NOAA Manual NOS NGS 3

Geodetic Leveling

Supersedes Coast and Geodetic Survey Special Publication 239,

Manual of Leveling, 1948

M. Christine Schomaker, Lt., NOAA
Ralph Moore Barry

Rockville, MD
August 1981
Reprinted June 2001

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PREFACE

The National Geodetic Survey (NGS) of the National Ocean Survey, a component of the National Oceanic and Atmospheric Administration (NOAA), establishes and maintains the National Geodetic Vertical Network. Specifications and instructions presented in this manual should be used by any agency collecting geodetic leveling data for inclusion in the national network. Revised material will be issued by NGS when procedures and instructions change.

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1.1 Introduction

Since ancient times, the ability to locate widely separated points on the Earth's surface has been of vital interest to public commerce and defense. Accurate positioning has become increasingly important to construction, boundary definition, navigation, and the prediction and monitoring of changes in the topography. To provide accurate positions over the entire globe, a detailed knowledge about the size and shape of the Earth is critical. The pursuit of this knowledge is the science of geodesy.

Traditionally, points on the surface of the Earth have been located by assigning geographic positions and elevations. Geographic position (latitude and longitude) is determined by surveying techniques such as triangulation, trilateration, and traverse, which determine horizontal distances and directions between points. Elevations are obtained by techniques that determine vertical differences between points. These include differential leveling, trigonometric leveling, and observing changes in atmospheric pressure. Modern satellite and inertial systems determine geometric distances, in three dimensions, that can be converted to both geographic positions and elevations above an ellipsoid.

Because of the differences in surveying techniques, separate networks of horizontal and vertical control have evolved. However, all geodetic networks have in common the fact that measurements are made with instruments oriented to the Earth's gravity field.

An interconnected system of points, each of which is assigned an elevation referred to a common surface, is called a vertical control network. Since geodetic leveling has in the past provided, and continues to provide, the most accurate means for measuring precise elevation differences, a vertical network typically consists of lines of control points, reflecting the progression of leveling from point to point.

Vertical control networks provide elevations for many purposes: from localized construction projects to studies of widespread motions of the Earth's crust. To establish a sufficiently accurate national network, survey techniques must be of geodetic quality: they must employ accurate equipment, maintain a high degree of precision when extended over large areas, and adequately define the gravity field.

1.2 Development of Geodetic Leveling

Leveling was successfully employed by the ancient Egyptians who attempted to connect the Nile River with the Red Sea, by the Babylonians who constructed an extensive irrigation system in the Euphrates Valley, and by the Romans who constructed extensive networks of aqueducts, both in Rome and such regions as Spain and the Middle East. In these cases, however, leveling depended on crude instruments that had provision for sighting along a water surface or that operated by some mechanical application of a plumb line.

The development of geodetic leveling as it is conducted today depended on the invention of three important items: the telescope, the reticle, and the level.

The invention of the telescope in 1608 is attributed to Lippershey, a Dutch spectacle maker. It was first used for scientific purposes by Galileo Galilei in 1609 for magnifying the image of a distant object, but it was not very useful as a pointing device until the introduction of the reticle.

The invention of the reticle, which provides "cross hairs" at the common focus of the objective lens and the ocular lens, was not possible until the invention of the "positive" ocular lens by Johann Kepler in 1611 and the actual placement of a measuring device at the common focus by the English astronomer William Gascoigne in 1639. In 1669, while working on a project to measure the length of a degree of latitude for the Royal Academy of France, Jean Picard first placed a reticle in a surveying instrument.

The invention of the level—a tube of glass with fluid sealed inside in such a way that a bubble is formed—is credited to Melchisedech Thevenot, who published information on instrumental details and manufacturing methods in 1666. Nearly a hundred years later, after procedures were perfected to manufacture level vials with uniform curvature, the telescope, reticle, and level were assembled in a spirit-level instrument similar to the one still used today in construction work. It is believed that such instruments were devised independently by Antoine de Chezy, a road and bridge engineer in France, and by Jesse Ramsden, a mathematical instrument maker in England.

Further developments in leveling are evident in early texts, such as Love's classical *Geodaesia* published in 1760, which recommends, under the heading "How to know whether water may be made to run from a Spring head to any appointed place," that: "It is better to get a water-level, such as you may buy at the Instrument-Makers." Later texts, such as the *Complete Manual of Leveling* by Simms, published in England in 1836, treat subjects such as corrections for curvature and refraction, and the calculation of earthwork from cross sections observed with a spirit-level instrument. References are made to both the "Y" level and the dumpy level, instrumental designs which are familiar to twentieth-century engineers. The "Y" level is believed to have been invented in 1740 by Jonathan Simmons of London, and the dumpy level in 1845 by William Gavatt in England.

The first leveling in Europe that is considered of geodetic quality was conducted in France under the direction of M. Bourdaloue between 1857 and 1860. His results were published in 1864. The complex observing procedure was designed to produce results with a high degree of accuracy by eliminating systematic errors and detecting and eliminating blunders. This work is said to have required that every two measurements agree to within $2\sqrt{K}$ millimeters, where K was the length of the leveling line in kilometers.

The French work inspired the Swiss to engage in a similar effort. In 1864, a Swiss recommendation for the execution of a precise leveling network over a large part of Europe was adopted by the International Geodetic Conference. The methods of observation and the use of a mean sea-level datum were prescribed in the resolution. For the observations, a precise spirit-level instrument was designed by Kern of Aarau, Switzerland. These instruments were widely used in Europe and later several were used by the U.S. Corps of Engineers.

Although some leveling was undoubtedly conducted locally in the United States (primarily to tidal bench marks) prior to the Revolutionary War and by the U.S. Coast Survey upon its establishment in 1807, the first recorded effort was a geodetic leveling project by the U.S. Coast Survey in 1856-57. To support detailed studies of the tides and currents in New York Bay and the Hudson River, a series of tide gages was established along the Hudson River which connected with a line of leveling established by G. B. Vose. Vose states in the 1857 *Report of the Superintendent of the Coast Survey:* "As you directed, a double series of levelings were made throughout the whole route and every doubtful step was retraced... It appears that the probable error for the entire distance from New York to Greenbush does not exceed two-tenths of a foot." A bench mark in this line provided the sea-level datum to which subsequent levelings by the U.S. Lake Survey were referred in determining elevations of the water surfaces in the Great Lakes.

In 1871 the Coast Survey formulated plans for a transcontinental arc of triangulation along the 39th Parallel. It quickly became apparent that accurate elevations would be required to reduce the triangulation data along the route. After Congress authorized the survey in 1876, new leveling instruments were designed and fabricated by the Coast Survey. These were used on the first transcontinental leveling in 1875. The first bench mark was set in the courthouse at Hagerstown, Md.

The line of "geodesic" leveling proceeded west with interruptions, reaching St. Louis, Mo., in 1882. A line leveled in 1881 from a tide gage at Sandy Hook, N. J., to Hagerstown, Md., provided a connection to mean sea level. In 1899 the transcontinental line reached within a few miles of Cheyenne, Wyo., and it was completed to the tide gage at Seattle, Wash., in 1905. Other lines were leveled, in cooperation with the Corps of Engineers, along portions of the Mississippi River and its major tributaries.

In 1875 the U.S. Lake Survey, requiring accurate elevations above mean sea level for the water levels in the Great Lakes and for bench marks in the adjacent harbor areas, extended geodetic leveling into the Great Lakes area. A line of control points was leveled in New York along the Erie Canal, various wagon roads, and the New York and Oswego Midland Railroad. The line extended from bench mark GRISTMILL, at the town of Greenbush, to bench mark A at the harbor in Oswego. During the same year, lines were leveled between Lake Ontario and Lake Erie, and a single line of leveling connected Lake Erie with Lake Huron. Thus, connections were made across the land between the Hudson River, Lake Ontario, Lake Erie, Lake Huron, and Lake Michigan.

An important procedure introduced in 1875 was water-level transfer in which the mean surface of each lake, averaged from data obtained at water-level gages during a 3- to 4-month period, was assumed to define an equipotential surface. An elevation relative to mean sea level, determined by leveling to a gage on a lake, was thus transferable to other gages on the lake. By this step-by-step process an elevation relative to mean sea level at Greenbush, N.Y., was assigned to the gage at Escanaba, Mich. For the first time, reasonably accurate elevations were available for all of the Great Lakes, except Lake Superior, based on the results of geodetic leveling and water-level transfers.

In 1877 the principle of "double-simultaneous" leveling was introduced along the Mississippi River by the Corps of Engineers. Two pairs of rods were used with one leveling instrument. The line of leveling was carried forward at each instrument setup with two independent observations of the backsight and foresight rods, on separate turning points. Thus, two separate levelings of the route were made, although only one observer and one leveling instrument were required. This provided continuous and effective checks against blunders as the work progressed. In 1882-83, J. B. Johnson of the Corps of Engineers introduced the observing procedure known as three-wire leveling.

In 1896 Congress authorized the U.S. Geological Survey to determine elevations in support of topographic mapping "above a base level located in each area under survey." Leveling was conducted in many States under this authorization. The first, and very important, line started at a tide gage at Morehead City, N.C., continued inland across the State through Raleigh, Greensboro, and Asheville; ran across Tennessee through Knoxville and Cleveland; extended southward into Georgia through Rome, Atlanta, and Macon, and ended on a tide gage at Brunswick, Ga.

Important contributions to the growing network of geodetic leveling were also made by railroad companies, which relied on leveling to support the construction and maintenance of their extensive track systems. Principal contributors were the Pennsylvania Railroad and the Baltimore and Ohio Railroad. In addition, almost all of the early leveling by the Federal Government was performed along railroad routes because they provided almost the only available cleared routes without excessively steep grades.

In 1898 an *ad hoc* committee of the Coast and Geodetic Survey (C&GS) was appointed to investigate the instruments, observational procedures, and results of geodetic leveling. After extensive analysis, the committee recommended a new design for the leveling instrument that had been used since 1877 and the adoption of the three-wire leveling procedures used by the Corps of Engineers. In 1899 three prototypes, modifications of the 1877 instruments, were produced in the C&GS shops and tested in the field.

The new spirit-level instrument was first used in 1900. It became known as the Fischer level, named for E. G. Fischer, chief of the C&GS Instrument Division, who was responsible for its design and manufacture. The instrument was designed to be very rigid and to change little in adjustment during temperature variations. Its freedom from mechanical failures and its high degree of accuracy soon became apparent.

The Fischer level was the standard leveling instrument of the Coast and Geodetic Survey until the mid-1960's when it was replaced by instruments equipped with optical micrometers.

Since 1900 further improvements in accuracy have been possible. These resulted from the introduction of Invar scales for leveling rods in 1917, instruments with optical micrometers in 1910, and compensator instruments in 1950. These features are still important today.

1.3 Height Systems

Before a network of geodetic leveling may be considered a network of vertical control, the observed elevation differences must be placed within a common frame of reference. The first step is taken when the differences are measured after leveling an instrument and a pair of rods; all measurements are then relative to the direction of gravity. The second step is to minimize discrepancies in the results obtained by leveling along different routes between the same two points. For this process, termed adjustment, an appropriate height system must be selected to account for the nonspherical and irregular nature of the gravity field. The third step is to define the surface datum to which heights may be referred.

Orthometric height. The gravity field can be represented as a series of surfaces of equal potential, termed "level" or equipotential surfaces, one of which is specified as the reference surface from which orthometric heights are measured. The orthometric height of a point is the distance from the projection of the point on the reference surface to the point itself, along a line perpendicular to every equipotential surface in between.

If an equipotential surface were consistently parallel to the reference surface, the orthometric height of every point on the surface would be the same. However, because of the Earth's rotation and anomalies in the gravity field itself, the equipotential surfaces defined by the gravity field are not parallel. The orthometric heights of points on a common equipotential surface vary, increasing most noticeably as a person moves towards the equator (fig. 1-1). For example, a level surface 1000 m above mean sea level at the Equator would be only 995 m above mean sea level at the poles.

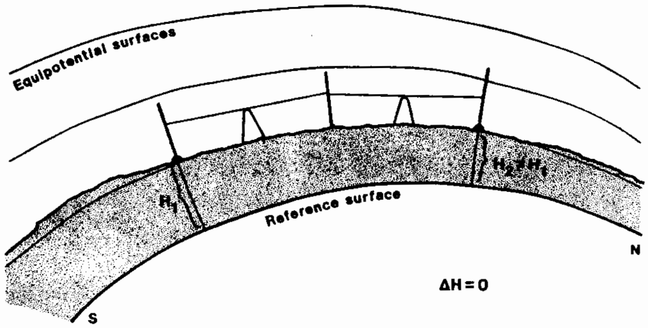

Figure 1-1.—Orthometric height, *H*, on a surface not parallel to the reference surface.

As a result, even though no elevation difference is measured when leveling between two points on an equipotential surface, the orthometric heights of the points are different if the points are at different latitudes. The elevation computed for the second point, by adding the observed elevation difference to the elevation of the first point, is not the orthometric elevation of the second point.

Furthermore, if the leveling follows a different route between the same two points, along a series of different equipotential surfaces, yet another elevation will be computed for the second point (fig. 1-2).

To obtain orthometric elevations that are consistent with respect to a single reference surface, this effect

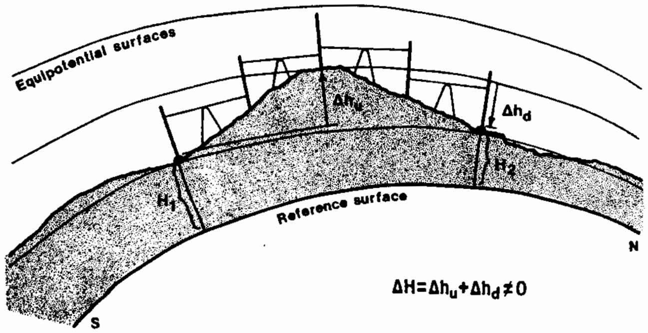

Figure 1-2.—Observed difference in orthometric height, ΔH, depends on the leveling route.

must be removed. One way is to correct the elevation difference for each section of leveling, subtracting the error predicted by using a model of the Earth's normal gravity field. (See end of sec. 5.5.2.) The elevations obtained in this manner are termed normal orthometric heights.

Geopotential number. A more accurate way to account for the effect of the gravity field is to compute geopotential numbers rather than orthometric heights. The geopotential number of a point is a measure of the difference in potential from the reference surface to the equipotential surface passing through the point. It is numerically equivalent to the work required to raise a mass of 1 kg against gravity, g, through the orthometric height, H, to the point:

$$\text{Geopotential number} = \int_0^H g\,dh.$$

The difference in height, dh, measured during each setup of leveling, may be converted to a difference in potential by multiplying it by the mean value of gravity for the setup. To compute accurate geopotential numbers, gravity must be determined with sufficient accuracy and density throughout the vertical network. Only since the development of the modern gravimeter has this become practical.

Although geopotential numbers are useful for the adjustment of vertical networks, for many purposes orthometric heights above a physically defined reference surface are still necessary. A geopotential number may be converted to an orthometric height by dividing it by the mean value of gravity along the plumb line between the point and the reference surface. Since such a value cannot be directly measured (because the reference surface lies within the Earth beneath the point), a model must be used to derive the value as a function of the geographic position, measured value of gravity at the point, and other variables.

Datums. Traditionally, mean sea level has been selected as the reference surface for computing heights because a water surface conforms to the gravity field by approximating an equipotential surface. A point on mean sea level is determined at a tide gage, where the fluctuations of water level are observed for many years to assess tidal effects adequately. If the elevation at such a gage is connected to a leveling network, and zero is fixed as the height of mean sea level, the reference surface thus defined is called a sea-level datum.

However, a variety of physical factors—changes in volume, currents, temperature and salinity gradients, atmospheric pressure, wind, and sea-floor topography—affect the mean water level determined at each tide gage. As a result, the points assigned to be on mean sea level at different tide gages do not necessarily lie on the same equipotential surface.

To avoid the implication that an elevation difference of zero will be obtained when leveling between two widely separated tide gages, the reference surface is simply referred to as the datum. It can be defined in many different ways that may or may not refer to mean sea level. As described in the following paragraphs, the datum of the National Geodetic Vertical Network has always been defined by a mean sea level. It has changed with each adjustment as additional tide gages have been connected.

1.4 National Geodetic Vertical Network

By 1900 geodetic leveling by the Coast and Geodetic Survey and other agencies had become so extensive that a general adjustment of the results became necessary to obtain consistent and accurate elevations for all control points. Data from 21,095 km of leveling were obtained by the Coast and Geodetic Survey from the U.S. Geological Survey, the Corps of Engineers (U.S. Lake Survey, Mississippi River Commission, Missouri River Commission, Deep Waterways Commission, and others), and the Pennsylvania Railroad.

The adjustment of this, the first national network, produced elevations for about 4,200 control points that were referred to mean sea level as determined at the following tide gages: Boston, Mass., New York, N. Y., Sandy Hook, N.J., Washington, D.C., and Biloxi, Miss. A connection to sea level on the Pacific coast had not yet been obtained.

Because of an expanded leveling program, more than 10,000 km of new leveling were completed by 1903, including the releveling of some previously unsatisfactory lines. A second adjustment was made, including approximately 6,900 control points in a network of 31,789 km.

In 1907 another 6,500 km of new work had been completed, including the transcontinental leveling through Wyoming, Utah, Idaho, Oregon, and Washington to connect with the tide gage at Seattle, Wash. To utilize the new data the adjustment of 1907 was made. At that time, the network included a total of 38,359 km of leveling and about 9,100 control points. As a matter of expediency, the elevations of most points in the eastern United States were not changed.

New connections to mean sea level on the Gulf coast and the Pacific coast, totaling 9100 km, became available in 1912, including a line across the southern United States from Louisiana to San Diego, Calif., and a north-south line. For the fourth general adjustment of the network, mean sea level was fixed at five tide gages on the Atlantic coast, two on the Gulf coast, and two on the Pacific coast. Orthometric corrections were applied to leveling data in the western States. The 1912 network included 46,462 km of leveling and 11,100 control points.

General Adjustment of 1929. After the previous period of comparatively short intervals between adjustments, 17 years elapsed before the network was adjusted again. In the meantime, it had become more extensive and complex, and included many more sea-level connections. The General Adjustment of 1929 incorporated 75,159 km of leveling in the United States and, for the first time, 31,565 km of leveling in Canada. The U.S. and Canadian networks were connected by 24 ties between Calais, Me./Brunswick, New Brunswick; and Blaine, Wash./Colebrook, British Columbia. A fixed elevation of zero was assigned to the points on mean sea level determined at the following 26 tide stations:

Father Point, Quebec	St. Augustine, Fla.
Halifax, Nova Scotia	Cedar Keys, Fla.
Yarmouth, Nova Scotia	Pensacola, Fla.
Portland, Me.	Biloxi, Miss.
Boston, Mass.	Galveston, Tex.
Perth Amboy, N.J.[1]	San Diego, Calif.
Atlantic City, N.J.	San Pedro, Calif.
Baltimore, Md.	San Francisco, Calif.
Annapolis, Md.	Fort Stevens, Ore.
Old Point Comfort, Va.	Seattle, Wash.
Norfolk, Va.	Anacortes, Wash.
Brunswick, Ga.	Vancouver, British Columbia
Fernandina, Fla.	Prince Rupert, British Columbia

[1] There was no tide station at Perth Amboy, but the elevation of a bench mark at Perth Amboy was established by leveling from the tide station at Sandy Hook.

The 1929 adjustment provided the basis for the definition of elevations throughout the national network as it existed in 1929, and the resulting datum is still used today. The elevations were referred to the "Sea Level Datum of 1929" until 1973, when the more appropriate name "National Geodetic Vertical Datum of 1929" (NGVD29) was adopted for the same reference surface.

The modern network. Since 1929, more than 625,000 km of leveling throughout the United States have been added to the national network. New results have been adjusted to fit into the network of 1929. If both old and new levelings were of similar accuracy and if the Earth's crust were perfectly stable, this procedure could have continued indefinitely. However, instrumental and procedural improvements make possible a higher degree of accuracy in modern leveling, and, increasingly, the national network has come to play an important role in monitoring crustal movements.

In many regions, withdrawal of underground fluids has led to land subsidence at the surface, which has resulted in significant economic losses. In tectonically active areas earthquakes cause sudden changes in elevation. Elevations are not fixed in time, and, therefore, elevation differences observed at widely different times cannot be expected to agree exactly. Leveling results must be adjusted by epoch and then compared to assess topographic change.

These considerations have led to the establishment of local programs in which regions of known crustal motion have been releveled periodically to assess the rate and extent of subsidence or uplift. To relate these surveys adequately and to determine where additional work is necessary, the national network itself must be resurveyed at regular intervals.

At present, a minimum network of 100,000 km has been targeted for releveling. This network, known as basic net A (fig. 1-3), is designed to satisfy the standards established by the Federal Geodetic Control Committee for first-order vertical networks (table 1-1).

Vertical control surveys (projects) are conducted continually by field parties of the National Geodetic Survey (NGS) to maintain and update the national network. The procedures by which such projects are accomplished—reconnaissance and bench mark setting, geodetic leveling, river crossing, and data processing—are the subjects of the remaining chapters of this manual.

Table 1-1.—Classification of vertical control networks

	First order class I	First order class II	Second order class I	Second order class II	Third order
Principle use	Basic net A		Area control		Local control
Line spacing,					
National net	100-300 km (60-190 mi)	50-100 km (30-60 mi)	20-50 km (10-30 mi)	10-25 km (5-15 mi)	As needed
Metropolitan net	2-8 km (1-5 mi)	2-8 km (1-5 mi)	0.5-1 km (0.3-0.6 mi)	As needed	As needed
Maximum length of leveling line between junctions	300 km (190 mi)	100 km (60 mi)	50 km (30 mi)	50 km (30 mi) double run, 25 km (15 mi) single run.	25 km (15 mi) double run, 10 km (5 mi) single run.
Control point spacing	Average 1.6 km (1.0 mi), maximum 3 km (2 mi).		Maximum 3 km (2 mi).		

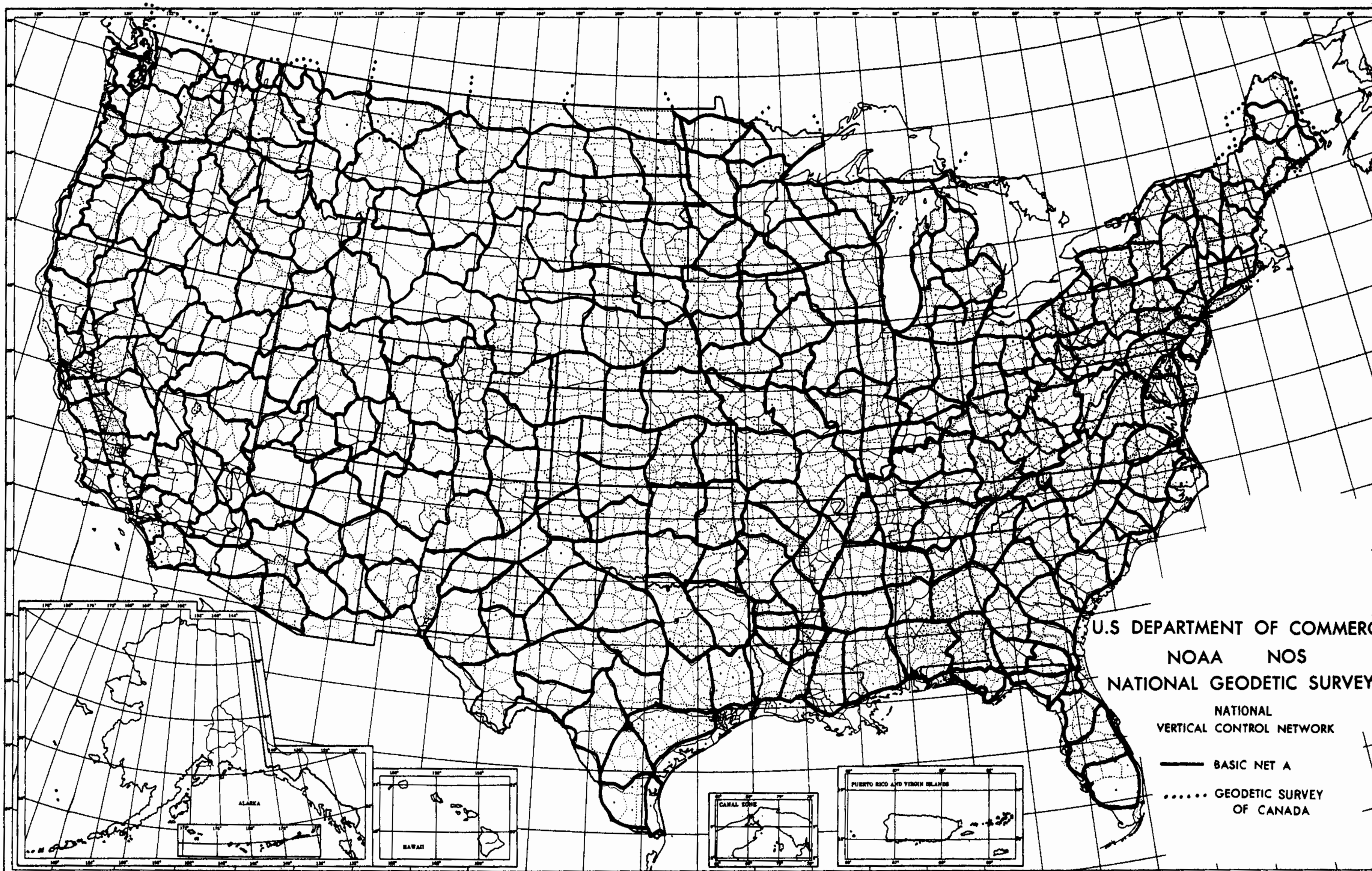

Figure 1-3.—Basic net A of the National Geodetic Vertical Network.

Chapter 2

RECONNAISSANCE AND BENCH MARK SETTING

2.1 Introduction

Before leveling proceeds in a network of vertical control, lines of control points must be defined. This is best accomplished by making a reconnaissance and setting the necessary bench marks a month or more before leveling begins. Detailed instructions for reconnaissance and mark setting are presented in this chapter.

The task is normally performed by personnel organized into mark setting units. Leveling routes are planned in detail to satisfy network requirements. Maps with written instructions (referred to as "logs") are prepared to illustrate the routes for the leveling units. Monuments are recovered, established, and described to mark permanently the lines of vertical control.

Sometimes a single monument must be removed and set elsewhere by a mark maintenance engineer or local surveyor before it is destroyed by highway construction or other activity. The chapter concludes with instructions for relocating such control points.

2.2 Organizing a Mark Setting Unit

At the start of a project, the mark setting personnel should be organized into units and each unit should be assigned a line to reconnoiter and prepare for leveling. Since a typical project encompasses a large area, each unit must be capable of working self-sufficiently, often several hundred kilometers from the project office. This section presents guidelines for organizing and maintaining such units.

2.2.1 Personnel

A foreman should be assigned to supervise the mark setting units and to manage their activities, particularly when many units are operating. Two persons are normally required on each mark setting unit, a mark setter and an assistant.

Mark setting foreman. The mark setting foreman coordinates, supports, and monitors the mark setting units. The foreman's skills should include technical competence in both mark setting and leveling, as well as proficiency in organizing unit activities and maintaining good relations with local agencies and the public. The foreman should set an example which ensures correct, safe, and efficient mark setting.

The foreman is responsible for coordinating the mark setting effort on all the lines in a project. From a study of the preliminary data, he or she should specify routes for each line and determine areas requiring special attention so that data can be organized and distributed to the appropriate units. Efforts of a specialized unit, such as a drilling-rig team, should be smoothly merged with those of the unit responsible for the line.

The foreman supports the units by providing liaison, training, and supplies. He or she should establish initial contacts with local agencies, and informs the mark setting units if follow-up contacts are necessary. Throughout the project, the foreman should provide technical guidance and training, and, in addition, inventory supplies regularly and place orders to meet future needs.

The foreman should ensure that the specifications and guidelines stated in this manual and in *NOAA Manual* NOS NGS 1, Geodetic bench marks (Floyd 1978), hereinafter referred to as the mark setting manual, are strictly followed. A monthly inspection should be made of each unit. Equipment should be examined to ensure that it is properly maintained. Mark setting techniques should be checked to ensure that safe and correct procedures are followed by the units. After receiving descriptions and logs from a unit, the foreman should routinely inspect the line, looking for poorly set marks, plotting mistakes, and description mistakes. The unit should be made aware of any deficiencies and should correct them immediately. Whenever there is substantial confusion or doubt about a route or mark setting procedure, the project director should be consulted.

The foreman should prepare a report of activities for the project director each month.

Often, if a project is small or the number of available personnel is limited, the foreman may be required to serve as chief of a mark setting unit or leveling unit. When this occurs, the project director should assume most of the responsibilities of the foreman.

Mark setter. As head of a mark setting unit, this individual is normally responsible for the correct reconnaissance and mark setting of an entire line. In addition, he or she must properly maintain and protect a large and varied selection of mechanical equipment. The mark setter who is in charge of a drilling rig has additional maintenance responsibilities. Training and experience in reconnaissance and mark setting as well as strong mechanical ability are necessary qualifications. Leveling experience is highly desirable.

The mark setter should follow the instructions stated in this manual and in the mark setting manual. The project instructions and preliminary data provided by the project director or foreman should be studied and understood. The mark setter should not hesitate to seek guidance when questions arise.

The mark setter should establish an organized work routine with the assistant, incorporating safety precautions into every procedure. Equipment failures should be reported promptly to the foreman or project director. The mark setter should submit monthly reports to the project office.

Assistant mark setter. The assistant mark setter is often a new employee or a member of a leveling unit who is receiving training. The assistant helps the mark setter in all phases of the work, in addition to being assigned certain routine responsibilities, such as truck maintenance and checking control point descriptions.

2.2.2 Equipment

Each unit should be assigned a truck, tools, and other equipment (as itemized in appendix A). When the equipment is issued, the mark setter should conduct an inventory and repeat it at least once every year thereafter. All equipment should be secured in an organized fashion in the assigned vehicle.

The mark setter is responsible for maintaining the truck and tools in good condition. In addition, he or she should study and use the equipment manuals. Equipment losses, damage, or defects should be reported immediately to the foreman or project director. Replacements and repairs should be noted on the monthly report.

Drilling rigs and gasoline-powered portable drills require especially rigorous maintenance if they are to function properly. They should be assigned only to responsible individuals who have demonstrated competence in operating such equipment.

2.2.3 Safety

Mark setting presents numerous situations where personal injury may occur. For this reason, a mark setting unit should always include two persons. All mark setters and assistants should be trained in basic first aid and cardiopulmonary resuscitation. They should know the location and telephone number of an emergency treatment center in the work area, so prompt assistance can be obtained if an accident occurs.

Accidents usually result from hazards created by the environment or the equipment. Anticipate potentially hazardous situations and be prepared to handle them.

The mark setting environment is normally the right-of-way of a highway or railroad. Obtain permission to work in these zones and find out from local officials what safety procedures are required. When working alongside a highway, stay well off the road, wear orange vests, and turn on the truck's warning light. If necessary, and only after consulting with local authorities, use traffic cones, warning signs, or flagmen to divert traffic while setting a mark. When working along a railroad, do not wear orange vests or use flashing lights because these may cause a train to stop unnecessarily. Cross the tracks only at designated crossings; otherwise keep clear of them.

Right-of-way zones are often used as routes for utility pipelines and cables. Select sites for marks with this in mind. Before using an auger or driving a rod, check the site with a pipe and cable locator. Similarly, be alert to the presence of wires *above* the work location, especially when maneuvering a drilling rig or erecting lengths of steel rod.

Mark setting equipment may present a hazard if used improperly. Learn the correct techniques for handling and operating all equipment. Use the right tools for the job and keep them clean. To prevent back injury, lift heavy tools and supplies from a squatting position; use the leg muscles, not the back. Get help if an object is too heavy to lift safely. Above all, stay alert. Do not allow a steady routine to make you complacent.

Two items of equipment deserve special mention: the gasoline-powered rock drill and the drilling rig. When driving rods or drilling with these tools, prevent eye injury by wearing goggles or shatter-proof glasses. Prevent hearing loss by wearing ear plugs or mufflers. Wear hard hats and steel-toed boots.

Mark setters operating a drilling rig must be thoroughly trained in the proper routines for starting and stopping the drill, leveling and stabilizing the truck, and assembling and disassembling the augers. These instructions are outlined in the maintenance manual accompanying the rig. Test the emergency stopping system each day. While drilling, do not wear gloves near the turning augers.

2.2.4 Reports

The project director or foreman should visit each mark setting unit at least once each month. At all other times the mark setters should maintain regular communication, either in person or by telephone, with the project director. In addition, the mark setter should submit the reports described in the following paragraphs.

Monthly report. Mail or submit in person, a report to the project office at the end of work each month. (See fig. 2-1.) If work involved more than one State or project, prepare a separate report for each activity. The report is normally forwarded to the headquarters of the National Geodetic Survey, where it provides statistical information about the progress of mark setting in the national network. Include the following information:

1. Number of points recovered in good condition.
2. Number of points recovered in poor condition.
3. Number of points recovered destroyed.
4. Number of points not recovered.
5. Number of bench marks set; types of bench marks set.
6. Notes about condition of equipment and maintenance needed or performed.
7. Inventory of existing supplies.
8. Notes on project activities.

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(3-82)

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

MONTHLY REPORT – BENCH MARK AND RECONNAISSANCE TEAMS

TEAM LEADER	PROJECT DIRECTOR	
TEAM ASSISTANT	PARTY	MONTH/YEAR 19____

Date	MARKS SEARCHED FOR									MARKS ESTABLISHED					TASK NUMBER:			
	VERTICAL CONTROL				HORIZONTAL CONTROL										STATE:			
	Recovered Good Cond.	Recovered Poor Cond.	NOT Recovered	Destroyed or Unusable	Stations	Reference Marks	AZ Marks	Other	NOT Recovered	Rock Marks	Quality A Rods Sleeved	Quality A Rods Unsleeved	Quality B Rods	Structure Marks	Line Number	Kilometers	Staff-Days	Remarks
1																		
2																		
3																		
4																		
5																		
6																		
7																		
8																		
9																		
10																		
11																		
12																		
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22																		
23																		
24																		
25																		
26																		
27																		
28																		
29																		
30																		
31																		
Total																		

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INVENTORY OF MARK SUPPLIES

PARTY	TRUCK NUMBER	MONTH/YEAR 19____
INVENTORIED BY	CONDITION OF TRUCK	

	BEGINNING INVENTORY	SUPPLIES ADDED	ENDING INVENTORY	SUPPLIES USED	SUPPLIES NEEDED
Rods					
Studs					
Rod Entry pts. (fluted)					
Rod Entry pts. (smooth)					
Datum pts.					
Logo Caps					
5" PVC					
1" PVC					
Disk (stem)					
Disk (rod)					
Witness Post					
Witness Sign					
Grease					
"O" Ring					
PVC Reducer					
PVC Cement					
PVC Schedule					
Pop Rivets					
BM Description					
Flagging					
Crayon					

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Figure 2-1.—Mark setting monthly report.

Maintain the report each day, recording the number of points searched for or set and a general statement about the day's activities. As appropriate, include remarks about weather, equipment malfunction, equipment repair, geological conditions, and suggestions. Be sure to list names, telephone numbers, and addresses of officials and individuals who were contacted.

At the end of the month, summarize progress along the assigned lines by totaling the distance of line reconnoitered and prepared for leveling, the number of 7.'5 position plots prepared (a 15′ plot counts as four), and the number of descriptions written. Do not count plots or descriptions that are not yet finished. Also, inventory specialized mark setting supplies, so their usage can be assessed and future stocks can be procured. List any supplies needed.

Truck report. Keep careful records of all expenses incurred for fuel, oil, repair, and maintenance of mark setting vehicles. At the end of the month, submit a report to the project office summarizing these expenses and include the receipts.

Miscellaneous reports. Be familiar with current organizational procedures for submitting per diem vouchers, accident reports, personnel reports, and expense receipts.

2.3 Routing a Line for Leveling

The reconnaissance and mark setting effort begins with the detailed planning of the leveling routes. The foreman (or project director) should organize and consolidate the preliminary data for a project into packages corresponding to each line or group of lines. A general reconnaissance of the area should be made and liaison should be established with local agencies. The mark setting units then proceed to route each line according to the specifications given in this section, recovering and establishing control points and preparing instructions for the leveling units.

2.3.1 Preliminary Data

For each line or group of lines in a project, a mark setting unit should possess the data listed in table 2-1.

Figure 2-2 shows the areas covered by some of these items. Most of the items are forwarded to the project director along with project instructions. Some items must be obtained in the field. Instructions pertaining to their use are described in the following paragraphs.

Project instructions. Project instructions are the starting point for any survey. They include information and instructions specific to the project which supplement the standard operating procedures given in this manual. Before beginning work, read the instructions thoroughly and consult them whenever in doubt about the project requirements.

Table 2-1.—Preliminary data for reconnaissance and mark setting

Requirements
Project instructions:
Maps for planning and organization:
NGS State index map for control leveling
NGS Geodetic Control Diagrams
USGS State index to topographic maps
State and county highway maps
Geological and soil maps, as available
Descriptions of control points:
NGS vertical control data, including a separate listing of archival cross-reference numbers if necessary.
NGS horizontal control data
NGS unpublished descriptions for points recently leveled or reset.
NOS tide and water-level station reports.
Descriptions of specially requested points.
Maps for logs and position plots:
USGS topographic maps
City and county street maps, as necessary.

Refer to the project instructions to obtain the following information:

1. A rough sketch of the proposed leveling routes.
2. Line numbers and titles.
3. Accession numbers ("L-numbers") of previous survey lines that coincide or connect with the proposed routes.
4. Specific control points to be leveled.
5. Survey-point serial numbers to be assigned to the points in each line.

A table similar to that in figure 2-3 is normally included in the project instructions.

Maps for planning and organization. Several types of maps, covering large parts or all of the project area, are useful for planning and monitoring progress along each line. These maps are most useful if the project lines have been highlighted and labeled in advance.

The Index Map of Control Leveling (fig. 2-4) for the State in which the project is located is available from NGS. It provides an overview of the project area, showing existing first- and second-order lines of leveling. On the index map are plotted and labeled the 30′ quadrants into which vertical control data are grouped.

Geodetic Control Diagrams (fig. 2-5) are also available from NGS. They are plots of existing geodetic control at a scale of 1:250,000 on the 1° by 2° quadrangle series of the U.S. Geological Survey (USGS). Control for Alaska is plotted at a scale of 1:500,000 on sectional aeronautical charts. Diagrams are designated by name, number, and two-letter area code. They show the locations of both vertical and horizontal control points and the boundaries of 30′ quadrants, 15′ topographic maps, and counties. The diagrams are particularly useful when planning connections between leveling lines and other types of control networks. An index of control diagrams for the United States is given in appendix B.

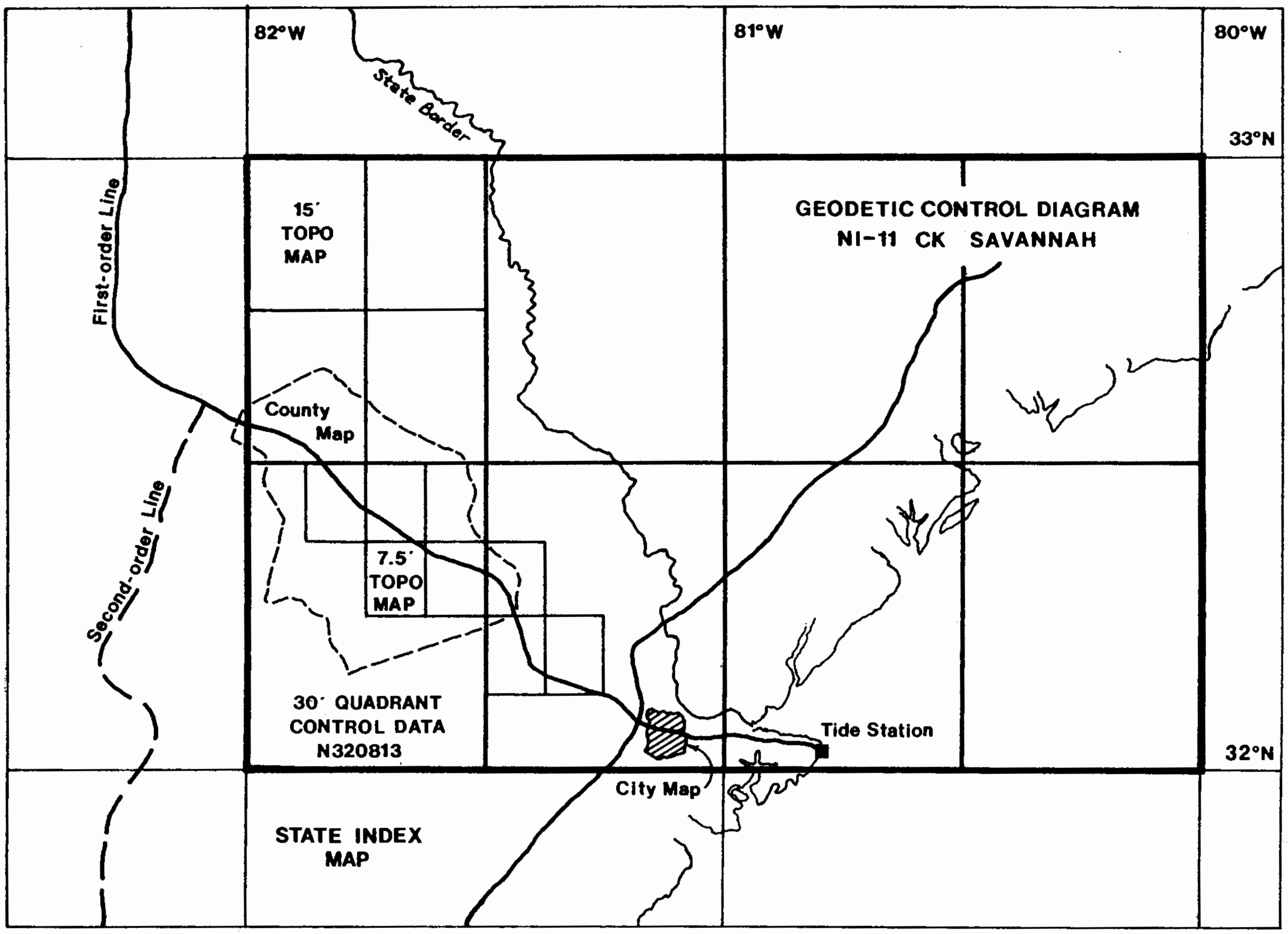

Figure 2-2.—Scale relationships of preliminary data.

TABLE 1

JC	LINE NO.	PROJECT TITLE/LINE TITLE
W2		HARRIS-GALVESTON COASTAL SUBSIDENCE AREA TX
W2	1	ADDICKS VIA HOUSTON TO CROSBY
W2	2	ADDICKS VIA BELLAIRE TO HOUSTON
W2	3	HOUSTON VIA LEAGUE CITY TO GALVESTON - PIER 21 TIDE STATION
W2	4	CROSBY VIA BAYTOWN AND TEXAS CITY TO LA MARQUE
W2	5	3 KM W QF PASADENA TO LA PORTE
W2	6	WEBSTER TO SEABROOK

TABLE 2

JC	LINE NO.	LENGTH KM.	OLD HGZ NO.	USE SPSN'S
W2	1	80	L24406-12,-19,-25	1-200
W2	2	60	L24406-10,-11	201-300
W2	3	80	L24406-7,-2	301-500
W2	4	100	L24406-19,-20,-4	501-700
W2	5	30	L24406-21,-22	701-800
W2	6	10	L24406-6	801-900

Figure 2-3.—Example of line assignments in project instructions.

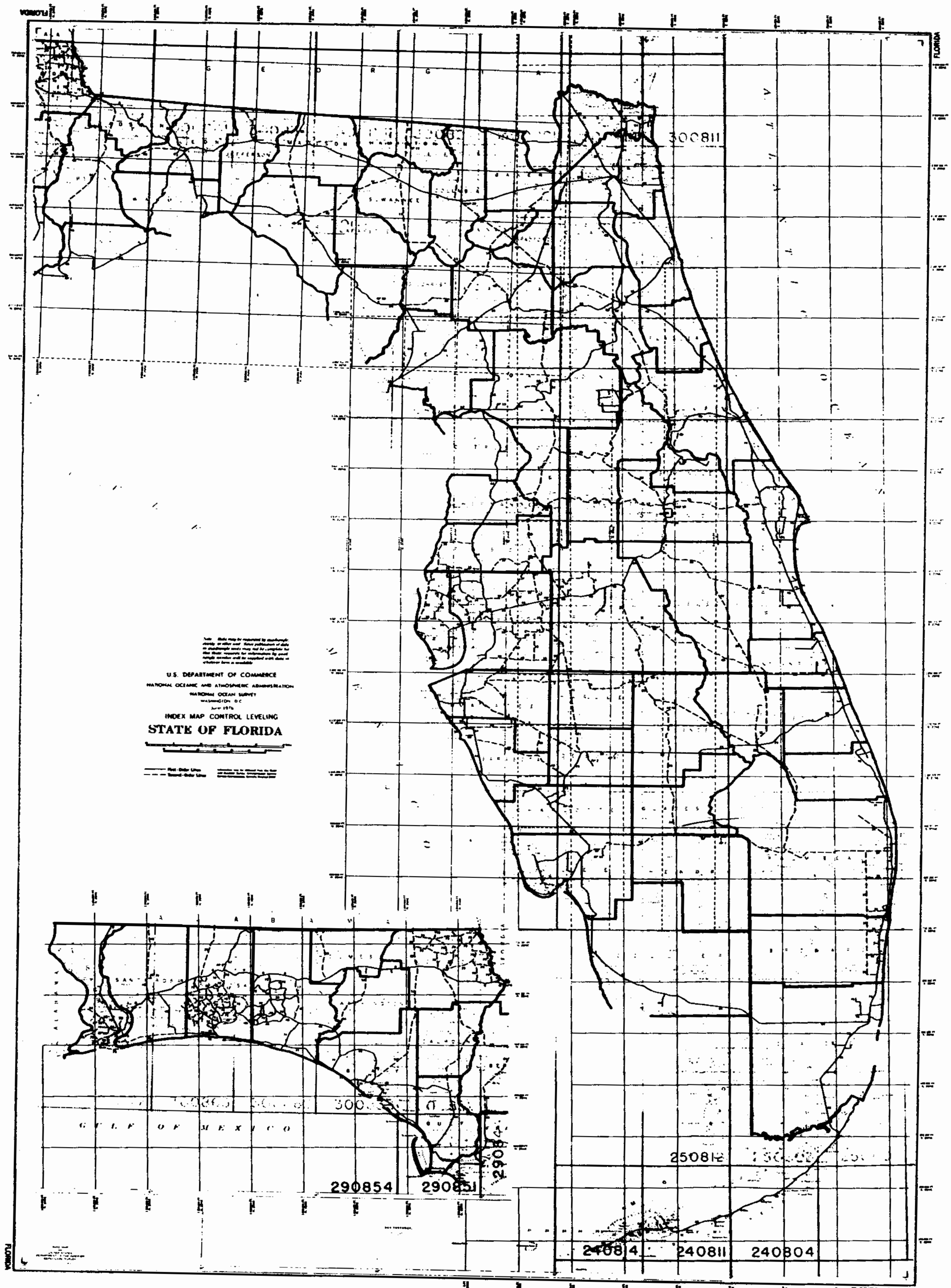

Figure 2-4.—State Index Map of Control Leveling.

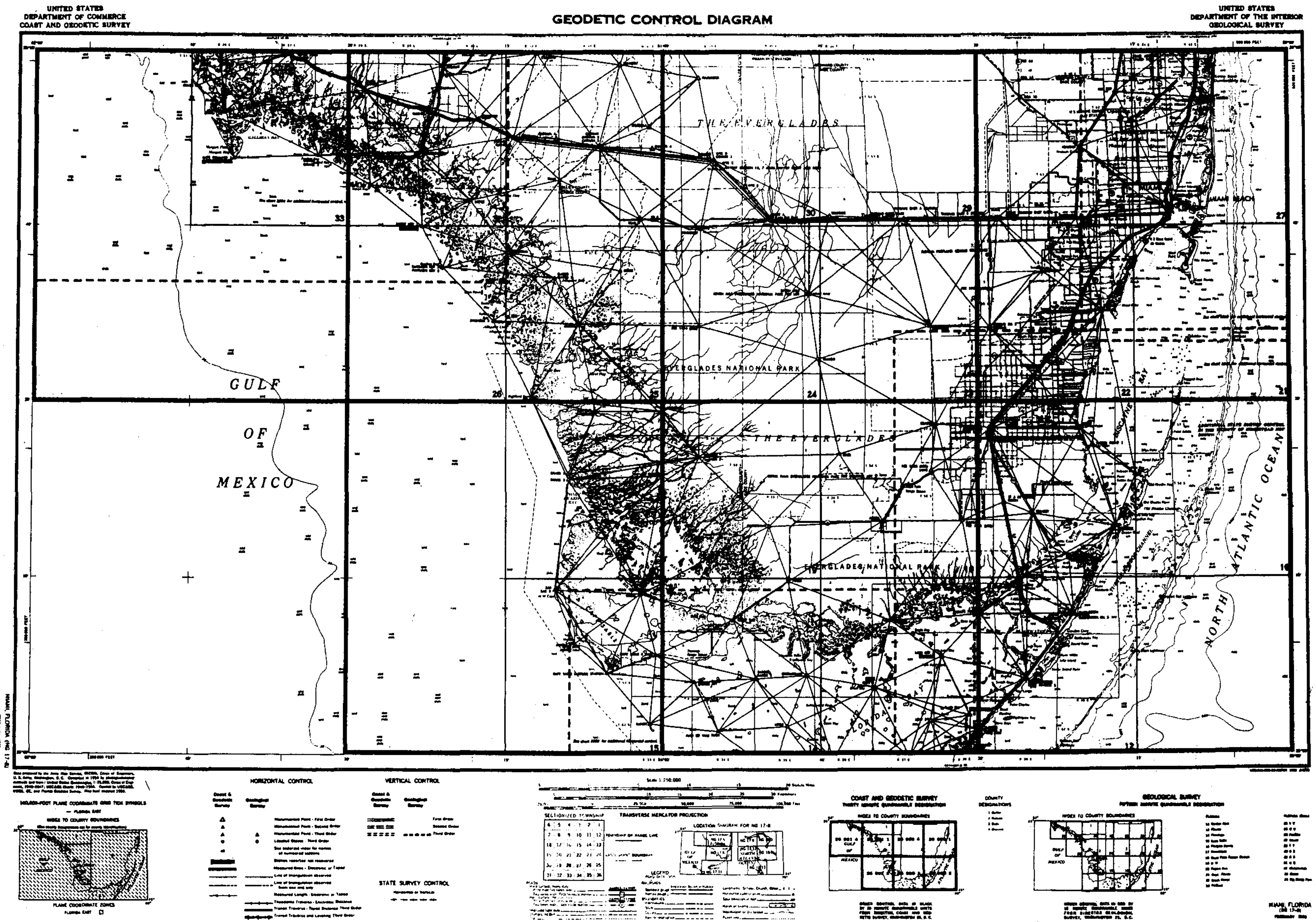

Figure 2-5.—Geodetic Control Diagram.

The Index to Topographic Maps for the State in which the project is located is available from USGS. Although the index does not show all geographical features, the project lines may be plotted approximately (fig. 2-6). It is extremely useful as an index and check-off list for logs and smooth plots.

State and county highway maps are usually available from district offices of a State highway or State transportation department. Since topographic maps do not always show the latest highway construction and route numbers, current State highway maps should be the final source for this information.

Consult geological and soil maps when selecting sites for bench marks. Basic information about the location and type of expansive soils throughout the United States is provided in the mark setting manual. Additional information, especially for locating bedrock and fault zones, can be obtained from geological maps published by the USGS or State agencies, such as departments of mines and natural resources. Information concerning soil types and depths is often available from local offices of the U.S. Soil Conservation Service.

Other sources that may be useful for certain projects include aeronautical charts, storm evacuation maps, and nautical charts. These are available from the National Ocean Survey (NOS).

Descriptions of control points. To recover existing vertical and other control points, published descriptions should be consulted. Because they serve different purposes, descriptions for vertical control, horizontal control, and tide or water-level stations are prepared, stored, and published separately.

Vertical Control Data, available from NGS, are published in sets. Each set ("quad" or "litho list") includes data for all the vertical control points in a 30′ quadrant. The quadrant is identified by hemisphere, latitude, longitude, and quarter. After the hemisphere symbol, "N" or "S", a 1° by 1° area is identified by the degrees of latitude bounding it on the south (on the north in the southern hemisphere) and by the degrees of longitude bounding it on the east. The quadrants are numbered clockwise within the 1° by 1° area, beginning with the northeast: "1" signifies the northeast quarter, "2" the southeast, "3" the southwest, and "4" the northwest.

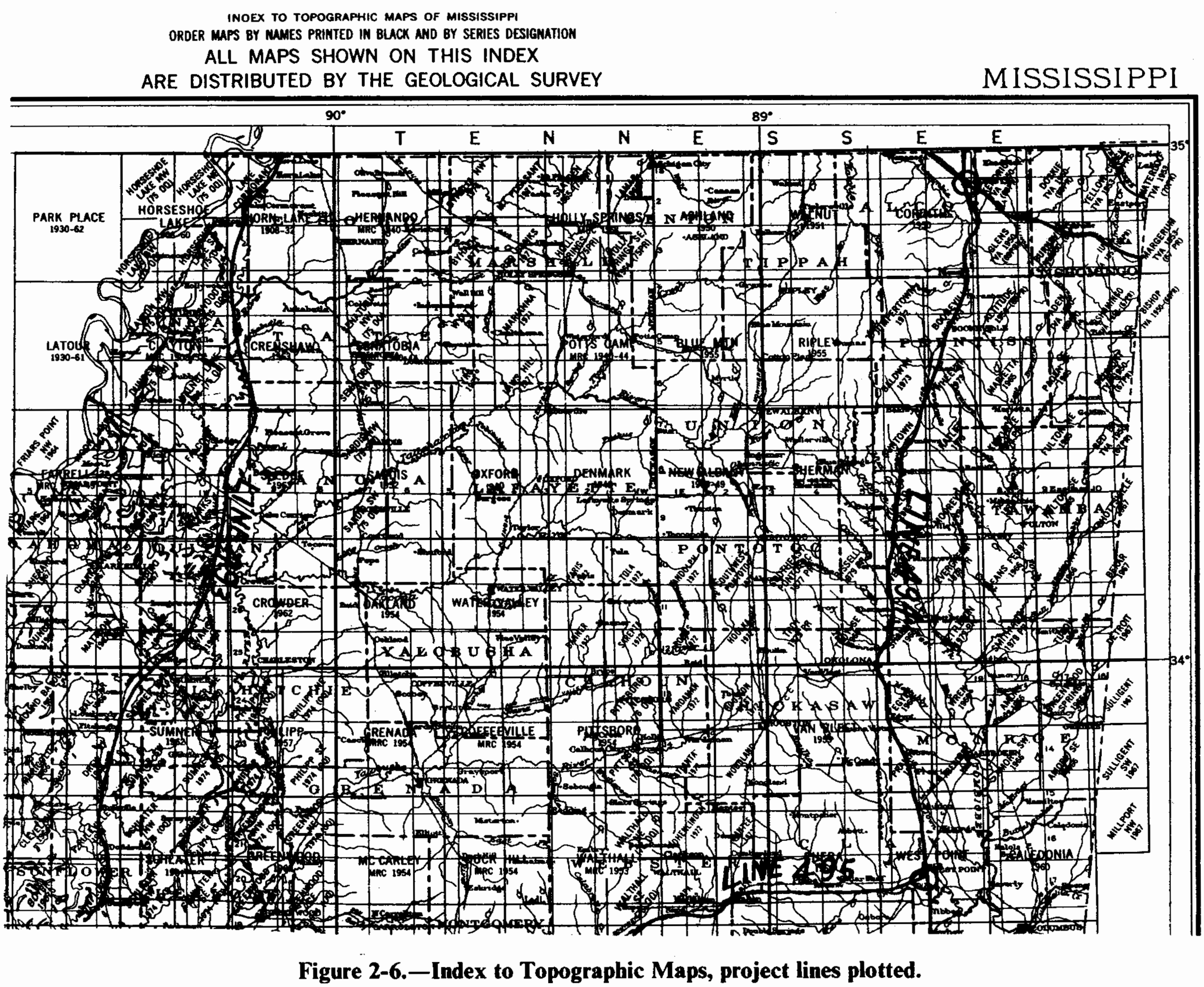

Figure 2-6.—Index to Topographic Maps, project lines plotted.

For example, "N340901" signifies a 30′ quadrant which is the northeast quarter of the 1° by 1° area bounded by 34°N on the south and 090°W on the east.

Each quadrant includes a plot showing the lines of control points described within the quad (fig. 2-7a). The line numbers given in the plot are unique only within the quadrant; they do not necessarily correspond to those of adjacent quadrants. Arrows indicate the order in which control points are described along each line. The plot is followed by two indexes that give the designations of points in alphanumeric order (fig. 2-7b) and line order (fig. 2-7c). Descriptions and other data (fig. 2-7d) are then given in line order.

Each control point is assigned a unique number (the archival cross-reference number) when it is initially included in the NGS data base. If the published descriptions do not include these numbers, obtain special listings of the synoptic file ("archive lists") from the NGS Vertical Network Division. Identified by two-letter area codes, these listings are available for each Geodetic Control Diagram. (See fig. 2-8a,b.)

Data for horizontal control points are also available in 30′ quadrants from NGS. Descriptions are usually arranged chronologically within each quadrant. An alphanumeric index of designations is included. After finding the designation of the desired point on a Geodetic Control Diagram, look it up in the appropriate index to find the page number of the description. Further information about horizontal control data is available in *NOAA Technical Memorandum* NOS NGS 5, National Geodetic Survey data: availability, explanation, and application (Dracup 1979).

For up-to-date information on points that have recently been recovered, established, or relocated, obtain descriptions from the NGS Vertical Network Division. It is possible that these data may not yet be included in the published descriptions. File the recent descriptions with the appropriate 30′ quadrant, and remember to use them. A reset mark may be mistaken for the original one if the mark setter is unaware of the complete history of the control point.

Descriptions for tide and water-level stations are available from the NOS Tides and Water Levels Division. Each station is identified by a unique seven-digit number. The first three digits identify the State or body of water. A list of these codes is shown in table 2-2. Station listings are published in *NOAA Tide Tables: High and Low Water Levels* (National Ocean Survey, annual publication). Request the most recent station reports for all tide and water-level stations in a project area. Station reports include the name, address, and telephone number of the station observer, bench mark descriptions, and a sketch of the station showing bench mark locations.

Leveling to specific control points may be specially requested either in the project instructions or by local agencies. The points may include junction points, international boundary monuments, airport monuments, compaction meters, deep-well casings, and monuments of USGS, State highway departments, counties, and cities. If the necessary descriptions are not provided with project instructions, the descriptions should be obtained from the appropriate agency.

Maps for position plots and logs. Positions of all points recovered or established must be plotted on the standard 7.′5 series of topographic maps available from USGS. If 7.′5 maps are not available, the 15′ series should be used. Because plots are difficult to see on orthophotographic maps, maps of this type should not be used.

The U.S. Geological Survey may be revising or compiling new 7.′5 maps for the project area. Check with USGS and request State progress maps, available from the USGS regional topographic division. Up-to-date 7.′5 "bluelines," which are preliminary compilations of topographic maps, can then be ordered.

In major metropolitan areas, logs should be prepared on up-to-date city or county street maps, available from the Chamber of Commerce, the city government, or local gasoline stations. However, plots must still be made on 7.′5 maps to ensure consistent and accurate scaling of positions.

2.3.2 Liaison

While routing the leveling line, a mark setting unit must communicate frequently with the owners, managers, and users of both public and private property. The purpose of these contacts is to obtain permission and make detailed arrangements for both mark setters and leveling units to work in the areas necessary for completion of the project. In addition, a systematic liaison effort may bring to light unusual environmental conditions or ongoing projects of other agencies which affect the choice of leveling routes. Maintaining good public relations is vital to the project.

The project director lays the groundwork for future cooperation and coordination by notifying the appropriate State highway or transportation department in advance of the project. The project director should send a letter requesting permission to operate in the right-of-way zones of the necessary State highways, plus a courtesy copy of the project instructions. If routes include railroad right-of-way zones, follow a similar procedure. The project instructions may list other agencies or individuals to contact.

Immediately after arrival in the project area, the mark setting foreman should visit officials of the agencies with which liaison is necessary. To set bench marks of the best possible quality, visit agencies and individuals who may provide maps or other specific information about the geology along the proposed routes. Determine if surveys exist that should be connected to the national network and inform the appropriate mark setting units. In addition, make arrangments to obtain sufficient supplies during the project.

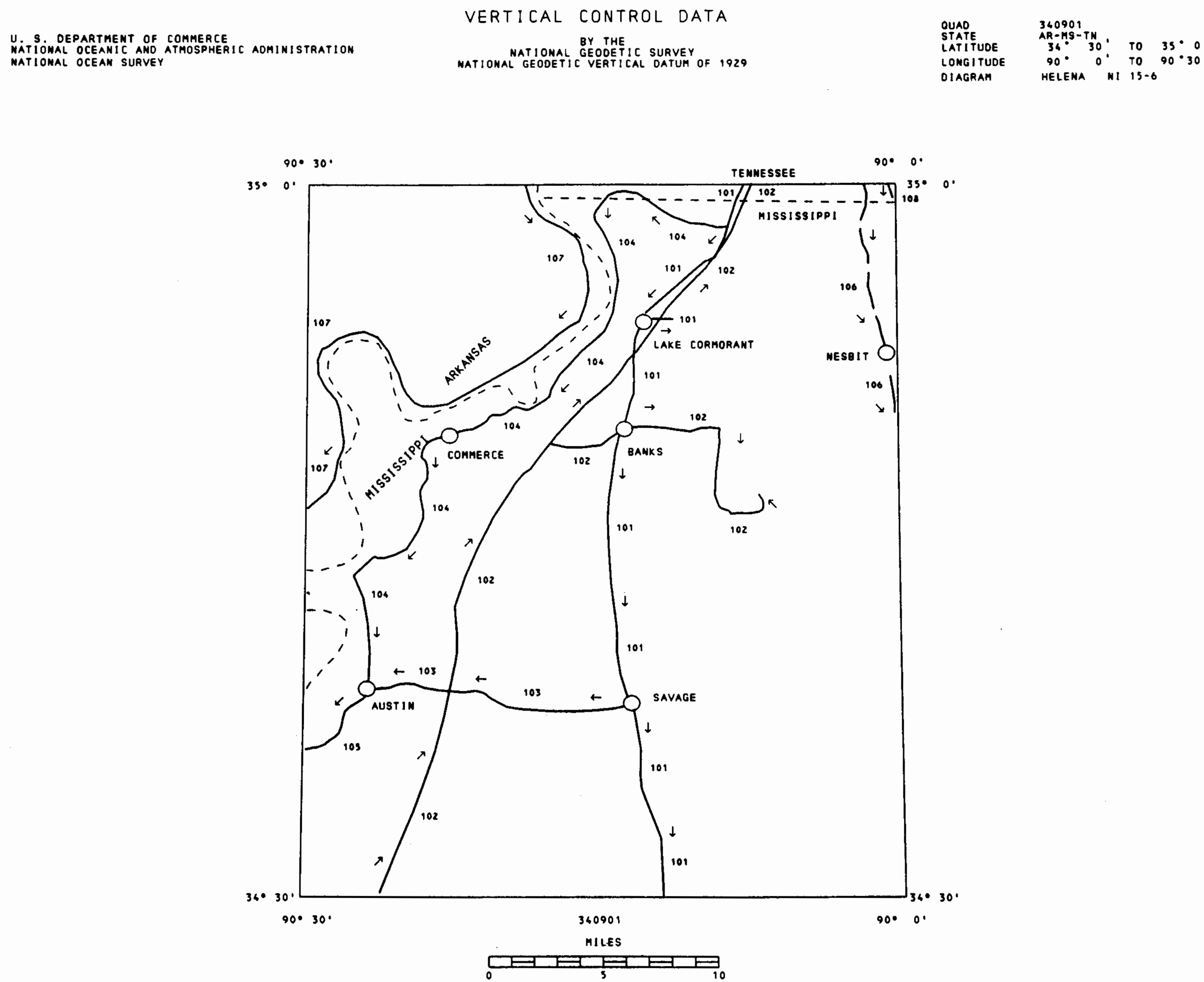

Figure 2-7a.—Index plot for 30′ quadrant of Vertical Control Data.

```
                                                                                        SEQN--051
US DEPARTMENT OF COMMERCE - NOAA              VERTICAL CONTROL DATA                QUAD--N34090100 LINE--101
NOS - NATIONAL GEODETIC SURVEY          NATIONAL GEODETIC VERTICAL DATUM 1929      STATE--MS  DIAGRAM--NI 15-6
ROCKVILLE MD 20852 - APR 1980             ADJUSTED BY--CGS    YEAR--1957           COUNTY--TATE
                                                   SOURCE--L15801

      BENCH MARK                               ORDER--1ST  MONUMENTATION QUALITY--B   APPROX  LAT  34-37-47N
      DESIGNATION--V 194                       ESTABLISHED BY--CGS    YEAR--1956    POSITION--LON 090-13-22W

          H - ELEVATION                                                                       NORMAL
         ABOVE NGVD 1929          MODELED BOUGUER       MODELED       NORMAL GRAVITY  GEOPOTENTIAL NUMBER
     (NORMAL ORTHOMETRIC HEIGHT)  ANOMALY   SIGMA   SURFACE GRAVITY   (1967 FORMULA)   (GPU=KILOGALMETER)
   58.238 METERS (   191.069 FEET)  -7.7 MGALS 0.9    979.682 GALS       979.702 GALS         57.056 GPUS

      ****************************** B E N C H   M A R K   R E C O V E R Y ******************************
  DESIGNATION--V 194                            STATE--MS  COUNTY--TATE                    QUAD--N340901 XRN--EH0051
  *************** MONUMENT BY--CGS **************** YR--1956 COP--UNK  MARK TYPE--BENCH MARK DISK **************
  *************** RECOVERY BY--MSHD **************** YR--1966 COP--UNK  CONDITION--GOOD **************************
                  STAMPING--V 194 (1956)
                  SETTING--PIER
                  LOCATED--0.3 MI SE     FROM THE CITY OR TOWN OF--SAVAGE
                  ********************************************************************************
      TO REACH FROM THE U.S. POST OFFICE IN CRENSHAW GO NORTH ON MISSISSIPPI HIGHWAY 3 FOR 7.55 MILES TO
      MISSISSIPPI HIGHWAY 4 ON THE RIGHT, CONTINUE NORTHWEST ON HIGHWAY 3 FOR 2.1 MILES TO THE BRIDGE OVER
      THE RAILROAD TRACK AND THE MARK SET VERTICALLY IN THE SOUTH FACE OF THE CONCRETE BASE OF THE FIRST
      PIER WEST OF THE TRACK.  IT IS 35.7 FEET WEST OF THE WEST RAIL OF THE MAIN TRACK AND 22 FEET WEST OF
      THE WEST RAIL OF THE SIDE TRACK.


                                                                                        SEQN--052
US DEPARTMENT OF COMMERCE - NOAA              VERTICAL CONTROL DATA                QUAD--N34090100 LINE--101
NOS - NATIONAL GEODETIC SURVEY          NATIONAL GEODETIC VERTICAL DATUM 1929      STATE--MS  DIAGRAM--NI 15-6
ROCKVILLE MD 20852 - APR 1980             ADJUSTED BY--CGS    YEAR--1957           COUNTY--TATE
                                                   SOURCE--L15801

      BENCH MARK                               ORDER--1ST  MONUMENTATION QUALITY--D   APPROX  LAT  34-36-50N
      DESIGNATION--W 194                       ESTABLISHED BY--CGS    YEAR--1956    POSITION--LON 090-13-11W

          H - ELEVATION                                                                       NORMAL
         ABOVE NGVD 1929          MODELED BOUGUER       MODELED       NORMAL GRAVITY  GEOPOTENTIAL NUMBER
     (NORMAL ORTHOMETRIC HEIGHT)  ANOMALY   SIGMA   SURFACE GRAVITY   (1967 FORMULA)   (GPU=KILOGALMETER)
   55.995 METERS (   183.710 FEET)  -6.5 MGALS 0.8    979.683 GALS       979.700 GALS         54.858 GPUS

      ****************************** B E N C H   M A R K   R E C O V E R Y ******************************
  DESIGNATION--W 194                            STATE--MS  COUNTY--TATE                    QUAD--N340901 XRN--EH0052
  *************** MONUMENT BY--CGS **************** YR--1956 COP--UNK  MARK TYPE--BENCH MARK DISK **************
  *************** RECOVERY BY--MSHD **************** YR--1966 COP--UNK  CONDITION--GOOD **************************
                  STAMPING--W 194 (1956)
                  SETTING--SHALLOW-SET PIPE
                  LOCATED--1.4 MI SOUTH  FROM THE CITY OR TOWN OF--SAVAGE
                  ********************************************************************************
      TO REACH FROM THE U.S. POST OFFICE IN CRENSHAW GO NORTH ON MISSISSIPPI HIGHWAY 3 FOR 7.55 MILES TO
      MISSISSIPPI HIGHWAY 4 ON THE RIGHT, CONTINUE NORTHWEST ON HIGHWAY 3 FOR 1.0 TO A SIDE ROAD LEFT
      RUNNING BETWEEN A GROCERY STORE AND A STORAGE GARAGE, TURN LEFT (WEST) AND CONTINUE ON AN
      ALL-WEATHER ROAD FOR 0.15 MILE TO A TENANT HOUSE ON THE RIGHT, CONTINUE WEST ON A FIELD ROAD FOR
      0.25 MILE TO THE RAILROAD TRACK, TURN LEFT SOUTH AND FOLLOW THE WEST EDGE OF A FIELD FOR 0.35 MILE
      TO THE MARK ON THE RIGHT AS DESCRIBED.  IT IS APPROXIMATELY 185 YARDS NORTH OF MILEPOST 41, 110 FEET
      SOUTHWEST OF POLE NO. 725, 57.5 FEET WEST-NORTHWEST OF POLE NO. 725 1/2, 23 FEET WEST OF THE WEST
      RAIL, 4.3 FEET SOUTHEAST OF A TELEPHONE POLE, AND 1.6 FEET SOUTH OF A METAL WITNESS POST.
```

Figure 2-7b.—Vertical Control Data sheet.

US DEPARTMENT OF COMMERCE - NOAA
NOS - NATIONAL GEODETIC SURVEY
ROCKVILLE MD 20852 - APR 1980

ALPHABETIC INDEX TO
VERTICAL CONTROL DATA
NATIONAL GEODETIC VERTICAL DATUM 1929

LAT SPAN 34-30.1N TO 34-59.9N
LON SPAN 90-00.1W TO 90-29.8W
DIAGRAM--NI 15-6 INDEX PG--06

SEQN	DESIGNATION	YEAR ESTABLISHED	LAST RECOV	ORD	ELEVATION (METERS)	SOURCE	APPROXIMATE POSITION LATITUDE	APPROXIMATE POSITION LONGITUDE	SURFACE GRAVITY	OTHER CONTROL	QUAD	QSN	ST
48	T 117 USE	UNK D	1966G	1ST	57.354	L15801	34-38-03N	90-13-35W	979.682		N34090100		MS
33	T 193	1956D	1966G	1ST	58.406	L15801	34-45-09N	90-14-38W	979.686		N34090100		MS
44	T 194	1956D	1966G	1ST	56.160	L15801	34-39-26N	90-14-08W	979.683		N34090100		MS
66	T 195	1956D	1966G	1ST	56.793	L15801	34-30-17N	90-11-42W	979.679		N34090100		MS
66	T 195 USGS	1956D	1966G	1ST	56.793	L15801	34-30-17N	90-11-42W	979.679		N34090100		MS
164	T 196	1956D	1966G	1ST	57.771	L15798	34-38-28N	90-26-22W	979.687		N34090100		MS
211	T 218	1976B		1ST	67.481	L24103	34-51-47N	90-21-22W	979.704		N34090100		AR
226	T 219	UNK B	1976G	1ST	58.989	L24103	34-49-01N	90-28-24W	979.709		N34090100		AR
131	T 238	1974C	1976G	1ST	61.147	L23280	34-50-24N	90-16-23W	979.696		N34090100		MS
126	T 243	1974C		1ST	89.054	L23280	34-46-57N	90-07-08W	979.681		N34090100		MS
146	T 260	1974C		1ST	74.821	L23280	34-59-42N	90-07-30W	979.712		N34090100		TN
54	TBM A STA 146	1953D	1966G	1ST	57.262	L15801	34-35-56N	90-13-00W	979.682		N34090100		MS
54	TBM A STA 146 USE	1953D	1966G	1ST	57.262	L15801	34-35-56N	90-13-00W	979.682		N34090100		MS
55	TBM B STA 146	1953D	1966G	1ST	57.319	L15801	34-35-56N	90-13-01W	979.682		N34090100		MS
55	TBM B STA 146 USE	1953D	1966G	1ST	57.319	L15801	34-35-56N	90-13-01W	979.682		N34090100		MS
174	TBM 44/45	UNK D	1966N	1ST	58.574	L00422	34-37-32N	90-28-08W	979.684		N34090100		MS
174	TBM 44/45 USE	UNK D	1966N	1ST	58.574	L00422	34-37-32N	90-28-08W	979.684		N34090100		MS
183	TBM 46/47	UNK D	1966N	1ST	57.356	L00422	34-36-02N	90-29-05W	979.681		N34090100		MS
183	TBM 46/47 USE	UNK D	1966N	1ST	57.356	L00422	34-36-02N	90-29-05W	979.681		N34090100		MS
35	U 193	1956D	1966G	1ST	57.642	L15801	34-44-25N	90-14-36W	979.686		N34090100		MS
45	U 194	1956D	1966G	1ST	55.362	L15801	34-38-40N	90-14-02W	979.683		N34090100		MS
65	U 195	1956D	1966G	1ST	54.124	L15801	34-31-04N	90-11-41W	979.680		N34090100		MS
165	U 196	1956C	1974G	1ST	59.402	L23280	34-38-27N	90-26-57W	979.686		N34090100		MS
212	U 218	1976B		1ST	67.214	L24103	34-51-31N	90-22-22W	979.704		N34090100		AR
227	U 219	1976B		1ST	66.326	L24103	34-48-19N	90-28-57W	979.706		N34090100		AR
132	U 238	1974C	1976G	1ST	61.488	L23280	34-50-45N	90-16-54W	979.697		N34090100		MS
139	U 243	1974C		1ST	61.652	L23280	34-54-41N	90-11-21W	979.702		N34090100		MS
1	U 75	1956D		1ST	71.125	L15801	34-59-55N	90-08-06W	979.714		N34090100		TN
36	V 193	1956D	1966G	1ST	58.295	L15801	34-43-29N	90-14-35W	979.685		N34090100		MS
51	V 194	1956B	1966G	1ST	58.238	L15801	34-37-47N	90-13-22W	979.682		N34090100		MS
64	V 195	1956D	1966G	1ST	57.061	L15801	34-31-58N	90-11-41W	979.680		N34090100		MS
213	V 218	1976B		1ST	67.183	L24103	34-51-22N	90-23-23W	979.705		N34090100		AR
133	V 238	1974B	1976G	1ST	62.172	L24103	34-50-46N	90-17-48W	979.698		N34090100		MS
140	V 243	1974C		1ST	61.631	L23280	34-55-18N	90-10-47W	979.703		N34090100		MS
5	W 193	1956D	1974N	1ST	62.927	L15801	34-57-49N	90-09-01W	979.710		N34090100		MS
52	W 194	1956D	1966G	1ST	55.995	L15801	34-36-50N	90-13-11W	979.682		N34090100		MS
62	W 195	1956D	1966G	1ST	55.713	L15801	34-32-34N	90-11-49W	979.681		N34090100		MS
82	W 237	1974B		1ST	57.969	L23280	34-38-26N	90-22-33W	979.687		N34090100		MS
135	W 238	1974C	1976G	1ST	60.369	L23280	34-51-01N	90-15-36W	979.696		N34090100		MS
141	W 243	1974C		1ST	64.680	L23280	34-55-51N	90-09-59W	979.703		N34090100		MS

Figure 2-7c.—Alphanumeric indexes for 30′ quadrant of Vertical Control Data.

