Laser Tracker Technology for Static Monitoring of Civil Infrastructure

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1 INTRODUCTION

Civil infrastructures are a vast category that includes simple constructions or big and complex objects like vehicular bridges, tunnels, factories, conventional and nuclear power plants, offshore petroleum installations, heritage structures, port facilities, and geotechnical structures. All structures begin to

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deteriorate once they are built and used, and different sudden external factors can affect their stability and safety; for example, explosions, excavations, earthquakes, etc. For this reason, monitoring and maintenance programs are periodically planned during the entire life cycle of a structure to test its integrity and level of safety.

A proper monitoring and maintenance program for each structure is influenced by different factors like location, importance, ownership, use, risk, and hazard connected to its failure and other aspects that may be regulated by law. The process aimed at tracking the presence and relevance of damages (cracks and movements) in the structure and assess the current state is often termed structural health monitoring (SHM) [1-2]. This process concerns the evaluation of different degradation phenomena like structural damage, water infiltration, chemical and mechanical degradation of materials, etc.

Monitoring such different scenarios requires different operators, from civil engineers and architects up to experts in the fields of mechanics, materials science, chemistry, physics, and so on; that is, a large group of experts all involved in the conservation process. Although each research field presents its own peculiarities, in a complete SHM project a general trend of present-day research is to develop effective and reliable methods of acquiring, managing, integrating, and interpreting structural performance data while trying to minimize the qualitative and subjective human aspects.

One of the parameters used to evaluate structural stability and safety is displacement detected at different epochs. Indeed, although movements are expected during the normal life of the structure (because of variation of superimposed loads, thermal changes, external vibrations induced by underground lines, subsidence, etc.), when a slow and constant movement reaches a significant magnitude, or in the case of sudden events (like earthquakes or explosions), evaluation with a metric measurement can be used as a parameter to assess the structure's safety. Ross and Matthews [3] and Mita [4] illustrate different cases in which structural monitoring may be required. They include modifications to existing structures, monitoring of structures affected by external works (deep excavations close to the inspected construction), monitoring during demolition, structures subject to long-term movement or degradation of materials, feedback loop to improve future design based on experience, fatigue assessment, assessment of post-earthquake structural integrity, decline in construction and growth in maintenance needs, and the move towards performance-based design philosophy.

Starting from the origin of the load acting on a structure, at least three different deformation phenomena can be identified: static, quasi-static, and dynamic movements. Different criteria can be used to classify these movements, but all are mainly concerned with the different time of application of a load and the associated deformation.

The work presented in this paper focuses on static monitoring; that is, those applications where deformation during data acquisition is sufficiently slow so that the structure can be considered fixed during the data acquisition time. Static monitoring campaigns usually last weeks, months, years, or even decades. The static nature of the monitoring depends on different factors such as loads, construction materials, temperature, etc. - the parameters to be evaluated during the planning phase.

Several standard tools able to detect and monitor structure movements [5-9] are available on the commercial market, but in recent years new instruments for different metrological applications have appeared on the market and can be used for some specific applications. Among them, laser tracker technology seems a very promising alternative for structural monitoring. Nowadays, laser trackers are mainly used in industrial applications and allow the collection of data with precisions superior to ±0.05 mm. However, their use in civil engineering applications is quite limited at the moment and the suitability under unstable environmental conditions needs more investigation. Indeed, structure monitoring can be carried out in areas where it is difficult to take into consideration external effects such as bad weather conditions, humidity, wind, and temperature.

This paper gives a brief state-of-the-art of the proven techniques in this field and then presents some experiments carried out on site to evaluate the usefulness of this innovative instrument, showing its potential but also raising some issues concerning its use for practical applications.

2 STRUCTURE MONITORING WITH TRADITIONAL SENSORS

Different data can be collected during a monitoring campaign. They encompass one-dimensional (1-D), two-dimensional (2-D) and three-dimensional (3-D), absolute or relative movements, and angular variations. Displacements are defined as absolute when they are estimated with respect to stable points without a direct connection to the structure. These points are physically materialized by means of benchmarks and it is assumed that they are fixed or that their movement is so small that it cannot be measured. These points are generally located in stable areas close to the object under investigation. Relative measurement instead concerns points fixed on the structure, where it is not possible to guarantee their stability over time. Some instruments can be used to detect both absolute and relative displacements while others are designed for relative measurements only (see Table 1 for a review).

Standard geodetic tools for structure monitoring are digital and optical levels (for detecting vertical variations), total stations, and global navigation satellite system (GNSS) systems (3-D movements). Other instruments traditionally used for structural monitoring are deformometres, extensometers, inclinometers, and pendulums and are generally used to measure relative displacements. In the last decade some new technologies like laser scanners, radar systems, digital cameras, strain gauges, linear variable differential

TABLE 1
Some instruments used in the field of static monitoring of constructions.

Instrument	Precision	Movement
Total station	± 0.30 – 0.40 mm	3-D
GNSS	± 20 – 50 mm	3-D
Level	±0.10 – 0.20 mm	1-D (Vertical)
Photogrammetry	Variable	3-D
Laser scanner	± 2.00 – 5.00 mm	3-D
Deformometer	± 0.01 – 0.10 mm	1-D
Comparator	± 0.01 – 0.10 mm	1-D
Plumbs	± 0.10 mm	2-D
Fibre optic	±1 μm – 0.10 mm	1-D
Distometer	± 0.01 – 0.10 mm	1-D

transformers (LVDTs), and fibre optic sensors were progressively introduced in the field of structure monitoring.

A real monitoring campaign includes multiple information gathered by several instruments distributed on the whole structure. Some techniques can provide information about the entire object (geometric levelling networks can reach many homogeneously distributed benchmarks), whereas some data remain localized to small areas (the measurement of crack apertures). The combined use of all metric data and their numerical study are fundamental to understand the behaviour of the structure.

Some common geodetic tools used for structure inspection include total stations, levels, and GNSS systems. When a monitoring project is aimed at detecting vertical variations, geometric levelling is undoubtedly the most-used method due to its limited cost and its sub-millimetre accuracy. A geometric (or spirit) levelling campaign is carried out with a level (that gives a horizontal line of sight) and a set of vertical rods. The difference of height between two benchmarks is given by the difference between rod readings.

Digital levels perform the reading in an automated way. After an initial setup of the instrument by means of a bubble, an image of a bar-coded scale printed on the rod is digitally captured. The reading is determined by comparing the acquired image with the predefined bar-code pattern of the rod stored inside the level. Nowadays, digital levels for monitoring applications can achieve height measurement with precision of about ±0.3 mm/km.

Alternatively, optical (mechanical) levels are still available on the market. The rod reading is performed by a human operator who collimates the regular graduation of levelling rods (generally 5.00 mm or 1 cm). The use of the integrated parallel plate glass micrometre allows the measurement with a precision of ±5 μm to 0.01 mm. Here, the collimation of the nearest graduated notch is directly connected to the displacement measured by a micrometre moved by an adjustment screw. When 5.00 mm graduations are employed, the height precision is about ±0.2 mm/km.

3-D movements can be measured instead with robotic total stations. Nowadays, instruments for static monitoring provide ±0.5” angle precision and ±0.60 mm distance precision when using phase-difference measurements and a retro-prism mounted on the structure. A proper geodetic network (with adequate geometry and redundancy) includes angular and distance measurements that are then adjusted by means of a rigorous Least Squares method to derive 3-D coordinates of a set of benchmarks. The main advantage concerns the high accuracy achievable with automatic target recognition and the opportunity to measure indoors and outdoors.

The differential global positioning system (GPS) (or more in general GNSS systems) allows the extraction of 3-D coordinates for some specific points [10]. An essential advantage of GNSS systems with respect to total stations is the opportunity to work without a direct line of sight (inter-visibility) between instrument and target. In addition, measurements can be carried out during the night and under varying weather conditions, which makes GNSS measurements practical, especially when multiple receivers can be installed on the structure. However, indoor measurements cannot be performed. Relative measurements are carried out with a master station placed on a stable location and some other rover antennas mounted on the structure. The location of the master has to be as close as possible to the structure to reduce baselines and increase differential measurement precision. In addition, both master and rover need good satellite visibility, absence of multipath (multiple signal reflection), and power and data links for all of the receivers. Differential measurements with good networks reach precisions better than ±3.00 mm, which can be improved when slow movements are expected. This is carried out by applying adaptive filtering algorithms on the signal to reduce data noise and to obtain more accurate static solutions.

