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Rome's Line C underground—dynamic monitoring of the Basilica di Massenzio

S. Moretti
IMG S.r.l., Rome, Italy

G. Pezzetti
Smak s.a.s., Bergamo, Italy

E. Romani & I. Mammone
Metro C S.C.p.A., Rome, Italy

ABSTRACT: Basilica di Massenzio is one of the most important monuments interested by the construction of the T3 stretch of the C Line Rome Underground. It is a masonry construction founded on a silty-clay and sandy soil. The Fori Imperiali station and the twin tunnels of the T3 stretch are to be built close to the Basilica which has been heavily instrumented to verify its behavior during construction activities. The paper describes the dynamic monitoring of the Basilica highlighting the process that has been used to define modal parameters, amplification factors and transfer function to assess compliance with current regulations and to define and verify intervention methods to increase the safety factor of the monument with respect to undergoing works and seismic actions.

1 INTRODUCTION

Basilica di Massenzio is situated along Via dei Fori Imperiali, in an intermediate position between the stations of Fori Imperiali and Venezia. In the examined area, the route of Line C is straight and external, sub-parallel to the longitudinal axis of the structure. The level of the Basilica is 27 m above sea level whereas that of Via dei Fori Imperiali, under which runs the gallery nearest to the monument, is between 22.5 and 23.5 m above sea level. The difference in level between the Basilica and Via dei Fori Imperiali is supported by the ancient Roman wall which bears the Velian hill, excavated for the opening of Via dei Fori Imperiali (Figure 1). The foundations are of a continuous type with a variable width of 4.0 to 6.0 and a level variable between 23.0 above sea level—Colosseum side —and 13 above sea level—Carinae side. The Line C galleries are sub-parallel to the facade of the Basilica, with a distance of 16 m between the axis of the gallery nearest to the monument and the monument itself in correspondence to the Colosseum side and of 35 m in correspondence to the Carinae side.

The axis of the galleries is situated at an average depth of 25.5 m from the level of Via dei Fori Imperiali. The Basilica di Massenzio is a particularly heavy structure, with significant contact stress transferred to the soil by the foundations that consist of simple extension of the bearing walls, with a very limited widening.

Figure 1. An internal and an external view of The Basilica di Massenzio now.

2 PRELIMINARY STUDIES

The Basilica had a rectangular plan of dimensions 100 x 65 m with an East-West oriented longitudinal axis which originally corresponded to the main axis. All that remains of the three naves is the northern one, which runs along via dei Fori Imperiali.

Preliminary studies have also examined the structural failures and the subsequent historical reinforcements, the geometrical survey, the structural and crack pattern survey, the material characteristics and the construction technologies with test investigations.

An important phase of the preliminary studies was the geotechnical investigation, aiming at defining the ground conditions. The geotechnical characterization of the foundation soils of the Basilica has been made using the results of site and laboratory tests carried out during several geotechnical investigation campaigns. The tunnels near the Basilica di Massenzio are mainly contained into medium and fine grained soils of the Paleotevere.

Both the preliminary analyses and the study of the wide range of documents found have pointed out a situation of widespread structural suffering. The fact that two thirds of the original monument have collapsed is a clue to structural deficiencies. The reasons of the collapse can also be reasonably connected to a density of vertical elements as compared to the surface which is significantly lower than that of other monuments of similar importance. The ratio between the resistant section and the area of the whole Basilica di Massenzio is about 12%, to be compared to 23% of the Pantheon and 26% of the Basilica of San Peter.

