Control Surveys by Geodimeter and Tellurometer in Canada

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•SEVERAL AUTHORS have reported on uses of electronic distance-measuring equipment in making basic control surveys in Canada (1, 5, 6); consequently, the writer will confine himself to his own experience.

Application of the Geodimeter and the Tellurometer to control surveying is discussed for two distinct projects. The first is for establishing a survey control system of plane coordinates in the Province of New Brunswick. The second is for accomplishing supplemental control for mapping by photogrammetric methods in the Rocky Mountains. An assessment of both cases will lead to a better evaluation of the capabilities of electronic distance-measuring procedures for control surveying. In conclusion, some thoughts are expressed as to the applicability of electronic distance-measuring instruments to surveying for highway engineering purposes.

PLANE COORDINATE SURVEY CONTROL SYSTEM IN NEW BRUNSWICK

General Aspects

New Brunswick, one of the Atlantic Provinces in Canada, covers 28,000 square miles. Its 600,000 inhabitants are living in scattered, more densely populated areas of the Province. Eighty-six percent of the land is forested. Timber and pulpwood production is the prime industry. Inasmuch as more than 50 percent of the land is administered by the government, timber leases constitute a major income, and an indisputable survey of property and lease boundaries is of major concern.

It is for this purpose that the Department of Lands and Mines of the Province has devised a plane coordinate survey control system, based on a stereographic projection onto which all government property records will be based. The private legal land surveyor will eventually follow suit by referencing privately owned land surveys to the same system. The engineer engaged in surveys, be it with the New Brunswick Electric Power Commission or with the New Brunswick Department of Highways, has already realized the importance and the convenience of such a control system for his surveying. Before use of electronic distance-measuring instruments in making control surveys, such survey constituted a luxury which only densely populated areas could afford. Geodetic control, too widely spaced for engineering use, usually could not fulfill a public purpose; the situations are now different. Engineers can make use of geodetic control, since economical densification of the existing trigonometric networks became possible. Such densification of control is a multipurpose proposition. It serves the planner, the engineer, the surveyor, and the mapper equally well, as it also serves the ordinary citizen and taxpayer. There is the possibility of using established plane coordinate positioned survey monuments, over which targets have been placed before photography for use in precise mapping by photogrammetric methods. Much of the usual supplemental control surveying on the ground can thus be greatly eliminated. This is but one of the advantages of such a system. The primary advantage, of course, is the complete recoverability of a point regardless of whether its immediate neighborhood has been affected by an outer force. We find many examples in new subdivisions, in areas of heavy construction, and in areas where destruction or fires have taken place, where position referencing by plane coordinates is the only origin from which certain points can be reset exactly where originally positioned in the ground.

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In New Brunswick, eventually the whole Province will be covered by a sufficiently dense network of monuments comprising points in a plane coordinate control system, which will serve as such reference. The establishment of reference monuments over the entire area is a project for a decade of work. It was started in 1959 using the Tellurometer and Geodimeter.

TABLE 1

SECONDARY TRIANGULATION AND TELLUROMETER TRAVERSING

Method of Control Surveying Including Monumentation, Erection of Towers	Cost per Monument ($)	Time Required per Point (day)
Secondary triangulation	2,160	16
Primary Tellurometer traverse	593	5

Survey Scheme

The primary geodetic network of 140 points, for which markers were set and position surveyed, consists of four quadrilateral chains surrounding the Province. These basic points comprise the starting points for further densification of markers in the network of control. In the experimental stage, it was considered necessary to densify the existing control, spaced at an interval of 30 miles or more, to a secondary triangulation network with control point markers 15 miles apart. From these points, traverses measured by Geodimeter NASM-4B could start. Because secondary triangulation proved to be expensive and slow, secondary control was provided more economically by Tellurometer-measured traverses. These had sides of 10 to 15 miles length, beyond the measurement reach of the Model NASM-4B Geodimeter. Table 1 gives a cost comparison between secondary triangulation and Tellurometer traversing.

The interconnection of secondary control surveying was started by traverses which directly provided the desired control points. These were measured, with control point markers set at an interval of one-half mile to one mile in more densely populated areas, and at an interval of from one to two miles in rural or desolate areas. In areas of difficult intervisibility, the spacing interval for station markers was often smaller. The placement points could always be selected along roads, mainly within their right-of-way. During 1959, 137 monuments were set and position surveyed and in 1960, 350.