In recent years several other sensors were proposed in the field of structure movement inspection. In fact, total stations and GNSS receivers, although very accurate, can measure only a very limited number of points. Terrestrial laser scanning [11-12] can partially overcome this limitation. Indeed, a laser scanner acquires dense point clouds constituted by several millions of unspecific points, providing an unprecedented amount of spatial information. This allows a more accurate investigation of the displacement on the whole surface of the structure; however, the 3-D accuracy of each point is generally lower than that achievable with a total station. Further issues arise for instrument calibration, scan registration, data processing, and filtering.

The overall accuracy of a multi-scan comparison can be enhanced by exploiting data redundancy with regular shapes that locally approximate the point cloud, especially when the structure features a simple and regular geometry. For more complex situations, the original point cloud can be triangulated and filtered to reduce data noise. Monitoring with laser scanners is performed by comparing the point clouds or the fitted models (either simplified or triangulated) at different epochs. A preliminary co-registration of the data in the same reference system is needed. If the structure movement is larger than the point cloud noise (usually few millimetres) deformations can be individuated and visualized in displacement maps.

Some specialists make use of photogrammetric techniques and vision metrology for structure monitoring and material testing [13-14]. Image-based methods often rely on the use of targets uniformly distributed on the object. Targets are usually employed because they form an array of points that can be surveyed at different epochs to detect the structure movement. The estimation of 3-D coordinates is carried out by means of Least Squares bundle adjustment, in which the image coordinates are input data and have to be measured with precise image-matching operators.

Image measurement can be performed in an automatic way by means of least Squares matching [15] with a precision superior to 1/50 pixel. The theoretical accuracy of 3-D points derived from bundle adjustment depends on several factors such as the number of cameras, network geometry, the size of the object, and the camera calibration quality. In some experiments performed under controlled conditions, theoretical accuracies from 1:100,000 up to 1:1,000,000 for hyper redundant network schemes were reached [16].

As previously mentioned, additional sensors are used to monitor in an accurate and reliable way the structural behaviour of a building. Some examples are metal wires, fibre optics, vibrating wire strain gauges, comparators, accelerometers and inclinometers, LVDTs, cable extensometers, and pendulums. They can be a valid support for several monitoring projects, especially when information has to be recorded in real time avoiding further data processing. For a complete review the reader is referred to Maas and Hampel [17] and Lynch and Loh [18].

3 LASER TRACKERS FOR STRUCTURE MONITORING

Although the tools briefly presented in the previous paragraph can be considered as ‘proven techniques’ for structure monitoring (a robust and established sensor technology), the laser tracking applications in the field of civil engineering are still limited [19], and their potential and accuracy have to be verified. Indeed, laser trackers are mainly used in industrial applications. These applications include: inspection and alignment, machine calibration and testing, computer-aided design (CAD)-based inspections, dimensional analysis, prototyping, measurements on aircraft or boats, surface inspections, verification of the design of manufactured structures, object tracking, reverse engineering, and other applications [20]. It should also be noted that laser trackers are very expensive instruments. The price of the sensor, reflectors,

FIGURE 1
Photographs of some laser trackers today available on the commercial market.

and other accessories (tripods, laptop and software for data acquisition and processing, etc.) can amount to £160,000. Cost is, therefore, a parameter of primary importance for planning future activities that are either more industrial or innovative types of research in other fields.

The laser tracker [21], or the tracking laser interferometer (see Figure 1), combines the displacement measurement accuracy of a laser interferometer with the three-dimensional angular measuring capability of a theodolite. It measures the 3-D coordinates of specific points that are materialized by special reflectors. Basically, a laser tracker measures two angles (vertical and horizontal) and a distance. The position of a target is given in spherical coordinates (d, θ, φ) that are then converted into Cartesian coordinates with simple relations (polar to Cartesian conversion).

Optical encoders on the mechanism provide azimuth, θ and elevation, φ, angles of a beam-steering mirror. The radial distance, d, typically can be measured in two ways: by an interferometer (IFM) or an absolute distance meter (ADM). The IFM or ADM laser beams are emitted by the instrument tracking the target [22], which is usually a spherically mounted retro-reflector (SMR). After the laser beam is reflected, it again reaches the tracker along the same path that it followed originally. SMRs consist of a special reflector made carefully so that the apex of the mirrors coincides with the centre of curvature of a precise tooling ball. These tracker reflectors then provide a well-defined interface between the optical measurement from the tracker and the mechanical system that is being measured. The identification of target centres is very precise due to the sensor's capability of detecting the shift between the original pulse and the reflected one.

The distance can be measured in an incremental or absolute modality. Incremental distance measurement is made with an interferometer (IFR) and a frequency-stabilized HeNe laser. Linear interferometers are standard industrial measurement tools; they work based on the principle of light interference. In a standard Michelson interferometer, a coherent light source is split into two beams. One beam is used as a reference, while the other beam is reflected back from a mirror or retro-reflector. The second beam is then merged with the reference beam, producing interference. The interference fringes are counted, as the external path length changes. Because the wavelength of the laser is known and is also highly stable, the distance can be calculated from the number of fringes. These devices are restricted to linear measurements. A laser tracker overcomes this limitation by using a beam-steering mirror to direct the laser beam in a wide range of directions. In particular, the laser beam is split into: (i) a component emitted by the instrument and reflected by the SMR; and (ii) another one that is directly sent into the interferometer. When the reflected component reaches the instrument, it is sent into the interferometer and interferes with the other component. With the interferometric measurement, a high accuracy in the distance measurement is obtained.

IFR is generally used to track a moving target. In order to allow the laser beam to follow the movement of a retro-reflective target, a feedback loop procedure is used. When the laser beam hits the retro-reflective target off-centre, it is reflected back parallel to the incident beam but is displaced (shifted). A two-dimensional sensor measures the displacement, allowing the laser tracker to adjust the beam-steering mirror to return the beam to its desired coaxial direction. When the beam hits the centre of the target, it returns without displacement, indicating that the beam has hit the correct location. Operating with the IFR requires the SMR to be locked onto by the laser tracker to establish its initial location. This is usually accomplished by starting with the SMR in the 'home', which is a known reference point on the body of the laser tracker. Once the initial SMR location is established, the SMR can be carefully moved to another location and ensures that the laser tracker stays locked onto the SMR. The interferometer will calculate the change in position. However, if the laser beam between the laser tracker and the SMR is lost, then the SMR home position must be re-established. This is the main limitation of using the DMI mode.

The absolute distance measurement (ADM) system independently measures the absolute distance to any unknown reflector location in an automatic way without the need of continuous contact between the laser beam and SMR. In a tracker with ADM, infrared light from a semiconductor laser is emitted, is once reflected from the SMR and re-enters the tracker. Then, an electronic circuitry analyses the signal to determine the phase of the retuned light. By gradually reducing the modulation frequency, the absolute distance of the SMR target can be determined with a high degree of accuracy, such as is found in other electro-optical distance meters (EDMs) adopted for total stations. There are different known modulation methods that superimpose this periodic structure on the measurement beam, such as modulating the linear polarization of the laser light. The high resolution and accuracy of the ADM is generally obtained by using the Fizeau principle. The key elements are that the measurements are always made at the same phase positions and the phase detection is based on a differentiation method. Therefore, the laser emitter must be able to set the modulation frequency in very small and accurate steps.

The main advantage of ADM measurement over incremental distance measurement is the ability to simply point the laser beam at the target and make the measurement; however, lower accuracy in the distance measurement is expected. ADM measurement can be used either for reinitializing the interferometer (IFM) or as a stand-alone device. In particular, ADM can be effectively used for measuring the relative position of two or more mechanical components, re-positioning them and re-measuring. In these applications, the combination of ADM and IFR can give very accurate measurement of the target displacements (when the displacements are sufficiently small). Indeed, first targets are placed at every location to be checked, and their positions are measured with ADM. After repositioning the points, a suitable search routine can be implemented to use ADM distances to re-initialize the interferometer in order to measure the new reflector coordinates in a very accurate way and determine displacement.

