Extending Control Surveys by Photogrammetry

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The U.S. Forest Service is interested in improving its methods and techniques of extending (bridging) horizontal and vertical control surveys by use of photogrammetry. The purpose of this paper is to report the results of a series of tests on production projects, carried on in cooperation with the Virginia Department of Highways. These tests were divided into two phases: (a) evaluating the extension of control by analog and analytic bridging and (b) expanding topographic mapping control to highway design photography by photogrammetry.

Twenty-five aerial photographs, at a scale of 350 ft/in., containing 77 horizontal and 210 vertical control points, were used in the first test. This material was bridged using measurements made with the Zeiss Stereoplanigraph, model C8, and the Mann Monoscopic Comparator, and each bridge was computed with identical varying amounts of control. Two scales of photography were used in the second test: 2,000 ft/in. (used for standard topographic mapping on a quadrangle basis) and 500 ft/in. (used for highway design mapping). Common image points were selected between the two scales of photography, and the small-scale photography was bridged, thereby establishing X, Y, Z coordinates for the common image points. The design photography was then bridged, using the common image points as control.

The standard deviation for the analytical bridging, with control every sixth model, was 0.59 ft horizontally and 0.47 ft vertically. The analytic method showed that errors are reduced about one-third horizontally and one-fourth vertically, as compared to the analog method.

Design mapping can be accomplished, using horizontal control established for the small-scale topographic mapping, to an accuracy of 1:4,700, but a datum shift can be expected.

•THE U. S. Forest Service is interested in improving its methods and techniques of extending (bridging) horizontal and vertical control surveys by use of photogrammetry. It is required to make engineering surveys throughout rugged topography and during adverse weather conditions; therefore, manpower must be used wisely to keep ahead of the ever-increasing demands on the engineers. For example, if photogrammetrically determined coordinates are used for targets along a preliminary route location, the engineer checks the "L" line each time he "ties" to a target, thereby saving "double measurement" of the line to ascertain its survey accuracy.

Photogrammetry has developed to the stage where coordinates determined for points by aerial triangulation can be used in lieu of coordinates for the same points measured by field control surveys. Too often errors in field control surveys are made because the photogrammetrist requests control where the topography or ground cover is not compatible with field methods or conditions. Together with good bridge planning, the photogrammetrist will request control in areas where it can be established accurately and

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identified correctly. Aerial triangulation can be used to establish control where the classic field surveys are not practical.

When the two types of surveys (field and photogrammetric) are planned simultaneously, the field survey will take full advantage of the terrain and consequently be a better control survey; and the flight plan will enhance the photogrammetric survey. The field survey should be in two parts (horizontal and vertical) and should be planned separately. The photogrammetric survey will wed the two together. Most of the time, the two field surveys will require different types of terrain for efficiency and accuracy.

The Forest Service is continually testing bridging equipment and methods to obtain guidelines for planning future projects. The purpose of this paper is to report the results of two production project tests. The first was to compare analog and analytic bridging, with varying control spacing, and the second was to expand topographic mapping control to design photography, by use of photogrammetry. The first test was divided into two parts:

1. Analog Plotter—The Zeiss Stereoplanigraph, model C8, with Ecomat (automatic readout device) was used to obtain bridge coordinates. This method was used to obtain two complete sets of measurements. Contact printed photographic transparencies (photographic images printed on glass) were used for the first set. The same diapositives were used for the second bridge, but the photographic image control points were drilled with a Wild PUG (stereoscopic point marking and transfer device).
2. Comparator (Analytic)—The Mann Monoscopic Comparator was used to measure X and Y coordinates on the diapositives of the PUG marked photographic image control points.

