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4 Reports

Subject Areas

21 Photogrammetry
54 Traffic Flow

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FOREWORD

The four papers included in this RECORD will be of interest to those persons engaged in survey, design, construction, and traffic engineering. The papers present new applications of and new procedures for photogrammetry and aerial surveys as well as reinforcement for existing procedures. All will contribute to continued advantageous use of photogrammetry and aerial surveys in the development and operation of highways.

Schultz summarizes four projects where comparisons were made between earthwork quantities computed from photogrammetrically obtained data and those computed from conventional field cross sections. He concluded that the photogrammetric methods would produce accurate estimates suitable for payment.

Schultz and Frantz illustrate the procedures used in Wisconsin to produce two-color aerial photomosaic contract plans and right-of-way plats. This unusual technique, a departure from traditional drafting methods, claims cost savings, greater clarity, and increased flexibility as benefits. The increased complexity of highway design and construction prompted this development and use.

The paper by Lee and Belkin introduces a comparison between two methods of ground control extension, trilateration versus conventional traverse. A FORTRAN computer program was developed to adjust the trilateration network. Results of the experiments showed that trilateration was more accurate than conventional traverse and was at least as economical. The authors recommend that a trilateration chain of quadrilaterals be used where a double centerline or double traverse is needed for highway surveying or mapping control.

In his paper, Cyra describes a traffic study that uses aerial photography to determine vehicle speeds, volumes, and accumulations on an urban freeway. Statistical analyses were made to determine the reliability of the aerial techniques compared to the conventional data collection procedures. Cost comparisons were documented in order to indicate the economic feasibility of each method. Conclusions revealed that oblique aerial photography is suitable for the collection of vehicle accumulations in a reliable and economical manner. Vertical aerial photography was also found to be reliable but had a varied economic condition.

—V. H. Schultz

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EVALUATION OF PHOTOGRAMMETRIC CROSS SECTIONS FOR EARTHWORK PAYMENT

Vernon H. Schultz, Division of Highways, Wisconsin Department of Transportation

Earthwork quantities were compared on four projects by using both photogrammetric final cross sections and conventional field-measured final cross sections. Original cross sections had been taken by conventional field procedures. On one project elevations were also compared. It must be recognized that any system of cross sectioning (field or photogrammetric) used to obtain earthwork quantities is an approximation. Therefore, the comparison made here is a comparison between two approximations. It was found that for dual roadway sections involving 7,500 cu yd or more per station the difference between photogrammetric and field-measured yardage was 2 percent or less. On those roadway sections involving 12,000 cu yd or more per station, the difference was less than 1 percent. In all cases the photogrammetric yardage was greater than the field-measured yardage. When interpolated photogrammetric elevations were compared to field elevations, it was observed that the average mean difference showed the photogrammetric to be 0.15 ft lower than the field. It was concluded that the difference in earthwork quantities was within the range of possible error because of the average end area method of computation, and therefore the photogrammetric method will produce an accurate estimate usable for payment.

•EARTHWORK quantities computed from conventional field surveys by using elevations determined by spirit levels have been used as the basis of payment to contractors for many years. Although this procedure has consistently produced accurate results, since the advent of freeway construction with heavier grading and wider cross sections, the cost of conventional surveys has steadily risen. Also, since the general public has become more involved in matters of design detail and because of the apparent need for greater analysis by the engineering staff of environmental conditions, conventional field surveys may no longer be the most efficient way of obtaining the data required for this new concept of maintaining design flexibility.

A research program was undertaken by the Wisconsin Division of Highways to determine whether sufficiently accurate results could be obtained from photogrammetric cross sections such that they could be used for pay quantities. Of particular emphasis here was that the investigation determine accurate results under typical production situations.

It was felt that there was adequate documentation of the basic theoretical research in this area and that the problem was one of determining whether these basic theories could be implemented on a production basis with sufficient results.

DESCRIPTION OF PROJECTS

Four construction projects were used in the analysis. The selection was based on availability of data within the time period, variability of field personnel and working conditions, and, to some extent, terrain. A special effort was made to select projects in different districts in order to determine the effectiveness of the method while working

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with many different people. It was felt that, with more people being involved, a greater variety of potential procedures and problems could be analyzed.

Only final cross sections were used in the tests. Yardage computations for both the photogrammetric method and the conventional method were made by using conventionally measured original cross sections to define the original ground.

US-14, Oregon-Madison Road, Dane County

This test section was 3 miles of a four-lane, divided freeway through rolling countryside. Cross sections varied from 250 to 465 ft wide with maximum elevation differences on any one section ranging to 60 ft. There were no interchanges involved on this test section, but there was variable median width throughout and independent reference lines for earthwork computation in some areas.

Photographs were taken on December 7, 1968. The subgrade was bare; the granular subbase placement had not yet begun. Grass cover was almost nonexistent on the slopes inasmuch as they had been seeded only a short time before.

US-10, Amherst Junction-East County Line, Portage County

This test section was 1 mile of two-lane, undivided highway through a flat-to-rolling area. Cross sections varied from 120 to 210 ft wide with maximum elevation difference on any one section ranging to 47 ft.

Photographs for original sections were taken on April 11, 1969, but because the contractor had already commenced clearing, grubbing, and removing topsoil it was decided not to use this for comparison. Photographs for final sections were taken on August 20, 1969. Because the pavement was already in place, "blue-top" elevations were substituted for the subgrade readings for the yardage computation. Grass cover on the slopes was minimal.

Wisc-15, Beloit-Milwaukee Road, Waukesha County

This project involved 3 miles of four-lane, divided freeway through gently rolling area. Only a partial analysis was done on this project because complete field sections were not available. Eleven random cross sections were field-checked, and one borrow pit immediately adjacent to the roadway was completely checked. Cross sections varied in width from 250 to 400 ft with maximum elevation differences of up to 30 ft. One diamond interchange and one directional interchange were included in the project.

Granular subbase placement had already commenced at the time of photography in August 1969. Thus, the "blue-top" elevations were substituted for subgrade readings for the yardage computation. Grass cover was at a minimum on new slopes.

US-53, Chippewa Falls-North County Line Road, Chippewa County

This test section involved 2.5 miles of four-lane, divided freeway through generally level terrain but with one 80-ft high river escarpment. Cross sections varied from 225 to 440 ft in width with maximum elevation differences on any one section ranging to 60 ft. There was one partial directional interchange involved. The subgrade was bare, and the slopes had a minimum of grass cover. Photographs were taken October 29, 1969.

EQUIPMENT AND TECHNIQUES USED

Conventional field cross-sectioning techniques were used to obtain field data. These consisted of establishing the stationing with a transit and steel tape, using a right-angle prism for "squaring out," and using a cloth tape for measuring distances from the reference line. Locke shots were used beyond the limits of the automatic level setup.

Prior to aerial photography, targets were placed by district survey personnel on the reference line at 500-ft intervals. Targets were made of white muslin 12 in. wide by 3 ft long in the shape of a cross. On the US-10 project where concrete pavement had been recently placed, targets made of heavy, dark brown paper were used. These contrasted very well with the brilliant white of the new concrete.

On the US-53 project, wing-point targets were used in lieu of image point control as was done on the other projects. These wing-point targets were placed at distances of 400 to 500 ft from the reference line and were roughly "square-out" from those targets on the reference line.

Photographs were made with a Zeiss RMK-A, 6-in. focal length camera from an altitude of 1,500 ft. On those projects where wing-point targets were not used, wing-point image control was designed in the photogrammetric unit, and field work was done by district survey personnel.

A reference line layout with a scale of 1 in. equal to 50 ft was prepared on vellum by drafting. Shown on this layout were the alignment stationing, target locations, target elevations, and locations where cross sections were desired. On the US-53 project, additional care was taken in the preparation of this layout by first computing the coordinate position of the various reference line targets and then by plotting these points by coordinate position. Thus it was found that, in interchange areas where several reference lines are used for cross sections, the relative position of each reference line was far more accurate.

By using the Kelsh Model 5030 stereo plotter, we scaled and made horizontal the stereo model by using the reference line layout and picture point control. Using an automatic scaler and digitizing equipment, the operator proceeded to take cross sections.

ANALYSIS AND EVALUATION PROCEDURES

Two methods of analysis and evaluation were used to compare the results of conventional field cross sectioning with the photogrammetric cross sectioning. First, it was desired to compare the ability of the stereo plotter operator to duplicate the elevation data obtained in the field; and, second, it was desired to compare the earthwork quantities.

It should be emphasized that any system of cross sectioning (field or photogrammetric) used to obtain earthwork quantities is an approximation. Therefore, the comparison made here is a comparison between two approximations, not between an approximate earthwork quantity and a correct one.

Almost all of the data used were generated by conventional production procedures, both in the field and photogrammetrically. Additional care and precision were used only in test cases to determine the effectiveness of the analysis procedures.

An analysis was made between the field elevations and interpolated photogrammetric elevations on about half of the cross sections of the US-14 project. On each cross section generally twice as many photogrammetric spot elevations were obtained as were field elevations. Thus it was felt that a valid comparison could be made by interpolating an elevation from the photogrammetric data at the specific distance of the field elevation. A test area was selected to determine the validity of this procedure. The results are given in Table 1. Where photogrammetric elevations were taken at the same dis-

TABLE 1

COMPARISON OF PHOTOGRAMMETRIC AND FIELD DATA USING INTERPOLATED ELEVATIONS

Elevation Location	Excavation			Embankment		
	Field	Photo-grammetric	Percent Difference	Field	Photo-grammetric	Percent Difference
Same distance from reference line	151,101	151,641	+0.4	5,622	5,686	+1.1
Different distance from reference line	151,101	151,796	+0.5	5,622	5,989	+6.5

Note: Length (8 stations) was 800 ft and number of points was 194 for each elevation location; average difference was -0.06 ft at same distance and -0.16 ft at different distance; and root mean square was 0.38.

tances from the reference line as field elevations, the mean of the average difference was -0.06 ft. When the photogrammetrist was allowed to select points at significant breaks in the ground, the mean of the average difference was -0.16 ft. Those figures for different distances were obtained by using interpolated photogrammetric elevations. There is a close comparison in the earthwork quantity.

The difference between the figures given in Table 1 can be attributed to two factors. One is that two different photogrammetrists did the interpretation. The second is the result of the error in the interpolated elevation caused by horizontal displacement. From this test we concluded that the method of using interpolated photogrammetric elevations would give sufficiently accurate results.

As the data were being compiled it became apparent that there were obvious blunders in both the field points and the photogrammetric points. As such, it was decided that any elevation comparison showing a difference of 1.0 ft or greater would be excluded from the study. Studies have shown conclusively that the photogrammetrist can interpret the ground elevation within ±0.25 ft; therefore, our elimination of obvious blunders of 1.0 ft or more may be rather conservative. A similar study by the Texas Highway Department eliminated all differences of 0.6 ft or more. The following table gives the distribution of points on the US-14 project for various elevations. Only 3.2 percent of the 1,598 points were eliminated as obvious blunders.

Elevation	Number of Points Compared
0.0 to ±0.5	1,323
±0.6 to ± 0.9	224
±1.0 and higher	51

We divided the US-14 project into various categories of grading and alignment. This was done to determine whether there were any significant differences in the results that might be attributed to the character of the highway design. The division between heavy and light grading was 15 ft at the reference line. The division between tangent and curve was 0 deg 30 min, where curves 0 deg 30 min and under were considered tangent sections and curves 0 deg 30 min and over were considered curved sections.

DISCUSSION OF RESULTS

Table 2 gives comparisons of the photogrammetric and field methods on the US-14 project. The average difference varied from -0.11 to -0.22 ft with the average for the

TABLE 2

COMPARISON OF PHOTOGRAMMETRIC AND FIELD DATA ON US-14 PROJECT

Section Type	Length (ft)	Number of Points	Average Difference[a]	Root Mean Square	Earthwork		
					Field	Photo-grammetric	Percent Difference
Heavy fill, tangent	500	117	-0.14	0.34	61,400	60,954	-0.7
Light fill, tangent	1,500	321	-0.22	0.39	72,148	70,055	-3.0
Light cut, curve	1,200	216	-0.15	0.35	37,496	39,850	+6.3
Heavy cut, curve	1,800	420	-0.20	0.38	267,092	270,825	+1.4
Heavy cut, tangent	2,664	435	-0.11	0.30	508,621	513,305	+0.9
Light cut, tangent	700	89	-0.15	0.23	63,039	64,185	+1.8
All special sections		1,598	-0.15	0.33			

[a]Includes sign.

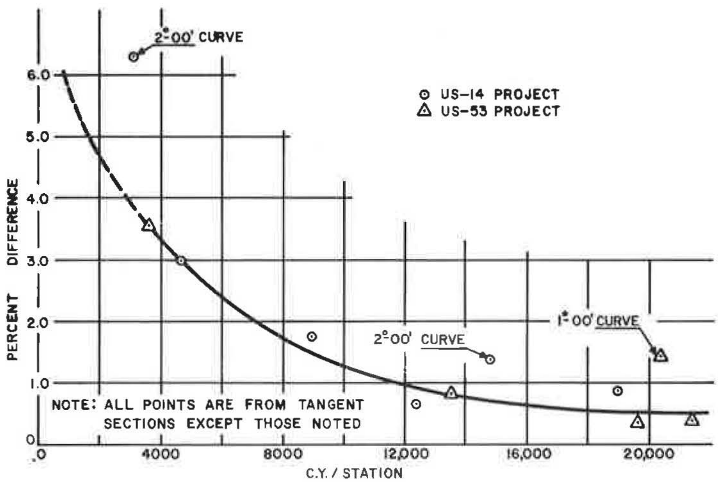

Figure 1. Percent difference between photogrammetric and field-computed earthwork quantity.

entire project being -0.15 ft. The yardage varied from -3.0 to +6.3 percent with the total for the project being +1.2 percent for excavation and -2.4 percent for embankment.

Several observations can be made from these results. There is no apparent correlation among the average differences, the root mean square, and the yardage differences. The percentage of yardage difference is more a function of the amount of grading and the depth of cut and fill. Figure 1 shows the amount of yardage per station and the percent difference between photogrammetric and field-computed yardage. With heavy grading, less than 1 percent difference can be expected. With light grading, minor absolute differences make substantial percentage differences. (Table 3 gives the results of the yardage comparison of all four test projects.)

The US-10 project produced a better correlation than did the US-14 project. The major difference in procedure here was that the "blue-top" elevations were used in the yardage computation. This was necessitated by the fact that the pavement had already been placed when the photographs were taken. This led to the conclusion that, on all photogrammetric final cross sections, the "blue-top" elevations as the project engineer has set them in the field should be used. These elevations are readily available, and their use will eliminate some of the coordination required to get photographs of bare subgrade.

