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Foreword

The seven papers comprising this Record deal with various problems in aerial photography, photogrammetry and new ground control survey techniques.

The first paper presents a method for automatic acquisition of traffic data from aerial photographs. Photographic images of traffic in selected areas were converted into digital images, as number patterns, and interpreted by digital computers. The process required electro-optical scanning of each photograph. Speeds, density and headway within a traffic stream were found to be the meaningful engineering outputs derived from the system.

An analysis of stereotriangulation for highway engineering mapping was the result of recent research in Texas. The second paper describes the development and explains that stereotriangulation can be used to map many areas where the surface is inaccessible because of unfavorable weather, terrain, vegetation or other adverse conditions.

The fact that highway engineers have found a large number of practical applications for photo interpretation is verified in "Utilization of Photo Interpretation in the Highway Field." The extensive list of applications given was compiled from replies by 53 highway organizations questioned by the Highway Research Board.

Another paper considers the total extent in which interpretation of air photos has been employed by highway organizations and describes some of the major uses, including evaluation of soils and surface geology, mapping of soils and sources of aggregates, study of drainage patterns, road condition surveys, traffic surveys, and land-use studies.

Cited in one report is a program conducted by Kansas to determine the most effective aerial film for various uses. The more significant studies included: (a) the effectiveness of color photography in road condition surveys to aid in identification and evaluation of pavement breakage; (b) correlation of various road "stains" with service condition; and (c) location of exposures of bedrock and deposits of sand and gravels for use in construction.

The author of one of the papers is a photogrammetric geologist in Ohio who has developed special techniques, including use of the Kelsh plotter, for mapping coal outcrop and overburden. The method is aimed at quicker and better solution of problems in land acquisition and highway location and design.

In "Accuracy of Field and Photogrammetric Surveys," data derived from surveys by photogrammetric methods and by conventional ground survey methods are compared. Although most of the data shown concern the accuracy of elevations within one extensive project, the authors conclude that the measurements by photogrammetric methods are satisfactory for computing volumes of earthwork for design and for payment purposes.

The last paper in the Record discusses the problem of adjustments in triangulation to perfect the controls for geodetic and other major surveying tasks. Explained are the advantages of using modern electronic distance-measuring instruments such as the tellurometer, Geodimeter, or Electrotape and their application in measuring and adjusting the sides of fundamental figures by trilateration. Angle measurements and trigonometric functions are unnecessary in the new trilateration technique.

Contents

Traffic Data Acquisition from Aerial Photographs By Photographic Image Processing

T. A. BAGGOT, Data Systems Division, International Business Machines Corporation, Kingston, New York

•THE APPROACH to traffic data acquisition known as image processing employs electrooptical scanning and conventional electronic data processing techniques to extract useful data from imagery. Basically, image processing involves three steps:

1. Digitizing—visual images are converted into digital images;
2. Transforming—digital images are converted into useful data; and
3. Processing—the useful data are used to achieve the desired output for a particular task.

Digitizing is the conversion of visual images into number patterns that can be automatically interpreted by digital computers. This conversion is accomplished by accurately directing a light beam to a tiny increment, as small as 0.001 in. in diameter, of a photographic negative or positive.

The quantity of light energy either transmitted or reflected by each increment is measured by a photoelectric cell and translated into a number. A basic scanning system is illustrated in Figure 1. The components of this scanning system are a light source, an optical arrangement, a phototube, a phototube amplifier, and an analog-to-digital converter. This system must automatically convert an area on the photograph

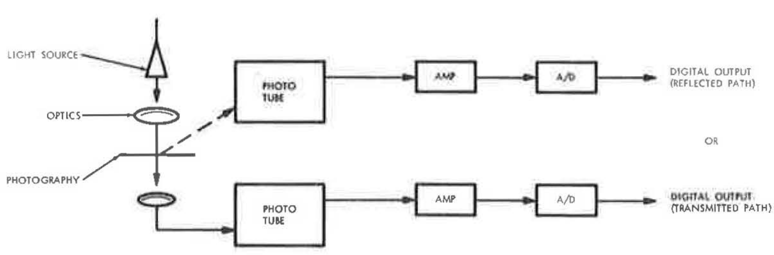


Figure 1. Basic scanning system.

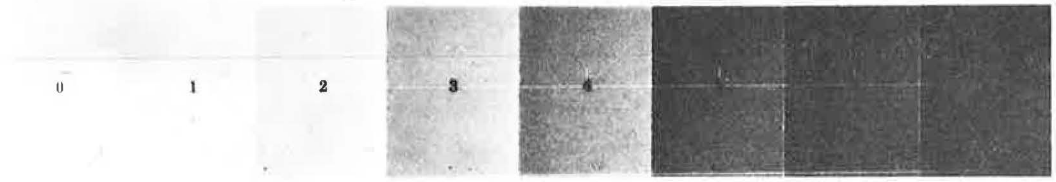


Figure 2. Typical gray scale.

Paper sponsored by Committee on Photogrammetry and Aerial Surveys.

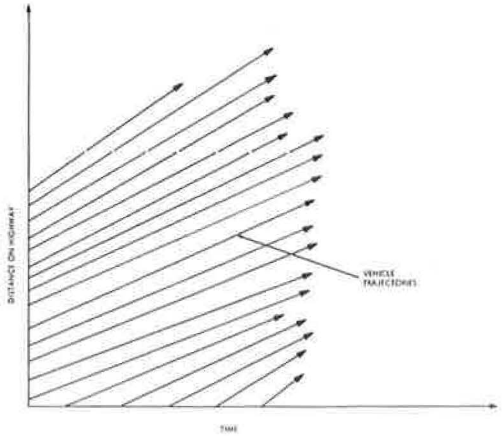


Figure 3. Space-time diagram.

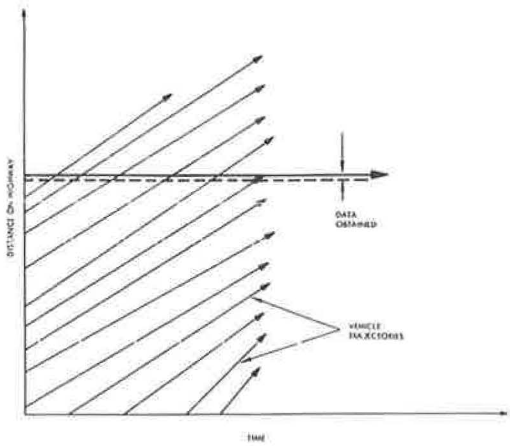


Figure 4. Space-time diagram for conventional traffic detector.

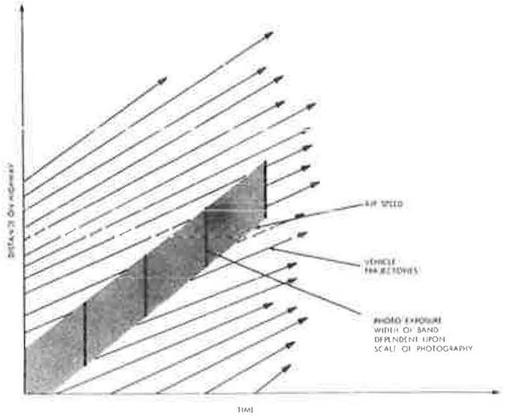


Figure 5. Space-time diagram for aerial photography.

into digital data and conveniently store the data so that it will be readily available for processing in a computer.

The number obtained by measuring the light energy corresponds to a predetermined scale and is called a gray scale. For example, a shadow cast by a vehicle might be given the value 7, whereas a 0 might represent a white line on a highway. Thus, digitizing an image gives a pattern of numbers that indicates the varying shades of gray in that image. Figure 2 illustrates a typical gray scale. It should be noted that this scale has only eight shades and serves only as an example. The number of levels established in a gray scale depends on the density range of the image and on the number of gray shades required for a particular application.

The result of digitizing imagery is a pattern of numbers, stored on magnetic tape, which corresponds to the varying shades of gray in the scanned image. A digitized image contains thousands of numbers per square inch, and, in using a spot size of one mil, the figure attained is 1 million per sq in.

TRAFFIC DATA

At this point, we should consider the traffic engineer and the information he requires. Figure 3, a space-time diagram, shows the total data available with respect to measurable parameters. Each trajectory indicates a vehicle traveling along a highway at a constant speed. From this graph we can obtain density, volume, and average velocity.

Figure 4 is a similar space-time diagram with the relationship of a conventional traffic detector added. The conventional traffic detector samples a short distance of the road for a long period of time. Measurements of speed and volume are recorded so that density can be estimated. Of particular interest is the small section of highway that is sampled. The solid and dashed horizontal lines illustrate the width of the sample strip, determined by the sensor spacing.

Figure 5 illustrates the traffic data obtainable from aerial photography on the space-time diagram. The density is readily available from a single photograph, and velocities can be calculated by tracking a vehicle from photograph to photograph. The extent of the area covered is determined by the scale of photography; a ½-mi stretch of highway is secured by a photographic scale of 300 ft to 1 in.

All the space diagrams used have been for a single lane and for constant velocities. They do not show lane changes or acceleration and deceleration of vehicles. A conventional detector will not normally detect these occurrences. However, these data will exist in the photographs.

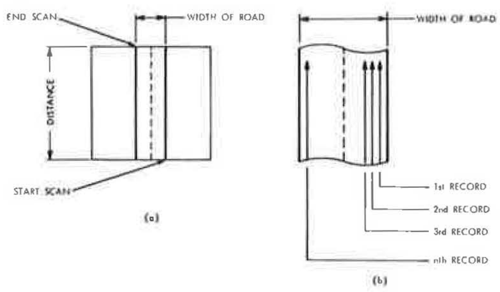


Figure 6. Scanning format.

TRAFFIC DATA AND IMAGE PROCESSING

In bringing the areas of traffic data and image processing together, we first establish a scanning format for traffic data acquisition (Fig. 6a). Starting at the

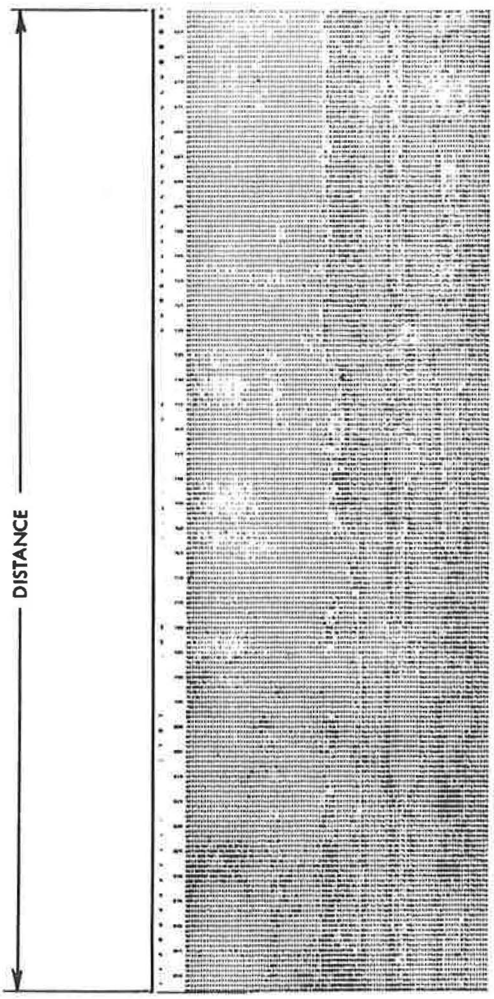


Figure 7. Printout of scanned data.

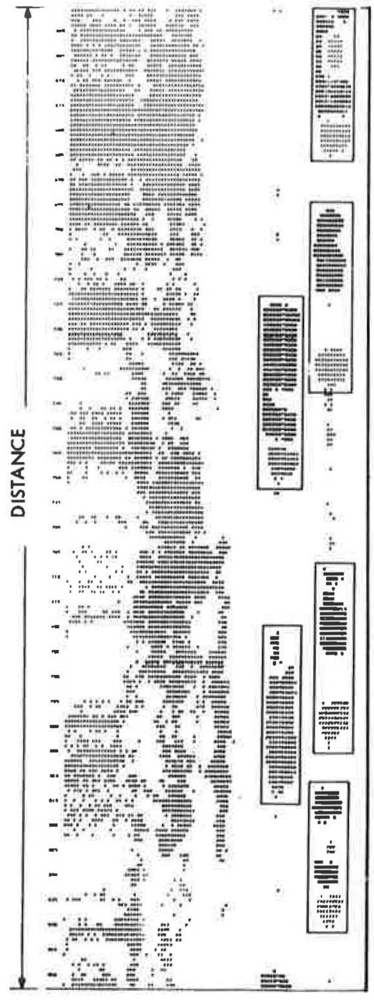


Figure 8. Printout of only extremes of scanned data (black and white).

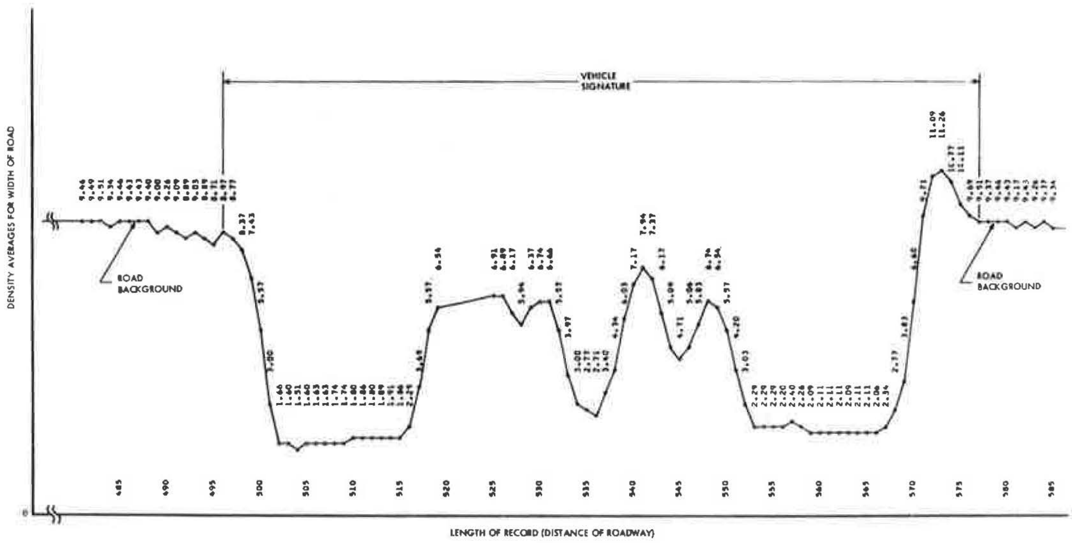


Figure 9. Output of averaging program.

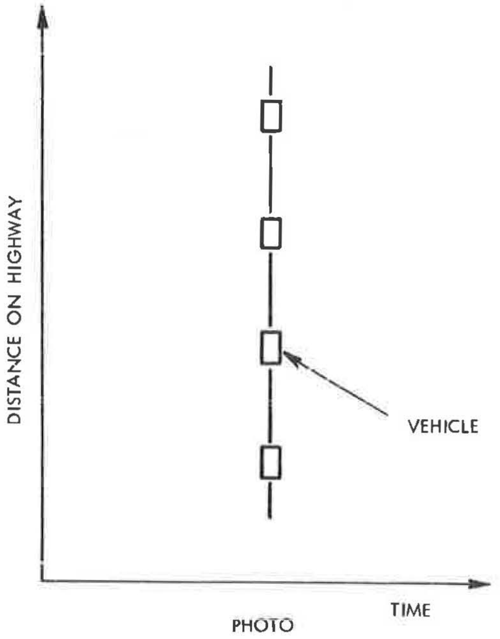


Figure 10. Data from single photograph.

lower right-hand corner of the photographed highway, we scan down the road for the desired distance. The last part of the highway to be digitized will be the upper left-hand corner.

Figure 6b illustrates the scanning and recording of the information. Each record is essentially a light profile of a portion of the highway. In computer language, such profiles are referred to as "data records." To clarify this process further, the figure shows an enlarged section of the roadway and the position of records as they are transferred to magnetic tape. A roadway represented on the photograph by an area 0.1 in. wide and 9.0 in. long and scanned with a 1-mil spot would be described by 100 records, each record containing 9,000 characters. The scanner used is capable of a 1-mil spot and of quantizing the light into 16 levels of gray. Each tape character represents the quantized gray level of a 1-mil spot on the photograph. Therefore, position across the roadway is located by record number, and position down the roadway is located by character position in a record. Computing distances on the roadway involves using the photographic scale factor. As an illustration, for a photographic scale factor of 1 in. = 300 ft, 1,000 characters of data are equivalent to 300 ft on the ground.

Figure 7 shows the data obtained in our first attempt to secure vehicular data from scanned aerial photographs. Patterns of zero's and F's can be seen in the right-hand portion of the illustration. To facilitate seeing the vehicle, we suppressed all digits except 0 and F. The result was Figure 8. Although the vehicles are outlined, it is ap-

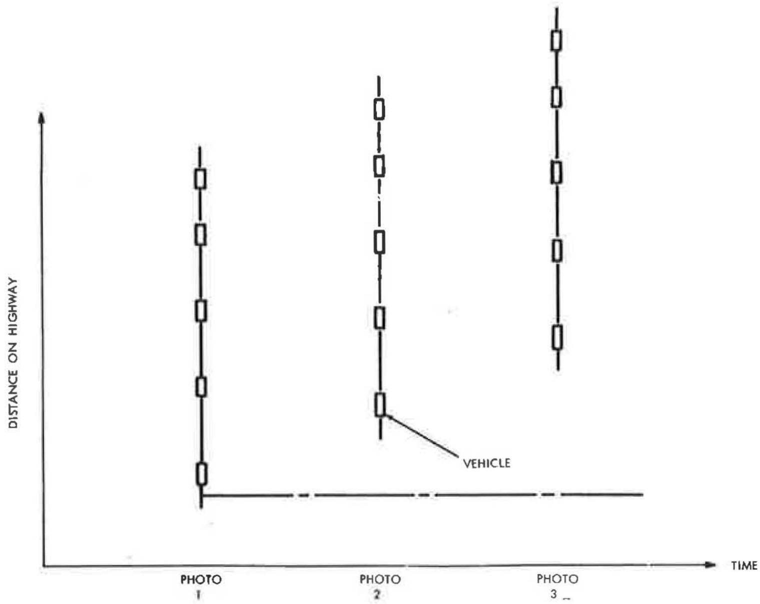


Figure 11. Data from a series of photographs.

OUTPUT OF DATA PROJECT - MODEL MARK II, VERSION 01

PHOTO TAPE NUMBER 1726

DIST	*	LGTH	IMAGE DE
56.8	1	20.1	DAAAARG
94.0	2	17.2	DAADAQ
124.4	3	17.8	CAAEAQ
151.1	4	20.1	CAACDIL
209.2	5	20.1	BAFJCCG
257.7	6	18.8	BCFFFGB
316.1	7	17.5	CAAEBO
422.4	8	18.8	CCFFFFB
470.2	9	29.7	CBDFFFF
552.1	10	17.5	EAEJCFB
645.5	11	20.1	IAAADAQ
740.5	12	32.3	BAABAEI
841.8	13	17.8	DAAFAN
926.6	14	19.5	EAAAAKL
1023.6	15	19.8	BBGFBI
1103.5	16	17.8	BBGFBI

PHOTO 1

TAPE NUMBER 1721, LANE 3

DIST	*	LGTH	IMAGE DESCRIPTION
46.5		19.5	DCEEEE
86.5		19.5	CAABANJ
124.4	1	19.1	DAAAAPG
161.7	2	21.4	CAACAMJ
194.7	3	17.5	CBBEDJ
236.6	4	19.5	DBCECIF
304.3	5	16.5	GGDADGBBG
366.3	6	19.1	CDEEFFB
459.7	7	17.5	DAAFCM
508.9	8	13.5	DDFFD
560.7	9	18.8	CDFFFFB
624.7		17.8	BDEFFE
723.0	10	17.5	BBGFBG
844.8	11	19.8	GBBCCEJ
942.1	12	20.1	EAACCFN
1018.7		15.5	DABELC
1075.8	13	17.2	DABGCJ
1167.9	14	18.8	FBBCBKD
1253.7	15	16.8	CAFICCF

PHOTO 2

* Correlation of Vehicles, not a printout.

Figure 12. Output of two vehicle detection programs.

parent that they would stand out by themselves. The data were obtained by scanning a photograph taken with a 12-in. focal length camera at an altitude of 7,200 ft, yielding a scale of 1 in. = 600 ft and a field of view of 5,400 by 5,400 ft on the 9- by 9-in. photograph.

The coding used on the printout for the 16 shades of gray ranges from 0 to 9 and from A to F, with 0 corresponding to the lightest shade and F corresponding to the darkest.

For the photograph under discussion, 65 veh were counted manually from 1,800 ft of 3 lanes of a 6-lane highway. From the suppressed printout data, a total of 62 patterns of 0's and F's was recognized. The vehicles not identified in the scanned printout included two in the shadow of an overpass and one in the shadow of a tree.

With this initial success of detecting 95 percent of the vehicles present, our next effort was to write a program to detect the existence of a vehicle on the roadway. This program calculates the average gray level across the lane for the scanned distance of roadway. For example, if the width of the lane is equivalent to 35 records or individual gray-level characters, the average of these characters is computed and stored. This process continues for each 1-mil increment along the roadway until the density averages for the entire length of scanned lane have been assembled. The results of this program present average gray levels for the distance of roadway under observation. Figure 9 shows the output of the program.

The establishment of these averages provides the criteria for detecting the presence of deviation in the road background. A vehicle-detection program operating on these data would yield vehicle location and density data. A deviation from the road background falling within minimum and maximum criteria determines that a vehicle exists at this point. The deviation, as shown in Figure 9, is called the vehicle signature. At this point, we have three factors necessary for obtaining traffic data from aerial photographs:

1. We have detected the vehicle;
2. We have determined the vehicle's location; and
3. We have the time at which the photograph was taken.

These data are shown graphically in Figure 10.

This information by itself is almost meaningless. However, we can continue repeating the procedure described previously and obtain data for a series of individual photographs. These plotted data are illustrated in Figure 11.

The space-time diagram is becoming crowded with individual data points, which by themselves are of minor significance. Our next step in this research was to track a vehicle from one photograph to another. The key to this problem was the vehicle's signature, mentioned previously. The signature is first coded; then, by correlating, we are able to locate a vehicle in the next successive digitized photograph. The technical feasibility of this procedure was verified by our last experiment, completed in July 1964.

The aerial photographs which were digitized contained 42 veh common to both photographs. The results were as follows:

1. Forty vehicles correlated;
2. Two vehicles did not correlate, because they passed under a tree; and
3. There were no false correlations.

Figure 12 is a sample of the vehicle detection program with the coded signature. The output displayed in this figure describes the location and the length of the vehicle and includes the image description. This is the coded vehicle signature. The vehicles listed in the output of photograph 1 were correlated to those in photograph 2. The time between photographs was 5 sec. The first two vehicles shown in photograph 2 are new to the scanned area. The vehicles in photograph 2 between 9 and 10 and between 12 and 13 moved from lane 2 to lane 3. This correlation, using the allowable variances in individual characters and the tolerances on overall descriptions, was performed by hand.

RECOMMENDATIONS

What information does the traffic engineer need? Speed, density, headway? These and other meaningful engineering outputs can be derived. With proper programming, the desired traffic characteristics or parameters can be obtained by extending the aforementioned techniques.

What has to be done to make this an operational system? Nothing; it is operational now. However, the term operational should be qualified; the system is operational only as an experimental tool. The drum scanner which has been used to investigate this application is essentially a laboratory device and not production equipment. For production work, the scanner would be inefficient and restrictive.

Further research is required on both the equipment and the programming. In the area of equipment, a scanning bed similar to an input-output XY plotter should be developed to control the scanning of curved highways and interchanges. The scanning head should be a variable-width line scanning device which would scan the photographs once for each lane. The programs now in use must be optimized and organized into a programming package. Additional programs must be written for obtaining the specific data required by the traffic engineer.

Automatic traffic data acquisition from aerial photographs by photographic image processing has been proved technically feasible. Further research and development are required, however, for an economical operating system.

ACKNOWLEDGMENTS

This report is based on the early work of L. M. Sista in traffic data acquisition, using image processing techniques. The author is also indebted to A. L. Fenaroli, F. A. Richell, and R. E. Ross, who continued Mr. Sista's research, and to others in traffic engineering who encouraged the effort. Valuable consultations and photographs were provided by the New York Port Authority. A major portion of this work was performed for the Massachusetts Department of Public Works in cooperation with the U.S. Bureau of Public Roads.

An Analysis of Stereotriangulation for Highway Engineering Mapping

TOMMIE F. HOWELL, Photogrammetry Section, Texas Highway Department

•THE TEXAS Highway Department has developed a procedure to use stereotriangulation for the procurement of intermediate control data for photogrammetric mapping. A Zeiss C-8 Stereoplanigraph is used for the stereobridges. Electrotapes (electronic distance measuring instruments) and other high-order surveying equipment are used for the primary ground control traverses. All data are adjusted by electronic computers for which special programs have been designed.

A research project was initiated to determine the feasibility of this method for obtaining supplemental or intermediate ground control for large-scale photogrammetric mapping to be used for engineering purposes. From an evaluation of the data obtained, information such as accuracies to be expected, limitations of adaptability, and economics were to be determined.

Ninety percent of all photogrammetric projects mapped by the Texas Highway Department are developed to a horizontal scale of 1 in. = 40 ft with vertical data shown by 1-ft contour lines and spot elevations to the nearest 0.1 ft. All planimetric features of the finished map sheets must be within 1.0 ft of their true horizontal position and that 90 percent of all contours must be within one-half the contour interval, in this case 0.5. The remaining 10 percent may approach the maximum deviation of one contour interval. Spot elevations may deviate a maximum of 0.30 ft.

To meet these rigid requirements, the basic ground control data must be very accurate. Ground control on such a photogrammetric project consists of a primary control traverse with points varying distances apart, depending on terrain and desired measurement spacing, but averaging about ½ mi apart, with secondary or intermediate control points approximately 300 ft apart to insure at least two points per stereo-model. These primary and secondary control points are paneled before photography with crosses having legs 4 ft long and 6 in. wide for easy photo identification. The primary field control traverses must meet at least second-order distance and angular closure requirements. The intermediate field control for stereo-model scaling must meet third-order accuracy requirements. The criteria used by the Texas Highway Department for second- and third-order accuracies are given in Table 1.

