The vertical transformation adjustment, which makes use of a general least squares curve fitting routine, should be accomplished on an electronic computer. The control point data are first read in and the co-efficients of the polynomial are determined using the curve fitting program. The pass-points are then read in and the adjustment for the elevations computed. Generally the control points are included with the pass-points in order to compare the computed elevations with the given data.

As a check on the electronic computer output, we utilize an automatic line plotter that operates directly from the output cards of the computer. The line plotter constructs the Z-z values in respect to a "zero datum" of the x distance. As the points are being plotted the operator numbers each point. A parabola is then constructed using a spline. This determines a point or points which might appear to be out of balance with other points in the strip.

In summary, in producing the Keystone Shortway topo maps, the use of aerotriangulation to extend ground-control to all stereo pairs of the compilation photography proved to be as accurate as conventional ground surveying methods of establishing control, and much faster and less costly.

*A Photogrammetric Cadastral Survey in Utah**

F. K. VAN ZANDT,
Bureau of Land Management,
U. S. Dept. of the Interior

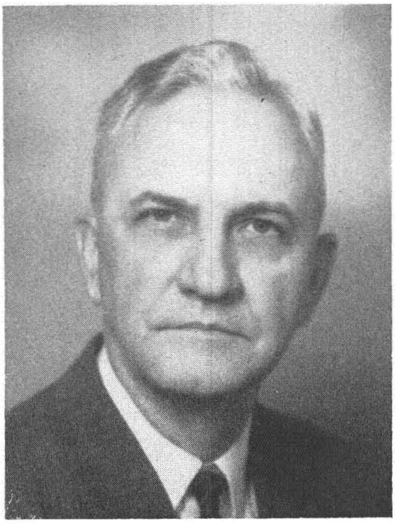
ABSTRACT: *The rugged topography of lands remaining to be surveyed and the difficulty of access increases the need for finding methods to cut costs of making cadastral surveys. The Bureau of Land Management has completed the survey described in this paper, using a combination of field and stereophotogrammetric methods. Original surveys were made covering ten townships. Five of these had been surveyed by conventional methods the year before. The comparison of positions obtained independently was used in appraising the reliability of the new procedure and as a criterion in the acceptance of the work on the other five. Attempt was made to take maximum advantage of maps, photography, and control done by other Government agencies.*

IN THE spring of 1957, the Bureau of Land Management decided to carry out an experiment in the use of photogrammetry for cadastral surveys. The primary purpose was to determine the specifications for photogrammetric procedures necessary to make original cadastral surveys within the degree of accuracy required by the Bureau of Land Management in extending the rectangular system of surveys over the public lands.

It is evident that with low-level photography and numerous control points, a high degree of accuracy can be obtained in the identification and determination of points on the ground. This was proved by the work done in the Tahoe test area of California by the U. S. Forest Service, with the Bureau of Land management assisting.[1] Careful position checks were made by parties of the U. S. Coast and Geodetic Survey. However, the problem was to determine whether the accuracy required in extending the rectangular network could be obtained by 1:20,000 photography and, if not, what scale of photography would be required. Also, the Bureau wished to know how the cost would compare with cadastral surveys made by ground methods, and whether manpower and time would be reduced in completing an assignment. Such a saving is especially critical at this time as the Alaska Statehood Bill provides for survey by the

[1] PHOTOGRAMMETRIC ENGINEERING, XXIII, 3, 493.

* Presented at the Society's 25th Annual Meeting, Hotel Shoreham, Washington, D. C. March 8 to 11, 1959.



F. K. Van Zandt

Bureau of Land Management of the boundaries of lands selected by the new State, amounting to more than 100 million acres to be selected over a period of 25 years. The size of this project may be realized when known that up to the present time, only about 2,700,000 acres have been surveyed in the entire State of Alaska. In making the survey, the land area will be computed after subtracting the vast expanse of water area. It is planned to measure the water areas by photogrammetry.

The desired area in which to carry on the test on accuracy and cost was found in southern Utah, northeast of the confluence of the San Juan and Colorado Rivers. For several reasons it seemed ideally suited to the purpose. See Figure 1.

