Monitoring of a SFRC retaining structure during placement

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ABSTRACT. This paper deals with a solution for monitoring rigid 3-D displacements and deformations of 4 SFRC innovative retaining structures designed for stabilizing unstable slopes. In particular, here the full monitoring system is described, whilst results achieved during the placement stage of every structure are given. It includes the periodic use of a robotic total station to perform geodetic measurements during both placement stage and standard life, with different acquisition rates. The application of standard techniques of Applied Statistics resulted in an over-estimate of the measurement accuracy, because positive contribute of correlations between readings taken at different epochs are not considered. An improved method to carry out this task has been defined and applied. The system is completed by local strain sensors (Vibrating Wire and Fibre Optic Sensors) which have been tested in order to define a suitable configuration for assessing deformations. The obtained results have shown that FOS are much less influenced by temperature change with respect to VWS, thank to the possibility of recording also temperature in a position close to the strain sensor. Finally, some load cells are used for measurement of the real tension on anchor cables that keep the panels fixed to the slope.

KEY WORDS: Monitoring, Engineering Geodesy, Retaining Structures, Risk Mitigation, Fibre Optic Sensor, Vibrating Wires.
1. Introduction

An innovative precast retaining structure for stabilizing unstable slopes has been designed at Dept. of Structural Engineering of Politecnico di Milano university (di Prisco et al., 2008b). This solution has been applied the first time to the reinforcement of a ground slope in Caslino d’Erba (Como, Italy). Here 4 panels were installed, integrated to other 11 smaller shelters to cover the upper part of the slope (di Prisco et al., 2008a). Both kinds of retaining structures are based on the high performance steel fibre reinforced concrete (SFRC) technology. In Figure 1 an image of the whole set of panels is reported.

This pilot installation has given the possibility to investigate and to assess the behaviour of these structures in a real operational environment. Indeed, the slope in Caslino d’Erba features some typical aspects of the Italian Prealps area, i.e. the presence of not homogeneous material, internal water circulation, and uneven external surface (Crosta et al., 2001). These characteristics resulted in a high complex definition of the geotechnical ground model and in a difficult forecast of the effective geotechnical response of the panels to external solicitations. In addition, during the placement stage some changes in the interface between ground and shelters have been introduced, and consequently the initial project was slightly changed.

This premise would like to stress the relevance of monitoring measurements to be carried out in here, concerning both placement phase and service ability condition. In particular the following issues devise to be considered and analysed.

First of all, a consolidated state-of-the-art technology for monitoring of retaining structures has not been defined yet, nor current technical specifications (e.g. EuroCode7) do give specific information and guidelines to be adopted in such applications. Even though several kinds of sensors and instruments can be supplied by Engineering Geodesy, Geotechnics and Engineering Geology, the design of a complete, durable and reliable monitoring system require a strong multi-disciplinary interaction. In many practical projects, the design of measurement systems is driven by instrument vendors, which in many cases propose off-the-shelf solutions that are not specifically tailored on a particular case. Conversely, the cooperation instantiated between the Surveying Dept. and the Dept. of Structural Eng. of Politecnico di Milano during the project in Caslino d’Erba allowed a deeper integration of competences. The basic concept is that the project of a monitoring system to be applied to a retaining structure should go together to the structural and geotechnical design. This approach presents the advantage that measurements can be focused on the analysis of really critical aspects, avoiding this way that redundant sensors were installed to measure secondary process. An important aim of this research is then to test different instruments and methods, in order to assess the achievable accuracy and the feasibility of their application for monitoring of the placement stage and for long-term control. In the former case, the whole monitoring system should be able to operate in harsh conditions typical of construction works (presence of vehicles, vibrations, obstacles) and to record data in short time. In the latter, some other aspects like durability, stability of the reference system, maintenance have to be dealt with. An expected outcome is then to give an appor in
the definition of a protocol for monitoring shelters applied to unstable slopes. This will be possible in a complete way after an observation period of a few years, involving more than one case study and different kinds of retaining structures. However, the Caslino d’Erba case study accounts for two different typologies of these, both to be monitored along the next five years. In this paper the full monitoring system is described, while results achieved during the placement stage will be presented and discussed. In Section 2 instruments, characteristics of the application and some methodological aspects to be used in similar projects are introduced. In the considered case study, periodical geodetic measurements by a robotic total station and different kinds of strain sensors (Vibrating Wires and Fibre Optic Sensors) have been used and experimented. In addition, some load cells are used for measurement of the real tension on anchor cables that keep the panels fixed to the slope.

Secondly, the availability of monitoring observations is fundamental for understanding the response of the panel under external solicitations. For this reason, a complete analysis of measurements recorded during the placement stage of each panel is reported in Section 3. In future, a similar analysis will be repeated by considering data recorded during the service ability of the retaining structures. Up today, measurements taken over the first 4 months of service ability are available, so that they have been retained to be a too small dataset to permit significant analysis.

The goal of the analysis which will be carried out along the paper are as follows:

Figure 1. A view of the slope in Caslino d’Erba with all retaining structures installed. On the lower part, the 4 bigger SFRC panels whose monitoring system is analysed in this paper.
- assessment of each sensor accuracy and reliability (operational and metrological) on an empirical basis, and comparison to design and theoretical expectations; this analysis is devoted to recognize which monitoring systems can provide the required information and which do not devise to be applied any longer in similar cases;
- analysis of correlations between different kinds of measurements;
- setup of a standard sensor configuration for monitoring of retaining structures, and especially for SFRC panels; this issue will comprehends the definition of specific operational procedures needed to reduce systematic errors and to improve the quality of measurements;
- check out of the response of shelters in Caslino d’Erba with respect to structural models.

All the metrological issues involved in the monitoring of all the 4 bigger SRFC retaining structures will be reported and discussed here in the following. The analysis of structural and geotechnical aspects related to these findings is out of the scope of this paper, but will be the object of another specific publication.

