Use of Modern Measuring Devices for Establishing Designed Alignment

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•THE Illinois Division of Highways purchased an MRA-1 Tellurometer in 1958. This instrument was used to make second-order horizontal control surveys to determine the position of control point monuments set at an interval of about 3 miles along 750 miles of the proposed Interstate Highways. This work was accomplished by a joint agreement with the U. S. Coast and Geodetic Survey, U. S. Bureau of Public Roads, and the Illinois Division of Highways. From the early part of 1959 until late in the year of 1961, the U. S. C. & G. S. used the instrument almost constantly in this second-order traverse surveying. There were short periods of time, however, when the State requested the instrument be made available for surveying basic horizontal control needed for topographic mapping by photogrammetric methods for highway location and design. Since late 1961, the instrument has been used by the State for control surveys and for checking distances measured by triangulation at major stream crossings. In highway work, it has been determined the MRA-1 Tellurometer has certain limitations. The distance to be measured must be long enough to achieve second-order accuracy. When such limitations are taken into account, unsatisfactory results are usually caused by something other than the instrument.

This paper is limited to the discussion of two survey projects. The MRA-1 Tellurometer was used on one project and the Tellurometer and Model 4B Geodimeter were used on the other. These instruments were used primarily to determine the accuracy in position of control points set along the staked centerline of the proposed highway.

Since topographic maps compiled by photogrammetric methods were made available for use by highway engineers, they desired maps of a corridor or band of topography on which a detailed alignment could be designed, staked on the ground in such a way the original map would be satisfactory for completion of design and preparation of highway construction plans, and for computation of construction pay quantities. If all control surveying and positioning on the ground of designed and plane coordinate defined highway alignment were accomplished with second-order accuracy procedure, this system would be entirely workable, but past experience indicates second-order accuracy is not generally obtained. In fact, second-order surveying methods are not employed. On the two highway survey projects to be discussed, a different approach than customary was taken to solve the problems of using the designed location as accurately delineated on the maps without sacrificing accuracy.

METHOD NO. 1

The first highway survey project to be discussed is a section of Federal-Aid Interstate 57, 10.04 miles in length. An engineering agreement was executed in February 1962, with a consultant to make the preliminary survey, accomplish the design, and prepare detailed construction plans. The consultant desired to use aerial methods in making the preliminary survey. As a result of a conference, in which certain procedures were agreed on, he was permitted to proceed. The alignment had been tentatively fixed and delineated on an uncontrolled photographic mosaic. The Illinois