It was the aim to survey the monuments with a relative accuracy of 1:20,000 or better. This objective, when achieved, would insure a superior control accuracy compared to accuracy obtainable by subsequent usual survey procedures, such as chaining (1:5,000) or optical distance measurement by subtense bar or horizontal rod tacheometers (1:10,000).

Monumentation and Survey

The control was established as follows:

A Tellurometer traverse-measuring crew selected sights for placement of secondary control station markers, erected towers where necessary, measured distances by use of the Tellurometer MRA-2 in daytime and angles by use of a T-3 theodolite to directional flash-lights at night. Whenever practical, nondirectional propane gas lights, which did not need to be attended and well visible at distances as large as 15 miles, were used at triangulation stations.

A reconnaissance crew determined the location for placement of control point markers to be coordinate position surveyed by Geodimeter measurement of traverses.

A monumentation crew erected the concrete markers on the spot at the chosen location.

A Geodimeter crew measured the distances between the traverse station markers at night. An average of 20 distances from alternate setup points could be measured per night.

Two theodolite-using crews measured horizontal and vertical angles in the traverse during the daytime. Only points where targets were farther than four miles away were position measured by observing on flash-lights at night.

A level crew measured the elevation of some of the station markers and connected them with the first-order geodetic level network so vertical angle measurements could be used to compute trigonometrically the elevation of each station marker.

A field computer checked the measured data and transcribed it onto data input sheets, which were then given to the University of New Brunswick for processing in an electronic computer. All survey crews resided in a centrally located camp.

Computation and Filing

A computer program, developed for the LGP-30 electronic computer at the University of New Brunswick, adjusted the traverse in the New Brunswick coordinate system, using all surveying data. The adjustment provided stereographic and geographic coordinates for each station marker. Also computed and tabulated were adjusted reference distances and azimuths, scale factors introduced by the adjustment and general, lateral, and longitudinal precisions of each traverse as well as its closing error. This information was transcribed onto record cards containing the station marker point sketch, and filed.

In 1961, the survey had progressed from the experimental to the production stage and a detailed analysis as to cost and accuracy could be made for the 500 station markers set and surveyed during the year. This comparison was published in 1962 (3, 4). Results showed operation of the Geodimeter NASM-4B was economical as well as more than sufficiently accurate. Use of the Geodimeter particularly justified use of zigzag traverses, which (under the classical concept) never would have been acceptable. The 1961 survey, which was conducted in Western New Brunswick, proved the Geodimeter to be a highly suitable instrument, although its use at night was inconvenient to the survey crew. In the survey of 1962, which took place along the Bay of Fundy Coast, it was realized local conditions of fog and haze were severely limiting the number of times the Geodimeter could be used. Consequently, at the end of the season, only slightly more than half as many distances were measured as during the previous year.

While the resultant backlog of unmeasured distances could be removed during winter months, it was decided to use the Tellurometer MRA-3 for future surveys. The MRA-3, with nearly the same resolution as the Geodimeter, operates on a 3-cm carrier wave. It is therefore less affected by reflections causing swing than the Model MRA-2, and it also incorporates the advantages of being able to measure distances under more adverse conditions of visibility than the Geodimeter.

TABLE 2

COST ANALYSIS OF NEW BRUNSWICK CONTROL SURVEY*

Cost Factor	No. of Men	No. of Months During the Year	Salary ($)	Living Expenses ($)	Mileage ($)	Materials and Supplies ($)	Amortization of Equipment, 10% ($)	Amortization of Vehicles, 25% ($)	Total ($)	Cost per Monument 1963 ($)	Cost per Monument 1961 ($)
Reconnaissance	2	4	2,800	400	2,000	50	-	-	5,250	8.75	9.00
Monumentation	13	4	8,500	2,600	1,200	5,000	-	400	17,700	29.50	26.20
Angle measuring T-2	5	4	5,000	1,000	1,600	10	480	-	8,090	13.48	14.20
Distance measuring MRA-3	4	4	4,200	800	1,200	20	1,100	250	7,570	12.62	10.00
Elevation measuring by leveling, Tower Bldg., M2	3	4	3,200	600	320	600	900	250	5,870	9.78	8.50
Measurement of primary traverses MRA-2											
Administration	5	4	5,500	1,000	480	30	-	250	7,620	12.10	6.00
Computations using LGP-30 electronic computer	0	12	-	-	-	1,200	-	-	1,200	2.00	1.90
Filing and other office work	3	8	7,800	-	-	150	-	-	7,950	13.25	6.50
Total, 600 monuments	3 fulltime 30 parttime		37,000	6,400	6,800	7,060	2,480	1,150	60,890	-	-
Cost per monument	3 tech. officers 16 students 14 laborers		61.66	10.67	11.33	11.77	4.13	1.92	-	101.48	82.30

*Acknowledgment is given to Col. W. F. Roberts, Director of Surveys of New Brunswick, for this information.