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SEQN	DESIGNATION	YEAR ESTABLISHED	LAST RECOV	ORD	ELEVATION (METERS)	SOURCE	APPROXIMATE POSITION LATITUDE	LONGITUDE	SURFACE GRAVITY	OTHER CONTROL	QUAD	QSN	ST
41	E 194	1956B	1966G	1ST	57.239	L15801	34-41-56N	90-14-12W	979.685		N34090100		MS
42	R 194	1956D	1966G	1ST	56.688	L15801	34-41-06N	90-14-13W	979.684		N34090100		MS
43	S 194	1956D	1966G	1ST	56.547	L15801	34-40-16N	90-14-10W	979.684		N34090100		MS
44	T 194	1956D	1966G	1ST	56.160	L15801	34-39-26N	90-14-08W	979.683		N34090100		MS
45	U 194	1956D	1966G	1ST	55.362	L15801	34-38-40N	90-14-02W	979.683		N34090100		MS
46	186	UNK D	1966X	1ST	56.668	00726	34-38-25N	90-13-51W	979.683		N34090100		MS
47	T 116	UNK D	1966G	1ST	58.793	L15801	34-38-10N	90-13-38W	979.682		N34090100		MS
48	T 117	UNK D	1966G	1ST	57.354	L15801	34-38-03N	90-13-35W	979.682		N34090100		MS
49	F 194	1956B	1966G	1ST	56.599	L15801	34-38-04N	90-13-32W	979.682		N34090100		MS
50	PTS 30	1908D	1966G	1ST	56.854	L15801	34-38-01N	90-13-34W	979.682		N34090100		MS
51	V 194	1956B	1966G	1ST	58.238	L15801	34-37-47N	90-13-22W	979.682		N34090100		MS
52	W 194	1956D	1966G	1ST	55.995	L15801	34-36-50N	90-13-11W	979.682		N34090100		MS
53	M 194	1956D	1966G	1ST	56.987	L15801	34-35-59N	90-13-03W	979.682		N34090100		MS
54	TBM A STA 146	1953D	1966G	1ST	57.262	L15801	34-35-56N	90-13-00W	979.682		N34090100		MS
55	TBM B STA 146	1953D	1966G	1ST	57.319	L15801	34-35-56N	90-13-01W	979.682		N34090100		MS
56	183	UNK C	1956N	1ST	55.768	00726	34-35-39N	90-12-59W	979.682		N34090100		MS
57	Z 195	1956D	1966G	1ST	56.316	L15801	34-35-00N	90-12-51W	979.682		N34090100		MS
58	Y 195	1956D	1966G	1ST	56.819	L15801	34-34-43N	90-12-49W	979.682		N34090100		MS
59	P-14-13	1940D	1966G	1ST	61.029	L15801	34-34-08N	90-12-30W	979.681		N34090100		MS
60	H 194	1954B	1966G	1ST	60.886	L15801	34-34-09N	90-12-30W	979.681		N34090100		MS
61	X 195	1956D	1966G	1ST	56.696	L15801	34-33-28N	90-12-10W	979.681		N34090100		MS
62	W 195	1956D	1966G	1ST	55.713	L15801	34-32-34N	90-11-49W	979.681		N34090100		MS
63	PTS 32	UNK D	1966X	1ST	55.306	00726	34-32-09N	90-11-42W	979.680		N34090100		MS
64	V 195	1956D	1966G	1ST	57.061	L15801	34-31-58N	90-11-41W	979.680		N34090100		MS
65	U 195	1956D	1966G	1ST	54.124	L15801	34-31-04N	90-11-41W	979.680		N34090100		MS
66	T 195	1956D	1966G	1ST	56.793	L15801	34-30-17N	90-11-42W	979.679		N34090100		MS
67	S 195	1956D	1966G	1ST	56.381	L15801	34-30-11N	90-11-51W	979.679		N34090100		MS
68	PTS 33	1908D	1967G	1ST	56.891	L15801	34-30-10N	90-11-39W	979.679		N34090100		MS
69	CONCRETE MON 2	UNK D	1956N	1ST	57.619	00726	34-30-07N	90-11-47W	979.679		N34090100		MS
70	H 245	1974B		1ST	54.296	L23280	34-30-30N	90-25-53W	979.677		N34090100		MS
71	C 245	1974D		1ST	54.362	L23280	34-31-33N	90-25-30W	979.679		N34090100		MS
72	D 245	1974C		1ST	55.091	L23280	34-32-22N	90-25-13W	979.680		N34090100		MS
73	F 245	1974C		1ST	56.735	L23280	34-33-13N	90-24-54W	979.681		N34090100		MS
74	G 245	1974D		1ST	55.160	L23280	34-34-13N	90-24-32W	979.683		N34090100		MS
75	J 245	1974B		1ST	56.363	L23280	34-34-59N	90-24-15W	979.683		N34090100		MS
76	CLAYTON RM 1	1956C	1974G	1ST	58.021	L23280	34-35-56N	90-23-53W	979.684		N34090100		MS
77	CLAYTON	1967C	1974G	1ST	57.727	L23280	34-35-51N	90-23-55W	979.684	H	N34090133	0002	MS
78	CLAYTON RM 2	1956C	1974G	1ST	57.790	L23280	34-35-56N	90-23-55W	979.684		N34090100		MS
79	CLAYTON AZ	1956C	1974G	1ST	57.555	L23280	34-36-18N	90-23-46W	979.685		N34090100		MS
80	K 245	1974C		1ST	57.281	L23280	34-37-10N	90-23-27W	979.686		N34090100		MS

Figure 2-7d.—Line order index for 30′ quadrant.

DESIGNATION	ACRN #	POSITION
A 10	BG0922	0300864
A 10 AZ MK	BG0921	0300864
A 10 BM	BG0524	0300864
A 10 RM 1	BG0923	0300864
A 10 RM 2	BG0924	0300864
A 11	BG2640	0300874
A 113	BG0222	0300871
A 115	BG0097	0300871
A 124	BG1001	0300864
A 125	BG1035	0300864
A 136	BG1036	0300864
A 137	BG2035	0300863
A 138	BG1115	0300864
A 139	BG1636	0300861
A 140	BG1458	0300861
A 141	BG1434	0300861
A 156	BG0907	0300864
A 158	BG0002	0300871
A 16 RM 1 A VITRO	BG1021	0300864
A 16 RM 1 B VITRO	BG1022	0300864
A 16 RM 2 A VITRO	BG1019	0300864
A 16 RM 2 B VITRO	BG1020	0300864
A 16 VITRO	BG1018	0300864
A 161	BG1817	0300872
A 162	BG0042	0300871
A 163	BG1706	0300872
A 163 RESET 1964	BG1707	0300872
A 165	BG0257	0300871
A 168	BG0498	0300864
A 169	BG2615	0300864
A 178	BG2255	0300802
A 179	BG2073	0300863
A 180	BG2081	0300863
A 181	BG1466	0300861
A 182	BG0207	0300862
A 182 RESET 1962	BG2275	0300862
A 25	BG1711	0300872
A 26	BG1772	0300872
A 27	BG2156	0300862
A 28	BG0800	0300864
A 29	BG1120	0300864
A 297	BG1974	0300863
A 299	BG1921	0300863
A 302	BG0137	0300871
A 303	BG0125	0300871
A 380	BG3435	030087123
A 395	BG3462	030086341
A 409	BG2644	0300874
A 436	BG2495	0300874
A 437	BG2528	0300874
A 84	BG0025	0300871
A 85	BG0387	0300864

Figure 2-8a.—Listing of synoptic file.

ACRN #	DESIGNATION	POSITION
BG0001	I 8	0300871
BG0002	A 158	0300871
BG0003	B 158	0300871
BG0004	J 8	0300871
BG0005	C 158	0300871
BG0006	K 8	0300871
BG0007	D 158	0300871
BG0008	E 158	0300871
BG0009	L 8	0300871
BG0010	F 158	0300871
BG0011	G 158	0300871
BG0012	M 8	0300871
BG0013	H 158	0300871
BG0014	N 8	0300871
BG0015	K 158	0300871
BG0016	J 158	0300871
BG0017	O 8	0300871
BG0018	L 158	0300871
BG0019	P 8	0300871
BG0020	M 158	0300871
BG0021	N 158	0300871
BG0022	Q 8	0300871
BG0023	P 158	0300871
BG0024	Q 158	0300871
BG0025	A 84	0300871
BG0026	B 84 RESET 1939	0300871
BG0027	B 84	0300871
BG0028	C 84 RESET 1939	0300871
BG0029	C 84	0300871
BG0030	D 84	0300871
BG0031	D 84 RESET 1939	0300871
BG0032	E 84 RESET 1939	0300871
BG0033	E 84	0300871
BG0034	F 84	0300871
BG0035	G 84	0300871
BG0036	H 84	0300871
BG0037	J 84	0300871
BG0038	K 84	0300871
BG0039	L 84	0300871
BG0040	R 158	0300871
BG0041	S 158	0300871
BG0042	A 162	0300871
BG0043	B 162	0300871
BG0044	T 8	0300871
BG0045	C 162	0300871
BG0046	U 8	0300871
BG0047	D 162	0300871
BG0048	QUINTETTE AZ MK	0300871
BG0049	QUINTETTE 2 AZ MK	0300871
BG0050	QUINTETTE RM 1	0300871
BG0051	QUINTETTE RM A	0300871
BG0052	QUINTETTE RM 2	0300871
BG0053	E 162	0300871

Figure 2-8b.—Listing of synoptic file.

Table 2-2.—Location codes for tide and water-level stations

States	Codes	States	Codes
Alabama (AL)	873	Montana (MT)	931
Alaska (AK)	945	Nebraska (NE)	924
Arizona (AZ)	938	Nevada (NV)	936
Arkansas (AR)	881	New Hampshire (NH)	842
California (CA)	941	New Jersey (NJ)	853
Colorado (CO)	934	New Mexico (NM)	937
Connecticut (CT)	846	New York (NY)	851
Delaware (DE)	855	North Carolina (NC)	865
District of Columbia (DC)	859	North Dakota (ND)	922
Florida (FL)	872	Ohio (OH)	892
Georgia (GA)	867	Oklahoma (OK)	927
Hawaii (HI)	161	Oregon (OR)	943
Idaho (ID)	932	Pennsylvania (PA)	854
Illinois (IL)	896	Rhode Island (RI)	845
Indiana (IN)	895	South Carolina (SC)	866
Iowa (IA)	912	South Dakota (SD)	923
Kansas (KS)	925	Tennessee (TN)	882
Kentucky (KY)	883	Texas (TX)	877
Louisiana (LA)	876	Utah (UT)	935
Maine (ME)	841	Vermont (VT)	843
Maryland (MD)	857	Virginia (VA)	863
Massachusetts (MA)	844	Washington (WA)	944
Michigan (MI)	901	West Virginia (WV)	864
Minnesota (MN)	911	Wisconsin (WI)	902
Mississippi (MS)	874	Wyoming (WY)	933
Missouri (MO)	884	United States	83x

Channels	Codes	Lakes	Codes
St. Lawrence River	831-1xxx	Lake Champlain	843-1xxx
Niagara River	906-3xxx	Lake Ontario	905-2xxx
Detroit River	904-4xxx	Lake Erie	906-3xxx
St. Clair River	901-4xxx	Lake St. Clair	903-4xxx
Saginaw River	901-xxxx	Lake Huron	907-5xxx
Grand River	908-xxxx	Lake Michigan	908-7xxx
Chicago River	896-1111	Lake Winebago	902-02xx
Fox River	902-01xx	Lake Superior	909-9xxx
Milwaukee River	902-03xx	Rainy Lake	911-01xx
St. Marys River	907-6xxx	Lake of the Woods	911-09xx
Rainy River	911-05xx		

A mark setting unit should also contact officials of the local highway districts, county and city transportation departments, Indian reservations, and other public and private agencies as required when traversing their property. Ask district officials about current and proposed highway widening projects. Enlist their assistance to avoid the effects of such projects when routing the line. Learn the construction details of culverts and bridges, and the locations of bedrock. Often, local officials can point out or specify existing monuments that will be destroyed, assist in recovery work, and recommend suitable locations for new bench marks.

Since highway right-of-way zones are often occupied by gas, water, or sewer pipelines, and electric or telephone cables, determine the location of such utilities before installing a monument. Look for signs marking pipelines or cables and obtain the assistance of a representative from the local servicing office if a rod must be driven near a pipe or cable.

Although most vertical control points are located on public land, occasional points may require that a private property owner or manager be approached as well. Sometimes, in addition to the owner's permission, special information must be obtained, such as the age and construction details of a building or the presence of active irrigation wells adjacent to a proposed monument site. When ownership cannot be readily determined, consult the county Recorder of Deeds. Never search for or establish a control point without first obtaining the property owner's permission.

When requesting permission from property owners to use their property, give them a fact sheet that describes the purpose of the survey. The fact sheet should also explain how the monument will be constructed and the importance of the monument to the vertical control network. An example is shown in figure 2-9. Always behave in a respectful and tactful manner and obey the homeowners' wishes concerning the use of their property.

Maintain a record of each contact made, including the designations of the relevant control points and the

UNITED STATES DEPARTMENT OF COMMERCE
National Oceanic and Atmospheric Administration
NATIONAL OCEAN SURVEY
Rockville, Md. 20852

NATIONAL GEODETIC SURVEY - INFORMATION NOTICE

The National Geodetic Survey (NGS), an office of the National Ocean Survey, NOAA, Department of Commerce, will be conducting a Geodetic Survey in your area in the near future. In preparation for this work, NGS field units are now installing survey points, known as bench marks, at intervals of approximately one mile along major highways and other thoroughfares. These bench marks will be tied to the national network of thousands of similar points throughout the United States which form the framework for a precise network of vertical control stations (points of precisely known elevations). This survey work is undertaken at the request of Federal, state, and local agencies for use in such projects as mapping, resource management, flood control, crustal motion studies (geodynamics), space exploration and large-scale engineering projects.

At selected locations throughout the country, special high quality bench marks are being established as part of a seven-year program to redefine the entire national geodetic vertical control network. These marks often take the form of brass disks cemented into bedrock outcrops or large buildings. Where this kind of mark is not feasible, a special truck-mounted drill is used to install a sleeved rod bench mark (as described in Figure 1.) Due to the stringent accuracy requirements placed on the elevations of these bench marks, particular care is taken when they are established. The mark must be well constructed, unlikely to be disturbed, and accessible to the general public.

Because of the above considerations, selection of an acceptable site is crucial. Thus, it is often necessary to rely on the public spirit of land owners for permission to establish survey stations on private property. It is the policy of the National Geodetic Survey to install bench marks only upon the informed consent of the property owners. An NGS bench mark can provide a valuable public service, and enhance property values by serving as a reference for any surveying done in the immediate area.

Once it is installed the elevation is precisely determined by a National Geodetic Survey vertical control party. This elevation is then published along with the description of the physical location of the bench mark and its position (latitude and longitude). The bench mark then takes its place in the national network as a basic reference point for a variety of surveying, mapping, planning, engineering and scientific applications.

Enclosure

Figure 2-9a.—Form letter distributed to property owners.

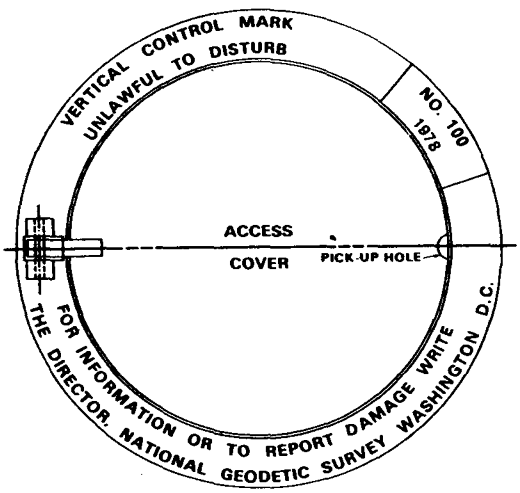
NATIONAL GEODETIC SURVEY VERTICAL CONTROL MARKER

In 1978 the National Geodetic Survey (NGS) introduced a new, improved bench mark into the National Vertical Control Network. The reference point for the elevation is the top of a stainless steel rod. The rod is located inside a protective aluminum casement that bears the NGS logo and the stamped bench mark designation. Users can obtain access to the rod by lifting a hinged access cover.

The bench mark is designed to prevent near-surface soil movements, e.g., frost heave, shrinkage, swelling, from disturbing the mark. This is accomplished by encasing the rod in a lubricated sleeve to the depth of expected soil movement and by anchoring the rod in the soil below.

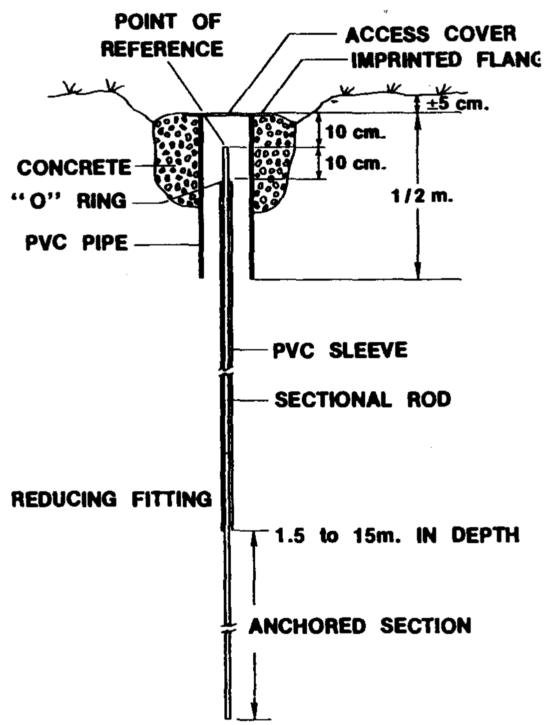
Top and side views of the bench mark are depicted below. Additional information about this mark can be obtained by writing to the Director, National Geodetic Survey, National Ocean Survey, NOAA, Rockville, Md. 20852.

TOP VIEW



BENCH MARK LOGO FLANGE & ACCESS COVER

SIDE VIEW



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

Figure 2-9b.—Form letter distributed to property owners.

name, address, and telephone number of the individual contacted. Upon completion of the project, this record is used to send published descriptions of the points to private owners and public managers as a courtesy. Requests for survey results and elevation data should be referred to the National Geodetic Information Center.

2.3.3 Routing Specifications

The actual routes to be followed by the lines of a geodetic leveling project depend primarily on the purpose of the project, as do the types and spacing of control points along each route. Given in this section are routing specifications appropriate to lines of the National Geodetic Vertical Network.

Junctions. Lines in a network begin and end at clusters of control points called junctions. In a large network, considerable time (sometimes several years) may elapse between the completion of one line and the start of another; therefore, the monuments at junctions must be permanent and stable. The most desirable locations for junctions are areas without significant subsidence or uplift relative to a majority of the area covered by the network. Though cities often serve as the logical intersections for lines routed along highways, cities may not be the best locations for junctions. Locate a junction in the most stable zone possible, even if it is necessary to detour as much as 10 km (6 mi) from the logical intersection of the lines. (See the examples in fig. 2-10.)

To establish the junction, recover or set three control points of the best possible quality. Locate them at intervals of at least 0.5 km (0.3 mi). This limits the possibility that they might all be affected by the same local disturbance. At an international boundary, locate the points along the border, if possible, so they are accessible to surveyors from either country. All three points must be leveled in every line beginning or ending at the junction.

At the end of the field season or project, if the leveling along a line must stop at some location short of the ending junction, the location should be chosen carefully and monmented with three control points of the best possible quality, as described in the previous paragraph. These are not strictly considered to be junction points, but they should be releveled for a tie when the line is resumed.

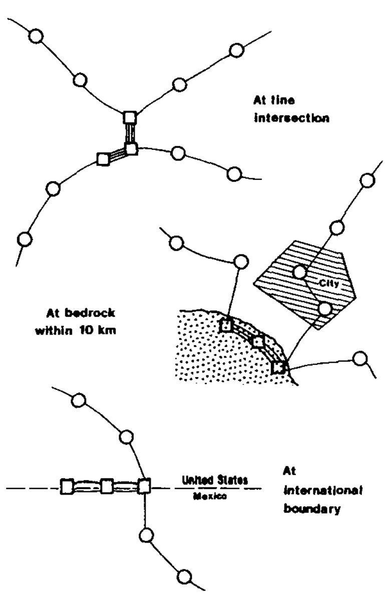


Figure 2-10.—Network junctions.

Routes. The mark setter must weigh many factors when routing the line. Paramount among these are the permanence, stability, and accessibility of the control points. Choose a route which permits the greatest proportion of control points to meet these standards. Since the lines of the national network are, for the most part, already in place, routing typically involves recovering or relocating an existing line.

In the past, lines of leveling usually followed railroad right-of-way zones, which often provided the only negotiable path through undeveloped areas. Today, these zones are not always accessible or convenient to the person who will be using local vertical control. The improvements in mark setting and leveling efficiency made possible by the use of vehicles depend on greater accessibility. Therefore, whenever a railroad does not have a continuous access road available to both public and private surveyors within five leveling setups (at the most 0.5 km or 0.3 mi), relocate the line to the edge of a highway right-of-way zone which parallels the railroad. Special connections should be made to the existing line. (See sec. 2.3.4, "Previous surveys.")

In the vicinity of an interstate highway or other controlled access roads, whenever possible, route the line so it is accessible from parallel roads outside the right-of-way fence. Avoid routing the line along the proposed route of a new highway, but if this is unavoidable, route the line as close as possible to the edges of the proposed right-of-way zones.

Leveling experience is helpful when selecting a route across rough terrain or through congested urban areas. Unlike a control point for triangulation, a control point for leveling must be readily accessible by land from the points that exist before and after it in the line. Avoid routing the line through short, extreme changes in elevation, such as down gullies or up and over embankments. When a monument must be set in such an inconvenient location to ensure permanence and stability, note the most efficient leveling route on the log.

If an obstacle is encountered that cannot be crossed by ordinary leveling—such as a river, bay, mudflat, marsh, or deep ravine—a "river crossing" technique may be required. This procedure may also be required if a bridge cannot be leveled across, either because it is subject to intense vibration or because traffic cannot be stopped for the length of time necessary. See sec. 4.2.1 for selecting and preparing river crossing sites.

Spurs. Portions of line that branch to control points not in the main line are called spurs. Spurs end on points not otherwise connected to the network, unless they are routed to make connections with previous surveys. Avoid creating spurs of less than 1.0 km (0.6 mi). Control points located closer than 1.0 km should be leveled as part of the line.

If routing a new line along an existing line that includes spurs, the spurs may be excluded unless they provide access to control points of unusually high quality or they are specified for inclusion by the project instructions.

Control points. The spacing between control points in the national network should average 1.6 km (1.0 mi). In dense networks, designed to measure such phenomena as movement near faults or local subsidence, spacing should average 1.0 km (0.6 mi) or less.

The distance between any pair of control points may normally range between 1.0 and 3.0 km (0.6 and 1.9 mi). In municipalities, housing or commercial zones, and other areas of intensive development, shorten the spacing to 1.0 km. In rural areas, with gentle terrain and only infrequent periods of windy or stormy weather, spacing may be increased.

If a portion of the line must follow a steep grade, space the control points to limit the number of leveling setups required to 40 or less. If the steep grade extends over several kilometers or the entire line, adopt an average spacing which corresponds to the slope. Estimate the slope (elevation difference divided by distance) from a topographic map and consult table 2-3 for a suitable spacing.

Control points should be marked with monuments of the best expected quality. However, new monuments of the best quality may be the most expensive and time-consuming to set. Furthermore, existing monuments of lower expected quality historically have often performed well and should not be removed from the line unless they are in unsatisfactory condition. Consequently, best-quality monuments need not be installed at every control point. Unless project instructions specify otherwise, recover or set a monument of the best possible quality at the following locations (fig. 2-11):

1. At network junctions, including international boundaries (set three monuments).
2. At one or both ends of a portion of line not to be leveled to both junctions during one field season (set three).
3. At the base of a spur or spurs connecting a previous survey that intersects the line (set one).

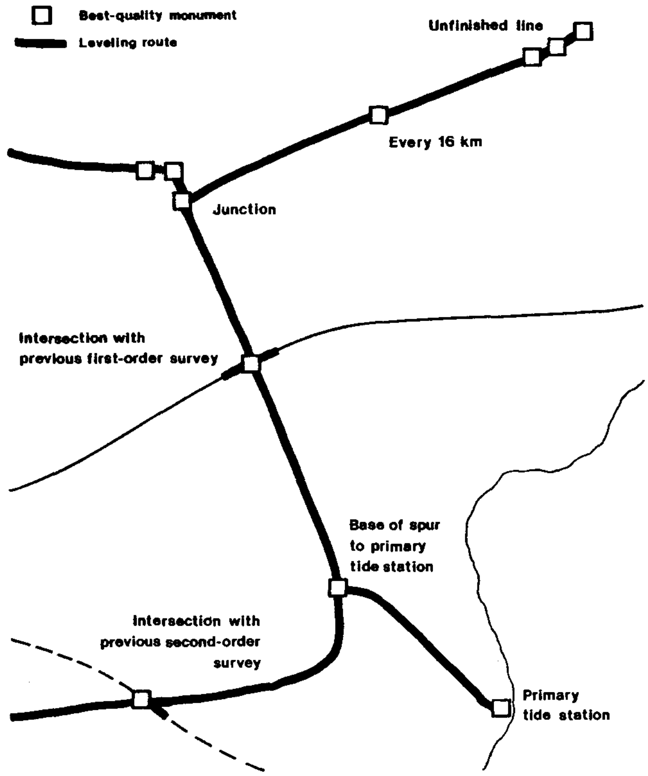


Figure 2-11.—Locations for monuments of the best possible quality.

4. At the base of a spur connecting a primary tide station (set one).
5. At a primary tide station (set one).
6. At least every 16 km (10 mi) along the line (set one).

2.3.4 Connections

To provide a common frame of reference in which elevations may be computed, two or more ties should be established between the line or lines of every level-

Table 2-3.—Spacing control points along slopes

This table is based on a change in elevation of 2 m (6 ft) during each setup, the normal maximum when following the specifications for geodetic leveling given in chapter 3.

Slope (percent)	Maximum leveling sighting distance m (ft)	Maximum distance between control points km (mi)	Recommended average spacing km (mi)
1	Maximum tolerance	3.0 (1.9)	1.6 (1.0)
2	50 (164)	3.0 (1.9)	1.6 (1.0)
3	33 (108)	2.7 (1.7)	1.1 (0.7)
4	25 (82)	2.0 (1.2)	0.8 (0.5)
5	20 (66)	1.6 (1.0)	0.6 (0.4)
6	16 (52)	1.3 (0.8)	0.5 (0.3)
8	12 (39)	1.0 (0.6)	0.4 (0.25)
10	10 (33)	0.8 (0.5)	0.3 (0.2)
15	6 (20)	0.5 (0.3)	0.2 (0.1)
30	3 (10)	0.3 (0.2)	0.1 (0.05)

ing project and the National Geodetic Vertical Network. A tie consists of two or more control points that, after they have been leveled and checked, connect a line to a previous survey.

When routing the lines of a project, connecting points must be recovered to permit ties to be made that span the area of the project. To the extent that they serve the purpose of the project, connections should also be routed to other types of control networks. Given next are specifications for connections to be made when routing lines of the national network. The requirements are summarized in table 2-4.

Previous surveys. When lines of a previous survey are releveled or intersected as part of a project, the ties established permit the previous survey to be readjusted and brought up-to-date with the current work.

If all or part of an existing line is to be releveled, route it by recovering the control points as they are described in the vertical control data. If the existing route is no longer accessible (as is the case with many lines following railroads) and the line must be relocated, recover the points of the best quality and include them in the new, parallel route. If possible, include such points at the following intervals along the line: 5 km (3 mi) if the distance separating the new and old routes is less than 0.5 km (0.3 mi), 10 km (6 mi) if the separation is between 0.5 and 2.0 km (0.3 and 1.2 mi), and 20 km (12 mi) if the separation is greater than 2.0 km (1.2 mi). (See fig. 2-12.)

Make a connection whenever the route intersects a line of a previous survey. If both current and previous surveys are of first-order precision, recover at least three control points that were leveled during the previous survey. Connect them with a spur (or spurs), recovering or setting a monument of the best possible quality at or near the point where the spur joins the main line (fig. 2-13). All three points must be leveled to establish a tie. If either or both of the surveys are of less than first-order precision, recover at least two control points for the connection.

Often "area surveys" are encountered, where two or more lines of a previous survey are intersected. In this case, make connections by recovering at least one control point from the previous survey at each intersection (fig. 2-14). At least three such points, spanning the area of the survey, must be leveled to establish a tie.

A local survey of good quality that has not previously been included in the national network may be connected by following these procedures. Consult the appropriate local officials to obtain descriptions of monuments that might serve as connecting points.

Do not extend a spur more than 8 km (5 mi) to make a connection to a previous survey, unless special project instructions apply.

Tide and water-level stations. Connections between the national network and primary tide stations are essential to compare and relate elevations to the sea-level surface at various points along the coasts of the United States. Similiarly, connections to water-level stations permit elevations to be obtained for the mean lake level at various points along the shores of the Great Lakes. Project instructions should provide a list of tide and water-level stations to be connected.

Numerous secondary and tertiary tide and water-level stations may be located near a line routed along a shore. Unless project instructions specify otherwise, connect these only if they are no more than 4 km (2.5 mi) from the line.

Table 2-4.—Requirements for connections

Connection	Spacing	Minimum number of connecting points
Previous surveys:		
Parallel, 0.5 km (0.3 mi) or less away	Every 5 km (3 mi)	1
Parallel, 0.5 to 2.0 km (0.3 to 1.2 mi) away	Every 10 km (6 mi)	1
Parallel, more than 2.0 km (1.2 mi) away	Every 20 km (12 mi)	1
Intersected, first-order line	At every intersection	3
Intersected, second-order line	At every intersection	2
Intersected, area survey	At two intersections that span the area	3
Tide and water-level stations:		
Primary	At every station	5
Secondary	At stations less than 4 km (2.5 mi) away	3
Tertiary	At stations less than 4 km (2.5 mi) away	3
Active airports	At airports not leveled before and less than 4 km (2.5 mi) away	1
Gravity stations	At every station	1
Horizontal control and three-dimensional stations	At stations not leveled before and less than 4 km (2.5 mi) away	1

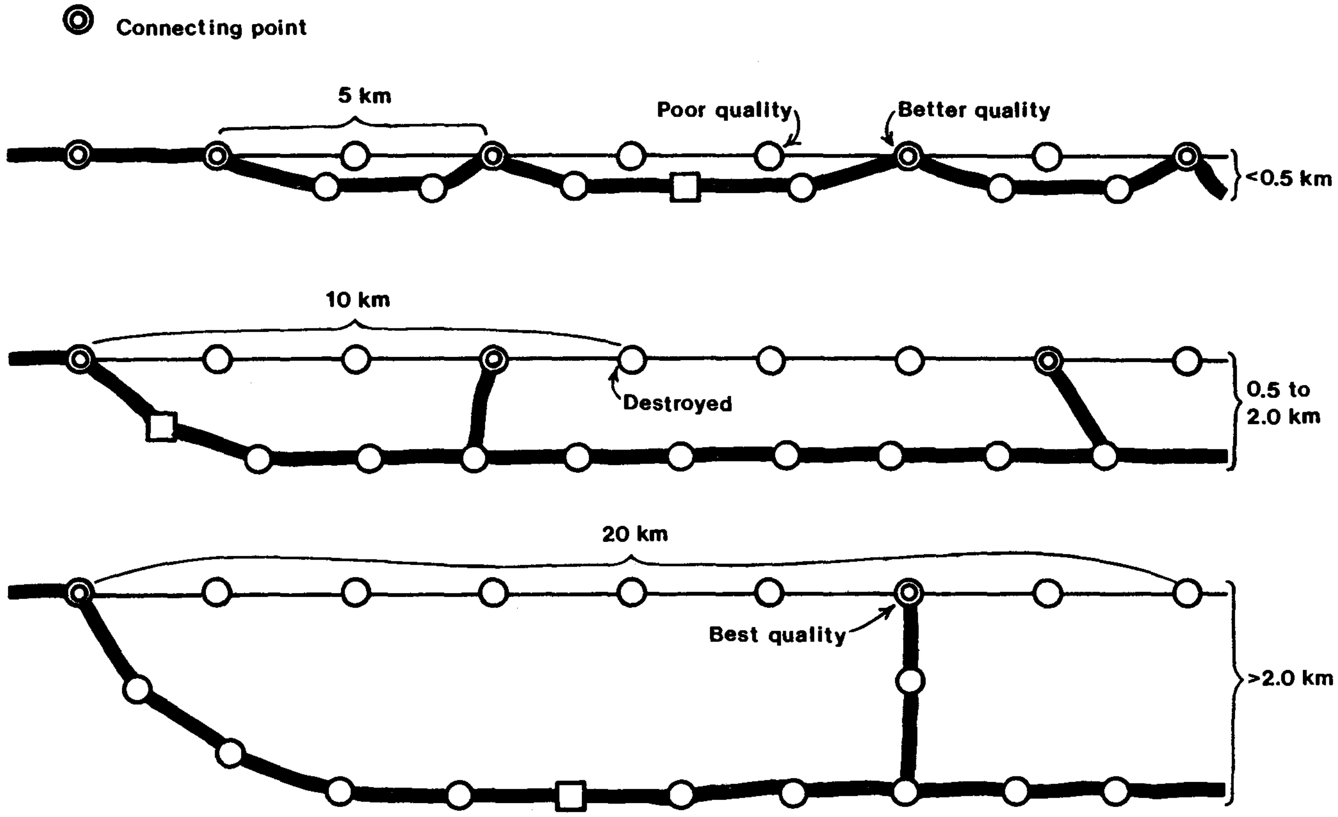


Figure 2-12.—Connections to parallel surveys.

Each primary or secondary station should have a cluster of at least five control points, a staff or electric-tape gage for measuring the height of the surface of the water relative to the points, and a water-level recorder that operates continuously. Obtain descriptions from the most recent station report. Contact the station observer to make arrangements for the leveling unit to level to the staff or gage. If no monument of quality A (sec. 2.4.4) is present at a primary station, set one. Connect the points to the main line with one spur, recovering or setting another mark of quality A at the point where the spur joins the line.

Airports. Connections to control points at airports provide elevations for aeronautical charting. If such points have already been leveled as part of the national network, they need not be releveled. Connect an active airport not previously leveled only if it is within 4 km (2.5 mi) of the main line, unless the connection is specifically requested in project instructions. Recover or establish only one control point, near the terminal building, if possible, or at the entrance to the airport. To locate an existing control point or to establish a new one, check with a responsible official at the airport.

Other control. Leveling connections to control points for gravity, three-dimensional, and horizontal networks are important for monitoring crustal motion and for continual development of a unified, precise, global positioning system.

Relative measurements of gravity are made at many control points of the vertical network, so special connections to gravity stations are usually not necessary. However, connect stations where repeated measurements of absolute gravity are made, when they are listed in project instructions.

Instructions should also list any three-dimensional ("satellite") and horizontal stations that are to be connected. Otherwise, connect these stations only if they have not previously been leveled, are within 4 km (2.5 mi) of the line, and are readily accessible to the leveling unit. Consult the appropriate Geodetic Control Diagrams to determine if such stations are near the line.

At these types of stations, more than one monument may have been established. Consult the horizontal control data or other descriptions that have been provided and, if practicable, include all the monuments in the line. For example, horizontal control stations normally include a station mark, two or more nearby reference marks, and an azimuth mark a few tenths of a kilometer away. If a spur is necessary, route it over all the monuments; avoid creating a "spur-on-a-spur."

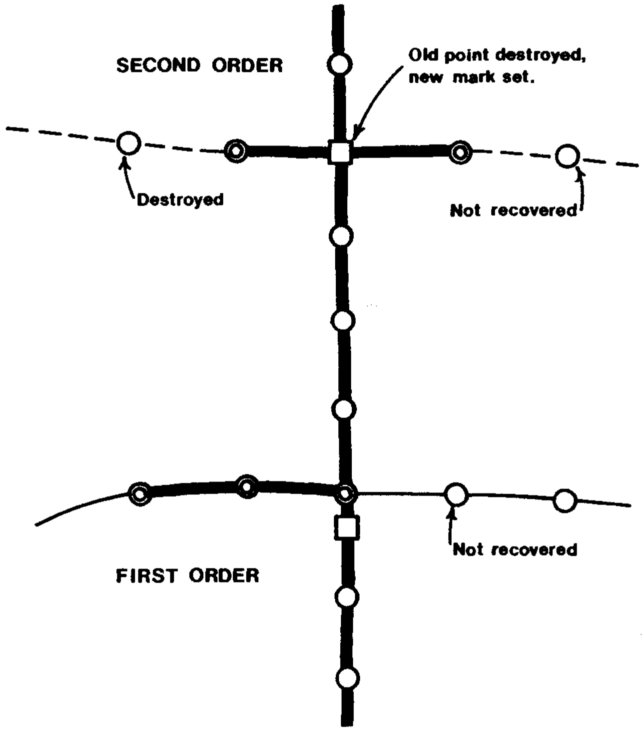


Figure 2-13.—Connections to intersected surveys.

2.3.5 Records

While routing a line, the mark setter must prepare numerous plots, descriptions, and logs, all critical to the future recovery and leveling of the control points. Plots and descriptions identify and provide permanent instructions for reaching the individual points; these are discussed in sections 2.4.3 and 2.4.4. Logs provide specific instructions to the leveling unit for leveling the line.

Logs. The logs must include clear and concise instructions that enable a unit to level efficiently to the control points along the route. Sometimes simple handwritten lists are sufficient, which briefly describe the locations of the points by providing driving instructions and measurements from the most obvious reference objects. Working position plots should be supplied with such logs. More convenient as logs are the working plots themselves, on which the plotted points and routes between them are briefly described (fig. 2-15). Prepare logs on working plots as explained here.

As each control point is recovered or established along the route, plot the control point on the appropriate topographic map. Provide a brief description by giving measurements to a witness post and other landmarks, the height of the point relative to the ground, any special leveling procedures required (such as for a vertically mounted disk or one requiring the use of spacers) and any special permission or contacts needed. If the monument site is cleared and marked with flagging, only a few of the most useful reference objects and distances supplied in the description need be noted on the log.

Use a broad, light-colored pen to connect the points in leveling order on the topographic map. Note the distances and the most efficient routes between them. Label a spur intended to tie a previous survey with the accession number of the survey. Whenever points are grouped in clusters that may cause confusion, attach a large-scale sketch. Tide and water-level stations nearly always require such a sketch; attach the station reports if possible. In an urban area, a large-scale city map may provide better information than a topographic map; however, a city map should not be used to scale positions.

After each map is completed, scale the positions of the points and assign survey-point serial numbers, taking care to match them with the correct designations. (See sec. 2.4.4, "Survey-point serial number.") Label the lower right-hand corner with the line number and a number corresponding to the location of the map in the line. Fold the map, printed side out, to a convenient size.

Submitting records to the project office. Keep all mark setting records up to date as work proceeds along a line. Use the Index to Topographic Maps as a checkoff list of the maps completed (fig. 2-6).

Collect and bind, in line order, descriptions of all points to be leveled, and label the binder with the line number. Bind descriptions of points not to be leveled as

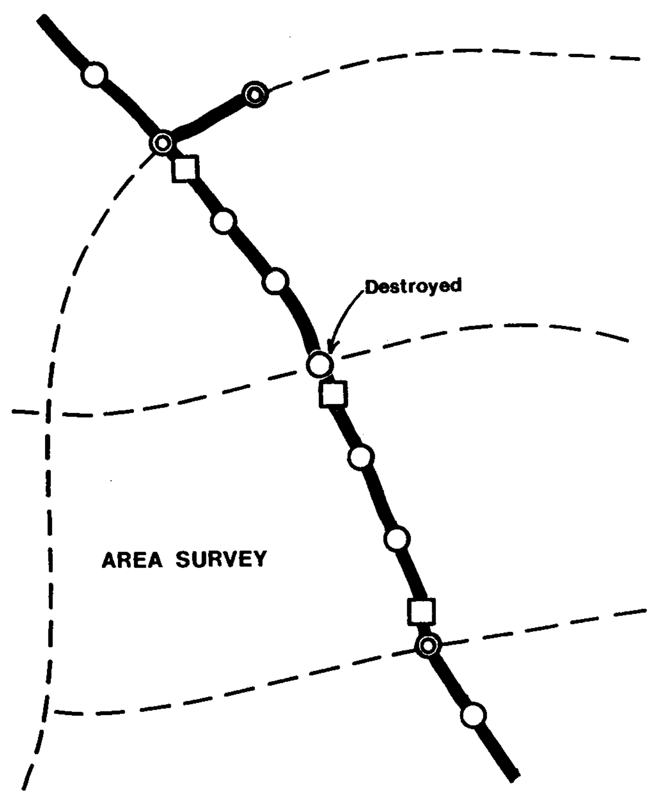


Figure 2-14.—Connections to an area survey.

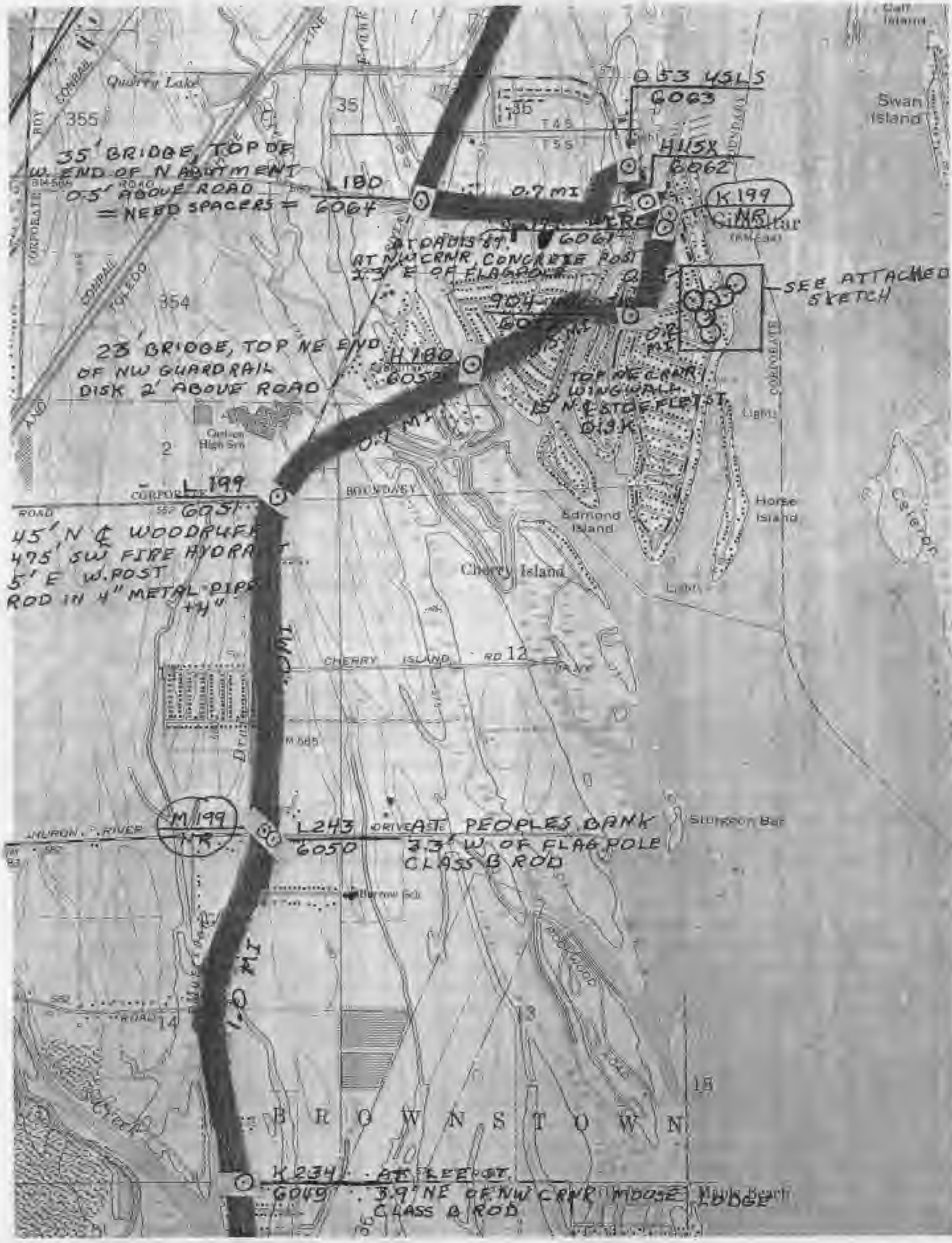


Figure 2-15.—Map prepared on a working plot.

each 30′ quadrant of vertical control data is completed. Label these binders with the quadrant numbers. Submit the binders to the project office together with logs and smooth plots covering the same area.

2.4 Vertical Control Points

More than 500,000 points define the lines of the National Geodetic Vertical Network. To mark these points, monuments of many types have been set. Most of these are bench marks—monuments designed to provide permanent and stable points within the vertical frame of reference. The remaining points are marked by monuments originally designed to provide other types of control.

When routing a line of a leveling project, after existing control points are recovered, bench marks must be set to complete the routing requirements described in the previous pages. Properly recovered and set control points are the most important product of the mark setting unit. They must be permanent, stable, accessible, and properly described.

2.4.1 Control Point Recovery

An efficient recovery effort depends on adequate preparation and organization, as well as skill and experience at interpreting previous descriptions and preparing new ones. In addition to recovering or setting control points for the lines assigned by project instructions, NGS mark setting units are sometimes expected to recover and prepare descriptions for all points listed in the NGS data base and located in the area covered by the 7.′5 maps that include the lines.

Preparation. First, collect and organize the data for existing control points along the line to be routed. Determine the topographic maps to be used and, if possible, plot the positions of the points to be recovered. As reminders, note on the maps the locations of line junctions and connections to be made with previous surveys, tide and water-level stations, active airports, and other control. Make arrangements to meet with appropriate officials to obtain permission and assistance. While recovering the points, keep track of their spacing, condition, and quality, noting where new bench marks should be set.

Recovering a control point. Follow the instructions given in the description for reaching each control point. Often a witness post and sometimes the monument can be seen from the vehicle after driving to the described location. However, this type of sighting is not an adequate recovery.

Before reporting a point recovered as described, be sure that the monument found is in fact the one previously described. Observe the stamping on the mark and check that it agrees exactly with that given in the description. Check the measurements and directions from nearby reference objects. If necessary, install a new witness post and make measurements to additional reference objects.

If the point is to be leveled, clear away brush or obstructions, install footholds for the rodman, and do whatever else may be necessary to make it accessible to the leveling unit. A disk recessed in concrete may not have sufficient clearance to permit a standard leveling rod to be set on the highest point. Note in the log that spacers will be required to level to such a point.

To assist the unit in finding the point, tie a single piece of brightly colored flagging at eye level around a nearby witness post, utility pole, tree, fence, or guardrail. Chalk may be used to draw attention to a mark set in stone or concrete, but only with the property owner's permission.

Before leaving the site, plot and identify the exact location of the control point and prepare a description. The type of description prepared depends on the type of control which will be provided by the point. For example, a point that is to provide both vertical and horizontal control must have two descriptions, one for each part of the data base. Vertical control descriptions are discussed in sec. 2.4.4. Horizontal control descriptions are discussed in the Federal Geodetic Control Committee publication, *Input Formats and Specifications of the National Geodetic Survey Data Base* (Pfeifer 1980: vol. I), the National Geodetic Survey Operations Manual (Greenawalt and Floyd 1980), and *C&GS Special Publication* 247, Manual of geodetic triangulation (Gossett 1959).

When recovering a relocated point ("reset"), confirm that the original monument has been destroyed. Search for both monuments if both descriptions are available. Since the two are likely to be close together, take care that they are not confused. Prepare a separate description for each monument.

If a control point apparently cannot be recovered, recheck the description, especially the starting location, distances, turns, landmarks, and supplemental notes. Compare distances to those given for points located immediately before and after it in the line. To find a likely site, compare elevations from map contours to the published elevation for the point. Look for remnants of flagging, signal cloth, wood, witness posts, or concrete. Pinpoint the likely location of the mark by measuring from the reference objects, then clear away any brush and dig. A metal detector may be helpful. Never report a monument destroyed unless enough of the metal surface is recovered for positive identification. If no identifiable part of a monument can be found, report in the station description that the monument was not recovered and note the amount of time spent searching.

Destroyed bench marks. If a destroyed or severely damaged NGS bench mark is recovered, NGS employees or other agents designated by the agency may remove what remains of the monument from the site. Report that it is destroyed in the description and return marks bearing a stamped designation to the project office for disposal. Report destroyed or damaged monuments of other agencies, but do not remove such monuments unless the agency specifically requests it.

Occasionally a monument that is scheduled to be destroyed (e.g., during highway widening), or one that is unlikely to prove reliable in the future because of low-quality construction (quality D) or poor condition, is recovered. Set a new bench mark of better quality in the immediate vicinity and route a spur from the new mark to the old. After the spur has been leveled and checked, destroy the old monument if the responsible agency so requests, and prepare a new description accordingly. If an unreliable monument is not destroyed, be sure to note in the original recovery description that it is not recommended for future geodetic leveling.

The cap-and-bolt is an example of a type of monument likely to prove unreliable because of its construction. The monument includes two control points, a bolt set in concrete 1 to 2 m beneath the ground surface and a cap on a pipe extending to the surface from the bolt. In the past, both points were leveled from a temporary bench mark set nearby and three elevations were published (cap, bolt, and reset cap). This practice was very costly and has been discontinued. Because the monument is of quality D, replace it as described in the preceding paragraph. Since standard leveling rods are too wide to fit into the pipe, instruct the leveling unit to level only to the cap.

2.4.2 Bench Mark Setting

Only bench marks of the best possible quality (A or B) should be set to complete routing requirements for a line. The mark setting manual provides instructions for selecting sites and constructing such monuments. Information presented here is intended to supplement and update the manual.

Designations for bench marks. Only NGS employees and agents may set brass disks and aluminum flanges precast with the NGS logo. Such marks must be stamped with designations supplied by the agency in project instructions or via computer. Each designation includes a letter of the alphabet and a number, up to four digits. It should be unique among all the designations in the State where the control point is located.

Designations are normally assigned to an individual mark setter in a quantity sufficient for the line to be routed. Several series are normally assigned; each includes one number and 24 letters of the alphabet. ("I" and "O" are omitted because they may be confused with "1" and "0".) For example: A 200-km leveling line, with the space between control points averaging 1.6 km and the recovery rate expected to average 50 percent, requires approximately 63 designations or three series, such as A 123 through Z 123, A 124 through Z 124, and A 125 through Z 125.

To avoid duplicate designations, keep a record of all designations that have been used (fig. 2-16). If a mark is set that bears a stamping duplicating that of another mark in the same State, stamp it with the letter "X" following the number if it has not been leveled. For example, "C 124" becomes "C 124 X." Be sure to correct the plots, the description, and the log. If the duplication is discovered after the mark has been leveled, do not stamp the mark but notify the project office. (See sec. 5.3.3, "Synoptic file comparison.")

Site selection. Set bench marks in bedrock whenever possible. In a region where expansive soil or local subsidence is prevalent, extend a spur as much as 4 km (2.5 mi) to set a high-quality bedrock mark.

RECORD OF BENCH MARK DESIGNATIONS
ASSIGNED BY MARK SETTER

STATE CALIFORNIA MARK SETTER JOHN SMITH

PROJECT NGVD REGION 6 YEAR 1980

Number	A	B	C	D	E	F	G	H	J	K	L	M	N	P	Q	R	S	T	U	V	W	X	Y	Z
123	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X
124	X	X	X																					
125																								

Figure 2-16.—Example of bench mark designations assigned by mark setter.

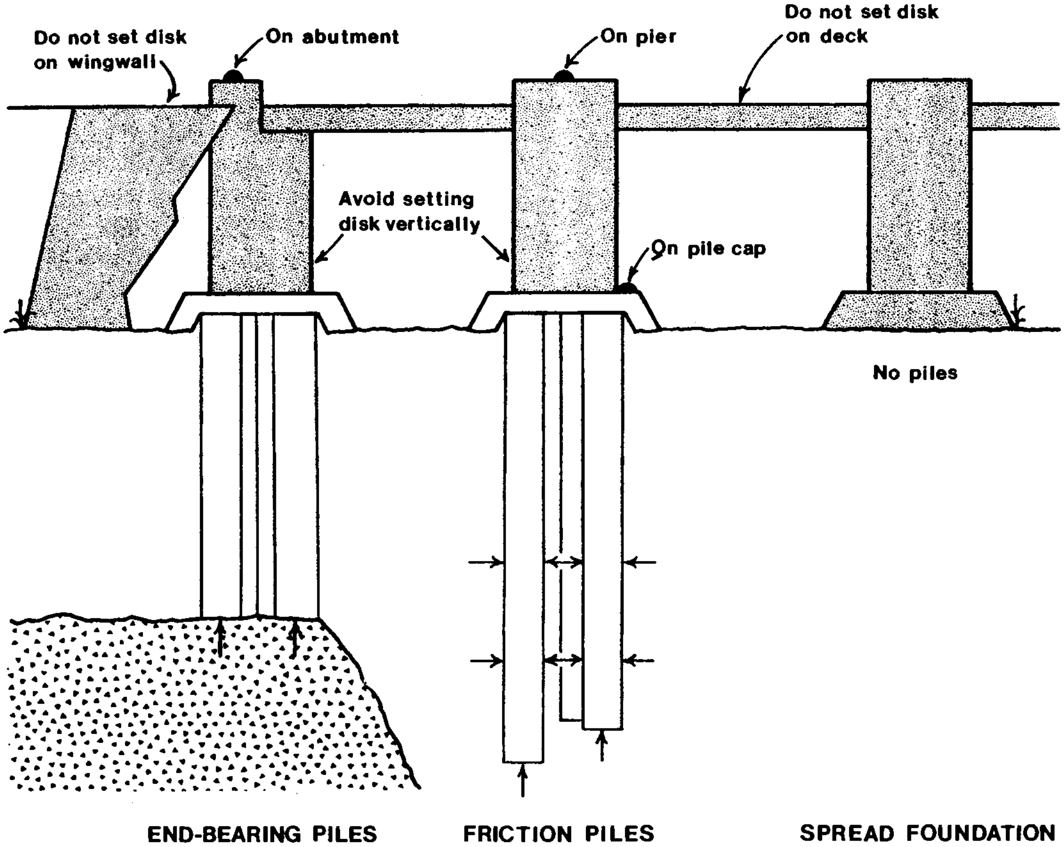


Figure 2-17.—Foundation types of massive bridges.

When selecting sites for monuments, remember the length (3 m or 10 ft) and width (7.6 cm or 3 in) of the standard leveling rod. Avoid setting marks that are awkward to level, such as those sketched in figure 3-78 (chapter 3). When setting a disk in bedrock, chip away the surrounding rock and level the disk so the highest point corresponds to the center of the disk. Hold a carpenter's level vertically, as though it were a miniature leveling rod, to test this. If a disk must be set vertically in a structure, place it about 1.0 m (3 ft) above ground and make sure the long line at the center is horizontal.

Massive bridges. A massive bridge across a wide river or valley or an overpass of an interstate highway may provide a high-quality site for a disk. Such a bridge normally has five structural members. The deck is the surface on which vehicles are driven. It is made up of several sections, separated by expansion joints, and is designed to yield to various forces. The abutments provide support to the ends of the deck. The piers provide support between the ends of the deck. Abutments and piers are in turn supported by foundations. At the ends of a bridge may be wingwalls, which are retaining walls built to hold back earthen materials.

Three types of foundations are commonly used (fig. 2-17). A spread foundation is essentially a slab that provides support by transferring the load over a large area of the soil. A friction pile transfers the load to the soil through friction at the interface of the pile and the soil. An end-bearing pile transfers the load to a resistant subsurface formation or bedrock. Piles are usually placed in groups and topped with a pile cap. Abutments and piers rest on the pile caps. A pile cap cannot be visually distinguished from a spread foundation. Check with the local highway engineer to learn the types and depths of the foundations of bridges you might use.

To be suitable for a monument of quality A, the foundation of the bridge should rest directly on bedrock or should be made up of end-bearing piles. Where soil is nonexpansive and maximum frost depth is less than 0.5 m (1.6 ft), the foundation may be made up of friction piles. The disk must be set on an abutment, pier, or pile cap.

Rod marks. Since the publication of the mark setting manual, various driving points (fig. 2-18) have been devised. When driving a string of stainless steel rods that have been screwed together, the wings of the driving point prevent the rods from unscrewing as a result of vibration and rotation induced by the hammering of the rock drill. Use these points when setting class A or B rod marks as described in the manual.

A rod string must be driven deeply enough to resist movement caused by hammering or pulling the rod at the surface. Initial experience in driving rods with a gasoline-powered rock drill indicates that, to ensure sufficient stability, a driving rate slower than 30 cm/min (1 ft/min) must be attained at or below the minimum

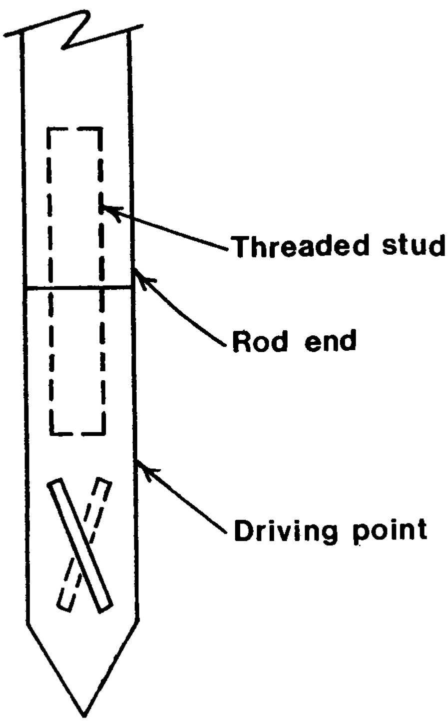


Figure 2-18.—Driving point for a rod mark.

depth required for the site. (See table 3 in the mark setting manual.) If driving through clay, do not drive more than 10 m (30 ft) when trying to attain this rate; after the rod has been in place for a few days, the clay should "set up" around it, anchoring the mark in place. If possible, return to such a mark to check that it has become anchored.

Two changes have been made to the instructions for a class B rod mark. First, instead of using a plastic cleanout fitting, protect and identify the control point with a casement exactly like the one used for a class A rod mark. (The 4-inch fitting used in the past is too narrow to permit a standard leveling rod to be placed with certainty on the highest point of every such mark.) Second, instead of crimping a brass disk to the top of the rod, round off the top end with a file or crimp a stainless steel cap onto the rod, as for a class A rod mark.

Before crimping on a stainless steel cap for either a class A or B rod mark, stamp the designation around the top of the cap, just below the edge of the rounded portion (fig. 2-19). This permits backup identification in the event that the flange of the casement is later mutilated or destroyed.

To crimp the cap, use a small hydraulic pump with a cutting attachment. After slipping the cap over the end of the rod, crimp it into the rod about halfway down from the top. Be sure that the cap cannot be raised, lowered, or twisted after crimping. Construct the casement so a leveling rod may be set and rotated on the control point without restriction.

2.4.3 Assigning Geographic Positions

The geographic positions of a small number of points in the national network of vertical control have been precisely determined by horizontal surveys. However, a position, even if approximate, should be assigned to every point for the following reasons: to permit automatic plotting and filing of vertical control data in the appropriate 30′ quadrants, to permit interpolation of gravity values for correcting the leveling data, and to assist recovery without relying solely on a description of the route to the point.

Working plot. To obtain approximate positions for control points, maintain a working plot ("rough plot"). This is a 7′.5 topographic map (or 15′ map if the larger scale is not available) on which latitude and longitude lines have been penciled. On the map, plot and verify the location of each point as it is recovered or set. If precise positions are available for some of the points, such as triangulation stations, these should be plotted in advance to assist in recovery. Plot points found destroyed, but do not plot those not recovered unless project instructions request estimated positions for them.

The accuracy of the plot depends on the mark setter's good judgment and experience. Use all the information available on the map to estimate the location of each point in relation to the surrounding topography. Consider the following elements: distance along the route from the last point; distance from one or more landmarks along the route, such as overpasses, road or highway intersections, prominent buildings, towers, powerline crossings, railroad crossings, bridges, marked

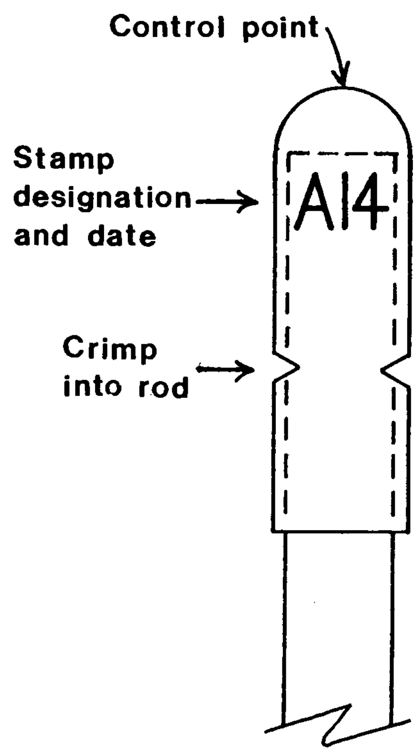


Figure 2-19.—Stainless steel cap for a rod mark.

boundaries, gullies, and hills; and distance of the point from the centerline of a highway or a rail of a railroad. When working across country, use the contour intervals to aid in locating points. After leveling, elevation differences and leveling distances should be used to check such plots. (See sec. 5.3.3, "Positions.")

Use only the most recent editions of topographic maps. If the map does not show a new highway, obtain local maps that show both the highway and section divisions (e.g., maps of the U. S. Forest Service, U. S. Bureau of Land Management, and route plans of State highway departments). Then, transfer the highway route to the topographic map by comparing section divisions. Use "bluelines" if they are available from the USGS (see sec. 2.3.1, "Maps for position plots..."), since they often show control points labeled with designations as well as new highways.

With waterproof black ink, indicate each control point with a dot, neatly surrounded by a 3-mm (1/8-in) circle. Label it with the designation. Indicate points that are not to be leveled with red ink. Indicate points that are destroyed with the note "DEST" and those that are not recovered with the note "NR." Well-maintained working plots may be used subsequently as logs for the line. (See sec. 2.3.5, "Records.")

Scaling positions. The position of each control point without a previously published position must be scaled from the working plot and recorded in the description. To ensure accuracy and consistency, scale positions only from 7.′5 or 15′ USGS topographic maps. The position of a single point may be scaled while preparing the description, or the positions for all the points may be scaled after work on the area is completed. In the latter case, ensure that the positions are matched correctly with the descriptions.

The scales illustrated in figure 2-20 are used to obtain both latitude and longitude. The following instructions apply to a 7.′5 map. The same procedure should be used for a 15′ map; however, use the single scale marked in increments of 2″ from 0′00″ to 5′00″.

1. Place the map on a flat surface. If latitude and longitude lines have not been previously drawn, draw them on the map using a straight edge and a sharp pencil. Draw latitude lines across the map at intervals of 2′30″. Draw longitude lines at the same interval. The intervals are marked on the edges and, with small crosslines, on the face of the map. (On a 15′ map draw the lines at intervals of 5′.)

2. From the edge of the map, read the degrees, minutes, and seconds of the nearest latitude line below the point. If the seconds read "00," use the scale marked from 0′00″ to 2′30″. If the seconds read "30," use the scale marked from 0′30″ to 3′00″.

3. Place the scale, oriented vertically, near the control point. Adjust the lower end (0′00″ or 0′30″) until it rests over the nearest latitude line below the point. Adjust the upper end (2′30″ or 3′00″) until it rests over the nearest latitude line above the point.

4. The scale is now slightly tilted. Maintaining the same angle of tilt, move it sideways until it intersects the point, as shown in figure 2-21. Ensure that the lower and upper ends rest over the lower and upper latitude lines, as before.

5. At the control point, read from the scale the minutes and seconds to the nearest second. Add this to the reading of the lower latitude line to obtain the latitude of the point.

6. Proceed similarly to obtain the longitude. From the bottom of the map, read the degrees, minutes, and seconds of the nearest longitude line to the right of the point. If the seconds read "00," use the scale marked from 0′00″ to 2′30″. If the seconds read "30," use the scale marked from 0′30″ to 3′00″.

7. Place the scale, oriented horizontally, near the control point. Adjust the right end (0′00″ or 0′30″) until it rests over the nearest longitude line to the right of the point. Adjust the left end (2′30″ or 3′) until it rests over the nearest longitude line to the left of the point.

8. Again, the scale is slightly tilted. Maintaining the same angle of tilt, move it up or down until it intersects the point, as shown in figure 2-22. Ensure that the right and left ends rest over the right and left longitude lines, as before.

9. At the control point, read from the scale the minutes and seconds to the nearest second. Add this to the reading of the right longitude line to obtain the longitude of the point.

When scaling positions, pay particular attention to the direction in which the scale is read. In the northern hemisphere, latitude always increases as you read up (north) from the bottom edge of the map. In the western hemisphere, longitude always increases as you move left (west) from the right edge of the map. Also, watch for consistent errors of 30″ caused by reading the wrong scale, errors caused by aligning one end of the scale with a latitude line and the other with a longitude line, and errors caused by reading the wrong latitude or longitude line.

To prevent such blunders, positions should be scaled by two persons independently before they are submitted to the project office. A recommended routine is for one member of the mark setting unit to plot, scale, and record the position while at the site of the control point. Then, the other member rescales and checks all the positions at the time survey-point serial numbers are assigned and logs are completed.

Smooth plot. The smooth plot is a clean version of the working plot, prepared on a copy of the same map (fig. 2-23). It is the permanent record illustrating the precise locations of the control points recovered. Label each point to be leveled with the survey-point serial number. Label the lower, right-hand corner with the line number and a sequence number corresponding to the location of the map in the line.

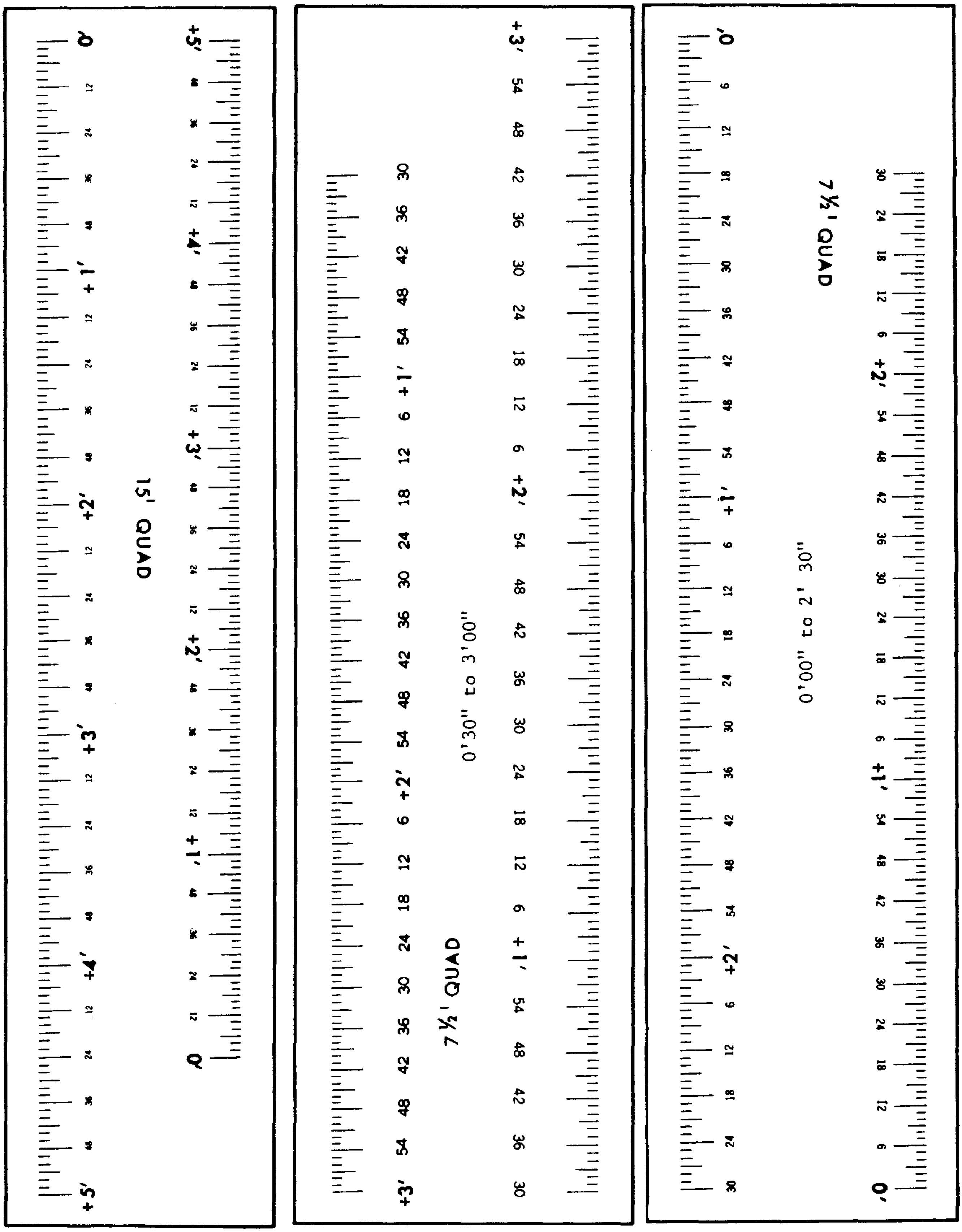


Figure 2-20.—Tools for scaling geographic positions.

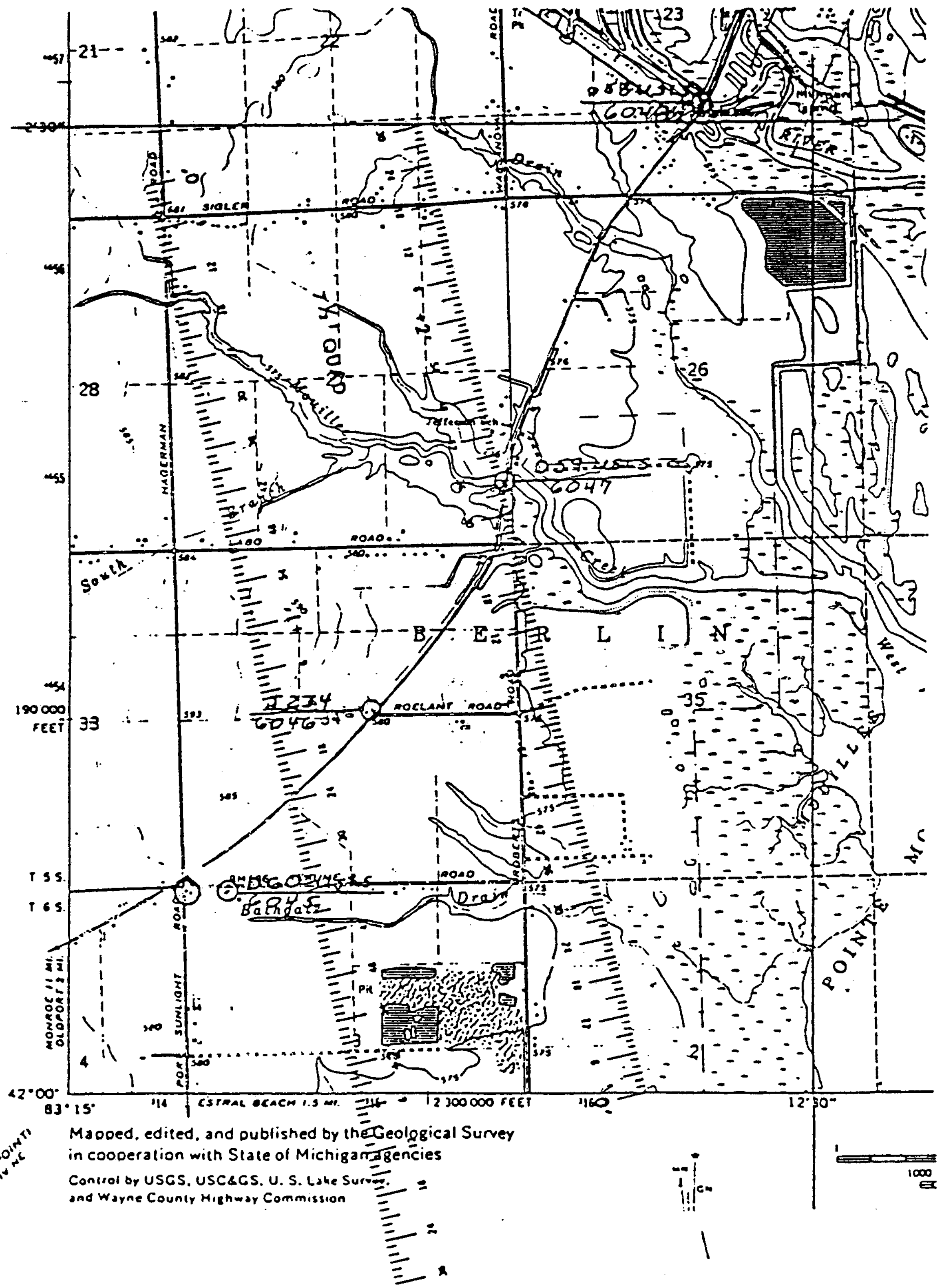


Figure 2-21.—Scaling latitude: D 59 USLS is 42°01'33''N.

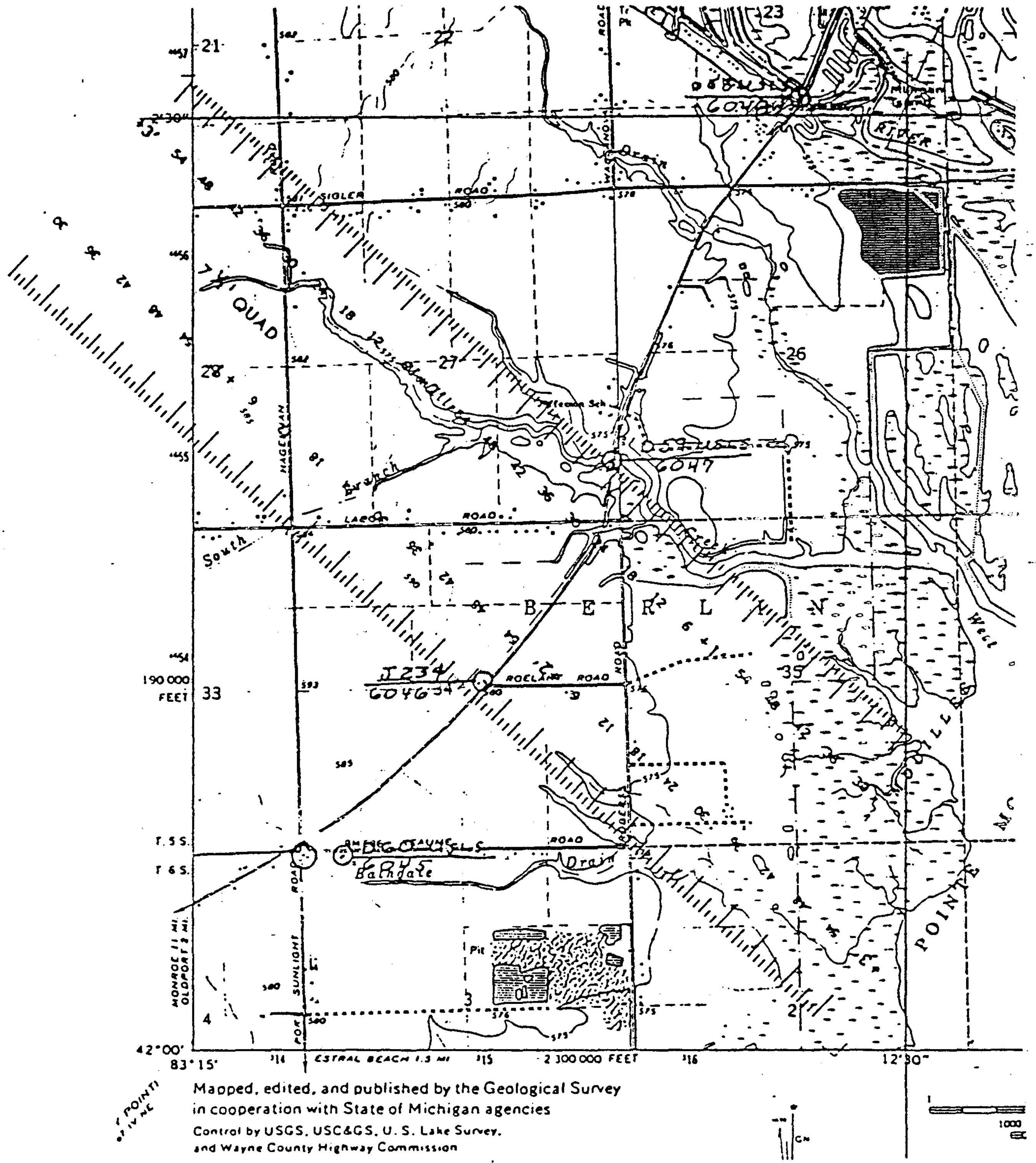


Figure 2-22.—Scaling longitude: D 59 USLS is 83°13'32"W.

