S. A. Veress, D.Sc.
University of Washington
Seattle, WA 98195
Lo-jung Lionel Sun
Gardner Engineers, Inc.
Seattle, WA 98104

Photogrammetric Monitoring of a Gabion Wall*

Terrestrial photogrammetry was employed to determine temporal deformations.

Introduction

In 1968, through the sponsorship of the Washington State Highway Department and the Bureau of Public Roads, a research project was initiated in order to discover whether or not motion and deflection of a retaining wall could be determined by photogrammetric means. This research showed that the photogrammetric monitoring method is capable of providing the required accuracy.

Abstract: *The gabion wall, which is being built as part of the Interstate Highway 90 east of Seattle, Washington, is being monitored by the terrestrial photogrammetric method. A modified KA-2 camera, f = 610 mm (24″), format size 23×23 cm (9″ × 9″), glass plate, is being used for this purpose. The establishment of the "base net" for the camera stations has been accomplished with EDM and theodolite of 1/10 of one second least reading. The design of the geometry of the camera stations and their measurements are discussed. The method by which the photogrammetric monitoring was performed is described and its accuracy is presented. A typical deformation of a target in the horizontal direction is shown.*

Similar projects have been implemented in Canada (Erez, 1971), in West Germany (Planicka, 1970), and in Rumania (Gutu, 1972). The U.S. Corps of Engineers, Seattle District, realizing the economical and technical potential of analytical terrestrial photogrammetry, began various investigations in 1970 to establish their method for routine applications (Erlandson and Veress, 1975).

The Washington State Highway Department in conjunction with the Federal Highway Administration is sponsoring a research project, which began in late 1975, to monitor the motion of a gabion wall. The wall is being built in the Snoqualmie Pass area near Seattle and will be a structural element for the planned westbound lanes of Interstate Highway 90.

One may ask the meaning of the word "gabion." In Webster's New World Dictionary, it is defined as:

(1) A cylinder of wicker filled with earth or stone formerly used in building fortifications; and

(2) A similar cylinder of metal used in building dam foundations, etc.

The gabions for this project are three feet by three feet by three feet of steel wire mesh filled with rocks. It was constructed as a gravity wall with native rock and weighs 100

* The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the Washington State Department of Highways or the Federal Highway Administration.

Photogrammetric Engineering and Remote Sensing,
Vol. 44, No. 2, February 1978, pp. 205-211.

pounds per cubic foot. The wall being monitored is 1200 feet long and varies in height from 5 to 54 feet depending upon the topography. This is an experimental design and is believed to be the largest in the nation. Part of the wall is shown in Figure 1 where the upper and lower portions of the wall are clearly distinguishable.

It was important to monitor the structure with various types of equipment and methods; among others was photogrammetry. The photogrammetric monitoring is to be described in this paper.

During the previously mentioned research projects, the photogrammetric methodology changed. In the first project (1968) precise control points were placed around the structures. With the use of these control points, the camera positions (coordinates of frontal nodal points and orientation matrix) were determined by using the collinearity equations, which were employed for space resection. Then, as the second step, the same equations were used to determine points on the structure by space intersection.

The research of the U.S. Corps of Engineers used a completely different approach. A modified Wild BC-4 camera was used to take the photographs and to measure the outer orientation elements (ϕ, ω, and κ). This permitted the simultaneous adjustment of geodetic and photogrammetric data as compared to the sequential adjustment employed in the previous project.

In this latest research, the camera stations were constructed so that the camera reoccupied the "same position" on each occasion when the photographs were taken. Therefore, once the camera positions (frontal nodal point of the lens and the orientation angles) were determined, only space intersection was required for determining the position of points located on the structure.

Camera and Camera Stations

Due to the topography of the terrain, the camera stations could be located only on the south side of the valley whereas the structure was being built on the north side. The photographic distance is about 3,000 feet (1,000 meters). Due to this distance and the required accuracy, a camera with a focal length of 24 in. (610 mm) and a 9½ × 9½ in. (23 × 23 cm) format size was used.