2. Monitoring equipment of SFRC panels

Different monitoring sensors have been installed before starting the placement phase. In Table 1, an overview of the adopted systems and the number of sensors per panel is reported. Broadly speaking, they can be divided into two groups. The first is based on geodetic measurements finalised to detect the 3-D global displacements of the retaining structures. To this aim, a set of control points (CP) have been positioned on the 4 panels’ external surfaces. To carry out these observations, a robotic total station (RTS) has been adopted, that has been repositioned at each epoch and georeferenced in a geodetic network in order to enable comparisons between measurements taken in a long-term period. In Sub-section 2.1, instruments, characteristics of the application and some methodological aspects to be used in similar cases are introduced. The second group of sensors can be addressed under the term local deformation sensors, and accounts for load cells (LC), vibrating wires (VW) and fibre optic (FOS) sensors. While LCs and VWs are normally used in this field, the use of the last one is quite innovative and experimental. Furthermore, as it can be seen in Table 1, geodetic measurements and LC readings are performed periodically according to a scheduled programme; on the other hand, observations with others sensors are carried on continuously with a high reading rate, thanks to a pair of remotely controlled acquisition units for data recording. A brief introduction to these instruments and the description of their use on shelters in Caslino d’Erba is given in subsections 2.2, 2.3 and 2.4, respectively.
**Table 1.** Number of actually operational sensors or geodetic control points (CP) on the panels in Caslino d’Erba. In parentheses after the number of CPs for RTS measurements it’s the number of these which have been directly fixed on the ground to check out relative displacements between panel and slope.

2.1. **Geodetic measurements**

The placement stage of 4 SFRC framed precast panels has been monitored by adopting **robotic total station** measurements and a set of retro-reflecting targets as control points. These will then be used for the monitoring during the ability service of the shelters. Different CP configurations have been adopted, as it is resumed in Table 2 and graphically shown in Figure 2. Panel 100 is equipped with 9 CPs, which have been previously evaluated as being the minimal configuration to assess the static condition of such a structure, considering its global deformation and not only rigid movements (rotation and shifts). On the other hand, due to the experimental character of this application, other panels have been equipped with a larger number of CPs (#13-15). Furthermore, some CPs have been directly fixed into the ground slope to check relative displacements of the panels with respect to the slope. The full set of targets has to be measured at each monitoring epoch during service ability, while a subset only has been used for measurements during the placement stage. This limit has been established in order to speed up the instrumental readings, considering the requirement of completing all loading steps of anchor cables in prefixed times.

Specific targets have been designed for this application, that are made up of two elements, one to be fastened in the concrete structure, and the other to be fixed with screws to the previous one (Fig. 3). The removable element reports a retro-reflecting surface for measurement by the electronic range-finder integrated into the RTS.

2.1.1. **Geodetic instrumentation**

The instrument adopted for all measurement stages is a RTC Leica TCA2003 (see Leica Geosystems 2008), that is capable of measuring azimuth readings (θ) and zenith angles (Ζ) with a nominal standard deviation of $\sigma_{\text{nom}} = \pm 0.15$ mgon (ISO 17123-3).
Figure 2. Different configurations of CPs and sensors for monitoring displacements and deformation of the 4 retaining structures. All of them size the same way, so dimensions are reported for the panel 400 only.
The phase-shift rangefinder enables it to measure distances with a nominal precision which can be evaluated by the formula \( \sigma_N = \pm (k_0 + k_1d) \), according to ISO 17123-4. The coefficient \( k_1 \) might be neglected in this application, because the distance \( d \) (in km) from total station to target keeps limited to a few decades of metres. The value of \( k_0 \) depends on the used reflector. In case of a high-precision prism (such those adopted for the materialization of the reference system – see Sub-sec. 2.1.2, \( k_0 = 1 \) mm; in case of “target tapes” used as CPs on the panels, \( k_0 = 3 \) mm). As described in the following, the TCA2003 is placed at every time on a topographic tripod an setup. Among the other facilities, this total station is equipped by motorized axes which allow it to autonomously aim on targets, if a list of approximate coordinates is already available. This option, integrated by the possibility of automatic target recognition (ATR), sharply speeds up the monitoring stage and makes readings independent from surveyor skillfulness. The total station can be georeferenced into an existing reference system by centring it on a point with known coordinates, and by azimuth orientation towards another known point. In addition, TCA2003 is capable to get itself georeferenced by 3-D inverse intersection. In this case, if a set of points (at least 2) with known coordinates is available, the total station can measure their position in its intrinsic reference system, and then computing a 3-D roto-translation to get the best fit into the ground reference system. Information about the achieved accuracy of georeferencing is also provided in real-time.

2.1.2. Reference system setup

A Ground Reference System (GRS) has been established in the area of the slope to be reinforced. Materialization of the GRS has been made by means of two 3-D...
monuments. When the system to measure displacements during both placement and service ability was initially designed, a solution based on a single total station standpoint was thought. Despite of the lack of redundancy on each CP measurement, this solution allowed a fast data acquisition, which was a mandatory prerequisite during the monitoring of the placement stage. In addition, the presence of several obstacles in front of the slope (a permanent crane, some yard vehicles) limited the area from which the whole surface of panels could be seen. Different is the case of monitoring during service ability after the end of construction works, when all existing obstacles will be removed. In this case, total station measurements could be also performed through intersection from two stand-points. A concrete platform at the base of the slope (point 100 in Fig. 4) was selected as total station stand-point. In order to reduce the setup error of the total station in the whole error budget (see Sub-sec. 2.1.3.1), the positioning of a stable (or removable) pillar would have been the best solution. Unfortunately, the platform where point 100 lies is placed on the roof of a facility construction of the public aqueduct. For this reason, no permanent structures were allowed to be built up over it. Then the adopted solution was to materialize the GRS through a set of 5 ground control points (GCP) widespread around stand-point 100. GCPs have been established in locations where no displacements are expected in future. On each of them, a high-precision retro-reflecting topographic prism can be accurately replaced at every measurement epoch (Fig. 3). The measurement of GCP coordinates was achieved at epoch “0”, before starting the placement of panels. As can be seen in Figure 4, a redundant geodetic network was setup, based on two instrumental stand-points (100 and 200). The mean accuracy of GCP measurement estimated in a Least Squares network adjustment resulted as ±0.6 mm in x direction, ±0.9 mm in y and ±0.3 mm in z. The lower accuracy along y is motivated by the larger contribute of the distance to the measurement of this coordinate. In Figure 4, 2-D error ellipses (1σ) of GCP coordinates are shown (Wolf and Ghilani, 1997).

The operational workflow to setup the total station into the GRS at each monitoring epoch is based on the following steps:

1. Approximate setup in correspondence of the nail on the platform (point 100), in order to take measurements from the same position. This solution has revealed to be very efficient in term of systematic error reduction, because all measurements are always performed with the same geometric layout.

