. The already acquired acceleration big data will allow to establish a data driven model of the Portella bridge behaviour in standard conditions, both regarding maximum accelerations and natural frequencies. This model will enable to highlight eventual changes in the response to the stresses induced by railway traffic, therefore evidencing possible changes in the structural state and damages occurred.

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References

1. Bàlazs, P., et al.: On the first year of implementation of the core network corridors. The Trans-European Transport Network (2015)

2. Morgese, M., Ansari, F., Domaneschi, M., Cimellaro, G.P.: Post-collapse analysis of Morandi's Polcevera viaduct in Genoa Italy. J. Civ. Struct. Heal. Monit. **10**(1), 69–85 (2019). https://doi.org/10.1007/s13349-019-00370-7
3. Salem, H.M., Helmy, H.M.: Numerical investigation of collapse of the Minnesota I-35W bridge. Eng. Struct. **59** (2014). https://doi.org/10.1016/j.engstruct.2013.11.022
4. Cruz, P., León, A.D. De, Wisniewski, D., Valente, I.: Sustainable Bridges Assessment for Future Traffic Demands and Longer Lives (2007)
5. Pipinato, A.: Innovative bridge design handbook: Construction, rehabilitation and maintenance (2015)
6. Moses, F., Schilling, C.G., Raju, K.S.: Fatigue evaluation procedures for steel bridges (1987)
7. Crespi, M., Giannone, F., Marsella, M., Sonnessa, A.: Automated geomatic system for monitoring historical buildings during tunneling in Roma, Italy. In: Life-Cycle and Sustainability of Civil Infrastructure Systems - Proceedings of the 3rd International Symposium on Life-Cycle Civil Engineering, IALCCE 2012 (2012)
8. Baiocchi, V., et al.: First geomatic restitution of the sinkhole known as 'Pozzo del Merro' (Italy), with the integration and comparison of 'classic' and innovative geomatic techniques. Environ. Earth Sci. **77**(3), 1–14 (2018). https://doi.org/10.1007/s12665-018-7244-6
9. Yu, J., Fang, Z., Meng, X., Xie, Y., Fan, Q.: Measurement of quasi-static and dynamic displacements of footbridges using the composite instrument of a smartstation and an accelerometer: case studies. Remote Sens. **12** (2020). https://doi.org/10.3390/RS12162635
10. Bedon, C., Bergamo, E., Izzi, M., Noè, S.: Prototyping and validation of MEMS accelerometers for structural health monitoring—the case study of the Pietratagliata cable-stayed bridge. J. Sens. Actuator Networks. **7** (2018). https://doi.org/10.3390/jsan7030030
11. Grimmelsman, K.A., Zolghadri, N.: Experimental evaluation of low-cost accelerometers for dynamic characterization of bridges. In: Pakzad, S. (ed.) Dynamics of Civil Structures, Volume 2. CPSEMS, pp. 145–152. Springer, Cham (2020). https://doi.org/10.1007/978-3-030-12115-0_19
12. Baraccani, S., Palermo, M., Azzara, R.M., Gasparini, G., Silvestri, S., Trombetti, T.: Structural interpretation of data from static and dynamic structural health monitoring of monumental buildings. In: Key Engineering Materials (2017). https://doi.org/10.4028/www.scientific.net/KEM.747.431
13. Zheng, H., Shi, G., Zeng, T., Li, B.: Wireless earthquake alarm design based on MEMS accelerometer. In: 2011 International Conference on Consumer Electronics, Communications and Networks, CECNet 2011 - Proceedings (2011). https://doi.org/10.1109/CECNET.2011.5768502
14. Domaneschi, M., Limongelli, M.P., Martinelli, L.: Interpolation damage detection method on a suspension bridge model: Influence of sensors disturbances. Key Eng. Mater. 569–570 (2013). https://doi.org/10.4028/www.scientific.net/KEM.569-570.734
15. Ali, A., Sandhu, T., Usman, M.: Ambient Vibration Testing of a Pedestrian Bridge Using Low-Cost Accelerometers for SHM Applications. Smart Cities. **2** (2019). https://doi.org/10.3390/smartcities2010002
16. Picozzi, M., et al.: Wireless technologies for the monitoring of strategic civil infrastructures: An ambient vibration test on the Fatih Sultan Mehmet Suspension Bridge in Istanbul, Turkey. Bull. Earthq. Eng. **8** (2010). https://doi.org/10.1007/s10518-009-9132-7
17. Rete Ferroviaria Italiana S.p.A.: Ponte Portella km 102+092 - rev. C. (2018)
18. Vagnoli, M., Remenyte-Prescott, R., Andrews, J.: Railway bridge structural health monitoring and fault detection: State-of-the-art methods and future challenges (2018). https://doi.org/10.1177/1475921717721137
19. Kok, R., Furlong, C., Pryputniewicz, R.J.: Development of a wireless MEMS inertial system for health monitoring of structures. In: Materials Research Society Symposium - Proceedings (2003). https://doi.org/10.1557/proc-785-d11.5

20. Yu, J., Meng, X., Yan, B., Xu, B., Fan, Q., Xie, Y.: Global Navigation Satellite System-based positioning technology for structural health monitoring: a review (2020). https://doi.org/10.1002/stc.2467
21. Im, S.B., Hurlebaus, S., Kang, Y.J.: Summary review of GPS technology for structural health monitoring. J. Struct. Eng. **139** (2013). https://doi.org/10.1061/(asce)st.1943-541x.0000475
22. Faulkner, K., Brownjohn, J.M.W., Wang, Y., Huseynov, F.: Tracking bridge tilt behaviour using sensor fusion techniques. J. Civ. Struct. Heal. Monit. **10**(4), 543–555 (2020). https://doi.org/10.1007/s13349-020-00400-9
23. Colosimo, G., Crespi, M., Mazzoni, A.: Real-time GPS seismology with a stand-alone receiver: a preliminary feasibility demonstration. J. Geophys. Res. Solid Earth. **116** (2011). https://doi.org/10.1029/2010JB007941
24. Webb, G.T., Vardanega, P.J., Middleton, C.R.: Categories of SHM deployments: Technologies and capabilities. J. Bridg. Eng. **20** (2015). https://doi.org/10.1061/(ASCE)BE.1943-5592.0000735