NOAA Technical Report NOS 88 NGS 19

Horizontal Control

Rockville, Md.
June 1980

U.S. DEPARTMENT OF COMMERCE
National Oceanic and Atmospheric Administration
National Ocean Survey

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NOAA geodetic publications

Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys. Federal Geodetic Control Committee, Department of Commerce, NOAA, NOS, 1974, reprinted annually, 12 pp (PB265442). (A single free copy can be obtained from the National Geodetic Survey, OA/C18x2, NOS/NOAA, Rockville, MD 20852.)

Specifications To Support Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys. Federal Geodetic Control Committee, Department of Commerce, NOAA, NOS, 1975, reprinted annually, 30 pp (PB261037). (A single free copy can be obtained from the National Geodetic Survey, OA/C18x2, NOS/NOAA, Rockville, MD 20852.)

Proceedings of the Second International Symposium on Problems Related to the Redefinition of North American Geodetic Networks. Sponsored by U.S. Department of Commerce; Department of Energy, Mines and Resources (Canada); and Danish Geodetic Institute; Arlington, Va., 1978, 658 pp (GPO #003-017-0426-1).

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NOS NGS-1 Leffler, R. J., Use of climatological and meteorological data in the planning and execution of National Geodetic Survey field operations, 1975, 30 pp (PB249677).

NOS NGS-2 Spencer, J. F., Jr., Final report on responses to geodetic data questionnaire, 1976, 39 pp (PB254641).

NOS NGS-3 Whiting, M. C., and Pope, A. J., Adjustment of geodetic field data using a sequential method, 1976, 11 pp (PB253967).

NOS NGS-4 Snay, R. A., Reducing the profile of sparse symmetric matrices, 1976, 24 pp (PB258476).

(Continued at end of publication)

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Horizontal Control

Joseph F. Dracup

Rockville, Md.
June 1980

U.S. DEPARTMENT OF COMMERCE
Philip M. Klutznick, Secretary

National Oceanic and Atmospheric Administration
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PREFACE

The precision required for today's scientific technology plus a pressing need for more accurate inventories of our Earth's resources have contributed to the fast growth of the National Geodetic Survey (NGS) data files. These demands have increased the number of requests received annually. Rapidly changing space systems and computer technology are replacing traditional methods employed for generations. In the near future the NGS geodetic data sheets, which are available to the public through the Geodetic Control Data Automatic Mailing List Agreement (NOAA Form 29–3), will be generated entirely by computer. (A copy of this form appears at the end of this publication. For further information, contact the National Geodetic Information Center at the address given on the form.)

This report provides the user with current information on horizontal control. The historical development is summarized, including some methods which are now obsolete. In general, the material is presented chronologically. A companion publication, *NOAA Technical Report* NOS 73 NGS 8, Control leveling (Whalen 1978) details the history of the National Geodetic Vertical Network.

Since 1832, responsibility for the U.S. national geodetic networks has remained with a single organization, although its name has undergone transformation several times to reflect changes in mission. Originally, responsibility for surveys along the U.S. coast was delegated to the Survey of the Coast, the first Federal scientific agency, established by the U.S. Congress in 1807. By 1845 the organization was renamed the Coast Survey. Congress enlarged the agency's mission in 1878 to include geodetic surveys over the entire country and at the same time changed the name to the U.S. Coast and Geodetic Survey (C&GS). A Federal reorganization in 1970 created the National Oceanic and Atmospheric Administration (NOAA). The C&GS was renamed the National Ocean Survey (NOS) and became a main component of NOAA. The NGS is an office within NOS.

HORIZONTAL CONTROL

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ABSTRACT: The horizontal geodetic control network of the United States consists of about 240,000 stations of first-, second-, and third-order accuracies. This vast network has been in a continuing state of development since 1832. Originally, progress was very slow, but as new, improved instrumentation developed, vast strides occurred. Today, surveys can be made to accuracies that were impractical only a few decades ago. This improvement is attributed to advances in electronics and to the utilization of satellites and quasars. The publication and maintenance of up-to-date data are a tremendous task, which has been made manageable by automation. When the new adjustment of the North American Datum is completed in the mid-eighties, the adjusted data will represent the optimum results obtainable.

INTRODUCTION

In the surveying and mapping of large areas the exact curvature of the sea-level surface of the Earth must be taken into consideration. Therefore, basic surveys are called geodetic surveys. An intricate network of precise geodetic surveys extends over the entire United States. It includes, in addition to the arc and area control established mostly by triangulation, the high-precision traverse with stations in 44 conterminous States. This vast traverse network of superior accuracy, observed between February 1960 and November 1976, is about 22,000 km in length and generally follows some routes of the principal triangulation. The high-precision traverse forms seven primary loops, somewhat rectangular in shape, with three smaller loops. One smaller loop encompasses sections of Arkansas, Louisiana, and Mississippi. The other two are located in California. In addition, a spur line juts into Florida and another extends from Maine to Maryland. The primary network includes the equivalent of about a dozen east-west arcs of triangulation and about 30 north-south arcs, although they are not in a regular pattern, as might be inferred from this statement. The aim has been to have the main arcs spaced at regular intervals over the country, leaving the intermediate areas well covered with supplemental control established by triangulation, trilateration, and traverse, or a combination of these methods. This precisely determined network serves as a basis for intermediate and local surveys and for accurate mapping. In the future, the classical methods for positioning stations will be employed less frequently as the accuracy of satellite, extraterrestrial, and inertial systems increases.

Geographic positions whose coordinates are expressed in latitudes and longitudes have been determined for many thousands of marked stations throughout the United States. Surveys of small areas can be based on any of these marked points. All points are correctly coordinated in position with all precise surveys and maps anywhere in the United States and with all local surveys connected to the network. The permanency of the results of the surveys connected to the national net is also assured. Any marked points which later may be lost or which may lose their integrity can be duplicated by new surveys based on nearby stations.

For many years triangulation was the most efficient method used for surveying extensive areas. This method is still preferred for certain situations and locales. Prior to 1952, when electronic distance measuring instruments (EDMI) were introduced in the United States, trilateration was totally impractical on a large scale. Then, traverse involved the tedious, time-consuming, and expensive operation of measuring with tapes the lengths of all lines entered into a survey. Triangulation, on the other hand, was comparatively simple and inexpensive to implement. Accurate control could be established over large areas in less time. The method consists of a system of connected triangles with all angles carefully observed, but with only an occasional length actually measured. Each measured length is known as a base. By using these measured angles and bases, the lengths of all other sides of the connected triangles can be computed mathematically. If the latitude and longitude of one point are known, together with the azimuth to one of the other stations, the latitudes and longitudes of all other points and the azimuths of other lines can also be derived.

Trilateration is a method of surveying in which the lengths of the triangle sides are measured. The angles of the triangles are computed from the measured distances, and the latitudes and longitudes for the stations are determined in the same manner as for triangulation. With the advent of EDMI, trilateration became a practical way to extend control expeditiously, often to a higher accuracy than previously obtainable from conventional triangulation and traverse.

Traverse is the surveying method most favored by surveyors and engineers. It uses a sequence of observed directions, or angles, and measured lengths to derive the latitudes and longitudes of stations under the same conditions stated for triangulation. The method has the advantage of following the terrain and therefore obstructions can be avoided. However, two disadvantages are evident. Since courses are often short, significant errors may be introduced in azimuth. In addition, because the observations are secured at ground level, atmospheric variations may be more pronounced. This does not necessarily mean that the ease of operation and the cost effectiveness of the method, which make traverse attractive to surveyors and engineers, are necessarily the same or should be examined in the same light when considering the method to extend the national network. For geodetic purposes, traverse must meet certain design requirements; e.g., stations more or less equally spaced and limited allowable deviations from a straight line that connects the established control stations. Often the traverse method is selected when a survey is to be performed in a sparsely populated area or when the principal reason for the survey is to strengthen the network.

During the past two decades, the classical methods for establishing geodetic control have been consolidated on numerous surveying projects. The merger of triangulation, trilateration, and traverse procedures is called a mixed-mode survey. In planning such work, the acceptability of the design can be ascertained by computer simulations and verified by sequential adjustments as the observations progress.

The surveying methods described here are simple to use over small areas where the Earth's curvature need not be considered. Over large areas, the computations may seem formidable to a local engineer or surveyor who is not experienced in the use of the required special formulas and computing practices, even when electronic computers are available. This difficulty can be overcome if the surveyor or engineer uses the State plane coordinate system. The data for horizontal control issued by the NGS include the x and y rectangular coordinates of each station as well as the latitude and longitude. In connecting to these stations, a local surveyor or engineer needs to consider only the plane coordinates and compute the surveys on the simple, familiar, rectangular system. The surveys will then be coordinated with the national net as effectively as if the more difficult coordinates of latitude and longitude had been used. Furthermore, the latitudes and longitudes of several important points can be easily derived later, if desired, by converting from one system to the other.

The data for all horizontal control established or adjusted by NGS are available to the user; e.g., engineers and surveyors. These data include station latitudes, longitudes, and plane coordinates, a detailed description of station locations, and, in most cases, at least one azimuth. In some instances the lengths and azimuths of the lines between contiguous stations are furnished. However, an important caution should be observed. A surveyor or engineer who uses the data should be careful not to confuse the geodetic azimuths of the lines with the plane coordinate azimuths. These two kinds of azimuths may differ considerably because the convergence of the meridians is considered when deriving the geodetic azimuth, but the plane coordinate or grid azimuths are strictly rectangular with reference to a central meridian.

NOAA Technical Memorandum NOS NGS–5, National Geodetic Survey data: availability, explanation, and application (Dracup 1979) describes current data and services available from NGS. Geodetic data may be obtained from: Director, National Geodetic Information Center (NGIC), National Ocean Survey, NOAA, Rockville, MD 20852.

FIELD METHODS AND EQUIPMENT

The following field operations are required to establish horizontal control: First, a reconnaissance is made to determine the best locations for the stations. Next, the necessary towers, stands, or masts are erected to make the stations intervisible, and the stations are marked. Then, the angles at specified stations are carefully measured with a theodolite. Any distances needed for the control of the triangulation, or lengths required to position points by means of trilateration or traverse, are measured next. Finally, the necessary astronomical observations are made if the scheme is long enough between connections to network stations previously adjusted to require a strengthening of the azimuths, known as Laplace control.

Reconnaissance

Reconnaissance can be described as the design of a horizontal control network. It is carried out by a small party and usually consists of an observer and one assistant or, occasionally, only the observer. The party selects the sites for all the main and supplementary stations; tests the intervisibility of all stations that must be seen for the angle or length observations; specifies the necessary heights for the signals; collects information regarding roads, climatic conditions, and other facts that may be useful to the signal-building and observing parties; and interviews property owners to secure permission before the horizontal control field party can enter the premises.

The work of the reconnaissance engineer is extremely important. If the stations selected for a triangulation network form poorly shaped triangles and quadrilaterals of low strength of figure, an excessive number of base lines will be required. When trilateration methods are employed, the configurations must conform to rigid specifications to avoid angle observations. In designing a traverse net, any abrupt changes in the direction or large variations in the lengths of the courses would weaken the system. On those occasions, when the design does not meet minimum specifications, it is often necessary to measure the directions and lengths at specific stations on more than one occasion and observe additional astronomic azimuths and positions. Where mixed-mode surveys are involved (and this type of network is becoming more common), the reconnaissance engineer must be certain that all procedural requirements are completely met. If the observing party finds that some of the lines are blocked by obstructions previously overlooked by the reconnaissance engineer, the revision of the scheme or the building of higher signal towers may seriously delay securing the observations and may increase the costs. A long period of training is essential for the success of this important part of geodetic surveying. U.S. Coast and Geodetic Survey *Special Publication* 225, Manual of reconnaissance for triangulation (Mussetter 1951), contains detailed reconnaissance instructions.

Signal Building and Marking Stations

In hilly or mountainous areas, the stations can usually be placed on elevated points intervisible from the ground. In flat wooded areas, towers must be built to elevate the instruments, reflectors, and the observer, as well as to elevate the lights on which the horizonal direction observations are made. When employing mixed-mode methods, various types of portable masts are used for mounting the lights and reflectors. In flat areas, the Earth's curvature is sufficient to obstruct a line even if no trees are in the way. The most convenient and economical type of signal is the Bilby steel tower. This structure, which was originally adopted in 1927, is described in U.S. Coast and Geodetic Survey *Publication* 62–3, Bilby steel tower for triangulation (Richards 1965). The tower consists of an inner tripod that supports the theodolite and distance measuring equipment, and an entirely independent outer tripod for the observer, recorder, lightkeeper, and mirror tender (figs. 1a and 1b). The tower can be erected in 4 to 5 hours, dismantles easily in even less time, and can be used over and over again. The height varies up to 35 m, depending upon the number of sections used. For shorter towers, one or more of the bottom sections are omitted. Superstructures, each about 3 m in length, are added when additional height is needed.

Several portable truck-mounted towers have been developed by NGS. At present, tower height is limited to about 15 m. Figures 2a and 2b show a typical truck-mounted tower and portable mast.