The borrow pit on Wisc-15 had an excellent yardage correlation. In addition, there were 11 random field cross section checks made on this project to verify the photo-

TABLE 3

COMPARISON OF EARTHWORK QUANTITIES ON ALL PROJECTS

Project	Length	Excavation			Embankment		
		Field	Photo-grammetric	Percent Difference	Field	Photo-grammetric	Percent Difference
US-14	12,900	925,747	936,519	+1.2	721,941	704,922	-2.4
US-10	5,400	107,333	107,988	+0.6	58,107	56,795	-2.2
Wisc-15	1,250	86,175	86,434	+0.3			
US-53		1,077,709	1,089,960	+1.1	1,320,998	1,304,476	-1.2
Main line	12,180	1,064,742	1,076,177	+1.1	781,819	773,753	-1.0
Ramps	12,000	12,967	13,783	+6.3	539,179	530,723	-1.6

TABLE 4

COMPARISON OF EARTHWORK QUANTITIES BY TYPE OF GRADING SECTION ON US-53 PROJECT

Station	Section Type	Earthwork		
		Field	Photo-grammetric	Percent Difference
61+00 to 71+00	Heavy fill, tangent	197,999	197,167	-0.4
71+00 to 78+00	Light fill, tangent	26,983	26,070	-3.4
86+00 to 109+88	Heavy cut, curve	480,758	487,401	+1.4
143+00 to 157+00	Heavy cut, tangent	543,891	546,366	+0.5
170+30 to 179+77	Heavy fill, tangent	131,145	130,169	-0.8

grammetric final sections. Although yardage could not be computed from these random sections, the end areas were compared. The field end area came to 35,700 sq ft, and the photogrammetric end area came to 35,614 sq ft, a difference of 0.2 percent.

On the US-53 project, evaluation was again made on several types of grading sections to determine the percent difference related to the volume of earthwork per station. Results similar to those of the US-14 project were obtained and are given in Table 4. Again we found a greater difference in the area of the 1-deg curve than was found on tangent sections.

Figure 1 shows that three points are well above the curve. The data for these points came from curved sections of highway. All of the other points represent data from tangent or slightly curving alignments (0 deg 30 min or less). Although we do not have any apparent reason for this particular phenomenon, we do believe that there is no difference in the photogrammetric procedure that could account for this.

Many leading textbooks indicate that the average-end-area method of computation will produce errors of up to 2 percent in earthwork computations. Thus it would appear that, by using this 2 percent figure as a guide, we could duplicate field-computed earthwork quantities with photogrammetric cross sections any time the grading involved about 7,500 or more cu yd/station on divided highways. This 7,500 cu yd/station would be represented by a uniform 10-ft cut for one station.

Translating the curve shown in Figure 1 into absolute values of difference per station, we find that it varies from about 100 to 150 cu yd per station. In other words, one might expect a difference of this amount regardless of the depth of grading for a dual roadway section. For a single roadway, a difference of maybe 50 to 75 cu yd per station might be expected.

When overlaying the two types of cross sections, we frequently observed one area of difference. Although this was relatively small it may have contributed to the fact that the photogrammetric excavation quantity was always higher than the field-measured quantity. In ditch bottoms the greater number of readings taken photogrammetrically would produce a more rounded section than that produced from field data. An example of this is shown in Figure 2. The quantity involved in this might run anywhere from 20 to 50 cu yd

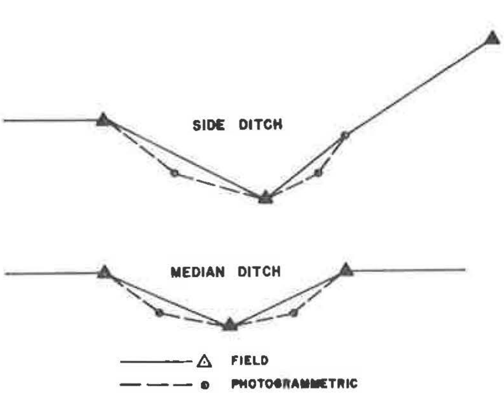

Figure 2. Difference between photogrammetric and field section.

for the dual roadway section. It might be reasonably concluded that, by the use of twice as many points, there will be a truer approximation of the cross section. Likewise, any blunder in the field method will produce twice the impact because of fewer points.

We agree with Dickerson and Warneck (2) that there are fewer sources of blunder in the photogrammetric method. With field sections, three sources of blunder appear: the person who observes the numbers on the level rod and tape; the one who records these observations; and the operator who punches these observations into cards. The photogrammetric method, on the other hand, combines all these operations into one by automatically recording and keypunching the data. Further, the stereo plotter operator does not observe numbers as does the field instrument man but merely places a measuring mark on the ground.

Early in our development of the photogrammetric cross section capability it was discovered that the accuracy of the reference line layout had an important bearing on the accuracy of the output. The more accurate the layout was, the more accurate the cross section was. This is especially true in interchange areas or where wide medians are used. This layout should be prepared to an accuracy of $1/40$ in. Thus, any particular dimension should scale within 1 ft of its true, on-the-ground distance.

Such things as paper shrinkage and poor drafting lead to scale errors. On the US-53 project, all reference line targets were plotted by coordinate position on stable-base drafting film. This method produced an excellent layout.

CONCLUSIONS

1. The percent of yardage differences between photogrammetric and conventional cross sections was more a function of the amount of grading and depth of cut and fill than of the average differences in elevations.
2. On all final photogrammetric cross sections, "blue-top" elevations as set by the project engineer should be used in lieu of photogrammetric elevations of the subgrade.
3. Photogrammetric cross sections should be subject to fewer blunders than conventional field-measured sections.
4. Earthwork quantities computed from photogrammetric final cross sections will produce an accurate estimate usable for payment.

REFERENCES

1. Merrill, R. L. Determination of Accuracies in Earthwork Quantities From Photogrammetrically Made Surveys. Texas Highway Dept., May 1968.
2. Dickerson, L. A., and Warneck, P. E. Comparative Accuracies of Field and Photogrammetric Surveys. Highway Research Record 109, 1966, pp. 49-58.
3. Turner, F. W., and Kassis, A. W. Earthwork Quantities by Photogrammetric Measurements—Foothills Parkway Project 862. Aerial Surveys Branch, Bureau of Public Roads.
4. Meyer, C. F. Route Surveying, 3rd Ed. International Textbook Co., Scranton, 1968.
5. Bouchard and Moffitt. Surveying, 5th Ed. International Texbook Co., Scranton, 1966.

DEVELOPMENT AND PRODUCTION OF TWO-COLOR AERIAL PHOTOMOSAIC CONTRACT PLANS

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Wisconsin Department of Transportation

The increased complexity of highway design and construction has prompted the use of aerial photography and two-color printing to depict proposed designs more easily. A soft green ink is used to show the aerial photograph background and black ink shows the proposed designs. The technique is a departure from traditional drafting methods in that a base drawing is used for one color (green) and an overlay drawing for the other color (black). This method is compatible with the modern "scissors drafting" or "stick-up" technique now being used by many state highway departments. Results have shown greater clarity and increased flexibility in the use of aerial photography and better results in reproductions from black-and-white microfilm.

•THE Division of Highways, Wisconsin Department of Transportation, has developed a procedure to produce right-of-way and construction site plan sheets by using aerial photomosaics and two-color printing. This development dates back many years when it was felt that considerable effort was being spent to acquire conventional survey data and to draft these data merely for "picture" purposes. As a result, experimentation into the use of aerial photomosaics for plan sheets was begun.

We recognized early that we could not expect a true scale picture without going to a fully controlled ortho-photographic process; otherwise, the picture would be subject to all of the distortions inherent in normal aerial photography. If the distortions could be minimized in the area of the highway construction, however, the result would be satisfactory.

In addition to the cost savings generated by reduced field survey time and reduced drafting time, it was found that identification and the ability to "read" the plan were much better with photographs. This was especially true among the nonengineers such as property owners and local officials who were not accustomed to viewing typical engineering, line drawings.

However, after plans were produced for a couple of years by this method, it became increasingly apparent that the full utilization was being impaired by the black-and-white printing. When both the photograph background and the drafted engineering information were printed in black ink, there were areas where it was difficult to distinguish between the two, especially where there were shadows of buildings or other dark spots. This problem became even more apparent when microfilming of "as-built" construction plans began. Through the microfilm process of reducing to a small negative and then enlarging, much of the line work was lost in the background, which became much darker.

Thus a procedure was developed to print the right-of-way and construction plans in two colors: the background aerial photomosaic in a light green and the right-of-way and construction information in black. The "readability" of this plan is vastly improved, and microfilming is no longer a problem. It was also found that the two-color process has an excellent application to complex highway construction plans where aerial photography is not used. The existing conditions are shown with green, and the new con-

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struction is shown with black. Thus, much of the confusion caused by the multitude of all-black lines is avoided, and the plan is easier to use.

The preparation of the two-color plan using aerial photography requires a completely different concept of plan production than the traditional production methods. These procedures are described. The Appendix gives definitions of terms used in this paper.

AERIAL PHOTOGRAPHY

Prior to photography, field crews place white targets on selected stations of the surveyed reference line. These targets show up on the photographs and can be used to enlarge the photographs to a reasonably accurate scale. Targets are placed at a maximum spacing of $2\frac{1}{2}$ times the photograph scale so that at least 3 targets appear on each photograph. Targets can be made of white muslin, white paint, or other suitable material. If the placement of targets is not feasible, identifiable photograph images can be used for scale rectification, but it is not so desirable.

Most projects are photographed at a height of 2,400 ft by using a 6-in. focal length camera. The 1-in. to 400-ft scale photographs are then enlarged four times to the plan scale. On urban projects where plan scales are either 1 in. = 20 ft or 1 in. = 50 ft; photographs are taken at 1,200 ft (1 in. = 200 ft). Flight height restrictions over urban areas prohibit lower flying. Photographs are exposed with 60 percent overlap, and, when only the centers are used, distortion can be minimized.

The majority of the photographs were taken in the spring and fall when foliage is limited; however, where there are few trees, summer photography has been done with good results.

ENLARGEMENT-RECTIFICATION

A scaled pencil drawing of the survey alignment must be prepared prior to the enlarging process. The scale should be that of the desired plan sheet. On this drawing must be shown the placement of all targets (or photographic images). This drawing can be on vellum or any other suitable drafting material. Although vellum is subject to some shrinkage and expansion because of changing humidity, this does not seriously impair the scale of the end product.

The enlargement-rectification is done on a HE-12 enlarger. This camera is equipped with a four-way tilting easel such that some of the X- and Y-tilt of the photograph can be removed. The scaled drawing is placed on the easel, and the photograph is projected onto the drawing. Adjustments are made with the easel and the scale until the best fit is obtained on all targets.

When a 65-line, squared-dot, gray screen is used, a straight-up (emulsion-up), screened positive is produced. These screened positives are then laid over the scaled drawing to ensure accurate butt-splicing between photographs. After it is spliced, the photomosaic is trimmed to proper size and fitted into a standard base vehicle, and a reverse (emulsion-down) contact cronaflex positive (CCP) plan sheet is printed. This becomes the original plan base drawing for two-color printing.

PLAN PREPARATION

The preparation of a two-color plan requires production of a base drawing and an overlay drawing for each sheet of the plan (one drawing for each color). The base drawing will be reproduced in green, and the overlay drawing will be reproduced in black.

The standard base vehicle for the plan and profile sheet contains orientation marks and registration marks (Fig. 1). The orientation marks will be used for aligning the overlay with the base drawing during the drafting process. The orientation marks show the location of the borders and the title block in the upper right corner of the sheet. The registration marks are used in the offset printing process for registering the paper printing plates.

Any information to be printed in green must be added to the original plan base drawing. If this base drawing contains the aerial photograph, the information must be drafted directly onto the base drawing; the "stick-up" technique cannot be used. All information

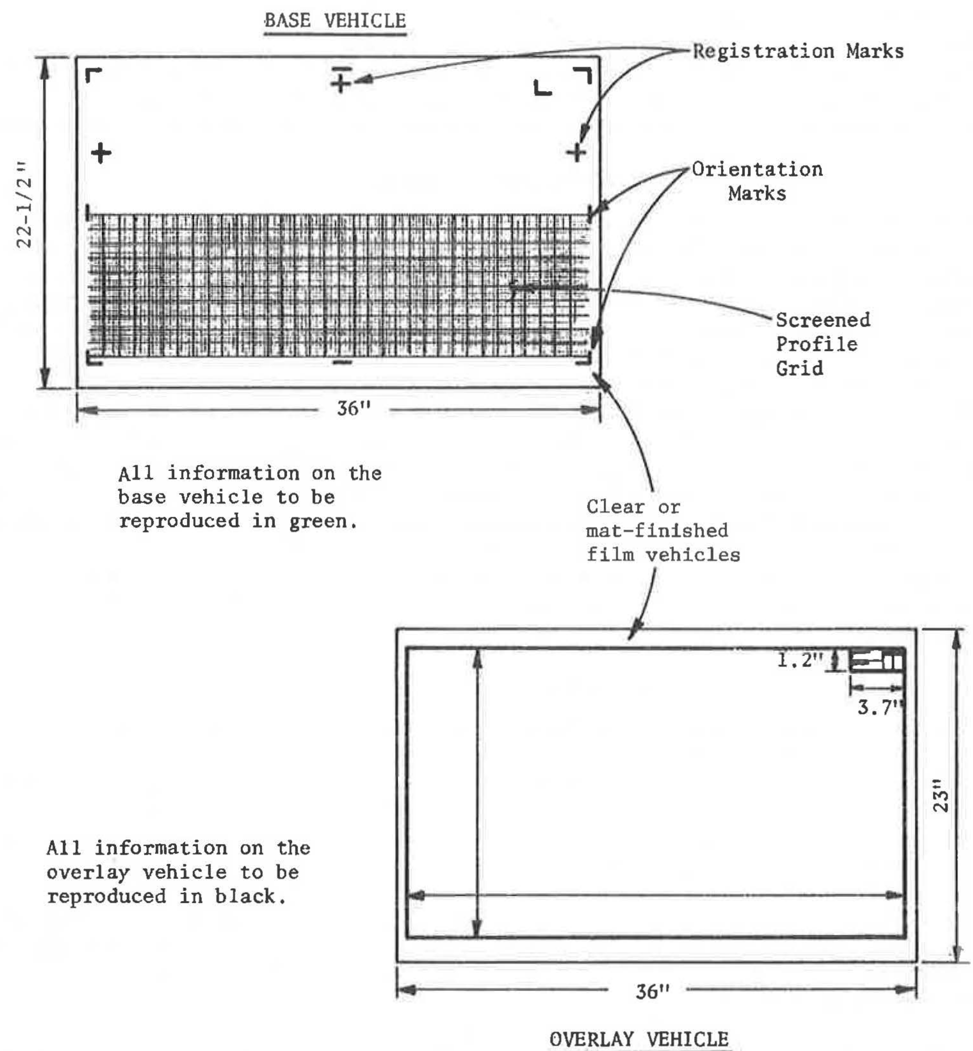

Figure 1. Standard Vehicles for Plan and profile sheets.

to be shown in black must be placed on the overlay. Two methods of preparing the overlay drawing are used. The first method is to use the stick-up technique and then to produce a mylar reproduction.