There are no metric terrestrial cameras manufactured to these specifications. Therefore, a KA-2 aerial camera was modified for this purpose. The modification consisted in adding two supporting rings and a magazine with plate holders. The camera is exhibited in Figure 2. The calibration of the camera was done by the U.S. Geological Survey in Reston, Virginia.

The camera shutter also was modified. The original shutter speeds, ranging from 1/25 to 1/400 of a second, were reduced to 1/8 to 1/250 of a second. The shutter then had the following speeds: open, 1/8 second, 1/15 second, 1/30 second, 1/60 second, 1/125 second, and 1/250 second. The shutter modification was required because the photographic plates are Kodak 215 mm IVF microflat spectroscopic high-contrast emulsion

Fig. 1. Gabion wall.

FIG. 2. Modified KA-2 camera and camera Station #2.

with ASA 50 sensitivity. These modifications were done by the Machine Shop of the Department of Civil Engineering, University of Washington.

Three permanent camera stations were anchored to bedrock by 3 cm × 3 cm reinforcement bars which were threaded on top in order to attach the camera mounts for each respective station. A drilling jig was prepared by the Washington State Highway Department's Soil Testing Laboratory to aid in alignment. The area was filled with concrete to provide a permanent camera station with mounts for the monitoring. The illustration of the number two camera station with the camera is shown in Figure 2.

FIELD CONTROL

The control net which determined the position of the camera stations was established along the existing Interstate 90 (to be eastbound when completed). This is a very narrow piece of land and it is difficult to establish a network of desirable geometry. The topographic map of the area is shown by Figure 3. Thus, it resulted in a network with ill-conditioned geometry. This situation required a combination of angle and distance measurements to arrive at an accuracy which is required for such a project.

The geometry is exhibited in Figure 4 where the measured angles are noted as 1 through 8. The smallest angle is 56 minutes. Because of the ill-conditioned geometry, a sequential adjustment was used in order to eliminate values having an adverse influence on the accuracy.

The sequences of adjustments were

- Adjustment of base lines (Points 1, 1a, and 3),
- Determining the position of Point 2a, and
- Adjustment of Camera Station 2.

(In Figure 4 CS 1, CS 2, and CS 3 were used as Camera Stations.)

For the adjustment of base lines, two sets of electronic surveying measurements were

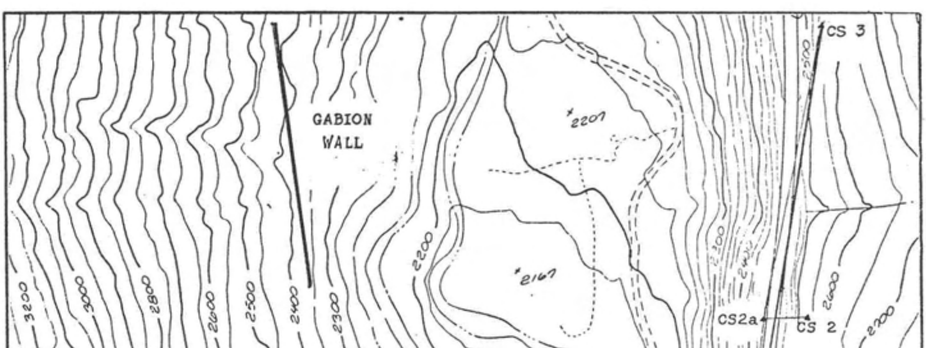

FIG. 3. Area topographic map.

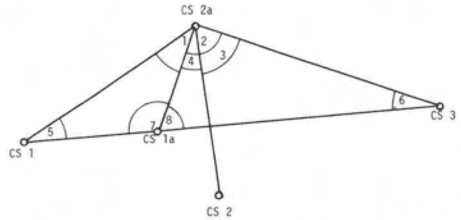

FIG. 4. Adjustment of camera stations.

used. One was measured in the fall of 1975 by using a Tellurometer DC-6 and the second set was measured in the spring of 1976 with a Kern DM 500. The results of these measurements are shown in Table 1. From the table it was concluded that there was no movement of points during the winter in spite of the geological faults which were close by.