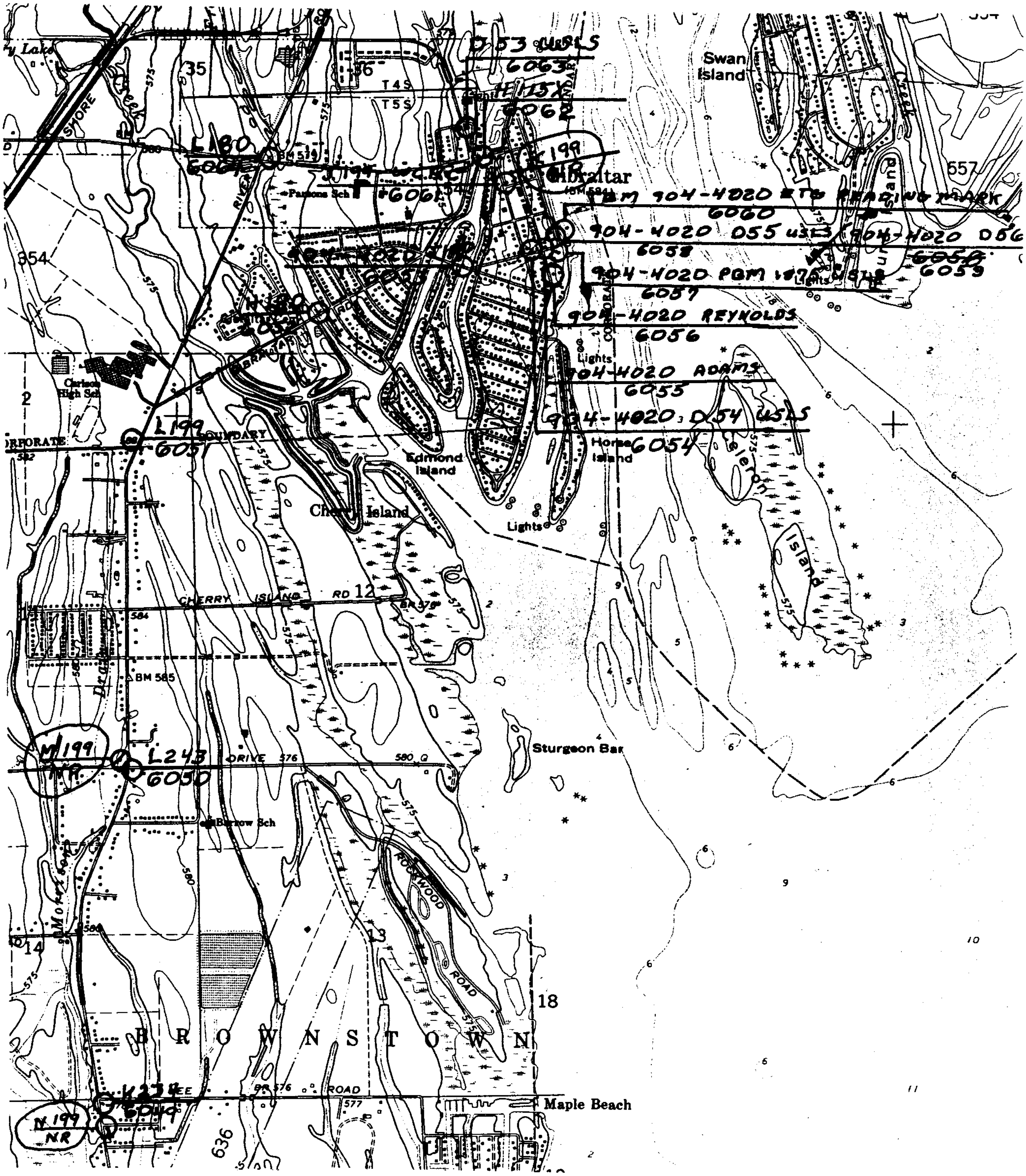


Figure 2-23.—Smooth plot.

2.4.4 Descriptions

To be useful, control points must not only be permanent and stable, but recoverable. Descriptions of points in the National Geodetic Vertical Network are the index for referring geodetic leveling data to actual points on the Earth's surface. Each description must independently provide, in a concise and consistent manner, all the information necessary to locate, positively identify, and level to a control point.

The degree of care with which a mark setter prepares descriptions is immediately evident to the other surveyors who use them. Do not repeat approximations, assumptions, and mistakes which may be present in previous descriptions. A description should contain only clear and accurate statements of facts that have been verified by the mark setter.

The National Geodetic Survey uses a form for coding much of this information, thus reducing the repetitive writing required when preparing numerous descriptions of similar points. It serves as a reminder, prompting the mark setter for detailed information about the location and the construction of the monument. It is designed to satisfy the requirements for vertical descriptive data presented in *Input Formats and Specifications of the National Geodetic Survey Data Base* (Pfeifer and Morrison 1980: volume II, ch. 7), hereinafter referred to as the input formats and specifications manual.

Prepare a description on NOAA Form 76-186, "Bench Mark Description," for each point searched for or set. Complete it at the site. Never write a description from notes or from memory; a vital piece of information may be missed or be remembered incorrectly. Only two items may be completed after leaving the site: the survey-point serial number and the position. These may be respectively assigned and scaled while compiling logs for a completely routed segment of line.

Descriptions are original records that are referred to repeatedly during the course of a project and are filed for further use afterwards. Protect them while in the field by attaching them to a clipboard with a waterproof cover. Write neatly, printing if necessary, in waterproof black ink. Do not use abbreviations or codes except those specifically permitted in the input formats and specifications manual.

Specific items of information required for the complete description of a vertical control point are explained here in the order in which they appear on the description form. If the point is already included in the national network and only slight changes must be made to the previous description, a recovery note may be prepared on the same form. It must provide information that verifies the identity and construction of the monument; instructions to reach the point are not necessary unless the previous description is inadequate or incorrect. If the point is not recovered, do not enter information that cannot be verified. Table 2-5 summarizes the standard entries for different types of descriptions.

To begin, in the upper right-hand corner write the project line number and the name of the topographic map on which the point is plotted. (See examples in figs. 2-24 and 2-25.)

Survey-point serial number. The survey-point serial number (SPSN) is an identifier used to match leveling data with the correct control points. Assign a unique

Table 2-5—Standard entries for descriptions

Action	Type of description	Survey Point Serial number? (*10*)	Description recovery code[1]	Archival cross-reference number?	Other condition (*22*)	Condition[1] (*25*)
Set new bench mark	Complete	Yes	D	No	Appropriate code	not required
Recovered, no change or slight change in description	Recovery[2]	Yes	R	Yes	Not required	G or P
Recovered, major change in description	Complete	Yes	R	Yes	Appropriate code	G or P
Recovered, first connection to the National Geodetic Vertical Control Network	Complete	Yes	R	No	Appropriate code plus X	G
Staff, gage mark, or other temporary control point	Temporary[3]	Yes	None	No	Not required	Not required
Recovered, not leveled by this survey	Recovery[2] or complete	No	R	Yes	Appropriate code	G or P
Recovered destroyed	Recovery[2]	No	R	Yes	Not required	X
Not recovered	Recovery[2]	No	R	Yes	Not required	N

[1] Refer to *Input Formats and Specifications of the National Geodetic Survey Data Base* (Pfeifer and Morrison 1980: vol. II).
[2] A recovery note requires the following entries: *10*, *13*, *15* (if any), *23*, *24*, *25*, *26*, *27*, *28*, and *30*.
[3] A temporary description requires the following entries: *10*, *13*.

LINE 3
STONY POINT

EXAMPLE

NOAA FORM 76-186
(3-80)

BENCH MARK DESCRIPTION

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

*10*SPSN-,6039, DRC CODE-,D, APPROX LAT-,415813N, APPROX LON-,0831720W *11*QUAD-,__________, ~~QSN~~ XRN -,MC0869 *12*LINE-,____,

*13*DESIGNATION-,E 234_______________________, *14*STATE CODE/COUNTY-,MI/,MONROE______________,

*15*ALIAS-,__________________________, (PARISH in Louisiana – State CENSUS DIVISION in Alaska)

*16*AREA-,__________________________________,

DISTANCE AND DIRECTION FROM

*17*NEAREST CITY OR TOWN-,OLDPORT__________________________, *18*NEAREST CITY OR TOWN-,2.8 KM (1.8 MI) S,

*20*CODE/MONUMENT BY AGENCY-,1,/,NGS_____________________, *21*YEAR-,1980, CHIEF OF PARTY-,LHW *22*OTHER CONTROL-,____

*23*CODE/RECOVERY BY AGENCY-,_,/,_________________________, *24*YEAR-,____, CHIEF OF PARTY-,___, *25*CONDITION OF MARK-,_,

*26*CODE/SETTING CLASSIFICATION-,64/,______________________________, *27*MONUMENTATION-,F, DISK TYPE-,__,

*28*STAMPING-,E 234 1980_________________________________, NGS QUALITY OVERRIDE-,_,

ORIGINAL OR RECOVERY DESCRIPTIVE TEXT

30,2.8 KM (1.75 MI) SOUTH ALONG DIXIE HIGHWAY FROM THE ST. CHARLES CATHOLIC___,
30,CHURCH IN OLDPORT, AT A PRIVATE ROAD LEADING EAST TO THE ENRICO FERMI
30,ATOMIC POWER PLANT,__________________________________,
30,____,
30,42.7 METERS (142 FT) EAST OF THE CENTERLINE OF DIXIE HIGHWAY,__________,
30,42.4 METERS (139 FT) EAST-SOUTHEAST OF THE JUNCTION,______________,
30,11.6 METERS (38 FT) SOUTH OF THE CENTERLINE OF THE PRIVATE ROAD,_______,
30,6.10 METERS (20.0 FT) SOUTH OF THE NORTHEAST CORNER OF A CORRAL,_______,
30,AND 0.61 METER (2.0 FT) EAST OF A WOODEN FENCE.___________________,
30,____,
30,____,
30,____,
30,____,
30,____,
30,____,

40(Units,_,E-English/M-metric) A DISK SET INTO THE TOP OF A CONCRETE POST,_,F-FLUSH WITH THE GROUND/P-PROJECTING/R-RECESSED,__,IN/CM

41(Units,M, Setting Code,64,same as in *26*) ROD/PIPE DRIVEN TO THE DEPTH OF, 6.0 ~~FT~~/M, IN A SLEEVE EXTENDING TO THE DEPTH OF,_4.8_~~FT~~/M (leave preceding entry blank if no sleeve), ENCASED IN A PIPE,F,F-FLUSH WITH THE GROUND/P-PROJECTING/R-RECESSED,__,IN/CM

42(Units,_,),____,FT/M,_,A-ABOVE/B-BELOW/L-ABOUT LEVEL WITH (Units and FT/M blank if 'L'),________________,

43(Units,M),_0.61_,~~FT~~/M,_W,(Point of Compass) FROM A WITNESS POST – else specify object,______________,
,____,Mi/Km,___,of Bench Mark,__________________________, (start entry with article (a,an,the) if applicable)

SUPERSEDES NOAA FORM 76-186 (9-78) WHICH MAY BE USED.

U.S. GOV. PRINTING OFFICE: 1980—668-690

Figure 2-24.—Complete description.

EXAMPLE   Line 3
GIBRALTAR

NOAA FORM 76-186
(3-80)

BENCH MARK DESCRIPTION

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

*10*SPSN-,6051,DRC CODE-,R, APPROX LAT-,420455N,APPROX LON-,0831239W *11*QUAD-,__________XRN ~~QSN~~-,NE0506*12*LINE-,____,

*13*DESIGNATION-,L 199________________________, *14*STATE CODE/COUNTY-,__,/,_______________________,

*15*ALIAS-,______________________________, (PARISH in Louisiana - State CENSUS DIVISION in Alaska)

*16*AREA-,___________________________________, DISTANCE AND DIRECTION FROM

*17*NEAREST CITY OR TOWN-,______________________________, *18*NEAREST CITY OR TOWN-,_______________,

*20*CODE/MONUMENT BY AGENCY-,_,/,____________________, *21*YEAR-,____, CHIEF OF PARTY-,___, *22*OTHER CONTROL-,____,

*23*CODE/RECOVERY BY AGENCY-,_,/,NGS_________________, *24*YEAR-,1980, CHIEF OF PARTY-,LHW, *25*CONDITION OF MARK-,G,

*26*CODE/SETTING CLASSIFICATION-,2,/,______________________________, *27*MONUMENTATION-,D, DISK TYPE-,01,

*28*STAMPING-,L199 1968______________________________, QUALITY OVERRIDE-,_,

************************************ ORIGINAL OR RECOVERY DESCRIPTIVE TEXT ************************************

30,OMIT PRIOR REFERENCE TO WITNESS POST.______________________________,
30,____,
30,____,
30,____,
30,____,
30,____,
30,____,
30,____,
30,____,
30,____,
30,____,
30,____,
30,____,
30,____,

40(Units,_,E-English/M-metric) A DISK SET INTO THE TOP OF A CONCRETE POST,_,F-FLUSH WITH THE GROUND/P-PROJECTING/R-RECESSED,__,IN/CM

41(Units,E, Setting Code,2,same as in *26*) ROD/PIPE DRIVEN TO THE DEPTH OF,_14_,FT/~~M~~, IN A SLEEVE EXTENDING TO THE DEPTH OF,____,FT/M (leave preceding entry blank if no sleeve), ENCASED IN A PIPE,P,F-FLUSH WITH THE GROUND/P-PROJECTING/R-RECESSED,4,IN/~~CM~~

42(Units,_,),____,FT/M,_,A-ABOVE/B-BELOW/L-ABOUT LEVEL WITH (Units and FT/M blank if 'L'),____________________,

43(Units,E,),___2,FT/~~M~~,__E,(Point of Compass) FROM A WITNESS POST - else specify object,________________,

,____,Mi/Km,___,of Bench Mark,____________________________, (start entry with article (a,an,the) if applicable)

SUPERSEDES NOAA FORM 76-186 (9-78) WHICH MAY BE USED.

☆U.S. GOV. PRINTING OFFICE: 1980—668-690

Figure 2-25a.—Recovery note.

*10*DRC CODE:

D-self-standing original description
R-self-standing recovery description

*14*STATE CODE: Use standard two-letter POSTAL CODE in the United States, its territories and possessions. Consult the User's Guide to the Input Formats and Specifications of the National Geodetic Survey Data Base for points elsewhere.

*20/*23*AGENCY CODE:

0-unknown
1-NGS or CGS (USC&GS)
2-U.S. Geological Survey (USGS)
3-U.S. Department of Defense (DOD)
4-other federal or interstate agency
5-state agency
6-county, city, or regional agency
7-commercial organization or private firm
8-National Ocean Survey (NOS)
9-foreign government agency

*22*OTHER CONTROL CODE:

A-astronomic station
F-fault monitoring site
G-gravity station
H-horizontal control point
M-magnetic station
N-no vertical control (not presently or anticipated to be connected to the National Vertical Control Network)
O-other (see descriptive text)
T-tidal bench mark
X-no vertical control (not connected to the National Vertical Control Network at time of recovery, but expected to be after this survey)

*25*CONDITION CODE:

G-GOOD or fair
N-NOT RECOVERED or not found, lost
O-other (SEE DESCRIPTIVE TEXT)
P-POOR or disturbed, mutilated
X-DESTROYED

*26*SETTING CLASSIFICATION CODE:

00-UNSPECIFIED
Shallow Settings (less than 10ft):
10-UNSPECIFIED SHALLOW
11-METAL ROD WITH BASE PLATE
12-CONCRETE POST
13-SHALLOW-SET PIPE
14-SHALLOW-SET METAL ROD (without base plate)
Unsleeved Deep Settings:
20-UNSPECIFIED DEEP
21-COPPER-CLAD STEEL ROD
22-GALVANIZED STEEL PIPE
23-GALVANIZED STEEL ROD
24-STAINLESS STEEL ROD
25-ALUMINUM ALLOY ROD
Rocks and Boulders:
30-UNSPECIFIED ROCK
31-ROCK OUTCROP, rock ledge, rock cut, or bedrock
32-BOULDER
Structures:
40-UNSPECIFIED LIGHT STRUCTURE
41-PAVEMENTS, such as streets, sidewalks, aprons, curbs, etc.
42-RETAINING WALLS AND BULKHEADS, including headwalls and wingwalls of culverts and small bridges
43-PILES AND POLES (e.g., spike in utility pole)
44-FOOTINGS AND FOUNDATION WALLS OF SMALL OR MEDIUM STRUCTURES
45-MAT FOUNDATIONS (bearing surface same size as structure), such as landings, platforms, steps, floors, tower foundations, bases of semaphores, etc.
50-UNSPECIFIED MASSIVE STRUCTURES
51-MASSIVE RETAINING WALLS, including headwalls and retaining walls of very large bridges
52-ABUTMENTS AND PIERS OF VERY LARGE BRIDGES, including overpasses and underpasses
53-TUNNELS
54-MASSIVE CONCRETE, MASONRY, OR STEEL STRUCTURES WITH DEEP FOUNDATIONS
55-LARGE CONCRETE, MASONRY, OR STEEL STRUCTURES WITH FOUNDATIONS ON BEDROCK
Sleeved Deep Settings:
60-UNSPECIFIED ROD/PIPE SLEEVE
61-COPPER-CLAD STEEL ROD IN SLEEVE
62-GALVANIZED STEEL PIPE IN SLEEVE
63-GALVANIZED STEEL ROD IN SLEEVE
64-STAINLESS STEEL ROD IN SLEEVE
65-ALUMINUM ALLOY ROD IN SLEEVE

*27*MONUMENTATION CODE:

B-BOLT
C-CAP-AND-BOLT
D-SURVEY DISK (any type)
F-FLANGE-ENCASED ROD
H-DRILL HOLE
I-METAL ROD
N-NAIL
O-CHISELED CIRCLE
P-PIPE CAP
Q-CHISELED SQUARE
R-RIVET
S-SPIKE
T-CHISELED TRIANGLE
V-STONE MONUMENT
X-CHISELED CROSS
Z-SEE DESCRIPTIVE TEXT

*27*DISK TYPE (blank if not survey disk):

00-unspecified SURVEY DISK
01-BENCH MARK DISK
02-TIDAL BENCH MARK DISK
03-TRIANGULATION STATION DISK
04-TRAVERSE STATION DISK
05-TOPOGRAPHIC STATION DISK
06-SURVEY DISK (not listed)
07-REFERENCE MARK DISK
08-AZIMUTH MARK DISK
09-GRAVITY STATION DISK
10-GRAVITY REFERENCE MARK DISK
11-MAGNETIC STATION DISK

*27*QUALITY OVERRIDE (for NGS use only)

40/*41*/*42*/*43*UNITS CODE:

E-English M-metric units

*43*POINTS OF COMPASS: N,NNE,NE,ENE, E,ESE,SE,SSE,S,SSW,SW,WSW,W,WNW,NW,NNW.

Figure 2-25b.—Recovery note.

serial number to every point that is to be leveled in the line. A series of numbers for each line is normally assigned in the project instructions. Assign the number "0000" to a point that is not to be leveled, such as one that is destroyed, unsuitable, or not recovered.

To prevent duplications, serial numbers may be assigned to all the points on a completed map at the same time that positions are scaled. They may be assigned in a sequence corresponding to the order in which the points should be leveled. However, they need not always appear in sequence. If points are added to the line after most serial numbers have been assigned, do not change the original numbers merely to correct the sequence.

Junction code. Write "JUNCTION" at the top of the description form whenever the control point is one of the following:

1. One of the three points at network junctions (including international boundaries).
2. A point at the base of a spur connecting a primary tide station.

A junction point leveled repeatedly within the same project must be described for each line in which it is leveled. If it is newly set, prepare a complete description for the first line leveled; prepare a recovery note for each subsequent line. For convenience, assign the same survey-point serial number to the point whenever it is described.

Description/recovery code (DRC). Enter a single letter on line *10* to indicate whether the control point was searched for, "R," or is newly set, "D." The code "R" indicates a recovery note or a complete description, written for a point that existed prior to the arrival of the mark setter. The code "D" indicates a complete description, written at the time the mark is set.

Position. Scale the latitude and longitude of the plotted position of the control point from the topographic map. Record each to the nearest second, latitude followed by the direction, "N" or "S", from the equator and longitude followed by the direction, "W" or "E", from the Greenwich meridian. The position is used by NGS to compute automatically the 30′ quadrant and the number of the quadrant line in which data for the point are filed.

If a point is not recovered, enter on line *11* the number of the quadrant and on line *12* the number of the quadrant line in which the previous description was published. Do not enter a position.

Archival cross-reference number. The archival cross-reference number (XRN or ACRN) is a unique identifier that is assigned to each control point in the national network. It permits the comparison of data collected during different epochs for the same point. It consists of two parts, a two-letter area code, indicating the Geodetic Control Diagram in which the point is positioned, and a four-digit number that is unique within the area. An index of two-letter area codes appears in appendix B.

Obtain the cross-reference number from either the published description or from a special listing of the synoptic file provided by NGS. Copy the number on line *11* of the description form. If a point has not previously been connected to the network, leave the space blank; a cross-reference number is automatically assigned after the project is completed.

Designation. To identify the control point, a designation is normally stamped on the mark by the agency which sets it. The designation is a unique name by which the point can be conveniently recognized and referenced. In general, the designation given in the description should be identical to that stamped on the mark. It should not, however, include redundant or nonspecific information that may appear in the stamping.

When the point is recovered, assign the designation given in the published description (or listing of the synoptic file). If necessary, correct mistakes such as misspellings or transposition of numbers, and record the previous, incorrect designation as an alias. Use the following guidelines to standardize the designations. (Examples appear in table 2-6.)

Do not include information that is normally precast on a mark, such as warnings or type of control.

Do not include a stamped elevation unless it is the only means of distinguishing the point.

Do not include a stamped date, unless the point has been relocated or the date is the only means of distinguishing between otherwise duplicate designations.

Limit the designation to 25 characters or less, including imbedded blanks. Abbreviate the designation if necessary to conform to this limit. Separate groups of alphabetic and numeric characters with one blank. Do not retain any punctuation that is stamped, except the following: A plus (+); a minus (−), only if it indicates a negative elevation; a period (.), only if it is embedded in a group of numerals; an equal sign (=); or a slash (/). Do not separate such symbols from the adjacent characters with blanks. Note: If a minus (−) is included, it can only be the first character of the designation.

If the mark is stamped with two designations, totaling 24 characters or less, concatenate the designations with an equal sign. If the characters total 25 or more, enter one of the designations as an alias.

If the mark belongs to an agency other than the NGS (or its predecessor, the Coast and Geodetic Survey), include at the end of the designation the acronym for the agency's name that is precast on the mark. If no name is precast, include the same acronym that is entered for the agency that set the mark. (See "Setting information," this section.)

If the mark provides control for a tide or water-level station, but was set primarily to provide control for the national network, include the station number as an alias. If the mark was set solely to provide control at the station, include the seven-digit number in the designation. Enter the first three digits, a space, and the last four digits. Add the word "TIDAL" if the station is a tide station. Then, enter the designation stamped on the mark, observing the guidelines given in this section. Newer

Table 2-6.—Examples of bench mark designations

Stamping as it appears on mark	Symbol for agency name as it appears on mark	Designation to appear on description
General:		
C 124 1980	NGS	C 124
2903	USGS	2903 USGS
H 325 1945 230.695 FT	USCGS	H 325
140B ELEV 95.3 FT	MORC	140 B MORC
T T17B 1965	USGS	TT 17 B USGS
TT-17B 1965	USGS	TT 17 B USGS
595 +00 1952	AHD	595+00 AHD
MI. 14.2	None	MI 14.2
4419	USGS	4419 USGS
5301.29 FT	USGS	5301.29 USGS
CH1174, 297+00A	USGS	CH 1174=297+00 A USGS
STA. NO. 3, MI. 182.5	None	STA NO 3=MI 182.5
No stamping	None	*Previous designation*
Tide station 872 8912:		
TIDAL 1 1937	USCGS	872 8912 TIDAL 1
BASIC 1937	USCGS	872 8912 TIDAL BASIC
H 14, TIDAL 3 1963	USCGS	H 14=872 8912 TIDAL 3
8912 A 1977	NOS	872 8912 A TIDAL
A 307 1974	NGS	A 307 *Alias* 872 8912 TIDAL
No stamping	USCGS	872 8912 TIDAL *Previous designation*
Staff, rod held at 6 ft stop		TBM 872 8912 STAFF 6 FT
Water-level station 906 3000:		
2 1934	USCGS	906 3000 2
BM 3 1934	USCGS	906 3000 BM 3
POOL 1934	USCGS	906 3000 POOL
M 104 1973	NGS	M 104 *Alias* 906-3000
3000 B 1977	NOS	906 3000 B
No stamping	USCGS	906 3000 *Previous designation*
Electric tape-gage reading mark		TBM 906 3000 ETG READ MK
Other control:		
BOULDER 1935	USCGS	BOULDER
BOULDER NO 2 1935	USCGS (Reference disk)	BOULDER RM 2
BOULDER 1935	USCGS (Azimuth disk)	BOULDER AZ MK
BOULDER NO 2 1935	USCGS (Azimuth disk)	BOULDER AZ MK 2
CHARLOTTE	USGS	CHARLOTTE USGS
CHARLOTTE NO. 1	USGS (Reference disk)	CHARLOTTE RM 1 USGS
CHARLOTTE	USGS (Azimuth disk)	CHARLOTTE AZ MK USGS
PALMER N.E. BASE 1940	USCGS	PALMER NE BASE
N WASH AZI	USGS	N WASH AZ MK USGS
Relocated points:		
Z 201 RESET 1980	NGS	Z 201 RESET 1980
3000 USGS RESET 1976	NGS	3000 USGS RESET 1976
CHICO 1948 1971	NGS	CHICO 1948 1971
CHICO 1948 NO. 3 1971	NGS (Reference disk)	CHICO RM 3 1948 1971
CHICO 1948 1971	NGS (Azimuth disk)	CHICO AZ MK 1948 1971
CHICO 1948 NO. 2 1971	NGS (Azimuth disk)	CHICO AZ MK 2 1948 1971
LIGHT 1950 RESET 1955	USCGS	LIGHT RESET 1955
WINSLOW 2 1962	USCGS	WINSLOW 2
LAKE WASHINGTON 1950 1970	USCGS	LAKE WASHINGTON 1970

tide and water-level marks are stamped with the last four digits of the station number and a letter; in this case, enter the three-digit state code followed by a space, the stamping, and the word "TIDAL."

If a point at a tide or water-level station has no stamping, append the designation given in the previous description to the station number. Because a staff or the reading mark of an electric tape gage is not a permanent monument, enter the letters "TBM" at the beginning of the designation, to indicate a temporary bench mark. Designate a staff by the station number, the word "STAFF," and the height of the stop on which the leveling rod is to be held. Designate the reading mark of an electric tape gage by the station number and the words "ETG READ MK."

If the mark was set to provide control for a horizontal network or other types of networks, assign the designation as it appears in the published description, again observing the guidelines given here. Designate a reference mark with the horizontal station name, the letters "RM," and the number stamped on the mark. Designate an azimuth mark with the horizontal station name, the letters "AZ MK," and the number, if any, stamped on the mark.

The mark for a vertical control point that has been relocated is normally stamped with the original designation, the word "RESET," and the year of relocation. Include all of this information in the designation. Other types of relocated control points may be stamped differently. For example, to reestablish a horizontal control point, a new surface mark may have been reset directly over an underground mark. The reset mark is usually stamped with two dates, the year the point was established and the year it was reestablished. The geographic position has not changed, but the elevation has. Indicate this in the designation by including both dates. If the designation exceeds 25 characters in length, include only the most recent dates.

A horizontal control point that has been entirely relocated and designated with the number "2" should not be confused with a reestablished point. Do not designate such a point with a date unless it, too, has been reestablished.

Alias. If the control point is known by more than one designation, enter the designation that is stamped on the mark on line *13* and enter all other designations as aliases. If the point has been designated incorrectly in a previously published description (or in the listing of the synoptic file), the incorrect designation should be shown as an alias in the corrected description.

General location. For control points in the United States enter a two-letter postal code indicating the State, commonwealth, province, or territory on line *14*. United States postal codes are given in table 2-2. Then enter the county, parish, census division, or independent city in which the point is located. Indicate a city with the phrase "C OF." For example, "C OF ST LOUIS" for St. Louis, Missouri.

For control points in other countries, enter the appropriate two-letter code and then the primary political division, such as a State, province, or district. The codes are published in annex A of the input formats and specifications manual.

If the point is part of a local network, such as a network for monitoring subsidence, an area title may be entered on line *16* to indicate this. The entry is optional and can be automatically added to all appropriate points after completion of the survey.

On line *17* enter the nearest city or town. It must have a post office, railroad station, or well-defined highway crossroads from which to begin distance measurements and it must appear on the official State highway map. If it is in a different State from the point, append the postal code for the state whenever the city or town is named. Also, append the code when necessary to avoid confusion. For example, "KANSAS CITY KS" as opposed to "KANSAS CITY MO."

The distance from the nearest city or town, which is to be entered on line *18*, is not a straight-line measurement. Instead, it is the total of the distances given in the instructions to reach the point, by following the most direct highway routes. Enter it to the nearest tenth of a kilometer, followed by the mileage in parentheses and the direction traveled. Abbreviate kilometer, "KM," and mile, "MI." Use the eight-point compass, spelling out the four cardinal directions and abbreviating the four intercardinal points. For example, "3.4 KM (2.1 MI) EAST," "10.2 KM (6.3 MI) NW," and "1.0 KM (0.6 MI) SE."

If the control point is located within city or town corporate limits, enter the word "IN" before the name of the city or town and do not give a distance and direction.

Setting information. Whenever the control point requires a complete description, enter information about the agency which set the monument and its purpose. This includes a code indicating the agency classification, the agency's name, the date (year) the mark was set, and the initials of the project director of the party which set it. Agency codes are listed on the back of the description form and in table 2-7. Acronyms for agency names, as standardized for the use of the National Geodetic Survey, are given in annex C of the input formats and specifications manual. If the date the mark was set and the initials are not available from a previous description, enter the date stamped on the mark and leave the initials blank. If the date is not known, enter "UNK" in its place.

When the point serves another geodetic or geophysical purpose enter the appropriate code on line *22*. The purpose should be evident from the project instructions for newly established points and from the previous description or type of disk for recovered points. Spe-

Table 2-7.—Agency classification codes designated by NGS for preparing NOAA Form 76-186

Agency	Code[1]
National Geodetic Survey	1/NGS
National Ocean Survey	8/NOS
U.S. Coast and Geodetic Survey (prior to 1970)	1/CGS
U.S. Geological Survey	2/USGS
U.S. Department of Defense	3/DOD
Other Federal or interstate agency	4
State agency	5
County, city, or public regional agency	6
Commercial organization or private firm	7
Foreign government agency	9
Unknown	0

[1] Refer to *Input Formats and Specifications of the National Geodetic Survey Data Base* (Pfeifer and Morrison 1980: vol. II).

cific horizontal, astronomic, or gravity data must be available in the NGS data base if the point is to be coded as providing such control. Control classifications appear on the back of the form and in table 2-8.

If a recovered point has not been previously connected to the national network, but it is connected by the current survey, enter "X" in addition to any other codes. If a point is described that has not been connected to the network in the past and is not connected by the current survey, enter "N". (A description is only required for such a point if it is marked by a disk or flange belonging to NGS).

Recovery information. In each recovery description, enter information about the agency which searched for the point. This includes a code indicating the type of agency, a standard abbreviation for the agency's name, the year the point is searched for, and the initials of the project director of the party conducting the search. Use the same codes and abbreviations that appear in tables 2-6 and 2-7 to denote the setting agency.

Enter a code for the condition of the monument on line *25*. If the monument appears to be undisturbed and is suitable for leveling, in good or fair condition, enter "G." If it has been disturbed, or if it cannot be positively identified because of mutilation, enter "P". If the monument is found destroyed enter "X". (See sec. 2.4.1, "Destroyed bench marks.") If it is not recovered, enter "N", even if it is presumed to have been destroyed. Explain any condition other than good in the descriptive text.

Construction details. Describe the construction of the monument by entering the two-digit code that denotes the setting classification. Table 2-9 summarizes setting classifications. If the monument is a mark installed in a preexisting structure (codes 40 through 55), describe the setting specifically with a short phrase on line *26*. For example, "51/ VERTICAL IN OVERPASS HEADWALL," "42/CULVERT," and "44/ VERTICAL IN BUILDING FOUNDATION."

Table 2-8.—Control classification codes designated by NGS for preparing NOAA Form 76-186

Other control	Code[1]
Fault monitoring	F
Tide or water-level	T
Horizontal	H
Astronomic	A
Gravity	G
Magnetic	M
Other (explain in text)	O
Not previously connected to the National Geodetic Vertical Network:	
Connected by the current survey	X
Not connected by the current survey	N

[1] Refer to *Input Formats and Specifications of the National Geodetic Survey Data Base* (Pfeifer and Morrison 1980: vol. II).

Table 2-9.—Monument classification codes designated by NGS for preparing NOAA Form 76-186

Code	Setting classification	Quality[1]
00	Unspecified	D
	Shallow settings (less than 3 m or 10 ft):	
10	Unspecified shallow	D
11	Metal rod with base plate	C
12	Concrete post	C
13	Shallow pipe	D
14	Shallow metal rod without base plate	D
	Unsleeved deep settings:	
20	Unspecified deep	C
21	Copper-clad steel rod	B
22	Galvanized steel pipe	B
23	Galvanized steel rod	B
24	Stainless steel rod	B
25	Aluminum alloy rod	B
	Rocks and boulders:	
30	Unspecified rock	B
31	Rock outcrops, such as a rock ledge, rock cut, or bedrock	A
32	Boulder	C
	Light structures:	
40	Unspecified light structure	D
41	Pavements, such as streets, sidewalks, aprons, curbs, and so forth	D
42	Retaining walls and bulkheads, including headwalls and wingwalls of culverts and small bridges	C
43	Piles and poles, such as a spike in utility pole	D
44	Footings, foundation walls, and abutments of small or medium size structures	C
45	Mat foundations, such as landings, platforms, steps, floors, tower foundations, semaphore bases, and other bearing surfaces of the same size as the structure	C
	Massive structures:	
50	Unspecified massive structure	B
51	Massive retaining walls, including headwalls and wingwalls of very large bridges	B
52	Abutments and piers of very large bridges	B
53	Tunnels	B
54	Massive concrete, masonry, or steel structures with deep foundations	A
55	Large concrete, masonry, or steel structures with foundations on bedrock	A
	Sleeved deep settings:	
60	Unspecified rod or pipe in sleeve	B
61	Copper-clad steel rod in sleeve	B
62	Galvanized steel pipe in sleeve	A
63	Galvanized steel rod in sleeve	A
64	Stainless steel rod in sleeve	A
65	Aluminum alloy rod in sleeve	A

[1] Refer to *Input Formats and Specifications of the National Geodetic Survey Data Base* (Pfeifer and Morrison 1980: vol. II).

If the monument is a concrete post (code 12), describe the height of the post with respect to the ground surface in metric units on line *40*. Similarly, if the monument is a rod or pipe which has been driven into the ground (codes 11, 13, 14, 20-25, or 60-65), give the depth, the sleeve depth (if any), and the height of the casement with respect to the ground surface in metric units on line *41*.

Describe the type of device marking the control point by entering the appropriate one-letter code on line *27*. If the mark is a survey disk, enter a two-digit code to indicate the type. A list of marks and corresponding codes is given on the back of the form and in table 2-10. Figures 2-26 and 2-27 show the bench mark disks used by the former Coast and Geodetic Survey and the National Geodetic Survey.

Quality classification. In addition to the variety of monuments set by NGS, monuments of even greater diversity, set by other Federal, State, and local agencies, have been and continue to be included in the national network. See appendix D of the mark setting manual. To assist the user of vertical control data in selecting the most reliable control points for the purpose of the intended survey, monuments have been classified according to their expected ability to remain stable in relation to the local topography. One of four quality codes is automatically assigned to each control point on the basis of the construction details provided in the description. The codes are defined as follows:

1. Quality A describes a monument of the most reliable construction, likely to be affected only by movement of the geological feature in which it is installed.

Table 2-10.—Bench mark classification codes designated by NGS for preparing NOAA Form 76-186

Type of mark	Code[1]
Survey disk:	
Unspecified	D 00
Bench mark (vertical control)	D 01
Tide or water-level bench mark	D 02
Triangulation station (horizontal control)	D 03
Traverse station	D 04
Topographic station	D 05
Reference mark	D 07
Azimuth mark	D 08
Gravity station	D 09
Gravity reference mark	D 10
Magnetic station	D 11
Other type (explain in text)	D 06
Flange-encased rod	F
Metal rod	I
Pipe cap	P
Cap-and-bolt	C
Bolt	B
Rivet	R
Nail	N
Spike	S
Chiseled cross	X
Chiseled circle	O
Chiseled square	Q
Chiseled triangle	T
Drill hole	H
Stone monument	V
Other (explain in text)	Z

[1] Refer to *Input Formats and Specifications of the National Geodetic Survey Data Base* (Pfeifer and Morrison 1980: vol. II).

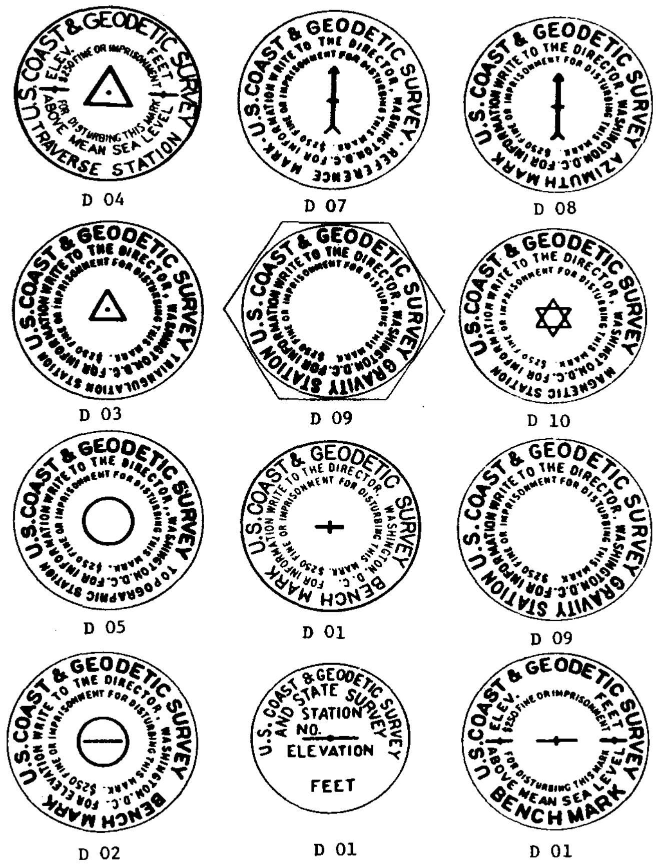


Figure 2-26.—Survey disks of the former U.S. Coast and Geodetic Survey.

2. Quality B describes a monument of construction likely to prove reliable.
3. Quality C describes a monument that may be affected by surface movement, such as frost heave or the shrinking and swelling of certain clays.
4. Quality D describes a monument not likely to prove reliable, or a monument of unknown construction.

Of course, generalizations have been necessary to implement this system. However, the experienced mark setter may greatly improve the value of the quality estimate by assigning the code in the field. A monument of low expected quality in one location might require a higher classification in another. For example, an unsleeved stainless steel rod (quality B) is subject to movement due to soil shrinkage or expansion in regions where the soil readily absorbs and retains water. In regions with low humidity, rare frost, and nonexpansive soil, the same monument may be upgraded to quality A.

Similarly, a monument of high expected quality might, when construction details are examined, require a lower classification. For example, a disk set in the wingwall of a very large bridge (quality B) is less likely to prove reliable if the wall is cracked and undercut by erosion (quality C).

More details on placement and construction of monuments are presented in section 2.4.2 and in the mark setting manual. Using table 2-9 as a guideline, apply your judgment in specific cases to assign a more realistic estimate of quality when it is appropriate. Enter the quality code after the setting classification.

Stamping. A control point marked with a disk flange-encased rod, pipe cap, or cap-and-bolt is normally stamped with the designation and year it was set. Sometimes an elevation may be stamped as well. A precast inscription is not considered part of the stamping.

The stamping serves as final verification of the identity of the control point. Enter it exactly as it appears on the mark. If no stamping appears, make note of it in the descriptive text.

From the precast inscription (if any), determine the originating agency for the mark. In the space following the stamping, enter the acronym for the agency as it appears on the mark.

Text. The text ("body") of the description must provide thorough and accurate instructions for reaching the control point. It should also describe features affecting the utility of the point for vertical control, clarifying information coded elsewhere in the description when necessary.

The fact that a control point is recovered by following instructions from the previous description does not, in itself, imply that the instructions are correct in every detail or that they will apply in the future. Verify all items in the previous instructions and assess their future usefulness. If no changes are necessary, the recovery note need not include a text entry. If brief corrections or additions are necessary, enter them. Here are two examples: "The 1935 description is adequate except that the sign 'JONES FARM' no longer exists"; and "The 1952 description is adequate. The point is 0.5 meter (1.6 ft) south of a witness post."

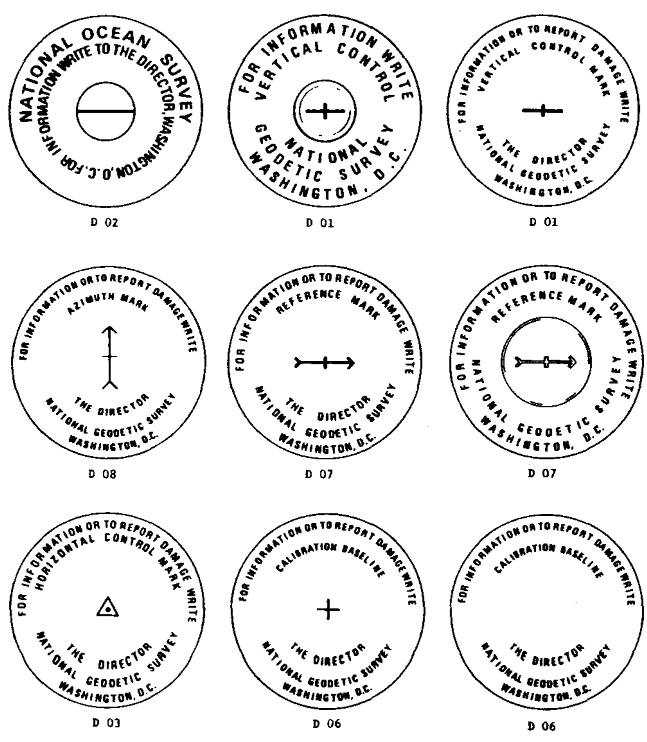


Figure 2-27.—Survey disks of the National Ocean Survey and National Geodetic Survey.

When a point is found destroyed or cannot be recovered, describe the situation. Here are three examples: "The concrete post was found broken off at the base and the disk was removed at this time." "After a 30-minute search by two persons, the point was not recovered. Reference objects given in the 1948 description were not found." "The bridge mentioned in the 1952 description has been rebuilt and the mark appears to have been destroyed."

If a point is particularly difficult to recover, the previous instructions are probably not adequate even though they may be correct. In such cases, and whenever major corrections are necessary, prepare a complete new description, including in the text a new set of instructions to reach the point.

Present instructions in a consistent, clear, and concise format. Start from a major highway intersection or a prominent, permanent landmark in the nearest city or town. Mention landmarks and reference objects in the order in which they will be used. Lead the surveyor first to the general vicinity of the monument and then to the exact point. For convenience, treat these two portions of the description separately, as shown in figure 2-24.

The instructions to reach the general vicinity should follow the most direct driving route (not necessarily the leveling route). Identify highways by Federal, State, or county number. To ensure that the measured distances between landmarks and turns are accurate, calibrate your vehicle's odometer. Enter distances to the nearest tenth of a kilometer, followed in parentheses by miles to the nearest half-tenth. Enter compass directions to the nearest cardinal or intercardinal point: north, northeast, east, southeast, and so forth. When describing a turn or the location of a landmark, enter a compass direction, followed by "right" or "left" in parentheses.

Since vehicle odometers differ, if driving distances are long, give distances to intermediate landmarks that can serve as check points. In addition to noting turns, mention a prominent landmark visible from the highway as near to the control point as possible, at least within the final kilometer. Within city or town corporate limits, include the address (if any) of the property on which the point is situated.

For a control point located about halfway along the line between two cities or towns, include driving instructions from each town, prefacing the second set of instructions with the word "also."

Once in the general vicinity, direct the surveyor to the point by listing measured distances and directions from nearby reference objects in an order convenient to the recovery of the mark. Begin with the most conspicuous object or the longest distance. A good practice is to enter first the distance and direction from the centerline of the highway, so the surveyor can search on the appropriate side.

Select and describe reference objects carefully. Consider the available objects when selecting the site for a new mark. (See page 6 in the mark setting manual.) Keep in mind that a monument may have become obscured by brush or may be buried under dirt and gravel. The lines of position for at least two of the objects should intersect at nearly a right angle. Select objects that are well defined, permitting such precise measurements that extensive digging or clearing of brush is not required to recover the point. Avoid objects to which the distance must be estimated. A total of three objects, plus a witness post, are sufficient for most control points. When describing reference objects, indicate whether sizes are standard, approximate, or measured by observing the following conventions: A standard size is given in standard units, with a hyphen between the numeral and the unit; for example, "1-inch pipe" or "2- by 4-inch timber." An approximate size is given in metric units, prefaced by the word "about"; for example, "a walnut tree about 15 cm in diameter" or "a timber about 15 by 25 cm." A measured size is given in metric units; for example, "a post 10 cm in diameter."

Measure distances horizontally (not along the slope) to the nearest hundredth of a meter. After the metric distance, enter the distance to the nearest tenth of a foot in parentheses. If an object is more than 30.5 m (100 ft) from the point or is not definitive (such as the centerline of a highway), measure to the nearest tenth of a meter (integral foot). Precede estimated distances with the word "about."

Measure from the center of objects such as trees and telephone poles and perpendiculary to objects such as fences and walls. Otherwise state the point from which the measurement was made. For example: "6.31 m (20.7 ft) north of the northeast corner of a semaphore base."

Be sure to describe the height of the mark relative to the ground or highway surface. A standard entry for this is available on line *42* of the description form. A similar standard entry for the measurement from a witness post is available on line *43*.

Enter additional notes as necessary to describe the monument or to explain unusual leveling procedures that may be required. Do not confuse culverts with bridges, piers with abutments, and retaining walls with foundations. Clarify information that may be somewhat misleading as presented in codes elsewhere in the description. For example, note the following: a mark which might be expected to bear a stamping but does not; a mark stamped with a designation duplicating that of another; a mark mounted vertically; the precise location of the control point on a large structure or an unusual monument; the presence of ferrous material in or adjacent to a monument; and the name of the owner on whose property the monument is set. Include your initials at the end of the text.

Throughout the text the following standard abbreviations may be entered for units of measurement: "m" for meter, "mm" for millimeter, "cm" for centimeter, "km" for kilometer, "ft" for foot, and "mi" for mile. Unless undue confusion would result, avoid entering quarter-quadrant compass directions, such as east-southeast or north-northwest. Use the accepted terminology found in a dictionary or engineering textbook; avoid the use of jargon.

2.5 Relocating Vertical Control Points

Each vertical control point represents a large investment of resources. Since it is intended to provide a continuous record of elevation change, as well as control for many local surveyors, its preservation is vital to the maintenance of the national network. Although monuments are constructed in locations where they are unlikely to be disturbed, many are destroyed or damaged by highway widening and maintenance, bridge rebuilding, railroad maintenance, and building demolition and construction. Such deterioration of the network must be minimized.

Although regional mark maintenance engineers monitor the condition of the network, the areas they cover include many States. To preserve specific points, the cooperation of local engineers and surveyors is important. When recovering a mark, prepare a brief report of its condition (fig. 2-28). If a monument is about to be destroyed, notify NGS at once. Before it is destroyed, a vertical control point should be relocated as described here.

First, a new monument is constructed in the vicinity. Then the elevation difference is measured by leveling from the old to the new control point. Finally, the old monument is destroyed. Data are normally recorded on NOAA Form 76-60, "Report on relocation of bench mark." (See fig. 2-29.)

Setting a new monument. After NGS is given the designation and location of the monument about to be destroyed, the agency will send to the reporter a brass disk, which is prestamped with the designation and year of relocation. For example, the disk for M 346 is stamped "M 346 RESET 1980."

Select a site for the new control point, if possible within one leveling setup from the monument to be replaced. Construct a monument of the best possible quality in which the disk may be set. (See the instructions in 2.4.2 and in the mark setting manual.) Recommended settings for this purpose are a class B rod mark (with the disk crimped to the top), bedrock, or a large and stable structure. If a witness post is near the old monument, relocate it to the site of the new one.

Prepare a complete description of the new point, either on the report form or on a standard description form (NOAA Form 76-186). On the report form, be sure to include instructions to reach the point from the nearest city or town, construction details, and a position. If the position cannot be plotted and scaled as explained in section 2.4.3, note the 30′ quadrant of Vertical Control Data and the number of the quadrant line in which the original point was described.

☆ U.S. GOVERNMENT PRINTING OFFICE: 1974-665-074/1047 Region 6

REPORT ON CONDITION OF SURVEY MARK

Form Approved
Budget Bureau No. 41-R1923

Name or Designation: U 150 Year Established: 1941

State: PA County: Huntingdon Organization Established by: C&GS

Distance and direction from nearest town: about 4.6 miles north along PA RR from station at Huntingdon

Description published in: *(Line, book, or quadrangle number)* Quad 400781

Mark searched for or recovered by: Name - Roger C. Poe

Organization - Michael Baker, Jr., Inc.

Date of report June 17, 1975 Address - P. O. Box 280, Beaver, PA 15009

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

Mark stamped:	Condition:
U 150 1941	Good

Marks accessible? [X] Yes [ ] No Property owner contacted? [ ] Yes [X] 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:

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

If additional forms are needed, indicate number required. ______

NOAA FORM 76-91 (11-72) U.S. DEPARTMENT OF COMMERCE - NATIONAL OCEANIC AND ATMOSPHERIC ADMINISTRATION - NATIONAL OCEAN SURVEY

Figure 2-28.—Report on the condition of a survey point.

Leveling to the new control point. Leveling should satisfy standards for second-order, class II accuracy or better. If the distance between the old and new points is greater than 140 m (460 ft), or if more than one setup is necessary, comply with the specifications for geodetic leveling given in chapter 3 of this manual. In most cases, however, leveling may be conducted as follows, with an engineer's level and a single rod. Be sure to record the type of instrument and the units of the rod scale (meters or feet). (See the example in fig. 2-29.)

1. Set up the instrument halfway between the old and new points, but no more than 70 m (230 ft) away from either point.

2. Plumb the leveling rod on the highest point of the old bench mark. Record the designation of the point. On the same line enter the published elevation.

3. Observe the intercept of the middle reticle line against the rod scale. Record the reading to the third decimal place. This completes the backsight, BS.

4. Compute the height of instrument, HI, which is the sum of the backsight and the elevation of the point. Record it on the same line.

5. Do not move the instrument. Plumb the rod on the highest point of the new bench mark. Record the designation of the point on the next line.

6. Again, observe the rod intercept. Record the reading on the same line as for step 5. This completes the foresight, FS.

7. Compute the elevation of the new point, which is the height of the instrument from the previous line minus the foresight. This completes the forward run.

8. Reset and relevel the instrument. Level backward, from the new point to the old, in the same manner as in steps 2 to 7. The elevation computed for the old point as the result of the backward leveling may differ from the published elevation by no more than ± 0.003 m (± 0.010 ft).

9. To compute the elevation difference from the old point to the new, subtract the mean of the two elevations for the old point from the elevation for the new point.

Destroying the old control point. Do not destroy the old point until the leveling data have been checked. Remove the old disk and any other parts of the monu-

NOAA FORM 76-60 (6-71)

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

REPORT ON RELOCATION OF BENCH MARK

(See instructions on reverse side)

DESIGNATION OF MARK: K 133

STATE: NEW MEXICO — COUNTY: LUNA

A. DESCRIPTION OF ORIGINAL MARK

FROM THE SOUTHERN PACIFIC COMPANY RAILROAD STATION IN COLUMBUS, NORTH ALONG STATE HIGHWAY 11 FOR 4.6 KM (2.9 MI), 11.6 METERS (38 FEET) EAST OF THE CENTERLINE OF THE HIGHWAY, 19.5 METERS (64 FEET) WEST OF A ROW OF POLES, AND 0.34 METER (1.1 FOOT) EAST OF A WHITE WOODEN WITNESS POST. A STANDARD DISK SET IN TOP OF A CONCRETE POST PROJECTING 0.30 METER (1.0 FOOT) ABOVE GROUND. DESTROYED THIS DATE.

PUBLISHED ON N321082 LINE 101 — STAMPING K 133 1934

B. WAS ORIGINAL BENCH MARK DESTROYED? ☒ YES ☐ NO — HORIZONTAL CONTROL POINT? ☐ YES ☒ NO

REMARKS

INSTRUMENT: NI 2 50132
ROD: PHILADELPHIA, FOOT UNITS

DATE OF LEVELING 23 MARCH 1977

(Feet or ~~Meters~~)*

POINT	B.S.	H.I.	F.S.	ELEVATION	REMARKS
K 133	3.791	4182.759		4178.968	CLEAR, CALM, WARM
K 133 RESET 1977	5.177	4182.745	5.191	4177.568	
			3.789	4178.956	.012 > .009 REJECTED
K 133	3.801	4182.769		4178.968	
K 133 RESET 1977	5.250	4182.820	5.199	4177.570	4177.570
			3.858	4178.962	.006 MEAN = 4178.965
					ELEVATION DIFFERENCE = − 1.395 FT

C. DESCRIPTION OF NEW MARK

STAMPING ON NEW MARK K 133 RESET 1977

FROM THE SOUTHERN PACIFIC COMPANY RAILROAD STATION AT COLUMBUS, NORTH ALONG STATE HIGHWAY 11 FOR 4.6 KM (2.9 MI). 30.1 METERS (99 FEET) EAST OF THE CENTERLINE OF THE HIGHWAY, 12.19 METERS (40.0 FEET) SOUTH OF MILEPOST 6.13, IN LINE WITH A ROW OF UTILITY POLES, AND 0.30 METER (1.0 FOOT) WEST OF THE HIGHWAY RIGHT-OF-WAY FENCE. A STANDARD DISK SET IN TOP OF A CONCRETE POST PROJECTING 0.30 METER (1.0 FOOT) ABOVE GROUND.

SIGNED NAME PHONE

AGENCY NAME AND ADDRESS

*Strike out unit NOT used.

Figure 2-29.—Report of a relocated bench mark.

ment that bear a stamping (such as a logo flange or stainless steel cap on a rod) and return them to NGS.

Prepare a recovery note on the report form or a standard description form (NOAA Form 76-186), identifying and explaining the disposition of the old monument.

Submitting relocation records. To include relocation data in the national network, the following records must be forwarded to NGS:

1. Complete description of the new control point.
2. Recovery note for the old control point.
3. Field records for at least two runs of leveling, one forward and one backward, including the computed elevation difference from the old to the new point.
4. Name, address, and telephone number of the individual who performed the relocation.

2.6 References

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Chapter 3
GEODETIC LEVELING

3.1 Introduction

In a widespread network of vertical control, geodetic leveling is the technique that provides the most reliable elevation differences between control points. It is a form of precise leveling where the observing team limits the magnitude of error by using calibrated instruments in combination with a rigorous, symmetrical observing procedure. In the following pages leveling procedures are described, the principal sources of error are discussed, and error tolerances are presented to provide a foundation for the instructions that comprise the rest of the chapter.

3.1.1 General Procedures

Along each line of a vertical control network, leveling is conducted in increments called sections. Each section is an unbroken series of setups, made between two permanent control points. A setup consists of a point supporting a backsight rod, a point supporting a foresight rod, and a leveling instrument positioned between them (fig. 3-1). Two heights are measured by sighting through the instrument toward a scale on each rod and recording the values intercepted by a line on the reticle. The height difference, backsight minus foresight, corresponds to the elevation difference between the two points. The foresight point of one setup becomes the backsight point of the next; thus, the sum of the elevation differences of the series of setups is the elevation difference for the section (fig. 3-2).

Three conditions must be satisfied for this technique to provide reliable elevation differences between points. First, the lines of sight from the instrument to the rods must be level; in other words, the lines of sight must be parallel at all times to the reference surface. Second, the values observed on the scales must accurately indicate heights above the points on which the rods rest, and third, the points in turn must be stable with respect to the topography.

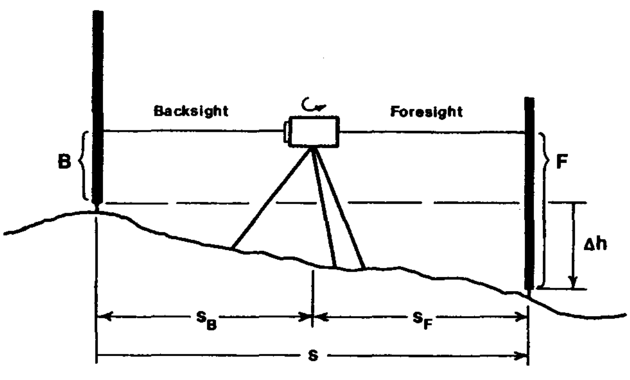

Figure 3-1.—Setup of leveling, $\Delta h = B - F$ and $s = s_B + s_F$

These conditions cannot be perfectly satisfied in the "real world"; however, they may be approximated by limiting the known sources of error. Leveling is classified by the degree with which error magnitudes are limited. (See sec. 3.1.3, "Tolerances for geodetic leveling.")

3.1.2 Sources of Error

Error may be characterized as random or systematic. Random error in leveling results represent the effect of unpredictable variations in the instruments, the environment, and the procedure of leveling. Random error cannot be completely eliminated, although it can be kept small. Therefore, it represents the "noise level," the limit on the precision with which leveling may measure elevation differences.

Systematic error represents the effect of consistent inaccuracies in the instruments or in the leveling procedure. It also results from consistent, though not always predictable, environmental effects. Although systematic error may be small in a single measurement, it accumulates when measurements made under similar circumstances are totaled. Thus, it can result in a significant discrepancy in the elevation differences measured between two control points by different leveling systems and/or routes. For leveling to provide accurate elevation differences, systematic error must be eliminated, either by procedure or by applying corrections to the data.

The sources of error in leveling can be classified into three groups: those affecting the line of sight, those affecting the heights observed, and blunders. The line of sight cannot be exactly level because of the effects of imperfections in the instrument and variations in the human eye, refraction, curvature, and tidal accelerations. The heights observed are not exact because of imperfections in the rod scales and the turning points; furthermore, a perfectly stable relationship cannot be maintained between the equipment and the topography because of environmental effects on the equipment. Blunders may occur while attempting to limit any of these errors.

Leveling instrument. The instrument used for geodetic leveling should consistently provide a horizontal

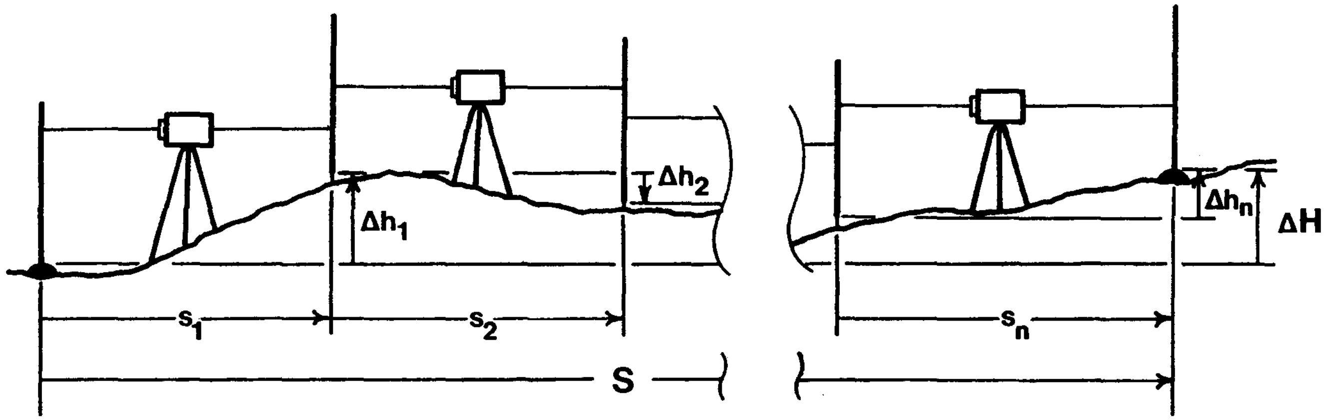

Figure 3-2.—Section of leveling, $\Delta H = \Delta h_1 + \Delta h_2 + \ldots + \Delta h_n$ and $S = s_1 + s_2 + \ldots + s_n$.

line of sight. The extent to which it achieves this determines its suitability for various orders and classes of leveling.

To be horizontal, the line of sight should be perpendicular to the direction of gravity, at the vertical axis of the instrument. If the line of sight is not horizontal, the angle by which it deviates from being horizontal causes an error in every observation (fig. 3-3). This angle is referred to as collimation error.

Collimation error can be limited by using a well-designed, properly maintained instrument. The angle should be measured and adjusted to specifications. The effect of collimation error on each observation can be reduced by limiting the sighting distance. Furthermore, if the sighting distances in each setup are balanced, the errors resulting from collimation error become equal. They cancel when the foresight is subtracted from the backsight to compute the elevation difference (fig. 3-4).

Although it is impractical to balance every setup exactly, the total contribution of collimation error can be limited very effectively while leveling by keeping the imbalance small and random in sign. Any systematic contribution that accumulates with distance may be eliminated by later applying corrections computed from the imbalance and a precisely determined value for the collimation error.

Collimation error should not change as the instrument is refocused or rotated about its vertical axis. A consistent difference between the collimation errors of the backsight and foresight causes a systematic accumulation of error (fig. 3-5). The precautions described here will limit this effect, but, because it is unpredictable, it cannot be eliminated. Additional precautions should be taken to ensure that a consistent difference does not exist.

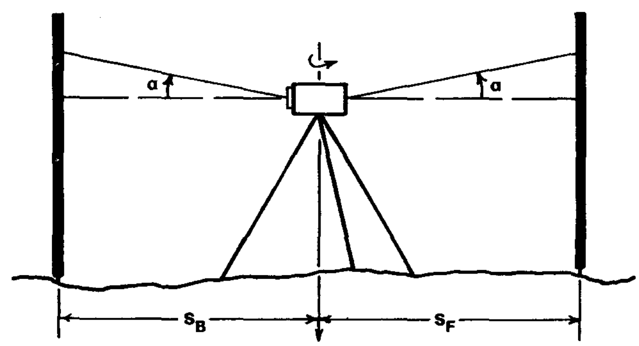

Figure 3-4.—Consistent collimation error cancels in a balanced setup since $s_B = s_F$.

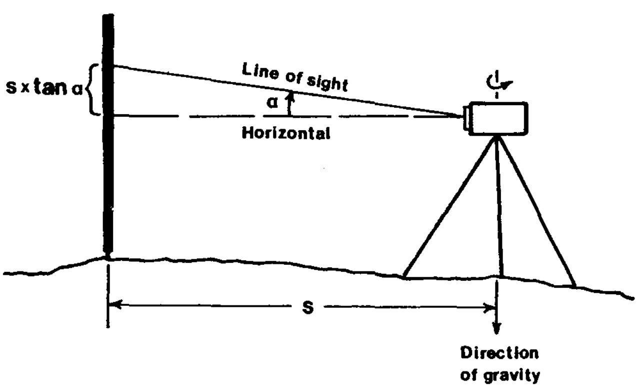

Figure 3-3.—Effect of collimation error, α.

In a compensator-type leveling instrument such a difference can occur if the compensator is not suspended properly in each direction. To prevent gross error the spherical level on the instrument should be properly adjusted, and compensation checks should be performed routinely. To eliminate smaller systematic effects, the compensator should be repositioned during each setup.

In a spirit-level type of leveling instrument, the error caused by imprecision in centering can become systematic if the bubble is consistently affected by a heat source in one direction. Shading the instrument should reduce this effect.

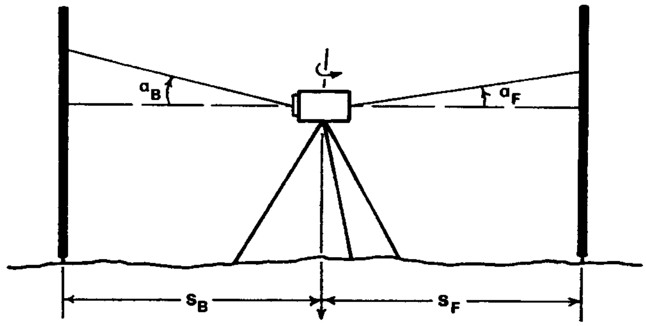

Figure 3-5.—Inconsistent collimation error does not cancel in a balanced setup since $\alpha_B \neq \alpha_F$, even if $s_B = s_F$.

Pointing. Another effect on the line of sight results from the human inability to repeat a pointing exactly. Both imperfections in the instrument and atmospheric refraction may contribute to this effect, combining with the imperfection of the human eye, to create pointing error.

The magnitude of pointing error is reduced by the use of a precise instrument with a micrometer and wedge reticle that has been adjusted to remove parallax. Limiting the sighting distance and instrument movement can also reduce the magnitude of the error.

Refraction. Variations in atmospheric density cause the line of sight to refract or bend in the direction of increasing air density. These variations seem to be primarily a function of air temperature.

Refraction is most noticeable when the line of sight passes through air of fluctuating density, as when "heat waves" are observed. The graduations on the scales appear to move up and down rapidly. This phenomenon, called shimmer (fig. 3-6), makes it difficult to intercept the scales precisely, thus increasing the magnitude of pointing error. It can be reduced by shortening the sighting distances or, in some cases, raising the height of the line of sight.

Whether or not shimmer is observed, the line of sight may be refracted. Since the error caused by refraction increases proportionately with the square of the sighting distance, refraction error may be reduced by limiting the sighting distance. As long as atmospheric conditions are similar along both the foresight and backsight, the error may be nearly eliminated by balancing setups.

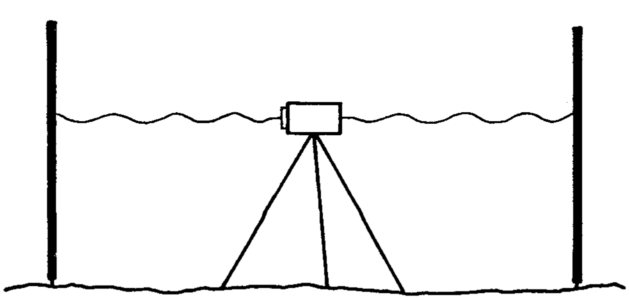

Figure 3-6.—Shimmer.

However, conditions are often not the same along both lines of sight. Air close to the ground changes in density more rapidly than air situated 1 m or more above ground. This can be visualized by imagining air layers, of equal density, conforming to the topography. On a slope, even if setups are exactly balanced, the conditions along the foresight differ from those along the backsight (fig. 3-7). Because the sight uphill passes through a greater change in air density, it is refracted more. Refraction error, then, accumulates with change in elevation.

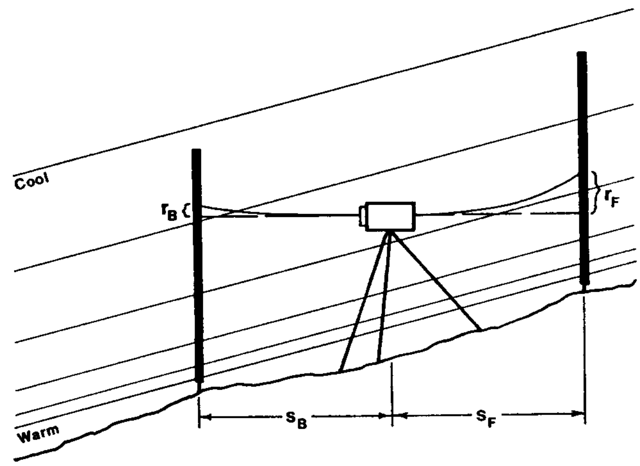

Figure 3-7.—Refraction error, *r*, does not cancel on sloping terrain since $r_B \neq r_F$, even if $s_B = s_F$.

Leveling results may be corrected at least partially for refraction if atmospheric conditions are determined and recorded with the observations. Of the many mathematical models that attempt to predict refraction error, the most successful require that air-temperature differences be precisely measured during every setup.

Two types of refraction error cannot be corrected. Therefore, the situations which cause them should be avoided when leveling. If the line of sight passes very close to the ground or to an intervening object, changes in air density cause the line of sight to be refracted in an unpredictable way. Similarly, when the air near the ground is cooler than the air above it, the relatively stable air layers may move slowly across the line of sight, causing long-period shimmer. The graduations on the scales appear to move up and down, but so slowly that an entire setup may be observed and checked before the movement is noticed. The observations may be significantly and unpredictably affected by the unnoticed shimmer. This effect usually occurs at night when the air is calm.

Curvature. Both the leveling instrument and the rods are oriented to the direction of gravity, to measure elevation differences with respect to the same reference surface. When the instrument is level and is rotated so that the line of sight intercepts each scale, the line of sight should sweep out a horizontal plane. It would be parallel to the equipotential surface if the gravity field, at each setup, also defined a plane. This is not the case, however, since the gravity field defines a curved surface. Thus, a small amount of curvature error is introduced into each observation (fig. 3-8).

Curvature error is proportional to the square of the sighting distance. Assuming the equipotential surface is evenly curved, curvature error can be reduced by making the backsight and foresight distances in each setup nearly equal. If setups are thus balanced, within a tolerance, correction for curvature need not normally be made while leveling.

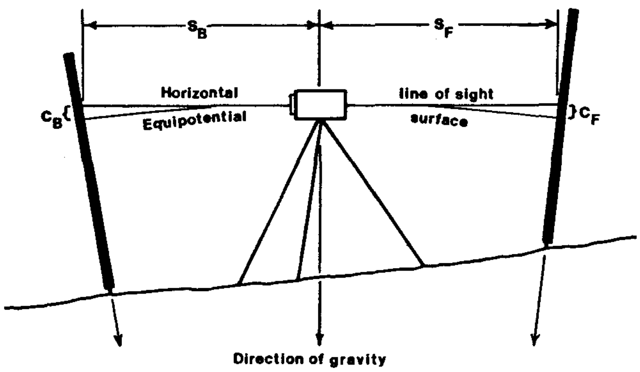

Figure 3-8.—Curvature error, *c*, where the line of sight is not parallel to an equipotential surface, cancels if $s_B = s_F$.

Because the surfaces defined by the gravity field are not evenly curved, minute differences in curvature error in every setup accumulate systematically in leveling conducted over large changes of elevation, particularly in a northerly or southerly direction. Even so, the magnitude of this error is so small that it may be neglected if the backsight and foresight distances are nearly equal.

Tidal accelerations. Because leveling instruments and rods are oriented to the direction of gravity, after curvature has been taken into account, the elevation difference of each section is computed along a route that approximately parallels an equipotential surface. However, the Sun and Moon create tidal accelerations that periodically distort this surface, generally more toward the equator than the poles (fig. 3-9).

The distortion is termed a deflection and is described by two component vectors. The vertical component affects only the magnitude of gravity along the route, resulting in a negligible effect on the elevation difference. The horizontal component, however, acts at 90° to the equipotential surface, resulting in a small error, especially if the section is oriented in a line with the Sun, Moon, and the north or south pole. The error accumulates significantly in leveling lines oriented north-south, particularly in the middle latitudes. To remove it, a correction must be applied.

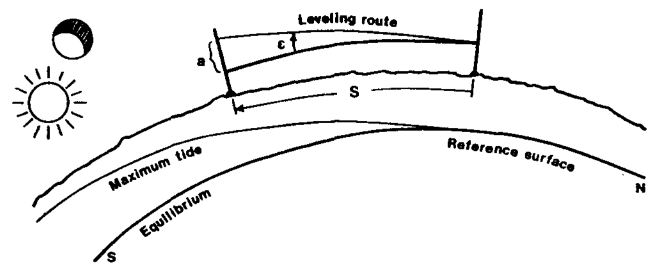

Figure 3-9.—Effect, *a*, of tidal deflection, ϵ, on a section of length and direction, $\vec{S}$.

Leveling rods. To observe accurate heights above the points on which the leveling rods rest, precise relationships must be maintained between the rods and the equipotential surface, and between the rods and the scales mounted on or within them.

The first relationship is ensured by plumbing, or aligning the rods with the direction of gravity, a task which is analogous to leveling the instrument. If they are not so aligned, an error is introduced into each observation. Although the error may be small, it accumulates systematically with change of elevation, especially on steep slopes where observations are made alternately low and high on the scale. (See figs. 3-10, 3-11, 3-12.) It can be limited only by plumbing the scales properly.

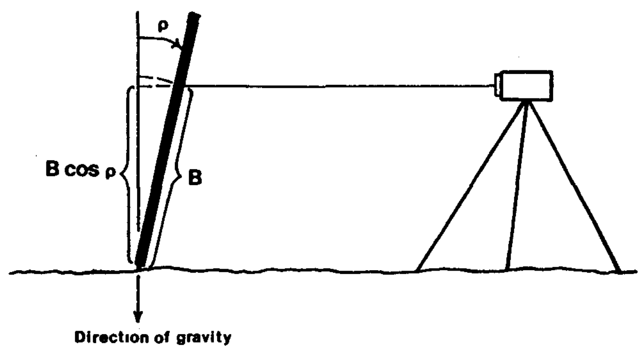

Figure 3-10.—Effect of rod plumbing error, ρ, on a height observation, *B*.

The second relationship depends upon the accuracy with which the scales are manufactured and mounted in or on the rods. If the graduations are not accurately marked above the scale zero or if the scale changes in length during leveling, error may accumulate in the observations. To limit the error, leveling rods should be well-designed, routinely calibrated, and properly handled and maintained.

The index error, which is the difference in height from the scale zero to the base plate of the rod, represents a constant portion of the error in the scale values. Index error can be eliminated by making an even number of setups for every section, thus using the same rod on

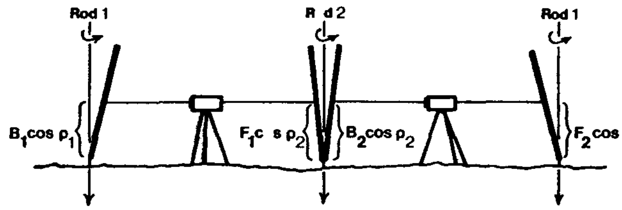

Figure 3-11.—Systematic effect of plumbing error (and scale errors) is small on flat terrain, since $B_1 \simeq F_2$ and $F_1 \simeq B_2$.

every bench mark during the leveling. (It can also be eliminated if only one rod is used if accurate calibration corrections are applied.)

In much the same way as an error in plumbing, error in the scale values causes systematic error to accumulate with change of elevation. For the most accurate results, observed scale values should be corrected to standardize them with respect to the National Standard of Length. To do this the rods must be accurately calibrated. Since the scales are subject to thermal expansion, the coefficient of thermal expansion must also be measured, and scale temperatures should be recorded during the leveling.

Turning points. The leveling rods must be set on clearly defined, stable points. Rather than using natural objects, which may or may not prove to be adequate, portable, standardized turning points should be used. They should provide definite points on which to rest the rods. Both the base plate of a rod and the top of a turning point should be cleaned before placing the rod on the point.

To prevent error when leveling to and from control points, they too should provide definite points. Since this is not always possible, special procedures may be necessary for leveling to awkward points.

Stability. During each setup, the leveling instrument and the rods may change elevation because of settlement or rebound caused by the type of ground cover. To minimize error from such movement, the backsight and foresight should be observed nearly simultaneously. This requires two rods and an efficient instrument and leveling unit.

Instrument movement can be reduced by proper use and maintenance of the tripod. Systematic error resulting from consistent movement of the instrument can be eliminated by using one of two observing procedures. In one procedure, known as micrometer leveling, two elevation differences are measured in two directions during each setup. If the instrument settles significantly during either or both measurements, the observations will not lie within a limit imposed on the difference between the results. Averaging the results practically eliminates any remaining systematic error.

In the other procedure, three-wire leveling, the first reading of the setup is made on the backsight of odd-numbered setups, and on the foresight of even-numbered setups. Consistent settlement of the leveling instrument will cause the results of odd setups to be too large and the results of even setups to be too small by a similar amount. Thus, over a section with an even number of setups, the errors nearly cancel.

Rod movement is minimized by placing the turning points so they provide sufficient stability. Systematic error caused by consistent movement during each setup can be limited somewhat by the procedures described previously. If the rods are allowed to rest on the points for 20 seconds before making an observation, any remaining movement should be negligible.

When proceeding from one setup to the next, the forward turning point must not move. This can only be assured by the rodman's diligence and by comparing repeated levelings of the section. Averaging the results of two runnings, made in opposite directions, nearly eliminates any systematic accumulation of error resulting from consistent movement.