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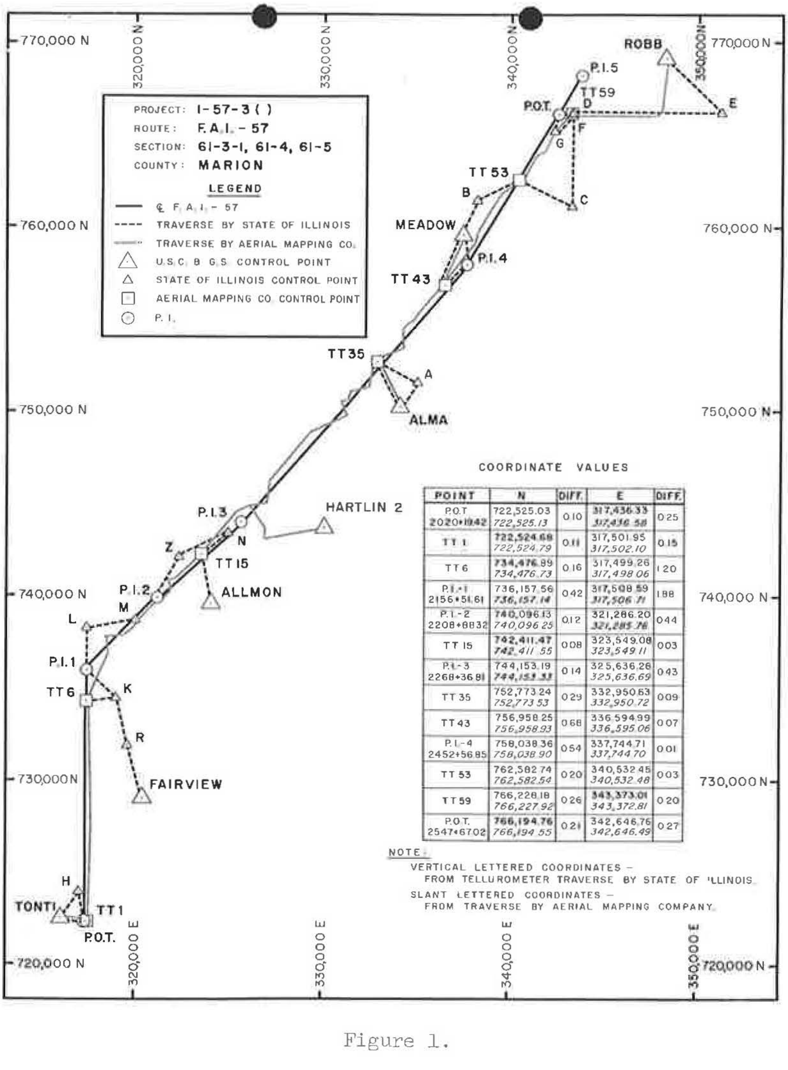
Division of Highways specifications for highway design mapping require the centerline be targeted at an interval of 400 feet and a profile to be measured on the ground for indexing the cross-sections when they are measured from the contours of the photogrammetrically compiled highway survey maps. In this instance it was apparent the tentatively positioned centerline was subject to minor adjustments. Thus the consultant was permitted to survey and target a traverse as much as 300 feet from that centerline in order to avoid certain obstructions such as timbered areas. The traverse was chosen and surveyed and targets were placed at stations marked with steel pins which were referenced sufficiently for subsequent recovery. For the topographic mapping to be done photogrammetrically, the control surveying was done later by the mapping contractor using third-order methods, originating and closing on U.S.C. & G.S. horizontal and vertical control. The surveying complied with third-order standards and a large-scale topographic map was compiled using field-measured elevations at the targeted stations to strengthen the vertical accuracy near the proposed centerline. After the map was completed, the highway alignment was adjusted to what was considered to be the optimum position. The consultant then staked the designed alignment on the ground by measuring from certain targeted stations (the coordinates of



POINT	N	DIFF.	E	DIFF.
P.O.T. 2020+10.42	722,525.03 722,525.13	0.10	317,436.33 317,436.58	0.25
TT 1	722,524.68 722,524.79	0.11	317,501.95 317,502.10	0.15
TT 6	734,476.89 734,476.73	0.16	317,499.26 317,498.06	1.20
P.I.-1 2156+51.61	736,157.56 736,157.14	0.42	317,508.59 317,506.71	1.88
P.I.-2 2208+88.32	740,096.13 740,096.25	0.12	321,286.20 321,285.76	0.44
TT 15	742,411.47 742,411.55	0.08	323,549.08 323,549.11	0.03
P.I.-3 2268+36.81	744,153.19 744,153.33	0.14	325,636.26 325,636.69	0.43
TT 35	752,773.24 752,773.53	0.29	332,950.63 332,950.72	0.09
TT 43	756,958.25 756,958.93	0.68	336,594.99 336,595.06	0.07
P.I.-4 2452+56.85	758,038.36 758,038.90	0.54	337,744.71 337,744.70	0.01
TT 53	762,582.74 762,582.54	0.20	340,532.45 340,532.48	0.03
TT 59	766,228.18 766,227.92	0.26	343,373.01 343,372.81	0.20
P.O.T. 2547+67.02	766,194.76 766,194.55	0.21	342,646.76 342,646.49	0.27

NOTE:
VERTICAL LETTERED COORDINATES – FROM TELLUROMETER TRAVERSE BY STATE OF ILLINOIS.
SLANT LETTERED COORDINATES – FROM TRAVERSE BY AERIAL MAPPING COMPANY.