An Electrotape field party established by the Photogrammetry Section of the Highway Design Division is made available to all District and Resident Engineer Offices located throughout the state to work in cooperation with their field personnel to develop primary control traverse data. This field party does not attempt to establish the intermediate control from which the photogrammetric mapping is accomplished because the volume of work and the physical size of the state make it unfeasible.

The establishing of the intermediate ground control points is expensive and has been a major difficulty in the development of control data. There are several reasons for this difficulty. Most District field parties have previous commitments such as construction projects which make it impossible to obtain this control in time to meet the mapping schedule, and in many cases, the areas to be mapped for highway engineering are inaccessible because of weather, terrain, vegetation, or uncooperative landowners. With these difficulties in intermediate ground control repeatedly encountered, the possibility of utilizing stereotriangulation merited a research program to determine feasible methods and techniques.

Paper sponsored by Committee on Photogrammetry and Aerial Surveys.

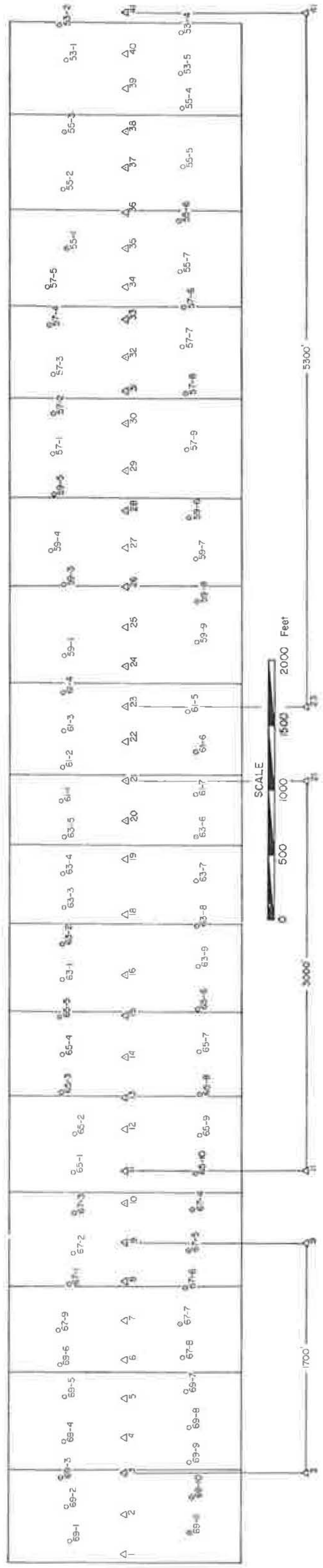


Figure 1. Control layout of stereotriangulation research area.

TABLE 1
ACCURACY CRITERIA

Order of Accuracy	Horizontal Dist. [a]	Horizontal Angles (sec)			Vertical Dist. (ft)
		Traverse Lines	Triangle Closure		
			Avg.	Max.	
2nd	1:10,000	$10\sqrt{N}$	3	5	$0.035\sqrt{M}$
3rd	1:5,000	$30\sqrt{N}$	5	10	$0.050\sqrt{M}$

[a] N = No. of angles between tangents of traverse.
[b] M = length of level circuit in miles.

An area approximately 2.7 mi long along I-35 north of Austin, Texas, was selected for the test area. Field control was established on this project (Fig. 1) similar to any other project designed to be mapped at 1 in. = 40 ft with a 1-ft contour interval. The only exception was that the Highway Design Division Electrotape field party, with its highly trained personnel and high-accuracy surveying equipment, established all the ground control including the intermediate control points. The traverse was run throughout the length of the project establishing primary control points at intervals of 1,700, 3,000, and 5,300 ft. The angles were measured with a 10-sec Kern theodolite and each angle was turned a minimum of 8 times. This basic traverse closed well within the tolerances of second-order specifications. Intermediate control points were chained in at distances varying from 270 to 330 ft. A total of 41 centerline points were established. Each of the intermediate points was slightly offset at varying distances from the tangent line between the primary control points. These deviations from tangent were induced to reduce any stereo operator "adjustments" for alignment. These centerline points were paneled with white crosses having legs 4 ft long and 6 in. wide.

The feasibility of vertical stereobridging was also studied. Wing points were placed throughout the length of the project and marked with panels of the same dimensions as those of the centerline. The wing points were located approximately 400 ft on either side of the centerline and were spaced approximately 300 ft apart, thus insuring enough for each stereo-model. A total of 76 wing points were paneled. Elevations were established on the centerline panels and wing point panels using a Kern level (Fig. 1).

After paneling and establishment of the ground control, a contract was awarded to a prequalified photogrammetric contractor to photograph the area following the Standard Specifications for Photography prepared by the Texas Highway Department. Specifically, the area had to be photographed with a "distortion-free" aerial camera having a 6-in. focal length producing a 9- by 9-in. aerial negative. The scale of the photography was required to be 1 in. - 200 ft, with an allowable 10 percent deviation. The negatives were to be of excellent image quality in all respects, and no negatives could have tilt exceeding 3 deg and accumulative tilt between successive negatives could not exceed 4 deg. The negatives were required to have an overlap between 55 and 65 percent. The photography received met these requirements in all respects.

Two qualified photogrammetric contractors having "first-order" stereoplotting equipment were selected to develop stereotriangulation data of the test area. The contractors had two different models of first-order plotters: a Galileo Santoni Model IV Stereocartograph and a Wild A-5 Autograph. Zeiss C-5 and C-8 Stereoplanigraphs were later incorporated into the study.

This research was initiated to determine the acceptability of stereobridged control for use in the development of photogrammetric maps. Also, data for stereobridging from Kelsh-type stereoplotters were to be compared with those obtained from the contracted stereoplotters. A skeleton control data tabulation sheet (Table 2) and a set of photography of the test area indicating the location of the vertical control points listed on the skeleton control sheet were furnished each of the photogrammetric contractors and the Texas Highway Department Kelsh operator. Also, the contractors were furnished the aerial negatives of the photographic flight for making diapositive plates adaptable to their equipment.

TABLE 2

SKELETON CONTROL DATA SHEET, I-35 STEREOBRIDGE PROJECT

Point	Elevation	Coordinates		Point	Elevation	Point	Elevation
		X	Y				
1	642.36	44,980.8	25,880.86	69-1	645.52	61-2	705.62
2	644.27	45,243.02	26,036.70	69-2	647.06	61-3	714.60
3	646.06	45,518.52	26,189.51	69-3	648.25	61-4	718.64
4	650.41			69-4		61-5	731.80
5	658.93			69-5		61-6	730.22
6	664.87			69-6		61-7	
7	666.04			69-7		59-1	
8	661.49			69-8		59-2	
9	654.95	47,007.80	27,052.03	69-9	641.66	59-3	
10	652.47	47,273.54	27,198.95	69-10	641.19	59-4	
11	656.27	47,487.65	27,328.88	69-11	641.18	59-5	
12	668.55			69-12	637.72	59-6	
13	674.35			67-1		59-7	
14	681.44			67-2	667.58	59-8	
15	689.33			67-3	658.58	59-9	
16	697.08			67-4	650.89	57-1	
17	703.98			67-5	647.61	57-2	
18	706.31			67-6	658.03	57-3	
19	708.32			67-7		57-4	
20	709.95			67-8		57-5	
21	710.31	50,092.70	28,820.72	67-9		57-6	
22	713.89	50,348.75	28,973.24	65-1		57-7	
23	723.13	50,589.56	29,106.15	65-2		57-8	
24				65-3		57-9	
25				65-4		55-1	
26	745.68			65-5		55-2	
27				65-6		55-3	735.94
28				65-7		55-4	740.30
29	735.52			65-8		55-5	
30				65-9		55-6	
31				65-10	660.34	53-1	725.71
32	728.40			63-1		53-2	715.42
33				63-2		53-3	709.28
34				63-3		53-4	719.23
35	710.67			63-4		53-5	730.62
36				63-5			
37				63-6			
38	737.55			63-7			
39				63-8			
40				63-9			
41	737.38	55,222.26	31,775.55	61-1			

TABLE 3

UNADJUSTED KELSH-TYPE PLOTTER TABULATION[a]

Point	Two-Projector Kelsh			Three-Projector Kelsh			K & E M-2 Stereoplotter		
	X	Y	Z	X	Y	Z	X	Y	Z
(a) Centerline Points									
1					Control furnished				
2									
3									
4	-1.0	-1.2		+0.4	-0.3		+0.4	-0.4	
5	-1.3	-1.6		+0.4	-0.4		+0.1	-0.7	
6	-1.5	-2.0		+0.5	-0.6		+0.3	-0.8	
7	-1.5	-2.6		+0.6	-0.1		+0.2	-0.4	
8	-1.5	-3.2		+0.6	-1.5		-0.3	-0.7	
9					Avg. (1 through 8)				
10	1.360	2.120		0.500	0.580		0.260	0.600	
11									
12					Panel destroyed				
13	-0.4	-1.1		+0.2	-0.1		-0.2	-0.2	
14	+0.5	-1.8		+0.3	+0.1		-0.3	-0.2	
15	+0.8	-2.0		+0.4	+0.4		-0.5	0.0	
16	+0.9	-2.6		+0.1	+0.5		-0.5	+0.5	
17					Panel destroyed				
18	+1.8	-3.2		-0.6	+0.5		-1.6	+1.5	
19	+2.2	-3.1		-1.0	+1.2		-2.0	+2.0	
20	+2.9	-3.1		-1.3	+1.9		-2.1	+2.2	
21					Avg. (12 through 20)				
22	1.35	2.41		0.55	0.67		1.02	0.94	
24	Data incomplete			-0.7	-0.6	+0.9	-0.6	-0.5	0.0
25				-0.4	-0.8	+1.3	-0.5	-0.6	+0.3
26				-0.6	-0.6	+1.7	-0.9	-0.5	+0.6
27				-0.9	-0.9	+2.7	-1.2	-0.4	+0.5
28				-1.4	-0.8	+4.0	-1.5	-0.2	+1.0
29				-2.6	-1.6	+4.2	-2.2	-0.7	+1.1
30				-2.7	-0.4	+4.5	-2.3	+0.4	+1.3
31				-3.1	-0.6	+4.7	-2.5	+0.6	+2.2
32				-3.8	-1.8	+5.0	-2.6	-0.5	+3.3
33				-4.8	-1.8	+6.6	-2.4	-0.2	+4.0
34				-3.9	-2.4	+7.0	-2.4	-0.2	+4.3
35				-4.9	-2.9	+7.7	-2.8	-0.3	+5.5
36				-5.0	-3.3	+8.5	-2.9	-0.2	+7.1
37				-6.6	-3.0	+8.0	-4.0	+0.7	+9.5
38				-7.0	-4.0	+8.5	-4.0	0.0	+9.9
				-8.2	-5.6	+9.1	-4.6	-0.8	+10.8
				-9.2	-5.4	+9.8	-4.9	-1.2	+11.8
				-10.0	-6.1	+10.6	-4.6	-0.7	+12.7
Avg.				(4.228)	(2.367)	(5.822)	(2.606)	(0.483)	(4.772)
(b) Vertical Wing Points[b]									
69-1									
69-2									
69-3									
69-4		-0.1			-0.2			-0.0	
69-5		-0.4			-0.3			-0.2	
69-6		0			-0.1			+0.1	
69-7		+0.3			+0.4			+0.3	
69-8		+0.2			+0.1			+0.1	
69-9									
69-10									
69-11									
69-12									
67-1		-0.2			-0.1			-0.1	
67-2									
67-3									
67-4									
67-5									
67-6									
67-7		-0.3			-0.1			-0.3	
67-8		+0.1			+0.2			0	
67-9		+0.4			+0.4			+0.3	
65-1		0.0			-0.1			+0.2	
65-2		+0.5			+0.2			+0.6	
65-3		+0.3			+0.2			+0.8	
65-4		-0.4			-0.3			-0.1	
65-5		+0.2			+0.1			+0.6	
65-6		+1.1			+1.4			+0.6	
65-7		+1.3			+0.7			0.0	
65-8		+1.0			+1.3			+0.7	
65-9		+0.9			+1.0			+0.3	
65-10		-0.6			-0.8			-1.4	
63-1		-0.2			-0.1			+0.1	
63-2		-0.7			0.0			+0.2	
63-3		-0.3			-0.1			+0.5	
63-4		-0.1			+0.2			+0.8	
63-5		+0.3			0.0			+0.9	
63-6		+1.1			+1.3			+0.4	
63-7		+0.9			+0.8			-0.1	
63-8		+4			+0.8			+0.2	
63-9		+0.6			+0.8			+0.1	
61-1		+0.1			-0.3			+0.9	
61-2									
61-3									
61-4									
61-5									
61-6									
61-7		+0.6			+0.4			-0.5	
Avg.		(0.437)			(0.430)			(0.380)	

[a] Area of centerline vertical control only.

[b] Wing point data in area of no centerline control is erratic; average errors and pattern are given in Table 4.

This skeleton control consisted of at least three horizontal and nine vertical control points at various distances throughout the length of the project. The initial model was fully controlled with additional horizontal and vertical control furnished at distances approximately 1,700, 3,000, and 5,300 ft apart. The primary reasoning for the varying control distances was to determine the capabilities of control extension on Kelsh-type stereoplotters. The stereobridge on the first-order equipment was set up similarly, with three different length bridges required. Vertical control was furnished throughout the 1,700- and 3,000-ft control gaps on the centerline panels, and every 1,000 ft thereafter. The differential control spacing was to assist in determining a preferred distance between the basic control points on a normal mapping project with a scale of 1 in. = 40 ft.

The test area was set up on a two-projector Kelsh plotter, a three-projector Kelsh plotter, and a three-projector K & E M-2 stereoplotter. All three bridging tests were run by the same operator to minimize the effect of operator techniques on the data differences obtained. The operator had no access to the known control data until all three bridges were completed.

The basic procedure used on all Kelsh-type bridging was simple control extension. The operator was provided a manuscript having all the furnished horizontal control points plotted by coordinates. The initial model was oriented as precisely as possible by the operator to the known horizontal and vertical control. This control was extended and tied to the next controlled model, which was set up and extended likewise until all three uncontrolled areas were spanned. As the operator tied one model to the next, all panel points were dropped on the furnished manuscript and elevations of each panel were noted. On completion of this bridge, the coordinates and elevations of the dropped points were manually taken off the manuscript and written in their appropriate blanks on the control skeleton sheets. A comparison of differences of the X, Y, and Z coordinate positions of these Kelsh-type stereobridges indicated what accumulative errors and patterns of errors might be expected (Table 3).

As indicated in Table 3, the three-projector plotters gave slightly better results. All three instruments had excellent results in the shorter bridges and some of the errors noted could be attributed to the manner in which the coordinates were manually obtained using an engineer's scale. However, no error greater than ½ ft was obtained. Therefore, this factor carried little weight in the evaluation.

This test indicated the necessity for abundant centerline vertical control. Level bubbles were utilized in the area where no centerline vertical control was furnished; however, the results were very unsatisfactory (Table 3). Deviations from the known control points obtained from the stereobridges on the three Kelsh-type stereoplotters were plotted to evaluate a possible pattern of deviation for adjustments. These tests indicated fair systematic error patterns for the horizontal control but very little for the vertical control, minimizing the amount of confidence to be placed on using this control for high-order accuracy photogrammetric mapping. This does not rule out using control data obtained by Kelsh-type stereobridging for areas not exceeding a three-model bridge between known horizontal and wing point vertical control, and having at least one centerline vertical control point per model. This control gap could be increased considerably with abundant centerline vertical control, providing the expected resultant map accuracies were relaxed accordingly. The extent of the use of Kelsh-type equipment for extending control would have to be governed by the intended use of the maps to be derived from this control.

Additional research on different techniques is hoped for in the immediate future, especially in the area of duplication of models on the three-projector plotters in conjunction with an adjustment computer program similar to the one now being used with stereotriangulation. This duplication of models technique was attempted, but without any graphic or computer adjustments. The initial results were similar to those obtained by the normal tie-in control extensions given in Table 3; however, the additional research in this area appears to be merited, and evaluation of existing data is being continued at this time.

Each of the contractors processing the test area on first-order stereoplotters was required to submit a synopsis and to complete the skeleton control sheet, whereon the X, Y, and Z coordinates of known panels on the project were furnished. The synopsis should contain information on the equipment used, procedure, and difficulties encountered. As stated earlier, the stereoplotting equipment employed was a Galileo Santoni Model IV Stereocartograph and a Wild A-5 Autograph. Both contractors used computer programs and their respective bridging procedures were very similar. The project was bridged by approximately scaling the initial model to the known control furnished and tying in each successive model without disturbing the previous model setting. Machine adjustments of BY, BZ, Swing, Tip, and Tilt were used in joining the successive models. Machine coordinates were established for all vertical and horizontal panel points throughout the length of the project.

The machine X, Y, and Z coordinates of the known and unknown control points were keypunched along with the true X, Y, and Z coordinates of the known vertical and horizontal control points for the computer input data. These cards were run through the computer using a program adapted to adjust and transform all machine coordinates to the basic datum coordinates furnished. These coordinate data were printed on a readout sheet supplying X, Y, and Z ground coordinates of all panels throughout the length of the project.

Difficulties encountered by the contractors as stated in their synopsis included excessive swing in the initial model, as evidenced by lack of BY movement to complete the bridge, adjusted for on subsequent bridge; excessive BZ movement, indicating climbing of aircraft in photography acquisition, adjusted for on subsequent bridges; and the possibility of excessive distortion in two isolated models, as evidenced by having to cross tilt the successive model to tie it to the previous model.

The completed skeleton control data sheets were received from the contractors with the required synopses. The initial data received for the Santoni stereotriangulation were analyzed, the field coordinates for the computer input were reversed, and the program was rerun. The revised data are referenced throughout this paper.

The data obtained by the stereobridges using the first-order stereoplotters were superior to the results obtained on Kelsh-type equipment. The data obtained on the variable lengths of stereobridges requested indicated little significant differences; however, no bridge was of sufficient length to provide conclusive results. The deviations of the coordinates obtained by stereotriangulation on the first-order plotters were similar, both horizontally and vertically, to the data obtained on the three-pro-

TABLE 4

FIRST-ORDER AND AVERAGE KELSH STEREOBRIDGE TABULATION DATA

Point	Kelsh (Unadjusted)			Santoni			A-5			C-5			C-8		
	X	Y	Z	X	Y	Z	X	Y	Z	X	Y	Z	X	Y	Z
(a) Centerline Points															
1	Control furnished			-0.1	-0.1	-0.2	-0.1	-0.1	-0.5	+0.3	+0.2	+0.1	+0.1	+0.1	-0.1
2				0.0	0.0	0.0	-0.1	-0.1	-0.1	-0.1	+0.1	+0.1	-0.2	0.0	-0.1
3				0.0	+0.2	0.0	+0.3	+0.2	+0.2	-0.3	-0.4	0.0	0.0	0.0	0.0
4	+0.4	-0.3		-0.1	-0.1	+0.6	+0.3	-0.2	+0.5	0.0	-0.4	+1.1	-0.1	-0.4	-0.2
5	+0.3	-0.5		-0.2	-0.5	+0.6	-0.1	-0.2	+0.7	-0.5	-0.3	0.0	-0.3	-0.4	0.0
6	+0.3	-0.7		0.0	-0.3	0.0	-0.1	-0.1	+0.2	-0.4	-0.4	-0.2	-0.2	-0.1	-0.2
7	-0.4	-0.2		+0.3	+0.2	+0.1	+0.1	0.0	+0.5	-0.3	-0.2	-0.1	0.0	0.0	+0.1
8	-0.4	-1.2		-0.1	+0.1	0.0	0.0	-0.2	+0.1	-0.1	+0.4	-0.1	-0.1	-0.2	-0.4
9	Avg. (1 through 8)			+0.2	-0.2	+0.1	-0.1	+0.2	+0.1	-0.2	-0.2	+0.2	+0.1	+0.2	+0.2
10				-0.3	0.0	-0.1	-0.1	-0.3	-0.1	+0.2	-0.4	+0.5	+0.2	-0.3	+0.5
11	0.36	0.6		-0.1	+0.1	-0.6	-0.1	0.0	-0.5	0.0	-0.2	-0.3	+0.3	+0.1	+0.1
12								Panel destroyed							
13	-0.2	-0.12		-0.2	+0.3	-1.0	-0.2	+0.2	-0.4	+0.1	+0.3	+0.5	+0.1	+0.2	+0.4
14	-0.3	-0.13		0.0	+0.1	-0.4	+0.1	+0.2	-0.3	+0.4	+0.3	+0.3	+0.3	+0.4	+0.4
15	-0.4	-0.2		-0.1	+0.1	-0.7	-0.2	+0.1	-0.5	+0.4	+0.2	+0.2	+0.1	+0.4	0.0
16	-0.3	+0.5		-0.2	+0.1	-0.8	-0.2	+0.1	-0.5	0.0	+0.3	+0.1	-0.2	+0.1	-0.4
17								Panel destroyed							
18	-1.2	1.0		-0.1	+0.2	-0.3	0.0	+0.3	-0.5	0.1	+0.2	+0.2	-0.2	+0.2	+0.3
19	-1.5	+1.5		-0.1	+0.2	-0.4	0.0	+0.3	-0.2	0.0	+0.5	+0.4	-0.3	+0.5	+0.3
20	-1.8	+2.0		+0.1	+0.1	-0.5	0.0	+0.1	-0.2	+0.2	+0.6	+0.2	-0.2	+0.4	0.0
21	Avg. (12 through 20)			0.0	+0.3	-0.6	+0.1	0.0	-0.2	-0.4	+0.4	+0.3	-0.1	+0.1	+0.4
22				0.10	-0.2	+0.1	-0.2	0.0	+0.1	-0.2	-0.4	+0.1	-0.1	0.0	+0.1
23				0.0	-0.2	+0.2	0.0	0.0	-0.1	+0.1	+0.3	+0.3	+0.1	0.0	0.0
24	-0.6	-0.5	+0.0		Data in process		-0.6	-0.1	+0.3		No additional data available		-0.2	-0.1	-0.6
25	-0.5	-0.7	+0.8				-0.4	-0.1	+0.5				0.0	+0.1	-0.8
26	-0.7	-0.5	+1.1				-0.4	0.0	+0.3				-0.1	+0.1	-1.1
27	-1.0	-0.6	+1.6				-0.2	-0.4	+0.6				0.0	0.0	-0.6
28	-1.4	-0.5	+2.5				-0.5	-0.2	+0.6				0.0	-0.4	-0.2
29	-2.2	-1.0	+2.6				1.5	-0.9	+0.2				-1.0	-0.6	-0.2
30	-2.5	-0.4	+3.0				-0.8	-0.5	+0.5				-0.4	+0.9	-0.2
31	-2.7	-0.6	+3.5				-1.0	+0.4	+0.5				-0.5	+0.7	+0.1
32	-3.2	-1.2	+4.1				-0.9	-0.7	+0.2				-0.2	0.0	-0.1
33	-3.5	-1.0	+5.3				-0.5	-0.3	-0.2				+0.1	+0.3	+0.1
34	-3.0	-1.6	+5.5				-0.4	-0.3	-0.4				+0.3	+0.3	+0.2
35	-3.7	-1.6	+6.7				-0.5	-0.3	-0.1				+0.2	-0.3	+0.1
36	-4.0	-1.7	7.8				-0.2	-0.2	-0.3				+0.6	+0.3	+0.1
37	-5.0	-2.0	+8.5				-0.9	+0.5	-0.1				0.0	+0.9	-0.1
38	-0.5	-2.2	9.0				-0.3	-0.4	+0.2				+0.4	-0.2	-0.2
39	-6.5	-3.2	10.0				-0.5	-0.5	-0.3				0.0	-0.2	-0.4
40	-7.0	-3.3	10.5				-0.5	-0.2	-0.2				-0.4	0.0	-0.2
41					Avg. (1 through 23)		-0.1	-0.1	-0.1				0.0	-0.1	0.0
Avg.	±2.9	±1.3		±0.1	±0.2	±0.3	±0.37	±0.26	±0.34	0.3	0.32	0.25	±0.2	±0.25	±0.24

(b) Wing Points—Vertical Only

Point	Kelsh (Unadjusted)	Santoni	A-5	C-5	C-8
		(0.295)	(0.403)	(0.254)	±(0.34)
69-1	All Kelsh readings + and - through point 61-7	-0.3	-0.3	-0.3	0.0
69-2		0.0	0.0	+0.1	0.0
69-3		+0.1	+0.1	+0.2	+0.1
69-4	0.1	0.0	+0.5	-0.2	-0.2
69-5	0.2	-0.3	+0.5	-0.5	-0.3
69-6	0.1	-0.4	+0.3	+0.8	-0.1
69-7	0.35		+0.7	0.0	-0.6
69-8	0.1	-0.3	+0.8	0.0	-0.5
69-9		+0.2	+0.3	+1.3	-0.3
69-10		0.0	+0.1	+0.2	-0.1
69-11		-0.1	+0.1	+0.2	-0.1
69-12					
67-1					
67-2		+0.2	+0.2	-0.1	-0.3
67-3		-0.3	-0.2	-0.1	0.0
67-4		+0.1	-0.2	+0.6	+0.5
67-5		+0.2	+0.2	+0.4	+0.3
67-6		-0.2	+0.2	+0.1	+0.2
67-7	0.2	-0.3	+0.2	-0.2	-0.1
67-8	0.1	-0.2	+0.3	-0.2	0.0
67-9					
65-1	0.15	-0.2	-0.4	-0.1	+0.5
65-2	0.4	+0.3	-0.4		-0.8
65-3	0.5	+0.3	-0.7	+0.7	+0.5
65-4	0.2	+0.1	-0.6	+0.2	-0.3
65-5	0.35	-0.2	-0.7	-0.1	-1.0
65-6	0.35	+0.5	-0.3	+0.7	-0.2
65-7	0.35	-0.2	+0.1	+0.5	+0.1
65-8	1.0	+0.3	-0.2	+0.7	+0.4
65-9	0.7	+0.4	0.0	+0.1	+0.2
65-10					
63-1	0.1	+0.3	-0.6	0.0	0.0
63-2	0.1	+0.5	-0.5	0.0	+0.4
63-3	0.3	+0.3	-0.5	+0.5	+0.3
63-4	0.5	+0.4	-0.5	+0.3	+0.4
63-5	0.5	-0.1	-0.2	-0.1	-0.2
63-6	0.8	-0.4	+0.2	+0.3	+0.2
63-7	0.4	-0.4	+0.1	+0.4	-0.5
63-8	0.5	-0.2	-0.3	+0.3	-0.4
63-9	0.4	+0.6	-0.2	+0.2	-1.6
61-1	0.6	+0.2	-0.3	-0.2	-0.2
61-2			+0.4	0.0	+0.3
61-3			+0.2	0.0	-0.2
61-4			+0.3	0.0	-0.3
61-5			+0.3	+0.1	-0.5
61-6			+0.7	-0.1	-0.4
61-7	0.4	-0.1	-0.2	+0.2	+0.5
59-1	0.4	-0.6	+0.4	No additional data available	-0.3
59-2					
59-3	1.1	-0.2	+0.8		-0.4
59-4	1.5	-0.1	+5		-0.8
59-5	2.5	-0.3	-1		-0.7
59-6	2.0	+0.3	+1.1		-0.4
59-7	2.1	+0.3	+0.7		-0.6
59-8	1.5	+0.5	+0.5		-0.8
59-9	0.7	-0.1	+0.8		-0.9
57-1	2.5	+0.1	+0.3		-0.4
57-2	3.3	+0.3	+1.1		-1.3
57-3	4.1	-1	+0.7		-0.2
57-4	5.1	-3	-0.5		-0.3
57-5	5.7	-3	-0.5		+0.1
57-6	5.6	-2	+0.5		+0.6
57-7	4.4	0.0	-0.1		+0.2
57-8	3.5	0.0	-0.3		+0.1
57-9	2.5	+0.5	+0.1		+0.4
55-1	6.5	-0.5	+0.2		0.0
55-2	8.4	-1.1	+0.2		0.0
55-3	9.0	-0.8	-0.9		-0.2
55-4	9.4	-0.9	+0.8		+0.1
55-5	10.0	-0.9	-0.2		-0.3
55-6	7.3	-0.4	-0.4		+0.1
53-1	10.4	0.0	-0.3		+0.4
53-2	11.5		+0.4		+0.9
53-3					
53-4	11.4		-0.5		+0.2
53-5	10.8		-0.3		0.0
Avg.	Inconclusive	±0.3	±0.4	±0.25	±0.34

jector Kelsh-type equipment for the initial three to four models. Beyond this distance, the accuracy of the data from the contracted plotters remained fairly constant, whereas the Kelsh accuracies decreased considerably. As indicated in Table 4, there were little significant differences in the X, Y, and Z coordinate results obtained on the two first-order machines and, assuming the same operator techniques and computer programs, very little difference in performance of the equipment.