Because a laser beam is used to calculate d between the laser tracker and SMR, the refractive index of the air, n, through which the laser beam is propagating must be accurately known, as well as the wavelength of the laser light, λ. These values are related by

$$d = N\left(\frac{\lambda}{n}\right) \tag{1}$$

where N is the number of wavelengths of light measured by the IFR. Changes in either λ or n will affect the measurements and provide sources of error for the measurements. For the actual laser tracker λ is generally very stable, the drift in the laser wavelength, $\Delta\lambda$, being up to $\Delta\lambda \sim 0.01$ ppm/24 hours; consequently, $\Delta\lambda$ is considered to be small enough to be negligible. This is not the case for n as it is a function of temperature, humidity and barometric pressure in the measurement environment [23]. To take into account the variation of n, laser trackers include a weather station to measure temperature, pressure, and humidity and allow calculation of n and post-correction of measurement results to a standard temperature, often 20°C. It should be noted that the refraction correction is not new in geodetic measurements; for instance, geometric levelling from the middle allows for a removal of errors by using reading differences, whereas trigonometric levelling (similar to measurement taken with a laser tracker) by means of total stations and electro-magnetic distance measurement (EDM) is corrected by using the temperature gradient and atmospheric pressure. This causes bending of the line of sight between sensors and reflectors. If the refraction in a vertical plane is given by an arc of constant curvature (simplified case for short distances), then the effect of refraction can be modelled by using a single parameter k, the refraction constant and gives a vertical correction, V_c:

$$V_c = \frac{ks^2}{2R} \tag{2}$$

where s is the horizontal sensor-target distance and R is the radius of the Earth. In the case of laser tracker systems, temperature and pressure sensors are fundamental to measure the correct wavelength of light. Calibration is therefore a very important task and can be carried out in a metrology laboratory (with very stable environmental conditions) or in-shop in order to obtain optimal measurements during the survey.

Laser trackers are generally equipped with *ad hoc* software for automatic measurement and data analysis. Indeed, these software packages can add a significant level of automation to the measuring processes in the case of repetitive tasks, as routines for planned measurements can be implemented. This is, for example, the case for monitoring the displacements of some points at regular intervals during the day or after some important events. In most cases, these packages also allow viewing of real-time measurements directly on a personal computer *via* wireless Internet connection and remotely controlling the laser trackers. The system provides real-time 3-D coordinates that can be immediately used for the specific application. If compared to total station data, a single station becomes sufficient to obtain sub-millimetre precision. The survey is therefore faster.

Specifically designed software is provided for data analysis such as corrections for ambient air conditions, angular scale errors, and misalignment of the beam steering mechanism. Laser tracker acquisition software relies on a model that describes the beam steering mechanism and its errors, allowing for correction. The parameters of the model are usually derived from a combination of calibrations performed at the factory. Other error parameters, particularly those related to angular scale errors, are considered as fixed at the time of manufacture and cannot be updated by the end user.

After these corrections are applied, measurements are performed from different stations in an instrumental reference system (polar coordinates). Cartesian coordinates in the instrumental reference system can also be simply derived. When measurements of the same points are performed from different stations, a bundle adjustment technique is generally used to derive instrument and point positions with a procedure which minimize the sum of the squares of measurement residuals (least squares adjustment). In the case of laser trackers, this situation generally occurs when measuring large structures with occlusions between points (i.e., presence of obstacles along the line-of-sight). In these cases, the measurements from different positions around the object are combined to obtain a complete model of the object.

Measurements from the different positions can be collected either by operating several trackers at the same time or by moving one tracker from each position to the next one.

When moving from a position to the next one, a set of points should be visible from both positions in order to solve for the new position of the laser tracker. Commercial software packages implement an adjustment for these situations. Redundancy, because more points than those necessary are available, allows for the computation of residuals, which, in turn, can be used to verify the assumed accuracy of the measurements. Commercial software has additional control/analysis utilities for correcting the target location for various offsets within the instrument and alignment features of the target (radius of the SMR) to indicate the resulting point of contact with the item being measured. Other tools allow for an effective interoperability with CAD and modelling software.

4 MEASURING BEAM DEFLECTION WITH LASER TRACKING TECHNOLOGY

This section illustrates two case studies in the field of civil engineering that were implemented under very different environmental conditions. In both cases, the aim was the direct measurement of beam deflections (Z-variations) for controlled (First test) and variable experimental conditions (Second test). In both cases, the tracker provided satisfactory results, which means the requests of structural engineers were fulfilled.

4.1 First test

The first test aimed to determine the intrinsic characteristics of a long beam during a laboratory inspection in which a progressive load was applied under controlled conditions. The laboratory is a big shed with a constant temperature of about 26°C. The beam size is approximately $17.0 \times 0.7 \times 2.5\,\text{m}^3$ and the weight is 109.45 kN (see Figure 2).

The laser tracker (AT901-Long Range; Leica Geosystems AG) shown in Figure 3 was used to measure the movements during the analysis. This instrument is able to work in the range of temperature between 0 and 40°C and humidity between 10.00 and 90.00%. The amount of time for system initialization (warm-up) is about five to eight minutes and depends on environmental conditions. The weight is about 22 kg, and the instrument is $620 \times 290 \times 240\,\text{mm}^3$. The load consisted of a series of metal bars with a weight of about 20.36 kN and a length of 12 m. The progressive distribution of all bars during the most predominant phases is shown in Figure 4; it is clear that a symmetrical scheme was adopted for the loading phase. The total weight after the positioning phase was 388.91 kN, while the theoretical weight used in the mathematical model was 391.79 kN. The small difference

FIGURE 2
Photographs of the beam at the beginning of the test and the final collapse.

FIGURE 3
Photographs of the laser tracker used and the progressive load applied by means of metal bars.

(+0.74%) achieved did not significantly influence the result, i.e. the comparison between the theoretical model.

As the objective of the laboratory testing on construction materials (for civil engineering applications) is the estimation of the characteristics of the analysed elements under different loading conditions, the laser tracker was used to study the deformation emerging during the loading period. In this experiment points were physically materialized by means of magnetic supports, in which the red-ring reflector (RRR) 1.5” was placed. Overall, 11 points were directly fixed on the structure with a homogeneous distribution, and an additional point (say Point 12) was placed on an external position in order to obtain a stable point to check the stability of the reference system (see Figure 5).

The tracker was programmed to measure the displacements in a 3-D reference system; therefore, movements were detected along the vertical direction (z-axis) and x- and y-axis components (longitudinal axis of the beam and orthogonal direction, respectively) were acquired. This a fundamental difference with respect to standard tools that were used in these experiments, such as LVDTs. LVDTs provide the magnitude of displacement through the mea-

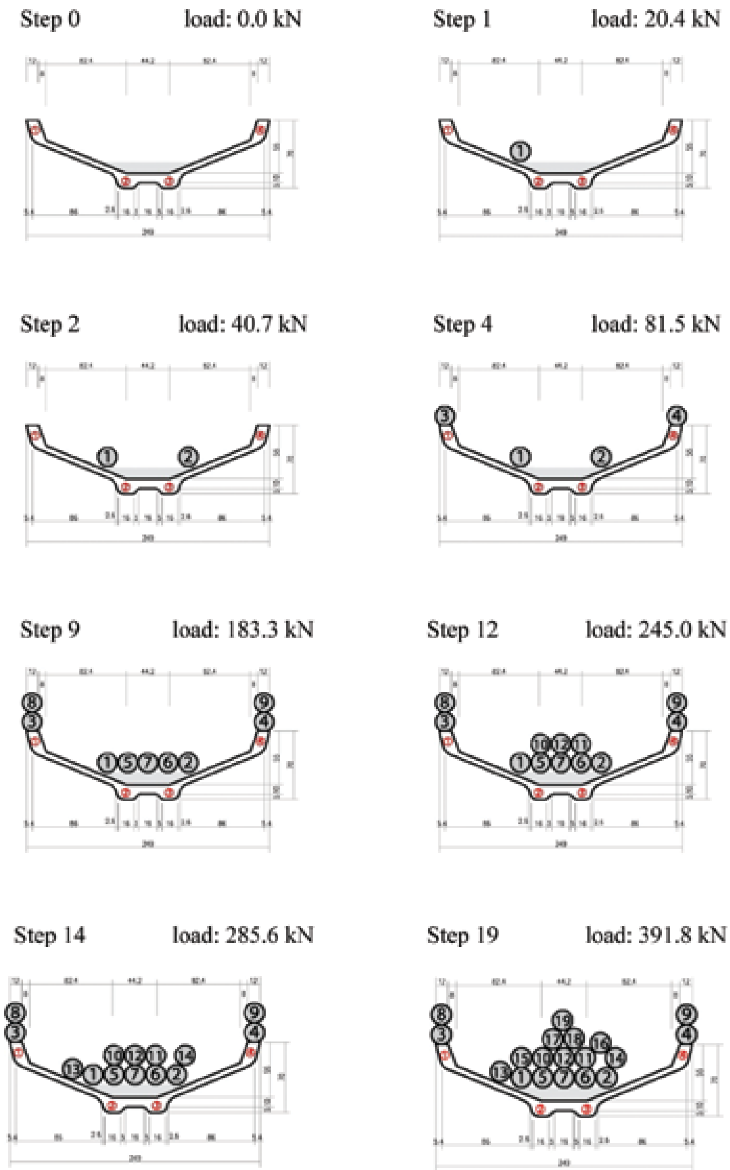

FIGURE 4
The progressive distribution of the bars on the beam.

