HIGHWAY RESEARCH RECORD

Number 65

Photogrammetry and Aerial Surveys 1963

13 Reports

Presented at the
43rd ANNUAL MEETING
January 13-17, 1964

HIGHWAY RESEARCH BOARD
of the
Division of Engineering and Industrial Research
National Academy of Sciences—
National Research Council
Washington, D. C.
1965

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Contents

Electrotape Use in Establishing Basic and Supplemental Control for Aerial Surveys

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•SINCE THE Federal Highway Projects Office of Region 9, U. S. Bureau of Public Roads, first began to make aerial surveys in 1956, a continual effort has been made to up date survey methods and equipment in order to reduce costs and to increase accuracies. The methods and equipment employed in obtaining distance measurements have varied. Conventional taping methods were initially employed in this phase. Due to ruggedness of the mountainous terrain and lack of expert chainmen, however, this method soon proved to be expensive and to be the source of many of the ground control errors. It is not difficult to assume that errors in tape-measured distances had existed prior to aerial surveys; however, their detection was much more difficult than with current photogrammetric methods of checking measurements.

To overcome taping mistakes and to speed up the field control surveying work, the 10-ft portable subtense bar was placed in operation in 1956. Later, in 1959, this bar was redesigned and lengthened to 12 feet. Although sometimes clumsy to operate in rugged terrain, the distances measured by use of the bar were usually much more accurate than distances measured by taping. The hours required to measure a distance were in many cases reduced by four to six times. Because the theory of the subtense bar is based on a set bar length and a horizontal angle measured between the ends of the bar, the most obvious source of error comes from measuring the angle. Herein lies the fault of the subtense bar. Adverse weather conditions, poor operation of the angle-measuring instrument, and improper handling of the bar all contribute to incorrectly measured angles and, thus to obtainment of distances which are in error.

To provide a check on the accuracy of the supplementary control, whether surveyed by tape or subtense bar measuring, markers of basic control check points were set at an interval of 2 to 3 miles. Their position was then measured by triangulation methods, using the T-2 theodolite for measuring the angles. For some highway survey projects, their remoteness or lack of geodetic monuments near them necessitated the establishment of entire triangulation networks. This operation proved to be time consuming, expensive, and, in some cases, required considerable adjustment to obtain a reasonable closure. The costs of surveying basic control in several cases exceeded the costs of surveying the supplemental control.

These occurrences led to investigating other means of establishing basic control.

TELLUROMETER

In the fall of 1961, a Tellurometer (Model MRA-1) electronic distance-measuring unit was used on a trial basis. To evaluate the equipment, accuracy, and the savings in costs, a contract was negotiated with a commercial firm for surveying the position of basic control points along 71.5 miles of aerial surveys in Yellowstone National Park and Hoback Canyon, Wyoming.

The probable cost of triangulating the basic control, using U. S. Bureau of Public Roads' personnel, was estimated and compared to the actual costs encountered through use of the Tellurometer. The savings accrued by use of the Tellurometer are given in Table 1.

Paper sponsored by Committee on Photogrammetry and Aerial Surveys.

TABLE 1

COSTS OF TRIANGULATION VERSUS COSTS OF USING THE TELLUROMETER FOR SURVEYING BASIC CONTROL

Project	Length (mi)	Estimated Triangulation Costs ($)	Actual Tellurometer Costs ($)	Savings ($)
East entrance	17.0	4,770.00	784.00	3,986.00
Norris-Beryl Springs	10.0	1,590.00	882.00	708.00
Northeast entrance	8.0	2,650.00	196.00	2,454.00
South entrance	11.0	1,325.00	686.00	639.00
Red Lodge-Cooke City	12.0	2,650.00	392.00	2,258.00
Hoback Canyon	13.5	4,033.50	320.00	3,713.50
Total	71.5	17,018.50	3,260.00	13,758.50

Later in 1961, a set of Tellurometer units, Model MRA-1, was leased on a monthly basis. U. S. Bureau of Public Roads' personnel were trained as operators by the leasing agency. These units were primarily used in surveying basic control for aerial surveys in New Mexico and Utah. On one survey in New Mexico, however, an attempt was made to survey both the basic and supplementary control by use of the Tellurometers. The basic control survey points were spaced 3 miles apart, whereas the supplementary points varied from 400 to 1,200 feet apart. Considerable trouble was encountered in obtaining reliable measurement of distances shorter than 1,000 feet. No better than third-order accuracies were obtained for the supplementary control, whereas the closures of all basic control surveying were second-order or better. On the basis of this project, the Tellurometer, Model MRA-1, was considered inadequate for measuring distances in accomplishing supplementary control surveys.

ELECTROTAPE

In 1961 the Cubic Corporation of San Diego, California introduced to the surveying profession a fully transistorized electronic distance-measuring instrument—Electrotape. The Electrotape system consists of two identical interchangeable units, each capable of transmitting or retransmitting microwave signals.

The transmitting unit is normally referred to as the interrogator and the receiving unit as the responder. These units are designed to operate on 12- or 24-v wet cell batteries. Total weight of each unit is 27 lb including a built-in radio-telephone system for communication between units. The Electrotape unit is designed to measure distances from 150 feet to 30 miles in length.

On basis of the performance record of the Electrotape with other Government agencies and the realization that short distances ranging from 300 to 1,000 feet could be accurately measured, two units, Model DM-20, were purchased in June 1962. Peripheral equipment also purchased included tripods, automatic psychometers, nickel-cadmium batteries, and a heavy-duty battery charger.

Since purchased, the Electrotape units have been used at altitudes varying from 5,000 to 12,000 feet and under various climatic conditions. Temperatures have ranged from a minus 10 F upward to plus 95 F. Terrain has varied from low, rolling topography to mountain canyons. Vegetational coverage has varied from sagebrush and scrub piñon to aspen and pine forests.

Observed adverse effects resulting from such operating conditions are as follows:

1. When temperatures are so low as to be classed extremely cold, a set of batteries provided power for only one hour of operation. To operate under this condition, additional battery sets have been purchased.

2. Moving objects, such as tree leaves and branches, in the line of sight affect the measurements, resulting in a difference of as much as 10 centimeters between forward and return measurements of the distance.

Inasmuch as the Electrotape units were in part replacing the subtense bar, a comparison was made of the two methods from the standpoint of the number of distances measured per day. During an average 8-hr day, a four-man survey party using a T-2 theodolite and two subtense bars could measure 20 traverse distances. The same number of men using the Electrotape units would measure only 15 distances. Average time required for making both forward and return measurements by use of the Electrotape has been 18 minutes per set-up. It should be pointed out that the horizontal angles are measured and recorded concurrently when the subtense bar is used, whereas measuring angles is an additional operation when using electronic distance-measuring equipment. Attempts to combine both operations have not proved fruitful.

TABLE 2

Electrotape Measured Slope Distance (M)	Total Field Correction (M)	Correction per Meter (M/M)	Total Correction Using Average Factor (M)	Difference Between Cols. 2 and 4 (M)
240.830	0.0091	0.0000379	0.0136	0.0045
199.705	0.0084	0.0000421	0.0113	0.0029
293.475	0.0136	0.0000464	0.0166	0.0030
297.325	0.0150	0.0000504	0.0169	0.0019
212.595	0.0107	0.0000504	0.0120	0.0013
342.645	0.0197	0.0000576	0.0194	0.0003
178.590	0.0104	0.0000585	0.0101	0.0007
226.060	0.0143	0.0000631	0.0128	0.0015
215.070	0.0135	0.0000630	0.0123	0.0012
216.280	0.0139	0.0000641	0.0123	0.0016
191.120	0.0143	0.0000748	0.0108	0.0035
245.190	0.0129	0.0000526	0.0139	0.0010
207.305	0.0122	0.0000589	0.0118	0.0004
241.230	0.0139	0.0000576	0.0137	0.0002
221.340	0.0141	0.0000636	0.0125	0.0016
257.920	0.0172	0.0000665	0.0146	0.0026
Average		0.0000567		
Corrected index	1.000263			

An analysis of field procedures revealed considerable time was lost by the operators of one Electrotape unit while the other unit was being moved from one control point marker to the next.

To overcome these lost man-hours and to accomplish the same amount of work as previously obtained when the subense bar was used, a third Electrotape unit, Model DM-20, was purchased in June 1963. With three units a leap-frog type of procedure was used.

Consider the three units as A, B, and C. While the distance between Units A and B was being measured, Unit C was being set up over the marker of the next control point beyond Unit B. When the measurements between Units A and B were completed, Unit A was moved to the marker of a control point beyond Unit C. In the meantime, Units B and C were being used. By this procedure 24 to 26 measurements can be completed in an average 8-hr day. This field surveying method with the Electrotape has proved to be very effective. Normally, two men are assigned to each unit—one as operator and measurement reader and the other as recorder and computer. Personnel permanently assigned to the units are responsible for their use and care. Due to the simplicity of the units, a man can be adequately trained to operate the Electrotape instrument in one day. Should one of the units fail electronically, no attempt is made to repair the interior components. Such units are shipped by air freight directly to the factory in San Diego, California for repair. During the period from the day they were purchased and first used to September 1963, a total of three breakdowns has been experienced. In each case, only three to four days of use time were lost. The availability of the third unit greatly reduces operational time loses; as, in the event of one unit failing, the other two units can still be used in measuring distances.

An index of refraction of 1.000320 has been applied to the internal circuitry of each Electrotape unit. Thus, each measurement is automatically modified by this factor to give absolute measurement under ideal atmospheric conditions. Unfortunately, these conditions never exist; therefore, the index must be adjusted for the various atmospheric changes. Wet bulb temperature, dry bulb temperature, and atmospheric pressure are used to determine the correction factor. These temperature and pressure measurements are recorded before and after each series of distance readings. This has proved to be quite time consuming and, as subsequently shown, unnecessary when surveying supplementary control. A review of several highway survey projects in which the supplementary control had been measured by use of the Electrotape revealed the amount of correction applied to each distance was consistently the same for each project. Table 2 contains corrections compiled from the Electrotape measurement notes on a survey project in Dinosaur National Monument.

Data in Table 2 reveal, for all practical purposes, that one factor can be used to correct all of the supplementary control survey measured distances on a particular survey. The Dinosaur survey project would have had a corrected index factor of 1.000263. When measuring distances of less than 1,500 feet, it can be concluded meteorological data need be recorded only four times during the average 8-hr day. From these four, an average index of refraction can be computed and applied to all of the measured distances.

Analysis of corrections applied to distances longer than 1,500 feet reveals it is not necessary to compute a new index for each distance measured. The same index can be applied to two consecutive measurements, provided atmospheric conditions do not change appreciably. The maximum error introduced by this procedure was 1 part in 50,000.

As of September 1963, the longest distance measured with the units was approximately 8 miles and the shortest distance was 162 feet. The poorest closure obtained in surveying supplementary control was 1 part in 7,000 for distances varying from 300 to 1,000 feet. The most consistent Electrotape measurements between interrogator and responder units have been obtained on the longer distance measurements.

As of September 1963, 130 miles of basic and 26 miles of supplementary control had been surveyed using the Electrotape units. Cost records are not complete enough to establish an average cost per highway route mile for such control surveying.

CONCLUSIONS

Model DM-20 Electrotape distance-measuring units are an effective means for measuring distances ranging in length from 150 feet to many miles, and only a minimum number of third-order accuracies will occur. The instruments are versatile, are constructed for rough usage and are operable when weather conditions are extreme, are simple to operate, and are relatively free from maintenance. In establishing supplementary control for highway surveys, three measuring units are much more effective and greater savings per highway mile may be realized than when using only two units. In measuring distances shorter than 1,500 feet, the amount of meteorological data obtained and used can be reduced without affecting the results.

REFERENCES

1. "Manual of Instructions for Electrotape Instruments." Cubic Corp., San Diego, Calif. (1962).
2. McBride, H. H., "Evaluation of the Tellurometer in Highway Horizontal Control Work." HRB Bull. 258 (1960).
3. Poling, A. C., "The Tellurometer for Highway Survey Control." HRB Bull. 258 (1960).
4. Winikka, C. C., "Tellurometer Use in Control Surveys for Mapping by Photogrammetric Methods." HRB Bull. 258 (1960).
5. Winsor, D. E., "Ten-Foot Portable Subtense Bar." HRB Bull. 199 (1958).

Evaluation of The Model 4 Geodimeter

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•IN THE FALL of 1957 the Photogrammetry Section at the Headquarters office of the California Division of Highways acquired a Model 3 Geodimeter for the purpose of measuring distances in making basic control surveys. The range and accuracy of the Model 3 made it possible to make survey ties directly to U. S. Coast and Geodetic Survey triangulation stations by traverse surveying procedures. In this manner the California State plane coordinate system could be established in the immediate vicinity of a highway mapping project without encountering costly and time-consuming conventional survey problems because of the remoteness of the triangulation stations or the ruggedness of the topography.

The Geodimeter has demonstrated its reliability and value for complementing precise surveying equipment. The development of the Model 4 Geodimeter which combines dependability, high accuracy at short ranges, and portability has made the instrument an invaluable tool, not only for basic control surveying but also for accomplishing all types of surveys.

DEVELOPMENT OF THE GEODIMETER

The initial model of the Geodimeter was developed to attain extreme accuracy in measuring distances of from 5 to 40 miles. The Model 1 instrument was introduced in early 1953, and was superseded by a refined version, the Model 2, in 1955. With the Model 2 Geodimeter, measurements can be made as accurately as by the highest precision in tape measuring of distances. Many users of the Model 2 requested development of a more portable unit which would be useful in making surveys not requiring utmost accuracy. Subsequent refinements of design, at the expense of range and accuracy, resulted in production of the Model 3 Geodimeter. This model is designed to measure distances in the 1- to 20-mi range, with a possible error of ± 0. 40 feet.

The Model 4 Geodimeter was designed primarily to meet requirements for an instrument of high accuracy in measuring distance of from 1, 000 feet to 4 miles. In clear air, distances up to 12 miles have been measured. For distances of 400 to 1, 000 feet the accuracy of measurement is 1 part in 10, 000 or smaller difference, and the proportional accuracy increases as the measured distance increases in length. The maximum error for a single observation at any distance is ± 0. 04 feet, plus 5 millionths of the distance.

Operating Principles

The basic principle of all models of the Geodimeter system of distance measurement is the indirect determination of time required for a pulsed light beam to travel between two stations.

A modulated beam of light is emitted from the Geodimeter at one end to a passive reflector placed at the other end of the line of sight. The reflector returns the light pulses back to the instrument where a phase comparison is made between the projected and reflected light pulses.

The precision of the Geodimeter relies on the fact that the velocity of light and the frequencies of the crystal oscillators are very accurately known and that they remain almost constant.

Paper sponsored by Committee on Photogrammetry and Aerial Surveys.

Changes in atmospheric temperature and barometric pressure produce a small known variation in the velocity of light. Temperature and pressure data are recorded for each observed line and resultant corrections are applied to the computed distance.

Variations in frequency constants are caused by ambient temperature. To maintain fixed frequencies, the crystals are enclosed in an electronic oven where a nearly constant temperature is maintained.

The reflector device is a 2-in. diameter solid glass prism mounted in a metal case. To afford the greatest flexibility and economy in operation, each prism is mounted in individual shockproof casings which can be placed in housings which will hold 1, 3, or 7 prisms. The housings will mount on any Wild theodolite tripod and with a special bracket a maximum of 21 prisms may be utilized. The prisms are easily portable and can be left unattended. The Geodimeter light will be reflected back to the instrument even though the axis of the prism is offset in its pointing by as much as 20°. Plastic reflectors, similar to those used on highway guide posts, may be used instead of prisms in measurement of distances up to 0.4 miles. It is possible, therefore, to select the most suitable reflector combination for the distance being measured.

Effect of Atmospheric Conditions

The largest source of error in Geodimeter measurement is caused by errors made in the determination of meteorological conditions. An error of 1 C in temperature or 3 mm of mercury in atmospheric pressure will cause a one part per million error in the measured distance. The influence of humidity is negligible within the accuracy limits and is not given consideration in the computations. For all practical purposes, the mean of the temperature and pressure data recorded at the Geodimeter and reflector stations is sufficient. In precise surveys requiring differences as small as one part in one million or smaller, additional data regarding temperature, atmospheric pressure, etc., must be obtained along the line of sight to minimize the effects due to variations in temperature and air pressure.

The range of the Model 4 varies with the state of atmospheric conditions. The instrument will measure a distance correctly as long as a sufficient signal is returned from the reflector. If the return signal is weak or not discernible, the instrument will merely stop operating. It is impossible to give specific range limits in measurement of distances under adverse weather conditions, such as in snow, rain, fog, mist, or haze. The maximum range will be determined by the density of the adverse factor and the number of reflecting prisms available. As an example, a distance of 11.2 miles was measured using nine prisms. On the following night a remeasurement was done under apparently similar atmospheric conditions, but 28 prisms were necessary. The following data are an indication of ranges in distance which can be measured under varying visibility conditions: fog: 100 to 2,000 feet; rain: 1,000 feet to 3 miles; heavy haze: 1,000 to 5,000 feet; medium haze: 1 to 3 miles; light haze: 3 to 7 miles; and clear air: 7 to 12 miles.

Under most atmospheric conditions, the normal time required to make a measurement is from 5 to 15 minutes. In extreme cases the time required to make a measurement may be 15 to 30 minutes.

THE MODEL 4 GEODIMETER

Three Model 4 instruments have been produced by the manufacturer which are essentially identical in basic design and operation. These units are designated as Models 4A, 4B, and 4D. After introduction of Model 4A, the manufacturer requested field evaluations of the instrument and many suggested changes and improvements were incorporated into the later models. Most of the refinements found in the latest models can be adapted to older Model 4's quite readily.

The important change between the 4A and 4B was the installation in the latter of an improved heating unit for the crystal oven. To maintain the rated accuracy of the instrument for measuring long distances, it is necessary to control the temperature of the crystals within a small range, which confines the modulating frequencies to their specified values at all times.

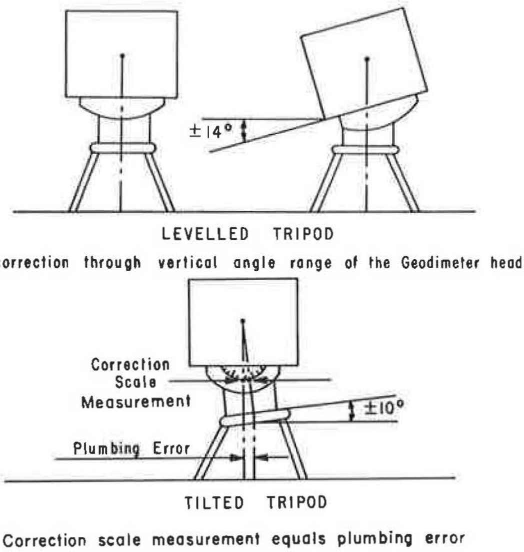


Figure 1.

Additional changes found in the 4B are subsequently mentioned. A tuning condenser was added to the Kerr-cell circuit which permits this circuit to be maintained at optimum level at all times and results in increased speed and stability of the null system. A system of three different aperture stops was developed for the receiver optics which allows either wide or narrow angle light acceptance from the area of the reflector. This has been an invaluable aid when operating in daylight or in areas which are highly illuminated at night.

The model 4A was designed with removable side and top panels which provide easy access to the instrument for repair and adjustment in the field. The housing for the model 4B is in one unit which attaches to the base of the instrument frame. If field repairs are necessary, the whole electro-optical system is exposed to the elements. One-piece construction of the housing, however, made the determination of plumbing eccentricities simple. The Geodimeter head is attached to a Wild surveying instrument tripod and allows the instrument to be pointed through a vertical range of ±14° if the tripod head is level. If the tripod head is not level, errors are introduced in the distance measurements, because the plumb-bob is suspended from the tripod and not the horizontal axis of the instrument. A plumb correction device has been adapted to the Geodimeter head to permit direct measuring of the plumbing error caused by tilt in the tripod head (Fig. 1). Also, a small hole has been drilled on the sides of the instrument housing at the horizontal axis of the Geodimeter. A pin is inserted into each hole and a bar plummet is suspended from the pins which will indicate the plumbing plane of the Geodimeter. This instrument "plumb-bob" is used when the tilt of the tripod head exceeds the range of the plumb correction device or when the Geodimeter is offset from the survey station marker.

The Model 4D is known as the daylight Geodimeter and is basically the same as its predecessors. The exciter lamp is a mercury gas lamp which is thirty times brighter than the incandescent lamps used in Models 4A and 4B. A discussion of this instrument is found in another portion of this article.

GEODIMETER FIELD EXPERIENCE

The maximum productivity and economy in Geodimeter surveying can be achieved only through efficient planning and reconnaissance work in the field.

For maximum productivity, the most efficient use of the Geodimeter is achieved utilizing two survey parties. One party operates the Geodimeter exclusively. The

other party uses theodolites to measure required horizontal and vertical angles. Experience has shown that a combined angle-measuring and distance-measuring party is not satisfactory. Combined operations are desirable only in those instances where occupying a station is so hazardous or time consuming that a joint effort is advantageous from the standpoint of economy and safety.

The Model 4 Geodimeter is best suited to surveying by measuring distances in a traverse. To make the accuracy of angle measurements compatible with the accuracy of distance measurements, it is advisable to measure the angles by use of a theodolite with a measurement increment of one second of arc. Radiating several lines of measurement from one station marker must be planned with utmost care, inasmuch as the ultimate surveyed positioning of survey control points may depend more on angle measurements between lines of sight than on the distance measurements between points. Also, the measured position of radiated points is difficult to verify because a closure is not made without further surveying.

The fundamental requirement for measuring a distance by use of a Geodimeter is a clear line of sight between points of measurement, the same as is required for measuring angles with a theodolite. It is especially important that a thorough reconnaissance be made to verify feasibility of the office-made plan for the control survey and to modify it as necessary, to select access routes to station markers chosen, and to mark the stations adequately so a minimum of time will be utilized in finding the station markers in the dark. The largest time-consuming factor in making control surveys by use of a Geodimeter is the time required to move from one survey station marker to another. The instrument moving time depends on the ease with which station markers can be located and occupied. It is therefore evident that the production of a Geodimeter survey party is directly proportional to the effort expended in the daytime planning and preparation for accomplishing the control of the surveying.

Personnel requirements are extremely flexible, depending mainly on the type of survey, its extent, and the nature of the topography. The basic party for Geodimeter surveying may consist of two men, the instrument operator and a reflector attendant. For making traverse-type surveys, the party usually required consists of the instrument operator, an assistant, and two reflector attendants.

Methods of Measuring

Methods and procedures in using the Geodimeter to measure distances vary in relation to the degree of precision desired on any given survey project. The basic procedure consists in making a series of 24 measurements (Fig. 2). Four measurements are made to the reflector (Ref.) and four are made on the internal base line of the Geodimeter (Cal) in each frequency. The effect of the internal measurement is to determine the zero point in each frequency and to eliminate any errors should electronic drift occur in the instrument.

The switching through the four phases will eliminate errors due to asymmetry of the Kerr-cell RF voltage and the phototube bias voltage. The connections to the switch are arranged so a 180° RF phase change will occur when switching from phase 2 to phase 3. The bias voltage changes between positions 1 and 2 and between positions 3 and 4. The manufacturer suggests a tolerance of 3 to 4 divisions between the sum of the measurement values of phases 2 and 3 and phases 1 and 4. To maintain consistent accuracies, however, it is recommended the measurement tolerance be limited to 1 division. If the stated tolerances are exceeded, the four measurements should be repeated. This procedure enables the instrument operator to establish the validity of each set of measurements and eliminates all errors in measurement observations.

The speed, sensitivity and null action of different Geodimeters will vary. The novice operator of a Geodimeter therefore should complete all 24 measurements for each distance until he is thoroughly familiar with the characteristics of his particular instrument. When the operation of the instrument is mastered and consistent measurement results are obtained, the following procedures may be followed: For extreme accuracy, measure all phases in each frequency and repeat the complete sequence of measurements at least 3 times. For the normal accuracy, which far exceeds require-

GEODIMETER OBSERVATIONS
NASM 4

Calibration No. 5 Serial 56

Date 10-1-63 Observer RAD Approx. Dist. 3.0 KM

District HQ Co. SAC Rte. — Sec. — W.O. —

Geod. Stat. A Refl. Stat. B

Type Refl. 1-3 PRISM Null GOOD Visibility GOOD

PH	F-1 Ref	F-1 Cal	F-2 Cal	F-2 Ref	F-3 Ref	F-3 Cal
1	+ 92.0	+ 40.0	+ 43.0	+ 218.0	− 104.0	+ 45.0
2	− 91.0	− 40.0	− 42.5	− 216.5	+ 104.0	− 45.0
3	+ 89.0	+ 41.0	+ 44.0	+ 216.5	− 104.0	+ 50.0
4	+ 88.0	− 41.0	− 43.0	− 214.0	+ 104.0	− 50.0
2+3	180.0	81.0	86.5	433.0	208.0	95.0
1+4	180.0	81.0	86.0	432.0	208.0	95.0
Sum	360.0	162.0	172.5	865.0	416.0	190.0
M	R_1 90.0	C_1 40.5	C_2 43.1	R_2 216.3	R_3 104.0	C_3 47.5

	Geod.	Refl.	Sum	Mean	P/PM	Sum P/PM
Temp	20° C	24°C	44° C	22° C		
Pres	50'	250'	300'	150'		

-G+ No OFFSETS +R-

HI - Geod. 4.9 Ft

HI - Refl. 5.1 Ft

Remarks

Theodolite At (G) R
D= 88 - 22 - 30
R= 271 - 37 - 42
Sum 359 - 59 - 72 +12
Mean(+) 01 - 37 - 36

STATE OF CALIFORNIA, DIVISION OF HIGHWAYS

Figure 2. Geodimeter observations, California Division of Highways.

ments for highway surveys, measure all phases on the reflector and measure phases 2 and 3 only on the internal light path. Because there is no atmospheric disturbance on the internal light path, the null action is highly stable. For surveys where nominal accuracies are acceptable, measure only phases 2 and 3 throughout the distance-measuring sequence. The latter procedure is recommended only for experienced instrument operators, because there is no check possible on the validity of measurements to the reflector unless all data are used in the field to compute the distance between station markers.

The most difficult measurements to make with a Geodimeter are those made under turbulent atmospheric conditions. These conditions are usually found when a cool night has been preceded by a hot day and the line of sight is over irrigated agricultural land or through successive temperature inversion layers. The null action becomes erratic because of changes occurring in the atmosphere, and tolerance in the set of measurements made to the reflector is difficult to achieve. Under these circumstances the measuring procedure should be modified so phases 2 and 3 are measured first and then phases 1 and 4.

Additional sensitivity and stability of the instrument may be achieved under marginal atmospheric or range conditions by increasing the input voltage approximately 10 percent.

At the reflector station the only error which can be introduced is in plumbing the tripod head. If the tripod head is leveled by eye or by the use of a bulls-eye level the expected error in the distance from this source will be in the order of ± 5 mm. This

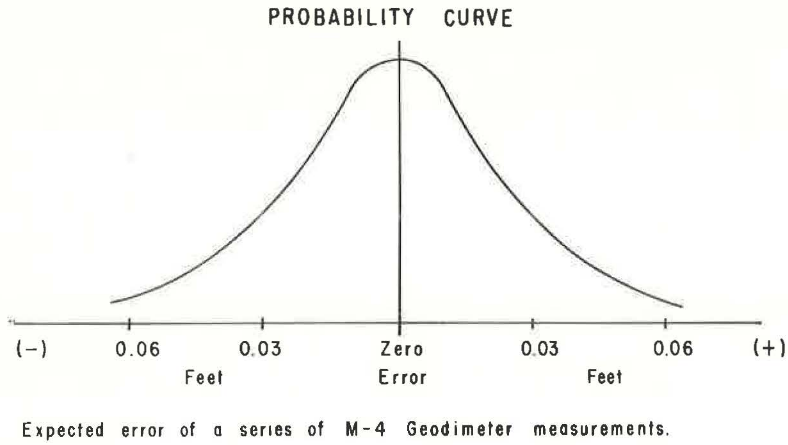


Figure 3. Probability curve.

error can be eliminated by using an adaptor which allows the reflector to be attached to the Wild traverse target optical leveling base.

Instrument Errors

Previously published data have indicated that the instrumental error of the Model 4 Geodimeter will closely approximate a probability curve as shown in Figure 3. The validity of this curve has been substantiated by numerous experiments. Because this error tends to be purely accidental, it is expected to cancel out through a series of courses. Experiments were conducted on test courses which were traverse segments approximately 4, 800 feet long, divided into increments of 800 feet to 1, 600 feet. After the individual segments were measured by increment, the full segment was measured in a single observation. The closing errors varied between 0. 01 and 0. 03 of a foot for three segments, and 0. 06 of a foot error occurred in a fourth segment.

The type of error previously noted can result in closures of one part in one million or smaller difference for surveys which close upon themselves. It is evident, however, the individual courses in the survey do not contain the same degree of precision.

Vertical Angles

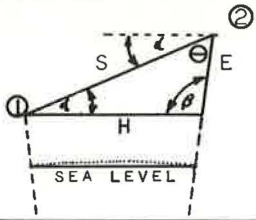
Inasmuch as the Geodimeter measures slope distances, the accuracy of the resultant horizontal distances will depend on the care with which the vertical angles are measured.

Over relatively flat topography, where the vertical angle does not exceed a few degrees, the theodolite is set up near the Geodimeter at the same instrument height. The mean of one direct and one reverse vertical angle measurement will provide the necessary accuracy. Where topography slopes are steep, extreme care must be taken in measuring the vertical angle. The theodolite must be set up over the Geodimeter plumb point and instrument height differences must be taken into consideration in the computations.

Computations for reducing a slope distance to a horizontal distance by exact methods are a cumbersome time-consuming procedure. A method (Fig. 4) developed by the California Division of Highways is a rapid and accurate solution. The equations used are close approximations which will yield results exceeding first-order accuracies. This is beyond the accuracy of the instruments involved in measuring the basic data.

Horizontal Angles

The Geodimeter is capable of measuring distances through any type of obstruction allowing passage of even a fraction of the light beam to and from the reflector. Included



MEASUREMENTS

	°	'	"
VERTICAL ANGLE ① TO ② = +			
VERTICAL ANGLE ② TO ① = −			

(S) SLOPE DISTANCE = METERS

(S) IN FEET = METERS x 3.28083333 = FEET

		°	'	"	SINE	DISTANCE [FT]	
*90°+(c) =	β	90					S (SLOPE DIST.)
**±V.A.+(c&R) =	α						E (DIFF. IN ELEV.)
***90°−(α+c) =	θ						H (HOR. DIST. @ ELEV. ①)
		180	00	00			

$$E = \frac{S}{\sin\beta} \times \sin\alpha$$

$$H = \frac{S}{\sin\beta} \times \sin\theta$$

*(c) IS CORRECTION FOR CURVATURE. $(c) = +4.93'' \times \frac{S}{1000}$, ADDED TO 90° TO FIND β

**(c&R) IS CORRECTION FOR CURVATURE & REFRACTION
$(c\&R) = +4.25'' \times \frac{S}{1000}$, ADDED ALGEBRAICALLY TO MEASURED V.A. TO FIND α
IF RECIPROCAL V.A.'S ARE MEASURED, THE MEAN VALUE = α DIRECTLY

***IN THE CASE OF V.A. MEASUREMENT AT ONLY ONE END OF LINE, $\theta = 180° - (\alpha+\beta)$
IN THE CASE OF RECIPROCAL MEASUREMENTS, $\theta = 90° - (\alpha + c)$

Figure 4. Reduction of slope distance to plane coordinate grid distance.

among these obstructions would be trees, brush, foliage, and power poles. Accuracy of the Geodimeter measurement will not be impaired by these terrain and ground cover features but horizontal angles may be unduly influenced by horizontal refraction which would result in loss of angular accuracy.

Calibration and Frequency Checks

Recalibration of the Geodimeter delay line is necessitated by changes which develop in the electronic circuitry of the instrument. The process of calibration is used to determine changes occurring in the relationship between wave length fractions as seen on the Geodimeter delay dial and the actual distance measurement these fractions represent with respect to a precisely measured base line. A calibration shift will cause a spread between the computed distances obtained for each frequency regardless of the length of line measured.

There are two methods by which a delay line shift can be ascertained. In the field, the instrument operator is able to make a rough check of the calibration by examining both the low and high ends of the delay dial while on the internal light path. The difference of the calibrated values of the delay line measurements will give the wave length of the frequency being checked, provided the calibration is still valid. In the office, the presence of a delay line shift is identified by a spread in the computed distance in the three frequencies beyond the normal range of 0-3 cm in one measurement out of ten or more.

The effect of changes in the frequencies of the crystals is not noticeable in field operations and may not be noticeable in computations. The effect of frequency drift is manifested only in measurement of long distances where a 30-cycle change in the frequency would cause approximately a 1 part per million error in the distance measurement. The expected change in frequency is in the range of ± 50 cycles within a 3-month period. If a sufficient change occurs in one frequency only, the effect would be discernible in the computations. If more than one frequency was affected, however, it is possible for the errors to either increase or cancel each other.

The time interval between calibrations and frequency checks is a variable dependent on the stability of each instrument and the accuracies required in the distances measured. As an experiment, a Model 4A Geodimeter which was out of calibration was used to measure some 30 distances. The instrument was then recalibrated and the distances were measured again and recomputed. The average deviation between the results was 1.2 cm and a maximum deviation of 3 cm occurred in only two measurements. To maintain reasonable accuracies, the Model 4 Geodimeter should have a recalibration and a frequency check every six months. To provide for maximum instrument accuracy, a calibration and frequency check should be made every two months.

Repairs and Maintenance

The major unknown factor at the time the first Model 4 Geodimeter was acquired was the frequency and cost of repairs which would be necessary. The advantages gained by using the instrument could have been nullified by excessive lost time due to mechanical or electronic malfunctioning. The Geodimeter has proved to be a remarkably trouble-free device.

The repair histories of ten instruments used by the California Division of Highways and two privately owned instruments have been maintained. Five of the instruments have been in use for approximately 80 percent of each work year since their acquisition and the remaining units are in use approximately 65 percent of the time. The heavy usage of these Geodimeters and the limited repairs needed under these conditions substantiates the dependability of the instrument.

The cost of repairs has been a nominal item. Under the terms of purchase, the major difficulties arising during the first year of operation are covered by the manufacturer's warranty. There has been no difficulty with the manufacturer or the West Coast distributor concerning warranty adjustment. The total cost of major repairs for 26 instrument years of operation has been approximately $1,600 or an average of $62 per year for each instrument. Additional costs will be incurred during routine maintenance, which include replacement of the exciter lamp and inoperative tubes and periodic calibration of the delay dial and frequency check of the crystals. Consideration must also be given to the probable replacement of the Kerr-cell and switching devices after two or three years of use.

If the cost of repair and maintenance is projected on the basis of 5 years of nearly continuous operation, it is reasonable to assume these costs will total about $500 annually.

Daylight Operation

Replacement of the conventional 5W projection lamp in the Model 4B with a 100-w mercury lamp has increased the night range of the instrument and given it added daylight capabilities.

The mercury lamp housing has been adapted to fit existing lamp adjustment devices, and the original condenser lens system has been modified. A lamp ignition system and a blower fan, which protect electronic components from the heat generated by the mercury lamp, is mounted on top of the Geodimeter. The additional power needed for the lamp is provided by a special power unit which is incorporated into a single housing with the standard power pack. Operation procedures in use of the converted instrument are identical to the standard Model 4B.

The California Division of Highways has seven Geodimeters utilizing the mercury lamp adaption. Preliminary data indicate maximum daylight range, using nine prisms,

will vary from between 1 and 2.5 miles during summer months. Difficulties are encountered when the distances to be measured are along lines over pavement or highly reflective ground, as found in the California desert areas, when the air temperature exceeds 80 F. Distances exceeding 2 miles over grassy range land have been measured with ease in temperatures exceeding 100 F. Under winter conditions, it is anticipated measurements during daylight hours will be made as readily as those at night.

TABLE 1

DAYLIGHT REPEATABILITY TEST
(April 10, 1963)

Station	Distance (m)	Variation from Mean (m)
Test to No. 1	618.900	+0.003
	618.898	+0.001
	618.894	-0.003
	618.897[a]	
Test to No. 2	864.677	+0.003
	864.679	+0.005
	864.667	-0.007
	864.673	-0.001
	864.674[a]	
Test to No. 3	548.143	-0.007
	548.156	+0.006
	548.150[a]	

[a]Mean.

The advantages of the Model 4D Geodimeter are (a) use of the instrument day or night to take advantage of weather or topographic factors, (b) increased range of measurement at night, and (c) simple convertibility back to 4B Geodimeter operation if desired. The only disadvantage of the Model 4D Geodimeter is the slight decrease in portability of the unit due to the added weight of the power pack.

Because daylight will tend to absorb a portion of the Geodimeter light beam, it is necessary to use glass wedges on the prisms to provide adequate signal deflection when measuring short distances so the light is reflected to the receiver optics. Under varying atmospheric conditions the use of wedges has been advantageous for measuring distances as long as approximately 4,000 feet. Beyond this distance, the deviation effect of the wedges becomes excessive and no useful light will be reflected. The choice of wedges is a function of both distance and atmospheric conditions so the combination of wedges needed may vary for measurement of distances of identical magnitude.

An interesting aspect of the model 4D Geodimeter is the fact the instrument can be converted back to model 4B operation with ease. From an operational standpoint there are two conditions which make this feature an important asset: first, if a mercury lamp malfunction should occur and a replacement lamp were not available; and second, if long packs were to be made into areas where the size and weight of the Geodimeter and power supply were critical.

Depending on atmospheric conditions and the experience of the Geodimeter operator, the time required for making a daylight distance measurement will vary between 5 and 30 minutes.

Repeatability of the Model 4 Geodimeter

The expected accuracy of repeated Geodimeter measurements of a distance will be on the order of ± 12 mm. The repeatability of Instrument No. 164 is given in Table 1. The distances were measured by two different operators at various times of the day. The weather pattern was changing from clear skies to rain, and the temperature varied between 11.5 C and 14 C.

Using the same instrument two separate distances were measured day and night by two different operators (Table 2). The distances given in Table 2 are conversions of slope measurements to horizontal distances. The spread in the measurements from Wruble to Whalen No. 2 was attributable in part to a discrepancy in the vertical angles as measured during the day and during the night.

TABLE 2

DAY-NIGHT REPEATABILITY TEST
(April 10, 1963)

Station	Time	Distance (m)	Variation from Mean (m)
Wruble to Whalen No. 2	Day	1211.275	-0.007
	Night	1211.290	+0.008
	Mean	1211.282	-
Wruble to Stump	Day	3890.336	-0.001
	Night	3890.338	+0.001
	Mean	3890.337	-

Numerous experiments have been made repeating with the Model 4D Geodimeter the measurements originally

made at night by use of Model 4A and Model 4B Geodimeters. Comparison of day and distances measured during day-light hours by use of the Model 4D with the distances originally measured during nighttime hours is similar to the results given in Tables 1 and 2.

Advantages of the Geodimeter

The primary advantage of the Model 4 Geodimeter lies in the fact that results achieved are positive. The validity of raw measurement data is established by the instrument operator in the field while the measurements are being made. The computations are also self-checking and any incorrect solutions are, in almost every case, caused by blunders made in reducing the field notes and transcribing measurement data to computation forms. Experienced computers can resolve virtually every possible field or office error. As an illustration, only one distance has been remeasured in more than 8, 000 Geodimeter measurements made by two California Division of Highways' Geodimeter parties.

Another significant benefit from using the Model 4 Geodimeter is when making control surveys in urban areas. With its great flexibility it is possible to measure from street to street or through residential areas without having to enter private property, as is necessary in ordinary measurement of distances by taping procedures. In short, elimination of taping effects the greatest savings in time and money.

Reports submitted by the various California Highway Districts and private users indicate use of the Model 4 Geodimeter results in a minimum monetary savings of 40 percent by comparison with conventional field methods of control surveying. The average amount of time saved, 60 percent, is not included in this figure, as some surveys would never have been accomplished without use of the instrument because of the nature of the topography, ground cover, and sparse distribution of U. S. C. and G. S. triangulation stations.

CONCLUSIONS

The results obtained from 26 instrument years of Model 4 Geodimeter use have proved the reliability and dependability of this instrument as a distance-measuring device. Accuracies greater than those required for highway survey purposes are achieved by normal operational procedures in measuring either short or long distances. In addition, use of the instrument effects substantial savings in the cost and time of accomplishing control surveys.

REFERENCES

1. AGA Corporation, Manuals and Brochures.
2. Katibah, G. P., and Radmanovich, D., "Meet the Geodimeter." California Highways and Public Works (May-June 1962).
3. Radmanovich, D., "Geodimeter Operators Training Course Manual." California Division of Highways (1961).

Evaluation of the Electrotape Distance-Measuring Device by the Texas Highway Department

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•THE PHOTOGRAMMETRY Section within the Texas Highway Department is a subdivision of the Highway Design Division and functions as a service organization to the 25 District offices located throughout the State. Requests for photogrammetrically compiled maps and for furnishing related products originate in the various Districts and are forwarded to this section in the Austin office for accomplishment. At present, six stereoscopic photogrammetric instruments are utilized for compiling maps and making measurements in the highway surveying programs.