1. The area to be surveyed was large enough to permit designing a practical project.
2. Surrounding lands were being surveyed by ground methods, so B L M personnel, a camp, and helicopters were already in the locality, and could be used for the needed control surveys on which the photogrammetric work was to be based.
3. Accurate and recent topographic maps of the Geological Survey covered the region.
4. Partial aerial photographic coverage already existed in the form of 6 inch, 1:20,000 scale photography taken in 1952 for the Atomic Energy Commission under a contract of the Geological Survey.
5. As the Geological Survey had at the time several photographic projects under contract in Utah, it seemed probable that the contractor might be willing to photograph the small additional area required to complete coverage of the B L M project at a figure favorable to the Government.

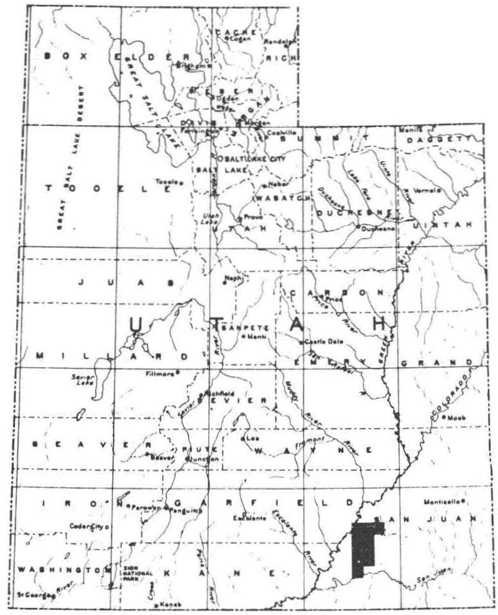


Fig. 1. Location of project.

6. The recent topographic mapping had necessitated establishing a fairly dense net of triangulation. The signals could easily be re-erected and flagged.
7. The ground cover over most of the area consisted of small juniper bushes up to a maximum of 12 feet high and of about the same diameter. These were sufficiently scattered that ground identification of a particular bush selected on the photo was generally certain.
8. The topography of the area was so rugged, with high cliffs and deeply cut canyons, that field work by tape and transit would be costly. See Figure 2. This led to believing that money could be saved by stereophotogrammetry here, if anywhere.

While it seems unlikely that so many favorable factors would be present in another project, the knowledge gained from this one might make possible savings in a later one that would make the project economically advisable.

The *Manual of Instructions for the Survey of the Public Lands of the United States*, 1947, Section 234, provides for limits of closure in making the official public land surveys. The basic limits of 21 minutes in departure from cardinal, or 50 links per mile allowable in measurement, have been modified to reflect a test of acceptable rectangularity. In ex-

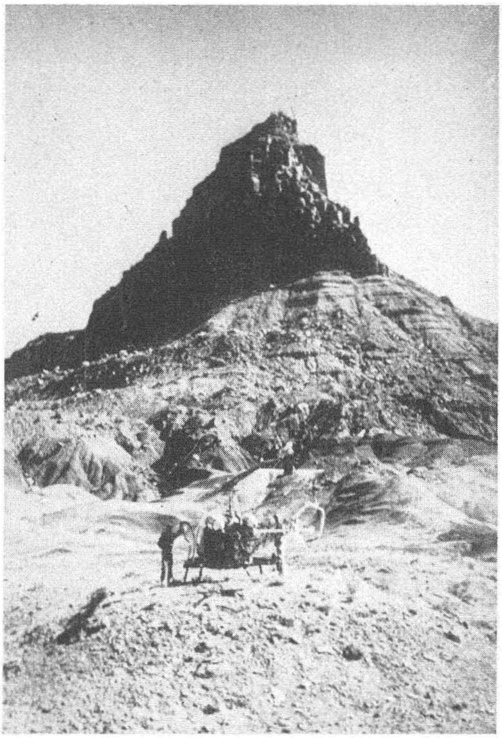


FIG. 2. Rugged topography in southern Utah.

tremely rough mountainous land, exceptionally difficult to survey, such as in southern Utah, it is expected that the average closure in latitude and departure of a survey surrounding a section should not exceed 25 links, equivalent to an error of closure of 1:1,280, in at least 23 sections of a township. The remaining 12 sections should close within 50 links or 1 part in 1:640. As previously stated, the test was directed toward determining if the accuracy required by the Bureau of Land Management could be obtained, and not to determine merely how accurate a cadastral survey could be made by photogrammetric procedure.