2. The precise 3-D coordinates of instrumental centre vector \( t^i = [x^i_0, y^i_0, z^i_0]^T \) and azimuth orientation \( (\alpha^i_0) \) are determined at each epoch (i) in indirect way by means of inverse intersection on all GCPs. To do this, firstly the coordinate vector \( x^i_j = [x^i_j, y^i_j, z^i_j]^T \) of each CP (j) are computed in an arbitrary reference system from observations of azimuth angles \( (\theta^i_j) \), zenith angles \( (\zeta^i_j) \) and tilted distance \( (d^i_j) \):

\[
x^i_j = \begin{bmatrix} d^i_j \sin \zeta^i_j \sin \theta^i_j & d^i_j \sin \zeta^i_j \cos \theta^i_j & d^i_j \cos \zeta^i_j \end{bmatrix}^T
\]  

[1]
3. A roto-translation is then computed in order to find the best fit between the coordinates of all GCPs measured in the arbitrary reference system and the GRS ($X_j=[X_j Y_j Z_j]^{T}$); here the coordinates of points are independent from the epoch (i) because they have been assumed to be stable. For each CP (j) the following equation can be written:

$$X_j = R(\theta_i) \cdot x_j^i + t^i$$

Where the rotation matrix $R(\theta_i)$ is only dependent on the azimuth orientation because the main axis of the total station has been accurately setup along the local plumb line. The redundant system of Eqs. [2] is setup by including all measured GPCs and it is solved by means of Least Squares. Indeed, the system contains 3 unknowns and 10 equations for planimetry, which arise from 2 scalar Eq. [2] per each GCP, and 1 unknown and 5 equations for the elevation. The estimated solution $(\hat{\theta}_i, \hat{t}_i)$ allows to compute the residuals. In case of measurement errors or unattended displacements of a ground control point, the classical data snooping techniques (Baarda 1968) enables to reject them from the solving system.

The operational workflow to get the RTS georeferencing can be directly performed on the field thanks to the on-board tools of the Leica TCA2003 instrument. However, a more rigorous computation can be performed after in post-processing.

The georeferencing parameters allow to compute the coordinates of all measured CPs at epoch (i) into the GRS, based on Eq. [2].

2.1.3. Analysis and testing of point displacements

The displacements of CPs between two epochs can be inferred only from identical points, measured in two epochs. The requirement for standard deviations for displacements is very essential. Indeed, if the estimated displacements are several times the size of the displacements standard deviations, the most probable displacements can be derived from differences in point positions. In addition, to determining the magnitude and direction of the displacements, the hypothesis testing for the displacement is also necessary. Consequently, these corresponding calculations must be performed.

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1 By adopting the usual convention for azimuths in Applied Geodesy, $q_0$ is taken clockwise with origin on the vertical axis y. The resulting rotation matrix is then (see Kraus 2008 for reference):

$$R(\theta_q) = \begin{bmatrix} \cos \theta_q & \sin \theta_q & 0 \\ -\sin \theta_q & \cos \theta_q & 0 \\ 0 & 0 & 1 \end{bmatrix}$$
In this Sub-section, an evaluation about the standard deviations of CPs coordinates evaluated at each epoch from the geodetic measurements described in previous Sub-sections is given.

The experience achieved here, supported by that coming from other similar previous experiences, demonstrated that the real empirical precision of monitoring measurements by RTS is better than the theoretical one. Indeed, if the expected precision of 3-D measurement of CPs can be estimated through the theory of error propagation, the same cannot be effectively applied to the evaluation of displacements between different epochs. The reason of this is to be found into the presence of not modelled correlated errors, which can give a positive contribute in term of precision of monitoring deformations, and to a not correct modelling of the statistical error distribution. An approach to overcome this problem in case of 2-D geodetic networks is proposed by Savšek-Safić et al. (2006). Here an empirical statistical distribution of errors is derived from repeated network simulations.
through Monte Carlo Method. Here an alternative approach has been adopted, due to the presence of not redundant observations to CPs, that is based on the empirical evaluation of standard deviations of CPs coordinates from additional field measurements at epoch “0”.

In next paragraphs, first the evaluation of theoretical CPs standard deviations and statistical testing of displacement significance based on them is reported and discussed. Then, an alternative solution to compute the same from empirical observations is introduced.

2.1.3.1. Theoretical standard deviations of CPs

The error budget affecting the CP measurement is made up of two main contributions. The first one concerns the uncertainty of RTS georeferencing into the GRS at each epoch, accounting for the precision of the instrumental centre vector \( \mathbf{t} \) expressed through its covariance matrix \( \mathbf{C}_t \), and the standard deviation \( \sigma_{\theta_0} \) of azimuth orientation. Both \( \mathbf{C}_t \) and \( \sigma_{\theta_0} \) can be predicted through Least Squares simulation of the system made up of Eqs. [2], in the reasonable hypothesis of normal errors only. In this problem, all GCPs were weighted by using standard deviations derived from the adjustment of geodetic network at epoch “0”. This computation resulted in the following standard deviations for the components of \( \mathbf{t} \):

\[
\sigma_{tx} = \pm 0.4 \text{ mm}, \quad \sigma_{ty} = \pm 0.7 \text{ mm} \quad \text{and} \quad \sigma_{tz} = \pm 0.5 \text{ mm}.
\]

These a priori estimated values have been confirmed when real measurements have been introduced. In this case, standard deviations resulted as:

\[
\sigma_{tx} = \pm 0.3 \text{ mm}, \quad \sigma_{ty} = \pm 0.5 \text{ mm} \quad \text{and} \quad \sigma_{tz} = \pm 0.3 \text{ mm}.
\]

The second contribute to the error budget is given by the precision of angles and range measurements performed by the total station, which can be expressed through the covariance matrix \( \mathbf{C}_\theta = \text{diag}(\sigma_\theta^2, \sigma_\zeta^2, \sigma_\delta^2) \). The diagonal form of \( \mathbf{C}_\theta \) is due to the stochastic independence among observations. While the standard deviation of range can be directly evaluated as described in Sub-section 2.1, in case of angles some further contributions have to be considered. In Scaioni et al. (2008) all theoretical details about the evaluation of both \( \sigma_\theta^2 \) and \( \sigma_\zeta^2 \) are reported and applied to the present case study.