A special personnel unit, known as the building party, erects the towers and masts as fast as needed. As soon as the observations have been completed on a station, the tower or mast is dismantled and moved forward to be erected at a new station. For a double-observing party working on arc-type surveys, 12

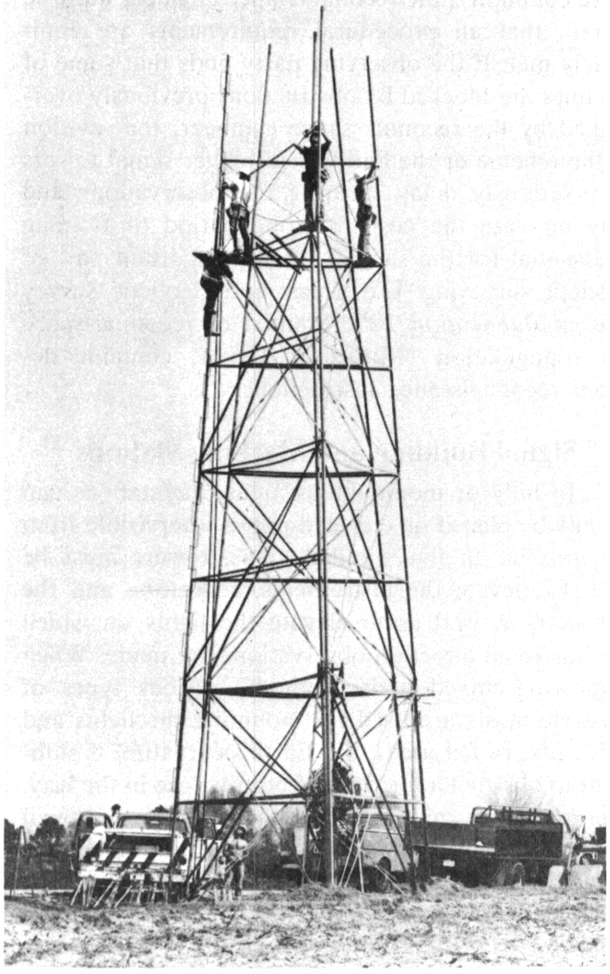


Figure 1a.—Bilby tower in the process of being erected

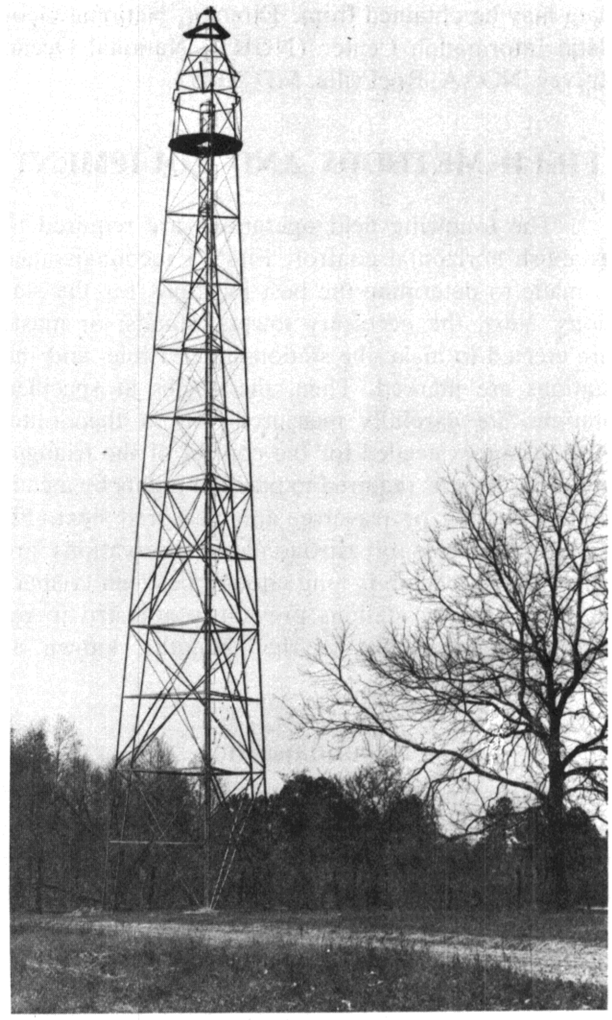


Figure 1b.—Completed tower

towers or masts will normally be used: eight towers will usually remain standing while the two towers at the rear are being dismantled and the two towers at the forward position are being erected at new stations. When area control is observed between arcs, additional towers or masts are required.

The building party also marks the stations and sets reference and azimuth marks. At each station an underground mark is set in addition to the surface mark, if conditions permit. The marks with inscribed legends are composed of bronze or brass disks. These are set in concrete posts or in large boulders or in bedrock. Some stations are established atop buildings. Various means are employed to attach the disks to the roofs or to reference the point so it can be replaced exactly. Reference and azimuth disks have inscribed arrows which are pointed toward the station. The reference marks, two for each station, are set fairly close to the station, usually within 30 m. An azimuth mark is placed about 0.4 to 3 km distant from the station to allow its use by local surveyors or engineers in obtaining an accurate starting azimuth. The azimuth mark is so placed as to be visible from the ground at the station; therefore, it can be used for local surveys without the need of towers or stands.

To aid in recovering stations and azimuth marks, witness signs or painted wooden posts are usually set at the sites. As a secondary purpose, they serve to alert construction and utility service personnel that a survey monument is nearby. Figures 3a and

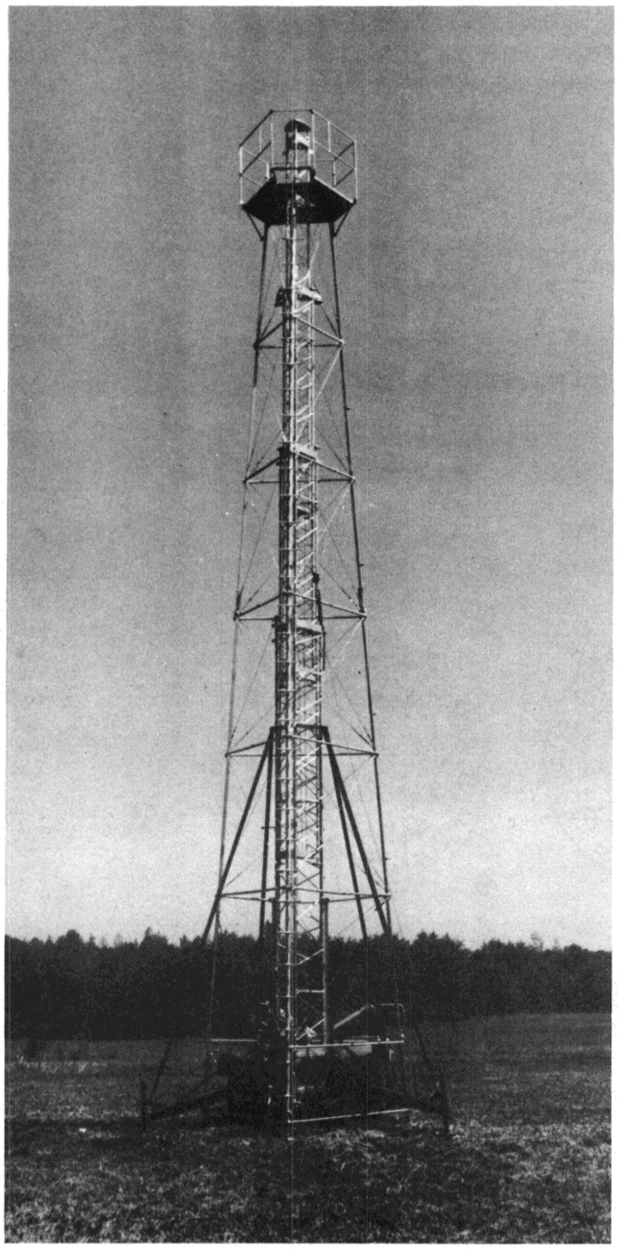


Figure 2a.—Truck-mounted observing tower

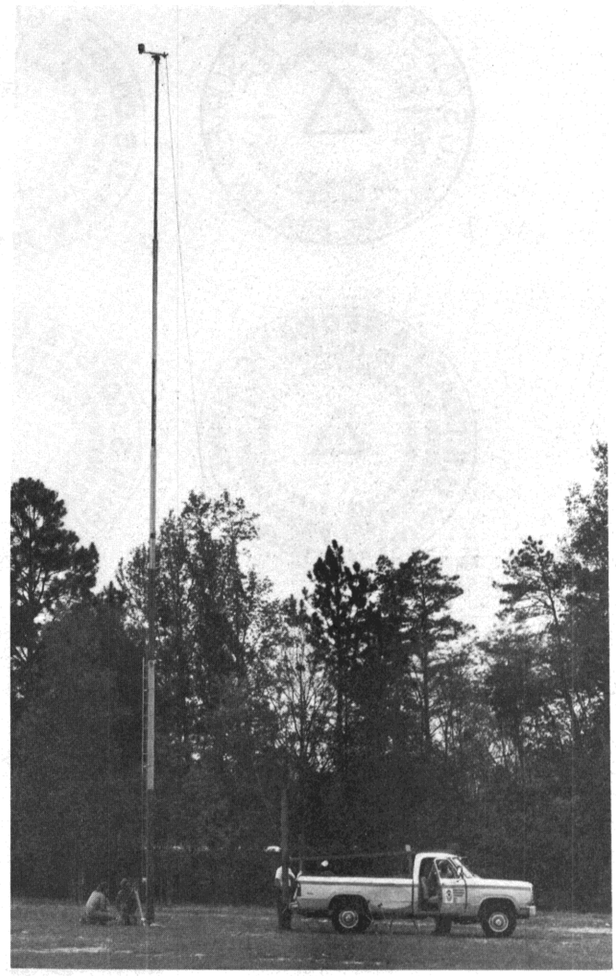
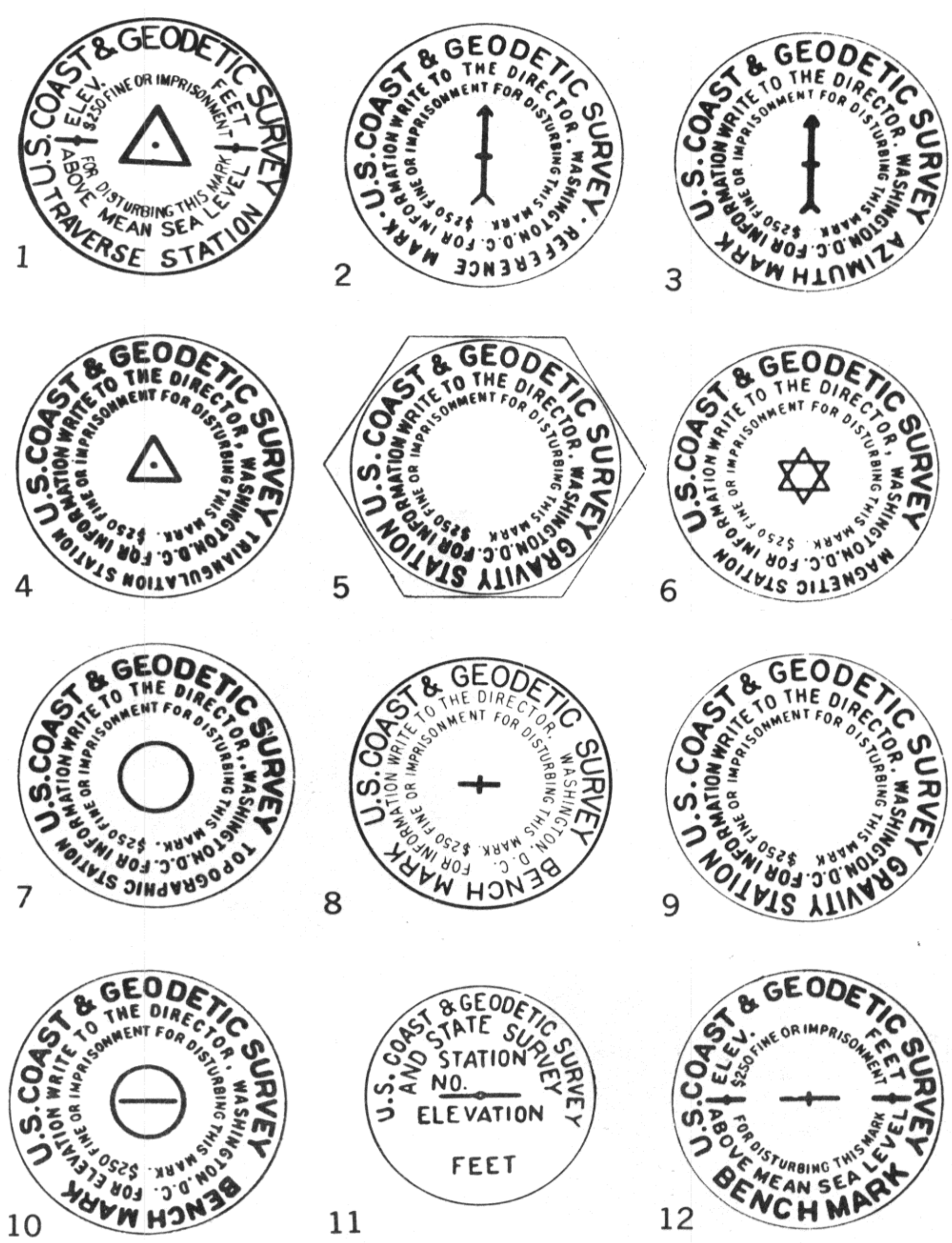


Figure 2b.—Portable mast elevates lights and reflectors

3b show the various bronze and brass disks used by the NOS/NGS and the former U.S. Coast and Geodetic Survey. The cooperation of engineers and surveyors is earnestly requested in preserving all stations and bench marks.

Horizontal Direction Observations

The angle observations on main scheme stations for first- and second-order surveys are made almost entirely at night. An efficient type of electric signal lamp has been developed for nighttime observations. The lamp is operated from dry-cell batteries and has a concentrated filament accurately placed at the focus of a parabolic reflector that produces a small but concentrated beam of light. A truck battery may also be used advantageously, although a rheostat is normally required. A lightkeeper at each station on which observations are being made directs the beam accurately toward the observer's station. If necessary, communications may be transmitted to the observer by radio. If a radio is not available, the light beam is obstructed to create a series of dots and dashes, according to the Morse telegraphic code. Nighttime observations are affected less by horizontal refraction and other atmospheric disturbances than daylight observations. Heliotropes, which are made visible by reflecting direct sunlight towards the observer, are sometimes used for daylight observations.



1. Traverse station mark.
2. Reference mark.
3. Azimuth mark.
4. Triangulation station mark.
5. Gravity station mark (old type).
6. Magnetic station mark.
7. Topographic station mark.
8. Geodetic bench mark (new type).
9. Gravity station mark (new type).
10. Tidal bench mark.
11. State Survey mark.
12. Geodetic bench mark (old type).