During the 1963 survey, the Geodimeter NASM-4B was replaced by the MRA-3. Measurements could, of course, now be made during daylight hours, which also would have been possible by converting the Geodimeter to a Model NASM-4D. But Maritime weather conditions in New Brunswick favored use of an electromagnetic-wave-using rather than an optical-wave-using instrument.

Cost Analysis

An analysis of cost incurred in the 1963 operation in comparison with cost of the 1961 survey is given in Table 2, which shows both instruments are comparable.

The increase in cost per monument from 1961 to 1963 is mainly due to use of three permanent employees in 1963 as compared to one in 1961. This is more desirable for a responsible operation. Most of the temporary summer employees are surveying and other engineering students at the University of New Brunswick.

Generally, the cost of a monument will also depend on the monument density within an area. Because of saving in time, distance, and expense, a monument in urban areas can be set and position measured cheaper than in desolate regions. In cities, survey markers set at an interval of about one-half mile will cost $50; set in agriculturally used areas at a 1-mi interval, $80; and set in desolate areas, $100 or more.

ACCURACY ANALYSIS

The survey operations from 1959 to 1963 allowed an accuracy comparison for the following three electronic distance-measuring instruments: Geodimeter NASM-4B, Tellurometer MRA-2, and Tellurometer MRA-3.

This accuracy analysis was obtained as a by-product of the survey and not as a separate investigation, which could and should analyze the limitations of accuracy more carefully. It should be stressed, however, that any large control surveying project can be considered incomplete if a fairly reliable assessment of accuracy, such as the one discussed now, has not been obtained.

Internal Observation Accuracy for Distance Measurement

Internal accuracy is obtained by making repeated measurements during one setup of the distance-measuring instrument. Internal accuracy will reflect all accidental errors of measurement and field note recording, and a small part of the systematic errors (swing from the use of various carrier frequencies). For the Tellurometer MRA-2 the standard internal measurement error for the mean out of the 18 finely read measurements distributed over the whole carrier frequency range becomes ± 0.034 m.

For the Geodimeter NASM-4B, the standard error of the mean distance measurement determined from three frequencies is ± 0.009 m. Both values are an average, determined from 20 measured distances each. The average Tellurometer-measured distance was 10 miles, and the average Geodimeter-measured distance was 2 miles.

The Tellurometer MRA-3, when distance measurements are made 20 times at 10 regularly distributed carrier frequencies (cavities), has a standard error of ± 0.013 m for the mean of the distances when measured from one or the other instrument.

Both sets of measurements, however, showed a constant delay difference of 0.038 m ± 0.012 m. Because of this, each distance measurement was repeated from the other station in the same manner, and the mean of both distance measurements was used.

Thus, the determination of a distance using an MRA-3 measurement consisted of 40 fine readings of the measurements. An identical procedure was used for the MRA-2, on which a total of 36 fine readings for a measurement was made.

Internal Accuracy of Distances Under Different Meteorological Conditions

The MRA-2 measurements gave a standard error of ± 0.163 m determined from the double measurement of 10 distances averaging 24 miles in length. This amounts to a precision of 1:235,000. Because different meteorological conditions cause a variation in the velocity of propagation of electromagnetic waves, this ratio is more indicative than the absolute amount of the error.

TABLE 3

DISTANCE MEASUREMENTS ON THE PRIMARY GEODETIC SIDE GREER-CARSON

Method of Distance Measuring	Distance (ft)	Difference from Geodetic Measured Distance (m)	Precision	Difference from NASM-4B Measured Distance (m)	Precision
Geodetic	62,069.26			+0.226	+1: 84,000
MRA-2	62,070.51	-0.381	-1:50,000	+0.091	+1:207,000
NASM-4	62,070.00	-0.226	-1:84,000		
MRA-3	62,070.81	-0.472	-1:40,000	+0.247	+1: 72,000