In the hypothesis that systematic errors have been completely removed, the covariance matrix \( \mathbf{C}_x \) of a CP can be estimated as follows (hereafter the index \( j \) which refers to a specific CP is omitted for clarity):

\[
\mathbf{C}_x = \mathbf{C}_j + \mathbf{C}_m = \mathbf{C}_j + \mathbf{J}_x \mathbf{C}_\theta \mathbf{J}_x^T \quad [3]
\]

where \( \mathbf{J}_x \) is the Jacobi matrix of derivatives of the system made up of Eqs. [2] relative to a single CP:

\[
\mathbf{J}_x = \begin{bmatrix}
\frac{\partial X}{\partial \theta} & \frac{\partial X}{\partial \zeta} & \frac{\partial X}{\partial \delta}
\end{bmatrix} \quad [4]
\]
From Eq. [3], the precision of a CP depends also on its relative position with respect to the total station stand-point. \( C_X \) gives the precision of point coordinates in the GRS where the total station has been georeferenced. Due to the inclination (\( \beta \)) of the horizontal profile of each panel with respect to the GRS (see Fig. 4), a new panel reference system (PRS) is needed to enhance displacements and precisions in tangential (t), normal (n) and vertical (Z) directions; as it will be described at Sub-sec. 3.1, all panels are placed with the main face approximate vertical, so that the t direction is aligned along the intersection of the local horizontal plane with the panel surface. The transformation to map coordinates of a point from GRS to PRS \( u = [u_t \, u_n \, u_Z]^T \) is given by:

\[
u = R(\beta) \cdot X \tag{5}\]

The rotation matrix \( R(\beta) \) is function of the horizontal tilt angle \( \beta \) defined from the X axis of the GRS towards the t axis of the PRS in anti-clockwise direction:

\[
R(\beta) = \begin{bmatrix} \cos \beta & \sin \beta & 0 \\ -\sin \beta & \cos \beta & 0 \\ 0 & 0 & 1 \end{bmatrix} \tag{6}
\]

The covariance matrix \( (C_u) \) of a CP referred to the PRS can be achieved through the transformation:

\[
C_u = R^T(\beta) C_X R(\beta) \tag{7}
\]

To discriminate between real displacements and measurement uncertainty a statistical test is performed (Teunissen 2000) considering the displacement of a generic coordinate \( k = \{t, n, Z\} \) between epochs (i) and (i+1):

\[
\Delta k_{i,i+1} = k_i - k_i^* \tag{8}
\]

The use of coordinates instead of the 3-D displacement vector (see e.g. Savšek-Safić et al., 2006) permits to keep the problem into linearity and to operate with the standard parametric inference. In the reasonable assumption that observations at two epochs are uncorrelated between them, each \( \Delta k_{i,i+1}^{k,t,n,Z} \) is still normally distributed as \( N(\mu_k, \sigma_k^2) \). The estimate of \( \sigma_k^2 \) can be expressed as function of the standard deviation at a single epoch which can be extracted from covariance matrix \( C_u \):

\[
\sigma_{\Delta k}^2 = (\sigma_k^t)^2 + (\sigma_k^{n,i+1})^2 + 2(\sigma_k^i)^2 \tag{9}
\]
To establish if a coordinate displacement $\Delta k_{i,i+1}$ has statistical significance, the following parameter is computed and compared to the critical value $T_{cr}=2$ according to the chosen significance level $\gamma=5\%$:

$$T = \frac{\Delta k_{i,i+1}}{\sigma_{i,i+1}} = \frac{\Delta k_{i,i+1}}{\sqrt{2}\sigma'_k}$$ \[10\]

Then it is assumed that the mean of $\mu_{\Delta k}$ is zero, i.e., no displacement happened (null hypothesis $H_0$). The test statistic $T$ is then tested according to $H_0$ and its alternative hypothesis $H_a$. If $T<T_{cr}$ at a chosen significance level $\gamma$, then the risk of rejecting the true null hypothesis is too high. Accordingly, it is established that the displacement is not statistically significant. If $T>T_{cr}$, the risk of rejecting $H_0$ is lower than the chosen significance level $\gamma$. Therefore $H_0$ is rightly rejected and the statistical significance of the displacements is thereby confirmed. In conclusion, displacements can be assumed as significant only if the following condition is fulfilled:

$$|\Delta k_{i,i+1}| \leq T_{cr}\sqrt{2}\sigma'_k = 2\cdot\sqrt{2}\sigma'_k = 2.83\sigma'_k = \Delta k_{cr}$$ \[11\]

In the case of CP displacements, estimated thresholds $\Delta k_{cr}$ are reported in Table 2. As it will be shown in Sub-section 3.2, point displacements involved in the considered case study are for the most much smaller than the computed thresholds. This would make not suitable the use of geodetic measurement for this application.

2.1.3.2. Empirical standard deviations of CPs

In reality, the theoretical evaluation of accuracy described in the previous paragraph disregards the fact that measurements are taken at different epochs through the same operational condition. Similar systematic errors in measurements are then repeated and can be removed when computing differences. In conclusion, the significance thresholds for displacements are inferior to the theoretical ones.

In order to estimate a more realistic value for the accuracy of total station measurements, an empirical approach has been adopted. First of all, in the expression of the covariance of a CP through Eq. [3], the contribute of the total station centring ($C_t$) as computed in the previous paragraph 2.1.3.1 is considered realistic. The covariance matrix ($C_m$) is then estimated by repeating a sample of $n$ measurements in a short period of time, during which no displacements occurred ($\mu_{\Delta k}=0$). During this period, the RTS remained on its setup. By this way, the influence of georeferencing accuracy was removed. The sample of readings obtained so that was then used to evaluate the relative measurement by Eq. [8]. Considering the generic coordinate $k$, the variance of the corresponding displacement $\Delta k^{0,i}$ between reference epoch “0” and a generic epoch $(i)$ can be estimated from the sample as (Mood et al., 1974):
\[
\sigma_{\Delta k}^2 = \frac{1}{n-1} \left[ \sum_{p=1}^{n} \left( \Delta k_{p}^0 \right)^2 - \frac{1}{n} \left( \sum_{p=1}^{n} \Delta k_{p}^1 \right)^2 \right]
\]