The adjustment of the base line was done by the observation method with the observation equation in the general form,

$$AX - L = V \ .$$

The probable values are

$$\text{CS 1} - \text{CS 1a} = 105.5204 \text{ m} \pm 1.9 \times 10^{-3} \text{ m}$$
$$\text{CA 1a} - \text{CS 3} = 631.3815 \text{ m} \pm 1.9 \times 10^{-3} \text{ m}$$
$$\text{CS 1} - \text{CS 3} = 736.9010 \text{ m} \pm 1.9 \times 10^{-3} \text{ m}$$

A combined intersection and resection method of adjustment was used to determine the position of point CS 2a by using angles 5, 6, 8, 1, and 2, respectively. The method of adjustment was the observation with quasi-normals in which the basic formulas are

$$\begin{bmatrix} AX - L = V \\ DX - J = V' \end{bmatrix}$$

and the normal equation matrix is—

$$[A^TPA, D^TP'D]\, X - [A^TPL + D^TJ] = 0 \ .$$

For a detailed solution of these equations, the reader is referred to Veress, 1974. This adjustment resulted in the most probable values of coordinates of point CS 2a with standard errors of

$$\sigma_x = \pm\, 7.5 \times 10^{-3} \text{ m}$$
$$\sigma_y = \pm\, 2.3 \times 10^{-3} \text{m} \ .$$

Finally, the position of CS 2 was determined by the least-squares adjustment of the observation method, resulting in the standard error of coordinates being less than 2 mm. The elevation of these points was determined by trigonometric leveling and adjusted by least-squares. The standard errors were found to be between ± 0.006 and ± 0.003 m.

TABLE 1. COMPARISON OF BASE LINE MEASUREMENTS

Station	CD-6 tellurometer	Kern DM 500
CS 1 − CS 3	736.919 m	736.9005 m
CS 1 − CS 1a	105.495 m	105.5217 m
CS 3 − CS 1a	631.421 m	631.3828 m
CS 2a − CS 2	52.742 m	52.7395 m

DESIGN OF THE PROJECT

The basic concerns of the design were to find the proper size, location, and mounting of the targets on the wall to be monitored. The targets were designed for an instrumental measuring mark of 40 μm with an annulus of 15 μm in circular form. The circular targets degenerate to an elliptical form due to the tilt of the photograph and, thus, exhibit two diameters, D and D', in opposite directions. D represents the minor axis and D' is the major axis of the ellipse. Both of these diameters are functions of the angles α and β. According to Figure 5, one can show that

$$D = D'_{\min} \frac{\sin(\alpha + \beta)}{\sin \alpha}$$

$$D'_{\min} = d\left(\frac{h}{f}\right)$$

where

$$h = \frac{h_1}{\sin \alpha}$$

and

$$D_{\max} = \frac{7d}{5f \sin \alpha} h_1$$

d is the diameter of measuring mark in these equations. (The derivation of these formulas is given in Erlandson and Veress (1975).)

Using the numerical data, the following results are obtained:

$$D_{\min} = 12.02 \text{ cm and } D_{\max} = 16.55 \text{ cm}$$

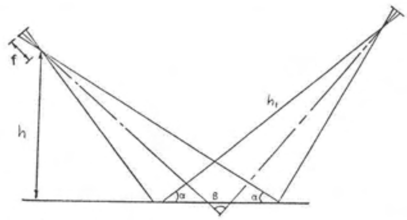

FIG. 5. Geometry of target design.

The target diameter was chosen to be 16 cm. The circles were painted with acrylic black, and acrylic white served as the background on a 9 × 9 × ⅛ in. aluminum plate. The targets were mounted on a steel bar which was 12 in. long and inserted into the gabion wall so that the targets remained on the surface of the wall. An array of these targets is shown in Figure 6. This form of target was found to be very satisfactory for all kinds of weather conditions.