The study of the interaction between the line galleries and the monument was carried out through the level 2 numerical analyses performed in bi-dimensional and tridimensional conditions (Figure 2). The starting model was created with Plaxis® 2D-3D finite element code for soil and rock analysis. The tridimensional structure-soil interaction analyses provide results consistent with those of the bi-dimensional analyses. For the central transversal wall – Carinae side – the subsidence value induced by the passage of the TBMs with a volume loss ratio of 0.5% is negligible, inferior to 2.0 mm. Instead, higher values were obtained for the central transversal wall – Colosseum side – with a maximum displacement of 3.5 mm and for the transversal wall – Colosseum side – with a maximum displacement of 4.8 mm. The variation of the stress in the vaults from the present condition to that after the intervention of the subsidence is almost unperceivable.

The strain of the apse is altered by the settlement imposed. The most relevant perturbation is concentrated in the dome, near the facing Via dei Fori Imperiali and on the Colosseum side, with tensile stresses of 3 MPa. The evolution of the tensional state in the longitudinal wall with triple-lancet windows shows a small perturbation with an increase in tractions, generally already present under its own weight. The deformation analysis provides a reliable picture of the perturbation induced by the settlement imposed in the structure. Under its own weight the maximum strain value amounts to about 3E-4, neglecting very localized and little representative effects. With the imposed subsidence we get to a maximum deformation value of the same order of magnitude, so we can conclude that the disturbance caused is negligible under design hypotheses (Figure 3).

Before the construction of the tunnels, according to Burland's classification, the structure is already in a state of slight damage, although the works do not further worsen the damage category.

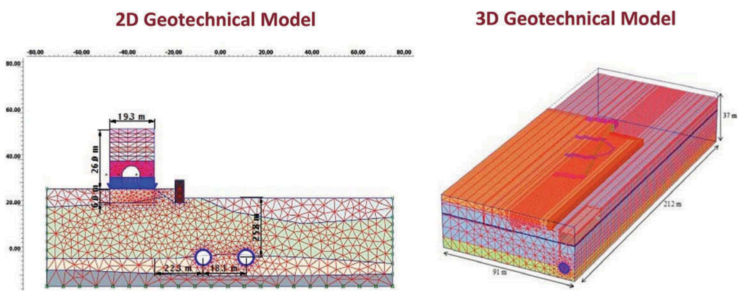


Figure 2. 2D and 3D Geotechnical Models for Monument-Soil interaction.

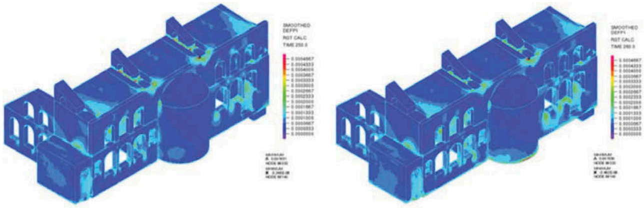


Figure 3. The result of the 3D structural models.

One of the most interesting aspects of the non-linear analysis is the numerical simulation of the cracking development. First of all, we can see a good correspondence between the crack localization in the model and that presently shown in the Basilica. The analyses suggest the opportunity to make localized definitive interventions for the structural safeguard aiming at the reduction of the barrel vault's thrusts.

The design of the structural reinforcements consists of the restoration of the brick facings with the use of fibers and structural injections and a post tensioning system with longitudinal and transversal Dywidag bars (Ø40) to reinforce the structure.

3 STRUCTURAL HEALTH MONITORING: DYNAMIC MEASUREMENTS

3.1 *Preliminary definition of the dynamic behavior of the monument*

In the aim of the dynamic characterization of the structure, reference was made mainly to the values of elastic modulus estimated starting from the ultrasound propagation velocity measurements. In fact, the expected deformation level for the underground induced vibrations is very low and close to the laboratory results tests. Indeed they often show an increase in ultrasound velocity going from top to bottom. The highest velocities were found at the pillars of the Carinae vault, with values often exceeding 2500 m/s, which indicates materials with good mechanical characteristics.