Variances of all coordinates estimated through Eq. [12] are then used to setup the empirical covariance matrix (\(C_{m,\text{emp}}\)) of measurements, neglecting correlations (alternatively, these can be assumed as in the original \(C_m\)):

\[
C_{m,\text{emp}} = \text{diag}(\sigma_{\Delta X}^2, \sigma_{\Delta Y}^2, \sigma_{\Delta Z}^2)
\]

Now, the general expression of the empirical covariance matrix of point displacements is given as:

\[
C_{i,i+1,\text{emp}} = C_{X,\text{emp}}^{i} + C_{X,\text{emp}}^{i+1} = C_{i} + C_{i+1} + C_{m,\text{emp}}
\]

which reduces to the term \(C_{m,\text{emp}}\) only if the instrument is kept fixed on the same stand-point during all epochs (e.g. during their placement of panels). To define the threshold \(\Delta k_{cr}\) for significant displacement detection, Eqs. [10] and [11] can be applied again, but tuned to consider that here the standard deviations \(\sigma_{\Delta}^{i+1}\) are extracted from the empirical covariance matrix \(C_{i,i+1,\text{emp}}\).

\[
\Delta k_{cr,\text{emp}} = T \cdot \sigma_{\Delta k,\text{emp}} = 2 \cdot \sigma_{\Delta k,\text{emp}}
\]

In Table 2, all significant displacements for different cases which have been evaluated with theoretical and empirical models are reported for the case study in Caslino d’Erba. As can be seen, empirical results give lower standard deviations and then reduce the size of significant displacement which can be recognized. This results increase the interest on RTS measurements, considering the displacements to detect are limited to a few millimetres.

<table>
<thead>
<tr>
<th>Configurations</th>
<th>Significant displacements – confidence level 5% (mm)</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>100</td>
</tr>
<tr>
<td>Model</td>
<td>(\Delta t)</td>
</tr>
<tr>
<td>Theoretical</td>
<td>Without</td>
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<td></td>
<td>With</td>
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<tr>
<td>Empirical</td>
<td>Without</td>
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<td></td>
<td>With</td>
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<tr>
<td>Range of azimuth directions (a)</td>
<td>379–392 gon</td>
</tr>
<tr>
<td>Range of zenith angles ((\zeta))</td>
<td>76–82 gon</td>
</tr>
<tr>
<td>Range of tilted distances (d)</td>
<td>24.5–27.8 m</td>
</tr>
<tr>
<td>Inclination of the panel in (XY)</td>
<td>47 gon</td>
</tr>
</tbody>
</table>

Table 2. Statistically significant displacements evaluated by theoretical (par. 2.1.3.1) and empirical models (par. 2.1.3.2).
2.2. Load Cells

The installation of load cells (LC) on 3 of the 4 anchor cables of panels 200, 400 and 500, allows to measure their effective tension and then to check out if they are correctly working on the ground. One anchor cable per panel remained without LC, as can be seen in Figure 2. This solution was motivated by the symmetry of panel structures. All panels were instrumented with LC before the placement stage, to be left there for the full ability service. The adopted cells have been constrained to 1 of 7 steel wires composing each anchor cable. In the hypothesis that all wires worked the same way, this information extends to the full cable. The sensor readings are performed manually.

Considering the use of this sensor is quite standard in monitoring of precast RC structures, no further detail are given here. In Table 3 the readings of LCs concerning the placement stage.

2.3. Vibrating wires

Vibrating Wire (VW) sensors are used to evaluate local linear deformation along a small baseline placed on the external surface of the panels. The functioning principle of VW is well known: a wire made of harmonic steel is constrained between two brackets fixed to the structures at a distance \( b \) (usually in the range 100-250 mm). The vibration frequency of the steel wire depends on its strain, that can be evaluated by measuring the frequencies at different epochs through the equation:

\[
\Delta \varepsilon = \frac{\Delta \sigma}{E_{VW}} = K_e \cdot \left( f_i^2 - f_0^2 \right)
\]  

[16]

where \( f_0 \) and \( f_i \) are the frequency readings at the epoch “0” and “i”, \( E_{VW} \) is the elastic modulus of the wire, and \( K_e \) is a transformation constant typical of the specific sensor, which is usually provided by the producer or that can be evaluated through laboratory calibration.

After describing the basic relationship [16], the influence of temperature variation on VW measurements can be dealt with, this representing the largest source of errors. Here the approach given in Pepe (2008) has been followed. Let us analyse the simplest case, i.e. a VW on a beam that is unconstrained along its longitudinal direction. In case of a temperature variation \( (\Delta T > 0) \), both points of the beam where brackets of the extensimeter are placed will have the following strain:

\[
\Delta \varepsilon_T = \alpha_c \cdot \Delta T
\]  

[17]

whilst the beam will not be affected by any strain, because it is free. In Eq. [17] \( \alpha_c \) is the thermal dilation coefficient of the concrete. At the same time, the VW should be
subjected to the same strain $\Delta \varepsilon'_{\Delta T}$, because it is constrained to the beam through the brackets. On the other hand, it is also affected by strain due to its intrinsic thermal deformation (dilation coefficient $\alpha_{VW}$):

$$\Delta \varepsilon'^{\prime}_{\Delta T} = \alpha_{VW} \cdot \Delta T > \Delta \varepsilon'_{\Delta T}$$

[18]

because $\alpha_{VW} > \alpha_c$. In reality, the coherence between both deformations $\Delta \varepsilon'_{\Delta T}$ and $\Delta \varepsilon''_{\Delta T}$ results in a slack effect on the wire:

$$\Delta \varepsilon'^{\prime}_{\Delta T} = \left(\alpha_{VW} - \alpha_c\right) \cdot \Delta T \cdot E_{VW}$$

[19]

Now we consider a beam which is constrained at both ends, so that it cannot freely deform along its longitudinal direction. An increment of temperature $\Delta T$ will result in increasing the compression stress in the beam:

$$\Delta \sigma_{\Delta T} = \alpha_c \cdot \Delta T \cdot E_c$$

[20]

where $E_c$ is the elastic modulus of the concrete. The same $\Delta T$ generates a compression stress on the wire, which is read by the extensimeter as a false compression stress on the beam:

$$\Delta \sigma_{\tau} = \Delta \varepsilon'_{\Delta T} \cdot E_{\tau} = \alpha_{VW} \cdot \Delta T \cdot E_c$$

[21]

