FRONTISPIECE. The Morin power dam.

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Deformation Measurements of Power Dams with Aerial Photogrammetry

Remarkably cost-effective three-dimensional measurements of deformations of a power dam can be performed by using numerical data from aerial photogrammetry with the support of very little ground control.

(Abstract on next page)

INTRODUCTION

HYDROELECTRIC POWER DAMS are of great economic significance to any energy conscious country. With increasing awareness of the importance of their movements and local deformations, the power dams are more and more subjected to periodic control surveys. The International Association of Dam Construction has recommended that 0.7 percent of the overall costs for the construction of such dams be spent for periodic control surveys. This requirement offers a challenge to the surveyor.

The conventional procedures of precision geodetic survey can be suitably replaced in these cases by using terrestrial photogrammetric methods (Brandenberger, 1972). These methods, due to their advantages of closeness and convenience in control in the field, offer the following advantages:

- Adequate accuracy (standard error on the order of about 1 cm in each of the three coordinates).

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- A complete and permanent record of the situations at the times of exposures.
- Instantaneous record of the entire dam as against point-by-point measurements with geodetic procedures.
- Supplementary points may be added to those already measured as may be necessary for studying possible critical dam movements.
- In view of the usual large number of necessary test points and deformation analyses desired by the engineers, terrestrial photogrammetry is more economical when compared with the geodetic procedures.

In order to develop a still more cost-effective and convenient procedure, it was decided to try aerial, rather than terrestrial, photogrammetry combined with digital techniques in a research project recently undertaken by the Department of Photogrammetry, Laval University, Quebec. This decision was prompted by the following principal considerations:

(1) In deformation studies, the required change parameters are relative rather than absolute. Thus, absolute field control is not essential. Independent geodetic control in terms of a known distance (for scale control) and a plane of reference (for spatial orientation) may be the adequate minimum requirements. A precisely measured distance would provide the former while, in the absence of elevation control, even the water level surface would provide the latter. In case an adequately wide and calm water surface is not available, it may be enough to determine by field measurements the elevations of three widely separated points identifiable in the stereomodel. This can be done with comparative ease when aerial photography is used.

(2) In repetitive measurements over a long period of time, all subsequent measurements can be referred to the initial one. Thus, identifiable reference points both on the dam and outside of it (on the assumed solid and fixed ground) are necessary. In this regard, uniquely identifiable natural points rather than targets would be advantageous from the point of view of logistics. Aerial photography in this respect is more advantageous than terrestrial photography. Difficulties in field work and targeting are minimized.

(3) In the natural surroundings (which is typical of Canadian power dams) seasonal problems, such as caused by vegetation and snowfall in particular, cause greater hindrances in case of terrestrial photography when compared with aerial photography. Consequently, it can be concluded that periodic measurements using aerial photogrammetry are advantageous, particularly in case of critical dam movements after heavy precipitation in summer or winter.

(4) Because the deformation studies are in general performed using numerous discrete points, one is inclined towards using digital techniques. In other words, numerical (rather than graphical) photogrammetry would be the more appropriate approach for such studies. Furthermore, such comparisons for the total body of the dam are better done with a set of points describing the whole or part of the feature which can be recognized and interpreted in a stereomodel. These are what Masry (1981) calls "entities," which are better treated by using the DTM technique. The technique is conveniently adaptable to stereophotogrammetry using precision plotters. Such plotters generally are more adaptable to aerial photography.

(5) Another desire was to curtail high costs involved in camera calibration and rigorous analytical approaches. This can be achieved by using a precalibrated camera, carefully designed flight plans, a precision stereoplotter, and most time- and accuracy-critical operations being done at a high speed computer. Aerial photogrammetric procedures, being now well established and customary, offer many advantages in this respect.