Figure 1.

which had been determined by field survey) to positions determined from their coordinates for points on the centerline as designed on the map. It was originally agreed the consultant would eventually measure a profile of the field-staked centerline and adjust the cross-sections measured from the map to the field-measured profile (1). Illinois Division of Highways Specifications for mapping require the consultant to measure fifteen consecutive cross-sections by precise field surveying methods within each five miles or portion thereof. Such cross-sections must comply with requirements set forth in Section 60 of the Reference Guide Outline. The possibility of slight discrepancies in position staking of the centerline on the ground created doubt as to the validity of testing the mapping in this manner. In consequence of this doubt, it was decided to measure an independent Tellurometer traverse through the U. S. C. & G. S. Control, some of the mapping contractor's control, and through the consultant's field established P. I. 's and P. O. T. 's to reconcile the ground positioning of the centerline with the designed and plane coordinate computed position of the centerline on the maps. The two Tellurometer-measured traverses attained closures between second-order U. S. C. & G. S. stations of 1:16, 000 and 1:19, 000, respectively. As shown in Figure 1, the plane coordinate position of points on centerline measured by Tellurometer traverse agreed with the plane coordinate position determined from the map within 0. 69 of a foot except for one P. I. which disagreed by 1. 93 feet (Fig. 2). At a point only 1, 681 feet away the mapping company's surveyed control station differed by 1. 21 feet in a similar direction leaving a difference of only 0. 7 feet, which indicated the P. I. had been properly located with respect to the position designed on the map and this map positioning was good locally, but probably not generally as good as all other positioning was throughout the remainder of the maps. Although the original control surveying was accomplished to an accuracy of third order or better, it had previously been determined this large error occurred in the area where the original control was weakest.

On the basis of results achieved in checking the horizontal surveying, it was decided to ask the consultant to check individual cross-sections in accordance with his agreement. A total of 31 cross-sections were measured in the field and compared

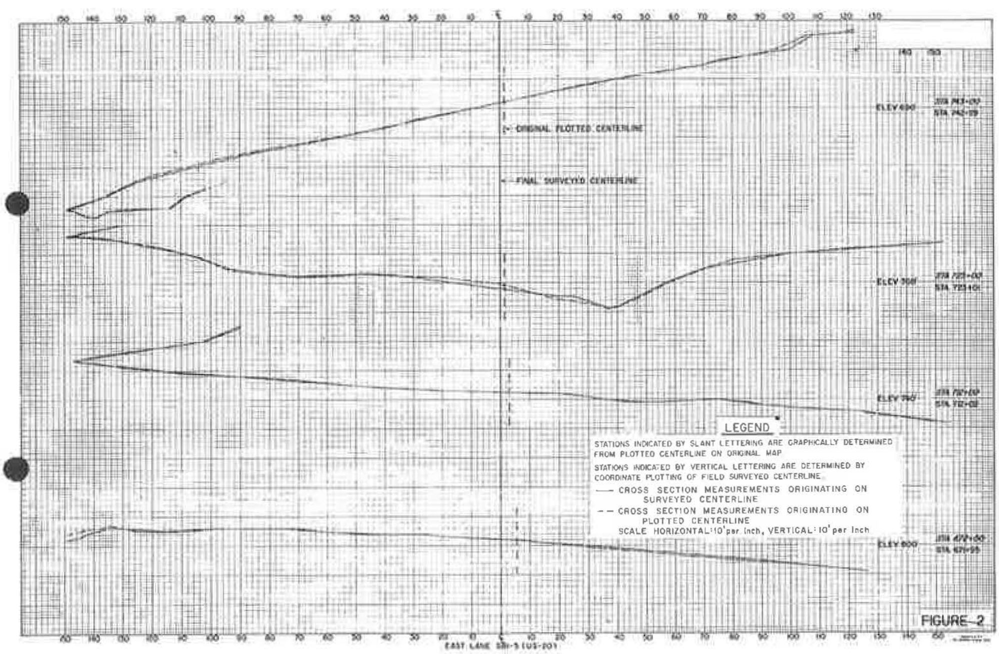


Figure 2.

with cross-sections measured from the maps. The mean difference between the cross-sections measured from the maps and the field-measured sections was (+) 0.25 of a foot, which was outside the limits set forth in Section 60 of the Reference Guide Outline for Aerial Surveys and Mapping by Photogrammetric Methods, 1958. At this point it was decided the consultant should establish at least one centerline elevation in each stereoscopic model before attempting adjustment of any cross-section data. Results of this check indicated a mean overall error of (+) 0.09 of a foot. On the basis of these results, attempts to adjust cross-section data could not be justified.

