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GEODETIC DEFORMATION STUDY
DOWN-VALLEY TAILINGS CONTAINMENT PROJECT
CYPRUS ANVIL MINE
FARO, YUKON

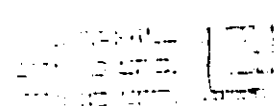
Conducted for
GOLDER ASSOCIATES
Consulting Geotechnical Engineers

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by
UNDERHILL ENGINEERING LTD.
Engineers and Surveyors
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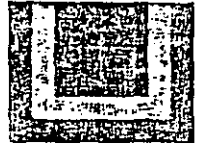
A technical report intended to critically examine the design and observation of the survey network established in 1981, with a view to prediction of the expected accuracy of detected movements.

File Y-2678



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OUR FILE: Y-2678

YOUR FILE:

Golder Associates
7017 Farrell Road S.E.
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Attention: W. J Burwash, P.Eng.

Dear Sirs:

Re: Your Request for a Report on the Deformation Study of the
Down-Valley Tailings Containment Project

We are pleased to submit herewith our technical report and accompanying plan in compliance with your request.

We trust the report satisfies your requirements.

Respectfully submitted,

UNDERHILL ENGINEERING LTD.

P. E. Thomson, BCLS

PET/pb
Encl.

ABSTRACT

The stability of the structures comprising the Down-Valley Tailings Containment project at Faro is of importance when environmental issues are under consideration.

The use of Geodetic deformation measurements to determine the position of the structures at various epochs of time will provide information relative to their stability. While a detected movement of one centimetre might not be significant when one is considering the safety of a structure, steps should nevertheless be taken to insure that the accuracy of the displacement vector is one centimetre, or as close to that figure as may reasonably be achieved. Thus the engineer will have meaningful data available when he is determining the long term stability of the constructed works.

This report examines the survey network established and the procedures followed on the project to date. Optimum network design parameters and methods for deformation studies in general are discussed and where and why some of these were deviated from will be explained. The general theory of Least Squares Adjustment is outlined as are certain statistical analysis techniques.

The results of the initial measurement of the network are presented and critically examined in order to show that the network and methods are capable of accuracies of the order of one centimetre.

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The survey commenced on November 28th with the determination of all vertical information by spirit levelling. This was completed on December 5th at which time distance measurement commenced. On December 11th this phase of the survey was completed. The direction observations were made during the period December 17th to December 20th thus completing all the field work entailed in epoch zero.

The reduction of observations and the computation and analysis of the results bring us to the time of writing of this report.

GENERAL CONSIDERATIONS

If any deformation study is to provide meaningful data, the observed (or computed) displacement vectors must be referred to some frame of reference or datum. The most convenient, both for ease of computation and clarity in the presentation of results, is a three dimensional system defined by three mutually orthogonal axes. Three coordinates therefore serve to explicitly define any point within such a space and any observed variances of these coordinates are evidence of the magnitude and direction of the displacement vector. For such a scheme to consistently provide accurate results, the stability of the monuments placed to reference the coordinate system must be proven. This can only be accomplished by repeated measurement of the network at subsequent epochs and showing that the fixpoints have no or only neglectable displacements. There must be at least four such fixpoints as it is possible (although highly unlikely) that three points may retain their positional orientation to one another but suffer some rotation with respect to the spatial coordinate system. It is also desirable that there be some distant fixpoint that serves as a directional reference only.

Within the network under consideration, there are five candidates for fixpoints, all of which are monuments placed in consolidated bedrock. They are stations 282, 284, 288, 289 and 292 and are shown on the plan accompanying this report. The one possible location for a distant fixpoint was rejected due to its altitude. This high elevation meant access to the point would be by helicopter only and that the probability of it being obscured by cloud was high.

GENERAL CONSIDERATIONS continued

All surveying observations or measurements are subject to two main types of errors which are classed as either random or systematic. Random errors are accidental errors and they tend to have a normal distribution. Therefore they have a standard deviation a priori which is either known from experience or is obtained by calculation from the physical makeup of the device or instrument used for the observation. Alternatively, the value may be provided by the manufacturer. Systematic errors, on the other hand, are constant errors which introduce a bias into all observations and while observational methods and techniques have been developed to minimize their effect, they cannot be totally eliminated.

There is a school of thought, therefore, which says that since deformation surveys are concerned with movement of points and not their absolute positions, then systematic errors are best dealt with by keeping them the same when possible and thereby eliminating their effect. To this end it is suggested that one should use the same instruments, equipment, observers, etc. for each element of the network and make the observations under the same conditions at each epoch.