The steps used to produce a mylar reproduction are as follows (numbers are keyed to those shown in Fig. 2): The trimmed aerial photograph halftone (step 1) is positioned on the clear base vehicle (step 2). The photographic contract print process produces a first-generation reverse halftone contact cronaflex positive (CCP)(step 3). Pieces of drafting material are positioned on the original plan base drawing (step 4). The plan and profile designs are drawn (step 5). When complete, the design drawings are removed from the original plan base drawing (step 6). A clear overlay vehicle is positioned and secured over the original plan base vehicle (step 7). Plan and profile designs are repositioned and attached to the clear overlay vehicle (step 8). When complete, the original plan base vehicle and "stuck-up" design overlay vehicle are separated (step 9). The "stuck-up" design overlay vehicle is reproduced (step 10). This

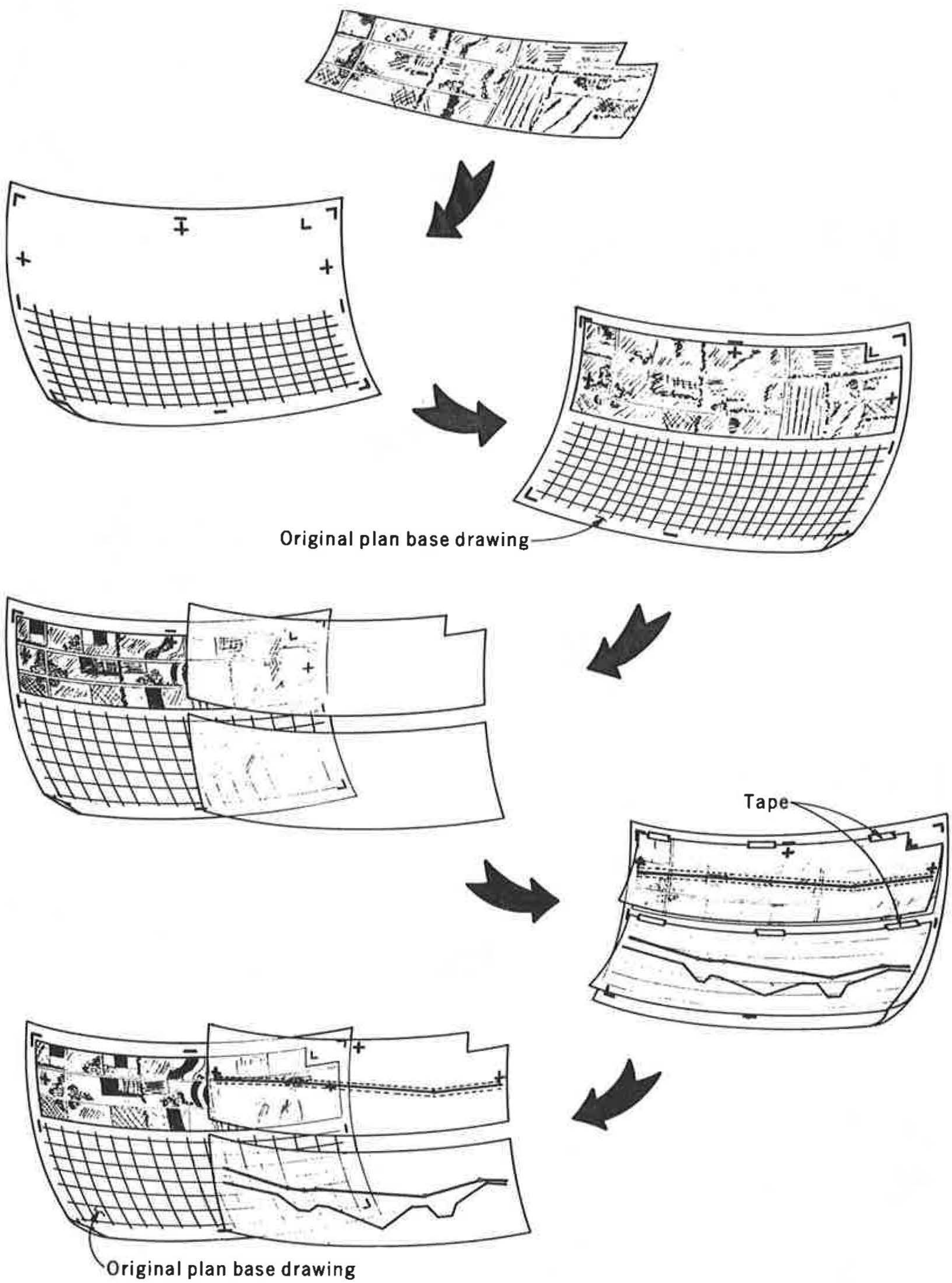

Figure 2. Steps in preparing mylar reproduction.

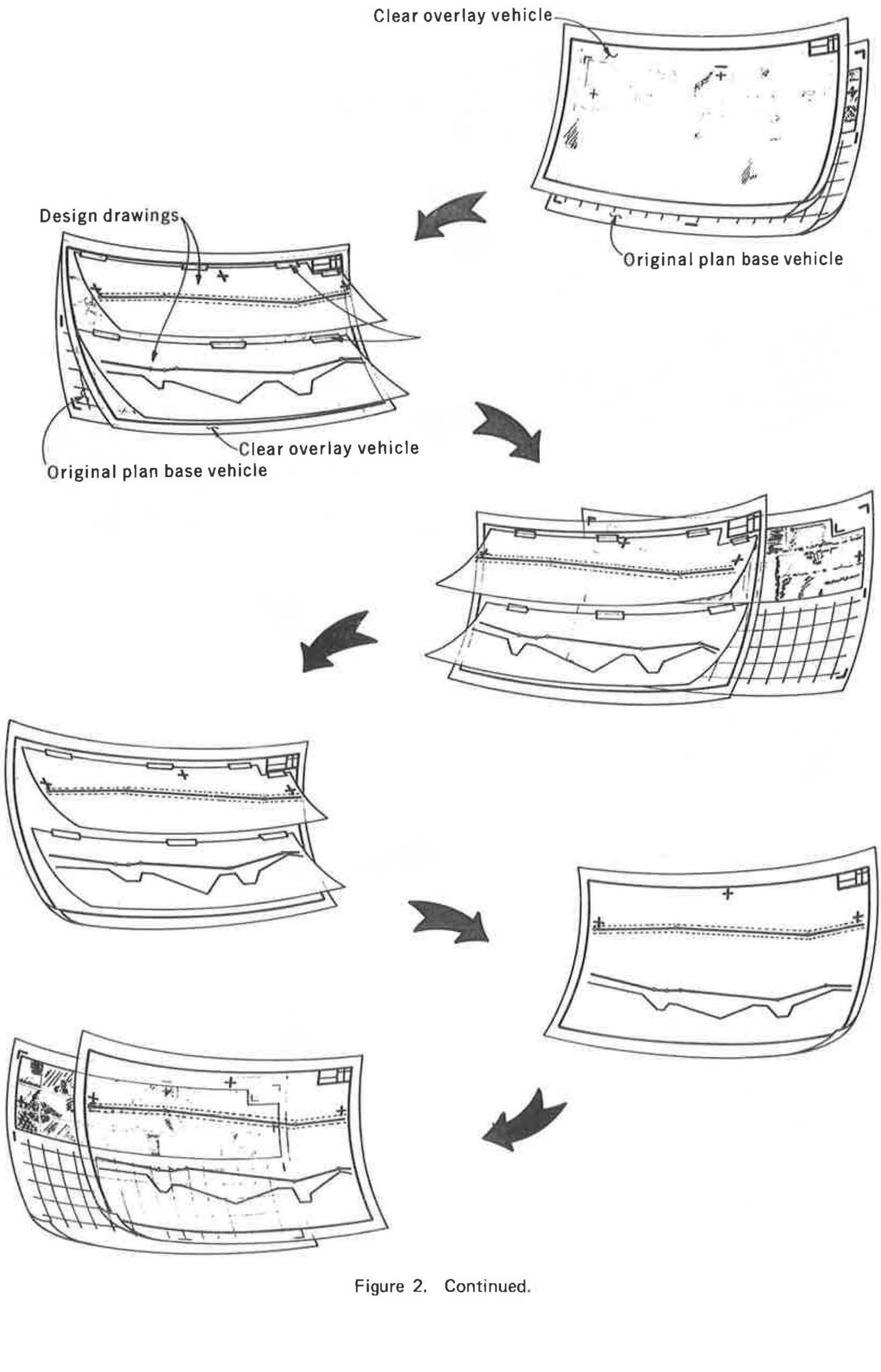

Figure 2. Continued.

yields a first-generation reverse mylar vehicle that is the original plan overlay drawing (step 11). This drawing, along with the original plan base drawing, is submitted for precontract administration (step 12).

After the drawings, original plan base drawings and original plan overlay drawings, are thoroughly checked for accuracy in precontract administration, the sheets are reproduced for distribution as contract plans. The sheets are separated, photographed, and reduced to half-size negatives from which paper plates for offset printing are made. On the first run through the offset press the green ink is printed; on the second run, the black ink is printed, producing the finished half-size plan sheet in black and green.

The second method is to draft directly onto the standard overlay. A mat-finished CCP of the standard overlay for the plan and profile sheets is placed over the base drawing, the border corners superimposed over the appropriate orientation marks. The two are taped together and the three registration marks are accurately traced onto the overlay. The remainder of the proposed plan and profile information is drafted directly onto the overlay.

At the time the plans are submitted for contract letting, there should be no visible discrepancy between the base drawing and the overlay vehicle. Normal expansion of the polyester material due to temperature and humidity will occur but will be much less than that of other drafting materials. Acceptable visual accuracy is achieved when registration marks are matched, when the proposed plan information falls into proper alignment with respect to the existing conditions on the base drawing, and when the profile line, corresponding elevations, and stationing fall on the proper profile grids.

PRINTING

When the plans have been approved for letting, they are sent to the reproduction unit. Here the base drawing and the overlay drawing are photographically reduced to half-size negatives. Paper offset printing plates are then made from the negatives. The plates for all of the base drawings are printed in green ink on a 15- by 18-in. offset press. Then the plates for the overlay drawings are used to overprint in black ink. This press will print only one color at a time, although presses are available that will print two colors in one operation.

The plates of the overlay drawing are adjusted to the print of the base drawing by the registration marks. The maximum allowable error for registering the plates is $\frac{1}{32}$ in. This degree of accuracy was found to be the most reasonable maximum error that could be maintained throughout the entire run with the methods and duplicating equipment used.

Accurate correlation between plan sheet information and registration marks is essential because the reproduction unit registers solely on the registration marks. The registration marks on the two drawings must match if overall accuracy is to be maintained.

APPENDIX

DEFINITIONS

Vehicle—a photographically processed polyester film sheet with a positive image used to hold drawings, details, notes, and so forth for reproduction. The vehicle may have a clear finish or a mat (frosted) finish.

Bass vehicle—a plan vehicle containing registration or orientation marks or both used to hold drawings, notes, mosaics, and the like that depict existing conditions for reproduction. A base vehicle is used with an overlay vehicle to produce colored plans.

Overlay vehicle—a plan vehicle containing the standard border and title block format for either a plan and profile sheet, a right-of-way sheet, or a construction detail sheet. An overlay vehicle is used with a base vehicle to produce a colored plan sheet.

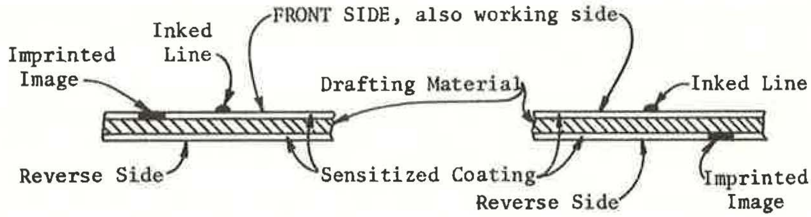

Figure 3. Straight-up versus reverse reproductions.

Original base/overlay drawing–any vehicle to which drawings or notes or both are attached. Also any original drawing with the base and overlay registration marks on which the plan information is presented in the form of an original ink drawing or mylar reproduction suitable for reproduction to half size for contract plan.

Registration marks–tick marks (+) printed on the base vehicles and used for registering the overlay vehicle to the base vehicle. These marks provide the guide for both the draftsman and the printer to register the two original drawings, one for each color.

Orientation marks–tick marks (⏋) printed on the base vehicles showing the corners of the standard border format. In the upper right corner of the base vehicle, additional tick marks are shown to outline the area occupied by the standard title blocks.

First-, second-, or third-generation reproduction–the number of times that an original drawing has been reproduced through successive intermediate reproductions from the original base/overlay drawings.

Straight-up and reverse–terms used to designate on which side of the reproduced drawing the image is printed. If the printed image is on the side of the sheet toward the reader, this is the straight-up side; if it is on the back or reverse side, this is the reverse side (Fig. 3).

Screening–the reproduction process used to allow the image, a continuous-tone photograph, or other drawing to be broken up into a dot-like or screened patterns so that it may be printed by the offset process.

Original plans–the linen sheets or mylar reproductions or both of the original base/overlay drawings that are submitted for processing into a contract plan.

AN EXPERIMENTAL INVESTIGATION OF HIGHWAY SURVEYING AND MAPPING CONTROL EXTENSION BY TRILATERATION AND CONVENTIONAL TRAVERSE

Shuh-Chai Lee and Anatol Leo Belkin, Ohio Department of Highways

A chain of 16 quadrilaterals approximately 6 miles in length along I-71 was used to evaluate ground control extension by trilateration and conventional traverse. Distances were measured with a Geodimeter, and angles were measured with a Wild T2 theodolite. A FORTRAN computer program was developed to adjust the trilateration network. The method is based on a previously developed and published method of trilateration adjustments. The traverse adjustment was accomplished by using the least squares method. Results of the experiments show that trilateration is more accurate than conventional traverse and is at least as economical. The authors recommended that a trilateration chain of quadrilaterals be used where a double centerline or double traverse is needed for highway surveying or mapping control. The methodology and computer program have permitted this method to be used wherever economically advantageous.