Then, in order to compute the effective axial force $N_{\text{beam}}$ related to a given strain $\varepsilon$ obtained from Eq. [16], this should be compensated for the thermal fictitious effect:

$$N_{\text{beam}} = \left[\varepsilon - \left(\alpha_c - \alpha_{VW}\right) \cdot \Delta T\right] \cdot E_c \cdot A_{\text{beam}}$$

[22]

The problem becomes more complex if the beam is constrained in not rigid and not modelled way. In such a case, the beam will be stressed under the effect of $\Delta T$, but a reaction would be induced by the stress. This results in the impossibility of correlating the beam’s stress to $\Delta T$, even though its thermal dilation coefficient ($\alpha_c$) is known. The knowledge of the deformation into the beam is strictly required to determine its stress. These considerations suggest that changes of temperature can strongly affect VW measurements. The presence of a temperature sensor close to each VW and the knowledge of thermal dilation coefficients of both the wire and the structure are mandatory to compensate for errors due to $\Delta T$, which can result in observed compression higher than the effective stress in the structure. The presence of protections made up of insulation material can be also useful. However, an accurate modelling of structure deformation under thermal effects is required to compensate VW measurements.
In Caslino d’Erba a set of VWs featuring $b_\varepsilon=165$ mm and $K_\varepsilon=4.062\cdot10^{-6}$ Hz$^2$ have been installed on 3 of the 4 panels, in positions depicted in Figure 2. The data acquisition is performed periodically (rate $5’-10’$) by an automatic device, which logs data on a PC operating as local Control Unit (CU). This can be remotely controlled via GPRS (Bao et al., 2006). The connection between each VW sensor and the CU is possible through cable links. Unfortunately, temperature sensors close to each VW has not been working during the placement stage.

2.4. Fibre optic sensors

The fibre optic sensor (FOS) technology accounts for several types of sensors capable to measure linear and angular displacements, strain, rotation, acceleration, vibration, temperature, humidity, pressure, on the basis of different measurement principles (Udd 1994). In the field of structural monitoring they are going to replace traditional sensors, pushed by some inherent advantages which include their ability to be lightweight, of very small size, passive, low power, and resistant to electromagnetic interference. On the other hand, their major disadvantages are the high cost and the unfamiliarity to the end users. In Ansari (2005) and Inaudi and Glisic (2008) some examples of FOS’ application to structural monitoring are shown.

One of the panels in Caslino d’Erba (no. 400) was equipped by 6 sensors (even though one of these went out of order just after the installation, reducing the number of operational sensors to 5 only) produced by Monitor Optics Ltd. (Dublin, Eirland), able to measure the displacement in a direction along a few decimetres baselines and contemporary to record the temperature. Specifically, the adopted sensors are based on advanced Fibre Bragg Grating (FBG) transducers, exploiting the dependence of the Bragg wave-length to the strain field applied to the grating, as first described by Butter and Hocker (1978). When an axial strain field is applied to a Bragg grating, it effects both the period because of the resulting elongation of the fibre and the refractive index because of the photo-elastic effect. The Bragg wavelength is also function of the temperature applied to the grating, therefore some kind of temperature compensation is necessary to accurately measure strain. The optical sensing elements are incorporated into telecom-grade optical fibre and integrated into a suitable engineering packaging (Whelan et al., 2002; Albrecht et al., 2004).

The use of dual FBG allows the acquisition of both strain and temperature: the former is achieved by a sensor constrained into the inner concrete structure, the latter by a second sensor close to the first one but unconstrained. Both readings are derived from decoding a wavelength-encoded optical signal that is serially output along the same optical fibre. The transformation into a digital, calibrated output is performed by an opto-electronic interrogation unit, which also provides to data recording and to the end user interface. In addition to above-mentioned advantages of FOS over traditional sensing technology applied so far in long-term structural monitoring, FBG presents further ones:
- They do not suffer from drift and guarantee a high accuracy over long periods of time (years);
- they are self-referencing so they can be powered down between readings without losing any reference;
- they exhibit high resolution and large dynamic range;
- FBG sensors and cable network are packaged to telecom standards which makes them highly resistant to harsh environmental conditions;
- they can be placed into the inner concrete structure, if they are installed during concrete casting;
- possibility to connect cables serially allows more sensors to exploit the same cable link, saving this way cost and reducing the total length of cables to lay down on the structure to monitor.

In Table 4 are reported some metrological features of the adopted FBG sensors in the present case study.

3. Analysis of measurements during placement of SFRC panels

3.1. Placement strategy

In order to simplify the placement of all panels and to guarantee a better contact with the ground, their installation followed the construction of four vertical RC beds. Firstly, both superior panels (100 and 200) was positioned and fixed. A preliminary loading of all central anchor cables was carried out in order to put each panel close to the RC bed. The first reading epoch occurred at this stage (epoch “0”). Then 6 loading steps have been contemporarily applied to central cables, up to reach the service ability condition. The procedure was first applied to panel 100 and then to panel 200. Secondly, all lateral anchor cables of both structures were tensioned through 3 loading steps for panel 100 and 6 for 200, respectively (see Table 3). The different number of steps are motivated by the not homogeneous soil condition under each panel which originated a diverse response. After three months, the ground has been prepared for the installation of lower retaining structures (400 and 500). These have been placed one after the other, by tensioning central and lateral cables of each panel. At this stage, a lower number of loading steps have been operated with respect to the installation of other panels; obviously, the experience previously achieved allowed construction work managers to better plan the process.

In a further step, loading of almost all panels have been refined by further steps, which are not reported in Table 3. Finally, the placement of other small shelters in the upper part of the slope (see Fig. 1) has been performed after.
<table>
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<th>load cells readings (ton)</th>
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<td>-</td>
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<td>342</td>
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<th>applied nominal load (kN)</th>
<th>mean stread elongation (mm)</th>
<th>load cells readings (ton)</th>
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<td>49</td>
<td>60</td>
<td>11.0</td>
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Table 3. Measurement of strain elongation and LCs during the placement stage of all panels.