The duties and responsibilities involved in establishing sufficient and accurate field control necessary for doing the mapping by photogrammetric methods rests with the District making the request, while the Photogrammetry Section acts as consultant and advisor for the field control surveying.

Depending on the intended utilization of the maps, the basic field control usually consists of a second- or third-order horizontal traverse and a third-order vertical network. For highway location and design programs in rural areas the third-order traverse is considered sufficient, but when it is intended as a control base for preparing deed descriptions and for compiling other right-of-way data, and for designing facilities in urban areas through use of dimensions scaled or calculated from plane coordinates of details on the maps, second-order horizontal accuracy is required in the basic control.

The attainment of second-order accuracies using ordinary field survey methods is extremely time consuming and expensive and proof of such attainment must be provided, either by surveying closed traverse loops or by making closure ties to existing station markers of known accuracy. Texas' land area, being as extensive as it is, prohibits the inclusion of ties to existing station markers in the planning of every survey project. Monuments at station markers are utilized at every point of existence, but in some cases to make traverse ties to the beginning and from the end of a particular survey project to the monuments of existing station markers would require as long, if not longer, than the traverse of the survey project itself, and therefore would be economically unjustifiable.

A traverse loop resulting in closure back to the beginning point adequately serves the proof postulation of accuracy; however, it does not present an efficient or economical solution to the problem. The inherent difficulties involved in a normal high-order transit and tape survey would seem to discourage the use of such procedures in any, but the most essential, control survey projects. The amount of time and money necessary to conduct a second-order transit and tape survey always discourages the resident engineer from undertaking the establishment of county or district-wide networks of basic control. Given certain highways to construct and maintain and a specific amount of money to defray the costs of accomplishing work, it is understandable why the resident engineer hesitates to undertake a program of surveying basic area control on an area basis when such an undertaking offers no immediate tangible results for the separate highway surveys which have to be made in scattered places and in successive years. The cost of property and the intricacies of intercity expressways allow the urban engineer to establish control survey networks throughout his area with the resultant costs assignable on an easily recognized need basis. Property values in rural

Paper sponsored by Committee on Photogrammetry and Aerial Surveys.

areas, however, do not require as great an accuracy. Consequently, control survey networks in rural areas are not so easily justified.

Even in urban areas where justification is not a problem, actual execution of basic control surveys can be difficult in the extreme. Traffic in the more congested sections almost prohibits the use of second-order taping, except during short and inconvenient periods of time. Triangulation is often used in these cases, but problems of position location and building obstructions often present difficulties which involve elaborate and therefore expensive solutions which do not readily lend themselves to highway survey project requirements.

As a partial solution to the problems of making accurate basic control surveys in urban and rural areas the Texas Highway Department decided to purchase an electronic distance-measuring device. Investigations were conducted to determine the instrument most suited for control surveying in Texas according to the intended use. The investigations were made by discussions with and by study of published reports of photogrammetric firms, consulting engineering firms, and governmental agencies.

The three major units on the market at the time were carefully studied. Particular attention was given to the size and weight of the instruments and availability of maintenance services and the method of instrument operation. Based on these three considerations, it was decided to purchase the Electrotape.

Because complete and detailed information concerning the Electrotape instrument (insofar as its regular field performance is concerned) was not readily available, it was decided to request participation of the U. S. Bureau of Public Roads in a research project designed to determine the device's suitability for making basic control surveys needed in the photogrammetric compilation of maps, and measurement of cadastral data and profile and cross-sections, and also its utility as an adjunct to regular highway surveying instruments.

Such assistance was granted under the stipulations of a research proposal outlining the procedures to be employed and with the end product consisting of a complete report of the methods used, problems encountered, and accuracies and limitations involved in the utilization of the device under varying conditions.

OPERATION OF THE ELECTROTAPES

A Cubic Corporation representative delivered the instruments and remained several days to instruct operators and to acquaint personnel with the ordinary procedures of maintaining the equipment in operation. Field and classroom instructions (with an emphasis on practical applications rather than operational theory) were given, inasmuch as it was planned to allow the factory to service the instruments during the extent of the warranty period of one year. Two engineers were trained as operators who were to conduct the initial portion of the evaluation program and who would also serve as instructors when the instruments were subsequently used in a program of basic control surveying. The simplicity of operation of the units was such that the instruction period was comparatively short.

The control panel of the Electrotape units has been specifically designed for ease of operation and for the rapid and logical completion of the sequence of manipulations necessary in obtaining the distance-measurement data. A method of instrument operation (in step-by-step form) may be obtained from the Texas Highway Department.

CREW PERSONNEL

Originally the makeup of the crew consisted of two operators from the Austin office with the remainder of the personnel being furnished by the district offices. This practice was initiated in order to create an opportunity to train operators in the field and thereby provide the research program with the data necessary to draw conclusions as to the ease, or lack of it, with which the instruments could be properly operated by inexperienced personnel. As the program developed, a third member was added to the crew from the Austin office to provide an extra man who could be made available for instruction and explanation as the work progressed uninterrupted.

The overall electrotape surveying crew size will, of course, vary according to individual project needs. It has been concluded, however, that a four-man crew is sufficient when the objective pertains to making linear measurements. If angles are to be measured or elevations established concurrently while using the Electrotape, the survey crew size must be increased accordingly. If elevations or angular data are• necessary, a full five-man crew is recommended.

The capital investment in the Electrotapes and the need to attain as near constant operation as possible make addition of the units to a regular survey crew, complete with operators, necessary (whenever possible) so the instruments and operators can be withdrawn for reassignment as the work demands without unduly disrupting the activities of the remaining personnel.

ACCOMPANYING EQUIPMENT

The electronic distance-measuring devices are completely new and different types of surveying instruments which may, in time, alter most present methods of establishing basic control in the field. It is not surprising, and it is only logical to assume, the employment of such instruments will require changes in the usages and types of accessorial equipment.

The horizontal distance accuracy now easily obtainable is almost completely wasted unless the supporting instruments are capable of producing results of at least a comparable nature. The attainment of these results requires a theodolite which indicates angular measurements directly to one second of arc and a precision level with equal capabilities.

The two-way communication system built into the Electrotape units is completely adequate and very practical. The communication facilities are usable, however, only after the Electrotapes have been set up and turned toward one another. During the remaining periods of time, it is beneficial to establish some sort of oral communication to facilitate orientation, lines of sight, point location, and the like. In addition, the nonoperating party chief should have some method of advising survey crew members about the possible future setups and the progress of the work in general. Small two-way portable radios provide an excellent means of fulfilling this requirement, and three such units are considered adequate for an average crew.

The small nickel-cadmium batteries supplied with the Electrotape have an average life of approximately two hours per recharge. It has, therefore, been necessary to supply an additional two batteries for use during recharging cycles and for work in areas where the terrain demands walking to and from a site.

Binoculars have proved to be an invaluable adjunct to the equipment to check lines of sight and to identify obstructions in the line between points for which the distance is to be measured and in visually locating the companion Electrotape situated at a long distance away. Thus, seven power, 50-mm binoculars were provided each Electrotape operator.

In addition to the small nickel-cadmium batteries, two regular automotive 12-v batteries are utilized for setups where vehicular transportation is possible. The batteries normally installed in the automobiles assigned to the crew have been used as a power source in the past, but experience has proved this practice to be inconvenient and inefficient because it demands that the automobile remain at one of the instrument sites as long as measurements are being made and prevents the use of the vehicle for reconnaissance or for shuttling between the units. A fully charged automobile battery will last approximately eight hours or through two normal working days before it requires recharge.

Survey traverses on the order of those measured by use of these electronic instruments should always be referenced to permanent monuments in the previously established network of basic control surveys of known accuracy or to the position of celestial bodies at some instant of time to preserve the effort and expense expended. Where monuments are inaccessible or nonexistent, observations on Polaris, a satellite or other foreign reference points are necessary and convenient. In such determinations, the exact time must be known to an accuracy beyond the capabilities of ordinary

mechanisms. The National Bureau of Standards operates powerful radio transmitters, which indicate by electronic tones, time intervals based on an atomic clock and these transmissions are discernible through a radio receiver on frequencies of 2. 5, 5, 10 and 15 megacycles. The use of a portable receiver is recommended.

The accuracies obtained by the Electrotape present a problem in field calculations. Multiplication and division of eight-digit figures become commonplace in checking the work in the field and require furnishing the survey crew with a simple field calculator. Slide rule or manual calculating is neither fast nor accurate enough to keep pace with the progress of the crew.

In checking the calibration of the instruments and in performing short-distance measuring, an invar or lovar tape, work tapes, chaining thermometers, tension handles, and related equipment are required. Of course, the typical surveying accessories are provided, but they are common to any survey crew and cannot be considered out-of-the-ordinary equipment.

DESCRIPTION OF SEVERAL PROJECTS

It has been the primary intention of the Texas Highway Department in accomplishing the objectives of this research project to test the instruments in as many diverse types of conditions as possible. Thus far, experience has ranged from the use of the Electrotapes in making an ordinary highway survey to the setting and measuring the position of recoverable reference monuments to a high degree of accuracy. The following is a partial listing of the projects conducted in this research:

US 59 in Fort Bend County

This is a combination of reconstruction on existing location and construction on new location of a highway traversing approximately 45 miles of Fort Bend County in southeast Texas.

Its value in the program lies in the different conditions encountered along the route. It offers flat open terrain, densely wooded areas, a major river crossing, and as mentioned before, both existing and new locations.

The Highway Design Division in Austin of which the Photogrammetry Section is a part was requested to furnish the Houston District Office with planimetric and topographic maps at a scale of forty feet to the inch. It was decided to utilize the Electrotapes in the establishment of second-order control to be used as a basis for compiling the maps. Using U. S. C. and G. S. monuments as reference points, an Electrotape traverse was measured along the proposed facility throughout the full length of the project. Iron pins were set in such a manner as to locate the P. I. 's of the alignment and P. O. T. 's were established at an interval of approximately 2, 000 feet where intervisibility would permit.

The Electrotapes were used to determine distances between the P. I. 's and P. O. T. 's and precise angular measurements were made at all tangent intersections to form a basic traverse with ties to existing U. S. C. and G. S. monuments as often along the project as existence of the monuments would allow.

The entire 45 miles of traverse consisted of four segments each of which began and ended on monuments of existing survey control points. Closure errors were calculated for each segment as if they were independent of one another and for the entire traverse as a single unit. A 10-sec theodolite was used for angular measurements at the P. I. 's and no special effort was made to secure unusual accuracies. Differences in elevations were obtained either by measurement of vertical angles or by differential leveling. Total errors of closure were as follows: Segment 1—1/14, 000; Segment 2—1/90, 000; Segment 3—1/20, 000; Segment 4—1/16, 000; Segment 1 through Segment 5—1/20, 000; and total number of Electrotape measurements—108.

Subsequent investigations revealed angular errors which, when corrected, improved the accuracies obtained.

I-45 in the City of Dallas

The project was located in a heavily congested urban area of one of the larger metropolitan centers. The project was selected primarily to study the effect of moving

traffic and the presence of large structures on the accuracy of Electrotape measurements.

The traverse was measured along sidewalks to avoid disrupting traffic and along alleyways, wherever possible, and was completed as a closed loop for the accuracy check. Occupied stations were preserved by chiseling survey marks in concrete. Distances too short to be measured by the Electrotape were taped to complete the traverse.

Angles were measured with a direct reading 10-sec theodolite, and differences in elevation were determined by spirit leveling.

TABLE 1

Line	Dist. Measured by (ft)		Diff. (ft)
	Tape	Electrotape	
ON	465.92	465.97	+0.05
OP	464.76	464.78	+0.02
FG	371.33	371.26	-0.07
EW	4,308.28	4,308.27	-0.01
CB	536.39	536.38	-0.01
CB-1	781.70	781.76	+0.06
AB	550.66	550.65	-0.01
AC	770.82	770.75	-0.07

The traverse closed on the beginning point with an accuracy of one part in twenty thousand. Twenty-one measurements were made for the entire loop.

To complete the test, most of the project was subsequently taped along the identical courses measured by the Electrotape to serve as a basis of comparison. Differences in lengths are given in Table 1.

State Highway 206 in Eastland County

It is proposed to reconstruct and realign State Highway 206 for a distance of approximately 20 miles along its present location and along totally new location. Planimetric and topographic maps at a scale of forty feet to the inch together with cross-sections were requested from the Photogrammetry Section by the District Engineer.

Part of the corridor of proposed location for the highway was densely wooded. Thus it was decided to utilize the Electrotape in establishing horizontal control in an effort to determine the instrument's capabilities under such conditions.

As in preceding projects, Electrotape stations were established at an interval of approximately 2,000 feet throughout the project by setting iron pins which were ¾ inch in diameter and 3 feet long, at each measurement station. The meander traverse was comprised of tangents as long as the location would allow and, where conditions required, measurement lines were cleared to a minimum degree.

Only two U. S. C. and G. S. monuments were recovered in the area and plans were made to include them in the traverse as closure stations.

A total of 75 measurements were required and final closure was 1 part in 18,000. After this basic traverse was closed and adjusted, targets were placed on points which were to serve as supplemental control and the position of these points was surveyed by transit and tape, using the stations in the basic control, as surveyed by the Electrotape, for closure and adjustment points.

TYPES OF SURVEYS

Use of Electrotape in Conjunction with Regular Surveying Instruments

The concept of acceptable use for instruments such as the Electrotape is based on the premise that their primary function should be establishing or extending horizontal control over great distances with high orders of accuracy in relatively short periods of time. As important as this function is realized to be, it cannot contribute to a diversified control surveying program with the flexibility necessary to alleviate the tedious, time-consuming tasks of surveying which have plagued highway organizations for years. It is the goal of the Texas Highway Department to fit this electronic instrument into normal highway surveying in such a manner as to permit new practices to be formed which could result in lower preliminary engineering costs and more effective use of available personnel through the revisions in theretofore normal procedures.

Efforts made thus far have been directed toward achieving such a goal. As a consequence, a tentative program has evolved which enables the resident engineer to

produce second-order surveys for all types of projects without the necessity of employing expensive techniques usually associated with transit and tape methods. Basically the program consists of the following steps:

As it becomes necessary to establish ground control for compiling maps by photogrammetric methods, a preliminary study is made in office and field of the area to determine the existence of referenced monuments, either Federal or State, and the most economical route the control surveying should follow. When this determination is made, a series of semi-permanent survey markers are set throughout the project in areas outside the anticipated limits of future construction. A theodolite is used in measuring position of the markers, and tangents are made as long as conditions permit. These markers, whether at points of intersection or points on tangent are spaced at an interval of approximately two thousand feet, and usually consist of reinforcing rods three to four feet long driven below the natural surface of the ground. Monuments marking existing control points which can be recovered in the area are included in the traverse and efforts are made to begin and end the surveying on markers of control points set by Federal agencies.

Electrotape measurements are made between the iron pins and to all existing monuments which can be recovered. The field notes of the basic traverse thus measured are reduced and the errors of closure are adjusted.

Assuming the topographic maps will be compiled at the normal scale of forty feet to the inch for design purposes, placement location for the targeted control points is established by typical chaining methods at an interval of three hundred feet. No effort is made to employ precision methods in the taping processes, since adjustments are subsequently made to the taped distances between each marker by use of the Electrotape measurements. In effect, the project traverse consists of numerous taped traverses none of which are longer than 2,000 feet. In this manner, taping errors are not allowed to accumulate and overall accuracies are always as great or greater than second-order accuracy.

The time expended (and therefore the cost) of such an operation is no greater than is normally required to make such surveys without the Electrotape, and yet the accuracies are usually much more dependable.

When the mapping operations are completed, the designed highway alignment is accurately plotted by coordinates on the map sheets, critical points of control along the centerline are identified by coordinates and established in the field using the iron pin markers as reference points, the position of each of which had been measured by use of the Electrotape. In this manner, the field notes for the centerline can be completed in the office permitting more attention to be given to particularly critical sections of the highway location which will be classified as priority areas during construction. Furthermore, construction staking is not dependent on long and involved transit-tape surveys, but may be accomplished in short segments, beginning at any point throughout the highway project during any phase of construction.

Use of Electrotape as a Separate Surveying Instrument

The Electrotape is well suited to function independently. Its primary independent use is in checking taped distances and in transferring control surveying measurements over long distances. Traverses measured by use of transit and tape in which closure errors indicate blunders exist are normally checked by remeasuring until discrepancies appear in comparative measurements. This can be a long and involved process requiring the time of experienced personnel whose services can more productively be utilized elsewhere. Using two Electrotape units and operators, the tangents of a survey can be checked in much less time than is otherwise required and at much less expense.

Another method of utilizing the Electrotape instruments as a separate unit consists in the making of a straight-line survey. For example, the control layout for a long structure for a river crossing is usually measured by triangulation with actual measurement along the centerline never having been made directly in the field. These crossings are very often over rugged topography where accurate taping is virtually impossible, and locations containing angles are usually measured and referenced using angle-measuring instruments.

The Electrotape is well suited to check the horizontal distances thus determined and proof of the instrument's measurement accuracy is provided from closed loops which are a part of these checks. One Electrotape unit is positioned on the centerline where observations can be made to points along as much of the alignment as is possible. This primary position should, for maximum effectiveness, be as near one end of the proposed structure as conditions will permit. The second Electrotape unit is moved from point to point along the centerline, and measurements are made between it and the first unit until the resultant series of measurements forms a distance between marked points which is equal to or exceeds the entire length of the structure. Variations of this simple procedure produce several measured segments of the centerline, the total of which can be compared to the overall length to form a straight-line closed traverse. The points selected to comprise the end of each segment may be those between which distances were already measured by other means or may be points at which markers were set at random as the Electrotape survey progressed. In either case, the distance measurements checked or initially measured can be used with full confidence in their accuracy and, if necessary, the position of the markers established thereby can form the basis for referenced control points to be used during construction.

Use of the Electrotape for Trilateration

The extension of horizontal control measurements over relatively long distances with a minimum of delay and expense appears to be one of the main benefits to be derived from use of an electronic distance-measuring device. As mentioned, it has been the primary purpose of the Texas Highway Department to study methods of utilizing the Electrotape in normal highway surveying. As this study continues, however, it becomes more obvious that it will be necessary to examine all potentials of the Electrotape to accomplish a complete and thorough evaluation of its multiple uses.

To realize the full advantages of trilateration, a complete electronic computer program is being developed which will accept the Electrotape-measured slope distances in centimeters and the vertical angle or difference in elevation between points for which the horizontal distance of each leg in a quadrilateral is required as input information and as output furnishes each horizontal distance in feet, each angle involved and its error of correction, and the X, Y, and Z coordinates of each point. This information may be obtained based on any horizontal reference plane which might be used.

Trilateriation by the field measurement of distances and the calculation of angles, accomplishes the same basic result as when triangulation is done by use of precision theodolites. Triangulation by trilateration, however, would appear to be much more economical and rapid in accomplishment than by other methods.

EVALUATION OF THE ELECTROTAPE

Training of Instrument Operators

The training of operators for the Electrotapes presents no serious problems. The instruments, whether acting as interrogator or responder, are simply and easily manipulated in a short time by the average surveyor. It is estimated acceptable measurement data can be obtained with the instruments by personnel completely unfamiliar with the procedures involved after one to two days of practice. Efficient and somewhat rapid operation can be expected in approximately one week. Recording the measurement readings on the data sheets in the proper location and the reduction of the initial data to slope distances in meters is easily understood and accomplished.

No correlation has been observed between the efficiency of individuals serving as Electrotape operators and the extent of their experience in surveying. It is obvious, of course, the experienced surveyor will require less supervision, when acting as an instrument operator on a field survey crew, than a person lacking such experience; but, it also became obvious it was not necessary to be an experienced surveyor to be an effective Electrotape operator. The same lack of correlation has been noted concerning the good operator and his educational background insofar as degrees of scholastic attainment beyond the high school level are concerned. Following the operating

procedures recommended by the Electrotape manufacturer, an experienced instrument operator can readily become aware of instrument malfunctions as they occur and may also recognize discrepancies in recorded data which indicate poor measurements.

The training of inexperienced operators should not be concentrated toward understanding the functions of the Electrotape, as much as toward establishing a pattern of operation to be followed consistently in making each successive measurement day after day. Knowledge of the function of each dial is easily transferred to the new operator, and no difficulty has been experienced in this regard. It has been noted, however, that the main objective in a training program should be indoctrination of a system and the sequence of actions which should be rigorously followed in making each measurement. Speed in operation will naturally follow as proficiency is gained through repetition.

Transportation and Portability of the Instrument

Transportation of the Electrotape requires no greater precautions than those exercised in transporting an optical surveying instrument. The units withstand the normal shocks involved in overland vehicle travel with no apparent effects. Vehicles used in transportation have included stationwagons equipped for surveying, pick-up trucks, regular passenger automobiles, carry-alls, and jeeps.

In no case has the Electrotape suffered internal damage as a result of handling even though no special or out-of-the-ordinary efforts were made to insure their safety. It can be concluded, damages to be expected as a result of transportation are so slight as to exclude this consideration in the overall evaluation of the instruments.

The units are completely self-contained and therefore absolutely portable. The weight involved (36 lb) is somewhat difficult for one man on long carries. This weight, however, is not prohibitive. Because a small nickel-cadmium battery is included as an internal part of the instruments, no external connections are required for operation and accessory equipment can therefore be held to a minimum where necessary.

Accuracy of the Electrotape

The extreme limits of accuracy of the Electrotape have been fully and completely investigated by other organizations and the results have been published. Therefore, this presentation pertains to accuracies in the range of second-order and the procedures necessary to attain this degree of accuracy consistently, and in a practical and economical manner.

Under this stipulation, only two primary conditions have been found to affect the instrument's ability to measure distances. These conditions are obstructions across the measurement line between the survey control points and the magnitude of the distance measured.

The effect of obstructions intervening between two Electrotape units upon the wave patterns emitted by any transmitter varies as the size, shape, density, and mass of the obstruction, its proximity to the transmitter or receiver, whether on the line to be measured or somewhat laterally removed therefrom (and if so removed the extent of the offset), whether of a reflective or absorbative nature and the degree of either, whether the obstruction is in motion or is stationary and, if in motion, the nature of the motion, whether the obstruction is singular in its geometric formation or part of a pattern that is repeated and, if repeated, the number of repeats, and so on, to what seems to be an infinite series of variables. Therefore no attempt shall be made here to define these effects. It can be said, however, rather limited experience dictates there should not be any on-line obstructions between the instrument units if maximum accuracies are to be obtained. When it is absolutely necessary to measure a distance for which the measurement line is obstructed, a careful study and comparison of the data which constitute the two measurements should be made to determine whether the difference in lengths would be normal for the conditions being encountered if no obstructions were present. This difference in measured length is generally termed the "spread" and an experienced operator can readily determine the spread which is and which is not acceptable.

In a great majority of the cases reported in this paper, the traverse must be clear insofar as optical line of sight is concerned for the measuring of angles and for the taping of short distances; therefore the problem of obstructions is not a major concern. For traverses through areas where the ground cover is comprised of woods, Electrotape measurements should not be attempted until an optical line of sight is effected between measurement points.

Measurements made in what may be termed "confined corridors" have been very successful. Most of the traverses measured in congested urban areas have required the making of tangent measurements parallel to buildings and approximately six to eight feet therefrom. This close proximity of concrete and steel had no noticeable effect on the traverse closures. Accuracies of more than second-order were obtained without difficulty. Continuous and intermittent streams of traffic parallel to the traverse being measured caused a slight fluctuation of the null meter indicator but again no appreciable adverse effects were realized.

The Electrotape's ability to maintain its usual accuracies in "closed corridor" traverses met its supreme test in a project along State Highway 87 in Jefferson County. A large oil refinery plant occupies the area immediately abutting the right-of-way on both sides of the highway. The air above the roadway and ground below the pavement are dense with high power transmission lines and pipe lines. To avoid the heavy streams of traffic as much as possible, measurements were made along side a 6-ft chain-link fence at the edge of the right-of-way. Ties to existing monuments at either end of the project proved the measured distances closed within one part in twelve thousand.

Measurements of relatively short distances are now being studied to determine the practicable limit of the instrument's capabilities in this extreme. Experience, so far, has indicated this limit is controlled by principles of economics rather than by accuracy.

An experienced taping crew has the capability of extending a traverse approximately 1,000 to 1,500 feet in the same length of time required to set up the Electrotapes, measure the distance twice, and move to the next point. Because the time involved for a taping crew to measure a given distance varies directly as the length of the line and the time involved for an Electrotape crew to measure a given distance is constant regardless of the length of the line, it logically follows that as the distances become shorter, use of the taping crew becomes more economical, comparatively, and the Electrotape's usefulness diminishes. Furthermore, the Electrotape's possible error ($\pm$1 cm) begins to become an important consideration in shorter measurements. Because of these two factors, economics and accuracy, Electrotape measurement of distances less than 1,200 feet is not attempted unless absolutely necessary.

Economics

The cost of surveying operations should never be considered the deciding factor in any project. Such cost, however, should be considered in relation to the expense involved in alternate types of surveys. It has been proved that the basic control established for a highway survey project is usually the most important, most underrated, and least expensive portion of the survey.

The costs of an Electrotape crew are extremely economical when compared with the benefits derived from accurately measuring the position of a series of identified monuments set throughout an area where no geodetic control otherwise exists.

As is evidenced (Table 2) the average cost of operating the Electrotape in making basic control surveys for photogrammetric compilation of maps is approximately $24 per man day. It should be pointed out that this cost is greater than can be ordinarily expected because the instruments have, for the purposes of this research project, been operated to a large extent by professional engineering personnel whose salaries are well above the range usually offered for this type of work.

The cost per measurement is very economical when it is considered one set of measurements could extend control for a distance of 20 to 30 miles over extremely rough and wooded topography. Comparisons between this cost and those incurred by a taping crew covering the same distance are given in the following two examples:

TABLE 2

COSTS OF DISTANCE MEASURING WITH THE ELECTROTAPE

Project	Man Days	Total Cost ($)	Cost/Man Day ($)	Number Measurements
A, rural	107	2,547	23	108
B, urban	8	217	27	21
C, rural	115	2,734	24	75
D, rural	105	2,462	23	100
E, urban	30	558	19	N.A.
F, urban	40	1,047	26	24
Average			24	

I-10 in Kerr County. —A base line was measured by taping for a distance of approximately 26.6 miles. Normal taping techniques were employed and the crew of five men required 30 days to complete the measurements. The Electrotape was employed to check the distances in lieu of other types of closures and for four men thus involved required four days to complete the check. Such procedure resulted in a savings of approximately 100 man days.

Ranch to Market Road 2475 in Jack County. —A traverse measured by taping and closure tied to existing monuments had an overall closure accuracy of 1 part in 11,000. Some doubt existed, however, as to the accuracy of individual courses, and the Electrotape was utilized to check each taped distance. The survey was checked in one working day, as opposed to a minimum of six working days which would have been required by a regular taping crew to do the same amount of checking. The resulting savings were 24 man days with an increase in closure accuracy to 1 part in 50,000.

Maintenance

The out-of-service or down-time record for these instruments is exceptionally small, and the average over the past year and one-half is approximately 4.7 percent. None of this lost time occurred as a result of rough field handling, although no unusual precautions were taken during transportation and use of the Electrotape units.

Maintenance records indicate the instruments were returned to the factory seven times for repair and adjustments. A majority of these cases involved simple problems that could not be considered out of the oridinary for complicated electronic equipment. The nature of the repairs ranged from loose electrical connections to a bent antenna.

Heretofore all repairs and adjustments have been accomplished at the San Diego factory by Cubic Corporation personnel. A repair shop, however, is now being outfitted at the Austin office to effect all future maintenance. Factory service for the Electrotape has been very excellent in regard to both time and effectiveness. On several occasions, the Electrotape units were shipped, repaired, and returned in a period of from three to four days. As this period was often arranged to include a weekend, only one to two working days were lost.

Field maintenance of the Electrotape units is extremely nominal. Ordinary care will preserve the appearance and proper mechanical functions of the instruments for a long period of time.

Electronic maintenance should never be attempted in the field due to the very complicated nature of the circuits and the critical frequency tolerances involved. It is recommended no adjustments or changes be made in the electronic components except by factory-trained personnel or by the factory itself.

The closure turnbuckles on the antenna cover are somewhat weak and breakage should be expected occasionally. The rubber sealing gaskets on the antenna cover and front panel cover are susceptible to displacement through usage and should be periodically checked for dust and waterproofness.

The tuning knobs are sturdily constructed and tightly secured to the spindles and should present no serious difficulties.

The attachment of the tripod adapter to the fiberglass case is of doubtful strength because of the inherent nonrigidity of the shell, and the connection should be checked

frequently for signs of fatigue cracks or ruptures. The bolts forming connections should be tightened occasionally to prevent the case from shifting on the adapter and thereby dislocating the electronic center.

For fullest utilization, the units should require no more care in field transporting and handling than would be ordinarily given an optical surveying instrument. Consequently no special precautions have been taken to protect the instruments from mistreatment other than precautions usually taken with a transit or theodolite.

The square shape and high center of gravity of the instruments create a problem when operating in high to moderate wind velocities. This is not a correctable condition insofar as mechanical modifications are concerned. Prudent care, however, should be exercised in the height of setup, implantation of the tripod shoes, and constant close proximity of the operator when such weather is experienced. Operation in a light rain is possible. The antenna disk is sealed against the case, and water infiltration from that direction is not likely. Excessive exposure of the dials and counter to rain will undoubtedly cause some difficulty, and the employment of a large umbrella is necessary when operation in the rain must continue. At present, insufficient data prevent the drawing of a conclusion as to the accuracy obtained when measurements are made during raining conditions.

The antenna is a rather delicate appendage and is easily bent or otherwise damaged. When such damage occurs, transmitting efficiency may be seriously impaired; consequently the antenna cover should always be attached except when measurements are actually being made.

It has not been necessary to utilize the large wooden cases in ordinary transporting, and their inclusion when the instruments are carried by a walking field crew adds weight that is totally unjustified. They have been reserved for those instances when it is necessary to ship the instruments to a distant destination.

CONCLUSIONS

In conclusion, the following points are offered in summary of the foregoing:

1. The Electrotape has a definite and valuable use in the work of a normal highway survey crew in:
    a. Establishing closure stations approximately 2,000 feet apart thereby insuring ultimate closure of at least second-order for the traverse at the expense and time usually required to make a third- or fourth-order survey by usual on-the-ground surveying methods.
    b. Extending control from the end and/or beginning of a transit and tape survey for closure on a monument in the previously established network of basic control, or in forming the return loop of a closed circuit to prove second-order accuracy.
    c. Performing in an efficient and economical manner to allow accomplishment of (a) and (b) preceding.
    d. Finding taping errors in completed transit and tape surveys more efficiently than could be accomplished by other ground surveying techniques.
    e. Determining distances over topographic areas inaccessible to a taping crew.
2. As a surveying instrument, the Electrotape is capable of:
    a. Being effectively operated by inexperienced personnel after a minimal instruction period.
    b. Withstanding normal field handling.
    c. Requiring very little maintenance.
    d. Achieving accuracies of second-order or better without resorting to the usual and expensive second-order surveying techniques.
    e. Requiring few accessories for efficient operation.

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

Paper sponsored by Committee on Photogrammetry and Aerial Surveys.

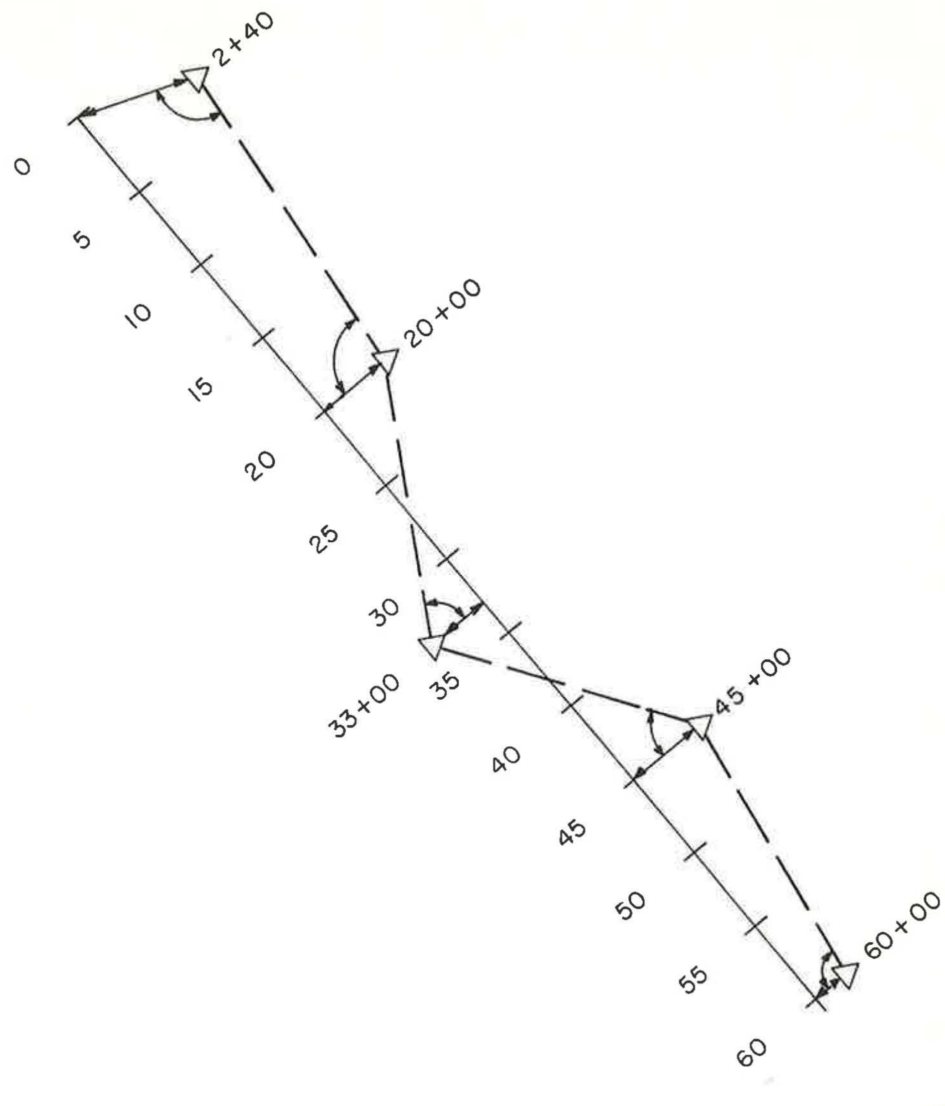

Figure 1. Establishing exact position of plane coordinate computed centerline in the field.

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 micro-switch 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.

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.

Paper sponsored by Committee on Photogrammetry and Aerial Surveys.

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

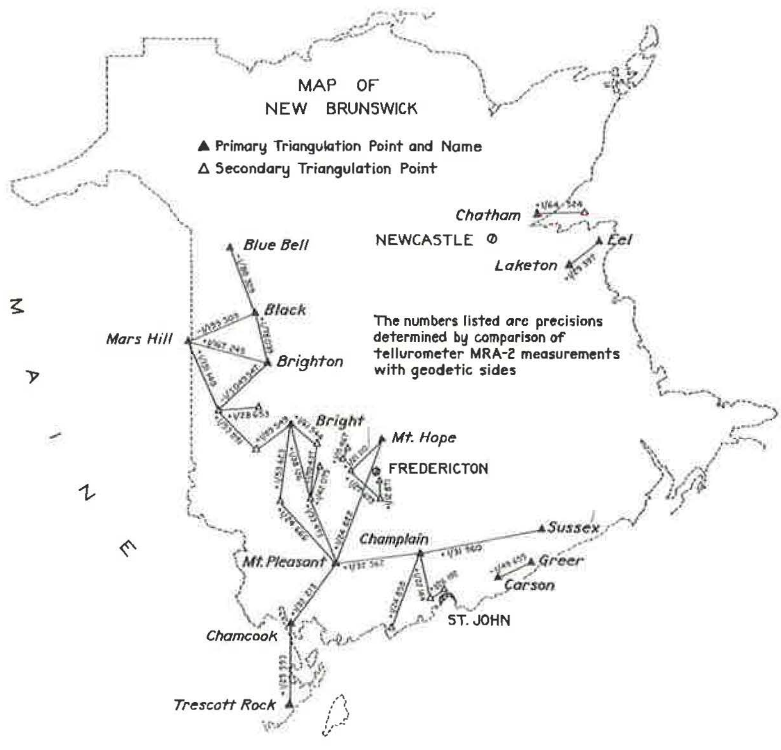

Figure 1. Triangulation sides measured by Tellurometer MRA-2 (Precisions).

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

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

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 (Δx and Δy) can be distributed. If the approximate coordinates are designated as:

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

and

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

the adjusted values become:

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

and

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

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

The closing error will be expressed as:

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

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

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

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

Terms also referred to are as follows:

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

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

$$1/Q = \frac{\Delta q}{\sqrt{(x_E - x_B)^2 + (y_E - y_B)^2}},$$ 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.

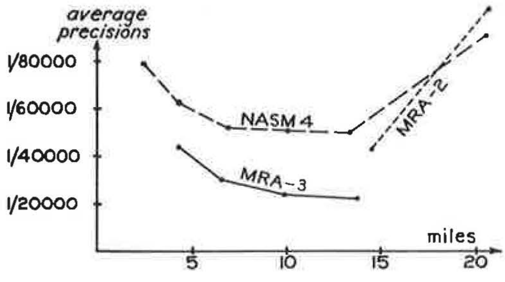

Figure 2. Lateral precision for traverses of various length.

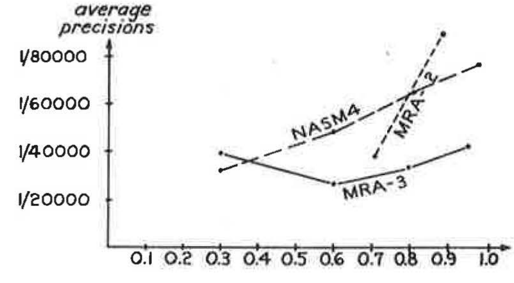

Figure 3. Lateral precision for traverses of various bending ratio

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

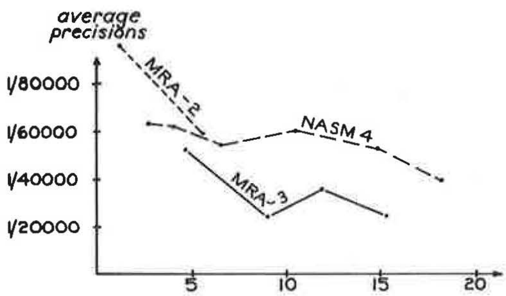

Figure 4. Lateral precision for traverses with varying number of legs.

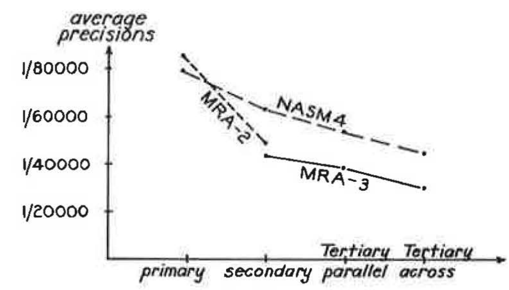

Figure 5. Lateral precision for traverses of various order.

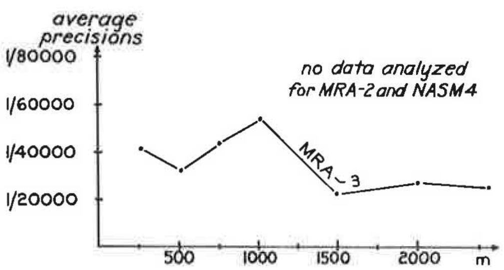

Figure 6. Lateral precision for traverses with varying shortest distance of leg.

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.

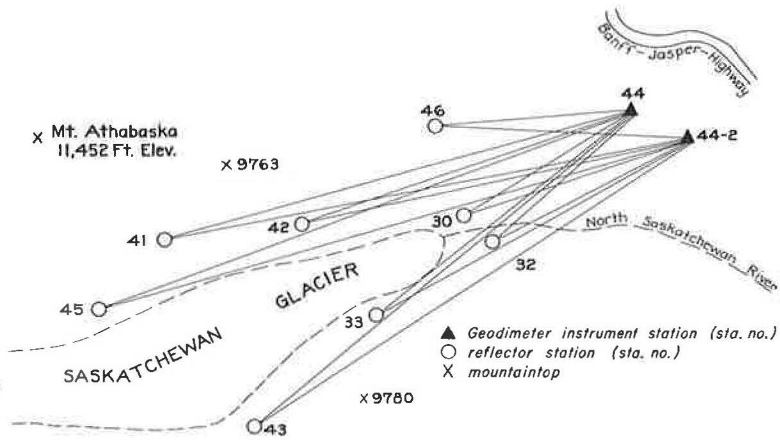

Figure 7. Control survey for photogrammetric mapping of the Saskatchewan Glacier, Alberta.

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

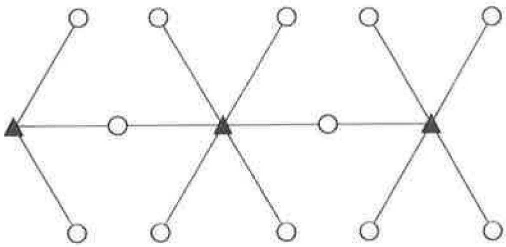

Figure 8. Layout of Geodimeter-measured control for orientation of photogrammetric models (ideal layout).