Data Acquisition and Reduction

In a previous section, it was indicated that a camera pedestal had been permanently fixed at each of the camera stations to ensure consistency in the orientation of the camera. It enables the camera to occupy the same location at any time without the need to redetermine the orientation parameters (particularly orientation matrices).

The orientation parameters have been determined by geodetic means by using Kern DKM 3 Theodolite. The detailed methodology is given in Sun (1976). A photogrammetric method also was applied to determine these parameters in order to check, and correct if required, the field measurements. Figure 7 shows the general data reduction flow chart.

The photographic coordinates were obtained on the AP/C analytical stereoplotter at the University of Washington. The accuracy of the photo coordinates was obtained by making five observations, and it varied according to the nature of the targets. The general evaluation of achieved results is shown in Figure 8. This figure shows that the standard error in the case of ambiguous targets is about 5 micrometers. In other cases the standard errors are less. These results proved to be satisfactory.

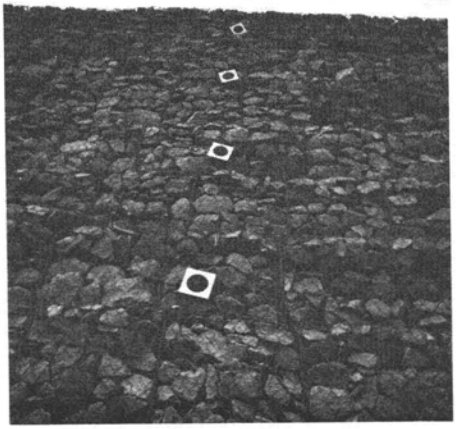

FIG. 6. View of target array.

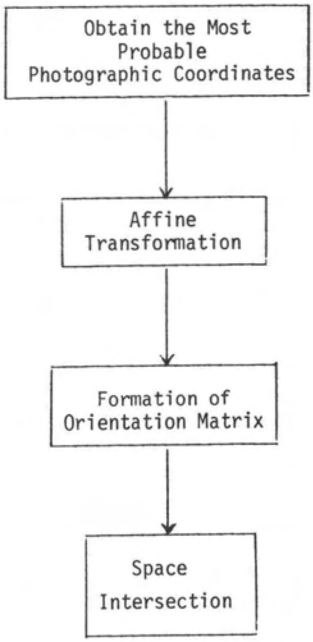

FIG. 7. Data reduction flow chart.

The instrumental photo coordinates were reduced to the center of the plate by a two-dimensional affine transformation.

The orientation matrix $\boldsymbol{M}$ for each camera station was determined by the vector method. This is summarized for camera station by the steps indicated below.

The vector from the center of the objective to the image point p is

$$\vec{S}_p = \begin{bmatrix} \dfrac{-x_p}{\sqrt{x_p^2 + y_p^2 + f^2}} \\[2ex] \dfrac{-y_p}{\sqrt{x_p^2 + y_p^2 + f^2}} \\[2ex] \dfrac{f}{\sqrt{x_p^2 + y_p^2 + f^2}} \end{bmatrix} = \begin{bmatrix} \cos xpO_i \\[2ex] \cos ypO_i \\[2ex] \cos zpO_i \end{bmatrix}$$

The vector components from the center of the objective to the ground point P (whose image is p) are

$$\vec{S}_P = \begin{bmatrix} \dfrac{X_{Oi} - X_P}{O_iP} \\[2ex] \dfrac{Y_{Oi} - Y_P}{O_iP} \\[2ex] \dfrac{Z_{Oi} - Z_P}{O_iP} \end{bmatrix} = \begin{bmatrix} \cos XPO_i \\[2ex] \cos YPO_i \\[2ex] \cos ZPO_i \end{bmatrix}$$