This point was carefully considered before the methodology for the survey was decided upon. No commitment could be made to dedicate any equipment or personnel to a project likely to span decades. Who cannot say that new and better methods and equipment will be available in years to come? The observational environment can contribute substantially to systematic errors. The present plan for epochal scheduling virtually guarantees severe differences of weather patterns. For these reasons it was decided to attempt to reduce systematic errors to a minimum by employing all the best known survey techniques developed for the purpose.

MONUMENTS

All of the monuments placed on the pertinent structures were designed and installed by Golder Associates. They comprise a 5 foot long $3/4$ inch rod with footplate and are protected from frost action by a neoprene sleeve and from surface disturbance by a 6 inch steel collar which protrudes about 1 foot above the ground. The monument itself only protrudes about 6 inches above the ground, thus it is effectively invisible from any of the control stations. Total reliance therefore must be placed upon optical plummets for centering of measuring apparatus over these points.

Good deformation survey practice suggests that the major control points be concrete pillars firmly attached to the native rock and be of such a design that forced centering may be conveniently utilized. Of the five locations on rock, three were on the south side of the Rose Creek diversion canal and one was on top of a steep hill across a diversion ditch. These factors combined with the extreme cold temperatures experienced at the time of installation of the monuments effectively eliminated any possibility of erecting such pillars. Again, optical means of centering had to be resorted to, but with the slight advantage that all centerings on the monuments, regardless of type, would now be of a consistent nature.

The five aforementioned monuments were placed by drilling an 18 inch deep hole in the rock and inserting a $5/8$ inch rod secured with non-shrinking grout. Where the surface rock was weathered to the extent that it was impossible to drill a hole, the unconsolidated material was chipped with a breaking hammer and an excavation made by hand to solid rock. A longer rod was then drilled into the rock and the hole backfilled with concrete to original ground level. All monuments placed, including those placed by Golder Associates, were either centre-punched or enscribed with a cross to facilitate centering.

Two monuments of the Golder design were placed at stations 290 and 291 (not on bedrock) to strengthen the design network. Additionally, an iron bar at station 814 which was placed in the Intermediate Dam during

MONUMENTS continued

construction was occupied and used in the network as it provided the only means of obtaining visibility to station 412. This monument was not of the Golder design and as such is not protected from frost action. As it will be in the network at epoch 1 and later, a displacement vector will be available for it but the significance of this vector is left for Golder Associates to decide.

CHOICE OF EDM

Two instruments were considered for the measurement of the distances in the network.

The first EDM considered was the Tellurometer model MA 100, an infrared device with resolution to 0.1 mm and a stated range of about 2 km. The manufacturer's claimed accuracy is $\pm 1.5 \text{ mm} \pm 2 \text{ ppm}$ making it theoretically possible to measure the longest distance in the network with a root mean square error of less than 5 mm.

The other EDM considered was the Geodimeter model 120, also an infrared device but with only 1 mm resolution. The Geodimeter offered better range, faster measuring time, and was considerably easier to transport and operate. The accuracy claimed was $\pm 5 \text{ mm} \pm 3 \text{ ppm}$ leading to a maximum root mean square error of less than 8 mm.

Both instruments were subjected to complete error analysis tests in Whitehorse before the choice of EDM was made.

EDM ERRORS

EDM errors fall into three classes: cyclic error, constant error and proportional error.

Cyclic error is caused by electrical and optical "crosstalk" between components within the instrument, leading to contamination of the detected phase shift. Since all electro-optical EDM equipment resolve the distance in terms of phase shift, each EDM will exhibit a cyclic error which is directly related to the modulation frequency imposed on the carrier wave. Most manufacturers claim that cyclic error is practically non-existent in their products and for the most part this is true; however aging of components can lead to this error and periodic checks on a suitable baseline should be carried out.

A precise baseline of 10 metres was established at Whitehorse comprising 10 bays of exactly one metre each. The stations were lead plugs in a concrete floor and were centered with copper tacks. This baseline is suitable for those instruments which have a characteristic wavelength of 10 metres (the usual case). The procedure is simply to set up the EDM off one end of the baseline and measure to each station in succession. The absolute distances obtained are not important but the differences between each measurement should be one metre exactly, otherwise cyclic error is present.

The data obtained from the Geodimeter 120 test for cyclic error was so consistent that no plot resembling a typical cyclic error curve could be constructed. The only conclusion that could be inferred therefore was that if cyclic error was present it most certainly had an amplitude of less than one millimetre and could be considered to be zero.