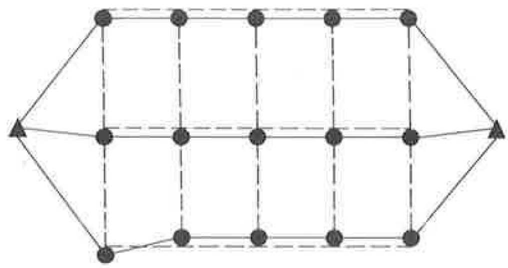

Figure 9. Layout of Tellurometer-measured control for orientation of photogrammetric models (ideal layout).

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.

REFERENCES

1. DeJong, S. H., "Research and Experience with Tellurometer and Geodimeter in Canada." Proc. 5th Nat. Surveying Teacher's Conf., ASEE Comm. VII (1962).
2. Gotthardt, E., "Uber die Eignun des Geodimeters NASM-4 zur Verdichtung von Festpunktfeldern für die Zwecke der Flurbereinigung." Zt. für Vermessungswesen, Stuttgart, pp. 125-138 (1962).
3. Konecny, G., and Roberts, W. F., "The Use of Tellurometer Observations in Establishing a Coordinate Survey Control System in the Province of New Brunswick, Canada." Proc. 1962 Tellurometer Sym., London (1962).

4. Konecny, G., and Roberts, W. F., "Electronic Surveys in New Brunswick." Jour. Surveying and Mapping Div., ASCE, pp. 17-36 (Oct. 1963).
5. Lilly, J. E., "Canadian Tellurometer Investigations 1957-59." Proc. Internat. Union of Geodesy and Geophys., Assoc. Geodesy, Helsinki (1960).
6. Saastomoinen, J., "Some Experimental Results on the NASM-4B Geodimeter." Canadian Surveyor, pp. 47-52 (March 1963).

Accuracy Requirement and Uses of Electronic-Measuring Devices for Surveying Photogrammetric Control

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•HIGHWAY DESIGN is a complex and integrated activity, and highway surveying is no exception. Different kinds of methods have to be employed simultaneously to achieve the desired results and to keep pace with the other phases of national production. These methods include surveys made in the usual manner on the ground, made by using aerial photogrammetry, and made by using electronic distance-measuring instruments; all of which are supplemented by use of high-speed electronic computers. To fulfill the demand for mass production, such methods must be harmonically organized among each other. Therefore it is necessary to have full knowledge of the nature of the methods as well as full knowledge of error propagation of the different methods. This paper contains an analysis of the methods from the error propagation point of view, providing mathematical and practical examples. All practical examples mentioned were sponsored by the Ohio Department of Highways.

AERIAL TRIANGULATION IN PRELIMINARY SURVEYING

One of the most discussed methods is aerial triangulation, which was not able to provide the required high accuracy for detailed preliminary location surveys. But relatively low accuracy is required in the reconnaissance stages of route surveys. Consequently a less accurate method, such as aerial triangulation, can be introduced to this phase of activity. An aerial triangulation method using double projection photogrammetric instruments (Kelsh type) has been investigated and published (12). This publication contains information about strip aerial triangulation by use of a double projection instrument. The photographs were taken at a scale of 500 feet per inch using a K-12 aerial camera.

The photography strip contained six stereoscopic models and was controlled at the beginning and at the end of the strip. The average standard residual error of a point was found to be ±1.0 feet and the average vertical error was about ±0.6 feet. This favorable error propagation makes it possible to compute the maximum bridging distance by use of the following empirical equation.

$$mp = h \sqrt{N} \, mp_o \tag{1}$$

in which

- mp = standard residual position error (in meters);
- h = flight height (in kilometers);
- N = number of stereoscopic models;
- mp_o = standard residual position error in meters for $N = 1$, which can be computed using the equation: $mp_o = m_o M_M$ in which M_M is the scale of the stereoscopic model, and m_o = standard error of observation.

Paper sponsored by Committee on Photogrammetry and Aerial Surveys.

Using the preceding equation, an example can be computed. If the desired horizontal accuracy is not greater than ±2 feet and the mapping scale is 200 feet per inch, the number of stereoscopic models usable for the aerial triangulation would be about six, and the bridged distance would be about ten miles. Two conclusions can be drawn. First, the employment of aerial triangulation would provide about 40 to 60 percent savings in the ground control measurements required. Second, the derived mathematical equations indicate 1:10,000 relative accuracy would be suitable for ground surveyed control, because the previously mentioned ±2-ft residual error in a distance of ten miles is representing about 1:26,000 relative accuracy. But before the final answer would be given to the question, how accurately should the ground surveyed control be measured, it is necessary to analyze the surveying for right-of-way acquisition and determine what maximum accuracy is needed.

MEASUREMENT BY PHOTOGRAMMETRY FOR RIGHT-OF-WAY ACQUISITION

Several attempts have been made in the United States, as well as in foreign countries, to employ photogrammetric methods to solve right-of-way problems. Most of the methods can be classified as graphical, which have their limitations. A map compiled by photogrammetric means usually contains three categories of errors (including photographic, plotting, and measurement) which get into the right-of-way data when measurements are made from the map. If a computational method is employed, the measurements required for right-of-way computation are not obtained from the photogrammetrically compiled map, but are measured on the stereoscopic model in a precision photogrammetric instrument. In this case, the measurements are free from map plotting errors as well as from the errors in measurement, and the photographic errors can be corrected for mathematically.

This possibility was investigated and the mathematical conception of the method has been published (13). In this method the stereoscopic model coordinates have been observed and corrected by use of the following equations:

$$X_P = 0.5\,(X''_P + X'_P) + 0.5\left[\frac{X'_P}{b}\left(\frac{X''_P - X'_P}{2}\right) + \frac{X''_P}{b}\left(\frac{X''_P - X'_P}{2}\right) + \frac{X''_P - X'_P}{2}\right] \tag{2}$$

$$Y_P = 0.5\,(Y''_P - Y'_P) + 0.5\left[\frac{Y'_P}{b}\left(\frac{Y''_P - Y'_P}{2}\right) + \frac{Y''_P}{b}\left(\frac{Y''_P - Y'_P}{2}\right) + \frac{Y''_P - Y'_P}{2}\right] \tag{3}$$

in which X_P and Y_P are the correct stereoscopic model coordinates of a point, and $X'_P\ X''_P$ and $Y'_P\,Y''_P$ are the coordinates measured on the normal and on the pseudo stereoscopic model.

The correction of the coordinates in the stereoscopic model by use of the equations leads to the result which indicated about 40 to 50 percent improvement in accuracy.

Eight different stereoscopic models were examined, and the results indicate the increase in accuracy is about 45 percent in both X and Y coordinates as well as in Z. These photographs were taken of the camera testing area of U. S. C. & G. S. in northern Ohio with a Fairchild F-501 (6-in. focal length) camera at the scale of 1:12,000 (1,000 ft per inch), and the stereoscopic model scale was 1:5,000. Targets were placed on all of the examined points before the photographs were taken, and their ground position coordinates were obtained from U. S. C. & G. S. in State Plane Coordinate System of Ohio. The measurement of coordinates of the control points in the stereoscopic models was done with Wild Autograph Model A7, and orientation of the models was accomplished by use of optical-mechanical orientation methods in the usual manner.

The residual position error of a point was found to be ±19 cm or 0.62 ft without correction, and about 13 cm or 0.43 ft with correction. The average residual error in elevation was found to be 1:7,000 of the photography flight height without correction and 1:13,000 after using the correction equations.

From these results a conclusion can be drawn. It is possible to achieve high accuracy by use of computational photogrammetric methods. Also this accuracy is suitable for solution of right-of-way problems, especially when it is realized the scale of these experimental photographs was 1,000 feet to one inch and the desirable scale for right-of-way measurement and description purposes would be 200 feet per inch. One could expect the maximum error to be reduced to 0.2 of a foot if the photography scale is 200 feet to one inch.

Such a result must be further analyzed because the residual error introduced by photogrammetry combines with the error in ground surveyed control.

Derivation of Error Propagation Equations

By having the correction equation, the error propagation in coordinates of the stereoscopic model can be derived. According to the error propagation law of the least-square theory, the standard error of a function is

$$\mu_X = \sqrt{\left(\frac{\partial X}{\partial x}\mu_x\right)^2 + \left(\frac{\partial X}{\partial y}\mu_y\right)^2 + \left(\frac{\partial X}{\partial z}\mu_z\right)^2} \tag{4}$$

if the function is $X = F(xyz)$, in which x, y, and z are the variables.

Substituting the partial derivatives obtained from the correction equation the final equations in simplified form become

$$\mu_{X_P} = \sqrt{(\mu_{X''_P})^2 + \left(\frac{X''_P}{2b}\mu_{X''_P}\right)^2 + \left(\frac{X'^2_P}{2b}\mu_{X'_P}\right)^2 + \left(\frac{X''^2_P - X'^2_P}{4b^2}\mu_b\right)^2} \tag{5}$$

$$\mu_{Y_P} = \sqrt{(\mu_{Y''_P})^2 + \left(\frac{Y''_P}{2b}\mu_{Y''_P}\right)^2 + \left(\frac{Y'^2_P}{2b}\mu_{Y'_P}\right)^2 + \left(\frac{Y''^2_P - Y'^2_P}{4b^2}\mu_b\right)^2} \tag{6}$$

and the standard position error of a point in a stereoscopic model is

$$P = \sqrt{\mu X^2_P + \mu Y^2_P} \tag{7}$$

Using such examples the following result has been obtained: $\mu_{X_P} = \pm 10$ microns, $\mu_{Y_P} = \pm 12$ microns, and $\mu_{m_P} = \pm 16$ microns.

Multiplying the standard position error by the 1:5,000 scale of the stereoscopic model the average standard position error is $\mu_P = \pm 8$ cm or about ± 0.26 ft.

Upon analyzing the average standard position error, it is evident such an error is much smaller than the average standard residual position error computed by using the actual errors. The difference is about ± 6 cm or 0.2 ft approximately. This is a clear indication a measurable error exists in the final result, which is due to causes other than photogrammetry. Consequently the result had to be given further analysis.

For the numerically achieving absolute orientation of the measured stereoscopic models, three control points have been used. The average distance between these points is about 1,300 meters. According to the U.S.C. & G.S. information, the coordinates of ground surveyed control points have been determined from second-order traverses attaining a relative accuracy of 1:25,000, which represents 5.2 cm or about 0.16-ft error at the distance of 1,300 meters between control points. This relative error corresponds well to what was obtained by theoretical consideration.

The conclusion can be drawn that the standard position error of a photogrammetrically determined point includes the errors of photogrammetric procedures as well as the errors of the ground survey. Therefore the ground control cannot be assumed as practically errorless as is the custom in common practice today.

The error propagation equations, including both ground and photogrammetric surveying, have been derived but do not represent high practical values because they are too complicated. They can be replaced by more practical equations, and the following is recommended for practical use:

$$\mu_P = \sqrt{\left(\frac{d}{2} M_M\right)^2 + \left(D R_C\right)^2} \tag{8}$$

in which μ_P is the standard position error of a photogrammetrically measured point, d is the diameter of the measuring mark of the instrument in space at the stereoscopic model scale, M_M is the scale of the stereoscopic model, D is the average distance between points, and R_C is the relative accuracy of the control points.

This equation has been checked, and proved to be accurate within about ± 10 percent for Wild Autograph Model A7, used by an experienced instrument operator. By using the equation, if the standard position error is given by a specification, as in the case of right-of-way surveying, the relative accuracy required in the control survey can be precomputed for the desired photography scale.

ERROR PROPAGATION IN PHOTOGRAMMETRY AND ELECTRONIC SURVEYING

The error propagation in photogrammetry is homogeneous or uniform, which means if a photogrammetric error is ±0.3 feet the error remains the same regardless of the distance between points.

It is a well-known fact the magnitude of an error in conventional surveying increases with the distance between points of measurement. Therefore the specifications usually given in relative accuracy, such as one in ten thousand, etc., are ideal to the nature of conventional surveying.

But the two errors, namely the error of photogrammetry and the error of conventional ground surveying, are completely different in nature and the final result is consequently unfavorable. The most important part of this situation is that the combination of these two kinds of errors can result in systematic errors, which cannot be avoided within a desired economy, because higher degrees of accuracy would unfavorably influence the surveying cost, unless an entirely new method is used. This new method is electronic surveying.

Electronic surveying is based on the fact the velocity of the electro-magnetic waves and of light rays is known. Thus the elapsed time during which the rays travel from one point to another and return is measured, and the distance computed from such measurements. Consequently, the error of an electronically measured distance is composed of the error of the velocity of the electro-magnetic waves or light rays and the error in time measurement. The error of the velocity is dependent upon the metrological circumstances, which can be corrected by the measurement of temperature, barometric pressure, and the humidity. After the correction, the residual error can be assumed to be constant. The error in the time measurement depends on the instrument in use and for a particular instrument it is constant. Consequently, the error in an electronically measured distance is constant regardless of its length. In other words, the error propagation in electronic surveying is homogeneous, such as in the photogrammetry. It is clear the combination of electronic surveying and photogrammetry is ideal.

ELECTRONIC SURVEYING TRAVERSE COMPUTATION WITH LEAST SQUARE ADJUSTMENT

Preliminary surveying executed by photogrammetry indicates a relative accuracy of 1:10,000 is suitable for ground surveying. Right-of-way application indicates the

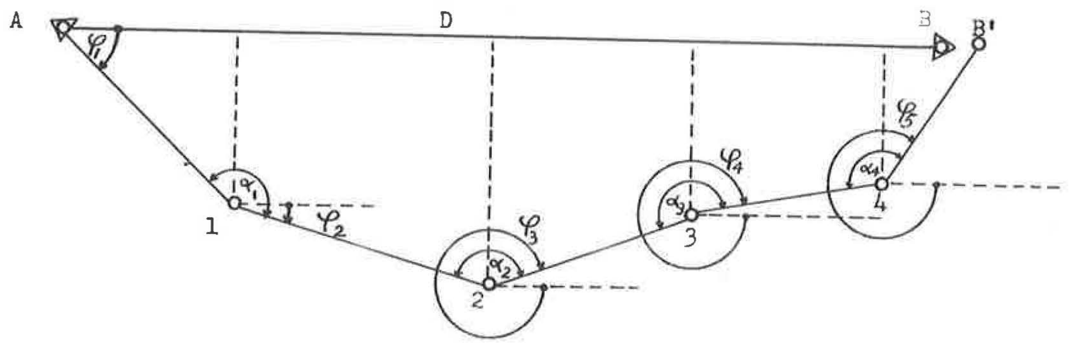

Figure 1.

relative accuracy of a ground survey should be about 1:25,000 to 1:50,000. To reduce the number of field measurements it is evident the different accuracy requirements must be combined into one, so each field measurement can be used for different purposes. This combination requires: first, the use of permanent station markers on which appropriate targets are centered before photography, and second, a universal measuring system which could be already available in the State Plane Coordinate System. The accuracy, or more specifically, the correct orientation of a measurement can be increased by computation. Least square adjustment is suitable for this purpose.

A suggested method for traverse computation by the least square method is subsequently given. It may be assumed that a traverse is measured between A and B triangulation points (Fig. 1), and the plane coordinates of these triangulation points were known previously. During the traverse surveying procedure accidental errors are introduced in the separate measurements as well as in the angle measurements. Due to these errors, the terminal B point is dislocated to B'. If the traverse has previously been computed or an azimuth has been measured, the φ_1 angle (which is the azimuth of AB line) can be obtained by computation or measurement. Knowing the angle φ_1, the other azimuth angles, such as $\varphi_2 \varphi_3$, etc., can be computed.

One condition equation can be written such that the sum of the projected traverse segments to the AB direction is equal to the D distance, computed from the plane coordinates of triangulation points A and B; mathematically,

$$d_1 \cos \varphi_1 + d_2 \cos \varphi_2 + d_3 \cos \varphi_3 + d_4 \cos \varphi_4 + d_5 \cos \varphi_5 = D \tag{9}$$

Actually this equality does not exist because of the introduced small accidental errors. A small correction must, therefore, be added to the traverse segments and the angles to fulfill the condition equation, as follows:

$$\begin{aligned}&(d_1 + V_1) \cos (\varphi_1 + \Delta\varphi) + (d_2 + V_2) \cos (\varphi_2 + \Delta\varphi + \Delta\alpha_1) + (d_3 + V_3) \cos \\ &(\varphi_3 + \Delta\varphi + \Delta\alpha_1 + \Delta\alpha_2) + (d_4 + V_4) \cos (\varphi_4 + \Delta\varphi + \Delta\alpha_1 + \Delta\alpha_2 + \Delta\alpha_3) + (d_5 + V_5) \\ &\cos (\varphi_5 + \Delta\varphi + \Delta\alpha_1 + \Delta\alpha_2 + \Delta\alpha^3 + \Delta\alpha_4) = D = \sqrt{(X_A - X_B)^2 + (Y_A - Y_B)^2}\end{aligned} \tag{10}$$

in which

$\Delta\varphi$ = correction to the initial azimuth,
$\Delta\alpha$ = corrections to the measured angles, and
V = corrections in the traverse segments.

Realizing that

$$\cos (\varphi + \Delta\varphi) = \cos \varphi \cos \Delta\varphi - \sin \varphi \sin \Delta\varphi \tag{11}$$

and because $\Delta\varphi$ is very small, thereby making $\cos \Delta\varphi = 1$ and $\sin \Delta\varphi = \Delta\varphi$:

$$\cos (\varphi + \Delta\varphi) = \cos \varphi - \Delta\varphi \sin \varphi \tag{12}$$

Substituting these into the condition equation the coefficients of $\Delta\varphi$ become zero.
To simplify the preceding equations, the following notation can be written:

$$\begin{aligned}
d_2 \sin \varphi_2 + d_3 \sin \varphi_3 + d_4 \sin \varphi_4 + d_5 \sin \varphi_5 &= N_1 \\
d_3 \sin \varphi_3 + d_4 \sin \varphi_4 + d_5 \sin \varphi_5 &= N_2 \\
d_4 \sin \varphi_4 + d_5 \sin \varphi_5 &= N_3 \\
d_5 \sin \varphi_5 &= N_4 \\
\cos \varphi_1 &= a_1 \\
\cos \varphi_2 &= a_2 \\
\cos \varphi_3 &= a_3 \\
\vdots \quad & \quad \vdots
\end{aligned}$$

Using these new symbols, the equation becomes;

$$\begin{aligned} a_1 V_1 + a_2 V_2 + a_3 V_3 + a_4 V_4 + a_5 V_5 + \Delta\alpha_1 \frac{N_1}{\rho} + \Delta\alpha_2 \\ \frac{N_2}{\rho} + \Delta\alpha_3 \frac{N_3}{\rho} + \Delta\alpha_4 \frac{N_4}{\rho} + W = 0 \end{aligned} \tag{13}$$

in which

$\rho = 206264.8$, and
$W = (d_1 \cos \varphi_1 + d_2 \cos \varphi_2 + \ldots + d_5 \cos \varphi_5) = D.$

In this problem, angles and distances are adjusted at the same time. Thus it must be assumed the measured angles and distances are weighted differently. Weights for measured distances are

$$P_{d_1} = \frac{C^2}{\mu^2_{d_1} + 1}, \quad P_{d_1} = \frac{C^2}{\mu^2_{d_2}}, \text{ etc.}$$

and the weights for measured angles are

$$P_{\alpha_1} = \frac{C^2}{\mu^2_{\alpha_1}}, \quad P_{\alpha_2} = \frac{C^2}{\mu^2_{\alpha_2}}, \text{ etc.}$$

in which μ^2's are the standard error of individual measurements, and C^2 is an arbitrarily chosen number depending on the computational convenience. Combining these equations, the normal equation is

$$\left(\frac{[aa]}{[P_d]} + \frac{\left[\frac{NN}{SS}\right]}{[P_\alpha]} \right) k + W = 0 \tag{14a}$$

From the normal equation

$$k = \frac{-W}{\frac{[aa]}{[P_d]} + \frac{\left[\frac{NN}{SS}\right]}{[P_\alpha]}} \tag{14b}$$

(In these equations the brackets represent the sum of the products.)

By knowing k, the corrections are computed in the following way:

$$V_1 = \frac{a_1}{P_{d_1}} k \qquad \Delta\alpha_1 = \frac{\frac{N_1}{\rho}}{P_{\alpha_1}} k$$

$$V_2 = \frac{a_2}{P_{d_2}} k \qquad \Delta\alpha_2 = \frac{\frac{N_2}{\rho}}{P_{\alpha_2}} k$$

$$\vdots \qquad\qquad\qquad \vdots$$

$$V_n = \frac{a_n}{P_{d_n}} k \qquad \alpha_n = \frac{\frac{N_n}{\rho}}{P_{\alpha_n}} k$$

The corrected angles and traverse distances are computed algebraically by adding the appropriate correction to the corresponding distances and angles. Because the corrected angles and distances are known, the traverse can now be computed with no discrepancy between points B and B′.

By employing such a computation procedure and the electronic surveying, a relative accuracy of 1:100,000 can be achieved easily which fulfills any requirement.

CONCLUSIONS AND RECOMMENDATIONS

The following conclusions result from the experiments and mathematical considerations.

1. Employment of electronic surveying in connection with the establishment of control for photogrammetric purposes is ideal.
2. Permanent use of station markers and targeting them before photography is taken are recommended for electronically measured traverses.
3. Use of State plane coordinate system is recommended.
4. A consequence of utilizing recommendations numbered 1, 2, and 3 will be, within five to ten years, a large amount of field control established throughout a State conforming to the accuracy of second-order triangulation. The increase in control points will occur as rapidly as precision surveying is done photogrammetrically without having any more than the minimum number of field surveyed measurements. Consequently, the use of this procedure will be highly economical.
5. Further investigations are required in: (a) aerial triangulation for precise surveying, (b) application of numerical photogrammetry for right-of-way land acquisition, and (c) specifications for ground surveying in which the new advanced methods are to be employed. Such research could be combined into one large research project and could result in a manual for modern highway surveying and an evaluation of methods and instruments.

ACKNOWLEDGMENT

The author expresses his gratitude to L. O. Herd, Chief Engineer of the Aerial Survey Section of the Ohio Department of Highways, for making it possible to perform this investigation.

REFERENCES

1. Ames, A., Jr., Ogle, K. N., and Gildon, G. H., "Corresponding Retinal Points, the Horopter and Size and Shape of Ocular Images." Optic. Soc. America (1932).
2. Bailey, D. A., "The Preliminary Location of Proposed Highway by Photogrammetric Surveys." Thesis, Purdue Univ. (1958).
3. Brandenberger, A. J., "Aerial Triangulation by Least Squares." Mapping and Charting Res. Lab., Ohio State Univ. (1956).
4. Brandenberger, A. J., "Fehlertheorie der Auseren Orientierung von Steilaufnahmen." Inst. Orell Fussly A. G., Zurich.
5. Fisher, F. P., "Fortgesetzte Studien über Binokularsehen (Tachermak). II. Über Assumetrien des Gesichtssinnes Speziell des Raumsinnes Beider Augen." Arch. Physiol. (1924).
6. Hazai, I., "Geodeziai Kezukonyy I, II, III." Kozgazadasagi es Jogi Kiado, Budapest (1956).
7. Ogle, K. N., "Binocular Vision." W. B. Saunders, Philadelphia & London (1950).
8. Punum, L. P., "Über die Einheifliche Vershmelzung Verschiedenartiger Netzhouteindrucke bein Sehen mit Zwei Augen." Arch. Anat. und Physiol. (1861).
9. Sebor, J., "Altalanos Geodezia. I. II." Mezogazdasagi Kiado, Budapest (1953).
10. Scudieri, P. F., "An Analysis of Photogrammetry Applied to Right-of-Way Surveys." Thesis, Purdue Univ. (1961).
11. Tarczy-Hornoch, A., Kiegyenlito Szamitasok, Kezirat Felso Oktatasi Jegyzet ellato Vallalat, Budapest.
12. Veres, S. A., "Aerial Traverse and Maximum Bridging Distance." ASCE Surveying and Mapping Div. Jour. (1963).
13. Veres, S. A., "The Effect of Fixation Disparity on Photogrammetric Processes." Photogrammetric Eng., 30(1).
14. Veres, S. A., "Handbook of Geodimeter Model IV." Ohio Dept. of Highways (1962).
15. Yzerman, H., "The S. D. I. A New Method for Stereoscopic Measurement and Plotting." Internat. Arch. of Photogrammetry, Vol. 12 (1956).

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.

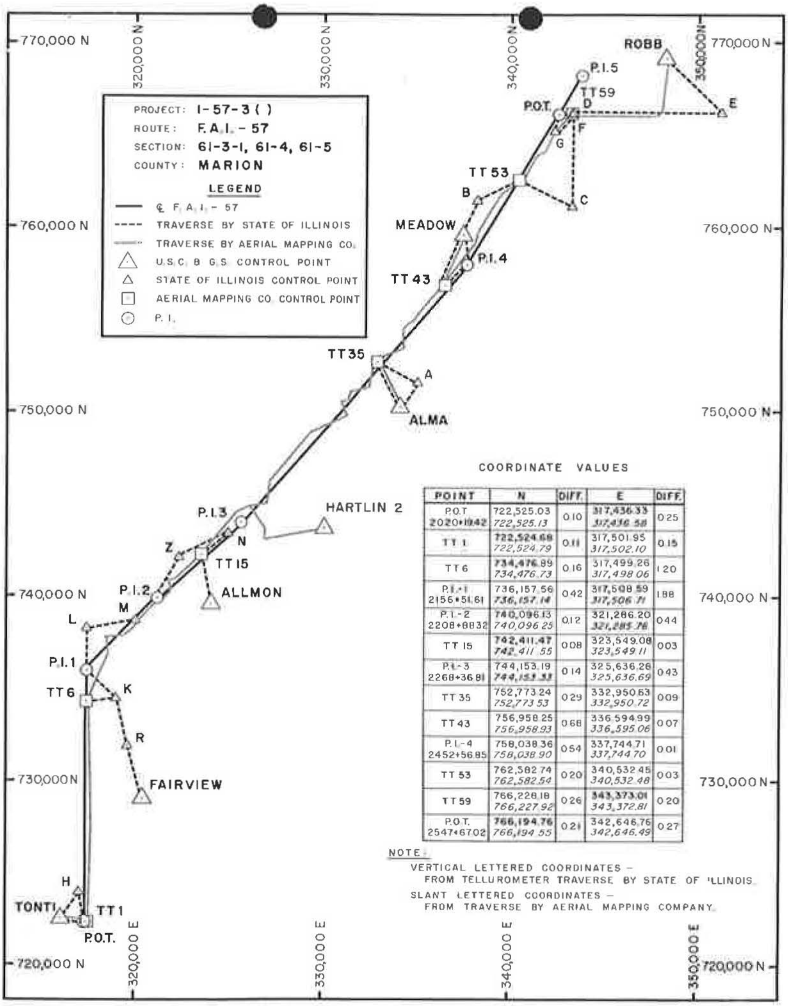
Since topographic maps compiled by photogrammetric methods were made available for use by highway engineers, they desired maps of a corridor or band of topography on which a detailed alignment could be designed, staked on the ground in such a way the original map would be satisfactory for completion 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

Paper sponsored by Committee on Photogrammetry and Aerial Surveys.

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+19.42	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

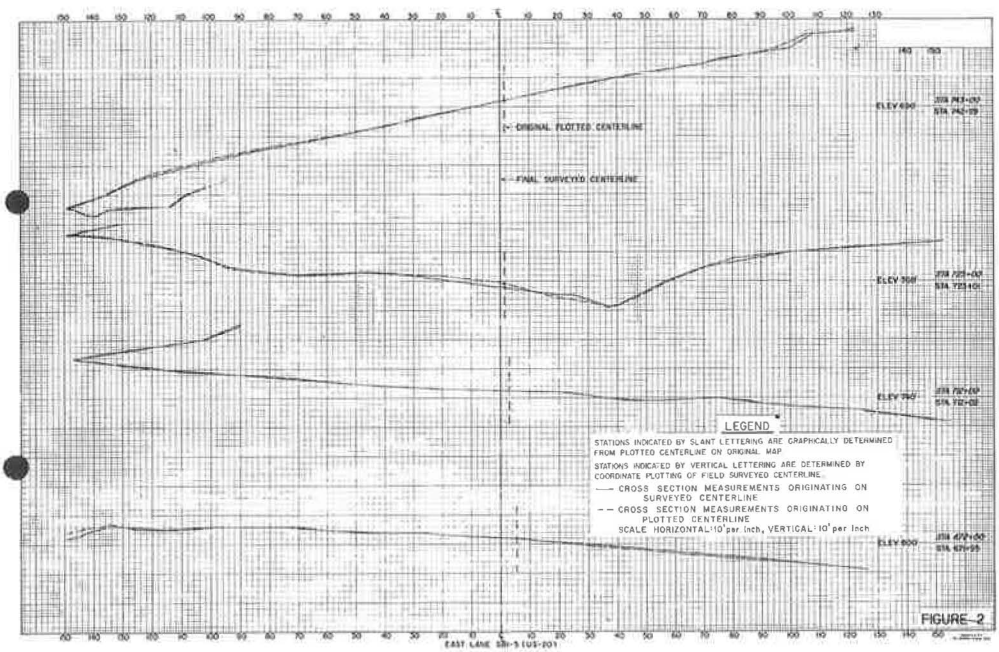


Figure 2.

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).

Idaho Highway Surveys Using the Tellurometer

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•THE State of Idaho has the mixed blessing of a wide variety of climatic and topographic conditions, ranging from semi-arid plains and mountains in the south to timbered mountains, fertile praries, and deep canyons of the central and northern portions. The highways range in elevation from 740 feet at Lewiston to 8, 701 feet at Galena Summit. Many of the highways follow rivers, whereas others cross plains or climb canyons and traverse mountain ridges or prairies.

Through the years, the survey work for highway projects has been slow and laborious in the rugged canyons and mountains. In past years, many traverses were loose-ended, some were closed in bearing by solar observations, and some were closed by costly circuit surveys. Arbitrary plane coordinate systems were used which started and ended on themselves, with no assurance constant errors did not exist in loose-ended surveys. As land values increased and surveying errors became more objectionable, it was evident new methods, tools, and procedures were required.

In 1958 two far-reaching decisions were made in the Idaho Department of Highways, both of which were made in an attempt to modernize preliminary engineering methods and procedures. First, it was decided all future survey projects, wherever practical, would be tied to markers of basic control surveys in the Idaho State Plane Coordinate System. Second, the location and/or design of future highway projects, wherever economically feasible, would be made by use of appropriate photogrammetrically compiled topographic maps.

The first major step in implementing this program was an immediate agreement with the U. S. Coast and Geodetic Survey to establish second-order horizontal and vertical control along all proposed Interstate highway routes with survey stations monumented at an interval of one to three miles. The establishment of this control on the Interstate System reduced, but did not eliminate, problems encountered in placing other highway survey projects on the State plane coordinate system.

After nearly a year of using laborious triangulation methods involving short bases and measuring traverse distances by taping and angles by angle-measuring instruments, it became apparent use of more modern surveying equipment and methods must be resorted to for fulfilling satisfactorily the desire for placing highway survey projects on the State plane coordinate system. It was decided some type of electronic distance-measuring instrument would be a natural teammate for use with the 1-sec reading theodolites already in use by the department. After considerable research, the following features seemed desirable for the instrument which should be selected:

1. Portability—the inaccessibility of many survey station markers would result in the instrument having to be backpacked.
2. Long range—many distances would have to be measured for tying and closing highway surveys to markers of isolated control points.
3. Ability to operate in nearly all weather conditions—the northern portion of the State is especially prone to extended periods of fog, rain, and snow.
4. Production—the general plan of future surveying operations was accomplishment of basic control surveying by traversing which would require measurement of numerous traverse segments per day.

The Tellurometer seemed best fitted for this program, except for its inability to measure short distances. This handicap could be overcome by taping, by short base

Paper sponsored by Committee on Photogrammetry and Aerial Surveys.

triangulation, and by employing the subtense bar, as appropriate and convenient.

In the fall of 1959, the department purchased one complete Tellurometer set, Model MRA 1/CW, with headsets and transistorized power packs. The headsets have since proved to be a wise investment, being instrumental in boosting the average daily number of measurements to 14 traverse segments.

The Tellurometer units were checked out locally and, after familiarization, were taken to an accessible U. S. C. and G. S. first-order base for making measurement accuracy checks and for possible calibration. The base was accurately extended by 50-ft increments and the total distance measured along this base and extensions. After reduction there appeared to be a constant error in the distance measurements of approximately 0.20 foot, always positive. Further checks were made between a series of U. S. C. and G. S. triangulation stations. These tests confirmed the former findings. Results of the latter tests confirmed reports which had been previously received from other highway departments. Tellurometers seemed to have a constant error of approximately the same magnitude and same sign. It was decided a calibration factor of -0.20 foot should be applied to all measurements. Consequently, it appeared possible to measure short distances and still retain an order of accuracy sufficient for most highway purposes, one part in ten thousand.

On the first survey project, subsequent to establishing the probability of the error constant, survey closures were computed using and not using the constant. Results attained upheld the validity of using the constant of -0.20 foot, which should be subtracted from the slope distance of each measured distance. A recent Tellurometer Field Reduction Computer Program received from the Arizona Highway Department incorporates the same constant in its calculations. From the first, it was apparent proper use of the Tellurometer would result in large savings in time and money.

Idaho, perhaps due to its rugged topography and sparse population, still has vast areas containing little, if any, horizontal and vertical control. An example of this is Owyhee County in the southwest corner of the State. In early 1960, a highway survey project was initiated for realignment and/or relocation of Idaho 51, which is the only major highway through this area. It was also desirable to place this entire survey project on the Idaho State Plane Coordinate System. Without the Tellurometer or some other electronic measuring device, this would have required a 40-mi triangulation arc to make adequate survey ties between the existing Snake River triangulation network to the Idaho-Nevada State line triangulation network. Due to the nature of the intervening terrain, a minimum of six to eight quadrilaterals would have been required to complete this control surveying project. A conservative estimate of the time required to accomplish this type of survey would be two to three months. Instead, a 40-mi Tellurometer and Wild T-2 traverse was measured from U.S. C and G.S. Triangulation Station SLICK in the State Line Net to Triangulation Station WICKAHONEY in the Snake River Net. This traverse consisted of making measurements between 19 main station markers and 13 auxiliary station markers which were tied to points in the proposed "L" line of the highway location. The Tellurometer work, 32 traverse segment measurements, was completed in 3 days. The azimuths were determined at night by measuring azimuth angles at 12 positions with a Wild T-2 theodolite and using a 5-sec rejection limit. All field surveying work on this project, including reconnaissance, was completed in slightly over two weeks. The azimuth closure, before adjustment was 02 seconds per station and the apparent position closure was 1:80,455 on U. S. C and G. S. triangulation stations. In addition to the tremendous saving in time and money, the Tellurometer-measured traverse had one other decided advantage over a triangulation network survey. It was possible to place station markers of the traverse on or very nearly on the proposed "L" line location for future control in photogrammetric work, whereas the positions of the triangulation stations would have been dictated by terrain, as it affected intervisibility and shape of the quadrilaterals.

On several other projects requiring extension of control from station markers in the basic network established by the U. S. C and G. S., various expedient forms of triangulation were combined with Tellurometer measurement of traverses. These triangulation forms, such as trilateration and two-point intersection, also employed the Tellurometer and Wild T-2. Trilateration by use of the Tellurometer has proved

to be very successful where weather conditions make angular measurements difficult or nearly impossible. As a check on the Tellurometer work, however, it is advisable to measure one angle. Two-point intersections, where azimuths and distance from two known stations are determined, have proved to be the most desirable method for position surveying of isolated control points situated at strategic locations for future expansion of control for highway surveying, including topographic mapping by photogrammetric methods for location, design, and preparation of detailed construction plans.

Although extension of basic control and establishment of basic control for highway survey projects has been a fundamental use of the Tellurometer, by far its greatest advantage has been in surveying supplemental control for mapping of highway survey and design projects by photogrammetric methods.

One of the early examples of this type of project was the survey made of the Lookout Pass highway route during the summer of 1960 for determining the best possible location for I-90 between Mullan, Idaho and Saltese, Montana. This project, lying in the heart of the beautiful Coeur d'Alene Mountains in the famous Coeur d'Alene mining district, was to be a joint project with the Montana State Highway Commission. The survey study was to include compilation and use of a 200 feet to 1 inch reconnaissance type topographic map containing a basic contour interval of 10 feet. Montana was to fly the route and take the photography and to compile the maps because, at the time, Idaho did not own any precision photogrammetric instruments. Both States, however, were to survey all necessary ground control in their respective areas of jurisdiction.

Idaho's segment of control for the route was approximately 8 miles long and 3 miles wide at its widest point. A deep canyon traversed its entire length from east to west. While being a tremendous tourist attraction, the area did not readily lend itself to any of the conventional methods of control surveying on the ground. It was therefore decided all control survey planning, both horizontal and vertical, should require fullest possible utilization of the Tellurometer and the Wild T-2 theodolite.

A careful and thorough study of the aerial photographs resulted in a plan for supplemental control containing nearly 60 image points for horizontal and 100 image points of vertical control. The points where images were selected for vertical control ranged in elevation from 3,000 feet above sea level in the bottom to 6,000 feet above sea level on either side of the canyon, with all possible passes being at an elevation of approximately 5,000 feet.

The extremely steep and rugged north side of the canyon, having been a portion of the area burned by the Great Kellogg fire of 1910, contained considerable low brush but very little timber. It was also nearly devoid of any roads or trails negotiable by anything other than a four-wheel drive narrow-track jeep. On the other hand, the south side of the canyon was heavily timbered and contained a fairly complete network of roads and trails, including present US 10, the Northern Pacific Railroad, and numerous Forest Service and logging roads. The south side of the canyon also contained all of the available vertical control and the bulk of horizontal control established by the U. S. C. and G. S. under the 1958 agreement.

The first step in surveying supplemental control for this project was to extend the horizontal control to the north side of the canyon. This was done almost entirely by two-point intersections using the Tellurometer to measure the distance and the T-2 theodolite to measure the azimuth angles. After a basic network of control comprised of strategically located points on the north side of the canyon had been surveyed, the supplemental horizontal control was measured to the objects of which the photographic image points were selected by making two-point intersections or by measuring short traverses with the Tellurometer. On this project all attempted measurements with the Tellurometer were accomplished successfully.

Simultaneously with this extension of horizontal control, two level parties were busy measuring the elevation of bench marks along the bottom and along roads and trails on the south side of the canyon. An elevation of additional bench marks along the top of the ridge on the south side of the canyon was also measured. These latter bench marks were later used to extend the measurement of elevations across the canyon by means of Tellurometer-measured distances and vertical angles measured by use of the Wild T-2 theodolite.

By this method of vertical control extension, the Tellurometer was used to measure distances from at least two points of known elevation on the south side of the canyon to the object or point for which a photographic image had been selected to serve as a vertical control point on the north side. Reciprocal vertical angles were then measured with a T-2 theodolite. If possible, these reciprocal measurements were made simultaneously. On some occasions, however, several hours duration occurred between making the separate vertical angle measurements. The ground points for which images were selected to serve as control points ranged from 500 to 2,000 feet higher in elevation than the nearest bench mark in the canyon bottom and were from one to three miles away (horizontally) from the points of known elevation on the south side. In all instances, the two or three independent elevations thus established for each selected image point agreed within one foot, thereby making them entirely acceptable as pass point elevations for leveling and orienting to scale the stereoscopic models used to photogrammetrically compile the maps. If conventional methods of differential leveling had been used to measure the elevation of these supplemental control points, some of them would have required 100 to 200 setups of the level for just one direction on the circuit. A conservative estimate would be an average of from four to five hours of survey party time were saved by use of the Tellurometer in measuring the elevation of each of such vertical control points.

It is also doubtful if any more accurate results could have been consistently attained by differential leveling. In all, the elevation of 37 vertical control points was measured in this manner, resulting in a possible net time savings of approximately 160 crew hours, or 20 crew days.

In surveying the supplemental horizontal and vertical control for this highway mapping project, approximately 80 traverse segments were measured by use of the Tellurometer without rejecting a single measurement. Ten men with two sets of level surveying equipment, two Wild T-2 theodolites, and one Tellurometer set completed surveying control for this highway mapping project in six weeks. This time included reconnaissance, selection and circle identification of selected image points, all field surveying work, and completion of all field computations.

Upon request from the Surveys and Plans Engineer, a close estimate was made of the apparent savings in time and money made possible with the aid of the electronic distance-measuring equipment. A conservative estimate indicated an approximate savings of 25 survey crew days at a minimum cost of $200 per day, comprising a total savings of nearly $5,000. This savings on one project would make reimbursement for one-half the initial cost of the Tellurometer equipment.

As previously mentioned, the reciprocal vertical angle measurements were not always performed simultaneously on this project, yet no apparent loss of accuracy was indicated. This can probably be attributed to the exceptionally stable atmospheric conditions encountered while the measurements were made and to the dense tree coverage over most of the area. Consequently, the vertical refraction was reduced to a minimum. Such conditions did not occur on some subsequent highway survey projects, as, for example, the Mountain Home to Bliss project surveyed in the fall of 1960.