•THIS is the abridged version of the fourth report of a trilateration study. The first paper (2) established the basic ideas of a trilateration scheme and its adjustment by use of the area equation for fundamental figures. The second paper (3) dealt mainly with the geodetic scheme of trilateration and its adjustment. The third paper (4) explained easy application of the area adjustment method for engineers and surveyors.

This paper reports a trilateration experiment using conventional traverse in highway mapping control. The experiment was done intermittently from the summer of 1965 through the winter of 1967 on a spare-time basis by the field survey party of the aerial engineering section of the Ohio Department of Highways. The computer programs were developed from the summer of 1967 through early 1969.

THE SCHEMES OF THE EXPERIMENT

The experimental network is located primarily along I-71 in Delaware County, Ohio, about 17 miles north of Columbus. The network spans between two U. S. Coast and Geodetic Survey (CGS) first-order triangulation monuments, Shannahan (established 1928, abbreviated S) at the south end and Galena (established 1933, abbreviated G) at the north end. The geodetic positions, the state plane coordinates, and other related data for these two points are listed in the Horizontal Control Data published by the CGS.

The two points were not directly connected and observed. S belongs to the north-south triangulation chain, and G belongs to the east-west triangulation chain. Direct measurement by Geodimeter of the distance between the two triangulation monuments was attempted but found to be impractical because too much work would be involved in establishing towers to overcome the obstacles along the line of sight.

According to the CGS (11), the scale factors at latitude 40 deg 10 min are $\frac{1}{16,200}$ too great for north zone plane coordinates and $\frac{1}{34,400}$ too great for south zone plane coordinates, and at latitude 40 deg 11 min are $\frac{1}{19,000}$ too great for north zone plane coordinates and $\frac{1}{32,000}$ too great for south zone plane coordinates. Therefore, the south zone

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plane coordinates for the state of Ohio were used in all computations because they were comparatively more accurate.

Reference mark 2 (abbreviated A) of station S was used as one vertex of the first quadrilateral in the network. This point serves not only as a point in the network but also as the azimuth mark from S in order to use the original grid azimuth value in the preliminary orientation of the network. The distance between X and A was also measured with the Geodimeter to check the accuracy of the original distance value obtained by CGS in 1928. The results are as follows:

Item	Year	Distance (ft)
CGS, original	1928	691.272
Geodimeter	1967	690.244
Geodimeter, reduced (sea level)	1969	690.212
Adjusted by trilateration	1969	690.214

To fit the usual highway control situation required that the network consist primarily of a chain of quadrilaterals with braced diagonals for trilateration investigation. A comparison was made with traverses formed by the external sides of the quadrilaterals. The basic geometric features and related field work of the two kinds of experiments are shown in Figures 1 and 2.

FIELD OBSERVATION AND REDUCTION OF GEODIMETER DISTANCES AND THEODOLITE ANGLES

Field observations of Geodimeter distances and theodolite angles for trilateration and traverse experimental networks were done in the usual manner as it has been practiced by the field crews of the aerial engineering section in recent years.

The reduced distances at sea level were used for trilateration and traverse computations. In the trilateration and traverse adjustment, a weight was assigned to each averaged distance according to the number of reduced sea level distances used for the average. (A weighting scheme according to the standard error of the measured distances was not used because maximum spread, and not standard error, was the only available value.)

All the horizontal angles have three sets of measurements. Therefore, the weights of the horizontal angles are assumed to be equal.

ADJUSTMENT OF THE TRILATERATION CHAIN

The method of adjustment of plane trilateration in fundamental figures by area equations developed in the first report of this series was used to adjust the quadrilateral chain of the experimental trilateration. The basic theory and equations have been treated thoroughly and published in previous reports. The practical application of the method was facilitated by use of an electronic computer in solving a large number of simultaneous equations. The FORTRAN IV computer program will be made available in a separate report (6). This report will deal only with data that have been used, the procedures followed, and results obtained.

As shown in Figure 1, there are 16 quadrilaterals in the network. The areas of triangles were computed by using the distance values of each side of the four triangles of each quadrilateral. The discrepancies between the area sums of each pair of opposite triangles in all of the 16 quadrilaterals and their relative ratios are given in Table 1. The three largest area errors are noted.

The purpose of the trilateration adjustment is to apply corrections to each side of the quadrilaterals so as to eliminate the area errors while keeping the sum of the squares of the corrections at a minimum.

In the computer program for trilateration adjustment, the input data were the 81 averaged sea level distances from the Geodimeter measurements and their proper weights. After the processing, the residual or the correction, the corrected value,

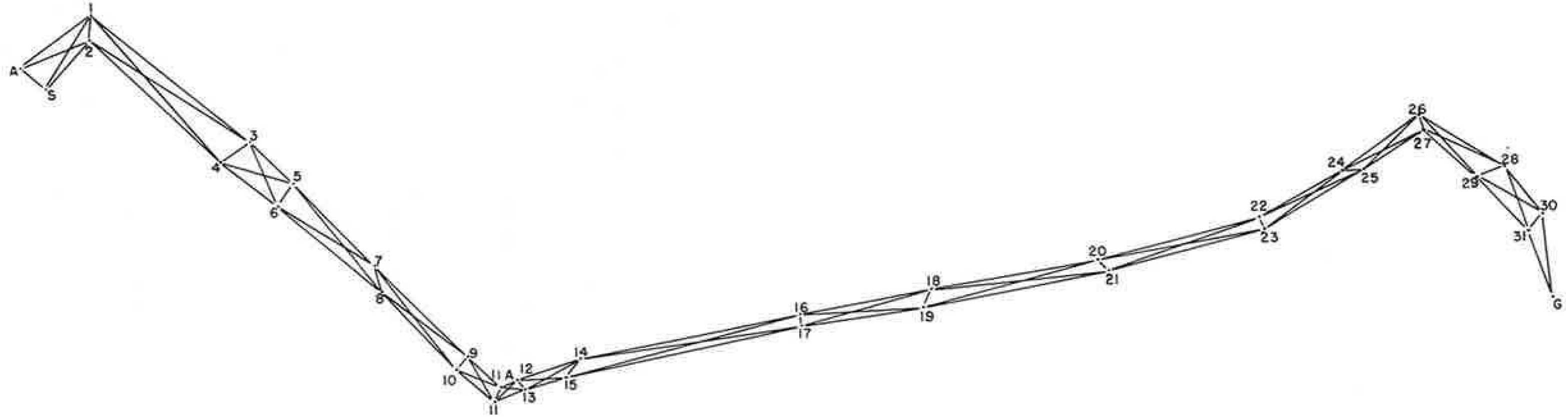

Figure 1. Geometric features and field work involved in trilateration network.

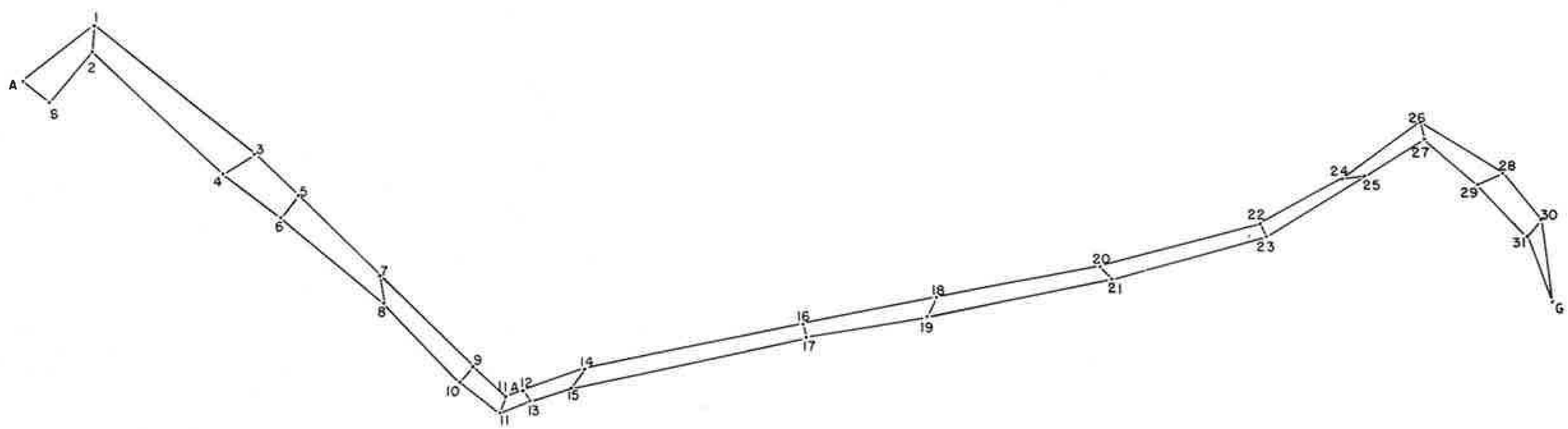

Figure 2. Geometric features and field work involved in traverse network.

TABLE 1

AREA ERRORS OF THE QUADRILATERALS OF EXPERIMENTAL TRILATERATION

Quadrilateral	Side-Length Ratio[a]	Area Sum (sq ft)	Area Error (sq ft)	Relative Error (1 in)
1	4.345	+645,178.209045 -645,200.004112	-21.795067	29,603
2	11.446	+1,728,819.656384 -1,728,821.294457	-1.638073	1,055,399
3	5.392	+536,354.382108 -536,379.650455	-25.268374	21,227
4	14.168	+608,719.033576 -608,664.446298	54.587278	11,151
5	8.483	+423,280.549834 -423,258.345555	12.204379	34,682
6	3.816	+448,860.854501 -448,876.703583	-15.839082	28.339
7	3.466	+240,672.253517 -240,649.371756	22.881761	10,518
8	3.616	+242,982.566471 -242,993.874676	-11.308205	21,488
9	21.655	+1,296,629.342527 -1,296,588.007834	41.334693	31,369
10	10.619	+517,974.509942 -517,822.283861	152.226081	3,402[b]
11	12.333	+1,021,591.253596 -1,021,555.186423	36.167173	28,246
12	13.136	+874,354.161284 -874,510.336224	-156.174940	5,599[b]
13	10.065	+505,890.469942 -505,515.604169	374.865773	1,349[b]
14	4.357	+455,782.294891 -455,762.123519	20.171372	22,595
15	4.122	+933,583.158365 -933,598.483824	-15.325460	60,918
16	3.808	+883,385.387404 -883,412.136686	-26.749282	33,025

[a]Side-length ratio = longest side/shortest side.
[b]Largest errors.

and the relative error of each distance are obtained. The accuracy of the trilateration will be discussed later.

In the adjustment of the experimental trilateration, orientation and scaling did not enter into the problem because the accuracy of the known azimuth angles and the distance between the two known triangulation stations were not necessarily better than the Geodimeter measurements. Because the rigorous simultaneous adjustment of distances, directions, and coordinates would mask the error contribution of the Geodimeter distances (which is the primary concern in this experiment), the authors did not use the rigorous method as used in geodetic adjustment.

ORIENTATION AND COMPUTATION OF THE UNADJUSTED AND THE ADJUSTED TRILATERATION

As stated earlier, the coordinates of a first-order triangulation point at each end of the trilateration network were known, and an azimuth mark at one end was also used as one point of the quadrilaterals. Thus, the trilateration was computed and oriented between these two known points.

The plane coordinates and orientation of the azimuths were computed by using the trilateration distances and the angles computed from the distances. The computer program used was the M.I.T. Integrated Civil Engineering System COGO (8). Two tests have been made with the computation. One test used the adjusted lengths of the trilateration; the other test used the observed distances of the original unadjusted trilateration. In both cases, computations were carried out through two different simple triangle chains of the trilateration scheme to the ending point G from the starting point S by using the preliminary grid azimuth from S to A, 90 deg 57 min 58 sec from south. By keeping the grid azimuth from S to G, 235 deg 04 min 49.31 sec from south, which was computed

from the given coordinates as fixed, we found the azimuth from S to A by trial and error as follows: northward through left chain of adjusted trilateration, 90 deg 57 min 04.52 sec; northward through right chain of adjusted trilateration, 90 deg 57 min 04.06 sec; northward through left chain of unadjusted trilateration, 90 deg 59 min 08.81 sec; and northward through right chain of unadjusted trilateration, 90 deg 56 min 56.74 sec.

It can be seen that the discrepancy is much less between the adjusted trilateration values than between the unadjusted trilateration values. Similar results were obtained with the computation of the grid distance from S to G by using the computed coordinates: northward through left chain of adjusted trilateration, 31,722.715 ft; northward through right chain of adjusted trilateration, 31,722.714 ft; northward through left chain of unadjusted trilateration, 31,721.122 ft; and northward through right chain of unadjusted trilateration, 31,724.047 ft.

The geodetic and grid distances from S to G as computed from the coordinates given by CGS are 31,721.473 ft and 31,722.127 respectively. The difference in the computations of the grid distance through either chain of the adjusted trilateration is only 0.001 ft, or 0.588 and 0.587 ft from the CGS grid distance. Therefore, use of the adjusted trilateration should be standard practice in orientation and computation of trilateration coordinates.

Because the trilateration was not adjusted by considering the orientation and scaling errors simultaneously, there was a discrepancy between the final coordinates of the adjusted trilateration at G and the coordinates of G given by CGS. To simplify the adjustment of this discrepancy, we computed two simple traverses, which can also be designated as west chain of legs and east chain of legs, for each simple triangle chain of adjusted and unadjusted trilaterations and adjusted them by COGO.

The computed coordinates of the points of both the adjusted and unadjusted trilateration of computation with chain of triangles and chain of legs along with the differences between these coordinates and the sums of squares of these differences are omitted in this account. The magnitudes of the sums of the squares of the differences of the coordinates are given in Table 2. From the magnitudes of the sums of the squares of the differences of the west from east coordinates of the adjusted chain of triangles and of the adjusted chain of legs, it can be seen that there is no appreciable difference between computing either from the west or the east chain of triangles or chain of legs of the adjusted trilateration.

The magnitude of the sums of squares of the differences of coordinates computed from the west chain of triangles and the west chain of legs for adjusted and for unadjusted trilateration and from the east chain of triangles and the east chain of legs for adjusted and for unadjusted trilateration is of the same order, but the values are larger in the computations obtained from the unadjusted trilateration. Therefore, the differences of computation between chain of triangles and chain of legs are not significant, but the differences between that of adjusted and unadjusted trilaterations are significant. The latter statement is strongly supported by the sums of squares of the differences of coordinates of the computations of the unadjusted trilateration either in chain of triangles or in chain of legs and the differences between the adjusted and the unadjusted trilaterations in the west chain of triangle and chain of legs. The small differences of the east chain of triangles and east chain of legs between the adjusted and unadjusted trilaterations may indicate that the computation from the unadjusted trilateration is irregular.