Figure 3a.—Standard marks of the U.S. Coast and Geodetic Survey

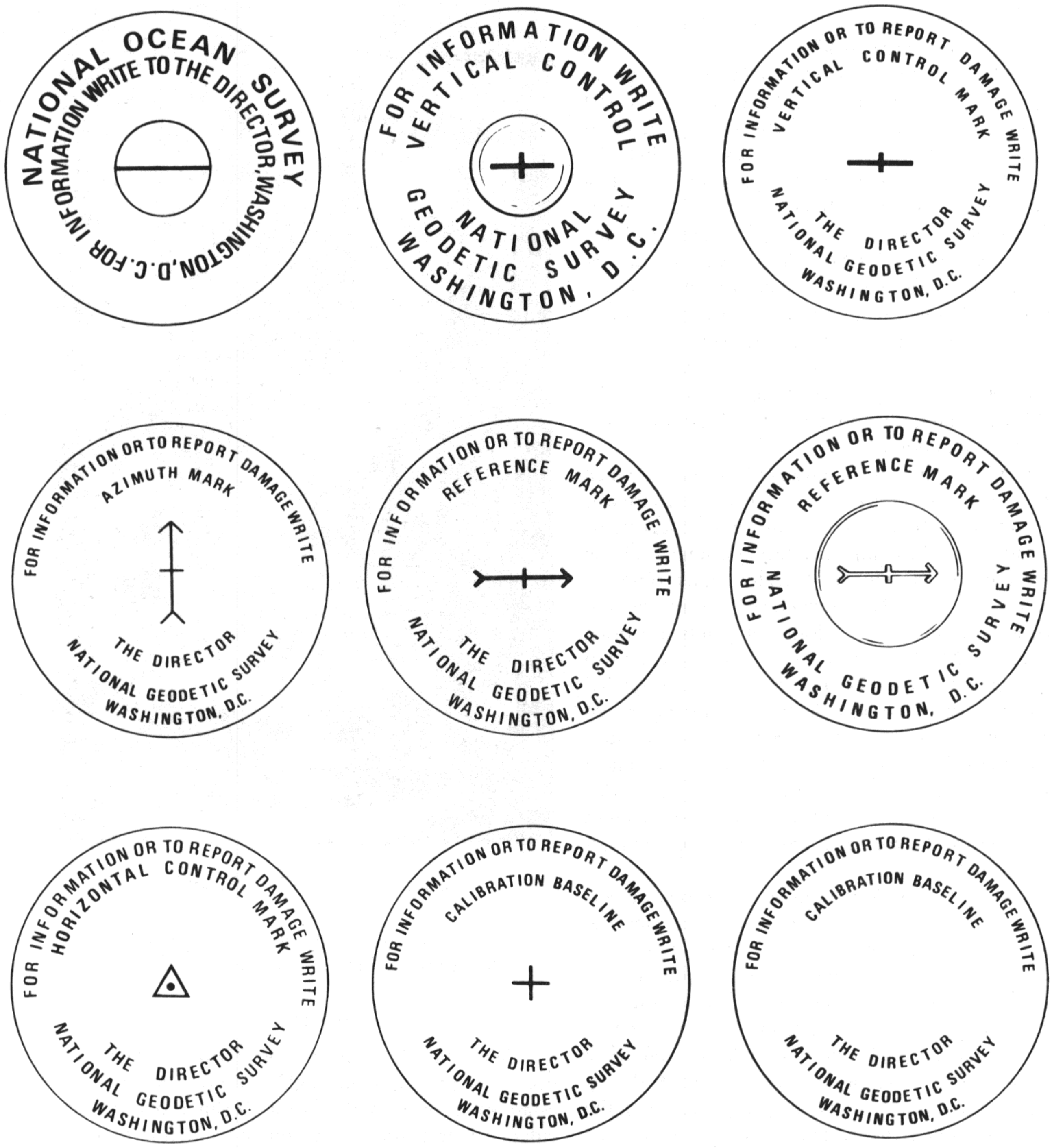


Figure 3b.—Standard marks of the National Ocean Survey/National Geodetic Survey

The angles are measured with theodolites. These instruments are similar to the surveyor's transit, except they are more precisely constructed and employ micrometers for circle readings. Two general types include the direction theodolite, which is used exclusively today, and the repeating theodolite. U.S. Coast and Geodetic Survey *Special Publication* 247, Manual of geodetic triangulation (Gossett 1959) describes both instruments. In the past, the repeating theodolite was used at stations, on high buildings or other structures, or wherever the movement of the observer was apt to disturb the position of the instrument. This was necessary because the observations on each angle required very little movement of the observer when using a repeating-type instrument. However, improved methods in constructing stands and supports for the observers have eliminated the need for repeating theodolites today. Figure 4 shows a direction theodolite.

If all the lights to be observed are visible when needed, or if the return signal from the reflectors or receiving instrument is satisfactory, the observations at a station may be completed during a single evening on all contiguous stations. Usually, other observations are necessary. To supplement the principal stations, a large number of additional points are often determined. Most of these extra points are known as intersection stations because they are not occupied by the observer, but are determined by means of intersecting observations from two or more stations of the network. If only two observations are made on the stations, they are known as "no check" stations because the accuracy of the observations cannot be checked by closures of triangles or agreements of derived lengths.

Intersection stations consist of various objects usually categorized as landmarks. These include smokestacks, water tanks, church spires, lighthouses, and standpipes. They may also include signals erected over stations of other organizations that are visible from the occupied stations. No attempt is made to mount signal lamps or reflectors on these structures. The observations are obtained during daylight hours, usually in the afternoon preceding the evening on which the regular observations will be made. A smaller number of observations (usually only one-fourth as many) are taken on the intersection stations than on the monumented stations. However, the resulting positions will be accurate enough for most local uses. "No check" points should be used with extreme caution in connecting them to local surveys. They should always be verified, if possible, by additional connections to at least one other station, preferably a checked point.

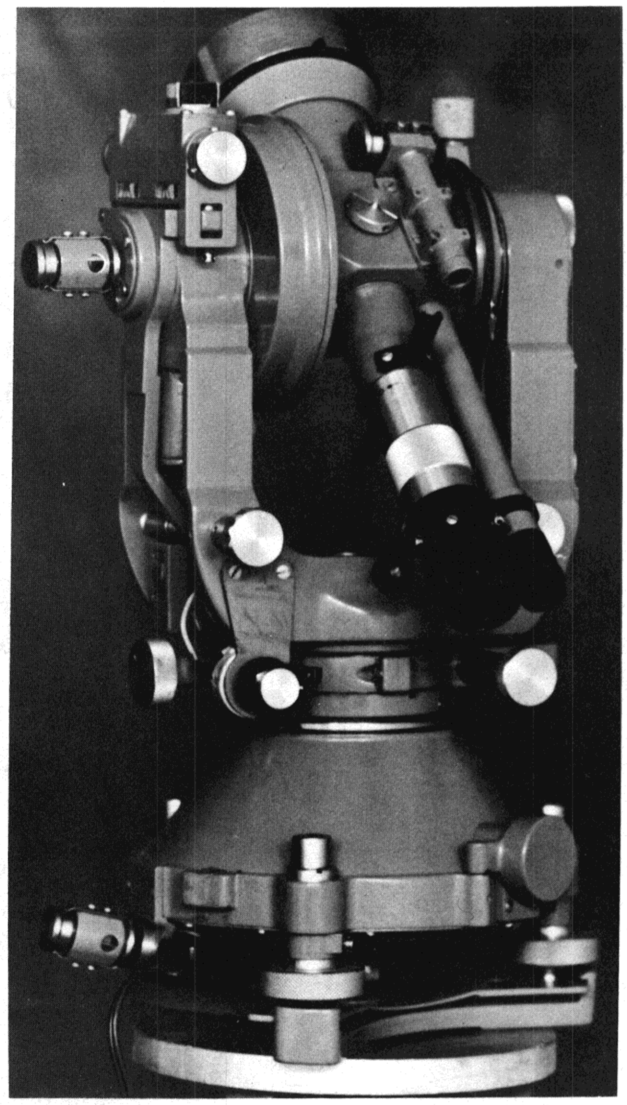


Figure 4.—Wild T–3 theodolite

Trilateration

With the development of EDMI, trilateration became a practical and highly accurate method for establishing horizontal control. When properly executed, trilateration is undoubtedly superior to both triangulation and traverse. The main disadvantage is the considerably smaller number of internal checks available, as compared to triangulation. For example, the classic triangulation figure, the quadrilateral containing both diagonals, has four triangle closures, of which three are independent. In addition, there are also other checks, specifically the agreements between common sides of the triangles. The total number of checks or redundancies for such a figure in triangulation is four. If trilateration is employed, there is only a single redundancy. Networks that contain the equivalent redundancies of traditional triangulation can usually be developed. Except for special projects the surveying costs are generally prohibitive because of the additional number of required points and observations. The group or ratio method of length measurement is adaptable to trilateration and can provide the same number of checks as triangulation. However, cost also is a determining factor because multiple measures of the lengths from both ends are required. This method is briefly described in the section on EDMI.

To maintain the internal strength of trilateration where conventional figures and methods are used, well-shaped configurations are required for both arc and area networks. For arcs, quadrilaterals must approximate a square with both diagonals measured. However, if only a single diagonal can be observed, a center point must be visible from the four vertices of the quadrilateral. In area systems, well-shaped triangles containing angles seldom less than 25° are mandatory for first- and second-order surveys. If these conditions cannot be met, then one or more of the larger angles in the quadrilaterals or the triangles that contain smaller than permitted angles must be observed. In addition, to reduce the length observations properly, accurate elevations are required.

When trilateration is used to extend the national network, the need exists to provide azimuth control at the stations and to position landmarks within the project area. Thus, the cost benefit ratio which favors trilateration over triangulation—if only the establishment of monumented points is considered—can be substantially reduced where other requirements must be met. However, for special purpose high-accuracy surveys, trilateration is often preferred. When used by local surveyors and engineers, this method can produce good quality control, with adequate internal checks as the work progresses, if the terrain permits acceptable configurations.

Traverse

Before the development of steel towers in 1927, most horizontal control surveys in flat, wooded country were made by extending traverses along railroads and highways. Triangulation in these areas would have required the construction of wooden towers, and the extra material and labor would have made the job too costly. During that period traverse differed from triangulation in that all lengths had to be measured in about the same manner as base lines. The method also lacked the automatic checks on the angle observations that were obtained from the triangle closures when the triangulation method was used. Traverses were observed to first- and second-order accuracy and usually followed railroads or highways. These routes had the advantage of a cleared right-of-way. The rail also provided support for the tape during length measurements.

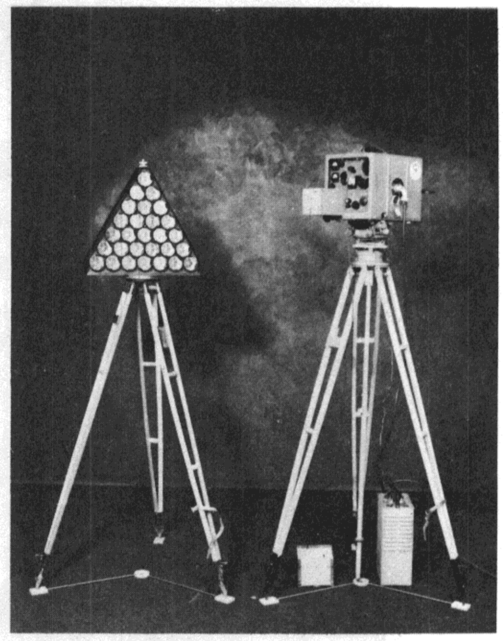
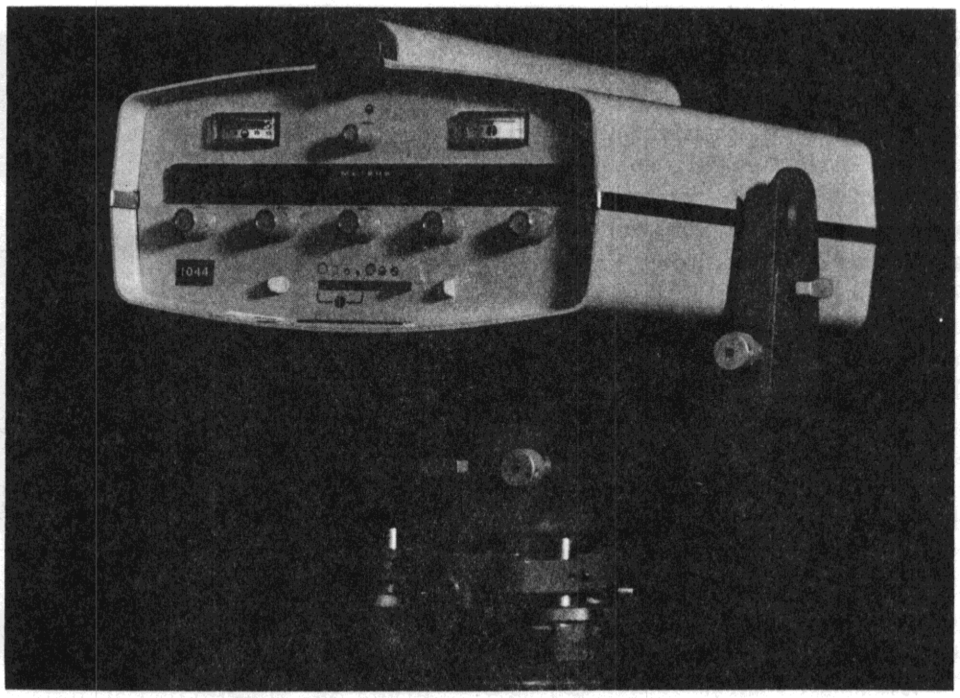


Figure 5a.—Electro-optical distance measuring instrument and reflectors

During the decade following 1917, a large number of traverse lines were measured and still remain part of the national net. However, after the adoption of the steel tower, very few first- and second-order traverse surveys were observed by the U.S. Coast and Geodetic Survey until EDMI was introduced in 1952. Since that time, thousands of kilometers of first- and second-order traverse have been observed. Many of the traverses were observed as part of the vast highway improvement program under cooperative arrangements with the States. For most of the surveys conducted under that program, the lengths were measured with microwave instrumentation. Today, for all traverses observed by NGS, the lengths are measured with electro-optical and infrared instruments. Microwave equipment is still used for certain classes of accuracy in trilateration. Figures 5a and 5b show some examples of electro-optical and infrared distance measuring instruments.



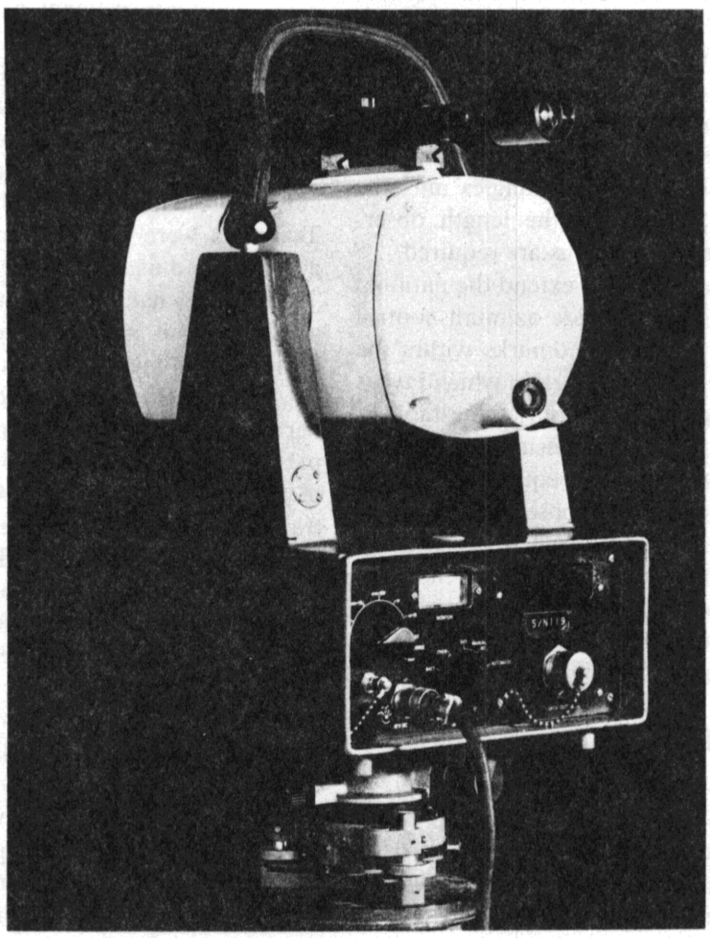


Figure 5b.—Infrared distance measuring instruments

Traverse is inferior to triangulation and trilateration in both execution and use. Extreme care must be exercised in taking the measurements to avoid blunders caused by a lack of automatic checks. The stations determined by traverse usually are along a narrow band and, therefore, do not serve as control for an area as large as arc networks of the same length. Area systems will normally provide about the same number of stations for any method. In opposition, however, is the fact that traverse stations are usually more easily accessible to the local engineer or surveyor than some stations established by other methods.