The latter highway survey project was to consist of reconnaissance type mapping at a scale of 400 feet to 1 inch with a basic contour interval of 10 feet. Its locale lay almost entirely within the arid to semi-arid region bordering the north bank of the Snake River in south central Idaho. The topography of this portion of the State consists chiefly of large sage brush covered prairies which are cut by deeply eroded canyons. The canyons are generally topped by nearly insurmountable rim rock cliffs. While the humidity over large portions of this area remains exceptionally stable, between 15 and 25 percent, the temperature range is extreme. The temperature may vary from a high of 120 F at 2 PM, to a low of 60 F at 2 AM. Even the temperature from sunny areas to shady areas, caused by intermittent cloud coverage, may change as much as 20 to 30 degrees. Perhaps this extreme range in temperature causes sufficiently large variations in the vertical refraction as to make it necessary to accomplish all reciprocal vertical angle measurements simultaneously. This limitation, however, did not reduce the effectiveness of the Tellurometer and Wild T-2 theodolite combinations in surveying accurate supplemental vertical control over a very wide

area at minimum cost. This method was used to measure the elevation of 81 vertical control points with sufficient accuracy. All vertical angles were measured by simultaneous reciprocal observations.

Also, on this project another variation of triangulation was developed for use of electronic distance-measuring equipment. Three of the prime, most strategically situated U.S. C. and G.S. Triangulation Stations, were either impossible or extremely difficult to occupy. One station was an airway beacon, another was located on top of a mesa completely surrounded by a rim rock cliff 60 feet high, and the other entailed a long rough jeep ride followed by a difficult hike on foot.

To fully utilize these strategically positioned stations without having to occupy them, with either the T-2 theodolite or the Tellurometer, two men, traveling very light, were sent to each station for the purpose of building a signal on it. This work was completed in one day with a minimum of effort. For the next several weeks, these stations in the basic network of control were utilized for surveying position and computing the State plane coordinates of approximately 40 image points without having to return to any of the three stations. In computing plane coordinates for each image selected to serve as a supplemental horizontal control point, one of the nearly inaccessible stations was used in conjunction with an easily accessible basic control point. The distance between the ground point imaged on the photographs and the accessible control point was measured with the Tellurometer and the angles at these two points were also measured to the signal on the inaccessible point. This procedure resulted in two known angles and two known distances (one computed and one measured) for each triangle. These data were sufficient for solving the triangle as well as for checking the results of the survey.

Since then this method has been used on other highway survey projects in proximity to which water towers, smoke stacks, radio towers, and similar objects were markers of basic control points established by the U.S. C. and G.S.

During the three and one-half years in which the Tellurometer has been in use by the Idaho Department of Highways, it has also proved to be extremely valuable for making survey ties between highway survey projects and land subdivisions, land survey section corners, and quarter corners, etc., for right-of-way purposes. In making ties to basic control for highway route mapping for design and for other highway survey purposes, these ties to land survey corners more often than not occur in some of the most rugged terrain where making such ties by conventional surveying methods would be extremely laborious and time consuming.

Within the past two years surveys made by the department with the Tellurometer have become more and more closely allied to the department's electronic computer program and have reached the point where all Tellurometer-made measurements are reduced to slope distances, all traverses are computed and adjusted, and all single triangles are solved by use of applicable computer programs. These three programs alone have reduced the office computing time by at least two-thirds of what former computation procedures required.

Perhaps Tellurometer users in the Idaho Department of Highways have been extremely fortunate when their experiences are compared with experiences reported by some other users of electronic distance-measuring devices, including Tellurometers. In measuring over 1,300 traverse distances during the past three and one-half years, not more than a dozen distances have had to be discarded because accurate results could not be attained. In some of these cases, it was felt local microwave units operating in the area could have caused the difficulties. In addition, 90 percent of all maintenance has been performed at the Department's own radio and radar repair facilities, thus reducing maintenance cost to a nearly negligible amount.

Principal users of the Tellurometer within the Idaho Department of Highways feel the versatility of any electronic distance-measuring equipment is limited only by the imagination and flexibility of its users.

Application of Precise Photogrammetric Methods To Right-of-Way Relinquishment Surveys

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• A METHOD which was developed in the San Francisco District Office of the California Division of Highways for obtaining right-of-way survey data using aerial photographs and simple scaling procedures has been described by Hovde (1). The measurement data obtained were sufficiently accurate to use for metes and bounds descriptions of right-of-way relinquished to local authority, such as frontage roads paralleling a completed freeway facility. This procedure offered savings over field survey methods and was conventional with this office for many years. Precise photogrammetric methods have now replaced the scaling procedures because of increased accuracies and significant reduction in man-hours of effort, which reflect further cost savings.

To provide a more complete introduction to the application of precise photogrammetric methods, a brief review of the scaling procedure described by Hovde is subsequently given. The "as built" location of a fence or other objects separating a freeway from a frontage road defines a line of reference for relinquishing right-of-way outside the freeway. The survey location of the fence is therefore important to make a realistic description of the property involved. Aerial photographs, taken at the proper scale, of a newly constructed freeway clearly show fence lines. It was from photographs of this type that the survey data were obtained.

Preliminary to taking photographs, existing construction survey points along the frontage roads were recovered and premarked with targets. Aerial photographs were then taken at a scale of 120 feet per inch and photographically enlarged for measurement purposes to the scale of 20 feet per inch. A line drawn between any two successive premarked survey points imaged on the photographic enlargement was used as the baseline for measurement of fence positions. This length of line, however, had to be compared with its known survey length to determine the correct scaling factor for adjusting all photographic measurements made within its terminal limits.

Offset measurements were made from the baseline along lines at right angle to it to selected fence posts to mathematically locate the fence. All measurements were adjusted by the scaling factor to arrive at X and Y survey coordinate values for the horizontal position of each selected fence post. From the coordinates, inversed distances and bearings were calculated to prepare a metes and bounds description, and a plot was made to document the relinquishment.

The precise photogrammetric method centers around use of the Zeiss Stereoplanigraph, model C8, an optical train photogrammetric instrument which permits determination of the X, Y, Z ground coordinates of points viewed in a stereoscopic model. The instrument is also capable of making an accurate plot of any point at the time its numerical coordinates are determined. This method requires less control, but which is of better quality, than control required for scaling from aerial photographs. The photogrammetric operation is done by the Photogrammetry Section in the Sacramento office.

RESEARCH PROJECT

As an initial project to investigate operational approaches to use of precise photogrammetric methods, and to evaluate results, a test section was chosen along a portion of State Sign Route 17 north of Santa Cruz, California. This section, about 0.9 mile

Paper sponsored by Committee on Photogrammetry and Aerial Surveys.

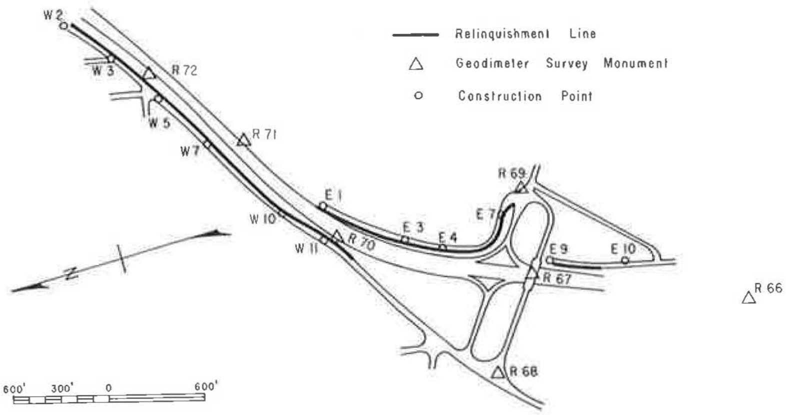


Figure 1. Layout of research project.

in length, had been previously surveyed and sufficient control existed, so further control surveying was not necessary. The layout of the project is shown in Figure 1.

Four main steps involved in conducting the investigation are described in the following sequence:

1. Selection and targeting of control points. The positions of existing survey monuments had been surveyed by Geodimeter measurement of a traverse throughout the test section within the freeway right-of-way. Supplementing monument-marked control points were construction points along the frontage roads. The survey had been made on a local coordinate system, and plane coordinate positions had been determined for each monumented control point and each construction survey point. The distribution of monumented and construction points is shown in Figure 1.

Because the monumented points position surveyed by Geodimeter were to be used to fix horizontal scale of the stereoscopic models formed by the photographs in the Zeiss C8 Stereoplanigraph, target design was considered very important to assure their accurate recovery. Figure 2 shows the target pattern and dimensions for the premarking monumented control points before photography was taken.

Each construction point was premarked with a white "arrow" target painted on the black pavement of the frontage roads, with the point of the arrow depicting the survey mark.

2. Aerial photography. A 6-in. focal length Zeiss RMK 15/23 aerial camera was used to photograph the project strip after placement of a target on each measurement point was completed. Aircraft flight-height was about 1,500 feet above the ground, which resulted in a photography scale of approximately 250 feet per inch. Cronar base aerial film was used.

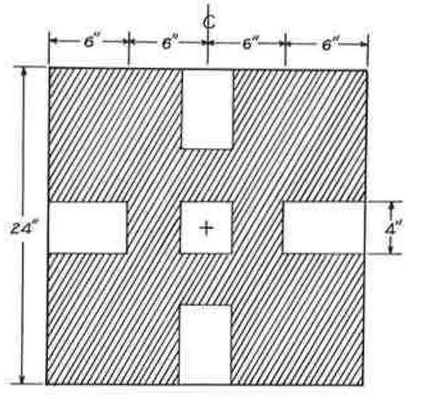


Figure 2. Target design for research project.

Photographic enlargements of alternate photographs were sent to the District Office for identification of all targeted points, and for instructions regarding relinquishment line data to be obtained.

3. Use of the Zeiss C8 Stereoplanigraph. Although a description of the Zeiss C8 Stereoplanigraph (Fig. 3) is not within the scope of this paper, a few pertinent remarks may clarify the matter of scales and measurement accuracy. There are three scales involved in numerical determinations, namely the photography scale, the measurement (stereoscopic model)

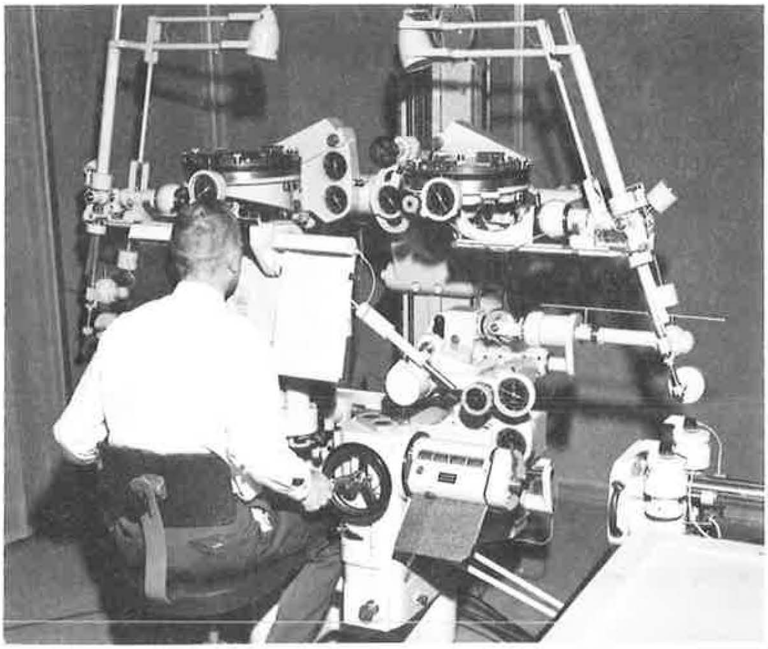


Figure 3. Zeiss Stereoplanigraph, Model C8.

scale, and the viewing scale. A fourth scale, the plotting scale, is considered only if a graphic plot is desired in conjunction with determination of numerical data regarding specific points.

At a photographic scale of 250 feet per inch the measurement scale is 125 feet per inch. The viewing scale is independent of the measurement scale and much larger. Because the measurement counter records x, y, and z measurement coordinates in increments of 0.01 mm, the equivalent X, Y, and Z coordinates, when the instrument measurement scale is 125 feet per inch, would represent 0.05 feet on the ground. This is the smallest possible measurement and, of course, is subject to random errors associated with any measurement technique.

The plotting scale can be varied mechanically by selecting gears of different ratios. It is possible to enlarge the plotting scale as much as 5 times the measurement scale (or 10 times the photography scale).

On the subject test section, six stereoscopic models covered the freeway strip. The aerial triangulation was accomplished in conformance with standard practice. The first stereoscopic model was leveled and scaled using out the best available control. When this absolute orientation was completed the photogrammetrically measured coordinates of all selected fence posts and targeted construction survey points were measured and recorded. Their positions were also plotted on mylar-base material at a scale 50 feet per inch, the same scale as the freeway construction plans.

On completion of measurements using the initial stereoscopic model, the second model was adjoined to it. The coordinates were measured photogrammetrically for all selected fence posts, construction survey points, and Geodimeter-measured control survey monument markers imaged on the stereoscopic model; their plotted positions on the map were determined in the same way as for the first stereoscopic model. Likewise, the remaining four stereoscopic models were also measured.

All photogrammetrically measured x, y, and z coordinate data were adjusted to fit only four of the seven monumented point positions which had been determined by Geodimeter surveying. The remaining three monuments could not be seen definitely in the stereoscopic models, despite precautions taken in placing the targets. It is noteworthy the the stereoscopic models were oriented to control and meas-

TABLE 1

DIFFERENCE IN PLANE COORDINATES[a]

Post	Difference (ft) X	Difference (ft) Y
8a	0.4	0.3
9a	0.1	0.3
10a	0.4	0.1
11a	0.2	0.2
12a	0.0	0.3
13a	0.1	0.4
14a	0.2	0.1
15a	0.3	0.2
16a	0.0	0.1

[a]Posts measured by use of two stereoscopic models, differences in photogrammetrically measured coordinates.

TABLE 2

DIFFERENCE IN PLANE COORDINATE[a]

Post	Difference (ft) X	Difference (ft) Y
E- 3	-0.2	0.5
E- 4	-0.3	0.1
E- 5	-0.3	0.4
E- 7	0.1	0.1
E- 9	-0.4	0.4
E-10	-0.1	0.5
W- 5	-0.1	0.2
W- 7	-0.3	0.1
W-10	-0.3	0.1
W-11	-0.3	-0.3

[a]Targeted construction survey points, differences between field and photogrammettically measured coordinates.

ured only once, and no further setups were made subsequent to the aerial triangulation and adjustment of the resultant data.

For this type of work, in which horizontal position only is desired, the Z coordinate measurement is not significant. Vertical control for orientation of the stereoscopic models was obtained from the USGS topographic maps published on a quadrangle basis of the area by assigning interpolated elevations from the contours of the maps to identifiable features on the photographs. This procedure provided elevations sufficiently accurate for the intended purpose.

4. Testing and evaluating results. As previously noted only four of the seven targeted basic control monument markers, which were position surveyed by use of the Geodimeter, were definitely visible in the stereoscopic models. Consequently the adjustment was held fixed at these four points: R66, R69, R70, R72. The photogrammetrically measured positions of 36 fence posts and 10 targeted construction survey points were accordingly determined from this adjustment in terms of X and Y coordinates on the local plane coordinate system.

Nine of the fence posts were visible in two adjoining stereoscopic models, and therefore two sets of plane coordinates were determined for each of the nine posts. Table 1 gives differences in each of the two sets of plane coordinates.

Each of the 10 construction survey points also had two sets of coordinates—the field measured and the photogrammetrically measured. Table 2 gives the differences in each of the two sets of plane coordinates.

The photogrammetrically measured coordinates were tested in the field by two different methods. Both methods consisted in measuring with a tape between a target point and some point on the fence line.

The first test involved measurements on a line perpendicular to the fence to a targeted point. The field measurement of this distance was done directly with the tape, whereas the photogrammetrically determined distance had to be inversed between the photogrammetrically measured plane coordinates of the targeted survey point and the coordinates of the normal point on the fence line. The latter had to be found by computation between photogrammetrically measured plane coordinates of fence posts either side of it. The results are given in Table 3.

The second test involved measurements along a line directly between a fence post and a targeted survey point. Here again, the photogrammetrically determined distance

TABLE 3

DIFFERENCE IN DISTANCE[a]

Targeted Point	Field Measured (ft)	Photo-grammetrically Measured (ft)	Incremental Diff. (ft)
W 2	18.6	18.5	-0.1
W 3	19.5	19.6	+0.1
W 5	18.7	18.8	+0.1
W 7	18.4	18.5	+0.1
W10	18.9	19.0	+0.1
W11	15.0	14.8	-0.2
R70	16.5	17.1	+0.6
R72	14.5	14.8	+0.3
E 1	23.5	23.7	+0.2
E 3	19.0	19.1	+0.2
E 4	19.1	18.5	-0.6
E 7	22.3	22.2	-0.1
R69	106.3	106.6	+0.3
E 9	19.3	19.8	+0.5
E10	25.3	25.4	+0.1
Corner	48.6	49.0	+0.4
		Average difference = +0.12	

[a]Right angle offset distance, differences between field measurements and photogrammetric measurements.

TABLE 4

DIFFERENCE IN DISTANCE[a]

Targeted Point	Fence Post	Field Measured (ft)	Photo-grammetrically Measured (ft)	Incremental Diff. (ft)
W 2	26b	18.8	18.5	-0.3
W 2	25b	20.2	20.0	-0.2
W 3	23b	20.1	20.2	+0.1
R72	22b	14.5	14.8	+0.3
W 5	21b	19.0	19.1	+0.1
E 1	1a	27.7	28.8	+0.4
E 5	8a	19.1	19.6	+0.5
E 7	14a	22.3	22.2	-0.1
E 9	17a	48.1	48.3	+0.2
E 9	18a	19.4	19.8	+0.4
E10	Corner	23.9	24.3	+0.4
			Average difference = +0.16	

[a]Between field measurements and photogrammetric measurements.

had to be inversed between the photogrammetrically measured coordinates of the fence post and the targeted survey point for comparison with the field measured distance. The results are given in Table 4.

Sources of error should be identified to evaluate properly these results. The arrow-type design of the target for construction survey points presented an indefinite shape in the stereoscopic model. The well-known halation phenomenon override of photographic images of white objects onto images of dark areas tended to make the targets appear too large. This in turn introduced some error in recovery of the survey point. On considering other sources of error, magnitude of such an error is probably not significant.

The most important source of error was undoubtedly caused by the fence post, because it is an "object" and not a "point." Hence, the "pointing" in the stereoscopic model cannot be exact, and the recorded measurement may not apply to the center of the post. Furthermore, the "lean" of the post, especially if brush obscures its base, contributes to position error. Another factor is the sun angle and resulting shadows. Posts of a chain link fence are particularly difficult to position in the stereoscopic model if their shadows fall in line with the fence. The differences given in Table 1 are undoubtedly attributable mainly to these sources of error, although they are of minor importance for this type of survey. (Subsequent projects photographed immediately on completion of construction have demonstrated a post newly set in concrete can be seen very clearly on the stereoscopic model, thus eliminating some of these sources of error.)

It has been reported test measurements made in the field may not be as reliable as desired. This comment is based primarily on the fact it was difficult to accurately locate the center of posts and normal points on fence lines. Although the results recorded in Tables 3 and 4 are considered entirely adequate for this type of survey, some measurements containing the larger differences should probably be rechecked in the field.

In any event, the fence line is not the line of relinquishment but only a reference to it. The question therefore resolves into whether or not photogrammetrically measuring the position of a fence is (a) of sufficient accuracy to serve as reference for mathematically determining a metes and bounds description of relinquishment property, and (b) cost savings attained thereby justify use of such measurement methods.

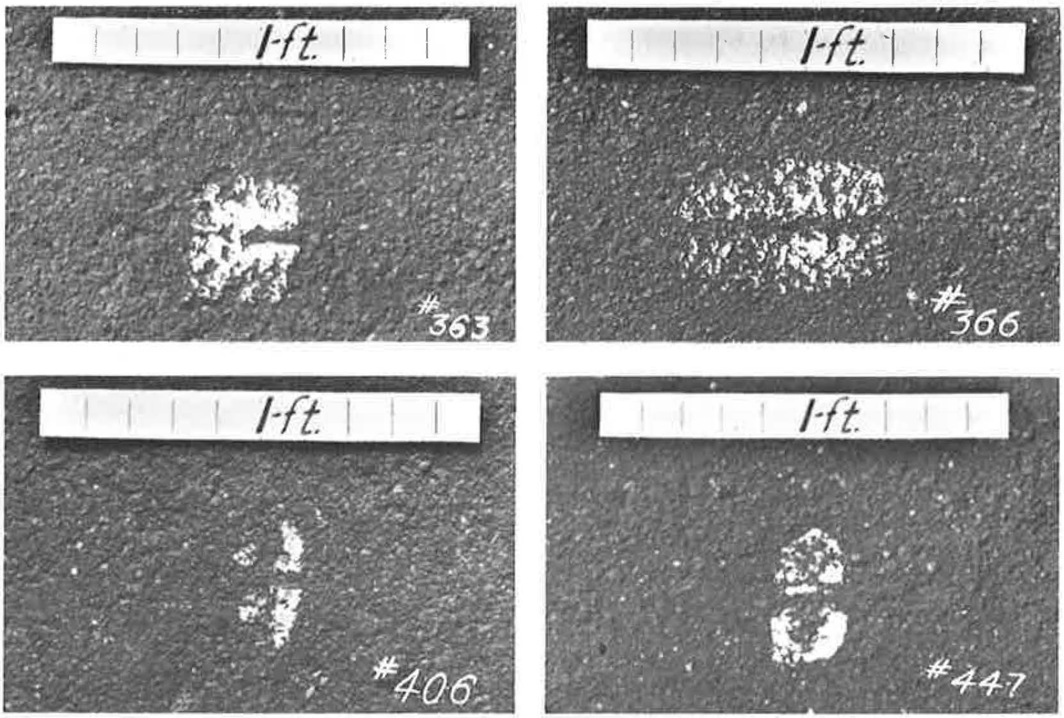
With respect to (a), accuracy of the precise photogrammetric method of measuring is unquestionably superior to scaling procedures using an engineer's scale on semi-controlled aerial photographs, and also probably superior to measuring by routine field surveys. By holding only to Geodimeter-measured control for aerial triangulation using the precision photogrammetric instrument, fence positioning is completely inde-

pendent of ties to construction survey points. The differences given in Table 2 are believed mainly caused by inconsistency in field values rather than in photogrammetric values.

With respect to (b), costs savings were difficult to assess on the research project. Costs, however, were analyzed on three subsequent projects and definite savings were realized, ranging from 50 percent for a rural highway project to 70 percent for each of two urban highway projects.

FURTHER INVESTIGATIONS

A particularly interesting operational project was recently completed which offered an excellent opportunity to investigate further the accuracies of photogrammetric methods of measuring by use of the Zeiss Stereoplanigraph, Model C8. This project was located on the Pacific Coast Highway (State Sign Route 1) on the east boundary of the newly established Point Reyes National Park. Survey records were not available for this road, and it became necessary to provide "as-built" location and right-of-way information in connection with establishing the Park boundary.



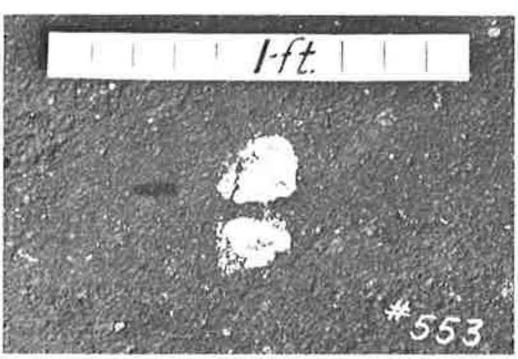


Figure 4. Example of centerline "cat-track" marks taken with a 35-mm camera from a distance of 4 feet. Note the 1-ft scale divided into $^1/_{10}$ths for an estimate of size.

With respect to photogrammetric procedures, this project was handled in the same way described for relinquishment surveys. A control traverse was measured with the Geodimeter paralleling the highway for the length of the survey project, and ties were made to station markers in the California State plane coordinate system. The monument markers of points in the traverse were located roughly 1,000 feet apart, and were targeted before photography using targets similar in design to the target illustrated in Figure 2. No other control points were used, except for a few points position measured with the Geodimeter which were situated at broad intervals transverse to the main traverse to provide stereoscopic model scaling bases for accomplishing the aerial triangulation.

Aerial photography on cronar base film was taken with a Zeiss RMK $^{15}/_{23}$ aerial camera from a flight-height of 1,500 feet. Resultant scale of the photography was 250 feet per inch.

When the stereoscopic models were viewed in the Stereoplanigraph, it was noticed centerline "cat-track" markings were actually visible on new sections of asphalt pavement. These marks served as suitable targets for photogrammetrically measuring coordinates where they were not obliterated by centerline strips. On older sections of pavement where only centerline stripes existed, measurements were made on the ends of stripes. Spacing of these coordinate measured points was approximately 50 feet, depending on the centerline markings.

Because of the finite size of the "cat-track" marks a field check was made of their photogrammetrically measured positions. It seemed expedient simply to measure between them for a comparison of photogrammetrically measured distances with field-measured distances. Figure 4 shows how these marks actually appeared on the pavement.

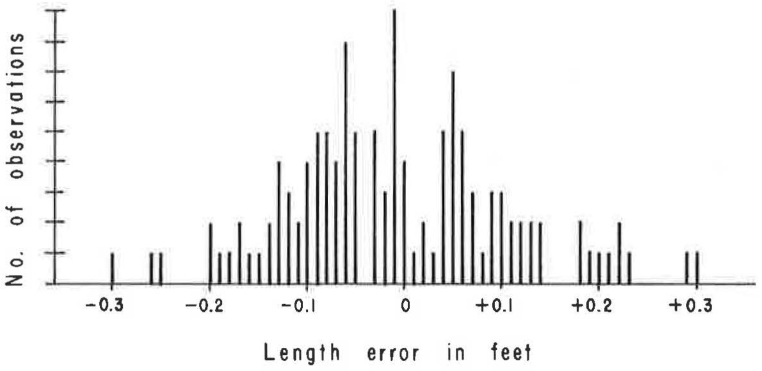
Because the marks were not centered on a survey point, an estimate was made as to the probable point of measurement by the operator of the Stereoplanigraph according to density or concentration of paint. This probable point was indicated with lumber crayon for reference in making measurements with a tape. A total of 123 measurements were then made between the crayon points for checking inversed lengths determined from photogrammetrically measured coordinates of the "cat-track" marks. Resultant errors in the photogrammetrically measured distances were, as follows: consecutive distances (123 measurements) (Fig. 5a); arithmetic mean, -0.009 feet (or -0.01 feet); and root mean square error, ± 0.11 feet.

Because the distances were measured consecutively, apparently a natural balancing effect was inherent in these results. Therefore, errors in alternate lengths were analyzed with the following results: alternate lengths, group A (62 measurements) (Fig. 5b); arithmetic mean, +0.007 feet (or +0.01 feet); root mean square error, ± 0.11 feet; alternate lengths, group B (61 measurements) (Fig. 5c); arithmetic mean, -0.027 feet (or -0.03 feet); and root mean square error, ± 0.11 feet.

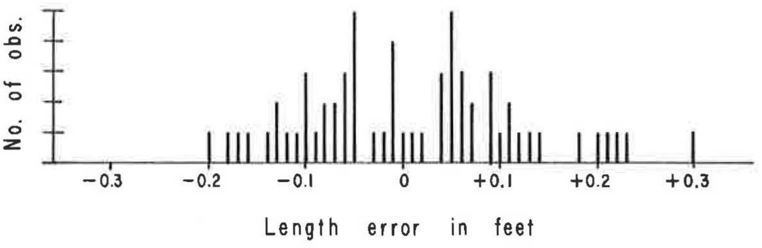
A rational analysis of errors indicates they are independent of distance measured. Even though the distances involved were about 50 feet, the same range of results would be expected if they were 900 feet (the airbase of a stereoscopic model formed by use of photographs taken at a scale of 250 feet per inch). Expressed in conventional terms of proportional accuracy, differences between exact measurements and these photogrammetrically made measurements should approach 1 part in 10,000 or smaller difference as the magnitude of the distance measured increases with the limits of a stereoscopic model. Because measured plane coordinates are being dealt with, however, perhaps errors should be expressed in terms of variation of coordinates, or "Absolute" accuracy rather than proportional accuracy.

Notes About Targets

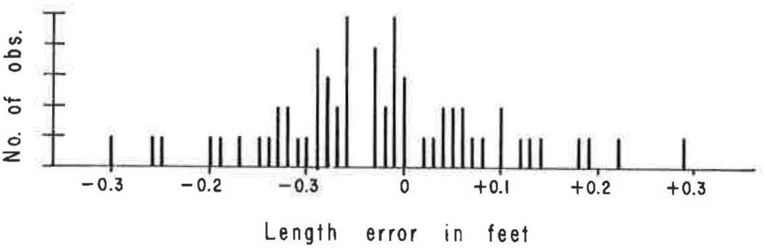
Aerial survey targets for placement before photography on control points or points for which plane coordinates are to be measured must be carefully considered for accurate work. The targets used for the projects reported in this paper have not been completely satisfactory. Most of these targets were printed on 10-point waterproof paper with printer's ink for attaining high contrast. Nevertheless they were not always distinctive in the stereoscopic models.



a. 123 consecutive measurements



b. 62 alternate measurements



c. 61 alternate measurements

Figure 5. Distribution of errors.

Figure 6 shows two targets of similar design but of different size and material. The right target is printed on waterproof paper, and the left target is printed on muslin cloth. It is readily noted the paper causes considerable reflection of light, whereas virtually no reflection occurs from the cloth. According to experience to date, the cloth target is far superior.

The cloth target size is 45 inches by 45 inches square. The crossarm width is 4 inches, and the white center is 4 inches square. The distance between the edge of the center square and the beginning of a crossarm is 7 inches. A target of this design provides a high proportion of black area to white area, which helps to balance halation caused by the white portions. The center square is easily visible on stereoscopic models in the Stereoplanigraph which are formed by using photography of a scale of 250 feet per inch

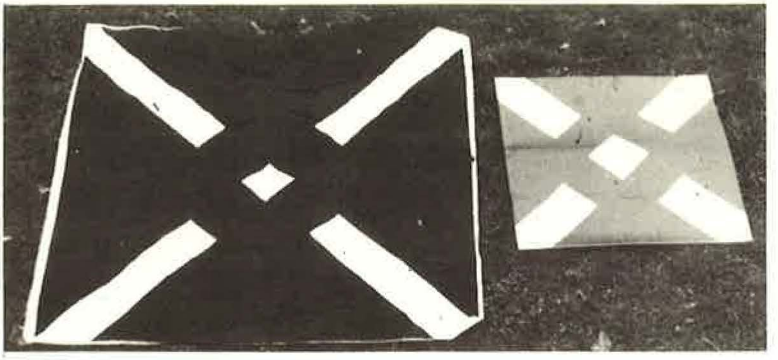


Figure 6. Aerial survey targets. Note the gray cast of target on right caused by light reflection.

and, under good conditions it is visible when the photography scale is 500 feet per inch. For any right-of-way or other type of cadastral surveying, it is necessary for the center of a target to be seen for making accurate measurements. The crossarms merely serve as reference identification for the center.

CONCLUSIONS

Results of investigations reported in this paper suggest precise photogrammetric methods can be adapted to making right-of-way and other cadastral surveys where high accuracies are required. Proper attention must be given, as in use of any other measurement technique, to particular phases of the entire operation if superior results are to be expected.

It is important that horizontal control points be position surveyed by use of the Geodimeter or other electronic distance-measuring equipment, and be targeted properly for absolute location on the stereoscopic models. Aerial photography should be taken using a scale stable base film with a cartographic camera of high resolution characteristics. High-quality photogrammetric instrumentation and procedure should be carefully considered. An electronic computer program for adjustment of photogrammetrically measured data is extremely helpful. And last, but not least, the photogrammetrist responsible for photogrammetric instrumentation must be thoroughly trained and skilled.

REFERENCE

1. Hovde, E. E., "Semi-Controlled Aerial Photographs as a Right-of-Way Surveying Tool." HRB Bull. 354, pp. 51-60 (1962).

Results of U.S. Forest Service Stereotriangulation Bridging on Virginia Highway Photogrammetric Test Project

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The Forest Service of the U. S. Department of Agriculture is constantly striving to improve the quality of the surveys it makes by photogrammetric methods. The tests described in this report were conducted using the usual methods and procedures. The report pertains to four basic factors which affect the quality of photogrammetrically made measurements: (a) identification of ground control (horizontal and vertical) points on the aerial photography, (b) movement of the aerial camera during negative exposure, (c) scale of the aerial photography, and (d) quantity of ground control.

Included are tabulations showing results of the bridges (extension of basic ground control by photogrammetric methods) using vertical photography taken with two different aerial cameras, two types of control identification, four scales of photography, and various spacings of ground control. These results are based on the photogrammetric measurement and electronic computation of 50 bridges.

Also presented is an outline of procedures to be followed for the location and design of highways based on surveys accomplished by photogrammetric methods, as well as conclusions based on these tests.

•IT IS IMPERATIVE that the Forest Service seek new methods of control extension by photogrammetry. Normal mountainous topography and short field survey season make it mandatory that photographs be used for inventory purposes, mapping, and engineering measurements. Aerial photographs to be used for making measurements by photogrammetric methods require some kind of ground control. Ground control is based on the U. S. Coast and Geodetic Survey (USC and GS) national network, with 1929 sea level datum for elevations and North American 1927 datum for horizontal control (with the geodetic positions converted to the State Plane Coordinates for each zone of application) inasmuch as this control is the most accurate and economical in the long run.

From 1947 to 1957, the Forest Service photogrammetrically extended vertical and horizontal control by the use of two scales of photography; i. e., field control was established for points on small-scale photography and, in turn, by photogrammetric use of this photography elevations and horizontal positions were established for selected points on larger scale photographs by use of common image points between the two scales of photography. The horizontal control was extended by the use of a stereo-templet triangulation plot. The stereo-templets were made by projection from the Kelsh stereoscopic plotter using the small-scale photographs. The Kelsh stereoscopic models were than reset (using position and distances determined by the stereo-templet triangulation plot) and vertical and horizontal control for the large-scale photographs was established from the stereoscopic models formed from the small-scale photographs.

From 1957 to the present, the Forest Service has been extending the horizontal and vertical control with the Zeiss Stereoplanigraph, Model C8. This method of control extension is called "bridging." (Wherever the word "bridging" is used in this paper it

Paper sponsored by Committee on Photogrammetry and Aerial Surveys.

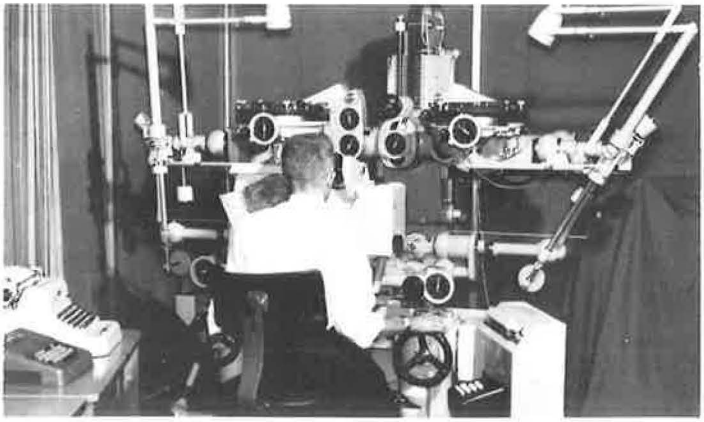


Figure 1. Zeiss Stereoplanigraph, Model C8.

means measurement by photogrammetric methods (stereotriangulation) and electronic computer adjustment of the measurements to establish both horizontal and vertical supplemental control for selected image points on the aerial photography between points on the ground and seen on the aerial photographs for which basic horizontal and vertical control has been surveyed on the ground.) By use of the C8, the photographs (printed as transparencies on glass) are cantilevered from the first model, with ground control through seccessive models to the end of the strip of photographs. The C8 has a system of optics and mirrors which allows one photograph to remain stationary while the next photograph is added to the stationary photograph. The "end product" of the C8 bridge is a stack of IBM cards with X, Y, and Z coordinates of identified points (control, pass points, center of each photograph and any other point). In other words, the Stereoplanigraph is an analogue computer that will take photographs of unknown scale, tip, tilt, and swing, and put them into a continuous strip of photographs with constant scale (horizontal and vertical) and on one datum. This statement is oversimplified, but it is in general what is being accomplished. The set of instrument-measured coordinates is computed with a digital computer to obtain the best least square fit on the ground coordinates. This paper gives the results of a series of tests on bridging by photogrammetric methods using aerial photography taken by two different aerial cameras, two types of control identification, four scales of photography, and various amounts of ground control within one strip.

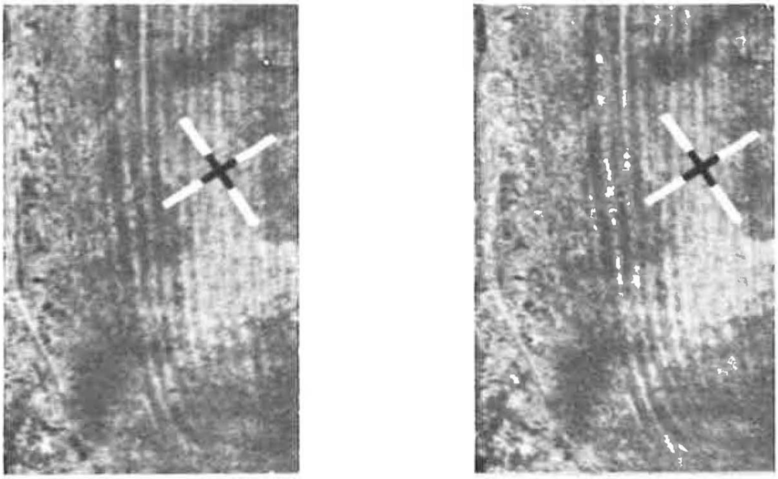
In early 1959, the Forest Service cooperated with the California Division of Highways in testing stereotriangulation bridging. The results of this test were inconclusive due to the few models available, and it was evident that further tests were needed.

In September 1959, the Virginia Department of Highways, U. S. Bureau of Public Roads, and the U. S. Forest Service agreed on a cooperative photogrammetric test project. The Virginia Department of Highways agreed to obtain aerial photography, establish horizontal and vertical control, set targets, identify "image" ground control points, and evaluate the bridging results. The Bureau agreed to plan the target placement, help set the targets and make stereotemplet bridges, using its three-projector Kelsh instrument. The Forest Service agreed to make stereotriangulation bridges with its Zeiss Stereoplanigraph, Model C8 (Fig. 1).

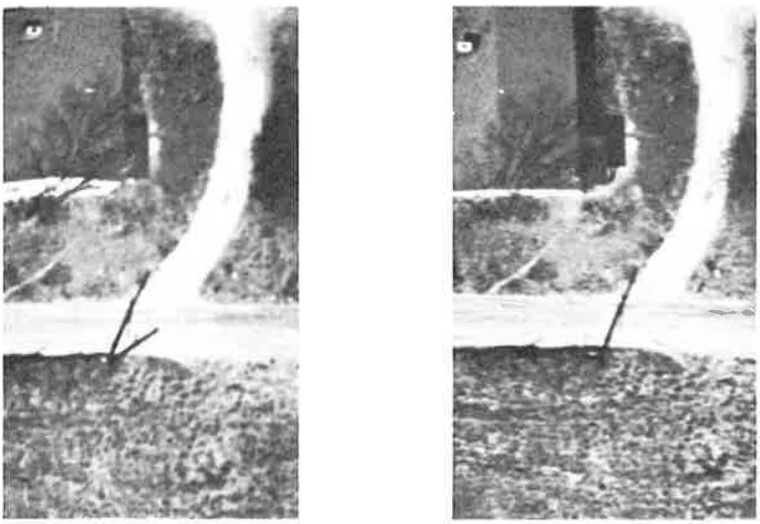
The area selected for the test is a preliminary survey segment of I-66 in Fairfax County, Virginia. Ground cover ranged from open farm land to wooded hills, and from a few roads and buildings to urban developments.

THE PROBLEMS

The Forest Service's part of the tests was planned for study of four problems in bridging to establish supplemental control for photogrammetric compilation of topographic maps and measurement of profile and cross-sections for highway design. The



This target is 27 feet long (9 ft white, 9 ft black, 9 ft white) and 23 in. wide.



The utility pole, with its shadow crossing the road, is an example of an image point.