To test the Tellurometer MA 100 the test base had to be modified somewhat as the expected wavelength of the error cycle was 2 metres due to a different modulation frequency. A fully graduated steel bar was laid between the points in the test base and measurements were taken at intervals of 20 cm. A plot of the results revealed a classic sine wave cyclic error curve of 2 metre wavelength that had an amplitude of 7 mm. This was almost 5 times the manufacturer's specifications and was interpreted as an indication of a serious problem within the instrument.

EDM ERRORS continued

No further tests were performed on the MA 100 and the choice of EDM fell to the Geodimeter providing it passed further tests.

Constant error is that property exhibited by an EDM whereby each result differs from the true value by an amount which does not vary in magnitude or in algebraic sign.

EDMs can self test themselves for this error on a suitable baseline. A baseline of 210 metres was laid down consisting of three bays of 70 metres each. These bays were not required to be exact distances but were kept within a few centimetres of being equal lengths so that any cyclic error would not contaminate the constant error test.

The principle of the test is simply that if the EDM measures each bay and then measures the total length, the sum of the first three results should equal the total, otherwise constant error is present. With three bays there are 5 combinations of distances which will give five values of the error.

The results of this test for the Geodimeter 120 were a constant error of -0.0015 mm with a root mean square error of 0.0023 mm. Whenever a statistical mean is less than its standard deviation it can still be interpreted as being the most probable value but our confidence level is very low. It was concluded that the instrument had no detectable constant error.

Proportional error in EDM equipment is that error due to an incorrect modulation frequency which affects directly the nominal wavelength. The result is that greater errors will be exhibited on greater distances and vice versa.

The only way to check for this error is to have a precisely known baseline against which to compare the results. Such a baseline exists in Whitehorse and a comparison of the Geodimeter 120 result with the known distance revealed that the instrument was performing within specifications.

The final test having been passed the decision was made to use the Geodimeter 120 for the deformation study.

TEST OF ANCILLARY EQUIPMENT

Barometers: were compared against the station pressure at the Whitehorse weather office. As a result, corrections were applied to all readings taken in the field,

Thermometers: were also compared against those on the screen of the Whitehorse weather station. Appropriate corrections were applied to all temperature readings.

Tribrachs

& Targets: all were tested and adjusted prior to their use in the field to insure accuracy of their optical plummets.

VERTICAL CONTROL

The vertical datum chosen was that used in the construction of the project, i.e. the Cyprus Anvil Mine datum. All elevations are referred to the bench mark at the mine pumphouse.

A Zeiss Ni007 first order Geodetic level and invar levelling rods were used to run differential levels from the pumphouse to all stations in the network and also to several other monuments which are to be monitored in the vertical direction only. Levels were also run to two bench marks placed on either side of the valley near stations 282 and 292.

The level loop was closed back to the pumphouse and the total misclosure in over 7 kilometres was only 7 millimetres. Therefore no rigorous analysis or adjustment of the results were necessary, and while it is apparent that an accuracy of one centimetre was obtained, one could most likely say with reasonable confidence that accuracies of the order of 3 or 4 millimetres are obtainable.

HORIZONTAL CONTROL

The horizontal datum chosen was also the Cyprus Anvil Mine datum. Horizontal ties were made to 3 existing control points to establish coordinates, in the local coordinate plane, for bedrock station 284. Station 284 was coordinated for use as the fixed station of the control network in the subsequent adjustment.

Control points were chosen on either side of the valley to give the most favourable geometric configuration possible and, at the same time, utilize all available rock outcropping to ensure stability of the network.

The base figure in the control network is a fully observed braced quadrilateral, 3 vertices of which are bedrock stations. Geometrically, this figure is very strong. This can be seen by looking at the error ellipses for station 282, station 292, and station 291; the 3 smallest ellipses resulting from the adjustment. This braced quadrilateral is centrally located with respect to the Golder monuments, giving a very strong basis for the subsequent traverses.

The 4 other major control stations were located with geometric considerations within the topographical limitations to allow as many redundant observations as possible.

Station 290 and station 291 are not bedrock stations (and therefore are not likely candidates for "fixed points") but are used to strengthen the network and thus the accuracy of the major control.

Within this major control lie five precision traverses which contain the Golder monuments installed to monitor the 3 main structures. The backslope monuments, and also those on the spoil piles were tied from the aforementioned traverses.

HORIZONTAL CONTROL continued

Further redundant observations were used to strengthen station 157 and station 559. A cross tie was made between the two dams. Distance and direction observation procedures were carefully designed so as to reduce systematic errors and improve the accuracy of the results as much as practically possible.