<table>
<thead>
<tr>
<th>Readings</th>
<th>Maximum gauge length</th>
<th>Resolution</th>
<th>Accuracy ($\sigma$)</th>
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<td>Strain</td>
<td>$L=250$ mm</td>
<td>$10^{-6}L$</td>
<td>$\pm 10^{-5}L$</td>
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<tr>
<td></td>
<td></td>
<td>(0.25 $\mu$m for $L=250$ mm)</td>
<td>(2.5 $\mu$m for $L=250$ mm)</td>
</tr>
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<td>Linear deformation</td>
<td>$10^6$ $\mu$e</td>
<td>$1$ $\mu$e</td>
<td>$\pm 10$ $\mu$e</td>
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<tr>
<td>Temperature</td>
<td>All operational air temperature scale</td>
<td>0.1 C</td>
<td>$\pm 1.0$ C</td>
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</table>

Table 4. Main metrological properties of FOS by Monitor Optics Ltd. adopted in Caslino d’Erba.

3.2 Analysis of monitoring measurements

The installation stage of retaining structures was monitored by RTS measurements, by applying the technique described in the previous Section 2.1. Observations concerned a subset of 6-7 CPs only (see Table 1), due to the small time available between loading steps. Figure 2 reports the positions of all CPs adopted during the placement stage for each panel. In addition, a CP fixed on the ground was measured to check out the displacements of the slope with respect to the structure. The selection of the subset of CPs to observe during placement was based on symmetry consideration about the panel geometry. Before the placement of each panel, the total station Leica TCA2003 was placed in correspondence of point 100 of the geodetic network, setup on a topographic tripod, and georeferenced. Then it wasn’t removed as far as the placement was completed. In this case, all measurements taken during the same session are likely not to be affected by systematic georeferencing errors due to instrument repositioning.

In Figures 5, 6, 7 the results in term of displacements along the axes of the PRS are shown for panels 200, 400 and 500. Hereafter displacements have been always referred to epoch “0.” As demonstrated in Sub-section 2.1.3, the significance of recognizable displacements is very high because $T_c$ has been evaluated at 1 mm level and also lower (see Table 2). This fact increases the interest in RTS measurements, opening to the possibility to detect the rigid motion component of each panel, and perhaps the elastic deformation in the normal direction of the PRS. The measurement of deformation along other axes is left to local sensors (VW and FOS), which are capable of resolutions and precisions at a few micrometers order. For this grounds, no comparisons between different RTS and the other sensors is devised.
Figure 5. Displacements of CPs on panel 200 measured by RTS; these have been computed with respect to step “0”. Threshold for significant displacements ($T_{cr}$) in $t,n,Z$ is 0.4 mm, 1.0 mm, 1.2 mm, respectively, according to Table 2.
Figure 6. Displacements of CPs on panel 400 measured by RTS; these have been computed with respect to step "0". Threshold for significant displacements (Tcr) in t,n,Z is 1.0 mm, 1.4 mm, 0.4 mm, respectively, according to Table 2.
Figure 7. Displacements of CPs on panel 500 measured by RTS; these have been computed with respect to step “0”. Threshold for significant displacements ($T_{th}$) in $t$, $n$, $Z$ is 0.4 mm, 0.8 mm, 0.4 mm, respectively, according to Table 2.
The RTS measurements for the panel 200 (Fig. 5) concerned 4 CPs on the ground and 4 on the structure. Generally, all displacements of CPs of the first group of CPs show agreement among them, barring a few cases where a small difference exist (e.g. 43 along n axis and 23 along Z axis at all epochs, while along t axis all of them follow a common trend of displacements with a few punctual discrepancies). These results show that the total station has not kept stable, then displacements on the panels should be compensated for those of points fixed on the ground. According to this, a small shift (about 1 mm meanly) can be recognize in t direction, a clear movement towards the slope in n direction, while an upwards movement of the left side of the panel along Z. Total station datasets for both inferior panels (400 and 500, see Fig. 6 and 7, respectively) show that a similar movement occurred between them. This can be decomposed in a vertical upwards shift amounting to an average of 7.5 mm in case of 400 and 5 mm for 500. Secondly, an inclination in the vertical plan around the main longitudinal axis has been observed, that in case of panel 400 presents a rotation axes positioned at 63 cm over its bottom edge; in case of panel 500, the rotation axis is 42 cm under the bottom edge, meaning that the whole structure has been pushed against the ground for an amount ranging from 4 to 18 mm. Rotation angles appeared as about 1 gon and 0.5 gon for panels 400 and 500, respectively. This rotation is motivated by different response of the ground bed against which the panel have been compressed. In particular, the presence of a strongly resistant concrete bed in the lower part resulted in a rotation of the upper edge of the panel towards the slope. In the direction tangential to the panel surface, an homogeneous shift of about 2 mm can be seen in case of panel 400, while for the other a small rotation around the normal direction occurred. Movements in t and Z direction with respect to the PRS are probably due to the settling of the interface structure-ground.

The analysis of displacements of CP fixed on the ground (“43”) has resulted not to be significant in t direction. In n, the ground has given a response similar to that of the structure, i.e. a depression in case of 400, and a compression for the other. Similarly, the CPs on the ground revealed a small displacement upwards along Z (about 2mm) as both whole panels. For the sake of completeness, the risk exists that movements of CPs on the ground are due to a not full stability of the total station. Indeed, to overcome this problem, a check of the RTS georeferencing should have been repeated at the end of placement.

In order to discriminate between the rigid displacement of the panel and its deformation due to loading, a further analysis was carried out. Relative 3-D displacements $\Delta n_{i,i+1}$ of each CP were considered. In case of only rigid movement, interpolation of all $\Delta n_{i,i+1}$ with a linear function should well fit data. On the contrary, the presence of higher residuals compared to $T_c$ would mean the presence of a deformation component. To avoid the influence of possible measurement errors, the estimate of the linear function interpolating relative displacements of CPs was performed by using a L1 robust technique (Barrodale and Roberts, 1973). In Figure 8, mean and standard deviations (error bars) of 3-D residuals with respect to the interpolating plane computed at different loading steps are reported for panel 500. Results show that relative residuals are always lower than the significance level of measurements $T_c=0.8$ mm reported in Table 2. However, the larger deformation
occurs after the first loading step (5) of lateral anchor cables. During the following steps, this deformation is partially adsorbed and residuals are small again. The same conclusion has been drawn from the analysis carried out on the other structures.

The availability of VW and FOS sensors on the most panels permitted to register strains in the tangential direction, where no significant information were achieved by geodetic measurement. Thanks to the automation of reading by the corresponding CU, both systems of sensors have been setup to record a reading every 1’ during placement. Then, all measurements have been averaged to give out only one observation per loading step.