The elastic modulus (E) is obtained from the following equation (1):

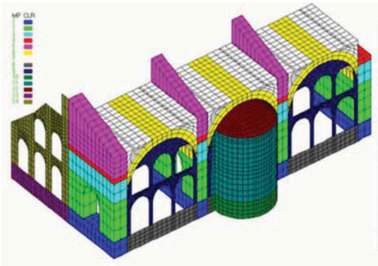
$$c_l = \sqrt{\frac{E(1-\upsilon)}{\rho(1+\nu)(1-2\upsilon)}} \tag{1}$$

where c_l = the ultrasound longitudinal velocity; E = elastic modulus; ν = Poisson coefficient and ρ = density of the mean.

Assuming for the masonry a Poisson coefficient of 0.15 and a density of 1.73 KN/m3/g, it has been obtained a mean value of the elastic modulus of 6555 MPa.A dynamic modal analysis has been performed for the first evaluation of the dynamic characteristics of the monument., The investigated main vibrational modes have been up to 20–30 Hz of frequency referred to the numerical model of the structure with the mechanical properties based on the results of the investigations carried out during studies on the effects of subsidence. Subsequently the finite element model has been updated (updating phase) on the experimental dynamic measures.

The finite element model was developed with solid elements at 20 knots and with shell elements at 4 and 3 knots, reproducing all the main constructive elements of the monument and the materials used. The model measures 88 m in a sub-parallel direction to Via dei Fori Imperiali, while in the perpendicular direction, 45 m on the Colosseum side and 20 m on the Carinae side, for a height of about 26 m from the current ground floor level.

The model has about 20,500 knots and 5241 elements, considering both the 20 knots and the various bricks shell types, for a total number of equations of about 64000. At the base of the model, at altitudes = -5.00 m, constraints have been inserted that block all degrees of freedom. Figure 4 shows an overview of the monument with the identification of the different mechanical properties of the various materials implemented in the numerical model



MP	E (kN/m^2)	υ	ρ ($tons/m^3$)	colore
1	2700000	0.15	2.2	blue
2	2700000	0.15	1.75	green
3	2250000	0.15	1.6	cyan
4	2250000	0.15	1.5	red
5	1800000	0.15	1.35	magenta
6	1800000	0.15	1.35	yellow
7	1800000	0.15	1.35	white
8	2700000	0.15	2.2	dark gray
9	2700000	0.15	1.75	dark blue
10	2700000	0.15	2.2	dark green
11	2700000	0.15	1.75	dark cyan
12	27000000	0.15	2.5	dark red
13	2700000	0.15	-	dark magenta
14	2700000	0.15	-	dark yellow

Figure 4. Base finite elements model of the basilica with identification of the different mechanical properties assigned (Benedettini et al. 2009).

(Benedettini et al. 2009). In Table 1 and Figure 5 the natural frequencies (periods) and first 3 vibrational modes of the structure are reported.

To validate the base finite element model, dynamic measurements has been used under the effect of environmental excitation. Subsequently the modal parameters obtained (fundamental frequencies) has been used to update the elastic modules of various zones of the model, in order to minimize the differences between measured quantities and those obtained by the model.

With regard to the excitation modalities best suited to the dynamic characterization of the monument, the electrodynamics exciters (such as the vibrodyne) may involve interferences with the normal use of the monument or directly trigger non-linear behaviors; otherwise, the vibrations of environmental origin, except for very particular cases, make the linear elastic behavior assumption easier. Obviously the study of environmental vibrations, which are much smaller than the forced ones, requires very sensitive measuring chains, able to capture the significant dynamics inside in the background noise. In this case, experimental layouts based on accelerometers and velocimeters were used (Benedettini et al. 2009).

The dynamic characteristics have been determined with the use of consolidated Operational Modal Analysis algorithms, implemented both in commercial calculation codes and in Matlab applications. In particular, Enhanced Frequency Domain Decomposition (EFDD), a technique that works in the frequency domain, based on the decomposition of the singular value of the spectral density matrix has been used; in this way the answer is decoupled in a series of responses of elementary oscillators, each representing the proper modes of the structure (Brinker, 2007).

