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DEVELOPING AND TESTING A METHOD FOR DEFORMATIONS MEASUREMENTS OF STRUCTURES

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Abstract. Dynamic monitoring of structures is an important task in civil engineering that aims to determine the stability and safety of a structure by using information about its deformations. This paper describes the development of a method for the determination of structures deformations. The proposed method is developed to add a new solution to traditional methods of angle intersection and trigonometric leveling. It is designed to provide a simultaneous solution to all observations in one step using least squares solution to improve the expected accuracy and to generate the necessary data for statistical analysis. A practical experiment was made, where the observations of 7 deformation points on a simply supported steel beam with concentrated load were measured using the proposed method, total station and linear variable displacement transducers (LVDTs). Deflections measured directly from LVDTs were used as a reference for assessment of the serviceability of the beam. The results show that for the maximum deflection at mid-span of the beam, the differences between the measured deflections from LVDTs and proposed method are less than 0.87 mm corresponding to an error of 4.3%, while they are less than 1.32 mm causing an error of 12.5% for the case of total station measurements. Based on root mean square error values, the accuracy of point displacements determination using the proposed method is much better than total station measurements. The proposed method is suitable for the accurate determination of horizontal and vertical displacements and provides a realistic solution for monitoring structures at both entire structure and member levels.

Keywords: angle intersection, deformation, monitoring, total station, theodolite.

Introduction

Dynamic structures monitoring is an important field of research that has great interest from government agencies for maintaining the safety of tunnels, buildings, dams and civil infrastructures.

Dynamic monitoring or deformation of an element can be defined as the variation of its position, size and shape with respect to its designed shape. The aim of measuring deformations is not only the determination of the exact locations of the observed element but also the variation of these positions with time. This is done to prevent the failure of large engineering structures.

Different surveying methods have been used in order to support the monitoring of the structures. However, the main purpose for the developed methods was to measure the displacements for a number of points. The difficulty in the displacements measurement is to find a 3D measurement method that satisfies numerous properties, such as reliability, precision, low cost and time consuming.

Several methods are available for accomplishing some of the above mentioned requirements, but it is difficult to find a method to meet all of them. Some of these methods are described as following (González-Aguilera *et al.* 2008):

- Topographic methods based on measuring angles, height variation and distances are commonly used in the surveying field. The instrument used consists of the different types of levels, theodolites, EDMs or total stations. For inaccessible points, indirect methods are used, for example: precise leveling traversing, single or multiple intersections (Ghilani, Wolf 2006), etc. Additionally, contact sensors can be used for these measurements (González-Aguilera *et al.* 2008; Lahamy *et al.* 2016), such as: an inclinometer, a pendulum, dials gauges or extensometers. However, this contact nature prevents them from use at the final stages of destructive load testing and they can only be used for acquiring observations in one dimension.

- The Global Positioning System (GPS) is used in structures monitoring with considerable range of displacements, as well as combined with other sensors (Breuer *et al.* 2002). GPS has two main limitations. Firstly, it cannot be used indoors or through above obstacles. Secondly, its current precision is limited to +/– 1 cm horizontally and +/– 2 cm vertically.
- Digital close-range photogrammetry is used to provide high accuracy (Detchev *et al.* 2016). It also offers a fast, remote, and spatial data acquisition with images that offer a permanent visual recording of the test. Using targets may not be suitable in some cases, especially when the access to the object cannot be reached or risky to operators. Measurements using additional equipments such as reflectorless total stations are necessary to determine the scale definition in the photogrammetric process.
- Terrestrial Laser Scanning (TLS) is a new methhod to the structures monitoring using novelty approaches and computation methods. The approaches noted above provide an accurate modeling strategy and have demonstrated their reliability for structural monitoring but they are not tested over complex structures such as bridges and high constructions. The reported analysis concentrates on two main problems: the first one is the accuracy and the stability of georeferencing, which is the base of making comparisons between different scans; the second one is the deformation computation based on the point-clouds. Generally, a comparison is performed using different surfaces types, such as: resample point cloud, mesh and polynomial surface (Herban 2009; El-Tokhey *et al.* 2013).