Blunders. Errors that result from failure to follow the specified procedures are termed blunders. In leveling, no unit is so experienced that the work performed by the group can be automatically considered free of blunders. Only conscientious attention to detail, combined with scrupulous checks and cross-checks, can ensure accuracy.

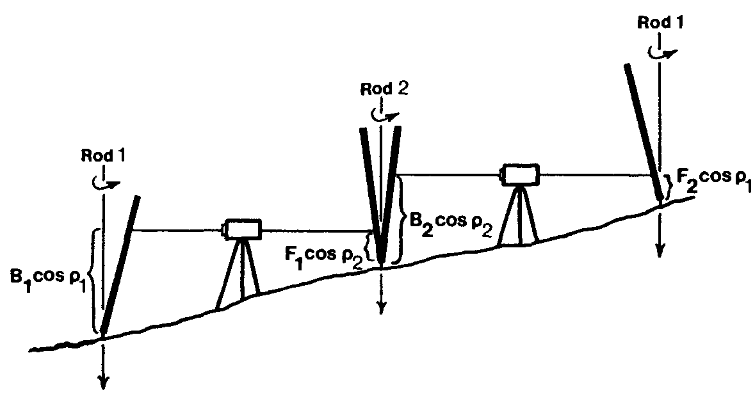

Figure 3-12.—Effect of plumbing error (and scale errors) accumulates on sloping terrain, since $B_1 \neq F_2$ and $F_1 \neq B_2$.

Random blunders are usually large and occur at single setups. They include (1) mistakes in reading or recording scale values, (2) observing the rods in the wrong order, (3) improper placement or leveling of the instrument, (4) gross movement of the instrument or turning points during the setups, (5) improper placement or plumbing of the rods, and (6) gross movement of the forward point between setups. The first four of these blunders may be detected and eliminated with confidence by using double-scale rods, computer-recording equipment, and the micrometer-leveling procedure given in section 3.7.2. In this system, observations are made in such a way that checks for internal consistency can be computed. The last two blunders, if unreported by the responsible rodman, may be detected only by comparing repeated levelings of the section.

Systematic blunders are more difficult to detect. They are primarily caused by failure to follow the procedures necessary for precise leveling, particularly maintenance of the instrument and the rods. If the collimation check and the compensation check are not performed correctly, the use of an improperly adjusted instrument may introduce error, undetectable in each setup, but accumulating over the line. Similarly, if the spherical levels on the rods are not properly adjusted, error may accumulate. If a rod is dropped, or otherwise damaged, previous calibration values may no longer apply and the data cannot be corrected with confidence.

3.1.3 Tolerances for Geodetic Leveling

To produce reliable elevations, the results of geodetic leveling must satisfy the appropriate standard of accuracy for the order and class of a survey. This standard is attained in three ways while leveling: first, by operating a well-organized and well-trained leveling unit; second, by selecting sufficiently precise equipment and calibrating it properly; and third, by adopting observing and recording routines that limit accumulation of error. Specific instructions for satisfying these requirements are presented in the remaining sections of this chapter. Tolerances, suitable for attaining each standard of accuracy, are presented in table 3-1.

Two types of tolerances must be applied. The first type is a set of practical limits on the dimensional factors within the leveling system that are known to increase the magnitude of error in the results. The second type is a set of limits on the precision of the results obtained from two or more levelings between the same two points. The tolerances should reflect the probable magnitude of random error that cannot be corrected or eliminated. Then, blunders and most systematic error can be detected and removed from the leveling data.

The second set of tolerances is defined with the expectation that the tolerances will be exceeded 5 percent of the time (95 percent test level). This permits a check on the validity of the limits imposed. If leveling results exceed the limits more than 5 percent of the time, either the first set of tolerances is not being satisfied or the second set has been made too restrictive (forcing observations that may not be accurate to appear precise). If leveling results exceed the limits less than 5 percent of the time, the second set of tolerances may not be restrictive enough.

Tolerances, revised periodically by the Federal Geodetic Control Committee, are published in *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).

3.2 Organizing a Leveling Unit

During a leveling project one or more units operate under the supervision of a project director who supports and monitors the activities. Like mark setting units, each leveling unit must be capable of working self-sufficiently several hundred kilometers from the project office. Guidelines are presented here for organizing and maintaining such units.

3.2.1 Personnel

A leveling unit normally requires five persons: the observer, the recorder, two rodmen, and a pacer. Certain equipment or conditions may require that additional personnel be assigned. An experienced unit, equipped with a compensator-type leveling instrument and special vehicles, can operate efficiently with four persons.

The leveling unit is exactly what the name implies, a unit, and each person in the group, whatever the position, must be constantly on the alert to ensure that all duties are performed promptly and carefully. Complete and continuous attention to the details of the work is required of all members of a unit if it is to operate smoothly and efficiently. Any member of the unit who is slow, careless, or indifferent can ruin the work of the whole unit.

Unit chief. The unit chief, who usually serves as the observer, is responsible for managing the unit in the field. The quality and quantity of leveling produced depends on the experience and good judgment of the chief. Practical proficiency in each of the leveling duties is a prerequisite for the job. This individual should set an example which ensures correct, safe, and efficient leveling.

The chief should ensure that the specifications and guidelines contained in this chapter are strictly followed by all unit members. New employees, in particular, should be thoroughly instructed in their duties. The chief should ensure that project instructions are read

Table 3-1.—Tolerances for geodetic leveling

	First order class I	First order class II	Second order class I	Second order class II	Third order
Lines of sight					
Maximum sighting distance	50.0 m	60.0 m	60.0 m	70.0 m	90.0 m
Maximum imbalance,					
per setup	±2.0 m	±5.0 m	±5.0 m	±10.0 m	±10.0 m
per section	±4.0 m	±10.0 m	±10.0 m	±10.0 m	±10.0 m
Leveling instrument					
Maximum collimation error, single line of sight	← ±10″.0 ($C \leq \pm 0.05$ mm/m) →				— —
Maximum collimation error, mean of two lines of sight	← ±4″.0 ($C \leq \pm 0.02$ mm/m) →				— —
Maximum angular difference in two lines of sight	← ±40″.0 ($Q \leq \pm 0.20$ mm/m) →				— —
Setting precision	±0″.25	±0.25	±0″.50	±0″.50	— —
Minimum reading	±0.1 mm Micrometer required	±0.1 mm Micrometer required	±0.5 mm	±0.5 mm	— —
Leveling rod					
Plumbing accuracy	±10.′0	±10.′0	±10.′0	±10.′0	
Maximum scale unit	1.0 cm	1.0 cm	1.0 cm	1.0 cm	
Calibration accuracy	±0.05 mm	±0.05 mm	±0.05 mm	±0.05 mm	
Agreement of observed elevation differences before correction, observed backward and forward during					
One-setup section	±0.40 mm	±1.00 mm	— —	— —	— —
Two runnings of a section less than 0.10 km in length	±0.95 mm	±1.26 mm	±1.90 mm	±2.53 mm	±3.79 mm
Two runnings of a section of one-way length K: $T \times \sqrt{K}$ mm, $T=$	±3.00	±4.00	±6.00	±8.00	±12.00
Three or more runnings of a section: Each accepted running must differ from the mean of all accepted runnings by no more than $t \times \sqrt{K}$ mm, t given below. For a section less than 0.10 km in length, let $K=0.10$.					
Number of levelings:					
3	2.10	2.81	4.21	5.63	8.44
4	2.33	3.10	4.66	6.23	9.34
5	2.48	3.31	4.96	6.64	9.95
6	2.59	3.46	5.19	6.94	10.4
7	2.68	3.58	5.36	7.18	10.7
8	2.75	3.67	5.51	7.37	11.0
One leveling of a loop of one-way length K, beginning and ending at the same point: $T \times \sqrt{K}$ mm, $T=$	±4.00	±5.00	±6.00	±8.00	±12.00

and understood by unit personnel, resolving any technical questions before work begins. When substantial confusion or doubt about a field procedure exists, the chief should seek immediate guidance from the project director.

In addition to technical concerns, the chief is responsible for the safety of personnel, and for the security and maintenance of all equipment assigned to the unit. Any deficiencies in these areas must be reported immediately to the project office. The chief submits weekly and monthly status reports, and is also charged with the safekeeping of the leveling data until they are properly transmitted to the project office.

The chief can improve the unit's efficiency by planning each day's activity in advance. By reviewing the logs in advance of the surveying date, sections which require unusual survey procedures can be pinpointed and the project director notified if any additional personnel or equipment are needed. It is helpful each morning to explain the day's goals to the unit. Whenever possible, duties should be rotated to provide opportunities for unit members to develop and upgrade their skills. When weather conditions or other factors prevent leveling, the chief should assign unit members other duties such as maintaining equipment or augmenting their skills and knowledge.

Observer. The unit chief or another individual who serves as the observer should have a thorough understanding of the leveling instrument, as well as the experience to operate it correctly and efficiently. The observer is responsible for maintaining the instrument and the tripod, and should follow the precautions outlined in subchapters 3.3, "Leveling instruments," and 3.6, "Atmospheric conditions." He or she should be constantly alert, to ensure that the route for the line is followed correctly, to plan ahead to avoid problematical setups, and to monitor the activities of the rodmen. The observer should check the recorder's work periodically and check and initial all records at the conclusion of each day's work.

Recorder. The recorder enters observation data on recording forms or into a portable computer. He or she should write legibly and be able to follow exactly the recording procedures outlined in subchapters 3.7, "Observing routine," and 3.8, "Field records." A clear understanding of the mathematical computations and checks is important, whether recording on a form or a computer, and the ability to compute these checks rapidly is essential when using recording forms. When using a computer-recording system, the recorder should be familiar with the various recording programs and skilled in operating and maintaining the equipment.

The recorder should be an alert and conscientious driver, since the truck that transports the rodmen and observer from setup to setup can be used as a recording station. In addition, the recorder should support the observer by checking constantly for blunders or oversights, which would lower the quality of the leveling, and by serving as acting observer in the absence of the unit chief.

Rodman. Although the position is often filled by an individual with little surveying experience, the rodman's duties are critical to precise leveling. With a computer-recording system, observing and recording blunders may be discovered and corrected before leaving each setup; however, no check can be made as to whether or not a turning point or leveling rod has moved between setups unless the entire section is releveled. The turning point and the leveling rod are the responsibility of the rodman.

He or she should be unquestionably reliable and conscientious, able to pace precisely, adept at handling the rod, and meticulous in setting turning points, following the precautions described in subchapters 3.4, "Leveling rods," and 3.5, "Turning points." The rodman is charged with the responsibility of maintaining the equipment properly and reporting deficiences promptly. When a mistake is made (e.g., pulling a turning pin too soon or accidently bumping a turning plate), the rodman should report it immediately. For motorized leveling, the rodman must be able to operate skillfully a specially equipped motorcycle or small vehicle, and perform minor maintenance.

Pacer. The pacer, an optional position which is usually rotated with the position of the rodman, marks out precisely balanced setups and sets the turning points ahead of the unit. He or she should be able to select a satisfactory leveling route, based on the information provided in the logs. The pacer also sets up warning signs and returns for the leveling truck after sections have been walked. In motorized leveling, this position is not necessary if the vehicles are equipped with accurate odometers.

Other personnel. A spirit-level instrument must be shaded during leveling, thus creating the need for a person to hold an umbrella. In three-wire leveling, this person assists the recorder by making the back check. (See sec. 3.7.4). He or she may also be needed to shield the instrument in extremely windy conditions.

If a traffic lane must be closed, two flagmen are required to divert traffic. These are usually the pacer and a sixth person who is temporarily assigned to the unit.

3.2.2 Equipment

Survey equipment, itemized in appendix B, should be readily available and routinely maintained by the personnel in the unit. An inventory should be conducted when the equipment is issued to the unit chief and at least once a year thereafter. All equipment should be stored securely, in an organized fashion, in the leveling truck or other assigned vehicles.

Items of survey equipment have been classified and coded according to type. The codes must be recorded together with the serial numbers of equipment used to collect data that are to be included in the national network. The codes are listed in *Input Formats and Specification of the National Geodetic Survey Data Base* (Pfeifer and Morrison 1980: vol. II, annex F).

In general, the individual who uses a piece of equipment should be responsible for its maintenance. Any defective or damaged equipment should be reported immediately to the project director. When the unit is working at a substantial distance from the project office, a spare leveling instrument should be available, especially if the instrument's collimation error cannot be adjusted in the field.

When equipment requires maintenance that cannot be performed by the leveling unit, the equipment should be returned to the project office. If the equipment cannot be delivered in person, it should be packaged properly to prevent damage in shipment. Instruments and computer-recording equipment are particularly vulnerable to such damage.

Motorized leveling. The term "motorized leveling" refers to using vehicles to transport all members of the leveling unit from one setup to the next. When personnel proceed between setups, they must usually wait for the backsight rodman, who has twice the distance to travel. Traditionally, the leveling truck, driven by the recorder, transports the rodmen half this distance to reduce the cycling time of each setup. When the rodmen are equipped with vehicles, the cycling time can be further reduced, especially if each vehicle is provided with a mechanism for setting the turning point and raising and lowering the rod without leaving the vehicle.

A specialized observing vehicle driven by the recorder may allow the observer to set up and level the instrument more efficiently, perhaps at a height sufficient to reduce the effect of refraction. A built-in intercom can prevent misunderstandings between the observer and the recorder. In addition, both the instrument and personnel are protected from the weather, permitting leveling to continue under a wider variety of conditions.

In the traditional, partially motorized routine, the backsight rodman proceeds to the leveling truck and rides to the instrument position of the next setup. From there the rodman proceeds to the foresight position, at which the pacer should already have placed a turning point. When only four persons are available, the rodmen must pace to and set their own points.

Vehicles, equipped with precise odometers, may improve the rodmen's efficiency. In the absence of a pacer, however, the rodmen must still set their own turning points.

3.2.3 Safety

Because lines of leveling typically follow highway and railroad right-of-ways, personnel must be alert at all times to the hazards of working close to traffic.

When operating along a road, each member of the unit must wear a highly visible, orange vest. Work should be conducted as far from the road itself as possible, avoiding setups which unnecessarily expose personnel to the traffic. For added protection, warning signs should be placed within 1.6 km of the unit, in each direction. Any other safety requirements should be identified at the time liaison with the State highway district office is established.

Any vehicles used to transport the rodmen and the observer during a section of leveling should be driven on the right shoulder of the road, oriented with the flow of traffic. Vehicles should never be backed against traffic. A warning light should be flashing and the vehicle's hazard lights should be turned on. If the road shoulder is too narrow or if the highway is too heavily traveled to accommodate the leveling truck or other vehicles, the section should be walked. If there is insufficient room for the unit to pass safely on foot, the procedures cited under "Controlling traffic" (next subheading) should be used.

When working along a railroad, do not wear orange vests or use flashing lights, since these may cause a train to stop unnecessarily. Each person should be alert to the presence of approaching trains and be ready to warn other unit members. Each person should also have an escape route planned, especially when working on terrain where a train is not visible until the last instant.

All personnel in the unit should be trained in basic first aid and cardiopulmonary resuscitation. At each work area they must be aware of the location and telephone number of the nearest emergency medical-treatment center, so prompt assistance can be obtained if an accident occurs. Any personal injury must be reported to the project director immediately. More detailed guidelines for safety are given in the National Geodetic Survey Operations Manual (Greenawalt and Floyd 1980).

Controlling traffic. When a traffic lane must be closed, as when leveling across a bridge with no sidewalks or leveling through a tunnel, permission to control the traffic must be obtained from local authorities. A suitable configuration for controlling traffic is shown in figure 3-13. Six personnel are necessary: two flagmen and a four-person leveling unit. The flagmen must be equipped with portable radios to coordinate their efforts. The recorder should drive the truck as for routine leveling. When added protection is necessary, two extra vehicles, each driven by an additional person, should precede and follow the unit.

After setting up warning signs, the unit proceeds down the right lane, as shown in figure 3-13. The forward flagman moves about one setup ahead of the unit, stopping traffic in the left lane (example 1). The rear flagman stops traffic behind the unit. After notification by radio that traffic in the left lane is stopped, the rear flagman begins directing traffic around the unit. When the setup is complete, the unit waits until the rear flagman has stopped traffic in the right lane (example 2). After notification of this, the forward flagman allows traffic to proceed in the left lane, and advances one setup ahead of the next setup. The unit advances at the same time to establish the next setup (example 3).

3.2.4 Reports

The project director should visit each unit at least once each month to inspect technical and safety procedures, and to provide personnel with updated information on project accomplishments. At all other times the unit chief should maintain regular communication with the project director. This can be accomplished in person when the unit is operating near the project office and by telephone when the unit is at a distant location. In addition, the reports described in the following paragraphs are required of the level unit.

Weekly leveling report. The weekly status report is submitted in person or mailed to the field office at the end of work each week. Any certified or registered-mail receipts for leveling data sent to the project office during the week should accompany the report. The report

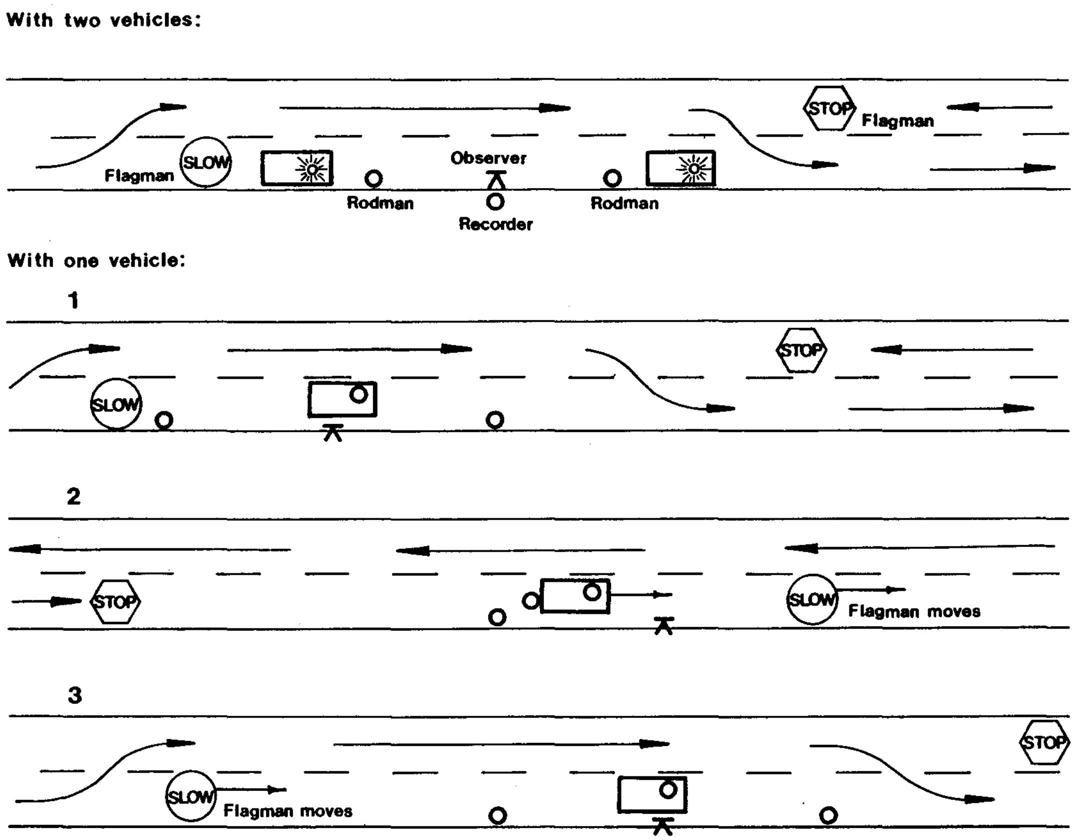

Figure 3-13.—Controlling traffic.

includes notes on progress, weather, activities, equipment condition and maintenance, suggestions, and requests for supplies. Figure 3-14 shows a properly executed NOAA Form 76-159.

When completing the form, enter distance in kilometers to one decimal place. Of primary interest is progress—the amount of the assigned line which has been completed. Report progress in two categories: double run and single run. Double-run progress is the total one-way distance of all sections that have been leveled in both the forward and backward directions. Single-run progress is the total one-way distance of all sections that have been leveled in only one direction.

When a section is releveled in the same direction as an earlier leveling, total the distance as rerun. For example, if a double-run section fails to close, count the distance for the third leveling of the section as the first rerun. If still more levelings are required to obtain closure, count the distances in the column for additional reruns.

The total distance represents the amount of work performed by the unit each day. It is the total of all the leveling distances, including both directions of double-run sections and all reruns. Check the statistic totals each day by this formula:

$$(2 \times \text{double-run progress}) + \text{single-run progress} + \text{rerun} = \text{total.}$$

The formula may be used to calculate double-run progress each day when an entire line is double run. Total the distance for the day, subtract all reruns, and divide what remains by two.

Report the number of setups of leveling completed each day. Do not count setups from the collimation check and setups from incomplete sections. Also report the number of section runnings completed.

The time at which the first setup began and the time when the last setup ended should correspond with the beginning and ending times recorded in the leveling data. Any delays of 15 minutes or more during the workday should be noted and explained in the remarks.

Note the number of collimation and compensation checks made. Also note the number of persons working with the unit for the day. Columns for weather conditions and remarks should be completed. Note such items as: cloud cover, wind speed, temperature, unusual climatic conditions, change of lines, change of observer, change of unit personnel, plumbing of rods, other adjustment and maintenance activities, equipment breakdowns, accidents, training activities, and any incidents which have a bearing on the quality of the leveling or condition of personnel and equipment.

Monthly leveling report. The monthly leveling report is submitted, either by mail or in person, to the project office at the end of work on the last day of each month. The report contains the same information as that given in the weekly report. Unlike the weekly report, however, the monthly report is forwarded to the headquarters of the National Geodetic Survey, where it serves as a source of statistical information and suggestions concerning the progress of leveling in the national network.

Figure 3-15 shows a properly executed monthly releveling report. At the end of each work day, the unit chief should fill out the line on the report corresponding to that day's activities. Then it is a simple matter to complete the totals at the end of the month.

Vehicle report. Careful records should be kept of all expenses incurred for the repair and maintenance of the leveling truck and other vehicles. A report summarizing these expenses should be submitted, together with all pertinent receipts, to the project office at the end of each month.

Miscellaneous reports. The unit chief should be familiar with current organizational procedures for submitting per diem vouchers, accident reports, personnel reports, and expense receipts.

3.3 Leveling Instruments

The instrument used for geodetic leveling must be precisely manufactured to ensure both sensitivity and ruggedness. This subchapter describes the mechanisms that are important in an optical instrument that is suitable for geodetic leveling. Specifications are included for proper maintenance and use of the instrument.

3.3.1 Classification

Optical leveling instruments are classified in two ways. The first is by the mechanism with which they provide a horizontal line of sight. Spirit-level instruments depend on a sensitive tubular level attached to the telescope. A fine-pitch tilting screw is turned manually to center the bubble in the vial of the spirit level. Compensator instruments depend on a pendulous reflecting component, the compensator, within the optical system of the telescope. Responding to the attraction of gravity automatically renders the line of sight horizontal.

Since the line of sight provided by either type of instrument cannot be exactly horizontal, the resulting collimation error must be regularly measured and ad-

N FORM 76-159
(3-82)

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

WEEKLY REPORT - LEVELING UNIT

OBSERVER: C. Smith
RECORDER: M. Cyr, M. Orenth
MONTH/YEAR: July 19 82
RODMEN, PACER: J. Sparks, M. Cyr, M. Orenth
PARTY: G-36
LOCATION(S): Cle Elum, Wa.; Wenatchee, Wa.
PROJECT: 83624100
LINE(S): 279
SURVEY ORDER: First
CLASS: II

DAY	DISTANCE (km) DBL RUN PROG	SGL RUN PROG	1ST RE-RUN	ADL RE-RUN	TOT ALL RUN	NUMBER SET UPS	SEC-TION	TIME 1ST SET UP	LAST SET UP	COLL AND COMP CKS	STAFF DAY	WEATHER CONDITIONS	REMARKS
Sun 4													
Mon 5													Holiday (4th of July)
Tue 6		8.4			8.4	125	9	0821	1620		4	Clear, Mild	Narrow winding mountain Road; Heavy traffic
Wed 7		8.2			8.2	107	6	0839	1520	✓✓	4	Cloudy - AM Pt.cloudy-Rain-PM	Good effort by Rodmen, Heavy traffic; Rain last section
Thu 8	1.95	2.9			6.8	82	5	0845	1615		4	Clear, Warm	Extremely Heavy traffic, Line tie spur; Move on line
Fri 9		6.4			6.4	91	7	0840	1528		4	Clear, Warm	Extremely Heavy traffic, 4 lane Hwy; Heat waves
Sat 10													
Total	1.95	25.9			29.8	405	27				16	km/Staff-Days = 1.86	

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Figure 3-14.—Leveling unit weekly report.

NOAA FORM 76-104
(3-82)

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

MONTHLY REPORT – LEVELING UNIT

OBSERVER C.SMITH	RECORDER M.CYR, M. DRENTH	MONTH/YEAR JULY 1982
RODMEN, PACER J.SPARKS, M.CYR, M. DRENTH	PARTY G-36	
LOCATION(S) CLE ELUM, WA.; WENATCHEE, WA.	PROJECT 83624100	
LINE(S) 279	SURVEY ORDER FIRST	CLASS II

DAY	DISTANCE (km) DBL RUN PROG	SGL RUN PROG	1ST RE-RUN	ADL RE-RUN	TOT ALL RUN	NUMBER SET UPS	SEC-TION	TIME 1ST SET UP	LAST SET UP	COLL AND COMP CKS	STAFF DAY	DOWN TIME	WEATHER CONDITIONS	REMARKS
1		4.6			4.6	146	7	0950	1610		4		CLOUDY, CALM	STEEP GRADE, WINDING MOUNTAIN PASS RD.
2		4.4			4.4	86	5	0814	1328		4		CLEAR-AM CLOUDY-PM CALM	STEEP GRADE, EXTREMELY HEAVY HOLIDAY TRAFFIC
3			W	E	E	K	–	E	N	D				T. HAILE OBS. BOTH 1ST & 2ND
4			W	E	E	K	–	E	N	D				
5			H	O	L	I	D	A	Y		~~4~~	~~8~~		4TH OF JULY HOLIDAY
6		8.4			8.4	125	9	0821	1620		4		CLEAR, MILD	NARROW WINDING MOUNTAIN ROAD, HEAVY TRAFFIC
7		8.2			8.2	107	6	0839	1520	✓✓	4	1	CLOUDY, AM PT. CLOUDY- PM RAIN	GOOD EFFORT BY RODMEN; HEAVY TRAFFIC; RAIN LAST SECTIONS
8	1.95	2.9			6.8	82	5	0845	1615		4	1	CLEAR, WARM	EXTREMELY HEAVY TRAFFIC LINE TIE SPUR; MOVE ON LINE *
9		6.4			6.4	91	7	0840	1528		4		CLEAR, WARM	EXTREMELY HEAVY TRAFFIC 4 LANE HWY.; HEAT WAVES
10			W	E	E	K	–	E	N	D				
11			W	E	E	K	–	E	N	D				
12			M	O	V	E	–	D	A	Y	4	8	CLEAR, WARM	SWITCH EQUIPMENT TO NEW TRUCK-AM; MOVE FROM *
13		7.1			7.1	74	5	0828	1409	✓✓	4	2	CLOUDY, WARM, RAIN	HEAVY TRAFFIC, 4 LANE HWY. DOWN TIME DUE TO RAIN
14		6.6			6.6	82	5	0807	1543		4	2	CLOUDY-RAIN-AM PT. CLOUDY, MILD-PM	DOWN TIME TO RAIN; VISITED IN FIELD BY CORPS OF ENGINEERS
15	3.25	2.5			9.0	114	4	0814	1627		4		CLOUDY, MILD	EXTREMELY HEAVY TRAFFIC; DBL RUN SPUR INTO WENATCHEE, WA.
16	4.1				8.2	134	3	0815	1648		4		CLEAR, WARM	DBL. RUN SPUR THRU WENATCHEE; TIME SPENT TO GET BACK TO TRUCK *
17			W	E	E	K	–	E	N	D				
18			W	E	E	K	–	E	N	D				
19	0.75	8.1			9.6	110	6	0752	1637	✓✓	4		CLEAR, WARM	20 MIN. MOVE ON LINE; HOT ALL DAY WITH GUSTY WINDS 75°-95°F
20		9.2			9.2	112	8	0822	1544		4		CLEAR, WARM	HIT BM ON ROCKY REACH DAM; HOT 71°-93°F; GOOD EFFORT BY CREW
21		8.7			8.7	90	7	0809	1502		4	1	CLEAR, WARM WINDY	DOWN TIME TO STRONG GUSTY WINDS - PM; HEAVY TRAFFIC
22		8.9			8.9	103	7	0810	1552		4		CLEAR, WARM	HEAVY TRAFFIC; WORKING THRU ENTIAT, WA.; BAD HEAT WAVES
23		8.4			8.4	100	6	0832	1609		4		CLEAR, WARM	HEAVY TRAFFIC ALL MORNING; BAD HEAT REFRACTION - PM.
24			W	E	E	K	–	E	N	D				
25			W	E	E	K	–	E	N	D				
26		7.2			7.2	87	5	0853	1642		4	2	CLEAR, HOT	75°-101°F; 1 HR. 45 MIN. LOST TO GOING BACK FOR TRUCK; WORK *
27		7.6			7.6	96	5	0832	1635	✓✓	4	1	CLEAR, HOT	72°-102°F; 1 HR. LOST TO MOVE ON LINE; WORK ALONG RR TRACKS
28		5.1			5.1	124	6	0840	1558		4		CLEAR, HOT	77°-102°F; WORKED ON STEEP NARROW GRADE; NO USE OF TRUCK
29		7.5			7.5	124	7	0902	1612		4		CLEAR, HOT	77°-103°F; WORKED ON STEEP GRADE; GOOD EFFORT BY CREW
30			M	O	V	E	–	D	A	Y	4	8		MOVE FROM WENATCHEE, WA. TO PATEROS, WA.
31			W	E	E	K	–	E	N	D	4	34		
Total	10.05	121.8			141.9	1587	113				88	km/Staff-Days = 1.69		* SEE REVERSE

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Figure 3-15.—Leveling unit monthly report.

justed within tolerance. Once the instrument is properly adjusted and the collimation error is known, the precision of the observations made depends in part on the consistency with which the spirit level or the compensator maintains the given line of sight. In a spirit-level instrument, this consistency is primarily determined by the degree of precision with which the bubble may be centered in the level vial. A coincidence prism or similar device enhances the precision of centering. However, since the bubble tends to "run" toward a point source of heat, the instrument must be shaded to ensure centering precision while leveling.

Compensator instruments maintain the consistency of the line of sight in various ways, some of which are described in this subchapter. In general, the automatic compensator mechanisms provide a setting precision superior to that attainable by centering a bubble. Thus, compensator instruments permit both greater productivity and improved leveling precision.

The second way in which leveling instruments are classified is according to whether or not they are equipped with micrometers. Without a micrometer, observation values may be estimated to, at best, 0.5 mm. The micrometer permits observations to be made with better precision. It permits values to be read directly to the equivalent of 0.05 mm or better.

For productive geodetic leveling, optical instruments should have the following features:

1. A telescope equipped with a large objective lens (50 to 70 mm diameter of the free aperture) to ensure brightness and good image definition.
2. A magnification power of 30 to 50 diameters.
3. A reticle etched between glass plates to ensure a consistent, dust-free image. It should include one vertical line, three horizontal lines (a middle line and two stadia lines) and, for use with scales with line graduations, a wedge. (See sec. 3.3.5, "Making an observation.")
4. Resistance to the effects of temperature change and vibration.
5. Units of the micrometer compatible with the graduations on the leveling rods. (See sec. 3.3.4, "Optical micrometer.")

Only instruments that are equipped with micrometers and that meet the appropriate standard of setting precision with respect to collimation error (table 3-1) can provide results considered to be of first-order accuracy. Although instruments with micrometers are preferred, instruments without micrometers can be used in combination with the three-wire leveling procedure to provide results considered to be of second-order accuracy.

Several different kinds of leveling instruments have been satisfactorily used for geodetic leveling in the United States. The Fischer level (fig. 3-16), designed by the Coast and Geodetic Survey, was the sole instrument used by that organization from 1900 to the early 1960's. Since then, commercial instruments that have been

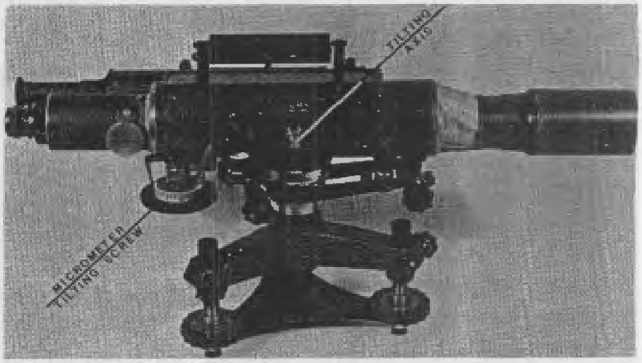


Figure 3-16.—Fischer level.

extensively used by the National Geodetic Survey include: Breithaupt NABON, Jenoptik Ni004, Zeiss Ni2, Zeiss Ni1, MOM NiA31, and Jenoptik NI 002 (figs. 3-17 through 3-21). Some features of these instruments are given in table 3-2.

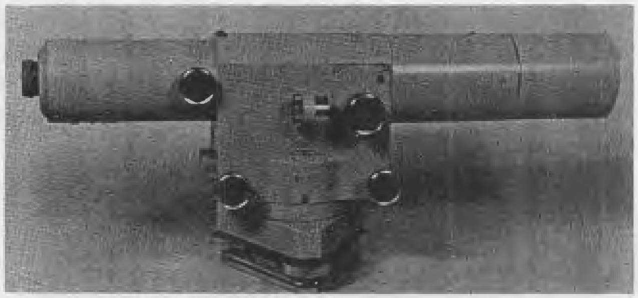


Figure 3-17.—Jenoptik Ni004.

3.3.2 Telescopic System

In a simple telescope, such as that used in older spirit-level instruments, the straight line between the optical center of the objective lens and the center of the reticle is called the line of collimation. A ray of light, passing through the optical center of a simple lens, continues to travel in its original direction as it proceeds beyond the lens. Thus, when the telescope is horizontal, the image of the center of the reticle, projected along the line of collimation to the center of the objective, defines a horizontal line of sight.

The ocular, or eyepiece, is a magnifying lens which permits the observer's eye to focus on the reticle lines and to see simultaneously on the reticle a magnified image of the scale of the leveling rod. The ocular and the observer's eye function jointly as a single lens. The ocular must be set at a fixed distance from the reticle, to enable the observer to see, without eyestrain or parallax, a sharper view of the reticle lines and the scale image. This setting must be adjusted to the eye of the individual observer. (See sec. 3.3.5, "Parallax adjustments.")

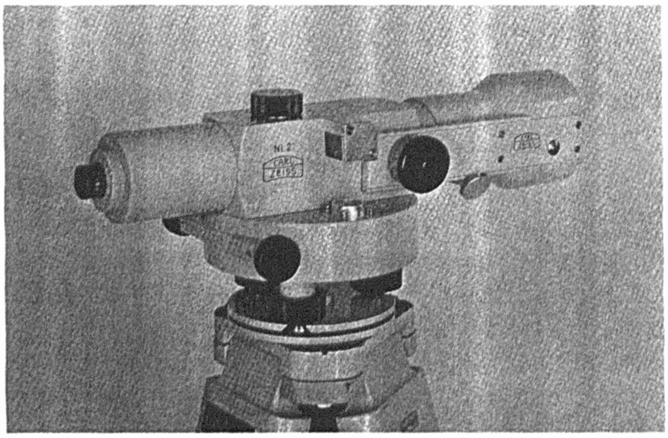


Figure 3-18.—Zeiss Ni2, with micrometer attachment.

To focus an instrument, the distance from the objective to the reticle, D_i, in figure 3-22, must be adjusted relative to the sighting distance from the objective to the rod scale, D_o. In older, external-focusing telescopes, the reticle and ocular are moved together in a combined mounting; the ocular is adjusted separately to focus distinctly on the reticle lines. In more recent versions, the reticle is in a fixed mount and only the objective lens is moved, usually by means of an external sleeve, to set the image distance as required.

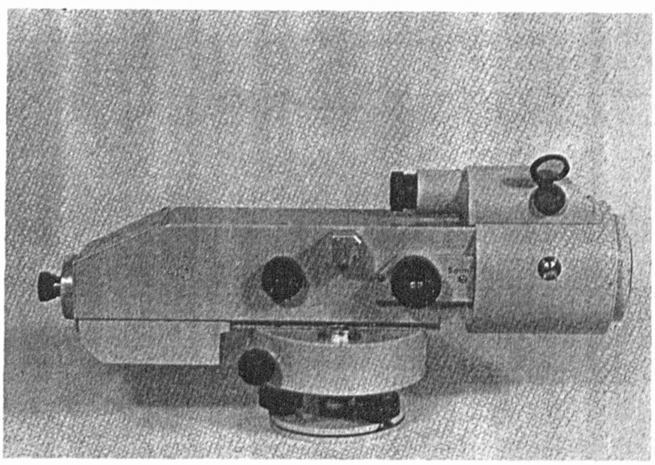


Figure 3-19.—Zeiss Ni1.

In internal-focusing telescopes, both the objective and the reticle are mounted in fixed positions (fig. 3-23). This system requires the introduction of a third, focusing lens between the objective and the reticle. The spacing between the objective and the focusing lens determines the focal length of the system. Focusing is accomplished by moving the focusing lens back and forth until the focal length is that required to make a sharp image at the reticle for the particular sighting distance. An inverted image of the rod scale is formed.

The internal-focusing system is much less likely to admit dust and moisture and is widely used in leveling instruments of high precision. In some instruments, the inverted image is made erect by a complex ocular system.

The presence of a focusing lens in the telescope requires a more general definition for the line of collimation. Because of imperfections in manufacturing, the center of the focusing lens cannot be set exactly on the line between the center of the reticle and the center of the objective. Neither can the sliding motion of the focusing lens be adjusted to coincide exactly with the line of collimation. Thus, the line of collimation deflects slightly as it passes through the focusing lens. The line of collimation can be redefined as the path followed by a ray passing through the center of the objective and deflected by intervening optical elements, finally intersecting the center of the reticle.

The line of sight provided by the instrument is the forward extension of the line of collimation, through the center of the objective to the sighted object. In the case of a deflected line of collimation, the line of sight is the extension of the segment immediately behind the objective.

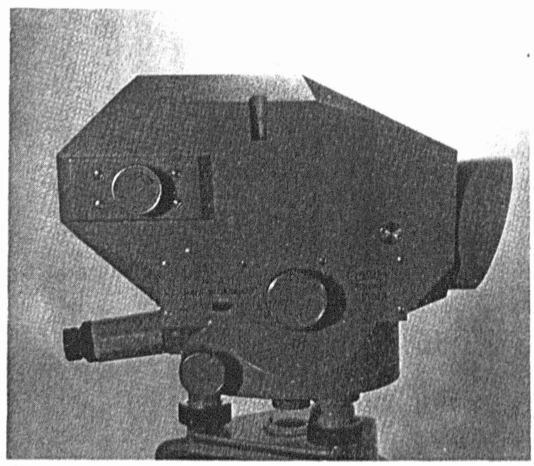


Figure 3-20.—MOM NiA31.

3.3.3 Compensator System

The presence of a compensator in an instrument eliminates the time-consuming operation of carefully centering the bubble in a level vial immediately before each observation. Instead, the instrument need only be roughly leveled with the tribrach screws, referring to a circular level rigidly attached to the instrument body.

Compensators can be grouped in two general classes: nonreversible and reversible. This manual explains two examples of nonreversible compensators: the Ni2 and Ni1 instruments. The NI 002 instrument is also described to provide an example of a reversible compensator.

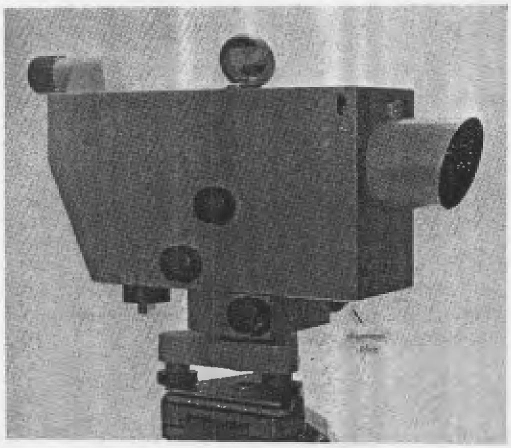


Figure 3-21.—Jenoptik NI 002.

General description. The compensator consists of one or more reflecting elements, usually prisms, set in the line of collimation of the telescope. One of the reflectors is suspended within the instrument by fine flexible links (usually Invar bands). The orientation of this movable element is changed within the telescope system by the action of gravity, whenever the alignment of the instrument body is changed with respect to the local gravity vector. The lengths and geometrical arrangement of the suspension links, the position of the center of gravity of the suspension links, and the position of the center of gravity of the movable element are carefully designed to tilt the reflecting surface. The tilt assures that the path followed by a ray of light, entering at the center of the objective, is deflected to intersect the center of the reticle.

Since the housing of the instrument is made only approximately at each horizontal setup (within a range depending on the sensitivity of the circular level), the movable element within the compensator must assume an attitude such that the segment of the line of collimation immediately behind the objective is consistently horizontal. The angle by which it deviates from the horizontal is the collimation error. The consistency with which the movable element repeats a given collimation error when the instrument is releveled, without otherwise being disturbed (e.g., in a laboratory), is termed the setting precision. It can be limited to less than ±0″.25 in a precise instrument. It compares with the centering error of about ±0″.5 for the tubular level of a precise spirit-level instrument.

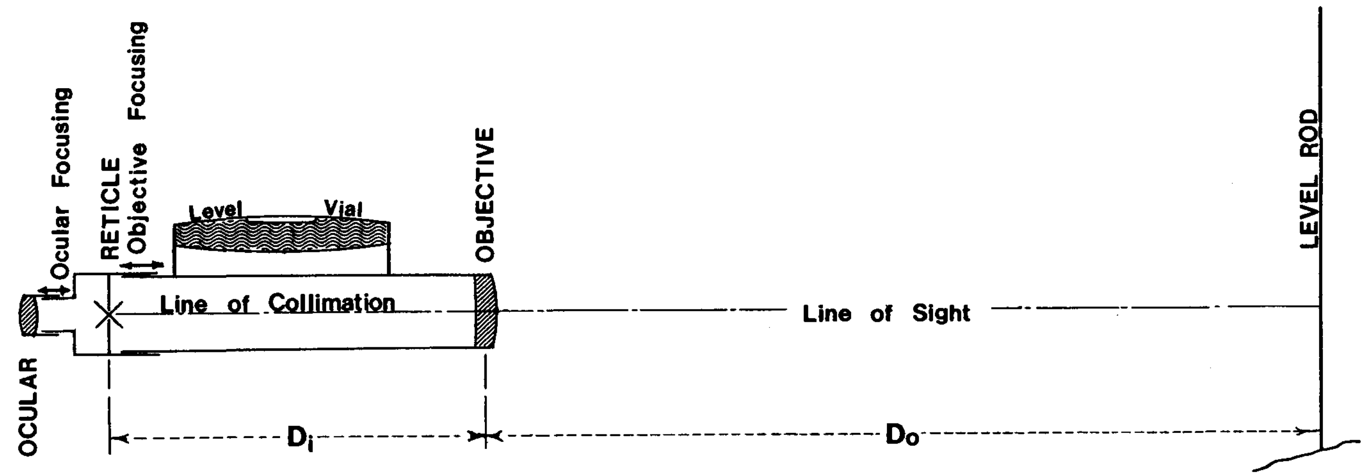
Zeiss Ni2. The first commercial compensator instrument was the model Ni2, produced in 1950 by Carl Zeiss in the Federal Republic of Germany. It has a relatively simple compensator system which contains three elements. Two are prisms rigidly attached to the telescope body. The third is an inverted pendulum, suspended by two pairs of fine Invar tapes on which the element can swing into a stable position with respect to the gravity vector. A reflecting prism is attached at the lowest part of the element. The telescope system, into which the compensator is inserted, is of the internal-focusing type. A micrometer attachment is available for the Ni2.

The basic configuration of the compensator system, with the telescope axis horizontal, is shown in figure 3-24. The fixed reflector prism and the movable prism serve, in effect, as plane mirrors. By simple reflection, the prisms cause large deflections in the line of collimation along its path from the objective to the reticle. Because of the "floating" action of the movable prism, the deflection varies in magnitude as the vertical axis of the instrument changes slightly from setup to setup. The fixed roof prism picks up the reflected line of collimation from the movable prism and redirects it to the reticle, where the image of the rod scale is formed. The roof prism not only deflects the line of collimation, but, by double lateral reflection, also reverses the image of the rod, left to right and vice versa.

The focusing system in front of the compensator generates an inverted image of the rod scale, but the prism

Table 3-2.—Features of some geodetic leveling instruments

Instrument	Micrometer	Image	Stadia factor	Stadia constant	Setting precision
Compensator instruments					
Jenoptik NI 002					
(reversible)	Integral	Erect	100	+0.4 m	±0″.05
MOM NiA31	Integral	Erect	100	+0.37 m	±0″.1
Zeiss Ni1	Integral	Erect	100	0.0	±0″.1
Zeiss Ni2	Optional	Erect	100/333	0.0	±0″.2
Wild NA2	Optional	Erect	100/333	0.0	±0″.3
Spirit-level instruments					
Wild N3	Integral	Inverted	100/333	−0.2 m	±0″.2
Jenoptik Ni004	Integral	Inverted	100	0.0	±0″.23
Breithaupt NABON	Optional	Inverted	100	0.0	±0″.2
USC&GS Fischer	None	Inverted	300/333	+0.6 m	±0″.5



$$\frac{1}{D_i} + \frac{1}{D_o} = \frac{1}{f}$$

Figure 3-22.—External-focusing telescope.

system erects the image from top to bottom and reverses it left to right. Thus, the observer sees an erect, direct image.

With the telescope axis horizontal, the line of collimation is horizontal before reflection by the fixed reflector prism. The line which connects pivot points *A* and *B* in the body of the instrument and the line which connects pivot points *C* and *D* by which the compensator is suspended are also horizontal. Points *C* and *D* are placed symmetrically with respect to *A* and *B*, by virtue of their suspension on either side of the movable prism. This requires four suspension ribbons in all, of nearly equal length. The line of collimation is deflected 45° by reflection from the first fixed prism, by 90° in the opposite direction from the horizontal surface of the movable prism, and again by 45° to coincide with the telescope axis. Thus, the image directed by the objective along the line of collimation is viewed at the ocular as if no deflection had occurred.

When the instrument is roughly leveled (fig. 3-25), the telescope axis is tilted within 4′ from a horizontal position and the line of the pivots, *AB*, is tilted by the same amount, α. Because of the flexible suspension ribbons, the supports *C* and *D* shift to a new position. The lengths, *AC, BD,* and *CD,* remain unchanged and the suspension ribbons bend slightly at *A, B, C,* and *D,* as if the supports were frictionless pivots. (Actually, the ribbons assume complex curved alignments, but if the axis is tilted only slightly, this causes no significant error.)

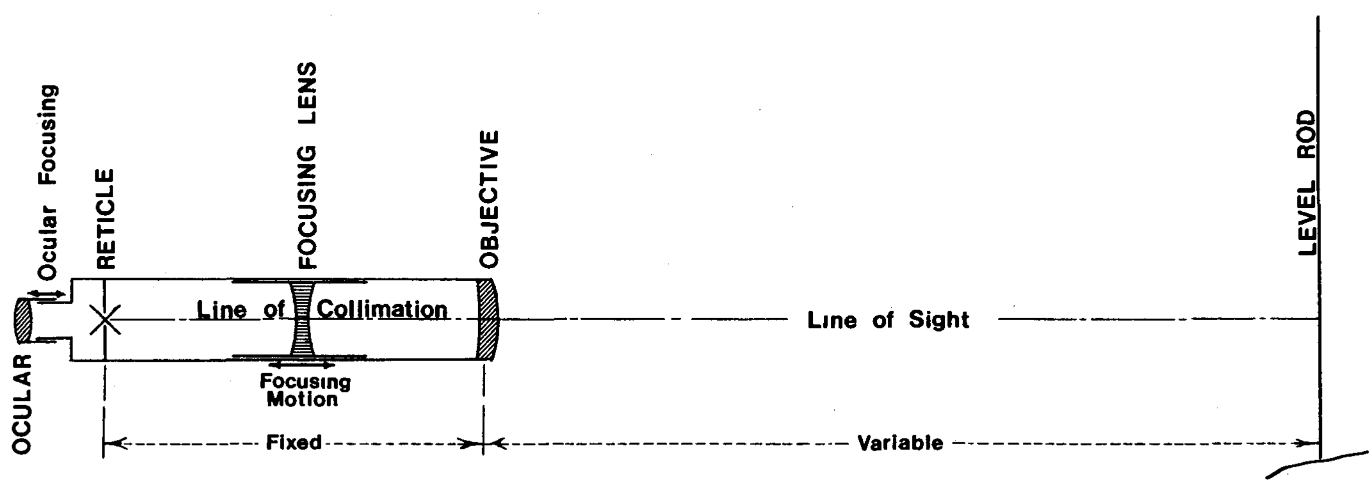


Figure 3-23.—Internal-focusing telescope.

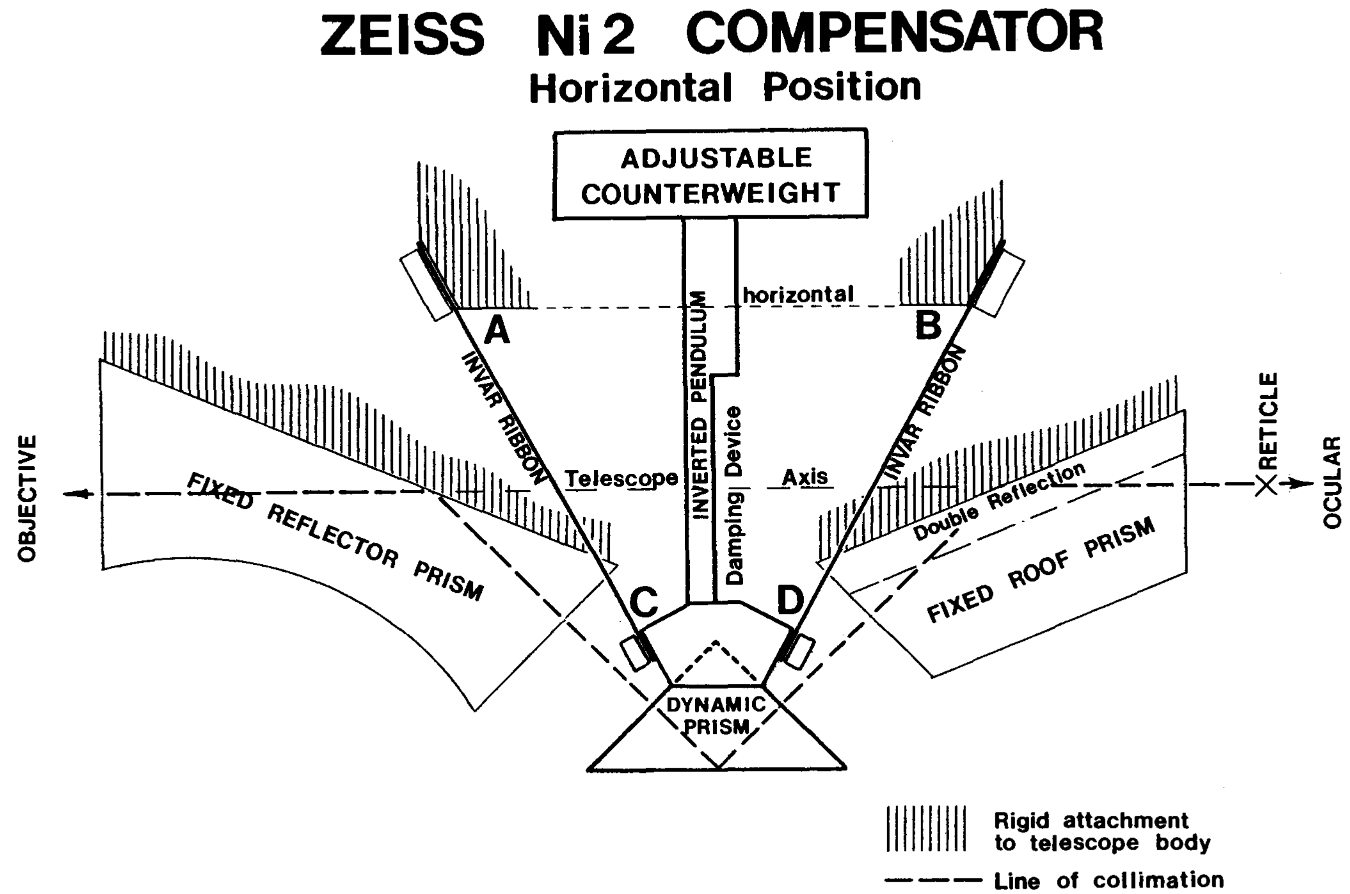


Figure 3-24.—Ni2 compensator system, telescope axis horizontal.

Because of the shift of the support, *CD*, the lower reflecting surface of the movable prism is tilted in the same direction as the line of pivots, *AB*, but through a larger angle, β, by the relationship:

$$\beta = n \times \alpha.$$

The factor, *n*, is a function of the dimensions of *AB* and *CD*, the perpendicular distance from *AB* to *CD* when *AB* is untilted, and the height of the center of gravity of the suspended system above the axis *CD*. The magnitude of the factor in a particular instrument depends on the location of the compensator in the telescope.

In essence, the compensator must deflect a ray of light, entering a telescope whose axis is tilted by a small angle from horizontal, through the angle necessary to redirect the ray to the center of the reticle, as shown schematically in figure 3-26. In the Ni2 compensator the ray is deflected three times, but the net effect is that of a single deflection of the correct magnitude.

The position assumed by the movable element for any tilt within its functioning range depends on the dimensions of the suspension system and the position of the center of gravity. Figure 3-27 shows the condition that must be met for stability of the movable element during a setup. *A, B, C,* and *D* are the pivot points shown with the same notation as in figure 3-24. *G* is the center of gravity at a height, *h*, above the line of pivots, *CD*. Stability (no oscillation) occurs automatically when the gravity vector, acting downward from *G*, passes through the point of intersection, *I*, of the extensions of the suspension links *AC* and *BD*.

When the axis is tilted at a small angle, motion occurs at all four pivots. Since they are essentially the equivalent of frictionless pins, the system swings past the stable position, swings back, and then continues to oscillate about the stable position until the motion is damped. To damp it quickly, a loose-fitting, air piston-and-cylinder damper is attached between the dynamic element and the instrument frame. Without mechanical contact between the piston and the cylinder, the device utilizes the resistance of the air pressure generated by the compression stroke to damp the oscillation in less than a second. Such a damper is vital to the productive use of a compensator instrument.

When stability is attained, the line of collimation is horizontal. Any small residual tilt resulting from imperfections in the instrument is adjusted by rotating a wedge-shaped optical element at the front of the objective. (See sec. 4.3.1, "Rotary-wedge attachment.")

This discussion of the basic configuration of the compensator system has been developed around the Ni2 instrument because the details of application are relatively straightforward. Application to other compensator instruments would follow the same general theory, but might vary considerably in detail.

Zeiss Ni1. The Ni1 instrument, like the Ni2, is produced by Carl Zeiss in the Federal Republic of Ger-

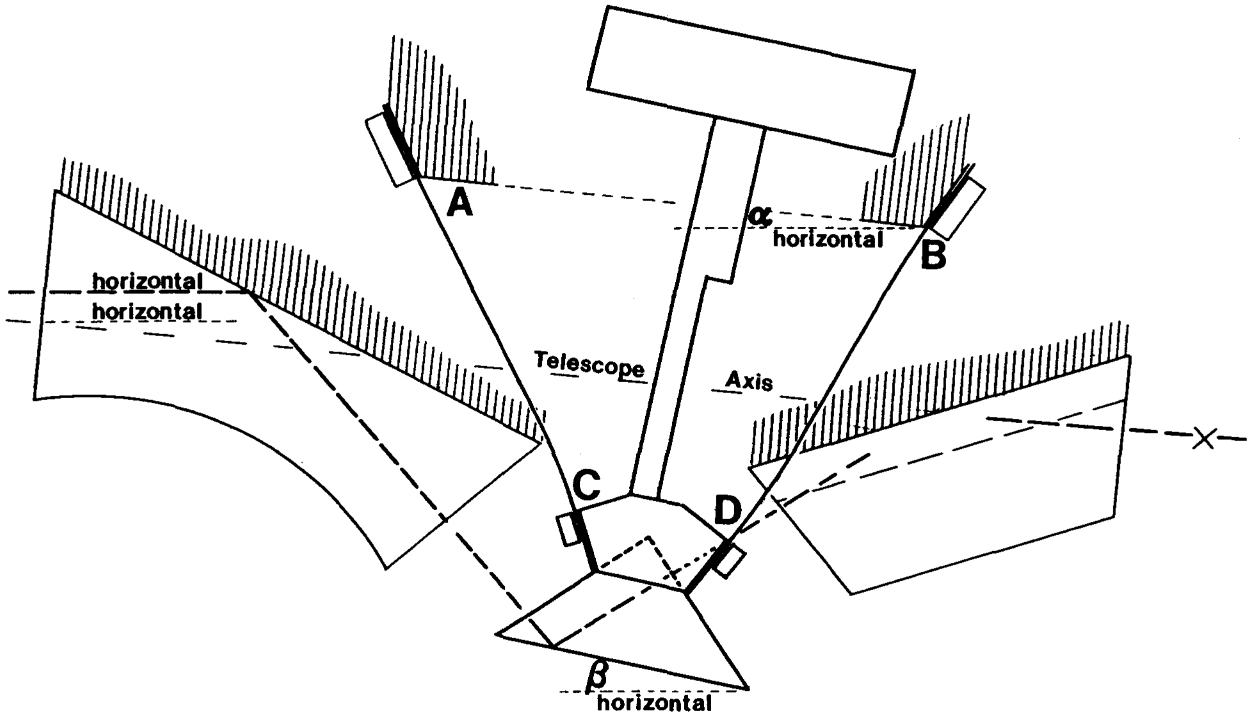


Figure 3-25.—Ni2 compensator system, telescope axis tilted.

many. Designed some years after the Ni2, it is larger, more precise, and intended specifically to produce leveling results of better accuracy. It is equipped with a high-powered telescope and a built-in, plane-parallel plate micrometer. The compensator (fig. 3-28) of the Ni1 is generally similar to that of the Ni2. The movable element is an inverted pendulum suspended on four Invar ribbons. The line of collimation is deflected three times by reflection from prisms. Figure 3-29 shows the schematic arrangement of the compensator system when the telescope axis is tilted.

The compensator varies from that of the Ni2 in the following particulars. First, the Invar suspension ribbons cross at a point above the base of the movable element. This "X-suspension" responds to tilting and to the gravity vector in a manner similar to that of the Ni2 (a "V-suspension"). However, the axis of the movable element tilts in the direction opposite to the tilt of the telescope axis.

Second, the three deflections of the line of collimation are accomplished by only two prisms. The roof prism reflects once; the movable prism reflects twice, internally.

Third, the movable element is not symmetrical about the support system. The roof prism is mounted fairly high above the base pivots and to the rear of the support system, near the ocular. The resulting eccentricity is balanced by the placement of a large damping system at a position on the objective side of the sup-

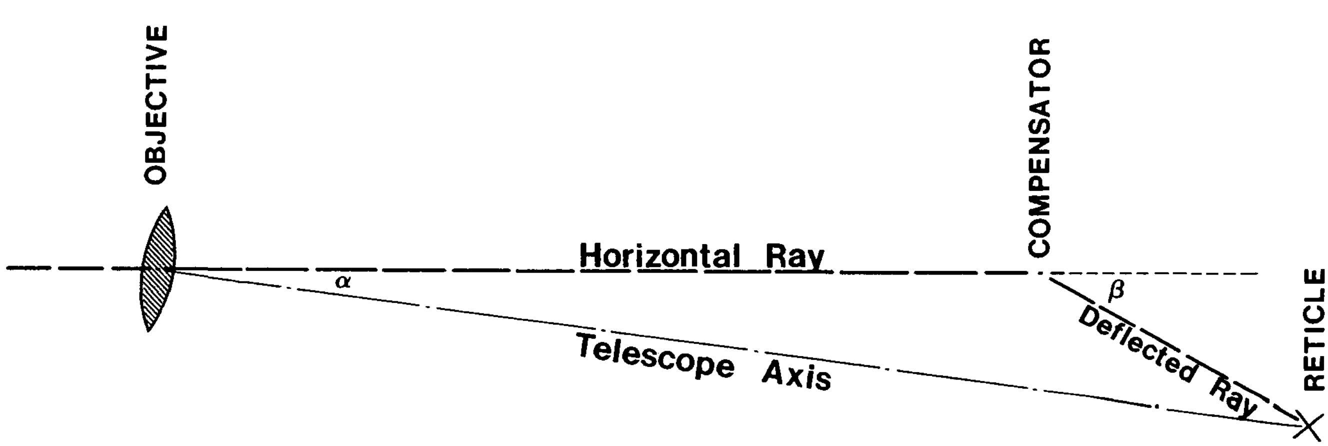


Figure 3-26.—Deflection of the line of collimation by the compensator.

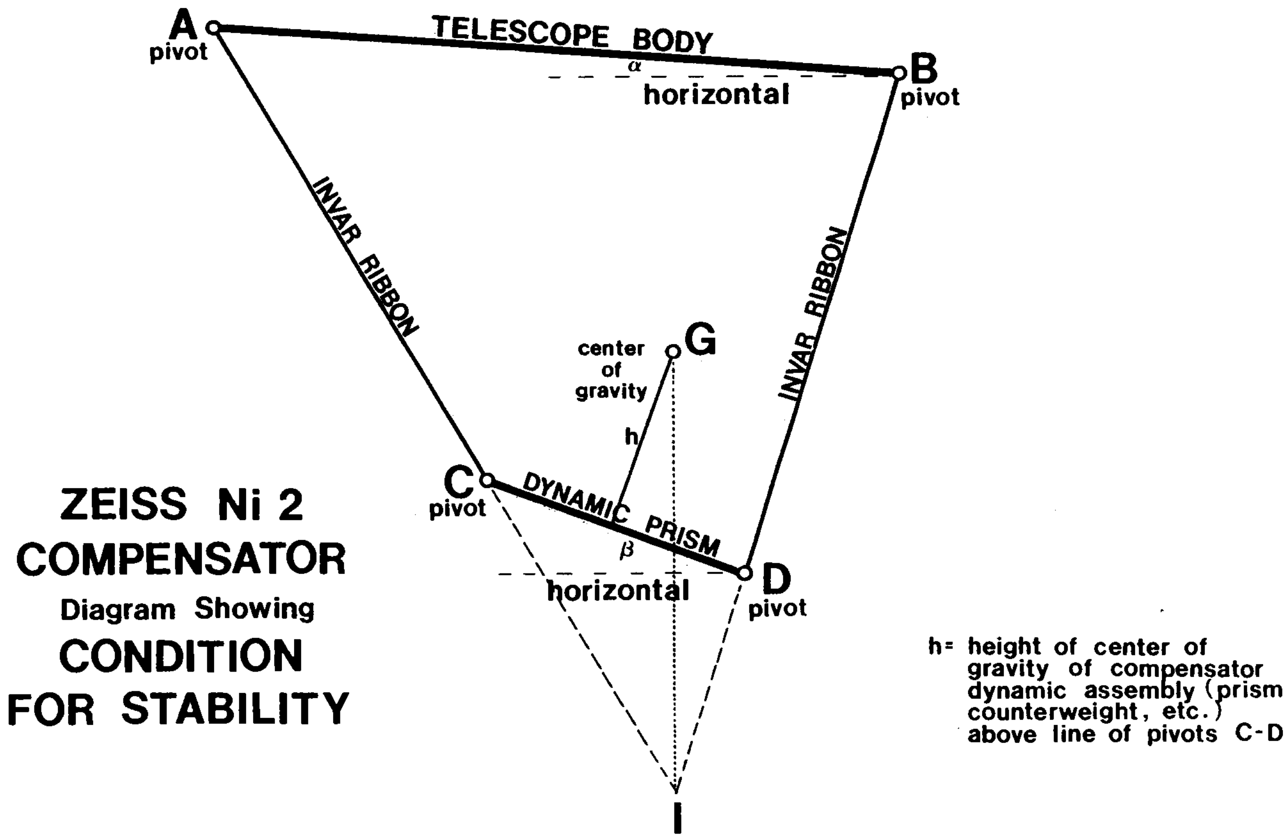


Figure 3-27.—Condition for dynamic stability of the Ni2.

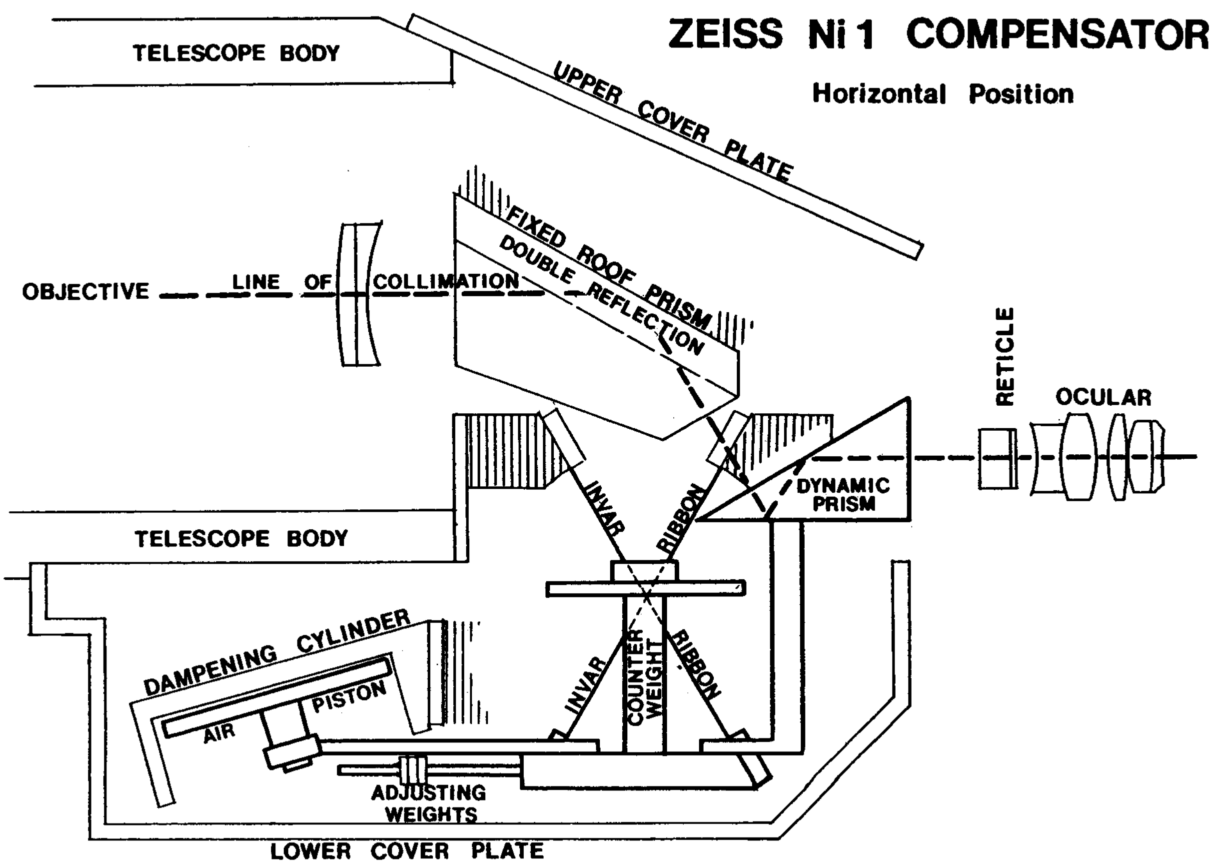


Figure 3-28.—Ni1 compensator system, telescope axis horizontal.

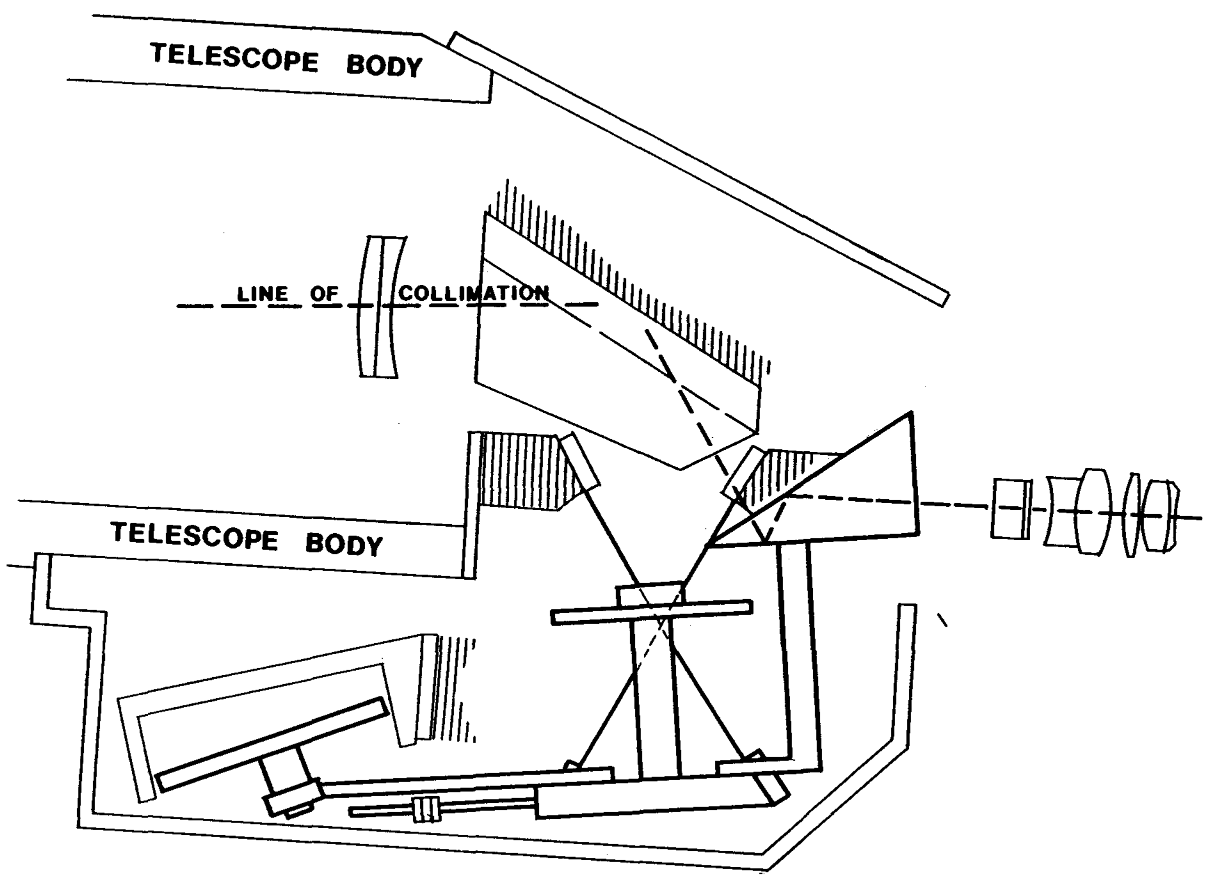


Figure 3-29.—Ni1 compenstor system, telescope axis tilted.

ports. This does not affect the response of the movable element to gravity. The condition for stability still requires that the gravity vector from the center of gravity of the movable element pass through the point of intersection of the supporting ribbons, as projected on a vertical plane. A major attribute of the Ni1 is that the mass of the element is greater than that in the Ni2, which reduces its natural vibration frequency. This makes the system more stable in the presence of external vibrations.

Fourth, the Ni1 has a larger objective aperture and a higher-powered telescope than the Ni2. These features provide a sharper and more distinct view of the rod scale, a factor that improves the precision possible when intercepting the graduations.

Jenoptik NI 002. A major innovation in compensator instruments is the Jenoptik NI 002, manufactured in Jena, German Democratic Republic. Although its optical system is complex, the compensator operates on a simple principle by adapting a vertically suspended, plane mirror.

Figure 3-30 shows that the optical system defining the line of collimation has been reduced to only a few components. The objective lens, 2, transmits rays of light to the optically flat, compensator mirror, 3, which is supported in an essentially vertical position by links, 15, in an "X-suspension" array, similar to that of the Zeiss Ni1. The instrument is focused by moving the mirror backward or forward. The reticle, 4, is engraved at the front of the objective lens. Therefore, the focusing distance is the total distance from the objective to the mirror and back to the objective.

The line of collimation, then, consists of two superimposed segments, both of which are horizontal if the compensator mirror is aligned with the direction of gravity. The important task of providing a horizontal line of sight is accomplished by the simple arrangement of directing rays from the objective to a suspended mirror, then reflecting them back to the reticle on the front surface of the objective. The only adjustment required is to distribute small counterweights so the compensator mirror hangs precisely in line with the direction of gravity.

The compensator mirror is manufactured to be a precise optical plane; both "front" and "back" surfaces are precisely flat and parallel within the fine tolerances of optical technology. The use of the double mirror as a reversible compensator is made possible by the provision of the rotary motion, *h*, with which, by the simple shift of a knob, the mirror and its suspension system can be rotated 180°. After rotation, the line of collimation is reflected from the reverse side of the mirror.

Since the line of collimation is tilted from horizontal by an angle, α_1, in one position, it is tilted by a nearly equal and opposite angle, α_2, after the mirror is rotated 180° (fig. 3-31). If the rod scale is observed twice, once

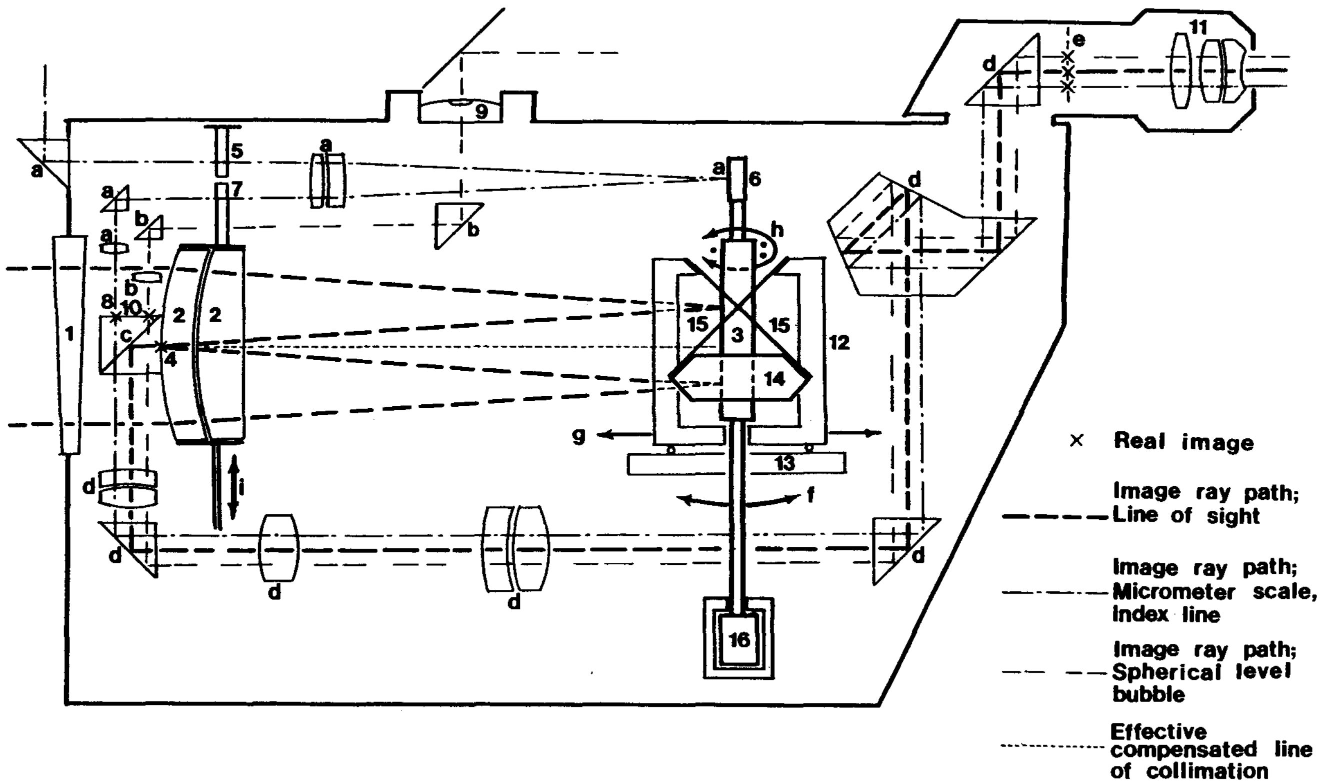


Figure 3-30.—Diagram of the NI 002.