Figure 2. Stereograms of a target and an image point. These photographic enlargements (8X) are at the same scale as viewed by the Stereoplanigraph operator.

tests must be considered a study of photogrammetric bridging methods and not basic research. These problems include:

1. Are targets required on control points for bridging or will natural images (picture points) suffice? (Fig. 2.)
2. Is the shutter speed on the aerial cameras a major source of error in bridging?
3. Can photogrammetric control be established from photography 2, 3, or 4 times smaller in scale than the design mapping photography and still be usable to measure

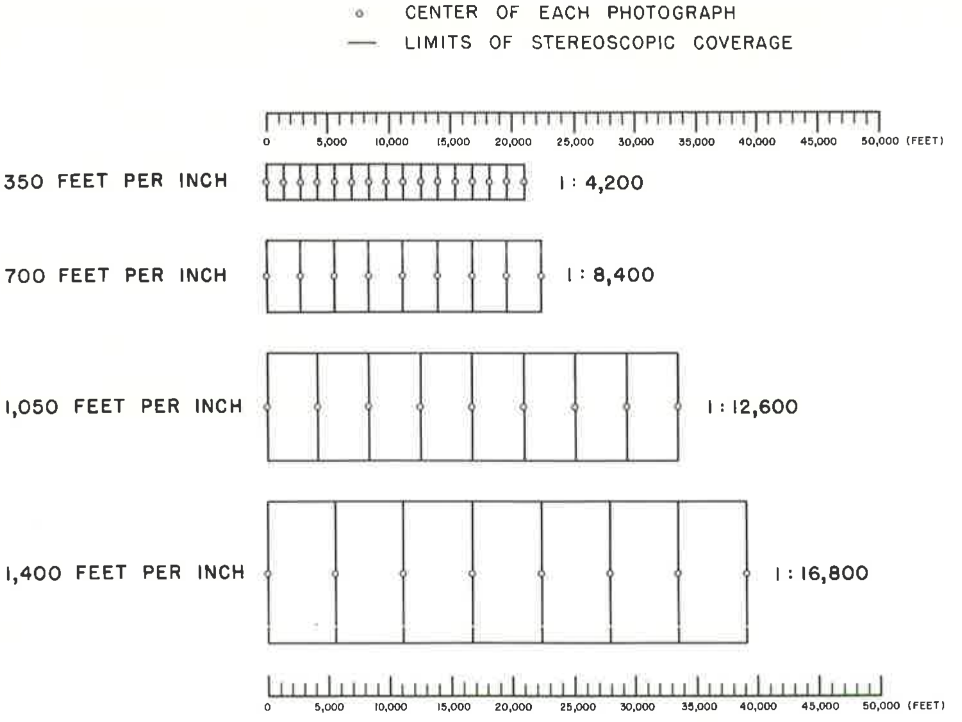


Figure 3. Scales of photography.

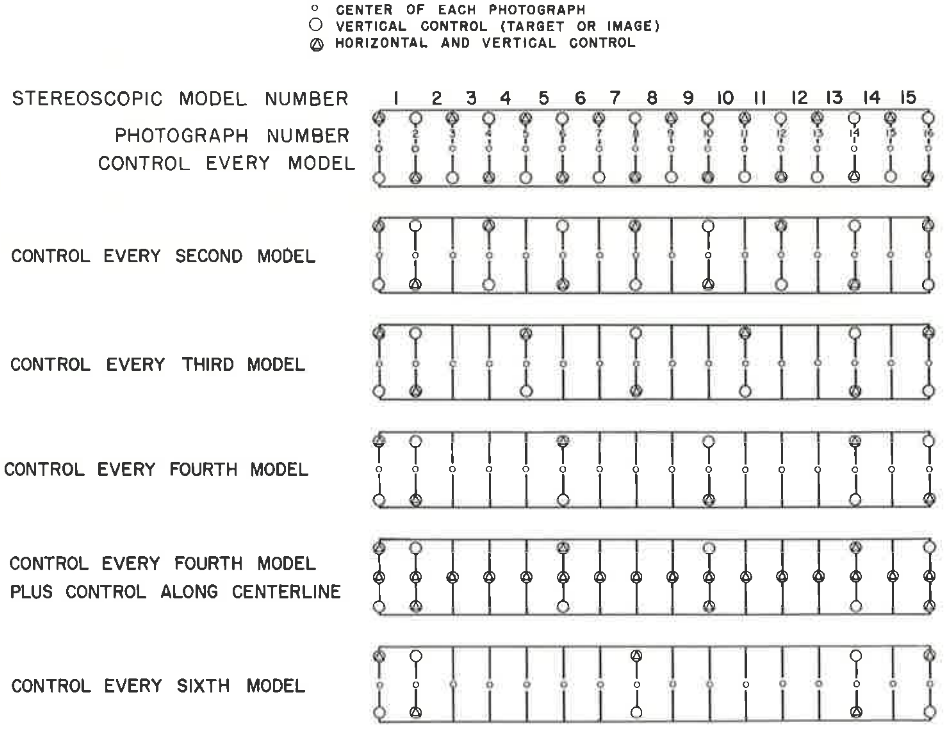


Figure 4. Control spacing for photography of 350 ft/in. scale.

and delineate planimetry and contours, and to measure profile and cross-sections? (Fig. 3)

4. Is field surveyed control required every 2nd, 3rd, 4th or 6th stereoscopic model used for bridging? (Fig. 4)

BRIDGING RESULTS USING THE 1959 PHOTOGRAPHY

The test area selected had been photographed by the Virginia Department of Highways (VDH) in the spring of 1959 with its Wild, 6-in. focal length aerial camera, equipped with a shutter which could be operated at a speed as fast as $^1/_{300}$ second To start the tests, this photography was used to measure and compute bridges with varying amount of image point control Tables 1 and 2 (with comments) give the results. At the time these bridges were measured and computed, the electronic computer programs were in two parts: (1) computation of the horizontal position of the bridge-measured points, and (2) computation of the vertical (elevation) position of bridge-measured points, using results from the electronic computer program for computing the horizontal bridge.

TABLE 1

HORIZONTAL BRIDGING RESULTS OF 1959 PHOTOGRAPHY[1]

Points (No.)	Largest Error (ft)	Least Error (ft)	Avg. Error (ft)	Standard Deviation (mean sq. error)	Total Error (ft)
Test Points					
48	2.244	0.102	0.927	1.056	44.508
47	2.360	0.098	0.818	0.968	38.467
47	2.887	0.022	1.027	1.199	48.264
46	2.640	0.045	0.886	1.052	40.778
44	2.304	0.106	0.828	0.955	36.452
38	2.304	0.120	0.882	1.002	33.519
12	2.140	0.530	1.174	1.684	14.088
Control Points					
3	0.104	0.041	0.083	0.088	0.248
4	0.311	0.041	0.144	0.177	0.576
4	0.120	0.030	0.071	0.079	0.282
5	0.368	0.061	0.192	0.230	0.959
7	0.683	0.141	0.359	0.397	2.516
13	0.582	0.188	0.380	0.399	4.945
39	1.153	0.136	0.585	0.689	22.799

[1] Horizontal results of the Forest Service Stereoplanigraph, Model C8, bridging of 12 stereoscopic models (stereotriangulation) using 1959, 6-inch focal length Wild camera photography at a scale of 350 feet per inch (1:4,200) which was taken at a shutter speed of $^1/_{300}$ second. The photogrammetrically measured bridge made using image point control was computed using the U.S.C. & G.S. horizontal bridging program in an IBM 650 electronic computer.

TABLE 2

BRIDGING RESULTS USING IMAGE POINTS

Bridging No.	Points (No.)	Largest Error (ft)	Least Error (ft)	Avg. Error (ft)	Standard Deviation (mean sq. error)	Arithmetic Mean (ft)	Total Neg. Error (ft)	Total Plus Error (ft)	Total Error (ft)
Test Points									
1	115	+4.85	0.00	1.109	1.479	0.554	95.61	31.95	127.56
2	113	+6.11	0.01	0.681	0.840	0.523	8.93	68.04	76.96
2A	113	+6.04	0.00	0.666	0.832	0.057	34.41	40.84	75.25
2B	113	+6.22	0.00	0.815	1.041	0.038	43.93	48.17	92.10
3	111	+5.58	0.00	1.104	1.455	0.535	31.60	90.94	122.54
4	107	+3.37	0.00	1.041	1.378	0.146	47.88	63.47	111.35
5	95	+2.72	0.06	1.010	1.357	0.385	29.70	66.29	95.99
6	82	+3.57	0.00	1.011	1.345	0.122	36.46	46.45	82.91
Control Points									
1	6	-0.07	-0.02	0.038	0.041	0.038	-0.23	0.00	0.23
2	8	+0.62	+0.14	0.358	0.401	0.010	1.47	1.39	2.86
2A	8	+0.26	+0.03	0.133	0.153	0.035	-0.39	0.67	1.06
2B	8	+0.20	+0.01	0.114	0.144	0.046	-0.27	0.64	0.91
3	10	+1.96	-0.20	0.914	1.123	0.035	-4.76	4.41	9.17
4	14	+2.12	-0.07	0.814	1.068	0.055	-6.08	5.31	11.39
5	26	-2.31	+0.03	0.963	1.164	0.048	-13.15	11.89	25.04
6	39	-2.97	-0.01	0.976	1.196	0.047	-19.96	18.12	38.08

Table 2

Results of the Forest Service Stereoplanigraph (C8) bridging of vertical control on 12 stereoscopic models (stereotriangulation), using image point control on 1959, 6-inch focal length, Wild camera photography taken at a scale of 350 feet per inch (1:4,200) with a shutter speed of $^1/_{300}$ second, were computed using the U. S. C. and G. S. vertical bridging program in an IBM 650 electronic computer.

Control Points. --Bridging no. 1 gives computed results using six control points: two at the west edge of the first stereoscopic model, two near the middle, and two at the east edge of the last model. Bridging no. 2 gives computed results using eight control points: two at the west edge of the first stereoscopic model, and two each near the fourth, eighth, and twelfth models. Bridging no. 2A gives computed results using eight control points, as in bridging no. 2, but three control points were changed; the horizontal control is along the flight centerline. Bridging no. 2B gives computed results using the same eight control points as in bridging no. 2A but with the horizontal control in the pass point area. Bridging no. 3 gives computed results using ten control points: two each on the first, third, sixth, ninth, and twelfth stereoscopic models. Bridging no. 4 gives computed results using fourteen control points: two each on the first, second, fourth, sixth, eighth, eleventh, and twelfth stereoscopic models. Bridging no. 5 gives computed results using twenty-six control points: four on the first and two for all other stereoscopic models. Bridging no. 6 gives computed results using thirty-nine control points spaced throughout the strip.

BASIC MATERIAL

Four scales of photography were obtained in the spring of 1961 (before the deciduous trees had leafed out): 350, 700, 1,050, and 1,400 feet to 1 inch. The photography scales selected were for compilation of topographic maps by use of a Kelsh stereoscopic plotter at a 7-diameter projection ratio (the ratio used by the Virginia Department of Highways; Forest Service uses a 5-diameter projection ratio). By use of the 7-diameter projection ratio, maps can be compiled at scales of 50, 100, 150 and 200 feet to 1 inch, respectively, using photography of the scales at which taken.

All four scales of photography were obtained with two precision 6-in. focal length cameras: the Virginia Department of Highways Wild camera with a shutter speed of $^1/_{300}$ second and a Zeiss camera with a shutter speed of $^1/_{1,000}$ second. Besides using photography taken at the four scales with the two cameras, the test project area was also photographed on clear and overcast days, with and without filters. In all some 43 flights of photography were obtained.

The U. S. Bureau of Public Roads and Virginia Department of Highways set the targets before any flights were made for taking the test project photography. The spacing of the targets on the photographs as taken did not agree exactly with plans, but was as good as could be expected on a production project. After the photographs were obtained, the Forest Service selected and circle identified image points and requested the Virginia Department of Highways to measure horizontal and vertical control positions for each of these images. As a whole, the results of this method were satisfactory. In one case, however, an image point was circled on the 1,050 feet per inch scale photography, described as a "fence corner," and the field party surveyed the position of the fence corner. Actually, the circled image was on a ditch crossing of the fence line, which was about 0.004 inch away (on the contract printed photograph) from the fence corner. Such misidentification of images is one of the pitfalls in using image points for horizontal control.

Photography image control points (common image points) were selected and circle identified on all scales of photography from both cameras before the bridging work was started. In other words, natural image points were selected for correlation of position between the 350 feet per inch scale photography and the photography of 1,400, 1,050, and 700 feet per inch scales. This procedure became a compromise in point selection which was necessary in order to do bridging with the photography of 350 feet per inch scale using image points for which control had been bridged using photography of the three smaller scales.

The Virginia Department of Highways furnished the horizontal and vertical control required for bridging with the 1,400 feet per inch scale photography. Control for the other scales was withheld by VDH until the Forest Service finished the bridges using photography of the 1,400 feet per inch scale and forwarded results to the VDH. The bridges were computed on the IBM 650 using basic formulas published by the U. S. C. and G. S. (1, 2).*

TARGET OR IMAGE POINTS

The test area selected in Northern Virginia is better than average for the selection of natural images. More image points are required for the same number of control points when natural images are used than when targets are placed on station markers because the identification of a triangulation station which is not targeted requires use of at least two nearby image points. This means that two bearing and distance measurements are needed for the identification of one station, where a target over the actual station marker would not require additional field measurements.

The use of image points for vertical control is sound because a flat area can be misidentified horizontally by several feet and still not impair the accuracy of the point. The actual field survey measurement of the elevation on an image point has a good chance of being accurate within 0.1 or 0.2 foot.

The use of image points for horizontal control is not sound. Further study is required, however, to determine the best size, shape, and color for targets, and the materials most suitable for their construction.

Most of the advantages of targets are lost if the photogrammetric instrument operator is unable to measure accurately both horizontally and vertically on the point.

The field surveyed positions of image points are not finite and the position of good image points may be in error by 0.5 foot. Control surveyed to second order accuracy is recommended, plus or minus 0.1/foot, so results from bridging will not be degraded by poor quality in the field surveyed control.

AIRCRAFT MOVEMENT DURING EXPOSURE

Two cameras were used on this test—a Wild with a shutter speed of $^1/_{300}$ second and a Zeiss with a shutter speed of $^1/_{1,000}$ second. At 120 mph, the aircraft with the Wild camera traveled 0.59 foot and the Zeiss 0.18 foot while the shutter was open. According to these figures, the Zeiss camera would be expected to enable achievement of better results, but the test showed bridging results were the same, regardless of which camera was used. The only way such an equality in results could be rationalized was to study the resolving power of the two cameras be-

*The Forest Service has been bridging with the Stereoplanigraph since 1957 and is now using the basic formulas of the U. S. Coast and Geodetic Survey. The horizontal and vertical control electronic computer programs were combined and a "borrow" feature added. This feature allows the borrowing of photogrammetrically measured control points from one strip and the use of them on an adjacent strip, which is a form of a modified block adjustment. While not used on the test project, the production bridging program has a horizontal and vertical rejection limit; i.e., the program is loaded to reject horizontal points with X foot error. For example, if a rejection horizontal error of two feet and vertical error of one foot are desired and if there are 18 horizontal control points and 28 vertical control points, the procedure would be as follows:

The horizontal portion of the formulas are computed first to get a printout (storage) of the least square fit of the 18 horizontal control points. The point with the largest error (if over 2 feet) is rejected and a printout (storage) with the least square fit of the 17 remaining points is obtained. This process continues, rejecting the largest error, one point at a time, until the remaining control points have less than two feet of error. The same process is repeated with the vertical control, except this time points with an error of over one foot are rejected, one point at a time; i.e., a fit with 28 points, a fit with 27 points, etc. After the rejection limit has been satisfied, the correction coefficients are applied to all points in the bridge. This program was not used on the Virginia Highway test, because it was for testing all elements of bridging, not just control. For this reason, it is believed better results can be obtained on a production job than on this test.

cause camera movement, the same as poor resolving power, would appear as fuzzy images.

Because there is no international test for cameras, the manufacturer's report was used to compute the number of lines per millimeter times the percent of the 9- × 9-in. format. For example, from the center of the lens out five degrees, approximately 1 percent of the 9 × 9 in. is covered. This percentage multiplied by the lines per millimeter, for both radial and tangetial resolution, gives the number used to compare the cameras. The Wild camera had approximately 15 percent better resolving power.

ESTABLISHING CONTROL IN PHOTOGRAMMETRY

It is difficult to say that horizontal and vertical control can be established from control photography two, three, or four times smaller in scale than photography taken for mapping purposes without some kind of a common denominator. The scale of some strip topographic mapping for design purposes is 40 feet per inch, another may be 200 feet per inch, and the contour interval may range from one foot to five feet. To establish a common denominator, the photography flight height divided by the average error was used. For example, if the average error was 0. 6 foot and the flight height was 4, 200 feet, the factor would be 1:7, 000; in other words, an error of one foot in vertical measurement can be expected for each flight height increment of 7, 000 feet.

About fifty bridges were measured and adjustment computed using a combination of the four scales of photography of 350, 700, 1, 050 and 1, 400 feet per inch taken separately with two aerial cameras (Wild and Zeiss); two types of control identification (image points and target) and two different spacings of the control (one with control every fourth stereoscopic model, and the other with control every fourth model plus horizontal and vertical control about 1, 400 feet apart along the centerline of the high-

TABLE 3

BRIDGING RESULTS FOR CHECK POINTS[1]

Camera	Control Identification	Scale of Photography (ft/in.)							
		Horizontal Results				Vertical Results			
		350	700	1, 050	1, 400	350	700	1, 050	1, 400
Control Every Fourth Model									
Zeiss	Picture point	1:1926	1:3853	1:3000	1:4389	1:4487	1:4773	1:5385	1:5156
Wild	Picture point	1:1944	1:3307	1:2800	1:3676	1:4516	1:5676	1:4598	1:5519
Avg.	Picture point	1:1935	1:3580	1:2900	1:4032	1:4502	1:5224	1:4992	1:5388
Zeiss	Target	1:2143	1:3529	1:4809	1:3992	1:4200	1:4200	1:6774	1:3406
Wild	Target	1:2658	1:3784	1:3462	1:3676	1:5000	1:6462	1:5040	1:3987
Avg.	Target	1:2400	1:3656	1:4135	1:3834	1:4600	1:5331	1:5905	1:3696
Avg. Total		1:2168	1:3618	1:3518	1:3933	1:4551	1:5278	1:5449	1:4517
Control Every Fourth Model Plus Along the Centerline of the Highway									
Zeiss	Picture point	1:2354	1:4343	1:6087	1:4819	1:4667	1:5250	1:6709	1:8580
Wild	Picture point	1:2515	1:4316	1:4828	1:5237	1:5357	1:5357	1:5620	1:6931
Avg.	Picture point	1:2434	1:4330	1:5428	1:5028	1:5012	1:5916	1:6164	1:7756
Zeiss	Target	1:2300	1:4375	1:5727	1:5138	1:4286	1:5250	1:7167	1:7939
Wild	Target	1:2482	1:4730	1:4609	1:5166	1:5541	1:6491	1:5375	1:6195
Avg.	Target	1:2391	1:4552	1:5168	1:5152	1:4914	1:5870	1:6271	1:7067
Avg. Total		1:2413	1:4441	1:5313	1:5090	1:4963	1:5894	1:6218	1:7411

[1] Results of the aerotriangulation by the Stereoplanigraph (bridging), using horizontal ground positions and elevations as check points. The control used to compute the bridges is not included in the tabulation.

way). The following comments and Tables 3 through 5 summarize results of these bridging tests.

In Table 4, results are given for the same bridges as compared in Table 3, but in Table 4 the flight height is related to the error on the ground control used to compute the bridges. For example: On the 350 feet per inch scale Wild (photography) target (control), control every fourth model shows 1:2, 658 (Table 3) while the control used to compute the bridge shows 1:7, 500 (Table 4). It should be noted from Table 4 that results are very erratic when control is used in every fourth model. When as few as

TABLE 4

BRIDGING RESULTS FOR CONTROL POINTS

Camera	Control Identification	Scale of Photography (ft/in.)							
		Horizontal Results				Vertical Results			
		350	700	1,050	1,400	350	700	1,050	1,400
Control Every Fourth Model									
Zeiss	Picture point	1:6364	1:89362	1:78750	1:125373	1:5676	1:17500	1:71590	1:64615
Wild	Picture point	1:9130	1:127273	1:57273	1:254545	1:16154	1:56000	1:21000	1:30000
Zeiss	Target	1:7500	1:52500	1:63000	1:105000	1:14000	1:13548	1:42000	1:27097
Wild	Target	1:7500	1:62686	1:94029	1:105000	1:12353	1:42000	1:42000	1:20000
Control Every Fourth Model Plus Along the Centerline of the Highway									
Zeiss	Picture point	1:3281	1:6774	1:6560	1:80000	1:6364	1:10220	1:8720	1:10500
Wild	Picture point	1:3962	1:6462	1:5620	1:5833	1:7143	1:9550	1:10700	1:9130
Avg.	Picture point	1:3621	1:6618	1:6090	1:6917	1:6754	1:9885	1:9710	1:9865
Zeiss	Target	1:3621	1:6462	1:7780	1:7568	1:7692	1:9550	1:12350	1:8155
Wild	Target	1:4545	1:8077	1:5430	1:6087	1:9091	1:10780	1:12350	1:8235
Avg.	Target	1:4083	1:7269	1:6605	1:6827	1:8392	1:10215	1:12350	1:8195
Avg. Total		1:3852	1:6944	1:6347	1:6872	1:7572	1:10025	1:11030	1:9005

TABLE 5

BRIDGING RESULTS FROM COMMON POINTS WITH THE PHOTOGRAPHY, 350 FEET PER INCH SCALE, AS A BASE

Control Identification	Scale of Photography (ft/in.)					
	Horizontal Results			Vertical Results		
	700	1,050	1,400	700	1,050	1,400
Bridging With Control Every Fourth Model						
Picture point	1:3621	1:3728	1:2675	1:9333	1:5294	1:4398
Target	1:3889	1:4375	1:4541	1:8077	1:6300	1:6269
Average	1:3784	1:4091	1:3485	1:8700	1:5833	1:5316
Bridging With Control Every Fourth Model Plus Along the Centerline of the Highway						
Picture point	1:5316	1:4286	1:3443	1:8571	1:5833	1:6043
Target	1:6176	1:5294	1:6087	1:11053	1:7241	1:11507
Average	1:5833	1:4884	1:4565	1:10000	1:6632	1:8235

three control points are used (as was done in using photography of 1,400 feet per inch scale for horizontal bridging) the bridge adjustment fits the control almost exactly and the ratio, flight height to error, is unbelievable. To prove this, the horizontal bridging done using Zeiss camera photography of the 1,400 feet per inch scale was recomputed using 5 control points (3 in the 1st stereoscopic model) instead of 3 (one every 4th model). The ratio decreased from 1:105,000 to 1:15,556 (present thinking is 1:8,000 to 1:12,000) which is still too accurate, but does show an average measurement is not obtained when too few control points are used.

To evaluate the results of establishing horizontal and vertical control points photogrammetrically, the average value obtained by use of photography of 350 feet per inch scale was used, with control every fourth stereoscopic model plus control along the centerline of the highway as a base. In other words, the position and elevation of the points established by using photography of 350 feet per inch scale were used to check the elevation and horizontal position of the same points obtained by separately using the other three scales of photography.

Table 5 gives the bridging results related to flight height, using natural image point and target marked control in photography used from the two cameras (Wild and Zeiss) combined.

All common points between the four scales of photography were used in the tabulation. The photogrammetric instrument operator had to select "compromise image points" on photography of one scale to get image points which would be common to photography of all four scales. It is believed that better results would be obtained if only two scales of photography were involved.

By using all common points, with no rejects, the tabulations appear to be erratic. One point on the photography of 1,400 feet per inch scale, included in the tabulation, had a 10-ft horizontal error. If this test project were an actual bridging and mapping project (with just two scales of photography to be used) selection of the best images for use as bridging points could be made.

Table 7 is a different approach to the question: "Can supplemental horizontal and vertical control be established photogrammetrically from small scale photography for control of large scale photography to photogrammetrically compile topographic maps?"

For such bridging previous tests showed small differences occurred between using photography from the Wild and Zeiss cameras, so for this test only photography from the Wild camera was used. In order for this test to be considered as being accomplished on a "production methods" basis, the control established photogrammetrically

TABLE 6

GUIDE LINES FOR PHOTOGRAPHIC SCALES[1]

Control Identification	Scale of Photography (ft/in.)					
	Horizontal Control			Vertical Control		
	700	1,050	1,400	700	1,050	1,400
Bridging With Control Every Fourth Model						
Picture point	2.3	3.2	6.4	0.9	2.6	4.4
Target	2.0	2.7	3.3	1.1	2.0	2.6
Average	2.1	2.9	4.8	1.0	2.3	3.5
Bridging With Control Every Fourth Model Plus Along the Centerline of the Highway						
Picture point	1.7	3.2	5.7	0.9	2.3	2.9
Target	1.2	2.2	2.7	0.9	1.7	1.6
Average	1.4	2.7	4.2	0.9	2.0	2.2

[1] Tentative guidelines in planning photography for bridging control and for topographic mapping. Accuracy (in feet) to be expected for 90 percent of the points for which horizontal position and elevation are established by bridging using the three scales of photography.

TABLE 7

BRIDGING RESULTS USING IMAGE POINTS COMMON TO ALL PHOTOGRAPHY AND THE GROUND SURVEYED CONTROL AS CHECKS[1]

Photo Scale (ft/in.)	Check or Control Points (No.)	Error (max.)	Error (min.)	Total Error	Avg. Error $\left[\frac{\text{Total Error}}{\text{No. of Points}}\right]$	Algebraic Arithmetic Mean $\left[\frac{\text{Algebraic Sum}}{\text{No. of Points}}\right]$	Standard Deviation $\left(\sqrt{\frac{\Sigma d^2}{N}}\right)$	90 Percent Within (1.65 × S.D.)	B Factor $\left[\frac{\text{Flight Height}}{\text{Avg. Error}}\right]$
					Horizontal				
700	70[2]	3.4	0.1	63.1	0.9	-	1.06	1.75	4,667
	18[3]	2.4	0.1	13.9	0.8	-	0.9	1.48	5,250
1,050	70[2]	4.0	0.3	84.4	1.21	-	1.36	2.24	5,207
	19[3]	3.7	0.4	27.4	1.4	-	1.6	2.64	4,500
1,400	70[2]	5.0	0.1	110.85	1.58	-	1.78	2.94	5,316
	19[3]	6.4	0.2	36.5	1.9	-	2.4	3.96	4,444
					Vertical				
700	121[2]	6.2	0.0	60.5	0.5	0.1	0.8	1.32	8,400
	18[3]	1.1	0.0	73	0.4	0.1	0.5	0.82	10,000
1,050	121[2]	5.7	0.0	128.9	1.1	0.7	1.3	2.15	5,727
	19[3]	1.8	0.0	12.2	0.6	0.0	0.8	1.32	10,000
1,400	121[2]	7.1	0.0	143.5	1.2	0.1	1.6	2.64	7,000
	19[3]	1.4	0.0	8.9	0.5	0.0	0.6	0.99	16,800

[1] Evaluation of aerotriangulation (bridging) with the Stereoplanigraph C8, using the Wild camera photography at a scale of 350 feet/inch. The control used for measuring and computing these bridges was established by photogrammetric bridging using Wild camera photography at the scales at 700, 1,050, and 1,400 feet/inch, which in turn, was controlled by use of ground surveyed control every fourth stereoscopic model.
[2] Check points.
[3] Control used.

consisted of image points. Common points were selected between the four scales of Wild camera photography, and horizontal positions and elevations were obtained from the bridges using control in every fourth model. The 350 feet per inch scale Wild camera photography bridges were computed using control established from bridging done by use of Wild camera photography of the scales of 700, 1,050, and 1,400 feet per inch. To check results obtained from the bridging accomplished using these three separate photography scales, all ground control appearing on the photography of 350 feet per inch scale was tabulated.

Results are erratic. Here again compromise image points were used in order to get conjugation of images on all four scales of photography. Often rejection of one or two points would improve results considerably. This is done on production jobs when two scales of photography are used, rather than four scales as in this test.

On a whole, these results are better than those shown in Table 6. The main differences occur with respect to bridging done using photography of the 1,400 feet per inch scale.

SPACING OF CONTROL FOR BRIDGING

All of the previously described bridges which were measured photogrammetrically were computed with surveyed control in every fourth stereoscopic model or every fourth model plus centerline. The Wild camera photography of 350 feet per inch scale used for this test contained 17 exposures for 16 stereoscopic models. Table 8 summarizes results of varying the surveyed control from every stereoscopic model to every sixth model.

It should be noted that surveyed control spacing has a very small influence on bridging results. In other words, 75 percent of the ground surveyed control could have been eliminated and results would have been the same.

Bridging results shown in Table 8 are better than those in Tables 1 and 2, which were obtained using photographs taken in 1959. The only reason for this difference is that the image points were selected by the Stereoplanigraph instrument operator and the field survey crew established horizontal position or elevation for these office selected image points.

An evaluation of Tables 7 and 8 shows that use of smaller scale photography for bridging gives better results when compared with the flight height. This is caused by "built-in" errors such as camera movement during negative exposure and field surveying errors in measuring control point positions (horizontal and vertical).

From these tests and other production projects, it is recommended that surveys for highway location and design by photogrammetric methods should proceed along the following lines:

TABLE 8

BRIDGING RESULTS FROM DIFFERENT CONTROL SPACING[1]

Control Spaced Every:	Check or Control Points (No.)	Error (max.)	Error (min.)	Total Error	Avg. Error $\left[\frac{\text{Total Error}}{\text{No. of Points}}\right]$	Algebraic Arithmetic Mean $\left[\frac{\text{Algebraic Sum}}{\text{No. of Points}}\right]$	Standard Deviation $\left(\sqrt{\frac{\Sigma d^2}{N}}\right)$	90 Percent Within (1.65 × S.D.)	B Factor $\left[\frac{\text{Flight Height}}{\text{Avg. Error}}\right]$
				Horizontal					
Model	50[2]	2.2	0.1	39.2	0.78	-	0.91	1.52	2,692
	19[3]	0.8	0.1	8.5	0.45	-	0.49	0.81	4,667
Second model	59[2]	2.0	0.0	44.0	0.75	-	0.87	1.43	2,800
	10[3]	0.7	0.2	4.4	0.44	-	0.46	0.76	4,773
Third model	61[3]	2.0	0.1	44.3	0.73	-	0.84	1.39	2,877
	8[3]	0.7	0.1	3.6	0.45	-	0.48	0.79	4,667
Fourth model	62[2]	2.1	0.1	48.1	0.78	-	0.89	1.47	2,692
	7[3]	0.5	0.1	2.5	0.36	-	0.38	0.63	5,833
Sixth model	63[2]	2.0	0.1	48.1	0.76	-	0.86	1.42	2,763
	6[3]	0.5	0.1	1.9	0.32	-	0.34	0.56	6,563
				Vertical					
Model	72[2]	1.4	0.0	31.5	0.44	0.04	0.58	0.96	4,773
	48[3]	0.8	0.0	13.0	0.27	0.03	0.33	0.54	7,778
Second model	103[2]	1.6	0.0	45.7	0.44	0.02	0.58	0.96	4,773
	17[3]	0.6	0.0	3.5	0.20	0.05	0.27	0.45	10,500
Third model	107[2]	1.5	0.0	47.8	0.45	0.08	0.58	0.96	4,667
	13[3]	0.4	0.0	2.2	0.17	0.05	0.20	0.33	12,353
Fourth model	109[2]	1.5	0.0	45.8	0.42	0.01	0.56	0.92	5,000
	11[3]	0.3	0.0	1.7	0.15	0.05	0.19	0.31	14,000
Sixth model	111[2]	1.5	0.0	46.5	0.42	0.08	0.55	0.91	5,000
	9[3]	0.2	0.0	1.3	0.14	0.06	0.15	0.25	15,000

[1] Evaluation of aerotriangulation (bridging) with the Stereoplanigraph C8 using the Wild camera photography at a scale of 350 feet/inch. The amount of ground control used to compute the photogrammetrically measured bridges was varied from every model to every sixth model. Total number of horizontal control points available was 69. Total number of vertical control points available was 120.
[2] Check points.
[3] Control used.

1. Outline the area for location of highway route alternatives on topographic maps which are published on a quadrangle basis.

2. Plan aerial photography flights to obtain photographic coverage of the outlined area and existing horizontal control. For example, if photography at a scale of 1,400 feet per inch is planned, photographs of the usual 9- by 9-in. format will cover a strip area width in excess of two miles for the length of the flight. On some projects, there will be a sufficient number of horizontal control stations to control the bridges; if not, additional horizontal control will have to be surveyed.

3. Set targets on the ground before photography for each horizontal control point. These targets should be in the shape of a cross (plus mark) each leg of which should be 14 feet long on the ground (0.01 inch long on the photography) and 3 feet wide (0.002 inch on the photography). The center 7 feet of the plus mark should be made of dull black material and the four tips (3.5 × 3 feet) of dull white material. The plus mark should be oriented so the legs will be parallel and normal to the photography flight line. Besides control identification, these targets will afford a check on camera motions during exposure and flight-line placement.

4. Obtain the planned 6-in. focal length "distortion free" photography at the scale of 1,400 feet per inch.

5. Identify both existing and essential additional vertical control by selected natural image points for controlling the bridge with two vertical control points in every fourth stereoscopic model (approximately 4 mile spacing along the photography strip).

6. Measure and compute the bridges. Plot the bridging results at a scale of 200 feet per inch and photogrammetrically compile the topographic maps, with contours at a 10-ft interval, of the route band outlined on the small scale quadrangle size topographic map.

7. Project proposed highway centerlines on these 200 feet per inch scale topographic maps and decide on the best alinement location.

8. Stake on the ground and monument a control survey base line, which will be near the proposed location of the highway centerline. This survey base line will be used to control topographic mapping to be done photogrammetrically for highway design and preparation of detailed highway construction plans.

9. Plan photography flight lines to obtain photographs of the proposed highway location corridor. For example, plan for taking photography at a scale of 350 feet per inch which will provide a width of photography coverage of more than 1,400 feet on either side of the flight line.

10. Set targets on the control survey (step 8) at an interval of about 1,000 feet along the base line. These targets would be in the form of a plus mark (like those for the 1,400 feet to one inch scale photography) 3.5 feet long (0.01 inch on the photography) and 0.7 of a foot wide (0.002 inch on the photography) with the center black and the tips white.

11. Obtain 6-in. focal length "distortion free" photography as planned in step 9.

12. Identify image points common between the photography of 1,400 feet per inch scale and the photography of 350 feet per inch scale. Reset the 1,400 feet per inch scale photography in the Stereoplanigraph and establish horizontal position and elevation for each of the common points which will be near the edges of the 350 feet per inch scale photographs.

13. Measure and compute the bridges using photography of 350 feet per inch scale and control from the control survey and control established for the image points common to both scales of photography by use of the 1,400 feet to one inch scale photography. Plot the bridged control points on a manuscript at the scale of 50 feet per inch and compile the topographic maps with contours at a 2-ft interval, or measure profile and cross-sections.

14. Design the highway based on the maps or stereoscopic model measured cross-sections.

15. Using the control survey as positioning origin, compute a description of the centerline of the designed highway and stake it on the ground for construction.

CONCLUSIONS

Stereotriangulation bridging for highway location and design is feasible and economical. It will improve the quality of each photogrammetrically made survey because the plotted position of bridged control points will result in a better individual solution for each stereoscopic model.

Targets will improve the accuracy of horizontal control bridging but may not improve the accuracy of vertical control bridging. Continued research on materials for, and colors, shapes, and placement of targets is needed. Being able to see a target on a contact printed photograph is a long way from making an accurate measurement when it is magnified eight or ten times in the view provided by the Stereoplanigraph for the instrument operator. Most of the usefulness of precise measurements for the horizontal and vertical position of targets is lost whenever the targets do not provide clear definite patterns of good contrast.

Use of photography taken with a camera having an efficient fast shutter should give better bridging results than photography taken with a camera having an efficient slower shutter. Camera movement during exposure causes a fuzzy appearance in the images, and poor lens resolution will also produce fuzzy images; therefore, in order to take advantage of a fast shutter, the resolving power of the lenses (with slow and fast shutter) should be approximately equal. The test results were approximately equal for the two speeds of shutters.

The use of multiple scales of photography for control extension is feasible and will give "by-products" that will more than pay for the photography. Tables 6 and 7 show results to be expected for three different scales of photography.

The spacing of field surveyed control required for stereotriangulation is somewhat nebulous but, at this time, one horizontal and two vertical control points every fourth stereoscopic model will produce results consistent with the photography quality, targets used, and field surveyed control usually obtained.

REFERENCES

1. Harris, W. D., "Aerotriangulation Adjustment of Instrument Data by Computational Methods." U.S.C. and G.S., Tech. Bull. No. 1.
2. Harris, W. D., "Vertical Adjustment of Instrument Aerotriangulation by Computational Methods." U.S.C. and G.S., Tech. Bull. No. 10.

Use of Aerial Photography in the Kansas Courts

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•AERIAL PHOTOGRAPHY frequently plays an important part in preparation of a condemnation appeal case for jury trial and in presentation of facts to the jury in Kansas. Generally, an aerial photograph is competent evidence in court on the same basis as oral testimony.

The aerial photographs discussed herein were furnished by the Photronics Department of the State Highway Commission of Kansas from photographs taken by the Photronics Department and printed in its laboratory. The Photronics Department owns and operates a Cessna 182 Skylane aircraft equipped with a Wild RC-8 camera. Cronar base film was used in taking the photographs at the usual image area size of 9 inches by 9 inches for each exposure. The current practice of the Photronics Department is to make two flights at a height of 6,000 feet above the ground—one along each side of the highway survey project. The ground area coverage of the adjacent matching side of each parallel strip of photographs contains the survey project corridor. In the preparation and trial of court cases involving real property, aerial photographs and enlargements are useful in several ways. Under Kansas law, compensation to landowners for real property taken or damaged by condemnation is determined by jury trial after an appeal is filed, by either the condemnor or the landowner, from the amount of compensation to the landowner determined by three court-appointed appraisers. Until January 1, 1964, expert appraisal witnesses appearing in condemnation appeal trials in Kansas were required to appraise the entire acreage under one ownership operated as one unit. The opinion testimony of these appraisal witnesses was the market value of the entire unit before the market value of the land taken, and the market values of the remaining land before and after the taking. After January 1, 1964, a new Kansas statute stipulates the measure of compensation shall be the difference between the market value of the entire property before and the market value of the remaining tract after the taking. To arrive at these values the appraiser must be aware of all pertinent features of the entire unit. If the taking is from a farm unit, this can involve hundreds of acres and all the improvements on the unit. Frequently, a review of an enlargement of an aerial photograph will reveal facts which were overlooked on actual inspection of the unit. Such features as erosion scars, watercourses, ponds, trials, roads, terraces, and trees are easily identified on an aerial photograph. Where improvements are altered or removed after the date of taking, an aerial photograph can be useful as a review and check of the inventory of improvements made by the appraiser (Fig. 1). Figure 1 is an enlargement of a photograph taken from a flight height of 1,500 feet during consideration of proposed alternatives for a highway location. This photograph was used to prepare a witness for anticipated cross-examination concerning the improvements on a farm unit when it was discovered he was not completely familiar with the improvements and had not inventoried them as of the date of taking. These buildings were more than a half-mile away from the highway right-of-way taken by condemnation, but since they were a part of the same unit, valuation witnesses were subject to cross-examination concerning the value of these buildings. This photograph was kept handy during trial as an aid in cross-examination of the landowner.

In highway condemnation appeal trials in Kansas, values are determined as of the date the amount of the court-ordered appraisal is paid into court. This date then becomes the "date of taking" of the property. The aerial photographs should be taken as nearly on that date as practicable. Any appreciable lapse of time between the date of

Paper sponsored by Committee on Photogrammetry and Aerial Surveys.

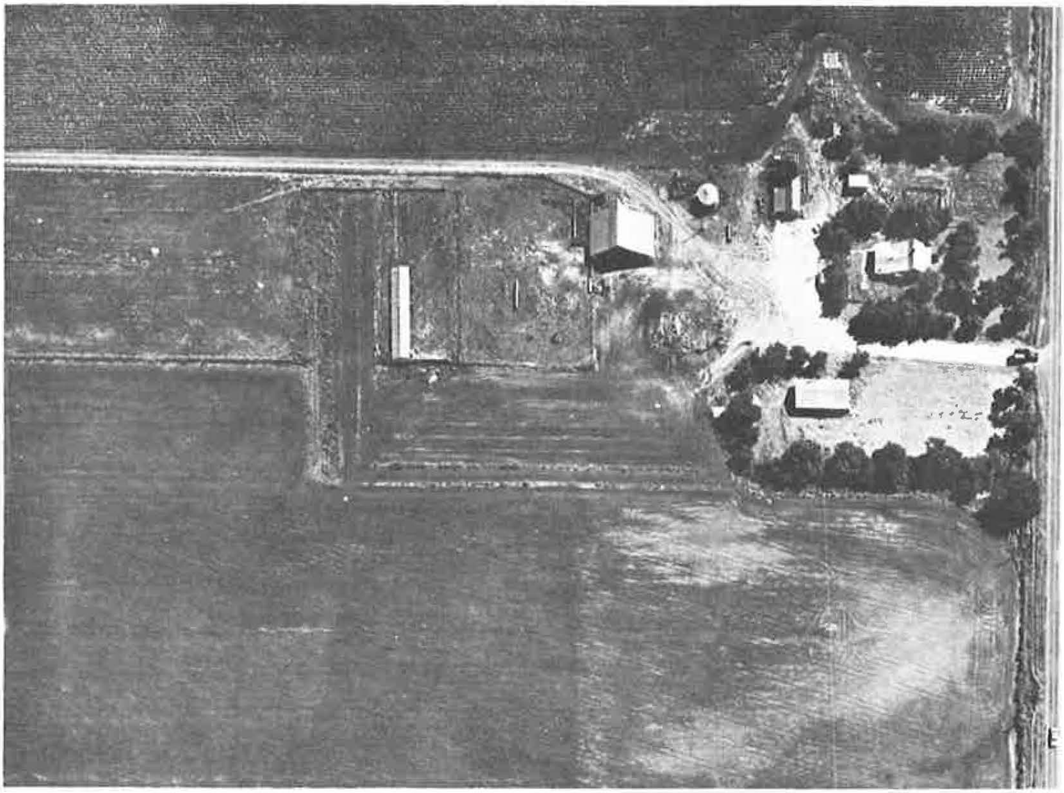


Figure 1.

taking and the date of photography increases the risk of material changes appearing in the premises and increases the risk of the court sustaining objections to the introduction of the photographs into evidence. Some photographs taken four years previously have been used by previous agreement with landowner's attorney.