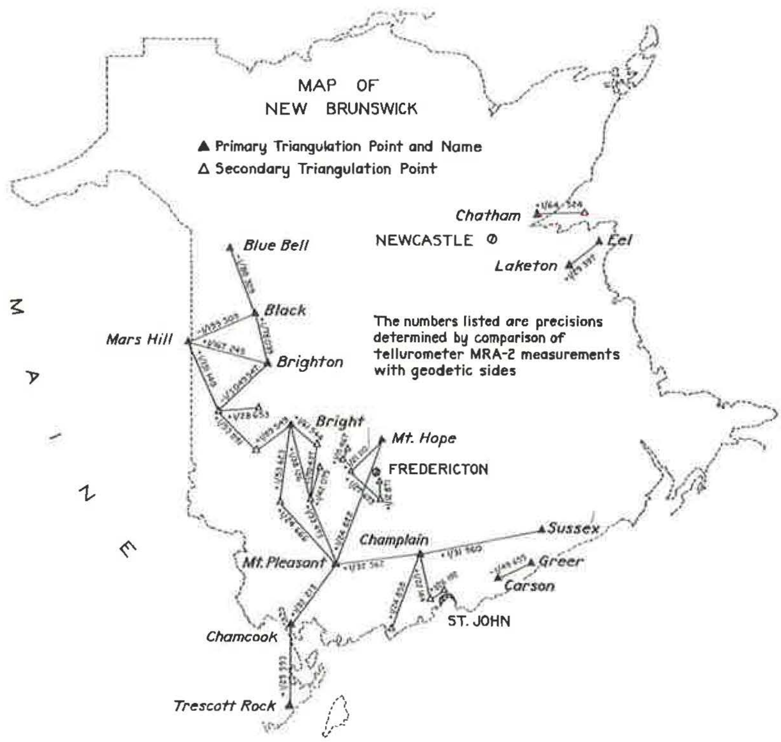

Figure 1. Triangulation sides measured by Tellurometer MRA-2 (Precisions).

A similar comparison was made for the Geodimeter NASM-4B and the MRA-3. The results are NASM-4B: ± 0.018 m for an average distance of two miles, 1:178,000; and MRA-3: ± 0.035 m for an average distance of one mile, 1:45,000.

The precisions, however, are not indicative in this case, because the distances are too short. Both centering and internal observation errors will tend to overshadow the effect of the meteorological conditions, which according to literature (5, 6) should be less than 1:200,000. Reference to this is made later.

Absolute Comparison

One line of the primary geodetic network, the line Greer-Carson, was measured by all three instruments. Table 3 gives the comparison.

There is close agreement between the measurements made with the NASM-4B and MRA-2 instruments. With respect to the geodetic measured distance, all electronic instrument measurements are too long. There may be three main reasons. First, the instruments could have a wrong crystal calibration of the modulating frequencies. Second, the meteorological data assumed along the path were incorrect. Third, the geodetic network is systematically distorted.

That the latter may be valid is indicated by a systematic pattern which prevails in the remeasurement of geodetic distances by use of the MRA-2 (Fig. 1).

Accuracy for Angles

Observing accuracy of the mean of eight sets of angle measurements for one direction was ± 0. 6 second with the theodolite Wild T3, while an accuracy of ± 0. 7 second was attained in use of the Wild T2 theodolite. Effects on the closure error (± m) in traverse surveying by this high accuracy in measurement of angles will only be sensible if provisions are taken for proper plumbing and centering of the theodolite and the sighting targets used.

Due to improper centering of the instrument by $\pm m_I$ and of the sighting target by $\pm m_T$, a standard error m_α for the observed angle will result. Its value in seconds is

$$m_\alpha = \pm\sqrt{m^2 + (\text{arc sin } 1'')^2 \cdot m_I^2 \cdot \frac{\left(1 + \frac{m_T^2}{m_I^2}\right)}{d_j^2}} \qquad (1)$$

This will require that forced centering be applied for the observation of angles if the distance between instrument and sighting target is less than one mile. In this procedure the targets and the theodolite are interchanged, while the tripods remain fixed. Because the requirements are not as critical for distance measurement, distances are more economically measured as a separate operation.

T-2 traversing equipment in conjunction with optical plumbing should be used for making angle measurements when the distance between points ranges to three miles. For distances longer than three miles, flag targetting becomes permissible for traverses. Better than flags are steel poles with metal cross-wings attached to the top. The poles are fastened to the ground by wires, which facilitate plumbing. Inasmuch as angular errors do not accumulate in triangulation, centering specifications need not be as rigid in a triangulation or trilateration network.