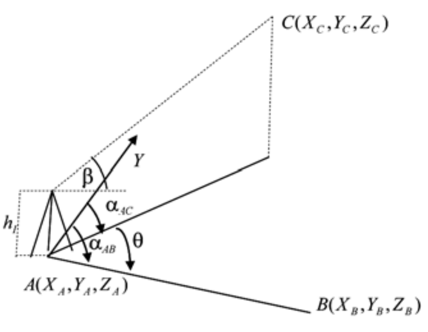

Fig 1. Observations to point C

The aims of the present research are (1) developing a proposed method depending on a mathematical model for computing the spatial coordinates from the measurements of separate monitoring points, (2) adjusting redundant measurements whose precision can be evaluated by a least squares adjustment, (3) employing the proposed method for the deformation monitoring of an I-shaped steel beam under different stages of loading, and (4) comparing the results of the proposed method to the results of total station technique.

1. Derivation of the mathematical model

Figure 1 illustrates the geometry for the determination of the unknown ground coordinates (X_C, Y_C, Z_C) of point *C*. In the figure, *A* is the instrument station and *B* is back sight station with known ground coordinates (X_A, Y_A, Z_A) and (X_B, Y_B, Z_B) respectively.

As shown in the Figure 1, a horizontal angle observation equation can be written as the difference between two azimuth observations, and thus for clockwise angles (Ghilani, Wolf 2006):

$$\theta = \alpha_{AB} - \alpha_{AC};$$

$$\theta + v_\theta = Tan^{-1}\frac{X_B - X_A}{Y_B - Y_A} - Tan^{-1}\frac{X_C - X_A}{Y_C - Y_A}, \quad (1)$$

where θ is the observed clockwise horizontal angle, v_θ is the residual in the observed horizontal angle, α_{AB} is the azimuth of line AB and α_{AC} is the azimuth of line AC.

Equation (1) is a nonlinear function of X_C, Y_C and Z_C only because control points coordinates are held fixed during the adjustment. Thus, Equation (1) can be rewritten as:

$$F(X_C, Y_C) = \theta + v_\theta, \quad (2)$$

where $F(X_C, Y_C) = Tan^{-1}\frac{X_B - X_A}{Y_B - Y_A} - Tan^{-1}\frac{X_C - X_A}{Y_C - Y_A}$.

The linearised first-order Taylor series expansion of Equation (2) can be expressed as:

$$F(X_C, Y_C) = F(X_C, Y_C)_o + \frac{\partial F}{\partial X_C}\Delta X_C + \frac{\partial F}{\partial Y_C}\Delta Y_C, \quad (3)$$

in which $\frac{\partial F}{\partial X_C}$ and $\frac{\partial F}{\partial Y_C}$ are the partial derivatives of F with respect to X_C and Y_C, respectively.

Evaluating partial derivatives of the function F and substituting into Equation (3), then substituting into Equation (2), results in the following equation:

$$v_\theta + \left(\frac{Y_C - Y_A}{AC^2}\right)\Delta X_C + \left(\frac{X_A - X_C}{AC^2}\right)\Delta Y_C =$$
$$Tan^{-1}\left(\frac{X_B - X_A}{Y_B - Y_A}\right)_o - Tan^{-1}\left(\frac{X_C - X_A}{Y_C - Y_A}\right)_o - \theta, \quad (4)$$

where $AC^2 = (X_C - X_A)^2 + (Y_C - Y_A)^2$ and terms are evaluated at the approximate values for the unknowns.

The coordinate Z_C can be obtained as following:

$$Z_C = Z_A + h_I + AC \times \tan\beta \text{ or}$$
$$Z_C = Z_A + h_I + \sqrt{(X_C - X_A)^2 + (Y_C - Y_A)^2} \times \tan\beta, \quad (5)$$

in which h_I is the instrument height, and β is the measured vertical angle (positive value for angle of elevation and negative value for angle of depression).