1. Rotatable optical wedge for adjusting mean line of collimation.
2. Telescope objective.
3. Compensator mirror for telescope image.
4. Reticle engraved at front surface of objective lens.
5. Transparent plate with micrometer index line.
6. Compensator mirror for micrometer index-line image.
7. Transparent plate with micrometer scale, attached to objective lens assembly.
8. Real image of combined images of micrometer index line, 5, and micrometer scale, 7, projected on face of prism *c* via optical train *a*.
9. Circular level, for roughly leveling the instrument.
10. Real image of bubble in circular level, projected on face of prism *c* via optical train *b*.
11. Ocular assembly.
12. Compensator support.
13. Compensator support base.
14. Compensator suspension block, attached to mirror, 3.
15. Flexible bands for compensator suspension block.
16. Air piston assembly for damping oscillation of compensator.

a. Optical train, transmitting image of micrometer scale, 7, and index line, 5; image formed on face of beam-splitting prism *c*, at 8.
b. Optical train, transmitting image of circular level bubble, 9; image formed on face of beam-splitting prism *c*, at 10.
c. Beam-splitting prism, collecting images from the objective, at 4, and from optical trains *a* and *b*, at 8 and 10, to be transmitted to image plane *e*, via optical train *d*.
d. Optical train, transmitting combined images from 4, 8, and 10, to image plane *e*.
e. Image plane for optical train *d*, with images of rod and reticle lines from telescope combined with images of micrometer scale and index line (from optical train *a*) and image of circular level (from optical train *b*).
f. Pendulous motion of compensator system.
g. Sliding motion of compensator system, to focus telescope image on reticle at 4.
h. Reversing motion of compensator system; provides 180° rotation about vertical axis, interchanging mirror surfaces "." and ":" on 3; also reverses micrometer mirror 6.
i. Vertical sliding motion of objective assembly, carrying micrometer scale 7, to set line of sight on integral rod graduation, thus serving as a mechanical/optical micrometer.
j. Location for plug to prevent vertical sliding of objective assembly during shipment.

in position one (.) and once in position two (:), the mean of the readings is essentially free from the effect of the collimation errors. Any small residual tilt in the mean line of collimation ("quasi-absolute horizon") may be adjusted within 1.″0 of horizontal by rotation of a wedge-shaped optical element (item 1 in fig. 3-30).

Note that the difference between the readings is proportional to the angular difference, $\alpha_1-\alpha_2$, between the collimation errors. This angle is normally kept within 20.″0 by adjusting the counterweights on the compensator assembly.

During a setup of micrometer leveling with an imbalance, Δs (backsight distance minus foresight distance), the angular difference causes a difference between the elevation difference observed in compensator position one, Δh_L, and that observed in compensator position two, Δh_H:

$$\Delta h_L-\Delta h_H=Q\times\Delta s.$$

This must be considered when checking observations for blunders. Since the angular difference fluctuates somewhat as the air temperature changes, its maximum magnitude should be used to compute the tolerance for the reading check. (See sec. 3.7.2, "Instructions: reading check.")

The precision of the NI 002 depends primarily on the relationship between the objective, the compensator mirror, and the reticle. The remainder of the optical system, which merely serves to transmit the scale image from the compensator to the ocular, is essentially described in the explanatory key to figure 3-30.

3.3.4 Optical Micrometer

Around 1910, Heinrich Wild made possible a significant improvement in the precision of leveling observations when he invented the plane-parallel plate micrometer. This optical device eliminates the need to estimate the intercept of the middle line of the reticle as it appears against the image of a rod scale. The micrometer permits the image of the scale to be shifted vertically until an integral graduation is precisely intercepted by the middle line. The amount of vertical shift may then be measured.

The function of the parallel-plate micrometer depends on Snell's law. A ray of light, incident on a plane air-glass surface at an angle, I, is refracted toward the perpendicular to the plane surface (fig. 3-32). After traversing a finite thickness of glass, t, and on passing through a glass-air surface parallel to the first, it is refracted through an equal and opposite angle. It thus emerges parallel to the original ray, but offset from it by a small distance, d. The distance may be computed from the index of refraction, n, for glass relative to air, as follows:

$$d=t[(n-1)/n]\tan I$$

On a leveling instrument, the plane-parallel glass plate is normally mounted at the front of the objective. The plate may be partially rotated about an axis perpendicular to the vertical axis of the instrument, by means of a knob. With the plate unrotated, the line of sight usually intercepts the rod scale at some point between two graduations, as shown in figure 3-33. Rotating the plate at an angle to the incident ray produces an apparent vertical shift of the field of view.

To measure the interval between the intercept of the line of sight and the first graduation below it, the plate is rotated until a graduation is shifted into coincidence with the middle line of the reticle (fig. 3-34). Since the interval depends on the tangent of the angle of rotation

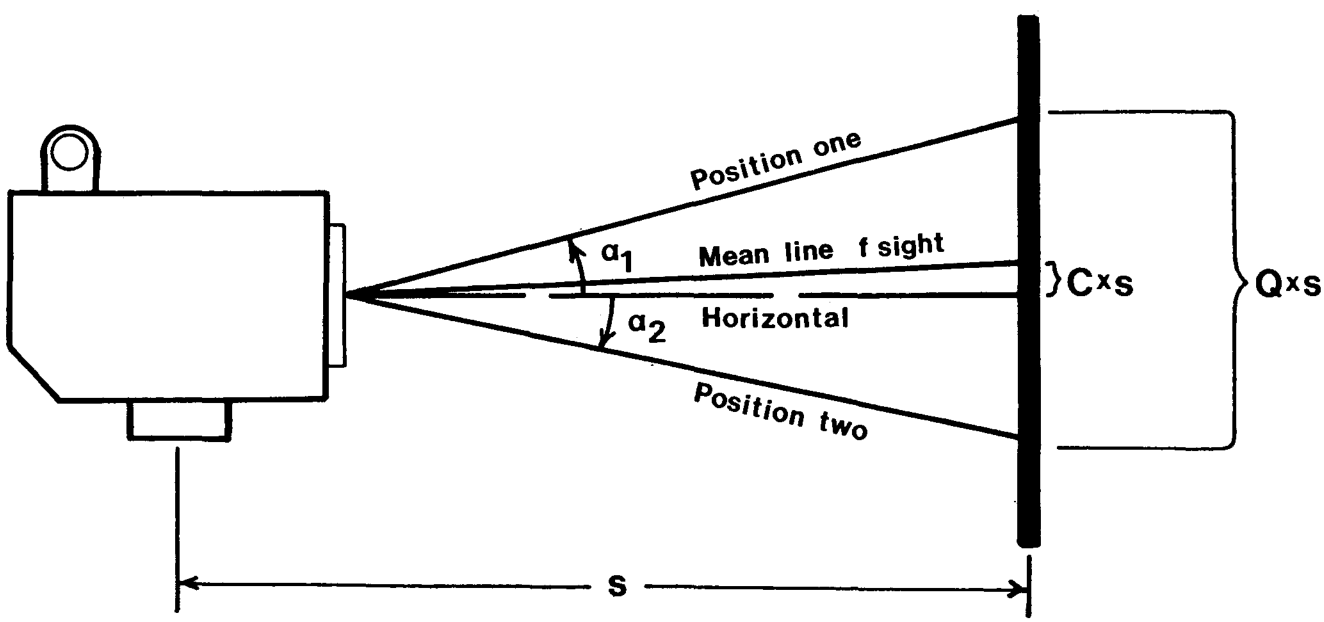


Figure 3-31.—Lines of sight provided by the NI002.

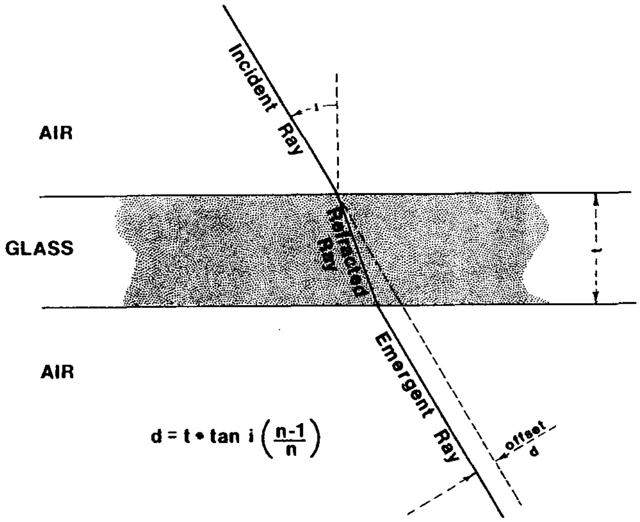


Figure 3-32.—Snell's law applied to a plane-parallel plate micrometer.

of the plate, uniform rotation of the knob generates a tangent function of the angle of rotation of the plate. A drum scale, attached to the knob, permits the values of the function to be read, in units that measure the vertical shift of the line of sight.

The rotating range of the micrometer plate is 10 micrometer units (fig. 3-35). To avoid exceeding the limits imposed by the approximations involved in the equation, the range provides equal displacement on each side of the unrotated position of the plate. Rotating the upper edge of the plate toward the eyepiece causes an apparent lowering of the field of view, or raising of the line of sight. Rotating the upper edge of the plate away from the eyepiece causes an apparent raising of the field of view, or lowering of the line of sight. For convenience, the micrometer units are usually numbered from 0, with the line of sight at its maximum upward displacement, to 10, at its maximum downward displacement. Thus, when the plate is unrotated, the reading is 5 units.

As a result of this system of numbering, a micrometer reading is positive and must be added to the value of the intercepted graduation. Since the readings are referred to its maximum upper displacement (not to the undisplaced line of sight) the readings are 5 micrometer units too large. However, since they are too large by a constant amount, the difference of elevation obtained from the difference of the foresight and the backsight is mathematically correct.

The maximum vertical range of shift of the line of sight is set equal to the interval between two adjacent graduations on the rod scale, i.e., one rod unit. Because of this, only one graduation may be intercepted for any fixed position of the instrument and the rod.

The foregoing details apply to the most commonly used micrometer system. Some instruments (e.g., the Jenoptik NI 002) achieve the same effect by shifting the objective lens itself vertically. The amount of shift is measured directly against an attached linear scale.

Optical micrometers are available for use with rods graduated in any of a variety of units. One micrometer unit is normally equivalent to one-tenth of a rod unit. Readings are made to the nearest one-hundredth of a rod unit (equivalent to 0.05 mm with half-centimeter rods), as indicated by subdivisions, or can be readily estimated. Always use a micrometer compatible with the units of the rod scale; otherwise confusion and blunders will result.

3.3.5 Use and Maintenance

During field use the leveling instrument is subject to continual physical stress, including shock, vibration, wind-blown dust and dirt, moisture, and temperature change. Even the most rugged instrument cannot withstand this assault indefinitely. The observer should take every possible precaution to protect it from these conditions.

Tripod. To ensure a stable platform for the instrument, the tripod on which it is mounted must be strong and rigid. It should be constructed of a material that is affected very little by temperature change. To provide stability in various types of soil, the feet of the tripod should have tapered metal shoes with projections that permit them to be pushed into the ground.

The legs should be of sufficient length to allow the observer to stand erect. For a given observer this means that the length necessary depends upon the location of the instrument eyepiece, which can vary considerably from instrument to instrument. The Jenoptik NI 002(fig.3-21) requires a shorter tripod than the MOM NiA31 (fig. 3-20) or the Zeiss Ni1 (fig. 3-19). For routine leveling, tripods with adjustable legs are not recommended.

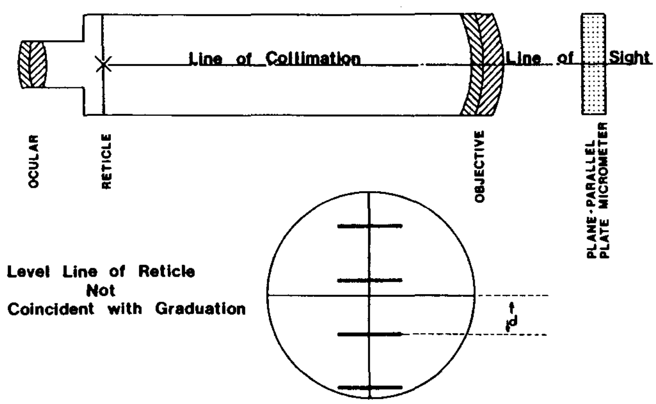


Figure 3-33.—Plane-parallel plate unrotated.

The observer should check the condition of the tripod head each day. A loose head makes it very difficult to maintain the instrument in a level position throughout the backsight and foresight of each setup. The hinge assembly should cover the legs snugly. Hinge tension is properly adjusted when each leg, raised to a horizontal

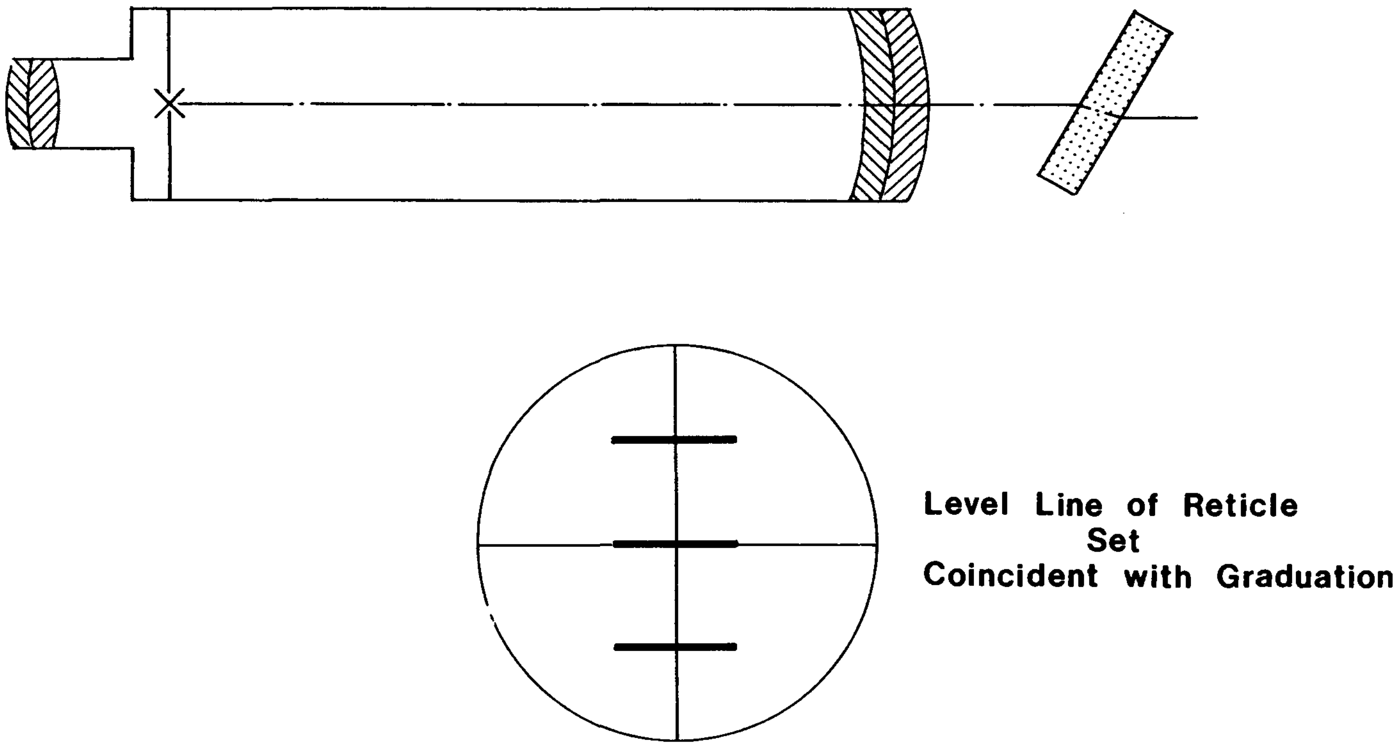


Figure 3-34.—Plane-parallel plate rotated to intercept a graduation.

position, slowly drops of its own accord.

Bolts, located between each pair of legs, should be tightened as necessary to reduce any play between the platform and the legs. Many modern tripods are constructed with plastic bushings that may crack or shatter if the bolts are overtightened, so care should be exercised when making this adjustment. The bushings may be replaced by taking the hinge assembly apart. The tripod hinge assembly should not be stressed by lifting a leg too high or by forcibly jamming the legs simultaneously into the ground. Shock from the latter activity may severely damage the leveling instrument.

The tripod should be placed firmly on the ground and each leg should be pushed in individually, to ensure a stable setup. Setups on bituminous pavement (such as asphalt) must be avoided, since significant tripod settlement may occur. The same precaution applies to frozen ground, where, if possible, the frozen layer should be broken through or shoveled away before setting up the tripod.

Set the tripod to afford as high a line of sight as possible. When leveling along a road shoulder that slopes down away from the road, if the instrument is used at the side (as is the NI 002), point one tripod leg away from the road and stand between the other two legs.

Tribrach. The tribrach, which supports the instrument, is another factor in maintaining instrument stability. The tribrach should be mounted on the tripod head so the foot screws are aligned with the tripod legs. Then, the bell-shaped bolt below the tripod head is screwed up through the two plates forming the bottom of the tribrach. It should be just tight enough to prevent the tribrach from shifting when transporting the instrument-tripod assembly. If overtightened, the plate may eventually loosen enough to wobble. When this occurs, tighten the screws that hold the upper plate against the footscrew pedestals. These screws should never be so tight that the downward pressure causes wear on the pedestals.

On many leveling instruments the tribrach is an integral part of the instrument, not removable in the field. Some instruments (e.g., the NI 002) may be detached from the tribrach by loosening a setscrew on the tribrach. The function of this screw should not be confused with that of the tangent screws immediately above it. Check periodically to ensure that the setscrew is tight.

Foot screws. Each of the three foot screws used for leveling the instrument should be adjusted to the tension that the observer finds is practical. First, turn each foot screw until the small hole on the inside is aligned with a similar hole on the outside. Then, insert an adjusting pin and turn the entire screw assembly to loosen or tighten the tension (the direction depends on the particular instrument). One-quarter to one-half turn in either direction is usually sufficient. The tension should not be so great that it is difficult to feel when a stop has been reached. Do not force the foot screws past their upper or lower stops because this damages the threads.

Figure 3-35.—Plane-parallel plate micrometer scale.

Carrying the instrument. After the instrument is mounted on the tripod, carry the entire assembly vertically, resting it against a shoulder and supporting it by gripping the tripod rungs with one or both hands (fig. 3-36). The tripod should not be balanced across the shoulder, since this puts the instrument in a more vulnerable position and subjects it to greater stress from shock or vibration. Use your body to cushion shocks to the instrument. When it is carried on the side step of a truck, rest the tripod feet not on the metal step, but on the toe of a boot or a thick elastic cushion (such as a section of tire) mounted expressly for this purpose.

Figure 3-36.—Carrying the Instrument-tripod assembly.

Leveling the instrument. At every setup the instrument must be roughly leveled. (See sec. 3.3.3, "Compensator system.") A circular level (known also as a spot or bull's eye) is attached to the body of the instrument for this purpose. The bubble in the level is centered to ensure that the compensator (or the bubble in the tubular level of a spirit-level instrument) is freely suspended.

To rough-level the instrument, first, turn it so the circular level and one foot screw form a line perpendicular to the line formed by the other two foot screws (fig. 3-37). Turn the two foot screws simultaneously, either toward or away from each other, to effect side-to-side movement of the bubble. Then, turn the first foot screw to move the bubble forward and backward, until it is centered.

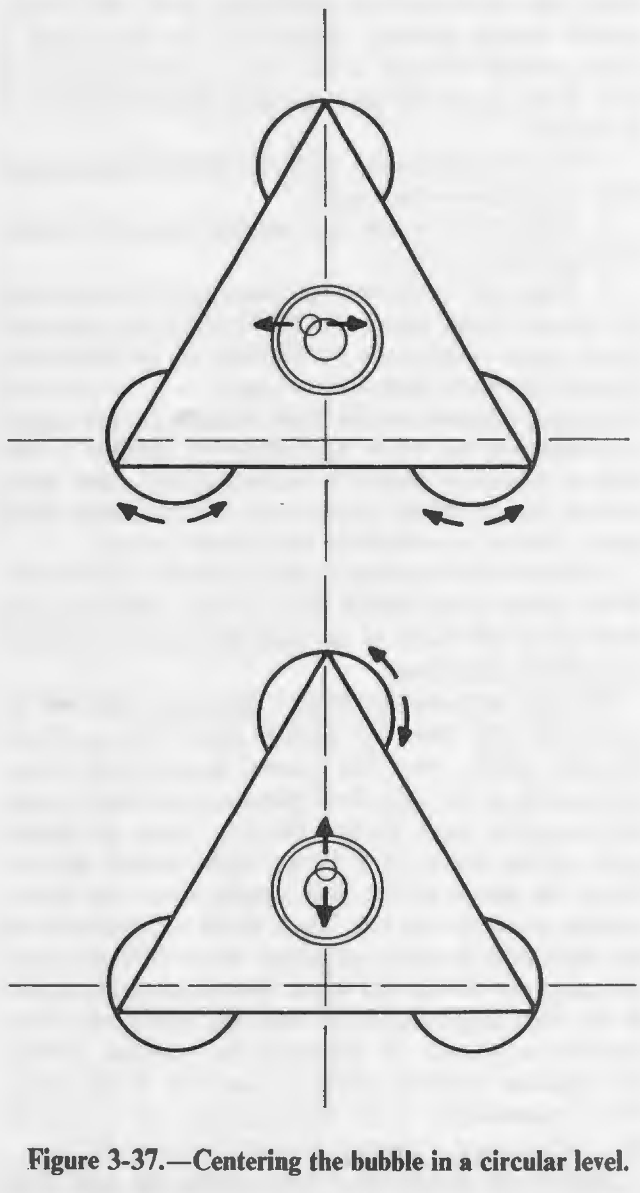


Figure 3-37.—Centering the bubble in a circular level.

Many instruments are equipped with mirrors or other optical mechanisms for observing the circular level. These mechanisms may invert or rotate the bubble image.

For example, the bubble on the NI 002 may be observed either in a mirror or through the eyepiece. In the mirror image, side-to-side movement is the same as, and forward-backward movement is opposite to, what the observer would expect when looking directly down on the bubble. The relationship of the eyepiece image to the direct view of the bubble is more complex, depending on the orientation of the eyepiece. If the eyepiece is turned to the side of the instrument, side-to-side movement appears forward-backward, and vice versa. With practice, the observer can learn to level the instrument efficiently while observing the bubble through the eyepiece.

Circular-level adjustment. At the start of work each day, and after any severe shock to the instrument, the observer should check that the circular level is properly adjusted. A circle is inscribed on the vial glass to provide a reference for assessing the bubble's movement. When the instrument is sufficiently level, the bubble should remain precisely centered as the instrument is slowly turned through a full circle about the vertical axis. If this is not the case, the level should be adjusted as follows:

1. Turn the instrument until the bubble displacement from center is at a maximum.
2. Using the tribrach foot screws, bring the bubble halfway back to center.
3. Using the three or four small screws supporting the circular level, adjust the bubble until it is centered. Each screw is adjusted by slipping an adjusting pin through the screw hole and turning it so as to maintain downward pressure on the base washer. Do not adjust by depressing one screw and retracting another. If the limit of downward motion is reached on the screw being turned, retract all the screws and start adjusting them again. Do not force the level vial against its base.
4. Rotate the instrument slowly through a full circle. If the center of the bubble does not stay within 0.2 mm (0.01 in) of the center of the level vial, repeat the entire adjustment procedure.

Parallax adjustment. Before observing with the instrument, the observer should check for parallax. Parallax results when the focused image of an object does not lie on the same focal plane as the focused image of the reticle lines. To test for this, point the instrument at the sky or at a distant light-colored surface. Focus the ocular so the lines appear sharp and black, without straining the eye. Then, focus the objective on the scale of a leveling rod about 40 m (130 ft) away, and move the eye up and down, slowly, across the ocular. If the lines appear to move over the rod graduations, parallax is present. To eliminate the parallax, refocus the objective until no parallax is evident. If the image of the graduations is not distinct, focus the ocular by the small amount necessary to make it appear distinct.

Making an observation. After setting up and leveling the instrument, the observer should point it quickly at the appropriate leveling rod. With most instruments this is done by sighting along the top of the telescope. Since the observer stands at the side of the NI 002, in this case the system of prisms on the front of the instrument is used to line it up with the rod. Once the instrument is roughly pointed, the rod is observed through the telescope and the tangent screw is turned to align the image of the rod scale with the reticle lines.

The reticle normally includes a single vertical line with three horizontal lines crossing it. The top and bottom stadia lines are used to measure sighting distance. The longer, middle line defines the horizontal line of sight. When leveling with scales having block graduations, a straight line is sufficient. However, with scales having line graduations, the reticle should include a pair of divergent lines, the wedge, marked to one side in lieu of the middle line. The wedge (fig. 3-38) is a more sensitive tool for precisely intercepting a graduation. There are two ways to use it.

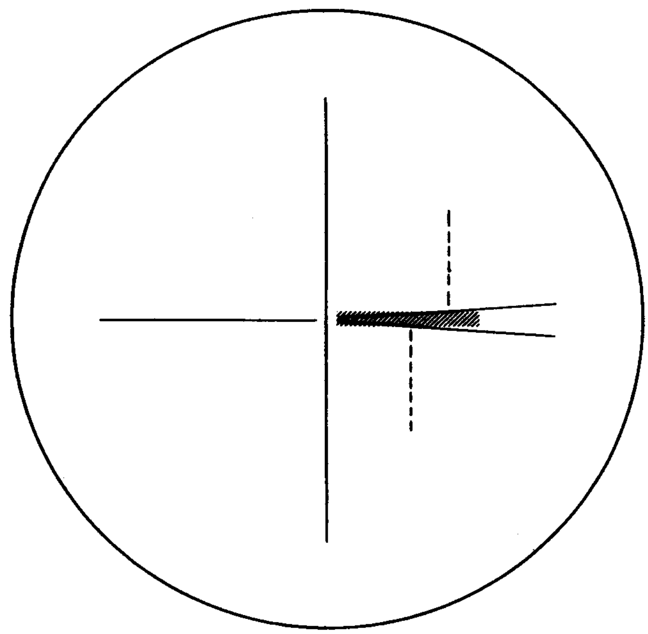
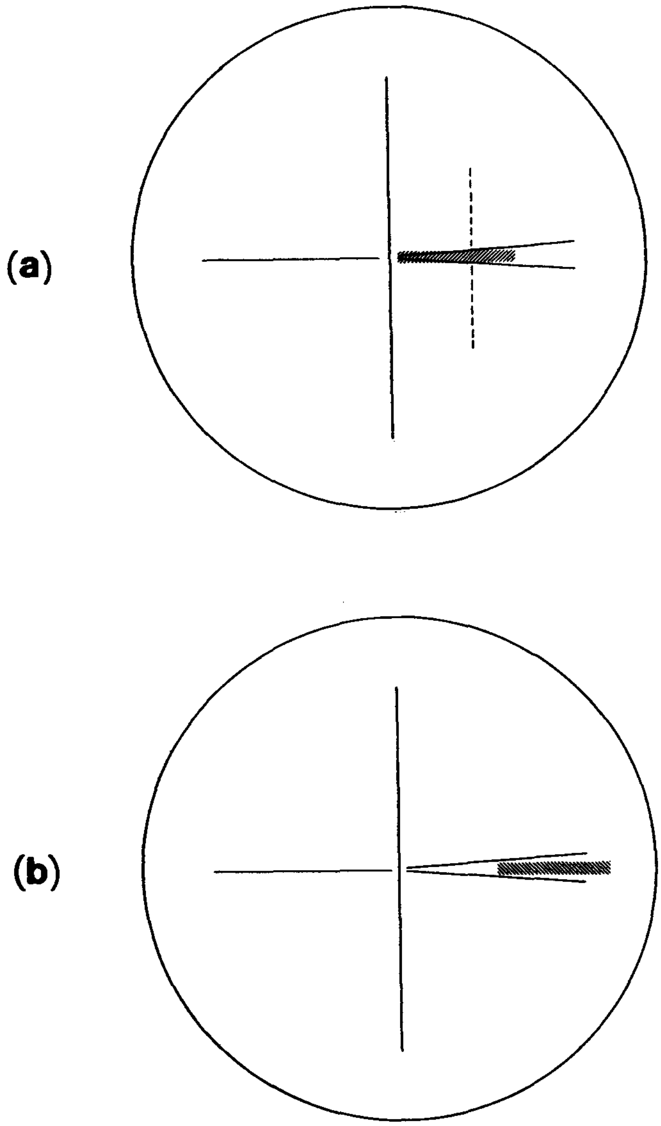


Figure 3-38.—Wedge reticle, graduation not centered.

In one method, set the wedge so the intersections of the upper and lower wedge-lines with the upper and lower edges of the graduation appear in vertical alignment (fig. 3-39a). In the second method, set the wedge so the interior edges of the wedge lines appear to just touch the corners of the end of the graduation (fig. 3-39b). Always make the final setting by turning the micrometer knob in the same direction.

In wind or on a vibrating surface, such as a bridge, it may be difficult to intercept a graduation precisely. If the vibration of the instrument is short in period, set the wedge so it appears to vibrate an even amount above and below the graduation. If the vibration is long in period or includes more than one graduation interval, mean the readings from a series of pointings to make each observation. If satisfactory results cannot be obtained, return to the section under better observing conditions.



Figures 3-39a. & 3-39b.—Wedge reticle, graduation centered.

Instrument maintenance. Before putting the instrument away each day gently wipe it with a clean soft cloth to remove dust and moisture. If the instrument has been exposed to rain or mist, it should also be allowed to stand at room temperature overnight. If the instrument is dirty or greasy, or if it has been exposed to salt water, clean all nonglass surfaces with a cloth dampened in denatured alcohol. Dust the objective and ocular lightly with a lens brush. Then, if necessary, wipe the lenses with lens paper that has been moistened with a small amount of lens cleaning solution. Care must be taken not to scratch the glass surfaces.

The instrument, with the lens cap attached, should be stored securely in a specially designed padded case. To protect the instrument from jolts and vibrations while the truck is in transit, the instrument should either be (1) carried in a foam-lined box bolted to the truck bed, (2) stored in its case and held on the lap of a passenger in the truck, or (3) strapped onto an empty seat in the truck. If the instrument is to be shipped to another location, it should be packed in its case and placed inside a well-padded carton, with an appropriate warning label affixed on the outside of the package.

At least once every 18 months, the instrument should be cleaned and adjusted on a collimator by a qualified technician.

Shipment of the NI 002. The National Geodetic Survey has found the objective assembly of the NI 002 to be particularly vulnerable to vibration during shipment. Damage to the stop at the base of the assembly can be recognized by the inability of the user to read a full scale on the micrometer when the instrument is leveled. If the stop has been damaged, the instrument should not be used until it is repaired.

To prevent damage to the stop, screw a small, hard-rubber plug into the instrument to limit movement of the assembly during shipment (fig. 3-21). The plug should be screwed in whenever the NI 002 is transported for long distances in a leveling truck or shipped via commercial carrier.

3.3.6 Sighting Distance

The sighting distance between the instrument and a leveling rod is normally computed by the stadia method. To use the method, a full or half stadia interval must be computed from stadia readings made while observing. To balance the sighting distances of awkward setups quickly, before any observations are recorded, the stadia interval may also be determined by counting the number of rod units observed between the stadia lines (full interval) or between the middle line and one stadia line (half interval).

Derivation of the stadia method. By the stadia method, short distances may be measured with a precision of ± 0.2 m (± 0.7 ft). The stadia method has been widely used for topographic surveys and is routinely used in geodetic leveling to measure sighting distances and control the imbalance between backsights and foresights.

Figure 3-40 illustrates the relationships which exist between the leveling rod, the objective, and the reticle when a stadia measurement is made with a leveled instrument. For the focused instrument, the distance, D_i, from the objective to the reticle, and the distance, D_o, from the objective to the rod, are governed by the Law of Conjugate Foci, where the focal length, f, of the objective, f, is a constant:

$$(1/D_o)+(1/D_i)=1/f.$$

The sighting distance, s, is equal to D_o plus the fixed distance, c, between the objective and the vertical axis of the instrument. By solving for D_o and adding c:

$$s=f\times(D_o/D_i)+f+c.$$

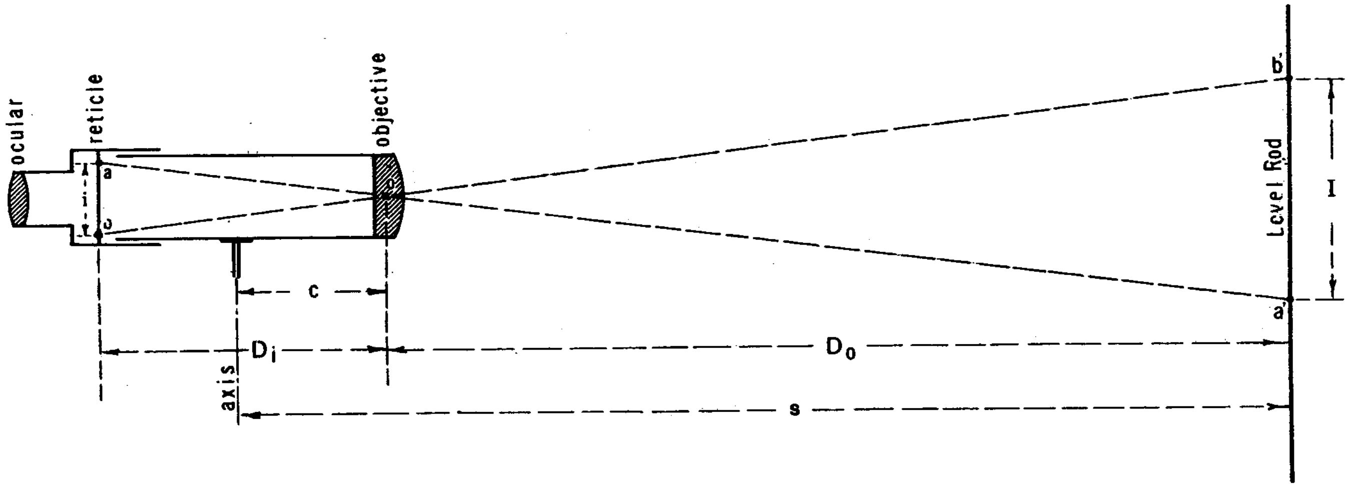


Figure 3-40.—Relationship of stadia interval to sighting distance.

To a sufficient approximation, the line connecting points *a* and *a'* is a straight line through point *o*; therefore, triangles *abo* and *a'b'o* are similar. Thus, all corresponding dimensions are in the ration D_o:D_i, including the full stadia interval, *I*, and the distance, *i*, between the upper and lower stadia lines in the reticle:

$$D_o/D_i = I/i.$$

By substituting the known ratio for the unknown:

$$s = (f/i) \times I + f + c.$$

The ration, *f*:*i*, is unique for the instrument and is called the stadia factor ("multiplication constant"). The constant, $f+c$, is also unique for the instrument and is called the stadia constant ("addition constant"). In most instruments with internal-focusing telescopes, the optical system is so designed that the stadia constant is essentially zero for the sighting distances used in leveling. Table 3-2 shows stadia factors and constants for some typical instruments.

Computing sight distance. From the full stadai interval, *I*, the sighting distance, *s*, is computed by the formula just derived:

$$s = (I \times \text{stadia factor}) + \text{stadia constant}.$$

For practical use in the field, the stadia constant is usually neglected. However, it should not be neglected when determining the stadia factor. During these operations sighting distance must be determined as accurately as possible. However, during routine leveling the error resulting from neglecting the stadia constant is insignificant when compared to the overall error of the stadia method.

To obtain the sighting distance in meters, a conversion factor must be included in the formula. Since the stadia factor is a unitless quantity and the stadia interval is measured in rod units (ru), the conversion factor required is the number of meters per rod unit. Thus:

$$s_m = I_{ru} \times \text{conversion factor}_{m/ru} \times \text{stadia factor}.$$

In the past, the product of the stadia factor and the rod-unit conversion factor has been referred to as the "stadia constant." Do not confuse such usage of the term with the definition given here.

This formula applies in three-wire leveling and whenever the full interval between the stadia lines is computed. For example, if the instrument has a stadia factor of 100 and the rod units are centimeters,

$$s_m = I_{cm} \times 0.01_{m/cm} \times 100 = I.$$

Thus a convenient relationship is derived: the number of rod units in the full stadia interval equals the sighting distance in meters.

However, if the rod units are in half-centimeters (hcm), the formula becomes

$$s_m \times I_{hcm} \times 0.005_{m/hcm} \times 100 = I/2.$$

In micrometer leveling, this result is employed to reduce the number of readings necessary. Instead of reading the intercepts of all three lines, only those of the wedge and one stadia line (usually the lower) are read. The difference of the readings is half of the stadia interval. As long as the rod units are half-centimeters, the difference of the readings is equal to the sighting distance. When using the micrometer-leveling procedure with centimeter rods, the half stadia interval must be multiplied by two to obtain the correct sighting distance.

Determining the stadia factor. The stadia factor is set by the instrument manufacturer for converting the stadia interval to sighting distance. In modern instruments used for geodetic leveling the stadia factor is typically 100. The reticle is etched in glass to ensure that the factor does not change. The stadia factor need not be determined in the field when using such an instrument.

Some older instruments may have a reticle consisting of fine strands of spider web, sandwiched between glass plates. The stadia factor for this type of instrument should be determined in the field whenever it is put to use and whenever the reticle is changed. To determine the stadia factor:

1. Lay out a course in a straight line along a level track, roadway, or sidewalk, marking the distances at 0, 25, 35, 45, 55, 65, and 75 m.
2. Position the instrument to allow for its stadia constant. For example, the Wild N3 has a stadia constant of −0.2 m; therefore, it should be set forward of the zero stake by 0.2 m. An instrument with a positive stadia constant should be set back from the zero stake by the corresponding amount.
3. Level the instrument. Read the low scale of a leveling rod at each of the six distances. Read all three reticle lines, top to bottom, at each distance.
4. Record the readings and identifying information as shown in figures 3-41 and 3-42, using the recording form appropriate to the type of leveling to be done. Compute each half-stadia interval.
5. Total the half-stadia intervals (in rod units). Total the measured distances (in meters) and divide by the total of the half-stadia intervals. The value obtained is the stadia factor, expressed in meters per rod unit, for the instrument-rod combination used.
6. The result of step 5 should be checked by computing a similar value for each distance. Divide each measured distance by the sum of the two half-stadia intervals at that distance. The mean of these six values should agree with the value computed in step 5.
7. To obtain the stadia factor of the instrument, independent of rod units, the value expressed in meters per rod unit must be multiplied by a conversion factor. This factor is the number of rod units per meter.
8. Submit data for the stadia factor measurement to the project office, along with other leveling data for the day.

3.3.7 Collimation Check

Under field conditions the collimation error of a leveling instrument (sec. 3.1.2, "Leveling instrument") is measured by obtaining a set of observations called the collimation check ("C-shot" or "peg test"). Collimation error is limited by adjustment of the instrument. Because the adjustment can change easily under field conditions, thus changing the collimation error, make the collimation check at least once a day with most instruments. In addition, make the check any time that an instrument sustains a severe shock or seems to function abnormally.

The collimation check has two purposes: to prove that the instrument is properly adjusted within the standard of accuracy required for the survey and to provide a collimation factor with which to correct data from unbalanced setups.

Collimation factor. During the collimation check the angular value of collimation error is not measured directly. Instead, the tangent of the angle, called the collimation factor (C-factor or C) is computed. A derivation of the formula follows.

Two elevation differences are observed in the same direction, but with different setups, between the same two turning points. Each setup has an imbalance of sighting distances. Since the elevation differences measured between the same two points should be equal, any difference between them is the result of the effects of collimation error, pointing error, refraction, and curvature.

The collimation error, α , causes an error that is proportional to the imbalance, Δs, in each setup:

$$\tan \alpha \times \Delta s = C \times \Delta s.$$

The pointing error is limited by a reading check. The sum, e, of refraction and curvature error is mathematically predicted for the appropriate sighting distances in each setup. Hence, if each observed elevation difference, Δh, is corrected for these errors,

$$\Delta h_2 - (C \times \Delta s_2) - e_2 = \Delta h_1 - (C \times \Delta s_1) - e_1.$$

Solving for the collimation factor,

$$C = [(\Delta h_1 - e_1) - (\Delta h_2 - e_2)] / (\Delta s_1 - \Delta s_2).$$

If the first setup is balanced and the second unbalanced (as for "Kukkamaki's method," given later in this subchapter, $\Delta s_1 = 0$ and $e_1 = 0$,

$$C = [\Delta h_1 - (\Delta h_2 - e_2)] / (-\Delta s_2).$$

If both setups are unbalanced by the same amount, and they are leveled in opposite directions (as for the "10-40 method," given later in this subchapter), the corrected elevation differences are opposite in sign. Thus,

$$C = [(\Delta h_1 - e_1) + (\Delta h_2 - e_2)] / (\Delta s_1 - \Delta s_2).$$

Since $\Delta s_1 = \Delta s_2$ and $e_1 \cong e_2$, the formula is simplified to the following:

$$C = [(\Delta h_1 + \Delta h_2) - 2e] / 2\Delta s.$$

The elevation differences, the refraction and curvature errors, and the imbalances are measured in the same units; the resulting collimation factor is dimensionless. However, for convenience, it is usually expressed in millimeters per meter:

$$C_{\mathrm{mm/m}} = \tan \alpha \times 1000_{\mathrm{mm/m}}.$$

For an instrument with a reversible compensator, a separate collimation factor may be computed for each position of the compensator. The collimation factor for the instrument is the mean,

$$C = (C_1 + C_2)/2.$$

If the collimation factor is not within the tolerance for the survey, the instrument must be adjusted. When the adjustment can be made in the field, the amount of

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GEODETIC LEVELING
MICROMETER OBSERVATIONS (Δh)

40	YR./MO./DAY	CODE	INSTRUMENT SERIAL NO.	CODE	ROD SERIAL NO.	CODE	ROD SERIAL NO.		Z	TIME
41	80 07 28	217	608134	316	277890			STADIA FACTOR EXAMPLE		

FROM	BM DESIGNATION	TO	BM DESIGNATION	Z	TIME BEGIN	TIME END	F/C	TEMPERATURE BEGIN	TEMPERATURE END	W	S	OBSERVER
				R	0801	0830	C	271	275	0	2	CWS

SET UP#	UPPER STADIA	S_B Σ	LOWER STADIA	S_F Σ	LOW SCALE BACK/FORE	Δh_L Σ	HIGH SCALE BACK/FORE	Δh_H Σ	$\Delta h_L - \Delta h_H$	REMARKS
25	401.0	25.0	376.0	25.0	B		B			25/50.0 = 0.500
	376.0		351.0		F		F			
35	411.1	35.1	376.0	35.0	B		B			35/70.1 = 0.499
	376.0	60.1	341.0	60.0	F		F			
45	420.0	45.0	375.0	44.9	B		B			45/89.9 = 0.501
	375.0	105.1	330.1	104.9	F		F			
55	431.0	55.0	376.0	55.0	B		B			55/110.0 = 0.500
	376.0	160.1	321.0	159.9	F		F			
65	440.0	65.0	375.0	65.0	B		B			65/130.0 = 0.500
	375.0	225.1	310.0	224.9	F		F			
75	451.0	75.0	376.0	75.0	B		B			75/150.0 = 0.500
	376.0	300.1	301.0	299.9	F		F			
300 m			600.0 hcm		300/600.0 = 0.500					MEAN 0.500
								CHECK		0.500 m/hcm x 200 hcm/m = 100

Figure 3-41.—Stadia factor determination, micrometer procedure, half-centimeter rods.

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GEODETIC LEVELING
THREE-WIRE OBSERVATIONS

40	YR./MO./DAY	CODE	INSTRUMENT SERIAL NO.	CODE	ROD SERIAL NO.	CODE	ROD SERIAL NO.		Z	TIME
41	80 07 28	217	30150	312	1305			STADIA FACTOR EXAMPLE		

FROM	BM DESIGNATION	TO	BM DESIGNATION	Z	TIME BEGIN	TIME END	F/C	TEMPERATURE BEGIN	TEMPERATURE END	W	S	OBSERVER
				R	0845	0914	C	280	285	0	2	CWS

SET UP	BACKSIGHT UPPER / MIDDLE / LOWER Σ	MEAN Σ	BACK CENTER	S_U / S_L	(U+L) Σ	FORESIGHT UPPER / MIDDLE / LOWER Σ	MEAN Σ	BACK CENTER	S_U / S_L	(U+L) Σ	COMMENTS
	133.7										
25	130.0			3.7							
	126.2			3.8	7.5			25/7.5 = 3.33			
	135.9										
35	130.7			5.2							
	125.4			5.3	10.5						
					18.0			35/10.5 = 3.33			
	137.7										
45	131.0			6.7							
	124.2			6.8	13.5			45/13.5 = 3.33			
					31.5						
	132.5										
55	124.3			8.2							
	116.0			8.3	16.5						
					48.0			55/16.5 = 3.33			
	126.2										
65	116.5			9.7							
	106.8			9.7	19.4			65/19.4 = 3.35			
					67.4						3.34 m/cm
	121.8									MEAN = 3.34	x 100 cm/m
75	110.6			11.2							
300 m	099.4		300/89.8 = 3.34	11.2	22.4			75/22.4 = 3.35			334
					89.8 cm			CHECK			

Figure 3-42.—Stadia factor determination, three-wire procedure, centimeter rods.

error to be removed is computed from the following formula. It is referred to the rod at the farthest sighting distance, s_F, and computed in rod units:

$$\text{error}_F = s_F \times C_{mm/m} \times \text{conversion factor}_{ru/mm}.$$

The error is subtracted from the last reading made on the far rod. Then the instrument is adjusted in such a way that the corrected reading is observed. After performing the adjustment, the entire procedure is repeated to compute and check the new collimation factor.

Note that the collimation-correction factor, "*C*", commonly used to correct data from three-wire leveling, is not equal to the collimation factor defined here. In the past, "*C*" has been defined as the product of the stadia factor and the tangent of the collimation error. The imbalance—expressed in rod units as a stadia interval, not as a distance—could be multiplied directly by "*C*" to obtain the correction for collimation error in rod units. Modern instruments and rods make this practice unnecessary, since they usually provide a one-to-one correspondence between stadia interval and distance.

General instructions. Two methods, sufficiently precise to satisfy the purposes of the collimation check, are presented here. The accuracy of the result with either of these methods depends on the assumption that the error observed is entirely a function of collimation error. Other effects that alter the line of sight, such as refraction, must be controlled. To achieve this, perform the following:

1. Make the collimation check on uniformly flat ground. The slope should be no more than 2 percent.
2. Allow the instrument and leveling rods to acclimatize for 5 minutes or more, if they have just been removed from their cases.
3. Make sure that the circular levels on the instrument and the rods are properly adjusted. (See secs. 3.3.5, "Circular-level adjustment" and 3.4.3, "Checking the circular level.")
4. Correct for refraction and curvature at each sighting distance. To use the values for refraction and curvature given in table 3-3, the collimation check should be made, if possible, when the air-temperature differential is less than 1°C (2°F) in magnitude and negative in sign (sec. 3.6.1). However, make a check immediately following an accident to determine if the instrument has been damaged, regardless of the atmospheric conditions. Then, make another check as soon as the conditions are suitable.

Kukkamaki's method. T. J. Kukkamaki of the Finnish Geodetic Institute developed this method for making a collimation check. The following instructions apply to any combination of instrument and rods. If the instrument does not have a reversible compensator, ignore references to compensator's position. Record the data either on a standard recording form for leveling or into a computer.

Table 3-3.—Refraction and curvature errors in a single sight

Sighting distance, *s* (m)	Sighting distance, *s* (ft)	Error in a rod reading, *e* (mm)	Error in a rod reading, *e* (ft)
0 to 28	0 to 92	0.0	0.000
28 48	92 157	0.1	0.000
48 61	157 200	0.2	0.001
61 73	200 240	0.3	0.001
73 82	240 269	0.4	0.001
82 91	269 299	0.5	0.002
91 99	299 325	0.6	0.002
99 106	325 348	0.7	0.002
106 113	348 371	0.8	0.003
113 119	371 390	0.9	0.003
119 125	390 410	1.0	0.003
125 131	410 430	1.1	0.004
131 137	430 449	1.2	0.004
137 142	449 466	1.3	0.004
142 147	466 482	1.4	0.005
147 150	482 492	1.5	0.005
160	525	1.8	0.006
170	558	2.1	0.007
180	591	2.3	0.008
190	623	2.6	0.009
200	656	2.8	0.009
210	689	3.0	0.010
220	722	3.3	0.011
230	755	3.7	0.012
240	787	4.0	0.013
250	820	4.3	0.014
260	853	4.7	0.015
270	886	5.0	0.016
280	919	5.4	0.018
290	951	5.8	0.019
300	984	6.2	0.020

1. On the flattest possible ground, lay out a setup with precisely 20 m between the turning points. Position the instrument in line between them, precisely 10 m from each turning point. (Use a tape to measure these distances.) (See fig. 3-43.)
2. As indicated on the recording form (line *40*), enter the date, instrument code and serial number, rod codes and serial numbers, time and time zone, type of temperature units, beginning air temperature, wind and sun codes, and the initials of the observer, recorder, and rodmen. Check all serial numbers against the equipment actually used. Label the recording form "Collimation Check." (See figs. 3-44 through 3-46 for examples.)
3. FIRST SETUP: Level the instrument. Check that the circular levels on the instrument and the rods are properly adjusted. Observe and record a set of readings by either the micrometer or three-wire procedure. Use rod 1 as the backsight, and be sure to change the compensator's positions between the low- and high-scale readings. Check that the imbalance is no more than ±0.4 m. The reading check for the micrometer procedure should be ±0.25 mm (±0.05 hcm for half-centimeter rods).

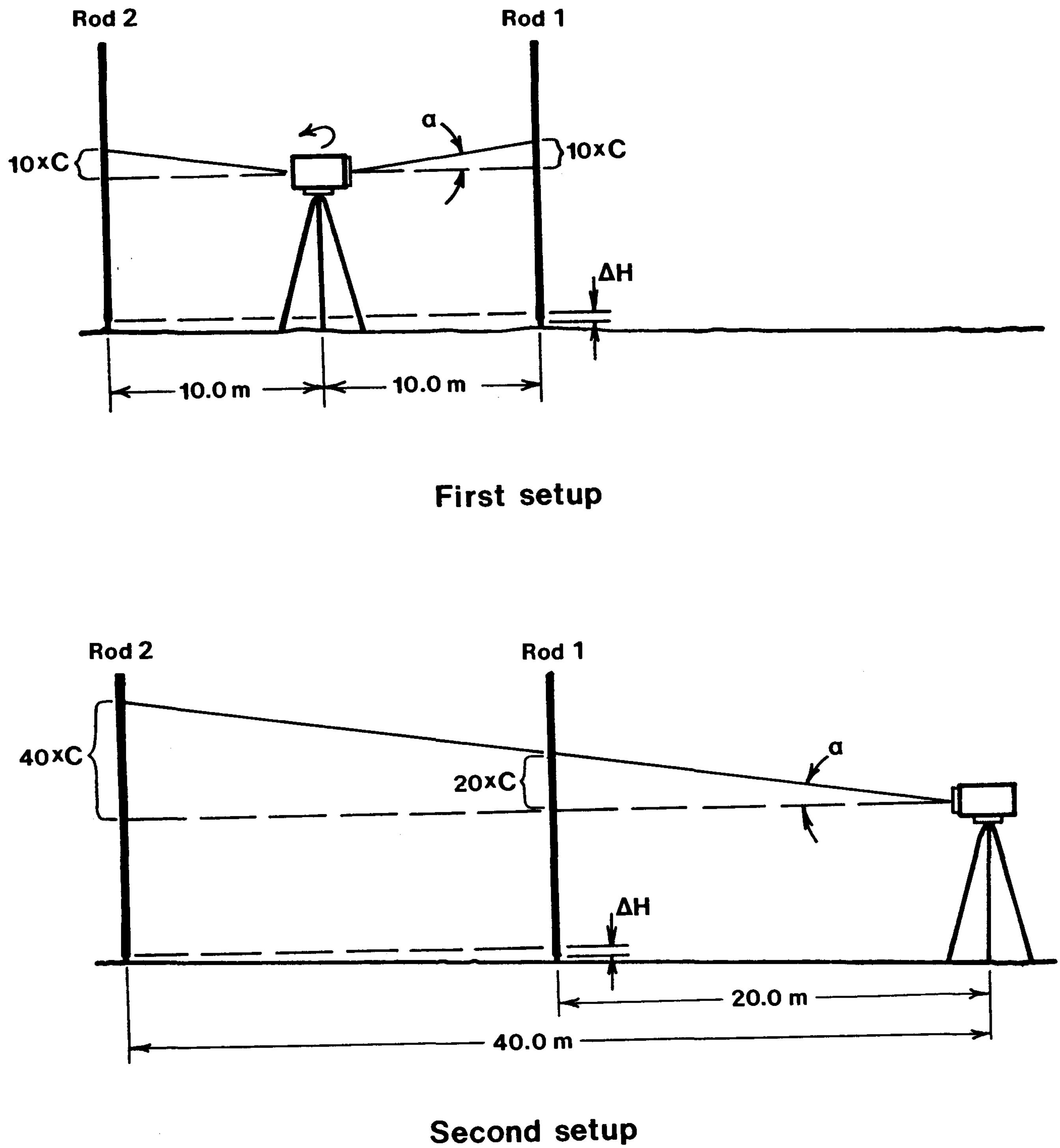

Figure 3-43.—Collimation check, Kukkamaki's method.

4. SECOND SETUP: Position the instrument in line with the turning points, 20 m from rod 1. Put the compensator in position one. With rod 1 as the backsight, observe and record another set of readings; however, leave the compensator in position one throughout the setup. Check that the magnitude of the imbalance is between 19.6 and 20.4 m. Enter the ending temperature. Remain in position until the collimation factor has been checked.

5. Convert the elevation differences, Δh_2 and Δh_1, from rod units to millimeters. Compute or obtain from table 3-3 the values, e, for refraction and curvature error at the sighting distances of the second setup, 20 and 40 m. Compute C, in millimeters per meter, by the formula:

$$C=[(\Delta h_1-\Delta h_2+(e_{20}-e_{40}))]/-\Delta s_2.$$

If the instrument has a reversible compensator, label the value "C_1" and skip to step 11.

6. Compare C with the tolerance (table 3-1). If the tolerance is exceeded, then adjust the instrument as follows.

7. ADJUSTMENT: Compute the error, in rod units, resulting from collimation error in the reading made at 40 m:

$$\text{error}_{ru} = 40 \times C_{mm/mm} \times \text{conversion factor}_{ru/mm}.$$

For half-centimeter rods, the error is $8 \times C$. For centimeter rods, it is $4 \times C$. Subtract this from the foresight reading, obtained on rod 2 (high scale) during the second setup. The result is the correct reading after adjusting the instrument.

8. Refer to the instrument manual for the mechanics of adjusting the instrument. Many compensator instruments have a setscrew on the front of the telescope that is loosened, after which a wedge-shaped lens is rotated to correct the angle of the line of sight.

9. Still in position for the second setup, point toward rod 2 (high scale) and adjust the instrument until the line of sight intercepts the corrected reading.

10. Return to the first setup (step 3) and repeat the entire collimation check to compute and check the new collimation factor.

11. THIRD SETUP (reversible compensator only): Put the compensator in position two and follow the same procedure as for the second setup (step 4). Compute C_2 by the same formula as C_1.

13. Compute the mean collimation factor and record it:

$$C = (C_1 + C_2)/2.$$

Check this against the tolerance for instruments with reversible compensators (table 3-1).

14. Compute the Q-factor and record it:

$$Q = C_1 - C_2.$$

Check this against the tolerance (table 3-1).

15. If the tolerances in step 13 is exceeded, the collimation check should be repeated to verify the original observations. If the tolerances are still exceeded, return the instrument to the repair shop or factory for adjustment.

10-40 method. Another procedure for the collimation check is called the 10-40 method because each setup is unbalanced, with one rod positioned 10 m away from the instrument and the other 40 m away. The following instructions apply to any combination of instrument and rods. Record the data either on a standard recording form for leveling, or into a computer.

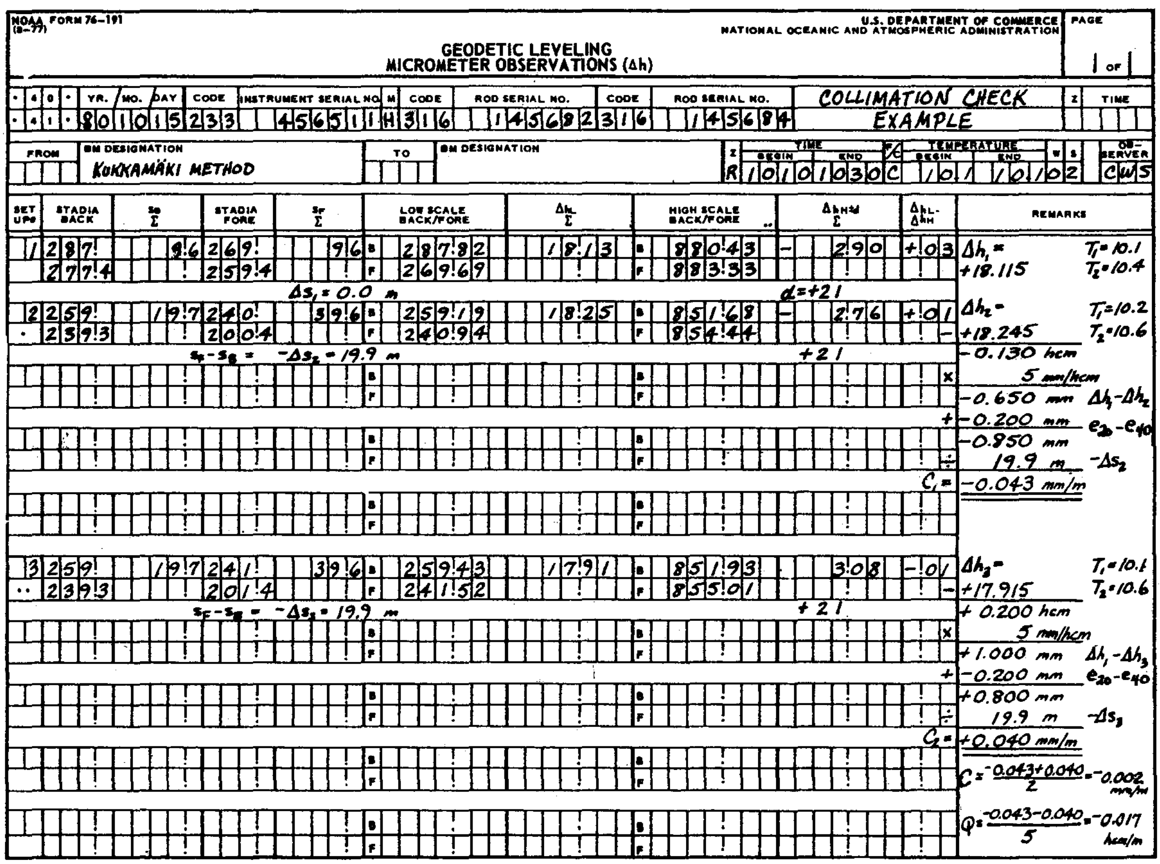

Figure 3-44.—Collimation check, Kukkamaki's method, micrometer procedure, half-centimeter rods, reversible compensator.

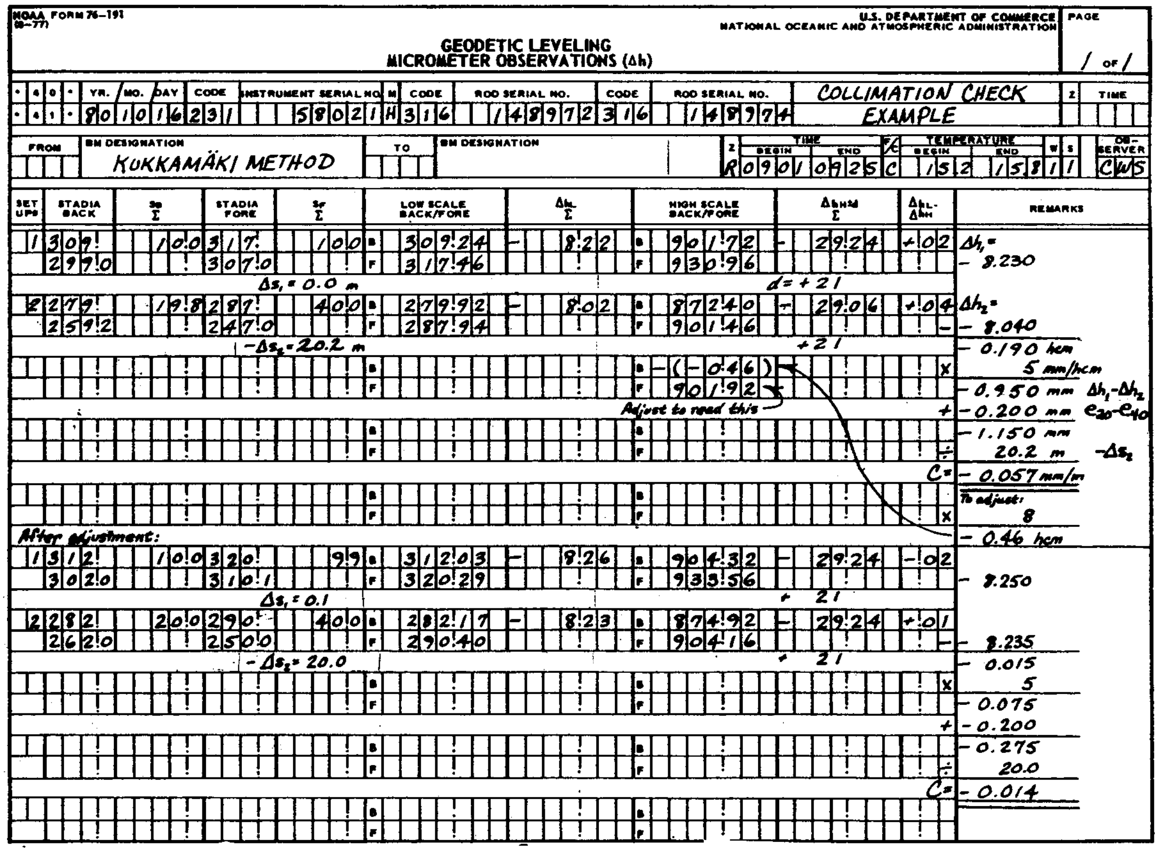

Figure 3-45.—Collimation check, Kukkamaki's method, micrometer procedure, centimeter rods, nonreversible compensator.

1. On the flattest possible ground, lay out a setup with precisely 50 m between the turning points. Position the instrument in line between them, precisely 10 m, as measured with a tape, from the turning point supporting rod 1. (See fig. 3-47.)
2. As indicated on the recording form (line *40*) enter the date, instrument code and serial number, rod codes and serial numbers, time and time zone, type of temperature units, beginning temperature, wind and sun codes, and the initials of the observer, recorder, and rodmen. Check all serial numbers against the equipment actually used. Label the form "Collimation Check." (See figs. 3-48 through 3-50 for examples.)
3. FIRST SETUP: Level the instrument. Check that the circular levels on the instrument and the rods are properly adjusted. Make a set of readings by the micrometer or three-wire procedure. Use rod 1 as the backsight. Check that the magnitude of the imbalance is between 29.6 and 30.4 m. The reading check for the micrometer procedure should be ±0.25 mm (±0.05 hcm for half-centimeter rods). Do not make the reading check if using a reversible compensator.
4. SECOND SETUP: Position the instrument in line between the turning points, precisely 10 m from rod 2. Make a set of readings as in step 3, using rod 2 as the backsight. Check the imbalance as in step 3. If using an instrument with a reversible compensator, check that

$$(\Delta h_L - \Delta h_H)_1 + (\Delta h_L - \Delta h_H)_2 \leq \pm 0.35 \text{ mm}.$$

The tolerance corresponds to ±0.07 hcm for half-centimeter rods. (If this reading check is not satisfied, begin again at step 1.) Remain in position until the collimation factor has been checked.

5. Convert the elevation differences, Δh_2 and Δh_1, from rod units to millimeters. Compute or obtain from table 3-3 the values, e, for refraction and curvature at 10 and 40 m. Compute the collimation factor, in millimeters per meter, by the following formula. Note that the imbalances are both negative.

$$C = [(\Delta h_1 + \Delta h_2) - 2(e_{10} - e_{40})] / (\Delta s_1 + \Delta s_2)$$

6. Check the collimation factor against the tolerance (table 3-1). If the tolerance is exceeded, then adjust the instrument as described in steps 7 through 10 of Kukkamaki's method.
7. If the tolerances in step 6 are exceeded, the collimation check should be repeated to verify the original observations. If the tolerances are still exceeded, return the instrument to the repair shop or factory for adjustment.

3.3.8 Compensation Check

When a compensator instrument is roughly leveled, the compensator should be freely suspended, unaffected by its suspension and damping mechanisms. The range

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GEODETIC LEVELING
THREE-WIRE OBSERVATIONS

40 *41*	YR.	MO.	DAY	CODE	INSTRUMENT SERIAL NO.	CODE	ROD SERIAL NO.	CODE	ROD SERIAL NO.		Z	TIME
	80	10	16	232	30910	312	106332	312	106334	COLLIMATION CHECK EXAMPLE		

FROM	BM DESIGNATION	TO	BM DESIGNATION	Z	TIME BEGIN	TIME END	F/C	TEMPERATURE BEGIN	TEMPERATURE END	W	S	OBSERVER
	KUKKAMÄKI METHOD			S	1030	1044	C	20.1	20.2	/	/	CWS

SET UP	BACKSIGHT UPPER / MIDDLE / LOWER Σ	MEAN Σ	BACK CENTER	S_U / S_L	(U+L) Σ	FORESIGHT UPPER / MIDDLE / LOWER Σ	MEAN Σ	BACK CENTER	S_U / S_L	(U+L) Σ	COMMENTS
	1596					1637					
1	1546	15460		5.0		1587	15870		5.0		
	1496			5.0	10.0	1537			5.0	10.0	ΔS_1 = 0.0 m
	1500					1640					
2	1400	14003		10.0		1440	14400		20.0		
	1301			9.9	19.9	1240			20.0	40.0	$-\Delta S_2$ = 20.1 m
		1457	$B_1 - B_2$		29.9		1470	$F_1 - F_2$		50.0	
	−	14.70	$F_1 - F_2$								
		−0.13	cm				14400				
	X	10.	mm/cm				−(−0.32)	Adjust to read			
		−1.30	mm $\Delta h_1 - \Delta h_2$				14432	this			
	+	−0.20	mm $e_{20} - e_{40}$								
		−1.50	mm								
	÷	20.1	m $\Delta S_1 - \Delta S_2$								
	C=	−0.08	mm/m								
	To adjust:										
	X	4									
		−0.32	cm								

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GEODETIC LEVELING
THREE-WIRE OBSERVATIONS

40 *41*	YR.	MO.	DAY	CODE	INSTRUMENT SERIAL NO.	CODE	ROD SERIAL NO.	CODE	ROD SERIAL NO.		Z	TIME
	80	10	16	232	30910	312	106332	312	106334	COLLIMATION CHECK EXAMPLE CONTINUED		

FROM	BM DESIGNATION	TO	BM DESIGNATION	Z	TIME BEGIN	TIME END	F/C	TEMPERATURE BEGIN	TEMPERATURE END	W	S	OBSERVER
	After adjustment:			S	1048	1105	C	20.2	20.1	/	/	CWS

SET UP	BACKSIGHT UPPER / MIDDLE / LOWER Σ	MEAN Σ	BACK CENTER	S_U / S_L	(U+L) Σ	FORESIGHT UPPER / MIDDLE / LOWER Σ	MEAN Σ	BACK CENTER	S_U / S_L	(U+L) Σ	COMMENTS
	1621					1662					
1	1571	15710		5.0		1612	16120		5.0		
	1521			5.0	10.0	1562			5.0	10.0	ΔS_1 = 0.0
	1536					1577					
2	1436	14360		10.0		1477	14773		20.0		
	1336			10.0	20.0	1378			19.9	39.9	$-\Delta S_2$ = 19.9
		1350			30.0		1347			49.9	
	−	13.47									
		+0.03									
	X	10.									
		+0.30									
	+	−0.20									
		+0.10									
	÷	19.9									
	C=	+0.005									

Figure 3-46.—Collimation check, Kukkamaki's method, three-wire procedure, half-centimeter rods.

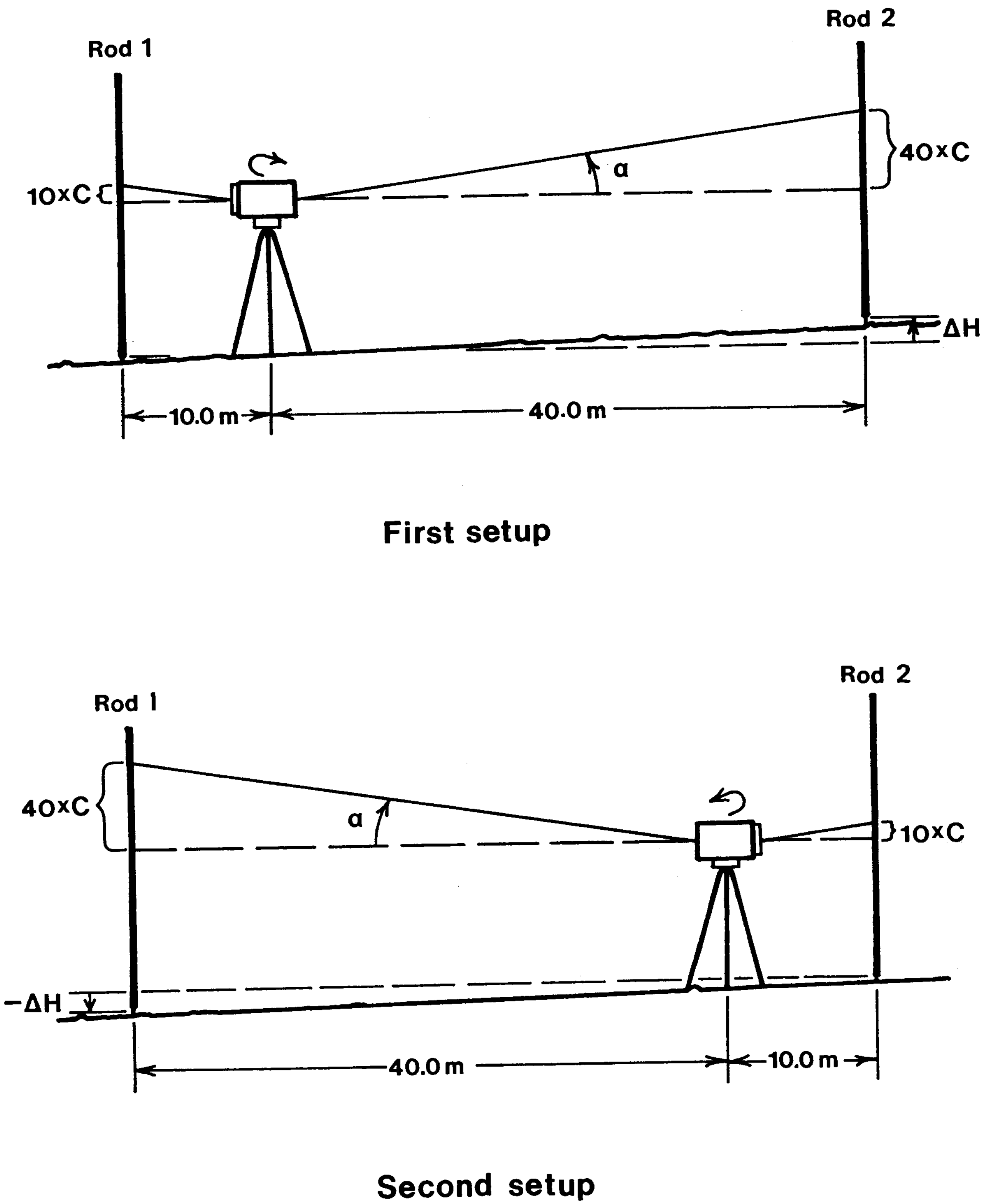
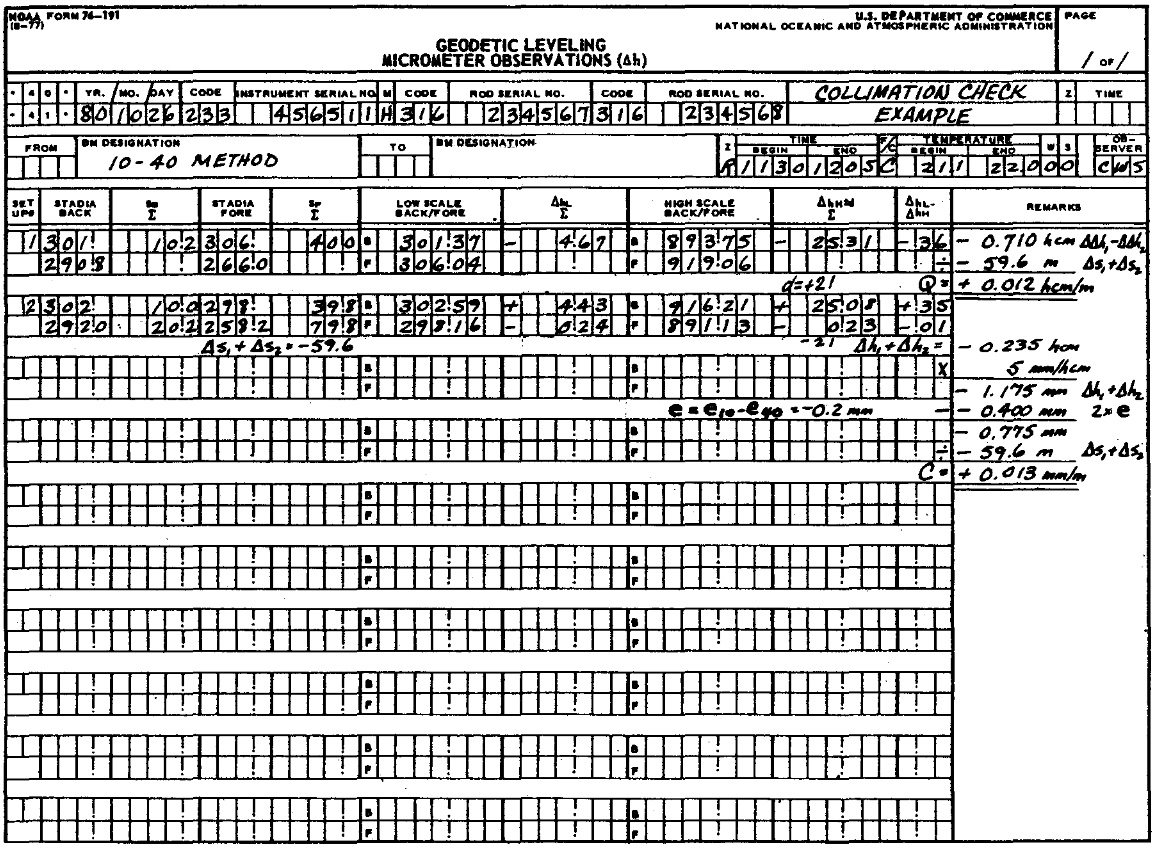


Figure 3-47.—Collimation check, 10-40 method.

of arc in which the compensator is expected to suspend freely should be somewhat greater than the arc described by a 2-mm movement of the bubble in the circular level. (Table 3-3 contains the sensitivities of the circular levels mounted on various instruments.) When the circular level is properly adjusted the compensator should provide a line of sight having a consistent collimation error, within 1.″0 (0.00485 mm/m) of that measured during the collimation check, no matter in which direction the instrument is pointed.