Finally, in order to see the change in distances after the final coordinates have

TABLE 2

SUM OF THE SQUARES OF THE DIFFERENCES OF THE COORDINATES

Trilateration	Sum of x^2	Sum of y^2
Adjusted trilateration		
West triangles and legs	5.150	3.477
East triangles and legs	5.125	2.808
West and east legs	0.0003	0.0003
West and east triangles	0.0002	
Unadjusted trilateration		
West triangles and legs	13.304	15.927
East triangles and legs	55.383	37.826
West and east legs	1,167.999	6,075.095
West and east triangles	952.672	4,857.954
Adjusted and unadjusted trilateration		
West triangles	957.894	4,762.219
West legs	1,086.693	5,441.824
East triangles	10.597	3.572
East legs	5.379	21.194

been computed through adjustment of orientation and scaling of the adjusted trilateration, we calculated the grid distances for corresponding Geodimeter distances of the trilateration quadrilaterals from the coordinates by COGO. The absolute change from the observed distances and their relative changes are also computed but are not shown here.

ACCURACY OF THE TRILATERATION IN THE EXPERIMENT

The accuracy of the trilateration depends on the accuracy not only of the individual distance measurement but also of the network distance measurement. Both aspects are of primary interest in this section. Let us first examine the accuracy of the individual distance measurements. For two measurements, it can be shown that the standard deviation is one-half of the "spread." Therefore, standard error and spread are directly related for two measurements.

For the Geodimeter measurements, a distance was observed with three different frequencies. The maximum spread for each pair of frequencies may be used as an indication of the accuracy of the Geodimeter distance. For easy comparison, the relative error, i. e., the maximum spread divided by the average distance and expressed as a unit fraction of the distance, was selectively computed for the short distances with the following significant maximum spread:

Station	Distance (ft)	Maximum Spread (ft)	Relative Error (1 in)
5-6	278.6631	0.1147	2,430
13-12	323.7927	0.1296	2,498
23-22	267.8335	0.2093	1,280
21-20	330.1614	0.1982	1,666

It seems that the errors of these four distances are relatively quite large. The statistics for the errors of all the distances are as follows:

Class of Relative Error (1 in)	Number of Geodimeter Distances
1,280	
	8
5,000	
	25
10,000	
	62
50,000	
	9
100,000	
	4
430,000	
Total	108

From statistics, we conclude that approximately 60 percent of the Geodimeter distances have accuracies in the range of $\frac{1}{10,000}$ to $\frac{1}{50,000}$.

The accuracy of the trilateration network as a whole can be examined in several ways.

1. The absolute and relative area errors of the quadrilaterals have the same pattern in magnitude (Table 1). Therefore, the area errors are independent of the area size. Quadrilaterals 13, 12, and 10 have, in that order, the largest absolute and relative area errors and, especially, quadrilaterals 12 and 13, where the distances 20-21 and 22-23, with the largest relative distance errors, lay. Therefore, the accuracy of the trilateration network depends on the accuracy of the individual distances. Also, there seems to be no apparent relationship between the area errors and the side-length ratio. Therefore, the area errors are also independent of the area shape.

2. The corrections of the distances in the trilateration adjustment is another indication of the accuracy of the network. The three largest absolute corrections of the distances are -0.0351, -0.0265, and -0.0262, and the three largest relative adjustments are only $^1/_{25,141}$, $^1/_{31,877}$, and $^1/_{36,009}$. The standard deviation for the distance of unit weight in the trilateration network as computed from the corrections is +0.0146 ft.

3. The grid distance from S to G as computed from the state plane coordinates is 31,722.127 ft. The discrepancies between the state plane coordinate distance and the trilateration distance of S to G (given earlier) and their relative errors are listed in the following:

Network	Absolute Discrepancy (ft)	Relative Discrepancy (ft)
Adjusted trilateration		
Left chain	0.588	54,041
Right chain	0.587	54,041
Unadjusted trilateration		
Left chain	1.005	31,563
Right chain	1.920	16,523

This table shows that the experimental trilateration distances are in excellent agreement with the values computed from the CGS data. There is some uncertainty as to which of these two distances is the more accurate inasmuch as a direct measurement between the two points could not be made.

4. The errors of closure of the chain of legs between the CGS triangulation points may be used as another indication of the accuracy of the trilateration. These are given in Table 3.

5. From the sums of squares of the differences of the coordinates computed from the adjusted and unadjusted triangulation through left or right chain of triangles and chain of legs, the accuracy of the trilateration coordinates can be computed as given in Table 4. Clearly, the coordinates computed from the adjusted trilateration have a higher degree of accuracy than do those computed from the unadjusted trilateration.

6. The three largest absolute changes of the grid distances from the observed distances are -0.083, +0.080, and -0.072 ft. Eight of the 83 relative changes are greater than $^1/_{10,000}$. The root-mean-square change of the grid distance is ±0.0326 ft.

ADJUSTMENT AND COMPUTATION OF THE TRAVERSE COORDINATES

The M.I.T. COGO program was used in the adjustment and computation of the experimental traverses. In the program, there are four methods for traverse adjustment: compass rule, transit rule, Grandall's method, and method of least squares. The method of least squares has been used throughout because it is comparable to the method used for adjustment of the trilateration.

In the method of least squares, a weighting scheme for angles and distances can be used. The preliminary angular closure error is adjusted first. The errors of closure in the latitudes and departures are then adjusted. The angular errors, linear errors, and relative errors for all the quadrilaterals in the network are given in Table 5. No relative errors are larger than $^1/_{10,000}$.

TABLE 3

ERRORS OF CLOSURE

Network	Error of Closure (ft)	Relative Accuracy (1 in)
Adjusted trilateration		
Left legs of left chain	0.588	66,927
Right legs of left chain	0.587	64,050
Left legs of right chain	0.587	64,088
Right legs of right chain	0.587	66,999
Unadjusted trilateration		
Left legs of left chain	1.005	39,141
Right legs of left chain	1.760	21,381
Left legs of right chain	1.920	19,598
Right legs of right chain	1.920	20,488

TABLE 4

ACCURACY IN TERMS OF THE ROOT-MEAN-SQUARE DIFFERENCES OF THE COMPUTED GRID COORDINATES OF EXPERIMENTAL TRILATERATION AND TRAVERSES

Trilateration and Traverse	x (ft)	y (ft)	Linear Resultant (ft)
Adjusted trilateration			
West and east triangles	0.002	0.003	0.003
West and east legs	0.003	0.003	0.004
West triangles and legs	0.389	0.320	0.504
East triangles and legs	0.388	0.287	0.483
Unadjusted trilateration			
West and east triangles	5.293	11.956	13.075
West and east legs	5.861	13.367	14.595
West triangles and legs	0.626	0.684	0.927
East triangles and legs	0.276	1.056	1.656
Unadjusted and adjusted trilateration			
West triangles	5.307	11.835	13.021
West legs	5.653	12.696	13.898
East triangles	0.558	0.324	0.645
East legs	0.398	0.790	0.891
Traverses			
Quadrilateral chain and long loop	1.034	0.546	1.169
Quadrilateral chain and simple traverse	0.449	0.865	0.975
Long loop polygon and simple traverse	1.194	1.182	1.680

The experimental traverse in the network consists of 16 quadrilaterals and one triangle as in trilateration but without the braced diagonals in the quadrilaterals. After the field work had been completed over a 2-year span, it was discovered during computation that there were discrepancies between the distance and angular measurements of quadrilaterals 12 and 13, which could not be checked with each other. Therefore, quadrilaterals 12 and 13 had to be combined into one polygon. Also, in quadrilateral 16

TABLE 5

ERRORS OF CLOSURE OF QUADRILATERALS AND POLYGONS OF EXPERIMENTAL TRAVERSES

Quadrilateral or Polygon	Angular Error (min)	Perimeter (ft)	Linear Error	Relative Error (1 in)
1	5 x 0.82	3,812.855	0.039	97,579
2	4 x 5.17	8,493.345	0.101	83,943
3	4 x 6.61	2,912.258	0.250	11,639
4	4 x 9.38	6,387.051	0.173	36,830
5	4 x 7.00	3,921.232	0.237	16,548
6	4 x 15.01	2,801.919	0.092	30,313
7	4 x 9.00	2,052.097	0.047	43,775
8	4 x 2.37	2,249.062	0.075	29,849
9	4 x 5.21	10,531.266	0.294	10,531
10	4 x 11.62	5,215.380	0.068	76,675
11	4 x 4.54	8,014.118	0.266	30,119
12-13	6 x 4.14	12,064.549	0.115	104,910
14	4 x 3.23	4,015.470	0.055	73,035
15	4 x 0.71	4,353.260	0.120	36,359
16	—	3,861.025	0.111	24,695
Triangle	—	2,592.559	0.115	22,455
Long loop polygon	33 x 4.10	71,700.172	2.102	34,105
Quadrilateral chain	—	3,861.025	0.108	35,633
Triangle	—	2,592.559	0.115	22,487

two angular measurements were missing. Therefore, quadrilateral 16 does not have an angular error of closure. These two exceptions are given in Table 5.

The traverse chain of quadrilaterals was also oriented and computed following the same procedure as for trilateration except that only one route of computation was possible for the adjusted traverse chain.

The azimuth of Shannahan to its reference mark 2 (S to A) was found to be 90 deg 57 min 57.01 sec from south. The grid distance from S to G computed from the coordinates of the adjusted quadrilateral traverse was 31,720.127 ft computed from CGS data.

The computed coordinates of the points of the adjusted quadrilateral traverse chain are omitted here. In order to compare the distances obtained by the adjusted traverse chain with those original Geodimeter distances obtained in the field, i. e., the unadjusted traverse or trilateration distances, we obtained the grid distances computed from the adjusted traverse.

No attempt was made to orient and compute the unadjusted traverse chain of quadrilaterals for experimental purposes, as was done with the trilateration, because it is not conventional practice. However, the orientation and computation of the long loop traverse, which is the overall polygon of the traverse chain without any intermediate connections, were tested. The azimuth from S to A in this long loop traverse is 90 deg 57 min 51.60 sec, and the grid distance from S to G is 31,720.889 ft. Both of these values have the same order of accuracy as the traverse chain of quadrilaterals.

The final discrepancies of the coordinates of G in the traverse chain of quadrilaterals with the coordinates given by CGS are also adjusted through two chains of legs or two simple traverses as was done for the trilateration. The coordinates of the traverse points from both the long loop and the simple traverse computations are computed but are not shown here. The grid distances computed from these adjusted traverses are also omitted in this paper.

The differences of the coordinates between any two of the three kinds of traverses—the quadrilateral chain, the long loop polygon, and the simple traverse—and the sums of squares of those differences were also computed. The magnitude of the sums of the squares of the differences of the coordinates between the long loop polygon and the simple traverse ($x = 48.448$ and $y = 47.506$) is the largest among the three pairs of sums. This suggests that the traverse chain of quadrilaterals was best for the computation of coordinates among the traverses tested.

Another way to compare the different traverses in adjustment and computation is to examine the changes of the computed distances from the field-observed distances. The sums of squared of the changes of the computed distances from the quadrilateral chain, the long loop polygon, and the simple traverse are respectively 0.439, 59.831, and 53.586. Therefore, the quadrilateral chain is undoubtedly the one with least alteration to the original field measurements.

ACCURACY OF THE TRAVERSES

The accuracy of the traverse depends not only on the Geodimeter distances but also on the horizontal angle measurements. The statistics of the relative errors of the observed Geodimeter distances for traverse use only are as follows:

Class of Relative Error (1 in)	Number of Geodimeter Distances
1,280	6
5,000	12
10,000	28
50,000	5
100,000	2
Total	53

From these statistics, it may be seen that approximately one-third of the traverse distance measurements have an accuracy greater than $1/10,000$.

The statistics of the maximum spread of the horizontal angle measurements are as follows:

Maximum Angular Spread (sec)	Number of Angles
1	9
5	31
10	10
15	9
20	4
26	
Total	63

From this list, it is seen that the most frequent angular error is in the range from 5 to 10 sec.

The accuracy of the horizontal angles can also be judged by the total angular error of closure of the traverse of the mean correction to each traverse angle as given in Table 3. The maximum mean correction is 15.01 sec, and the minimum is 0.71 sec. The most frequent mean correction is in the range from 5 to 10 sec.

The combined effect of the observed errors of traverse distance and horizontal angle measurements can be expressed by the linear error of closure or the relative error of the traverses as given in Table 5. The absolute linear error of the long loop traverse is a large value, 2.102 ft, but its relative error is only $1/_{34,105}$. Generally, the absolute linear error of closure and the relative error of the quadrilateral traverses are very small. No errors of quadrilateral traverses have exceeded 0.3 ft or $1/_{10,000}$.

The accuracy of the individual coordinates of the traverses can be examined by the differences between the computed grid coordinates of the experimental traverses. The accuracy of the coordinates of the individual traverses as a whole may be expressed in terms of the root-mean-square difference (Table 4).

The accuracy of the final adjusted traverse of quadrilateral chain may also be represented by the changes of the horizontal angles and the grid distances computed from the final coordinates. For the traverse of quadrilateral chain, the maximum change in distance is 0.625 ft or $1/_{1,805}$ relative to its distance value. The root-mean-square change of the grid distances is ±0.0768 ft.

COMPARISON AND EVALUATION OF THE TRILATERATION AND THE TRAVERSE EXPERIMENTS

As stated earlier, the scheme of the experiments is primarily to fit the highway surveying and mapping control situation. The network consists of a chain of quadrilaterals with braced diagonals for the trilateration experiment and without diagonals for the traverse experiment. The basic geometric features and field work involved in the two kinds of experiments are shown in Figures 1 and 2.

First, to compare and evaluate these require that the actual field and office work be analyzed in terms of the personnel employed and time expended. According to the field notes and time sheets submitted by the field and office personnel who worked on this trilateration research project, a total of 246 hours was spent on reconnaissance and layout of the network, 484 hours on horizontal and vertical angle measurements, 372 hours on Geodimeter measurements, and 208 hours on checking and reducing field notes, including theodolite angles, hand computation of Geodimeter distances, angular closure of traverse, and preliminary computation of triangle areas. The record of hours includes the time spent resetting missing station marks and remeasurement of the angles and distances.

By estimation, about one-quarter of the time spent on angular measurements was for vertical angles used in Geodimeter distance reduction. Of the 83 Geodimeter distances of the trilateration network, 51 were also used in the traverse experiment. In regard to the network reconnaissance, the chain of quadrilaterals without diagonals in traversing required one-third less time than that required for the chain of quadrilaterals with diagonals in trilateration.

The time spent checking and reducing field notes is not easily allocated between traversing and trilateration. However, approximately one-third of this time was assigned to traversing and two-thirds to trilateration, based on the degree of difficulty of the work. Therefore, the overall time required for experimental traverse and trilateration excluding the network adjustment and coordinates computation was 831 hours for trilateration and 863 hours for traverse.