Equation (5) can be rewritten as:

$$\beta + v_\beta = Tan^{-1}\frac{Z_C - Z_A - h_I}{\sqrt{(X_C - X_A)^2 + (Y_C - Y_A)^2}}. \quad (6)$$

The observation equation for Equation (6) after linearization is:

$$v_\beta + \frac{g \times (X_C - X_A)}{AC(AC^2 + g^2)}\Delta X_C + \frac{g \times (Y_C - Y_A)}{AC(AC^2 + g^2)}\Delta Y_C +$$
$$\frac{-AC}{(AC^2 + g^2)}\Delta Z_C = Tan^{-1}\left(\frac{g}{AC}\right)_0 - \beta, \quad (7)$$

in which $g = Z_C - Z_A - h_I$ and terms are evaluated at the approximate values for the unknowns.

In order to obtain the ground coordinates of point C, three measurements should be available for solving three observation equations. More than three measurements would enable a least squares solution.

The observation Equations (4) and (7) can be written for least squares method solution as (Mikhail 1976):

$$V + B \cdot \Delta = \varepsilon, \quad (8)$$

where: Δ is the correction vector to the current values set for the unknowns (the object space coordinates of the deformation points) in the iterative solution; B is the matrix of the partial derivatives of Equations (4, 7) with respect to the unknowns; V is the residual vector, i.e., the correction vector to the observations; and ε is the discrepancy vector.

The principle of the least squares method needs the minimizing of the quadratic form $V^t \cdot W \cdot V$, where W is the weight matrix whose elements are the weights associated with each of the observations. The least squares solution of an equation similar to Equation (8) can be given as (Mikhail 1976; Ghilani, Wolf 2006):

$$\Delta = N^{-1}C, \quad (9)$$

where

$$\left.\begin{aligned} N &= B^t \cdot W \cdot B \\ C &= B^t \cdot W \cdot \varepsilon \end{aligned}\right\}. \quad (10)$$

Variance of unit weight can be computed as:

$$\hat{\sigma}_o^2 = V^t \cdot W \cdot V / (N - U), \quad (11)$$

where $\hat{\sigma}_0^2$ is the variance of unit weight; N is the number of observations; U is the number of unknowns and equals to $3n$ in which n is the number of deformation points; $(N - U)$ = Degree of freedom.

2. Experimental work

The derived mathematical model for deformations measurement is general and can be applied to any kind of building or bridge structure or structural members. In this paper, it was applied for the structural monitoring of I-shaped steel beam. The beam dimensions are 100mm depth, 100mm flange width, 6 mm web thickness, and 8 mm flange thickness.

The beam was simply supported with a span length of L = 4 m shown in Figure 2. The beam was loaded with a concentrated load at the mid span. Three values of the load 0.5, 1.0 and 1.5 ton were used in this experiment.

Seven deflection points were chosen at L/8, L/4, 3L/8, L/2, 5L/8, 3L/4 and 7L/8 locations. Seven paper prisms (Fig. 2) and seven LVDTs were installed on the deflection points.

Two observation points (control points) were chosen on the floor 5m apart from the beam. From the several techniques for determining the level differences in the field, the well known and widely accepted one

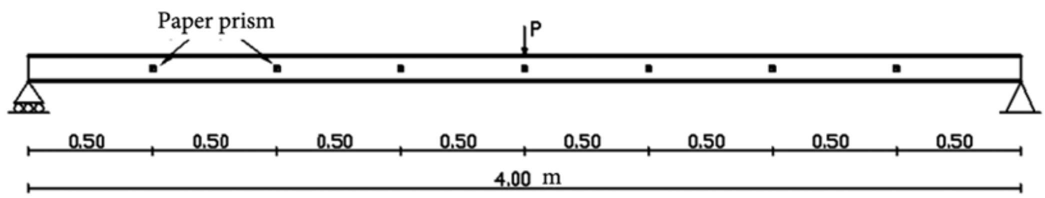

Fig. 2. The beam, loading, and locations of the deflection points

was chosen. Level differences can be measured with one level and readings performed on the invar rod using two different instrument horizons. This procedure is strongly related to the need to ensure the proper accuracy required to measure the level differences in the field (Nistor, G., Nistor, I. 2007). The level difference between the two observation points was achieved with the precise leveling using a GPLE3 geodetic invar staff with 10 mm graduations and Leica NA2 automatic level with a Leica (10 mm) GPM3 parallel plate micrometer attachment. To measure the horizontal distance between the two observation points, a Topcon GTS710 total station was used. A total of six horizontal distances (3 direct and 3 reverse) were observed. The final horizontal distance was taken as the mean of all measured values. By knowing the horizontal distance and the height difference between the two observation points, the coordinates of the two control points were assigned to a local coordinates system.