Traverse Adjustment and Traverse Closures

The discrepancies shown in Figure 1 represent systematic scale errors. If traverse are to be applied they can be eliminated by choosing a traverse adjustment procedure which will allow for a conformal change of coordinates between beginning and end points (B and E). After distributing the angular closure, the coordinate differences (Δ x and Δ y) can be distributed. If the approximate coordinates are designated as:

$$\bar{x}_i = x_B + \sum_{j=B}^{i-1} d_j \cdot \sin \alpha_j \qquad (2)$$

and

$$\bar{y}_i = y_B + \sum_{j=B}^{i-1} d_j \cdot \cos \alpha_j \qquad (3)$$

the adjusted values become:

$$x_i = \bar{x}_i + \frac{\Delta x(\bar{x}_F - x_B) + \Delta y(\bar{y}_F - y_B)}{(\bar{x}_F - x_B)^2 + (\bar{y}_F - y_B)^2}(\bar{x}_i - x_B) + \frac{\Delta x(\bar{y}_F - y_B) - \Delta y(\bar{x}_F - x_B)}{(\bar{x}_F - x_B)^2 + (\bar{y}_F - y_B)^2}(\bar{y}_i - y_B) \qquad (4)$$

and

$$y_i = \bar{y}_i + \frac{\Delta x(\bar{x}_F - x_B) + \Delta y(\bar{y}_F - y_B)}{(\bar{x}_F - x_B)^2 + (\bar{y}_F - y_B)^2}(\bar{y}_i - y_B) - \frac{\Delta x(\bar{y}_F - y_B) + \Delta x(\bar{x}_F - x_B)}{(\bar{x}_F - x_B)^2 + (\bar{y}_F - y_B)^2}(\bar{x}_i - x_B) \qquad (5)$$

d_j resembles the distance, starting from point j and αj is the azimuth of this distance.

The closing error will be expressed as:

$$e = \sqrt{(x_F - \bar{x}_F)^2 + (y_F - \bar{y}_F)^2} \qquad (6)$$

and it can be split up into its longitudinal and lateral components, Δl and Δq:

$$\Delta l = \frac{\Delta x(\bar{x}_F - x_B) + \Delta y(\bar{y}_F - y_B)}{\sqrt{(\bar{x}_F - x_B)^2 + (\bar{y}_F - y_B)^2}} \qquad (7)$$

$$\Delta q = \frac{\Delta x(\bar{y}_F - y_B) - \Delta y(\bar{x}_F - x_B)}{\sqrt{(\bar{x}_F - x_B)^2 + (\bar{y}_F - y_B)^2}} \qquad (8)$$

Terms also referred to are as follows:

$$1/\textcircled{P} = \frac{e}{\sum_{j=B}^{E-1} d_j},$$ as general precision,

$$1/\textcircled{L} = \frac{\Delta l}{\sqrt{(x_E - x_B)^2 + (y_E - y_B)^2}},$$ as longitudinal precision, and

$$1/\textcircled{Q} = \frac{\Delta q}{\sqrt{(x_E - x_B)^2 + (y_E - y_B)^2}},$$ as lateral precision of a traverse.

The general precision refers to the distance actually measured and its errors, whereas longitudinal and lateral precisions refer to the geometrical relation of starting and ending points and are thus more suitable for an assessment of the accuracy within the traverse network. If systematic scale errors are present, then these will affect the longitudinal precision only. The adjustment procedure, however, eliminates their effect completely. More indicative for the overall accuracy of a traverse network is thus the lateral precision, unless all measured distances, d_j, can be referenced to the scale of the primary triangulation network in the area. This can be done by applying a systematic factor to the measurements, further to reducing a distance for slope, elevation above sea level, and the projection system.

The magnitude of the closing error, its components, and its precisions will generally depend on a number of factors, such as the length of the traverse, the bending of the traverse, the number of measurement segments in the traverse, the order of a traverse within the network (primary, secondary, etc.), and the shortest leg of the traverse, to name but a few. The values for the various traverses of the network can accordingly be analyzed to find indications of how to improve the procedure. Figures 2 to 6 show such an analysis. The examples are only given for lateral precisions for traverses measured by the various instruments.

The various traverse precisions obtained in the New Brunswick survey are given in Table 4.