EFDD technique, includes damping estimation, improved natural frequency estimation, and improved mode shape estimation. After the experimental determination of the dynamic

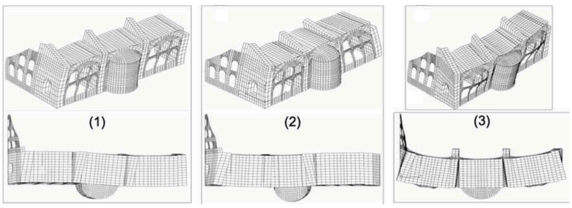


Figure 5. First 3 vibrational modes of the structure.

Table 1. Natural frequencies (periods) of first 3 vibrational modes of the structure

Frequency Number	Frequency Cycles/Sec	Period Seconds
1	1.231	0.812
2	1.834	0.545
3	2.177	0.459

properties of the monument, the fundamental frequencies and the modal forms of the structure were compared with those provided by the a priori model, in order to verify the existence of a reasonable similarity, to be taken as a starting point for the updating of the model itself. The choice of the parameters to be changed was made on the basis of engineering assessments and knowledge of the structural behavior of the building. Tables 2 and 3 summarizes the initial and identified values of the various parameters. The reduction of the elastic modulus between the a priori model and the identified model (Table 2) suggests that the transverse walls and vaults are more damaged than the other structural elements. This circumstance is confirmed by the observation of the presence on the monument of a significant and well documented crack pattern.

The comparison of the values reported in Table 3 between numerical frequencies and identified frequencies shows a good correspondence. Figure 6 shows, instead, the comparison between numerical and measured modal shapes, which correspondence appears much better.

3.2 *Dynamic measurement for post tensioning system control*

During the design phase, all necessary precautions have been used for the forecasting of the influence that, both during construction and in the future exercise phase, will occur on sensitive receptors due to the realization of the C line. After providing for the implementation of an

Table 2. Parameters of the finite element model: initial and identified values (elastic modules, MPa)

Parameter	E Initial value	E Identified value
Walls with windows	2700	4410
Vaults	1800	1400
Transversal walls, foundations	2700/1800	2050/1370

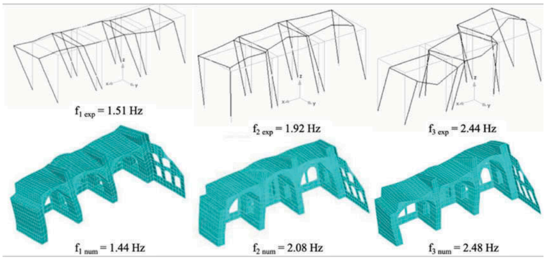


Figure 6. First 3 vibrational modes of the structure.

Table 3. Experimental fundamental frequencies (in brackets) and identified, relative errors [in brackets], experimental modal damping

Mode	f (Hz)	ζ
1	(1.51) 1.44 [-4.6%]	1.79%
2	(1.92) 2.08 [8.3%]	2.27%
3	(2.44) 2.48 [1,6%]	1.68%

extremely accurate static monitoring system and after having completed the preliminary study of the dynamic response of the structure, a dynamic monitoring system was installed in parallel. The final design of the dynamic monitoring system involved the installation of 7 accelerometers, arranged on the structure according to the diagram shown in the following Figure 7.

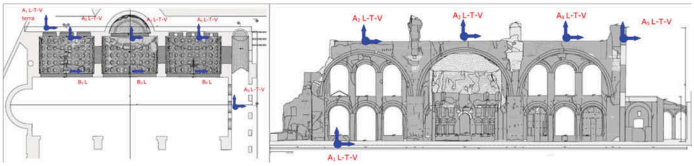


Figure 7. Schematic representation of the accelerometric sensor installed on the monument.