The observations of the deflection points were taken before loading the I-beam and after each stage of loading. The deflection points were observed using Wild (Lieca) T2 with 1” least count. The horizontal angles to the deflection points were observed by direction and closing the horizon methods and reading the horizontal circle in both the left and right faces. Multiple observations of the angle are made, with the circle being advanced prior to each reading to compensate for the systematic errors. Each angle was determined for each observations set and the final horizontal angle value was taken as the average of all measured values. The vertical angle of each deflection point was determined from the vertical circle readings in both left and right faces and taking the mean value.

To compare the results of the proposed method with the total station results, the coordinates of the deflection points were measured directly using Topcon GTS710 total station.

The surveying instruments used were technically checked before the execution of the measurements and found that they are in good condition.

3. Results

Vertical deflections of the simply supported beam subjected to the action of concentrated loading stages are

Table 1. Values of vertical deflections and discrepancies (in mm)

Loading Stage	Stage 1 0.5 Ton					Stage 2 1.0 Ton					Stage 3 1.5 Ton				
Distance	Proposed method		Total station		LVDTs	Proposed method		Total station		LVDTs	Proposed method		Total station		LVDTs
	Deformations	Discrepancies	Deformations	Discrepancies		Deformations	Discrepancies	Deformations	Discrepancies		Deformations	Discrepancies	Deformations	Discrepancies	
1L/8	2.84	0.24	3.84	1.24	2.60	5.76	0.29	7.52	2.05	5.47	8.74	0.35	10.55	2.16	8.39
L/4	5.05	0.27	6.06	1.28	4.78	9.22	0.12	11.20	2.1	9.10	14.52	0.89	16.22	2.59	13.63
3L/8	6.81	0.31	7.40	0.9	6.50	12.79	0.54	13.69	1.44	12.25	18.57	0.27	19.96	1.66	18.3
L/2	7.2	0.34	7.72	0.86	6.86	13.93	0.59	14.66	1.32	13.34	20.84	0.87	21.29	1.32	19.97
5L/8	6.71	0.32	7.24	0.85	6.39	12.78	0.57	13.34	1.13	12.21	18.64	0.38	19.96	1.7	18.26
3L/4	5.17	0.29	5.79	0.91	4.88	9.66	0.42	11.05	1.81	9.24	14.36	0.5	16.48	2.62	13.86
7L/8	3.04	0.39	4.10	1.45	2.65	5.64	0.24	7.32	1.92	5.40	8.83	0.29	10.73	2.19	8.54
RMSE	0.31		1.09		–	0.43		1.72		–	0.56		2.08		–
RMSE for all observations	0.45		1.68												

used as serviceability assessment criterion. For each loading stage, direct measurements of vertical deflections from LVDTs at L/8, L/4, 3L/8, L/2, 5L/8, 3L/4, and 7L/8 positions of the steel beam were used as the reference for the comparison.

The measured deflections at the deflection points for each one of the three loading stages, 0.5, 1.0 and 1.5 ton, using LVDTs are compared with obtained values using the proposed method and total station as shown in Table 1.

Furthermore, Figure 3 shows the deformed shapes of the beam generated from the measured vertical displacements using the different methods and loads.

From Table 1 and Figure 3, the following conclusions can be drawn:

- The vertical displacements obtained from the proposed method are in good agreement with the directly measured displacements from LVDTs.
- For the maximum deflection at mid-span of the beam, the discrepancies are in the range of 0.34–0.87 mm corresponding to an error of less than 4.3% for the proposed method results, while they are in the range of 0.86–1.32 mm causing an error of up to 12.5% for the case of total station measurements.
- The maximum values of discrepancies are 0.89 mm and 2.62 mm for the proposed and total station methods respectively. This leads to conclude that the results of the proposed method are comparable to the results of LVDTs.
- Values of Root Mean Square Error (RMSE) show that the accuracy of point deformation determination using the proposed method is much better than using total station.