The acquisition of a Zeiss C-8 Stereoplanigraph by the Texas Highway Department enabled further study of stereotriangulation in areas not explored on the contracted portion of the project. The stereobridges required of the contractors were negotiated and limited in scope. The data and procedures obtained from the contractor's stereobridges were definitely informative and indicated that intermediate control could be obtained for some photogrammetric mapping projects by stereotriangulation. These encouraging preliminary data were a prime factor in determining that first-order plotting equipment would be of more use to the Department than the Kelsh-type stereoplotting equipment then in operation.

The identical test area and skeleton control data were initially utilized on the C-8 Stereoplanigraph to obtain an equipment evaluation and to check out the difficulties indicated by the respective contractors in their synopses. The data obtained on the C-8 Stereoplanigraph are included in Table 4. The Army Map Service Branch Plant in San Antonio allowed the Texas Highway Department's operator to use of one of their first-order machines. During the 3 weeks spent working on the Army Map Service C-5 Stereoplanigraph equipment, the operator stereobridged a portion of the test area. The data obtained were adjusted utilizing the Army Map Service computer program. The program is basically the same as the adjustment program the Texas Highway Department uses; however, the horizontal and vertical input data had to be run through the computer separately. The readout-adjusted X, Y, and Z coordinates were similar to the results obtained by the Zeiss C-8, Wild A-5, and Santoni stereobridges, and are included in Table 4.

At the conclusion of this phase, the test area along I-35 had been stereobridged, using the same control interval, on a two-projector Kelsh plotter, a three-projector Kelsh plotter, a K & E M-2 stereoplotter, a Wild A-5 Cartograph, a Galileo Santoni Model IV Stereocartograph, a Zeiss C-5 Stereoplanigraph, and a Zeiss C-8 Stereoplanigraph. A thorough analysis of the data obtained from these various stereobridges gave a fair indication of the accuracies which could be expected from stereotriangulation utilizing known control in the input data at distance intervals of 1,700, 3,000, and 5,300 ft.

All data thus far evaluated had been obtained from stereobridges developed through one certain area using the same photography and control. In general, the X and Y deviations from the X and Y coordinates established from the ground survey were very small, averaging less than ±0.3 at any one point established by stereotriangulation on first-order equipment. At a horizontal map scale of 1 in. = 40 ft, this ±0.3-ft deviation would amount to slightly less than $^1/_{120}$ in. This fractional difference would be extremely difficult to plot on a map sheet and would have little effect on individual model scaling on a Kelsh-type plotter. Also, this variation of the stereotriangulated points from true coordinate position is plus and minus, and the possibility of accumulative errors is eliminated by the computer program which adjusts the intermediate control points to the coordinate positions of the furnished ground control points throughout the length of the project.

Factors taken into consideration when analyzing and evaluating the horizontal errors of ±0.3 ft incurred from the horizontal stereobridge data obtained on the various types of equipment are as follows:

1. Possible resultant errors in basic control data,
2. Distortion in the aerial photography,
3. Correlation of the bridging equipment lens system and the photographic camera lens,
4. Financial limitations on the photogrammetric contractors for input adjustments, and
5. Control spacing not ideal for first-order bridging.

TABLE 5

DIFFERENTIAL CONTROL INTERVALS STEREOBRIDGED ON C-8

Point	1,200			1,500			2,000			2,500			3,000		
	X	Y	Z	X	Y	Z	X	Y	Z	X	Y	Z	X	Y	Z
								(a) Centerline Points							
1	+0.1	+0.3	-0.1	+0.2	+0.1	-0.1	0	+0.2	+0.2	0	+0.2	+0.1	+0.1	0	-0.2
2	-0.2	0	-0.2	+0.2	0	-0.2	-0.3	0	+0.2	-0.3	+0.1	-0.2	-0.2	-0.1	-0.2
3	0	+0.1	-0.1	0	-0.1	-0.1	0	0	0	-0.1	0	-0.1	+0.1	-0.2	-0.1
4	-0.2	-0.5	-0.2	-0.2	-0.5	-0.2	-0.1	-0.4	-0.2	-0.2	-0.4	-0.2	-0.1	-0.6	-0.2
5	-0.4	-0.5	0	-0.5	-0.5	0	-0.4	-0.4	-0.1	-0.4	-0.4	-0.2	-0.3	-0.5	0
6	-0.2	-0.3	-0.2	-0.3	-0.3	-0.2	-0.2	-0.2	-0.3	-0.3	-0.2	-0.3	-0.2	-0.3	-0.2
7	0	-0.1	+0.1	-0.1	-0.1	0	0.0	-0.1	0	-0.1	-0.1	0	0	-0.1	+0.1
8	-0.1	-0.4	-0.4	-0.2	-0.4	-0.4	0.0	-0.3	-0.4	-0.1	-0.3	-0.5	-0.1	-0.3	-0.4
9	-0.1	0	+0.2	0	0	+0.2	+0.1	+0.1	+0.1	+0.1	0	+0.1	+0.1	+0.1	+0.2
10	+0.3	-0.5	+0.5	+0.2	-0.5	+0.5	+0.3	-0.4	+0.5	+0.2	-0.5	+0.4	+0.2	-0.4	+0.6
11	+0.3	-0.2	+0.1	+0.2	-0.2	+0.1	+0.4	-0.1	0	+0.3	-0.2	-0.1	+0.3	-0.1	+0.1
12								Panel destroyed							
13	+0.2	0	+0.5	+0.1	-0.1	+0.5	+0.2	0	+0.4	+0.1	-0.1	+0.3	+0.1	0	+0.5
14	+0.3	+0.2	+0.4	+0.2	+0.2	+0.4	+0.3	+0.2	+0.4	+0.3	+0.1	+0.3	+0.3	+0.3	+0.5
15	+0.1	+0.1	0	0	+0.2	0	+0.1	+0.2	-0.1	-0.1	-0.1	-0.1	0	+0.2	+0.1
16	-0.2	-0.2	-0.3	-0.2	-0.1	-0.3	-0.1	-0.1	-0.4	-0.1	-0.2	-0.5	-0.3	-0.1	-0.3
17								Panel destroyed							
18	-0.2	+0.1	+0.1	-0.2	0.0	0.0	-0.1	-0.1	+0.2	-0.1	-0.1	-0.4	-0.2	+0.3	+0.2
19	-0.2	+0.2	+0.3	-0.2	+0.2	+0.3	-0.2	+0.2	+0.3	-0.2	+0.1	+0.1	-0.4	+0.3	+0.4
20	-0.1	+0.1	+0.1	-0.1	+0.1	+0.1	-0.1	+0.1	0	-0.1	0	-0.1	-0.3	+0.2	+0.1
21	0	-0.2	+0.4	0	-0.1	+0.4	0	-0.2	-0.3	0	-0.3	+0.3	-0.2	-0.1	+0.5
22	+0.1	-0.2	+0.2	+0.1	-0.2	+0.2	0	-0.3	+0.1	0	-0.2	0	-0.2	-0.2	+0.2
23	+0.2	-0.3	+0.1	+0.2	-0.3	+0.1	+0.2	-0.3	0	+0.2	-0.4	-0.1	0	-0.2	+0.1
24	-0.1	-0.3	-0.6	-0.1	-0.4	-0.5	-0.1	-0.4	-0.6	-0.1	-0.5	-0.7	-0.3	-0.3	-0.5
25	+0.2	-0.2	-0.8	+0.2	-0.2	-0.7	+0.1	-0.2	-0.8	+0.2	-0.3	-0.9	-0.1	-0.1	-0.7
26	+0.1	-0.1	-1.0	+0.1	-0.2	-1.0	0	-0.2	-1.1	+0.1	-0.3	-1.2	-0.2	-0.1	-1.0
27	+0.2	-0.2	-0.5	+0.2	-0.3	-0.5	+0.1	-0.3	-0.6	+0.2	-0.4	-0.7	-0.1	-0.2	-0.5
28	+0.2	+0.2	-0.1	+0.2	+0.1	-0.1	+0.1	+0.1	-0.2	+0.1	0	-0.2	-0.2	+0.2	-0.1
29	-0.8	-0.8	-0.2	-0.7	-0.9	-0.1	-0.8	-0.9	-0.2	-0.7	-0.9	-0.3	-0.1	-0.7	-0.1
30	-0.2	-0.8	-0.1	-0.2	+0.6	-0.1	-0.3	+0.6	-0.1	-0.2	+0.6	-0.2	-0.5	-0.8	-0.1
31	-0.3	+0.6	+0.2	-0.3	+0.4	+0.3	-0.4	+0.5	+0.2	-0.3	+0.4	+0.1	-0.6	+0.6	+0.2
32	0	-0.2	0	-0.1	-0.4	+0.1	-0.1	-0.3	0	0	-0.4	-0.1	-0.3	-0.2	0
33	+0.3	+0.2	+0.1	+0.3	0	+0.2	+0.2	+0.1	+0.1	-0.4	0	+0.1	0	+0.2	+0.2
34	+0.5	+0.2	+0.3	+0.4	-0.1	+0.4	+0.5	+0.1	+0.2	+0.6	0	+0.1	-0.2	+0.2	+0.3
35	+0.4	+0.2	+0.2	+0.3	-0.1	+0.2	+0.4	0	+0.2	+0.5	0	+0.2	+0.2	+0.2	+0.2
36	+0.7	+0.2	+0.2	+0.6	-0.1	+0.2	-0.3	+0.1	+0.2	+0.9	+0.1	+0.1	+0.5	+0.2	+0.1
37	+0.1	+0.9	0	-0.1	+0.4	+0.1	+0.1	+0.7	+0.1	+0.3	+0.7	0	-0.1	+0.9	0
38	+0.5	-0.1	-0.1	+0.2	-0.4	0	+0.5	0	-0.1	+0.7	0	-0.1	+0.3	+0.1	-0.1
39	+0.1	-0.2	-0.3	-0.2	-0.8	-0.2	+0.1	-0.3	-0.2	+0.3	-0.4	-0.2	0	-0.2	-0.3
40	-0.4	0	-0.1	-0.7	-0.6	0	-0.2	-0.1	-0.1	0	-0.1	0	-0.3	0	-0.2
41	0	-0.1	+0.1	-0.4	-0.8	+0.2	+0.1	-0.2	+0.2	+0.3	-0.2	+0.2	0	0	0
Avg.	±0.2	±0.25	±0.22	0.21	0.25	0.22	0.18	0.27	0.21	0.23	0.22	0.22	0.2	0.23	0.23
								(b) Vertical Wing Points							
69-1			-0.2			-0.1			-0.1			-0.1			-0.1
69-2			0			0			-0.1			-0.1			0
69-3			0			+0.1			+0.1			0			+0.1
69-4			-0.2			-0.3			-0.3			-0.3			-0.3
69-5			-0.3			-0.3			-0.3			-0.4			-0.3
69-6			-0.2			-0.1			-0.1			-0.1			-0.2
69-7			-0.6			-0.6			-0.7			-0.7			-0.4
69-8			-0.5			-0.6			-0.6			-0.6			-0.5
69-9			-0.3			-0.4			-0.4			-0.4			-0.3
69-10			+0.1			-0.2			-0.1			-0.1			-0.1
69-11			-0.1			-0.2			-0.2			-0.2			-0.2
69-12															
67-1								No Panel							
67-2			-0.3			-0.3			-0.2			-0.2			-0.3
67-3			0			0			0			+0.1			0
67-4			+0.5			+0.6			+0.4			+0.4			+0.6
67-5			+0.3			+0.3			+0.2			+0.2			+0.4
67-6			+0.2			+0.2			0			+0.1			+0.2
67-7			-0.1			-0.1			-0.2			-0.2			0
67-8			0			0			-0.1			0			+0.1
67-9								Bad point—no panel							
65-1			+0.5			+0.4			+0.5			+0.4			+0.4
65-2															
65-3			+0.5			+0.4			+0.6			+0.4			+0.4
65-4			+0.4			+0.3			+0.4			+0.2			+0.3
65-5			0			-0.1			+0.1			-0.1			-0.1
65-6			+0.5			+0.6			+0.3			+0.3			+0.7
65-7			+0.1			+0.2			+0.1			-0.1			+0.3
65-8			+0.4			+0.5			+0.2			+0.2			+0.5
65-9			+0.2			+0.3			0			0			+0.3
65-10								Bad point—no panel							
63-1			0			-0.1			+0.1			-0.1			-0.1
63-2			+0.5			+0.4			+0.6			+0.4			+0.4
63-3			+0.4			+0.2			+0.4			+0.2			+0.3
63-4			+0.5			+0.4			+0.6			+0.3			+0.4
63-5			-0.1			-0.1			0			-0.2			-0.2
63-6			+0.2			+0.4			0			0			+0.4
63-7			-0.5			-0.3			-0.7			-0.7			-0.3
63-8			-0.5			-0.4			-0.5			-0.7			-0.4
63-9			+1.4			+1.6			+1.2			+1.2			+1.6
61-1			0			-0.1			+0.1			0			0
61-2			+0.4			+0.3			+0.5			+0.2			+0.3
61-3			-0.1			-0.2			0			-0.2			-0.2
61-4			-0.3			-0.3			-0.2			-0.3			-0.3
61-5			-0.5			-0.4			-0.7			-0.7			-0.3
61-6			-0.4			-0.3			-0.6			-0.6			-0.2
61-7			+0.5			+0.7			+0.3			+0.3			+0.7
59-1			-0.2			-0.3			+0.1			-0.3			-0.3
59-2															
59-3			-0.3			-0.3			-0.2			-0.4			-0.4
59-4			-0.6			-0.7			-0.6			-0.8			-0.8
59-5			-0.6			-0.6			-0.5			-0.7			-0.7
59-6			-0.4			-0.2			-0.4			-0.5			-0.2
59-7			-0.6			-0.4			-0.7			-0.8			-0.4
59-8			-0.8			-0.6			-0.9			-0.9			-0.6
59-9			-0.9			-0.7			-1.0			-1.0			-0.7
57-1			-0.2			-0.3			-0.2			-0.3			-0.3
57-2			-0.1			-0.2			-0.1			-0.2			-0.2
57-3			0			0			0			-0.1			-0.1
57-4			-0.2			-0.1			-0.2			-0.2			-0.3
57-5			+0.3			+0.3			+0.3			+0.2			+0.2
57-6			+0.6			+0.3			+0.6			+0.6			+0.7
57-7			+0.2			+0.3			+0.2			+0.1			+0.3
57-8			+0.1			0.3			+0.1			+0.1			+0.2
57-9			+0.4			+0.6			+0.4			+0.3			+0.6
55-1			+0.1			+0.2			0.1			+0.1			0
55-2			+0.2			+0.3			+0.1			+0.1			+0.1
55-3			-0.1			0			-0.1			-0.1			-0.2
55-4			+0.1			+0.2			+0.2			+0.2			+0.1
55-5			-0.3			-0.1			-0.1			-0.2			-0.2
55-6			+0.1			+0.3			+0.2			+0.2			+0.2
53-1			+0.5			+0.7			+0.5			+0.5			+0.4
53-2															
53-3															
53-4			+0.2			+0.3			+0.5			+0.4			+0.2
53-5			0			+0.1			+0.2			+0.2			0
Avg.			0.27			0.29			0.27			0.28			0.28

The basic control traverses were established by an experienced field party using precision equipment and the possiblity of error was minimized in the actual survey and in the placement of the panels. A panel might have been displaced off the absolute center of the control point since the test area is in a developed section of the highway and, due to the number of panels, it was difficult to maintain surveillance of all points before photography. The consistently high deviations noted on certain points, with the necessity of cross tilting on some model ties, could have been due to minor localized photographic distortion. Distortion would, very definitely, have some effect in an overall stereobridge—especially when dealing with tenths of a foot.

It is very doubtful that more than one of the stereobridges was run on equipment calibrated in exact correlation with the photographic camera lens. The Texas Highway Department used corrective plates for the average of the type of lens, but these were not correlated with the specific lens of the camera. This source of error should have little effect on the horizontal points but should be considered in an overall evaluation.

The final readout data furnished by the contractors might have been improved by removing certain control points from the computer input data. This would have required additional computer time and additional funds, and was not part of the contract.

It was recognized that the control data furnished for the development of the stereobridging were not ideal for first-order equipment. Better results probably could have been obtained if the control had been spaced at different intervals; however, this interval was preferred for a simultaneous evaluation of the Kelsh-type plotters and the first-order equipment.

The machine X, Y, and Z coordinates of all panel points on this test area had been recorded on the initial C-8 stereobridge, and with only minor computer input data revisions, stereobridge data were obtained at varying distance intervals of 1,200, 1,500, 2,000, 2,500, and 3,000 ft. A comparative analysis of the effect of this differential control spacing on the resultant adjusted X, Y, and Z coordinates is given in Table 5.

Elevation deviations from true ground elevations or Z coordinates noted in evaluating the various stereobridges from the test area were somewhat greater than the X and Y coordinate deviations. The average error incurred was less than 0.4 ft. This elevation differential exceeds the vertical control accuracy desired on mapping projects at a scale of 1 in. = 40 ft. The effect of this average vertical error on the intended use of certain highway mapping projects might be negligible.

Again, the factors mentioned previously that possibly affected the horizontal accuracies obtained were considered in the analysis of the vertical errors incurred. In fact, these factors would have more effect on the vertical data than the horizontal data. Distortion in the aerial photography and correlation of the bridging equipment lens system and photographic camera lens system could greatly affect vertical accuracies.

Additional stereotriangulation research was conducted along F. M. Highway 1604 in Bexar County. Map sheets were currently being developed on this project at a scale of 1 in. = 40 ft with a 1-ft contour interval. The basic control for the project had been established by standard field methods and most models had set up relatively well. A 3-mi tangent was selected and stereobridged on a Zeiss C-8 Stereoplanigraph. Machine coordinates were recorded on all vertical wing points and centerline panels. Known ground control data were inserted into the input computer data at different intervals. The results obtained on this stereobridge are given in Table 6. Unfortunately, none of the wing vertical control points were paneled and identification of the exact location of these points was extremely difficult. Mislocation of a vertical point on this project by a very small amount could easily result in input errors in excess of 1.0 ft. Inasmuch as the successive models were tied by pass points in stereotriangulation and the individual models were not leveled (control not furnished), misidentified vertical control points could not be detected during the bridge. This problem of exact control point identification would definitely indicate that all control points, including vertical wing points, to be used as the basic control on any stereobridge should be paneled before photography for easy photo identification.

Research into several areas of stereotriangulation is continuing. The Texas Highway Department recently purchased a Wild RC-8 aerial camera with an aviogon lens and obtained correction plates for the Zeiss C-8 Stereoplanigraph ground especially

TABLE 6

F. M. 1604 STEREOTRIANGULATION RESEARCH AREA[a]

Point	Three Single Control Bands			Three Cluster Control Bands			Four Single Control Bands		
	X	Y	Z	X	Y	Z	X	Y	Z
1205+00.2	0.0	+0.2	-0.1	+0.1	0.0	-0.4	+0.1	0.0	-0.2
1208	0.0	0.0	+0.5	0.0	-0.2	+0.2	0.0	-0.2	+0.5
1211	-0.1	+0.2	+1.0	-0.2	0.0	+0.8	-0.2	0.0	+1.0
1214	+0.2	+0.2	+1.2	+0.1	+0.1	+0.9	0.0	+0.1	+1.2
1217+15.4	0.0	+0.1	+1.1	-0.2	+0.1	+0.8	-0.2	+0.1	+1.0
1220	0.0	+0.2	+1.4	-0.2	+0.1	+1.1	-0.2	+0.1	+1.3
1230	-0.2	+0.2	+1.9	-0.5	+0.1	+1.6	-0.4	+0.1	+1.8
1233	+0.2	+0.4	+1.5	-0.2	+0.3	+1.2	-0.1	+0.4	+1.4
1236	-0.9	+0.3	+0.9	-0.4	+0.2	+0.6	-0.4	+0.4	+0.7
1239	-0.2	+0.1	0.0	-0.6	0.0	-0.1	-0.4	+0.2	+0.1
1240+34.4	+0.1	+0.3	+0.2	-0.5	+0.2	-0.1	-0.3	+0.4	+0.1
1243	-0.2	+0.2	-0.1	-0.6	+0.1	-0.3	-0.4	+0.3	-0.2
1246	-0.1	+0.4	-0.8	-0.4	+0.3	-1.0	-0.2	+0.5	-0.9
1249	0.0	+0.2	-0.4	-0.3	+0.2	-0.7	0.0	+0.3	-0.6
1252	0.0	+0.2	+0.4	-0.3	+0.1	-0.9	+0.1	+0.4	-0.8
1255	-0.2	+0.2	-0.9	-0.4	+0.2	-1.1	0.0	+0.5	-1.1
1258+02.5	-0.1	+0.3	-0.7	-0.3	+0.2	-1.0	+0.1	+0.5	-1.0
1261	-0.1	+0.3	-0.6	-0.2	+0.3	-0.9	+0.3	+0.5	-0.9
1264	0.0	+0.2	-0.6	-0.1	-0.1	+0.8	+0.4	+0.4	-0.8
1267	-0.2	0.0	-0.7	-0.3	-0.1	-1.0	+0.4	-0.2	-1.0
Avg.	(0.140)	(0.210)	(0.750)	(0.295)	(0.145)	(0.775)	(0.210)	(0.280)	(0.830)

[a]Centerline vertical data indicative of vertical data on wing points.

for the lens characteristics of this camera. An area similar to the I-35 Test area will be controlled, paneled, and photographed with the Wild RC-8 in the immediate future, and numerous research tests of stereotriangulation accuracies on the Zeiss C-8 Stereoplanigraph will be conducted on this area. Stereotriangulation tests on a smaller scale map project (1 in. = 200 ft) are also scheduled. Research is continuing on the three-projector Kelsh plotters utilizing the duplication of models (setting the tie-in model twice), in conjunction with a computer adjustment program.

As a result of a thorough evaluation and analysis of the data obtained on stereotriangulation during this research project, the Texas Highway Department plans to obtain and use control from stereotriangulation for photogrammetric mapping projects to be utilized in highway engineering. Minor limitations will be placed on vertical control data at this time.

Utilization of Photo Interpretation in the Highway Field

HAROLD RIB, Highway Research Engineer, U.S. Bureau of Public Roads

Many papers have been written in the past 20 to 25 years indicating the various possible applications of photo interpretation in the highway field. The purpose of this paper is to indicate to what extent these applications are being utilized by the various highway organizations. Summaries of the use of photo interpretation by major areas of photo interpretation and by stages of highway engineering are included. The paper also includes discussions on research under way and special applications of photo interpretation in the highway field, as well as the highway department's cooperative work with the Bureau of Public Roads utilizing Highway Planning and Research funds.

•THE VALUE of photo interpretation in the highway field has been discussed and demonstrated for over 20 years, yet its adoption by highway organizations has been slow. Only a limited number of highway organizations were utilizing photo interpretation extensively in their work before the enactment of the Federal-aid Highway Act of 1956. The enormous amount of work involved in the planning, design, and construction of the 41,000 mi of highways in the Interstate System, coupled with the shortage of engineering manpower available for this large undertaking, has resulted in a greater use of photo interpretation by the highway organizations. An analysis of a survey of highway organizations made in 1955-1956 (1) indicated that 33 highway organizations used photo interpretation in some phases of their work; however, only 10 indicated a fairly extensive use. Analyses of recent surveys (2, 3, 4) and projects of the highway organizations indicate that 50 are now employing photo interpretation in some phase of their work and 22 of these are using it fairly extensively.

The objectives of this paper are to (a) indicate the extent to which photo interpretation is currently being utilized by the highway organizations, (b) point out some of the special applications and research under way in photo interpretation by various highway organizations, and (c) indicate the type of work performed by the U.S. Bureau of Public Roads (BPR) and highway organizations cooperating with BPR utilizing the $1\frac{1}{2}$ percent planning and research funds. No attempt is made to show how photo interpretation can be used in particular areas of highway engineering. This type of information can be found in the references to this paper especially in Refs. 2, 5, and 6.

The summarizations in this paper deal with 53 highway organizations: the 50 state highway departments, the District of Columbia Highway Department, the Puerto Rico Department of Public Works and BPR. In the summaries of the use of photo interpretation by the various highway organizations, only the number of highway organizations utilizing this is shown. A listing by individual organizations is not included, because a major portion of the information utilized in the summaries was obtained from answers to questionnaires, and the answers by the individual organizations to one of the questionnaires were confidential.

