The Geodimeter and Highway Surveying

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•DURING THE EARLY 1950's, the Missouri State Highway Commission experimented in the use of photogrammetry as a means of highway surveying by obtaining project mapping from consultant firms. After experimenting on several highway survey projects, the advantages of this method of surveying were apparent and the Commission decided in 1958 to establish a photogrammetric unit within the Department. For highway design and preparation of detailed construction plans, the Photogrammetric Unit would concentrate on mapping at scales of 50 feet per inch in urban areas and 100 feet per inch in rural areas.

The first method of obtaining horizontal control consisted of establishing a centerline and targeting it at a predetermined spacing interval on the ground equal to one-half the airbase of the photography. This assured three targets would appear on each stereoscopic model, and random errors of a greater magnitude than two feet would be apparent when the stereoscopic models were oriented to the vertical and horizontal control. This method of obtaining horizontal control assured all measurements made by use of each stereoscopic model were referenced to the surveyed centerline since it had been staked on the ground and targeted before photography. The disadvantages of this method are (a) the centerline must be surveyed and staked, and this sometimes delays taking the photography; and (b) relocations for which centerline description and point positions are computed in the office are difficult to stake on the ground because errors may exist in the initial preliminary survey traverse.

For relocation surveys, the targets for mapping at the scales of 50 feet per inch and 100 feet per inch consist of muslin crosses, the legs of which are one foot wide and six feet long. Wherever targets are placed on pavements, however, they are painted V-targets and circles reduced in size about 30 percent. Target size must be enlarged proportionately whenever photography is to be taken for mapping at smaller scales.

This method of obtaining horizontal control has worked exceedingly well and continues to be used. In urban areas and in many instances in rural areas, however, it is impossible to establish its position and stake the centerline on the ground in advance of mapping for detailed design purposes, or the best detailed location for the centerline is not evident when using the available reconnaissance survey material. Under either of these circumstances, it is desirable to control the photography for mapping by surveying random traverses and reference tying the identifiable finite image points on the photography to these traverses and using such image points for orienting the stereoscopic models to scale. The preferred method is to survey a traverse between U. S. Coast and Geodetic Triangulations Stations, keeping the traverse as close to the reconnaissance established and recommended centerline as topography and other obstacles and conditions will permit. Permanent monuments to serve as station markers are set at each point of intersection in the traverse and the points for controlling the individual stereoscopic models are obtained using polar surveying from these markers. Plane coordinates of the traverse station markers are computed in the State Plane Coordinate System, as established by the U. S. Coast and Geodetic Survey for the State of Missouri. This procedure allows much latitude over the first method for improving the highway location and for detailed design. It requires, however, more exact horizontal control, which must be computed and closer obtained to the desired accuracy before mapping is started. If a large random error in control is discovered during map compilation work, it is necessary to correct the traverse by resurvey