surement of electrical resistance variation due to load. These tools are considered to be proven techniques with micrometric accuracy and real-time data processing. On the other hand, the tools are capable of measuring only 1-D deformations. In addition, a cable connection to a data acquisition unit is necessary, and in the case of destructive tests, the sensor can be easily dam-

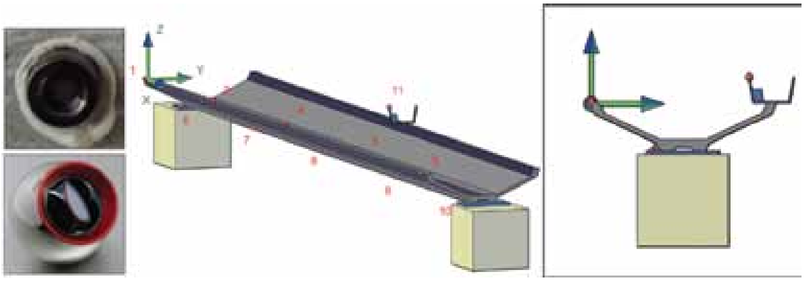

FIGURE 5
Photographs and diagrams of the reflector used along with a scheme with point distribution.

FIGURE 6
Photographs of the RRR reflector and some LVDTs. They were installed close each other in order to inspect the same part of the beam.

aged. The location of some of the standard sensors in this experiment is shown in Figure 6. As shown, laser reflectors and LVDTs were placed in close proximity, although it is physically impossible to determine the movement of the same point.

The comparison between the data acquired with different sensors was made by using the vertical movements (LVDTs are placed with the longitudinal axis along the vertical direction). Data are available at different epochs, corresponding to different loading conditions. One of the main advantages of the tracker is the opportunity to capture 3-D movements, whereas linear sensors can measure only the variation along the sensor axis. The results for the points on top and bottom of the beam, acquired at different epochs, are illustrated in Figure 7. The knowledge of this additional information gives planar coordinates useful data to understand the rotation of the beam during its inspection.

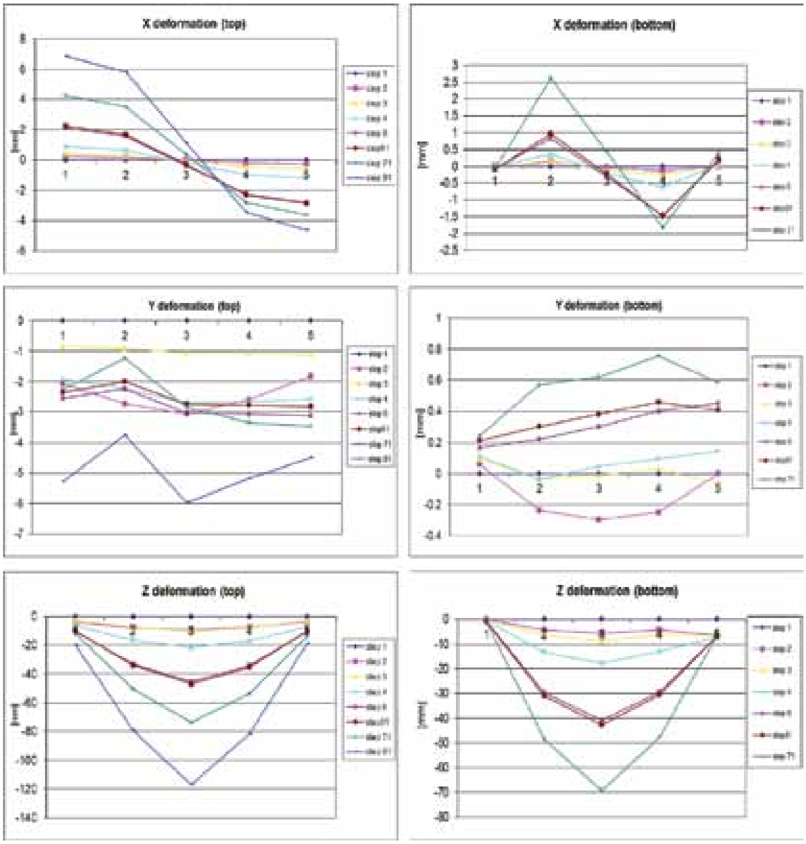

FIGURE 7
Graphs of the displacements measured by the laser tracker for the different *x*-, *y*- and *z*-coordinate components. These data cannot be measured with traditional sensors.

A graphic visualization of the comparison in the vertical direction is shown Figure 8. As shown, there is a very good consistency between points close to the supports, and a discrepancy can be found in the middle. It is important to mention that linear sensors and the tracker did not measure the same quantities; the laser captured *Z*-variations, whereas the linear sensor can be affected by a rotation, which means that its measurements are referred to with a variable reference system. From this point of view, the 3-D information given by laser-tracking technology can give more information about the deflection.

Another interesting result was found for the two points close to beam supports - Point 6 and Point 10, about 10 cm from the metal support. The movement of these points was expected to be zero, but a different result was found for: (i) the collapse of the concrete support for extreme loads; (ii) the deformation of the metal plate where the reflector was placed; and (iii) a rotation

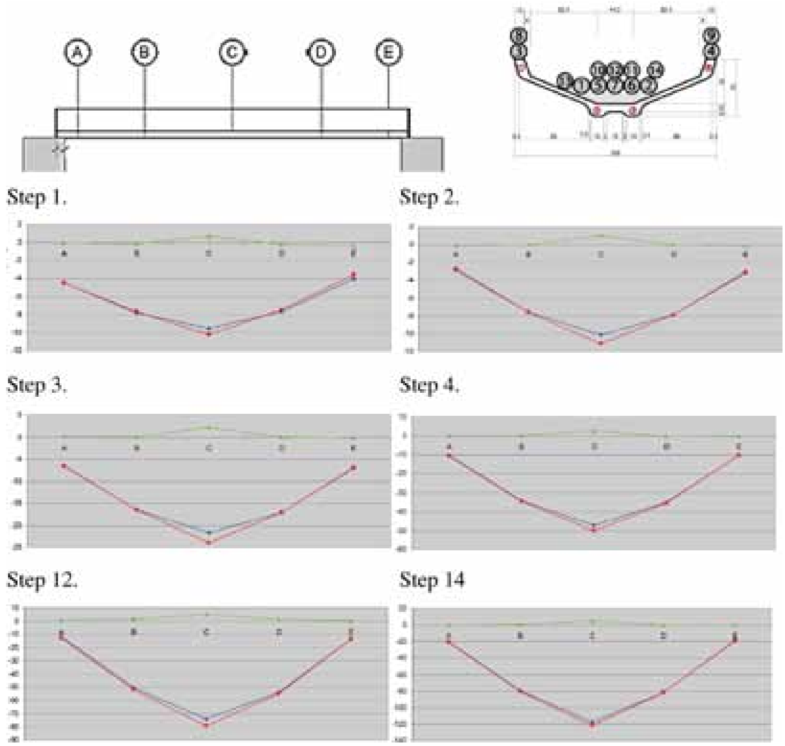

FIGURE 8
The comparison (in mm) between LVDT (red) and tracker (blue) data for some of the most significant load conditions. The green line represents the difference between the measured values. As can be seen, the trend is very similar for points close to the supports, whereas in the middle the rotation of the beam gives a discrepancy between the measurements (sensors measures different quantities).

of the metal plate on the corner of the support. For Point 6 this displacement was quite limited (see Figure 9); however, Point 10 had a sudden movement for the local collapse of the concrete, which remained quite stable for the experiment.

4.2 Second test

The objective of the second experiment was instead the assessment of a new bridge in Parma, Italy. The basic purpose of this analysis was to determine if the bridge was able to carry the traffic with adequate margins of safety. This application includes a pedestrian bridge and another vehicular bridge belonging to the same structure (see Figure 10). The adequacy of the bridge had therefore to be determined during (simulated) extreme load conditions, in which real measurements were compared to the theoretical model to discover hidden strengths and weaknesses.