The tests were made on a production project using production methods. The Virginia Department of Highways furnished test material (camera report, flash plate, photographs, glass diapositives, and horizontal and vertical control) for a portion of Interstate 81 near Christianburg. This material consisted of 25 photographs (24 stereoscopic models) at a scale of 350 ft/in. (1:4,200) taken with the Wild, RC8, 6-in. focal length, aerial camera, using a shutter speed of $^1/_{300}$ 0.03-sec. Aircraft speed was 168 ft/sec, or 0.62 ft of forward movement during exposure. The strip bridged by use of the aerial photographs was 29,764 ft long, along a bearing of N66° 30′E. The 77 horizontal control points (spaced about 400 ft apart) were identified by targets; about 40 percent of the 210 vertical control points were targeted, and the remaining 60 percent were identified by natural images. Photographic exposures were printed through the film base on photographic glass plates which had a thickness of $^1/_4$ in.

This material was used by two instrument operators using the Stereoplanigraph to make the measurements for computing each separate photogrammetric bridge:

1. Operator A measured each point once for bridging from west to east.
2. Operator A measured each point once for bridging from east to west.
3. Operator A measured each point four times for bridging from west to east.
4. Operator B measured each point once for bridging from west to east.

Each of these four bridges was computed five times by varying amount of control as follows:

Adjustment A—Control on every stereoscopic model; i.e., 25 horizontal and 50 vertical control points were used to compute the bridge.
Adjustment B—Control spaced every second model; i.e., 13 horizontal and 26 vertical control points were used to compute the bridge.
Adjustment C—Control spaced every third model; i.e., 9 horizontal and 20 vertical control points were used to compute the bridge.
Adjustment D—Control spaced every fourth model; i.e., 7 horizontal and 16 vertical control points were used to compute the bridge.
Adjustment E—Control spaced every sixth model; i.e., 5 horizontal and 12 vertical control points were used to compute the bridge.

TABLE 1

STANDARD DEVIATION IN FEET

Bridge Number	Adjustment	All Control Points	
		Horizontal (77 points)	Vertical (210 points)
1	A	0.74	0.54
	B	0.78	0.55
	C	0.78	0.56
	D	0.79	0.60
	E	0.87	0.62
2	A	0.74	0.64
	B	0.75	0.65
	C	0.77	0.66
	D	0.85	0.66
	E	0.92	0.67
3	A	0.88	0.88
	B	0.91	0.90
	C	0.95	0.91
	D	0.97	0.95
	E	1.15	1.03
4	A	0.87	0.71
	B	0.91	0.71
	C	0.95	0.71
	D	0.99	0.76
	E	1.11	0.73

Table 1 summarizes results by standard deviation in feet. The results are similar except for Number 3, which is somewhat ou of line. Ordinarily, measuring each point four times would produce better results. These results (1) are approximately the sam as obtained on the Interstate 66 test.

The Forest Service continued the test by negotiating a contract with a private compan to do the bridging, using the same material. The contractor was furnished a set of photographs and all points were identified and labeled. The contractor used the Wild PUG to drill an 80-μ diameter hole (0.003 in.) for each photographic point on the diapositives. Point coordinates of the drilled holes were measured with the Mann Monoscopic Comparator. The aerial analytic triangulation com putations were based on the U. S. Coast and Geodetic Survey's equations. The bridges were computed using the same varying amoun of control as used in the analog instrument bridging.

After the contractor furnished the results of the five bridges, the Forest Service bridged the photographs, using the same glass diapositives and identical control.

Table 2 gives three bridge results: Mann with PUG, Stereoplanigraph, model C8, b Operator A (1), and Stereoplanigraph, model C8, with PUG Operator A.

These results are not a true comparison of instruments, as the computation equation for the analytical procedure, using comparator measurements, are more sophisticated than those used for the analog bridging, using Stereoplanigraph measurements, althoug both sets of equations were developed by the USC & GS.