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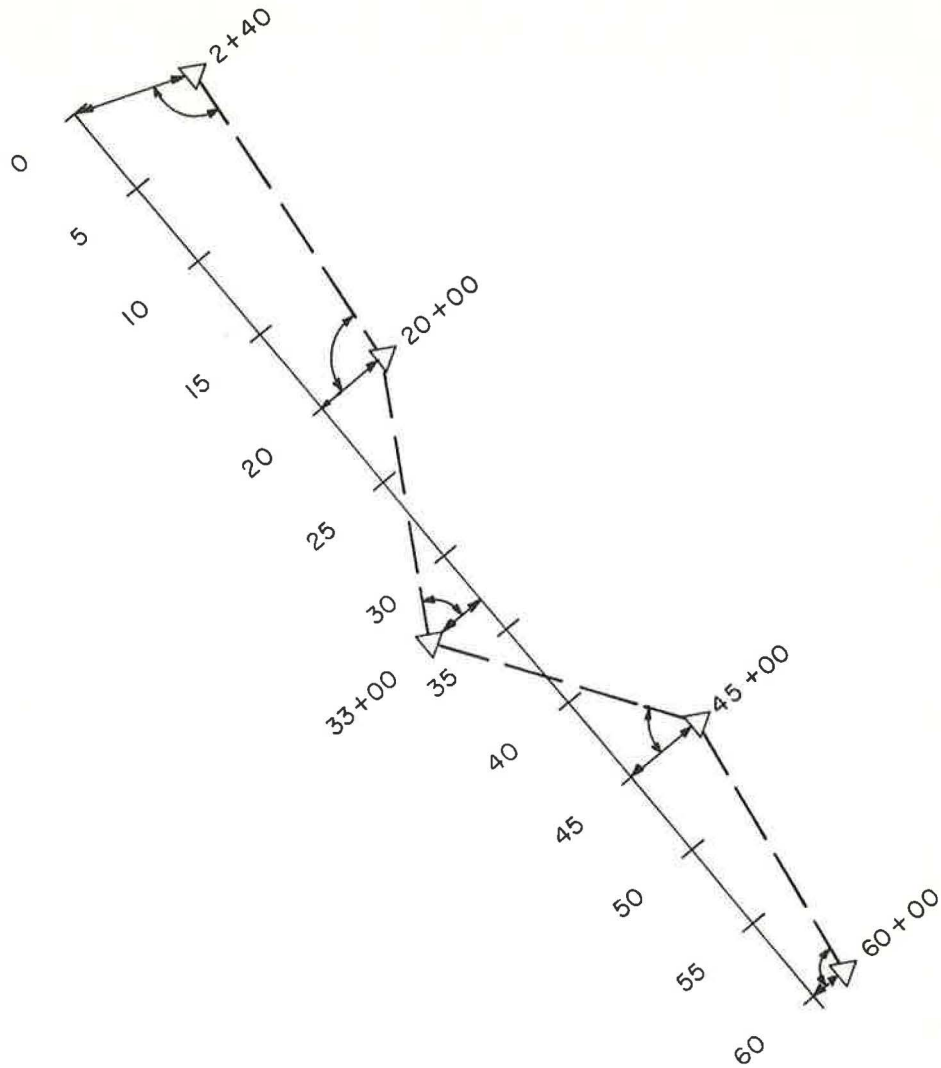

Figure 1. Establishing exact position of plane coordinate computed centerline in the field.

procedures and to recompile the maps affected, because the error has been distributed throughout the entire traverse when the closure adjustments were made. If the horizontal control consists of a highway location centerline staked on the ground, which has been targeted before photography, an equation can be inserted where the error occurred so as to preserve the compilation. After the maps have been compiled, and the description and plane coordinate position of the highway location centerline have been computed and plotted on the map manuscripts, extra care is required to establish the computed centerline in the field in the same respective position in which it was plotted by plane coordinates on the maps. Angles and distances are computed from the traverse marker monuments to instrument station points of the centerline and these points are accurately surveyed and staked in the field. The centerline is then measured and staked between these points; as this work is done corrections are applied as necessary to keep all errors localized (Fig. 1).

A few projects were attempted using conventional chaining and second-order surveying procedures for measuring and staking random traverses. Second-order closures were consistently obtained. Due to the lengths of the traverses, however, some random, locally compensating, errors in chaining were large enough to cause gross errors in the mapping, although the traverses closed within second-order accuracy. It became apparent that, to locate the computed centerline in the same relative position on the ground, traverse surveying without blunders or large random errors was necessary. It was decided a maximum error of two feet in horizontal position would not materially affect the accuracy of design and earthwork computations in most cases in rural areas, and on long traverses it would be desirable to have an unadjusted closure of two feet or less, instead of using the criteria of second-order accuracy which would allow larger errors.

Achieving an accuracy in which the error does not exceed two feet is hardly economically feasible using conventional chaining methods. For this reason, a study was made of electronic distance-measuring devices in 1960. The Model 4B Geodimeter was chosen as the instrument best satisfying the needs for reasons of accuracy, portability, and initial cost. The Geodimeter requires a clear line of sight, must be operated at night, and measurements can be prevented by ground fog. These disadvantages are outweighed, however, by the reasons previously discussed.

A Geodimeter was obtained on a rental purchase agreement to gain experience with its capabilities. After three months of use, it was subsequently purchased along with allied equipment.