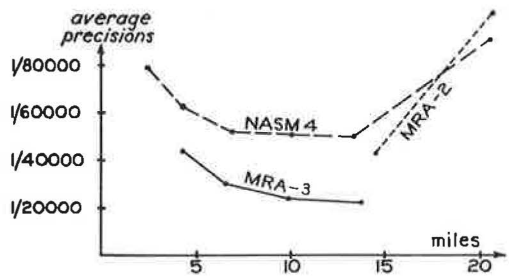

Figure 2. Lateral precision for traverses of various length.

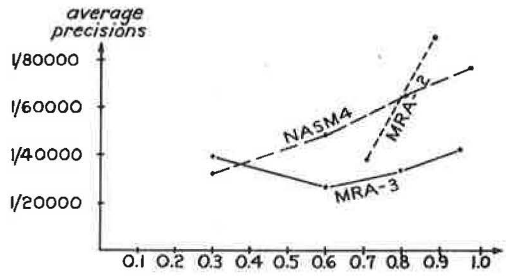

Figure 3. Lateral precision for traverses of various bending ratio

$$\frac{\sqrt{(X_E - X_B)^2 + (Y_B - Y_E)^2}}{\Sigma \, dj}.$$

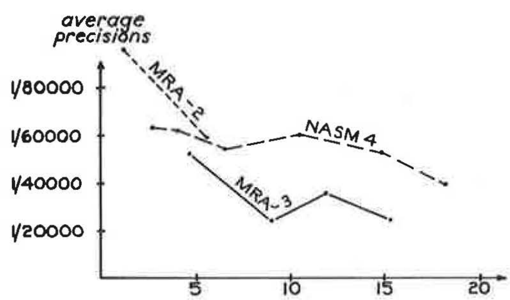

Figure 4. Lateral precision for traverses with varying number of legs.

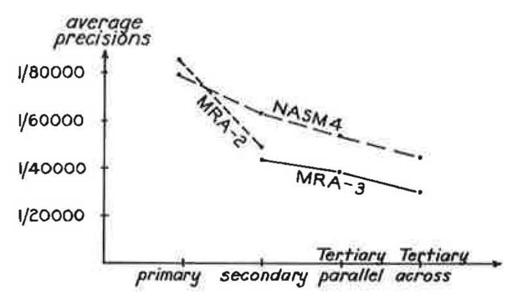

Figure 5. Lateral precision for traverses of various order.

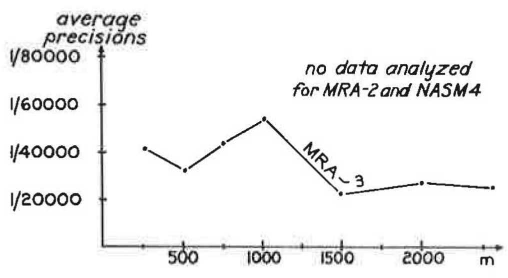

Figure 6. Lateral precision for traverses with varying shortest distance of leg.

TABLE 4

Instruments	1/P	1/L	1/Q
Tellurometer MRA-2 (from 11 traverses)	±1:65,000	±1:88,000	±1:70,000
Geodimeter NASM-4B (from 106 traverses)	±1:37,700	±1:34,000	±1:54,000
Tellurometer MRA-3 (from 31 traverses)	±1:42,000	±1:46,000	±1:42,000

These precisions are based on an average closing error, which is $4/5$ of the standard error, and represent discrepancies before the adjustment. The standard error of a point will be considerably smaller.

A more accurate assessment of accuracy can still be obtained from a least squares adjustment of the traverse loops. The various-covariance matrix will then represent point accuracies. At present, neither the adjustment involving the setup and solution of hundreds of normal equations, nor an analysis of point accuracy was considered necessary or feasible for the 1:20,000 survey accuracy objective.

Beyond any doublt, use of Tellurometer or Geodimeter provides a more economical and more accurate multipurpose system of survey control than any other previous method of making the essential measurements.

CONTROL FOR PHOTOGRAMMETRIC WORK IN THE ROCKY MOUNTAINS

Providing control for photogrammetric work in specific areas is distinctly different from establishing a plane coordinate system of basic control. The aerial photography coverage will determine the location and the density of points for which position must be measured. The type and scale of the photography and character of the topography will determine how the ground control is to be established. Various papers have been written on the subject so it is possible for the author to restrict himself to the task of providing control in the high mountains for mapping using photogrammetric methods. The survey and mapping project was undertaken to determine glacial retreat and volume loss. The work was done jointly by the University of New Brunswick and the Department of Northern Affairs and National Resources, and was supported in part by the National Research Council of Canada.