The analog bridge computations are based on the USC & GS Technical Bulletin No. 1 (Jan. 1958) and Technical Bulletin No. 10 (Sept. 1959). In 1963, Olin D. Bockes combined the equations and programmed them for use in an IBM 7074 electronic computer. Aerial analytic triangulation equations (USC & GS Bulletin No. 21) include corrections for film distortion, perspective center, symmetric and asymmetric lens distortion, atmospheric refraction, and relative orientation and adjustments for earth curvature, all of which are not included in the analog bridging program for use in the IBM 7040. The analog bridging procedure arbitrarily considers: perspective center by aligning th diapositive on the fiducial marks, film distortion by changing the focal length, lens dis tortion by using a correction plate, refraction and earth curvature by predetermined tip, and relative orientation by the parallax solution; but does not consider cross tilt (averaging the Stereoplanigraph measurements of carry-over points as they affect the total bridge) in the bridge computations. All of these arbitrary corrections would tend to make the bridging results from Stereoplanigraph measurements less accurate than those from analytical bridging. Another factor which tends to improve the accuracy of the comparator is that blurred images, due to movement during exposure, are better "centered" monoscopically when compared to stereoscopic "pointings."

Jesse R. Chaves of the Bureau of Public Roads recomputed these same analog C8 bridges, using the Stereoplanigraph measurements and the new USC & GS equations (Technical Bulletin No. 23), and the results were approximately the same. The unanswered question is: Why didn't Bridge 3 (each point measured four times) give better results than single measurement for each point? The only apparent answer is that witl this number of check points (77 horizontal and 210 vertical), single measurements ave aged more accurately than multiple measurements.

These results show the possibility of using photogrammetry to establish control. Existing ground control, as well as new control, should be targeted. Photogrammetry can be used to determine control position for natural image points, but some of the

TABLE 2

EXTENDING CONTROL SURVEYS BY PHOTOGRAMMETRY

BRIDGE [24 MODELS-SCALE OF PHOTOGRAPHY 1:4200-FLIGHT HEIGHT 2100 FEET]	INSTRUMENT	AVERAGE ERROR IN FEET $\left(\frac{\text{SUM OF ERRORS}}{\text{NUMBER OF POINTS}}\right)$		B FACTOR $\left(1:\frac{\text{FLIGHT HEIGHT}}{\text{AVERAGE ERROR}}\right)$		STANDARD DEVIATION (S) IN FEET $\left(S=\sqrt{\frac{\Sigma d^2}{N}}\ \ d=\text{ERROR}\ \ N=\text{NUMBER OF POINTS TESTED}\right)$		90% ACCURACY IN FEET (1.65 X S)	
		HORIZONTAL (77 POINTS)	VERTICAL (210 POINTS)	HORIZONTAL	VERTICAL	HORIZONTAL (77 POINTS TESTED)	VERTICAL (210 POINTS TESTED)	HORIZONTAL	VERTICAL
ADJUSTMENT "A" [CONTROL EVERY MODEL; I.E., 25 HORIZONTAL (H) AND 50 VERTICAL (V) CONTROL POINTS USED TO COMPUTE BRIDGE]	MANN/PUG	0.46	0.32	1:4565	1:6562	0.52	0.45	0.86	0.74
	C-8	0.67	0.42	1:3152	1:4954	0.74	0.54	1.22	0.89
	C-8/PUG	0.65	0.54	1:3231	1:3889	0.72	0.64	1.19	1.06
ADJUSTMENT "B" [CONTROL EVERY SECOND MODEL; I.E., 13 H AND 26 V CONTROL POINTS USED TO COMPUTE THE BRIDGE]	MANN/PUG	0.47	0.33	1:4468	1:6364	0.52	0.46	0.86	0.76
	C-8	0.68	0.43	1:3099	1:4886	0.76	0.55	1.25	0.91
	C-8/PUG	0.70	0.56	1:3000	1:3750	0.75	0.66	1.24	1.09
ADJUSTMENT "C" [CONTROL EVERY THIRD MODEL; I.E. 9H AND 20 V CONTROL POINTS USED TO COMPUTE THE BRIDGE]	MANN/PUG	0.48	0.32	1:4375	1:6562	0.52	0.46	0.86	0.76
	C-8	0.69	0.43	1:3031	1:4836	0.78	0.56	1.29	0.92
	C-8/PUG	0.75	0.55	1:2800	1:3818	0.80	0.66	1.32	1.09
ADJUSTMENT "D" [CONTROL EVERY FOURTH MODEL; I.E., 7 H AND 16 V CONTROL POINTS USED TO COMPUTE THE BRIDGE]	MANN/PUG	0.49	0.34	1:4286	1:6176	0.54	0.47	0.89	0.78
	C-8	0.71	0.47	1:2971	1:4461	0.79	0.60	1.30	0.99
	C-8/PUG	0.81	0.57	1:2593	1:3684	0.87	0.67	1.44	1.11
ADJUSTMENT "E" [CONTROL EVERY SIXTH MODEL; I.E., 5 H AND 12 V CONTROL POINTS USED TO COMPUTE THE BRIDGE]	MANN/PUG	0.53	0.35	1:3962	1:6000	0.59	0.47	0.97	0.78
	C-8	0.78	0.49	1:2709	1:4272	0.87	0.62	1.44	1.02
	C-8/PUG	0.89	0.60	1:2360	1:3500	0.97	0.72	1.60	1.19