An intensive theoretical and operational course by a factory representative can train two men in the operation of the Geodimeter in two days and two nights. Afterward, some difficulty will no doubt occur but, with added experience, each operator will become very proficient. Two skilled operators are assisted by regular survey personnel from the district in which each survey project is located.

The Geodimeter transmits light of known wave lengths to a reflector which returns the light to the receiving optics of the instrument. Three different wave lengths of light are transmitted consecutively. These wave lengths are divided into four equal light pulses or segments called "Unit Lengths." The operator reads on the delay dial that portion of the last unit length transmitted from the instrument. The distance can be obtained from these unit lengths and the sign of the reading because these unit lengths and signs can be obtained for only one distance every 1,000 meters. True, the distance must be known within 2,000 meters, which can be measured roughly by use of a scale on any reliable map. The important point to remember is the entire distance is obtained by measuring within the span of one-fourth of a wave length (roughly 2.5 meters), and large random distance errors are not introduced into the traverse.

The time necessary for setup of the instruments and making two measurements at an instrument station ranges from 20 to 30 minutes depending on the warm-up time required by the instrument. An additional distance can be measured for each additional five minutes the Geodimeter is at one station, if the reflectors are in place over the additional points to which a distance measurement is required. Very short distances between the points selected for use in orienting the stereoscopic models to scale are measured, using a tape, by the angle-measurement party. The Geodimeter operator can measure a distance of 200 feet quicker than the angle-measurement party can get the tape, unwind it, and rewind it after use if the Geodimeter is already set up over a station marker of the traverse station and is at operating temperature. Undesirable chaining conditions and traffic can make it more advantageous to use the Geodimeter. An experienced Geodimeter operator can measure distances of up to one-half mile in length during daylight hours with the Model 4B. To do so, however, requires wedges inserted into the prisms and sometimes requires a partial masking of the receiving optics to keep out disturbing light. Newer models of the Geodimeter have a built-in aperture to keep out this light. Primarily the Model 4B Geodimeter is an instrument for nighttime use and should be used accordingly.

The Model 4b Geodimeter introduces an error not previously encountered in surveying. The error is a maximum ± 0.04 of a foot in each distance measured regardless of the length. This limits the minimum distance which can be measured in a

TABLE 1

COMPARISON OF TWO CONSECUTIVE GEODIMETER-MEASURED DISTANCES FROM THE SAME INSTRUMENT SETUP POSITION

Setup	Measurement (ft)		
	1st	2nd	Difference
1	1,199.398	1,199.384	0.014
2	993.292	993.296	0.004
3	2,204.961	2,204.971	0.010
4	2,391.090	2,391.067	0.023
5	1,089.383	1,089.406	0.023
6	3,749.683	3,749.666	0.017
7	831.417	831.463	0.046
8	1,398.731	1,398.744	0.013
9	3,970.095	3,970.082	0.013
10	4,020.240	4,020.244	0.004
11	3,376.281	3,376.281	-
12	10,463.683	10,463.696	0.013
13	385.063	385.080	0.017
14	7,929.018	7,929.041	0.023
15	2,995.543	2,995.516	0.027
16	6,770.804	6,770.768	0.036
17	1,691.495	1,601.459	0.036
18	599.852	599.849	0.003
19	1,194.175	1,194.195	0.020
20	594.652	594.632	0.020
21	2,113.652	2,113.629	0.023
22	537.774	537.801	0.027
23	1,037.563	1,037.593	0.030
24	1,389.279	1,389.322	0.043
25	1,726.293	1,726.319	0.026
26	387.979	387.960	0.019
27	1,215.190	1,215.177	0.013
28	2,496.207	2,496.230	0.023
29	9,613.469	9,613.463	0.006
30	1,044.399	1,044.392	0.007
31	4,614.226	4,614.213	0.013
32	2,684.848	2,684.822	0.026
33	2,879.998	2,880.040	0.042
34	2,005.543	2,005.520	0.023
35	2,448.616	2,448.609	0.007
36	1,717.318	1,717.328	0.010
37	3,041.517	3,041.540	0.023
38	2,026.796	2,026.793	0.003
39	787.798	787.821	0.023
40	1,164.941	1,164.915	0.026
41	5,688.564	5,688.597	0.033
42	873.631	873.628	0.003
43	140.815	140.779	0.036
44	1,730.432	1,730.422	0.010
45	1,066.880	1,066.906	0.026
46	694.842	694.829	0.013
47	3,760.290	3,760.284	0.006
48	2,252.761	2,252.771	0.010
49	828.207	828.121	0.086
50	2,879.798	2,879.776	0.022
51	1,603.329	1,603.253	0.076
52	990.474	990.440	0.034
53	2,274.310	2,274.324	0.014
54	4,030.240	4,030.210	0.030
55	3,609.262	3,609.282	0.020
56	1,524.586	1,524.609	0.023
57	22,297.846	22,297.852	0.006
58	1,408.616	1,408.617	0.001