Height differences rather than observed zenith angles were used in reducing slope distances to the horizontal. This method necessitates the measuring of the height of the reflector and the EDM above their respective stations, the elevations of which were determined by spirit levels. As a check against errors in these measurements, heights were also measured to the head of the tripod. A quick subtraction would verify the constant relationship between the height of the tripod and the reflector. A simple but effective check against human error.

All prisms used were of identical design and were mounted in the previously tested tribrachs. All distances were measured 3 times. Both the reflector and the instrument tribrachs were rotated by $\frac{360^\circ}{n}$ between measurements; where n is the number of observations. This serves 2 purposes:

- 1) it forces 3 independent levellings and centerings of the reflector and the instrument, thus randomizing the centering errors involved in setting up the tripods.
- 2) the mean of the 3 distance measurements is virtually free of unadjusted optical plummet errors.

This actually reduces the precision of the measurements but increases the accuracy and this is, of course, the main concern.

HORIZONTAL CONTROL continued

Temperature and barometric pressure were measured at both ends of the line for each measurement. The mean of the measurements was used to give an average refractive index along the line of sight. The meteorological correction was calculated using a rigorous mathematical formula rather than relying upon the built-in computer in the EDM.

The measurements were observed in a mode known as "D Stroke". In this mode the instrument automatically repeats its measurement while the built-in calculator integrates the measurements and displays the continuous arithmetic mean.

When a spread of greater than 1 cm was discovered between successive distance measurements, the distance was re-measured. A plot of these spreads showed a normal distribution. This indicates a good set of observations with no suspected blunders or systematic errors. Distances were initially reduced to Mean Sea Level and then factored to a common plane at 1080 m a.s.l. (the average elevation of the job site).

Direction measurements were performed in a similar fashion to that of the distances. Signal tribrachs were rotated by $\frac{360^\circ}{n}$ as in the distance measurements. A Kern DKM-3 was used to observe the directions. This is a first order theodolite, reading direct to 0.5 seconds. Rather than an optical plummet this instrument has a plumbing rod. This necessitated rotating only the bubble on the plumbing rod by $\frac{360^\circ}{n}$,

to effectively eliminate plumbing errors. Three sets of directions were measured; i.e. there were 6 pointings on each target, 3 on each face. The standard deviation of the mean of the observations at each set up was checked before moving to the next point.

The horizontal circle was also rotated between sets. The instrument is designed to mean the readings on opposite sides of the circle, therefore, it need only be rotated by $\frac{180^\circ}{n}$ to eliminate

inconsistencies in circle graduation. Possible inconsistencies in the micrometer were reduced in the same way; i.e. micrometer readings were changed by $\frac{5'}{n}$ between sets.

The sighted signals were opaque, tribrach mounted, plate targets which have a white geometrically symmetric configuration on a black background.

LEAST-SQUARES ADJUSTMENT

To calculate the coordinates of new points (unknowns) the number of observations must be at least equal to that of the unknowns. Geodetic networks typically contain observations far greater than the number of unknowns. An adjustment of the observations, which is necessary before any definitive coordinate values can be computed, should be performed by a rigorous method such as the parametric least-squares method, which allows for a full statistical analysis of the accuracy of the network.

This is only possible on large scale computers. The adjustment of this network was performed at the University of New Brunswick on their IBM computer using the program GEOPAN (Geodetic Plane Adjustment and Analysis).

The basic criterion of the least-squares adjustment is that the sum of the squares of the residuals of the observations is a minimum. This leads to a unique and the most probable solution to the network that contains more observations than unknowns. Additionally, we have the ability to weigh the observations in proportion to the inverses of their a priori standard deviations.

The direct product of the adjustment is the residuals of the observations. This leads to the corrected observations from which the final adjusted coordinates may be computed. A by-product of the adjustment is the covariance matrix from which we can determine many statistical quantities.

ACCURACY ANALYSIS

The accuracy of a surveying network is fully defined if errors of relative positions between any two points in the network are known at a certain confidence level (probability level). Usually a 95% probability that the error of the relative position does not exceed the given value is accepted as the confidence level. The relative position of one point with respect to another point is the difference in the adjusted positions of the two points as defined by the differences of the correspondingly adjusted coordinate values. If the positional accuracy of a point is related to the reference points that are treated as fixed (errorless) in the process of the network adjustment, the term absolute positional accuracy is applied.