PHOTO INTERPRETATION IN THE HIGHWAY FIELD

Photo interpretation is used here in the broad sense employed by many highway engineers, i.e., to include all types of activities where qualitative information is

Paper sponsored by Committee on Photogrammetry and Aerial Surveys.

obtained from photographs. This is necessary since most of the information utilized in this paper is derived from questionnaires, and there is generally no reliable method whereby the various levels of photo interpretation—photo reading, photo analysis, and photo interpretation (5, p. 6; 6, p. 852)—can be differentiated.

There are numerous applications of photo interpretation in the highway field and various methods of summarizing the uses by the highway organizations. For this study, two types of summaries are made: (a) by the major areas of photo interpretation applicable to highway engineering, and (b) by the stages of highway engineering in which photo interpretation can be utilized.

The major areas of photo interpretation applicable to highway engineering include:

1. Location of construction materials (sand, gravel, borrow, clay);
2. Evaluation of soils and geology and preparation of engineering soil maps;
3. Evaluation of ground conditions affecting highway alignment, including topography (cut and fill, amount of bedrock and common excavation), soils (plastic, organic, erosive), drainage (seepage, flooding, high water table), landslides, and faults and earthquake zones;
4. Drainage studies and preparation of drainage maps;
5. Land-use studies;
6. Traffic surveys and analysis;
7. Planning subsurface exploration and surveying programs; and
8. Highway condition and inventory, and damage surveys.

Table 1 gives the summary by the major areas of photo interpretation in the highway field and indicates how many highway organizations are utilizing photo interpretation in these major areas. This summary is based on an analysis of several recent questionnaires (2, 3, 4), as well as on the author's knowledge of the activities of various highway organizations through their cooperative work with BPR.

The stages of highway engineering in which photo interpretation may be used are as follows:

1. Highway planning;
2. Traffic surveys;
3. Highway location surveys, including reconnaissance survey of areas to determine feasible routes, reconnaissance survey of alternatives to select route, prelimi-

TABLE 1

HIGHWAY ORGANIZATIONS UTILIZING PHOTO INTERPRETATION IN HIGHWAY FIELD BY MAJOR AREA OF USE

Major Areas of Use	No. of Organizations
Location of construction materials	32
Evaluation of soils and geology—preparation of engineering soil maps	42
Evaluation of ground conditions affecting highway alignment	36
Drainage studies—preparation of drainage maps	38
Land-use studies	38
Traffic surveys and analyses	11
Planning subsurface exploration and surveying programs	39
Road condition and inventory and damage surveys	15

nary survey of the selected route for design and preparation of construction plans, and location survey—staking of designed location on the ground;

4. Construction surveys;
5. Condition and inventory surveys; and
6. Maintenance surveys.

The number of highway organizations utilizing photo interpretation in the various stages of highway engineering is shown in Figure 1. The extent of use—slight, moderate or extensive—is also indicated. This summary is based on the answers by the highway organizations to the "Photographic Interpretation Surveys" portion of the 1962 Questionnaire on Use of Aerial Surveys (2).

An evaluation as to the extent of use of photo interpretation within each stage was accomplished by grouping the various items listed in each stage into the major areas of photo interpretation (land use, soils and geology, drainage, etc.). A weighted percentage was then determined based on the number of major areas of photo interpretation included within each stage and the possible degree of utilization of photo interpretation within each major area. The criteria on which the ratings of slight, moderate or extensive are based are as follows:

Slight—Utilized in up to one-third of the total possible uses in each stage;

Moderate—Utilized from one- to two-thirds of the total possible uses in each stage; and

Extensive—Utilized for more than two-thirds of the total possible uses in each stage.

The total number of responses on which the analysis is based is 50. In this number, BPR is considered as one organization; however, the degree of utilization by the

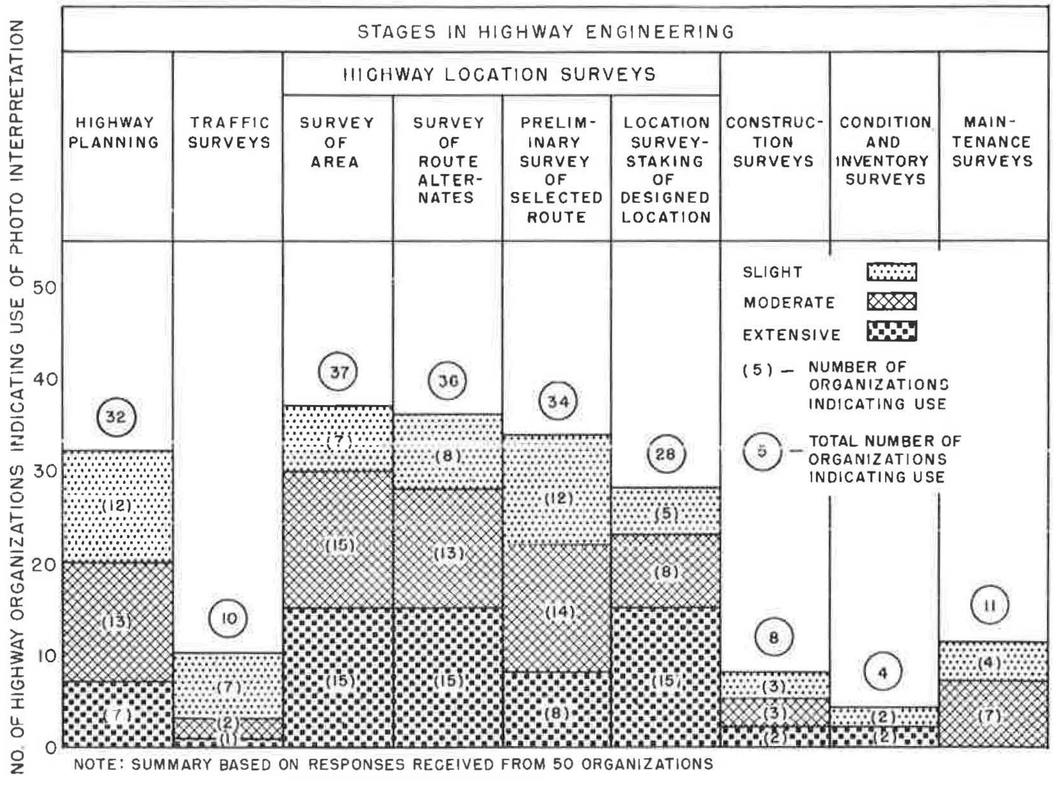


Figure 1. Highway organizations utilizing photo interpretation in the highway field, by stages in highway engineering.

Bureau is based on an average of the answers received from four of the Bureau's regional offices.

ANALYSIS OF SUMMARIES AND CONCLUSIONS INDICATED

It is generally considered that the greatest value of photo interpretation in the highway field is in the early stages of highway location, that is, highway planning, reconnaissance survey of area and preliminary survey of alternate routes. The reason for this is that several of the environmental factors controlling the economics of highway location, such as topography, soils, land use and availability of materials, can be evaluated by photo interpretation and the information thus developed can be utilized to arrive at the best and most economical solution. An analysis of Figure 1 indicates that in these stages, 32 to 37, or approximately 65 to 75 percent, of the responding highway organizations utilize photo interpretation in their work; however, only 7 to 15 organizations or approximately 15 to 30 percent of the responding highway organizations make extensive use of photo interpretation in these stages.

Photo interpretation is also of great value in the preliminary survey of a selected route for evaluating the environmental factors as well as for planning a soil survey program to obtain information for the final design of the highway. As indicated in Figure 1, 34 of the highway organizations, or 68 percent of those responding, utilize photo interpretation in this stage. However, only 8 organizations, or 16 percent of the responding organizations, utilize the method extensively.

Considerable use is also being made of photo interpretation in location surveying and staking of the designed location. Twenty-eight of the highway organizations indicated the utilization of photo interpretation in this stage, and more than half of these indicated a fairly extensive use.

The application of photo interpretation in the remaining stages, traffic surveys, construction surveys, condition and inventory surveys, and maintenance surveys, is very limited. Comparatively few highway organizations are experimenting with or developing procedures or techniques for the use of photo interpretation in these stages. Some examples of the use of photo interpretation in some of these stages have been given (7, 8, 9).

An analysis of Table 1 indicates that photo interpretation is utilized by up to 42 of the 53 highway organizations for the determination of one or more of the environmental factors that control the economics of highway location (i.e., ground conditions, soils, availability of materials, land use, etc.), as well as for drainage studies and planning field exploration and surveying programs. However, it is evident from Table 1 and Figure 1 that there is a limited number of highway organizations utilizing photo interpretation in the areas of traffic surveys, road condition and inventory studies and damage surveys.

Although Figure 1 and Table 1 indicate that, in the majority of the stages of highway location and major areas of photo interpretation, one-half to three-fourths of the highway organizations are utilizing photo interpretation in their work, only one-fourth to one-half of these are using it extensively. Most important in determining the extent of its use by a highway organization are the presence of photo interpreters on the staff of the organization and the knowledge, background, and experience of these interpreters. It is interesting to note that, in a questionnaire published in 1962 (3), only 12 highway organizations indicated they had photo interpreters on their staff (BPR not included in this summary); however, from the author's contacts with the state highway departments, it is estimated that today between 20 and 25 highway organizations have photo interpreters. This may well be the reason for the lack of extensive use of photo interpretation by most of the highway organizations.

The need for more trained photo interpreters has been recognized for many years by the highway organizations. Several have sent personnel to the special short courses on photo interpretation given by several universities (e.g., Cornell and Purdue). In addition, BPR has given short courses. Also, many universities are increasingly emphasizing photo interpretation. In a survey of university engineering schools made in 1962 (10), 29 out of the 96 answering indicated they planned to increase their emphasis on photo interpretation.

In summary, based on an overall study of the information available and known to the author, the use of photo interpretation by the 53 highway organizations is as follows:

1. Fifty, or 94 percent, utilize photo interpretation;
2. Three, or 6 percent, indicated no use;
3. Fourteen, or 26 percent, use photo interpretation to a slight extent;
4. Fourteen, or 26 percent, use photo interpretation to a moderate extent;
5. Twenty-two, or 42 percent, utilize photo interpretation extensively;
6. Only two have indicated a use of photo interpretation in all stages of highway engineering.

RESEARCH AND SPECIAL STUDIES

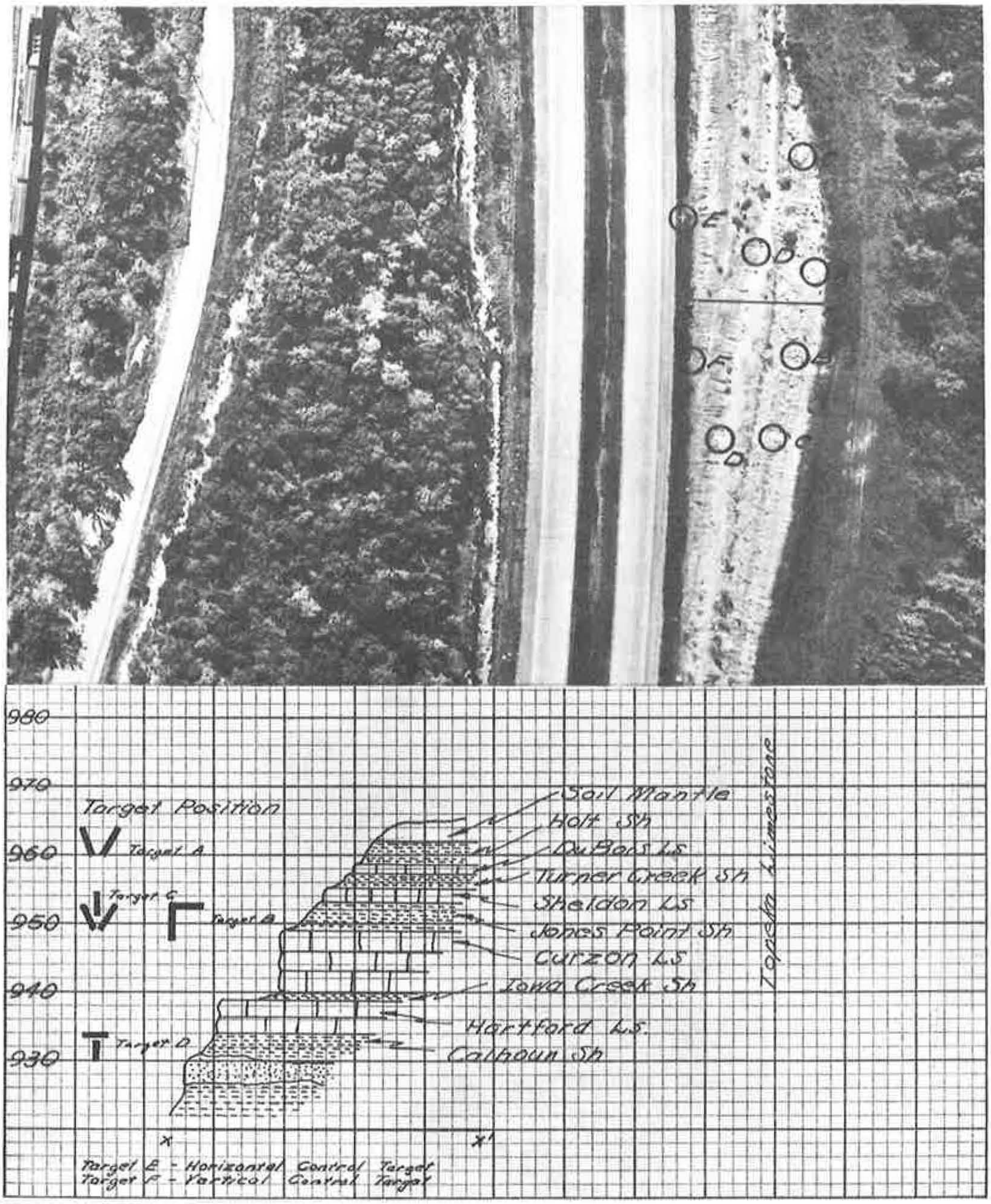
In all phases of photo interpretation as applied to the highway field, information is obtained in practically all cases by the stereoscopic examination of black-and-white panchromatic aerial photographs or from the study of mosaics prepared from the black-and-white aerial photographs. However, within recent years, several highway organizations have been investigating the use of other types of photography, i.e., black-and-white infrared, color, and color infrared.

Several organizations, including BPR Indiana, Kansas, Michigan, Montana, Ohio, Virginia and Wyoming, have been investigating the use of color aerial photography for photo interpretation purposes. The use of color photography as reported in the Bureau's projects (11, 12) has proved so successful that the Region 9 office in Denver now uses color aerial photography for photo interpretation purposes for practically all of their projects in national forests and parks.

Research in the value of color photography is being performed by the Ohio Department of Highways and Ohio State University under a cooperative agreement with BPR. In this project, the university is evaluating various film-filter-scale combinations to determine the usefulness of photo interpretation in obtaining design information for the highway engineer. Areas under study are landslide-susceptible terrain and organic or soft subsoil areas. Multisensor photography (panchromatic, infrared, color, and color infrared) are being evaluated at scales of 1/9,600, 1/4,800 and 1/2,400. In addition, various filter combinations are being studied. Preliminary reports on this project (13, 14) have indicated that for organic soils, the best differentation of surface moisture conditions occurred on panchromatic film with no filters, and for landslide areas color photography was best for differentiating the type of rock outcrops and for determining details in the shadows. The report also indicated that a scale of 1/9,600 was satisfactory in both landslide and organic area, but for evaluating items such as depth of organic layers and for differentiating types of rock, a scale of 1/4,800 was preferable.

Interesting research projects utilizing photo interpretation techniques are also being conducted by the State Highway Commission of Kansas. Some typical uses, as well as special uses, of photo interpretation of black and white aerial photographs have been reported (8). Some of the special projects discussed are (a) the use of special targeting techniques in conjunction with photo interpretation for the identification of geologic formations and evaluation of the use of these geologic materials in highway construction; (b) the study of channel changes over a period of years to try to evaluate the effect of a highway on channel changes or the effect of channel changes on an existing or proposed highway; and (c) a bridge deck condition survey. The meander patterns of streams are also being studied in an attempt to correlate the meander angle (function of degree of curvature of the stream meander) as seen on the photograph and the type of material the stream is cutting through. Some examples of the work done by State Highway Commission of Kansas are shown in Figures 2 and 3.

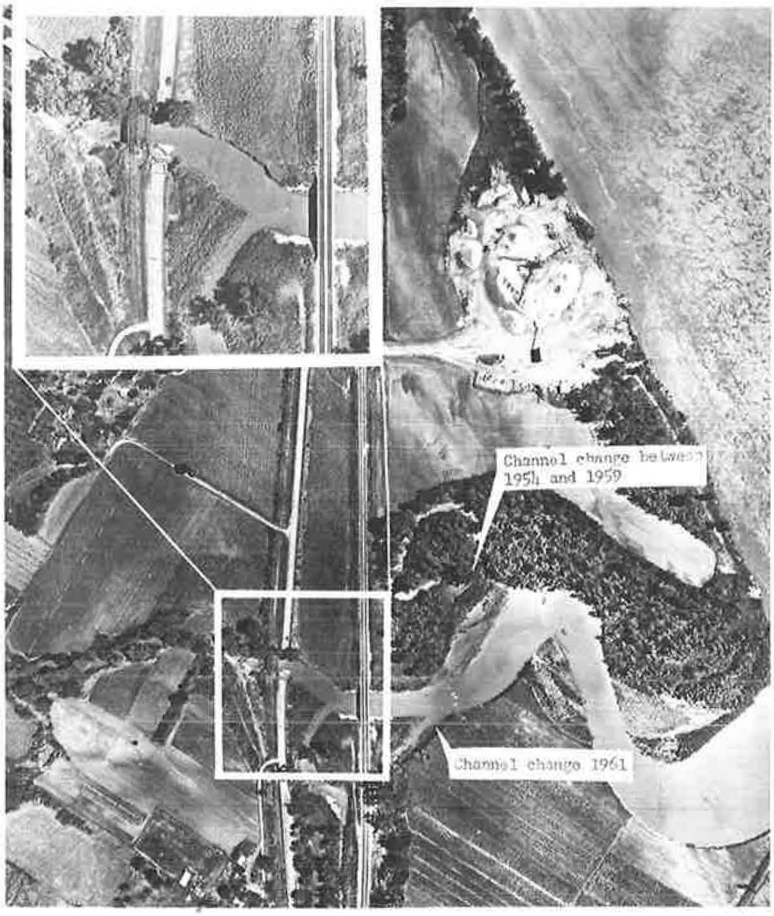
Figure 2 shows an example of a geologic exposure that has been targeted so the thickness, strike, dip and extent of the various geologic formations can be determined and mapped. This geologic information may then be used in the design of rock slopes, estimation of the quantity of rock that will be encountered during construction, and the determination of the possibility of slides and other maintenance problems that may be encountered in existing facilities.



COURTESY STATE HIGHWAY COMMISSION OF KANSAS

Figure 2. Example of targeting of specific geologic formations for determination and mapping of thickness, strike, dip and extent of various geologic formations.

An example of the use of photo interpretation for the evaluation of channel changes over a 21-year period and the effect of the channel changes on an existing road is shown in Figure 3. Aerial photographs taken at five different times during this period were studied to ascertain the stream activities. From photo interpretation and the evaluation of the stream gradient, it could be determined that the collapse of the highway

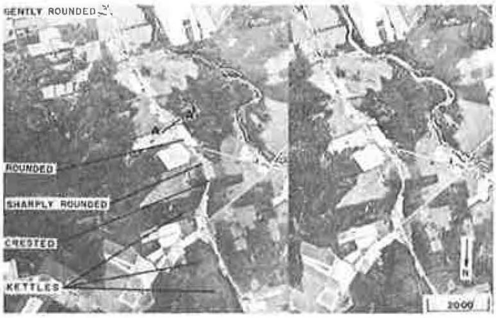


COURTESY STATE HIGHWAY COMMISSION OF KANSAS

Figure 3. Use of interpretation of five sets of photographs taken over 21-year period to determine reason for collapse of highway bridge.

bridge was caused by the increased erosive action of the stream within recent years due to channel changes occurring in this period.

The Maine State Highway Commission and the University of Maine under a cooperative agreement with BPR have recently completed two research projects on the use of photo interpretation techniques for obtaining information of value to highway engineers. In the first study, various features, such as size, orientation, elevation and type of vegetative growth, were evaluated to determine if reliable estimates could be made on the depth of peat by photo interpretation techniques. A statistical analysis made on data obtained from 174 sites indicated that none of the features studied were statistically reliable for predicting the depth of peat, although general trends were noted (15). In the second study certain features of eskers discernible by photo interpretation techniques such as longitudinal slope configuration, transverse shape, and height were evaluated to determine if some relation existed between these features and the grain size of the materials in the esker at a given point. Results indicated that the combination of longitudinal slope configuration and transverse shape were very valuable in predicting locations of gravelly materials, and for the areas checked, the prediction of gravelly materials was correct in 70 percent of the cases (16). Figure 4 is a stereogram showing a 2-mi section of an esker indicating four of the seven basic transverse shapes evaluated in this study (gently rounded, rounded, sharply rounded and crested).



COURTESY MAINE STATE HIGHWAY COMMISSION

Figure 4. Stereogram showing 2-mile section of esker and several shape configurations analyzed to correlate shape with grain size of materials encountered.

In addition, this study also developed criteria for defining the limits of excessive overburden on esker flanks so that a photo interpreter could determine realistic boundaries for material volume estimates.

PHOTO INTERPRETATION AND THE BUREAU OF PUBLIC ROADS

Throughout the years, BPR has been one of the leading proponents of the use of photo interpretation in the highway field. For over 15 years, BPR has been promoting the use of photo interpretation through education and research and has been encouraging the highway organizations to use Highway Planning and Research funds available to them for research in this area.

In 1946, the Bureau entered into a cooperative agreement with the New Jersey State Highway Department for development of engineering soil maps for the state. Work on this project was performed by Rutgers University under contract with the state highway department, and photo interpretation was used extensively for the development of the engineering soils maps. Rhode Island completed a similar project in 1956 using photo interpretation extensively. Cooperative work with Maine State Highway Commission was first initiated in 1948 for the development of engineering soils and materials maps using photo interpretation techniques. This work is still under way, but is mainly utilized for highway location studies. In addition, as previously indicated, techniques have been investigated for increasing the amount of information that can be derived by photo interpretation. At present there are 19 states in which photo interpretation techniques are being utilized using the $1\frac{1}{2}$ percent Planning and Research funds. Some of these projects are (a) materials mapping on a strip, county or state basis; (b) engineering soils mapping on a strip, county or state basis; (c) evaluation of terrain investigation techniques; and (d) special projects, such as the engineering classification of geological materials. Two procedures manuals for performing material studies have been developed under the cooperative program (17, 18).

In addition to promoting the use of photo interpretation by the state highway departments, the Materials Division of BPR has conducted or participated in 14 schools of a 1- to 2-week duration to train highway personnel in the techniques and applications of photo interpretation.

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Evaluation of Color Aerial Photography in Some Aspects of Highway Engineering

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To understand the usefulness of color aerial photography in its application to specific highway engineering problems, the State Highway Commission of Kansas has conducted a color aerial photography evaluation program. An evaluation criterion is established to guide the photographic interpreter and to emphasize the various facets of the photo interpretation techniques most instrumental in evaluating the various types of aerial photography for road condition surveys and material inventories. Procedures used in the program and a discussion of factors affecting the interpretation process used for each engineering project are presented. Most of the findings are based on the fact that the usefulness of color aerial photography is governed by the objectives of the investigations, the photographic interpretation pattern elements most instrumental in the photographic analysis, and the natural conditions existing in the area being investigated. In essence, these factors determine whether or not the color imagery or a change of color of the imagery provides additional information pertinent to the objectives of the investigation as opposed to the quality and quantity of information extracted from black-and-white photography.

•COLOR AERIAL photography provides the photographic interpreter with the additional dimension of color contrast and a more natural image to observe and study. However, the usefulness of such photography is not fully known in many fields of endeavor.

It is the purpose of this paper to evaluate color aerial photography for specific uses in the engineering field. The evaluation pertains to photography used on two specific engineering studies, materials inventories and road condition surveys. The former is associated with a statewide materials inventory program being made by the State Highway Commission of Kansas in cooperation with the U. S. Bureau of Public Roads. The manuscript of the color evaluation report has not been reviewed by the Bureau.

The road condition surveys are a part of a research project on the performance of concrete pavement being conducted by the Research Department of the State Highway Commission of Kansas. The use of color aerial photography as a phase of this study is being investigated by the Photogrammetry Section of the Commission. Special emphasis is being given to a particular type of surface deterioration associated with a surface staining of the pavement. Various sections of pavement are characterized by different stages of stain development; i.e., some sections have only a few stains that can be detected, whereas others have a large percentage of the surface covered with a stain associated with extensive pavement disintegration. The rate at which the staining spreads on the concrete surface, the pattern of stain development, and the stage of stain development at which initial distress is first detected are all factors that may be useful in determining the cause of such stains and, ultimately, the cause of the deterioration.

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To learn more about stain "activity," aerial photography of selected sections of highway pavement will be studied periodically for stain appraisal and mapping purposes. In this paper, the term road condition survey refers to the proposed stain appraisal and mapping process.

To ascertain the most effective aerial film for road condition surveys, an evaluation program was conducted. The objective of the program was to determine the degree to which a given type of photography will increase the usefulness of a given pattern element. To complete the evaluation, the information derived from the pattern element must be appraised as to its significance to the objective of the project.

The general procedures used to evaluate the usefulness of color aerial photography on the two engineering studies are similar; however, the nature and magnitude of the problems encountered on each were quite different. The objectives of the road condition surveys are less extensive but more detailed in nature, and the ground conditions are uniform. The objectives of the materials inventories are larger in scope and the ground conditions are highly variable. Therefore, even though similar procedures are used, each evaluation is discussed separately.

To provide a consistent evaluation procedure throughout the investigation, an evaluation criterion was established. As each facet of the investigation was being completed, the following questions included in this criterion were answered:

1. What information is desired or what are the objectives of the investigation?
2. What area and, consequently, what type of ground conditions are being investigated?
3. How much and what type of ground information is available?
4. How instrumental is ground information in the photographic interpretation process in obtaining the objectives of the investigation?
5. What photographic interpretation clues are most essential or are used the most in the photo analysis?
6. Does color photography make any particular clue or pattern element more perceptible or meaningful to the objectives of the investigation?
7. Does color photography provide any additional clues that cannot be detected or used on black-and-white photography?
8. How much time is required to complete the photo analysis using the various types of aerial film?
9. What is the cost of each type of film?