An aerial photograph may become the very center of the trial. An enlargement of a photograph was described by a local attorney representing the highway commission in his final argument as the most important one item in the trial. An enlargement often is used by both parties and by most witnesses during their testimony. In some counties all exhibits, including the aerial photographs, are delivered to the jury room when the jury retires to deliberate, and in other counties an exhibit is delivered to the jury during deliberation only when that specific exhibit is requested. In the majority of the cases the aerial photograph is requested. Aerial photographs have become familiar enough to the average person serving as a juror to allow him to understand and utilize the information available through visual examination of the aerial photographs.

In his book, "Trial Technique," Irving Goldstein has this to say about the value of exhibits: "A good exhibit will continue to argue the merits of the attorney's cause long after his voice has been stilled. A jury may forget some of the oral testimony, but members of the jury cannot very well overlook or forget the exhibit which serves as an ever present reminder of the truth of testimony contentions."

Under Kansas law, the court exercises its discretion as to whether or not the jury is shown the land in question.

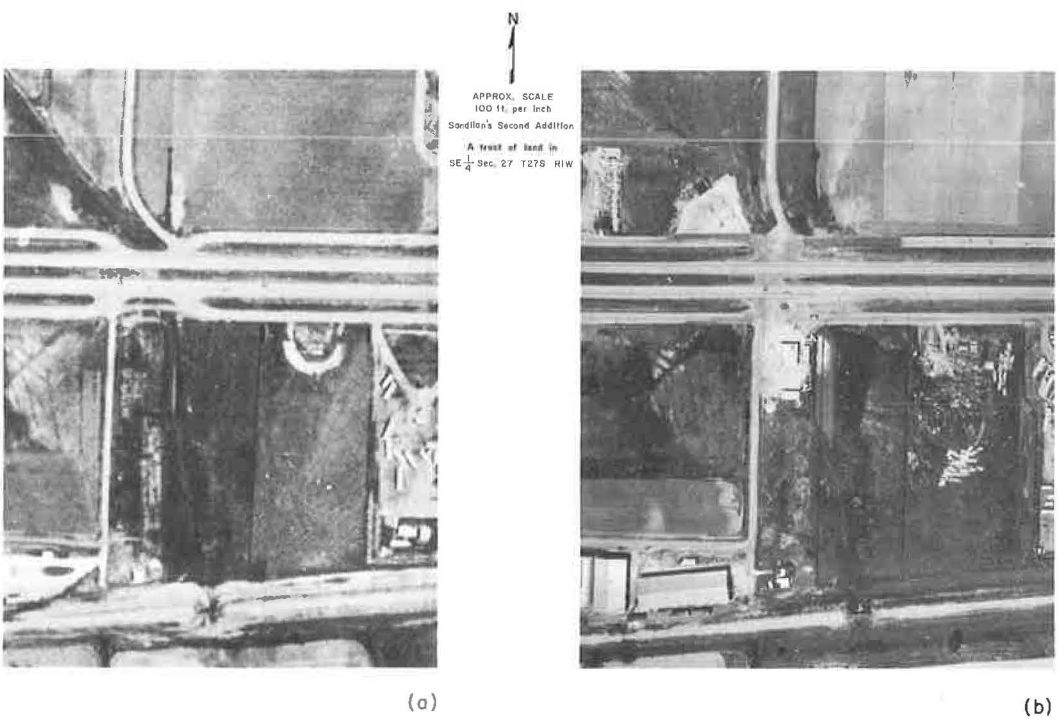
In some counties a view is never permitted, and in some counties the general practice is to show the land to the jury. Where the jury does not view the land, photographs are the only guide a jury has concerning actual conditions, apart from verbal descriptions and opinions expressed by witnesses who are presented by one party or the other.

Where the jury does view the land, the view is generally quite limited and does not detract from the usefulness of the photograph.

At present, Kansas has an inverse condemnation case approaching the trial stage. This case alleges that the highway commission took property and property rights without compensation by highway construction in 1959. This action was filed against the highway commission in 1961, and the hunt for convincing evidence of what occurred in 1959 followed. The Photronics Department was able to locate and obtain 1,000 feet to 1 inch scale contact print of an aerial photograph taken in 1956 for the United States Government. From this print a negative was made, and the portion pertaining to the area in question was enlarged to a scale 100 feet per inch (Fig. 2a). To people who work with photography this enlargement may be something less than a work of art, but to an attorney this enlargement is a thing of beauty, as it is an eyewitness who will not forget, and may even accompany the jury into the jury room during their deliberations. As a comparison, the Photronics Department made a photographic enlargement of the same ground area from a portion of an aerial photograph in its own files, which had been taken in 1960 (Fig. 2b). These enlargements provide a photographic record of "before" and "after" for comparison. The major point to be established is the location of the traveled ways before and after the construction in 1959.

In a condemnation appeal tried before a jury, the court generally instructs the jury that the burden of proof is on the landowner. Nevertheless, it is often thought the jury subconsciously expects the condemnor to pay for an item of damage which is testified to, or prove it does not exist. In other words, it may be the prevailing thought the landowner should not be forced to speculate or stand the risk of any loss. This idea is often forceably argued by landowners' counsel in final argument, where it is emphasized the landowner can never come back into court and sue again, if damage develops later from the improvement beyond any then anticipated. An aerial photograph often will place enough additional information before a jury to reduce speculation.

In what might be called a typical condemnation situation, the Highway Commission of Kansas, as a condemning agency, has a very limited possibility of reducing the origi-



(a) (b)

Figure 2. Representation of exhibit, not to scale.

nal award or even of maintaining it. It only takes an occasional substantial increase, however, to forcefully remind the attorney how essential it is to present all the information he possibly can to the jury

In a recent case, an aerial photograph was available but was not used because the local highway resident engineer professed to regard any aerial photograph or enlargement as "distorted" and declined to verify any photograph or markings on any photograph. The small amount of the original award did not appear to justify the expense or inconvenience to another department to bring an engineer from some distance away to testify. The landowner brought in testimony in excess of $14, 000 compared to a court award of $1, 600 appealed from. The jury awarded $4, 800 additional. It would be pure speculation to claim the presence of a photograph in the court would have reduced the verdict any definite amount. One portion of the land taken and adjacent damage to the remainder, however, could have been shown much more effectively to have been a swampy area. In another case, the sight of an aerial photograph (Fig. 3) on the attorney's table prompted the landowner to revise his testimony concerning his "level farm land" and to describe the terraces accurately. This occurrence is indicative of the respect and weight given any photograph, but more especially, an aerial photograph. In a third instance, a licensed real estate broker testified the land "sloped gently to the south," in support of his valuation testimony. A few minutes later, on cross-examination, this witness was confronted with an enlargement of an aerial photograph (Fig. 4) which clearly showed a drainageway across the tract and several large terraces. Mem-

Figure 3.

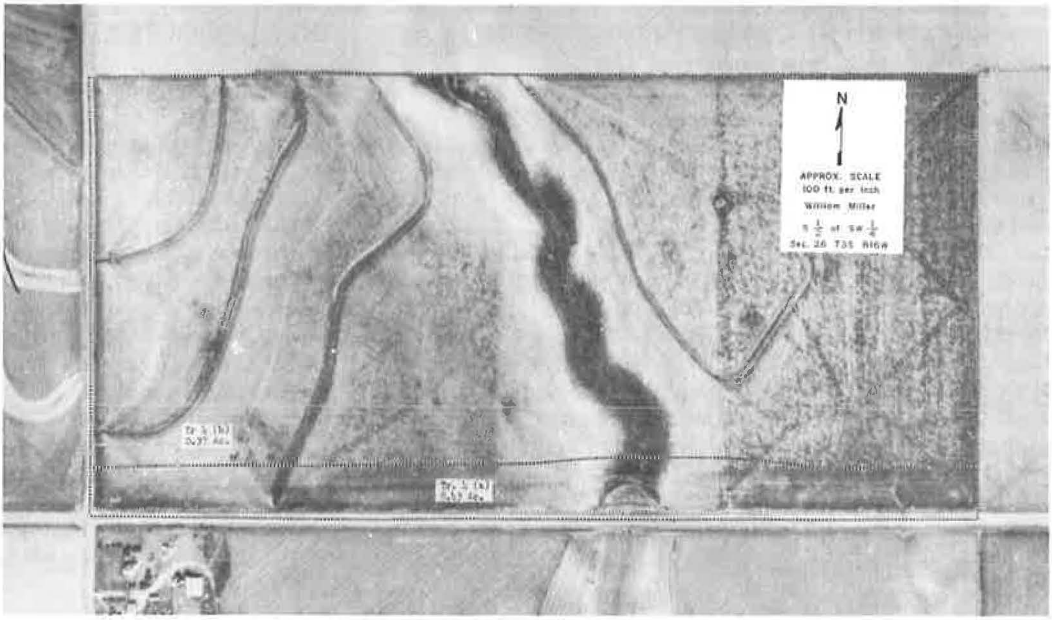


Figure 4. Representation of exhibit, not to scale.

bers of the jury might have seen this topographic condition when they viewed the land, but the jury would view the land at some convenient interval during the trial and possibly would not readily recall the discrepancy, whereas the photograph was instantly available and the contradiction was immediate and apparent.

An aerial photograph or a photographic enlargement of it is not a diagram or map, or a substitute for either one, but is a record of what all the property looked like on the date of photography, and stands as continuing testimony of the appearance of the property each time the jury's attention is directed toward the photograph, both during trial and during the jury's deliberations at the conclusion of the trial. It should be noted a properly taken aerial photograph speaks impartially. The fairest verdict should be one based, however, on the least amount of speculation and guesswork.

The process of properly preparing a photographic enlargement of an aerial photograph and marking it for use as an exhibit is a task requiring extreme care and double checking. A most important consideration is accuracy of the work. It should be stressed this does not mean precision. A photographic enlargement will not be to an exact scale, so delineations on it should be done with great care, supplemented by as much checking by measurements from landmarks to land survey description corners or property and other lines as is feasible.

The photographic enlargements used have ranged in scale from 50 feet per inch to 200 feet per inch, with the most common scale being 100 feet per inch. As a minimum, a square print of 28 inches dimension along each side would cover a quarter section tract of land. A larger print is desirable, as it tends to place the land in question in its proper setting in relation to surrounding properties and landmarks. It should be kept in mind use of the photographic enlargement is primarily for the benefit and information of the jury, and some of the jurors will be 10 to 20 feet away from the enlarged aerial photograph when it is most important that they see it during presentation of opinion testimony concerning the value of the property. Generally, at a trial, the attorney for the landowner attempts to focus all attention on the area taken, whereas the attorney for the condemnor seeks to show the area taken is small in comparison to the owner's entire acreage. This is important because one element of damage to the remainder of the owner's land frequently asserted is the injury to the owner's business operation and

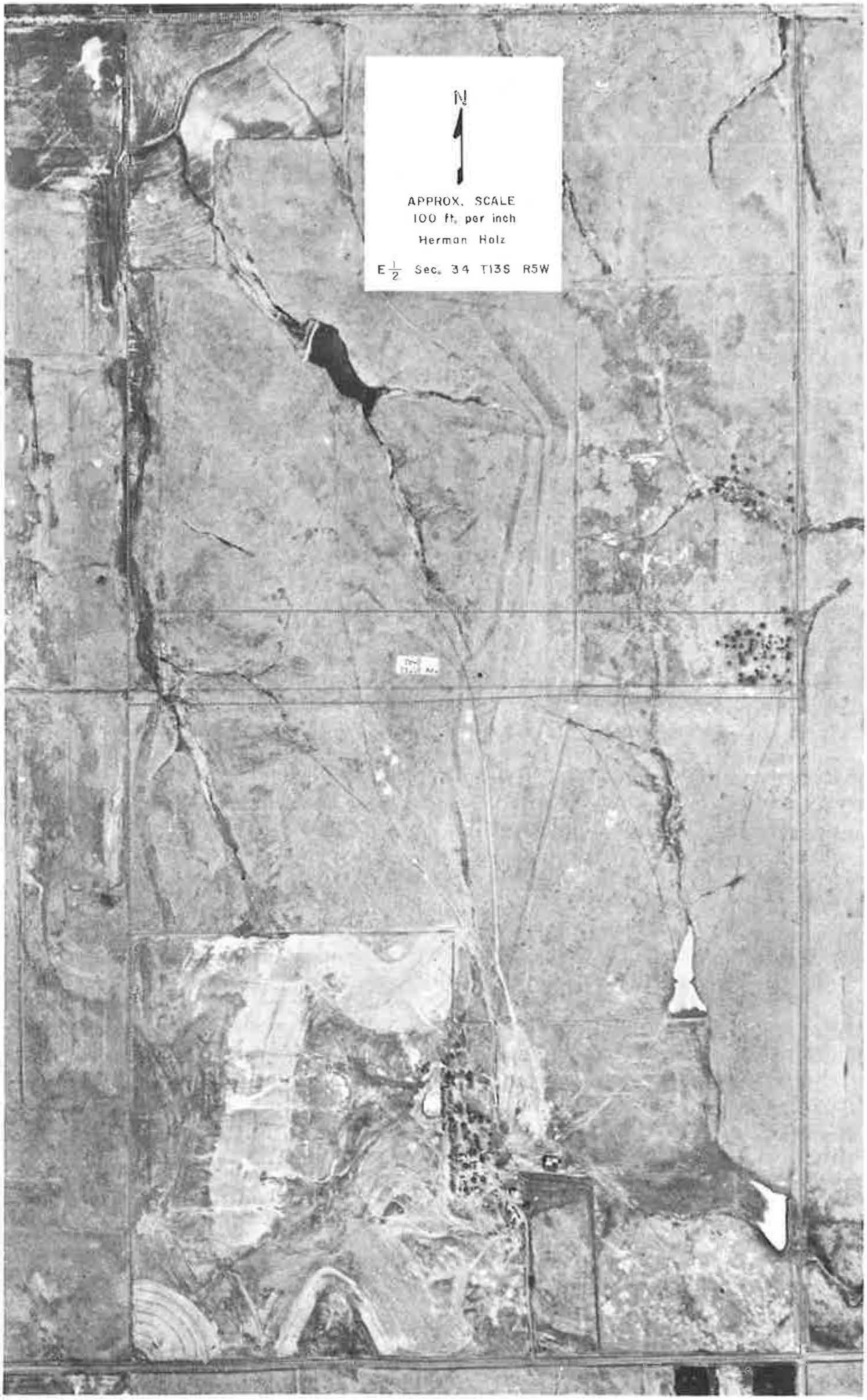


Figure 5. Representation of exhibit, not to scale.

income resulting from the reduction in size of the owner's remaining land. Unless there is a specific object to be illustrated, the aerial photograph or enlargement should show the entire unit serving as the basis for valuation of testimony and for the jury to achieve a verdict.

On a case involving two adjacent quarter sections of land, the Photronics Department printed a photographic enlargement at the scale of 100 feet to 1 inch with the print measuring approximately 30 inches by 60 inches in size (Fig. 5). This photographic enlargement revealed a large portion of the pasture land in question had once been cultivated and later allowed to grow back in native pasture grass, a fact which was not noticed on actual inspection. This type of pasture is referred to as "grow back," and its market value is usually reduced.

To place the area taken in its proper perspective, the boundary of the entire acreage of the owner should be outlined on the photograph exhibited, also the boundary of the area taken should be similarly outlined with colored tape. Commercial tapes in a variety of colors and widths are available. The Photronics Department uses tape $\frac{1}{32}$ in. wide on photographic enlargements printed to the scale of 100 feet per inch. When a tape width of $\frac{1}{64}$ in. was used it became evident the tape was too narrow to be visible to some of the jurors. Yellow or orange tape is most commonly used to outline the boundary of the entire parcel of land ownership, and red is used for indicating the boundary of the areas to be taken. Yellow and orange are thought to be the dominant colors. By use of the dominant color for the entire unit emphasis is placed on the unit in place of spotlighting the area to be taken. The coloring material on this tape sometimes will separate from it after a few months. After the coloring material peals off a white tape is left. Such occurrences have not presented a serious problem because trial exhibits are stored by the clerk of the court after trial and rarely ever unrolled again. Each land area taken is marked by a small typewritten note, for example:

Tr. 3(b)
3.32 Ac

This note is trimmed down as small as possible and is taped within the area taken so nothing of significance on the photograph is obscured. At some location on the photographic enlargement away from the taking, where no landmark is covered, a short identifying description, the scale, and a north directional arrow are affixed.

To avoid as far as possible any grounds for objection by the opposing party only a minimum number of markings should be added. The photograph should be allowed to speak for itself. The identifying number, the photograph used, and the date of photography may be written on the back of the photographic enlargement or omitted.

One chronic problem in use of aerial photographs in jury trials has been introduction of the photographs into evidence. In Kansas, the Highway Commission selects a local attorney for each project in the original condemnation proceeding. All appeals from the condemnation are assigned to the selected local attorney, and the Highway Commission's staff attorney is assigned to assist that attorney. This procedure provides a wide variety in the manner of handling and use of the aerial photographs. In many cases, an aerial photograph is admitted into evidence by agreement among the attorneys. In occasional cases, such admission gives the landowner's attorney the opportunity to put his own exhibit into evidence in exchange for allowing the aerial photograph into evidence, or, as a part of his strategy, simply prevents admission of the photograph or enlargement into evidence. The alternative is to be prepared to call a witness to verify the photograph and the tape delineations on it.

In situations where the one party seeks to place a photographic enlargement in evidence at a trial over the objections of the opposite party, there are two problems to be faced. First, the photograph itself, and second, any markings placed on the photograph must be verified by some witness. What is expected of a witness in verifying a photograph in a trial can best be explained by the following quote from the American Law Reports (9 ALR 2nd, pages 903-904):

General statements in the cases to the effect that the party offering photographs in evidence must show that they are correct and accurate must be taken in a relative sense. It seems to be impossible to produce a photograph which is correct in the minutest details, because there are certain natural limitations on correct representations through photography. For example, if a photograph is taken of two identical automobiles, and the front of one is ten feet from the camera and the front of the other is fifty feet from the camera, the one fifty feet from the camera will appear to be much smaller than the one ten feet from the camera. And there are various inaccuracies or differences in results depending upon the type of equipment and photographic aids employed and the use made of them, which are explained and illustrated by pictures in Part I. of Scott on Photographic Evidence. In view of the practical impossibility of obtaining photographs which perfectly represent their subject, it would seem that when the courts state that one offering photographs in evidence should prove that they are accurate and correct, they really mean that it must be shown merely that the photographs are sufficiently correct to be helpful to the court and jury. In accord with this view it was said, in Hassam v. Safford Lumber Co. (1909) 82 Vt 444, 74 A 197, that photographs must be "properly verified; that is to say, preliminary evidence is required to show that they are sufficiently accurate to be helpful to the jury." It was similarly stated in Leland v. Leonard (1921) 95 Vt 36, 112 A 198, wherein it was conceded that there were certain defects in the taking, developing, and printing of pictures, that all that is required to entitle photographs to admission in evidence is that they be "sufficiently accurate to be of aid to the trier in ascertaining the truth." And in Blake v. Harding (1919) 54 Utah 158, 180 P 172, the court said: "As a matter of course, before a photograph is admissible under the circumstances disclosed in this case, it must be made to appear that it is a true or correct picture or representation of the objects photographed and in question. By that is not meant that it must be shown that the photograph is a true and correct picture or representation of the object photographed in the minutest details, but it must be made to appear that the photograph is a substantially true and correct picture or representation of the objects, and not a distorted or false one." There has not been any complete judicial definition of "verification" in connection with the introduction of photographs in evidence, but much may be inferred from the decisions. Primarily a verification of a photograph consists of evidence that the photograph is substantially correct with respect to the matters concerning which it is offered in evidence, and this includes an "identification" or statement concerning what the photograph shows. See §3, infra. The verification of a photograph sometimes seems to include evidence concerning matters not readily apparent from the photograph, such as additions to or deletions from the original negative, heights and distances, and inherent inaccuracies, distortions, and defects in the photographs; and a verification may include evidence concerning the circumstances under which the picture was taken and the equipment and method used in taking the picture, developing the negative, and printing the photograph.

In some States the test of admissibility of a photograph into evidence is expressed as follows: "A photograph or copy thereof is receivable in evidence when it is shown to look like the person or object sought to be identified." It should be emphasized that a witness in verifying a photograph is expressing an opinion. The witness verifying the photograph generally needs no qualification except sufficient knowledge of the property to recognize it and a photograph of it sufficiently to be able to express his opinion that the

photograph is a fair representation of the land on the date of taking. The foregoing discussion outlines the minimum requirement.

Regardless of the court's requirements on verification, a witness should collect all the background information he can obtain on flight height, who took the photograph, purpose for the photography, who made the photographic enlargement, equipment used, etc., to enable the witness to answer questions from either party. A witness who verifies a photograph or photographic enlargement, and then must answer "I don't know" to elementary questions concerning details seen on the photograph lowers the prestige of both himself and the photograph in the eyes of the court and the jury.

The second problem concerns admittance into evidence of the markings and boundary delineations on the photograph; the lines drawn or taped on an aerial photograph represent property lines and bound the areas taken. Are these lines to be considered as a scale drawing superimposed on an aerial photographic enlargement or are they more properly considered as only illustrative markings on a photograph? Maybe the answer is a bit of both. From the standpoint of insuring the photograph or its photographic enlargement will be admitted in evidence, the safer approach seems to be to consider the taped property lines as "markings" or "boundary delinations."

Any person with sufficient knowledge to testify the markings or boundary delineations are reasonably accurate and fairly represent the property lines and outline the area taken can verify the markings and delineations on the witness stand. This is the opinion of the witness. It is perhaps more impressive to have the engineer in charge of the project as the verifying witness.

Immediately following the verification, helpful identification and explanation of the existing roads, improvements, landmarks, areas taken, etc., can be brought into the case by testimony of the verifying witness. The more familiar a verifying witness is with the subject property, at the date of taking, the more effective his testimony is.

Another theory in respect to the use of photographs in evidence has been advanced. This theory is that a photograph, markings and delineations on a photograph, diagrams, plats, etc., are only a method by which the witness testifies and illustrates his testimony, and that the exhibit introduced is a part of his testimony. The end result seems to be the same inasmuch as the photograph is placed before the jury during the verifying testimony of the witness and is shown to the jury at various times after the witness leaves the stand.

Most of the recorded court cases, and the total is small, concerning aerial photographs place them in the same category as an ordinary photograph taken from a position on the ground or from an object or building supported by the ground. One aerial photograph, either admitted or excluded as evidence would rarely appear important enough to furnish the sole basis for an appeal case; therefore, most cases concerning aerial photography treat it as only one point among several to consider on appeal.

Effective January 1, 1964, the State of Kansas has adopted both a new code of civil procedure and a new eminent domain code. It is anticipated admission of aerial photographs into evidence will be easier under the new code. It does not appear that enlargements of aerial photographs will be any less useful under the new laws.

The courts have been aware of the impact of photographs upon jurors and have referred to them as "dumb witnesses," "mute witnesses," and to what appeared on the photograph as "unvarnished testimony." The thought has been expressed that people are likely to accept any photograph as completely accurate.

It seems the maximum potential use of aerial photographs and photographic enlargements has not, as yet, been realized. As recently as 1956, the Kansas Supreme Court had this to say concerning photography. "Photography is recognized more and more by the courts as being helpful in presenting facts." With added experience, it is felt witnesses, judges, jurors, engineers, and attorneys will rely on aerial photographs more and more.

Comparison of Two Techniques of Aerial Photography for Application in Freeway Traffic Operations Studies

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•EACH YEAR it is becoming more apparent that many sections of urban freeways will have to be controlled during hours of peak traffic. The need for knowledge on how, when, and where to control traffic has resulted in development and installation of several types of surveillance systems to monitor traffic operations on freeways. Electronic sensing and automatic control devices provide the best solution, but equipment now available cannot accurately describe traffic operation over long sections of freeways. It is doubtful that suitable systems will be developed until a more complete criterion for the operation of a freeway system has been established. Television monitoring systems, now in operation in several locations, are providing much of the needed data, but not all study sections can, nor should be, equipped with complete television coverage. It is for such a reason that aerial surveys, which approximate the coverage provided by continuous television monitoring, were used to study traffic movement on the Gulf Freeway in Houston.

The use of aerial photography in making traffic surveys is not a recent innovation. Reports of the use of aerial photography were available as early as 1927 (1). Serious consideration has not been given to this survey method, however, because of two factors: cost of the field survey, and the difficult and time-consuming task of data reduction.

The same objections were raised when proposals to study freeway operation by motion picture film from ground locations were being considered. But the results of these film studies have provided a better understanding of the traffic movement characteristics, improved designs of entrance and exit ramps on the freeways, and improved signalization of the intersections within the interchanges.

The taking and use of motion film photography from ground locations, however, have limitations which must be considered. Numerous studies of freeway operation have documented the fact that a small disturbance at one location on the freeway, during the time when a peak number of moving vehicles is on the freeway, can create complete stoppages at some distance preceding the site of the disturbance. Therefore, a study which is limited to a small section of a freeway will reflect the change in operation without recording a cause to relate to it. The use of aerial photography permits the expansion of the study area not only along the freeway but also laterally to cover the frontage road or roads and supporting street system.

Objectives of the Survey

Aerial surveys were made of traffic on a 6-mi section of the Gulf Freeway in Houston (Fig. 1). Two types of aerial photography were investigated: (a) strip photography comprising two continuous stereoscopic photographs taken simultaneously from the beginning to the end of the traffic survey section, and (b) time-lapse photography where individual fixed-size photographs are taken at short intervals of time (Figs. 2 and 3).

Paper sponsored by Committee on Photogrammetry and Aerial Surveys.

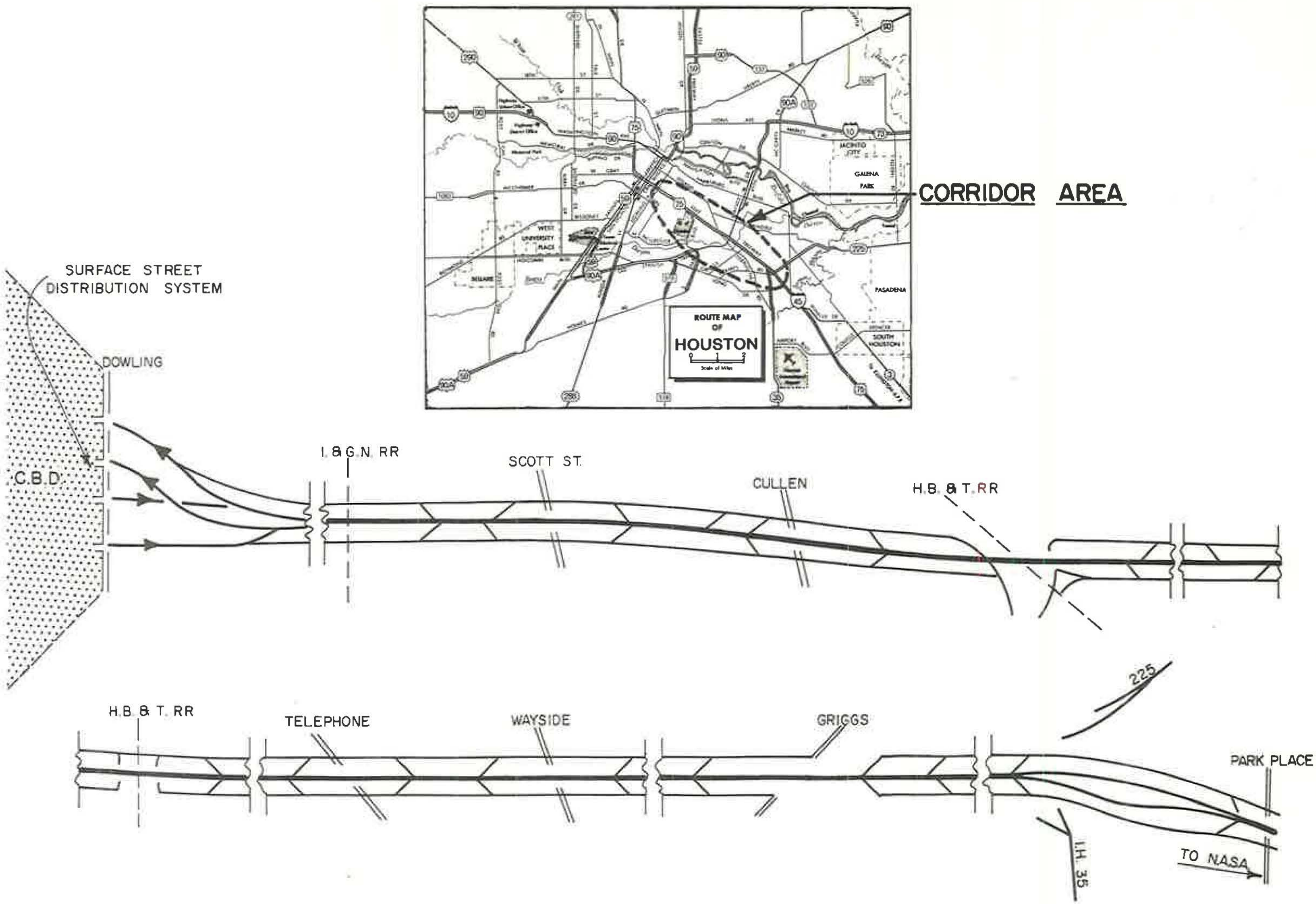


Figure 1. Study area—Gulf Freeway, Houston, Texas.

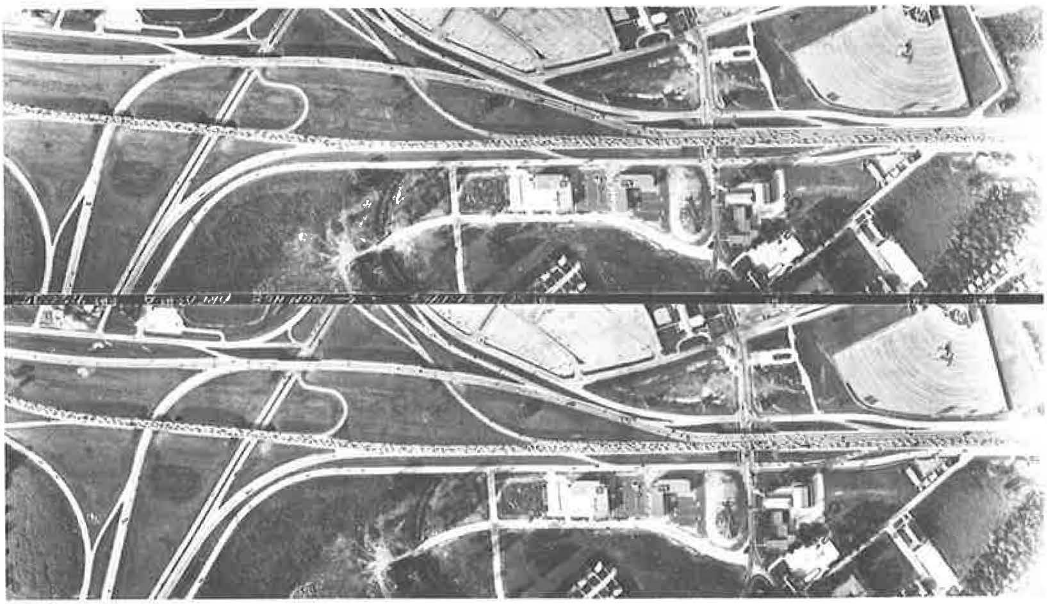


Figure 2. Continuous-strip stereoscopic photography.

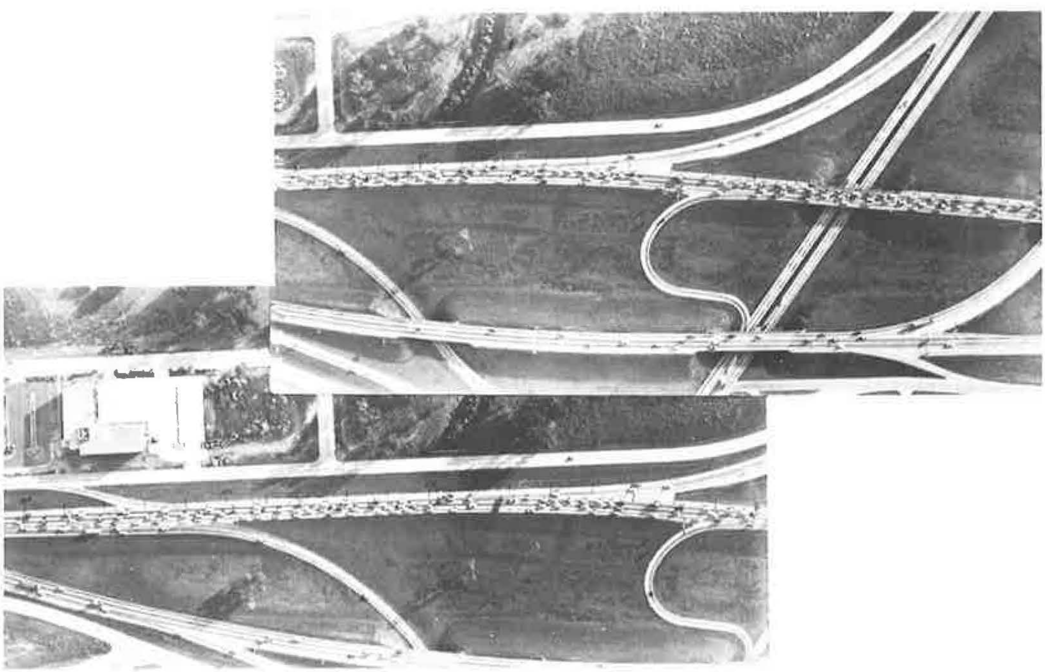


Figure 3. Time-lapse photography.

The objectives of this survey and study were (a) to determine operational characteristics of traffic on the freeway and factors which affect the level of service offered to the motorist, and (b) to evaluate two techniques of aerial photography and their applications in making traffic surveys and studying traffic movement.

The purpose of this report is to compare the two types of aerial photography used for their application to surveys for traffic study purposes. It is very difficult to evaluate the performance or applicability of these two aerial photographic techniques from this limited study because many of the disadvantages experienced in this study can be eliminated with more detailed planning and advance preparation of the test area based on the experience gained in this work. This paper reports the results of the survey made for the studies accomplished and the changes needed for making future studies of traffic operations.

Previous Work

Only in the last two years have any extensive studies of traffic operations by aerial photography been undertaken, although many experiments have been reported (2—4). Wohl (5) presented the application of strip photography for traffic studies and the relationships of speed, number of vehicles, and density as measured from the film. The fundamental principles for the continuous-strip stereoscopic photography are covered extensively in Wohl's paper and, with the exception of Figures 6 through 8 are not included in this report.

Wagner and May (6) reported the results of a density study of freeways in California using time-lapse photography. A significant development in this paper is the method of presenting density conditions on the freeway by a time-distance-density contour map.

Howes (7) reported an extensive study in the application of aerial photography to the collection and analysis of highway traffic flow data. Results from using time-lapse photography were compared to those obtained by employing conventional on-the-ground techniques. Analyses of the cost and procedure of data reduction and the accuracies of the results were made and included continuous-strip aerial photography, although no surveys were made using this technique.

There have been other reports on the application of aerial photography to studying traffic operations (8) and freeway design (9), but these studies are primarily concerned with some form of road or equipment inventory.

PROCEDURES

Description of Field Survey

The section of the Gulf Freeway in the traffic survey area extends from the Reveille Interchange at the intersection of US 75, Tex. 225, and Tex. 35, to the downtown distribution system at Dowling Street (Fig. 1). The photography flights were made in the morning peak period from 6:30 to 8 a.m. for two weekdays in September.

Two airplanes were used for obtaining film negative exposures of the aerial photography; one equipped with a 24-in. focal length camera for taking time-lapse photographs and the other equipped with a 4-in. focal length camera for taking the continuous-strip photographs (Figs. 4 and 5). Both airplanes were fixed wing Cessna 195's.

The photography flight plans required the two airplanes to be separated by at least a two-minute interval. The continuous strip photographs were taken from a flight height of 1,000 feet above the ground and the time-lapse photographs were from a flight height of 2,500 feet. Only one photographic flight was made each day in the outbound direction against the peak traffic flow. All other aerial photography flights were made in the direction of peak traffic flow. This is an important consideration in the design of photography flights for traffic survey purposes for the following reasons:

1. Time-lapse photographs will contain the images of a larger number of different vehicles, but will require fewer measurements per vehicle when photography flights are made against the flow of traffic.

2. Continuous-strip photography will have a larger number of different vehicles and the correction for density will be negative, that is, true density is less than vehicle spacing seen on the photography. This is caused by time lag, which is the length of time required to take a strip of such photography. Also, the size of the image of

Figure 4. Time-lapse camera.

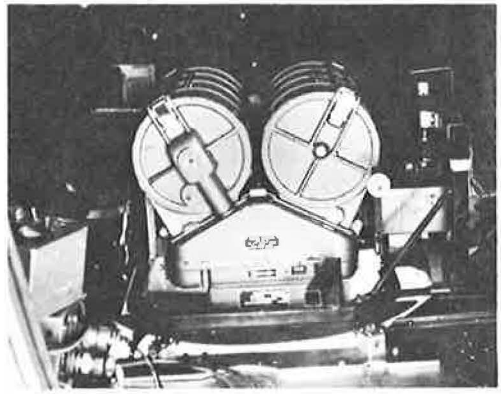


Figure 5. Continuous-strip camera.

the vehicle will be shortened. This is especially important in attempting to identify the vehicles by type or by markings.

During the photography flights by the aircraft, counts of the traffic were made by ground observers at several locations on the freeway lanes and ramps. Control vehicles with distinguishing markings on their roofs made travel time runs. Two of these vehicles were equipped with speed recording equipment for tracing speed profiles.

There was no ground-to-air communication during the time photography was being taken. A time reference was established by synchronizing watches one-half hour before the start of each photographic flight.

Analyses of Photography

The purpose of this type of traffic surveying was to determine whether or not aerial photography is practical to use for making freeway operation studies. In the design of the data reduction procedure all information which could be of any value in traffic studies was obtained by examination of the photographs. A time reference and the location of each vehicle by station number were recorded each time a vehicle was photographed. The location by lane, direction of travel, and type of vehicle were also recorded at the same time. Computer programs were developed to use these basic data and compute desired traffic movement characteristics such as speed, number of vehicles, density, headways, etc. The methods used in reducing the data from the photography and the accuracy of measurements are discussed in other sections of this report.

The important feature of the data reduction procedure was determining the location of each vehicle by roadway station numbers. These numbers were very difficult and costly to obtain, but more detailed analyses by electronic computers were then possible for specified sections of roadway, certain numbered groups of vehicles, or for each individual vehicle.

Improvement of Procedure

Many problems were encountered in this first traffic survey which can be eliminated in subsequent projects. One difficulty over which there was no control, however, was the weather. Aircraft flying at low flight heights over populated areas near the Houston International Airport required a minimum visibility of five miles which was difficult to obtain because of smoke and haze. The flights were delayed three to four days.

Communication between ground and air units is advisable. In some locations it is possible to use the control tower at a nearby airfield to provide the communication link between the airplane and the traffic survey area.

Reference points on the ground should be established at intervals which are short. Beginning and ending markers and two sets of intermediate reference points were used, and station numbers were determined from construction plans of the freeway. Each photography flight had to be numbered separately. Transferring these station numbers to the photographs is a time-consuming task subject to errors which can be eliminated by placing station markers on the pavement before the traffic survey photography is taken.

Reduction of traffic data from the aerial photography is still a time-consuming task. The work can be alleviated by obtaining photographs at scales not smaller than 200 feet to one inch, using magnifying lenses while making essential measurements, and using a recording procedure which will give working relief at specified intervals to the persons using the photographs.

The problem of traffic data reduction from the photography has been lessened by the use of electronic computer programs and a reduction procedure which records at the same time all basic information needed to make all anticipated analyses.

ANALYSIS OF THE TWO AERIAL PHOTOGRAPHIC TECHNIQUES

One of the primary objectives of this traffic survey was to develop the best procedure for obtaining traffic data from the aerial time-lapse photography and from aerial continuous-strip photography. It was further stipulated in development of the survey techniques that all traffic data available from the photography should be recorded.