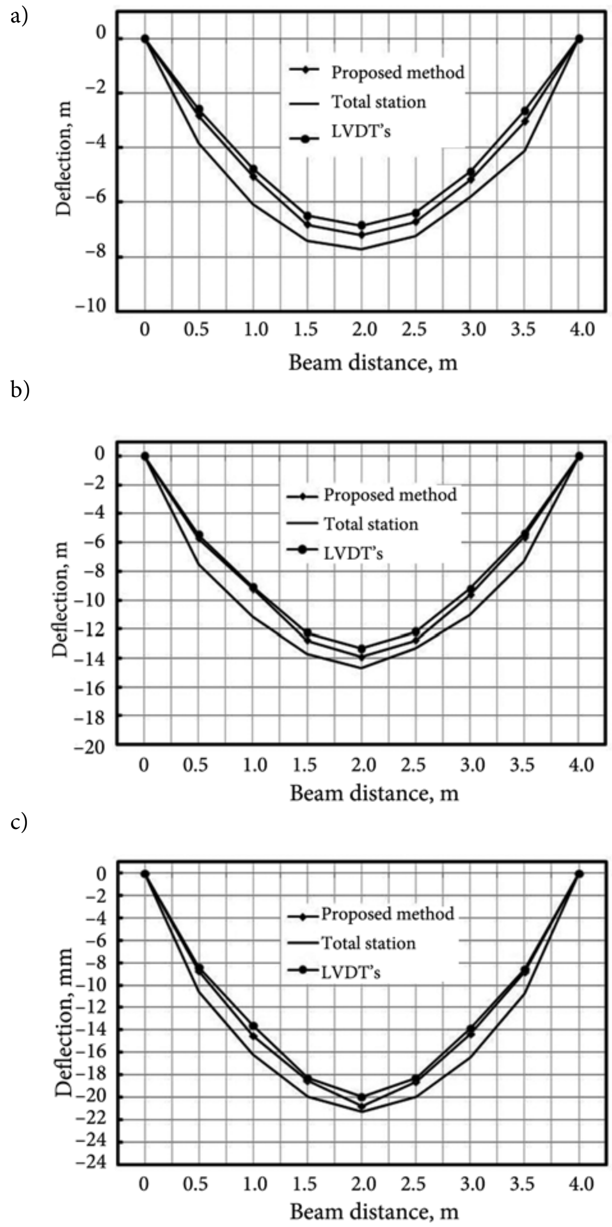

Fig. 3. Plots of the beam deflections for a) 0.5 ton load, b) 1.0 ton load and c) 1.5 ton load

Conclusions

The proposed method is developed to add a new solution to traditional methods of angle intersection and trigonometric leveling. It is designed to provide a simultaneous solution to all observations in one step using least squares solution to improve the expected accuracy and to generate the necessary data for statistical analysis.

Measurements of deflection in steel beam subjected to a concentrated load using LVDTs, the proposed method and total station have been successfully demonstrated.

Deflections measured directly from LVDTs can be used as an accurate measure for computing the discrepancies of deformations of the proposed and total station methods results.

The results show that for the maximum deflection at mid-span of the beam, the discrepancies are in the range of 0.34–0.87 mm corresponding to an error of less than 4.3% for the proposed method results, while they are in the range of 0.86–1.32 mm causing an error of up to 12.5% for the case of total station measurements, for the maximum deflection at mid-span of the beam. Based on RMSE values, the accuracy of point displacements determination using the proposed method is much better than total station measurements.

The maximum values of discrepancies are 0.89 mm and 2.62 mm for the proposed and total station methods respectively. This leads to conclude that the results of the proposed method are comparable to the results of LVDTs.

Therefore, according to the improvement in the measurement accuracy of the developed method presented in this paper and comparing the methods of GPS and close range photogrammetry, the presented method can be used as an effective approach for structural monitoring with the following advantages: (1) suitable for indoor and outdoor applications, (2) using simple surveying instruments (theodolite), (3) no need for expensive GPS receiver antennas or metric or non-metric cameras, (4) no in situ instrumentation of sensors, (5) no difficulties to reach structures or structural members, (6) simple in its performing by surveyors not by specialists or photogrammetrists, and (7) no wiring cost.

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