With regard to the need for periodical deformation checks of hundreds of such power dams in

ABSTRACT: *The use of aerial photogrammetry, due to its operational and economic advantages, offers much towards periodic control surveys to determine deformations in power dams. An increased use of aerial photogrammetry for such deformation measurements can be foreseen. The paper deals with the development of a digital method for measuring differential movements in three dimensions of a power dam. This method emphasizes the use of a digital model data rather than ground control by way of using the initial model for absolute orientation of subsequent models and using spatial "entities" (a set of points describing the whole or part of a feature which can be interpreted or recognized on the stereo-model) for studying the differential movements. Data analyses with statistical control are discussed. The results are analysed and some ideas are presented for further refinement of the developed working system. This system seems to offer a cost-effectiveness better than that of precise geodetic microtriangulation or that of terrestrial photogrammetry.*

Quebec, and in view of the prevailing time and cost constraints, a somewhat relaxed accuracy requirement of around ±1 inch (2.5 cm) standard error was considered acceptable and this in consultation with the Provincial Department of Environment. The main objective of the research project was to find out whether, apart from its methodological advantages, the numerical aerial photogrammetric approach would produce a comparable accuracy. However, and if so, certain innovations are necessary and were developed in the course of the research project described in this paper.

This project was concerned with deformation studies of the *Barrage du lac Morin*. After a brief description of the project, practical conclusions derived from the project are presented in the paper.

Description of the Project

THE BARRAGE

Located approximately 200 km northeast of the city of Quebec in the region of Rivière-du-Loup, this *barrage* controls the flow of the Fourchue River. Made entirely of reinforced concrete, the *Barrage du lac Morin* is about 195 metres long and has a height of approximately 20 metres above the river bed. The central core of the *barrage* is anchored into bed rock to a depth of about 10 metres in the center and about 6 metres on both sides.

Built in 1973, the dam has so far served its purpose well. Being concerned about its continued performance, the Province of Quebec Department of Lands and Forests requested the Department of Photogrammetry, Laval University to measure the dimensional changes in the surface of the dam once a year.

GROUND CONTROL

As indicated before, for such studies absolute field control is not necessary. However, in order to achieve a good absolute orientation of the initial (1979 photography) model, it was decided to establish such control in the field. This would also permit an assessment of the displacements of the datums caused by considering a point on the dam as the bench mark. Nine control points were established from field observations. Of these, four (1, 3, 3A, and 7, see Frontispiece and Figure 1) are situated on the dam. Five of them (1, 3, 3A, 7 and F1) were established by triangulation, each of which was used as a theodolite observation station, with the rest of the points being established by intersection. All angles in the triangulation network were observed. The angles, both horizontal and vertical, were observed in two sets (with both direct and reverse observations) with a Wild T2 theodolite.

It may be noted that the choice of the locations of the points outside the dam was limited due to the existence of dense vegetation.

A local right-handed rectangular system of coordinates was established with the X-axis being parallel to the line joining points 1 and 7 and with point 1 having arbitrarily the coordinates, $X = 100.00$ m and $Y = 100.00$ m. Elevations (Z coordinates) above sea level were used and determined. Point number 3 was considered as the bench mark for this local system. This consideration was made in order to connect all height measurements to the system of

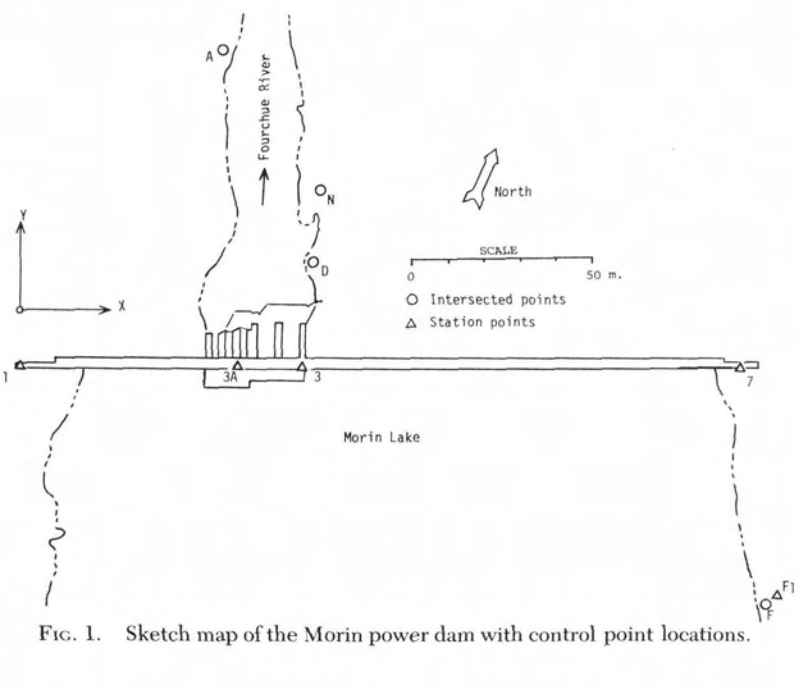

Fig. 1. Sketch map of the Morin power dam with control point locations.