METHOD NO. 2

The other project to be discussed presented a different and unique problem. In 1955 an aerial survey company was engaged to map 17.5 miles of US 20 in JoDaviess County. Two types of topographic maps were compiled. One was a reconnaissance type map at a scale of 200 feet to one inch containing contours at a 5-ft interval for route location purposes. The other was a large-scale map at a scale 50 feet to one inch containing contours at a 1-ft interval for design and preparation of detailed construction plans. These maps were compiled photogrammetrically using photography taken in 1955, without the benefit of targets.

In 1962 additional control was surveyed by State forces for extending this mapping to the east for location of an additional section of the proposed road. In so doing, the traverse was tied to one of the control points used for the original mapping and a 10.8-ft datum shift was discovered. At this time it was decided additional checking of the mapping was necessary. A traverse was measured by use of the Tellurometer through the entire 17.5-mi section and ties were made to several control points used for the original mapping. This survey also verified a datum shift in control used for the mapping.

In order to measure cross-sections from the large-scale maps and be assured there was no appreciable shift in the alignment or the plane coordinate grid, positions from the plane coordinates were established for centerline points by Geodimeter-measured traverse. Positioning for these control points on the ground was determined from the large-scale maps by scaling distances from various planimetric features on the maps. The Geodimeter-measured traverse was tied to several of the Tellurometer-measured traverse points through the 17.5-mi area and to additional original control points.

It was decided additional control points used for the original mapping should be incorporated into the Geodimeter-measured traverse so further checks could be made on any datum shifts. Positions determined from the Geodimeter-measured traverse permitted the accurate plotting of the designed highway alignment on the large-scale maps and, at the same time, correct for known errors in datum by shifting the plane coordinate grid on the maps. The primary objective of this procedure was to evaluate the maps in terms of their suitability for measuring cross-sections from them. The mapping company measured cross-sections by scaling offsets and interpolating elevations from the map for a 4.4-mi section of this route.

In the spring of 1963 photography was obtained at the negative scale of 250 feet per inch using a Wild RC8 aerial camera, for the purpose of checking the maps by photogrammetric methods. This checking is patterned after the method reported by Katibah (2) except new photography was obtained. This photography covered the 4.4-mi section over which the mapping company had measured cross-sections from the maps. Before this photography was taken, targets were placed at an interval of 400 feet on the field-surveyed and staked centerline. The elevation of each target was measured by field survey. Table 1 contains a comparison of the field-surveyed and staked centerline. The elevation of each target was measured by field survey. Table 1 contains a comparison of the field-surveyed elevations with elevations interpolated, measured from the contours of the maps. It will be noted the average error is 0.592 feet, and the mean error is (-) 0.016 feet. In this test the mean error is significant in that it indicates earthwork quantities derived from cross-sections measured from the maps would probably be correct, because the plus and minus differences are nearly in balance for the fifty points tested.