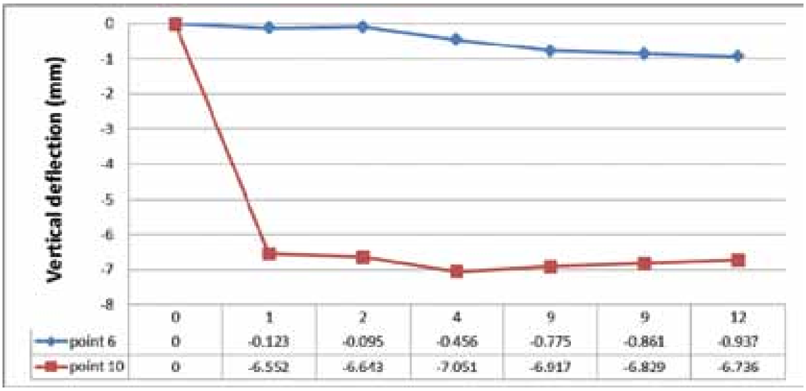

FIGURE 9
Graph of the vertical movement for the points close to beam supports.

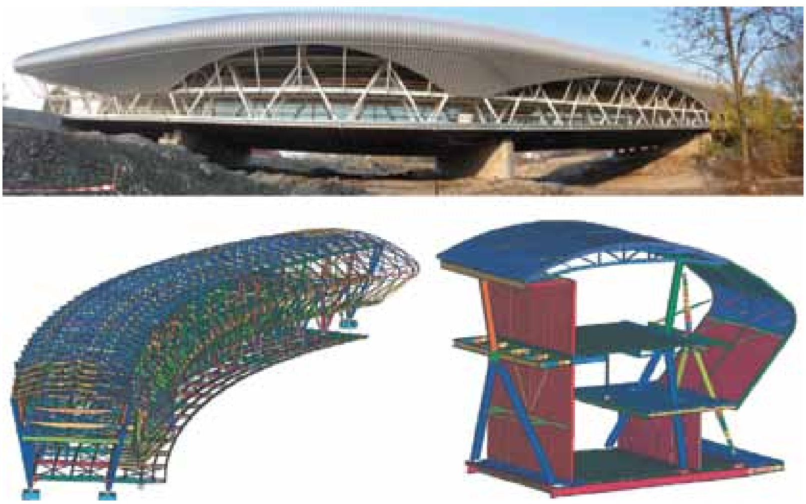

FIGURE 10
A panoramic view of the bridge in Parma and the scheme of the structure.

Different instruments were used to detect the structure movement: a laser tracker (AT 401; Leica Geosystems AG) was placed under the bridge, and a total station (TS30; Leica Geosystems AG) was mounted on a stable pillar in front of the bridge. This is a very important point and demonstrates that a single technique is not sufficient for a complex case like the one illustrated in this section. It is important to acknowledge that this inspection was a real civil engineering application that was meant to determine the behaviour of the new structure and was not only a test for the tracker. For this reason, the use of several instruments and the comparison of the obtained data become essential. This test was also very interesting to prove the potential of the tracker with bad environmental conditions for repetitive measurements (about three days).

The tracker and total station were needed to reach 28 prisms installed under the bridge (see Figure 11), while spirit levelling provided the vertical variations for the vehicular bridge. These additional benchmarks were installed on the bridge for some specific locations that were difficult to reach with both laser tracker and total station. The analysis was carried out only to detect the movements during the applied loading conditions. The thermal characterization of the structure was not needed because the movements checked (without external load) during a whole day were about 0.10 mm; that is, smaller than the expected movements during the test.

Some points were measured by using different techniques in order to obtain a cross-evaluation of the achieved results. This was feasible, as the reflectors were installed on specific supports that were able to carry different kinds of targets. For the case of spirit levelling, particular rods were used. This analysis is surely more complete than the analysis typically obtained with distometers (or other sensors able to measure only the vertical deflection) placed under the bridge.

The location of the tracker and total station are shown in Figure 12 (Point C and Point B, respectively). In general, the automation provided by the tracker was noteworthy, as the instrument is extremely rapid and fully automated, especially for applications where the same measurements must be taken at different epochs with different loading conditions. After setting the program for the initial phase, the sensor can automatically repeat the same operations (in some cases for 10 different loading conditions *per* day) without manual measurements. Only the initial set up was needed (displacements

FIGURE 11
The equipment for this monitoring application: A benchmark for spirit level here a rod and a prism can be installed together, the installation of the reflector under the bridge, the total station TS30 on a pillar and laser tracker AT 401 under the bridge, some prisms under the bridge (in all 28 different reflectors).

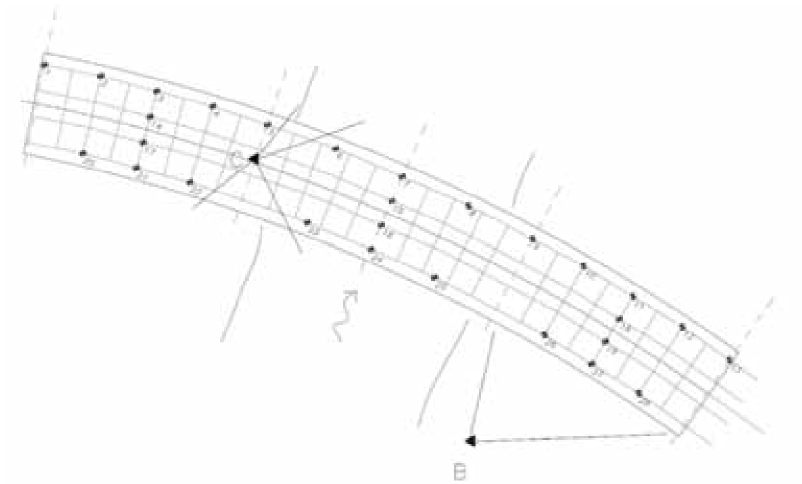

FIGURE 12
Work flow chart of the vehicular bridge where the laser tracker (Point C) and total station (Point B) where simultaneously used.

are elastic in the range of few centimetres). Then, the sensor completed the different loading epochs in a fully automated way. The precision was better than ±0.20 to ±0.30 mm (this result was determined by repeated measurements of the same point), which is an acceptable value for such applications that are carried out in adverse environmental conditions. In fact, the temperature during the day (in December) varied from 2 to 8°C.

A typical result for a loading configuration during the inspection is shown in Figure 13. In this case, the load was applied on the left part of the bridge (first span). The expected deformation was confirmed by real data; there was a large deformation for the first span, an opposite behaviour for the second one, and small displacements for the last one.

5 VERTICAL DISPLACEMENT MEASUREMENT VIA LASER TRACKING TECHNOLOGY

As mentioned in the state-of-the-art, the estimation of heights or height changes can be made with different techniques. In the field of structure monitoring, these techniques include spirit (geometric), trigonometric, barometric, mechanical and hydrostatic levelling, 3-D traversing, as well as GNSS systems and gravity data.

Spirit levelling is still undoubtedly the most used method for vertical structure monitoring, due to its limited cost (level, rods, tripod, and a set of benchmarks are the only equipment needed) and its sub-millimetre accuracy.

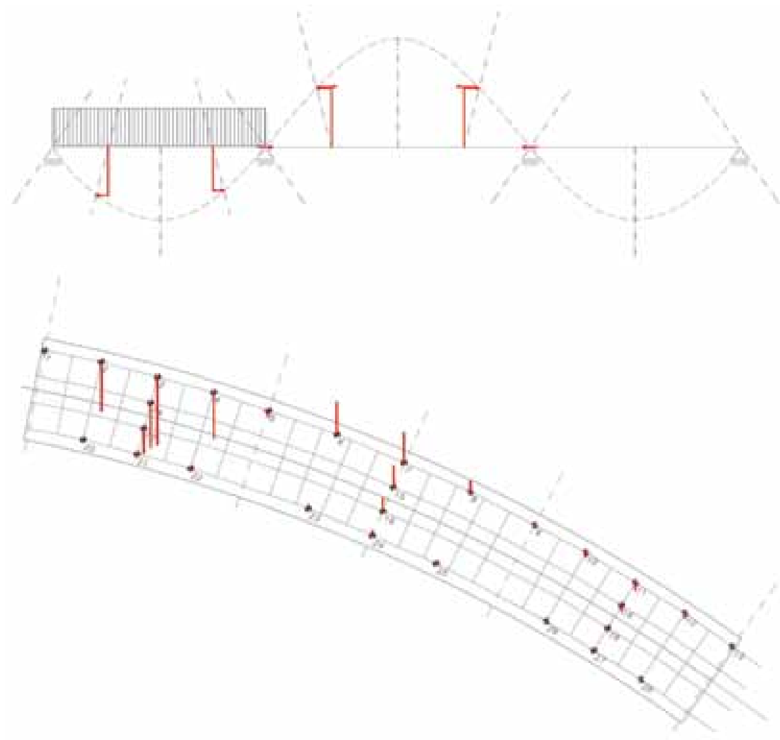

FIGURE 13
A load condition and the measured displacements.