Electronic Distance Measuring Instruments

Electronic distance measuring instruments determine the length of the lines by timing the passage of visible and invisible light or radio waves transmitted between two points. In a technical sense, invisible light does not exist; but for simplicity the terminology is applied to infrared radiation, which falls outside the visible spectrum. Instrumentation falls into two general categories: lightwave and microwave. Lightwave equipment is further subdivided into electro-optical and infrared instruments, although infrared can be classified as electro-optical. Most modern electro-optical measuring devices now use a laser light source. In the past, the light source was provided by tungsten and mercury vapor lamps.

Although some electro-optical and microwave instruments are capable of measuring distances greater than 100 km, today lines longer than 50 km are seldom needed to extend the national network. Infrared devices are limited to about 4 km. However, under ideal conditions distances twice this length have been measured. Properly calibrated electro-optical equipment with good accessory instrumentation, operated by experienced personnel, can measure distances to about one part in a million (usually expressed as 1 ppm or 1 in 10^6). Under some meteorological conditions microwave instruments can also meet this accuracy, but normally one part in 200,000 to 300,000 (5 to 3.3 ppm) is the expected average. This is caused by the uncertainty in determining relative humidity, which significantly affects microwave measurements, but seldom amounts to more than 0.5 ppm in distances observed by electro-optical means. A distance that has been measured using infrared equipment, following minimum recommended practices, is generally accurate to a centimeter or less. Special observing techniques can be applied to some instruments that utilize an infrared light source to secure distances accurate to less than 5 mm.

Multiwavelength instruments can measure distances accurate to 1 in 10^7 and better. In the near future, two- and three-wavelength instrumentation will replace other types of measuring equipment in those instances where even higher accuracy is required.

The major difficulty in measuring distances is the necessity of obtaining a representative value for the temperature over the observed line. Occasionally, aircraft equipped with highly sensitive thermistors are flown over the line at the height of the ray path during the time of observation to obtain temperature data. The best results are obtained with instruments that utilize two lasers of different wavelengths. Two distances are obtained that differ by small amounts. These differentials are then applied in a mathematical process to determine the average temperature which is accurate to 0°.1 C or better. In addition, multiwavelength equipment can include microwave devices. These obtain the relative humidity component to a higher accuracy than by conventional practices.

The ratio or group method for measuring distances can be applied where two or more lines are observed from the same point. The observing routine is similar to that used in observing horizontal directions. This method relies on the phenomena that, within a limited area and time period, changes in the refractive indices over the various lines tend to be alike. This results in the ratios of the distances remaining almost constant. Observations made on different days over the same lines may vary by several parts per million, but their ratios will remain essentially unchanged and variances in the measurements can be attributed to temporal changes in scale. The ratio method is especially useful in measuring small changes, such as those involved in dam deformation studies and crustal motion investigations. It is also recommended where superior accuracy length measurements are required in horizontal control surveys. *NOAA Technical Report* NOS 74 NGS 9, Survey of the McDonald Observatory radial line scheme by relative lateration techniques (Carter and Vincenty 1978), contains a discussion of the procedures and the results obtained on a specific project.

Calibration Base Lines

The NGS has a program for establishing base lines, in cooperation with local organizations, for use in calibrating and testing EDMI. To provide an optimum testing pattern, the range usually consists of monuments set at an initial point (or zero monument) and at 150 m, 400 to 430 m, and 1000 to 1400 m. Measurements are made in all combinations, and the adjusted results are furnished in computer-generated tabulations. *NOAA Technical Memorandum* NOS NGS 8, Establishment of calibration base lines (Dracup et al. 1977) and *NOAA Technical Memorandum* NOS NGS 10, Use of calibration base lines (Fronczek 1977) provide further information.

Bases

The effective accuracy of triangulation depends on the precision with which the various observations are made and on the shapes of the triangles and quadrilaterals or other figures of the network, technically the strength of figure. U.S. Coast and Geodetic Survey *Special Publication* 225, Manual of reconnaissance for triangulation (Mussetter 1951) discusses strength of figure.

Maintaining the strength of figure over long distances is no longer critical because EDMI now provides the means to measure base lines almost anywhere. In the past, suitable sites for base lines were not always readily available and the costs involved were substantial. Most base lines were considerably shorter than the length of the triangulation sides. Usually it was necessary to observe a small network —identified as a base expansion figure—to project the measured distance into the triangulation. The locations of the stations included in these expansion figures had to be selected carefully to ensure the least degradation as the measured length was expanded through a series of triangles to the main triangulation. In some instances, the ratio of the triangulation line to the base was 10 to 1. However, numerous electro-optical measurements have shown that the lengths of the triangulation determined from this type of expansion net are seldom worse than 1:100,000 and often considerably better for modern first-order triangulation.

The history of base measurements in the United States contains numerous descriptions of ingeniously devised apparatus contrived for carrying out measurements. For the initial first-order base, which was measured on Fire Island, N.Y. in 1834, four iron bars were employed, each 2 m in length and mounted end-to-end with unique accessory equipment. This 14.1-km base was incorporated in a modern, multiple base-line, first-order network with no undue constraints. Later, base lines were measured with apparatus that consisted of bars or tubes of different metals having unequal coefficients of thermal expansion. These were arranged so that (1) the temperature effects on the lengths of the component parts were largely neutralized, or (2) the component parts formed a metallic thermometer, from which the temperature of the apparatus was determined and the length corrected accordingly. A very precise and extremely accurate measurement device was the iced bar apparatus. It consisted of a steel bar maintained at a constant temperature by surrounding the instrument in melting ice. This type of equipment was used in laboratories to standardize tapes and, to a limited extent, to establish field comparators.

Around 1900, steel tapes were used for measuring base lines, but their erratic behavior required that the measurements be made at night or under cloudy conditions. Numerous observations were necessary to obtain satisfactory results. In 1906, the C&GS obtained a set of invar tapes, constructed of nickel-steel alloy which had a very low coefficient of expansion. Tapes of this type were used until 1956 to measure all first- and second-order bases in the national net; some are still being employed for special purposes. During this period, most bases were measured along railroads or highways. Special supports were required to hold the tape above the rail to avoid errors caused by friction. Along highways, heavy stakes were driven into the ground to support the tape. A uniform tension of 15 kg (33 lb) was applied to the tape by means of carefully tested spring balances. The temperature of the tape was read and recorded for each tape length. Three or more tapes, each of which had been carefully standardized at the National Bureau of Standards, were used on each base. For measurement purposes, base lines were usually subdivided into sections of 1 km in length. Rarely did a base contain the exact number of kilometer sections; the overall length normally included a shorter segment. In carrying out the measurements, this segment was treated the same as the sections. Each section was measured using one tape in the forward direction and another tape in the backward direction. By selective scheduling, each tape measured an equal portion of the base. If the agreement between the two measurements of each section of the base did not agree within a certain specified limit, additional measurements were made. U.S. Coast and Geodetic Survey *Special Publication* 247, Manual of geodetic triangulation (Gossett 1959) contains instructions for base measurements. Today all bases that provide scale to the national net are measured with electro-optical instruments. Table 1 shows the accuracies required for different orders of triangulation.

Laplace Azimuths

In spite of the high precision with which the angles of triangulation are measured, there is a tendency for arcs of triangulation to swerve from their true orientation. The same condition exists in traverses and arcs of trilateration. In fact, the sway is usually more pronounced in traverse. This swerving could readily be corrected by making astronomical azimuth determinations at suitable intervals and including these azimuths in the adjustment of the observations if it were not for the effect of deflections of the vertical. We ordinarily think of the plumb line as pointing toward the center of the Earth, or at least toward the axis of the Earth. This is not exactly the case. Large mountains or other topographic features exert a sidewise attraction on the plumb lines or on the level vials of instruments. This may affect the astronomical observations by several seconds of arc, in fact, by 20″ or 30″ (seconds of arc) for extreme cases in the conterminous United States. Deflections on the order of 1′ (minute of arc) are found in Hawaii and Puerto Rico.

Uncorrected astronomical azimuths, therefore, cannot be used as true azimuths. Fortunately, there is a method to determine accurately the effect of the deflection of the vertical on astronomical azimuth observations at a station. If the astronomical longitude is determined at such a station and compared with the geodetic longitude, or the longitude carried through the network, the deflection of the vertical in an east-west direction becomes known within a small fraction of a second. This value for the deflection of the vertical can then be used to correct the astronomical azimuth observations and thus an azimuth can be obtained which is used as an observed quantity in the adjustment of all the observations. This corrected azimuth is known as a Laplace azimuth and the station is called a Laplace station.

Longitude and azimuth observations are made at specified intervals. Theoretically, because the astronomic latitude is required in order to compute the azimuth and to define the deflection completely, the latitude is also observed. Experience has shown that a spacing of about six or eight quadrilaterals between Laplace stations will provide sufficient azimuth control for first-order triangulation and trilateration. More frequent spacing is needed for traverses of the same accuracy class. Tables 1 and 2 show the accuracy specified for Laplace azimuth observations with a limiting standard error of 0.45″ for first- and second-order control surveys.

Other Methods for Establishing Control

In 1971, technological advances made it possible to determine accurate geodetic positions using signals emitted by satellites of the U.S. Navy Navigation Satellite System (NNSS). These positions can be observed anywhere, under almost any atmospheric conditions, by measuring Doppler shifts of the satellites' transmissions. No established control is required because the positions are based on a worldwide reference system. Further comments are found in the section entitled New Adjustment of the North American Datum. *Specifications to Support Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys,* (Federal Geodetic Control Committee 1980) contains specifications for Doppler surveys.

Inertial surveying systems (ISS) are now rapidly furnishing positions, deflections of the vertical, and gravity data. While Doppler receiving instruments still require (as of 1980) a solid terrestrial base, ISS equipment is adaptable to land and air vehicles. For best results, ISS require control that is spaced at 50–70 km, but this restriction is almost certain to be modified as instrumentation and techniques improve.

High-precision photogrammetric surveys (HPPS) position the points directly where terrain permits systematic targeting and where control is available at about 35–km intervals. It is expected that improvements in techniques and equipment will eventually permit more widely spaced control.

An adaptation of the interferometric phenomena, known as very long base line interferometry (VLBI), which employs radio waves from quasars, provides accurate azimuths and distances of extremely high precision (several centimeters). Because very long lines can be observed (thousands of kilometers), this method is particularly useful in crustal motion studies between distant locations. A variation of this technique will likely be used when advanced satellite systems are in operation for directly positioning the points.

STANDARDS OF ACCURACY

Control surveys are classified according to the accuracy of the resulting lengths and azimuths of the lines. Because the absolute errors of these quantities cannot be ascertained, indirect gages must be used.

Table 1.—Classification, standards of accuracy, and general specifications for triangulation

Classification	First-Order	Second-Order		Third-Order	
		Class I	Class II	Class I	Class II
Strength of figure					
R_1 between bases					
Desirable limit	20	60	80	100	125
Maximum limit	25	80	120	130	175
Single figure					
Desirable limit					
R_1	5	10	15	25	25
R_2	10	30	70	80	120
Maximum limit					
R_1	10	25	25	40	50
R_2	15	60	100	120	170
Base measurement					
Standard error	1 part in 1,000,000	1 part in 900,000	1 part in 800,000	1 part in 500,000	1 part in 250,000
Horizontal directions					
Instrument	0".2	0".2	0".2 or 1".0	1".0	1".0
Number of positions	16	16	8 or 12	4	2
Rejection limit from mean	4"	4"	5" or 5"	5"	5"
Triangle closure					
Average not to exceed	1".0	1".2	2".0	3".0	5".0
Maximum seldom to exceed	3".0	3".0	5".0	5".0	10".0
Side checks					
In side equation test, average correction to direction not to exceed	0".3	0".4	0".6	0".8	2"
Astro azimuths					
Spacing-figures	6-8	6-10	8-10	10-12	12-15
No. of obs./night	16	16	16	8	4
No. of nights	2	2	1	1	1
Standard error	0".45	0".45	0".6	0".8	3".0
Vertical angle observations					
Number of and spread between observations	3 D/R—10"	3 D/R—10"	2 D/R—10"	2 D/R—10"	2 D/R—20"
Number of figures between known elevations	4-6	6-8	8-10	10-15	15-20
Closure in length (also position when applicable) after angle and side conditions have been satisfied, should not exceed	1 part in 100,000	1 part in 50,000	1 part in 20,000	1 part in 10,000	1 part in 5,000

Table 2.—Classification, standards of accuracy, and general specifications for trilateration and traverse

TRILATERATION

Classification	*First-Order*	*Second-Order*		*Third-Order*	
		Class I	*Class II*	*Class I*	*Class II*
Geometric configuration					
Minimum angle contained within, not less than	25°	25°	20°	20°	15°
Length measurement					
Standard error	1 part in 1,000,000	1 part in 750,000	1 part in 450,000	1 part in 250,000	1 part in 150,000
Vertical angle observations					
Number of and spread between observations	3 D/R—10″	3 D/R—10″	2 D/R—10″	2 D/R—10″	2 D/R—20″
Number of figures between known elevations	4-6	6-8	8-10	10-15	15-20
Astro azimuths					
Spacing-figures	6-8	6-10	8-10	10-12	12-15
No. of obs./night	16	16	16	8	4
No. of nights	2	2	1	1	1
Standard error	0″.45	0″.45	0″.6	0″.8	3″.0
Closure in position					
after geometric conditions have been satisfied should not exceed	1 part in 100,000	1 part in 50,000	1 part in 20,000	1 part in 10,000	1 part in 5,000

Table 2.—Con.

TRAVERSE

Classification	*First-Order*	*Second-Order* *Class I*	*Second-Order* *Class II*	*Third-Order* *Class I*	*Third-Order* *Class II*
Horizontal directions or angles					
Instrument	0″.2	0″.2 or 1″.0	0″.2 or 1″.0	1″.0	1″.0
Number of observations	16	8 or 12	6 or 8	4	2
Rejection limit from mean	4″	4″ or 5″	4″ or 5″	5″	5″
Length measurements					
Standard error	1 part in 600,000	1 part in 300,000	1 part in 120,000	1 part in 60,000	1 part in 30,000
Reciprocal vertical angle observations					
Number of and spread between observations	3 D/R—10″	3 D/R—10″	2 D/R—10″	2 D/R—10″	2 D/R—20″
Number of stations between known elevations	4-6	6-8	8-10	10-15	15-20
Astro azimuths					
Number of courses between azimuth checks	5-6	10-12	15-20	20-25	30-40
No. of obs./night	16	16	12	8	4
No. of nights	2	2	1	1	1
Standard error	0″.45	0″.45	1″.5	3″.0	8″.0
Azimuth closure at azimuth check point not to exceed	1″.0 per station or 2″ $\sqrt{N}$	1″.5 per station or 3″ $\sqrt{N}$	2″.0 per station or 6″ $\sqrt{N}$	3″.0 per station or 10″ $\sqrt{N}$	8″ per station or 30″ $\sqrt{N}$
Position closure after azimuth adjustment	0.04m $\sqrt{K}$ or 1:100,000	0.08m $\sqrt{K}$ or 1:50,000	0.2m $\sqrt{K}$ or 1:20,000	0.4m $\sqrt{K}$ or 1:10,000	0.8m $\sqrt{K}$ or 1:5,000

For triangulation, the principal criterion is whether the discrepancy between a measured base and its length, as computed through the scheme from the next preceding base, is less than a certain fraction of the length of the base itself. In a rigorous sense, this discrepancy is derived after the side and angle conditions through the net between the two bases are first satisfied by a minimum constrained adjustment. However, in practice this is only done when the field computations indicate marginal results. Table 1 contains the limiting values for the ratio of this discrepancy to the length of the base for the different orders of triangulation. For trilateration and traverse, a similar criterion for the closure in position is specified. These criteria are at about the 2 sigma (2σ) or 95-percent confidence level.