TABLE 1

COMPARISON OF ELEVATIONS MEASURED FROM MAP WITH ELEVATIONS MEASURED BY FIELD SURVEY

Targeted Point No.	Elevation Feet from Map	Elevation Feet by Field Survey	Difference (ft)	
			+	-
H 3	854. 20	854. 97		0. 77
H 4	852. 40	852. 92		0. 52
H 5	860. 30	861. 21		0. 91
H 6	842. 60	842. 86		0. 26
H 7	808. 60	809. 09		0. 49
H 8	823. 30	823. 88		0. 58
H 9	808. 00	807. 84	0. 16	
H 10	841. 00	841. 66		0. 66
H 11	820. 80	820. 81		0. 01
H 19	747. 00	747. 50		0. 50
H 20	774. 70	775. 00		0. 30
H 21	763. 60	763. 38	0. 22	
H 22	799. 30	799. 16	0. 14	
H 23	789. 40	790. 44		1. 04
H 24	794. 30	794. 75		0. 45
H 25	789. 20	790. 32		1. 12
H 26	791. 20	791. 54		0. 34
H 27	797. 00	797. 42		0. 42
H 28	774. 90	774. 07	0. 83	
H 30	724. 00	723. 73	0. 27	
H 31	665. 90	666. 06		0. 16
H 32	637. 50	637. 29	0. 21	
H 33	624. 00	624. 84		0. 84
H 34	621. 70	622. 32		0. 62
H 35	612. 50	611. 84	0. 66	
H 36	677. 00	674. 30	2. 70	
H 37	727. 95	727. 24	0. 71	
H 38	737. 40	737. 03	0. 37	
H 39	673. 60	674. 14		0. 54
H 40	729. 10	727. 54	1. 56	
H 41	800. 90	801. 36		0. 46
H 42	778. 00	778. 51		0. 51
H 43	726. 30	725. 76	0. 54	
H 44	718. 00	719. 62		1. 62
H 45	714. 90	714. 60	0. 30	
H 46	676. 20	675. 40	0. 80	
H 47	630. 50	630. 72		0. 22
H 48	602. 30	601. 84	0. 46	
H 49	601. 30	601. 16	0. 14	
H 50	642. 20	642. 36		0. 16
H 51	639. 40	639. 52		0. 12
H 52	687. 60	687. 32	0. 28	
H 53	704. 80	704. 74	0. 06	
H 54	703. 00	702. 23	0. 77	
H 55	701. 20	701. 46		0. 26
H 56	738. 20	735. 86	2. 34	
H 57	765. 45	766. 53		1. 08
H 58	802. 90	802. 67	0. 23	
H 59	864. 80	865. 04		0. 24
H 60	900. 45	899. 80	0. 65	
		Total	14. 40	15. 20

Note: Average error 0. 592 (ft). Mean error -0. 016 (ft).

Additional checks were made of the cross-sections measured by the mapping company. This was done by measuring eleven cross-sections from the map in referencing to the field-surveyed and staked centerline. Figure 2 shows a comparison of four cross-sections considered typical of the area. The eleven cross-sections remeasured indicate the field surveyed centerline was properly correlated with the designed centerline in the position intended for it, according to the planimetric and topographic features of the maps. It further indicates the actual position plotting was correct within reason. The second check consisted of using two stereoscopic models in the Kelsh stereoscopic plotter and measuring 17 cross-sections. These cross-sections were compared with sections measured from the maps by determining the end areas of each of the two sets of cross-sections. Results of this test were a mean difference of 0.10 of a foot and an average difference of 0.64 of a foot, which is well within the limits set forth in the Reference Guide Outline.

Nine additional cross-sections were measured using a third stereoscopic model; the results of all 26 cross-sections tested are given in Table 2. This stereoscopic

TABLE 2

COMPARISON OF CROSS-SECTIONS MEASURED FROM TOPOGRAPHIC MAP WITH CROSS-SECTIONS MEASURED PHOTOGRAMETRICALLY USING RECENT PHOTOGRAPHY