The main goal is the estimation of the height differences between different points by means of an optical or digital level (that is set up on a tripod and provides a horizontal line of sight) and two vertical graduated rods.

According to the basic principle of levelling, the difference between two readings is the height difference:

$$\Delta_{i,i+1} = L_{i+1} - L_i \tag{3}$$

The process is repeated to obtain the height difference between the back sight and foresight:

$$\Delta_{i+1,i+2} = L_{i+2} - L_{i+1i} \tag{4}$$

so the total height difference between widely separated points can be measured by combining the height differences of all the intermediate points. Obviously, redundant measurement schemes are adjusted *via* standard least squares method.

As a measurement campaign is based on the progressive acquisition of several height differences, the errors should be reduced to a minimum to avoid a progressive accumulation. The technical literature reports several experiences of and possible solutions to this problem. Therefore geometric levelling can be assumed to be a proven technique, not only for land surveying, but also for structure monitoring. It is also noteworthy that most errors can be reduced by taking a series of *ad hoc* readings (backwards-forwards-forwards-backwards) from the centre when the line of sight is not horizontal; for example: (i) to correct the errors due to symmetric atmospheric refraction; (ii) to compensate for the Earth's curvature; and (iii) to remove the collimation errors. Other sources of errors, which had effects that were extremely significant in the past, were eliminated with the introduction of more sophisticated instruments.

The design of an appropriate measurement scheme, coupled with precise measurements, allows for the determination of heights (and height changes for data taken at different epochs) with sub-millimetre precision. The scheme of the network has a direct impact on the precision: a series of closed loops with common points must be preferred to: (i) improve the accuracy and (ii) to obtain an immediate check, based on misclosures.

The input data of a project are the differences in elevation, $\Delta_{i,j}$, and the elevations, H_b, of some benchmarks (at least one) that are considered to be fixed points. The linear observation equations give a configuration matrix $\mathbf{A}$ made up of zero and ± 1. The problem is in the form

$$\mathbf{A}\hat{\mathbf{x}} = \mathbf{b} + \hat{\mathbf{v}} \tag{5}$$

with

$$\hat{\mathbf{v}}^T \hat{\mathbf{v}} = \min \tag{6}$$

and the least squares estimate is

$$\hat{\mathbf{x}} = (\mathbf{A}^T\mathbf{A})^{-1}\, \mathbf{A}^T\mathbf{b} \tag{7}$$

assuming the same precision for all differences in elevation. The residual vector is

$$\hat{\mathbf{v}} = \mathbf{A}\hat{\mathbf{x}} - \mathbf{b} \tag{8}$$

and the unit weight variance, σ_0^2, is

$$\sigma_0^2 = \hat{\mathbf{v}}^T \hat{\mathbf{v}} / r \tag{9}$$

where r is the redundancy. Finally, the covariance matrix of parameters (x) and residuals (v) can be estimated as

$$\mathbf{C}_{xx} = \sigma_0^2 (\mathbf{A}^T\mathbf{A})^{-1} \tag{10}$$

and

$$\mathbf{C}_{vv} = \sigma_0^2 (\mathbf{I} - \mathbf{A}(\mathbf{A}^T\mathbf{A})^{-1}\mathbf{A}^T) \tag{11}$$

In the case of structure monitoring, geometric levelling provides the vertical displacements of a series of benchmarks; that is, points that are well-tied to the structure. Although very accurate, geometric levelling is quite slow, with productivity limited to a small number of points *per* hour. In some applications that aim to determine the safety conditions of a structure, time becomes an essential factor and limits the number of points that can be checked. From this point of view, the use of a laser tracker for detecting vertical movements is very attractive. Obviously, it is fundamental to check if this instrument can replace standard spirit levelling. A traditional benchmark can be substituted with special supports for retro-reflectors that allow the correct repositioning at different epochs. The precision achievable with the laser tracker is quite similar to that of a level, and experiments aimed at determining the feasibility of the method are not completely new [24], notwithstanding there is no real practical application of this technique at the present.

One of the problems that should be considered is the Earth's curvature. In fact, the tracker measures coordinates in a Cartesian reference system. The system is 'in plumb' on the station point. Therefore, the instrumental z-axis coincides with the vertical line in the station point. Geometric levelling, with readings that are collected from the middle, can instead remove this effect. It is well known that the effect of the Earth's curvature depends on the distance according to the relationship

$$h = \frac{s^2}{2R} \tag{12}$$

where h is the vertical error and s is the laser-reflector distance. Because of the high precision of a laser tracker, the effect on short distances has to be taken into consideration (in Figure 14 the error for a radius $R = 6378$ km is shown).

The laser tracker was tested inside the Cathedral of Milan (il Duomo di Milano) in order to detect the static movement of the columns. In this test a spirit levelling network is physically materialized, and several monitoring campaigns were implemented. The precision achievable after the least squares adjustment is about ±0.10 mm and measurements are periodically

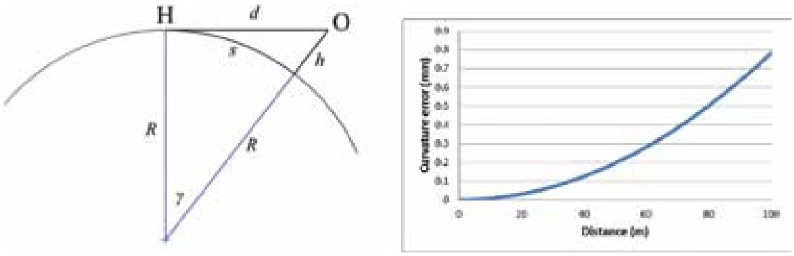

FIGURE 14
The VERTICALITY error *h* for Earth curvature as a function of the laser-reflector distance *s*.

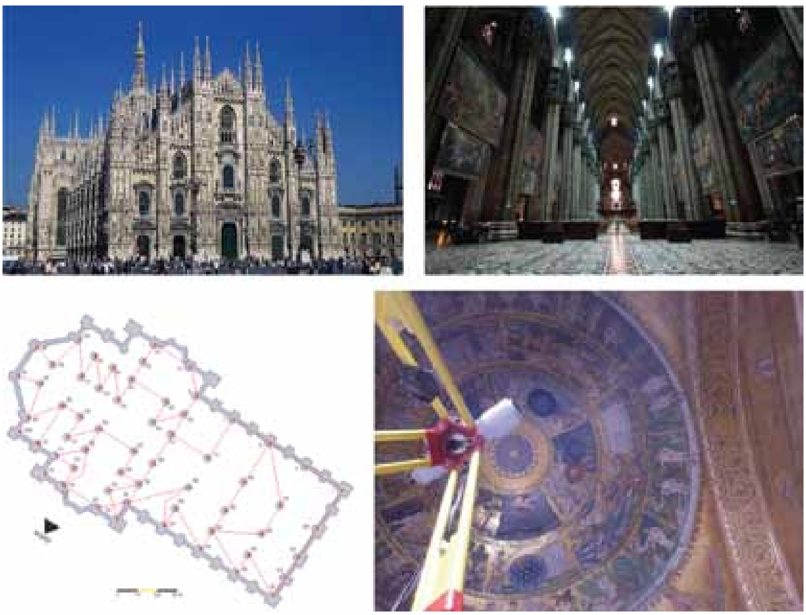

FIGURE 15
The Cathedral of Milan, the original levelling scheme and the optical level used.

carried out to inspect vertical movements. A single measurement campaign can be completed within a working day. Measurements are carried out with an optical level (Ni1; Carl Zeiss AG) and 5.00 mm graduated rods placed on benchmarks fixed on the columns. The complete levelling network is shown in Figure 15.