In Figure 9 the readings of VWs concerning the placement stage of panels 200, 400 and 400 are reported; Figure 10 reports readings of FOS on the panel 400 during the placement stage, after compensation for the temperature effects. In addition the amount of thermal deformation in correspondence of FOS is also reported.

First of all, a preliminary consideration about the data quality must be addressed. Both strain sensor are affected by temperature variation, especially VWs which are positioned at open-air and cannot be compensated because the temperature sensor has started working after placement. In Figure 11, readings of FOS and VW sensors which are placed in close positions on panel 400 are reported. By looking at the temperatures recorded by FOS and reported on the same Figure 11, two different trends can be enhanced. The first groups sensors “S1”, “S4” and “S6” which are placed on the external side of the structure; their temperatures decrease according to a lowering of the air temperature (the placement started at 14:50 and ended at 16:25 of 25th October). On the other hand, the temperature of sensor “T2”, which is placed next to the soil, has got higher throughout the same period. This is motivated by a warming effect given by the ground to the panel. This analysis put in evidence that temperatures strictly depends on specific positions, then those in correspondence of VWs cannot be inferred. In addition, VWs are placed on the external surface and are affected by direct sunlighting. According to this observations, differences which can be noticed in Figure 11 are partially motivated by the lack of temperature compensation for VWs, and then by the different positions with respect to the neutral axes of the sections where they are placed. To improve the measurements of VWs by applying to them a corrective model accounting for temperature changes will be future work.

However, results given by both systems are coherent with the theoretical structural behaviour of the panel. In case of FOS, the following results can be outlined:

1. sensor “S1” is placed on the lower beam close to the side facing the valley, on a section which is compressed during loading of central cables, and tensioned during loading of lateral ones. This behaviour is exactly that has been reported in Figure 6;
2. sensor “S4” revealed a constant slightly tensioning during loading of both central and lateral cables;
3. sensor “S6” registered a very small compression during loading of central cables and a tension just after; however, these results are lower than the significance level, which is $T_{cr}=2\sigma=20 \mu e$ according to Table 5;
Figure 8. Mean and standard deviation of residuals with respect to the interpolating plane of CPs’ relative (on the left) and absolute (on the right) displacements; these have been computed with respect to step “0”.

4. sensor “T2”, aligned along the Z axis, has not showed any deformation, as expected;
5. sensor “T5” registered the largest tension during the loading of central cables, which is fully adsorbed again during loading of lateral ones.

The lower graphic in Figure 10 reporting the contribute of temperature change to the deformation measured by FOS, shows somehow this can be relevant if compared to the effective strain. This effect is emphasized on sensors placed on the external surface (“S1”, “S4”, “S6”). The role of measuring the temperature in correspondence of the sensor is then really important.

4. Final considerations and future work

In the paper a case study made up of 4 SFRC retaining structures installed for the reinforcement of an unstable slope has been presented, focusing in detail on the monitoring systems to be used during placement stage and service ability. The case study is located in Caslino d’Erba (Italy), in the Italian Prealps area.

In Section 2 all adopted instruments and methods have been presented and problems related to their application discussed. Mainly, the monitoring systems is based on the use of a total station which is capable to detect 3-D displacements of a set of control points on all structures. Here this kind of instrument has been setup during each placement session of a panel, which consisted in a first loading of two central anchor cables, and in a second one concerning two lateral cables. In future, the total station will be periodically reinstalled (e.g. monthly) in order to achieve displacements throughout a long term period. The realisation of a geodetic network allows one to setup the instrument into a stable reference system, needed to compare data between them. The classical technique adopted in Applied Geodesy to estimate
the precision of such measurements, which is based on covariance propagation of theoretical standard deviations of instrumental readings, has revealed to underestimate the quality of achievable results. An alternative approach consisting in the on-site estimation of measurement precision through the acquisition of a sample of repeated observations under the same external conditions has been proposed and applied. The application of this method to real data taken on the case study confirmed its validity and allowed to enhance the 3-D displacements of all panels during different loading steps of placement.

In addition, the retaining structures have been equipped with different configuration of sensors capable to detect local displacements with a high accuracy. Load cells have been installed on 1 of the 7 wires of each anchor cables, in order to measure their tension. Fibre optic sensors (FOS - specifically based on Fibre Bragg Grating technology) have been placed in 1 of the 4 panels during casting, so that they can record the deformation in the inner structure with a precision in the order of $10^{-6}$. This performance is possible thanks to the presence of a dual sensor based on the same fibre optic technology, which is capable to contemporarily works as extensometer and temperature sensor. Reading of FOS is performed automatically by means of a data acquisition unit, which can be also remotely controlled. Deformations recorded by the 5 FOS installed has agreed to the expected structural behaviour of the panel where they have been placed. A second system of extensometers of different kind (vibrating wires – VW) has been adopted on 3 of the 4 panels with a larger number of sensors. Unfortunately, this kind of sensor has resulted to be strongly influenced by temperature variation and, during the placement stage, no thermal sensors were available close to each VW. On the other hand, the mere knowledge of the temperature is not enough to compensate for errors on VW measurements, aim requiring the modelling of the structure’s response under thermal solicitation. The integration to each VW of a temperature sensor during the service ability will permit to calibrate them. Indeed, in period with constant external load condition of the panels, it will be possible to correlate deformations recorded by VW to temperature changes. The presence in close positions of a few FOS will give reference deformation for calibration purpose.

The system made up of the tested monitoring instruments is capable to provide a complete overview of retaining structure deformation and movements during the placement stage. The automatic data recording offered by FOS and VW will allow to observe the panel structural behaviour during service ability, integrated by periodical measurement of control points performed by a total station. The availability of this dataset will be useful for safety maintenance and for analysis purpose. Indeed, the case study will be observed for a period of 5 years.
Figure 9. Deformations recorded by VW sensors during placement of panels 200, 400 and 500.
Figure 10. Deformations recorded by FOS during placement of the panel 400, after correction of thermal contribute (temperature was contemporarily registered by the FOS themselves); the lower graphic shows the amount of thermal deformation.

Figure 11. Comparison between FOS and VW measurements on close sensors; on the right, temperatures recorded by FOS are displayed. The same colour corresponds to a pair of close sensors of different kind.
Acknowledgements

To be completed after review process.

5. References


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