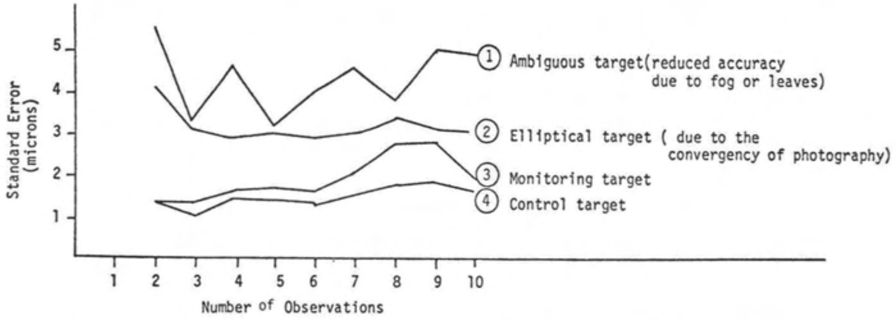

FIG. 8. Pointing analysis. Note: The data obtained from 100 man-hours experience.

The relation between the two vectors is

$$M \vec{S}_p = \vec{S}_P$$

From the equation, the $\boldsymbol{M}$ orientation matrix is obtained in the conventional manner.

The last step of the data reduction is the determination of targets by space intersection. The space intersection can be briefly described from camera stations i and j as follows. The vectors from the camera stations to the object points are designated as

$$\vec{S}_i = (\cos XPO_i, \cos YPO_i, \cos APO_i)$$

$$\vec{S}_J = (\cos XPO_j, \cos YPO_j, \cos ZPO_j) \quad (1)$$

The relation between the vectors of picture and object space are

$$M_i\vec{S}_i = \vec{S}_i \text{ and } M_j\vec{S}_j = \vec{S}_J \quad (2)$$

The equations of line O_iP and O_jP are

$$\frac{X_P - X_{Oi}}{\cos XPO_i} = \frac{Y_P - Y_{Oi}}{\cos YPO_i} = \frac{Z_P - Z_{Oi}}{\cos ZPO_i} \quad (3)$$

$$\frac{X_P - X_{Oj}}{\cos XPO_j} = \frac{Y_P - Y_{Oj}}{\cos YPO_j} = \frac{Z_P - Z_{Oj}}{\cos ZPO_j}$$

By using Equation 2, the direction cosines of Equation 1 are computed and the X_P, Y_P, and Z_P of the target point coordinates are obtained from Equation 3 by means of least-squares adjustments.

DATA EVALUATION

The gabion wall was targeted by 40 visible artificial targets as was described and by 50 "natural" targets which consisted of easily recognizable rocks in the wall. The experiment shows that there are no major differences between natural and artificial targets under ideal photographic conditions. However, when the area is covered with snow or ice, only the artificial targets are usuable.

The evaluation of each individual target was done by accepting the results of the first photography (September 7, 1976) as fixed or "zero" data. Every later result was compared to these. A three-dimensional local coordinate system was chosen so that its X-axis was parallel to the wall, the Y was in the vertical direction, and the Z was perpendicular to the wall. Due to the three-dimensional observation, the settling of the wall as well as any other motion can be determined. A typical motion of a point in the Z direction is shown by Figure 9 as a function of time when the photograph was taken.

Besides the photographic monitoring, several other instruments were used to mon-

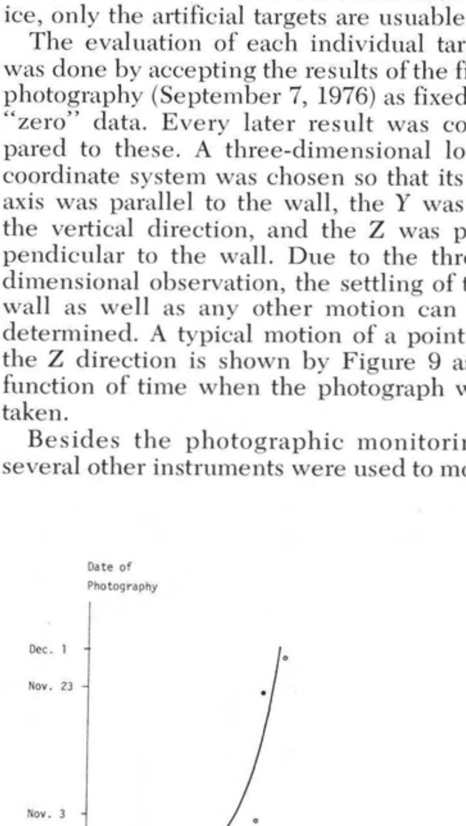

FIG. 9. Deflection of a point during the construction.

itor movement. For example, two inclinometers were placed in the wall. The photogrammetric data closely corresponds to that of the inclinometer readings.