The inspection of vertical movements was also carried out with the AT 401 laser tracker. In this case, the instrument was placed on a stable metal tripod, which has a position that is fixed (see Figure 16). This allows for a stable repositioning of the instrument for measurements taken at different epochs (the legs are fixed). Then, a sub-portion of the global network was measured

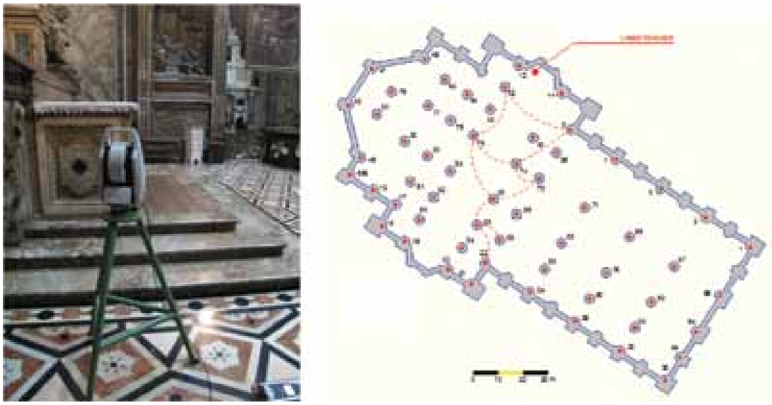

FIGURE 16
The laser tracker inside the Cathedral of Milan and the new levelling scheme for laser tracker data.

with the laser tracker in order to get the coordinates of the benchmarks and perform a comparison.

Spirit levelling was repeated for only these points, and after the least squares adjustment, the height of the benchmarks (nine in this case) was estimated with a precision better than ±0.10 mm. This is consistent with the instrument used and network geometry. In particular, the least squares adjustment provided a sigma-naught of ±55 μm and the height precision of the order of ±0.10 mm (number of observations is 10, number of unknowns is eight). A synthesis of the achieved results is reported in Table 2 and Table 3. The benchmarks were also measured with the laser tracker by placing its spherical reflector in the spherical benchmark of the levelling rod. In this

TABLE 2
Results after LS adjustment for the network inside the Cathedral of Milan.

Point	Height (cm)	Precision (cm)
9 (fixed benchmark)	0.00000	0
22	4.70080	0.011640
42	–0.15627	0.004752
55	–2.40320	0.008597
56	–0.03168	0.010232
73	–2.94590	0.006408
74	1.40830	0.004752
75	5.03640	0.005361
85	–0.27293	0.006566

TABLE 3
Least Squares residuals for the network inside the Cathedral of Milan.

From	To	Residual (cm)	Standardized residual
42	9	–0.00048	–0.16867
9	74	–0.00048	–0.16867
74	75	0.003533	1.00680
75	42	–0.00048	–0.16867
75	85	0.004017	1.40170
85	73	0.004017	1.40170
73	74	0.004017	1.40170
85	55	8.88E-16	Infinity
56	22	0.00000	0.00000
55	56	0.00000	NaN

case, it is important to clarify that a hand-held reflector was used. In fact, the spherical reflector can be placed in the spherical benchmark, removing re-positioning errors.

Two dataset measurements were acquired, and the reflector was placed by two different operators. First, the distances $d_{i,j}$ between the targets for the two datasets should remain invariant, notwithstanding the different reference systems that are employed. This comparison was made by checking all distance combinations $d_{i,j}$ ($i \neq j$) and obtaining an average of 0.09 mm and a standard deviation of the differences of ±0.19 mm. The differences in values, which are representative of the precision of the inter-distances at different epochs, are shown in Figure 17.

Another common problem is the registration of laser points (from multiple stations) into a common reference system. As mentioned, laser tracker technology produces point coordinates (x-, y- and z-coordinates) by using range d and angular information horizontal direction θ and vertical angle α, that are subsequently transformed into Cartesian coordinates. Cartesian coordinates are referenced to with the intrinsic reference system (IRS) and are therefore determined in an instrumental system. To capture the geometry of a complex object, multiple points are normally needed. This means that multiple stations must be aligned into a common reference frame. This operation is called 'registration' and requires the estimation of a six-parameter transformation:

$$\mathbf{X}_2 = \mathbf{R}\mathbf{X}_1 + \mathbf{T} \qquad (13)$$

where $\mathbf{R}$ is a 3×3 rotation matrix and $\mathbf{T}$ is a 3×1 translation vector. Given a sufficient number of 3-D point correspondences (at least two), the parameters

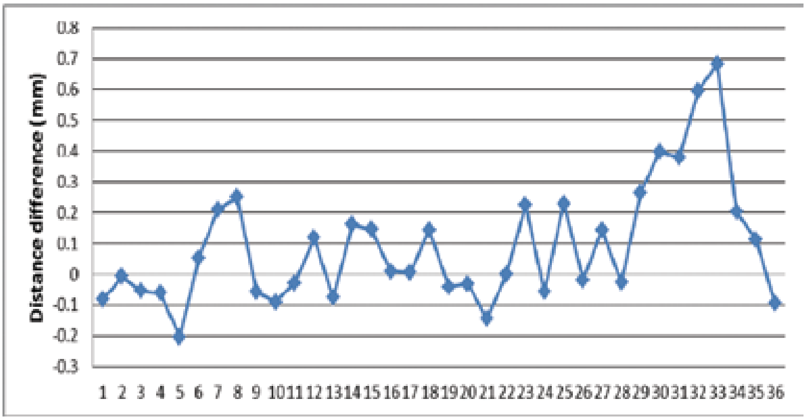

FIGURE 17
Graph showing the comparison between inter-distances (in mm) for the two laser tracker dataset.

can be estimated. The set of corresponding points $\mathbf{X}_1 \leftrightarrow \mathbf{X}_2$ must have a good distribution in the space to efficiently estimate the unknown values. If the results of a survey must be transformed into a ground reference system (GRS); that is, a predefined Cartesian reference system, then the operation mapping of each station into the GRS is called ‘georeferencing’. In fact, in many cases all of the tracker locations are first mutually registered between them, and eventually, the resulting final point structure must be georeferenced in a unique final step (see Figure 18) with a global adjustment. Disregarding the method that is adopted for registration, most of them work using scan pairs, where: (i) all scans are pair wise organized according to their relative overlap or proximity; (ii) each pair is registered; and (iii) all pairs are ultimately concatenated together.

The least squares estimate of the roto-translation can be used to check the consistency of the *x*-, *y*- and *z*-coordinates that are acquired from the same location in this case. All points were used in the adjustment and provided a sigma-naught of about ±0.30 mm (the residuals for the different components are shown in Figure 19). It is also possible to reduce this transformation to a 2-D comparison, as the instrument is levelled with a high precision electronic bubble. The unknown parameters reduce to a single rotation - one angle instead of a 3-D rotation matrix - and a 2-D translation vector. This comparison provided a sigma-naught of 0.15 mm that is much better than the previous 3-D results (2-D residuals are shown in Figure 20). This can be intended as an indicator for a problem on the *z*-coordinates.

As the Point 9 was assumed as a fixed benchmark, it was interesting to compare the inter-distances between the fixed point and the remaining ones. The results are shown in Table 4 for the different datasets. As shown, the average value of the difference is almost zero (±0.01 mm), which means there is no systematic error, and the standard deviation is ±0.15 mm.

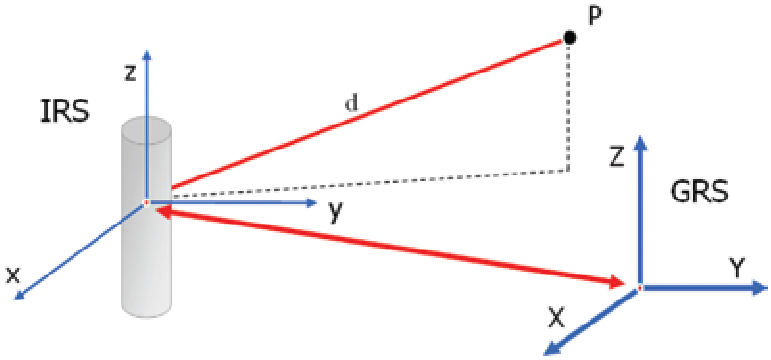

FIGURE 18
The use of a fixed reference system needs a roto-translation of laser points.