height values in use for this dam by the Quebec Department of Lands and Forests. One can argue that locating the reference bench mark on the dam is undesirable because dam movements are to be monitored in three dimensions and the bench mark itself is liable to move. However, for our purpose, this constitutes re-establishing the coordinates X, Y, and Z of the bench mark. These shifts could still be determined by using ground control of the dam as well as outside of it.

The planimetric misclosure in the triangulation network was ± 3 mm and the standard error in the heights was ± 13 mm. These being considered acceptable, the least-squares adjusted coordinates of the ground control were used to orient the initial (1979 photography) stereo model.

PHOTOGRAPHY

Two sets of stereo-pairs (of 60 percent forward overlap each) with the same flight direction (east-west) were obtained in July 1979 and 1980 with Wild RC8 cameras of focal lengths 152.176 mm and 153.367 mm, respectively. Similar photographs (with a Wild RC8 camera) were taken in July 1981. The approximate photo scale in each case was 1:3,000. It may be noted that with a more careful planning the photography for this dam could have been taken at a scale as large as 1:1,500 by either flying lower or by using a camera with a longer local length. This, as contemplated for the future, would have yielded yet better results. Glass diapositives prepared immediately after each photo mission were used for the measurements.

PHOTOGRAMMETRIC RESTITUTION

The stereomodels were oriented at the Wild A7 Autograph of Laval University's Department of Photogrammetry. The three-dimensional coordinate data are registered automatically with the EK-5a electrical data acquisition system and are simultaneously recorded on computer compatible cards with an on-line IBM 029 card punching outfit. The model scale in each case was 1:1,000. Corresponding maps for graphical references were plotted at the scale 1:500 with a contour interval of 1 m.

The initial model (1979 photography) was oriented at the Wild Autograph A7 by using the available ground control. However, the absolute orientation was further refined by the following computations:

- Affine transformation in three-dimensions;
- Least-squares fit by using all available control points; and
- Weighted transformation in order to fit the model to points considered as absolutely fixed.

Several recent research studies indicate that, in a stereo-model, scales along different directions are indeed different and that one obtains better results when using additional parameters to correct for model deformations caused by these scale variations. In a specific system (materials, camera, and instruments combined) such errors tend to be systematic and consistent (Ebner, 1976; Ghosh, 1971; Morgan, 1971; Nanayakkara, 1970).

The principle of the three-dimensional affine transformation is expressed by the following equation (see also Ghosh, 1979):

$$\begin{bmatrix} X^* \\ Y^* \\ Z^* \end{bmatrix} = \mathbf{R} \begin{bmatrix} k_x X \\ k_y Y \\ k_z Z \end{bmatrix} + \begin{bmatrix} X_0^* \\ Y_0^* \\ Z_0^* \end{bmatrix} \tag{1}$$

X^*, Y^*, Z^* are the coordinates after transformation;
X, Y, Z are the coordinates before transformation (i.e., the model coordinates at the instrument);
X_0^*, Y_0^*, Z_0^* are the coordinates of the origin of the X, Y, Z system in the X^*, Y^*, Z^* system, determining the translation vector;
k_x, k_y, k_z are the scale factors along the X, Y, and Z axes, respectively; and
$\mathbf{R}$ is the rotation matrix of dimensions 3 by 3, containing sine and cosine functions of the three rotations Ω, Φ, K, around the three axes, X, Y, Z, respectively.