It was decided that 10 townships would be worked by the proposed method, and that 5 of these should also be surveyed by conventional methods as a check. The area is shown in Figure 3. As is the practice with other recent work in Utah, only the township exteriors and sections 2, 16, 32, and 36 were to be surveyed, These are the sections to be granted to the State for schools. If agreement between the two determinations was sufficiently close, then monumentation would proceed on the remaining 5 townships, based on the photogrammetric data.

PHOTOGRAPHY

The photography used for mapping was at a scale of 1:35,000 or 1:40,000, obviously too small for cadastral surveys. In fact, the Tahoe test had shown the desirability of a scale as large as 1:10,000. However, in this land of great distances, no existing property lines, and little cultural detail, it was desirable to know just what accuracy could be obtained from the 1:20,000 scale photography. The existing 1:20,000 scale photography covered three-fourths of the area and could be secured for the price of the prints and the diapositives. It had been taken with a six-inch metrogon lens.

The remainder of the area was to be photographed under contract, using a planigon lens at 1:20,000. It was decided not to specify large-scale photography due to the much greater number of stereo models involved and the many additional control points that would be required. Specifications were, in general, standard, as called for by the Geological Survey. The B/H ratio was 0.63, and the W/H ratio 1.00. The time of the photography was to be in the summer or fall, prior to December 1. The presence of shadows caused no

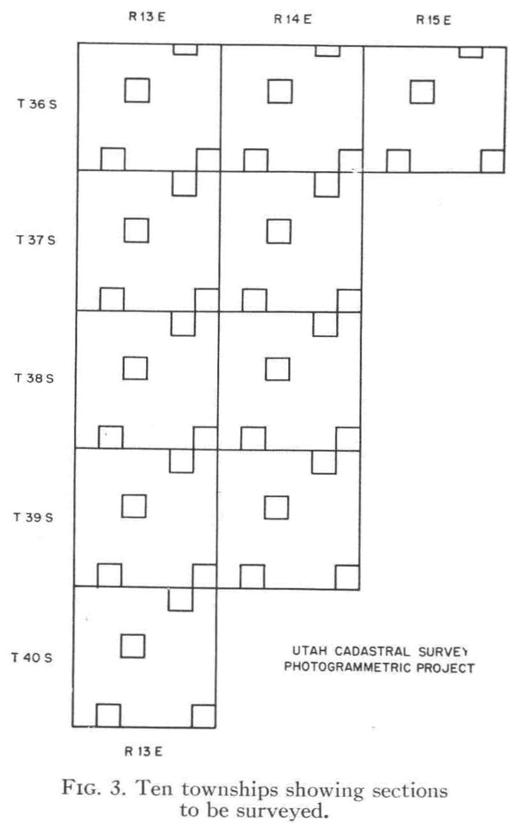


FIG. 3. Ten townships showing sections to be surveyed.

great concern, as it would not be necessary to draw contours, and a witness corner would be set at the top or bottom of the escarpment if the section corner happened to fall on a very steep slope. Photography was taken about the middle of November.

Control

Twenty-one triangulation stations were on the area to be stereotriangulated. It was necessary to put in and observe 18 more. Most of these new positions were determined by 3-pointing, using a small Askania theodolite, reading to one-tenth of a minute. Three panels of white cloth were laid down extending radially from the station, as shown on Figure 4.

The triangulator photographed the markers and the area surrounding each station from a helicopter. He used a 35 mm. camera, held in the hand, and as nearly vertical as possible. To do this, it was necessary to leave the door off the 'copter and to lean out while the pilot banked. A height of about 2,000 feet above the station seemed to give a picture most useful for identification on the six-inch photography. Transportation of the observer to the point was by helicopter, and had to be scheduled after all the line crews had been ferried to work and before the time to bring them in.

At the observing time, a sketch was made at each station showing its relation to surrounding photo-identifiable objects. This was a precautionary measure, and in nearly every case the small photo was much preferable as a means of determining the position of the control point on the 6-inch photograph.