On the basis of the answers of these questions, the usefulness or the effectiveness of each type of aerial photography used in this investigation is presented in the conclusions of this paper in terms of time required to complete the photographic interpretation, the quality and quantity of information derived from the interpretation process, the cost of the film, and other factors relevant to the objectives of the investigation.

ROAD CONDITION SURVEYS

Area of Investigation

The test areas in which this investigation took place were a 2-mi strip of a 4-lane highway in northeast Kansas, and two test strips in the Topeka area. The former was characterized by advance stage staining; the latter was characterized by initial stage stains. These areas were photographed by the State Highway Commission of Kansas using Kodak Aero Ektachrome film and Agfacolor Negative film at an approximate scale of 1:1,200. The conditions under which the two types of photography were taken were as uniform as possible; i.e., all photography was taken at nearly the same time of day under similar atmospheric conditions.

Color transparencies were obtained from the Kodak Aero Ektachrome film and both color and black-and-white prints were printed from the Agfacolor Negative film. The black-and-white photographs were printed on a single weight matte as well as a glossy photographic paper to evaluate both types of prints.

Procedure

Initially, all stains were classified on the basis of information obtained through field work and the discernible stain characteristics observed on the aerial photography. The stain classification system was developed for identification purposes and to provide a basis of pavement panel classification. Subsequently, pavement panels were classified as to the type and number of stains in each panel that could be detected on aerial photography. It is beyond the scope of this paper to present a detailed description of the stain classification system.

Each test strip was mapped and appraised by using the various types of photography, the poorest quality photography being used first and the best quality last. The mapping indicated the number and length of pavement panels included in the test strips and each stain was classified according to the classification system adopted for this investigation. The test strips were then mapped in the field using the same mapping procedure and classification system. The field data were used to evaluate the results obtained from the interpretation of the aerial photography.

Evaluation of Photography

When conducting road condition surveys, the interpreter has to associate the severity of the pavement disintegration with the discernible characteristics of the pavement stains. Initially, it was thought that a variation in stain color would be indicative of the severity of the pavement's condition. This was found to be true only to a limited extent. Early stage or initial stains (when the stains are first detectable) are very light compared to the more advanced stains. This is true not because of color or tone difference, but because of the degree to which small individual stained areas have merged or interconnected. The merging of the smaller spot stains ultimately governs the size of the area which the stain covers. For the most part, it is the degree of concentration of spot stains that governs the detectability of this particular highway stain on aerial photography. The stain, especially in the advanced stage of development, is fairly consistent in color and, consequently, is quite readily discernible on color prints.

Severity association is accomplished by noting stain width and number during the stain and panel classification. To classify the stains and panels accurately, the interpreter had to be able to distinguish significant highway stains from other highway discolorations. Typical discolorations discernible on aerial photography are finishing marks, "road scum," asphalt stains, areas ground off for leveling purposes, tire marks, and the concrete itself. Many of the discolorations have either a distinct color of their own or a very inconsistent color pattern. However, some are rendered in a tone similar to significant highway stains on black-and-white photography. Figure 1 illustrates finishing marks and a significant stain on black-and-white photographs. The insignificant finishing marks and the highway stain are rendered in nearly the same shade of gray on the black-and-white photography; however, a color change is observed when comparing the same two discolorations on color photography.

Figure 1. Black-and-white photograph portraying insignificant pavement finishing mark and significant highway stain.

In essence, the biggest problem of completing road condition surveys was differentiating between significant highway stains and insignificant highway discolorations. The effectiveness of the various types of film used in this study was measured in terms of number of significant stains detected compared to the number present and the number of insignificant stains erroneously mapped on a given section of pavement.

Black-and-White Photography (Single Weight Matte Prints).—Only 24 percent of the stains that were in the initial stage of development were detected when black-and-white single weight matte prints were being used for the photo analysis of the selected pavement test sections. On many occasions, the stains in the more advanced stage of development were difficult to detect; however, because of their larger size, approximately 50 percent of the advanced stage stains were detected and mapped accurately.

When mapping the advanced stage stains, little difficulty was encountered in differentiating between significant and insignificant stains; however, during the process of mapping initial stage stains, 62 insignificant stains were mistaken for significant stains.

The main defect of black-and-white photography printed on matte photographic paper for road condition survey purposes is poor resolution. All highway stains, regardless of their nature, were obliterated by the grain size of the photographic paper which contributed to the difficulties discussed previously.

Black-and-White Photography (Glossy Prints).—When compared with matte surfaced photographs, glossy prints greatly improve the resolution of black-and-white photography, resulting in a higher percentage of stain being detected. Approximately 57 percent of the initial stage stains and 95 percent of the advanced stage stains were detected when using the glossy prints. No insignificant stain was mistaken for a significant one when working with the advanced stage of stain development; however, 57 insignificant stains were recorded on pavement in the initial stage of staining.

The black-and-white glossy prints improve the detectability of all stains, significant and insignificant; however, the main difficulty encountered when using this type of photography is differentiating between these types when mapping pavement characterized by stain in the initial stage of development.

Color Transparencies.—When comparing the mapping results produced from black-and-white glossy prints with those obtained from color transparencies, a lower percentage of significant stains was detected, and a greater number of insignificant stains were erroneously mapped. When the initial stage stains were being mapped by use of the color transparencies, only 43 percent of the significant stains were detected, and 91 insignificant stains were mapped. Better mapping results were obtained from stains in the advanced stage. In all cases the color transparencies provided better results than the photography printed on single weight matte photographic paper.

Even though color transparencies provided the interpreter with color contrast and excellent resolution, several factors influenced the quality of the information extracted from the transparencies. The main factor detrimental to the interpretation process was the quality of the photography for road condition survey purposes. The Kodak Aero Ektachrome film exposure was based on the terrain adjacent to the highway and not on the light-colored concrete pavement. Consequently, the pavement was overexposed and the adjacent terrain was rendered in natural color. The pavement appeared to be washed out on the color transparencies. Much of the stain was not recorded on the film and some of the more conspicuous insignificant stain was mistaken for significant stain. This discrepancy was especially apparent on the pavement characterized by stain in the initial stage of development.

Better results could probably have been obtained if better equipment had been used to interpret the color transparencies. For example, a light table equipped with lights of different intensities and different colors would provide optimum conditions for the detection and evaluation of a given size and color of stain.

Agfacolor Negative Film.—Color prints proved to be far superior to any other type of photography in detecting and mapping initial stage stains. Approximately 84 percent of the initial stage stains were detected, and only 24 insignificant stains were erroneously recorded. When working with the advanced stage stains, the color prints proved to be no more effective than the black-and-white glossy prints; i.e., approximately 95 percent of the advanced stage stains were detected and few if any insiginificant stains were erroneously mapped.

The color prints were of good quality. The areas adjacent to the highway pavement were slightly underexposed and good quality color prints for road condition survey purposes were obtained. The better photography was a result of experience gained from previous test photography.

MATERIALS INVENTORIES

Procedures

For the purpose of this investigation, three counties (Ellis, Mitchell and Brown) included in the current State Highway Commission of Kansas statewide county materials inventory program were selected as test areas. Each county has different geological conditions. Materials inventories were completed in all three counties using black-and-white photography printed from DuPont Cronar Safety Film at a scale of 1:24,000.

As each county was being investigated for the location of construction materials, areas providing typical problems encountered during the photo analysis of the county (approximately 10 percent of the area of the county or 90 sq mi) were selected to be photographed with color aerial photography. These areas were selected on the basis of the type of geological source beds present, amount of overburden, and other properties that may be peculiar to the specific area.

The selected areas in Ellis and Brown Counties were photographed with black-and-white photography (DuPont Cronar Safety film) and Kodak Aero Ektachrome film at a scale of 1:12,000. Black-and-white photographs were printed on single weight matte photographic paper from the Cronar Safety film, and color transparencies were developed from the Kodak Aero Ektachrome film. The same procedure was followed in Mitchell County; however, instead of using DuPont Cronar Safety film to obtain the larger scale black-and-white prints, Agfacolor Negative film was used. Black-and-white prints and a limited number of color prints of the area being investigated were made from the Agfacolor Negative film. The selected area in Mitchell County was also photographed using Kodak Aero Ektachrome film to obtain positive color transparencies. The photography used in each of the test counties was as follows:

Ellis and Brown Counties—DuPont Cronar Safety film, black-and-white single weight matte prints at scales of 1:12,000 and 1:24,000; and Kodak Aero Ektachrome film, color positive transparencies at a scale of 1:12,000.

Mitchell County—Agfacolor Negative film CN 17, black-and-white single weight matte prints and color prints at a scale of 1:12,000; DuPont Cronar Safety film, black-and-white single weight matte prints at a scale of 1:24,000; and Kodak Aero Ektachrome film, color positive transparencies at a scale of 1:12,000.

After the procurement of the evaluation photography, each test section of each selected county was analyzed using the black-and-white matte prints and the color photography. The results obtained from each type of photography were compared with the results of a ground investigation of these same areas (ground reconnaissance and exploratory drilling).

Criteria

The usefulness of color aerial photography is best realized when the color of the image improves and increases the amount of information above that normally obtained from the tonal qualities of the black-and-white photography. The evaluation, therefore, requires a knowledge of not only the type of geology (ground conditions) and, consequently, the construction material source beds in an area, but also the pattern elements used to accomplish the interpretation. It must also be determined whether or not a color image improves the pattern element or elements to such an extent that an additional amount of pertinent information is obtained to justify the higher cost of the color aerial photography. To have a consistent evaluation from one county to another, the evaluation criterion set forth previously was followed.

The objectives of material inventory investigations are consistent throughout the evaluation program—namely, detecting, mapping and describing construction material source beds in the counties being investigated. The ground conditions of the sites being investigated were generally known, and a review of the existing information added to the general knowledge of the area. Such information includes data obtained from material quality tests, geological publications and maps, and soil reports based on field work accomplished in the county to be investigated or in adjacent counties. Geological reports, preliminary soil surveys, and groundwater reports were also available for all three counties included in the evaluation program. Usually, when ground information is available and can be correlated to the natural features as detected on the aerial photographs, the photographic interpreter uses the information derived from the pattern elements to supplement the ground information or vice versa, depending on the availability of the ground information. Consequently, the degree to which the photo interpreter relies on the information obtained through the interpretation of pattern elements will vary with the amount and type of ground information available.

The cost of each type of photography used in the evaluation program is given in Table 1. The evaluation criterion is referred to and discussed in the separate county discussions and in the conclusions of this report.

Evaluation of Photography

Ellis County. — The geology of Ellis County is characterized by interbedded shale and chalky limestones of Cretaceous Age capped in places by silt and fine sand of the Tertiary Age. Silt of the Pleistocene Age blankets most of the county.

A small quantity of poor quality chalky limestone can be produced from the strata of Cretaceous Age and a limited amount of poor quality fine sand and silt can be produced from the Tertiary beds. The main sources of construction material in Ellis County are the terraces of various ages located in the Smoky Hill and Saline River valleys.

The specific tasks used for evaluation purposes in the test area in Ellis County are as follows:

TABLE 1

PHOTOGRAPHY COST

Film	Factor	Cost ($)
DuPont Cronar Safety	Cost of roll (9½ × 230 ft), approx. 300 frames	71.50
	Negative film	0.24 per exposure
	Paper and chemicals[a]	0.10 per print
	Aircraft and labor (avg.)	0.36 per print
		0.70 per print for negative and first set of prints
Kodak Ektachrome Aero	Cost of roll (9½ × 75 ft), approx. 90 transparencies	126.00
	Color reversal negative film	1.40 per transparency
	Chemicals	0.36 per transparency
	Aircraft and labor (avg.)	0.45 per transparency
		2.21 per transparency
Agfacolor Negative CN 17	Cost of roll (9½ × 100 ft), approx. 115 frames	152.59
	Film cost	1.33 per frame
	Chemicals	0.22 per frame
	Aircraft and labor	0.38 per frame
		1.93 per negative frame
	Color prints	
	On contract[b]	3.00 per print
		1.93 per negative
		4.93 per print and negative
	State Highway Commission lab.	
	Cost of paper	0.28 per print
	Labor and chemicals	0.80 per print
		1.08 per print
		1.93 per negative frame
		3.01 per print and negative
	Black-and-white prints	
	Labor, paper, chemicals	0.10 per print
		1.93 per negative frame
		2.03 per print and negative
	Color and black-and-white prints[c]	
	On contract	
	Agfacolor negative	1.93 per negative
	Color print	3.00 per print
	Black-and-white print	0.10 per print
		5.03
	State Highway Commission lab.	
	Agfacolor negative	1.93 per negative
	Color print	1.08 per print
	Black-and-white print	0.10 per print
		3.11

[a] Add $0.02 per print if glossy prints are desired.

[b] At present all color prints are contracted; State Highway Commission of Kansas plans to print its own color prints on completion of proper facilities.

[c] If both color and black-and-white prints desired, cost of Agfacolor negative included only once.

1. Locating volcanic ash for mineral filler,
2. Differentiating between the Ogallala Formation (Tertiary Age) and the Ft. Hays limestone formation (Cretaceous Age) when both formations have a thin veneer of Pleistocene silt; and
3. Differentiating between different aged terraces (Pleistocene Age) containing different types of construction materials located in the Smoky Hill and Saline River valleys.

Ground information played a very important role in the materials inventory of Ellis County. The photo interpreter had a good knowledge of Ellis County geology before the interpretation process was started. Most of the material source beds were known and after the results of the available quality tests were correlated with the source beds, the quality of the material that could be produced from each source bed was also generally known.

Several different pattern elements were utilized in different areas for interpretation purposes. Volcanic ash deposits can be located by having some knowledge of the mode and time of deposition of the formation in which the ash is found; however, unless the ash is exposed, prospective sites can not be located using aerial photography. If the ash is exposed, a light colored band can usually be located. Volcanic ash deposits are unconsolidated and, consequently, do not form a distinctive ledge but blend into the relief of the surrounding terrain. Ash deposits are normally found in unconsolidated material. Even though Cretaceous bedrock has a similar color, the bedrock forms a distinctive ledge. Therefore, if the ash deposit is exposed, color imagery would not be required to differentiate between bedrock and the ash deposit. Ash deposits in Ellis County are rarely exposed, and if they are near the surface, they are discolored by the overlying silts and are not easily distinguishable on any type of aerial photography.

The Cretaceous limestone mapped in Ellis County was distinguished by a distinct tone pattern combined with a very distinctive topographic ledge and topographic position. The Ogallala Formation (Tertiary) was detected and mapped predominantly on the basis of topographic expression combined with a tone pattern peculiar to that formation.

Terraces of different ages were detected and mapped on the basis of topographic position, topographic expression, land use, and drainage characteristics. The first two pattern elements were utilized the most.

Color photography made the terrain easier to observe and more pleasant to study. Consequently, the elements used for interpretation were easier to analyze. This is especially true for interpreters with limited experience; however, although some additional information could be extracted from the color photography by virtue of the color images, it is generally conceded that the additional information would not alter or add enough to the materials inventory in Ellis County to justify the increase in cost. Figure 2 portrays the Fort Hays Limestone of the Cretaceous Age and Pleistocene terraces

Figure 2. Black-and-white photography of Fort Hays Limestone and Pleistocene terraces in Ellis County, Kansas.

along the Smoky Hill River valley in southeast Ellis County as they appear in black-and-white photography. Color is an important factor in delineating Cretaceous bedrock in Ellis County; however, the color of the bedrock is rendered in a very distinctive tone on black-and-white photography.

Brown County.—The geology of Brown County is characterized by interbedded limestone and shale of the Permian and Pennsylvanian Ages capped by heterogeneous glacial drift deposited by the Nebraskan and Kansan glaciers. A thin veneer of silt of the late Pleistocene Age covers most of the high terrain with thicker deposits located in the northeast corner of the county.

The best sources for crushed limestone aggregate are the Permian limestone beds located in the western third of the county. The Pennsylvanian limestones are softer, have more overburden, and are characterized by thinner limestone units. A poor quality of clay-bound siliceous sand and limestone gravels can be produced from the glacial drift. A limited amount of clay-bound chert gravel can be produced from high terraces of the late Pliocene Age in the northwest quarter of the county, and locally derived limestone gravels can be produced from Recent terraces and floodplains of some of the larger drainage channels in the area.

Ground information was used extensively in the mapping and correlating of limestone beds suitable for construction material. A limited amount of ground information was useful in detecting, mapping, and describing the high chert gravel terraces located in the larger drainage channels. Ground information in the form of drill logs was available in areas characterized by thick glacial drift. Because of the heterogeneous nature of the glacial drift, this information supplied data for only the point where the hole was drilled and could not be used for correlation purposes.

Various pattern elements were utilized for the detection and mapping of the different source beds in Brown County. The high chert gravel terraces were mapped predominantly on the basis of topographic position and expression, whereas the limestone units were mapped on the basis of landform, tone and ground information. Limestone gravel deposits in Recent terraces and floodplains were detected and mapped primarily on the basis of landforms, drainage characteristics, and general similarities to locations of known deposits and ground information. Prospective material sites were selected in the glacial till on the same basis, and when the deposits were near the surface, on gully analysis.

All of the primary pattern elements used in the Brown County evaluation were more perceptible on color than on the black-and-white photography. However, the elements used to detect and map bedrock and unconsolidated material in terraces were just as meaningful on the black-and-white photography as the color photography insofar as the objective of this investigation is concerned. This was not true of material investiga-

Figure 3. Black-and-white photograph of glacial till in Brown County, Kansas.

tions in the glacial drift. Since the use of ground information was limited, the photo interpreter was largely dependent on the pattern elements of the photo interpretation process to detect prospective material sites. In some areas, this method was limited because of the thickness of the overburden. The inconsistent nature of the glacial drift made correlation of ground information from one area to another nearly impossible. The main pattern elements used in the interpretation of the drift areas were tone, topographic expression, and gully analysis. During this evaluation, it was discovered that different colored drift material (red clays, yellow sand, etc.) would render a similar gray tone on the black-and-white photography. Certain deposits of coarse and fine sand in the glacial deposits can be associated with a red silty clay to clay deposit. Consequently, it was the detection of these red areas on the color photographs that tentatively located a prospective material site. Although this association may or may not be true in other drift areas, similar associations involving different types of material may be possible.

Figure 3 illustrates an area characterized by glacial till in Brown County, Kansas. Initially, this area, as observed on black-and-white photography, was not considered a prospective material site until the color photography was studied. A very distinctive orange material associated with siliceous sand and gravel was detected at point "A" by use of color photography. Because of the heterogeneous nature of the glacial till, color photography can be profitably used if a potential material source of desirable quality can be located on the basis of material color change. However, unless the photo interpreter has a detailed knowledge of the material present, the association of color of material to location and quality of material may not be established until a portion of the investigation has been completed or color photography was initially used on an experimental basis.

Other than a more extensive use of tone as a pattern element, no other photo interpretation clues other than those used with the black-and-white photography were used or discovered on the color photography for the exploration of construction materials in this particular county.

Mitchell County.—The geology of Mitchell County is characterized by interbedded shale and chalky limestone of the Cretaceous Age capped in places by silt of the Pleistocene Age.

A poor quality chalky limestone can be produced from the Cretaceous bedrock. The main sources of construction material in Mitchell County are terraces of the Illinoisan Age (Crete terraces). These terraces are composed of locally derived limestone from some Cretaceous bedrock units and siliceous sands and gravels derived from the Ogallala Formation. These terraces were laid down by the Solomon River and some of its larger tributaries.

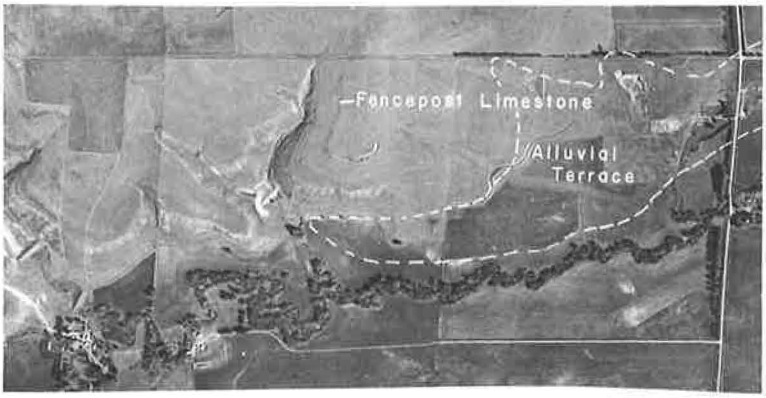

Figure 4. Black-and-white photography portraying Fencepost Limestone and alluvial terrace in Mitchell County, Kansas.

Ground information played a very important role in the material inventory of Mitchell County. The photo interpreter had a good knowledge of the area geology before the interpretation process was started. Most of the material source beds were known and after the available quality tests were correlated with the source beds, the quality of the material that could be produced from each source bed was also generally known.

Some different pattern elements were utilized in different areas for interpretation purposes. Cretaceous limestone formations were mapped on the basis of a distinct tone pattern and topographic position. The Crete terraces were mapped on the basis of ground information, topographic position and expression, general similarities to locations of known deposits, and in some areas, land use.

Color photography greatly improves the usefulness of tone, but tone was not considered a primary pattern element for exploration of construction material in Mitchell County. No other photo interpretation clues other than those used with the black-and-white photography were utilized or discovered on the color photography for the exploration of construction material in this county. The additional information that could be extracted from the color photography by virtue of the color imagery was not relevant to the objectives of the material inventory and, therefore, did not alter or add to the investigation.

Figure 4 is a portion of a black-and-white photograph portraying the Fencepost Limestone member and an alluvial valley in Mitchell County. Even though the limestone source bed is easier to detect on the color photography, the unit can be mapped just as effectively on the black-and-white photography.

SUMMARY AND CONCLUSIONS

Color aerial photography is pleasant to view, and it is conceded that when speaking in general terms, more information can be extracted from it than can be extracted from black-and-white photography; however, unless this additional information is pertinent to the objectives of the investigation, the color photography would only add to the expense of the investigation. The additional information that can be extracted from the color photography is obtained by virtue of color contrast that cannot be detected as a significant tonal change on black-and-white photography. Color photography, in most cases, improves the discernibility of all pattern elements, but unless the color imagery increases the quantity and quality of the information deciphered from the pattern elements, the same information can usually be obtained from good quality black-and-white photography. Concerning the objectives of this investigation, color aerial photography has a distinct but limited use in Kansas.

The conclusions derived from this investigation pertain to two specific engineering problems. Even though these conclusions can be applied to similar projects in different areas, different conclusions may result when the same photography is evaluated for different problems with different objectives. For example, color photography of a given area may not be necessary to complete an accurate construction material investigation but may be indispensable when analyzing and mapping soils of the same area.

The effectiveness of color aerial photography when used to complete road condition surveys and materials inventories can be measured in terms of cost of the film (Table 1), the time required to complete the photo analysis, the amount and type of information extracted from the photography, and other factors that may be peculiar to a given project. The conclusions presented in this paper are based on these factors.

During this investigation, emphasis was placed on the type of information that could be extracted from the various types of film; however, although no detailed time log was maintained during the interpretation process, the relative amount of time required to complete the evaluation, using the different types of aerial photography, was noted. Less time was required to complete the interpretation process when using color photographs printed from the Agfacolor Negative film. The color image, when used to extract information pertaining to the objectives of the projects, would on many occasions identify or delineate a particular item of information by virtue of its color, and the time-consuming activity of further studying and correlating other features was not required. Compared to the color transparencies, all color and black-and-white prints were easier to handle in the office and in the field. Because of better resolution, black-and-white glossy prints were easier and faster to interpret than the black-and-white photographs printed on single weight matte paper. Color transparencies took the longest time to interpret, but the authors feel that this time would be substantially reduced if better quality transparencies (for road condition survey purposes) and better interpretation equipment were available. Assuming good quality transparencies were available, they would provide the photographic interpreter the added benefit of the color images, as explained previously in the color prints discussion, and excellent resolution. However, the added size of the transparencies (all transparencies were inclosed in a plastic jacket) hampered their positioning for steroscopic vision. This situation could be improved by proper interpretation equipment.

Road Condition Surveys

Of all the photography used during this evaluation program, Agfacolor Negative film is best adapted to road condition surveys. One of the outstanding qualities of this film is the possibility of obtaining color or black-and-white prints. Color prints are a necessity to complete an accurate survey on pavements characterized by initial stage stains. Black-and-white glossy prints would suffice for pavement characterized by advanced stage stain. Even though color prints are more costly, the speed of interpretation and the higher quality of data that can be extracted from the color image of initial stage stains offset the higher cost. Because a similar quality of data can be extracted from good quality glossy black-and-white prints when mapping and evaluating the advanced stage stains, these lower cost prints can be utilized.

Good color transparencies can conceivably provide the same quality of image as the color prints at a slightly lower cost; however, the additional time to complete the photo analysis and the fact that a single purpose negative is used (only color transparencies or prints can be obtained from Kodak Aero Ektachrome film), the Agfacolor Negative film is considered more adapted to this particular project. Therefore, if a given pavement is to be surveyed, the portion of the pavement having advanced stage stains may be mapped and evaluated using black-and-white glossy prints, and the portion of the pavement having initial stage staining would have to be mapped using color photography. If Agfacolor Negative film was being used, black-and-white glossy prints, and color prints could be obtained from the same negative. The difference in cost of the color print and the black-and-white prints is such that savings could be realized in areas having widespread advanced stage staining. This situation is more flexible and, consequently, more adaptable to road condition surveys. One flight would suffice for both black-and-white and color photography.

It should be noted, however, that if the pavement to be surveyed is characterized by predominantly advanced stage stains, Cronar Safety film could be used. It is less expensive than the Agfacolor Negative film and has better resolution than the black-and-white photographs printed from Agfacolor Negative film.

Materials Inventories

In general, color photography did not significantly add to the materials inventory investigation except in isolated cases. Areas where the use of color photography is most beneficial are characterized by inconsistent and erratic geology as in southeast Kansas or by thick overburden consisting of heterogeneous material as in glacial terrain in northeast Kansas. Ground information pertaining to the same areas is difficult to project and correlate and, consequently, more reliance must be placed on photographic interpretation techniques.

Ordinarily, the photographic interpreter in Kansas has knowledge of the general conditions of any county that might be investigated for construction material sources. Therefore, the need for color aerial photography can be anticipated once the location of the county to be investigated is known.

The procedure currently being used for materials inventory investigations in Kansas involves investigation of photographs printed from black-and-white DuPont Cronar Safety film. The cost is relatively low, and with only a few exceptions, the quality of data extracted is as good as information extracted from color photographs. The amount of time to interpret the black-and-white prints for materials inventory purposes is less than that required for color transparencies and nearly the same as the time required to interpret color prints.