Another important indirect gage of the accuracy of the final results of triangulation is the average closure of the triangles. After allowance has been made for spherical excess; i.e., for the slight increase in the angles caused by the spherical shape of the Earth, the sum of the three angles of a triangle should be exactly 180°. The permissible variations from 180° for the different orders of triangulation are given in table 1, along with other specifications governing the accuracy of horizontal surveys.

Requirements for Horizontal Control

Tables 1 and 2 contain the basic Federal standards for horizontal control. The two FGCC publications, *Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys* (Federal Geodetic Control Committee 1974) and *Specifications to Support Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys* (Federal Geodetic Control Committee 1980) provide further information.

FIELD AND OFFICE COMPUTATIONS AND ADJUSTMENTS

During earlier periods in the extension of horizontal control surveys, only limited computations were carried out in the field. Those computations verified that the observations met the specifications for triangle closures and side and length checks. All calculations were performed by hand, employing logarithms to carry the lengths through the triangulation. It was a cumbersome process. Progress was slow because lengthy observation programs were required to secure accurate observations. Other contributing factors were the primitive means of transportation and an equally primitive road system. During a typical field season, only two or three figures of the primary net would be completed.

With the introduction of mechanical calculators complete computations of the field observations became possible, including geographic positions. Lists of field positions were distributed to users upon request. These positions could be used in the interim period prior to the adjustment and publication of the data, which was delayed sometimes for long periods. Logarithms continued to be the principal means for computing the triangle sides and geographic positions until about 1945. Then a gradual trend began towards using natural functions. By 1960, most field computers limited the use of logarithms to the calculation of side equations to isolate observational errors. This procedure was required when evaluations could not be completed by examining the common triangle sides.

In 1960, the amount of necessary field computations increased appreciably as numerous length measurements were secured in many projects and the observations of astronomic azimuths became commonplace. The work load on many field parties was so heavy that often two computers were required to meet the needs. Desktop programmable calculators were introduced around 1966, and many computations which had previously taken hours were now completed in minutes. Programs for these calculators were placed on magnetic cards with the results tabulated on tape. Programmable calculators reduced the effort involved, but the amount of field data continued to increase. The task of completing all the field computations was formidable. Often several months were required to prepare the data for office use.

In 1975, NGS installed portable terminals for field party use. This enables direct communication with centrally located computers via telephone lines. Through these terminals field parties can store data for an unlimited time, including results of the computations. Sequential least-squares adjustments, used to evaluate segments of the net, can be made as the work progresses. When necessary, the office can access the information without contacting the field unit. Finally, the data are placed in the prescribed office format and the adjustments (involving all or part of the observations) are made while the field party is still on the project site. Figure 6 shows a typical arrangement.

Two primary objectives must be met by the office processing of the field observations. First, the data must be made consistent throughout and must

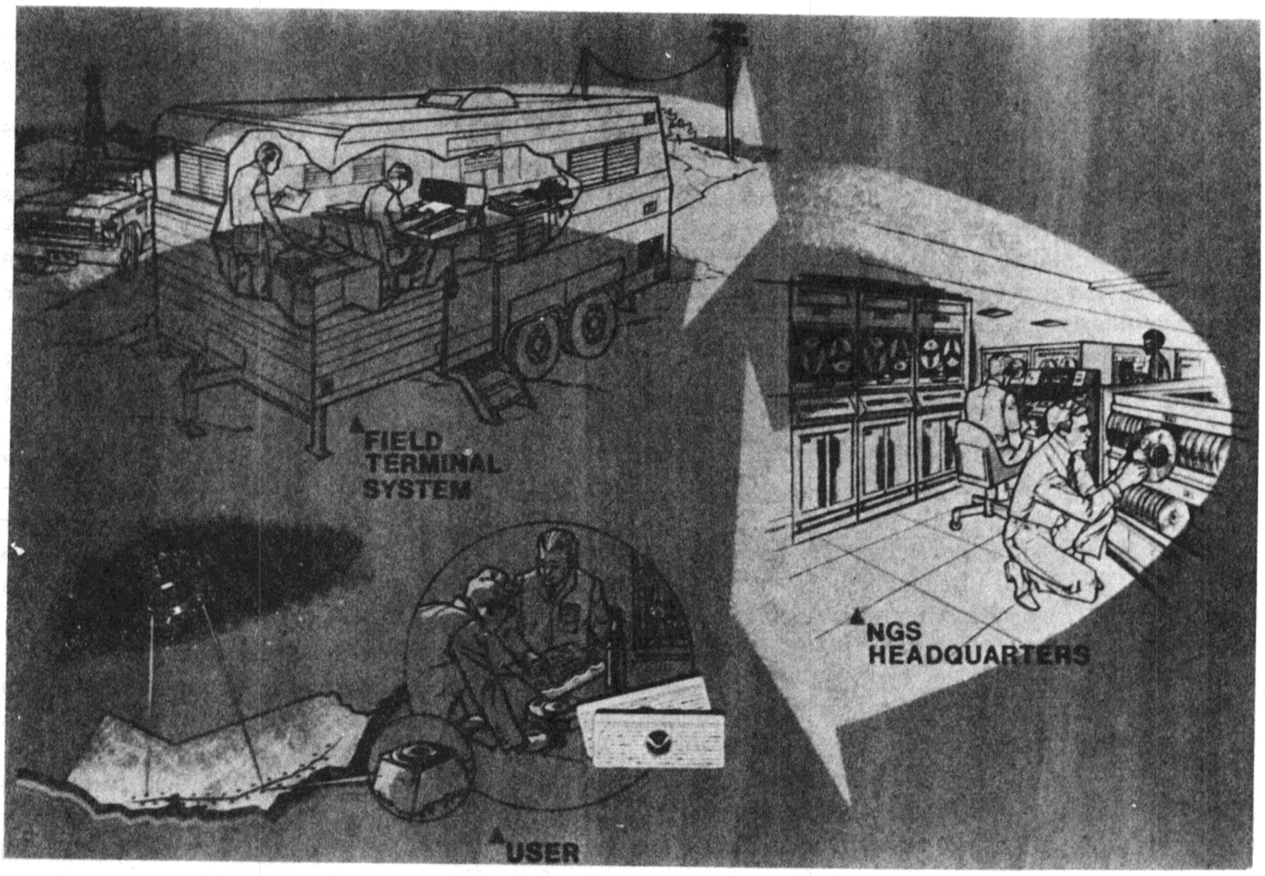


Figure 6.—Remote terminal system used by horizontal control field parties

be fitted to the existing adjusted control. Second, the data must be placed in a convenient form for use by surveyors and engineers.

The first objective is accomplished by means of special computations, known as least-squares adjustments. These adjustments derive the smallest possible corrections to the observations to produce the following results: (1) The sum of the three angles of each triangle equals 180° plus the spherical excess. (2) The lengths involved in each quadrilateral, or more complex figures, are consistent with the angles entering in the computation of these lengths. (3) The network is consistent in length, azimuth, and latitude and longitude with the previously adjusted control to which it is joined. (4) Finally, the net is consistent with measured bases and Laplace azimuths included in the adjustment.

For more than a hundred years, special equations were required to account for these conditions. These equations were then solved simultaneously. This procedure is called the method of conditions. U.S. Coast and Geodetic Survey *Special Publication* 138 (Reynolds 1934) contains additional information. Although this publication describes computational procedures employed prior to the development of present-day computers, *Special Publication* 138 remains an excellent reference for understanding the method-of-condition equations.

With the introduction of electronic computers, the method-of-condition equation was discontinued because the automatic formation of the equations was, and still is, difficult to handle on a computer. In its place, the method of variation of coordinates was adopted almost universally because of its ease in forming the observation equations. Here, too, the equations are solved in a simultaneous fashion by similar procedures used for condition equations. When a network is adjusted by the method-of-condition equations, the geographic positions are determined by applying corrections resulting from the adjustment to the observed angles of the triangles, solving these triangles, and finally computing the geographic positions from the adjusted angles and sides. In the method of variation of coordinates, the adjusted geographic positions are obtained immediately following the solution of the equations, because the results are actually corrections to the assumed positions which were used in the formation of the observation equations. Although the residuals or corrections to the observations are computed, the triangles are rarely calculated. Lengths and azimuths between stations are derived from inverse computations using the adjusted geographic coordinates. The method of variation of coordinates provides the means for easily computing accuracy estimates for the positions, lengths, and azimuths. These accuracies are much more difficult to obtain from condition equations. U.S. Coast and Geodetic Survey *Special Publication* 28, Application of the theory of least squares to the adjustment of triangulation (Adams 1915), details the development of the basic equations used in adjustments by the method of variation of coordinates.

After the adjustment is completed, the final essential data for each station must be compiled and placed in a form readily available for use by surveyors, engineers, and others. The following information is tabulated in an easy understandable format: the geographic position (latitude and longitude) with the corresponding plane (x and y) coordinates of each station on the State plane coordinate system in use for that particular area including, as a general rule, the geodetic and grid azimuth of at least one line radiating from the station with description and recovery notes. The latest listings include the State plane coordinates in feet and meters, and universal transverse Mercator (UTM) coordinates (fig. 7).

Descriptive information for established stations is now being placed in computer-readable form, and descriptions for stations positioned in new surveys are received in the standard computer-readable format. Soon all relevant data will be completely computerized and available to the user in the usual manner, or by access to the NGS data base via onsite terminals.

Before describing in more detail some of the operations involved in the computations, a brief review of the national control network and datums used in the United States seems appropriate.

HISTORY OF THE NATIONAL HORIZONTAL CONTROL NET

The horizontal control surveys of the United States were begun during the early part of the nineteenth century at a number of points mostly along the coasts. These existed first as separate surveys, each based on one or more astronomical determinations of latitude, longitude, and azimuth. Examples of such detached surveys are the early triangulation in New England and along the Atlantic coast, a portion of the transcontinental triangulation along the 39° parallel in the vicinity of St. Louis, Mo., the same arc in the Rocky Mountain region, and three separate surveys in California near San Francisco, Santa Barbara Channel, and San Diego. These separate pieces of triangulation were later extended until several of them touched or overlapped. Finally, the transcontinental arc was completed and joined all of these detached surveys into one conterminous triangulation.

With the completion of the transcontinental arc around 1900, it was possible to compute the net as a single coordinated survey and replace the previous independent systems which, of course, did not fit together properly at the junctions. The recomputation of all triangulation that had been completed up to that time would have been a fairly heavy piece of work. Considerable thought was given to devising the best method possible in order to adopt a datum that could be held fixed for a long time into the future. After careful study, it was decided to extend the datum that had been used in New England and along the Atlantic coast between 1880 and 1901 through the entire network. This decision avoided much recomputation and, at the same time, gave an almost ideal datum for the Nation.

The New England Datum had as it origin the geographic position for station PRINCIPIO in Maryland and was oriented by the geodetic azimuth from PRINCIPIO to TURKEY POINT. Computations were made on the Clarke spheroid of 1866. The geographic position for PRINCIPIO and the geodetic azimuth to TURKEY POINT were determined by relating all astronomic latitudes, longitudes, and azimuths observed in the eastern triangulation to the station. With the adoption of the United States Standard Datum in 1901, the geographic position for MEADES RANCH, in Kansas, and the azimuth to WALDO, as computed through the triangulation from PRINCIPIO, defined the origin.

Standard Datums

Before horizontal control can be computed over a large area, two fundamental factors must be known or determined: (1) the exact shape of the mathematical figure, which is a close approximation to the sea-level surface of the Earth, and (2) the latitude and longitude of one station of the net together with the azimuth of the line to one of the adjoining stations. This is the historical definition of a datum.

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US DEPARTMENT OF COMMERCE - NOAA          HORIZONTAL CONTROL DATA              PAGE 029
NOS - NATIONAL GEODETIC SURVEY           NORTH AMERICAN DATUM 1927      QUAD N25080123  QSN 0023
ROCKVILLE MD 20852 - JUL 1980        PROJECT ACCESSION NUMBER 14831     CONTROL DIAGRAM NG 17-8
                                                                        FL-DADE COUNTY

HORIZONTAL CONTROL STATION:  SOLDIER KEY 4
GEODETIC POSITION DATA-----------------------     STATION INFORMATION--------------------------------------------
                      DEG MIN SECONDS                  TYPE:  1ST-ORDER TRIANGULATION
          LATITUDE:    25 35 24.62246N                 OBSERVATIONS BY NATIONAL GEODETIC SURVEY IN 1972
          LONGITUDE:   80 09 41.40569W                 ADJUSTED BY NATIONAL GEODETIC SURVEY IN 1979
          AZIMUTH 1:   16 09 19.7  FROM SOUTH          AZIMUTH REFERENCE OBJECT 1:  SANDS POINT USE RM1 1972

          ELEVATION:       1.0  METERS                 ELEVATION DETERMINED BY TRIGONOMETRIC LEVELING
                           3.3 FEET
          GEOID HEIGHT:   10.1 METERS

STATE PLANE AND UNIVERSAL TRANSVERSE MERCATOR COORDINATE SYSTEMS------------------------------------------------------
                  X           Y        X/EASTING   Y/NORTHING   POINT SCALE  CONVERGENCE  GRID AZIMUTH 1  GRID AZIMUTH 2
 GRID  ZONE     FEET        FEET        METERS       METERS       FACTOR     DEG MIN SEC   DEG MIN SEC     DEG MIN SEC
FL-TM  E     776364.67   457594.23   236636.424   139475.000   1.00002877    +0 21 43.9    15 47 35.7*
 UTM   17                            584207.385  2830404.835   0.99968756    +0 21 43.9   195 47 35.7*
 UTM   18                            -18782.210  2840249.930   1.00292510    -2 14 03.9   198 23 23.5*

                                                                      *CAUTION - ARC-TO-CHORD CORRECTION ASSUMED ZERO

STATION DESCRIPTION---------------------------------------------------------------------------------------------------
ORGANIZATIONS MARK:  NATIONAL GEODETIC SURVEY                                  HEIGHT OF TELESCOPE:     1 METERS
YEAR DESCRIBED:  1972    CHIEF OF PARTY:  HLM    REACHED BY:  BOAT             PACK TIME:  00 HRS 00 MIN

CODE-- MARK-------- TYPE OF MARK----------------- SETTING/LANDMARK TYPE---------------------------- MAGNETIC PROPERTY
     SURFACE       TRIANG STA DISK        SEE DESCRIPTION                                              UNKNOWN
D55  REFERENCE     SURVEY DISK            SET INTO DRILL HOLE IN ROCK OUTCROP
D00  REFERENCE     SURVEY DISK            SEE DESCRIPTION
L23  REFERENCE     LANDMARK               VOR ANTENNA
L55  REFERENCE     LANDMARK               TOWER

  CODE------ REFERENCE OBJECT------- HEADING------------ DISTANCE------------ DIRECTION------- MAGNETIC PROPERTY
  D55    SANDS POINT USE RM 1                                                  000 00 00.0       UNKNOWN
  D00    SOLDIER KEY 4 RM 4           S              21.46 FEET   6.541 MTRS   004 56 05.0       UNKNOWN
  D00    SOLDIER KEY 4 RM 5           W              30.20 FEET   9.203 MTRS   097 07 36         UNKNOWN
  L23    BISCAYNE BAY VORTAC BSY      N    ESTIM APPROX 5.25 MI                153 34 10.0       UNKNOWN
  L55    CAPE FLORIDA OLD TOWER FINIAL N   ESTIM APPROX 4.75 MI                167 27 46.7       UNKNOWN
            SOLDIER KEY 4 RM 4
         TO SOLDIER KEY 4 RM 5                       37.70 FEET  11.493 MTRS

STATION IS ABOUT 12 MILES EAST OF PERRINE, 12 MILES SOUTHEAST OF MIAMI, ON SOLDIER KEY ISLAND WHICH IS THE FIRST ISLAND
SOUTH AND ABOUT 4-3/4 MILES FROM THE SOUTH END OF KEY BISCAYNE ISLAND (CAPE FLORIDA STATE PARK).