accuracy will be lost to both the photogrammetrist and the field engineer when a finite point is not established. The analytic approach shows that errors will be reduced about one-third horizontally and one-fourth vertically, as compared to the analog method. The X corrections (E-W) were much larger than the Y corrections (N-S), which could be attributed to camera motion during exposure, as the flight direction was N66° 30'E.

The purpose of the second test was to determine if material (photography and control) from recent standard topographic mapping on a quadrangle basis can be used to control design photography at a scale of 1:6,000 (500 ft/in. for design mapping at a scale of 1:1,200; i.e., 100 ft/in.). For this test, we chose a project in the George Washington National Forest in Virginia, on which had been used the mapping photography (1:24,000—ft/in.) and control for making route investigations leading to route selection and preliminary design. This preliminary location was based on the use of a 1:4,800 (400 ft/in.) scale topographic map with a 10-ft contour interval.

This 1:4,800 scale topographic map was used to locate targets for both the control survey and preliminary (P) road location. Intervisible control targets were located some distance from the proposed route for identification of a field-surveyed traverse. This traverse extended back and forth across the South Fork of the Shenandoah River and had very limited use for route location. Intervisible targets were also set near the preliminary location for the road.

Again, the Virginia Department of Highways cooperated and furnished the photography, using the same camera as on the first test.

A spirit level elevation was measured for each targeted point along the preliminary location for the road. Positions, with vertical angle elevations, were measured along the control traverse.

Common image points were selected between the mapping photography (1:24,000 scale) and the design photography (1:6,000 scale), using the Zoom stereoscope. The Zoom stereoscope was also used to transfer the target positions from the 1:6,000 to the 1:24,000 scale photographs. This was quite difficult, as some of the ground cover (trees and bushes) had been cleared to set the targets.

The mapping photography was bridged using only the control established for the topographic mapping done on a quadrangle basis. This bridge established X, Y, and Z coordinates for both the common image points and the targets transferred from the 1:6,000 scale to the 1:24,000 scale photography. Thus 35 targets were transferred from the design photography to the mapping photography. All 35 targets had field measured elevations (either spirit level or vertical angle), of which 14 had field measured horizontal position. A comparison of field surveyed and bridged results showed an average elevation (Z) error of -2.8 ft. The average X (E-W) error was +8.6 ft, and the average Y (N-S) error was +14.7 ft. The datum shift in X was +8.1 ft, Y +12.3 ft, and Z -0.7 ft. When a ground measured distance of 11,788.8 ft was compared with the photogrammetrically measured distance between the same points, the error was 6.2 ft, or about one part in 1,900.