If color photography is desired for use in material inventories, Kodak Ektachrome Aero film would be most desirable. Color transparencies are more time consuming to interpret but the lower cost of the transparencies would offset the extra cost of interpretation. Although Agfacolor Negative film is a dual purpose film (since both black-and-white and color prints can be printed from the same negative), this dual purpose would not, under existing conditions, be utilized. As mentioned previously, the photo interpreter is able to anticipate the use of color photography when conducting material inventories in Kansas. Once the counties are studied on black-and-white photography, the areas to be analyzed on color photography can be selected. On many occasions, color photography may not be required. The areas selected will usually be relatively small compared to the area of the county being investigated. Consequently, to photo graph the entire county using Agfacolor Negative film would only add to the cost of the project because only a small percentage of the film would be used to print color photographs.

Coal Outcrop and Overburden Mapping with Kelsh Plotter

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The techniques developed for mapping coal seams and overburden using the Kelsh plotter are described. Applicable to terrain to be studied for highway relocation, the method is economical, utilizing the same photogrammetric materials used to produce the topographic map for the relocation study.

The coal seams were deposited under swampy conditions, the base describing an approximate plane which has been subjected to diastrophic forces. Determination of the dip and strike within the mapping limits is done by using existing geological information and photo interpretation of surficial coal manifestations.

Retrieval of the original coal base through leveling the Kelsh models results in a segmented mapping of the coals and overburden, each model leveled to the best information and connected to adjacent models in the flight strip.

•ACQUISITION OF rights-of-way for highway relocation can be an involved process. When mineral deposits occur in the landforms in the vicinity of the relocation, the determination of value, or damage, is even more difficult. Frequently the value of the mineral is many times the land value of adjacent tracts lacking the mineral deposit. Determination of the identity and areal extent of minerals and depiction in a measurable form was the objective of this project.

The original idea for mapping coal seams along proposed highway rights-of-way was formulated by Lloyd O. Herd of the Ohio Department of Highways.

Coal seams were deposited as vegetal matter in low swampy areas and the base of the coal originally occurred in an approximate plane. Subsequent diastrophic forces have both regionally and locally warped this depositional plane. In Ohio, the present dip of the rock strata is generally southeast, interrupted by gentle reversals and small anticlines. Resurrection of the original depositional coal base and leveling this plane in the Kelsh double projection plotter would permit mapping the coal's extent and the overburden.

The highway route selected for exploring and developing this technique was an 8-mi segment of proposed I-70 between the Guernsey County line and Morristown in Belmont County, Ohio. Topographic maps previously compiled at the scale of 1 in. = 200 ft by the Aerial Engineering Section, with the proposed centerline for I-70 superimposed, were available, as were the aerial photography, horizontal and vertical control, and the glass plate transparencies for Kelsh double projection instrument work.

The terrain is dissected sedimentary rock with valleys as low as 950 ft and hilltops frequently exceeding 1,300 ft. Exposed are upper Conemaugh and Monongahela, Pennsylvanian system, and lower Permian system rocks.

Before mapping the coal seams, information was gathered from the following sources:

1. Soil and rock reports by the Ohio Department of Highways Testing Laboratory which pertain to materials along the proposed centerline;

Paper sponsored by Committee on Photogrammetry and Aerial Surveys.

2. Stratigraphic sections measured in the area and on file at the Geological Survey of Ohio;
3. Publications relating to geology of this area prepared by the Geological Survey of Ohio;
4. Coal outcrops delineated by the Geological Survey of Ohio on 1:62, 500-scale topographic maps of the 15-min quadrangle series for Pittsburgh No. 8 Coal and Meigs Creek No. 9 coal;
5. 1:24, 000-scale topographic maps in the 7½-min quadrangle series by the Geological Survey of Ohio;
6. Aerial photography by the U. S. Geological Survey; and
7. Aerial photography taken at the scale of 1 in. = 800 ft and 1 in. = 200 ft by the Ohio Department of Highways.

In the Ohio soil and rock reports, test cores containing coal were representatively plotted on a profile at the vertical scale of 1 in. = 10 ft and the horizontal scale of 1 in. = 200 ft. This profile constituted the basic framework for establishing the dip of the coal seams.

The first step in preparation for accomplishing the coal seam and overburden mapping was review of the available geological information to gain an overall concept of the highway route corridor. Primary objectives in making this review were to determine: (a) the major coal seams which outcrop within the limits of the corridor to be mapped; (b) intervals between coal seams; and (c) possible intermittent seams. The existence of the less persistent coal seams such as the Redstone 8a and the Fishpot was known. These seams are sometimes smut streaks or coal blossom and occasionally may thicken to a true coal which can be mined. Care was required to prevent correlating these coal seams with others of the stratigraphic column as this would have resulted in erroneous leveling in the mapping phase.

Stratigraphic sections were studied and their location was annotated on the topographic maps at a scale of 1 in. = 200 ft, which served as a base for the coal seam mapping. Coal seams reported in the highway segment selected for such mapping were plotted on the profile at the longitudinal position and elevation where they occurred. Occasionally, positioning of the stratigraphic sections was complicated by antiquated descriptions with references to terrain features which no longer exist. Aneroid barometer measurements were used on older sections making vertical positioning less than precise.

Coal outcrop maps for the Pittsburgh No. 8 and the Meigs Creek No. 9 coal mining areas were examined. Recorded on the map are spot elevations at which the coal beds occur. These were recorded on the profile at the position indicated.

The 1:24, 000-scale topographic maps of the 7½-min quadrangle series, dated 1961, were an excellent source of information. Where coal has been stripped, the pit forms a water-filled basin between the high wall and the spoil bank. The maps, produced by photogrammetric methods, show the pit water and by interpolating the contours on the high wall and the spoil bank, a coal seam base was ascertained within 10 ft of its true elevation. Where two pits were mapped on the same slope, the approximate interval between the coal seams was determined and the seams were tentatively identified. This method was used to advantage in the valley of Stillwater Creek where slopes exceed 300 ft in elevation.

Coal pit elevations interpolated from the 1:24, 000-scale topographic maps were tentatively plotted on the profile wherever they were located within the proposed right-of-way corridor for the highway.

Photographs obtained from the U. S. Geological Survey, which had been used photogrammetrically to compile topographic maps in the 7½-min quadrangle series of 1:24, 000-scale, were used in conjunction with such maps. Slopes were examined stereoscopically for mine entries at the approximate elevation of the stripping. Pits located up to 3 mi left and right of the centerline of the proposed highway location aided in determining the strike on an areal basis. The term dip as used here is the descent of the coal seam along the proposed centerline and the strike is the descent at right angle to the centerline.

Coal seams and the average interval between them in the stratigraphic column (Fig. 1) as determined from the study were Waynesburg No. 11 to Uniontown No. 10, 50 ft; Uniontown No. 10 to Meigs Creek No. 9, 112 ft; Meigs Creek No. 9 to Fishpot, 24 ft; Fishpot to Redstone No. 8a, 35 ft; Redstone No. 8a to Pittsburgh No. 8, 31 ft; and a total interval from No. 9 to No. 8 of 90 ft.

Attention was next directed to the profile. The Testing Laboratory core information formed the basic framework for establishing the dip of the coal seams as the seams were precisely positioned. Tentative information from stratigraphic sections and pit elevations were subject to vertical revision when measured with the Kelsh plotter.

A crude but entirely satisfactory method of correlating the coal seam information on the profile was used. The profile was on an 18-ft long piece of paper laid out on a long table. A string was stretched from the lowest coal seam reported in a core at the west end of the profile to the lowest coal from the same source at the east end of the profile. The string represented the dip of the Pittsburgh coal along the proposed centerline between the exterior points. A significant anomaly was apparent at the west

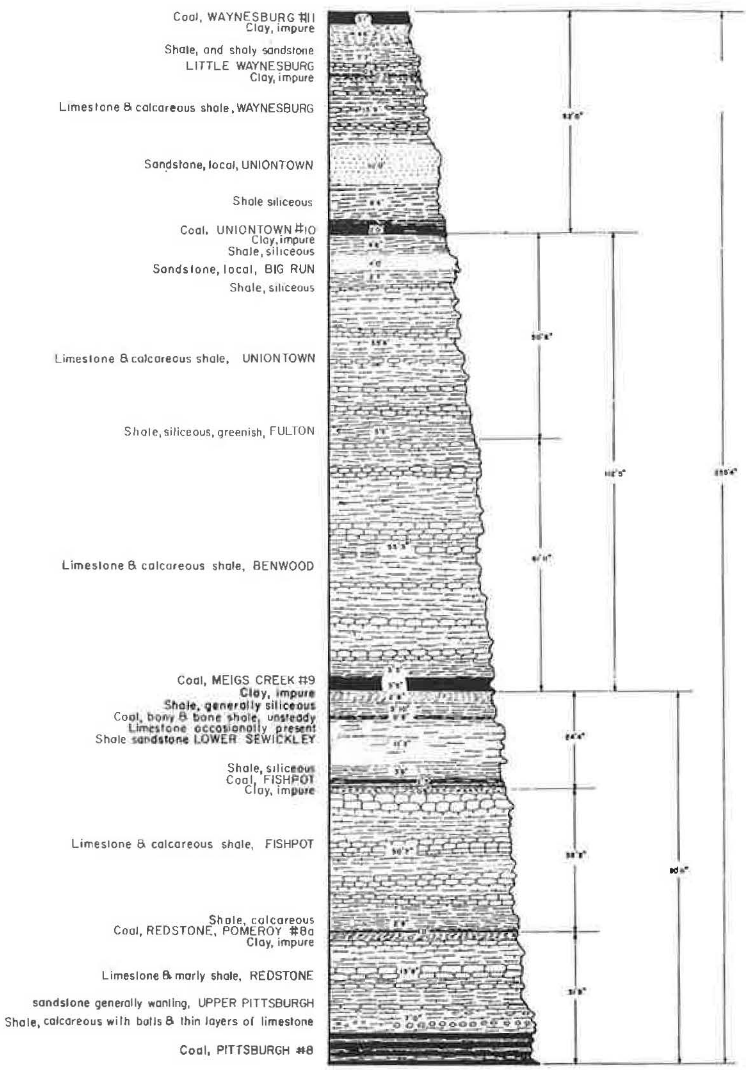

Figure 1. Stratigraphic column in Belmont County.

end of the profile. If the line representing the No. 8 coal was correct, the interval to the No. 9 was in excess of 120 ft. There was coal reported about 30 ft above the line and could have been the Redstone No. 8a. After adjusting the string to these coals, it was discovered that a small anticline existing at this point which rose about 30 ft, remained level for 4,000 ft, then resumed a normal dip. Along the remainder of the profile smaller anomalies were discerned. The string was adjusted to pass through the coal reported at the point where the anticline flank began to dip east. All evidence of coal seams which fell in the proximity of the string was labeled No. 8. The base of all these coal seams was connected to form the dip along the centerline, subject to change where Kelsh measurements were made on mine entries and pits, situated near the centerline.

The same procedure was repeated for the Meigs Creek coal seam. Knowledge of the general 90-ft interval was found reliable and correlation was simplified.

The Uniontown No. 10 and Waynesburg No. 11 coal seams were encountered in the eastern 1½ mi of this project near the proposed grade.

The Uniontown coal is erratic in thickness, content and vertical position. The line representing the Uniontown coal was an average taken from the four reported occurrences along the proposed centerline. A subsequent core recorded after map completion only served to substantiate the inconsistency of the Uniontown. On an areal basis, it is felt that the coal outcrop line mapped represented the best average within the mapping limits.

The Waynesburg No. 11 coal which outcrops in the hilltops at the east end of the area mapped was identified through the research, the core reports, and spot elevations from coal outcrop maps at a smaller scale.

PHOTO STUDY

On completion of this preliminary work, the aerial photography from which the Kelsh diapositives were produced was examined stereoscopically (Fig. 2). The manifestations of coal in the landforms were delineated and tentatively identified as to coal seam encountered. Evidence considered included strip mines, drift mine entries, tipples, and test pits.

The photo patterns of the strip mines with the unmistakable high wall, pit, and spoil make identification a simple task. Inherent in this simplicity are potential pitfalls. Not all strip mines are operated solely for coal recovery. Underlying most coal seams is a clay bed. In some operations, both the coal and the clay beneath are removed, resulting in a pit elevation well below the coal base.

At other locations, where only the clay is stripped, a visual comparison of the amount of spoil with that from coal stripping reveals a usable photo pattern. Clay stripping usually results in small quantities of spoil while the reverse is true for a coal operation.

Clay is also mined by drifting. Where this is done, there is no way of differentiating clay mine entries from coal mine entries with aerial photos. If kilns are seen during photo study, the existence of clay mining should be anticipated in the vicinity.

Tracing of old wagon roads and haul roads to their apparent termini made location of abandoned mines possible in some wooded areas. Easily located were those mine entries where piles of mine debris had been dumped on the slopes near the entry. They form an eroding anomaly on the slope which was generally bare of vegetation in contrast to the wooded area surrounding.

Test areas were bare soil areas where the soil had been removed along a slope seeking the coal seam.

MAPPING PROCEDURE

From the photo interpretation it was apparent that some stereoscopic models contained adequate coal base leveling control and others lacked it; consequently, bridging was required. The stereoscopic model with the best distribution of visual coal information was selected.

Figure 2. Stereo-pair showing surficial coal evidence; notations are 1, strip mines; 2, drift mines and debris; 3, test for coal; and 4, haul roads.

Using standard setup procedures for the Kelsh double projection instrument, the first model was scaled and leveled to the control used in producing the 1 in. = 200 ft topographic map.

The topographic map was aligned to the planimetric detail of the projected model and taped down. A semi-transparent Mylar sheet of the same length as the map was taped over it, permitting them to be moved as a unit when the model was releveled.

The proposed centerline, visible through the overlay, was plotted and stationed on the overlay. Subsequently, coal outcrop and overburden was mapped in relation to this proposed centerline.

At this time the model projected the terrain as leveled to the horizontal and vertical control used in making this segment of the topographic map. All visible pits, mine entries, and test pits were located in the model and plotted, and the elevations were recorded on the overlay.

The locations of cores containing coal, reported by the Ohio State Highway Testing Laboratory, were plotted on the overlay at the position indicated in the soil profile; for example, 80 ft right of Sta. 162+00. These locations were symbolized, and the "top of hole" and the "base of coal" elevations were recorded.

LEVELING TECHNIQUES—X DIRECTION

The next step was to level the model to conform to the Pittsburgh No. 8 coal using the information measured during the normal model projection and the dip indicated by the profile.

Leveling in the X direction (along the centerline) was accomplished in the following manner. Coal reported in the cores would not be visible for leveling, nor would coal base elevations gained from the profile. To permit the use of this underground information, several techniques were employed. For example, if at Sta. 107+00 the coal base was found at elevation 1102 and the terrain permitted, an identifiable object on a slope in the vicinity of that station at 1102 was plotted to represent one of the X direction leveling points.

Occasionally, a situation existed within a model where the terrain would not permit transfer of the coal elevations to hillside objects adjacent to the station. Sometimes this was due to a large landform occupying the area of the proposed centerline at one or both ends of the X direction. The leveling was done using the procedure shown in Figure 3. At Sta. 146+00 the ground elevation was 1020 and the coal base occurred at elevation 910. At Sta. 182+00 the ground elevation was 1,000 ft and the coal base occurred at elevation 895. The coal dipped 15 ft in the X direction. To level the coal

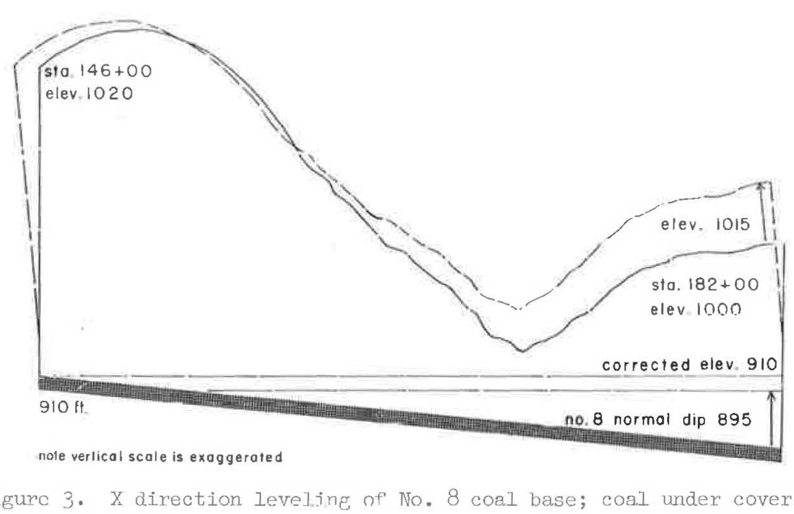

Figure 3. X direction leveling of No. 8 coal base; coal under cover.

base, the second point had to be raised 15 ft in the model. This was accomplished by indexing the floating mark at the ground elevation at Sta. 146+00 and raising the ground at Sta. 182+00 until it read 1015 on the counter. The coal base was then level in the X direction and was 110 ft below the index point at Sta. 146+00.

Variations of this technique were used where one or both X direction leveling positions were eroded below the coal base, but leveling in this direction was essential.

The model was next leveled in the Y direction using the visible coal manifestations previously listed (Fig. 4). After the usual small X and Y adjustments, the model was level to the best available coal information for that seam. The model projected a segment of terrain representing an area approximately 4,300 ft in the X direction and 7,200 ft in the Y direction.

The floating mark was indexed to the Pittsburgh No. 8 coal base and the tracing table was locked at this elevation. The No. 8 coal outcropped on all landforms where the floating mark encountered the slope. With the floating mark locked at this horizon, pass points (drop points) were located along the edges of the neat model (match lines). Small identifiable dots or a short line representing the coal base were plotted, to be used in the adjoining models. These were subject to adjustment when other models contained visual evidence indicating a change in the coal base elevation in the Y direction. Before mapping, these adjustments were prorated through the bridged models where visual Y direction information was lacking.

Actual mapping of the coal outcrop and overburden was performed if the model was considered leveled to sufficient control. With the floating mark locked at the coal base, the outcrop was plotted on the overlay and this pseudo-contour was labeled No. 8. The counter was then set so the floating mark encountered the landforms 10 ft above the coal outcrop line. This line represented the soil and oxidized coal within the landforms. Next a line 20 ft above the coal base was mapped, with this line representing 20 ft of coal, soil and rock overlying the coal base. In increments of 20 ft, these overburden lines were plotted to the top of the landform or to the next coal in the landform if one existed.

Mapping the Meigs Creek No. 9 coal base required releveling the model to the information available for this seam. Procedures followed were those cited for the No. 8 seam, but less visual information was found and the 90-ft interval was relied on extensively. Only slight modifications of the No. 8 setup were felt necessary due to the landforms being subjected to the same regional deformation.

The depositional variables which altered the interval between the No. 8 and No. 9 coals were virtually unknown except from the profile and direct measurements made with the Kelsh plotter on landforms containing evidence of both seams.

After all mapping was completed on the model, it was releveled to the vertical control used in topographic mapping, all pass points were measured, and their true elevations were recorded on the overlay. This completed the mapping for that model.

Succeeding models in the flight strip were worked when sufficient coal information occurred within the neat model. Where visible evidence of a coal seam elevation was

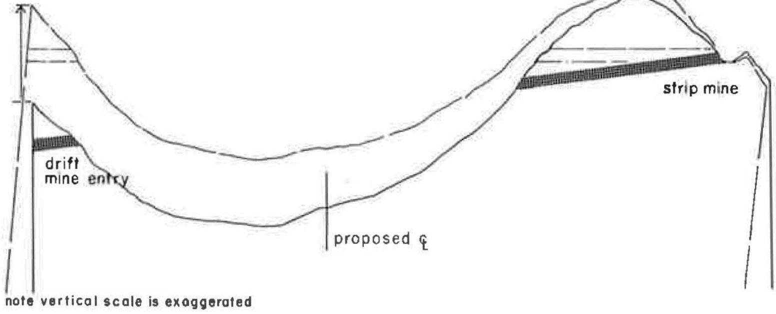

Figure 4. Y direction leveling of No. 8 coal base.

measured within the proximity of the centerline, a correction was made to the tentative dip originally established for the profile in that vicinity.

BRIDGING PROCEDURE—Y DIRECTION

Bridging across models lacking Y direction leveling information was accomplished as indicated in the following example. The figures used are not the true elevations for the No. 8 coal, and are used only for illustration. Visual coal measurements were plotted on the profile at their true position. For example: 2,300 ft left of Sta. 120+00, the No. 8 coal base was measured in a mine entry at elevation 1110. In another model, 2,100 ft left of Sta. 200+00, a pit measured 1078. The model lying between these stations lacked any indication of coal for leveling left of the centerline. A line was drawn on the profile connecting the coal measurements cited. The line intersected Sta. 160+00 at elevation 1094. A terrain point 2,200 ft left of Sta. 160+00 at elevation 1094 was located in the model and the model was leveled to this point (in the Y direction).

This example is a simplification of the procedure. Sometimes the bridging spanned two or more models. Landform conditions similar to those encountered in leveling in the X direction, namely, coal under deep cover or one end of the model occupied by a valley, also occurred in the Y direction. The methods described for leveling in the X direction were used in Y direction leveling where necessary.

The use of the procedures developed, where applicable, resulted in a model-by-model mapping operation. Each segment of the strip depicted the coal outcrops and overburden in relation to the proposed centerline as dictated by the evidence gathered from all the sources mentioned (Fig. 5).

At this time, construction is in the beginning stages. One cut in the vicinity of Sta. 53+00 uncovered weathered coal between the No. 9 outcrop line and the 10-ft overburden line.

An unexpected check on the dip of the No. 8 coal materialized when the Department of Highways Testing Laboratory secured a core 1½ mi east of the easternmost No. 8 coal used in producing the profile. The coal base dip was projected east along the profile to the core location without benefit of any adjustments. The projected No. 8 dip intersected the core 11 ft above the true base of the No. 8 coal. No other checks of

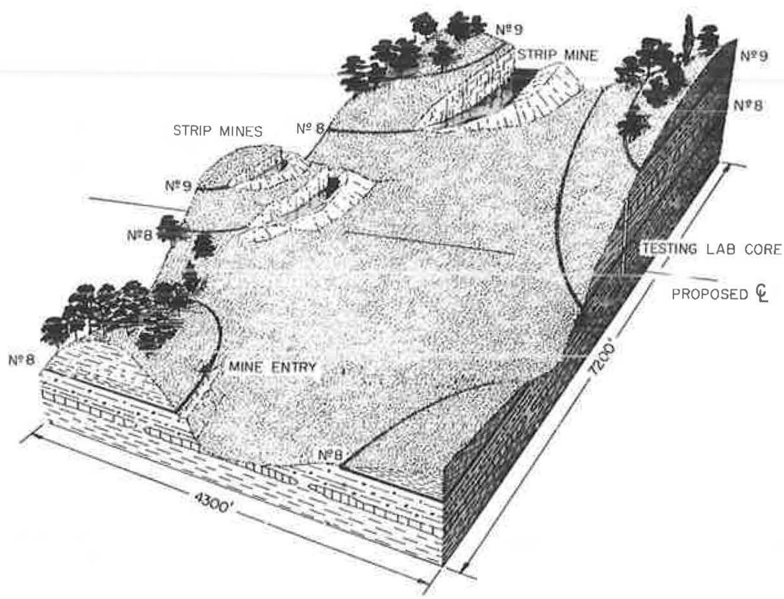

Figure 5. Segment of Kelsh coal base mapping illustrating ideal distribution of information; vertical scale greatly exaggerated.

outcrop accuracy have been made. During construction, when the coals are exposed in the cuts, the map will be systematically checked using photogrammetric methods.

NEW APPLICATIONS OF PROCEDURES

Original research frequently results in extensions of the original work. For example, after the dip of a coal seam is established, the stratigraphic sections reporting coal in the measured column can be positioned with the coal located at its true position. All other rock strata reported are properly placed vertically by this technique. Where two or more coal seams are reported, the technique serves to bracket the strata. With this information, predictions of rock types in proposed cuts is aided. The subsequent boring program can be planned with greater efficiency and economy by selective investigations based on the stratigraphic profile. Vertical delineation of stratigraphic rock types will also aid in slide prone slope analysis.

Consideration is being given to future coal outcrop mapping. On the preliminary survey maps at a scale of 1 in. = 200 ft, abandoned coal mine maps, as well as the location of the proposed centerline for the highway, will be superimposed. The coal mine maps are on file at the U.S. Bureau of Mines and are at various scales. All mine maps, however, can be brought photographically to the scale of 1 in. = 200 ft, and would then be of value where the highway grade line is at or near the elevation of a coal seam.

For highway design and construction plan preparation, topographic maps are compiled at a scale of 1 in. = 50 ft. Using the coal profile, the coal outcrop line can be delineated on these maps using the techniques to level the stereoscopic models in the X and Y directions employed for similar work at the 1 in. = 200 ft scale previously explained. Then within each property boundary and the right-of-way lines the amount of coal actually taken by the highway from mining possibility can be determined.

On completion of the coal seam and overburden mapping at the scale of 1 in. = 200 ft, an analysis was made of the advantages of this research and the results attained.

The coal seam mapping was found to be inexpensive. All necessary photography, mapping control, glass plate transparencies printed from the photography, and the base map were available. The supporting literature and core information regarding the coal seams and their overburden were procured very cheaply.

The topographic maps at the 1 in. = 200-ft scale covered a route band of topography approximately 1 mi wide and 8 mi long. Within the mapped area are leased tracts of land in which the strippable coal seams will be interrupted by the new highway, thereby affecting continuity of stripping operations. Large stripping machinery will be isolated on one or the other part of originally continuous tracts of land containing the coal seams. The possibility of litigation is anticipated.

Other products of this endeavor include: (a) coal seam and overburden outlined by contours on a 1 in. = 200-ft scale map of the highway route corridor approximately 1 mi wide and 8 mi long; (b) identity of coal seams affected; (c) extent of coal seams affected; (d) overburden measurements; (e) location of possible drift mine entries; and (f) location of strip mines.

The coal seam mapping techniques reported herein were developed by a geologist-photographic interpreter with extensive experience in operation of a Kelsh stereoscopic plotter. Although this combination is ideal, the same procedures can be used by people with one or more of these skills working with others who have complementing skills. To achieve the best coal outcrop line, numerous small decisions were made while the coal seam and overburden mapping was being done. Considerable time is saved if all skills required are possessed by the Kelsh instrument operator.

APPLICATIONS IN OTHER FIELDS

The techniques and procedures listed can be used in a regional coal study where long-range planning of availability is required to supply a facility.