Computer time records for the network adjustment and computation of coordinates were not available. However, if the trilateration adjustment can be incorporated into the COGO program, there would be little difference in the computer time required. Therefore, it can be concluded that trilateration for highway control is no more time-consuming than traversing, and with additional field experience on trilateration it may require considerably less time than traversing.

A bottleneck for trilateration field work is that the vertical angles must be observed separately by theodolite. If the Geodimeter or any other electronic distance-measuring instrument had a vertical-angle-measuring device attached to it, an appreciable amount of field time could be saved in distance measuring.

Second, the accuracy achieved by either trilateration or traversing should be analyzed. For the same number of control points and the same size of field crew, the trilateration establishes more reliable positions than does traversing. There are several facts displayed in the experiment that can be summarized to support this conclusion, including the following:

1. From an analysis of the observed values, the angular measurements by theodolite were subject to personal errors in pointing or sighting and in reading operations, while the distance measurements by Geodimeter were independent of personal errors.
2. Both Geodimeter and theodolite measurements are subject to errors for short distances. A quadrilateral traverse has four angles to be measured, and each angle has one short line of sight. But for the trilateration quadrilateral, with two long diagonals, only two of the six distances to be measured are short.
3. The discrepancies between the distances from S to G computed from the CGS-listed coordinates and from either the experimental trilateration or the experimental traverse were 0.588 ft for trilateration and 1.564 ft for traversing.
4. As given in Table 4, the accuracies in terms of the root-mean-square differences of the computed grid coordinates of the adjusted trilateration are definitely higher than those of the traverses.
5. The accuracy in terms of the sum of the squares of the changes in the distances computed from the adjusted trilateration from the field Geodimeter distances, 0.088, is much less than the sum of the squares of the changes of the distances computed from the adjusted traverse of the quadrilateral chain from the field Geodimeter distances, 0.489.

RECOMMENDATIONS FOR APPLICATION OF THE TRILATERATION TO THE CONTROL OF THE HIGHWAY SURVEYING AND MAPPING

In the preceding sections, the experimental trilateration and traverse have been investigated in detail. It was concluded that the trilateration is more accurate than the traverse and is at least as economical.

For highway surveying and mapping, as well as for other surveys such as railways and waterways, the area under consideration is usually a long narrow band. If a simple open traverse is not accepted as safely accurate enough for the surveying or mapping control, a double simple chain closed traverse such as the double centerline in the location survey of the Interstate highway, may be adopted. The latter traverse is the same as the long loop extension so named in our experiment. Obviously, the long loop extension without middle connecting or check lines is still not safely accurate as shown in the experiment. As the middle connections or check lines are increased, the maximum connection will be reached as the traverse that consists of a chain of quadrilaterals without diagonals as in the experiment. This is the traverse chain of quadrilaterals with all distances and angles measured.

A simple chain of triangles may also be called a simple chain trilateration if all sides of the triangles are observed in the field. A simple chain trilateration is enough to fix the points as for a simple open traverse. However, a simple chain trilateration run between two known control points has no self-check for each triangle.

A trilateration chain of quadrilaterals is a traverse chain of quadrilateral polygons braced with double diagonals or a chain of overlapping triangles. It has a self-check for each quadrilateral, which was termed the fundamental figure of adjustment in the first report. The quadrilateral chain of trilateration should be run between two known points. However, if only one point and one azimuth are known, these are enough to fix the orientation and position of the trilateration with respect to the existing control system, but not enough to give a check.

A trilateration chain of quadrilaterals has the same number of points as the traverse chain of quadrilateral polygons or as the long loop extension or as the closed double simple chain of legs as mentioned before, and is also more accurate and at least as economical. Therefore, it is recommended that the trilateration chain of quadrilaterals be used where a double centerline or double traverse is needed for highway surveying and mapping control.

To apply the scheme of trilateration to mapping and surveying control of highway engineering, we recommend the following specifications and procedures:

1. A regular route survey control by trilateration chain in highway engineering should start from at least one first- or second-order CGS triangulation monument with known azimuth marks (or should both start and end at such a monument?).

2. In all cases, a chain of quadrilaterals with braced double diagonals should be used except that a simple chain of triangles may be used if two known monuments at the termini are available.

3. The line to the azimuth mark may be used as one line of the trilateration. In exceptional cases, the orientation of the trilateration may be determined or checked by trilateration astronomical observations.

4. For independent surveys or small projects such as bridge sites, known monuments may not be available in the vicinity, and orientation and positioning with state plane coordinates may not be practical. One or several quadrilateral networks may be used as an independent network of trilateration.

5. The shape of the triangles in all of the trilateration networks may be acute with the ratio of the longest side to the shortest side as large as 30:1. The length of the shortest side should never be less than 200 ft.

6. All of the distances in the trilateration can be measured with electronic or optical distance-measuring instruments.

7. The difference of the sums of the two pairs of opposite triangles in a quadrilateral is an indicator of the accuracy of the field work. It can be used as a check for blunders. This area discrepancy should not exceed 100 sq ft at any time.

8. The area errors of the trilateration network of quadrilaterals may be adjusted by the method of area equations as described in the first three reports. A single quadrilateral trilateration may be adjusted by use of a desk calculator with the steps prescribed in the third report. A computer program will soon be available for any sophisticated network.

9. The adjustment and computation of the coordinates of the points of the trilateration may use any computer program available for these purposes.

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REFERENCES

1. Geodimeter Model 6 Operation Manual. AGA Corp., March 1965.
2. Lee, Shuh-Chai. Adjustment of Trilateration in Fundamental Figures. Highway Research Record 109, 1966, pp. 59-67.
3. Lee, Shuh-Chai. Adjustment of Geodetic and Engineering Trilateration by Area Equations. April 1964, unpublished.
4. Lee, Shuh-Chai. Adjustment of Distance Measurements of a Plane Quadrilateral by Area Equations. Presented at 1964 Regional Convention of American Congress on Surveying and Mapping, Kansas City, Sept. 24-26, 1964.
5. Lee, Shuh-Chai, and Belkin, A. L. A Report of Experiments on Highway Surveying and Mapping Control Extension by Geodimeter Trilateration and Conventional Traverse With Chain of Quadrilaterals. Unpublished.
6. Lee, Shuh-Chai. Computer Programs for Trilateration Adjustment by Area Equations of Fundamental Figures. Unpublished.
7. Lee, Shuh-Chai, and Goodman, J. Geodimeter Distance Reduction Program. Bureau of Data Processing, Ohio Dept. of Highways, 1969.
8. Engineer's Guide to ICES COGO I, 1st Ed. Civil Engineering Systems Laboratory, M.I.T., Cambridge, Aug. 1967.

TRAFFIC DATA COLLECTION THROUGH AERIAL PHOTOGRAPHY

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In April 1969, the Wisconsin Department of Transportation conducted a comprehensive traffic study on the freeway in the state with the highest volume. This input-output study consisted of manually recording volumes and speeds during peak periods over a $5\frac{1}{2}$-mile freeway section. Vehicle accumulation and speed data were collected and processed. An aerial study using time-lapse photography collected similar data during the same peak period. Oblique aerial photography was used to collect vehicle accumulation data, and vertical aerial photography was used to collect vehicle speed data. Statistical analyses were made to determine the reliability of the aerial photographic collection techniques as compared to the conventional collection procedures. In addition to method reliability, actual cost comparisons were made and indicated that oblique aerial photography is a reliable and economical method for collecting vehicle accumulations and that vertical photography is a reliable method for collecting vehicle speeds and headways. The vertical method allows traffic flow evaluation based on the performance of individual vehicles in the traffic flow but is an expensive method of collection when only speed data are considered. However, when data on vehicle accelerations, headways, and platoon behavior are required, vertical photography is convenient and economical as well.

•THE comprehensive freeway study of April 1969 on the East-West Freeway in Milwaukee served two purposes: It provided a quantitative inventory of peak-hour traffic data to be used in a freeway control program, and it served as an excellent basis for comparing manual and aerial photographic methods of collecting vehicle accumulation and speed information. The purpose of this comparison was to test the reliability of the aerial photographic collection method against the conventional method of collection. In addition, the actual costs incurred with each of these collection methods were documented for the purpose of establishing a general cost guideline that can be used in future freeway studies of this sort. The purpose of this paper is not only to investigate the reliability of the aerial photographic collection technique but also to present a practical guide based on the types of traffic data needed and the cost associated with fulfilling this need.

STUDY LOCATION

The East-West Freeway in Milwaukee carries the highest traffic volumes of any freeway in Wisconsin. The study section used in the comparison of the manual traffic data collection method and the aerial photographic collection method was from the Marquette Interchange in the east to the Zoo Interchange in the west (Fig. 1), a distance of 5.5 miles. Congestion occurs regularly on that section during the peak hours of 7:00 to 9:00 a. m. in the eastbound direction and of 3:30 to 5:30 p. m. in the westbound direction.

Sponsored by Committee on Photogrammetry and Aerial Surveys and Committee on Traffic Flow Theory and Characteristics.

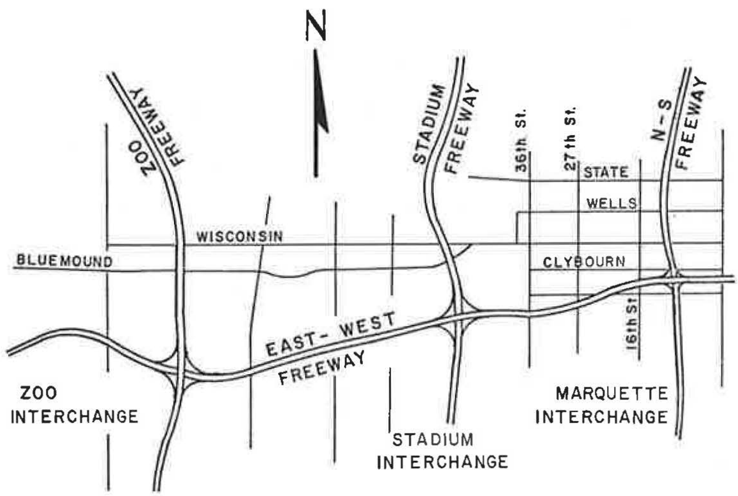

Figure 1. Study location.

STUDY TECHNIQUE

Time-lapse photography, where pictures are taken at short intervals of time, was the aerial method of traffic data collection. For the purposes of comparison, two types of traffic data were collected through aerial photography. The first type, vehicle accumulation, which represents the number of vehicles on the freeway at some given time, was collected through the use of oblique aerial photography. The second type was vehicle speed data. The acquisition of speed data through photographic means requires a controlled collection technique. Aerial vertical photography affords the control necessary for data collection of this type, especially over the 5½-mile study section. This collection represented a microscopic study that permitted an investigation of the interaction of individual vehicles and their behavior in the traffic stream. Individual vehicle speed data and also headway, or that distance between the front bumpers of the lead and following vehicles, were collected in this study.

Oblique Aerial Photography

The requirements of oblique photography were that some overlap be provided and that the photographs permit the identification of individual vehicles. The study technique used was similar to the technique presented by Wattleworth and McCasland (7).

For the purpose of comparison, the study section was identical to the one used in the manual input-output study. The oblique photographic equipment and procedure used to collect vehicle accumulations were as follows.

The plane was a Cessna 172 Skyhawk. Two 35-mm cameras were used—a Kodak Retinette IA with a 55 mm/F 1:2.8 lens and a Pentax (H-1-A) with a 55 mm/F 1:2 lens. The flight plan was to photograph only in the direction of traffic and make as many runs over the section as possible during the study period. The plane flew at an altitude of 1,000 ft and approximately 500 ft to the side of the freeway. The flight crew consisted of a pilot and two photographers who shared the responsibility of photographing the freeway. In this way, while one photographer was taking the pictures of the freeway the other had time to reload his camera and record the time of the beginning and ending of each flight along with any appropriate notes regarding the flight. The first exposure of each flight contained the beginning of the study section. The following overlapping exposures were taken in order to achieve a mosaic of the entire length of the study section.

After the film was exposed it was taken to a commercial photofinishing firm that had a 1-day film developing service. This firm processed the film into 3- by 4-in. prints. These prints were taken to the Wisconsin DOT office where one man assembled the prints by proper sequence and flight number. Each flight used one roll of film (about 32 prints), and these prints furnished a complete picture of the study section. The vehicle accumulations were extracted from the study section in the following manner.

1. The study section was broken into subsections, and the vehicles were counted with respect to subsections. The total accumulation of vehicles on the freeway was the sum of the vehicles in the subsections.
2. The time of day for this total vehicle accumulation was that time when the center photograph was exposed. This technique assumed that there was no change in vehicle accumulation during the entire flight, which was approximately 4 min.

Vertical Aerial Photography

The vehicle speed data collected by vertical aerial photography were the speeds of vehicles traveling through 1,000-ft speed traps painted on the freeway at four bottlenecks. The equipment and procedure used for the vertical photography were the following.

A twin-engine Cessna Skymaster with a push-pull engine arrangement made as many passes over the study section as possible during the peak period. The camera used was a Zeiss RMK 15/23A with a 6-in. focal length and a maximum shutter speed of $\frac{1}{1,000}$ sec and a minimum automatic cycling capability of 2 sec. The photographs were taken in the direction of the peak traffic flow. The desired scale of the photography was 1 in. equal to 600 ft. This scale dictated a flying height of about 3,600 ft. The intervalometer was set at an exposure rate of one photograph every 8 sec. These photographs were 9 by 9 in. and were taken with a 60 percent overlap to ensure workable enlargements of a consistent 1 in. equal to 100 ft scale.

Only two successive prints of each of the four bottleneck areas were enlarged to the hundred scale. Each flight required eight enlargements; a total of 10 flights were flown. Each of these 80 enlargements measured 2 by 3 ft. This size permitted vehicle identification quite readily inasmuch as a foot of ground measure was represented by 0.01 in. on the enlargement.

The next step was to identify the same vehicle on two successive enlargements. Each vehicle was given an identification number. This number consisted of four digits that represented, reading from left to right, lane, vehicle type, and placement in the queue. That is,

Lane	Type	Queue Placement
0	0	00

On each enlargement there were basically three primary reference points: (a) the beginning of the speed trap designated with a paint stripe on the freeway, (b) the ending of the same speed trap, and (c) the middle of the front bumper of each vehicle. These reference points were used to determine the position of the vehicle in reference to the speed trap.

To measure the distance between reference points, we used a coordinatograph to assign relative x and y coordinates to each reference point. After the coordinates of each vehicle were assigned, they were keypunched onto data processing cards. Each vehicle on each photograph had its own data card that represented flight, photograph, lane, vehicle type, and placement in the queue.