TO REACH THE STATION BY BOAT FROM THE HOMESTEAD BAYFRONT PARK ABOUT 9 MILES EAST OF HOMESTEAD, GO NORTHEAST FOR ABOUT 13
MILES ON AN AZIMUTH OF 53 DEGREES FROM TRUE NORTH TO A CONCRETE BOAT DOCK ON THE NORTHWEST SIDE OF SOLDIER KEY ISLAND
AND THE STATION AS DESCRIBED.

STATION MARK, STAMPED SOLDIER KEY 4 1972, IS A STANDARD DISK CEMENTED IN A DRILL HOLE IN THE NORTHEAST CORNER OF THE
BOAT DOCK.  IT IS 107 FEET WEST OF THE SOUTHWEST CONCRETE PILLAR OF AN OLD HOUSE FOUNDATION, 44 FEET NORTH OF A 12-INCH
PINE TREE, 23.7 FEET NORTH OF THE OUTSIDE NORTHEAST CORNER AND 3.2 FEET SOUTHWEST OF THE OUTSIDE NORTHEAST CORNER.

                                           (CONTINUED ON NEXT PAGE)
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Figure 7.—A computer-generated data sheet

Most modern datums, including the one that will be used for the new adjustment of the North American Datum, will be oriented by means of satellite position determinations with local orientation provided by astronomic azimuths. This will be discussed in detail later. However, for continuity, an explanation of the determination of earlier datums follows.

The mathematical figure of the Earth, technically referred to as a spheroid or an ellipsoid, is derived by comparing the differences between astronomical determinations with the corresponding distances measured on the Earth and through gravity observations. The astronomical determinations are particularly useful in deriving the size of the Earth and the gravity values to obtain its exact shape or the amount of flattening at the poles. The precise solution of this problem requires tedious work and difficult mathematics. (See U.S. Coast and Geodetic Survey *Special Publication* 82, The figure of the Earth and isostasy from measurements in the United States (Hayford 1909).)

Since the beginning of the nineteenth century, the dimensions of the ellipsoid (spheroid) have been computed at least 20 different times with considerable accuracy. The early computations were based on comparatively small amounts of data and naturally did not represent the true shape and size of the Earth quite as accurately as later computations. In 1924, at the triennial meeting of the International Geodetic Association, held in Madrid, Spain, an international ellipsoid was adopted for use by all member countries of the Association that might be in a position to adopt a new datum or to recompute their triangulation nets. The international ellipsoid is based on dimensions derived in 1909 by John F. Hayford of the former C&GS.

In 1880, C&GS adopted the Clarke spheroid (ellipsoid) of 1866 as a basis for its triangulation computations. By the time the international ellipsoid was adopted by the International Geodetic Association, thousands of stations in the United States were based on the Clarke spheroid, and numerous computation tables had been computed and published on this spheroid. The work involved in changing to a new ellipsoid would have been very great. Because the ellipsoid already in use differed only slightly from the new one, it was decided that no change would be made. The principal dimensions of these two ellipsoids are given in table 3. The ellipsoid dimensions that have been adopted for the new adjustment of the North American Datum are also included.

Another prerequisite for computing the horizontal control for a country is the adoption of the most nearly ideal position for a starting station—a matter of great importance and considerable difficulty. As explained earlier in the section on Laplace azimuths, astronomical determinations are seriously affected by the sidewise attraction of topographic features (deflections of the vertical) and, therefore, cannot be depended upon to give the best position for any single station. The ideal starting position is one that would make the average algebraic difference between the resulting geodetic and astronomical positions of identical stations approximately zero for the whole country.

The first datum adopted for the entire United States was known as the United States Standard Datum. It was based on the Clarke spheroid of 1866 and had as its basic station a point in Kansas known as MEADES RANCH. The following position was adopted for this station:

$$\begin{aligned} \text{Latitude} &= 39^\circ\ 13'\ 26''.686 \\ \text{Longitude} &= 98^\circ\ 32'\ 30''.506 \\ \text{Azimuth to station WALDO} &= 75^\circ\ 28'\ 14''.52 \end{aligned}$$

Although the azimuth to station WALDO was included in the fundamental data for MEADES RANCH, this azimuth is now of secondary importance because the azimuths in the net are controlled by the many Laplace azimuths scattered throughout the network. The adopted position of this fundamental point was tested by comparing all available astronomical positions connected to the net with the corresponding geodetic positions. It was found that the average algebraic residuals in both latitude and longitude were very nearly zero.

In 1913, the same datum was adopted by Canada and Mexico. This was a very important step in international cooperation because almost all of North America was then on a single geodetic datum, allowing triangulation surveys to be coördinated for practically the entire continent. In recognition of its new continental extent, the name of the datum was changed to the North American Datum. At that time, such an ideal arrangement had never been accomplished anywhere else in the world. Later, following World War II, much of Europe and parts of Africa were adjusted on a common datum.

North American Datum of 1927

In 1927, the national control network was recomputed. Although the Clarke spheroid and the position originally adopted for MEADES RANCH were still satisfactory, the adjustment of the net was far from ideal. The Nation's triangulation net had been built up by continually adding new arcs to those already measured, and each new arc was adjusted by making it fit the existing net which had been previously adjusted. This method forced the entry of distortions into the new arcs which should have been distributed through loops including both the new and the old arcs. However, this was the only feasible method that could be used until completion of the main framework of the national network, because it would have been impracticable to readjust the entire net or even a large portion each time a new arc was added.

In 1927, the western half of the control network (extending from the 98° meridian to the Pacific coast) had been completed to the extent that a complete new adjustment could be made. This adjustment would later provide a control framework of such high accuracy as to permit new arcs to be added without undue distortion. It was very desirable to undertake this new adjustment before the net became more complex because of the great amount of computation involved. A special method for making the adjustment was suggested by William Bowie and solved mathematically by O. S. Adams. U.S. Coast and Geodetic Survey *Special Publication* 159, The Bowie method of triangulation adjustment as applied to the first-order net in the western part of the United States (Adams 1930), explains the procedure. By using junction figures the adjustment was divided into a number of different parts that could be carried out simultaneously, which allowed the work to be completed within a reasonable time.

A short time after the western half of the national net had been readjusted in this manner, the eastern half was recomputed in the same way. In the meantime the field work for the main framework of the eastern part of the country had also been completed. The two adjustments resulted in a very accurate net for the entire country.

The reader may wonder why the name of the datum was changed from the North American Datum to the North American Datum of 1927 when no changes were made in the dimensions of the spheroid or in the position of the initial station. The principal reason was to guard against confusing the readjusted data with the old data. MEADES RANCH was the only station held fixed in position in the readjustment. All other stations in the net were changed in position. The changes were small in the vicinity of MEADES RANCH but were fairly large at greater distances. In the State of Washington, for example, the change in position was slightly over 1″ in latitude and nearly 1″.4 in longitude. The new name for the datum indicated new positions for the stations of the net rather than changes in the fundamental properties of the datum, except that the azimuth from MEADES RANCH to WALDO was changed about 5″. This change was caused by the introduction of a nearby LaPlace azimuth in the computation that had not been previously used. The original azimuth defining the datum was carried almost 1500 miles through the triangulation from Maryland.

After the completion of the adjustment of the basic net in 1932, few major difficulties were encountered in fitting new surveys to the adjusted control until 1952. Rather large readjustments were necessary in Connecticut, the Hudson River Valley in New York, South Carolina, North Dakota, and South Dakota. In general, new work could be adjusted to the published positions without introducing serious distortions. However, by 1955, as more and more new triangulation was added to the national net, it became increasingly evident that a new adjustment of the entire framework would be required. Fortunately, experience gained in the period following the last adjustment dictated a waiting period for several reasons: The missile age had commenced. It was obvious that higher accuracy surveys would be mandatory and EDMI, which had the best potential for accomplishing these surveys, was being perfected. To meet the requirement for a continental framework of superior accuracy, the high-precision traverses were designed. Initial work began in Florida on February 23, 1960. The network was completed in Michigan on November 15, 1976, and is more than 22,000 km in length. Further remarks concerning this survey, which has an estimated accuracy of 1:1,000,000, are given in the section titled New Adjustment of the North American Datum. Skyrocketing costs will probably deter any country from attempting in the future to observe a survey of this length to the accuracy obtained in the high-precision traverses.

EXPLANATION OF HORIZONTAL CONTROL DATA

The National Geodetic Survey computes the final results of its horizontal control surveys on two different systems of coordinates, one based on geographic latitudes and longitudes, and the other on plane rectangular coordinates. A major advantage of geographic coordinates is that they constitute a universal system for the entire world and any point located in this system is related to any other point in the system.

For limited surveys, rectangular coordinates can be used in place of geographic coordinates. The main advantage of rectangular or grid coordinates is their simplicity. These coordinates are computed on a plane surface and expressed as x and y values, which are familiar to most surveyors and engineers with little or no experience in control surveys. Where UTM coordinates are provided, the quantities are identified as northings (N) and eastings (E).

Geographic Positions

The horizontal control data published by the National Geodetic Survey are first computed as geographic positions, latitudes, and longitudes. Latitudes are referred to the equator of the ellipsoid and longitudes to the meridian of Greenwich. For many years these data were tabulated in listings that contained, in addition to geographic positions, azimuths and lengths to all stations involved in a particular adjustment. As a convenience to the user, the lengths between the stations were given in meters and feet. In many instances the logarithms of the distances in meters were also provided. Two azimuths were given for each line: one identified as the forward azimuth and the other the back azimuth. The forward and backward azimuths of geodetic lines do not differ by exactly 180° because of the convergence of the meridians passing through the two stations. The forward and back geographic azimuths are exactly 180° apart only for stations on the equator or for stations on the same meridian. Azimuths are given in a clockwise direction with reference to true south. South is 0° (or 360°), west is 90°, north is 180°, and east is 270°. Plans are being made to reference azimuths clockwise from the north when the new adjustment is completed.

In the late 1950's, all data for stations of the national net were gathered and listings, called data sheets, were prepared. The data sheets were assembled by latitude and longitude in 30′ blocks or, where massive amounts of control were available, in 15′ blocks. These blocks are identified as quadrangles (quads). A unique numbering system for each station within a specific quad forms the basis for the NGS data base, which will be described later. Within a short time it became apparent that the cost of revising a data sheet simply to add additional azimuths and distances was prohibitive. Modifications produced a data sheet containing only those azimuths necessary to provide orientation at a station. Usually, this is the azimuth to the azimuth mark. Horizontal control data for most States have been converted to the data sheet quad format. Data sheets for Puerto Rico and the Virgin Islands, American Samoa, Guam, and other U.S. possessions will be prepared later after the data for the States are converted. As mentioned earlier, the new data sheet will include the State plane coordinates in both feet and meters, and UTM coordinates (fig. 7). Except for these additions, the new format is essentially the same as the previous version.

State Plane Coordinate Systems

The plane rectangular coordinates of horizontal control stations are derived from their geographic positions. Although surveyors and engineers who use these coordinates do not need to know exactly how they are computed, they may wish to understand the methods employed and how the size of the area included in each independent system is determined.

About 1932, a decision was made to compute plane coordinates from the geographic positions and to publish this information as part of the horizontal control data. A careful study was made by O.S. Adams to determine what types of projections would be most suitable as bases for plane coordinate systems, as well as the most desirable size for the area to be included in each independent zone. He found that the Lambert conformal conic projection was excellent for areas of greatest extent in an east-west direction and the transverse Mercator projection for areas of greatest extent in a north-south direction. Regarding the size of the area, the individual counties were considered to be too small, as their use would have involved frequent zone changes. Many States were too large to be considered as a single zone. The criterion finally adopted made the area such that the maximum scale error would rarely exceed 1 in 10,000. This meant that the larger States had to be divided into two or more zones with widths not greater than about 158 miles. The boundaries between zones were made to follow county boundaries so that only one zone would be needed in any one county. Liberal overlaps of the zones were provided in the computation of coordinates. Today, this policy is still followed by NGS and will be retained in the new adjustment.

Lambert Projection

Each Lambert zone is based on a cone that intersects the Earth along two standard parallels.

These parallels are selected so as to make the maximum scale errors between the parallels about equal to or a little less than the maximum errors outside them, the two being of opposite sign. That is, the compression of areas between the parallels is made about equal to the expansion outside (fig. 8).

Transverse Mercator Projection

The transverse Mercator projection is similar to the ordinary Mercator projection but is related to a central meridian instead of to the equator. The projection intersects the Earth along two lines parallel to the central meridian in order to balance out the scale errors, as in the Lambert projection. It, too, compresses the areas between these two lines and expands the areas outside as in the Lambert projection. Figure 9 illustrates how the transverse Mercator projection was used for the three plane coordinate zones in Idaho.

Combined Projection Systems

Two States, Florida and New York, use both the Lambert and the transverse Mercator systems. Northern Florida lends itself more readily to the Lambert projection as does Long Island, N.Y. The remaining areas of these two States use the transverse Mercator system. Michigan was originally on the transverse Mercator projection, but rather than have two zones in the southern portion of the State, which encompasses the largest population, a Lambert system was developed and adopted as the official State grid. Plane coordinates for both systems will continue to be published until the results of the new adjustment are available. At that time, the transverse Mercator projection will be dropped.