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GEODETIC LEVELING
MICROMETER OBSERVATIONS (Δh)

PAGE 1 OF 1

·40·	YR./MO./DAY	CODE	INSTRUMENT SERIAL NO.	M	CODE	ROD SERIAL NO.	CODE	ROD SERIAL NO.		Z	TIME
·41·	801026	233	456511	H	316	2345673	16	234568	COLLIMATION CHECK EXAMPLE		

FROM	BM DESIGNATION	TO	BM DESIGNATION	Z	TIME BEGIN	TIME END	F/C	TEMPERATURE BEGIN	TEMPERATURE END	W	S	OBSERVER
	10-40 METHOD			R	1130	1205	C	211	220	0	0	CWS

SET UPS	STADIA BACK	Ss Σ	STADIA FORE	Sf Σ	LOW SCALE BACK/FORE	Δh_L Σ	HIGH SCALE BACK/FORE	Δh_H Σ	$\Delta h_L - \Delta h_H$	REMARKS
1	301	10.2	306	40.0	B 301.37	− 4.67	B 893.75	− 25.31	− .36	− 0.710 hcm $\Delta h_1 - \Delta h_2$
	290.8		266.0		F 306.04		F 919.06		÷	− 59.6 m $\Delta s_1 + \Delta s_2$
							d = +21		Q =	+ 0.012 hcm/m
2	302	10.0	298	39.8	B 302.59	+ 4.43	B 916.21	+ 25.08	+ .35	
	292.0	20.2	259.2	79.8	F 298.16	− 0.24	F 891.13	− 0.23	− .01	

$\Delta s_1 + \Delta s_2 = -59.6$ −21 $\Delta h_1 + \Delta h_2 =$ − 0.235 hcm

X 5 mm/hcm

− 1.175 mm $\Delta h_1 + \Delta h_2$

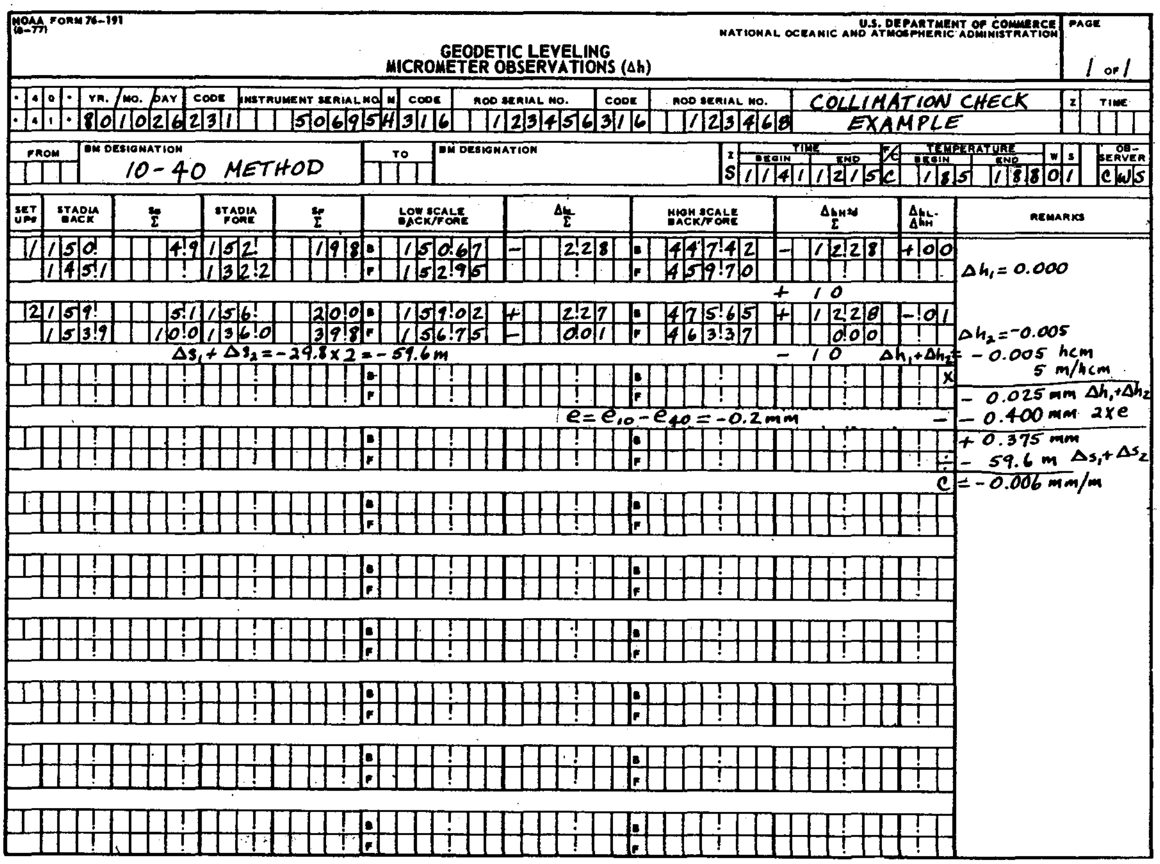
$e = e_{10} - e_{40} = -0.2$ mm − − 0.400 mm 2×e

− 0.775 mm

÷ − 59.6 m $\Delta s_1 + \Delta s_2$

C = + 0.013 mm/m

Figure 3-48.—Collimation check, 10-40 method, micrometer procedure, half-centimeter rods, reversible compensator.



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GEODETIC LEVELING
MICROMETER OBSERVATIONS (Δh)

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·40·	YR./MO./DAY	CODE	INSTRUMENT SERIAL NO.	M	CODE	ROD SERIAL NO.	CODE	ROD SERIAL NO.		Z	TIME
·41·	801026	231	50695	H	316	123456	316	123468	COLLIMATION CHECK EXAMPLE		

FROM	BM DESIGNATION	TO	BM DESIGNATION	Z	TIME BEGIN	TIME END	F/C	TEMPERATURE BEGIN	TEMPERATURE END	W	S	OBSERVER
	10-40 METHOD			S	1141	1215	C	185	188	0	1	CWS

SET UPS	STADIA BACK	Ss Σ	STADIA FORE	Sf Σ	LOW SCALE BACK/FORE	Δh_L Σ	HIGH SCALE BACK/FORE	Δh_H Σ	$\Delta h_L - \Delta h_H$	REMARKS
1	150	4.9	152	19.8	B 150.67	− 2.28	B 447.42	− 12.28	+ 00	
	145.1		132.2		F 152.95		F 459.70			$\Delta h_1 = 0.000$
							+ 10			
2	159	5.1	156	20.0	B 159.02	+ 2.27	B 475.65	+ 12.28	− .01	
	153.9	10.0	136.0	39.8	F 156.75	− 0.01	F 463.37	0.00		$\Delta h_2 = -0.005$

$\Delta s_1 + \Delta s_2 = -29.8 \times 2 = -59.6$ m − 10 $\Delta h_1 + \Delta h_2 =$ − 0.005 hcm

X 5 m/hcm

− 0.025 mm $\Delta h_1 + \Delta h_2$

$e = e_{10} - e_{40} = -0.2$ mm − − 0.400 mm 2×e

+ 0.375 mm

÷ − 59.6 m $\Delta s_1 + \Delta s_2$

C = − 0.006 mm/m

Figure 3-49.—Collimation check, 10-40 method, micrometer procedure, centimeter rods, nonreversible compensator.

Systematic change in the collimation error, resulting from consistent variation in the response of the compensator to gravity, is limited by strictly following the micrometer leveling procedure. The instrument is leveled while pointing in one direction, and then releveled while pointing in the opposite direction. The releveling is omitted if the compensator can be mechanically repositioned, as is the case with an instrument having a reversible compensator.

However, a compensator may "hang" or "stick" in such a way that releveling or repositioning may not remove all of the compensation error that is introduced (fig. 3-5). With a nonreversible compensator, the observer should lightly tap the side of the instrument or tripod leg during each pointing to make sure that the image oscillates. This indicates that the compensator is freely suspended. In addition, a compensation check should be performed weekly.

With a reversible compensator, whenever the compensator is repositioned the observer should check that the image oscillates. Compensation error in such an instrument results in a lack of symmetry in the two lines of sight. The error is revealed if the mean collimation factor exceeds the tolerance during the collimation check; therefore, a separate compensation check is not necessary.

Instructions. The form commonly used for this test is NOAA Form 77-81, "Geodetic Leveling Compensation Check." The form is designed for use with the Nil leveling instrument and half-centimeter rods. An example is given in figure 3-51. The following instructions apply to any compensator instrument with a micrometer, used with double-scale rods. If a computer program is not used, prepare a standard recording form as illustrated in figures 3-52 and 3-53.

1. Make sure that the circular levels on the instrument and the rods are properly adjusted before beginning this test. (See sec. 3.3.5, "Circular-level adjustment " and 3.4.3, "Checking the circular level.")

2. Set up a leveling rod 40 m from the instrument. For convenience, use rod 2 from the last setup of the collimation check.

3. Point the instrument at the rod. Bring the bubble in the circular level to position "a," where the left

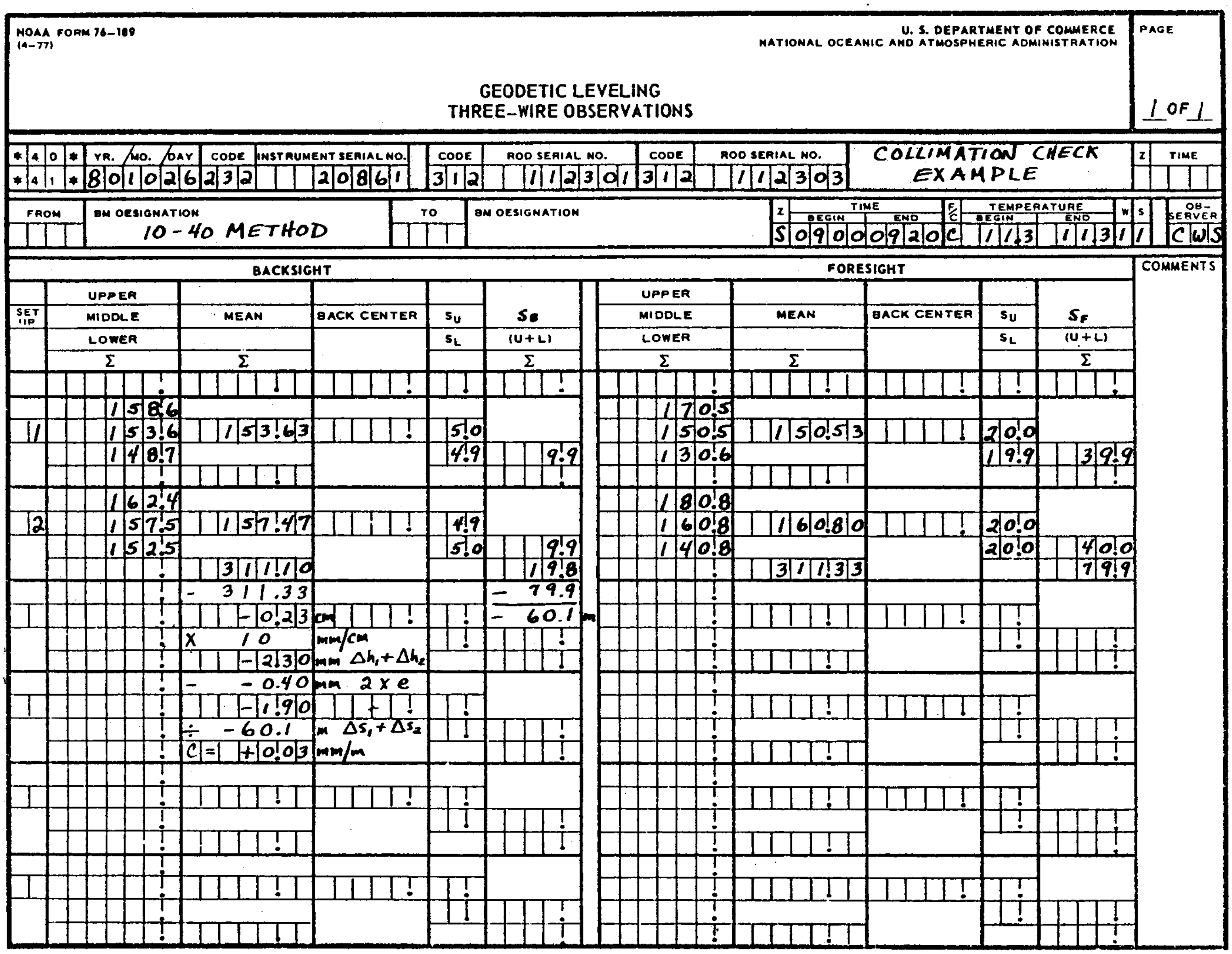

Figure 3-50.—Collimation check, 10-40 method, three-wire procedure, centimeter rods.

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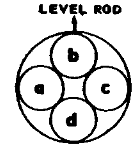
GEODETIC LEVELING COMPENSATION CHECK EXAMPLE

LEVEL ROD



BUBBLE POSITION

INSTRUMENT: Zeiss Ni 1/ 20816

WIND	SUN	TIME	DATE	ROD SERIAL NUMBER
0	2 %	0950	80/10/29	123468

If tan α or tan β < 0.0000097 the compensator is in good adjustment.

POSITION OF BUBBLE	HALF STADIA	ROD LEFT HALF cm	ROD RIGHT HALF cm	HALF STADIA INTERCEPT (HSI)
a	288	288.722	881.212	43.4
	2446	.700	.237	
	a	288.711	881.224	
	c	288.760	881.294	
	a - c	-0.049	-0.070	

Mean $(a\text{-}c)_{cm} = \dfrac{(a\text{-}c)_{left}+(a\text{-}c)_{right}}{4} = -0.030$ cm

S_{cm} = HSI x Stadia ~~Constant~~ Factor = 4340 cm

$\tan\alpha = \dfrac{a\text{-}c_{\,cm}}{S_{cm}} = \dfrac{-0.030}{4340} = -0.0000069$

POSITION OF BUBBLE	ROD LEFT HALF cm	ROD RIGHT HALF cm
c	288.750	881.305
	.769	.282
	288.760	881.294

REMARKS (OBSERVER/RECORDER)

POSITION OF BUBBLE	HALF STADIA	ROD LEFT HALF cm	ROD RIGHT HALF cm	HALF STADIA INTERCEPT (HSI)
b	288	288.937	881.455	43.3
	2447	.922	.436	
	b	288.930	881.446	
	d	288.484	881.004	
	b - d	+0.446	+0.442	

Mean $(b\text{-}d)_{cm} = \dfrac{(b\text{-}d)_{left}+(b\text{-}d)_{right}}{4} = +0.2220$ $S_{cm} = 4330$

$\tan\beta = \dfrac{b\text{-}d_{\,cm}}{S_{cm}} = \dfrac{+0.2220}{4330} = +0.0000513$

POSITION OF BUBBLE	ROD LEFT HALF cm	ROD RIGHT HALF cm
d	288.497	881.007
	.470	.000
	288.484	881.004

REMARKS (OBSERVER/RECORDER)
Instrument must be adjusted.

Figure 3-51.—Compensation check for the Ni1 and half-centimeter rods.

edge of the bubble is just under the black line of the left edge of the circle.

4. Read the low scale of the rod. Estimate to one-thousandth of a rod unit (the third decimal place) on this and all subsequent scale readings during the test. Record the reading in column C. Record the integer portion in column A.

5. Read a stadia intercept. Record it in column A and compute the half stadia interval, HSI, in column B.

6. Read the high scale of the rod twice (two pointings). Record the readings in column E. Compute the mean in column F.

7. Read the low scale again, record the reading in column C, and compute the mean of the two low scale readings in column D.

8. Bring the bubble in the spherical level to position "c," where the right edge of the bubble is just under the black line of the right edge of the circle. Repeat the rod readings in the same order (steps 4, 6, and 7), omitting stadia readings.

9. Convert rod units to millimeters. Compute the sighting distance, s, in meters. Compute tan α in millimeters per meter by the formula

$$\tan\alpha = [(a-c)_L + (a-c)_H]/2s.$$

Tan α is the maximum change that may be introduced into the collimation error when the compensator is suspended within the maximum left-right range of arc permitted by the circular level.

10. Tan β is observed and computed in the same way. Substitute bubble position "b" for "a" and bubble postion "d" for "c," and follow steps 3 through 9. Tan β is an expression for the maximum change that may be introduced into the collimation error when the compensator is suspended within the maximum forward-backward range of arc permitted by the circular level.

11. Both tan α and tan β must be less than ± 0.0097 mm/m (± 0.0000097). If this is not the case, the instrument should be adjusted at the repair shop or factory.

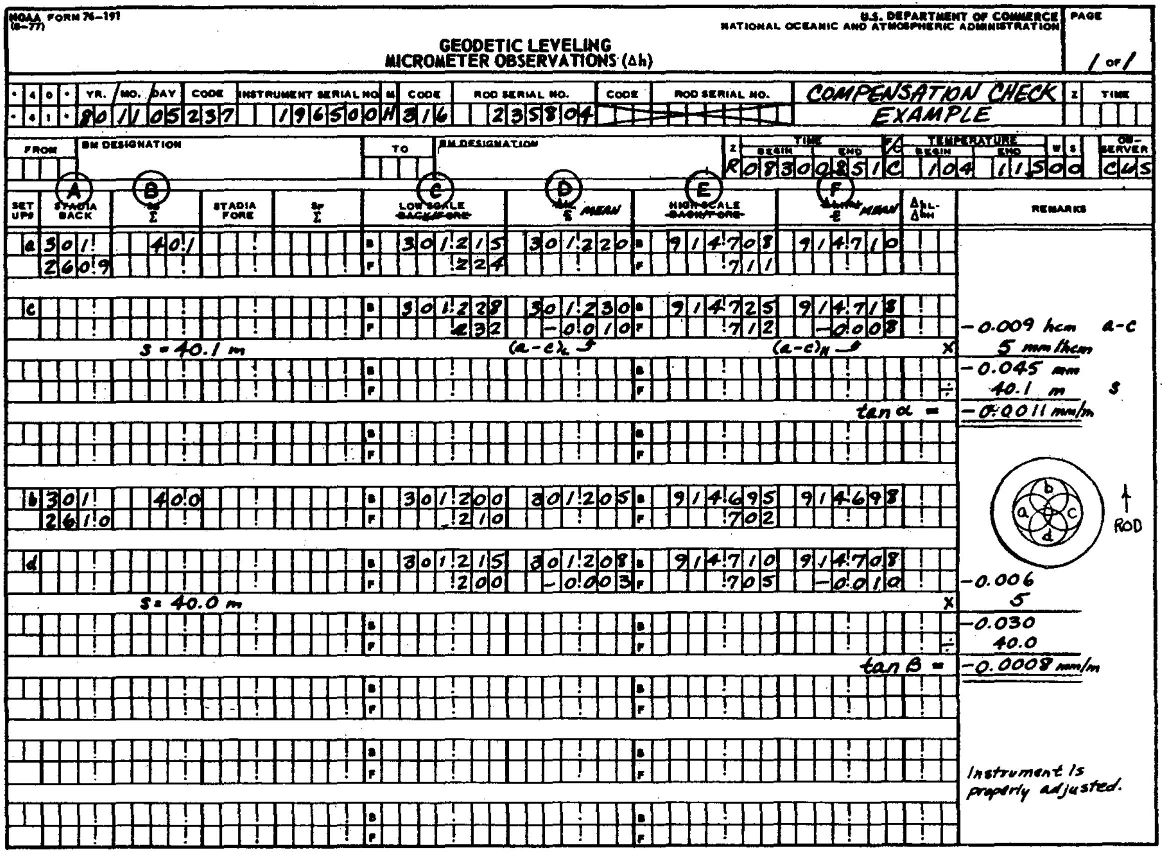
12. Submit data from the compensation check to the project office along with the leveling data acquired that day.

3.4 Leveling Rods

Leveling rods must be carefully designed, precisely manufactured, regularly calibrated, and properly used if they are to provide accurate heights above turning points and control points. In the following pages leveling rods are described, and the requirements for their use, calibration, and maintenance are presented.

3.4.1 Description

Leveling rods have often been made of seasoned hardwood, with a scale printed or stamped directly on the wood. Typically, such a rod is constructed of two or



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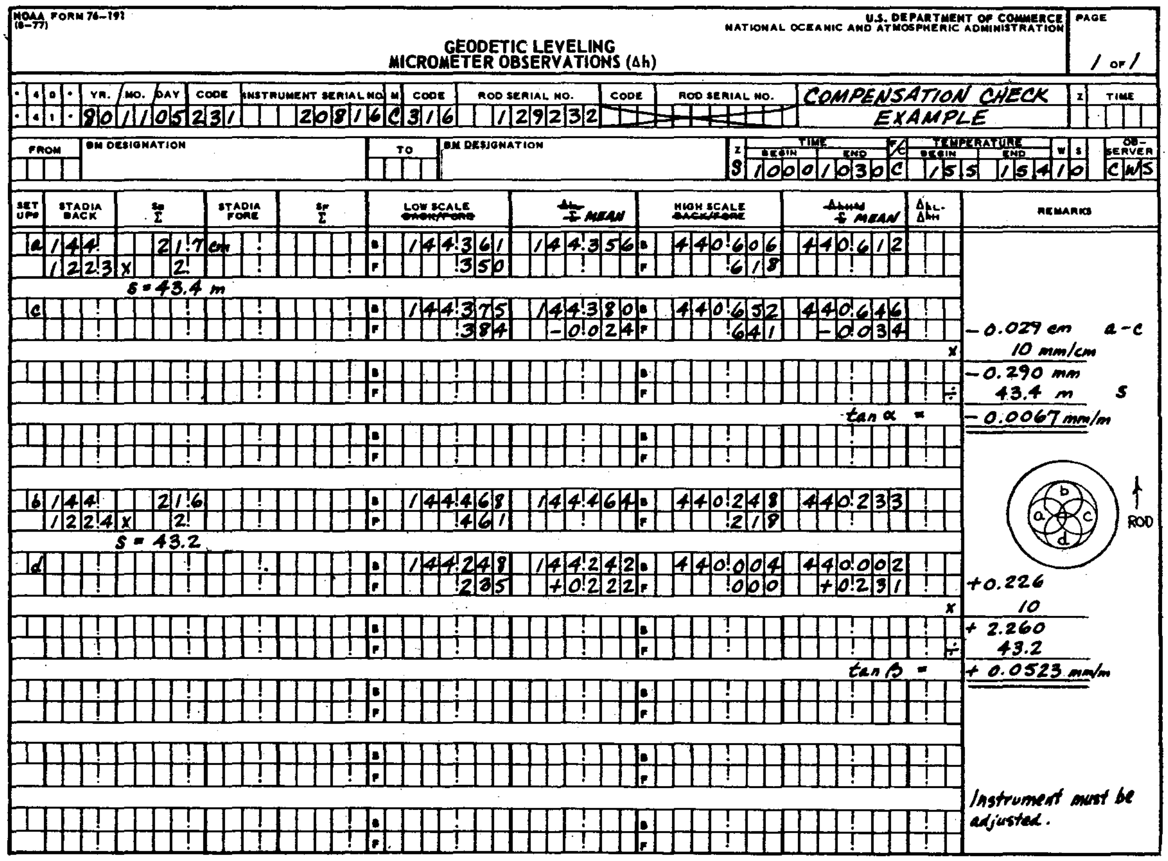
GEODETIC LEVELING
MICROMETER OBSERVATIONS (Δh)

	YR.	MO.	DAY	CODE	INSTRUMENT SERIAL NO.	M	CODE	ROD SERIAL NO.	CODE	ROD SERIAL NO.		Z	TIME
•41•	80	11	05	237	196500	H	316	235804			COMPENSATION CHECK EXAMPLE		

FROM	BM DESIGNATION	TO	BM DESIGNATION	Z	TIME BEGIN	TIME END	F/C	TEMPERATURE BEGIN	TEMPERATURE END	W	S	OBSERVER
				R	0830	0851	C	104	115	0	0	CWS

SET UPS	A STADIA BACK	B Σ	STADIA FORE	Σ		C LOW SCALE	D MEAN		E HIGH SCALE	F MEAN	$\Delta_{hL}-\Delta_{hH}$	REMARKS
a	301	401			B	301215	301220	B	914708	914710		
	2609				F	224		F	711			
c					B	301228	301230	B	914725	914718		
					F	232	−0010	F	712	−0008		−0.009 hcm a−c
	s = 40.1 m						$(a-c)_L$			$(a-c)_H$	×	5 mm/hcm
					B			B				−0.045 mm
					F			F			÷	40.1 m s
										tan α =		−0.0011 mm/m
b	301	400			B	301200	301205	B	914695	914698		ROD
	2610				F	210		F	702			
d					B	301215	301208	B	914710	914708		
					F	200	−0003	F	705	−0010		−0.006
	s = 40.0 m										×	5
					B			B				−0.030
					F			F			÷	40.0
										tan B =		−0.0008 mm/m
												Instrument is properly adjusted.

Figure 3-52.—Compensation check, micrometer leveling, half-centimeter rods.



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GEODETIC LEVELING
MICROMETER OBSERVATIONS (Δh)

	YR.	MO.	DAY	CODE	INSTRUMENT SERIAL NO.	M	CODE	ROD SERIAL NO.	CODE	ROD SERIAL NO.		Z	TIME
•41•	80	11	05	231	20816	C	316	129232			COMPENSATION CHECK EXAMPLE		

FROM	BM DESIGNATION	TO	BM DESIGNATION	Z	TIME BEGIN	TIME END	F/C	TEMPERATURE BEGIN	TEMPERATURE END	W	S	OBSERVER
				S	1000	1030	C	155	154	1	0	CWS

SET UPS	STADIA BACK	Σ	STADIA FORE	Σ		LOW SCALE	MEAN		HIGH SCALE	MEAN	$\Delta_{hL}-\Delta_{hH}$	REMARKS
a	144	21.7 cm			B	144361	144356	B	440606	440612		
	1223 ×	2			F	350		F	618			
	s = 43.4 m											
c					B	144375	144380	B	440652	440646		
					F	384	−0024	F	641	−0034		−0.029 cm a−c
											×	10 mm/cm
					B			B				−0.290 mm
					F			F			÷	43.4 m s
										tan α =		−0.0067 mm/m
b	144	21.6			B	144468	144464	B	440248	440233		ROD
	1224 ×	2			F	461		F	218			
	s = 43.2											
d					B	144248	144242	B	440004	440002		
					F	235	+0222	F	000	+0231		+0.226
											×	10
					B			B				+ 2.260
					F			F			÷	43.2
										tan β =		+ 0.0523 mm/m
												Instrument must be adjusted.

Figure 3-53.—Compensation check, micrometer leveling, centimeter rods.

more telescoping sections. The overall length of the scale on a wooden rod changes significantly with normal variations in atmospheric humidity. Furthermore, when assembling two or more telescoping sections, mismatches at the junctions introduce further errors in the scale. Therefore, for geodetic leveling, a much more precise rod is required. The elements essential to such a rod are described next.

Scale. The scale should be a continuous (not collapsible or folding) metal strip with a small coefficient of thermal expansion. Steel-nickel alloy (Invar) is a suitable, commonly used metal for this purpose. A scale length of 3 m (10 ft) is efficient when the height of the line of sight is expected to average from 1.5 to 2.0 m (4.9-6.6 ft). A 3.5 m (11.5 ft) rod may be preferable if the line of sight averages 2.0 to 2.5 m (6.6-8.2 ft), as in the case when observations are made from a vehicle in motorized leveling. The length of the scale sets an upper limit on the amount of scale-related error that may be accumulated in a setup.

Graduations must be accurately marked on the scale during its manufacture, and the scale must be calibrated regularly thereafter. The intervals indicated by the graduations, termed rod units, should be no larger than 1 cm (0.03 ft). Half-centimeter or centimeter units are preferred for geodetic leveling, although "centifeet" (0.01 ft) and "centiyards" (0.01 yd) have been used. The units must be compatible with the units of the micrometer in the leveling instrument. (See sec. 3.3.4, "Optical micrometer.")

The scale or its housing is usually labeled with numbers corresponding to tens of rod units. To obtain a reading, the observer must count the rod units between the point where the line of sight intercepts the scale and the first labeled number below that point. The reading is the sum of the labeled number and the number of rod units counted. To prevent blunders as a result of miscounting, every graduation could be labeled. This is possible with centimeter rods, but not with half-centimeter rods since the numbers are too small to be read at the usual sighting distances. The orientation of the labeled numbers must be compatible with the instrument, depending on whether the image provided by the instrument is upright or inverted. An inverted scale, for use with an inverting instrument, is pictured in fig. 3-54).

The type of scale must be compatible with the leveling procedure. For micrometer leveling, two parallel scales with line graduations are necessary on each rod. Line graduations are lines of finite height (0.8 to 1.6 mm), the centerlines of which define their actual positions on the scale. When intercepted with the wedge reticle (sec. 3.3.5, "Circular level adjustment"), precise readings can be made without estimation. The parallel scales are marked on the same metal strip, one offset slightly from the other, so a constant relationship exists between them even though the overall length of the scale may change. The graduations are labeled so the base of the low scale is zero, and the corresponding point on the the high scale is about 300 cm (600 hcm). The resulting offset, in rod units, is termed the rod constant. (See fig. 3-55.)

The pair of double-scale rods must be unmatched; each rod must have a different rod constant. This permits a check to detect observations that are transposed during a setup. The rod with the smaller constant is always labeled rod 1. It should be marked with a piece of flagging or reflective tape so it can be easily recognized.

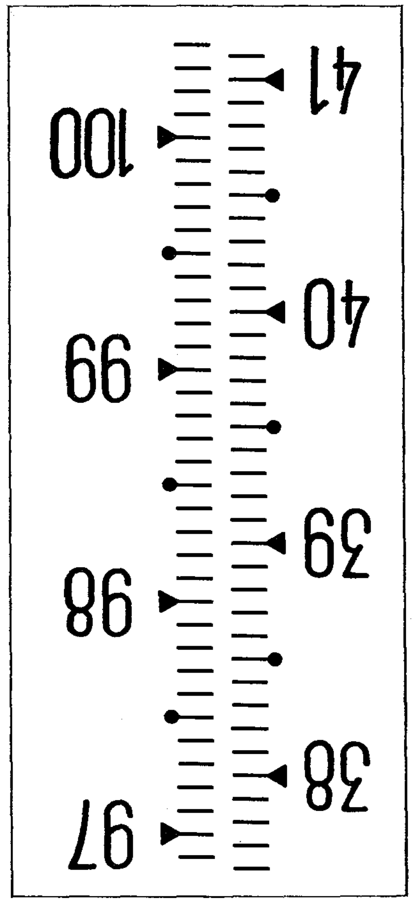


Figure 3-54.—Inverted scale for a leveling rod.

Because the scales are independent of the rod housing on which the graduation values are labeled, the rod constant may be changed if necessary. If this is done, a record must be included in the observation data specifying the date of change and the new constant. For example, if both rods of a pair have identical rod constants of 592.50, one constant can be changed to 602.50 by affixing a metal strip over the high-scale graduation values marked on the housing. The strip should be engraved with values increased by one ("60" becomes "61," etc.), to obtain the desired 10 unit change in the constant.

For three-wire leveling, rods must have single scales and block graduations. In addition, each rod should have a check scale in different units on the back. Block graduations are the borders between rectangular spaces, which usually alternate black and white in color. Each block is of a height equal to one rod unit. This type of

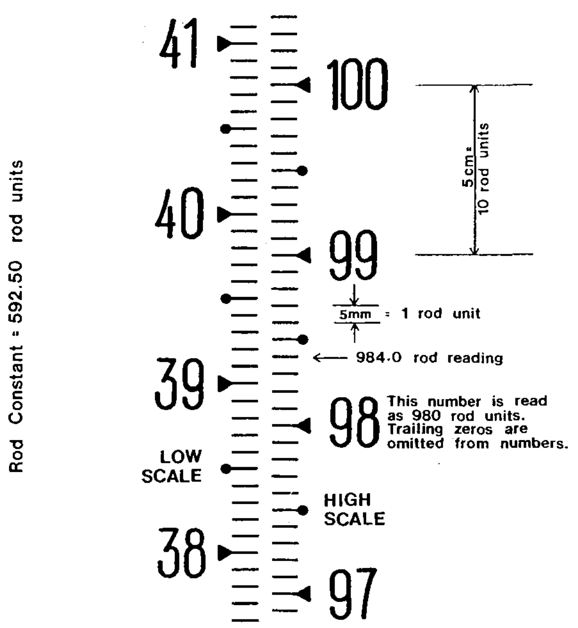


Figure 3-55.—Double scale with half-centimeter line graduations.

graduation permits the intercept of the line of sight to be estimated to one-tenth of a rod unit. Black blocks must not be thought of as black graduations on a white background.

Figure 3-56a shows a type of block graduation (known as the Philadelphia graduation) commonly used in engineering surveys, with a rod unit of 0.01 ft. At short sighting distances, readings may be estimated to 0.001 ft. Figure 3-56c shows a similar type of graduation, in which the unit is 0.01 yd.

Figure 3-56b shows the block graduation devised and used by the U. S. Coast and Geodetic Survey from 1916 to 1962. The unit is 1 cm, but the rod appears to have two adjacent, parallel scales. In each centimeter, there are a black block and a white block side by side, causing a checkerboard appearance. The purpose of this scale is to make a white block available, in every centimeter, against which the black reticle line shows distinctly, thus facilitating estimation of tenths of units. This scale must not be confused with the offset parallel scales used for the micrometer-leveling procedure.

Housing. The housing of a precise leveling rod is made of durable metal or seasoned hardwood. It provides a frame in which the scale may be suspended under constant tension, independent of expansion or contraction of the housing. Padding between the edges of the scale and the housing prevents wind-induced vibration and the resulting wear on the scale and decrease in reading precision.

A base plate ("footpiece") made of noncorrosive metal should be an integral part of the housing. It should be designed so that the rod may be placed easily on the various types of control points. The bottom surface of the base plate should correspond to the zero of the scale. Typically, the scale is attached rigidly at the foot of the rod and attached flexibly at the top, by a spring-loaded clamp.

Circular level. A circular level should be mounted on the back of the housing, where it can be readily observed by the rodman. It should have a sensitivity of 10′.0 or less per 2 mm displacement of the bubble. Thus, the rod may be plumbed within 10′.0 (equivalent to 1.0 cm at the top of a 3-m rod) of the direction of gravity. A mirror or reflecting prism placed above the level facilitates centering.

Brace poles. The rod should be equipped with a pair of adjustable length poles, attached by swivel joints to the center of the top of the rod (fig. 3-57). The pole assembly should be lightweight, but it must be sufficiently strong to withstand the stresses of bracing the rod against the wind. Mounting the assembly at the top center permits the rod to be pivoted on the turning point between setups, without moving the poles. (If the circular level is not properly adjusted, the fact should become evident while pivoting the rod.) The use of brace poles minimizes swaying and unsteadiness of the rod, allowing it to be plumbed more precisely.

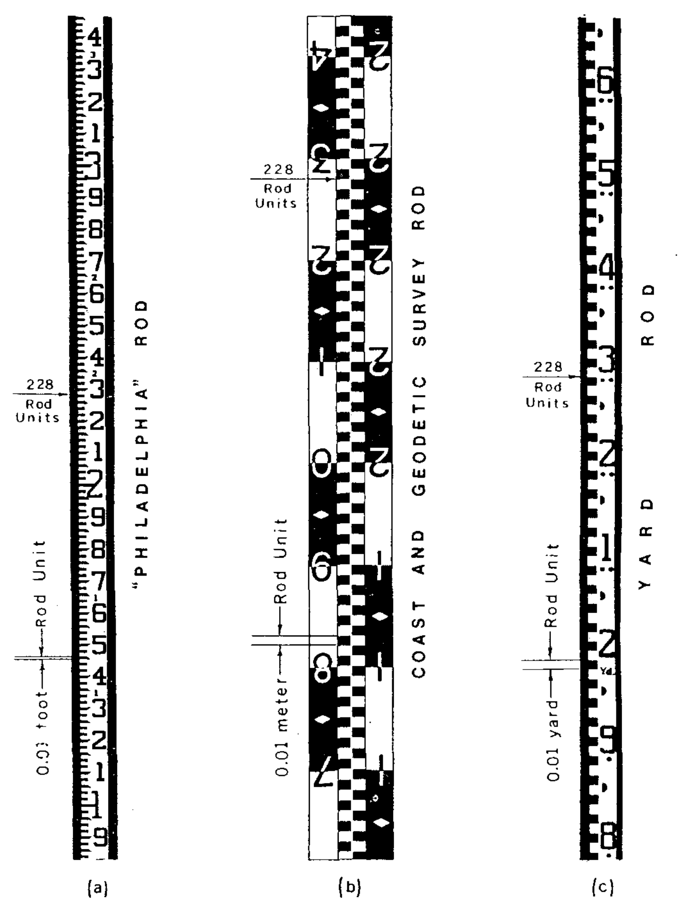


Figure 3-56.—Leveling rods with block graduations.

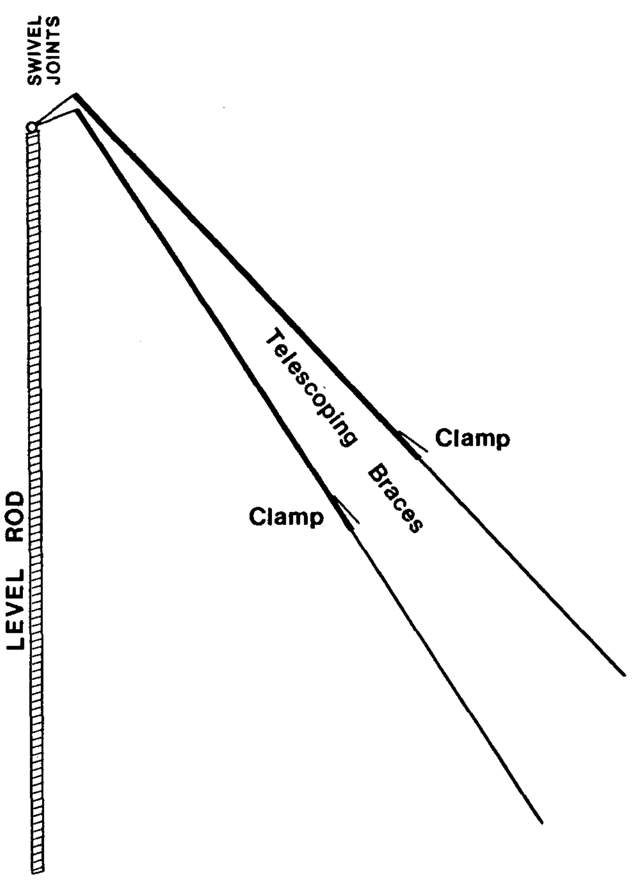


Figure 3-57.—Brace poles for a leveling rod.

3.4.2 Calibration

The rods used for geodetic leveling must be calibrated routinely by a reputable laboratory against a standard that has been compared to the National Standard of Length. A complete set of calibration data should be obtained for each rod before the rod is first used, every 5 years during its use, and just before the rod or scale is retired. The data should include the index error, length excess, and coefficient of thermal expansion. In addition, during each year of use, at least one calibration should be made to determine the index error and length excess. If a rod is dropped, or otherwise handled in a way that may have damaged it, it should not be used again until it has been recalibrated.

Each calibration consists of the precise measurement of at least four overlapping intervals on the scale (usually between the base plate and the bisected graduations at 0.2, 1.0, 2.0, and 3.0 m). The measurements, with respect to the National Standard of Length, should be accurate to ± 0.05 mm.

From one calibration a value for the index error and a factor for the length excess can be computed. The differences between the calibrated and assigned lengths of the graduations are plotted versus the assigned lengths (fig. 3-58). The intercept of the plotted line with the axis of the differences is the index correction which is equal and opposite in sign to the index error. The slope of the line is the length excess.

To obtain a complete set of calibration data, at least four calibrations must be made, each at a different temperature between 15° and 35°C. The variations in graduation values due to changes in temperature are described by the coefficient of thermal expansion, computed from these data. To verify the amount of scale offset, at least one additional calibration of the high scale of a double-scale rod is required.

Calibration with a laser interferometer. A very accurate method for calibrating the rod scale is employed by the National Bureau of Standards, using a laser interferometer. All leveling rods used by the National Geodetic Survey should be calibrated by this method at least once during each year of use. Instead of values for four intervals, a calibrated value is obtained for every graduation on the scale. With this information, observed heights may be converted directly to "calibrated" heights.

Submitting calibration data. The complete set of calibration data must accompany any leveling data submitted to the National Geodetic Survey for inclusion in the national network. Formats for the NGS "Leveling Instrument and Rod File" are presented in *Input Formats and Specifications of the National Geodetic Survey Data Base* (Pfeifer and Morrison 1980: vol. II, ch. 6).

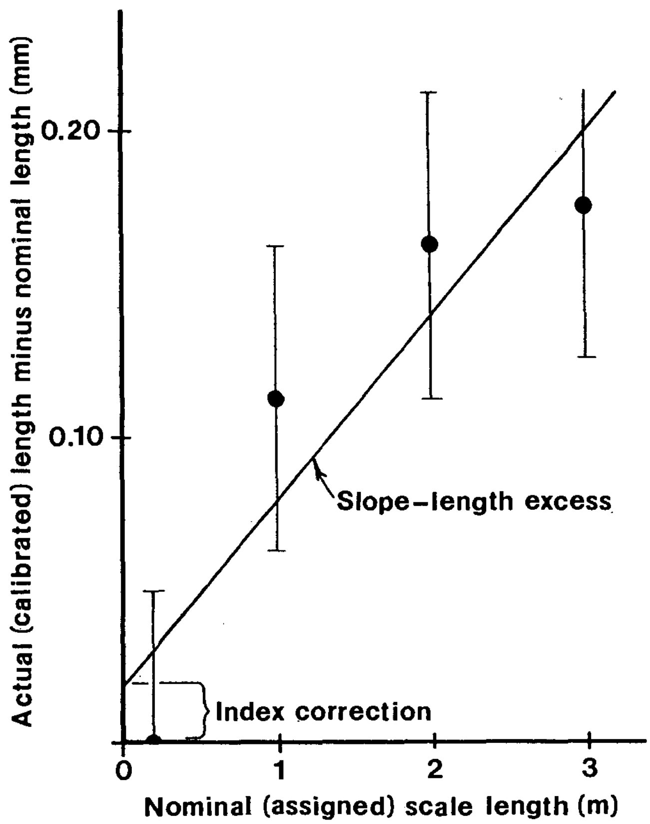


Figure 3-58.—Rod calibration data.

Calibration data should include the actual measurements and the temperatures at which they were made for each rod used. All data should be identified by the serial number of the rod. If a rod housing is reused with a new scale, the rod should be assigned a new serial number and recalibrated. The same applies if a scale is reused in a new rod housing.

3.4.3 Use and Maintenance

The leveling rod, like the leveling instrument, must be properly maintained if good quality data are to be collected. The rodman should guard the rod against physical damage and protect it from the effects of exposure to the outdoor environment. The guidelines and adjustment procedures given in this section should be strictly followed.

General handling. Because of its length, the leveling rod is so easily struck against objects that great care must be exercised when handling it. Paint does not stick easily to the special materials used for rod scales. As a result, striking the rod against any hard object (such as poles, trees, signs, rocks, vehicles, or the ground) may chip paint off the scale, making it difficult to continue observing precisely and efficiently. Such shocks may also damage the inner spring assembly or the housing, causing unpredictable change in the length of the scale.

Carry the rod by holding it with one hand at the middle and balancing it face up across your shoulder (fig. 3-59). Do not touch the painted scale. Use your arm as a spring to absorb shocks. When riding on the rear step of a truck, hold the rod parallel to the side of the truck, taking care that it does not hit the truck or any obstacles along the leveling route.

During a halt in the leveling operation, hold the rod vertically on the toe of your boot. Otherwise, place the rod in a protected location where it can be easily seen. Do not place it near the path of a vehicle. Lay it gently on the ground, with the scale facing up and the circular level protected from stress. See that no dirt or rocks are kicked on the rod while it is on the ground.

Figure 3-59.—Carrying the leveling rod.

At the end of work each day, gently wipe the rod to remove moisture and dirt. Store it, supported firmly by padding, in a box large enough to accommodate a pair of rods with brace poles and turning point guides attached. Protect the circular levels and any attached mirrors with padding or the rod handles (which may fold over the attachments).

Setting up the rod. Since the base plate corresponds to the zero point of the scale, it must be kept perfectly clean and free from corrosion if precise heights are to be observed. Before setting the rod on a point, carefully wipe away any dust or dirt on the base plate and the point. Do not allow the base plate to rest directly on the ground, since scratches may result. A rod with a defective or damaged base plate should be repaired and calibrated.

Set the rod gently onto the point; never drop it. Dropping a rod on a turning point or control point is comparable to hitting the point with a 7-kg sledgehammer. Even the best quality bench mark may suffer from such treatment.

Place the rod so that the exact center of the base plate rests on the highest point of the turning point or control marker. (See fig. 3-76 for the locations of control points on various types of monuments.) The National Geodetic Survey uses a turning-point guide ("flipper"), attached to the bottom of the rod, to assist with this centering (fig. 3-60). The guide is designed to fit around the standard turning point. If the rod is set against a flatter, broad surface, such as a bench mark disk, the guide flips back out of the way, exposing the entire base plate.

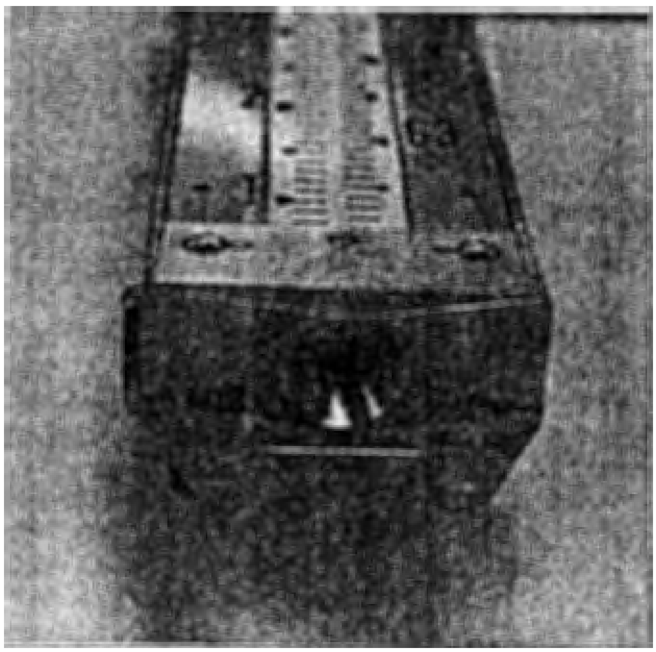


Figure 3-60.—Turning-point guide.

Be alert to the possibility that the rod can be set down in such a way that some part of the turning-point guide, instead of the base plate, is the contact point. Prevent this by turning and jiggling the rod after placing it on the turning point. A visual examination of the point of contact may be necessary. The guide can also "hang up" inside the protective 4- or 5-inch pipe which encircles many bench marks. When setting the rod on a mark of this type, remove the turning-point guide by loosening the two finger screws.

Always place the brace poles downwind from the rod, and arrange them so they do not interfere with the observer's line of sight. Face the rod toward the instrument and plumb it by adjusting the lengths of the poles until the bubble in the circular level is centered precisely. After completing one setup, without moving the poles, rotate the rod to face the instrument at the next setup. Do not remove the rod from the turning point at any time during this cycle of two setups. If necessary, replumb it by adjusting the poles again.

Lightly hold both poles throughout each setup. Stand where the circular level can be seen to ensure that the rod is plumb, and always be ready to prevent the rod from falling (fig. 3-61). Do not press down on the poles or lean against them, since this can cause movement of the turning point.

Figure 3-61.—Setup of leveling rod.

The top assembly of the brace poles must withstand a great deal of stress during leveling. Examine it regularly for loose, bent, or broken parts. The large screw, which secures the assembly to the top of the rod, may loosen and its shaft may bend if it is subjected to strain. Prevent this by checking at least once a day to see that the screw is tight. Do not twist the poles, since this may bend the fittings which connect the poles to the top assembly. The grips lock the poles at the desired length by means of friction. They may become smooth with use and should occasionally be taken apart and roughened with sandpaper.

Checking rod plumb. During a setup each leveling rod must be plumb, in other words, vertically aligned with the direction of gravity. Achieve this by centering the bubble in the vial of the circular level attached to the rod housing. To center the bubble precisely, use a mirror or reflecting prism mounted on the rod to observe the level from as nearly overhead as possible, not from the side.

Each day, while leveling, the observer should check the plumbing of each rod by comparing the alignment of each rod to the vertical line in the reticle of the leveling instrument. Check the rod while it faces the instrument; then, after the rodman turns the rod 90° (at a right angle to the instrument), check it again. This procedure quickly reveals gross errors in the plumbing of the rod.

If the circular level is not properly adjusted, centering the bubble does not ensure that the rod is plumb. Check the level by one of the following methods at least once a week and immediately after any shock to the rod. Include a note in each weekly report, stating the time and date that the check was performed, and explaining the adjustments required.

If the level cannot be adjusted within tolerance, the rod housing may be warped or the level may be defective. In this case, check the housing by stretching a string from one end of the rod to the other and looking for bends or twists. If the rod has such defects, replace it immediately. If not, replace the circular level.

Checking the circular level. The simplest method for checking the adjustment of the circular level is the brace-pole method. To use this method, the rod must be equipped with a turning-point guide and the brace poles must be in good condition. In particular, the shaft of the large screw which centers and secures the poles to the top of the rod must be straight. If this is not the case, three guy wires, attached to the top center of the rod, may be used in lieu of the brace poles.

Set the rod on a turning point and precisely center the bubble by adjusting the length of each pole or guy wire. Lock the poles and slowly rotate the rod 360°. The bubble should not move more than 2 mm (0.08 in) from the center, a distance which represents an angular deviation from vertical of 10.'0 or less. If the bubble moves more than this distance, adjust it in the same way as the one on the leveling instrument. Follow the instructions in section 3.3.5, "Circular-level adjustment," and adjust the poles or guy wires as though they were the tribrach foot screws of the instrument.

Another method for checking the circular level is the suspension method. Begin this method by suspending the rod from an eave or other similar structure with a short piece of strong line fixed at the top center of the rod. Anchor a piece of paper under the rod. (The back of the weekly report form is suitable.) The check and adjustment follow:

1. Plumb the rod, with the base plate resting against the paper. Trace around the front and sides of the base plate.
2. Without moving the paper, rotate the rod to face the opposite direction. Plumb it again and retrace the front and sides of the base plate. The two tracings should be parallel. If not, the rod has not been rotated completely.
3. Measure the distance between the corners of the tracings (fig. 3-62). For a 3-meter rod, if the distance is more than 2 cm (0.8 in), the circular level should be adjusted.

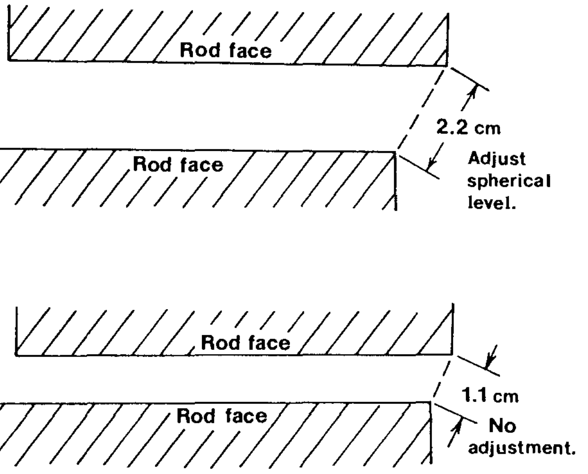


Figure 3-62.—Tracings for checking the adjustment of the circular level.

4. Before adjusting the level, move the base plate of the rod halfway back toward the first tracing and hold it there.
5. Center the bubble in the level by adjusting the small screws supporting the vial with an adjusting pin. Always turn the screws downward, to maintain pressure on the base washer. Do not retract one screw and depress another. If more adjustment play is needed, retract all the screws and begin adjusting them downward again. Do not force the vial tight against its base.
6. Turn the rod back to the original direction and perform the test again.
7. Submit the tracings and measurements to the project office with the day's leveling data or with the weekly report.

3.5 Turning Points

A turning point is the temporary support on which a leveling rod is placed during a setup. The foresight point for one setup becomes the backsight point for the next, "holding" the elevation while the leveling instrument is moved between setups. Therefore, turning points must be stable, well defined, and properly spaced to measure accurately elevation differences from one setup to the next. The degree of care taken by the pacer and rodmen in setting and using turning points greatly affects the quality of the leveling. This is especially true in single-run leveling, since movement of the turning points between setups is not easily detected.

3.5.1 Selection and Use

A turning point should not settle or rebound. It should have a definite high point on which to hold the rod and should be easy to set and remove. Do not use objects such as stones, fire hydrants, railroad spikes and ties, or marks on the pavement. Instead, use a pair of standardized points, suitable for the terrain likely to be encountered. Any of the three types described in this section is acceptable. The most satisfactory of these for routine use, under a variety of conditions, is the turning pin. On concrete sidewalks and gravel, the turning plate is suitable. If the route crosses sandy or marshy ground (and the line cannot be relocated to avoid this), a wooden stake with a double-headed nail provides the most reliable turning point.

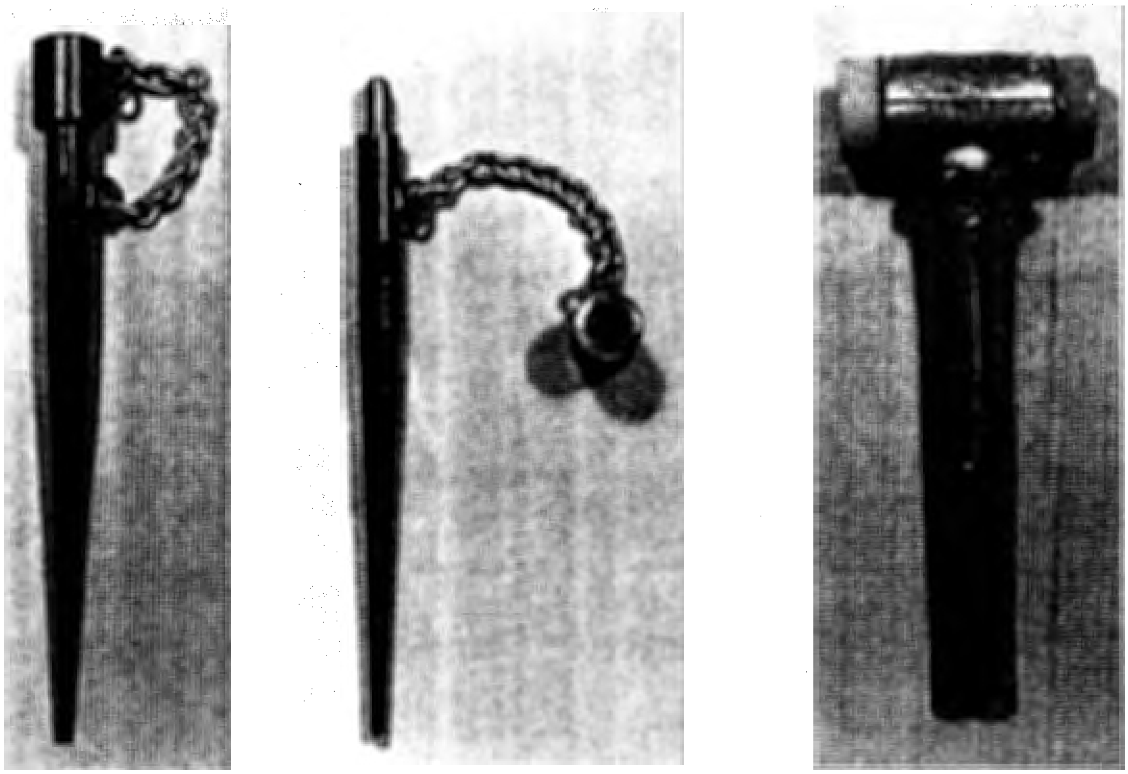
General instructions. When setting a turning point of any type, avoid loose soil, sand, or unpacked gravel. If necessary, dig away the loose layer with a shovel. Avoid frozen ground, but if it cannot be avoided, dig through the shallow frozen layer to unfrozen ground in which a turning point may be placed.

Once set, any turning point can settle, rebound, or be displaced surprisingly easily. Often these changes are not detectable. Be alert to the possibility of such movement by observing the following precautions: Do not drop the leveling rod onto the turning point. Do not remove the rod once it has been placed, since removing and replacing the rod can cause a significant change in the elevation of the point. Maintain a constant weight on the point by allowing the rod to stand free from downward pressure as much as possible. Do not walk around the point unnecessarily.

Turning pin. A turning pin and hammer for driving it are illustrated in figures 3-63 and 3-64.

To set a turning pin, place the driving cap over the high point and use a 2-kg (4 lb) hammer to drive the pin into the ground. The faces of the hammer should be made of replaceable soft plastic, to prevent damage to the driving cap. Do not hold the hammer with its head crossways when hammering, since this may damage the metal part of the head and make replacement of the faces difficult. Drive the pin straight, not at an angle, until the driving shoulder is within 5 cm (2 in) of the ground. If the pin is set at an angle, or not driven deeply enough, it is more likely to rebound or settle.

After setting the pin, remove the driving cap and carefully place the rod on the high point. After the setup has been checked, gently strike the side of the pin from



(a) (b)

Figure 3-63.—Turning pin (a) and hammer (b).

two directions. This should loosen the pin in the ground so it can be pulled "by hand." Avoid pulling the pin by tugging on its cap or chain.

Turning plate. When a turning pin cannot be driven efficiently, such as in the hard-packed gravel of a road shoulder, in very firm clay, or on a concrete sidewalk, use a turning plate (turtle). A turning plate should have (1) a definite high point, (2) three flat feet, and (3) weigh at least 7 kg (15 lb). Figure 3-65 shows a recommended design.

Exercise caution when setting the turning plate. Simply dropping it onto the ground will not provide enough stability. Stamp the turning plate firmly in place, even on concrete. Do not set it over small clumps of grass. Instead, use a shovel to clear a space, then tamp the plate down. When rotating the rod, maintain a constant weight on the plate, so it is not disturbed. Do not place your feet near the handle of the plate.

Wooden stake. In sandy or marshy ground or the loosely packed soil sometimes encountered on highway and railroad embankments, use a wooden stake. The stake is made from a piece of 2- by 2-inch wood cut 60-90 cm (2-3 ft) long. To use the stake as a turning point, hammer it into the ground to a firm depth, leaving about 5 cm (2 in) exposed. Then, drive a double-headed nail into the top, until the bottom head rests against the wood. The top of the nail serves as the turning point.

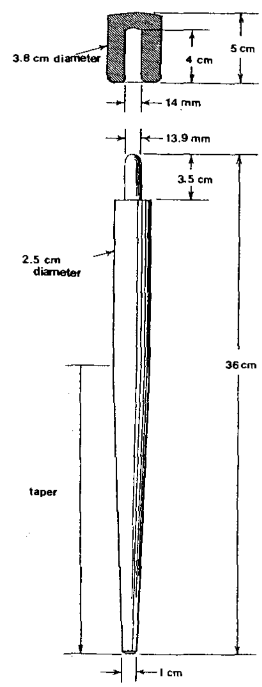


Figure 3-64.—Turning pin dimensions.

3.5.2 Pacing Balanced Setups

Many errors in leveling are reduced in magnitude by balancing setups. The distance from the leveling instrument to the foresight point must equal the distance from the instrument to the backsight point within the tolerance for the survey. When a pacer is a member of the leveling unit, he or she should lay out a balanced setup and set the foresight point before the rodman arrives from the previous setup. This permits leveling to proceed more efficiently than when rodmen set the turning points.

To lay out a balanced setup, count paces from the backsight point to the next position for the instrument and make a mark. Then, take an equal number of paces to the spot where the foresight point is to be set.

Paces need not be of any specific length; rather, they must be consistent. A beginning pacer (or rodman) should mark a 50-meter course on level ground and pace it several times, striving to develop consistency in the number of paces taken. Most individuals take paces about a meter in length; therefore, to make a change of less than 5 m in the length of a setup, .take the number of paces equivalent to the change.

Some individuals find that they routinely pace "long" (foresight distance longer than backsight distance) or "short" (foresight distance shorter than backsight distance). Learn to compensate for such tendencies, to prevent accumulating a large total of imbalances during a section of leveling. For this reason, do not move more than one setup ahead of the rest of the unit. If the observer or recorder alerts the pacer as soon as the total imbalance for the section exceeds the tolerance for one setup (see table 3-1), the pacer can reduce the total gradually instead of making one or two extremely unbalanced setups.

There are several ways to mark positions for the instrument and the turning points. If using lumber crayon ("keel"), draw a "$\wedge$" to show where the instrument should be placed and draw a line, circle, or "X" for the turning point. Do not use the crayon on buildings or

Figure 3-65.—Turning plate.

private property without explicit permission from the owner. Flagging may be useful for marking setups in brushy areas. In dirt or gravel, use a shovel or the heel of your boot to scrape a line marking the instrument setup and to clear a spot for the turning point. Do not use spray paint for marking setups. In general, the leveling unit should leave little or no evidence of its passing.

In addition to marking setups and setting turning points, use the instructions provided by the mark setting unit in the log to route the unit to the next control point. Consider both the observer's line of sight and terrain irregularities, such as slope and brush, when laying out the route. Slopes may require shorter sighting lengths than usual; a hand level is helpful when determining just how short to make them. To level to some points, it may be necessary to clear brush or hold branches away from the observer's line of sight. When an even number of setups are required between control points, the pacer must plan the final setups of a section carefully.

Four-person unit. When a leveling unit operates with four persons instead of five, the rodmen assume the tasks of pacing and setting their turning points. First, the observer paces to a position for the leveling instrument. Then the backsight rodman (from a previous setup) counts the number of paces from the last foresight point to the instrument. He or she takes the same number of paces to the next foresight point.

Specialized vehicles for motorized leveling have precise odometers with which to balance setups. Used carefully, the odometers can prevent the troublesome task of correcting a poorly paced, unbalanced setup. When advancing to the next setup, set the odometer at zero upon passing the backsight point, to count up the backsight distance. Upon passing the instrument, switch the odometer to reverse, to count down the distance to the foresight position.

When on foot or using a single truck, count regularly spaced objects such as railroad rails or highway markers to establish balanced setups.

Hand level. The hand level (fig. 3-66) is a small telescope in which a spirit level is mounted in such a way that, by means of a prism, the level may be viewed at the same time that an object is viewed through the telescope. When the bubble is centered in the spirit level, a reticle line indicates the horizontal line of sight.

When leveling up or down a slope, use the hand level to select the positions for the foresight rod and the instrument at the next setup. The following instructions apply to standard, 3-meter leveling rods. (See fig. 3-67.)

1. Begin at the foresight rod of the previous setup. When leveling uphill, point and level the hand level toward the next foresight position. Select a reference object intersected by the horizontal reticle line. Proceed to the object, counting paces. (When leveling downhill, pace a conservative distance to an estimated position for the next foresight.) This procedure limits the elevation

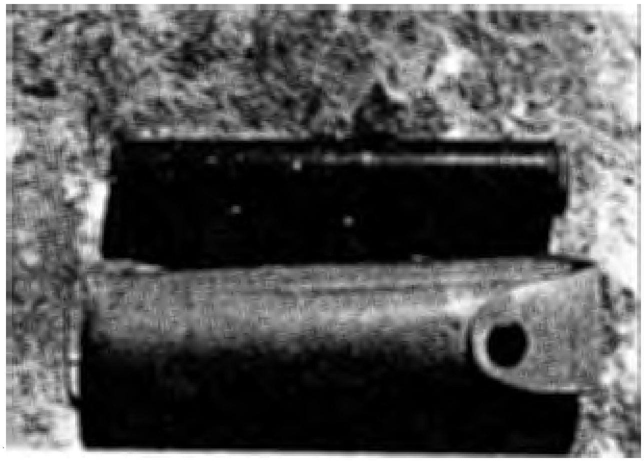


Figure 3-66.—Hand level.

difference for the setup to the height of eye of the pacer (1.6-1.8 m). With experience, the pacer can take a few extra paces to increase the elevation difference to a maximum of 2.0 m.

2. Check the foresight position by pointing and leveling the hand level toward the previous foresight rod. Looking downhill, the line of sight should pass no more than 0.5 m (1.6 ft) above the top of the rod. (Looking uphill, it should be just above the base of the rod.) Set the turning point.

3. Pace halfway back to a position for the instrument. Check it by pointing and leveling the hand level toward the previous foresight rod. Looking downhill, the line of sight should be below the top of the rod. (Looking uphill, it should be at least 0.5 m (1.6 ft) above the ground level at the rod.) Mark the position for the instrument. The pacer can omit this step and simply call out the necessary number of paces to the observer.

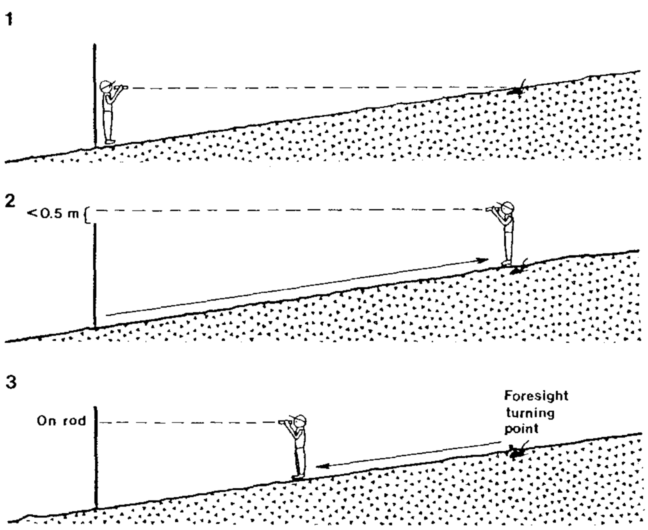


Figure 3-67.—Pacing a balanced setup on a slope.

3.6 Atmospheric Conditions

During geodetic leveling, atmospheric conditions must be determined for two purposes: to correct for the effects of atmospheric refraction on the line of sight and to correct for the effects of thermal expansion of the scales in the rods. Three measurements, recorded at the beginning and end of each section, can provide satisfactory corrections. These are air temperature, intensity of solar radiation, and wind speed. To correct more accurately for refraction, special equipment must be used to determine conditions at every setup. A method for measuring temperature differences, essential to many mathematical models of refraction, is presented here. Certain extreme atmospheric conditions may require special precautions or procedures when leveling. These are also discussed.

3.6.1 Air Temperature, Sun, and Wind

Measure the air temperature at the beginning and end of every section, as required for the observing procedure. To measure the air temperature, use a mercury thermometer with a range of at least −10° to 45°C (14° to 113°F), accurate to ±0.1°C (±0.2°F). Mount it rigidly in a shaded, protected place on the tripod (fig.3-68). If a pair of special sensors are available for measuring temperature differences, use the top sensor. Read and record the temperature to the nearest tenth of a degree.

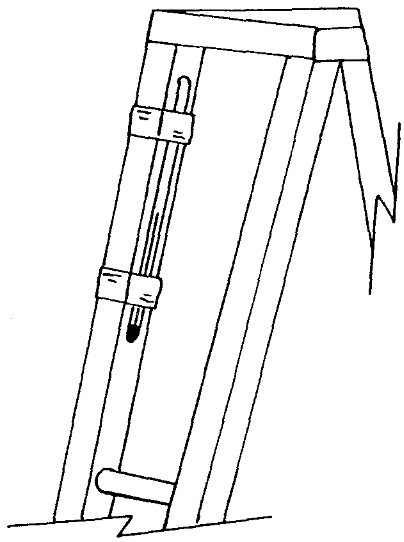


Figure 3-68.—Thermometer, mounted on tripod.

Record the intensity of solar radiation, as a single-digit sun code, at the beginning and end of every section. The sun code is defined as follows:

1. "0", if less than 25 percent of the setups are performed in sunny conditions.
2. "1", if 25 percent to 75 percent of the setups are performed in sunny conditions.

3. "2", if more than 75 percent of the setups are performed in sunny conditions.

If the day is clear and sunny, but all the setups of a section are made in the shade (in other words, the entire line of sight is shaded), the code is "0". If the sky is half covered with clouds, but all the setups are made in the sun, the code is "2".

Record the wind speed, as a single-digit wind code, at the beginning and end of every section. The wind code is defined as follows:

1. "0", if wind speed averaged less than 10 km/hr (6 mi/hr).
2. "1", if wind speed averaged 10 to 25 km/hr (6 to 15 mi/hr).
3. "2", if wind speed averaged greater than 25 km/hr (15 mi/hr).

Measuring air-temperature differences. The best mathematical models for computing refraction corrections depend upon measuring at least one air-temperature difference during every setup. Temperatures are measured at two different heights above the ground. The difference is computed by subtracting the temperature at the bottom from that at the top. It is usually negative during daylight hours when the Sun heats the ground, warming the lower air layers. It is positive when the air near the ground is cooler than that above it.

Use aspirated thermometers, accurate to $\pm 0.1\,^{\circ}C$ ($\pm 0.2\,^{\circ}F$), to measure the temperature difference. A typical measuring system includes two thermistors, each shaded by a metal tube and aspirated by a small, battery-powered fan (fig. 3-69). A digital display permits readings to be made from either thermistor.

Air movement across the thermistors should be equal at all times. Check for air movement by placing your hand in front of each thermistor. A small voltmeter should indicate when batteries must be replaced to power the fans adequately. In addition, the display should indicate when its batteries must be replaced. A display battery having low voltage can cause the observed temperatures to be in error by several degrees.

Once a week, and whenever a problem is suspected, check the thermistors for consistency. Place them next to each other at the same height and pointed in the same direction. After allowing them to stabilize for 3 minutes, the temperatures should agree within $\pm 0.1\,^{\circ}C$ ($\pm 0.2\,^{\circ}F$). Note the result of this check on the recording form. If the thermistors fail to check and replacements are not available, continue leveling, but do not measure temperatures with them.

Locate the thermistors 1.3 m and 0.3 m above the ground unless specified otherwise in project instructions. These heights have been selected so the thermistors can be mounted on a tripod leg. If the tripod is used by different observers, relocate the thermistors as necessary for each observer. The thermistors must be at the correct heights above the ground when temperatures are measured.

Figure 3-69.—Aspirated thermistors for measuring air-temperature differences.

Keep the thermisters inside their shields and pointed into the wind and out of sunlight at all times. Allow them to stabilize for at least 1 minute during each setup, then read the display after the leveling observations are checked. Record the temperatures to the nearest tenth of a degree, top one first. Stand away from the thermistors while reading the temperature; body heat can falsify the results.

If the tripod must be positioned temporarily in a way that lowers or raises the thermistors from the required heights, reset the tripod after the observations are complete and allow the thermistors to stabilize for 1 minute. Then, read and record the temperatures.

3.6.2 Leveling in Extreme Conditions

Do not level if the weather becomes so severe that the health or safety of personnel is threatened. Adverse weather may also affect the correct operation of equipment and result in unreliable observations. Safe working conditions are discussed in the National Geodetic Survey Operations Manual (Greenawalt and Floyd 1980).

Leveling equipment is designed to operate correctly within a broad range of temperature. However, precipitation and wind may interfere with normal operations. If precipitation is light (less than 0.5 mm/hr) continue leveling by using an umbrella to protect the instrument. Afterwards, be sure to dry each piece of equipment thoroughly before storing it. Do not level in heavy precipitation.

Wind often causes vibrations in the leveling instrument which makes observing difficult. At wind speeds in excess of 30 km/hr (19 mi/hr), leveling may be impossible. In windy conditions, use the leveling truck or an umbrella as a shield. Each rodman must be particularly attentive to maintain the rods plumb. If possible, find a less windy section of line on which to continue work until conditions improve.

Observations are not reliable if made when long-period shimmer is present. It occurs most frequently when the air is calm and the air near the ground is cooler than the air above it. It is unlikely to occur during daylight hours between 1½ hours after sunrise and ½ hour before sunset, when the air near the ground is warmer than the air above it. For this reason, restrict leveling to these daylight hours whenever possible.

Sometimes the unit must level when long-period shimmer may be present, such as in calm air over frozen ground or snow, over water, or at night. In these circumstances, double run every section.

3.7 Observing Routine

This subchapter describes the routine to be followed when leveling a line of the National Geodetic Vertical Control Network. First, general instructions are given. Second, two procedures are described: micrometer leveling and three-wire leveling. Third, precautions and special instructions are given for control points. Finally, instructions are provided for releveling and closing sections.

3.7.1 General Instructions

The leveling unit is normally assigned a line or portion of a line within a vertical control project. The line should be leveled by sections, each section beginning and ending on described, permanent control points. Each section is an unbroken series of setups, observed by a procedure that meets specifications for the order and class of survey given in the project instructions. Micrometer leveling is preferred by the National Geodetic Survey for all first- and second-order surveys. Three-wire leveling produces results of lesser precision; therefore, it is included here only as a second-order technique. The following instructions apply to either procedure.

Leveling the line. Plan the leveling route by examining the logs (instructions) prepared by the mark setters for the assigned line. Chapter 2 contains detailed requirements for routing lines of control points.

Establish the direction of leveling by giving the line a title corresponding to the intended beginning and ending points. Normally, a line begins and ends at junctions with previous or concurrent leveling. Sections leveled away from the beginning point and toward the ending point are termed forward runnings. Sections leveled in the opposite direction are termed backward runnings.

Check the project instructions and the National Geodetic Survey Operations Manual (Greenawalt and Floyd 1980) to determine the order and class of the survey, special requirements, and changes to standard procedures. Determine if the line is to be double run or single run. Review the tolerances given in table 3-1.

When a line is to be double run, level each section once forward and once backward, under different environmental conditions if possible. A good practice is to level in one direction in the morning and return in the afternoon, leveling in the other direction. Alternate the starting direction each day. The two runnings of each section must close as described in section 3.7.6, "Rejection procedure." This procedure minimizes blunders and the accumulation of most types of systematic error.

When a line is to be single run, level each section once. Alternate the direction of the runnings from one section to the next. An alternative procedure is to alternate the direction of leveling from one day to the next, maintaining an equal distance of forward and backward runnings along the line. This procedure prevents the accumulation of most types of systematic error for more than one section or day. Single-run leveling must form loops with other leveling, preferably during the same field season, to enable the detection of blunders.

Double run all spurs. A spur is a portion of line that branches off the main line of leveling, usually ending at one or more points not otherwise connected to the network. The second running provides the only check on the leveling. If a spur requires only one setup, change the height of the instrument by several centimeters between the forward and backward runnings. If the purpose of a spur is to tie a previous line of leveling, a single running is sufficient, provided it closes with the previous work.

Make single-run ties at each connection with a previous or concurrent line of leveling. Connections are normally noted in the logs. To tie to a first-order line, level to three points in the line (two sections). To tie to a second-order line, level to two points in the line (one section). To tie to an area survey, with several lines crossing the new work, level to three points from the survey that span its area.

At each connection, check the new leveling for closure with the previous, unadjusted leveling, using tolerances for whichever survey has the lower order and class. Elevation differences for the previous survey may be obtained from an unadjusted abstract for the survey. (See sec. 5.2.3, "Preliminary data."). If the sections do not close after one running, relevel them until they close to verify the new work and contact the project office immediately to find out if additional points must be leveled to complete the tie.

If the leveling of a line is interrupted for 2 weeks or more, establish a tie, as just described, before resuming work on the line. If the leveling is resumed within the same field season, a two-mark tie at least 1 km (0.6 mi) in length is sufficient, regardless of the order and class of survey. The tie ensures that no significant movement of the control points has occurred since the leveling was interrupted.

Leveling the section. The entire leveling unit must exercise the greatest possible attention to detail while leveling each section of the line. Each day check and maintain the instruments and equipment as described in this chapter. A checklist appears in table 3-4.

Explain specifically any deviation from the leveling routine. Record the following: (1) date, (2) time at which the leveling of each running began and ended, (3) personnel, and (4) equipment involved. Describe any symptoms of potential equipment failure at the time they occur.

Keep sighting distances within the prescribed tolerance for the order and class of the survey. If shimmer presents a challenge, shorten the sighting distance until acceptable readings can be made. In addition, do not allow the line of sight to pass closer than 0.5 m (1.6 ft) to the ground or to any intervening object. These precautions should reduce the error caused by refraction.

Balance the sighting distances as closely as possible on every setup, at least within the prescribed tolerance. This reduces the effects of collimation error, refraction, and curvature.

Keep the total of the setup imbalances as small as possible, at least within the prescribed tolerance. If the accumulated imbalance becomes large during the levelings of the section, adjust the remaining setups to diminish it gradually. Do not try to correct for it with a few extremely unbalanced setups.

Complete the section as efficiently as possible, with no breaks during the series of setups. Only properly described, permanent monuments may serve as beginning and ending points. If the section cannot be completed or if a blunder occurs, reject the data collected so far. Enter on the recording form "999" for the next setup number and write a note explaining the rejection.

Except in the case of an incompleted section, do not reject any data in the field. If a blunder is found to have occurred after the fact, document it as clearly as possible on the recording form or by letter to the project office.

After completing one section, advance without a break to the first setup of the next section whenever possible. The recorder may complete the ending and beginning running records from the new setup.

Table 3-4.—Checklist for leveling

Equipment	Check	Section reference
Leveling instrument		
Tripod condition	Daily	3.3.5
Tribrach condition	Daily	3.3.5
Footscrew tension	Daily	3.3.5
Circular-level adjustment	Daily	3.3.5
Collimation check,		
Nonreversible compensator	Daily	3.3.7
Reversible compensator	Weekly	3.3.7
Compensation check	Weekly	3.3.8
Clean	Weekly	3.3.5
Stadia-factor determination	As necessary	3.3.6
Leveling rods		
Condition (base plate, brace poles, housing)	Daily	3.4.1
Circular-level adjustment		
Check with instrument	Daily	3.4.3
Check precisely	Weekly	3.4.3
Clean	Weekly	3.4.3
Calibrate	Annually	3.4.3
Condition of turning points	Daily	3.5.1
Temperature sensors		
Battery condition	Daily	3.6.2
Calibrate	Weekly	3.6.2
Computer-recording equipment		
Charge batteries	Daily	3.8.2
Precondition data tapes or disks	Daily	3.8.2
Clean recording head	Weekly	3.8.2
"Cycle" batteries	Monthly	3.8.2
Clean	Monthly	3.8.2
Check programs	At start of project	3.8.2

Check all of the above whenever equipment is changed.