We reduced the photographic reference point data to ground coordinate data and then computed speeds and headways of the vehicles by writing a computer program for the IBM 360/50 computer. The format of the output data included flight number and time, photograph pair number, vehicle identification number, headway on photograph number one, headway on photograph number two, average headway, and average speed.

Time synchronization of personnel on the ground and in the air was a necessity for the successful completion of this study. During this study the 8-sec aerial speed sample by lane (about 20 vehicles) was compared to the 1-min speed sample by lane of a single vehicle taken on the ground. Speed data were compared by analysis of variance to indicate level of significance and by a standard error of the net difference to represent the difference between speed data collected aerially and manually.

STATISTICAL ANALYSIS

Vehicle Accumulation

The comparative plotting of these data, oblique aerial versus manual, is shown in Figures 2 through 7. Table 1 gives data collected for the westbound direction, and Table 2 gives data collected for the eastbound direction. Generally speaking a combined 5 percent error is associated with the eastbound and westbound directions with the aerial method usually providing the higher accumulation.

To investigate the level of significance between the means of the two collection methods, we conducted an independent t-test. This test assumes (a) a homogeneity of variance (8), (b) no difference in the vehicle accumulation collection methods due to time during the peak period and day of the week, and (c) normal vehicle accumulation distribution during the peak period.

The hypothesis statement assumes that there is no difference between the manual and aerial collection techniques. The alternative hypothesis is that the manual is not equal to the aerial. That is, the null hypothesis is $\mu_1 = \mu_2$, and the alternate hypothesis is $\mu_1 \neq \mu_2$.

TABLE 1

INCREMENTAL VEHICLE ACCUMULATION COMPARISON FOR WESTBOUND DIRECTION

Date[a]	Time	Manual Survey	Aerial Survey	Difference
April 18, 1969	4:01	556	517	-39
	4:12	625	625	0
	4:21	544	579	+35
	4:32	600	593	-7
	4:41	860	884	+24
	4:50	830	903	+73
	4:59	790	845	+55
	5:11	860	922	+62
	5:23	820	844	+24
	5:31	652	698	+46
Total		7,137	7,410	+273
April 21, 1969	3:48	673	671	-4
	3:59	520	552	+32
	4:09	550	628	+78
	4:19	500	550	+50
	4:28	540	527	-13
	4:38	731	766	+35
	4:56	820	835	+15
	5:06	883	870	-13
	5:15	928	888	-40
	5:24	862	880	+18
Total		7,007	7,167	+158
April 22, 1969	3:37	597	623	+26
	4:00	403	416	+13
	4:22	488	497	+9
	4:32	643	631	+12
	4:43	910	997	+87
	5:06	712	735	+23
	5:26	588	692	+104
Total		4,341	4,591	+274

[a]Percent error for combined westbound direction = 3.81 = (3-day absolute difference/3-day manual sum) or 3.82 for April 18, 2.25 for April 21, and 6.31 for April 22.

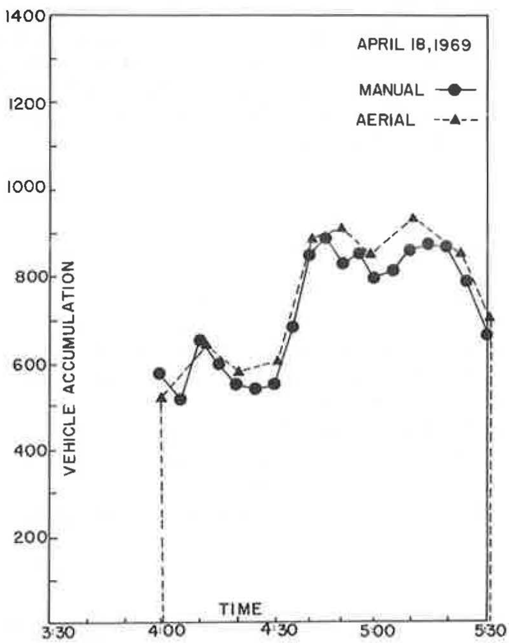

Figure 2. Vehicle accumulation in westbound direction on April 18, 1969.

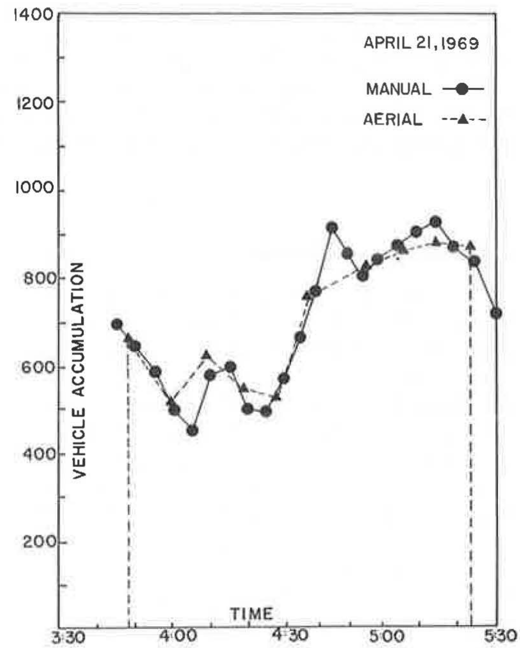

Figure 3. Vehicle accumulation in westbound direction on April 21, 1969.

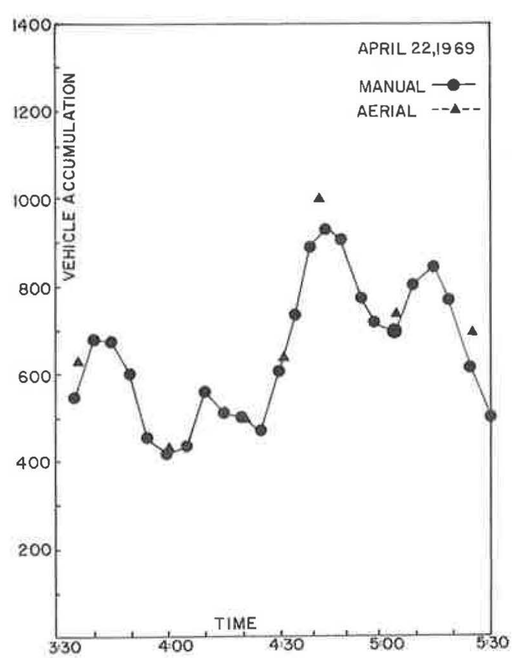

Figure 4. Vehicle accumulation in westbound direction on April 22, 1969.

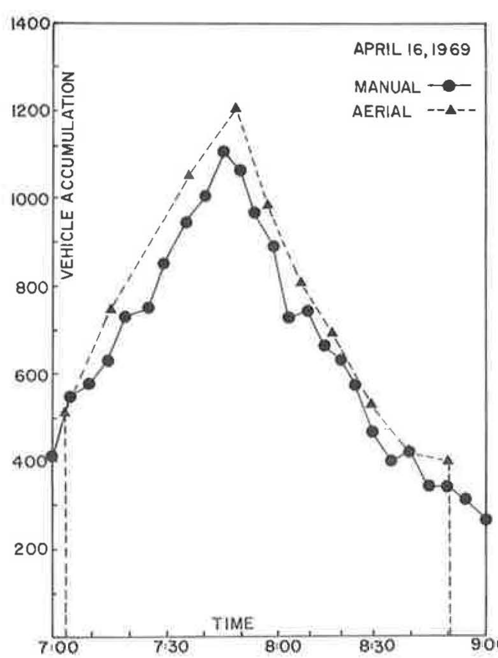

Figure 5. Vehicle accumulation in eastbound direction on April 16, 1969.

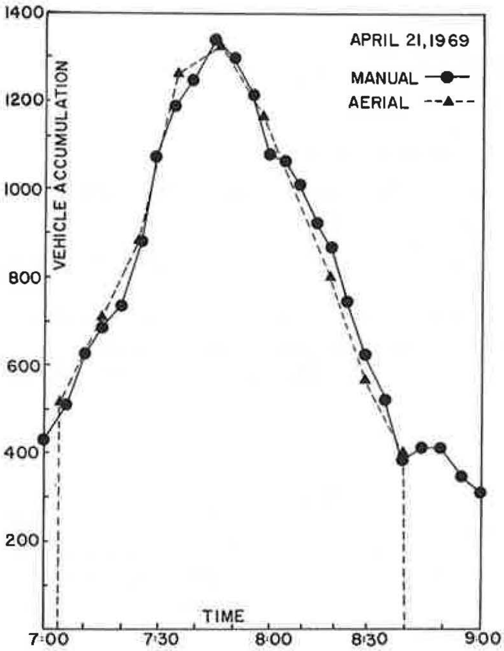

Figure 6. Vehicle accumulation in eastbound direction on April 21, 1969.

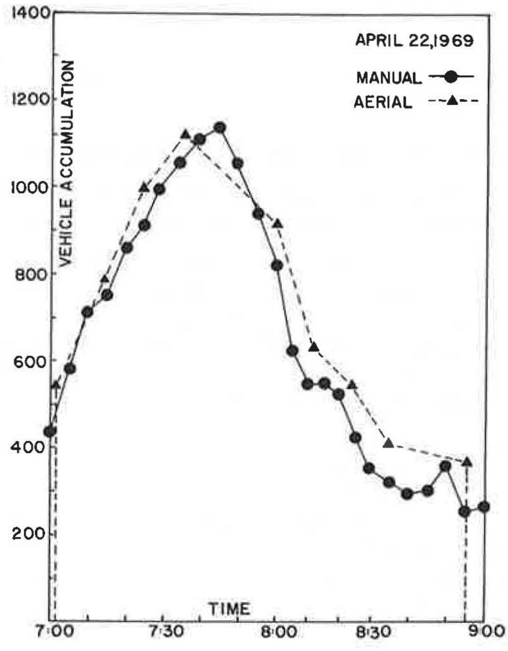

Figure 7. Vehicle accumulation in eastbound direction on April 22, 1969.

The following procedure was used in the testing:

1. Obtain the mean accumulations.

For manual,

$$\overline{X}_1 = 700.67$$

For aerial,

$$\overline{X}_2 = 735.40$$

2. Determine the variance of each technique.

$$S_i^2 = \frac{(X_i - \overline{X}_i)^2}{n_i - 1} \qquad S_1^2 = 54,996 \quad S_2^2 = 55,395$$

where n_i is the number of observations tested.

3. Pool the variances.

$$S_p^2 = \frac{(n_1 - 1)\, S_1^2 + (n_2 - 1)\, s_2^2}{(n_1 - 1) + (n_2 - 1)} = 55,196$$

4. Find the V-statistic.

$$V(\overline{X}_2 - \overline{X}_1) = S_p^2 \, \frac{1}{n_1} + \frac{1}{n_2} = 2,004$$

5. Compute a t-statistic.

$$t = \frac{(X_2 - X_1) - E\ (\text{statistic})}{V\text{-statistic}} = 0.7759$$

6. Test at a 5 percent significance level (± tabulated = 1.96).

By using the results of the two comparative tests—the percent error test and the analysis of variance test—we derived the following conclusions:

1. For the percent error test, the aerial method is generally 5 percent higher than the manual method of collecting vehicle accumulations.
2. With the analysis of variance test, at the 5 percent level of significance, there is no significant difference between the means of the aerial and manual methods.
3. Therefore, it is reasonable to assume that either method can be used to collect vehicle accumulation data.

Vehicle Speeds

The speed data recorded during the study are given in Tables 3 through 6. These tables give the variability of speed at each bottleneck location dependent on time of the peak period, method of collection, and lane for which the data were taken. To analyze the relationship that exists between speed and these variables in terms of statistical significance required that an analysis of variance technique with the index F as a test

TABLE 2

INCREMENTAL VEHICLE ACCUMULATION COMPARISON FOR EASTBOUND DIRECTION

Date[a]	Time	Manual Survey	Aerial Survey	Difference
April 16, 1969	7:04	515	515	0
	7:16	655	755	+100
	7:38	988	1,057	+69
	7:48	1,085	1,218	+133
	7:58	920	984	+64
	8:09	743	803	+60
	8:18	654	692	+38
	8:30	463	528	+65
	8:40	428	417	-11
	8:50	356	402	+46
Total		6,807	7,371	+564
April 21, 1969	7:04	493	513	+20
	7:14	675	712	+37
	7:26	925	885	-40
	7:37	1,235	1,268	+33
	7:46	1,342	1,329	-12
	7:58	1,142	1,168	+26
	8:20	868	800	-68
	8:30	612	563	-49
	8:40	387	402	+15
Total		7,679	7,640	-68
April 22, 1969	7:02	490	529	+39
	7:14	740	787	+47
	7:25	910	997	+87
	7:37	1,065	1,107	+42
	8:00	807	906	+99
	8:12	545	624	+79
	8:24	439	546	+107
	8:35	320	402	+82
	8:56	250	370	+120
Total		5,566	6,268	+702

[a]Percent error for combined eastbound direction = 6.31 = (3-day absolute difference/3-day manual sum) or 8.28 for April 16, 0.88 for April 21, and 12.61 for April 22.