One State, Alaska, employs three projections. Southeast Alaska is covered by a single zone using a skew Mercator grid. The major land mass has eight zones on the transverse Mercator system, and the Aleutian Islands have a single zone on the Lambert projection. (For publications pertaining to the State plane coordinates for each State, see the following reference: U.S. Coast and Geodetic Survey—publication date varies for individual States.)

The State systems will be retained as the basic national grid following the new adjustment. None of the fundamental constants will be changed unless a request to do so is received from an individual State. The most precise formulas will be employed and the coordinates will be expressed in meters. The NGS will also publish UTM coordinates for all stations.

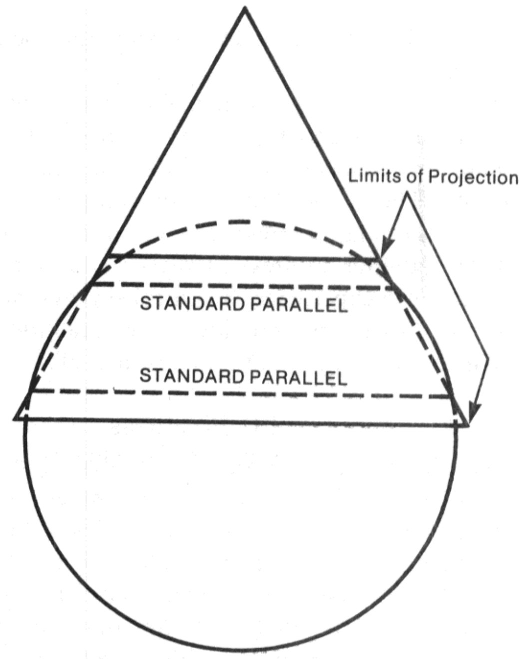

Figure 8.—Sphere and intersecting cone illustrating the Lambert conformal conic projection

Using the State Plane Coordinate Systems

Plane coordinates for stations established or for values published by the NGS are computed immediately after the adjustment is completed. To avoid negative values for the station coordinates, a constant is added to the x values derived by the computations.

The essential plane coordinate data for a station consist of the following items: the name of the State, the designation of the zone used as a basis for the computations, the x and y coordinates of the station in feet (latest listings also shown in meters), the mapping angle or the difference between geodetic and grid azimuths at each station, which is known as the θ or $\Delta\alpha$ angle, and finally the grid azimuth of at least one line radiating from the station. Other azimuths may be derived by direct computation simply by differencing the coordinates and using standard trigonometric formulas that are well known to surveyors and engineers. On those occasions when geodetic azimuths are available, the grid azimuths can be obtained by applying the mapping angle for the station of interest. Forward and back grid azimuths should differ by exactly 180°. However, when employing the mapping angle, there may be small differences, especially when the stations are located at some distance north or south from the central parallel for the Lambert projection and east

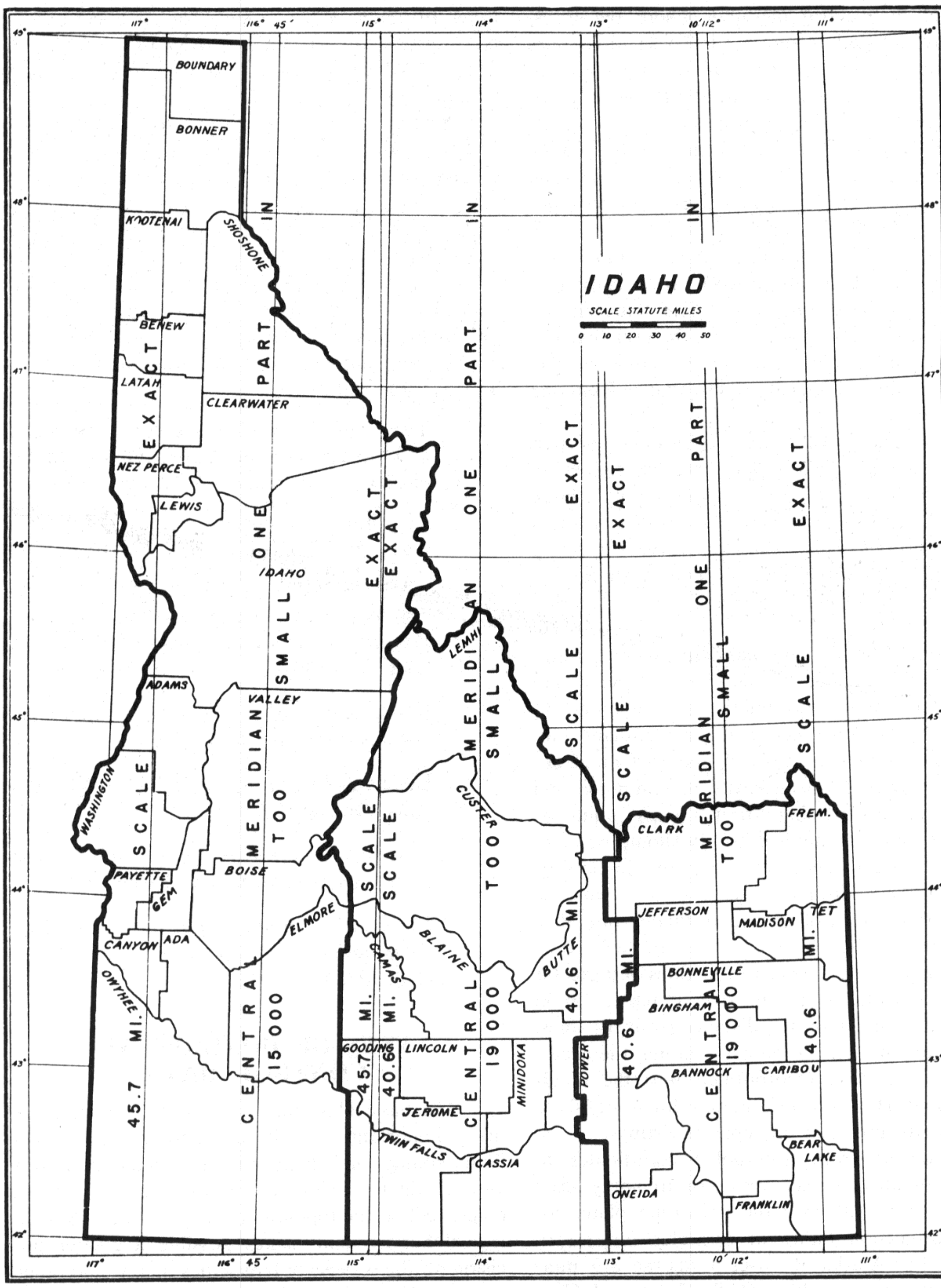

Figure 9.—Use of transverse Mercator projection for plane coordinate zones in Idaho

or west of the central meridian where the transverse Mercator grid is employed. These differences are identified as the second term or (t-T) corrections. They can be computed by the formulas given with the plane coordinate projection tables available for each State, Puerto Rico, the Virgin Islands, and American Samoa (U.S. Coast and Geodetic Survey). Acceptable approximations may be computed using simplified formulas given in *Technical Report* 36, Geodetic and grid angles—State coordinate systems (Simmons 1968). If a station is near the edge of a zone, coordinate data on the adjacent zone are also given so that the user may use whichever zone is most convenient and avoid the additional computations required to change from one zone to another.

One difficulty with plane coordinates is that the area covered by a single zone is limited in extent, because it is impossible to represent a large area of the Earth's surface on a plane surface without serious distortions. This means that surveys covering fairly large areas will sometime include stations in more than one plane coordinate zone. Another difficulty is that the grid azimuths at a station will not be referred to the true meridian except for stations located at the central meridian on which that particular State plane coordinate system is based. In other words, the convergence of the meridians cannot be incorporated in a rectangular system of coordinates. However, it can easily be computed and the true meridian passing through each point determined. One other minor difficulty should be mentioned. If optimum accuracy is desired in surveys based on these coordinates, scale factors must be used in certain areas of each coordinate zone. The scale factor is unity along two lines in each zone, but is less than unity between these lines and greater than unity outside them. The size of the factor in each case depends upon the distance from the lines.

For most stations in the national net, the necessary data have been incorporated into the quadrangle format. However, information for some island stations is still provided on separate listings.

To convert from geographic coordinates to plane coordinates, or vice versa, or to apply scale factors the user will need the individual plane coordinate projection tables or formulas for a specific State. For Alaska, the user needs the 2½ minute grid intersection tables (U.S. Coast and Geodetic Survey).

Electronic computers and calculators have eliminated many of the former tedious hand computations. Formulas and constants for all plane coordinate systems developed by NOS are included in ESSA *Publication* 62–4, State plane coordinates by automatic data processing (Claire 1973).

Several publications describing the plane coordinate systems are available. They explain how the systems were originally computed and how they may be used. It is not necessary for a surveyor or engineer to make a detailed study of these computations. Except for special problems, encountered only rarely, the x and y coordinates can be used without concern to the derivation of the values from the geographic positions. However, for users interested in the computational methods, the following publications are helpful:

The State coordinate systems, U.S. Coast and Geodetic Survey *Special Publication* 235 (Mitchell and Simmons 1977).

Conformal projections in geodesy, U.S. Coast and Geodetic Survey *Special Publication* 251 (Thomas 1968).

Geodetic and grid angles—State coordinate systems, ESSA C&GS *Technical Report* 36 (Simmons 1968).

Fundamentals of the State plane coordinate systems (Dracup 1973).

Understanding the State plane coordinate systems (Dracup 1977).

Horizontal control as applied to local surveying needs (Dracup and Kelley 1973).

Surveying instrumentation and coordinate computation workshop lecture notes (Dracup et al. 1973).

USES OF HORIZONTAL CONTROL DATA

The basic U.S. horizontal control network has many important uses. It serves to locate national, State, county, and many private boundaries. It provides a rigid framework for all types of accurate maps, such as the topographic maps of the U.S. Geological Survey, and makes possible a greater precision for the surveys of large cities than would otherwise be obtainable. These data coordinate into a single related system all the local surveys connected to the network and ensure the perpetuation of all marks established by such surveys. Information from the net is used to study crustal motion, and serves as an adjuvant in the earthquake prediction program and related investigations involving plate tectonics and continental drift. It strengthens the defense needs of the Nation by providing an accurate survey base for many military activities. The high-precision traverses are used as test ranges to evaluate new and very accurate survey systems.

The State plane coordinate systems were devised to enable engineers and surveyors to connect stations to the control net with great facility wherever stations of the net are within a reasonable distance from a proposed local survey or engineering project. Many States have legalized the definition of boundaries of private and public property in terms of plane coordinates on the zone specified for the particular area. The NGS retains and will continue to retain these definitions unless otherwise requested by the individual States. Examples of the manner in which a local survey can be connected to the control net are given in U.S. Coast and Geodetic Survey *Special Publication* 235, The State coordinate systems (Mitchell and Simmons 1977) as well as other publications listed in the references.

The utility of the national control net is increasing rapidly. In the formative years of this Nation, the cost of land was very cheap. Property owners were not greatly concerned with having their boundary lines precisely determined. Surveying costs would have been excessive in relation to the value of the land. In many areas compass surveys were considered adequate for the purpose. Today the picture is very different. Land values have risen almost everywhere. Inaccurate surveys cannot be tolerated. The property owner demands accurate boundary surveys, including land recordation procedures for future use and possible litigation. The most economical way to accomplish these results is to have the surveys based on stations of the national net. Accuracy and permanency are ensured. Frequently the surveys can be carried out in less time than by employing other procedures.

METRIC AND ENGLISH UNITS

Geodetic surveys performed by the NGS (including its predecessor agencies) have always used the meter as the unit of length. For the convenience of the user, many published lengths are also given in the more familiar English units.

Station descriptions usually give the distances to reference marks and other measured distances to nearby objects in meters and, then, in feet. If the distances were originally measured only in meters or feet, the converted values for the particular units are enclosed in parentheses. Other distances listed in a description, which may have been only roughly determined or (occasionally) estimated, are given only in the units used in the field descriptions. For example, a field description may state that a station is about 100 yards from a fence, or 35 paces from the corner of a barn. These distances are not converted to other systems of units. This practice is slowly being discontinued.

Although the meter was used as a standard of length in horizontal control surveys since 1816, no legal definition equated the meter and the foot until 1866 when U.S. Congress enacted legislation which stated that 1 meter equals exactly 39.37 inches. Since 1893, the United States' unit of length has been derived from the metric standard, as stated in the Act of 1866. In 1959, the definition for the length of a foot was changed from 1200/3937 meters to 0.3048 meter exactly, which shortened the old value by about 1:500,000. At the same time, however, any data derived from and published as a result of geodetic surveys within the United States continued to use the old standard. By this executive decision all land measurements are based on the earlier relationship of 1200/3937 meters and the unit is identified as the U.S. Survey Foot. This definition will continue in effect in the future, although length data published as the result of geodetic surveys will be in the International System of Units (SI), i.e., meters. Any surveyor or engineer who finds it necessary to convert land measurements from feet to meters, or vice versa, should use the ratio that defines the U.S. Survey Foot. To encourage public awareness, Congress enacted the Metric Conversion Act of 1975, which established a national policy of voluntary conversion to the metric system.

COOPERATION IN PRESERVING MARKS

Cooperation between the NGS and the public in establishing and maintaining monuments of the national geodetic control networks is extremely important. Accordingly, NGS field parties are instructed to obtain the permission of a property owner before establishing a station, and they are authorized to pay for any damages incurred in the process. However, it is not always possible for NGS field personnel to locate the property owner, and sometimes a station is established without permission. When a station is established on cultivated ground, or where the monument might interfere with lawn maintenance or detract from the landscape, the mark is placed below the ground surface.

The U.S. Government makes no claim to the land on which the station is located. Permission is always given to an owner to destroy a station, if necessary, in the process of property improvement. However, if informed in sufficient time, NGS prefers to relocate a station to preserve its use in the future.

Many thousands of marked horizontal control stations are widely scattered over the entire United States, and are very valuable. These marks and their positional data represent the final results of all field and office work involved in horizontal control surveys. A station is useful only as long as it can be recovered. Therefore, great care is used in marking and describing each established station. Inscribed bronze or brass (figs. 3a and 3b) disks are set in bedrock, or in concrete, to provide a station mark and to furnish reference points. These reference points can be used to recover and verify the station's position or to relocate precisely a destroyed station.

Usually a station can be quickly located during the first few years following its establishment. But after a longer lapse of time, the changes in nearby natural and cultural features make it increasingly difficult to find the mark by relying on the original description. Interested persons who have occasion to visit or use a horizontal control station and find that the description needs modification would be performing a public service to report to NGS the present condition of the mark and its surroundings. The report should be addressed to: Director, National Geodetic Survey, Rockville, MD 20852. For your convenience, copies of NOAA Form 76–91, Report on Condition of Survey Mark, are included at the end of this publication. Additional forms are available on request.