traverse to obtain a specific order of closure. Two slope distance measurements have been made during setup of the instrument over a traverse station marker and the same vertical angle measurement is used in computation of the horizontal distance from the slope distance. It is interesting to note the distances varied more than the allowable 0.08 of a foot only once (Table 1). This maximum allowable difference was exceeded by 0.006 of a foot for this one distance. These measurements were made just before the Geodimeter was returned for repair and recalibration, and the D-1, D-2 and D-3 difference in the computations exceeded the allowable spread of 0.10 of a meter. This of course does not prove the correct distance was actually obtained, but it does prove agreement in the separate measurements within the limits specified by the instrument manufacturer. A few errors of 1, 5, 10, 50, and 100 meters were encountered in the computing of the Geodimeter-measured distances. Occurrence of such errors has since been eliminated by rewriting the electronic computer program used to reduce Geodimeter measurements to horizontal distances.

A wealth of material was not available to check Geodimeter-measured distances with distances measured accurately by taping, however, occasionally base lines precision measured by taping for triangulation at bridge sites have been checked. Table 2 gives a comparison between tape-,

TABLE 2

COMPARISON OF TAPE, TRIANGULATION, AND GEODIMETER-MEASURED DISTANCES

Distances (ft) Measured by			Difference (ft)
Taping	Triangulation	Geodimeter	
1,891.09		1,891.062	0.028
1,023.20		1,023.242	0.042
902.19		902.204	0.014
910.73		910.765	0.035
1,200.05		1,200.071	0.021
364.897		364.855	0.042
680.109		680.107	0.002
442.808		442.759	0.041
	2,358.195	2,358.184	0.011
	210.100	210.060	0.040
	2,125.650	2,125.670	0.020
	2,131.536	2,131.643	0.107
	3,018.286	3,018.290	0.004
	2,393.962	2,393.992	0.030
	2,360.990	2,361.010	0.020
	2,265.087	2,265.213	0.126
	2,335.750	2,335.726	0.024

triangulation-, and Geodimeter-measured distances. The difference between the taping and the Geodimeter measuring of distances was in all cases 0.04 foot or less. Two of the triangulation-measured distances differed from the Geodimeter-measured distances slightly more than one-tenth of a foot. It is believed the angle measurements were contributory to this occurrence.

Traverses varying in length from two to 17 miles have been measured with closures varying from 0.2 of a foot to 2.4 feet (Table 3). It is interesting to note 70 percent of these closures were one foot or smaller distance. These closures gave representative fractions of error varying from one in 18,000 to one in 194,000, and the majority of the closures were of the magnitude of one in 40,000. These closures were obtained before any adjustment was applied to the traverse. Other organizations have reported representative fractions expressing error of closure which were smaller than one in 100,000 are the exception. However, errors in closures which are that small are not consistently obtained. These Geodimeter-measured traverses were estimated to be 50 to 70 percent less in cost than were the cost of measuring traverses by conventional taping methods and saved approximately the same percent in time.