A program (Bougouss, 1981) written in FORTRAN IV permits one to obtain the following:

- the transformation parameters;
- the residuals in the observation data;
- the *à posteriori* variance of unit weight;
- the variance-covariance matrix of the adjusted parameters;
- the matrix of the correlation coefficients of the parameters;
- the variance-covariance matrix of the adjusted observations; and
- transformation of all observed points into the ground coordinate system.

At the Laval University computer center's IBM 370 system, an acceptable convergence was obtained in three iterations for the seven used control points.

It was decided to consider points 1 and 7 as being the fixed ones, i.e., they are required to have the same ground coordinates (without errors/misclosures) after transformation. This was accomplished by using weighted observation data. This weighting was achieved indirectly by using *à priori* standard errors at the control points. The weights were chosen based on (a) the evaluation of the field work and (b) according to their use in the stereo-model. Comparative reliability of the points with regard to each coordinate (X, Y, Z) was considered separately. This was done in view of the basic assumption that a mathematical correlation between the coordinates can be neglected in practice (see Maarek, 1973). Consequently, the selected weights follow a variable distribution pattern. Fifteen different patterns having been tried, the pattern given in Table 1 was found to yield the best results, *viz.*, $\sigma_x = \pm 7.0$ mm, $\sigma_y = \pm 2.0$ mm, and $\sigma_z = \pm 1.9$ mm at ground scale, which are considered to be sufficiently small. One must note, however, that this accuracy indicates the quality of fitting the initial model to the ground control and is not relevant to the study of the movements of the dam. The additional error contribution resulting from the field work must be considered also.

A similar transformation procedure was used for the 1980 photography. However, this model was transformed to fit the transformed ground coordinates obtained from the 1979 (initial) model, in which points 1, 7, 2201, and 2204 were considered as fixed. The weights yielding the best results in this case were in accordance with Table 2. It may be noted that points 2201 and 2204 were not obtained from ground survey. These points were selected from the initial (1979 photography) model. Point no. 2204 was used as a check against blunders.

This yielded the following results at ground scale: $\sigma_x = \pm 1.9$ mm, $\sigma_y = \pm 1.9$ mm, and $\sigma_z = \pm 2.6$ mm. At the photo scale 1:3000, these figures correspond to values less than 1 μm, which is considered excellent.

One may note here that, unlike the results associated with Table 1, the above results are very much relevant to the job at hand. For studying the movements of the dam, a very tight fit between the stereo-models (referred to fixed ground outside the dam) is absolutely necessary.

In each of the above transformations a chi-square test was performed to compare the *à priori* and *à posteriori* variances in order to reveal significant anomalies, if any. In each case the null hypothesis could not be rejected for the 95 percent confidence level. [Note: *Null Hypothesis*, definition given by James & James; *Mathematical Dictionary*, 3rd. Ed., 1968, van Nostrand Co.: It is a particular statistical hypothesis usually specifying the population from which a random sample is assumed to have been drawn, and which is to be nullified if the evidence from the random sample is unfavorable to the hypothesis, i.e., if the random sample has a low probability under the null hypothesis and a higher one under some admissible alternative hypothesis].

Table 1. *A priori* Standard Errors at Control Points in the 1979 Model

Points	$\sigma_{x(\mu m)}$	$\sigma_{y(\mu m)}$	$\sigma_{z(\mu m)}$
1	2	2	2
3A	50	90	15
3	50	90	15
7	2	2	2
D	50	90	15
N	50	90	15
A	50	90	2

Table 2. *A priori* Standard Errors at Control Points in the 1980 Model

Points	$\sigma_{x(\mu m)}$	$\sigma_{y(\mu m)}$	$\sigma_{z(\mu m)}$
1	2	2	2
7	2	2	2
2201	2	2	2
2204	40	40	40

The second transformation (of one stereomodel to another) yielded comparatively better results than the first transformation (of one stereomodel to the ground control).

DATA ACQUISITION

As previously indicated, all observations were made at the Wild A7 Autograph. Each point was observed twice, once stereoscopically and once pseudoscopically. This was done to check blunders and to minimize the effects of instrumental and observational errors.