Establishment of dip and strike on an areal basis will permit economical positioning of test borings. Leasing will be expedited.

Where coal mining has been conducted at proposed dam sites, procedures discussed can aid in locating mine entries, air shafts, and in geological investigation.

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Comparative Accuracies of Field and Photogrammetric Surveys

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Data resulting from the examination of surveys made by photogrammetric and by conventional field survey methods for highway design are compared. The greater part of the data presented compares the accuracy of elevations on one highway survey project for design of a section of Interstate highway approximately 7 mi long. The results of one evaluation indicate that the frequency and magnitude of differences between two field parties are approximately the same as those between either field party and the photogrammetric approach. Data from a less extensive examination of three other projects are also analyzed and compared.

Both field and photogrammetric methods can be used to make surveys and compile maps for highway design, as far as elevations are concerned. Either method may produce errors in elevation, and the field survey method is likely to cause the larger ones. However, in both cases, the errors are of little consequence during design and construction. Elevation measurements made by photogrammetric methods are sufficiently accurate for computing volumes of earthwork, both for design and for payment purposes.

Basic field survey control is necessary for horizontal measurements made by either conventional field or photogrammetric surveying methods. Aside from this, and to the extent horizontal measurements are shown by plotted positions, photogrammetric methods result in better local and overall horizontal accuracy. Both methods, however, are adequate for the purposes of highway design and construction.

•IT IS the purpose of this paper to present data resulting from the examination of surveys made by photogrammetric methods for the design of highways. Since the principal means of determining accuracy is by comparison with measurements accomplished by conventional field survey methods, pertinent data on these field surveys were also secured and are presented. Most of the data given concerns the accuracy of elevations on one extensive highway survey project. Data from less extensive examinations on three other projects and a very small amount of data on comparative accuracies of horizontal positions are given.

BACKGROUND

In the course of making surveys by photogrammetric methods, many queries are received as to the accuracies to be expected from such methods. Although practicing photogrammetric engineers obtain in the course of their work many indications of accuracy, these tend to be somewhat qualitative and are seldom in a form suitable for conclusive presentation. With one exception known to the authors, there is no con-

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clusive factual data published on the accuracy of photogrammetrically made measurements and field surveys as applied to large-scale topographic mapping for highway design purposes. The exception is a paper recently published by L. L. Funk after completion of the data given here. In the course of their work, the authors have collected the results of several tests and comparisons, and it was felt these would be of value to others and serve to answer the queries encountered concerning accuracy of surveys made by photogrammetric methods.

The bulk of the data given here was obtained from one Interstate highway survey project. As the project work moved on through design and into the construction stage, it became convenient to secure the results of the pre-construction field-measured profile and cross-sections from the supervising engineer, as well as the cross-sections he plotted using the photogrammetrically made measurements. None of the engineer's original work of plotting cross-sections, based on the photogrammetrically made surveys, was checked for horizontal positioning, interpolation of elevations from the contours, or plotting. This work provided a mass of data which could be used to evaluate the accuracy of the photogrammetric work for the original design. It then developed that the contractor on the job had also measured pre-construction cross-sections and was willing to make his data available for study. This then provided an opportunity to compare field surveys against one another as well as against surveys made by photogrammetric methods.

As the results of the comparative examination became available, the need for examining other projects was indicated. Without undertaking costly field work for this purpose, two other projects were on hand where some limited comparison of field survey and photogrammetrically made measurements could be made. These comparisons were made and studied along with the more extensive data already referred to; the resulting data are being presented herein. Data from a fourth project are also presented. These resulted from a vertical accuracy test performed under circumstances which necessitated only a minimum of field survey cost.

It should be noted that the data given are largely comparative and leave unanswered the question of absolute accuracies. It would have been desirable to settle that question in the case of the first project mentioned but by the time need for such a comparison became evident, construction was under way and the original ground was no longer available for re-survey. Cost considerations ruled out further field survey test work on two other projects. It is felt, however, that the data gathered are still of considerable value even though the absolute accuracy question still remains unanswered.

SCOPE

The first project to be examined was about 7 mi of highway route surveyed by photogrammetric methods for design of a section of the Interstate Highway System. The photography was taken during the fall flying season from a height of 1,500 ft. A K-17 camera equipped with a 6-in. wide-angle Metrogon lens was used. The maps were compiled using Kelsh stereoscopic plotters at a scale of 1 in. = 50 ft with contours at an interval of 2 ft. Except for one section of rough topography, approximately 2 mi long, the topography was generally rolling and the land was under cultivation or in pasture. Some scattered stands of light to medium hardwood cover were encountered. The rough area contained heavy stands of hardwood timber and medium stands of conifers which were particularly dense in the drainage ways.

On completion of the photography, examination indicated that the vegetative cover presented no problem to mapping by photogrammetric methods except in the rough area. It was obvious that field survey work would be required to complete the maps for the sizable areas scattered throughout the rough topography section where the ground was invisible. During the photogrammetric phases of the survey work, contours were measured and delineated in all but these areas. Copies of the photogrammetrically compiled manuscripts were then taken to the field to be used as plane table sheets; the contours were completed in the missing areas by plane table and alidade surveying methods.

Following design of the highway and as construction approached, the supervising engineer established the centerline on the ground, staking it while using normal survey procedures. He also measured cross-sections across the route using the normal on-the-ground survey procedures. His field notes of the cross-sections became the source for positions and elevations of the points used in making the comparison reported herein. Also, before any construction work on the highway, the contractor on the job had the original ground cross-sections independently measured by his engineering staff. The contractor did not establish an independent centerline but used as a base the centerline previously staked by the supervising engineer. The contractor used normal procedures in his cross-section measurement work. These cross-sections, as measured and plotted by the contractor, were the second source of data subsequently reported.

With a rather unlimited amount of data on hand, it became apparent that all the information available could not be analyzed and some selection would have to be made which would constitute a fair sampling. It was determined that it would be appropriate, as a first comparison, to examine the centerline profile from one end of the survey project to the other. It was realized this approach would tend to show the field work in a favorable light because both field parties had the advantage of measuring elevations at the same horizontal point, whereas at the time of map compilation, information was not available as to where the centerline would be established.

It was then determined that if the cross-sections were examined, the comparison of Party A to Party B would have no advantage of horizontal positioning except at the base line. It was impractical to examine each cross-section in its entirety for the entire project, so an arbitrary decision was made to examine, as a second comparison, a complete cross-section at each interval of 1,000 ft throughout the length of the survey project.

In the data collecting phases, it became apparent that one portion of the rough wooded territory, previously mentioned as requiring plane table survey, was producing gross differences in all possible comparisons. In view of the magnitude and frequency of the differences uncovered, it was decided to treat one part of this area separately. A segment some 1,900 ft long, containing about 1,100 ft of plane table work, was isolated and examined separately. The data examined in this area are not included in the two sets of data previously mentioned, but are considered in a third comparison. The area mentioned was not completely surveyed by plane table because the area requiring such work had a random outline and generally followed the gullies.

It was then decided as the fourth and final analysis of Project 1 to examine a segment of designed highway completely in cut or fill. An arbitrary segment length of 1,000 ft was selected as being a representative sample. Subsequent investigation of the construction plans revealed that in only one place was the condition satisfied and the area involved was entirely in cut.

FACTUAL DATA

The first examination of data was made of the centerline profile. Here the engineer established the centerline and staked it at a stationing stake interval of 50 ft. Elevations were extracted from his field survey records for the stations. The contractor's data were extracted and recorded on the same basis. The map elevations were taken from the cross-sections measured from the topographic maps and plotted on separate sheets by the engineer during the design phases of the highway engineering work. Comparisons were made between the contractor's and the engineer's data, between the map and the contractor's data, and between the map and the engineer's data. When the engineer's data were involved in comparison, it was held as a base and the deviations found indicate the departure of the other set of data from the engineer. When comparison was made between the contractor's data and map data, the contractor's data were held as the base. This basic procedure was followed for all projects examined. A portion of the plotted base line profile is shown in Figure 1 and the results of the comparison are given in Table 1. It is pertinent to note that in each of these comparisons there were 10 differences greater than 2 ft.

The second examination of data consisted in analysis of each elevation along each cross-section at even intervals of 1,000 ft for 7 mi. The engineer's cross-section point distances from centerline and elevation measurements were taken directly from

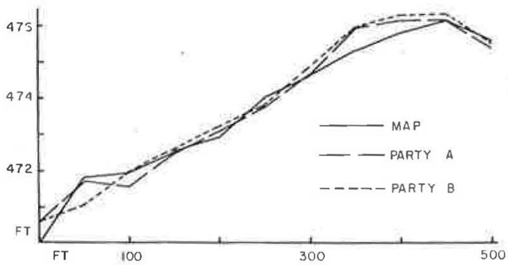

Figure 1. Typical segment of base line profile.

his field survey notes. The contractor's elevation measurements were taken from his plotted cross-sections, interpolating where necessary, using points the same distance from the centerline as the engineer's. The elevations for the map were taken from cross-sections measured from the map and plotted by the engineer, but again the engineer's field measured distance to each elevation point was considered to best portray the surface of the ground. It should be noted that the separate distances from base line to points selected by the engineer for elevation measurement on each cross-section are not necessarily the same points selected by the contractor or, as could normally be expected, selected based on map content alone. It is pertinent to note that in each comparison there were 13 elevations differing by more than 2 ft. The results of the comparison of the three sets of elevations are given in Table 1 and samples of plotted cross-sections are shown in Figures 2 and 3.

TABLE 1

COMPARISON OF ELEVATIONS, PROJECT 1

Data Source	No. Points	Arith. Mean of Diff.[a]	Avg. Diff.	Std. Dev.	Maximums
(a) Along Staked Centerline[b]					
Party A to B	532	+0.04	0.23	0.75	-70.0 to +10.6
Map to Party A	532	+0.05	0.59	0.93	-10.1 to +5.9
Map to Party B	532	+0.02	0.61	1.13	-10.1 to +10.6
(b) For Complete Cross-Sections, 1,000-Ft Intervals Throughout Project[b]					
Party A to B	227	+0.03	0.50	0.98	+3.9 to -7.5
Map to Party A	227	+0.10	0.67	1.07	-4.0 to +7.5
Map to Party B	227	+0.08	0.65	0.961	+4.3 to -4.1
(c) Along Every Cross-Section for 1,900 Ft of Centerline[c]					
Party A to B	465	0.0	1.1	2.2	-10.5 to +13.9
Map to Party A	465	-0.1	1.6	2.8	-16.8 to +15.3
Map to Party B	465	0.0	1.8	2.9	-14.4 to +11.3
(d) Along 1,000-Ft Segment of Highway in Cut					
Party A to B	171	-0.03	0.29	0.48	-2.0 to +1.6
Map to Party A	171	+0.2	0.4	0.53	±1.7
Map to Party B	171	+0.2	0.4	0.44	±1.2

[a] Disregarding sign.
[b] Plane table surveyed areas excluded.
[c] In rough wooded area, mapping partially by plane table method.

The third set of data compared consisted of elevation data along each highway segment for 1,900 ft of centerline in a rough wooded area. The mapping of approximately 1,100 ft of this area was accomplished by plane table surveying methods. The three sets of elevations were analyzed and comparison results are also given in Table 1. Figure 4 shows a plotted cross-section in this area. It should be noted that all gross map differences occurred in areas where plane table surveys were performed and differences encountered where surveys were made by photogrammetric methods fell in the same general range as encountered elsewhere in this paper.

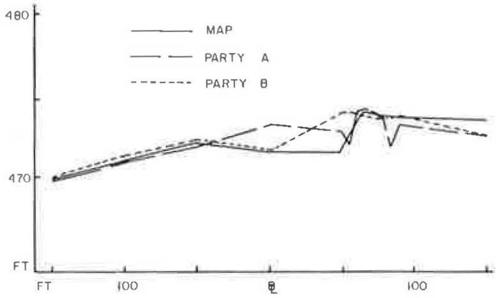

Figure 2. Typical section, average terrain.

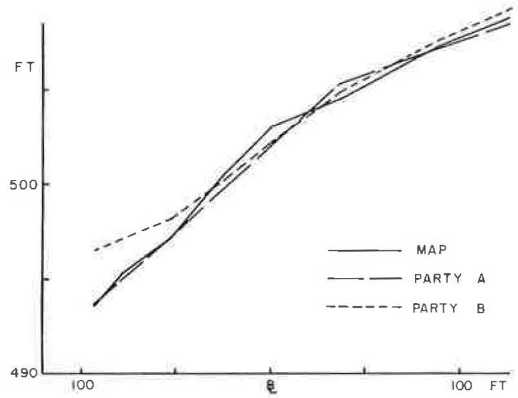

Figure 3. Typical section, average terrain.

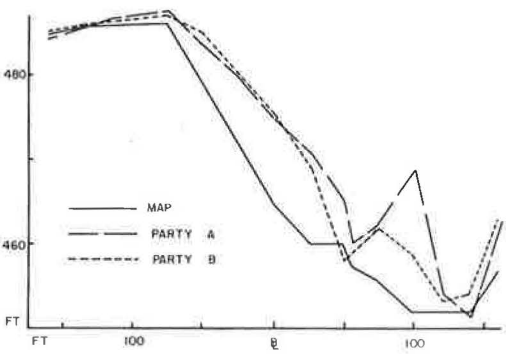

Figure 4. Typical section, rough wooded area.

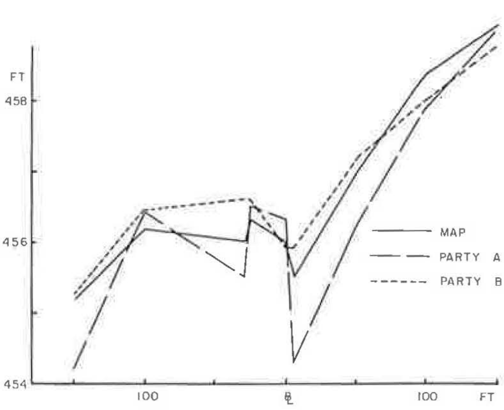

Figure 5. Typical section, cut area.

The fourth set of data examined on this project consisted of the 1,000 ft of highway designed to be completely in cut. Here again the engineer's data were considered to be datum and for elevation comparison purposes his distances from centerline to elevation measurement points were assumed to be the best representation of the ground. The engineer established the staked centerline as well as grade stakes 150 ft right and left of the centerline. The contractor had these stakes to provide horizontal positioning 150 ft left and right of the centerline as well as a measurement line along each cross-section. The map elevations were taken from cross-sections measured from the map and plotted by the engineer and appropriate elevations were taken from the contractor's plotted cross-sections. The results of the analysis are given in Table 1 and a typical cross-section is shown in Figure 5.

In addition to analyzing the data as in the previous three cases, earthwork volumes were also computed using the design templates for this section of highway. The engineer's and contractor's were both data taken directly from their field survey records. This is not the case in the previous studies where it is assumed that the contractor's distances from centerline to elevation measurement points on each cross-section were his best evaluation of the ground surface and should be used. Map elevations were taken from the cross-sections measured from the maps and plotted by the engineer. Volumetric data were computed electronically and the following values were found:

Based on engineer's data, 98,640 cu yd;
Based on contractor's data, 98,628 cu yd; and
Based on map data, 100,235 cu yd.

The difference between map data and field data was +1.62 percent. The average depth of cut was 20 ft, of which 2 to 4 ft were overburden and the remainder was rock. A typical cross-section is shown in Figure 5.

The second project to be examined was about 1,800 ft of a highway project where mapping by photogrammetric methods overlapped mapping by conventional field survey methods. The aerial photography was taken during the fall flying season from a height of 1,200 ft. A precisionized K-17 camera equipped with a 6-in. wide-angle Metrogon lens was used. The maps were compiled using Balplex stereoscopic plotters at a scale of 1 in. = 50 ft with a contour interval of 1 ft. The topography was generally rolling and the land consisted of cultivated fields or pasture. No additional field topographic information was required in the area of this study.

Field data were obtained from cross-sections plotted from the original survey. Ground elevations from the maps were obtained by measuring the profile of the original base line plotted on the maps compiled by photogrammetric methods. The cross-section lines were then erected perpendicular to the centerline and elevations were interpolated from contours on the maps at the same distance from the centerline as recorded for elevation measurement points in the field survey cross-section notes. The results of the study are given in Table 2.

In plotting the field measured cross-sections on the map, it was discovered that where the cross-sections were on curves and one cross-section line crossed the plotted position of another, field surveyed elevations on the different cross-sections fell very close to each other. Vertical differences in the neighborhood of 2 ft were observed in several places. The horizontal accuracy of this project is discussed in this paper.

The third project to be examined was 1½-mi segment of mapping on a 10-mi survey project. The photography was taken during the spring flying season from a height of 1,200 ft. A precisionized K-17 camera equipped with a 6-in. wide-angle Metrogon lens was used. The maps were compiled using Balplex stereoscopic plotters at a scale of 1 in. = 50 ft with a contour interval of 1 ft. The topography was generally flat and the land was under cultivation and pasture. Although some field survey elevation data were required to supplement photogrammetrically obtained data in several other areas, none was required here. For comparison purposes, the field survey data were taken from the field survey notes prepared by the organization which photogrammetrically compiled the maps. A second-order base line survey had been made through the project, and side base lines, established by second-order survey methods, were also established on several intersecting streets. Station marker monuments were established at intervals of approximately 500 ft along these base lines for which elevations were measured by third-order levels. Positions for profile measurements were established by stadia methods at intervals of about 50 ft along the base line. Using a Wild N-2 level, a series of unchecked side shots were made to establish the elevation of the profile points. The base line was plotted on the maps, and using field measured distances, elevations were interpolated from the contours of the maps. The results of the analysis are given in Table 2.

TABLE 2

COMPARISON OF ELEVATIONS, OTHER PROJECTS

Proj.	No. Points	Arith. Mean of Diff.	Avg. Diff.[a]	Std. Dev.	Maximums
2	136	+0.61	2.3	1.51	+6.4 to -9.4
3	145	-0.02	0.18	0.26	+0.5 to -1.2
4	28	-0.02[b]	0.29[b]	0.37	-0.9 to +0.5

[a]Disregarding sign.

[b]Here differences classed as errors because positive horizontal and vertical position assured.

The last project to be examined was a segment of an 11-mi section of mapping performed for design of the Interstate Highway System. The photography was taken during the spring flying season at a height of 1,500 ft. A precisionized K-17 camera equipped with a 6-in. wide-angle Metrogon lens was used. The maps were compiled using Kelsh stereoscopic plotters at a scale of 1 in. = 50 ft with a contour interval of 2 ft. The topography was generally rolling. Most of the mapped area was under cultivation or was in pasture. In several areas, light to medium hardwood timber covered the land. No additional field work was required to supplement the photogrammetric work in the wooded areas. On completion of the mapping, the centerline of the northbound land was staked on the ground. Using a Wild N2 Precise level, a closed line of levels was measured over two portions of the line, each about ½ mi in length. Stations on the centerline at intervals of 200 ft were turned through, these points were plotted back on the 1 in. = 50-ft maps, and elevations were interpolated for the appropriate stations. Approximately 40 percent of the points tested were in the woods. The analysis of the errors is given in Table 2. This is the only test reported here in which positive horizonal and vertical positions are assured.

DISCUSSION

It might be wise here to consider the causes of errors that affect topographic mapping by photogrammetric methods. L. L. Funk states that major errors in the photogrammetric system stem from either large systematic errors or blunders. He also states that random errors together with small systematic errors, which may be impossible to eliminate, determine the basic accuracy of the system. Inasmuch as the magnitude of the small systematic error is in the range of 0.20 ft or less, it is not felt that this type of error will noticably affect map accuracy. Large systematic errors together with small random errors, however, would have an effect on map accuracy. By maintaining tight control over the field and map compilation phases, these errors and blunders can be kept to a minimum. Random errors, some large systematic errors and blunders will, however, escape detection and appear in the completed map. As long as the frequency of this type of occurrence remains low, highway design will not be materially affected.

It is also appropriate to mention some of the advantages and disadvantages of the stereoscopic model. Vertically, the least elevation measurement that can be repeatedly read on the instrument is about 1/7,500 of the flight height. In other words, if a surveyed project was photographed from a flight height of 1,500 ft to prepare a map, the best possible measurement reading of a known elevation on the ground would be ±0.1 ft. To set up the stereoscopic model, at least three known elevations are required and four should be used in best practice. The stereoscopic model is leveled to these field-surveyed points to within ±0.1 ft; when this is done, the model is brought to scale by means of at least two known horizontal positions. An area about 2,000 by 900 ft is then prepared for mapping. The photogrammetric instrument operator moves his measuring mark through the stereoscopic model delineating planimetry and contours in separate phases. Inasmuch as the entire area is fixed horizontally and vertically, the instrument operator must measure and delineate something that he cannot see if he is to make a mistake. He can, however, still be guilty of an omission or carelessness. In actual practice every movement of the tracing table, while measuring and delineating contours, is measuring and connecting spot elevations. The process then takes an infinite number of spot elevations which the instrument operator connects together as he moves the measuring mark over the spatial model. At any one time he may be a little above or below the ground. This type of error can be expected to be within the range of less than one-half the contour interval. Vertical errors in excess of this amount are caused by the inability of the operator to see the ground or to blunders or carelessness.

In comparison to field survey methods then, we have combined the observing, measuring and plotting phase into one operation and have eliminated two possible sources of error.

It is also appropriate to examine the field survey procedures that would be used to gather similar data. In all probability, mapping by field survey methods stems from a base line which may or may not be closed on itself, let alone being tied to high order control surveys established by governmental agencies. This then is the first place that errors can occur in the field work, and although solar observations may tie down azimuth, there may be no check made on distance measurements. After base line staking at stations and ground line breaks, it is the usual practice to measure a base line profile and gather cross-section data. Considering only the vertical aspect for a moment, elevations along the base line and the cross-sections will be measured by use of a series of backsights and foresights. Each one of these separate elevation measurements is unchecked and subject to question. Although an occasional wild reading of the level rod will not materially affect highway design, it is certainly not desirable.

The horizontal positioning of all cross-section points for which elevation measurements are made and all other data to be plotted is open to suspicion. Although both field survey parties worked from a staked base or centerline and were supposedly measuring ground point data on the same cross-sections, differences of from -10.5 to +13.0 ft did occur in one of the tests. Although the measured elevations were unchecked single measurements, it is difficult to assume that the differences all stem from poor leveling. It is more reasonable to assume that different cross-sections were actually measured with the only common point being at the centerline. In one of the other sets of data presented, Party B had the advantage of having a centerline stake and a grade stake set 150 ft right and left of the centerline. In several instances, elevations measured for the same grade stakes varied as much as two ft between the work of two field survey parties. Obviously in this case the differences were caused by poor leveling procedures.

In comparing maps compiled by photogrammetric methods with those compiled from field survey data, it has been observed that horizontal positioning of lineal features at the base or centerline is usually good. It is very common to observe large differences appearing, however, as the distance from the centerline increases. Figure 6 is illustrative of this. Fence and building positions and size were measured from the maps prepared from field survey data and maps compiled by photogrammetric methods. Both of these were then overlaid and the discrepancies became obvious. Field investigation proved that the map compiled by photogrammetric methods was correct. At this point, it is impossible to determine whether or not the surveying instrument measurements were incorrectly read or recorded, whether a right deflection was recorded as a left deflection or whether the data were just plotted wrong. This type of horizontal error has been found with fences, houses, the centerline of intersecting roads, railroads and all other planimetric detail. The authors have never tied into adjoining maps prepared by field methods without finding at least one such discrepancy. It must be pointed out that while this gross type of horizontal error is not likely to be found in photogrammetric work, it is impossible by the photogrammetric approach to provide absolute dimensions of features. It is possible, however, to assume that good photogrammetric work in making measurements and compiling topographic maps of a built-up area would overlay identically a good map prepared using data obtained by field survey methods, for the drafting tolerances would be the controlling factor.

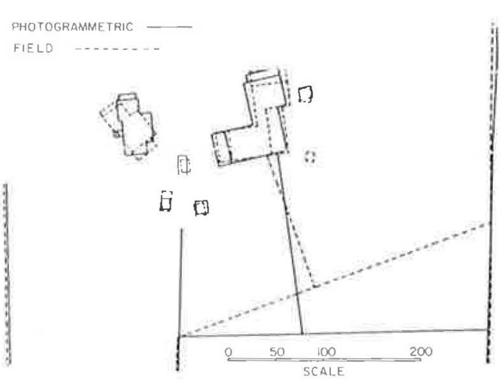

Figure 6. Comparison of horizontal positions.

It is vitally important to remember this horizontal weakness in evaluating all the vertical comparisons included in this paper. In only one case can it be said that a test has been applied to the mapping; the rest of the data is presented as comparisons only. While these comparisons tend to indicate someone is wrong, no proof has been presented to identify which one is wrong or how much each one is wrong.

An overall look at the first project examined indicates that the maximum differences in all cases, be it Party A to Party B or either party as compared to interpolated elevations from the maps, are large, but all maximums for any one examination are in the same range. The original data tend to indicate that gross type differences were made by all parties concerned. In fact, all comparisons for any single test are of the same general magnitude. The one place where the comparison of data indicated that the field parties were producing better results between themselves than either did with the maps was in the computation of the standard deviation. Even here, however, in two cases, a lower standard deviation was obtained by comparison of the map elevations with measured elevations of one field survey party than between elevations measured by the field survey parties themselves.

In the test where 1,000 ft of centerline was designed to be in cut, the arithmetic mean of the comparison of photogrammetrically compiled map data with either field survey party data indicated that the contours of the maps were 0.2 ft above datum. Subsequent volume determinations bear this out. The field survey parties had greater maximum differences between themselves, but their overall results appear to be more uniform.

A comparison of the volumes computed from cross-section data of the engineer and of the contractor indicates that for a planned excavation of some 99,000 cu yd, their cross-sections produced a volume difference of 13 yd and the maps indicated a difference from the measurements by either field survey party of 1,606 cu yd. This amounts to a difference of 1.63 percent, and both the engineer and the contractor felt that, in this case, such an amount was negligible. Had the cut been shallower, however, this percentage could have risen to a point of significance.

The real test of a map prepared for highway design and construction lies not in statistical data but in how well the job can be done. That the highway discussed has been designed and constructed and both the engineer and the contractor feel only the usual number of modifications were made in the construction phases prove the topographic mapping done by photogrammetric methods was as satisfactory as if the mapping had been done by conventional field survey methods.

The results of the second project examined tend to indicate that the datum of the topographic mapping done photogrammetrically was high by about 0.6 ft. It should be noted on this project that data have been presented which prove the cross-section measurements were in error and, in addition, proof has been presented to show errors also occurred in the horizontal measurements.