The inherent error in Geodimeter-measured distances is not systematic but is of such small magnitude it can, under normal circumstances, be disregarded in the photogrammetric compilation of topographic maps for most of the highway design which has to be done. The advent of electronic distance-measuring devices made measured angles the weakest link in traverse surveying and the utmost care must be exercised in measuring angles if traverse closures are to be maintained within the previously mentioned 2-ft requirement. Care must be exercised in setting the traverse station markers in the ground at positions where long backsights and foresights are provided for the angle measuring and where the effects of heat waves on such sighting and measuring will be held to a minimum. A slight angular error extended through the course of a long traverse will result in an error of closure exceeding the previously mentioned two feet. To check the accuracy of azimuths, and in some cases to help locate angle-measurement errors, the electronic computer program has been written to compute the traverse distances using measurements made forward and in reverse. If the unadjusted closures in plane coordinates between the forward and reverse measurements differ considerably, an angular error should be suspected near one end of the traverse, either at the starting or ending azimuth. Distance-measurement errors, angle-measurement errors in the center of the traverse and systematic angular errors throughout the traverse tend to be compensating and will result in approximately the same error of closure in both directions. These observations regarding errors are not made as statements of fact, but are used as aids in analyzing traverses for error. Much depends on the bearing of the individual courses in the traverse, the azimuth between the beginning and ending triangulation stations, and faith that the Geodimeter-measured distances contain no large blunders. The time added in computing both the forward and reverse measurements of each portion of a traverse is very small compared to the information gained toward obtaining more accurate results.

The Geodimeter has been returned for repairs, recalibration, and tuning two

TABLE 3

ACCURACY OF GEODIMETER-MEASURED TRAVERSES

Traverse No.	Angular Error (sec)	Denominator of Representative Fraction of Closurer Error	Error of Closure (ft)	Length of Traverse (mi)
1	10	71,000	0.5	6.7
2	3	41,000	1.5	11.4
3	13	18,000	2.3	7.5
4	2	40,000	2.2	16.3
5	11	46,000	1.0	8.8
6	6	28,000	1.6	8.4
7	19	21,000	2.0	8.1
8	8	194,000	0.2	8.8
9	1	50,000	0.4	3.6
10	10	38,000	0.8	6.0
11	5	53,000	0.4	4.1
12	17	44,000	1.4	11.5
13	8	82,000	0.7	10.2
14	20	29,000	1.0	5.6
15	22	42,000	0.4	3.4
16	8	52,000	0.4	4.1
17	2	62,000	0.2	2.7
18	5	19,000	0.5	3.6
19	4	160,000	0.1	2.0
20	3	81,000	0.1	1.2
21	9	92,000	0.1	1.9
22	4	29,000	0.3	2.0
23	9	139,000	0.1	1.6
24	2	94,000	0.3	5.0
25	20	20,000	2.4	9.5
26	15	30,000	1.3	7.6
27	1	48,000	0.3	3.2
28	7	68,000	0.5	6.2
29	26	27,000	0.8	3.9
30	7	46,000	0.7	6.1
31	17	73,000	1.1	15.7

times since it was purchased in April 1961. The first time was for repair of a microswitch which had failed. The cost of such repair, recalibration, and shipping was $254.65. The Geodimeter was shipped on July 11, 1962 and returned on August 8, 1962. It was again necessary to return the Geodimeter to the factory in the summer of 1963 because it was intermittently dead when switched to frequency No. 2. The total cost of a tube, resistor, power cable, calibration, and shipping was $114.19. The Geodimeter was shipped on Thursday and returned the following Thursday--a considerable improvement over the previous time duration for repair. Other difficulties encountered include a broken power cord, faulty tubes, mirror out of adjustment and faulty Kerr-cell heater. These items were repaired by unit personnel. Based on past experience, the Geodimeter probably should be returned yearly to the factory for overhaul, and should be recalibrated every six months according to the manufacturer's recommendations. Due to a recent sudden change in calibration of the Geodimeter, it has become apparent that calibration is very important and should be accomplished whenever a spread of more than 0.06 of a meter occurs regularly during computation of horizontal distances using the field-made measurements. Instructions for calibration have been recently published by the manufacturer.

Too little attention has been given the accuracy of horizontal control for mapping by photogrammetric methods. This is especially true for large-scale mapping where a designed centerline must be staked on the ground, where positioned and plane coordinate computed on the map. The Missouri State Highway Commission has used only the Geodimeter. There are other instruments available, however, which can probably do as well. It is believed that horizontal control established by use of accurate electronic distance-measuring devices will aid in bringing further acceptance of such methods for highway surveying by photogrammetric methods.