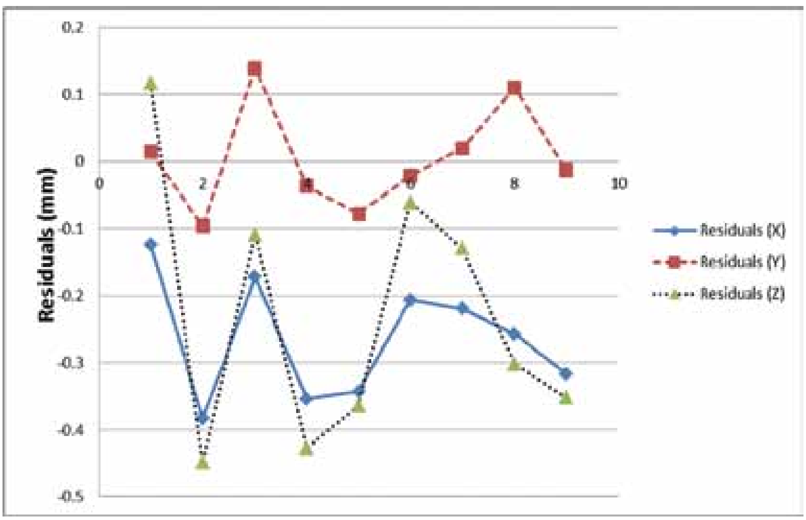

FIGURE 19
Graph showing the residuals of the 3-D least squares registration based on the whole dataset of common points.

The laser tracker data (height values) were then compared with the levelling information. The first laser dataset shows large discrepancies of about ±0.50 mm (standard deviation of the differences). In addition, there is a systematic error (the average is –0.70 mm). The second dataset is more accurate and has an average of 0.11 mm and standard deviation ±0.30 mm. It is important to mention that for the second experiment, the computed differences (accuracy evaluation) are less than 0.20 mm, except for two only points (0.40 and 0.80 mm, respectively). The final statistics carried out after removing

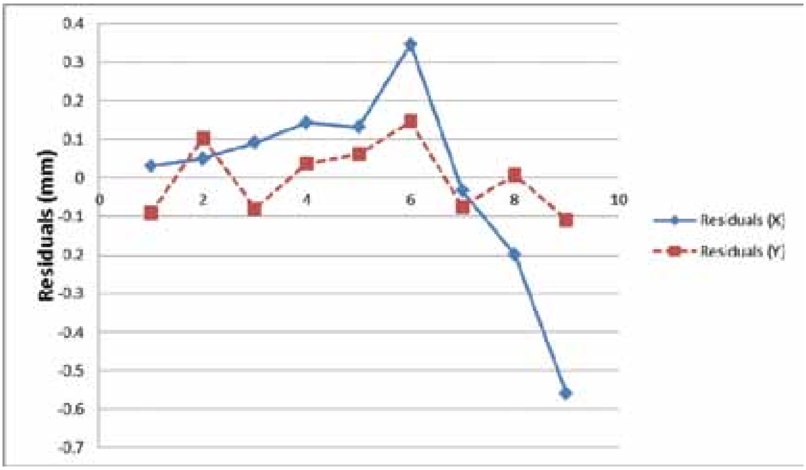

FIGURE 20
Graph showing the residuals of the 2-D registration based on the whole dataset of common points.

TABLE 4
Distance between point 9 (fixed) and the remaining ones for the different datasets.

Point	Dataset 1	Dataset 2	Difference (mm)
22	55887.89	55887.98	–0.08
42	26580.94	26580.95	–0.01
55	46781.51	46781.56	–0.05
56	47085.99	47086.05	–0.06
73	17593.52	17593.72	–0.20
74	18933.48	18933.42	0.05
75	32536.20	32535.99	0.21
85	38779.32	38779.07	0.25

these two points gave a standard deviation measurement of about ±0.15 mm, whereas the systematic error was removed (the final average 0.02 mm). These results are similar to those that were achievable on the optical level, and a visual comparison is shown in Figure 21.

The results of this experiment are not completely satisfactory, as some gross errors were found in both datasets, and some points were removed. Because different operators obtained different results, the use of a more stable reflector, instead of a hand-held sphere, will be investigated. Indeed, a stable reflector provided much better results notwithstanding the long sensor-target distance. A sphere was therefore placed at a distance of about 65 m (which is sufficient to cover a variable target distribution inside the Cathe-

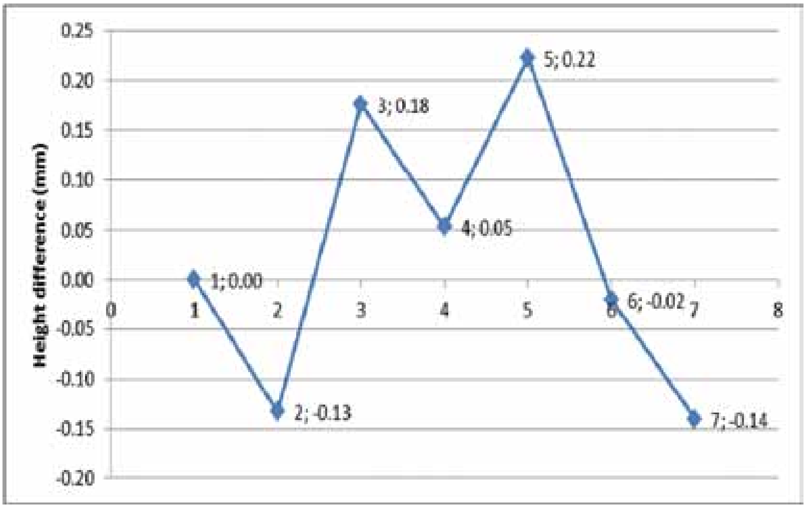

FIGURE 21
Graph showing the height difference between laser data (Dataset 2) and optical level.

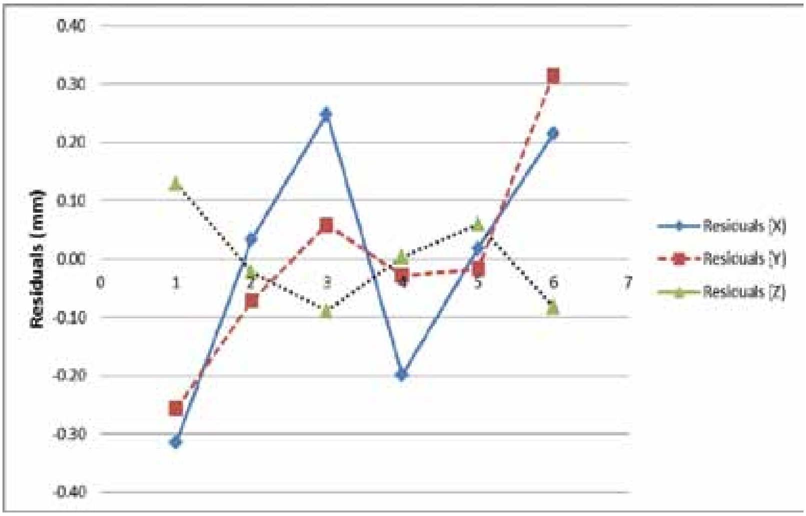

FIGURE 22
Graph showing the residual distribution for a long tracker-reflector distance (65 m).

dral) and was measured five times. Then, the obtained coordinates were compared to the average value for the different components, and the results are shown in Figure 22. The standard deviations were ±0.22, ±0.19 and ±0.08 mm, for the *x*-, *y*- and *z*-coordinates, respectively, and are consistent values for the long distance employed. This means that new exhaustive analysis is therefore needed and will be carried out by using a multi-epoch project; data will be collected in different seasons to check thermal variation movements in order to verify if the tracker can effectively substitute geometric levelling.

6 CONCLUSIONS

The use of laser tracking technology in civil engineering applications, such as structure monitoring, is very attractive due to the high precision that is achievable. Different case studies that were implemented with two trackers were illustrated and discussed. Many typical problems, such as bad weather conditions, unstable temperature, and humidity were considered in the proposed examples.

The first case study (beam deflection during material testing) proved the advantages of the system for experiments that were conducted with controlled and stable environmental conditions. The analysis of bridge movements was instead much more complicated with the bad weather conditions (low temperature). The last example implemented in the Cathedral of Milan will continue with a more stable setup of the reflector in order to determine the reasons behind some unexpected results.

The first results illustrated in this paper (that are quite innovative in the field of structure inspection and monitoring, as this sensor is relatively unknown) were quite satisfactory. After a preliminary warm-up of the sensor and the creation of an acquisition program, the tracker can take the measurements in a fully automated way. This is really important for monitoring applications at different epochs, as the time for data acquisition (one of the most important parameters in some cases) can be reduced. The tracker can also provide three-dimensional (3-D) movements. A good distribution of the reflectors can be a valid alternative to substitute some traditional sensors that are used to detect only the relative displacement of the object.

The achieved precision was sufficient for several civil engineering applications, notwithstanding some unexpected results for some points were also found. For this reason, more exhaustive analysis is needed to better understand the potential of this technology, which seems quite promising and attractive.

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