Variances and covariances of coordinates of the analyzed network are a basis for calculations of the absolute and relative positional errors. They allow the calculation of the following quantities that are frequently used in describing the positional accuracies:

- Standard deviations of coordinates or differences in coordinates.
- Semimajor and semiminor axes of standard-error ellipses (absolute or relative)

The most general description of the positional accuracy of a point is given in a graphical form by an error curve. Brief definitions of the above terms and quantities are given below.

The variance of a coordinate value or of an observation is a statistical measure of reliability of the value. It is also called mean square error.

The covariance of a pair of coordinate values belonging to one or two points is a measure of statistical dependence of the two values. The same applies to the covariance of two observations. If two coordinates or two observation values are uncorrelated their covariance equals zero. In most cases, coordinates of points that are calculated using the same set of observations are highly correlated.

ACCURACY ANALYSIS continued

The standard deviation of a coordinate value, a difference in coordinates, or an observation is the positive square root of the corresponding variance. It is also called standard error or root mean square error. Standard deviation represents a probability (confidence level) of 68% that the difference between the given (calculated) coordinate and its true value is not more than the standard deviation. To increase the confidence level to 95%, the standard deviation must be multiplied by 1.96.

The standard-error ellipse of a calculated absolute position is an ellipse described by the semimajor and the semiminor axes. If drawn around a point, it is usually interpreted as depicting the region in which we have 39% confidence that it contains the position of the corresponding point determined from errorless observations. To increase the confidence to 95%, the semiaxes of the error ellipse must be multiplied by 2.45. If an error ellipse is calculated on the basis of variances and covariances of differences of coordinates of two points, it is called a relative standard-error ellipse, which describes the relative positional accuracy between the points.

The error curve, also called pedal curve, if drawn around a point describes positional standard deviations in any desired direction at an angle measured clockwise from the semimajor axis of the error ellipse. Error curves are only very slightly larger than error ellipses and the more circular the ellipse, the closer it approximates the error curve.

NETWORK ADJUSTMENT

Input into the GEOPAN program were the approximate coordinates of the stations, the reduced observations, and the a priori estimates of the standard deviations. Using this information the program was run giving us the results of the adjustment.

Included in the output, of interest to us, was the following:

- the final adjusted coordinates of the stations.
- a summary of reduced observations, residuals and adjusted observations.
- the results of various statistical tests.
- the station confidence ellipses.
- the covariance matrix of the parameters.

ANALYSIS OF RESULTS

The most significant fact revealed by the least-squares adjustment was that none of the observations were flagged for rejection at the 99% confidence level.

This is proof that the survey was performed in a meticulous manner. It also indicates that the a priori estimates of the standard deviations were reasonable.

The chi-square test on the variance factor failed marginally at the 99% confidence level. This was not unexpected and is not serious. It is an indication that the systematic errors were not totally eliminated. It also shows that the a priori estimates at the standard deviations might have been slightly optimistic.

The plan accompanying this report shows the error ellipses for each station at the 99% confidence level. It is important to remember that these ellipses are station ellipses and represent the positional accuracy relative to the fixed station 284. These ellipses do not represent the accuracy with which deformations can be determined. Rather they show the normal propagation of errors resulting from increased distance from the fixed station. The plot of the error ellipses provides a graphical means of drawing meaningful conclusions from the adjustment. It is quite apparent, even with minimal study of the plot, that the orientation of the ellipses is evidence of the superior determination of directions as opposed to distance. This, of course, was a direct result of the choice of EDM. However, when one considers for example that movement of the dams in the down-valley direction is of more importance than the cross-valley direction, it is readily apparent from the plot of the ellipses that we have attained a high accuracy in the direction of concern. A similar argument applies to the canal dyke as any movement towards the valley represents the main interest. The stations with the largest error ellipses are those whose movement might represent the least concern, i.e. the backslope and spoil pile monuments.

CONCLUSIONS

The conceptual design of the survey network and the subsequent observations satisfied the initial requirements. This is concluded from the average dimensions of the error ellipses on the two dams in the down-valley direction. At the 95% confidence level the positional accuracy in this direction is 11 mm on the average.

Re-measurement of the network at epoch 1 will provide the information required to determine the fixpoints. It is expected that stations 282 and 292, among others, will become fixpoints with the result that the error ellipses for monuments between these fixpoints will be considerably reduced in size.

One may conclude therefore, that the design and observation of the network is suitable for the detection of deformations, with an order of accuracy of one centimetre. It is recommended that the methods and equipment used at epoch 1 be unchanged from those utilized in the initial observation programme.

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