3.7.2 Micrometer Leveling

Micrometer leveling is the most precise procedure currently available for geodetic leveling. Readings are made directly from the instrument's micrometer, which provides more precise results than those provided by estimation in the three-wire leveling procedure.

The primary feature of the procedure described here is the measurement of two elevation differences during every setup. The first difference is measured from backsight to foresight, the second from foresight to backsight. Since two runnings are completed in opposite directions during every setup, each section is leveled twice, simultaneously. When this procedure is used for a single leveling of a line, it is sometimes called double-simultaneous leveling. It can be used for any order and class of survey by varying the tolerances specified in table 3-1.

To provide two different lines of sight, an instrument with a reversible compensator is preferred. An instrument without a reversible compensator can be used to provide two lines of sight if, between the two sets of observations, it is disleveled, adjusted to a slightly different height, and releveled.

To provide two different readings at each turning point, double-scale rods are used. Two rods are necessary, not only to improve efficiency, but to permit nearly simultaneous observation of the backsight and foresight. The constants of the two rods must differ, to permit a mathematical check ensuring that rods are observed in the correct order. The rod-constant difference, rod 2 minus rod 1, is labeled "d" on the recording form.

During each setup, six readings are made. First, wedge and stadia intercepts are read, backsight then foresight, from the low scale of each rod. From the two half stadia intervals the sighting distances, s_B and s_F, and the imbalance, $s_B - s_F$, are computed and checked. From the wedge intercepts the elevation difference of the low scales, Δh_L, is also computed.

Second, after changing the line of sight, wedge intercepts are read, foresight then backsight, from the high scales of each rod. From these two readings a value, $\Delta h_H \pm d$, is computed. To obtain the elevation difference of the high scales, Δh_H, the rod-constant difference, d, must be added or subtracted, depending on which rod is observed as the backsight. Then Δh_L and Δh_H are compared.

Balancing the sighting distances should reduce the effects of collimation error, moderate refraction, and curvature to immeasurable amounts during the setup. However, error in the pointings may still cause a slight difference between Δh_L and Δh_H. The difference, $\Delta h_L - \Delta h_H$, is compared to a tolerance to check for a value greater than might be expected as a result of random pointing error. This comparison is called the reading check:

$$\Delta h_L - \Delta h_H \leq \text{tolerance}.$$

To determine a suitable tolerance for the reading check, the other tolerances for the leveling procedure must be considered. If the collimation error does not change between the low- and high-scale observations, the tolerance may be derived from the standard deviation of a single pointing, which, in turn, depends on the sighting distance and the optics of the instrument. With a sufficiently powerful computer-recording system, a formula might be used to compute the tolerance appropriate to the actual sighting distances of each setup. Usually, however, a single tolerance is derived, appropriate to the maximum sighting distance permitted for the order and class of the survey. The tolerances given in table 3-1 are of this type.

If the collimation error is different for the low- and high-scale observations, as is the case with a reversible compensator, the setup imbalance affects the tolerance for the reading check. For example, the difference in the collimation errors for position one and position two of the Jenoptik NI 002 affects $\Delta h_L - \Delta h_H$ by an amount that depends on the setup imbalance and $C_1 - C_2$. Since the angular difference, $C_1 - C_2$, changes somewhat unpredictably both in magnitude and sign, the tolerance for the reading check must be derived from the maximum setup imbalance permitted for the order and class of the survey and the maximum magnitude of $C_1 - C_2$ in addition to the standard deviation of each pointing.

After satisfying the reading check, temperatures are recorded so a refraction correction can be computed and applied to the data before adjustment. Then, the two elevation differences are averaged to obtain the elevation difference for the setup:

$$\Delta h = 0.5 \times (\Delta h_L + \Delta h_H).$$

As each setup is completed, the new difference is added to the sum of the previous differences. Thus, at the end of the section, the total elevation difference between the marks is obtained. For convenience when setup data are recorded on paper, Δh_L and $\Delta h_H \pm d$ are summed separately. The elevation difference is then computed at the end of the section.

Though this leveling procedure is designed to prevent blunders and the accumulation of systematic error, both may still occur without warning. Note, for example, that there is no check for movement of the turning points between setups. As with any procedure, strict attention must be paid to the precautions and guidelines presented in the rest of this chapter.

Instructions. The following instructions apply to a properly adjusted leveling instrument with a micrometer, used with a pair of calibrated, double-scale rods. If the instrument does not have a reversible compensator, substitute the following procedure whenever instructed to change the position of the compensator: Dislevel the instrument using the footscrew closest to the foresight rod, then relevel it using the other two footscrews.

If possible, record the observations in a computer and, if required, prepare a backup recording form (NOAA Form 77-82, "Geodetic Leveling"). The instructions presented here include the procedure for using NOAA Form 76-191, "Geodetic Leveling Micrometer Observations." Use a separate recording form for each running. (See figs. 3-70 through 3-72 for sample records.)

GEODETIC LEVELING

LINE L 24582 Line 364A TAPE NO. NM 12
PROJ. NO. NGVD Region 4 PAGE 1 OF 8

ST 0 DATA	ST 1 YR. MO. DY.	ST 2 ZONE	ST 3 INST. NO. CODE
40	800404	20	45651 1.233

ST 4 ROD NO. 1 CODE	ST 5 ROD NO. 2 CODE
269722.316	277926.316

ST 6 OBSERVER LHT	ST 7 TIME	ST 8 COLLIM. FACTOR	ST 9 C F	ROD UNITS	STAD
120820	0630	-0.002	3	HC	✓ C ✓ F

0 – WRITE ON TAPE – 9 BLOCK 01-07

ST 0 DATA	ST 1 FROM:	DESIGNATION
41	2210	M 198 1935
	ST 2 TO: 2212 (1)	S 260 1980

ST 3 TIME	ST 4 TEMP	ST 5 WIND	ST 6 SUN	ST 7 DATE	BLOCK
0653	-20	0	1	800404	01-08

0 – WRITE ON TAPE – 7 5 – CLR

J.S.S

ELEV. DIFF. RCL 7	KM. RCL •	NO. RCL. 0
-5.76435	1.28	12

ST 0 DATA	ST 1 TIME	ST 2 TEMP	ST 3 WIND	ST 4 SUN	OBS.	REC.	BLOCK
41	0805	41	0	2	LHT	CWS	03-05

0 – WRITE ON TAPE – 4 ROD #1 TJJ ROD #2 RMD

REMARKS

Wrong SPSN on tape. "2212" should be "2211."

REMARKS

M 198 to checkpoint:
382.31
- 371.02
+ 11.29
800403: - + 11.41
- .12 hcm
x 5
- 0.60 mm

NOAA FORM 77-82 (11-79) U.S. DEPARTMENT OF COMMERCE – NOAA
SUPERSEDES NOAA FORM 77-82 (8-79) WHICH SHOULD BE DESTROYED.

Figure 3-70.—Micrometer leveling, backup form for a computer-recording system.

1. EQUIPMENT RECORD: At the start of each day, or at any change of the observer, equipment, or collimation factor, prepare the survey-equipment record (line *40*). Include the date, the instrument code and serial number, the rod codes and serial numbers, the local time zone and time, the type of temperature units, the collimation factor at the last check, and the initials of the observer, recorder, and rodmen. CAUTION: Each work day check all serial numbers against the actual equipment used. Also, check that the record of the collimation factor is correct.

2. Establish a balanced setup with the backsight rod plumbed on the beginning control point of the section, the foresight rod plumbed on a turning point, and the instrument set in line between them. Check for parallax. Using the instrument, check that the rods are plumb.

3. BEGINNING RUNNING RECORD: Immediately before leveling begins, prepare the beginning running record (line *41*). Include the following items of information: The survey-point serial number of the beginning control point, the stamping on the point (or designation if no stamping exists), the local time, the air temperature, the wind code, the sun code, and the number of the rod on the control point. CAUTION: Check that the rod identified is, in fact, on the point identified. The recorded stamping must correspond to that on the mark leveled. Check and initial the survey-point serial number entered on the form.

4. BACKSIGHT, LOW SCALE: Level the instrument while pointing at the backsight rod. In compensator position one (odd-numbered setup) or two (even-numbered setup), read the wedge intercept on the low scale. Pause slightly between the three- or four-digit rod-unit reading and the two-digit micrometer reading, so the recorder can enter the decimal correctly. The recorder should call out any uncertain numbers for the observer to verify. Record the entire reading in column F, backsight. Record the rod-unit reading only in column B.

5. Read the intercept of the lower stadia line, estimating to tenths of rod units, on the low scale. Again, pause during the reading to indicate the decimal. Record it in column B. Compute the half-stadia interval by subtracting the stadia reading from the rod units of the first reading. With half-centimeter rods this value is the backsight distance, s_B. With centimeter rods, multiply the interval by 2 to compute the distance.

6. FORESIGHT, LOW SCALE: Point toward the foresight rod Make sure the rod has rested on the point for 20 seconds or more. Still in the same compensator position, read the low scale as before. Record the readings in column F, foresight, and column D. Compute the foresight distance, s_F. Check that the imbalance, $s_B - s_F$, is within tolerance for the survey. If it is not, relocate either the foresight rod or the instrument and begin again at step 4. Compute the low-scale elevation difference, Δh_L, by subtracting the foresight from the backsight.

7. FORESIGHT, HIGH SCALE: Still pointing toward the foresight rod, change the compensator position (or dislevel and relevel the instrument). Read the high scale. Record the reading in column H, foresight.

8. BACKSIGHT, HIGH SCALE: Point toward the backsight rod, in the same compensator position, and read the high scale. Record the reading in column H, backsight. Compute $\Delta h_H \pm d$ by subtracting the foresight from the backsight.

NOAA FORM 76-191 (8-77)

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GEODETIC LEVELING MICROMETER OBSERVATIONS (Δh) EXAMPLE First Order, Class I — PAGE 1 of 1

	YR.	MO.	DAY	CODE	INSTRUMENT SERIAL NO.	M	CODE	ROD SERIAL NO.	CODE	ROD SERIAL NO.
40										
41	80	11	14	231	5802	H	316	148972	316	148974

FROM	BM DESIGNATION	TO	BM DESIGNATION	Z	TIME BEGIN	TIME END	F/C	TEMPERATURE BEGIN	TEMPERATURE END	OBSERVER
1236	TOP NO.2 1948	1237	M 306 1979	R	1420	1445	C	172	1780	CWS

(A) SET UP#	(B) STADIA BACK	(C) S_B Σ	(D) STADIA FORE	(E) S_F Σ		(F) LOW SCALE BACK/FORE	(G) Δh_L Σ		(H) HIGH SCALE BACK/FORE	(J) Δh_{H} Σ	(K) $\Delta h_L - \Delta h_H$	REMARKS
1	357	514	255	530	B	35772	10224	B	94953	8127	-03	Reading check=±.08 hcm
	3056		2020		F	25548		F	86826			Imbalance=±2.0 m
		$\Delta s = S_B - S_F = -1.6$								d=+21		Section imbalance=±4.0 m
2	372	525	250	510	B	37280	12188	B	98697	14283	+05	
	3195	1039	1990	1040	F	25092	22412	F	84414	22410	+02	
										-21		
R 3	379	525	254	510	B	37936	12489	B	97119	10380	+09	R
	3265		2010		F	25447		F	86739			
				+1.5						+21		
3	379	525	254	510	B	37935	12481	B	97120	10380	+01	
	3265	1564	2010	1550	F	25454	34893	F	86740	32790	+03	
				+1.5						+21 5		
4	358	543	225	540	B	35856	13317	B	97273	16413	+04	
	3037	2107	1710	2090	F	22539	48210	F	81860	48203	+07	
				+0.3						-21		
R 5	346	545	261	511	B	34674		B				R
	2915		2099		F	26190		F				
				+3.4 > 2.0 m						+21		
5	346	545	261	550	B	34672	8483	B	93859	6385	-02	
	2915	2652	2060	2640	F	26189	56693	F	87474	54588	+05	
				-0.5						+21		BM vertical. Used 1-meter scale.
6	360	500	057	520	B	36014	30253	B	97428	32348	+05	
	3100	3152	1090	3160	F	05761	86946	F	65080	86936	010	
		+3160		-2.0		ΣΔs=-0.8 m				-21 Mean ΔH =		+ 869.410 hcm
		6312 m			B			B				÷ 200 hcm/m
		x .001 km/m			F			F				+ 4.34705 m
		S = 0.63 km										

Figure 3-71.–Micrometer leveling, half-centimeter rods.

NOAA FORM 76-191 (8-77)

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

GEODETIC LEVELING MICROMETER OBSERVATIONS (Δh) EXAMPLE First Order, Class II — PAGE 1 of 1

	YR.	MO.	DAY	CODE	INSTRUMENT SERIAL NO.	M	CODE	ROD SERIAL NO.	CODE	ROD SERIAL NO.
40										
41	80	11	14	231	58021	C	316	108322	316	108324

FROM	BM DESIGNATION	TO	BM DESIGNATION	Z	TIME BEGIN	TIME END	F/C	TEMPERATURE BEGIN	TEMPERATURE END	OBSERVER
6013	B 301 1967	6012	M 31 1930	S	1302	1330	C	153	16002	CWS

SET UP#	STADIA BACK	S_B Σ	STADIA FORE	S_F Σ		LOW SCALE BACK/FORE	Δh_L Σ		HIGH SCALE BACK/FORE	Δh_{H} Σ	$\Delta h_L - \Delta h_H$	REMARKS
1	155	240	164	241	B	15520	- 896	B	45191	- 1894	-02	Reading check=± .10 cm
	1310		1399		F	16416		F	47085			Imbalance ≤ ± 5.0 m
		$\Delta S = S_B - S_F = -0.1 \times 2 = -0.2$ m								d=+10		Section imbalance ≤ ±10.0 m
2	148	225	146	234	B	14856	213	B	45484	1210	+03	
	1255	465	1226	475	F	14643	- 683	F	44274	- 684	+01	
				-0.9 x 2 = -1.8						-10		
3	188	240	173	230	B	18816	1417	B	48455	416	+01	
	164[illegible]	705	150[illegible]	705	F	17399	734	F	48039	- 268	+02	
				+1.0 x 2 = 2.0						+10		
R 4	164	186	161	160	B	16421		B				R
	1454		1450		F	16109		F				
				+2.6 x 2 = 5.2 > 5.0						-10		
4	164	171	162	170	B	16465	264	B	47115	1265	-01	
	1469	876	1450	875	F	16201	498	F	45850	997	+01	
				+0.1 x 2 = 0.2						-10		
R 5	147	225	157	225	B	14767	- 987	B	44408	- 1973	-14	R
	1245		1345		F	15754		F	46381			
				0.0						+10		
5	147	225	157	225	B	14766	- 989	B	44398	- 1988	-01	
	1245	1101	1345	1100	F	15755	009	F	46386	- 991	00	
				0.0						+10		
6	166	234	143	230	B	16625	2262	B	47275	3263	-01	
	1426	1335	1200	1330	F	14363	2271	F	44012	2272	-01	
				+ 0.5 X 2 = 1.0						-10		
999					B			B				Rodman Jones pulled foresight pin.
					F			F				

Figure 3-72.—Micrometer leveling, centimeter rods.

9. READING CHECK: Compute $\Delta h_L - \Delta h_H$. To do this, if rod 1 is the backsight rod, add d to $\Delta h_H \pm d$. If rod 2 is the backsight rod, subtract d from $\Delta h_H \pm d$. Then subtract Δh_H from ΔH_L. Compare the result to the tolerance for the survey. CAUTION: Do not move the backsight turning point until the reading check is complete. If the point is moved, even only slightly, the data collected so far must be rejected and the running of the section must be begun again at step 2.

10. If the readings fail to check, begin the setup again at step 4. If they repeatedly fail to check, try one or more of the following procedures:

(1) Check for parallax and remove it if present.

(2) Check the plumb of the rods and the leveling of the instrument, including the adjustment of the circular level.

(3) Check that the reversible compensator is correctly positioned.

(4) Reposition the instrument to balance the setup exactly.

(5) Shorten the sighting distances by repositioning the instrument and the foresight rod.

(6) Check that the rods are being observed in the correct order (backsight firs).

(7) Check that the rod-constant difference, d, is entered with the correct magnitude and sign.

(8) Check that the starting rod for the section was identified correctly and that the current setup number is correct.

(9) Make a collimation check (sec. 3.3.7).

(10) Make a compensation check (sec. 3.3.8).

(11) Consider the possible presence of unusual environmental conditions (sec. 3.6.3), such as long-period shimmer. Return to the section when conditions improve.

11. After obtaining a reading check, record the air temperatures for the refraction correction if the appropriate equipment is available (sec. 3.6.2). Similarly, record temperatures of the rod scales if the appropriate equipment is available. With a computer-recording system, check that the temperatures are reasonable.

12. Check the accumulated imbalance for the section. If it is more than the tolerance for one setup, alert the pacer or rodmen to adjust the next setup or setups accordingly. When recording on paper, sum the backsight and foresight distances separately; then compare the overall imbalance. Also, sum the low-scale and high-scale elevation differences separately; at the end of each even-numbered setup the differences should agree closely (though not necessarily within the tolerance for the reading check).

13. NEXT SETUP: Leaving the compensator in the same position, advance to the next setup. Leave the foresight rod (and turning point) in place to become the next backsight rod. Move the former backsight rod and turning point to the next foresight position. Set the instrument on a line halfway between the two rods. CAUTION: Do not remove the former foresight rod from its turning point. Keeping it centered on the point, pivot it to face the instrument.

14. Repeat steps 4 through 13 at the next setup. Remember that the backsight, low scale is observed first on every setup, but the rods, the compensator positions, and the sign of d alternate on every setup. (See table 3-5 for a summary of the procedure.)

15. As the unit approaches the end of the section, if required, adjust the sighting distances to make an even number of setups for the section. The final setup should have rod 1 on the control point and rod 2 on the last turning point. An even number of setups is not required if all graduations of both rods have been calibrated by laser interferometry. (See sec. 3.4.2, "Calibration with a laser interferometer.")

16. ENDING RUNNING RECORD: After completing the last setup of the section, prepare the ending running record (line *41*). Include the following items of information: the survey-point serial number of the ending control point, the stamping on the point (or designation if no stamping exists), the local time, the air temperature, the wind code, and the sun code. Compute and record in meters the elevation difference of the section. Compute and record in kilometers the length of the section. Record the number of setups. If the leveling is the second running of a section, check for closure (sec. 3.7.7). CAUTION: Make sure the stamping entered for the ending point corresponds exactly to that on the mark on which the rod is resting.

Table 3-5.—Summary of the micrometer leveling procedure

During each setup

1. Balance setup.
2. Point instrument at backsight.
3. Level instrument and plumb rods.
4. Read backsight, low scale: wedge and stadia.
5. Point to and read foresight, low scale: wedge and stadia.
6. Check sighting distances and imbalance against tolerances.
7. Change compensator position or dislevel-relevel.
8. Read foresight, high scale: wedge.
9. Point to and read backsight, high scale: wedge.
10. Check low- and high-scale elevation differences against tolerance.
11. Read temperatures.
12. Check accumulated imbalance against tolerance.
13. Move to next setup.

During the section

Number setup	Scale observed	Compensator position
1	low	•
	high	••
2	low	••
	high	•
3	low	•
	high	••
.	.	
.	.	
.	.	

3.7.3 Three-Wire Leveling

Until the advent of leveling instruments equipped with micrometers, three-wire leveling was the most precise method for measuring elevation differences. However, without a micrometer, scale readings must be estimated and only one elevation difference can be measured efficiently during each setup. The method described here can provide differences with a precision sufficient for second- or lower-order surveys.

The instrument need only provide a single, consistent line of sight during each setup. Two nearly identical rods are used to permit nearly simultaneous observation of the backsight and foresight. Only one calibrated scale is necessary on each rod; however, a second uncalibrated scale on the back of the rod should be read as a check for blunders. To estimate readings with the greatest precision, the calibrated scale is graduated with blocks, preferably in a checkerboard pattern. (See sec. 3.4.1, "Scale.").

During each setup, eight readings are made. First, the intercepts of the upper, middle, and lower reticle lines ("wires") are read from the calibrated scale of rod 1. Then, the intercept of the middle line with the back scale is read. The four readings are estimated to the nearest tenth of a rod unit.

Two checks are made. To check the internal consistency of the three precise readings, the half stadia intervals, s_U and s_L, are computed and their difference is compared to a tolerance. To detect gross blunders of decimeters or meters, the reading of the middle reticle line is compared to the back-scale reading. Since the precise readings are made nearly simultaneously, a consistent error in the first or second digit might otherwise go undetected. This is particularly likely when leveling moves from flat to sloping terrain and the readings increase abruptly by a full meter.

Second, four more intercepts are read and checked from rod 2. The sighting distances are computed by summing the half stadia intervals for each rod. Then, the imbalance is computed and checked against the tolerance.

Rod 1 is observed first during each setup. Thus, the backsights and foresights are observed in alternate order on alternate setups, to reduce systematic error. This is especially important when using a compensator instrument, to prevent the accumulation of systematic error due to consistent variation in the collimation error. (See sec. 3.3.8, "Compensation check.")

The elevation difference for each setup is the difference between the mean of the three backsight readings and the mean of the three foresight readings. It is not usually computed until the entire section is completed.

With three-wire leveling certain blunders cannot be detected. For example, a mathematical check to detect transposition of the foresight and backsight observations is not possible. Neither is a mathematical check to detect disturbance of the instrument between the backsight and foresight observations.

Instructions. The following instructions apply to a properly adjusted instrument without a micrometer (or with the micrometer locked in position), used with a pair of calibrated rods, each having one calibrated scale and a check scale. A spirit-level instrument must be shaded with an umbrella.

Record observations on NOAA Form 76-189, "Geodetic Leveling Three-Wire Observations." (See figs. 3-73 and 3-74 for sample records.) If observations are recorded in a computer, maintain a backup recording form as required by project instructions.

1. EQUIPMENT RECORD: At the start of each day or at any change of the observer, equipment, or collimation factor, prepare the survey-equipment record (line *40*). Include the date, the instrument code and serial number, the rod codes and serial numbers, the local time zone and time, whether temperature is measured in Fahrenheit or centigrade units, the collimation factor at the last check, and the initials of the observer, recorder, and rodmen. CAUTION: Each work day check all serial numbers against the actual equipment used.

2. Establish a balanced setup with the backsight rod plumbed on the beginning control point of the section, the foresight rod plumbed on a turning point, and the instrument set in line between them. Check for parallax. Using the instrument, check that the rods are plumb.

3. BEGINNING RUNNING RECORD: Immediately before the leveling begins, prepare the beginning running record (line *41*). Include the following items of information: The survey-point serial number of the beginning control point, the stamping on the point (or designation if no stamping exists), the local time, the air temperature, the wind code, the sun code, and the number of the rod on the control point. CAUTION: Check that the backsight rod is, in fact, on the point identified. The recorded stamping must correspond to that on the mark leveled. Check the survey-point serial number entered for the point.

4. ROD 1: Level the circular level on the instrument while pointing at rod 1. With a spirit-level instrument, use the tilting screw to center the bubble precisely in the tubular level. Read the intercepts of the three reticle lines with the precise scale, top to bottom. Estimate each reading to tenths of a rod unit.

5. When rod 1 is the backsight rod, record the readings in the backsight column, column B in the example. When rod 1 is the foresight rod, record the readings in the foresight column, column G. CAUTION: The recorder should watch to ensure that rod 1 is observed at the beginning of the setup. Be sure the readings are recorded in the correct column because there is no mathematical check for this type of error.

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GEODETIC LEVELING THREE-WIRE OBSERVATIONS EXAMPLE Second Order, Class I

*	40	*	YR./MO./DAY	CODE	INSTRUMENT SERIAL NO.	CODE	ROD SERIAL NO.	CODE	ROD SERIAL NO.	Z	TIME
*	41	*	77 10 04	232	7762	313	7765	313	7766		

FROM	BM DESIGNATION	TO	BM DESIGNATION	Z	TIME BEGIN	TIME END	F/C	TEMPERATURE BEGIN	TEMPERATURE END	W	S	OBSERVER
0001	052477 TBM 853 7562 STAFF 6 FT	0002	853 7562 A 1977	R	1100	1120	F	62	62	1	2	PDW

(A) SET UP	(B) BACKSIGHT UPPER / MIDDLE / LOWER / Σ	(C) MEAN / Σ	(D) BACK CENTER	(E) S_U / S_L	(F) (U+L) / Σ	(G) FORESIGHT UPPER / MIDDLE / LOWER / Σ	(H) MEAN / Σ	(J) BACK CENTER	(K) S_U / S_L	(L) (U+L) / Σ	COMMENTS
	478.7					281.4					Reading check ≤ ±3 hcm
1	474.1	474.07		4.6		276.6	276.57		4.8	Δs = − 0.4	Imbalance ≤ ±5 m
	469.4			4.7	9.3	271.7			4.9	9.7	x 200 hcm/m
	1422.2	474.07			9.3	829.7	276.57			9.7	1000 hcm ÷ 333
	251.8					437.3					≤ ±3
2	238.8	238.80		13.0		425.2	425.17		12.1	+ 1.7	
	225.8			13.0	26.0	413.0			12.2	24.3	
	2138.6	712.87			35.3	2105.2	701.74			34.0	
	− 2105.2	− 701.74			− 34.0					+ 35.3	
	= + 33.4	= + 11.13 hcm			= 1.3					= 69.3	
	÷ 3.				x 333.					x 333.	
	= + 11.13 hcm				= 432.9 hcm					= 23076.9	hcm
	÷ 200. hcm/m				÷ 200 hcm/m					÷ 200000	hcm/km
	ΔH = + 0.0556 m				ΣΔs = 2.2 m					S = 0.12 km	

Figure 3-73.—Three-wire leveling, half-centimeter rods.

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GEODETIC LEVELING THREE-WIRE OBSERVATIONS EXAMPLE Second Order, Class I

*	40	*	YR./MO./DAY	CODE	INSTRUMENT SERIAL NO.	CODE	ROD SERIAL NO.	CODE	ROD SERIAL NO.	Z	TIME
*	41	*	80 12 04	217	10827	312	428	312	436		

FROM	BM DESIGNATION	TO	BM DESIGNATION	Z	TIME BEGIN	TIME END	F/C	TEMPERATURE BEGIN	TEMPERATURE END	W	S	OBSERVER
0001	X 350 1957	0002	X 350 RESET 1980	R	1550	1610	C	13.4	13.3	1	1	RMB

SET UP	BACKSIGHT UPPER / MIDDLE / LOWER / Σ	MEAN / Σ	BACK CENTER	S_U / S_L	(U+L) / Σ	FORESIGHT UPPER / MIDDLE / LOWER / Σ	MEAN / Σ	BACK CENTER	S_U / S_L	(U+L) / Σ	COMMENTS
	163.8					137.0					Reading check ≤ ±.3 cm
1	154.9	154.90	50.8	8.9		128.2	128.20	42.0	8.8	Δs = + 0.2	Imbalance ≤ ±5 m
	146.0			8.9	17.8	119.4			8.8	17.6	x 100 cm/m
	464.7	154.90			17.8	384.6	128.20			17.6	500 cm ÷ 332 S.F.
	170.0					145.0					≤ ±1.5
2	161.5	161.50	53.0	8.5		136.0	135.97	44.5	9.0	− 1.1	
	153.0			8.5	17.0	126.9			9.1	18.1	
	949.2	316.40			34.8	792.5	264.17			35.7	
	165.7					154.9					
3	156.9	156.87	51.3	8.8		145.9	145.90	47.8	9.0	− 0.3	
	148.0			8.9	17.7	136.9			9.0	18.0	
	1419.8	473.27			52.5	1230.2	410.07			53.7	
	168.0					124.9					
4	159.1	159.07	52.1	8.9		116.7	116.67	38.2	8.2	+ 1.4	
	150.1			9.0	17.9	108.4			8.3	16.5	
	1897.0	632.34			70.4	1580.2	526.74			70.2	
	− 1580.2	− 526.74			− 70.2					+ 70.4	
	= + 316.8	= 105.60 cm			= 0.2					= 140.6	
	÷ 3.				x 332.					x 332.	
	= + 105.60 cm				= 66.4 cm					= 46679.2	cm
	÷ 100. cm/m				÷ 100 cm/m					÷ 100000	cm/km
	ΔH = + 1.0560 m				ΣΔs = 0.7 m					S = 0.47 km	

Figure 3-74.—Three-wire leveling, centimeter rods.

6. READING CHECK: Compute and record in column E (or K) the half stadia intervals. The upper interval, s_U, is the difference between the upper and middle readings, and the lower interval, s_L, is the difference between the middle and lower readings. Check that $s_U - s_L$ is no more than 0.3 rod unit. If it exceeds this tolerance, reobserve rod 1, beginning at step 4.

7. If the tolerance is satisfied, compute the full stadia interval, $s_U + s_L$, and record it in column F (or L). Compute the sighting distance and check that it is within the tolerance for the survey. For convenience, if recording on paper, convert the tolerance to a permissible stadia interval in rod units, and use this value to make the check. For example, if the rod units are centimeters, the stadia factor is 333, and the tolerance for sighting distance is 60 m, then the tolerance for the stadia ininterval is 18.0 cm [(60 m×100 cm/m) ÷333=18.0 cm].

8. Compute the mean of the three readings. The mean may be computed quickly by examining the last digits of s_U and s_L. If s_U is three tenths larger than s_L, add 1.0 to the middle reading. If s_U is two tenths larger, add 0.7. If s_U is one tenth larger, add 0.3. If s_U equals s_L, the middle reading is the mean. Similarly, if s_U is three tenths smaller than s_L, subtract 1.0 from the middle reading. If s_U is two tenths smaller, subtract 0.7. If s_U is one tenth smaller, subtract 0.3. The mean always ends with the numeral "0", "3", or "7".

9. BACK CHECK: Read the intercept of the middle reticle line with the back scale. Record the reading in column D. The recorder or the person holding the umbrella must convert the reading to the units of the precise scale, then compare the result to the reading of the middle reticle line made in step 4. If no blunders have been made, sum the three readings together with the sum from the previous setup. Also, sum the means computed so far. CAUTION: Do not move the backsight rod and turning point until the data for the entire setup have been checked.

10. ROD 2: Point the instrument toward rod 2. Do not relevel the circular level. With a spirit-level instrument, recenter the bubble in the tubular level. Make three readings as before.

11. When rod 2 is the foresight rod, record the readings in the foresight column, column G. When rod 2 is the backsight rod, record the readings in the backsight column, column B. CAUTION: Again, check that the readings are recorded in the correct column.

12. Perform the reading check and back check as in steps 6 through 9.

13. Compute and check the imbalance (backsight distance minus foresight distance.) If it exceeds the tolerance for the survey, reobserve the setup, changing the position of the foresight rod or the instrument.

14. Check the accumulated imbalance for the section. If it is more than the tolerance for one setup, alert the pacer or rodmen so the next setup or setups can be adjusted accordingly.

15. NEXT SETUP: Advance to the next setup. Leave the former foresight rod (and turning point) in place to become the next backsight rod. Move the former backsight rod and turning point to the next foresight position. Set the instrument on a line halfway between the two rods. CAUTION: Do not remove the former foresight rod from its turning point. Keeping it centered on the point, pivot it to face the instrument.

16. Repeat steps 4 through 15. Remember to read rod 1 first on every setup, regardless of whether it is in the backsight or the foresight position.

17. As the unit approaches the end of the section, if required, adjust the sighting distances to make an even number of setups for the section. The final setup should have rod 1 on the control point and rod 2 on the last turning point. An even number of setups is not required if all graduations of both rods have been calibrated by laser interferometry (sec. 3.4.2), which permits sufficiently accurate correction for index error.

18. After the last setup, compute and record the elevation difference for the section. Total the backsight readings and the foresight readings separately. Subtract the foresight total from the backsight total, and divide by 3. Similarly, total and subtract the backsight and foresight means. The two elevation differences should agree within 1.0 mm (0.003 ft).

19. Compute and record the length of the section. Total the backsight and foresight stadia intervals, multiply the result by the stadia factor, and convert it into meters. A summary of the three-wire procedure is given in table 3-6.

20. ENDING RUNNING RECORD: After completing the last setup of the section, prepare the ending running record (line *41*). This includes the survey-point serial number of the ending control point, the stamping on the point (or designation if no stamping exists), the local time, the air temperature, the wind code, and the sun code. Compute and record in meters the elevation difference of the section. Compute and record in kilometers the length of the section. Record the number of setups. If the leveling is the second running of a section, check for closure. (See sec. 3.7.7, "Releveling and closing sections."). CAUTION: Make sure the stamping entered for the ending point corresponds exactly to that on the mark on which the rod is resting.

3.7.4 Precautions to Take at Control Points

The beginning and ending control points of a section are critical in three ways. First, the marks must be positively identified. Second, the control points must be clearly defined and properly used. Third, the stability of a mark that represents the end of a previous day's or week's leveling must be proved.

Identify the mark. Because of the proliferation of survey marks of all types and the possibility that control points may have been relocated without the knowledge of the leveling unit, each mark must be identified carefully. At the start of a section, identify only the

Table 3-6.—Summary of three-wire leveling procedure

During each setup

1. Balance setup.
2. Point instrument at rod 1.
3. Level instrument and plumb rods.
4. Read three reticle lines, top to bottom.
5. Check half-stadia intervals and sighting distance.
6. Read back scale: middle reticle line.
7. Check mean of three readings against back-scale reading.
8. Point instrument at rod 2.
9. Read three reticle lines, top to bottom.
10. Check half-stadia intervals and sighting distance.
11. Read back scale: middle reticle line.
12. Check mean of three readings against back-scale reading.
13. Check imbalance and accumulated imbalance.
14. Move to next setup.

beginning mark; identify the ending mark when the section is completed. When the rodman prepares the control point for the leveling rod, he or she should call out the stamping to the recorder. The recorder enters the stamping, as it is called out, and the corresponding survey-point serial number as it is given in the log.

Locations where marks may be easily misidentified include triangulation stations and tide or water-level stations. At these locations, marks are set in clusters, often only a few meters apart. To further complicate matters, many marks may be stamped with similar designations, perhaps bearing only one or two inconspicuous symbols to distinguish them. Level through such clusters with care. Avoid unnecessary spurs. For example, at a triangulation station, do not level separate spurs to the reference marks. Instead, level from one reference mark to the station, to the next reference mark, and then to the next control point in the line (fig. 3-75).

If an apparent mistake or duplication is found in the log, investigate and correct it, adding a thorough explanation on the recording form for leveling. Some examples follow: If a control point is not found where it was plotted on a map, explain the situation and plot the point correctly. If a survey-point serial number has not been assigned in the log, use "0000" and note the fact. If a mark does not exactly match the description given in the log, check for a similar mark in the vicinity, and be sure to record the stamping that appears on the mark actually leveled.

Sometimes the unit may find a control point that is not indicated in the log. In general, do not level to such a point unless it is already included in the national network. All control points in the national network should have been accounted for by the reconnaissance team. If, in fact, the point represents an accidental omission by the mark setter or an amendment to the original leveling route, it should be leveled. Prepare a complete description in the standard format (sec. 2.4.4) and submit it with the leveling data.

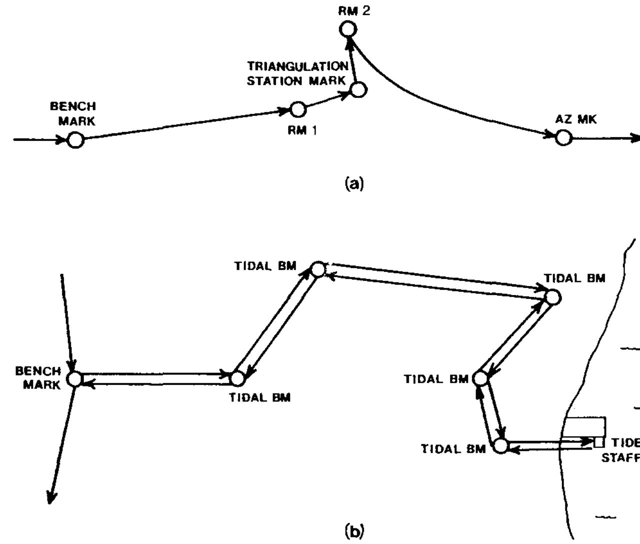


Figure 3-75.—Leveling routes through a cluster of control points at a triangulation station (a) and a tide station (b).

Identify the control point. The leveling rod must rest on a clearly defined point. On a horizontal surface, the control point is the highest point of the mark. On a vertical surface, it is at the intersection of a pair of crossed lines (the exact center of a bench mark disk, fig. 3-76). The one exception to these rules occurs if some other point is specifically defined in the description of the control point. If any other point is used, describe it clearly in the leveling records.

Center the base plate of the rod precisely on the control point. If the point is in a location that does not permit this, use the center of the back or front edge before resorting to a corner of the base plate. If a mark is very flat, use a spacer placed exactly at the center. Sometimes the concrete around a disk may prevent centering or rotating the rod. Use a spacer in this situation as well.

A spacer ("plug") is a calibrated cylinder of solid metal, often magnetized. (If observations are recorded on a magnetic tape or disk, keep the spaces away from the recording media.) The spacer is used to elevate the rod above the control point in a restricted setting, thus permitting the rod to be centered and rotated. When a spacer is used, either the same spacer or another one identical in height must be used on the turning point of the same setup. Make a note on the recording form whenever a spacer is used.

To use a single spacer, place it under the backsight rod of the setup. After reading the backsight, low scale, place the spacer under the foresight rod. After the foresight readings, return it to the backsight rod for the final reading. The height of the spacer is included in

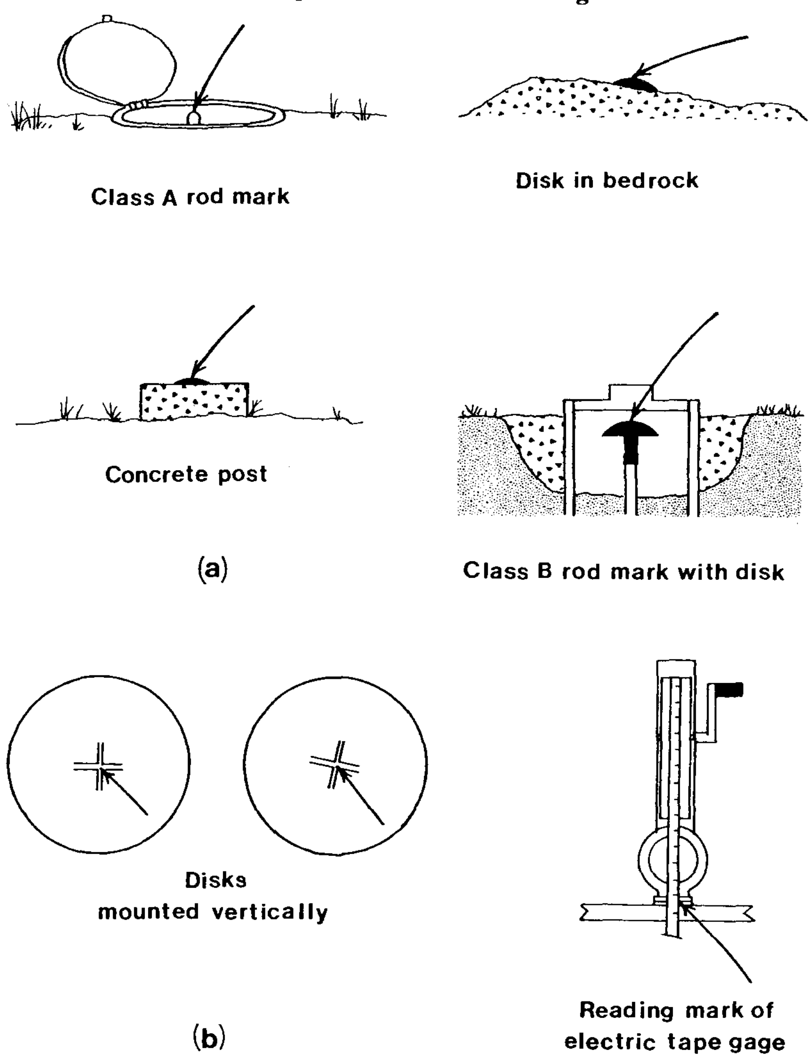


Figure 3-76.—Control points on horizontal surfaces (a) and vertical surfaces (b).

the observed heights of both the backsight and the foresight; therefore, the extra height is canceled when the elevation difference is computed (fig. 3-77).

A pair of spacers, manufactured and calibrated to agree in height within 0.05 mm (0.0015 ft), can be used simultaneously for a more efficient operation.

Set a check point. The elevation of the control point at the end of a line segment leveled during one day must remain unchanged until leveling resumes the next day. To ensure this, set a check point wherever a day's leveling is not connected to previous leveling. The check point should be a solid point, clearly defined, and not on the same structure as the control point. Locate it no more than one setup away. Examples of check points include a marked point on a building foundation, a double-headed nail driven into a telephone pole, or a marked point on a rock outcrop.

From a balanced setup between the control point and the check point, read the low scales of each rod. Write the readings in the "remarks" column of the recording form and compute the elevation difference, control point to check point. Also, write the difference on the field abstract for future reference.

When resuming work, observe a new difference between the control point and the check point, recording the difference as before. It should not differ from the first difference by more than 1.0 mm (0.0030 ft). If this check indicates movement has occurred (or that the rod was incorrectly placed on the control point), the original section leveled to the questionable point should be releveled in the opposite direction and closed.

If leveling is single run, set check points as follows: When leveling forward on the line, set one check point at the end of work each day; when leveling backward, set one check point at the start of work and set another at the end of work only if the previous day's leveling is not connected.

If leveling is double run, a check point need only be set at the end of a completed segment of line. It is practical to level the last section of the day in only one direction, setting no check point. The next day, level the section in the opposite direction, thus checking the stabil-

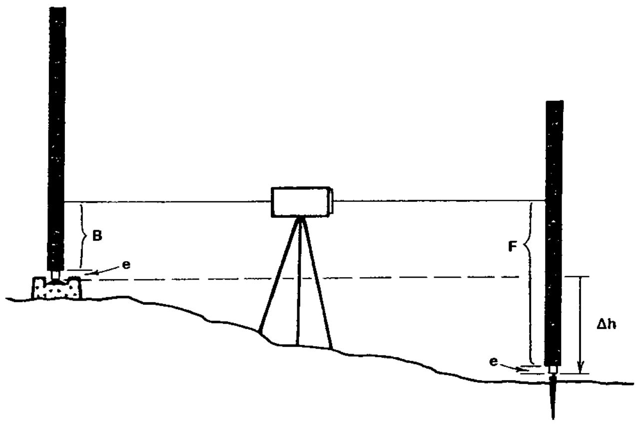


Figure 3-77.—When using spacers the height, e, of each spacer cancels when the elevation difference, Δh, is computed: $\Delta h=(B+e)-(F+e)=B-F$.

ity of the end mark, as well as satisfying the double-run requirement.

3.7.5 Leveling to Awkward Control Points

In the National network many existing control points are located where they cannot be easily leveled. These awkward points fall into two categories: (1) those mounted or etched on vertical surfaces, and (2) those mounted or etched on horizontal surfaces in positions requiring unusual equipment or procedures.

Locations where a control point may be found on a vertical surface include foundations and footings of large buildings and headwalls, and abutments of highway overpasses. The best way to level to a point in such a location is to intercept it directly. Another way is to use a short scale in lieu of the standard leveling rod.

Awkward locations for points on horizontal surfaces include the following: the top of a post or pedestal (too high to plumb the leveling rod properly and not large enough to support a rodman); a point less than 3 m (the length of a leveling rod) below an overhang or ceiling; and the underside of an overhang or eave. The best way to level to a point in the first location is to set the line of sight tangent to the highest point of the mark. At the second location, use a short rod. The third location requires that a rod be read while placed upside down against the point. Figure 3-78 illustrates each of these cases.

The following instructions explain how to level to awkward points.

Level directly to the point. Follow the micrometer leveling procedure. (See fig. 3-79 for an example.)

1. During the setup, which includes the control point, adjust the instrument height so that when the instrument is leveled the line of sight is within one full turn of the micrometer (one rod unit) of the control point.
2. To make the low-scale reading on the point, intercept it with the middle reticle line. Read the micrometer. Record the observation as "000." plus the micrometer reading.
3. To make the stadia reading, hold a short scale at the point. Read a stadia intercept on the low scale.
4. To make the high-scale reading on the point, intercept it with the middle reticle line again. Read the micrometer. Record the reading in the remarks column, and add the appropriate rod constant to the reading. Enter the sum as the high-scale reading.

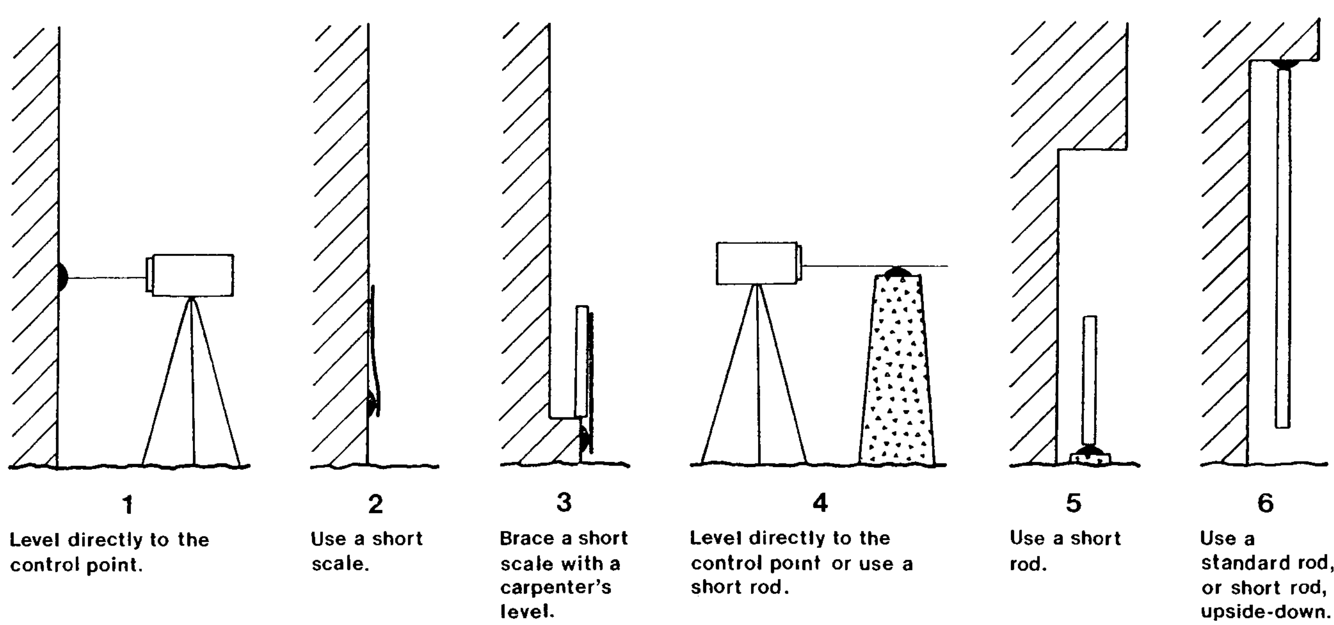


Figure 3-78.—Leveling to awkward control points.

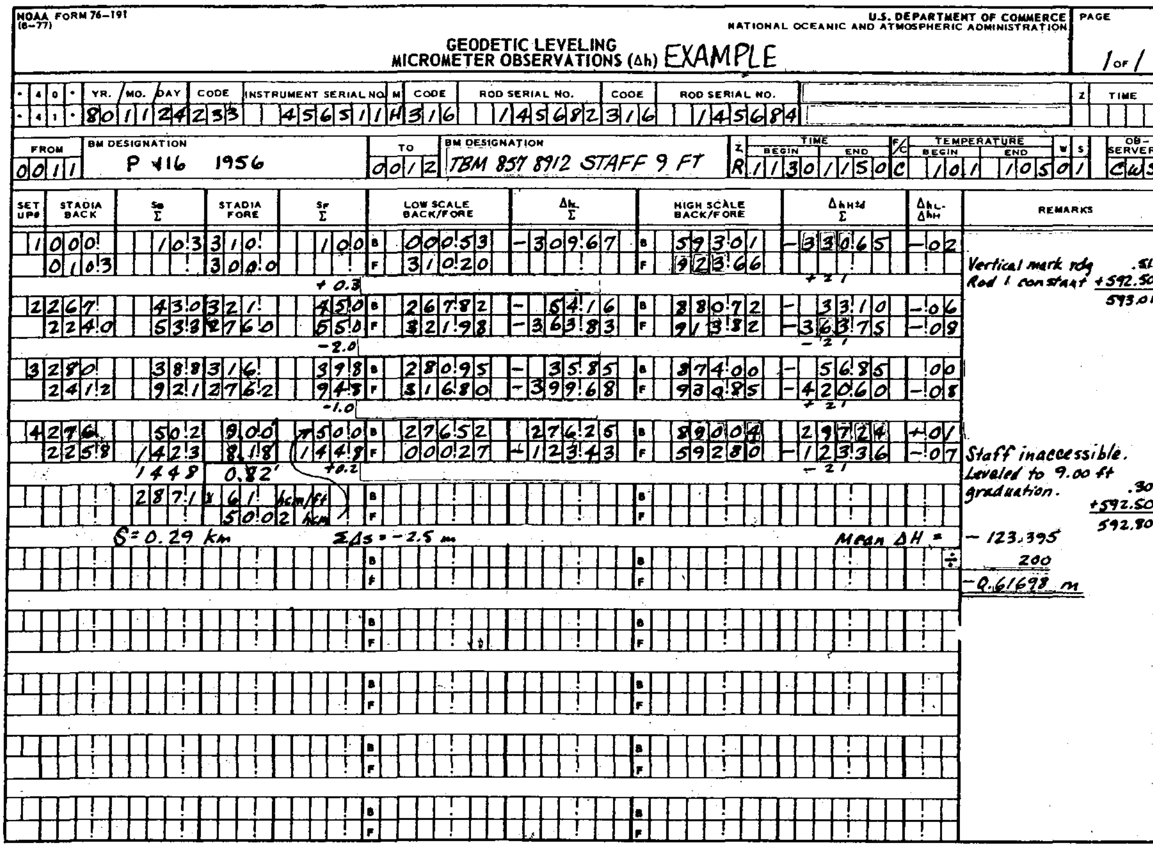


Figure 3-79.—Leveling directly to a control point.

5. Note in the remarks column that the point was intercepted directly.

Use a short scale. A short scale is shown in figure 3-80. For micrometer leveling use a double scale with a constant identical to that of rod 1.

1. During the setup, which includes the control point, adjust the instrument height so that when the instrument is leveled the line of sight is as close to the control point as possible.

2. Two people should hold the scale, one at the top and the other at the bottom. Center the index line of the scale on the point as illustrated in figure 3-81.

3. Plumb the scale by observing it through the instrument and aligning the scale with the vertical reticle line. When necessary, as in case 3 of figure 3-78, support and plumb the scale with a carpenter's level.

4. Observe the scale exactly as though it were rod 1 in a routine setup. Make a note that the short scale was used.

Note that the short scale must not be used with the weighted end placed on the control point. In such a position the scale is upside down. If it is used on both the backsight and foresight of the setup including the control point, the resulting elevation difference will have the wrong sign.

Use a short rod. A short rod should be identical to rod 1, except in length. The auxiliary scale included with Zeiss River Crossing Equipment is suitable. Use it as explained in section 4.3.1, "Auxiliary scale."

Read down from the point. If the rod or short scale cannot be plumbed in an upright position, it may be observed upside down. However, a special computation is necessary. (See fig. 3-82 for an example.)

1. Align the index line of the scale or place the base plate of the rod on the control point. Use a hand level to plumb the rod. If an upside-down, circular level is already mounted on the rod, be certain it is properly adjusted.

2. Follow the usual leveling routine, but on the upside-down scale count graduations down. If a micrometer is used, read it as usual. Record each upside-down reading with a negative sign.

3. If a micrometer is used, correct the upside-down readings. (With a computer this should be done automatically.) The micrometer effectively counts up, but the negative sign implies that it counted down; therefore, add twice the micrometer value to each reading. For example, if the reading is -313.61, add 2×0.61 ($=1.22$) to the value. The corrected reading becomes -312.39.

4. Compute the stadia distances and checks as usual.

5. Compute the elevation differences and reading checks as usual, remembering that the upside-down readings are negative. With the micrometer leveling procedure, the difference between the high-scale readings must be converted to $\Delta h_H \pm d$ as follows: If the upside-down scale is the backsight, add twice the rod constant. If the scale is the foresight, subtract twice the rod constant.

3.7.6 Tide and Water-Level Stations

Leveling connections from the national network to tide and water-level gages are important in two ways: The connections make it possible to refer elevations to the surface of a local body of water, and they provide a means to monitor relative changes in the levels of rivers, lakes, and oceans. Leveling to connect the national network to a gage is distinguished from leveling to monitor movement of the gage relative to the control points at the station. The latter type of work, conducted by the National Ocean Survey, is discussed in *Publication* 30-1, Manual of tide observations (Coast and Geodetic Survey 1965) and *User's Guide for the Establishment of Tidal Bench Marks and Leveling Requirements for Tide Stations* (Bodnar 1977).

A description and sketch of each station to be leveled should be attached to the log, showing the exact location of the points to be leveled and a recommended route through them. (See sec. 2.3.4, "Tide and water level stations.") Since the points are numerous and the stampings sometimes duplicated, identify the points with extreme caution. Level to the staff or gage and make a water-level measurement according to the following instructions.

Before leveling to a tide or water-level station, make an appointment to meet the station observer at the station. The station observer should be present to calibrate the water-level recorder.

Staff. Level to a staff by using a well-defined point on the staff to support the leveling rod, or level to the staff directly. The first method is preferred, but the second may be used if the staff is inaccessible.

Mounted on most tide staffs is a rod stop, a galvanized metal angle with a round-head bolt through the top. The highest part of the bolt is the point on which to place the rod. If possible, confirm the location of this point with the station observer. Include the height of the point, above or below the zero of the staff, in the designation.

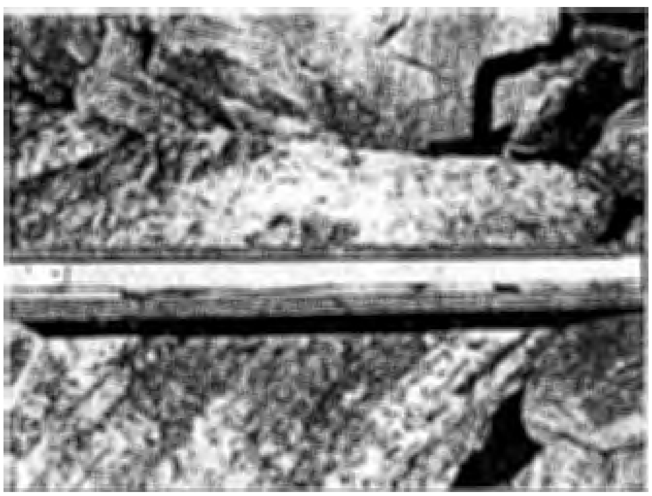


Figure 3-80.—Short scale.

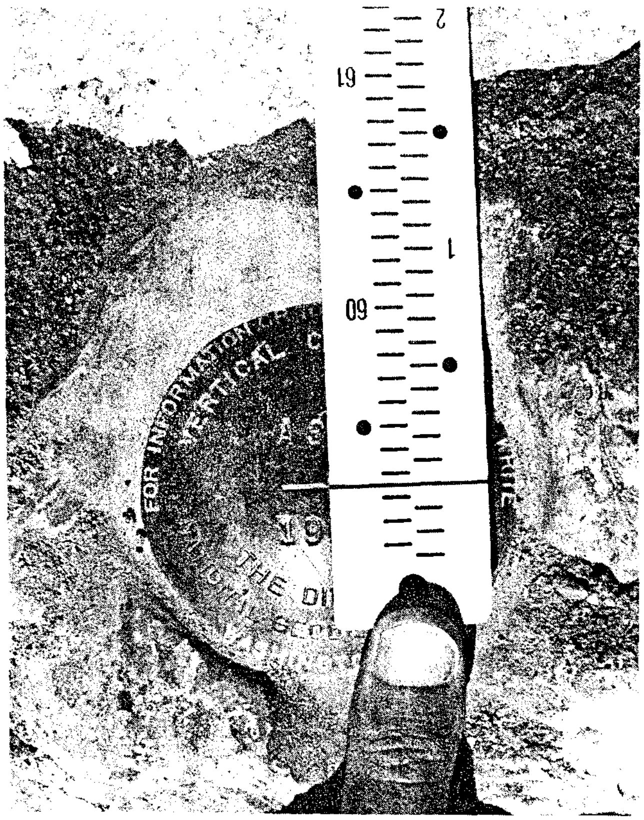


Figure 3-81.—Centering the index line of a short scale on a control point.

If no rod stop is available, or the staff is inaccessible, level directly to the staff. Follow the procedure described in the previous subchapter, using a convenient graduation as the control point. Include the graduation value in the designation (not in the readings). To make a stadia reading, count the number of staff graduations in the half stadia interval, estimating to tenths. Record this in the remarks column and convert it to rod units, then enter it as the stadia reading. For example, if the staff is graduated in tenths of feet and the rod units are half-centimeters, then 2.3 staff graduations are counted in the half stadia interval, corresponding to 0.23 ft. Multiply this by a conversion factor, 60.96 hcm/ft, to compute the stadia reading, 014.0 hcm.

Immediately after leveling to the staff, record on the recording form the water-level height in staff units. If possible, record the staff's identification date, which should be stamped on an attached brass tag.

Electric-tape gage. The electric-tape gage is usually located in a small house along with the water-level recorder. Level to the reading mark, directly if possible. It is usually a horizontal line, etched on the vertical edge of a metal plate supporting the electric-tape reel (fig. 3-76). Perform the accompaning water-level measurement as follows:

1. With one hand, hold the crank handle of the tape reel. With the other hand, release the ratchet pawl and hold it clear.

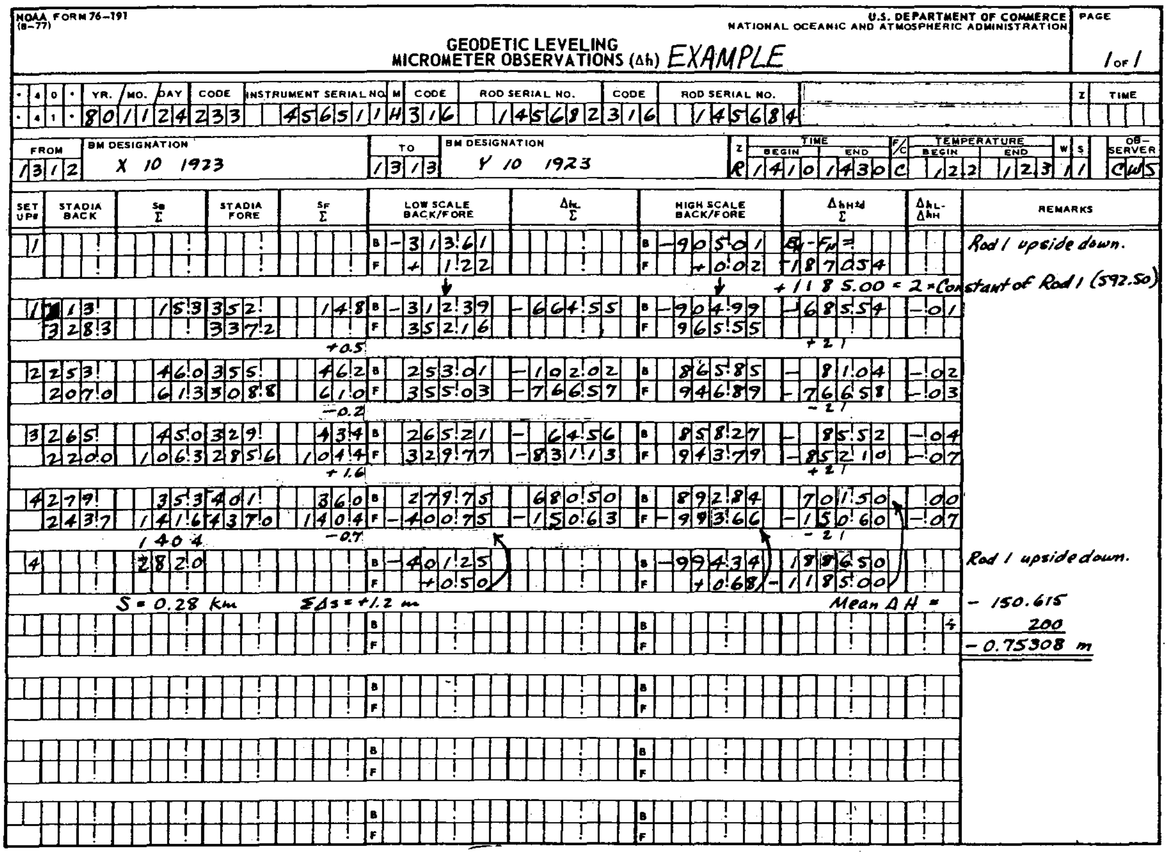


Figure 3-82.—Leveling to an upside-down scale.

2. Slowly lower the weighted tape until the voltmeter moves. This indicates that contact has been made with the water surface.
3. Read the tape value at the reading mark. Record in the "remarks" column the value (usually in feet), the time, and the gage identification date (which should be stamped on an attached brass tag).
4. Reel the tape back up to about 0.5 ft below the reading mark, and secure it.

3.7.7 Releveling and Closing Sections

Often in single-run leveling, and always in double-run leveling, two or more runnings of a section are required. Just as two elevation differences observed on a single setup must be checked for agreement, the elevation differences obtained from multiple runnings of a section must be checked for agreement. The check is analagous to closing a loop (sec. 5.4.3, "Checking loops") by summing the differences of a series of setups beginning and ending on the same point and comparing the result to a tolerance. Thus, the check for agreement is called "closing the section."

If all leveling errors were eliminated, section runnings would agree exactly. However, since small amounts of random error are always present, tolerances have been set to ensure that runnings retained in the uncorrected leveling data disagree by no more than an amount consistent with the precision of the prescribed leveling procedure.

When multiple runnings of a section disagree by more than the tolerance, one or more blunders may have occurred. Blunders occur when prescribed procedures are not followed. The section must be releveled to obtain a sample of data, excluding blunders, that more accurately provides the elevation difference.

Requirements for releveling and procedures for closing sections are discussed next.

Releveling. If any of the following situations occurs during single-run leveling, relevel the section in the opposite direction and close the section. Releveling may be required by the project office for other reasons as well, after the data have been checked and analyzed.

1. A procedural mistake is reported or suspected.
2. A tie check with previous leveling is not within tolerance.
3. One or more readings are missing or unclear, either in the computer or on a recording form.
4. The stamping recorded for a control point is missing or apparently incorrect, either in the computer or on a recording form.
5. One or more observations that do not meet specifications have been accepted because reading checks were computed improperly.

Closing the section. After a section is double run, check that the elevation differences from the two runnings agree within the tolerance for the order and class of the survey.

First, compute the difference, the misclosure, between the two elevation differences. Because they are opposite in sign, the misclosure is the sum of the backward running plus the forward running.

Then, compute the tolerance by the formula $T \times \sqrt{K}$, where T is the factor from table 3-1 for the order and class of the survey and K is the length of the section in kilometers. The length of the section is computed by totaling the distance leveled and dividing by the number of section runnings. For example, if the distance leveled in each running is 1.60 km and the survey is first order, class I, the tolerance is $3.00 \times \sqrt{1.60} = \pm 3.79$ mm.

If the misclosure is within the tolerance, additional runnings are not required. If it exceeds the tolerance, relevel and recheck the section to satisfy two criteria: (1) All runnings likely to contain blunders (as determined by the rejection procedure) are rejected, and (2) at least one forward running and one backward running are accepted. A standard rejection procedure is used to reject statistically unreliable runnings.

When releveling to close the section, alternate the direction of leveling on each running, to maintain an equal number of forward and backward runnings. If systematic error persists in the leveling, the mean of the elevation differences may be biased in favor of the running direction that is in the majority. Equalizing the number of runnings in each direction prevents this bias.

Notify the project office if more than four runnings are required to close the section. To prevent excessive releveling, check that the leveling rods are being placed on the control points correctly and check that the points themselves are stable. Record any unusual features of the points in the leveling data. Check and adjust the rods and the instrument, especially the circular levels. On subsequent runnings, try shortening the sighting distances, balancing setups more precisely, varying the routes, and changing the duties of the unit members. Try to make subsequent runnings when the air-temperature difference is negative and less than 1 °C (2 °F). (See sec. 3.6.1, "Measuring air-temperature differences.")

Rejection procedure. After three or more runnings of a section, check for agreement and reject blunders as follows. (See example in table 3-7.)

1. Compute the mean of all the runnings (disregard the signs), including those that have been rejected before.
2. Compute the differences between the mean and each running, the mean value minus each running value.
3. Compute the allowable difference from the mean. It is based on the order and class of the survey, the number of runnings that were meaned, and the length of the section. The formula is $t \times \sqrt{K}$, where t is the appropriate factor from table 3-1 for the order and class of the survey, and K is the section length in kilometers. For example, if the survey is first order, class I, four runnings were meaned, and the section length is 1.51 km, and the allowable difference is $2.33 \times \sqrt{1.5.} = \pm 2.85$ mm.
4. Compare the differences from step 2 to the allowable difference. If any runnings exceed it, reject only the running that exceeds the allowable difference by the largest amount.
5. Check the runnings to see if there are at least one forward and one backward running remaining. If this is the case, and no running was rejected in step 4, the section is closed. If a running was rejected in step 4, compute a new mean with the remaining runnings and return to step 2. If only runnings made in one direction remain, relevel the section and begin again at step 1.

Notice that certain runnings, rejected when the number of runnings is three or four, may be accepted when the number increases. This is acceptable because the accuracy of the mean improves with a larger sample of data; it becomes easier to recognize if a running is different enough to be considered a blunder.

3.8 Field Records

Original records of field observations are the primary source of information for all future analysis and adjustment of a survey. As such field records must accurately and completely present the results and conditions of leveling. The records may include both data recorded on forms and printouts of data recorded in computer memory. To be incorporated into the National Geodetic Vertical Control Network, they must provide data sufficient to meet the requirements of the Federal Geodetic Control Committee, which are published in *Input Formats and Specifications of the National Geodetic Survey Data Base* (Pfeifer and Morrison 1980: vol. II, ch. 6). Whatever form is used, the records must be carefully preserved by the leveling unit until they are submitted to the project office. Their preparation and submittal are the subjects of this subchapter.

3.8.1 Recording Observations

Leveling observations can be collected by using one of two methods. Observations may be either (1) written on a recording form, or (2) keyed into computer memory and recorded on a magnetic tape or disk, supplemented with a backup form if required. The computer-recording method is preferred to reduce errors and to simplify checking and archiving the data. Nevertheless, both methods require that the recorder prepare clear, correct, and complete records.

When observations are written, the recording form serves both as a record and as a reminder of the checks and computations to be made. When observations are keyed, the backup recording form serves as a record to verify information that cannot be checked by the computer. Use standard recording forms, as illustrated in the figures in this manual.

Be sure to record correct identifying information. This cannot be overemphasized. Mistakes in any of the following entries will cause false elevation differences to be computed when leveling data are processed.

Table 3-7.—Example of rejection procedure for first-order, class II leveling

Date	Direction (km)	Distance (m)	Elevation difference (mm)	Difference from mean	Explanation
10/11	F	0.82	−2.43152		Two runnings of a section are made.
10/12	B	0.83	2.43752		Their difference is $B+F=6.00$ mm.
					The tolerance is $3\times\sqrt{0.82}=\pm2.72$ mm.
					Since the difference exceeds the tolerance,
					the section is releveled.
10/11	F	0.82	−2.43152	+1.67	The mean of the three runnings and the differences
10/12	B	0.83	2.43752 R	−4.33	from the mean are computed. The allowable
10/12	F	0.82	−2.43052	+2.67	difference is $2.10\times\sqrt{0.82}=\pm1.90$ mm. The second
			Mean=2.43319		and third runnings are both outside the allowable.
					Rejecting the worst leaves only two forward
					runnings, so the section is releveled.
10/11	F	0.82	−2.43152	+2.90	The mean and differences are computed for all four
10/12	B	0.83	2.43752	−3.10	runnings. The allowable is $2.33\times\sqrt{0.82}=\pm2.11$ mm.
10/12	F	0.82	−2.43052 R	+3.90	All are outside the allowable. Rejecting only the
10/13	B	0.82	2.43810	−3.68	worst, the third running, a new mean is computed.
			Mean=2.43442		The allowable for three runnings is, as before
	F	0.82	−2.43152 R	+4.19	±1.90 mm. Rejecting the worst, (the first)
	B	0.83	2.43752	−1.81	leaves only two backward runnings, so the section
	B	0.82	2.43810	−2.39	is releveled.
			Mean=2.43571		
10/11	F	0.82	−2.43152	+3.64	The mean and differences are
10/12	B	0.83	2.43752	−2.36	computed for the five runnings. The allowable
10/12	F	0.82	−2.43052 R	+4.64	is $2.48\times\sqrt{0.82}=\pm2.25$mm. Rejecting the
10/13	B	0.82	2.43810	−2.94	worst, a new mean is computed.
10/13	F	0.83	−2.43812	−2.96	
			Mean=2.43516		
	F	0.82	−2.43152 R	+4.80	The allowable for four runnings is, as before,
	B	0.83	2.43752	−1.20	±2.11 mm. The first running is rejected, and
	B	0.82	2.43810	−1.78	then a new mean computed. The allowable
	F	0.83	−2.43812	−1.80	for three runnings, is as before, ±1.90 mm.
			Mean=2.4632		Since all differences are within the allowable,
	B	0.83	2.43752	+0.39	and at least one forward running and one
	B	0.82	2.43810	−0.19	backward running are accepted,
	F	0.83	−2.43812	−0.21	the section is closed!
			Mean=2.43791		

The recorded stamping, or designation of each control point, and its corresponding survey-point serial number determine the order in which the line is abstracted. This order must correspond to the way the line was actually leveled.

The recorded date, time zone, and time must be accurate because observations are stored and sorted chronologically. Corrections for collimation error, refraction, tidal accelerations, and scale error are applied accordingly. Time zones should be coded according to the alphabetical system of the U.S. Navy. (See table 3-8 for the codes applying to North America.)

Finally, recorded descriptions of the equipment and the corresponding serial numbers determine which collimation and calibration corrections are to be applied. The make and model of rods and instruments should be specified by code if possible. (See sec. 3.2.2, "Equipment.")

Write observations and remarks neatly in ink. Never recopy original records. If a mistake is made when computing, draw a straight line through the error and write the correct value above it. (See fig. 3-71.) If a mistake is made when recording a rod reading, reobserve the

Table 3-8.—U.S. Navy time zone designations

Standard time		Daylight time		Time meridian	Time Zone description	U.S. Navy designation
Atlantic	AST	Eastern	EDT	60W	+ 4	Q (Quebec)
Eastern	EST	Central	CDT	75W	+ 5	R (Romeo)
Central	CST	Mountain	MDT	90W	+ 6	S (Sierra)
Mountain	MST	Pacific	PDT	105W	+ 7	T (Tango)
Pacific	PST	Yukon	YDT	120W	+ 8	U (Uniform)
Yukon	YST	AK/HI	HDT	135W	+ 9	V (Victor)
AK/HI	HST	Bering	BDT	150W	+10	W (Whiskey)

setup and record the new readings on a new line. Crossed-out or illegible readings in written data are like readings missing from a computer-recorded tape: the entire section must be rejected.

When computing, round results to the appropriate number of decimal places. Elevation differences should usually be expressed in meters to five decimal places (four decimal places in three-wire leveling). Distances should be expressed in meters to one decimal place or in kilometers to two decimal places. Some calculators do not automatically round the result when displaying fewer decimal places than were computed. If using such a calculator, look at one extra decimal place and then round the result. If the number to be rounded ends with the numeral "5", round to the nearest even number. For example, the collimation factor +0.0135 mm/m rounds to +0.014 mm/m, and the elevation difference −2.148145 m rounds to −2.14814 m.

Compute reading checks carefully. If they are computed incorrectly and one or more setups are accepted which do not meet the tolerance, the section must be releveled. This type of blunder can occur anytime that observations are written; it can also occur when preliminary information is stored incorrectly in a computer. The observer should check for such blunders as work progresses.

Use the "remarks" column (or computer comment lines) freely. Record any activity or event which may affect the quality of leveling, such as frequent failure of reading checks, unusual atmospheric conditions, or difficulties with the computer-recording equipment. Give a complete description of the situation: time at which the activity occurred, the setup number, what happened, and the way it was handled. Be specific. Include the names of personnel involved and the serial numbers of the affected equipment.

At the end of the day, the unit chief should check and initial the recorder's work. Be sure to check that the designations and survey-point serial numbers are correct, as well as the other identifying information. If mistakes are found that may require editing of a data tape or disk, do not attempt to correct them. Instead, note and explain the correction required on a comment record in the computer or on the appropriate backup form.

3.8.2 Computer-Recording Equipment

Portable computers have revolutionized recording procedures. Specially designed computer-recording equipment permits observations to be checked at the time they are made, nearly eliminating reading and recording blunders from accepted data. The equipment also permits all field observations to be stored in data packages that can be easily transferred to more powerful computers, simplifying the subsequent checks, correction, analysis, and adjustment of the data.

To obtain the greatest benefit from a computer, the equipment selected should meet the general requirements given in this section. To collect data reliably, the computer and its power supply must be maintained properly. To collect data accurately, the computer must be programed and operated correctly.

General requirements. Computer-recording equipment for leveling should be manufactured for the field environment. The equipment must withstand frequent exposure to dust, water, and vibration. It should be portable, weighing no more than 10 kg. It should be capable of operating continuously from an internal power supply for at least 6 hours and from both 115 V AC and 12 V DC external power supplies. It should not lose stored data if the power supply is interrupted or exhausted.

Data may be recorded and stored on a tape or disk, or in the computer memory. If recorded in memory, data must be easily retrieved for rapid transfer onto a tape or disk or by telephone into a central computer. The output format should be compatible with the computer facilities of the project office.

The computer should be programmable and should have an alphanumeric keyboard with a numeric cluster. The keyboard permits the stampings on control points to be entered in addition to survey-point serial numbers. It also permits direct entry of remarks and simplifies the coding of other information. The display should be easy to read and should have at least 32 characters. At least 20,000 bytes of memory should be available for storing both programs and data.

Maintenance. One person in the leveling unit (usually the recorder) should be responsible for maintaining the computer-recording equipment. Carefully follow the instructions provided with the equipment. Protect it from exposure to dust, water, and vibration. The more self-contained and rugged the equipment is, the better; however, it must still be kept clean and be handled with care.

Sticky or dirty keys on a computer keyboard make recording difficult. Protect the keyboard by taping a thin sheet of plastic over it. When leveling data are recorded on magnetic tapes or disks, the data are highly vulnerable to alteration or erasure caused by dirty recording apparatus. Therefore, learn the proper method for cleaning the equipment, and clean it at least weekly.

Data may also be altered or erased if the equipment is exposed to shock or excessive vibration during the recording process. As a result, do not transport the equipment while data are being recorded. When data are stored in the computer memory, dropping the equipment or striking it against objects may erase or render inaccessible the results of an entire day's work. For the same reason, keep magnetic material away from the computer and data packages.

Battery care. For computer-recording equipment to operate reliably, the batteries must be properly charged and maintained. The nickel-cadmium batteries used in most portable computers should be recharged at regular intervals, either nightly or as instructed. They should be protected from overcharging by a voltage regulator. They should also be "cycled," or discharged completely and recharged, at least once a month and after a period of storage. They should be stored in a cool, dry place.

Built-in batteries, if properly charged, should provide enough power for a day's work. However, to prevent loss of time if they fail, keep two alternate power supplies available: a spare battery pack and a hookup to the battery of the leveling vehicle. Maintain the spare pack in the same way as the built-in batteries.

Data packages. For submittal to the project office, leveling data are normally stored on a tape or disk. Use a separate tape or disk to record each day's data. If data are recorded directly onto a tape, precondition the tape before work begins. Keep a supply of clean tapes in the leveling vehicle, in case a tape is defective. If data are stored in computer memory, print out or record the data onto a tape or disk immediately after the day's leveling is completed. Instructions for submitting data packages appear in section 3.8.4.