In the actual case, control was provided for making a terrestrial photogrammetric research survey of the Saskatchewan Glacier in Alberta. The control surveying problems would not have been different basically for establishing ground control for aerial photogrammetric work in the area.

The Saskatchewan Glacier is nearly 15 miles long. It is part of the Columbia Icefield, which covers an area of 100 square miles along the British Columbia-Alberta boundary. The mountain tops are at elevations above 11,000 feet and the valleys extend down to an elevation of 6,000 feet. The only road of the area, the Banff-Jasper Highway, passes three miles east of the toe of the glacier. The toe itself can be reached by four-wheel-drive vehicle on a barely passable trail.

Over such inaccessible terrain, the making of control surveys using electronic distance-measuring instruments has not been done before except when survey equipment and personnel were transported by helicopter. The weight of the Geodimeter or Tellurometer and their accessories are prohibitive for an economical application of these instruments in the usual manner. Contrary to flat and hilly terrain, triangulation is still highly competitive in the mountains.

Because a helicopter, particularly for operation in elevations above 8,000 feet, was far too expensive, this eliminated the use of the Tellurometer.

The Geodimeter has an advantage over the Tellurometer. As compared to both Tellurometer sets, only one Geodimeter unit has excessive weight. Reflector and tripod can be carried easily by a mountaineer. Eight reflectors (4 housings with 7 prisms and 4 housings with 3 prisms) and ten tripods were available for the survey, which was conducted in the following way by a crew of eight:

First, station markers were set at the selected control points. Inasmuch as glacial behavior was to be studied these were not permitted to be on easily accessible moraines, which partially moved with the glacier, but had to be placed on difficult rocky terrain, which was stable. These points were selected where they would be visible from a point accessible by a vehicle, or as close to such a point as possible. They were marked by bronze plugs, drilled and cemented into rock, because they were to be reused. Alongside of the plug an eccentric cairn of two to three feet in diameter and five feet in height was erected out of rocks and painted orange in direction to the terrestrial phototheodolite photography.

Tripod and reflector were then set on top of the plug. Due to high winds, centering was difficult, and the tripod legs had to be fastened by heavy rock piles. After the reflectors were turned in direction to the point from which measurements were to be made by use of the Geodimeter, the station was left. Eight stations could thus be erected in two days, encountering mountain hikes of up to 20 miles per day, with two on the rope.

Second, the Geodimeter station was established in such a way that the equipment had to be carried only a minimum distance. In the case of the Saskatchewan Glacier, this amounted to an elevation difference of 700 feet which was unusually difficult; but with the help of eight people this was not insurmountable.

For check purposes, a second station 1,000 feet away was selected at about the same elevation.

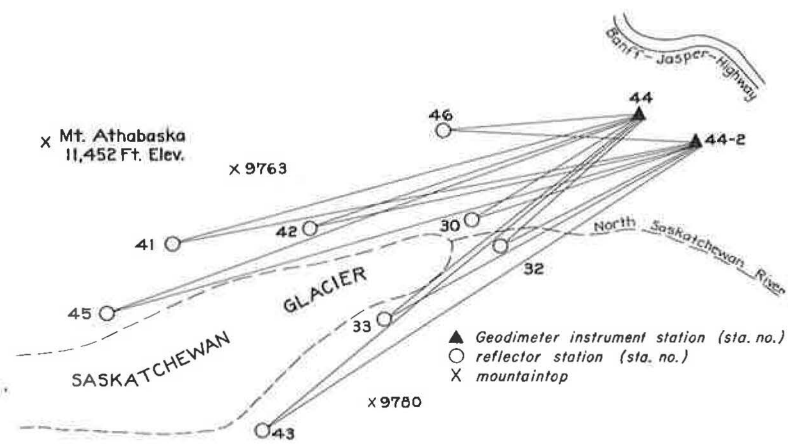

Figure 7. Control survey for photogrammetric mapping of the Saskatchewan Glacier, Alberta.

By use of the Geodimeter, distances could be measured from both of these stations during one night, and the horizontal and vertical angles could be measured the following morning. It is significant that the distance check could be provided without having to reorient the reflectors.

Remeasurement of the eight distances during the following night, despite severe wind conditions, common for this area, agreed within an average of 1:163,000, the longest distance being seven miles.