The procedure adopted required basic measurements to determine vehicle location. An electronic computer program was written to calculate all traffic movement characteristics. It is only possible to compare the two photographic techniques in terms of the measurements made, resulting cost of the reduction, and accuracy of data obtained. From these traffic survey examples, it was not possible to determine which method required fewer man-hours to determine only vehicle speeds, space headways, or some other flow characteristic. A survey requiring only one or two of these parameters to be measured would probably use a different data reduction procedure.

This procedure, which records all data in a form for calculation and processing by electronic computers, permits faster and more complete analyses of the aerial photographs. This compensates to some degree for the slow process of making essential measurements by use of aerial photography.

Method of Data Reduction

Description of Methods. — To determine all traffic movement characteristics available from aerial photography, there are only two parameters to be measured: The location of each vehicle at some known time, t, and the location of each vehicle at some known time, $t + \Delta t$. This information determines the speed of the vehicles from which other traffic characteristics, such as number of vehicles, density, headways, etc., can be calculated. To obtain a complete analysis of the data available from the aerial photographs, additional information regarding lateral location of each vehicle, type of vehicle, and time of day are recorded.

Strip Photography. —There are two ways of determining vehicular speeds from continuous-strip photography. The conventional method is to measure the distance between two images of the vehicle and divide by the time lag between photographs. The other method is to measure the elongation of the image of vehicles due to the exposure time. This is a function of the vehicle speed relative to the known speed of the aircraft.

Both of these methods were investigated and the displacement method was selected. The vehicle elongation procedure required an accurate measurement of the length of the vehicle image. This required determining of the scale of the photography. Also, the true length of each vehicle measured had to be known. Because vehicle images were blurred and the photography scale was large, it was difficult to determine an exact identification for vehicle and thus apply its actual length in measuring the elongation. Figures 6 through 8 present the time-distance relationships of the continuous-

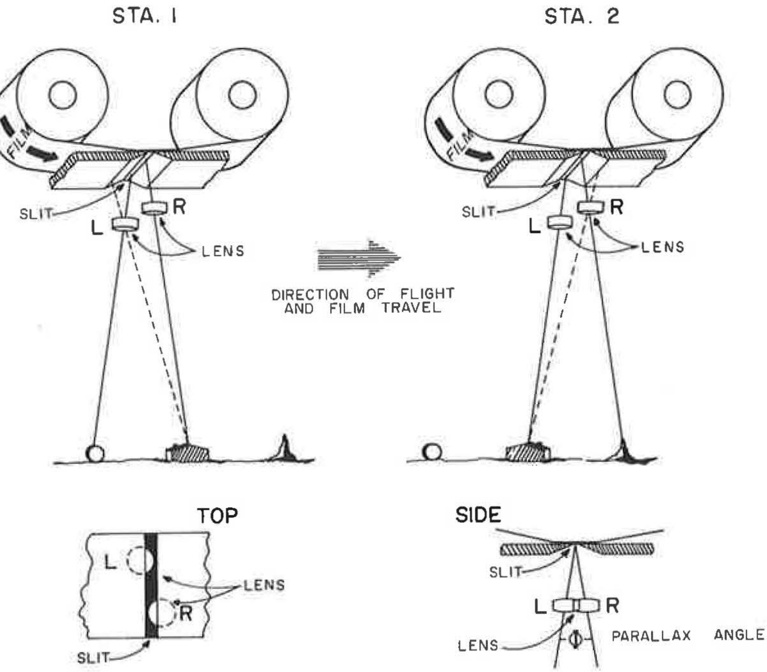


Figure 6. Plan and side views of continuous-strip aerial camera, with two stereoscopic lenses.

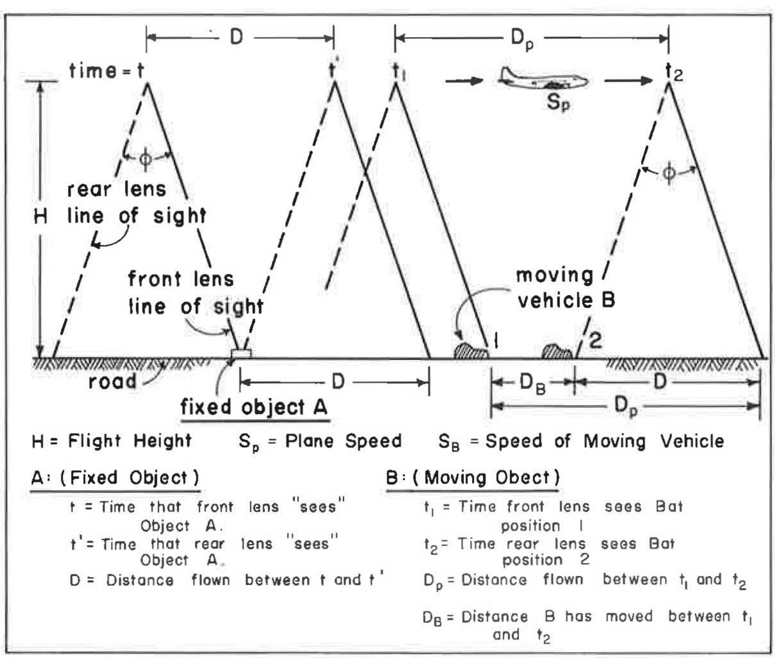


Figure 7. Time-distance relationships, continuous-strip photography.

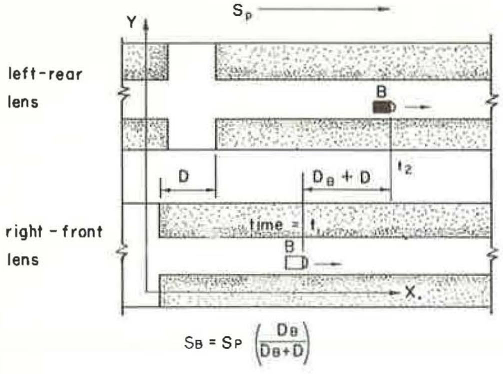
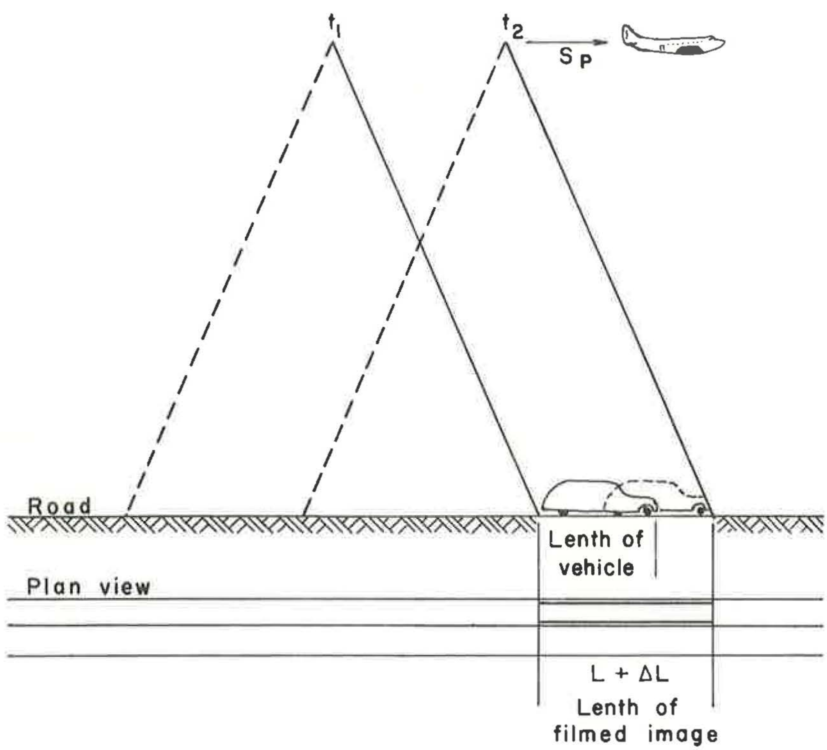


Figure 8. Time-distance relationships, continuous-strip photography.

strip photography. The precision of the speed measurements was a direct function of the precision of the displacement measurements. Therefore, several techniques were investigated for making these measurements. Figure 9 illustrates the calculation of speed using measurement of the vehicle image.

A stereoscopic viewer, capable of making displacement measurements to $\frac{1}{1,000}$ inch was considered. This equipment, which relies on stereoscopic depth perception of the user, should be used by personnel with extensive training in its operation. This method was rejected because precision of measurements could not justify cost of training instrument users and renting the equipment.



$$t_2 - t_1 = \frac{L + \Delta L}{S_P}$$

$$\Delta L = (t_2 - t_1)(S_B)$$

$$S_B = \frac{\Delta L}{L + \Delta L}(S_P)$$

S_P = Plane speed
S_B = Speed of moving vehicle
L = True lenth of vehicle
$L + \Delta L$ = Lenth of vehicle image on continuous strip film

Figure 9. Time-distance relation of strip photography.

The use of the stereoscopic view on a contractural basis was also investigated, but the cost of data reduction was too high and the information received was not as complete as needed; that is, lane usage, location with respect to stationing, and number of vehicles in the survey area, were not included in the data reduction procedure.

The displacement method, used by our staff, requires measurement of the vehicle displacement with an offset scale.

Because the scale was small, photographic enlargements were made of one strip of photographs to determine what improvements could be expected in the accuracy of measurements. Because the photography strip used was a positive transparency, contact prints made directly therefrom are negative in appearance.

This is not a serious problem, but it induces some discomfort to the person who is examining the photography for long periods of time. Preparation of a film negative from which to make positive prints increases the cost of photographic enlargements by 25 percent.

The maximum photographic enlargement which could be processed by use of the equipment available is three to one. The scale would be approximately 100 feet to 1 inch. This method was rejected because the cost of making the enlargements could not justify the increase in accuracy, and there was no significant decrease in the data reduction costs.

The technique selected was measurement of data directly from the photography film transparencies. Figure 10 shows the light tables constructed for this work. A plexiglass illuminating frame was developed for making displacement measurements. The offset scale etched in the plexiglass was viewed through a magnifying glass. To make displacement measurements a reference mark on the glass was positioned over a vehicle in the lower film strip. The frame was positioned perpendicular to the film border. The displacement was measured at the location of the vehicle on the scale in the upper film strip. Measurements were made to $^1/_{100}$ inch.

<u>Time-Lapse Photography.</u> — Vehicular speeds, headways, and densities were calculated from measurement of ground displacement of the vehicles from one photograph to another photograph, exposed at known time intervals (Fig. 11). All other time-dependent parameters were obtained from these measurements. The time interval between exposures in time-lapse photography varied from one photography flight to another and had a range of from three to four seconds. Variation in the time interval within one photographic flight was ±0.1 second.

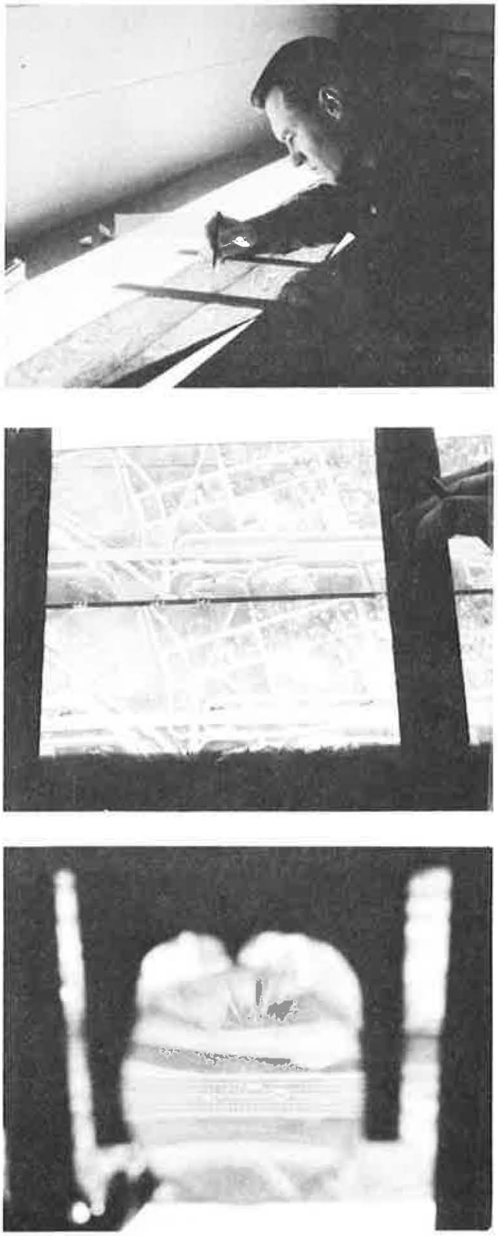


Figure 10. Data reduction from continuous-strip photography.

Figure 11. Data reduction of time-lapse photography.

PRECISION OF MEASUREMENTS

Continuous-Strip Photography

Two measurements were made for each vehicle and are represented in Figure 8. Photography displacement, D, is measured on the film positive in inches. The vehicle dimension plus its photographic displacement, $D_B + D$, are dimensions measured from the film positive in inches.

Speed is calculated from the equation:

$$\text{Vehicle speed} = \text{place speed}\left[\frac{\text{vehicle displacement}}{\text{vehicle plus photographic displacement}}\right] \quad (1)$$

The accuracy of the vehicular speeds and all traffic movement characteristics dependent on speed are influenced by the precision of these measurements and by other factors subsequently explained.

Variation in Observers. —Equipment used in making measurements limited recording of displacement measurements to the nearest $\frac{1}{100}$ inch. The measurement precision depends somewhat on the instrument user's judgment. By comparing measurements made by two different recorders, it was evident that the difference between observers is negligible. The average difference and standard deviation for two observers were approximately the same as for two measurements by the one observer.

Limited Accuracy of Equipment. —Limitations of the equipment made it necessary to estimate measurements to the nearest $\frac{1}{100}$ inch. Figure 12 illustrates the error in vehicle speed resulting from errors in measurements. Low speeds of 20 miles per hour or less have considerable error, in the range of 10 to 15 percent, and can be attributed to the limited accuracy in measurements.

Variation in Scale. —As in all types of aerial photographs, there is difficulty in obtaining a uniform scale on the continuous-strip film. Variations in the scale, however, have no effect on vehicular speeds because the scale of the film does not appear in the calculations.

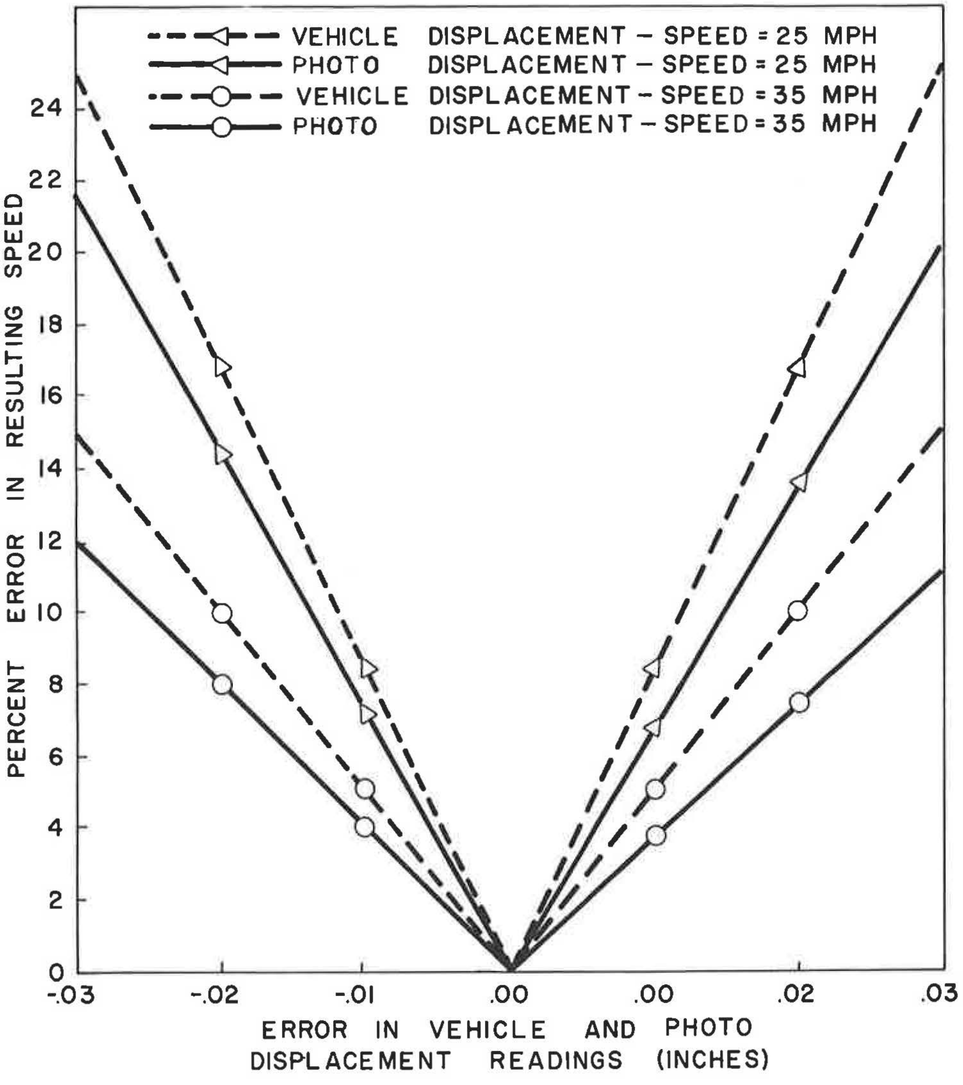


Figure 12. Errors in speeds resulting from errors in readings.

Space headways and vehicle densities cannot be measured directly because of a difference in time between the photographing of two vehicles. The correction necessary to position the vehicles in their true relative positions is small for high density conditions. The variation in scale must be considered in determining the original location of each vehicle as it appears on the photographic film positive.

Variation in Airplane Speed. —The airplane speed was calculated by dividing the length of the elapsed time during the photographic flight. This average speed was assumed to be constant throughout any one flight. This assumption is valid because the airplane speed was fast and the variations would only be a very small percentage of the true speed. The vehicle speed calculated from the continuous-strip photograph is obtained by equating the ratio of vehicle movement to airplane movement to the ratio of vehicle speed to airplane speed. Therefore, an error in airplane speed would result in the same percentage error in vehicle speed.

Variation in Flight Height. —The angle between the two camera lenses remains constant, so the only condition which would result in a change in photographic dis-

placement of images is a change in flight height. This is one of the most difficult conditions to control and, as a result, the photographic displacement varies constantly.

It was impractical to make a photographic displacement measurement for every vehicle. Measurements were made at every 100-ft station along the roadway, and the photographic displacement for each vehicle was obtained from a straight line interpolation of the displacement measurements. Using the station numbers as reference points, the same photographic displacement was used for all three freeway lanes.

Measurements of vehicle displacement caused by its movement and photographic displacement caused by tilt were made directly on the film positive transparencies with a scale etched in a plexiglass overlay. The measurements were made to an accuracy of $\frac{1}{1,000}$ inch.

The accuracy of these measurements was checked by selecting 100 images of "test" vehicles at random and sending them to the contracted aerial survey company for processing with the stereoscopic viewer. Data were measured to an increment of $\frac{1}{1,000}$ inch. Three members of the traffic survey project staff made measurements for the same vehicle using the plexiglass overlay.

The data from the survey company were assumed to be correct and were compared to the measurements made by use of the overlay (Table 1). Figures 13 through 15 indicate results of the three test observers were consistently larger than from measurements made by the aerial company. The fact that speed differences were not distributed around a mean difference of zero indicated a bias in the measuring technique, or testing procedure.

A check of the accuracy of measurements made by the aerial survey company was obtained using a second aerial photography sample of 100 vehicles. Fifty vehicles in the sample were part of the vehicles in the original set. The measurements obtained for the fifty duplicated vehicles were different from the original (Table 2).

Because these differences occurred, there was no basis with which to determine the accuracy of the test observers' measurements. A comparison was made, however, which indicated accuracy of the test observers' measurements relative to the accuracy of the measurements supplied by the aerial survey company.

For this comparison, only the measurements of the two test observers, which were employed in the actual data reduction, were used. One of these two observers was asked to make a second set of measurements for the fifty vehicles, as this comparison was based only on the fifty vehicles for which there were duplicate measurements.

TABLE 1

SUMMARY OF DIFFERENCES BETWEEN AERIAL SURVEY COMPANY AND TEST OBSERVERS

Observer	Veh. Speed (mph)	Veh. Displacement (in.)	Photographic Displacement (in.)
		Average Difference	
1	2.7	0.006	0.040
2	1.7	0.002	0.037
3	1.7	0.001	0.036
		Standard Deviation	
1	4.4	0.020	0.021
2	7.8	0.039	0.022
3	5.0	0.025	0.020

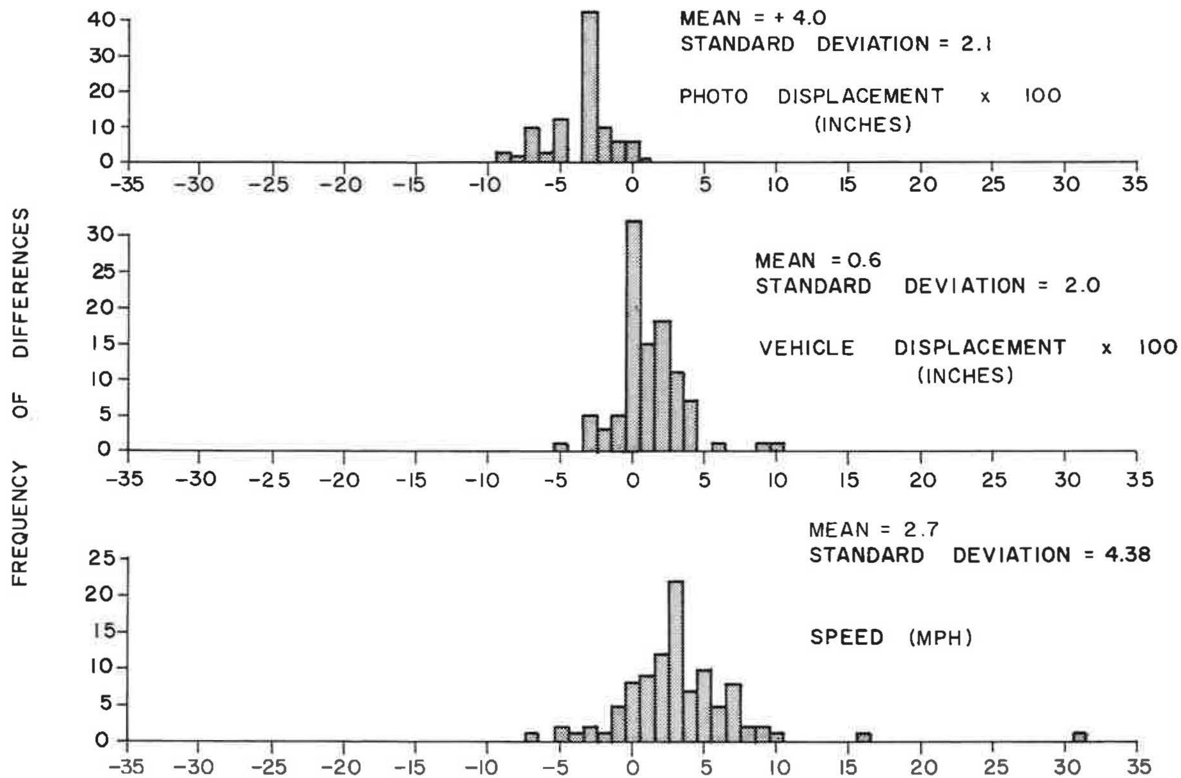


Figure 13. Distribution of differences between Test Observer 1 and aerial survey recorder.

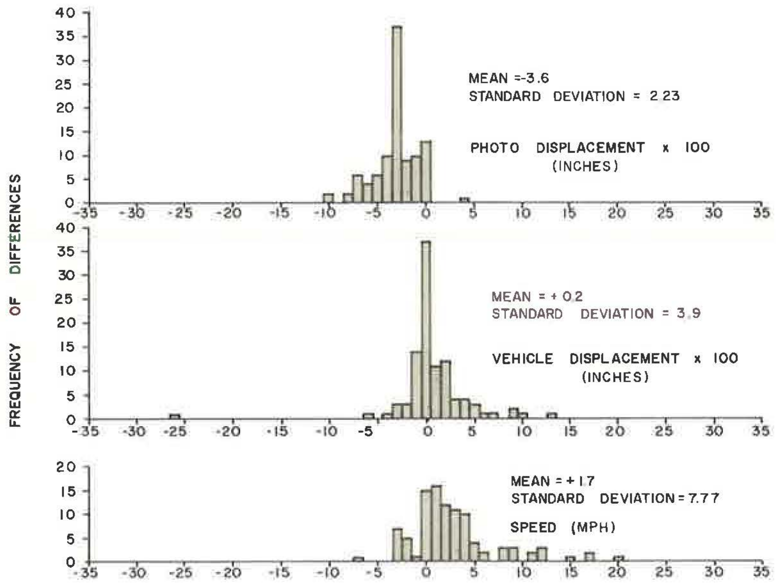


Figure 14. Distribution of differences between Test Observer 2 and aerial survey recorder.

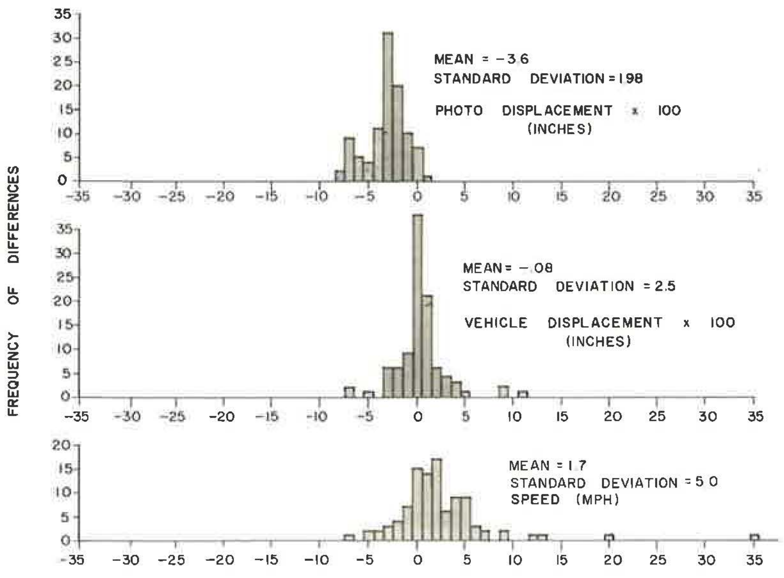


Figure 15. Distribution of differences between Test Observer 3 and aerial survey recorder.

TABLE 2

COMPARISON OF OBSERVERS

Case	Veh. Speed (mph)	Veh. Displacement (in.)	Photographic Displacement (in.)
		Mean Difference	
A[1]	-0.03	-0.002	-0.009
B[2]	0.73	0.006	0.001
C[3]	-0.85	-0.007	-0.005
		Standard Deviation	
A[1]	2.33	0.014	0.014
B[2]	4.24	0.026	0.021
C[3]	4.78	0.026	0.022

[1] Difference between two aerial survey company measurements.
[2] Difference between two test observers' measurements.
[3] Difference between two measurements of Observer 1.

The differences between the two separate measurements by the aerial survey company, the two measurements by Observer No. 1, and the measurements of Observer No. 2, were compared and are summarized in Table 2.

Table 2 gives an indication of the degree of reproducibility of the measurements. The results of the two observers are about the same as the results of two measurements by the same observer. The mean differences in speed determined from measurements by the aerial survey company are considerably less than the mean differences in speed determined from measurements by the test observers. The standard deviation of speed differences, which is the measure of scatter around the mean, was found to be in the range of four to five miles per hour for the comparisons involving test observers. The standard deviation of speed differences obtained from measurements by the stereoscopic viewer was found to be 2.3 mph.

A fundamental theorem of statistics states: If two distributions are normal, then the sums of differences of these distributions are normally distributed. Therefore, the distribution of these differences can be expected to follow a normal distribution. It could then be expected that about 70 percent of the differences will be within one standard deviation of the mean difference. The mean difference is less than 1 mph in all three comparisons.

To give an indication of the type of data used in this comparison, the speeds of the fifty test vehicles had an average of 34.7 mph, within a range of 52.6 mph, from a maximum speed of 59.7 mph to a minimum speed of 7.1 mph.

Time-Lapse Photography

The data measured from the time-lapse photographs were subject to the same factors affecting accuracy of measurements from the continuous-strip film photography. Measurements of vehicle displacement were made to the nearest $\frac{1}{100}$ inch with an engineers' scale. The time interval between photographs had an accuracy of ± 0.1 second. The maximum effect of these two measurements on the determination of vehicular speeds would be ± 3 mph at 70 mph to ± 1 mph at 15 mph.

The photographs used were ratio printed to the same scale so variations in flight height did not affect their scale.

The only major problem in the data reduction was the transfer and identification of reference points on different photographs. The movement of a vehicle was measured from one distinguishing roadway feature. If this reference point failed to appear in all

photographs in which the vehicle was imaged, a new reference point was established. Frequently, the third or fourth vehicle position was located by measuring from the wrong reference marker. The electronic computer programs noted large changes in vehicle speed, which pinpointed these errors.

Measurements from these reference markers were made using an average scale for the photographs. There was image displacement at the edges of the photographs but errors in distance it caused were less than two percent.

ECONOMIC COMPARISONS

Cost of Field Survey Work

It is the general practice of aerial survey companies to negotiate contracts. Consequently, the cost of a traffic survey, such as the one accomplished for the freeway in Houston will depend on many factors. Among these are the length of the survey project and its location in relation to a base of operations, the time of the year, and the availability of equipment. The contracts for the Houston survey contained the following provisions:

> Strip Photography.—Beginning at 6:30 a.m., the airplane shall make as many photographic flights as possible until 8 a.m. This schedule is to be repeated as soon as possible. This plan resulted in 22 flights being made over the same 5-mi section of the freeway. One positive film transparency for each flight at a scale between 200 and 300 feet per inch was obtained at a total cost of $2,500.
>
> Time-Lapse Photography.—Beginning at 6:30 a.m., the airplane shall begin making the prescribed photographic flights. A total of nine flights shall be completed before 8 a.m. One set of scale ratioed photographic prints at a scale of 100 feet to one inch for each flight was provided at a cost of $2,300.

First indications were that the continuous-strip film transparencies provided much more photography coverage for the money expended. There are other factors, however, which must be considered. Table 3 contains some statistics regarding the two surveys.

On the 22 continuous photography film strips 13,774 vehicles were photographed. Only a few vehicles were imaged on two consecutive photographic flights, so most of the 13,774 measurements for speed determination pertained to different vehicles.

Nine of the continuous-strip photographs were taken at the same time the time-lapse photographs were taken. Therefore, the same number of vehicles was studied by use of photographs taken on these flights. The time-lapse photography has the added feature of tracking vehicles for several seconds. Most of the vehicles were imaged in three consecutive photographs, and a few of the vehicles traveling at fast speeds were image recorded in four or five. This represented a time interval of twelve to fifteen seconds during which speed changes could be determined.

Table 4 indicates the cost per mile for the aerial surveys. The continuous-strip photography provided the least cost, considering the length of roadway covered by the airplane. The unit cost per length of roadway, however, as measured from the pho-

TABLE 3

COMPARATIVE STATISTICS

Photography	Photographic Flights (No.)	Flight Height (ft)	Flying Time (min)	Photography Time (min)	Cost of Taking Photographs ($)	Veh. Observed (No.)	Measurements Made to Determine Veh. Speed (No.)
Time lapse	9	2,500	55	26	2,300	7,186	15,700
Continuous stereoscopic photography	22	1,000	175	50	2,500	13,774	13,774

TABLE 4

COMPARISON OF COST/MILE FOR AERIAL PHOTOGRAPHY

Photography	Overlap (%)	Scale (ft/in.)	Cost per Freeway Mile Photographed ($)	Cost per Mile Corrected for Overlap ($)
Continuous strip—Houston[1]	0	300	18.90	18.90
Time lapse—Houston[1]	60	100	41.90	16.75
Time lapse—California[2]	15	200	20.49	17.40

[1] Traffic studies.
[2] Inventory study.

tographs after correcting for the overlap, indicates the time-lapse surveys are less expensive.

Cost of Data Reduction

Continuous-Strip Photography.—The continuous-strip photography was in the form of positive film transparencies which require a light table or film examination unit for viewing. Several different approaches to the problem of data reduction of the aerial film positives are subsequently outlined. The methods which give greater accuracy are usually more time-consuming and more expensive than others.

Stereoscopic Viewer.—A stereoscopic viewer with the capability for making measurements to within 1/1,000 inch was available from the aerial survey company which took the photography for the survey. The two alternatives available were either to rent the viewer and provide a number of the project staff with instruction and training in its use or to contract the measurement work to the survey company. The estimated cost of each proposal is, as follows:

Renting Stereoscopic Viewer:

Rental Cost:	
6 Months at $120 for first month 80 for each additional month	$ 520
Cost of training viewer operator	500
Salary for viewer operator	1,500
Total	$2,520

Contracting Data Reduction:

Cost of Data Reduction 13,774 vehicles at $1.25 per vehicle	$17,200

Enlargements of Continuous-Strip Film Positive Photography.—Another approach to improving the accuracy of measurements taken from the continuous-strip film positive photography was to improve the scale of the film. The Texas Highway Department made available their photographic equipment which has the capacity of enlarging film strips threefold. Sample prints were made from the film for study. At a 3:1 enlargement ratio, 15 sheets were required to make prints of each film strip. The cost for enlarging the photography for the entire traffic survey would be 330 sheets at $4.50 per sheet or $1,485.

Because the film strips were positive transparencies, positive contact prints could not be obtained directly. The cost of a set of negatives would be 330 negatives at $1.50 per negative or $495.

Limited investigations regarding the cost of measuring basic data from these sheets of photographic enlargements indicate the man-hour requirements for the measuring work was not significantly different from measuring data using the original film positive strips.

Measurements from the Film Positive Strips. —The technique which was used on this traffic survey project was to make measurements directly from each film strip transparency. Cost of equipment and personnel used in this data reduction technique are, as follows:

Equipment:	
2 light tables 60 × 24 in.	$160
2 magnifying glasses	6
2 engineers' scales	10
Data Reduction:	
13,774 vehicles at $0.05 per vehicle	$690
Total	$866

The cost of data reduction was calculated from a pay rate of $1 per hour for student workers. The $0.05 per vehicle rate was determined after the data reduction procedure was well established. The rate includes cost of identifying the station numbers and measuring displacement caused by movement of each vehicle and the photographic displacement for each station number and recording the vehicle type, lane, and direction of travel. The cost of position identifying the station numbers was approximately $6 per film strip. This cost was not included in the per-vehicle rate because it is independent of the number of vehicles photographed and because this reduction task can be eliminated by placing station markers on the pavement before the traffic survey photography is taken.

Time-Lapse Photography. —Measurements of vehicle movements during the time interval between photographs were made directly from the contact prints made from the aerial film negatives by the aerial survey company. These prints had a scale of 100 feet to 1 inch, which permitted making measurements with an engineers' scale to the acceptable degree of accuracy.

The cost of traffic data reduction using the time-lapse photography was 15,700 measurements for determining vehicle speed at $0.07 per vehicle or $1,100.

The cost of data measurement and reduction was calculated using a pay rate of $1 per hour for student workers. The $0.07 per vehicle rate was determined after the data reduction procedure was well established. The rate includes the cost of measuring and calculating the station number of the vehicle, the vehicle type, lane, and direction of travel. The cost of position identifying the station numbers is approximately $25.00 per flight strip of time-lapse photography. This cost item was not included in the unit cost because it is a reduction task which can be eliminated by placing station markers on the pavement before the traffic survey photography is taken.

The cost of data reduction using time-lapse photography was higher than when using the continuous-strip positive film transparencies because of difficulty in referencing vehicles to station numbers. Electronic computer programs were prepared for computing all traffic flow data from the basic measurements made for each vehicle, which were the location of the vehicle and the time of day. Locating the position of vehicles by station number on each photograph was complicated by scale distortions at the edge of the photographs and by inaccuracies of the station numbers. Station numbers were transposed onto the aerial film positives from a set of construction plans for the freeway. Because of scale distortions, more than two reference markers should have been used, but on some photographs there were no distinguishing roadway features to establish references. This problem can be averted by placing station markers on the pavement before photography is taken for the traffic.

Because no station markers were placed for use in making this survey, a special reduction procedure was used whereby the first vehicle position was determined from the transposed station markers. Each succeeding location was calculated by measure-

ments from a common reference marker. This process of having to repeatedly make reference to the photograph in which the vehicle first appeared resulted in loss of efficiency.

DATA OBTAINED FROM THE TWO AERIAL SURVEYS

Speed

Both photographic methods provide means of obtaining vehicle speeds. The accuracies of these measurements are different because of the time interval used to calculate speeds and the scale of the photographs.

Density

Density must be calculated using the data measured from the continuous-strip film positive photography. The data have to be corrected for the time transpiring while photography was being taken throughout the length of roadway section along which the traffic survey was being made. This correction is very small, in the order of five to ten seconds, for freeway segments one or two thousand feet long.

The time-lapse photography furnishes a true measure of traffic density within a freeway section approximately 1,800 feet long, which is included in the image of coverage of one photograph. Measurements over a longer section required use of two or more photographs for which their instant of exposure is separated by a short interval of time. In this event the same correction for the elapsed time must be made to determine density of traffic.

Space Headways

Determining the distance between two vehicles, space headways, is subject to the same conditions and corrections applying to measurements for determining traffic density.

Time Headways

The time separating two vehicles can be calculated by dividing the space headway by the speed of the trailing vehicle.

Volume

The volume, the number of vehicles comprising the traffic, can be determined from aerial photographs if continuous coverage was maintained for one hour or more. Rates of traffic movement (flow) can be calculated for each flight strip of photography from the following expression (9):

$$V_t = \frac{\bar{s}\,(n\ 1)\,S_p}{D + (S_p - S_d)}$$

in which

V_t = rate of flow of traffic in vehicles per hour for a time period of $t = D_t/S_p$;
$\bar{s}$ = average overall speed of n vehicles;
D_t = distance between end vehicles (measured on photography);
n = number of vehicles in distance, D_t;
S_p = speed of the airplane in mph; and
S = speed of last vehicle in traffic lane.

The determination of n must be done carefully to avoid counting the same vehicle more than one time.

Acceleration--Deceleration

Speed change by individual vehicles cannot be measured by use of the continuous-strip film positives. Each vehicle is photographed only twice each on each flight strip of stereoscopic photography.

The time-lapse method of aerial photography can be used to make multiple measurements for determining the speed of each vehicle. There are several factors which influence the number of times a vehicle will be photographed: (a) the percent overlap between successively adjacent photographs, (b) the difference in airplane and vehicle speeds, and (c) the direction of aircraft flight relative to the direction of the traffic movement.

In the survey made in Houston, sixty percent overlap and an airplane speed of 120-140 mph resulted in several vehicles traveling in the same direction being photographed five times. From these several measures of speed, the acceleration and deceleration of each vehicle can be calculated. These speed traces can be of value in measuring the build up to, or relief of, congested areas.

These measures of speed change can also be used in calculating traffic density. Rather than making an assumption of constant speed, the acceleration or deceleration of the vehicle can be projected for the time interval involved to get a more accurate location for density determination.

Vehicle Classification

Large commercial vehicles can be identified from both the continuous-strip film positive and time-lapse photography. Small single-unit and pickup trucks are difficult to distinguish from passenger vehicles on the strip film positive photography especially when traffic is moving fast.

Markings, which had been placed on the top of control vehicles, were easily noted in the time-lapse photography, but required very close inspection to be seen on film strip positive photography.

CONCLUSIONS

The application of aerial photography to making traffic surveys has been demonstrated and has proven to be an excellent means of gathering traffic data. Through the use of this technique, the traffic survey areas have been expanded in breadth and length, and inclusion of traffic on the street systems supporting the freeway is now possible. Traffic flow characteristics requiring measurements of distance, such as density and space headways, can be determined more accurately.

There are disadvantages which must be considered in designing a traffic survey for accomplishment by use of aerial photography. Although the traffic conditions are recorded pictorially in true relationship to the geometric design of the traffic facilities, aerial photography does not provide continuous motion data throughout a period of time as motion pictures will. This fact should be carefully considered in determining the length of freeway section to be surveyed and the frequency for making the aerial photography flights.

Data measurement and reduction from aerial photographs is still a time-consuming task. But an electronic computer program utilizing only a few basic measurements to calculate all traffic flow characteristics eliminates any necessity for using the photographs time after time.

Traffic surveys by aerial photographic techniques are limited to the hours of good lighting and flying conditions. This prevents getting aerial photography during hours of peak traffic during winter months under the worst driving conditions.

Selecting the type of aerial photography to use requires considering the information to be obtained from the photography. The following conclusions are drawn from the aerial surveys made in Houston:

1. Time-lapse photography is more suited for making measurements to determine traffic density. The aerial photography should be taken from the flight height which will provide ample coverage over the roadway segments to be included in the traffic

density calculations. The amount of photographic overlap in line of flight can be adjusted to requirements of the traffic survey.