TABLE 3

SPEED COMPARISON, 19TH STREET

Time	Median				Middle				Shoulder			
	Aerial Survey		Manual Survey Speed	Difference	Aerial Survey		Manual Survey Speed	Difference	Aerial Survey		Manual Survey Speed	Difference
	Vehicles	Average Speed			Vehicles	Average Speed			Vehicles	Average Speed		
4:00	8	54.4	62.5	-8.1	14	51.4	52.5	-1.1	6	48.3	47.5	+0.8
4:10	19	50.7	57.5	-6.8	17	52.5	42.5	+10.1	13	46.0	52.5	-6.5
4:21	14	52.1	57.5	-5.4	15	50.2	47.5	+2.7	13	41.0	52.5	-11.5
4:31	22	51.4	57.5	-6.1	19	46.4	52.5	-6.1	10	42.0	47.5	-5.5
4:41	47	22.7	12.5	+10.2	41	23.2	27.5	-4.3	33	29.2	32.5	-3.3
4:50	43	19.7	32.5	-12.8	34	29.7	22.5	+7.2	27	30.2	37.5	-7.3
4:59	21	44.6	52.5	-7.9	23	42.1	52.5	-10.4	17	37.2	47.5	-10.3
5:10	24	28.3	27.5	+0.8	26	29.4	32.5	-3.1	21	27.0	32.5	-5.5
5:22	58	10.9	12.5	-1.6	60	9.4	2.5	+6.9	60	7.9	2.5	+5.4
Total	281	37.2	41.4		279	37.1	36.9		223	34.3	34.9	
Error				-37.7				+1.8				-43.7
Percent error		10.1				0.54				14.15		

statistic be used. This technique is conducted as a multifactorial design. It is realized that the statistical experimental error may be appreciable in this analysis of variance because of the small manual speed sample (one) compared to approximately 20 aerial samples and because a comparison of this type is generally true only during relatively dense periods. However, this comparative analysis seems to be the most reasonable considering the method of data collection. Table 7 gives the results of the analysis of variance investigation of speed in relation to time during the peak period, method of collection, and freeway lane. The following results were obtained from the analysis of variance:

1. There is generally no significant difference between the aerial and the manual speed collection methods. However, at one bottleneck, Hawley Road, the difference was significant, which prompted a test of the difference of the means at all bottleneck locations by lane. The results of this test indicate that the shoulder lane at Hawley

TABLE 4

SPEED COMPARISON, 29TH STREET

Time	Median				Middle				Shoulder			
	Aerial Survey		Manual Survey Speed	Difference	Aerial Survey		Manual Survey Speed	Difference	Aerial Survey		Manual Survey Speed	Difference
	Vehicles	Average Speed			Vehicles	Average Speed			Vehicles	Average Speed		
4:00	10	54.0	52.5	+1.5	11	50.7	42.5	+8.2	8	50.0	47.5	+2.5
4:10	20	47.5	52.5	-5.0	17	44.3	52.5	-8.2	17	38.1	47.5	-9.4
4:21	13	52.1	52.5	-0.4	12	47.1	42.5	+4.6	10	47.5	42.5	+5.0
4:31	13	57.5	52.5	+5.0	14	50.0	52.5	-2.5	12	45.8	42.5	+3.3
4:41	25	30.3	37.5	-7.2	25	29.3	27.5	+1.8	19	20.6	17.5	+3.1
4:50	27	29.4	32.5	-3.1	31	25.9	32.5	-6.6	28	21.4	27.5	-6.1
4:59	28	25.9	37.5	-11.6	27	25.6	22.5	+3.1	21	25.6	27.5	-1.9
5:11	22	24.3	27.5	-3.2	25	24.5	22.5	+2.0	20	20.2	17.5	+2.7
5:23	21	45.8	37.5	+8.3	16	44.1	22.5	+21.6	14	42.9	22.5	+20.4
Total	199	40.8	42.5		199	37.9	35.3		175	34.7	32.5	
Error				-15.7				+24.0				+19.6
Percent error		4.28				7.03				6.28		

TABLE 5

SPEED COMPARISON, HAWLEY ROAD

Time	Median				Middle				Shoulder			
	Aerial Survey		Manual Survey Speed	Difference	Aerial Survey		Manual Survey Speed	Difference	Aerial Survey		Manual Survey Speed	Difference
	Vehicles	Average Speed			Vehicles	Average Speed			Vehicles	Average Speed		
4:01	17	44.0	42.5	+1.5	15	42.8	37.5	+5.3	19	38.3	32.5	+5.8
4:11	20	51.5	47.5	+4.0	17	50.1	37.5	-12.6	10	50.0	42.5	+7.5
4:21	21	54.2	42.5	+11.7	19	51.2	42.5	+8.7	16	47.2	32.5	+14.7
4:32	19	50.1	52.5	-2.4	17	46.6	42.5	+4.1	20	44.5	37.5	+7.0
4:41	23	39.9	37.5	+2.4	23	37.7	37.5	+0.2	20	35.0	32.5	+2.5
4:51	34	29.7	27.5	+2.2	26	33.7	32.5	+1.2	24	35.0	27.5	+7.5
5:00	25	29.7	32.5	-2.8	25	27.5	32.5	-5.0	25	27.3	22.5	+4.8
5:11	36	31.3	32.5	-1.2	33	29.2	32.5	-3.3	27	35.1	27.5	+7.6
5:23	21	45.8	52.5	-6.7	16	44.1	47.5	-3.4	14	42.9	37.5	+5.4
Total	236	41.8	40.8		208	40.3	38.0		194	39.5	32.5	
Error				+8.7				+20.4				+62.8
Percent error		2.31				5.62				17.68		

TABLE 6

SPEED COMPARISON, 92ND STREET

Time	Median				Middle				Shoulder			
	Aerial Survey		Manual Survey Speed	Difference	Aerial Survey		Manual Survey Speed	Difference	Aerial Survey		Manual Survey Speed	Difference
	Vehicles	Average Speed			Vehicles	Average Speed			Vehicles	Average Speed		
4:01	8	55.0	47.5	+7.5	11	50.7	52.5	-1.8	12	46.7	47.5	-0.8
4:12	26	35.6	37.5	-1.9	22	34.3	37.5	-3.2	21	32.3	42.5	-10.2
4:22	10	50.0	42.5	+7.5	14	52.5	57.5	-5.C	10	46.5	47.5	-1.0
4:33	21	43.7	32.5	+11.2	11	46.6	42.5	+5.9	10	47.5	47.5	0
4:42	29	34.7	37.5	-2.8	23	36.4	37.5	-1.1	24	34.6	27.5	+7.1
4:51	16	44.7	62.5	-17.8	13	43.3	47.5	-4.2	18	40.0	42.5	-2.5
5:01	18	47.8	42.5	+5.3	18	40.6	37.5	+3.1	27	28.2	42.5	-14.3
5:12	22	41.6	42.5	-0.9	23	36.8	32.5	+4.3	24	31.0	32.5	-1.5
5:24	11	53.0	47.5	+5.5	8	50.6	52.5	-1.9	9	51.4	47.5	-3.9
Total	179	45.1	43.6		159	43.5	44.2		171	39.8	41.9	
Error				+13.6				-3.9				-19.3
Percent error		3.35				1.00				5.39		

TABLE 7

ANALYSIS OF VARIANCE SUMMARY AT THE 5 PERCENT SIGNIFICANCE LEVEL

Location	Source	F (calculated)	F (0.05) (tabulated)	Results
19th Street	Time during peak period	56.47	2.59	Significant
	Aerial versus manual	4.47	4.49	Not significant
	Freeway lanes	1.34	3.63	Not significant
28th Street	Time during peak period	83.13	2.59	Significant
	Aerial versus manual	1.58	4.49	Not significant
	Freeway lanes	32.61	3.63	Significant
Hawley Road	Time during peak period	61.80	2.59	Significant
	Aerial versus manual	31.74	4.49	Significant
	Freeway lanes	26.25	3.63	Significant
92nd Street	Time during peak period	26.75	2.59	Significant
	Aerial versus manual	0.26	4.49	Not significant
	Freeway lanes	7.01	3.63	Significant

Road was the only lane of all 12 tested to have a significant difference at the 5 percent level. The reason for this difference cannot be determined.

2. Speed with respect to time during the peak period is the relationship that had the most significant results. This means that speed is most apt to change because of the change in traffic flow that occurs regularly during a time change in the peak period.

3. The fact that there is no significant difference among speeds for the three lanes of 19th Street appears to be reasonable; i.e., speed data collected in all three lanes at this bottleneck were generally about the same. Normally the speed on the freeway is distributed such that the fastest speed occurs in the median lane and the slowest occurs in the shoulder lane. At 19th Street, probably as a result of congestion caused by its proximity to the CBD and also caused by the stop-and-go driving at the gore of a high-volume entrance ramp, there was no large difference in the speed data collected by lane during the same time period.

To examine the range in the difference in speed between the arerial and manual methods of collection required that the standard error of the net difference be computed. This procedure assumes equal variances and normal distributions of the difference between the two samples. The standard error of the net difference indicates the range about the mean speed expressed as one standard deviation. One standard deviation represents approximately 68 percent of the vehicles observed traveling at some speed about the mean speed. Table 8 gives a summary that compares the speed data collected by the two methods of collection. The comparison is in terms of net difference—algebraic sum of differences between aerial speed data and manual speed data; percentage of error, which is net difference divided by total of average aerial speeds; mean speed; and one standard error and two standard errors.

Based on the data presented there is no significant difference at the 5 percent level of significance between the speed data collected manually and through aerial photography. One standard deviation of the net difference generally represents ±6 mph around a mean speed of 39 mph. In addition, an average 5½ percent error represents a difference between collection methods of about 2.2 mph when an average speed of 40 mph is used. The results would indicate the general acceptability of aerial photography for the collection of speed data. That is, speed data can be collected through either manual or aerial methods.

DATA APPLICATION

The application of aerial photography as a potential tool in traffic operations rests directly with the traffic engineer. His decision to use this collection method would

TABLE 8

AERIAL VERSUS MANUAL SPEED DATA

Location	Lane	Net Difference (mph)	Percent Error	Mean Speed (mph)		One Standard Deviation of Net Difference (mph)	Two Standard Deviations of Net Difference (mph)
				Manual	Aerial		
19th Street	Median	-37.7	10.1	41.4	37.2	±6.6	±13.0
	Middle	+1.8	0.5	36.9	37.1	±6.9	±13.5
	Shoulder	-43.7	14.2	39.2	34.3	±5.3	±10.5
29th Street	Median	-15.7	4.3	42.5	40.8	±6.1	±12.0
	Middle	+24.0	7.0	35.3	37.9	±8.8	±17.3
	Shoulder	+19.6	6.3	32.5	34.7	±8.4	±16.4
Hawley Road	Median	+8.7	2.3	40.8	41.8	±5.2	±10.2
	Middle	+20.4	5.6	38.0	40.3	±5.9	±11.6
	Shoulder[a]	+62.8	17.7	32.5	39.5	±3.3	±6.6
92nd Street	Median	+13.6	3.3	43.6	45.1	±8.7	±17.0
	Middle	-3.9	1.0	44.2	43.5	±3.9	±7.7
	Shoulder	-19.3	5.4	41.9	39.8	±6.5	±12.8

[a]Speeds proved to be significantly different; therefore, the results are unrealistic.

TABLE 9

COMPARISON OF DATA COLLECTION METHODS

Method	Data Collected	Advantages	Disadvantages	Cost
Manual	Freeway volumes, ramp volumes freeway spot speeds, vehicle accumulation, vehicle type	Provides continuous volume counts on main line as well as on ramps, provides continual speed observations	Requires many people (approximately 50), is difficult to coordinate, is not reliable because of large number of personnel and because of counting equipment	$600 for 1 hour of data collection plus $1,500 for data processing time to program, sort, and summarize data (developmental in nature)
Oblique aerial photography	Vehicle accumulations, ramp queues, shoulder use, lane density, vehicle type	Requires few people (2), requires no programming, is reliable, provides photographic record, enables density contours to be plotted quite readily	Depends on weather, requires 1-day photofinishing time, requires 4 hours to obtain vehicle counts from approximately eight flights (1 hour of flying time), requires 4 hours to obtain density contours from eight flights	$100 for 1 hour of data collection plus $30 to calculate and plot densities from vehicle accumulations that occur during 1 hour
Vertical aerial photography	Lane densities, vehicle speeds, headways, vehicle type, shoulder use, queues at ramps	Is reliable, is flexible in regard to the types of data collected, can provide very large samples, provides photographic record	Is expensive, requires trained personnel and special equipment, depends on weather, requires time to retrieve speed and headway data (approximately 1 month to receive printout of headways and speeds after vehicles were identified and sent to coordinatograph)	$3,000 for 1 hour of speed data collection plus $150 for acquisition of acceleration and headway data

probably be based principally on two factors, reliability and cost. The hourly costs incurred for each method are given in Table 9. A general evaluation by method follows.

1. The manual method (input-output study) requires a large number of personnel and, because of this, is inconvenient to use and becomes very expensive.
2. The oblique aerial method uses two men to gather and extract vehicle count data. The time expended in the collection and extraction of the data is less than 3 days. In addition, the photographs can be used to examine other traffic-related activities. Therefore, the oblique aerial method is very economical, about one-sixth as expensive as the input-output study, and very convenient to use, affording the flexibility of acquiring other traffic data when needed.
3. The vertical aerial method is not very economical when only vehicle speeds are collected; in fact, speed data collection is about 20 times more expensive by vertical aerial photography than by manual methods. However, vertical photography offers the greatest flexibility in traffic data collection, and, when traffic data concerning accelerations, headways, and platoon behavior are needed, this method of collection is not only convenient but also economical.

CONCLUSION

The comparative analyses of traffic data collection methods used in this study have indicated the acceptance of aerial photography as a collection method in the following ways.

1. Oblique aerial photography can reliably collect vehicle accumulations.
2. Oblique aerial photography is a convenient, practical, and economical method of collecting traffic data.
3. Vertical aerial photography can reliably collect vehicle speed data representative of the speeds occurring on the freeway at ±30 sec at the time the picture is taken.
4. Vertical aerial photography affords the flexibility of collecting a broad spectrum of traffic data with economic feasibility depending on the amount and types of traffic data collected.

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REFERENCES

1. Drake, J. S., Schofer, J. L., and May, A. D., Jr. A Statistical Analysis of Speed Density Hypotheses. Highway Research Record 154, 1967, pp. 53-87.
2. Fisher, R. The Design of Experiments. Hafner Pub. Co., Inc., New York, 1960, pp. 12-105.
3. Palmer, R. M. The Development of Traffic Congestion. In Quality and Theory of Traffic Flow, Bureau of Highway Traffic, Yale Univ., 1961, pp. 107-114.
4. Underwood, R. T. Speed, Volume, and Density Relationships. In Quality and Theory of Traffic Flow, Bureau of Highway Traffic, Yale Univ., 1961, pp. 145-188.
5. Salam, Y. M., Gillfillan, W. E., and May, A. D., Jr. Bay Area Freeway Operations Study. Institute of Transportation and Traffic Engineering, Univ. of California, Third Interim Rept., 1968, pp. 1-19.
6. Wagner, F. A., Jr., and May, A. D., Jr. Use of Aerial Photography in Freeway Traffic Operations Studies. Highway Research Record 19, 1963, pp. 24-34.
7. Wattleworth, J. A., and McCasland, W. R. Study Techniques for Planning Freeway Surveillance and Control. Highway Research Record 99, 1965, pp. 200-225.
8. Wardrop, J. G. Some Theoretical Aspects of Road Traffic Research. Proc., Institute of Civil Engineering, Pt. 2, Vol. 1, 1952, pp. 325-362.
9. Weinberg, M. I. A Commentary on Traffic Surveillance Systems and Techniques. Traffic Engineering, March 1967, pp. 24-27.
10. Winer, B. J. Statistical Principles in Experimental Design. McGraw-Hill Book Co., New York, 1962, pp. 24-25.
11. Following Another Vehicle. In Manual for Motorists, Wisconsin Division of Motor Vehicles, 1970, p. 25.
12. Ybarra, W. A., and May, A. D., Jr. Bay Area Freeway Operations Study. Institute of Transportation and Traffic Engineering, Univ. of California, Second Interim Rept., 1968, pp. 1-13.