The design photography was then bridged, using only common image points as control. Thirty-seven targets, with field measured elevations (26 by spirit levels and 11 by vertical angles), and 15 targets with field measured horizontal position, were used to evaluate this bridge. The average X error was +3.3 ft, Y +13.8 ft, and Z +2.0 ft. The datum shift was X +3.3 ft, Y +13.8 ft, and Z +0.5 ft. When the same ground measured distance of 11,788.8 ft was compared with the photogrammetrically measured distance, the error was only 0.7 ft, or one part in 16,800. The datum shifts were similar to the preceding bridged results.

The bridge was recomputed using only the targets along the preliminary road location; i.e., those with spirit level measured elevations. Eleven targets, with vertical angle field elevations, were used to evaluate this bridge. The average error in Z was -0.8 ft, of which -0.4 ft was datum. It should be remembered that none of these targets were located along the proposed road, but were located near the pass-point (edges of the photographs) area. The X and Y errors for this bridge were the same as for the previous bridge, as the same common image points were used to compute the horizontal portion of both bridges.

TABLE 3

TRAVERSE RESULTS

	Bridge Computed Using Ground Surveyed Control		Bridge Computed Using Common Image Points as Control	
	Station in Feet	Delta Angle	Station in Feet	Delta Angle
T-1	0		0	
T-2	462.79	20°04'27" Rt	462.75	20°04'16" Rt
T-3	1,294.19	12°17'57" Rt	1,294.08	12°18'02" Rt
T-4	2,065.88	2°09'40" Rt	2,065.66	2°08'51" Rt
T-5	2,800.77	0°36'02" Lt	2,800.55	0°36'07" Lt
T-6	3,800.13	17°47'04" Lt	3,799.87	17°47'23" Lt
T-7	5,093.96	30°08'13" Lt	5,093.65	30°08'27" Lt
T-8	6,389.37	17°46'43" Lt	6,389.04	17°45'01" Lt
T-9	7,354.68	23°08'23" Lt	7,354.00	23°09'46" Lt
T-10	8,034.67	25°01'08" Lt	8,034.08	25°00'49" Lt
T-11	8,484.59	11°25'13" Lt	8,484.04	11°26'25" Lt
T-12	8,990.94	0°43'35" Lt	8,990.33	0°43'02" Lt
T-13	9,333.44	0°16'07" Rt	9,332.80	0°16'37" Rt
T-14	9,998.61	1°09'42" Lt	9,997.88	1°04'38" Lt
T-15	10,428.67	20°02'01" Lt	10,427.60	20°08'08" Lt
T-16	10,786.42	18°01'15" Lt	10,785.25	17°59'57" Lt
T-17	11,167.79	5°18'35" Lt	11,166.45	5°19'10" Lt
T-18	11,708.61	4°01'12" Rt	11,707.05	4°01'07" Rt
T-19	11,952.60	13°17'55" Lt	11,950.93	13°17'28" Lt
T-20	12,815.54	1°37'10" Rt	12,813.29	1°37'08" Rt
T-21	13,290.78	4°38'43" Lt	13,288.18	4°39'04" Lt
T-22	13,732.09		13,729.17	
			2.92	
			(1:4,771)	

For the final test, the bridge using the design photography was recomputed, using the field surveyed control; i.e., the 37 vertical and 15 horizontal control points, which were identified by targets. This bridge was compared to the bridge using common image points as control. The comparison was made by computing a traverse of the 22 targets along the P line. This was an important test, as the L line is usually staked by use of computed offsets from the P line. Table 3 indicates what errors can be expected.

These results are most promising, and the traverse checks show that an accuracy of one part in 4,700 should be obtained. The tests indicate a datum shift; i.e., all coordinates would be shifted by 3.3 ft in X, 13.8 ft in Y, and 0.7 ft in Z, if the field surveyed control is eliminated. The datum shift becomes a problem when the surveyor determines an azimuth from a geodetic station or by making a solar or polaris observation. The only way in which this shift could be eliminated would be to increase the accuracy of the topographic mapping bridges.

REFERENCE

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