Table 4. Experimental fundamental frequencies, relative errors and experimental modal damping for each measurement campaigns

	June 2016		July 2017		January 2018	
Mode	f (Hz)	ζ(%)	f (Hz)	ζ(%)	f (Hz)	ζ(%)
1	1,565	1,22	1,547	1,70	1,548	1,55
2	1,940	2,23	1,985	2,29	1,970	1,06
3	2,512	1,71	2,502	2,09	2,477	1,77

The dynamic monitoring system has been designed for the collection, processing and archiving of continuously measured vibrational variables, as well as for the generation and real-time data communication for the evaluation of the impact of ongoing activities. In the application of a real time monitoring, the measured data will be processed by the acquisition and storage units in order to check for any exceeding of the thresholds indicated by the sector standards in reference to the structural damage (UNI 9916, DIN 4150/3). In particular, these standards provide for differentiated thresholds according to the structure type, the foundation, the ground and the type of stress.

In the case of Basilica di Massenzio, which can certainly be classified as 'Historical Building', the permissible velocity values are included, for short-term vibrations, between 3 mm/s for frequencies below 10 Hz, and 10 mm/s for frequencies above 100 Hz. In the case of long-lasting vibrations, the standards provide a limit value for the measured velocity of 2.5 mm/s.

For the vertical component of the vibrations the reference value for the vibrational velocity is 20 mm/s limited. This value is independent of the frequency content of the recording and may be lower for the third class. A further application of the monitoring system provides for the execution of periodic measures aimed to verifying the stability of the modal parameters of the structure during all construction phases as for example the phases of the tensioning system setting up.

In this regard, after the execution of the tensioning of the reinforcing bars, 3 dynamic measuring campaigns were carried out, according to the technique of operational modal analysis, to verify the stability of the modal parameters. The results of the measurements performed (June 2016, July 2017, January 2018) are shown in the following Tables 4 and 5. These results show an increase of natural frequencies for the first vibration modes, which highlight a global structure stiffening as a consequence of the consolidation interventions. Furthermore, an excellent correspondence between the modal forms calculated during the 3 measurement campaigns and those ones obtained during the preliminary investigation phase is noted. In addition to the calculation of the modal parameters, the transfer functions has been determined from the following equation (2):

Table 5. Experimental Frequencies ratio between value after (f) and before (f_0) tensioning activities

	June 2016	July 2017	January 2018
Mode	f/f_0 (%)	f/f_0 (%)	f/f_0 (%)
1	3,51	2,39	2,45
2	1,03	3,27	2,54
3	2,87	2,48	1,49

$$H_{re}(f) = \frac{S_{x_{rPe}}(f)}{S_{p_{PePe}}(f)} \quad (2)$$

where: H_{re} is the transfer function; f is the frequency; S_{XrPe} is the cross spectral density between input and output signal, S_{PrPe} is the spectral density of the input signal.

3.3 *Dynamic measurement during earthquakes of Central Italy (2016)*

On 24 August 2016 a strong earthquake occurred in Central Italy. This earthquake, with magnitude 6.0, was recorded along the Valle del Tronto with its epicenter located between the municipalities of Accumoli (RI) and Arquata del Tronto (AP) and hypocenter at the depth of 8 km. The earthquake, perceived in all the main towns of central Italy, has caused about 300 victims among the local population and extensive damage to the residential and cultural heritage of the affected area. Two powerful replicas took place on 26 October 2016, with epicentres on the Umbrian-Marche border. On 30 October 2016, the strongest quake (magnitude 6.5) was recorded with the epicenter in the Province of Perugia. The two main shocks of 26 August and 30 October were also perceived in Rome and recorded by the dynamic monitoring system installed on the Basilica di Massenzio. The 24 August 2016 earthquake seismograms and amplitude spectrum recorded by two accelerometric sensors (A2 placed on the top of the monument and A1 on the foundation). The maximum amplitudes and the dominant frequencies of the two seismic events are instead reported in Tables 6 and 7.