Station	Area		Length of Cross-Section (ft)	Mean	Error	Error	
	(+)	(-)		(+)	(-)	Avg.	Max.
712+97.2	130.1	31.3	370.0	0.27		0.44	1.2
713+98	82.3	76.4	353.0	0.17		0.45	1.6
714+98	8.7	211.8	359.0		0.57	0.61	2.8
715+97.3	53.8	198.4	374.0		0.39	0.67	2.5
716+98.5	20.6	116.1	347.0		0.28	0.39	1.8
717+98.5	78.9	508.4	414.0		1.04	1.42	5.6
718+99.5	252.4	136.2	362.0	0.32		1.07	3.0
719+98.5	331.1	17.6	387.0	0.81		0.90	3.0
720+98.7	358.7	33.9	328.0	0.99		1.20	4.8
773+98.8	33.3	38.2	272.0		0.02	0.03	1.0
774+98.9	166.4	64.1	276.0	0.37		0.84	1.5
775+98	238.3	2.8	280.0	0.84		0.86	1.6
776+98.4	244.5	10.0	282.0	0.84		0.90	1.9
777+97.8	323.2	19.7	319.0	0.95		1.07	2.7
778+98.8	376.6	52.8	303.0	1.07		1.42	7.5
779+98	172.7	105.7	294.0	0.23		0.95	4.0
780+98.5	232.7	111.6	291.0	0.42		1.18	3.0
781+98.4	378.1	145.5	298.0	0.78		1.27	14.7
791+97	60.0	51.2	301.0	0.03		0.37	1.2
792+96	180.3	74.9	299.9	0.35		0.85	2.4
793+97	127.2	74.3	299.0	0.18		0.67	2.2
794+97	36.8	52.3	296.0		0.05	0.30	1.0
795+97	15.9	63.2	294.0		0.16	0.27	1.2
796+98	73.1	25.7	291.0	0.16		0.34	1.3
797+98.5	171.3	1.9	289.0	0.58		0.60	1.2
798+98	115.5	1.7	287.0	0.40		0.41	0.9

Note: Photogrammetrically measured cross-sections are assumed to be correct. Mean difference = 0.28 (ft), average difference = 0.75 (ft), and maximum difference = 14.7 (ft).

model contained difficult terrain and a comparison of the elevation field measured for targeted points with the elevation measured from the map for similar stationed points of the designed centerline indicated there might be trouble. This combined test does not meet the requirements set forth in Section 60 of the Reference Guide Outline.

The mean difference for the separate cross-sections of any one centerline stationing point was determined by using the algebraic sum of the end areas of the two sets of sections (the cross-sections measured photogrammetrically were assumed to be correct) and dividing by the total length of the separate cross-sections. The maximum difference is the discrepancy between the elevation of points on cross-sections measured from the maps and measured directly from the stereoscopic models. When due consideration is given to the comparison of elevations in Table 1, and the test of 17 cross-sections, there is a strong indication that, if more cross-sections were tested, the results would meet requirements set forth in the Reference Guide Outline.

The field costs for surveying and staking the designed alignment on the ground were as follows:

Method No. 1—$398.41 per mile. This work was done by the consultant and these costs were obtained from him.

Method No. 2—$956.88 per mile. This work was done by State forces except for the Geodimeter work which was done by agreement with a consultant. He was paid $2,998.71 for this work, which is included in the cost per mile.

No attempt has been made to compare the cost of office computations for the two methods. The necessary office work to accomplish the staking in Method No. 1 amounts to approximately $100 per mile. At this time we have spent almost this much per mile in Method No. 2, but have only completed about 4 ½ miles. We estimate the cost will amount to well over $200 per mile. Part of this additional cost is explained by the fact the plane coordinate grid will necessarily have to be shifted some to make the map good locally throughout the last 12 miles. This will be accomplished by comparing the original plane coordinate positions for the control with the corrected positions, as determined by use of electronic and light source distance-measuring devices.

None of the costs for testing either by field surveying methods or by photogrammetric methods have been included. It was assumed that, if the designed alignment were properly positioned, the cost of accuracy tests would be similar regardless of how the staking was accomplished.

After all the foregoing tests were completed and fully evaluated, the following conclusions were made:

1. A field-surveyed and staked centerline can be correlated with the designed and plane coordinate computed position of the centerline on the maps by using precise field surveying methods for measuring the plane coordinate position of actual centerline points when the staking is done on the ground.
2. A topographic map compiled before the designed centerline was established on the ground can be used for measuring cross-sections.
3. A datum shift can be accurately determined using modern electronic distance-measuring equipment.
4. Advance targeting on the ground of stationing points on the centerline staked for the proposed highway (sometimes referred to as a base line) yields more reliable results at a lower cost than is generally realized where remote control is established after photography is acquired.

REFERENCES

1. Funk, L. L., "Adjustment of Photogrammetric Surveys." HRB Bull. 228, pp. 21-27 (1959).
2. Katibah, G. P., "Photogrammetric Map Checking." HRB Bull. 312, pp. 18-28 (1961).