There were several modifications and methodological improvements required during this project. Among the most important was the illumination of the collimation marks of the camera. Under adverse weather conditions (which exist often in the construction area), some of the collimation marks could not be observed due to the low level of illumination. This problem was solved by illuminating the collimation marks with (green) light-emitting diodes.

Another significant problem encountered during this project was that the orientation matrix was not as stable as it had been expected to be for several occupations of the same camera station. The reason for this was found to be at the front suspension of the camera system where a stainless steel circular cross section device was placed on a flat surface. This suspension has now been changed to a conical surface placed in a circular shaped hole, permitting no side motion. This system is presently under evaluation.

Finally, the achieved accuracy should be mentioned. With the modifications mentioned above, the minimum accuracy was found to be 1/50,000 of the photographic distance and the best accuracy obtained was 1/140,000 of the photographic distance in the X, Y, and Z directions.

The photographic distance which changes from camera station to camera station ranges from 22 to 3200 feet while the residual errors (standard error of coordinates) are nearly constant. This results in the rather large variation in relative accuracy. The standard error on the retaining wall ranges from $\pm$ 15 mm to $\pm$ 6 mm. Only for a few natural targets was it found to be $\pm$ 20 mm.

Conclusions

The method developed for this project can provide the required accuracy for continuous monitoring of a structure. The advantage of photogrammetric monitoring over other data collection techniques is that it does not depend upon the number of points to be determined. Thus, the analysis of a large number of points is possible in a short period of time. The extra cost of adding points is extremely small. A specified area on these photographic plates can be re-observed at a later data, as happened in this project, to establish more points or to check the accuracy of preciously obtained data. The photogrammetric method provides a permanent time record.

A further advantage is that the results obtained are three-dimensional. Thus, it provides any information required for structural analysis. Due to the fact that the photographs are taken at a distance from the structure, it is ideal to use both during and after the construction.

Acknowledgments

The authors wish to express their appreciation to the Washington State Highway Department for sponsoring this project, particularly to Mr. Newton Jackson whose excellent organization made this project a valuable combination of construction and research. Further, the excellent work of the Photogrammetric Branch, Roadway Development Division of the Washington State Highway Department should be recognized for developing and providing the University of Washington with photographic material, thus increasing the proper flow of work which was so important the research executed during the construction. Acknowledgment should also be given to Mr. Edward Flint for the construction of the camera stations and to Mr. John Hatzopoulos for providing a large number of observations and for the general data collection.

References

1. Erez, M. T.; *Analytical Terrestrial Photogrammetry Applied to the Measurement of Deformations in Large Engineering Structures,* Dissertation, University of Laval, 1970.
2. Erlandson, J. P., and S. A. Veress, "Monitoring Deformations of Structure," *Photogrammetric Engineering and Remote Sensing,* Vol. 41, No. 11, November 1975, pp. 1375-1384.
3. Flint, E., *Design for Photogrammetric Monitoring of a Gabion Wall,* Thesis, University of Washington, 1975.
4. Gutu, A., "Photogrammetric Measurement Accuracy of Wall Pillar Cracks in Rock Salt Mines," *Bulletin de Photogrammetry,* 1972.
5. Sun, L. L., *Photogrammetric Monitoring of Gabion Wall,* Thesis, University of Washington, 1976.
6. Veress, S. A., *Adjustment by Least-Squares,* American Congress on Surveying and Mapping, 1974.
7. Veress, S. A., *Determination of Motion and Deflection of Retaining Walls,* University of Washington, Final Technical Report on Research Conducted for Washington State Highway Commission Department of Highways in Cooperation with U.S. Department of Commerce, Bureau of Public Roads.