The control points 1 and 7 are clearly identifiable, and each is located at the midpoint of the principal wall of the dam. Line 1-7 being parallel to the X-axis of the local ground coordinate system, the ΔX difference between the coordinates of points 1 and 7, when divided by 99, would give 100 interpolated points at equal distances apart in X on the dam. Because the dam is not exactly straight, necessary slight ΔY shifts of the instrument base-carriage would bring the measuring mark along the edge of the dam at the interpolated locations. At each tenth point a cross-profile was considered, along which the X, Y, and Z coordinates were observed at each break point of the dam surface (on the average, six points per profile). This gives an "entity" of a total of 150 points on the surface of the barrage, none of which is targeted but all of which are determinable by observation and interpolation at the instrument. Three-dimensional model (instrument) coordinates were read at each of these points.

The model coordinates of all these 150 points are used in the transformation in order to obtain their true ground coordinates.

The dam does not present any discontinuity, and therefore the displacements can be considered as situated on continuous curves obtained by interpolation and giving the real movement of the dam determined from the second and subsequent models relative to the first model.

Several interpolation methods were considered and tried. These include functions of one and more variables. Ultimately a third-order polynomial interpolation was used. One may note, however, that the graphical method by using "french curves" would even be adequate for such interpolations when one considers the few data to be treated (such as around 100 points in this case).

DETERMINATION OF DISPLACEMENTS

Spatial coordinates of all points are obtained after the transformation in each case. The displacements and deformations are deduced by comparing these values. However, before they are used for a meaningful interpretation and analyses, it is necessary that certain refinements of the data be performed, such as:

(1) *Elimination of gross errors*. Certain points revealed unusual discrepancies which could not be justified. These (only seven points) were simply rejected as having gross errors, i.e., blunders that could be in either of the two models (that are compared), although their causes were undeterminable.

(2) *Elimination of constant parts of the differences*. A careful inspection of the data revealed constant shifts of all the points in each of the three directions, X, Y, and Z. This could be interpreted as if either the entire dam moved or that some systematic and constant errors in the photogrammetric process existed in either or both of the two models. The shifts were -4.50 cm in X, -10.75 cm in Y, and -8.31 cm in Z. However, one should be aware that in the present case these shifts were mainly caused by the fact that point no. 3 was used as the bench mark, having moved itself, and consequently resulted in shifts of the datums of reference.

The final data obtained after these shifts yield the dam deformations, i.e., the changes determined from the 1980 model as compared with the 1979 model. One would, however, understand that these data still contain systematic (or regular) and random (or irregular) errors. In order to illustrate the results obtained from the 1980 model, deformation vectors in planimetry (X and Y) for only 21 points and their corresponding systematic pattern are presented in Figures 2 and 3, respectively.

The residuals between the systematic errors (obtained by total interpolation, as illustrated in Figure 3) and the values obtained from the first differences (as illustrated in Figure 2) are used in order to obtain the standard errors (which give an idea of the final quality of the work performed) according to the following formula:

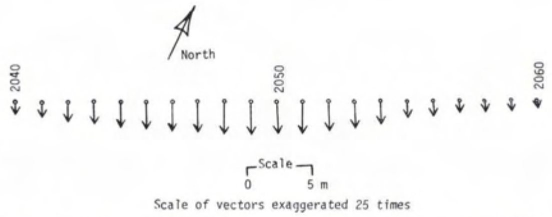

FIG. 3. Systematic planimetric displacement vectors for the 21 central points of the dam.

$$\sigma = \sqrt{\frac{[v\ v]}{n - 1}} \tag{2}$$

where σ is the standard error,
v are the residuals, and
n is the number of points (around 150 in each case).

The corresponding values obtained from the 1980 photography are $\sigma_x = \pm 0.75$ cm, $\sigma_y = \pm 2.85$ cm, and $\sigma_z = \pm 2.66$ cm. Their average, ± 2.09 cm, more than satisfies the requirement criterion of ± 2.5 cm as indicated before.

In a subsequent study using photographs obtained in June 1981 (as compared with the 1979 photography), the corresponding values were even better, *viz.*: $\sigma_x = \pm 0.05$ cm, $\sigma_y = \pm 1.31$ cm, and $\sigma_z = \pm 1.71$ cm. The excessive difference between σ_x on one hand, and σ_y and σ_z on the other, still remains to be accounted for. However, at the photo scale 1:3,000, these represent average accuracies of ± 7 μm for the 1980 photography and ± 4 μm for the 1981 photography, which must be considered as being very satisfactory.