2. Time-dependent parameters can be measured more readily by use of continuous-strips film positive photography.

3. Vehicular speed changes can be measured by use of the time-lapse photography.

4. Land distribution and vehicle classification can be obtained by use of both types of photography, but vehicle identification by examination of the continuous-strip film positive photography is more difficult because of blurred images.

5. When freeway sections less than 2,000 feet long are photographed, the effect of time lag in the continuous-strip film photography is slight. The assumption of uniform speed for the traffic gives results within an acceptable level of accuracy.

6. Continuous-strip film photography provides more coverage for the money expended, 22 flight strips of photography in three hours of true time. Time-lapse photography provided nine flight strips in one hour of true time.

7. More speed readings were obtained by use of the time-lapse photography.

REFERENCES

1. Johnson, A. N., "Maryland Aerial Traffic Density Survey." Proc. HRB, 7(1): 244-246 (1927).
2. Greenshields, B. D., "The Photographic Method of Studying Traffic Behavior." Proc. HRB, 13: 382-399 (1933).
3. Greenshields, B. D., "The Potential Use of Aerial Photographs in Traffic Analysis." Proc. HRB, 27: 291-296 (1947).
4. Forbes, T. W., and Reiss, R. J., "35 Millimeter Airphotos for the Study of Driver Behavior." HRB Bull. 60, pp. 59-66 (1952).
5. Wohl, M., "Vehicle Speeds and Volumes Using Sonne Stereo Continuous Strip Photography." Traffic Eng. (Jan. 1959).
6. Wagner, F. A., Jr., and May, A. D., Jr., "Use of Aerial Photography in Freeway Traffic Operations Studies." HRB Res. Rec. 19, pp. 24-34 (1963).
7. Howes, W. F., "Photogrammetric Analysis of Traffic Flow Characteristics on Multilane Highways." Purdue Univ. (July 1963).
8. Rice, J. F., "Adoption of Aerial Survey Methods for Traffic Operations." HRB Res. Rec. 19, pp. 47-52 (1963).
9. Tamburri, T. H., "California's Aerial Photography Inventory of Freeways." HRB Res. Rec. 19, pp. 17-23 (1963).
10. Wohl, M., and Sickle, S. M., "Continuous Strip Photography—An Approach to Traffic Studies." Traffic Eng. (July 1959).

Photogrammetry and Highway Law

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•THE INCREASINGLY large number of land measurement transactions for land purchases, various government programs, etc., has been accompanied by the application of new scientific techniques. One of these measurement techniques is the art and science of photogrammetry, the calculation of vertical and horizontal distances from measurements made by use of photographs. As the use of techniques associated with this process becomes more widespread, photogrammetry impinges upon established legal criteria in some instances. Chief among such criteria is the use or adaptability of photogrammetric evidence in administrative or judicial proceedings.

Photogrammetric evidence may be considered as a form of demonstrative evidence. Such evidence, though lacking substantive value, can be the most effective medium through which the trial attorney may express his client's case to the court. Many lawyers who are active in trial work and whose task it often is to convince a jury or commission of the legal and equitable rights of their client generally agree with the old adage that "one picture is worth a thousand words." In some jurisdictions demonstrative evidence which was properly admitted during the trial may be called for by the jury when they are deliberating upon their verdict in the jury room. Physical or material evidence will convince a jury of the alleged facts of a case much more readily than does the most efficient orator or the most "honest looking" witness who will testify. Consequently, the effect of demonstrative evidence is worthy of the attention of every trial attorney.

The major purpose of this report is to indicate some of the ramifications of the use of photogrammetry in legal and quasi-legal proceedings, in administrative determinations of a legal nature, and to indicate trends in the statutory and case law illustrating recognition of the reliability of photogrammetrically made measurements in the courts and elsewhere. Highway engineers and legal counsel will then understand the boundaries, in a legal sense, of photogrammetry and will be aware of new developments that have implications for highway personnel.

Highway engineers, city planners, land-use specialists, and various survey and defense activities have utilized photogrammetric techniques with varying degrees of regularity. It was in pre-World War II Italy (1931), however, that systematic application of photogrammetric methods to cadastral or land surveys was first achieved. By 1939, this became the regular means of producing cadastral maps in Switzerland. In France, photogrammetric methods have been used for the delineation of properties and determination of boundaries in large-scale maps for many decades.

The United States cadastral survey techniques have lagged behind those in other countries, for it was not until 1937 that the first complete standard quadrangle was mapped by Fairchild for the Tennessee Valley Authority. The results of this experiment were indicative of some of the later uses of this technique (1). For while the results differed from those of ordinary topographic maps, the aerial results were correct in every instance of disagreement with maps compiled from conventional plane table surveys.

Most progress in developing uses of aerial photography in the United States as a tool in surveying and mapping has occurred in the last 25 years. Since its establishment in 1935, the Maps and Survey Division of the Tennessee Valley Authority has

Paper sponsored by the Special Committee on Highway Laws.

utilized aerial photographs in nearly all of its mapping activities. The TVA's use of aerial photographs in property surveys has been employed on a wholesale basis because it has involved relatively large areas of land (2). The TVA, from 1933 to 1952, for example, made cadastral surveys of more than 2,000,000 acres of privately-owned land representing some 40,000 individual ownerships in the Valley area. Aerial photography was used in practically all of these surveys. Later, as the topographic program of TVA progressed, manuscripts showing fence lines, wood lines, hedgerows, streams, and other features likely to represent property lines were shown; facsimile copies of the manuscript were made into plane table sheets and used as a base for property surveys.

In Canada, the purpose of the legal survey has been to define and mark boundaries of properties on the ground but has not been used to prepare large-scale maps (3). As an experiment, the Alnwich Indian Reservation was surveyed by ordinary field methods and by photogrammetric methods in 1958 and compared in both cost and accuracy. In order to perform the survey by photogrammetric methods, it was still necessary to establish property corners in the field. It was found that, for short distances, photogrammetry was not as good as a field survey but was considered adequate.

The situation in Switzerland is unique (4). There is a complete absence of flight problems because all of the photography is performed by Swiss Federal Government organizations. The government takes care of supervision and coordination of the various projects and setting up of rules, has technical supervision of photogrammetric projects and their control, and holds examinations for cadastral surveyors. For control point determination, the photogrammetrist has at his disposal an excellent network of triangulation points throughout the country and does not have to make a fresh start for each project.

Although photogrammetry was used in conjunction with many aspects of planning and engineering highways in the United States, perhaps the greatest impetus to photogrammetry was given by the Federal Highway Act of 1956, providing for the accelerated completion of the 41,000-mile Interstate Highway System. In recognition of its potential and its accuracy, Section 121 of the Act was prescribed to foster the use of photogrammetry in mapping.[1] When one considers the enormous problem involved in the development of various topographic measures, the laying out of rights-of-way, and the fact that 750,000 parcels of land will have been acquired by the completion of the Interstate program, one can obtain a better appreciation of the need to adopt accurate and rapid measurement techniques which have the possibility of reducing cost to Federal and State Governments and to the community at large.

It should be pointed out that few instances have arisen which have culminated in court actions where the issue to be decided hinged directly upon the admissibility or validity of aerial photographs or maps compiled by photogrammetric techniques. It is reasonable to assume, however, that such court actions will increase proportionately to the number of instances in which photogrammetry is utilized. The legislature may, by proper legislative provision, minimize the burden of increased litigations and attempt to clarify certain elements of doubt which might arise among prospective litigants due to the prevailing statutory void in the subject of photogrammetry.

Photogrammetry in Highway Design and Development

Before discussing the legal implications of photogrammetric measurement, it might be well to describe briefly how this method has been used in highway construction. In the location and design phase, photogrammetry has proven particularly valuable. A general reconnaissance survey is first made in order to select terminal points between which a continuity of design is indicated. This entails detailed study of existing maps to ascertain type of terrain, populated areas, bodies of water, and other

[1] Federal-Aid Highway Act of 1956 ch. 462, P.L. 627 § 121, Mapping. In carrying out the provisions of this title, the Secretary of Commerce may, whenever practicable, authorize the use of photogrammetric methods in mapping, and the utilization of commercial enterprise for such purposes.

features which may have an influence on selection of a route and subsequent design of the highway. After a general route is selected, photographs are made with precision aerial cameras. The pictures are taken from a predetermined aircraft flight height in order to secure coverage of the route alternatives which may require study and comparison. A wealth of information can be gathered from detailed examination and interpretation of these photographs. With a three-dimensional viewer, the topography and land use features can be studied in detail, contours may be measured and delineated and stream and river crossings selected. Photogrammetrists then compile topographic maps by use of special stereoscopic plotting instruments which require only a few vertical and horizontal control points. The instruments are also used to extend the control. Contours on the topographic maps can be measured and delineated at 5- and 10-ft intervals or, for special and detailed design purposes or in urban areas, at 2- and 1-ft intervals. Finally, with the selected route band appropriately mapped and the highway location design completed, the map sheets are given to the land negotiators who are now in a position to proceed with permits and title searching (5).

Another benefit derived from aerial photography and photogrammetry is in the appraisal area. In the initial stages of highway engineering, appraisals are usually made for cost estimation and comparison purposes. More often negotiations are made with the aid of aerial photographs, photographic mosaics, and maps during or subsequent to completion of the preliminary survey and preparation of construction plans. This type of presentation presents a clear and concise plan to the parties involved in the negotiations (6).

Other Uses of Aerial Photography

The use of aerial photographs for cadastral surveying is but one of the many ways in which aerial photography is utilized. It has advanced to such a degree that it is no longer a matter of what can be obtained in the way of planimetric detail, but more a matter of deciding what should and should not be shown. It is possible, when detailing with a 6-in. accuracy to show such things as individual steps on houses, projections from buildings such as window air conditioners, and even the positions of switches in railroad yards will be shown as open or closed (7). With such clarity of detail, the utility of aerial photographs is extended immeasurably.

One of the most obvious advantages of photogrammetry in cadastral surveying is the determination of locations of natural features such as streams, rivers, shore lines, ridges, and swamp lines. Rivers nearly always become natural property boundary lines and are quite troublesome when their courses change. Shore lines confront the cadastral surveyor with problems of riparian rights, and the laws controlling riparian rights differ from state to state. Photogrammetry will not only give the present location of the river but will also reveal data that will enable the surveyor to chart and sometimes date each change of course (8). This is important to a land surveyor particularly if his survey is being contested in court.

An extremely important use of aerial photographs which has not been developed to its fullest extent is in the preservation of man-made monuments. Because all conveyances must somehow be related to known monument positions, certainty of location is endangered if the monument is destroyed. One means of preserving this evidence is by aerial photography. Very often the photograph itself is of more value to the property surveyor than the measurements made from it. The many uses of a photographic history of land include the following:

1. Identification. The Forest Service, Bureau of Land Management, U. S. Geological Survey, ties the location of a found corner or other boundary evidence to other monuments on the ground. Reference ties to three or more points that are easily seen on a photograph will reference the corner to all images on the photograph. This forms a permanent record for location on the ground long after all man-made references have disappeared.

2. Land-use development. Aerial photographs may be used to reveal direction of city growth, rural growth, population density, urban concentration, population, zoning,

data, and culture where ground information is otherwise unobtainable or unavailable or where there is a need to bring available material up to date.

3. Use of old photographs. After a road has been obliterated, or a fence removed, traces may still be seen on a photograph even though no evidence appears on the ground. A comparison of an old photograph with a new one indicates some of the changes. It is apparent that an old photograph can be more valuable than an old map because there is no danger of a surveying or drafting blunder. Accordingly, the aerial photographer can build up a library of perpetual visual evidence with a degree of accuracy otherwise unobtainable.

4. Riparian evidence. Comparison of old and new photographs will show the action of water with some certainty, e.g., accretion and dereliction. The photographs will evidence shallow areas as well as relative beach and shore lines. The location of shore lines at the original survey or time of conveyance is essential information for determining riparian rights.

5. Evidence undetectable from the ground. Pipelines and field drains may be valuable title evidence but can become completely invisible from the ground. Their locations are usually evident from aerial photographs even when the lines are abandoned for many years. Infrared photography will reveal the subtlest change in the character of the land.

6. Detection of encroachments. A building wall or corner may appear to be over a property line; the extent of the overhang is clearly illustrated in an aerial photograph.

7. Identification of lost tracts. Tracts which are described by metes and bounds sometimes have insufficient title identity. If the parcels are plotted to the same scale as an aerial photograph and if the shape is tried like a jigsaw puzzle until a similar pattern on the photograph is discovered, title identity can often be determined.

8. Location of monuments. Search for ancient cornerstones, landmarks, and section corners can be aided by a thorough study of an aerial photograph. Faint field lines can be projected, and their intersection will localize the area to be searched (9).

Photographs of any type have been admitted into trial and into administrative proceedings as a means of providing a representation of the particular property, direction of growth, or in particular, of some facet applicable to the case involving highway construction, zoning, land changes, land values, and highway location. The need for any type of representation arises from the need to illustrate some contentious point clearly to the agency, court, and/or the jury.

In the law, a model, map, or photograph is of course considered to be demonstrative in that it serves merely as a visual aid to the court or jury, especially in comprehending the verbal testimony of a witness.

The unique value of photogrammetry lies in providing visual explanations and a means of accurate measurement of visual subject matter.[2] Demonstrative evidence such as this has no probative value in itself and hence is distinguished from substantive evidence which goes beyond a mere aid to understanding. An excellent example of this point was discussed in Barnes v. North Carolina State Highway Commission, 250 N. C. 378, 109 SE2d 219 (1959), where maps of a registered civil engineer, showing a residential subdivision that could have been placed on land previously taken by eminent domain and also showing the reduced number of lots after the taking, were properly admitted as evidence. The maps were used to illustrate and explain testimony previously given by an expert realtor who testified that the property before and after taking was adaptable to residential subdivision. Although the maps were not admissible as substantive evidence to show a practical subdivision, they were nevertheless admitted for a more definitive explanation of an expert witness' testimony.

Foundation for Admissibility

Prior to the admission of any aid during a trial, technical procedural rules prevail; thus, a proper foundation must be laid. Demonstrative evidence must be identified

[2] Smith v. Ohio Oil Co., 10 Ill. App.2d 67, 134 NE2d 526 (1956).

by a witness and verified as being an accurate and reliable representation. Maps prepared for testimonial purposes are of a circumstantial nature, and the question of sufficiency of the testimony offered as a foundation for them is addressed to the discretion of the court. They may be excluded where the court finds that, notwithstanding their relevance and competency, the probative value of the exhibit is outweighed by the risk of undue influence, confusion, or waste of time entailed in its use. Where the qualifying testimony is sufficient for a map and the accuracy of the map is then disputed, the question of accuracy must be answered by the jury.

In administrative proceedings, zoning hearings, county commissioners' courts, and many others, of course, these technical rules may not apply or do so in various degrees.

Certain generally accepted circumstances tend to give foundation for the admission in evidence of a map.[3]

1. The map must be prepared according to scale. In addition, any variance between horizontal and vertical scale would perhaps mislead the jury.

2. It must be verified by a witness as being a reliable and correct representation of the area in issue. In most kinds of evidentiary presentations, courts generally favor what has been termed the "best evidence" rule. Thus, if a particular document is to be used to prove a point, then that document (and not a copy) is to be preferred. In the case of a surveyor's plot, however, this rule does not apply. In an action against a city to recover for damage to the plaintiff's property caused by the construction of a viaduct, plaintiff called a witness who produced a plat made by him of the lots and surrounding area. The witness testified that it was a correct plat according to a survey originally made by him. But the plat had been made by him by copying the lot lines and dimensions from the original plan on record in the recorder's office. An objection was made that, under the best evidence rule, the original plat should be produced. Admission of the copy was upheld. The court said:

> The object was to show the jury the location of the property by the surveyor, who had measured it. It would have been competent for him while upon the stand to have made a plat of the property as he found it for the inspection and information for the jury.[4]

3. The map must be of such a nature as to be explanatory of verbal testimony. Where a witness testifies that the land is adaptable to the erection of certain buildings, a map of the land with the supposed buildings depicted thereon is admissible, supported by his testimony, for the limited purpose of showing such adaptability.[5] But even though it is verified as correct by the surveyor, the plat is not admissible unless it is somehow related to relevant landmarks and thus, as it were, attached to the soil.[6] But when the survey or plat is of official origin and conforms to the code section pertaining thereto, it is admissible as presumptive evidence of the facts.[7] Thus, the courts have taken judicial notice of topographic maps made by the U.S. Geological Survey.[8]

4. The map must be of such a nature as not to mislead the jury or cause confusion or undue influence. Formal survey maps are the cause of much misunderstanding by counsel in controversies over land. A plat produced by an expert surveyor and supported by his testimony that it is correct, especially if it purports to show boundaries in favor of the party who called him, challenges objection because of its seeming legal effect. Unless some statutory authorization, official recognition, admission by the opposing party, or documentary reference to it gives it such effect, it is admissible only as any other diagrammatic medium may be, to illustrate the testimony of the

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[3] Foundation for admission of map or diagram, see, 7 Am. Jur. Proof of Facts, Maps, Diagrams, and Models, Proof I (1961).
[4] Chicago v. LeWayne, 119 F. 662 (7 Cir. 1902).
[5] Campbell v. City of New Haven, 101 Conn. 173, 125 Atl. 650 (1924).
[6] Hagaman v. Bernhardt, 162 N.C. 381, 78 SE 209 (1913).
[7] Durden v. Kerby, 41 SE2d 131 (Ga. 1947).
[8] Union Transportation Co. v. Sacramento County, 42 Cal.2d 335, 267 P.2d 10 (1954).

surveyor who made it. If the surveyor testifies that the plat correctly represents the location of the objects marked thereon and the measurements made by him of distance, it is rendered admissible as part of his testimony.[9]

5. The witness must be qualified to testify as to the accuracy of the proposed map. It is not an objection, though subject to the court's discretion, that the witness is not skilled in the making of maps.[10] He must have had observation of the land in question, must collect his observations, and must correctly express his observation and recollection. It must appear that there is a witness who has competent knowledge, and the map is affirmed by him to represent it.[11] An owner with no qualifications as a surveyor or engineer may support an issue as to what land is in his adverse possession by measuring his fences, making a diagram of them, and producing and testifying to it in court.[12] In a condemnation proceeding by a railroad company, the owner of the property was permitted to introduce in evidence, as part of his testimony, a map or diagram of his property made by him, showing the location of his various improvements—his house, barn, etc.—and the location of the railroad across the land relative to these improvements. He was also permitted to testify by referring to the drawing.[13]

6. The nature of the testimony must be such that reference to a map is necessary to the understanding of the testimony by the jury. In a condemnation proceeding, it was held that the trial court did not err in admitting in evidence a copy of a plat of several blocks of a city including the property in question, where the plat was admitted for the sole purpose of showing the location of the property in reference to the streets. It was admitted on the theory that the plat was nothing more than a verified pictorial representation of matters about which the witness had testified, and a desirable expedience by which to illustrate the witness' testimony as to the location of the land in question.[14]

The rules of law dealing with photographs differ from those rules dealing with maps because the situation and surrounding circumstances are subject to change. Photographs, to be admissible as evidence, must have been taken at the time of the transaction or before the situation and circumstances have undergone a change. Frequently, photographs have been held inadmissible on the ground that they were taken at too remote a time and conditions had changed.[15]

There is no distinction between aerial and other types of photographs insofar as their admissibility is concerned, and no case has been found which admitted or excluded a photograph on the sole ground that it was taken from the air.[16] The basic elements of authentication are the same for pictures taken from airplanes as for ordinary photographs although emphasis may differ.[17] Aerial photography presents problems that are not encountered on the ground; the qualifications of the photographer and the quality of his equipment can be expected to play a greater role in admissibility because it is more difficult to make useful and accurate photographs from airplanes.[18] In addition, courts often require that aerial photographs possess some additional advantages over ground-level photographs of a scene in order to be admissible. On principle there is no reason photographs taken from airplanes should not be admitted in evidence under the same rules governing all photographs provided they are relevant to some issue in the case and verified as correct representations of the scene they

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[9] Seidschlag v. Town of Antioch, 207 Ill. 280, 69 NE 949 (1904).
[10] Redman v. Cooper, 160 SW2d 318 (Tex. Civ. App. 1942).
[11] Wigmore, Evidence, § 793 (3rd ed. 1940).
[12] Ibid.
[13] Chicago K. and W.R.R. v. O'Dill, 41 Kan. 736, 21 Pac. 778 (1889).
[14] Aycock v. Fulton County, 95 Ga. App. 541, 98 SE2d 133 (1957).
[15] Chandler v. Russell, 164 Va. 318, 180 SE 313 (1935).
[16] 20 Am. Jur. § 727 (Cum. Supp. 1962); 57 A.L.R. 2d p. 1352, § 1351 (1958).
[17] Scott, Photographic Evidence, § 628 (1942).
[18] 9 Am. Jur. Proof of Facts, Photos as Evidence, Proof 6, p. 199 (1961); Scott, Photographic Evidence, p. 191 (1942).

portray.[19] When the prerequisites of relevancy and verification are present, the question of admissibility rests upon the discretion of the trial court.

A California district court of appeals, commenting on the admissibility of land-level photographs, in a 1959 case, said:

> In ruling upon the admissibility of photographs the trial judge had two primary duties, one, to determine whether the photograph is a reasonable representation of that which it is alleged to portray, and second, whether the use of the photograph would aid the jurors in their determination of the facts of the case or serve to mislead them.[20]

Very accurate maps for use as evidence in real property controversies can be compiled from properly prepared aerial vertical photographs. But this being a highly specialized form of photography, the photographs to be used must be made by a photographer skilled in taking aerial photographs for mapping by photogrammetric methods, the airplane must be piloted by a man skilled in piloting a plane on photography missions undertaken for mapping purposes, and the maps must be compiled from the photograph by someone trained in photogrammetric instrument operation and map compilation.[21] The photographs are also valuable assets in negotiation. The parcel can be studied prior to contact with the owner and compared to the parcels and oriented to existing roads, the proposed construction, streams, and other features. Damages and enhancements as assessed during the appraisal process may be examined as well as neighborhood characteristics and the general terrain involved (7).

This is well illustrated in Orange County Water District v. Riverside, 173 Cal. App. 2d 137, 343 P. 2d 450 (1959) where, in an action by a county water district for declaration of water rights of cities to take water from the district river system, admission of a map compiled by use of aerial photographs was held not to be a prejudicial error where the effect was purely cumulative. The expert who prepared the map was present to verify and attest the validity of the map, and the jury was not present. Another case, which was appealed in 1945 in an action by the plaintiff to perpetually enjoin the defendant and his successors from using a certain ditch, admitted a tracing from an aerial photograph. The court based its decision upon cases dealing with the discretion of the trial court to admit maps and charts as competent evidence to illustrate relative locations and objects as an aid to the jury.[22]

Whether an aerial photograph should be admitted as evidence depends a great deal on the circumstances of the individual case. For example, aerial photographs showing condemned property and the neighborhood surrounding it were admissible as evidence where they were properly identified and where they accurately portrayed the condition on the ground.[23] Such photographs were qualified for admission by testimony of a registered professional engineer employed by the condemning city, who was familiar with the property in question.[24] Aerial photographs were considered admissible where they showed the location of the land, roads, and buildings involved from a high altitude, and witnesses testified that the area "looked about the same" as the photographs showed.[25] It is for the trial court to determine whether a photograph offered is a preliminary question of fact to be decided by the trial judge.[26]

A leading case of the subject of admissibility of aerial photographs is Department of Pub. Works & Bldgs. v. Chicago Title & Trust Co., 408 Ill. 41, 95 NE2d 903 (1950),

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[19] United New Jersey R. & Canal Co. v. Golden, 104 N.J.L. 385, 140 Atl. 450 (1928); Chandler v. Russell, 164 Va. 318, 180 SE 313 (1935).

[20] Anello v. Southern Pacific Co., 344 P. 2d 843 (Cal. Dist. Ct. App. 1959).

[21] Scott, Photographic Evidence § 628 (1942).

[22] Williams v. Neddo, 66 Idaho 551, 163 P.2d 306 (1945).

[23] Frankfurt v. Dullas, 299 SW2d 722 (Tex. Civ. App. 1957).

[24] See also, 3 Wigmore, Evidence § 793 (3rd ed. 1940), as to competency of witness.

[25] Trachta v. Iowa State Highway Comm'n., 249 Iowa 374, 86 NW2d 849 (1957).

[26] Howe v. City of Boston, 41 NE2d 1 (1942); Hyde v. Town of Swanton, 72 Vt. 242, 47 Atl. 790 (1900); Adams v. State, 28 Fla. 511, 10 So. 106 (1891).

where the Department filed a petition to condemn certain land for park purposes. The court entered judgment directing the petitioners to pay a certain amount to the county treasurer for the benefit of the property owners as full compensation and the owners appealed, contending that the court erred in allowing three topographic maps made from aerial photographs as exhibits to show what portion of apellant's property was under water, what portions might be subject to flood, and what portion was on or above the feasible building plane. The Supreme Court of Illinois ruled that plats, photographs, drawings, and diagrams that illustrate the subject matter of the testimony may be received into evidence for the purpose of showing a particular situation, explaining the testimony, or enabling the jury to apply the testimony more intelligently to the facts shown; but the particular photographs involved were held inadmissible on the basis of the trial judge's discretion.[27]

Aerial photographs taken from the files of the U. S. Corps of Engineers to show improvements on land or a lack thereof have been held admissible as evidence in Louisiana where the plaintiff produced an expert witness to testify to the validity of the photographs.[28] This was the only case uncovered during the study concerning admissibility of aerial photography in which the court cited a statute in support of its reasoning. However, the statute cited deals entirely with the admissibility of Federal enactments, regulations, or documents as evidence but does not mention photographs.[29]

Photographs used as a comparison for similar conditions have been admitted also. In an action to condemn land for the purpose of extending a municpal airport, the defendant landowner objected to the admission into evidence of a photograph of a municipal airport in another city on the ground that it was immaterial and irrelevant and that there was no evidence by the photographer that he took it correctly. The court held the photograph admissible where the witness testified that he formerly was an aviator and had, over a long period of time, flown in and out of the airport photographed, that the photograph was a true and correct picture of the airport, and that it reflected the true condition on the ground.[30]

Because the admission of aerial photographs is a matter within the trial court's discretion, there are a number of cases in which such photographs have been held inadmissible. The courts have ruled that they were inadmissible when there was ample evidence previously submitted to give the jury a proper perspective of the site,[31] where the photograph was not accurate (not representative) in that it failed to show the complete condition of the land,[32] the photograph was not sufficiently verified,[33] the accuracy was not properly and sufficiently shown,[34] and where the photographs have not been verified or authenticated by some other evidence before they are admitted.[35]

In a few actions aerial photographs have been excluded from evidence, at the discretion of the court, where other evidence gave a sufficiently accurate picture. In Buchanan v. McHurdle,[36] the court stated:

> The aerial pictures would have thrown little, if any light upon the essential facts, in addition to that disclosed by the other evidence —certainly not enough to work a reversal and retrial of this cause.

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[27] "While such exhibits (aerial photographs) might properly have been admitted under the rule of the Smith case ... , it was still a matter within the discretion of the court, and its failure to do so is not such an error as to warrant reversal of the judgment." Department of Pub. Works & Bldgs. v. Chicago Title and Trust Co., 408 Ill. 41, 95 NE2d. 903 (1950).

[28] Airway Homes, Inc. v. Boe, 140 So.2d 264 (La. Ct. App. 1962).

[29] L.S.A. - R.S. - 13:3713 (1951).

[30] Wise v. Abilene, 141 SW2d 400, 9 A.L.R.2d 928 (Tex. Civ. App. 1940).

[31] Galt v. Dept. of Pub. Works & Bldgs., (71 Sup. Ct. 804, 341 U.S. 931). (Ill. 1951, certs. denied.) Reported below: 408 Ill. 41, 95 NE2d 903 (1950).

[32] Chandler v. Russell, 164 Va. 318, 180 SE 313 (1935).

[33] United New Jersey R. and Constr. Co. v. Golden, 104 N.J.L. 385, 140 Atl. 450 (1928).

[34] Department of Pub. Works and Bldgs. v. Chicago Title and Trust Co., 408 Ill. 41, 95 NE2d 903 (1950); Chandler v. Russell, 164 Va. 318, 180 SE 313 (1935).

[35] Moore v. McConnell, 105 Ga. App. 758, 125 SE2d 675 (1962).

[36] 209 Miss. 722, 48 So. 2d 354 (1950).

It is evident, however, that with a proper foundation an aerial photograph can be admitted as demonstrative evidence in much the same manner as a map or an ordinary photograph. Care must be exercised in having an expert prepare and verify the aerial photograph in court to avoid any possible exclusion.[37]

In 1962, the Photogrammetry and Aerial Surveys Committee of the Highway Research Board prepared and circulated a questionnaire (6) to state highway departments and Federal agencies to determine the utilization of aerial surveying in each of the principal highway engineering stages. Of the replies received from 47 states, only 17 states reported use of aerial photographs for cadastral purposes, and nearly one-third of the states use aerial photographs for appraising and negotiating for rights-of-way. Less than 10 percent of the states use aerial photographs for preparation of deeds, however (6).

Other questionnaires (10) to state highway departments show that 47 states use some form of photogrammetric techniques in their highway programs.

Legislative Reform and Judicial Recognition

With respect to admissibility as evidence, the products of photogrammetry are treated almost universally by the courts as any other photograph or stereoscopic model. There has been to our knowledge little effort to lay guidelines for the development of uniform rules for photogrammetrically made measurements as cadastral data or to recognize the technical ability of the photogrammetric methods to meet and even surpass the various accuracy and minimum standard requirements presently demanded of cadastral surveys by the state title associations and the judiciary.

A majority if not all of the states statutorily define land surveying and the surveyor under some title such as professions and occupations. Accordingly they institute licensing and registration requirements for the professions and occupations regulated. The courts generally rely on these statutory provisions as providing a reliable level of competence. Consequently, the products of the land surveyor or professional engineer are recognized judicially as the work of an expert and thus receive a certain air of veracity. Some courts have already extended judicial notice to topographic maps prepared by the U. S. Geological Survey.[38] Yet these statutes, regulating everything from pediatricians to morticians, have not recognized the newer techniques of surveying and land measurement as described in this paper.

California appears to be the only state which does statutorily recognize photogrammetry. The statutory change in California began in 1961 when the land surveying definition was amended by adding the words, "or photogrammetry," to paragraph "(d)" of Section 726.[39] Two additional sections, subsequently added, read as follows:

> § 8730.5. Preparation and delivery of topographic maps produced by photogrammetric process; licensing.
>
> This chapter does not require licensing to prepare and deliver topographic maps produced by the photogrammetric process or data connected therewith under contract with an individual, firm, corporation, association, or public agency if the following conditions exist:

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[37] For other cases involving verification, see, Adams v. State, 28 Fla. 511, 10 So. 106 (1891); United New Jersey R. and Canal Co. v. Golden, 104 N.J.L. 385, 140 Atl. 450 (1928); Moore v. McConnell, 105 Ga. App. 758, 125 SE 2d 675 (1962); 3 Wigmore, Evidence § 794 (3d ed. 1940).

[38] Union Transportation Company v. Sacramento County, 42 Cal.2d 335, 267 P.2d 10 (1954).

[39] 4 Cal. Code Ann. § 8726 (Cum. P.P. 1962), Amend., Stats. 1961, c.2225, p. 4579 § 1. "§ 8726 Land Surveying defined a person practices land surveying within the meaning of this chapter who, either in a public or private capacity, does or offers to do any one of the following: (d) determines the configuration or contour of the earth's surface or the position of fixed objects thereon or related thereto, by means of measuring lines and angles, and applying the principles of trigonometry <u>or photogrammetry</u>." [Emphasis added.]

(a) Field surveys to be done under the contract are performed by registered civil engineers or licensed land surveyors.

(b) A registered civil engineer or licensed land surveyor is the official of the individual, firm, corporation, association, or public agency responsible for the approval of the performance under the contract, or the work is to be delivered to a registered civil engineer or licensed land surveyor.

§ 8730.6. Termination of licensing exemption.

After June 30, 1962, the exemption from licensing provided in Section 8730.5 shall apply to only those persons who hold a certificate of exemption issued by the board. The board shall receive applications for certificates of exemption filed on or before June 30, 1962, but not after that date. A certificate of exemption may be issued to any person who shows to the satisfaction of the board that he has had six years or more of professional level photogrammetric mapping experience. The certificate shall be on a form prescribed by the board and shall be accompanied by the application fee prescribed by this chapter for land surveyor's license.

As a result of these statutory changes California clearly intends that photogrammetry be recognized as a method of measurement for calculating land dimensional data. Before these specific statutory provisions were enacted California recognized the work of the photogrammetrist, but not on the same plane as the work of the registered land surveyor or civil engineer. In 1954 the Attorney General of California rendered the opinion that topographical maps prepared by aerial mapping firms under contract with the Department of Public Works, for use by the Division of Highways and Division of Water Resources, did not necessitate the services of a licensed surveyor, because the service rendered by the aerial mapping firm did not constitute land surveying as defined in Sections 8726 and 8627 (as of 1954).[40]

Section 8727, as enacted in 1954, except certain surveys from the provision of Section 8726. Aerial photography and photogrammetry were among the excepted categories. This section was changed in 1959, however, and the categories of aerial photography and photogrammetry were removed.

A similar point came up in Hill v. Kirkwood,[41] a 1958 taxpayer's suit to enjoin payment on a certain contract. It held that a contract requiring a company performing aerial survey work to furnish maps prepared by a process known as photogrammetry was not illegal because the company engaged in the work was not registered as a civil engineer or licensed as a land surveyor even though the mapping did require use of conventional land surveying methods for the ascertainment of ground control data because this work did not constitute land surveying as [then] defined by Sections 8726 and 8727. The subsequent statutory changes, however, tend to indicate that the same factual situation today would require a different holding.

Idaho also has interpreted the service of photogrammetry as not being within the state's statutory definition of land surveying.[42] Idaho has a statute,[43] as do a great majority of states that defines land surveying in terms similar to those used by California but not including the word photogrammetry. The general terms in which most of these statutes are drafted perhaps would permit judicial interpretation to include photogrammetry as a method of measurement (within the meaning of the statute) if the court desired to so construe the present statutes. It could be well argued, however, that such interpretation would be burdensome upon the litigative process. The alternative solution appears to lie within the command of the legislature, for proper legislation could eliminate the majority of doubt. Photogrammetric techniques in their pres-

[40] 23 Ops. Cal. Atty. Gen. 86 (1954).
[41] 161 C.A.2d 346, 326 P.2d 599 (1958).
[42] Aero Service Corp. (Western) v. Benson, 374 P.2d 282 (Idaho 1962).
[43] Idaho C.54-1202 (1957).

ent state, as compared to conventional land surveying techniques, lend themselves to statutory recognition and regulation with equal definitiveness.

The changes made by California are indicative of a trend which may possibly result in the adoption or amendment of licensing and registration regulations including examinations directed toward evaluating the capability of photogrammetric instrumentation and techniques and setting a standard level of ability and competence for the photogrammetrist.

Conclusion and Forecast

It is apparent that photogrammetry has greatly advanced in the United States since its beginnings a few decades ago. Some of the advantages and improvements which have been made through its use are improved accuracy of cadastral surveys, an increase in planimetric detail, a minimizing of costly omissions, reduction in time, elimination of cumulative type errors, elimination of delays in schedules due to weather, and an increase in size of the area mapped to include adjoining properties without trespassing or significantly increasing the cost of the survey. Despite the recent technological advances in the perfection of high-speed cameras, the use of photogrammetry in litigation is relatively rare. It is probable, however, that with increasing public interest and lowering of expense together with easier and more accurate methods of using aerial photographs, the use of aerial photographs as evidence will become more extensive.

When the TVA needed maps of an area within which it was working, it was proved through use that photogrammetry could be a useful tool in compiling small-scale maps of large areas. Since then, mapping has been a major application of photogrammetry for small surveys but large-scale detail work by photogrammetric methods was not readily accepted until proven feasible, economical, and reliable. Only in recent years has acceptance of the use of photogrammetry by the design engineer, land surveyor, landscape architect, city planner, and municipal engineer occurred. Much of the credit for demonstrating to the design engineer and surveyor the capability and accuracy obtainable from photogrammetric surveys must go to the design survey maps compiled in conjunction with the greatly expanded highway program of the past decade.

The question of admissibility is based upon the discretion of the trial judge in any given litigation. The criteria upon which that discretion generally hinges may be concisely stated as reasonable accuracy, proper verification, and relevancy. Accuracy is a relative matter and, with modern development in aerial photography, photogrammetry supersedes the requirements set for the admission of land-level photographs as demonstrative evidence. The proper verification of an aerial photograph generally can be supplied by the testimony of the expert engaged in the taking of the photograph or by one who can attest to the veracity with which the given photograph reflects the actual subject. The requirement of relevancy may be satisfied by showing that the photographs are beneficial to the proper understanding of the subject under litigation.

The question of cost is diminishing as technical developments increase the accuracy of the equipment and measurement and decrease the time factor (11).

In the field of eminent domain in particular, evidence in the form of aerial photographs has great potential. Aerial oblique photographs can clearly show aspects and features which could not be as effectively shown to the jury or special commissioners in eminent domain proceedings and zoning hearings by any other means. Vertical exposures may be used to show comparable properties, but generally they are not as readily understood by laymen as the oblique view. Photographs with overlays or with the artist's conception of the completed facility, perspectively fit into bordering photographic details, can show benefits to property remainders to offset claims of excessive damages in severance suits. A valuable use can be made of photographs showing in perspective the completed highway construction and abutting property development that is comparable to the area in question. In partial takings where the project is in operation before eminent domain proceedings are completed, photographs of the completed project can be most effective, particularly in determining if enhancement in value has taken place in the area by reason of the highway construction (7).

In many suburban and rural areas, maps compiled at a scale of 40 feet to 1 inch are considered acceptable for use in right-of-way development when the properties involved are large tracts (7).

Consequently, it is reasonable to assume that the use of photogrammetry will steadily increase in such areas as planning, surveying, designing, and procuring rights-of-way for highway development as its possibilities become evident and as the legislatures recognize the need for statutory revision to provide for the acceptance of photogrammetry as a metrical science.

In the words of another in concluding a discussion of photogrammetry:

> More recent developments in the area of photography such as photogrammetry ... also promise to assume favored positions as illustrative aids in establishing market value. There is no reason to believe that the courts will require anything more for their admissibility into evidence than is required for the admission of courtroom exhibits in general--that they be relevant and verified as accurate.
>
> Assuredly the primary objective of awarding just compensation in a proceeding in eminent domain, and the general objective of accomplishing justice, requires the continued use, encouragement and further development of illustrative assistance in the varied forms of demonstrative evidence. For, in this dynamic age, we must, to borrow a phrase from "Alice in Wonderland," "... run awfully fast to stand still." (12)

REFERENCES

1. Eliel, L. T., "One Hundred Years of Photogrammetry." Photogrammetric Engineering, p. 359 (June 1959).
2. Frierson, R. E., "The Use of Aerial Photography in Property Surveys." Surveying and Mapping, p. 70 (April-June 1948).
3. Blachut, T. J., "Use of Photogrammetry on a Legal Survey at Alnwich." Canadian Surveyor, p. 336 (July 1959).
4. Pastorelli, A., "Large Scale Photogrammetry and Economy of Precision Photogrammetry in Switzerland." Photogrammetria, 11(3) (1954-55).
5. Johnson, H. E., "Aerial Surveys Aid Land Acquisition." Right of Way, p. 35 (Feb. 1959).
6. "Results of 1962 Questionnaire on Use of Aerial Surveys." HRB Res. News 6, 50 pp. (June 1963). Pryor, W. T., "Evaluation of Aerial Photography and Mapping in Highway Development." 9th Pan Amer. Highway Cong., Doc. 20 (1963).
7. Huff, T. S., and Christian, A. H., "Photogrammetry." Acquisition for Right-of-Way, AASHO, p. 196 (1962).
8. King, I. E., "Advantages of Photogrammetry in Cadastral Surveying." Surveying and Mapping, p. 97 (March 1962).
9. Brown, C. M., and Eldridge, W. H., "Evidence and Procedure for Boundary Location." 313 pp., J. Wiley, New York (1962). Pryor, W. T., "Highway Engineering Applications of Photogrammetry." Photogrammetric Eng., p. 523 (June 1954).
10. Avera, H. Q., "Photogrammetry and the Small Land Surveyor." Photogrammetric Eng., p. 96 (Jan. 1963).
11. "Increased Use of Photogrammetric Service Saves Department Upwards of One Million Dollars." Hy-Lighter (Michigan State Highway Pub.), p. 4 (Sept. 1958). Clarke, L. E., "Photogrammetry in Preparing Right of Way Maps and Deeds." Texas Highways, 7(8):3 (Aug. 1960). Goodwin, W. A., "Aerial Photographs and Some of Their Application to Highway Engineering." Presented at 2nd Ann. Oklahoma Joint Highway Eng. Conf., p. 35 (1959). Lowe, T. M., Jr., "Photogrammetry for the Highway Engineer." Proc. of the 2nd Ann. Alabama Joint Highway Eng. Conf., p. 41 (Mar. 1954). Segelke, V., "Use of Photogrammetry

for Highway Land Acquisition Purposes." HRB Bull. 314, pp. 78-81 (1961).
12. Hadley, G. C., "Demonstrative Evidence in Eminent Domain Proceedings." Inst. on Eminent Domain, p. 132 (1960).