As the additional field work was done pre-

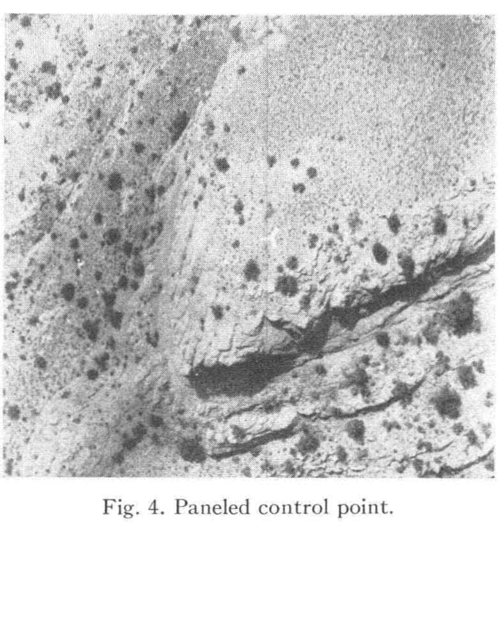


Fig. 4. Paneled control point.

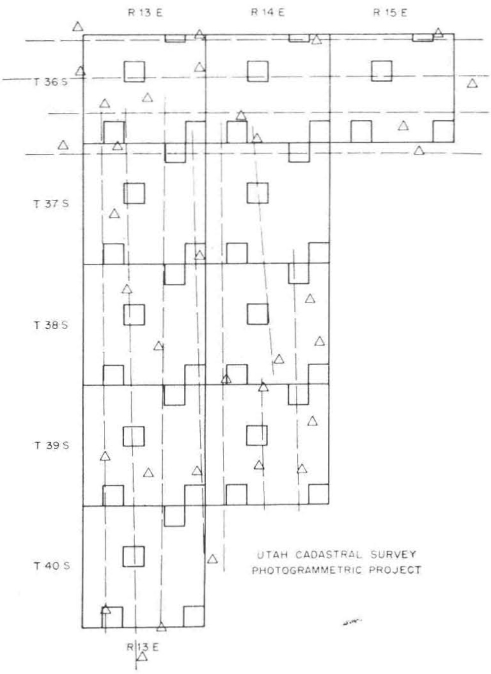


FIG. 5. Photographic flight lines control points.

vious to the date of the new photography, many panels showed up on it. Those that did not may have been destroyed or may not be recognizable, due to lack of contrast against the light-tan color of the soil and rock at the station site. Photographic flights and control points are shown in Figure 5. The new flights are east-west.

Adjustment and Computation of Section Corner Positions

As township boundaries completely surrounding the project area had been previously surveyed, it was first necessary to adjust them to the triangulation. Some ties of land corners to triangulation had been made, but others were needed. The ties indicated that the chained distances were too long. In some cases, it was possible to throw all the adjustment into the distances, leaving the bearings unchanged. In this type of country, the chaining is subject to greater error than the transit work.

The positions of all control points and of all section corners already established and to be established were computed on state plane coordinates. The positions of the corners to be set were adjusted to those of the existing corners, so as to follow the instructions prescribed for the survey of the public lands.

Stereotriangulation

The only photogrammetric method that would give the accuracy required for cadastral surveys dictated the use of a first-order plotting instrument. As the Bureau had no instrument of this accuracy or personnel to operate it, it was decided to ask the Geological Survey to handle the stereotriangulation on a repay basis. It was glad to cooperate on this basis; and the project was assigned to the Sacramento office to be done on the C5 stereoplanigraph.

All control, and the positions of all section corners and quarter corners, both existing and planned, were plotted on a base sheet at a scale of 1:20,000. This permitted an approximate scale solution. All final determination of positions was taken from machine coordinates. The stereotriangulator, during the course of bridging a strip, selected 3 photo-image points within about 300 feet of the section corner to be used as reference points for the location of the corner in the field later. He was able to spot the approximate location in the stereo model, by observing when the microscope of the coordinatograph arrived over the plotted position of the corner. This operation was one factor in the decision to use a stereoplotting instrument rather than an analytic method.