The reflectors and the tripods were then collected. A layout of the survey is given in Figure 7.

The Geodimeter survey was later extended to connect the survey network of the Saskatchewan Glacier with that of the Athabaska Glacier, 10 miles to the north. A trilateration study was included, and a total of 57 distances was measured. An internal accuracy of 1:182,000 was attained for an average distance of 2,800 meters. The external accuracy was 1:62,000, primarily resulting from centering errors as large as ± 0.040 meter, which were very difficult to reduce because of the wind conditions. In all cases, the Geodimeter instrument stations were kept very close to the road, while the reflectors were carried to the mountain tops, 2,000 to 3,000 feet higher than the road. For this, a crew of two climbed the mountain in the afternoon, established the point and set the reflector. The Geodimeter occupied three to four stations during the night, and subsequent to radio communication the reflectors were reoriented. The crew camped overnight on top of the mountain and measured angles in the morning. Results of this research survey will be published at a later date. Measurement lines on a vertical angle as large as 35° were included in the survey. These could, of course, be measured only by deliberately tilting the tripod of the instrument, by plumbing its center, and by recording height and eccentricity and making the necessary reductions afterwards.

COMMENTS TO THE APPLICATION OF ELECTRONIC SURVEY PROCEDURES TO HIGHWAY ENGINEERING

Survey procedures in highway engineering may include conditions such as those reported in this paper. To identify highway engineering survey problems with those encountered in making a basic control survey would be incorrect.

In highway engineering there is first the work of surveying ground control for photogrammetrically mapping the possible routes at small scale. Second, there is the work of comparing the route alternatives and selecting a route for the highway. And, third, there is the work of surveying ground control for compiling large-scale maps by photogrammetric methods for design of the highway location and preparation of

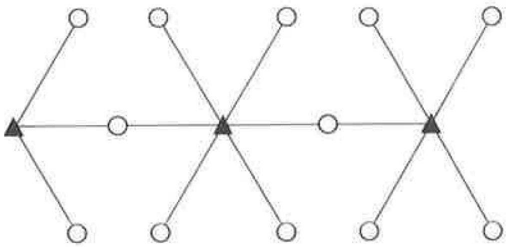

Figure 8. Layout of Geodimeter-measured control for orientation of photogrammetric models (ideal layout).

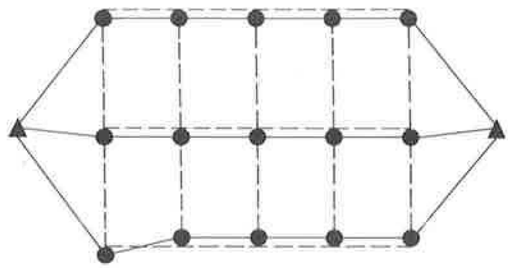

Figure 9. Layout of Tellurometer-measured control for orientation of photogrammetric models (ideal layout).

detailed construction plans. In some areas, however, preliminary surveying for such purposes is done by the usual methods on the ground.

There is no question, however, a multipurpose plane coordinate system of control will make surveys of the first type much easier and less expensive to accomplish. Also the second kind will be greatly facilitated by having a general control survey system for originating and closing each highway survey. Whether it is beneficial to use electronic distance-measuring instruments for making the third type of survey may remain questionable. The limitation is certainly not accuracy, instead it is cost and time. Making a measurement by use of an electronic distance-measuring instrument takes at least 20 minutes, while measuring a short distance by taping or by stadia may be done in less time.

Reference is made to an interesting article published by Gotthardt (2). He compares the applicability of the NASM-4 to problems of making a detailed survey where distances to be measured are only a few hundred feet long. He concludes the strength of making measurements by use of electronic distance-measuring instruments lies in providing basic control, but not in accomplishing detail surveys.

If supplemental control is necessary for orientation and use of stereoscopic models in photogrammetric instruments, the control can be established easily by use of electronic surveying instruments, as indicated in Figures 8 and 9, if the aerial photography scale is not too large. The limiting scale for such photography is yet to be established. For photography scales of 1:2,400, optical distance measuring might be more compatible to both taping and electronic surveying techniques.

Regardless of this, electronic distance-measuring instruments should be used to supercontrol these surveys, so the geodetic principle of working from large areas down to the small areas should be fulfilled for the benefit of maintaining order and reliability. Only then can a survey be termed as being truly professionally done and of service to the general public.

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