As examples, two typical profiles are presented in Figures 4 and 5 in order to illustrate their respective movements between 1979 and 1980 along the Y-Z plane. Figure 4 illustrates profile no. 7 located near the middle of the dam and Figure 5 illustrates profile no. 10 located at the extreme end of the dam where the movement is nil. In these figures, the Z scale has been exaggerated 10 times over the Y scale.

From the economy point of view, the developed method permits a considerable reduction of cost and time for the following reasons:

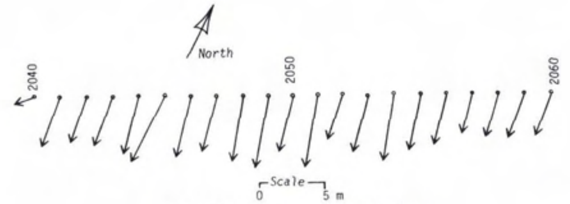

FIG. 2. Displacement vectors in planimetry for 21 central points of the dam before refinement of the data.

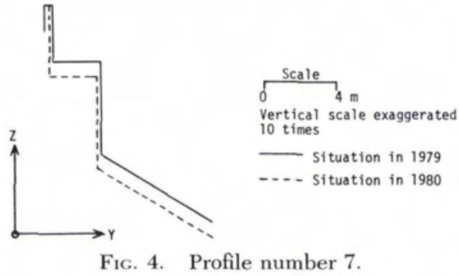

FIG. 4. Profile number 7.

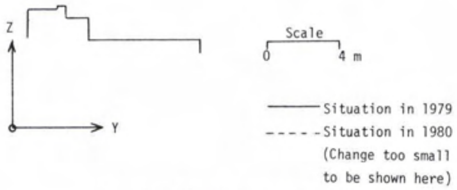

FIG. 5. Profile number 10.

- The field work for the establishing of the ground control need be done only once and no elaborate work or targeting is necessary.
- The orientation and procurement of data in each model at the stereoplotter requires little time, usually not more than about 4 hours.
- The emphasis being on the interior and relative orientations at the stereoplotter and on the absolute orientation at the computer, the refinements of the data for the least-squares fit of the control, the interpolation of points, and the determination of coordinate data are operations which are performed with comparatively high accuracy in little time.
- The final data, being computer generated, are adaptable to all sorts of interpretation and analyses with the help of the computer at comparably little cost and time.

CONCLUSIONS AND RECOMMENDATIONS

From the results obtained it can be concluded that the objectives of the project have been achieved by using the developed method.

The method or the procedure can be further improved by considering the following recommendations:

(1) Selection of all the points (control points in particular) in such a way that their identification and reading can be made without any ambiguity.

(2) Location of the points used as control for the transformation to be as wide apart in the model as possible (such as in the extreme corners of the model with possibly one or more in the middle) such that a maximum of accuracy for the absolute orientation is obtained.

(3) If possible, location of the reference point (bench mark) on solid ground away from the dam.

(4) Consideration of additional sources of photogrammetric errors, particularly those due to lens distortion, film deformation, and image blur due to air-craft movement with the aim of improving the accuracy of the results. For continued routine type work, these errors can be predetermined and appropriately applied for the correction of the model coordinates.

(5) Increase of the base/height ratio and the photo scale by flying lower and/or using wider angle cameras or by using helicopters or balloons to improve further the obtainable accuracy.

(6) Enhancement of the surface texture of the dam. Such concrete dams generally yield bland photographic texture and thus a poor stereo view, which affects the locating and reading of the selected points. Some painting or otherwise changing the surface texture of the dam would help in improving the measurement capability.

(7) Versatile use of aerial photography because the advantages of using aerial photography need not be limited only to the determination of deformations of dams. It can also constitute a means for the classification of dams. Such classification could be done according to the dimensions of the dams, their capacity of water storage, the dam height, and the possible risk of dam collapse. Such indications furnished by the DTM data would serve a great many purposes in continued surveillance and maintenance of such dams.

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