The first criterion for selecting a reference point is the certainty of its identification in the field. The scattering juniper bushes in the area offer the best reference points. While a small bush permits a more exact measurement, it must be of sufficient size to be distinguishable from those nearby. If there is none suitable in the close vicinity, then it may be necessary to choose something else, such as a drainfork, or a prominent rock.

The stereotriangulation involved 12 strips of about 18 models each. Four or more control points fall in each flight, being distributed along the strip. The positions of the points were dictated mainly by the topography, as the survey work was by triangulation. As the desired end result is horizontal position only, any vertical data needed was taken from the quadrangle maps. In addition to the corner reference points, it was necessary to read on all control, on two points near each photocenter for bridging, and on points near each edge of the flight for ties between models and between strips. X and Y machine coordinates for all points were listed and delivered by the Sacramento Office, U. S. Geological Survey, to Bureau of Land Management. Reference points and pass-points were marked and pricked on the photos and the reference-points were described on the back. Where there was a chance that the needle hole might obliterate the photographic image, it was placed to the side away from the approximate corner position. The approximate corner position, as indicated by the position plotted on the projection sheet, was marked on the contact print with a tiny red pencil mark. This proved very convenient in the field later on.

Adjustment to control and conversion to state plane-coordinates was done by the method described in *Technical Bulletin* No. 1, January 1958, of the U. S. Coast and Geodetic Survey. This operation had been programmed for its electronic computer; the Coast Survey processed the data and delivered plane-coordinate values in feet, for all points on each strip.

As these adjustments are independent for each strip, comparisons were made of the X and Y values of tie points as determined from adjacent strips. Deviations in feet were plotted as ordinates on graphs, on which distances along the line of flight were the abscissas. The graphs so constructed provided data for a logical adjustment of the coordinate values of the reference points.

Final Location of the Corners

The process of locating and marking the corner positions on the ground was started the middle of September 1958. A contract was let for the service of a helicopter. A pilot and a mechanic arrived with the 'copter on September 18. A trailer camp was established on the area to be surveyed. Camp was moved once during the progress of the work. It might be mentioned here that with electric lights, trailers, running water, and a helicopter, camping isn't what it used to be—it's much better. It was necessary to haul the water some distance over poor roads, however, so it was used sparingly.

A plane-table sheet was prepared on cross-section paper for each corner at as large a scale as the distances would permit, and the three reference points and the corner position were plotted on the sheet by their state plane-coordinate values. A compass line was drawn on each sheet with the declination for the locality. See Figure 6. This sheet was fastened on a 15 by 15 plane-table board with a compass on the side. This board fits on a Bumstead tripod. Distances were scaled between reference-points and from each reference-point to the corner. The distances between reference-points were chained on the ground, if it was necessary to check on the identification of the points. The procedure was to set up, level, and orient the plane-table. A sight alidade provided a means of changing the orientation from magnetic to true north. The distance from each reference-point was chained in the direction of the corner location. Theoretically, the three arcs described by these radii should intersect in a common point. The size of the triangle between the arcs is a measure of the accuracy of certain steps of the work.

Manual of Surveying Instructions, approval was given for setting corners in the north five, unsurveyed townships.

In analyzing these deviations, it should be borne in mind that means were not obtainable for comparing the ground-measurements, nor the photogrammetric results, with absolute values. Those errors which are inherent in the ground-surveys are not reflected in this comparison.

The topography, as required for the town

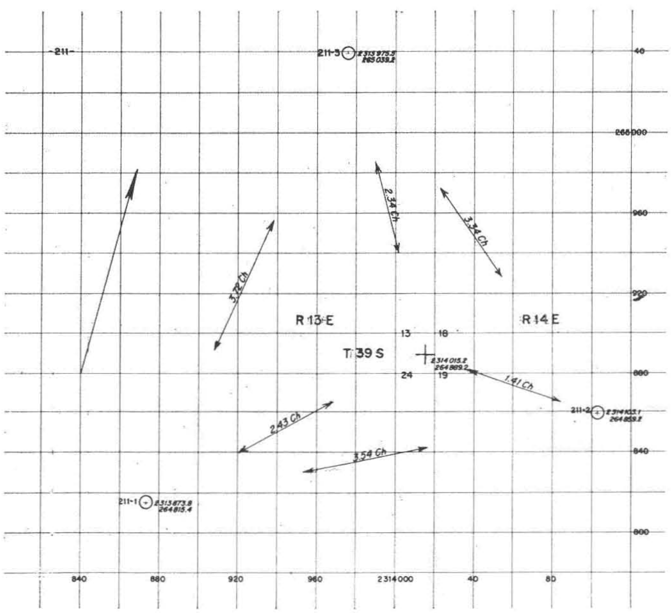


FIG. 6. Plane table sheet.

ship plats, will be taken directly from the quadrangle maps.