Surveyors, engineers, and other persons interested in the national network can aid in preserving the stations by placing witness signs at the station sites (fig. 10). The signs and posts are furnished free of charge by NGS. Anyone who emplaces a sign is requested to return the same card used for reporting modifications of station descriptions, noting briefly the location of the witness sign in relationship to one or more of the marks at the station location. These signs have been very effective in saving survey monuments by alerting construction crews

Figure 10.—Witness sign at a typical station site situated next to marker

and maintenance equipment operators of the existence of nearby monuments.

The preservation of accurately determined survey stations should concern every American. Each person has paid for these stations through tax revenue and, therefore, is a joint owner. Needless litigation and expenses are eliminated where these stations are used as basic starting points to define clearly the location of public and private boundaries. Communities benefit by the more accurate and efficient surveying and mapping of all local projects connected to these stations. Some typical examples are surveys for industrial development, flood control, sites for hydroelectric and nuclear power plants, highways, surface and underground rail systems, delineations of city property boundaries, creation of suburban residential communities, and farms.

Relocation of Marks

Horizontal control stations are placed where they are not likely to be disturbed or destroyed and yet can still be found without too much difficulty. New construction, relocation of roads, or beach erosion are anticipated prior to the establishment of a station and these locations avoided. However, in spite of these precautions, station marks are frequently and unavoidably destroyed. If destruction can be foreseen, it is often possible to establish a new station nearby in a safe location. The new station can be connected to the old station so precisely that it can be used in place of the original station.

The NGS has a small number of highly experienced geodetic surveyors residing in strategic geographic locations throughout the conterminous States. The regions assigned to them for maintenance are quite large. Public cooperation is essential to the success of their mission. As soon as it is known that a monument will be destroyed, a letter should be sent to the Director, National Geodetic Survey, Attention: Network Maintenance Branch, Rockville, MD 20852. The mark should be described in sufficient detail so that a positive identification can be made. Reason for the destruction should also be stated. Whenever possible, 2- to 3-weeks' notice is requested. However, if the need is urgent, collect telephone calls (area code 301–443–8319) will be accepted by NGS.

NATIONAL GEODETIC SURVEY DATA BASE

The NGS data base is designed to be user-oriented. It will eventually hold all data associated with the national geodetic control networks, including data for local surveys that may not be published as part of these networks. Because the data base management system (DBMS) is of modular design, new applications and data types can be added whenever necessary. Various modes will be available for users to access the data base, including inhouse terminals.

The foundation for the data base is the quad identifier (QID) and the quad station number (QSN) system associated with the data sheet format (fig. 7). All information for a particular point will be held under the unique QID/QSN assigned to that point.

When the DBMS becomes fully operational, the vast amount of observational data and other information held by NGS will be available to investigators in many disciplines. Surveyors and engineers will have access to details related to the published data, which were previously difficult to provide.

NEW ADJUSTMENT OF THE NORTH AMERICAN DATUM

Although many scientists in the North American geodetic community realized as early as 1950 that a new adjustment of the datum was necessary, no action was taken for more than two decades. It is fortunate that no action occurred during the intervening period because vast technological advances were made and a large number of new stations established. With the introduction of EDMI in the United States in 1952, it was possible to perform surveys to accuracies previously considered impossible, except on an extremely limited scale. A national need for high accuracy surveys became apparent in 1959, when the missile program accelerated. Accordingly, specifications were drawn for a traverse network consisting of about eight loops which covered the entire country. The anticipated accuracy of the net was 1:1,000,000. The work began in 1960 and was completed in 1976. Tests show conclusively that the accuracy requirements have been met.

During this same period surveying systems employing satellites and radio waves from outer space were being developed. This research is still continuing. In the period from 1966 to 1970, a 45-station network encompassing much of the world was established in cooperation with several other countries. Observations were obtained using sophisticated camera systems and very precise timing equipment for photographing a passing satellite against a star background. From these observations, directions between stations were determined and a network formed. This network had shape but no scale. The high-precision traverses in the United States and long accurate base lines in Europe, Africa, and Australia were used to scale the net. At the conclusion of the project, much of the world was connected with an accuracy between stations of 5 to 10 m in latitude and longitude, or about 1:500,000 of the average distance between points.

The photographic technique relied on a weather-dependent system, which was abandoned as independent all-weather surveying devices were developed. These systems include Doppler satellite techniques and very long base line interferometry (VLBI). By 1977, Doppler positioning devices were lightweight, portable, and capable of producing positions to better than a meter in latitude, longitude, and height. With the NAVSTAR global positioning system (GPS), which will eventually consist of 18 satellites in three different orbits, positional accuracies at the decimeter level are highly probable. Figure 11 shows a late-model instrument, which is typical of the equipment being manufactured by various companies for Doppler satellite observations. Today VLBI produces highly accurate distances and directions, but the method cannot be considered portable. Further developments will almost certainly produce instrumentation usable at almost any site. When GPS becomes operational, it is possible that interferometric procedures may be employed using the GPS signals to obtain positional accuracies better than 5 cm.

In 1969, the National Academy of Sciences and the National Academy of Engineering established a committee to study whether a new adjustment of the United States geodetic network was required. The 1971 published report (National Academy of Sciences and National Academy of Engineering 1971) was overwhelmingly in favor of a new adjustment. With this mandate the National Ocean Survey initiated plans for a new adjustment of the North American Datum.

Since the effort would be continental in scope, meetings were held in June 1973 between representatives of NGS and its Canadian and Mexican counterparts. Preliminary plans were discussed and communication links established. Later, Denmark (on behalf of Greenland) and the republics of Central America joined in the cooperative venture.

The new adjustment of the North American Datum will contain observations involving all stations of the United States network and primary stations in Canada, Mexico, and the republics of Central America. There are more than 240,000 stations in the United States. The island networks will be oriented by Doppler satellite positions using the same system and ellipsoid as the main adjustment.

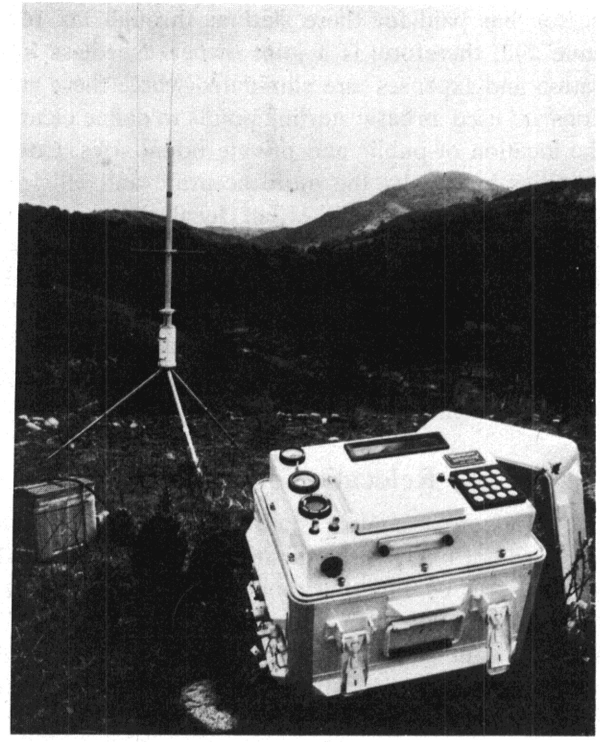

Figure 11.—A Doppler satellite surveying instrument, representative of equipment now being used for Doppler surveying

No single datum point such as MEADES RANCH will be used. Instead, possibly as many as 500 Doppler satellite positions will be introduced in the adjustment as observations and permitted to accept corrections. The network will also be oriented and scaled by several thousand Laplace azimuths and base lines. A new Earth mass-centered ellipsoid (known as GRS1980), using parameters listed in table 3, has been adopted as the reference surface.

Table 3.—Comparison of ellipsoids

Name of ellipsoid	*Equatorial radius (a)*	*Polar radius (b)*	*Flattening or ellipticity (a-b)/a*
Clarke spheroid of 1866	6,378,206.4	6,356,583.8	1/294.98
International ellipsoid	6,378,388.0	6,356,911.9	1/297.00
Geodetic reference system (GRS) 1980	6,378,137	6,356,752.3*	1/298.257*

*Computed from the adopted parameters.

All observations will be included in a simultaneous least-squares solution containing about 500,000 equations accomplished by a partitioning method known as Helmert blocking. This massive effort will provide the surveyors and engineers of North America with the best possible results from the available observations.

Surveying is on the threshold of vast changes that will affect the entire profession. In addition to satellite and VLBI surveying systems, laser lunar-ranging and laser-ranging to satellites hold great potential for providing positions to extremely high accuracies. Inertial surveying devices are furnishing rapid positioning to accuracies acceptable for many projects with the promise of even greater improvements in the future. High-precision photogrammetric surveying techniques have been perfected to the point that densification networks can be accomplished at minimum cost.

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NOAA FORM 29-3 (9-79)

U.S. DEPARTMENT OF COMMERCE
NATIONAL OCEANIC AND ATMOSPHERIC ADMIN.

GEODETIC CONTROL DATA AUTOMATIC MAILING LIST AGREEMENT

1. REQUISITION OR ORDER NO.

2. NOAA AGREEMENT NO.

INSTRUCTIONS – (See reverse side). Return original to the National Geodetic Information Center, C18, National Ocean Survey, NOAA, Rockville, Maryland 20852.

3. DATE

4. APPLICANT'S NAME AND ADDRESS

5. ADDITIONAL MAILING INSTRUCTIONS

6. **AGREEMENT – The applicant agrees to accept all requested geodetic control data and to pay for the same upon receipt. Prices will be based on the current price list.**

7. ☐ Check here if Federal Government agency – Review your requirements and confirm your subscription agreement by completing item 12.

8. ☐ Check here if other than Federal Government subscriber – Review your requirements and confirm your subscription agreement by completing item 12.

9. INDICATE AREA AND TYPE OF DATA DESIRED.

☐ NUMBER OF COPIES ☐ VERTICAL CONTROL ☐ HORIZONTAL CONTROL ☐ DIAGRAMS

LATITUDE °

LONGITUDE °

10. ☐ Check here if the applicant desires field data and recently adjusted projects in manuscript form for the area.

11. ☐ Check here if the applicant desires an initial shipment of above requested data. The purpose of this shipment is to establish an initial file for new subscribers.

12. AUTHORIZED OFFICIAL *(Applicant)*	NAME	TITLE
	SIGNATURE	DATE
13. ACCEPTED BY NATIONAL GEODETIC SURVEY	NAME	TITLE Director, National Geodetic Information Center, National Geodetic Survey, NOS, NOAA
	SIGNATURE	DATE

GEODETIC DATA
AUTOMATIC MAILING SERVICE

The automatic mailing service for geodetic control data provides the mechanism through which users maintaining active files receive newly published data automatically for a specific area. To facilitate this service, it is necessary that the desired area be composed of complete quadrangle units.

The standard quadrangles of geodetic control data are 30' of latitude by 30' of longitude. However, in congested control areas, standard quadrangles are 15' of latitude by 15' of longitude, and in some areas of Alaska, due to the sparsity of control, quadrangle units are 1° of latitude by 1° of longitude. Data are now available in these formats for approximately 85 percent of the United States with the remaining 15 percent being in a different format and available by county. In the latter areas, it will be necessary to furnish complete county coverage for all counties falling in your defined area until the data has been converted to the quadrangle format. Unpublished projects are also available in some areas through this service, but only on special request.

The prices for initial data furnished through this service are the same as for individual orders. Revised or additional published data for the requested area will be furnished thereafter for an annual subscriber charge based on the number of sheets mailed during the year (not to exceed $8.00). Handling and postage costs involved for each supplemental data shipment will be charged to the subscriber. There will be no charge, in the event that no data is mailed to the user.

A bill for the initial shipment will accompany the data. A bill for all revised or additional published data for the requested area (not to exceed $8.00), based on the number of sheets mailed during the year, will be forwarded annually thereafter. It is necessary that a copy of each bill accompany your payment to insure proper credit.

Diagrams are not included in the annual subscriber charge for geodetic control data. Subscribers requesting diagrams, whether new or revised editions, will be billed according to the current price list.

Federal Government agencies may be furnished a complimentary copy of geodetic control data and diagrams for their jurisdiction as required.

INSTRUCTIONS FOR COMPLETION OF FORM

Items 1 - 3	For agency use only.
Item 4	Correct if necessary. It is preferred that the mailing address be an office or position rather than an individual. Please include your zip code.
Item 5	Any special instructions for mailing data, if other than shown in Item 4.
Items 6 - 8	Self-explanatory.
Item 9	Area requirements may be described by a detailed written description or by filling in the appropriate boxes and outlining the geographic area on the grid. A small attached map outlining the area of interest is also acceptable. Data will be furnished to meet your requirements as previously specified.
Items 10 - 13	Self-explanatory.

REPORT ON CONDITION OF SURVEY MARK

Form Approved: OMB No. 41–R1923
Approval Expires: April 30, 1983

Name or Designation: ____________ Year Established: ____________

State: ____________ County: ____________ Organization Established by: ____________

Distance and direction from nearest town: ____________

Description published in: ***(Line, book, or quadrangle number)*** ____________

Mark searched for or recovered by: Name - ____________

Organization - ____________

Date of report ____________ Address - ____________

Condition of marks: List letters and numbers found stamped in (not cast in) each mark.

Mark stamped:	Condition:

Marks accessible? ☐ Yes ☐ No Property owner contacted? ☐ Yes ☐ No

Please report on the thoroughness of the search in case a mark was not recovered, suggested changes in description, need for repairing or moving the mark, or other pertinent facts:

This report is authorized by law (Title 33 USC Section 883e). While you are not required to respond, your cooperation is needed to make the results of this survey comprehensive, accurate, and timely.

If additional forms are needed, indicate number required. ____________

Witness Post? Yes ____ No ____
Witness Post set ___ feet ___ of ______ mark.
Witness Post set ___ feet ___ of ______ mark.

NOAA FORM 76–91
(5–78)

U.S. DEPARTMENT OF COMMERCE · NATIONAL OCEANIC AND ATMOSPHERIC ADMINISTRATION · NATIONAL OCEAN SURVEY

REPORT ON CONDITION OF SURVEY MARK

Form Approved: OMB No. 41–R1923
Approval Expires: April 30, 1983

Name or Designation: ____________ Year Established: ____________

State: ____________ County: ____________ Organization Established by: ____________

Distance and direction from nearest town: ____________

Description published in: ***(Line, book, or quadrangle number)*** ____________

Mark searched for or recovered by: Name - ____________

Organization - ____________

Date of report ____________ Address - ____________

Condition of marks: List letters and numbers found stamped in (not cast in) each mark.

Mark stamped:	Condition:

Marks accessible? ☐ Yes ☐ No Property owner contacted? ☐ Yes ☐ No

Please report on the thoroughness of the search in case a mark was not recovered, suggested changes in description, need for repairing or moving the mark, or other pertinent facts:

This report is authorized by law (Title 33 USC Section 883e). While you are not required to respond, your cooperation is needed to make the results of this survey comprehensive, accurate, and timely.

If additional forms are needed, indicate number required. ____________

Witness Post? Yes ____ No ____
Witness Post set ___ feet ___ of ______ mark.
Witness Post set ___ feet ___ of ______ mark.

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