The data analysis shows that the difference in magnitude (0.5) between the two earthquakes corresponds to an increase in terms of maximum amplitude recorded on the accelerometers placed on the top of the monument on all the components equal to about 2.

Furthermore, the amplification coefficient has been calculated as:

$$A_i = \frac{A_{2max_i}}{A_{1max_i}} \quad (3)$$

where: A_i is the amplification coefficient in i direction; A_{2maxi} is the maximum amplitude in i direction on the top of the monument; A_{1maxi} is the maximum amplitude in i direction at the base of the monument.

Table 8 values show how the amplification factors relating to the same direction remain almost constant for the two seismic events.

Finally, the maximum amplitude and dominant frequency values for each direction were compared with the limits indicated by the UNI 9916 and DIN 4150–3 standards. The threshold values have been exceeded, but both the static monitoring data and the campaigns of dynamic measurements, following the earthquakes, show no evidence of structural damage to the monument.

Table 6. 24/08/2016 earthquake. Max Amplitude (MA) and Dominant Frequency (DF)

	MA (mm/s)	DF (mm/s)	MA (mm/s)	DF (mm/s)	MA (mm/s)	DF (mm/s)
	x axis		y axis		z axis	
A2	11,4	1,44	8,73	1,78	1,82	1,34
A1	1,98	1,62	2,39	0,93	1,44	1,34

Table 7. 30/10/2016 earthquake. Max Amplitude (MA) and Dominant Frequency (DF)

	MA (mm/s)	DF (mm/s)	MA (mm/s)	DF (mm/s)	MA (mm/s)	DF (mm/s)
	x axis		*y axis*		*z axis*	
A2	24,4	1,32	19,5	1,7	4,7	1,9
A1	4,68	1,7	4,42	1,18	3,14	1,52

Table 8. 30/10/2016 earthquake. Amplification coefficient

Seismic event	Amplification factor		
	x axis	y axis	z axis
24/07/2016 earthquake	5,76	3,65	1,27
30/10/2016 earthquake	5,21	4,41	1,39

4 CONCLUSIONS

Dynamic monitoring of Basilica di Massenzio has been performed in order to assess the structural behavior both in "natural conditions" before the starting of the works related to the new Metro Line and without any structural reinforcement, also in "modified" conditions.

Modified conditions included the installation of steel pre-stressing chains and hoops to improve the strength of the structure by letting the different part of the structure to collaborate.

The structure was instrumented by both static and dynamic monitoring instruments and data were recorded during specific interventions and natural events, such as the two earthquakes which stroke the Central Italy area in 2016.

Monitoring data have been compared with numerical models and results have been analyzed to evaluate the status of the structure and the impact of the reinforcement which have been designed in view of the interaction with the new underground tunnels excavation.

The main conclusions are:

- The structure without reinforcement is behaving as a group non collaborating elements: aps, walls, pillars;
- The first 3 natural frequencies as per structural model were respectively: $f1 = 1{,}231$ Hz, $f2 = 1{,}834$ Hz and $f3 = 2{,}177$ Hz;
- Measured frequencies – assuming different material properties – have been evaluated to be: $f1 = 1{,}51 \div 1.44$ Hz, $f2 = 1{,}92 \div 2.08$ Hz and $f3 = 2.44 \div 2.48$ Hz;
- The structural reinforcement (chains and hoops pre-stressing) has increased the stiffness of the structure; consequently, the natural frequencies increased of a percentage between 1.03 to 3.51;

The effect of the earthquake have been evaluated and compared to the assumed structural limits;

The structural amplification factors have been evaluated for the recorded earthquakes, obtaining the following factors:

- 24/07/2016 earthquake: x axis = 5.76 – y axis = 3.65 – z axis = 1,27
- 30/10/2016 earthquake: x axis = 5.21 – y axis = 4,41 – z axis = 1,39

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