Total costs on the project were divided between the unsurveyed townships at the north of the project and the five used as a check, in proportion to the time and labor involved. Positions of quarter-corners were not determined in the five "check" townships so this was considered in the apportionment. The cost of the aerial photography contract was charged entirely to the north five townships covered by it. The costs of the corner monuments and of setting them in the ground was entirely chargeable to the north five. Results were somewhat disappointing as a summation of the costs for these townships showed that the cost per mile exceeded the cost by conventional methods by about 50 per cent.

Several things that increased the cost of the work are evident. Some of these could be

CONCLUSIONS

For an appraisal of the accuracy of the methods, the displacement of the photogrammetric position from the established corner was applied to each end of each mile of line that had been surveyed by the conventional ground surveys. Accepting the field measurements as correct, values were computed for the bearing and distance between the photogrammetric positions. The average deviation between these values and the corresponding ones on the approved township plats was 5 minutes in azimuth and 13 links in distance. The average closing error around a section for the photogrammetric work in the five townships where the ground work had been previously executed, amounted to 1:2,840 for those sections where the checks were made. However, as these closures conformed to the tolerances prescribed in the avoided if another project were undertaken while some others could not. One time-saving device highly desirable would be a printing counter for machine coordinates, now standard on the newer first-order instruments. Experience gained by personnel would permit greater efficiency in a similar undertaking. As was pointed out at first, it will probably be difficult to find another area with so many factors favorable to photogrammetry. The expense of going into the area twice and setting up a camp is a disadvantage of this method. A complete survey would permit a comparison more favorable to photogrammetry than the skeleton survey described here. One conclusion seems certain, the method is not adapted to a survey of a small area. The extent to which it will be used for other original surveys is a matter for further study.

This project is an example of one of a number of techniques under consideration by the Bureau of Land Management as an aid in speeding up surveys in Alaska and the unsurveyed lands in the other States of the West.

*The Use of Doppler Radar in Present and Future Mapping Operations**

HANS MEIER,
Aero Service Corporation,
Philadelphia, Pa.

ABSTRACT—*The removal of the military security restrictions on the Doppler Radar Navigation System has made possible adding a new tool to the commercially available electronic distance-measuring devices.*

The purpose of this paper is to explore the possibilities which this new instrument offers to the surveying and mapping profession. Since only a very limited amount of practical experience exists in this new application of Doppler, the ideas presented herewith should be considered as suggestions for studies to obtain the necessary statistical data to verify and prove our fondest hopes.

AT PRESENT, our position is the same as that of enthusiastic and optimistic pioneers after the Second World War, when Shoran was a navigation instrument and its development as a geodetic measuring instrument was proposed. The theory and instrumentation of the Doppler system which measures ground speed and drift angle by means of radar signals emitted from the aircraft and reflected from the terrain is assumed to be known. However, a short description of the Doppler principle seems to be in order.

Figure 1 shows an aircraft emitting two pencils of radiation downward and which are reflected back to the aircraft. The difference between the emitted signal-frequency and the received signal-frequency from a forward and backward-looking pair of radiation beams is measured. This difference is proportional to the aircraft's ground speed. If we use four beams radiating symmetrically to the aircraft axis, covering the ground in an x-shaped pattern, the drift-angle of the aircraft can be measured. The Doppler echo frequency-shifts from the two diagonal pairs of pencils are compared. If they differ, the antenna is not aligned with the actual path being flown, and the aircraft is drifting. The frequency difference actuates a servo mechanism, which rotates the antenna, aligning the beams with the actual path flown. The angle through

* Presented at the 25th Society's Annual Meeting, Hotel Shoreham, Washington, D. C. March 8 to 11, 1959.