The data of the third project tend to indicate topographic mapping was very good, which was all that could be hoped for. The only explanation which can be offered for the small differences shown in this project, where the elevations are all of the unchecked nature, is that the field survey work was under control of the mapping organization and the work was carefully done. It is a little incongruous to make this statement when all previous statements made condemn loose field surveying techniques. Once again it should be emphasized that although the field survey data were gathered after the mapping was completed, it is considered a comparison and not a test.

The last project examined was a test and is positive proof the arbitrary line, as staked, produced errors of the indicated magnitude. The size of the sample is small, but the distribution of points is reasonable and the accuracy in all types of cover was tested.

CONCLUSION

Both field and photogrammetric methods can be used to make surveys and compile maps for highway design, in as far as elevations are concerned. Either method may produce large errors in elevation with the field survey method likely to cause the large ones; but in both cases, such errors are of little consequence during design and construction. Elevation measurements made by photogrammetric methods are sufficiently accurate for computing volumes of earthwork, both for design and payment purposes.

With respect to horizontal measurements, we must recognize that the basic field survey control is necessary in surveys made by either conventional field or photogrammetric methods. Aside from this, and to the extent horizontal measurements are shown by plotted positions, photogrammetric methods result in better local and overall horizontal accuracy. Both methods, however, are adequate for the purposes of highway design and construction.

Adjustment of Trilateration in Fundamental Figures

SHUH-CHAI LEE, Aerial Engineering Section, Ohio State Department of Highways

A fundamental figure is a quadrilateral or a central-point triangle which forms the basis of the problem of adjustment in trilateration. Any trilateration may be planned, analyzed and adjusted in fundamental figures. Area equations are used for the formation of the condition equations in the least squares adjustment of the sides of the triangles of fundamental figures. Evaluation of the coefficients of the error equation or correction equation is the first task in the adjustment and can be done systematically as shown. Cases for both geodetic and engineering trilaterations are presented.

•TRIANGULATION has been the conventional method used for the horizontal control of geodetic surveying and for some of the control of engineering surveys since Willebroad Snellins (1591-1626) used it in Holland in 1617. In such an operation, few bases and all of the angles are measured because long-distance measurement to the required accuracy is tedious, time consuming and difficult. The adjustment of triangulation is also complex and requires high technique.

Several years ago, the author tried to simplify the routine work of the adjustment of triangulation in fundamental figures for the convenience of engineers and recommended its use to geodesists (7, 8). Since then, the advancement of the electronic distance measuring instruments such as Shoran, Hiran, Geodimeter, tellurometer and Electrotape and the increasing interest in the adoption of trilateration have caused him to consider applying his ideas on this adjustment to the adjustment of trilateration.

Distance is the basic geometric element in the position science of surveying and geodesy (6). We can not determine horizontal positions by triangulation measurements without at least one known length, but we can determine positions by a trilateration scheme without any angular measurement. We have been using triangulation because we did not have a handy and reliable method of obtaining a great quantity of precise distances. Since the revolution of distance measurement by electronic instruments, trilateration has become increasingly significant. Field operations with electronic instruments have been cautiously carried on. The problem of adjustment has been along the line of traditional triangulation method.

Mechanically, angular measurement with the optical theodolite has its limitations. Even the electrooptical Geodimeter can substitute for the theodolite without difficulty. Other electronic distance measuring instruments are limited very little by weather and can measure long distances (6).

Current literature in the fields of geodesy and surveying generally contains two basic analytical approaches to the adjustment of trilateration: (a) indirect adjustment by calculation of the variation of plane or geodetic coordinates (2, 4, 13, 14, 17); and (b) conditional adjustment by conversion of the lengths of the trilateration into the angular condition equations (1, 11, 15, 16, 18, 19, 20). There are still many other graphical methods, analogue methods (5, 10) and methods for which the Laplace conditions are attached (3, 4), but they are variations of the basic analytical approach.

In this paper, the problem of adjustment of pure trilateration (no angular observations) is attacked by the basic analytical approach of conditional adjustment by area

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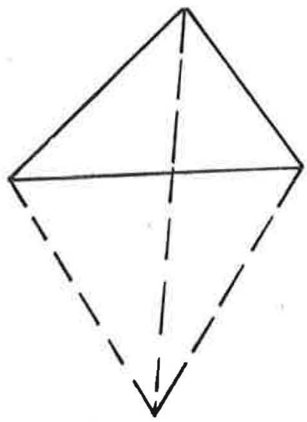

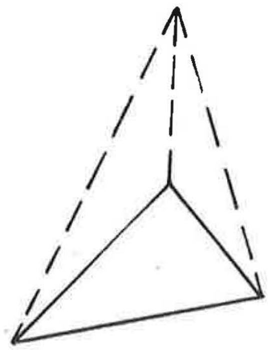

Figure 1. Formation of quadrilateral and central-point triangle.

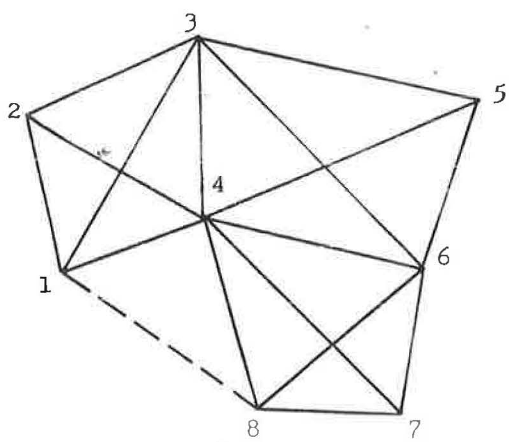

Figure 2. Formation of complicated trilateration.

equations of direct linear measurements. There are no angles, no coordinates and, hence, no trigonometric functions involved in the plane trilateration case in this approach. In the development of this method, both geodetic and engineering purposes are considered.

FUNDAMENTAL FIGURES

A simple triangle is the basic element of trilateration as it is in triangulation, but unlike triangulation, there is no redundant observation when the three sides are measured instead of three angles. When a new point is attached to a triangle to form a two-triangle trilateration, there is still no redundant observation unless the attachment is led to all three vertices to form a four-triangle overlapped quadrilateral or central-point triangle as shown in Figure 1.

The number of redundant observations is equal to the number of conditions in the problem of adjustment. According to the theory of adjustment, there can be no adjustment if there are no redundant observations. The quadrilateral or the central-point triangle, each having one geometrical condition, starts the problem of adjustment in trilateration. We shall call them the fundamental figures.

For one geometric condition, unlike the traditional method of forming equations by using angles in terms of triangle sides indirectly, there is only one way to form the area equation in terms of sides directly for a fundamental figure. Therefore, it is unique and consistent in adjustment and accuracy. This is another meaning of fundamental figures.

The fundamental figures are the fundamental units of more complicated trilateration in the geometric consideration of the formation of the figures and also, as we shall see later, in the mathemetical handling of a large number of equations. As shown in Figure 2, 1 to 8 is a waste measurement unless we make another measurement from 1 to 7 or 8 to 2 to form one more fundamental figure, or from 1 to 6 or 8 to 3 to form two more fundamental figures, in addition to the original three fundamental figures.

Of course, we can measure from 1 to 6, 1 to 7, 8 to 2 and 8 to 3 to form six more fundamental figures. If 1 and 8 are known fixed points, the original three fundamental figures are sufficient for the adjustment of the observed sides to fix the six unknown points.

The use of fundamental figures as an index to identify the number of conditions involved and to study the accuracy of the figures of the more complicated geodetic trilateration, with or without restraints and with or without Laplace orientation, can be developed further. In this introductory paper, however, only the theory of using area equations of fundamental figures and the possible application of these equations to the systematic adjustment of geodetic and engineering trilaterations are presented.

AREA CONDITION EQUATIONS

The area equations to be used in this new approach of adjustment of trilateration are, for plane triangles,

$$f_p = \sqrt{s(s - \ell_1)(s - \ell_2)(s - \ell_3)} \tag{1a}$$

or as given earlier (9)

$$f_p = \sqrt{[(\ell_2 + \ell_3)^2 - \ell_1^2][\ell_1^2 - (\ell_2 - \ell_3)^2]}/4 \tag{1b}$$

and for spherical triangles, according to Lhuilier's formula,

$$\begin{aligned} f_S &= R^2E \\ &= 4R^2 \text{ arc tan } \sqrt{\tan \tfrac{1}{2}\left(\frac{s}{R}\right) \tan \tfrac{1}{2}\left(\frac{s - \ell_1}{R}\right) \tan \tfrac{1}{2}\left(\frac{s - \ell_2}{R}\right) \tan \tfrac{1}{2}\left(\frac{s - \ell_3}{R}\right)} \\ &= 4R^2 \text{ arc tan } c \end{aligned} \tag{2}$$

where

ℓ_1, ℓ_2, ℓ_3 = sides in straight-line distances for plane triangles or in reduced spherical distances for spherical triangles,

$s = \tfrac{1}{2}(\ell_1 + \ell_2 + \ell_3)$;

R = mean radius of earth's sphere = 3, 959 mi = 6, 371 km;

E = spherical excess; and

$$c = \sqrt{\tan \tfrac{1}{2}\left(\frac{s}{R}\right) \tan \tfrac{1}{2}\left(\frac{s - \ell_1}{R}\right) \tan \tfrac{1}{2}\left(\frac{s - \ell_2}{R}\right) \tan \tfrac{1}{2}\left(\frac{s - \ell_3}{R}\right)}. \tag{3}$$

For the fundamental figures, the conditional equations in terms of areas as shown in Figures 3a and 3b are for a quadrilateral

$$\omega_Q = f_I - f_{II} + f_{III} - f_{IV} = 0 \tag{4}$$

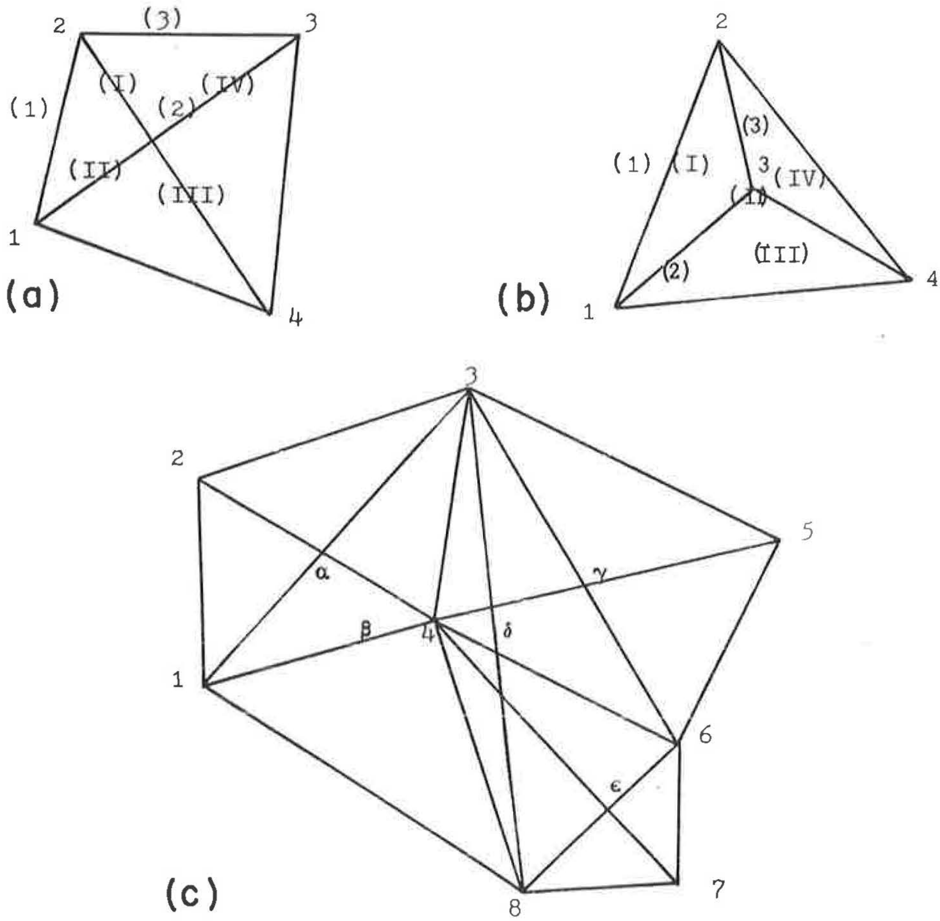

Figure 3. Area conditions.

and for a central-point triangle,

$$\omega_T = f_I + f_{II} - f_{III} - f_{IV} = 0 \tag{5}$$

For a more complicated figure of trilateration, the condition matrix of areas can be easily written according to a number of r fundamental figures (e.g., Fig. 3c, with $r = \epsilon$)

$$\begin{vmatrix} \omega_\alpha \\ \omega_\beta \\ \omega_\gamma \\ \omega_\delta \\ \omega_\epsilon \end{vmatrix} = \begin{vmatrix} f_{\alpha\text{-I}} - f_{\alpha\text{-II}} + f_{\alpha\text{-III}} - f_{\alpha\text{-IV}} \\ -f_{\beta\text{-I}} + f_{\beta\text{-II}} - f_{\beta\text{-III}} - f_{\beta\text{-IV}} \\ f_{\gamma\text{-I}} - f_{\gamma\text{-II}} + f_{\gamma\text{-III}} - f_{\gamma\text{-IV}} \\ f_{\delta\text{-I}} - f_{\delta\text{-II}} + f_{\delta\text{-III}} - f_{\delta\text{-IV}} \\ f_{\epsilon\text{-I}} - f_{\epsilon\text{-II}} + f_{\epsilon\text{-III}} - f_{\epsilon\text{-IV}} \end{vmatrix} = W_o = 0 \tag{6}$$

where

$f_{\alpha\text{-III}} = f_{\beta\text{-I}}$,

$f_{\beta\text{-IV}} = f_{\delta\text{-II}}$,

$f_{\gamma\text{-I}} = f_{\delta\text{-I}}$, and

$f_{\delta\text{-IV}} = f_{\epsilon\text{-II}}$ (see Table 2).

THEORY OF ADJUSTMENT

The relations of the one-column matrices of the observed lengths of the side ℓ's, their error e's or correction v's and the most probable values ℓ_o's of the sides are

$$L - L_o = E = -V$$

and

$$L + V = L_o \tag{7}$$

The condition matrix W_o is a function of area and in turn a function of length. Through expansion by Taylor's theorem and omission of the terms of and over second order, the condition matrix W_o in terms of a number of n observed lengths and their corrections becomes:

$$\begin{aligned} W_o(\ell_o) &= W_o(\ell + v) \\ &= W(\ell) + \frac{\partial W}{\partial L} V \\ &= W + B'V = 0 \end{aligned} \tag{8}$$

where B' is a transposed matrix of B,

$$\underset{r\times 1}{W} = \begin{vmatrix} \omega_\alpha \\ \omega_\beta \\ \cdot \\ \cdot \\ \cdot \\ \omega_\rho \end{vmatrix} \tag{9}$$

$$\underset{n\times 1}{V} = \begin{vmatrix} V_1 \\ V_2 \\ \cdot \\ \cdot \\ \cdot \\ V_n \end{vmatrix} \tag{10}$$

$$\underset{r\times n}{B'} = \frac{\partial W}{\partial L} = \begin{vmatrix} \frac{\partial \omega_\alpha}{\partial \ell_1} & \frac{\partial \omega_\alpha}{\partial \ell_2} & \cdots & \frac{\partial \omega_\alpha}{\partial \ell_n} \\ \frac{\partial \omega_\beta}{\partial \ell_1} & \frac{\partial \omega_\beta}{\partial \ell_2} & \cdots & \frac{\partial \omega_\beta}{\partial \ell_n} \\ \cdot & \cdot & \cdots & \cdot \\ \cdot & \cdot & \cdots & \cdot \\ \cdot & \cdot & \cdots & \cdot \\ \frac{\partial \omega_\rho}{\partial \ell_1} & \frac{\partial \rho}{\partial \ell_2} & \cdots & \frac{\partial \omega_\rho}{\partial \ell_n} \end{vmatrix} = \begin{vmatrix} \alpha_1 & \alpha_2 & \cdots & \alpha_n \\ \beta_1 & \beta_2 & \cdots & \beta_n \\ \cdot & \cdot & \cdots & \cdot \\ \cdot & \cdot & \cdots & \cdot \\ \cdot & \cdot & \cdots & \cdot \\ \rho_1 & \rho_2 & \cdots & \rho_n \end{vmatrix} \tag{11}$$

and there are only six non-zero α's, β's, ... , ρ's in each row for the six sides of each fundamental figure. By the method of least squares using Causs' correlate k's and differentiating the conditional minimum,

$$\Phi = V'PV - 2K'(W + B'V) = \min.$$

$$\frac{\partial \Phi}{\partial V} = V'P - K'B' = 0$$

we obtain

$$V = \bar{P}BK \tag{12}$$

where $\bar{P}$ is the inverse matrix of weight. By substituting into Eq. 8, we obtain the normal equation matrix

$$B'\bar{P}BK = MK = -W \tag{13}$$

and then

$$K = -\bar{M}W \tag{14}$$

Knowing $\bar{P}$ and B, we can compute

$$M = B'\bar{P}B \tag{15}$$

By solving for K, V can be evaluated.

EVALUATION OF B MATRIX

In computing M, the matrix B has to be evaluated if $\bar{P}$ has been already assigned or assumed to be a unit matrix, as is justified in the case of trilateration where the three sides of each triangle are measured with equal accuracy.

As stated in Eq. 10, the elements of B are derivatives of ω's, the condition equations of areas, with respect to the sides of each fundamental figure. For the ith side of the αth fundamental figures, e.g., a quadrilateral as in Figure 3a, the differential coefficient is:

$$\alpha_i = \frac{\partial \omega_\alpha}{\partial \ell_i} = \frac{\partial f_I}{\partial \ell_i} - \frac{\partial f_{II}}{\partial \ell_i} + \frac{\partial f_{III}}{\partial \ell_i} - \frac{\partial f_{IV}}{\partial \ell_i} \tag{16}$$

and, in turn, this coefficient is reduced to the problem of evaluating the derivative of the area of the Ith triangle with sides 1, 2 and 3 with respect to the ith side, e.g., side 1. This has been derived from Eqs. 1a and 2 as:

$$\frac{\partial f_I}{\partial \ell_i} = \frac{(s_I - \ell_i)(s_I - \ell_2)(s_I - \ell_3) + [(s_I - \ell_i) - (s_I - \ell_2) + (s_I - \ell_i)(s_I - \ell_3) - (s_I - \ell_2)(s - \ell_3)]s_I}{4f_I} \tag{17a}$$

or in a new simplified form for a plane triangle (9):

$$\frac{\partial f_I}{\partial \ell_i} = \ell_i\,(\ell_2^2 + \ell_3^2 - \ell_i^2)/8f_I \tag{17b}$$

and for a spherical triangle:

$$\begin{aligned} \frac{\partial f_I}{\partial \ell_i} = \frac{R^2}{2(1+c_I^2)c_I} \Bigg\{ & \tan\tfrac{1}{2}\left(\frac{s_I-\ell_i}{R}\right) \tan\tfrac{1}{2}\left(\frac{s_I-\ell_2}{R}\right) \tan\tfrac{1}{2}\left(\frac{s_I-\ell_3}{R}\right) \left[1+\tan^2\tfrac{1}{2}\left(\frac{s_I}{R}\right)\right] + \\ & \tan\tfrac{1}{2}\left(\frac{s_I-\ell_i}{R}\right) \tan\tfrac{1}{2}\left(\frac{s_i-\ell_2}{R}\right) \left[1+\tan^2\tfrac{1}{2}\left(\frac{s_I-\ell_3}{R}\right)\right] \tan\tfrac{1}{2}\left(\frac{s_I}{R}\right) + \\ & \tan\tfrac{1}{2}\left(\frac{s_I-\ell_i}{R}\right) \left[1+\tan^2\tfrac{1}{2}\left(\frac{s_I-\ell_2}{R}\right)\right] \tan\tfrac{1}{2}\left(\frac{s_I-\ell_3}{R}\right) \tan\tfrac{1}{2}\left(\frac{s_I}{R}\right) - \\ & \left[1+\tan^2\tfrac{1}{2}\left(\frac{s_I-\ell_i}{R}\right)\right] \tan\tfrac{1}{2}\left(\frac{s_I-\ell_2}{R}\right) \tan\tfrac{1}{2}\left(\frac{s_I-\ell_3}{R}\right) \tan\tfrac{1}{2}\left(\frac{s_I}{R}\right) \Bigg\} \end{aligned} \tag{18}$$

If spherical excess E's instead of area f's are used in forming the condition equation ω for spherical triangles, computations will be saved for the factor R^2 in all related equations.

By deduction, any $\partial\omega/\partial\ell$ can be written or computed from equations similar to the forms of Eqs. 16, 17a and 18 for any side of the triangle in a fundamental figure. Thus, the B matrix can be formed.

EXAMPLE

The method discussed in the last sections can be accomplished systematically either by a desk calculator or by an electronic digital computer. An example for the analysis of a trilateration net based on Figure 3c is given in Tables 1 through 4. The numbering system of the points, the sides, the triangles and the fundamental figures or the condition equations, which are being tested in a computer program, is self-explanatory in the tables. The results of the adjustment of the trilateration net of Figure 3c according to the basic principle presented in this paper are also shown. The detailed sample computation of the adjustment of a plane quadrilateral has been given earlier (9).

TABLE I

SIDES OF TRILATERATION NET

Observed Side	Side No.	Distance (ft)	Figure-Triangle-Side No.
1-2	1	6,973.270	α-I-1 = α-II-1
1-3	2	10,201.010	α-I-2 = α-III-1 = β-I-1 = β-II-1
1-4	3	4,474.850	α-II-2 = α-III-2 = β-I-2 = β-III-1
1-8	4	7,061.170	β-II-2 = β-III-2
3-2	5	9,418.530	α-I-3 = α-IV-1
4-2	6	8,376.250	α-II-3 = α-IV-2
3-4	7	6,599.990	α-III-3 = α-IV-3 = β-I-3 = β-IV-1 = γ-I-1 = γ-II-1 = δ-I-1 = δ-II-1
3-5	8	9,101.230	γ-I-2 = γ-III-1
3-6	9	11,054.280	γ-II-2 = γ-III-2 = δ-I-2 = δ-III-1
3-8	10	12,443.930	β-II-3 = β-IV-2 = δ-II-2 = δ-III-2
4-5	11	10,972.020	γ-I-3 = γ-IV-1
4-6	12	8,540.750	γ-II-3 = γ-IV-2 = δ-I-3 = δ-IV-1 = ε-I-1 = ε-II-1
4-7	13	10,142.850	ε-I-2 = ε-III-1
4-8	14	6,362.350	β-III-3 = β-IV-3 = δ-II-3 = δ-IV-2 = ε-II-2 = ε-III-2
6-5	15	6,601.030	γ-III-3 = γ-IV-3
6-7	16	5,288.990	ε-I-3 = ε-IV-1
6-8	17	7,103.720	δ-III-3 = δ-IV-3 = ε-II-3 = ε-IV-2
7-8	18	4,970.630	ε-III-3 = ε-IV-3

TABLE 2

TRIANGLES OF A TRILATERATION NET

Triangle Point No.	Triangle No.	Area (sq ft)	Figure-Triangle No.	Side Used (No.)
1,2,3	1	31,767,883.8270	α-I	1, 2, 5
1,2,4	2	15,597,596.3250	α-II	1, 3, 6
1,3,4	3	10,754,874.0270	α-III = β-I	2, 3, 7
1,3,8	4	36,014,781.3490	β-II	2, 4, 10
1,4,8	5	13,984,375.4720	β-III	3, 4, 14
2,3,4	6	26,924,985.4690	α-IV	5, 6, 7
3,4,5	7	29,996,444.6060	γ-I	7, 8, 11
3,4,6	8	28,148,470.0840	γ-II = δ-I	7, 9, 12
3,4,8	9	11,288,330.9630	β-IV = δ-II	7, 10, 14
3,5,6	10	30,020,311.5120	γ-III	8, 9, 15
3,6,8	11	39,010,020.0090	δ-III	9, 10, 17
4,5,6	12	28,172,289.2010	γ-IV	11, 12, 15
4,6,7	13	22,580,657.0760	ε-I	12, 13, 16
4,6,8	14	22,145,704.9730	δ-IV = ε-II	12, 14, 17
4,7,8	15	12,698,281.8090	ε-III	13, 14, 18
6,7,8	16	13,133,108.2150	ε-IV	16, 17, 18

TABLE 3

INFORMATION ON FIGURES OF TRILATERATION NET

Figure Point No.	Figure No.	Triangle Used (No.)	Side Used (No.)	ω
1,2,3,4	α	1, 2, 3, 6	1, 2, 3, 5, 6, 7	-176.0600
1,3,4,8	β	3, 4, 5, 9	2, 3, 4, 7, 10, 14	12,689.1130
3,4,5,6	γ	7, 8, 10, 12	7, 8, 9, 11, 12, 15	53.7890
3,4,6,8	δ	8, 9, 11, 14	7, 9, 10, 12, 14, 17	4,059.9150
4,6,7,8	ε	13, 14, 15, 16	12, 13, 14, 16, 17, 18	-125.6970

TABLE 4

ADJUSTMENT OF A TRILATERATION NET

Observed Side	α	β	γ	δ	ε	Correction v
1-2	-1,632.533	-	-	-	-	0.197
1-3	3,463.391	-8,362.605	-	-	-	0.021
1-4	-3,094.385	9,448.741	-	-	-	-0.122
1-8	-	-4,451.629	-	-	-	0.234
2-3	-1,277.335	-	-	-	-	0.154
2-4	2,313.738	-	-	-	-	-0.279
3-4	-2,643.833	17,269.190	51.578	6,649.493	-	-0.337
3-5	-	-	66.738	-	-	-0.039
3-6	-	-	-85.728	3,223.571	-	0.185
3-8	-	-9,717.541	-	-9,047.636	-	0.129
4-5	-	-	-86.388	-	-	0.050
4-6	-	-	71.117	-4,387.762	-1,870.126	-0.169
4-7				-	3,873.248	-0.118
4-8	-	12,799.095	-	8,148.742	-2,477.302	-0.253
5-6	-	-	51.571	-	-	-0.030
6-7	-	-	-	-	-1,952.866	0.059
6-8	-	-	-	2,633.135	2,674.465	0.029
7-8	-	-	-	-	-3,263.620	0.099
ω	-176.0600	12,689.1130	53.7890	4,059.9150	-125.6970	-

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