Programs. When recording observations into a computer, use only the standard programs specified in the project instructions. The recorder should become thoroughly familiar with the programs by studying accompanying documentation and instructions. Although knowledge of programing techniques is not essential, it is helpful when troubleshooting in the field.

Recording programs should prompt the recorder for the necessary data at the appropriate time in the observing routine. They should incorporate as many checks for blunders as possible. They should record observations twice on tapes or disks, to increase the chance of retrieving at least one complete set of observations if some data are destroyed or made inaccessible by imperfections in the system. For leveling data to be included in the National Geodetic Vertical Control Network, the programs should format the data as described in *Input Formats and Specifications of the National Geodetic Survey Data Base* (Pfeifer and Morrison 1980: vol. II, ch. 6).

Depending on the capabilities of the equipment, a system of programs can control the observing routine at one of three levels: the setup, the section, or the day.

The most basic system prompts the recorder for the observations, checks them for blunders, and records them at each setup. Using a backup form as a guide, the recorder must enter the running records for each section and the survey-equipment records for each day. Separate programs are used for each order and class of survey, and to observe and compute the collimation and compensation checks. Although this system nearly eliminates blunders at each setup, it allows blunders in section and equipment information to remain undetected in the field. Therefore, a backup recording form, such as shown in figure 3-70, must be maintained to permit verification of the data.

More blunders can be eliminated if the system includes prompts and checks for running records and survey-equipment records. In such a system the recorder must again select the appropriate program to use, but programed control of the observing routine is extended to include the entire section.

The most sophisticated system should prompt for and check all the data collected by the leveling unit each day. Prompts should include the following: date, line number, survey-equipment record, daily equipment checks (such as the collimation check), weekly checks (such as the compensation test), running records, and observations. Subroutines, such as the collimation check, should be accessed by the master program when required. Constants, such as the collimation factor, should be computed and retained in computer memory from day to day. Separate options should be available for each order and class of survey. The results of multiple runnings of a section should be checked automatically.

Precautions. When using computer programs, the recorder must observe the following precautions.

Learn to glance at each displayed rod reading immediately after keying it and before entering it. Gross mistakes (such as misplaced decimal points and double-keyed numbers) may then be deleted and corrected before the entire setup is rejected.

Never write over or erase the data for a setup. Enter a remark if necessary. If a section must be terminated, enter a rejection record. Learn how to avoid erasing data accidentally. Learn how to reprogram the computer if the program or data are lost because of a battery change or other power failure.

Avoid using computer-recording equipment near a source of high-energy radio-frequency emissions. Depending on their frequency, such sources as air-navigation, television, and radio transmitters may interfere with normal computer operation.

3.8.3 Field Abstract

The field abstract is a complete summary of the elevation differences measured between the control points in a line. On the abstract, points are listed in the order in which they are connected by leveling. For every pair of points, the elevation differences from one or more runnings are recorded. These are added to the elevation assigned to the beginning point, to obtain the field elevation for each point on the line.

If all observations are written on recording forms, a field abstract must be prepared and submitted to the project office. If observations are recorded with a computer, the abstract, although not required, should still be prepared. Since observations must be submitted daily, the abstract provides the leveling unit with the only record of which sections require additional leveling and which portions of the line have been completed. Multiple runnings, check points, and ties can be recorded and checked on the abstract.

The abstract, by itself, is not an acceptable substitute for the actual observations, no matter how they are recorded. It must always be accompanied by the original recording forms or printouts of the original observations recorded on a computer.

Instructions. For leveling to be included in the national network, use NOAA Form 76-188, "Geodetic Leveling Line Identification" (fig. 3-83), and NOAA Form 76-187, "Geodetic Leveling Field Abstract." A single-run portion of line (fig. 3-84) is abstracted in figure 3-85. A double-run portion of line (fig. 3-86) is abstracted in figure 3-87. Prepare and compute the abstract as follows:

1. Prepare a cover sheet for the abstract. Only the portions of the form that are circled in the example need be filled out by the leveling unit. When observers, recorders, rods, or instruments change, add the initials or serial numbers and dates of change to the form. Submit the abstract form to the project office when the leveling line has been completed.

2. On the abstract, enter the accession number for the line, if known. If the accession number is not available, enter the line number, job code, and project title. Enter the line title, corresponding to the direction of leveling. Be sure to circle the appropriate units of distance and elevation difference. Include this information on every sheet of the abstract.

3. On the first sheet, skip the first line. Enter the survey-point serial number and designation of the beginning point on the second line. Enter the beginning distance, 0.00, in column D. In column H, enter the adjusted elevation from the most recently published description of the point. If the point is new to the national network, do not enter a beginning distance or elevation until a point with a published elevation is leveled. The beginning point and elevation are normally assigned by the project office, as described in section 5.4.2.

4. Enter the results for each section on subsequent records. The ending point of one section must correspond to the beginning point of the next. Write the designation of the beginning point for a forward running on line *30*. Write the survey-point serial number and designation of the ending point on the line labeled "TO." As shown in columns C, D, E, and F, write the date, direction, distance, and elevation difference for each running. Always write the results of the first forward running on line *30*. If more than two runnings are made, write the designation of the ending point on the next available line labeled "TO."

5. Check that sections with more than one running are closed. Indicate rejected runnings with the letter "R" in red ink.

NOAA FORM 76-188
(8-76)

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

GEODETIC LEVELING LINE IDENTIFICATION

	HGZ NUMBER	LINE/PART	RELEVEL	FIELD SURVEY DATES YEAR	MO.	DAY	TO YEAR	MO.	DAY	UNITS	TOLERANCE FACTOR	ORDER & CLASS	STATE CODES	PARTY CHIEF	AGENCY RESPONSIBLE FOR SURVEY
10	L24133	5	R	1976	05	11	1976	12	09	MM	3.0	1 1	LA	BKM	NGS

PROJECT/LINE TITLE

11 LEVELING FOR CORPS OF ENGINEERS NEW ORLEANS DISTRICT

12 LOTTIE TO 9.8 KM (6.1 MILES) W OF KROTZ SPRINGS

13

14

COMMENTS

15

15

TASK NUMBER: R362 5200

CHIEF OF PARTY: BKM

OBSERVERS: RLW

RECORDERS: DBC

RODS: 316-121178; 316-127951

INSTRUMENTS: 231-90825

☆ U.S. Government Printing Office: 1976—765-005/1082, Region No. 6

Figure 3-83.—Leveling line identification.

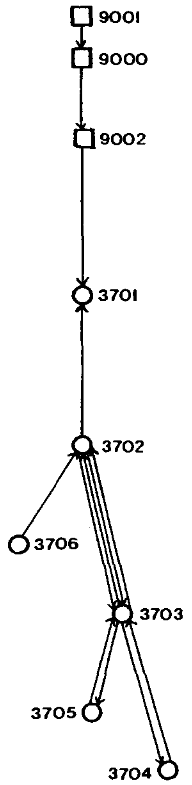
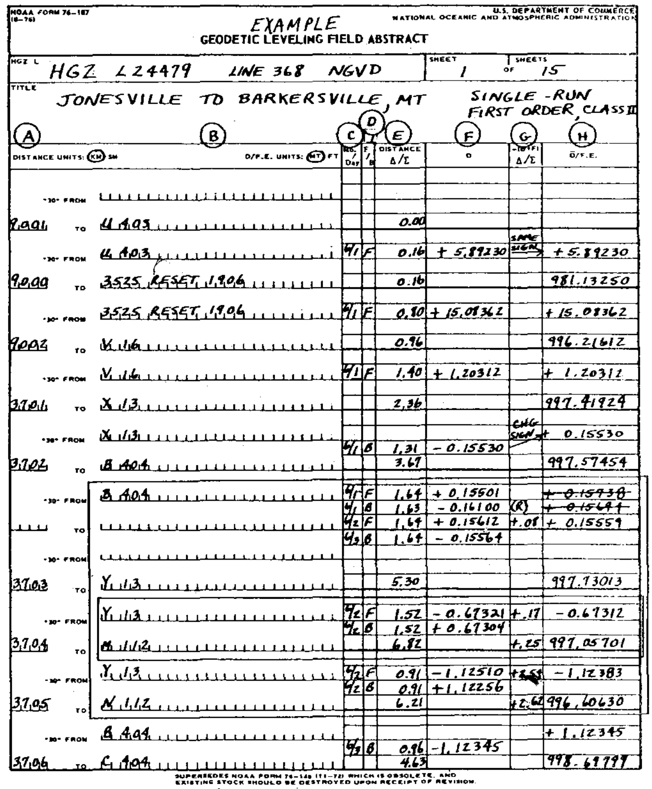


Figure 3-84.—Sketch of single-run leveling.



NOAA FORM 76-187 (8-76)

EXAMPLE

GEODETIC LEVELING FIELD ABSTRACT

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

HGZ L: HGZ L24479 LINE 368 NGVD — SHEET 1 OF SHEETS 15

TITLE: JONESVILLE TO BARKERSVILLE, MT — SINGLE-RUN FIRST ORDER, CLASS II

DISTANCE UNITS: KM SM — D/F.E. UNITS: MT FT

(A)		(B)	(C) No./Day	(D) F/B	(E) DISTANCE Δ/Σ	(F) D	(G) ~(B+F) Δ/Σ	(H) D̄/F.E.
	30 FROM							
9001	TO	U 403			0.00			
	30 FROM	U 403	4/1	F	0.16	+ 5.89230	SAME SIGN	+ 5.89230
9000	TO	3525 RESET 1906			0.16			981.13250
	30 FROM	3525 RESET 1906	4/1	F	0.80	+ 15.08362		+ 15.08362
9002	TO	V 16			0.96			996.21612
	30 FROM	V 16	4/1	F	1.40	+ 1.20312		+ 1.20312
3701	TO	X 13			2.36			997.41924
	30 FROM	X 13					CHG SIGN	+ 0.15530
			4/1	B	1.31	− 0.15530		
3702	TO	B 404			3.67			997.57454
	30 FROM	B 404	4/1	F	1.64	+ 0.15501		~~+ 0.15738~~
			4/1	B	1.63	− 0.16100	(R)	~~+ 0.15644~~
	TO		4/2	F	1.64	+ 0.15612	+.08	+ 0.15559
			4/3	B	1.64	− 0.15564		
	30 FROM							
3703	TO	Y 13			5.30			997.73013
	30 FROM	Y 13	4/2	F	1.52	− 0.67321	+.17	− 0.67312
			4/2	B	1.52	+ 0.67304		
3704	TO	M 112			6.82		+.25	997.05701
	30 FROM	Y 13	4/2	F	0.91	− 1.12510	[illegible]	− 1.12383
			4/2	B	0.91	+ 1.12256		
3705	TO	N 112			6.21		+2.62	996.60630
	30 FROM	B 404						+ 1.12345
			4/3	B	0.96	− 1.12345		
3706	TO	C 404			4.63			998.69799

SUPERSEDES NOAA FORM 76-149 (11-72) WHICH IS OBSOLETE, AND EXISTING STOCK SHOULD BE DESTROYED UPON RECEIPT OF REVISION.

Figure 3-85.—Abstract of single-run leveling.

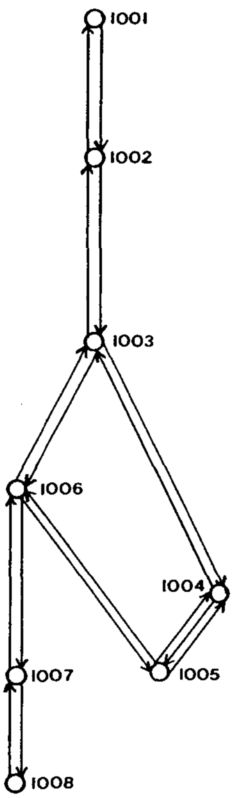
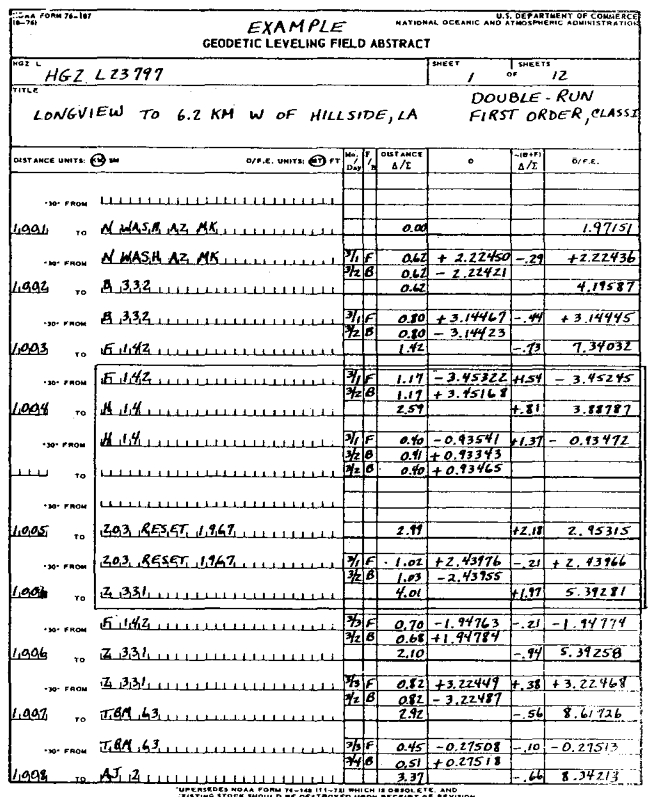


Figure 3-86.—Sketch of double-run leveling.



NOAA FORM 76-187 (8-76)

EXAMPLE

GEODETIC LEVELING FIELD ABSTRACT

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

HGZ L: HGZ L23797 — SHEET 1 OF SHEETS 12

TITLE: LONGVIEW TO 6.2 KM W OF HILLSIDE, LA — DOUBLE-RUN FIRST ORDER, CLASS I

DISTANCE UNITS: KM SM — D/F.E. UNITS: MT FT

			No./Day	F/B	DISTANCE Δ/Σ	D	~(B+F) Δ/Σ	D̄/F.E.
	30 FROM							
1001	TO	N WASH AZ MK			0.00			1.97151
	30 FROM	N WASH AZ MK	3/1	F	0.62	+ 2.22450	−.29	+2.22436
			3/2	B	0.62	− 2.22421		
1002	TO	B 332			0.62			4.19587
	30 FROM	B 332	3/1	F	0.80	+ 3.14467	−.44	+ 3.14445
			3/2	B	0.80	− 3.14423		
1003	TO	F 142			1.42		−.73	7.34032
	30 FROM	F 142	3/1	F	1.17	− 3.45322	+1.54	− 3.45245
			3/2	B	1.17	+ 3.45168		
1004	TO	H 14			2.59		+.81	3.88787
	30 FROM	H 14	3/1	F	0.40	− 0.93541	+1.37	− 0.93472
			3/2	B	0.41	+ 0.93343		
	TO		3/2	B	0.40	+ 0.93465		
	30 FROM							
1005	TO	203 RESET 1967			2.99		+2.18	2.95315
	30 FROM	203 RESET 1967	3/1	F	1.02	+ 2.43976	−.21	+ 2.43966
			3/2	B	1.03	− 2.43955		
1006	TO	Z 331			4.01		+1.97	5.39281
	30 FROM	F 142	3/3	F	0.70	− 1.94763	−.21	− 1.94774
			3/2	B	0.68	+ 1.94784		
1006	TO	Z 331			2.10		−.94	5.39258
	30 FROM	Z 331	3/3	F	0.82	+ 3.22449	+.38	+ 3.22468
			3/2	B	0.82	− 3.22487		
1007	TO	TBM 63			2.92		−.56	8.61726
	30 FROM	TBM 63	3/3	F	0.45	− 0.27508	−.10	− 0.27513
			3/4	B	0.51	+ 0.27518		
1008	TO	AJ 2			3.37		−.66	8.34213

SUPERSEDES NOAA FORM 76-149 (11-72) WHICH IS OBSOLETE, AND EXISTING STOCK SHOULD BE DESTROYED UPON RECEIPT OF REVISION.

Figure 3-87.—Abstract of double-run leveling.

6. Compute the mean of the accepted elevation differences for each section. Write the mean in column H, on the same line as the first forward running. Assign to the mean the sign of the forward running.

7. If there are two runnings, compute the difference in millimeters between the forward and backward runnings. Enter the difference in column G, with the sign of whichever number is smaller. If there are more than two runnings, compute the difference between the means of the forward and backward runnings.

8. Draw a box in red ink around every spur of the line, as shown in the example. For a spur-on-a-spur, draw an additional, nested box. If any sections connect to form a loop, show the longer route as a spur and the shorter route as part of the main line.

9. To the beginning distance, add the shortest distance for each of the sections leveled so far. Write the total in column E, on the dark line across from the designation of the point leveled to. When the end of a spur is reached, be sure to return to the point at the base of the spur to continue computing the distance along the main line.

10. Similarly, to the beginning elevation add the elevation differences of the sections leveled so far. Write the total in column H, on the same line as the distance. When the end of a spur is reached, return to the point at the base of the spur to continue computing the elevations of points on the main line. Total column G in the same manner.

11. Start subsequent sheets of the abstract on the first line, beginning with the point from the last line of the previous sheet.

3.8.4 Submitting Records to the Project Office

The unit chief is responsible for field records until they have been submitted to the project office. Submit records of observations daily. Submit completed abstract sheets weekly, together with the weekly status reports.

Package each day's records separately. Thus, if a package is lost or damaged, only one day's work at most need be repeated. Label the package with the following information: accession number or line number, date, initials of the unit chief, and, if data are recorded on a tape or disk, the serial number of the recording equipment. If many units are operating from the same project office, each unit should use labels of a unique color, so the data packages can be easily distinguished.

When working near the project office, submit records in person at the end of work each day. When working at some distance from the office, transmit observations recorded in a computer directly over a telephone line. Send other types of records, including backup material (duplicates of transmitted data) by certified mail. Retain the receipts and send them with the weekly status report. Arrange the work schedule to allow visits to the post office when the facility is open. If this is not practical, send records by first class mail. When records are needed immediately, as at the end of a project, make special arrangements to send them to the office by courier or express mail.

Magnetic tapes and disks. Data recorded on magnetic tapes or disks are particularly vulnerable to alteration or loss. This may occur especially if they are stored for long periods under field conditions or exposed to X-rays or strong magnetic fields while enroute to the project office.

If data packages must be collected and stored in the field for any length of time, keep them in a rigid, nonmagnetic container. When they are sent through the mail, enclose them in an appropriate mailer. Magnetic tapes may be sent in aluminum boxes and placed with the accompanying backup forms inside preaddressed, padded envelopes. Magnetic disks should be similarly packaged.

3.9 References

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Coast and Geodetic Survey, 1965: Manual of tide observations. *Publication* 30-1, National Ocean Survey (C233), NOAA, Rockville, Md. 20852, 77 pp.

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Chapter 4

RIVER OR VALLEY CROSSING

4.1 Introduction

In geodetic leveling, when the sighting distance and setup imbalance cannot be confined to the maximum specified for the order and class of the survey, a special procedure must be followed. Because it is most commonly required when a line of leveling crosses a river, the procedure is termed a river crossing. However, the procedure may be required over any type of terrain. In the following paragraphs, general procedures and requirements for a river crossing are explained. Detailed instructions for two river-crossing routines are presented in the remainder of the chapter.

Leveling errors that are a function of sighting distance are greatly magnified when an unusually long sight is observed, as shown in figure 4-1. The graduations on a geodetic leveling rod either cannot be seen or do not provide a sufficiently definite image, especially when shimmer is present. As a result, pointing error is greatly increased. Since the sighting distances of such a setup usually cannot be balanced, the effects of collimation error and curvature cannot be limited in the usual manner. (See sec. 3.1.2, "Leveling instrument.") Furthermore, even if the setup can be balanced, refraction may be large and unpredictable because of variations in the atmospheric conditions along the lines of sight, particularly if the line of sight passes over water.

Intercepting targets. To reduce pointing error when the sighting distance exceeds the limits for the order and class of the survey, a series of repeated scale readings should be made for each required observation. Pointing error may be further reduced by observing the crossing under favorable environmental conditions. Even so, a scale with line graduations does not provide an image which can be intercepted with sufficient precision at sighting distances greater than about 80 m (260 ft). Similarly, a scale with 1-cm block graduations does not provide a sufficiently precise image at distances over about 160 m (530 ft).

A better image is provided by a target or directional light, attached to the rod or other vertical support. A target should provide an image similar in size to that of a scale graduation viewed at about 50 m (165 ft) through the instrument. It should have a sharply defined shape and be symmetrical about the horizontal axis. An isosceles triangle is practical because the slopes of the sides define the center of the target throughout a wide range of sighting distances.

A precise method for making scale readings employs two targets. One target is placed above and the other below the point where the horizontal line of sight is estimated to intercept the support. The line of sight is then tilted by the amounts necessary to intercept each target and to return to a horizontal position. The inter-

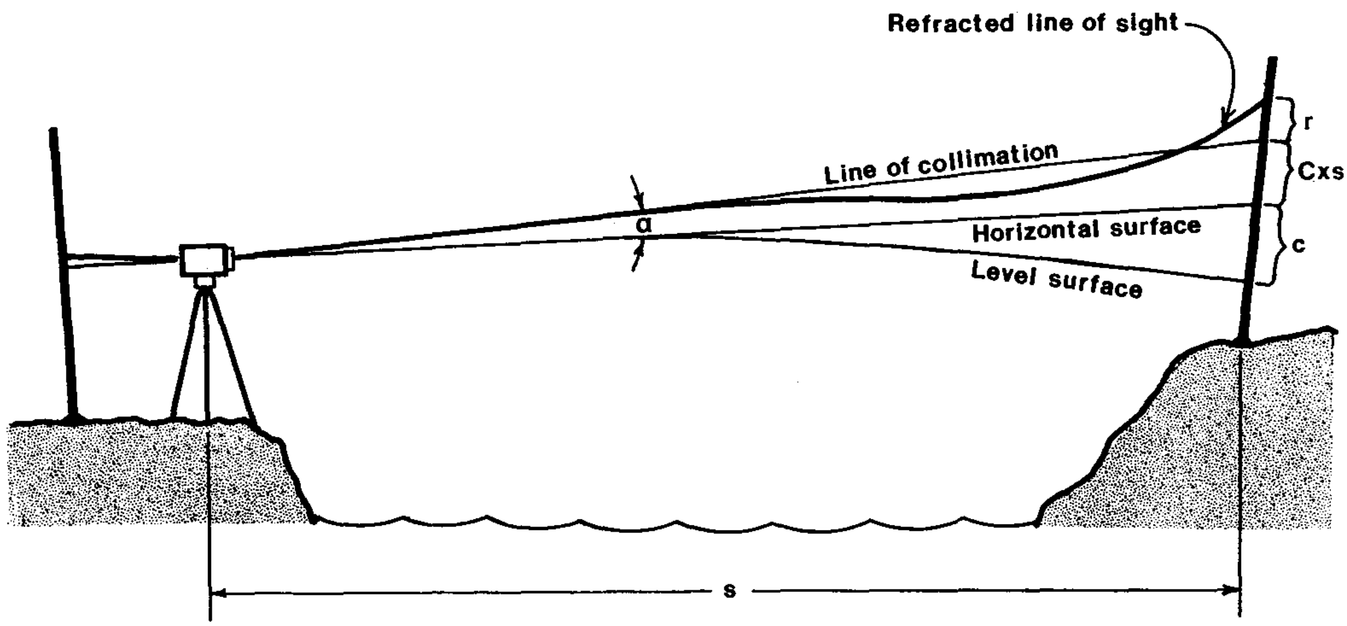

Figure 4-1.—Curvature, *c*, refraction, *r*, and the effect, $C \times s$, of collimation error, α, are magnified by a long sighting distance.

cept of the horizontal line of sight is computed from the proportional relationship between the distance intercepted on the support and the vertical angle of tilt, as illustrated in figure 4-2.

Three tilt measurements are made: (1) when the line of sight intercepts the upper target, a; (2) when it is horizontal, r; and (3) when it intercepts the lower target, b. The distance, $F-T$, between the unknown foresight intercept and the lower target is related to the angle, $b-r$, from lower tilt to horizontal in approximately the same ratio as the distance, D, between the targets is related to the angle, $b-a$, from lower to upper tilt (provided both angles are measured in radians):

$$(F-T)/(b-r)=D/(b-a).$$

Solving for the foresight intercept,

$$F=T+[D\times(b-r)/(b-a)].$$

The sighting distance, s, is computed without stadia observations by the following equation:

$$s=D/[\tan(b-r)+\tan(r-a)].$$

Since the angles of tilt are very small,

$$\tan(b-r)+\tan(r-a)\cong(b-r)+(r-a)\cong(b-a)_{\text{radians}}.$$

Thus, to a sufficient approximation,

$$s=D/(b-a)_{\text{radians}}.$$

This method for making scale readings is suitable for the longest sighting distances and is employed by the river crossing routines outlined in subchapters 4.3 and 4.4. For the most precise results, the angular difference $b-a$ should be between 100″ and 250″, and D should be no more than 1.0 m (3.3 ft). This is possible at sighting distances of 2.0 km (1.2 mi) or less.

Reciprocal setups. To limit the effects of curvature, two setups should be observed in opposite directions, one from each side of the crossing (fig. 4-3). Each setup should be unbalanced by the same amount, $\Delta s=s_B-s_F$. Such reciprocal setups are like those used in the 10-40 method for making a collimation check.

To compute the mean elevation difference, ΔH, the observed difference from the backward setup, Δh_B, must be subtracted from the observed difference for the forward setup, Δh_A. The effects of curvature then cancel. If the adjustment of the instrument does not change between the setups, the effects of collimation error cancel. The effects of refraction might be expected to cancel as well, but atmospheric conditions may change significantly, particularly over water, during the time required to move the instrument from one setup to the other. Another technique must be employed.

Simultaneous observations. To reduce the effects of refraction, the two reciprocal setups should be observed simultaneously through the same atmosphere. This condition can be approximated by employing two instruments, one on each side of the crossing, in positions that permit the lines of sight to be equal in length, parallel, adjacent, and at the same height above the water or ground. In addition, to ensure that the lines of sight pass over similar terrain, the bench mark and the instrument on one side should be positioned at distances from the edge of the water that are equal to the corresponding distances on the opposite side.

Such a set of simultaneous reciprocal observations is not enough, however, to limit the effect of any difference in the respective collimation errors of the instruments (α_1 and α_2 in fig. 4-4). The mean ΔH becomes

$$\Delta H=[\Delta h_{A1}-\Delta h_{B2}-(C_1-C_2)\times\Delta s]/2.$$

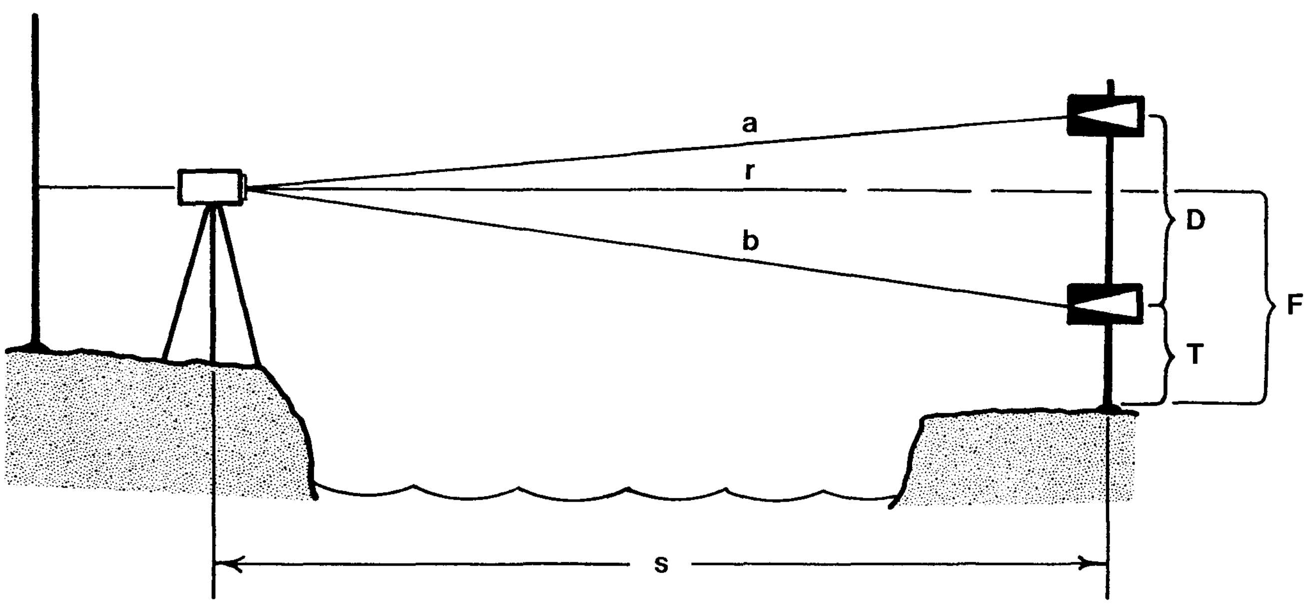

Figure 4-2.—Intercepting two targets to reduce pointing error.

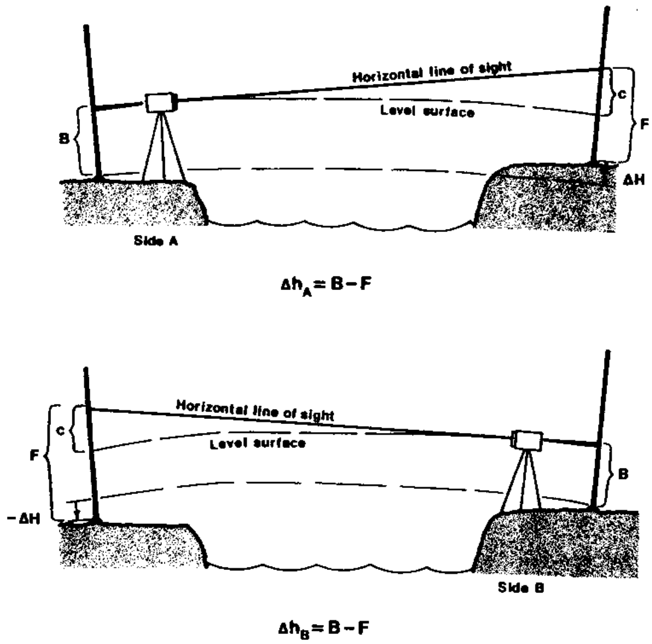

Figure 4-3.—The effects of curvture, *c*, cancel if a pair of reciprocal setups are observed and the results, Δh_A and Δh_B, are meaned.

Carefully determined values for collimation factors C_1 and C_2 might be used to correct the result, but at long sighting distances the magnitude of the correction is very large, as is the corresponding uncertainty.

Reciprocal collimation. To limit the effect of different collimation errors more effectively, four instruments with special attachments should be employed, two on each side of the crossing. During one set of simultaneous observations, each instrument pair should be repeatedly placed in reciprocal collimation; that is, the collimation error of one instrument should be adjusted to be equal and opposite to that of the other. (The lines of sight thus obtained are analogous to those provided by the two compensator positions of the NI 002, section 3.3.3. In this way, the mean collimation error of the instruments is minimized, and, as shown in figure 4-5,

$$\Delta H = (\Delta h_A - \Delta h_B)/2.$$

This principle is applied in the observing procedure used with Zeiss River Crossing Equipment. (See sec. 4.3.)

Another technique for reducing the effect of different collimation errors, requiring only two ordinary instruments, is to interchange the instruments between two sets of simultaneous observations, each set including the setups illustrated in figure 4-4. The mean of the results of the sets is the elevation difference for the section:

$$\Delta H = [(\Delta h_{A1} - \Delta h_{B2}) + (\Delta h_{A2} - \Delta h_{B1})]/4.$$

The collimation errors of the instruments can and do change somewhat when they are transported from one side to the other (as a result of temperature changes, vibration, and shocks during transport). increasing the uncertainty of the results. However, when less than first-order precision is required, a procedure employing this technique may be used. (See 4.4.)

Multiple runnings. One complete running of a river crossing, then, must include the equivalent of four setups: two sets of simultaneous reciprocal observations, with the instruments for one set providing lines of sight reciprocally collimated to those provided by the instruments for the other set. To permit a check of the results, a second complete running must be made. Although both runnings are computed as though they were observed in the forward direction, they constitute a double-run section because forward and backward levelings (the two reciprocal setups) are incorporated within each running. The results of the two runnings should close within the tolerance for the order and class of the survey, as described in subchapter 3.7.7.

4.2 Preparations

In most vertical control projects, river crossings occur infrequently. As a result, when a river crossing is necessary it requires special preparations. To ensure that it is conducted correctly and efficiently, the project director should first review the procedures outlined in

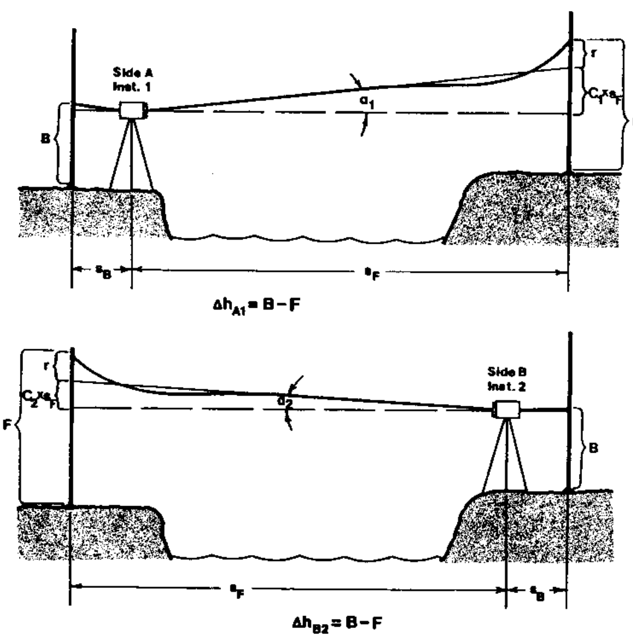

Figure 4-4.—The effects of refraction, *r*, cancel when the results of a set of simultaneous, reciprocal observations are meaned; however, the collimation error of one instrument, α_1, is only partly offset by the collimation error of the other instrument, α_2.

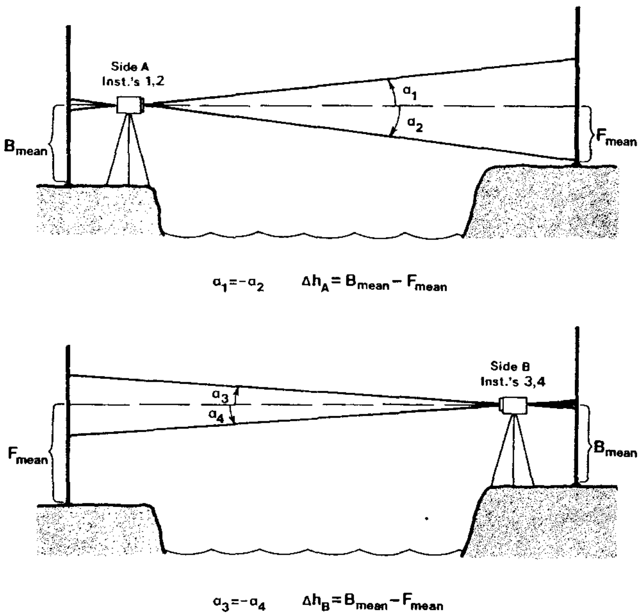

Figure 4-5.—Simultaneous reciprocal observations with four leveling instruments, the two instruments on each side having reciprocal collimation errors.

this chapter when planning the crossing. Then, the site must be selected and prepared, the necessary personnel and equipment must be assigned, and the work must be scheduled under suitable environmental conditions.

4.2.1 Reconnaissance and Mark Setting

An unusually long line of sight may be necessary when a leveling route crosses a river, the mouth of a bay, a deep ravine or valley, or a passage between two islands. When possible, the route should bypass such obstacles, taking advantage of bridges, dams, breakwaters, or even land exposed at low tide. However, if a river crossing is unavoidable, select a location to satisfy the following conditions:

1. The crossing is readily accessible to the leveling lines to be connected. Any connecting spurs or lines should be as short as possible.

2. The lines of sight, from one side to the other, are as short as possible.

3. The lines of sight are as high as possible, clearing water by at least 3 m (10 ft) and clearing land or objects (such as trees, buildings, or bridges) by at least 0.5 m (1.5 ft). Consider the possibility of difficulties resulting from wave action, vessel traffic, or extreme refraction. For crossings of 1.0 km (0.6 mi) or more, the lines of sight should be at least 4.5 m (15 ft) above the high-water line to ensure sufficient clearance.

4. The lines of sight pass over similar terrain. For example, avoid a site with a steep bank on one side and a gentle beach on the other.

Additional requirements for ordinary instruments. In most cases, special instruments are necessary for river crossings; however, ordinary leveling instruments may be used if all of the following conditions are satisfied:

5. The instrument has a mechanism for tilting and measuring the tilt of the line of sight. (Compensator instruments, such as the NI 002 and Ni1, generally do not have such a mechanism.) Two such instruments and four targets must be available for this type of crossing.

6. Less than first-order precision is required for the survey.

7. The instruments can be quickly transported (preferably within an hour) from one side of the crossing to the other.

8. On each side, a bench mark can be set in such a position that a 3-meter leveling rod placed on the control point is entirely visible from the instrument station on the opposite side.

Setup specifications. On each side of the crossing, mark a location for the instrument station and set a permanent bench mark (sec. 2.4.2) to satisfy the following specifications. The setups for Zeiss River Valley Crossing Equipment are shown in section 4.3.2, "Setups," and the setups for ordinary equipment are shown in section 4.4.1, "Equipment and setups."

1. The lines of sight between each instrument station and the opposite bench mark should be equal in length, parallel, and at the same elevation.

2. The instrument stations should be at equal distances from the edge of the water or obstacle.

3. The bench mark should be 5 to 50 m (16-160 ft) from the instrument station. This backsight distance should be the same on each side of the crossing.

4. The bench mark should be no more than 1.0 m (3.3 ft) above or below the instrument station, to permit observation of a standard leveling rod. Furthermore, conventional geodetic leveling must be possible between the bench mark and the target station (which will be located next to the instrument station).

4.2.2 Personnel

Six individuals are normally required to conduct a river crossing: two observers, two recorders, and two rodmen. If an extra truck and equipment are available, a leveling unit assisted by one or two additional persons may perform the crossing. The unit chief coordinates the overall operation and serves as observer or recorder for one side. If two leveling units perform the crossing, one unit chief should be designated as the coordinator, and advantage should be taken of the opportunity to train the extra individuals present.

Observers. The observers must follow the appropriate crossing procedures, paying particular attention to the details of intercepting the targets. They must ensure that observations on each side are made simultaneously. Before leaving the site, the observers must verify that all required measurements have been accurately recorded.

Recorders. Recorders should ensure that all steps in the crossing routine are completed in the correct sequence. They must be thoroughly familiar with procedures for recording and computing the data, either on paper or with a computer-recording system. Accuracy and efficiency are especially important, since many readings must be entered during a short period of time.

Rodmen. At each side of the crossing a rodman is responsible for one leveling rod. The rod is guyed in place over the bench mark for the duration of the crossing, the rodman ensures that it is plumb and secure at all times. If more than one setup is required between the target station and the control point, the rodman holds a second rod on a turning point. The rodman may be responsible for setting the target(s), and measuring and reporting the target heights, as required.

4.2.3 Equipment

Each side of a river crossing should have a leveling truck and a complete set of geodetic leveling equipment (appendix A). Equipment appropriate to the crossing routine must be available. (See subchapter 4.3 or 4.4.) Even if Zeiss River Crossing Equipment is used, an ordinary leveling instrument and rods should be used to level between the bench mark and the target station.

In addition, two-way radios should be available to permit communication between the two stations. Signal flags may be used if radios are not available. The observers raise the flags to indicate when foresight observations are being made.

When instruments must be interchanged between stations, as for the method employing ordinary instruments, a boat and operator may provide the most efficient service. Other means of transport, which may have to be obtained by contract, include a ferry or helicopter.

4.2.4 Environmental Conditions

Refraction effects are most unpredictable when a large vertical air-temperature difference exists, as is usually the case over water on a sunny, windless day. Shimmer can be extreme, making observations difficult. A crossing should be conducted when refraction effects are apt to be small and uniform—when the sky is overcast and the wind is moderate. Air-temperature differences (sec. 3.6.1, "Measuring air-temperature differences") may be measured to assist in determining whether conditions are favorable.

Other environmental conditions may interfere with visibility, such as vessel traffic, large swells, wind waves, fog, or having to observe directly into the Sun. Whenever possible, plan the location and time of the crossing to avoid these difficulties.

The leveling unit(s) assigned to a crossing should check each morning to see if conditions are suitable; if not, the units should resume routine leveling. To permit this, the necessary equipment should be available in the leveling trucks when a crossing is assigned.

4.3 Instructions for Zeiss River Crossing Equipment

Because the compensator instruments employed by the National Geodetic Survey have no mechanism for precisely tilting and measuring the tilt of the line of sight, Zeiss River Crossing Equipment is normally used to perform a river crossing. The equipment includes instruments with rotary-wedge attachments that enable the observers to tilt the line of sight by a precisely measured amount.

The equipment has two other versatile features. First, so instruments need not be transported from one side of the crossing to the other, four instruments are employed. Each pair of instruments is placed in reciprocal collimation, enhancing accuracy by limiting the mean collimation error on each side to $\pm 0.''2$.

Second, to permit more flexibility when selecting sites for bench marks, target stations are used. The stations are located close to the shoreline, where in methods using ordinary equipment the marks must be set. As a result, the marks may be set farther away from the shoreline, improving their quality and potential for survival.

4.3.1 Description of Equipment

Two complete sets of Zeiss River Crossing Equipment are necessary, one for each side of the crossing. Each set, labeled "A" or "B," includes the following:

1. Two Zeiss Ni2 leveling instruments, marked "1" and "2."
2. Two rotary-wedge attachments for the Ni2 instruments.
3. One base plate for the Ni2 instruments.
4. Two targets.
5. One target column.
6. One tribrach.
7. One tripod, with adjustable legs, for the instrument station.
8. One tripod, for the target station.
9. One auxiliary scale, 0.6 m in length, graduated in units compatible with the ordinary leveling equipment.

Ni2 instrument. The Ni2 is illustrated in figure 3-18 and described in section 3.3.3. Before the river crossing begins, the collimation error of each instrument should be checked. If one of the instruments is to be used for leveling from the nearby target station to the nearby bench mark, a micrometer attachment may be required. If so, the micrometer units should be compatible with the units of the leveling rod used. (See sec. 3.3.4, "Optical micrometer.")

Rotary-wedge attachment. Each rotary-wedge attachment slips over the objective end of an Ni2. The attachment includes a rotatable optical element, the wedge, to which is attached a graduated scale. To be sufficiently precise, the attachment must be of the best optical quality.

The wedge is a slightly deviating prism, cut in the shape of a circle and mounted to permit rotation about an axis parallel to the line of collimation of the instrument. It functions in the same way as the prism that is used to adjust the collimation error of such instruments as the Zeiss Ni1.

The wedge deflects the line of sight by a small angle (257″) in a plane perpendicular to the line of intersection, or "edge," of the two main faces. (The "edge" does not physically exist because it is cut away when the prism is cut to a circular shape.) When the wedge is rotated so the "edge" is parallel to the vertical axis of the instrument (fig. 4-6a), the deflection is entirely in the horizontal plane of the instrument, causing no "tilt" in the line of sight.

However, when the wedge is rotated to some other position (fig. 4-6b), the line of sight is deflected vertically as well as horizontally. The vertical deflection, δ, is observed as a tilt of the line of sight and is proportional to the sine of the rotation angle, ω:

$$\sin \delta = \sin 257'' \times \sin \omega.$$

The attachment is designed to permit a maximum rotation angle of $\pm 51°6'$. Thus, the line of sight can be smoothly tilted as much as 200″ above or below the horizontal plane of the instrument.

Units representing equal increments of sine of the rotation angle are graduated on the attached scale (fig. 4-7), that rotates with the wedge. To avoid difficulty with algebraic signs, the unrotated position (no vertical tilt) is numbered "10", the position of maximum upward tilt is numbered "0", and the position of maximum downward tilt is numbered "20". Each wedge unit is graduated into ten subunits, and each subunit is large enough to permit estimation of tenths of a subunit. Therefore, the angle of tilt can be measured to one hundredth of a unit. Since each unit represents 20″ (0.0001 radian), the smallest angle that can be measured is $\pm 0\overset{''}{.}2$ (0.000001 radian).

The wedge is rotated by turning a knob that is provided with both coarse and fine modes of motion, similar to the focusing knob on the instrument itself. In the coarse mode, which is available for the entire range of rotation, the knob is slightly difficult to turn. Reversing the knob shifts the mode from coarse to fine. The knob remains in the fine mode, where it is much easier to turn, for only 0.5 to 1.0 wedge unit (depending on which part of the scale is in use), after which it reverts to the coarse mode. The line of sight can be directed most precisely with the knob in the fine mode. Therefore, make all final pointings in this manner.

Base plate. The base plate is an elongated metal casting designed to support the pair of instruments. It screws onto the head of an ordinary tripod. On the plate, positions

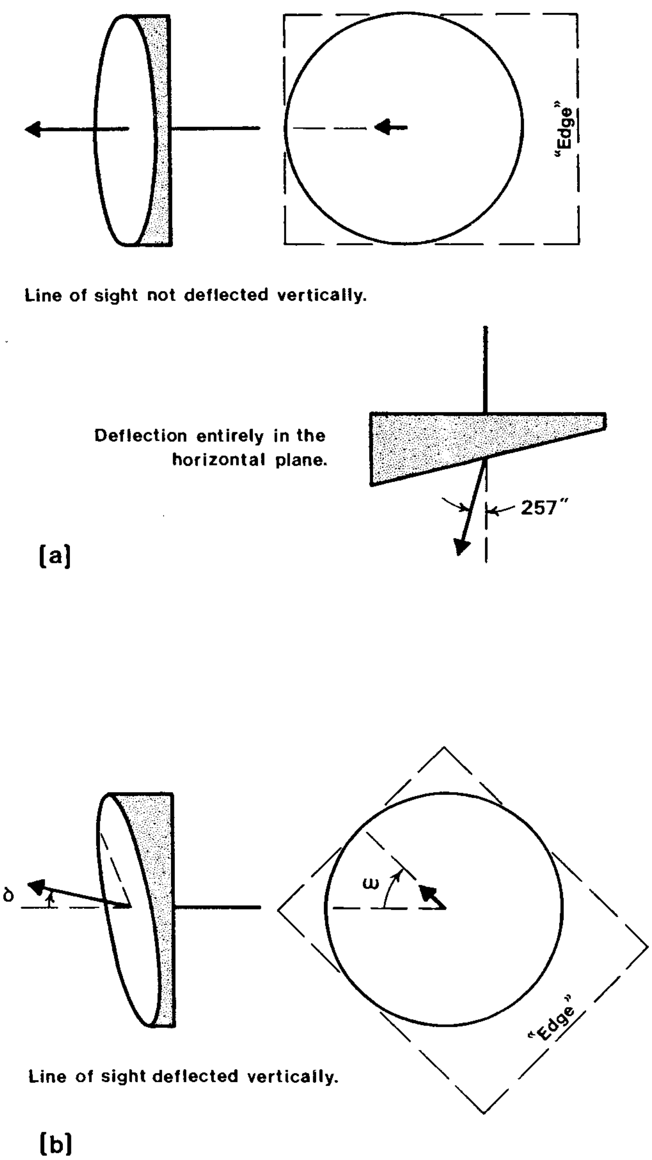

Figure 4-6.—Rotary wedge, unrotated (a) and rotated (b).

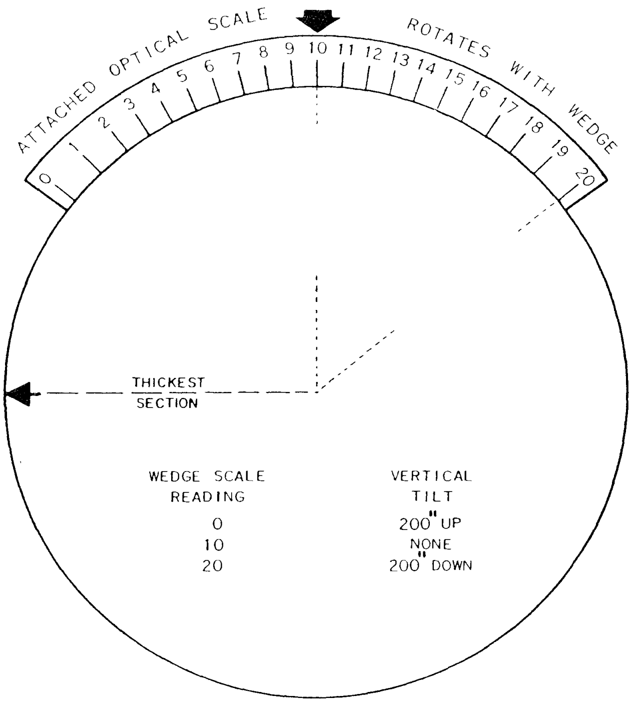

Figure 4-7.—Rotary wedge scale.

for each instrument are labeled and marked with red lines. The attached clamping screws, similar to those found on tripods, are used to secure the instruments.

Tribrach. The tribrach is an ordinary, separate, three-screw, leveling base intended for use with any surveying instrument provided with a matching stub. In this application, it provides the means for mounting and plumbing the target column on a tripod.

Target column. The target column (fig. 4-8) is a piece of channel section, with a stub suitable for mounting it in the tribrach. A circular level at the foot of the column provides a reference for precisely aligning the column with the direction of gravity. On the column are four pairs of precisely spaced studs on which the targets can be mounted.

The height stud is attached horizontally near the base of the column and terminates in a circular knob. The top of the knob is at precisely the same height as the target center defined by the lowest pair of studs. It serves as the control point from which leveling is conducted to the nearby bench mark.

Targets. Two metal targets, each 20 by 30 cm, are provided with each set of equipment. Each target is black, with a white isosceles triangle painted on it. Each target is equipped with a mounting bracket that fits over a pair of studs on the target column. The horizontal centerline of each target can be mounted 0.00, 40.00, 80.00, or 120.00 half-centimeters (hcm) above the height stud (fig. 4-9).

Auxiliary scale. The auxiliary scale is a short, wooden leveling rod, 0.6 m in length. It is used to level from the height stud on the target column to the nearby bench mark. When in use, it is held vertically against the target column, with the base resting on the top of the knob of the height stud (fig. 4-10).

The scale is graduated in half-centimeter units. The graduations are grouped and numbered in sets of ten blocks, which alternate black and white, forming a checkerboard pattern. The first ten sets of blocks are numbered from 0 through 9, to be read as tens of rod units (010., 020., 030., etc.). The second four sets of blocks are numbered from 0 through 3, with a large dot over each number to indicate that the readings should be preceded by "1" (100., 110., 120., and 130).

4.3.2 Setups

On each side of the crossing, position an instrument station opposite a target station located on the other side. A correct pair of setups is illustrated in figure 4-11. The foresights, from each instrument to the opposite target station, must be as similar in length as possible, within 1 m in height and adjacent within 3 m.

Instrument station. Set up the instrument station as follows:

1. Set the adjustable-leg tripod firmly in the ground. It must not move during the entire running.
2. Orient the base plate on the tripod so the long axis is perpendicular to the line of sight to the opposite target station, with the indicated position for instrument 1 on the left.

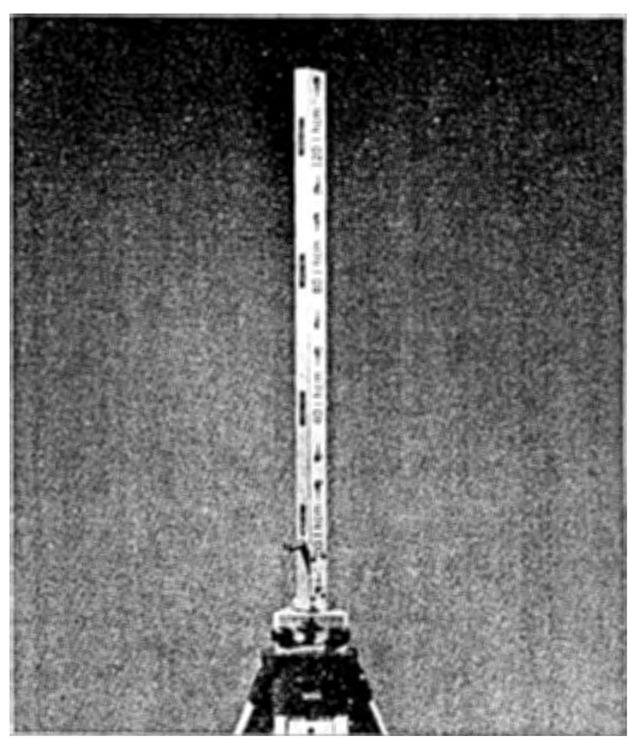

Figure 4-8.—Target column.

3. Level the base plate roughly by centering the bubble in the circular level. There are no foot screws or similar devices on the base plate; therefore, level it by varying the lengths of the tripod legs or by changing the positions of the tripod feet in the ground. This ensures that the lines of sight of the two instruments will be at nearly the same elevation.

4. Secure the rotary-wedge attachments to the instruments.

5. Attach the instruments to the base plate in the indicated positions, aligning the red marks on the instrument tribrachs with the numbered, red marks on the base plate (fig. 4-12). In this way, one foot screw of each instrument is positioned in line with the long axis of the base plate and the other two are aligned perpendicularly to it.

6. Set the three foot screws on each instrument to the middle position of their range of movement. The top edge of each screw should be even with the indexing ring marked on the dust skirt. This procedure limits the elevation difference between the two instruments. Make all subsequent leveling adjustments by using only the screws aligned perpendicularly to the long axis of the base plate. Do not use the screws that are aligned with the long axis again during the entire running.

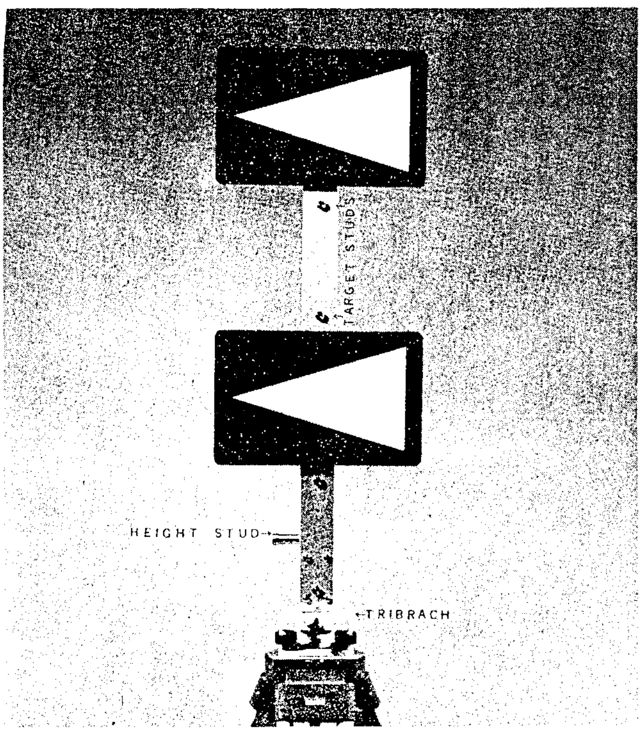

Figure 4-9.—Targets mounted on target column.

Target station. Set up the target station as follows:

1. Attach the target column to the tribrach and mount it on the tripod (fig. 4-9). It must not move during the entire running.

2. Secure two targets to the target column. Position them at the same heights on each side of the crossing, placing the lower target at zero, if possible.

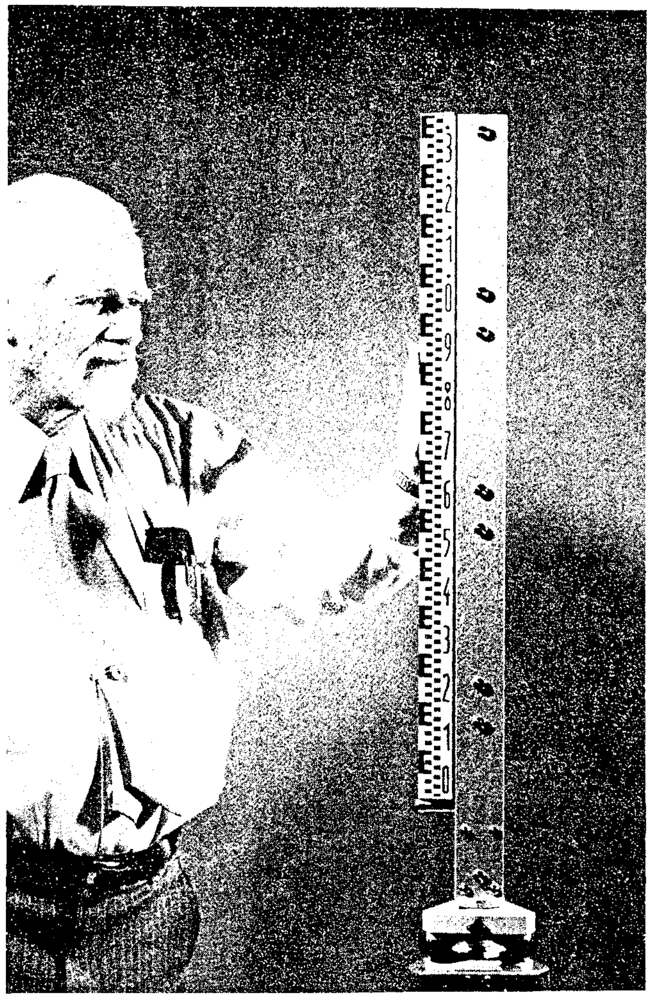

Figure 4-10.—Using the auxiliary scale.

3. One target must be above, and one below, the mean line of sight from the opposite instrument pair. The angle of tilt from the lower target to the upper target should be five wedge units or more. Consult with the observer on the opposite side to ensure that these conditions are satisfied. Adjust the height of the tripod or targets as necessary.

4. Plumb the target column by centering the bubble in the circular level, using the three leveling screws on the tribrach.

Leveling rod. Set a geodetic leveling rod on the nearby bench mark. To preserve the usual value for the rod constant difference when leveling from the nearby target station to the mark with only one setup, use rod 2 of the standard pair. If two setups are necessary, use rod 1. During the series of observations from the bench mark to the opposite target station, the rod constant is not important because only the low scale is observed.

Plumb and guy the rod in place with three wires attached at the top center. Rotate the rod to face the nearby instrument station.

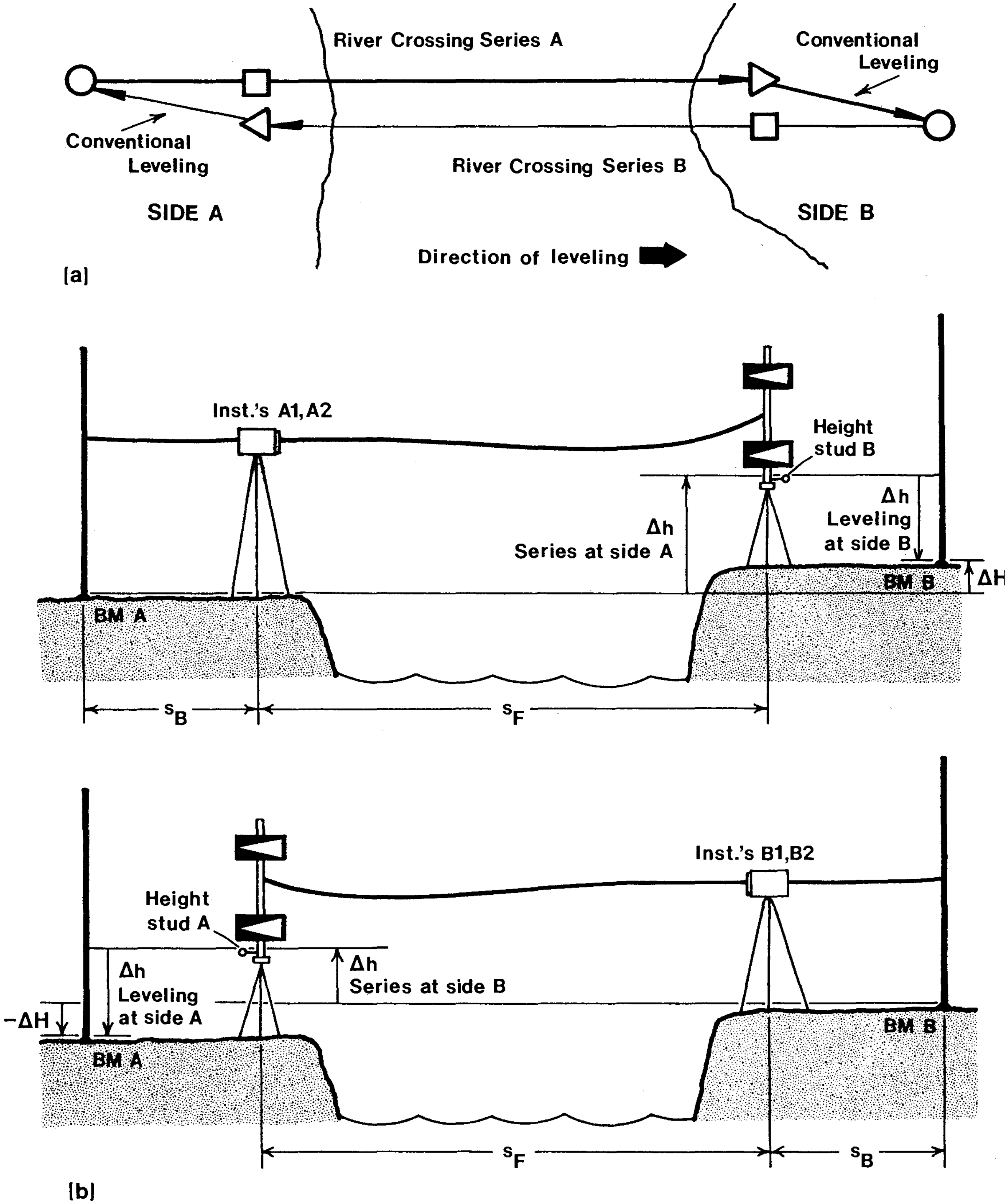

Figure 4-11.—Setups for Zeiss River Crossing Equipment, top view (a) and side view (b).
$\Delta h = \frac{1}{2}[(\Delta h_{\text{Series A}} + \Delta h_{\text{Leveling B}}) - (\Delta h_{\text{Series B}} + \Delta h_{\text{Leveling A}})]$.

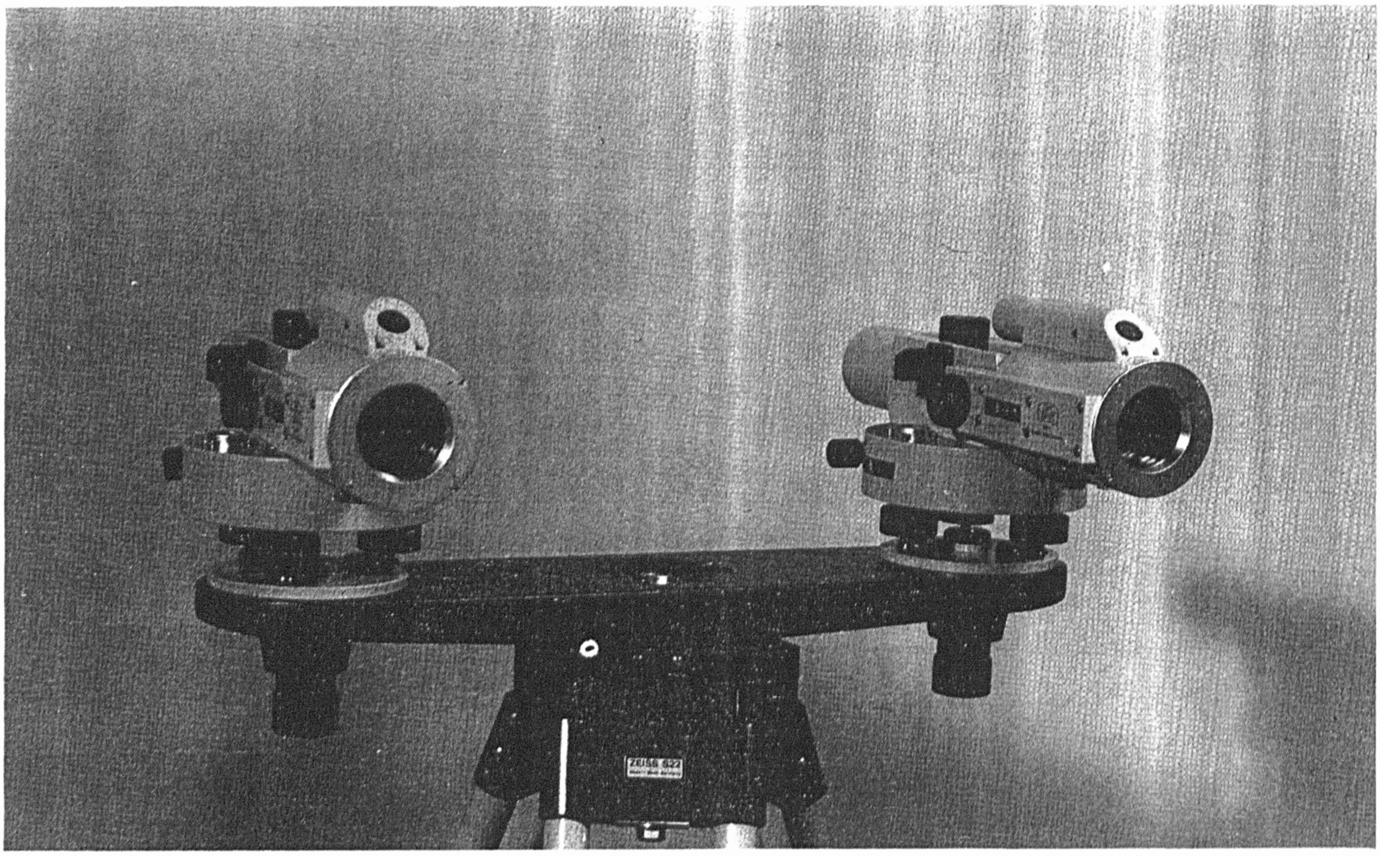

Figure 4-12.—Instruments mounted on base plate.

4.3.3 Observing Routine

One running of the observing routine includes, on each side, the measurement of two elevation differences. The first is obtained by conventional leveling from the nearby target station to the nearby bench mark. The second is obtained by a series of observations from the nearby bench mark to the opposite target station. The first is then measured again to ensure that the nearby target station has not moved. To compute the total elevation difference from the nearby bench mark to the opposite bench mark, the result of the series of observations from one side is added to the result of the conventional leveling from the opposite side. The total computed at one side is meaned with the total computed at the other side to obtain the elevation difference for the running. (See fig. 4-11.)

For a complete river crossing, two runnings are required that close as described in section 3.7.7. If both runnings cannot be completed in one day, the second running may be conducted another day. (See table 4-1 for a summary of the observing routine.)

If possible, record the observations on a computer, using backup forms if necessary. The instructions that follow include procedures for using the standard recording forms.

Collimation checks. On each side of the crossing, check the collimation errors of the pair of Ni2 instruments, following a procedure described in section 3.3.7. Since the instruments are to be set in reciprocal collimation, adjustment is not necessary unless the difference in collimation errors exceeds 40″ ($C_1 - C_2 > 0.20$ mm/m).

Perform a collimation check with the instrument to be used for the nearby leveling. One Ni2 may be removed from the base plate of the instrument station and used for this purpose. Adjust the instrument if it does not satisfy the tolerance for the order and class of the survey (table 3-1).

Leveling, nearby target station to nearby bench mark. On each side of the crossing, double run a section of leveling from the height stud of the nearby target station to the nearby bench mark. Follow the usual observing routine, satisfying specifications for the order and class of the survey and preparing a survey equipment record (i.e., line *40*) and beginning and ending running records (i.e., line *41*) (Pfeifer and Morrison 1980: vol. II.)

The auxiliary scale must be used on the height stud. Make sure the units of the scale are compatible with the leveling equipment; convert the scale readings to appropriate rod units, if necessary. If employing the micrometer leveling procedure, described in section 3.7.2, a special computation is necessary to obtain a high-scale reading. Read the scale in the same manner as for the low-scale reading. Then, add the rod constant of rod 1 to the reading before recording it. (See fig. 4-13 for an example.)

Series, nearby bench mark to opposite target station. On each side of the crossing, make a series of observations to determine the elevation difference from the nearby bench mark to the height stud of the opposite target station. This series follows the pattern backsight-foresight-backsight. The backsights each include one set of observations to "lower" and "upper" graduations on the leveling rod placed on the bench mark. The foresight includes four sets of observations to the lower and upper targets on the opposite side. Every set includes observations from both instruments at the station. Do not interrupt a series once it has started.

1. Prepare the beginning series record (line *42*). Include the following information: date, the instrument codes and serial numbers, the rod code and serial number, the survey-point serial number and stamping of the bench mark (or designation if no stamping), the survey-point serial number and designation of the opposite height stud, the running number, the local time zone and time, the temperature units and temperature, the wind code, the sun code, and the initials of the observer, recorder, and rodman. (Refer to the annotated recording form shown in fig. 4-14. An example is given in fig. 4-15.)

2. Record the positions of the nearby targets as indicated on the form. Communicate this information to the recorder on the opposite side.

3. Obtain the positions, T and t, of the targets on the opposite side. Enter the positions as indicated on the recording form, in units identical to those of the leveling rod. For example, with a half-centimeter rod, enter 0, 40, 80, or 120 hcm for each target. Compute the difference, D, between the target positions.

4. At the instrument station, turn both instruments to point away from the target station on the opposite side. This procedure is indicated on the recording form as "↓". Carefully center the bubble in each circular level, using only the foot screws aligned perpendicularly to the long axis of the base plate.

5. RECIPROCAL COLLIMATION: Perform a reciprocal collimation immediately before every set of observations in the series, as follows:

a. Looking through instrument 1, focus precisely on a distant, sharply defined object. Then, turn the instruments toward each other. Looking through instrument 2, focus instrument 2 until the reticle lines of instrument 1 are sharply defined (fig. 4-16a). This procedure focuses both instruments at infinity.

b. Rotate the wedge knob on instrument 1 until the scale is set at 10.00. Make the final setting with the knob in fine mode. (See sec. 4.3.1, "Rotary-wedge attachment.")

c. Looking through instrument 2, rotate the wedge knob to bring the middle reticle line into coincidence with the image of the middle line of instrument 1 (fig. 4-16b). Make the final setting by turning the knob clockwise, in fine mode. Record the reading of the wedge scale.

d. With the knob still in fine mode, move the middle line of instrument 2 out of coincidence by a small amount and set it again. This time make the final setting by turning the knob counterclockwise. Record the second scale reading. Compute the mean, r, of the two readings.

6. BACKSIGHT: Turn the instruments to point at the leveling rod placed on the nearby bench mark. First, with instrument 1 and then with instrument 2 obtain a set of observations by carefully following the instructions here. With each instrument, intercept the same lower and upper graduations on the low scale. Make all final settings with the wedge knob in fine mode.

a. Refocus the instrument to obtain a sharp image of the rod scale.

b. Set the wedge scale to 20.0 by turning the wedge knob clockwise (the scale moves counterclockwise). The setting need not be precise.

Table 4-1.—Summary of crossing routine for Zeiss River Crossing Equipment

During each running, on both side A and side B:

Set up instrument station, target station, and leveling rod.

Make COLLIMATION CHECKS with the leveling instruments.

Double run a section of LEVELING from the nearby target station to the nearby bench mark.

Observe a SERIES, simultaneously with the opposite side:
1. Point away from the opposite target station and level the instruments.
2. Observe and record reciprocal collimation.
3. Observe and record backsights to nearby leveling rod.
4. Point toward the opposite target station and level the instruments.
5. Observe and record reciprocal collimation.
6. Observe and record foresights to opposite target station.
7. Point away - reciprocal collimation - foresight observations.
8. Point toward - reciprocal collimation - foresight observations.
9. Point away - reciprocal collimation - foresight observations.
10. Point toward - reciprocal collimation - backsight observations.

Double run a section of LEVELING from the nearby target station to the nearby bench mark.

During one day:		OR During two days:	
COLLIMATION CHECKS LEVELING SERIES	RUNNING 1	COLLIMATION CHECKS LEVELING SERIES LEVELING	RUNNING 1
LEVELING SERIES LEVELING	RUNNING 2	COLLIMATION CHECKS LEVELING SERIES LEVELING	RUNNING 2

c. Observe the lower graduation: turn the knob counterterclockwise until the middle reticle line, moving upward on the low scale of the rod, appears at a position slightly above the first graduation encountered. The knob is probably in coarse mode. (If the line already intercepts graduation when the wedge scale is set at 20.0, go to the next graduation.)

d. Reverse the knob and, in fine mode, intercept the lower graduation. Record the graduation value, G, and the reading of the wedge scale.

e. Continue turning the knob (clockwise) until the middle line is slightly below the same graduation. The knob should remain in fine mode. Release your fingers and grasp the knob in another place. Then, reversing it, intercept the graduation. Record the scale reading. Compute the mean, l, of this reading and the reading obtained in the previous step, 6d.

f. Set the scale to 0.0, turning the knob counterclockwise. The setting need not be exact.

g. Observe the upper graduation: Turn the knob clockwise until the middle reticle line, moving downward on the low scale of the rod, appears at a position slightly below the first graduation encountered. The knob is probably in coarse mode. (As before, if the line intercepts a graduation when the wedge scale is set at 0.0, continue on to the next graduation.)

h. Reverse the knob and, in fine mode, intercept the graduation. Record the graduation value, g, and the reading of the wedge scale.

i. Continue turning the knob (counterclockwise) until the middle line is slightly above the same graduation. The knob should remain in fine mode. Release your fingers and grasp the knob in another place. Then, reversing it, intercept the graduation. Record the scale reading. Compute the mean, u, of this reading and the one obtained in the previous step, 6h.

7. This step may be performed after all crossing observations are completed. Compute the mean backsight interval, I, and the sighting distance, s_B, as follows (fig. 4-17).

a. Compute the distance, d, between the upper and lower graduations, in rod units:

$$d = g - G.$$

b. Compute the angles of tilt from the settings on the lower graduation to the collimated lines of sight: $l_1 - 10$ for instrument 1, and $l_2 - r$ for instrument 2.

c. For each instrument, compute the angle of tilt from the lower graduation to the upper graduation: $l_1 - u_1$ and $l_2 - u_2$.

d. For each instrument, compute the interval, i, from the lower graduation to the intercept of the collimated line of sight, in rod units:

$$i_1 = [d \times (l_1 - 10)]/(l_1 - u_1)$$

$$i_2 = [d \times (l_2 - r)]/(l_2 - u_2)$$

NOAA FORM 76-191 (8-77)
RUN 1
SIDE B

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

GEODETIC LEVELING
MICROMETER OBSERVATIONS (Δh) EXAMPLE First order, Class I RIVER CROSSING

PAGE 5 OF 7

• 4 0 •	YR.	MO.	DAY	CODE	INSTRUMENT SERIAL NO.	M	CODE	ROD SERIAL NO.	CODE	ROD SERIAL NO.	Z	TIME
• 4 1 •	80	07	08	233	456599				316	119363		

FROM	BM DESIGNATION	TO	BM DESIGNATION	Z	TIME BEGIN	TIME END	F/C	TEMPERATURE BEGIN	TEMPERATURE END	W	S	OBSERVER
0002	HEIGHT STUD B	0900	WINDSOR	Q	0940	0950	C	7.4	7.5	1	2	RLT

SET UP#	STADIA BACK	S_B Σ	STADIA FORE	S_F Σ		LOW SCALE BACK/FORE	Δh_L Σ		HIGH SCALE BACK/FORE	Δh_H Σ	$\Delta h_L - \Delta h_H$	REMARKS
1	069	205	352	200	B	069.21	−283.24	B	661.75	−304.23	−01	Auxiliary scale 69.25
	048.5		332.0		F	352.45		F	965.98			Rod 1 constant +592.50
				+0.5						+21		661.75
		205			B			B				
		200			F		−283.24	F		−283.23		
(a)	S = 40.5		ΔS = 0.5 m								Δh =	−283.235
					B			B			÷	200
					F			F				−1.41618 m F
											−04	
2	349	198	66	207	B	349.50	283.22	B	963.04	304.26	−04	Auxiliary scale 66.28
	329.2		45.3		F	066.28		F	658.78			Rod 1 constant +592.50
				−0.9						−21		658.78
		198			B			B				
		207			F		283.22	F		283.26		
(b)	S = 40.5 m		ΔS = −0.9								Δh =	+283.240
					B			B			÷	200
					F			F				+1.41620 m B
					B			B				Section check:
					F			F				−1.41618 F
												+ 0.02 mm
					B			B				Tolerance, K ≤ 0.1 km
					F			F				is 0.95 mm.

Figure 4-13.—Leveling from the nearby target station to the nearby bench mark, forward (a) and backward (b).

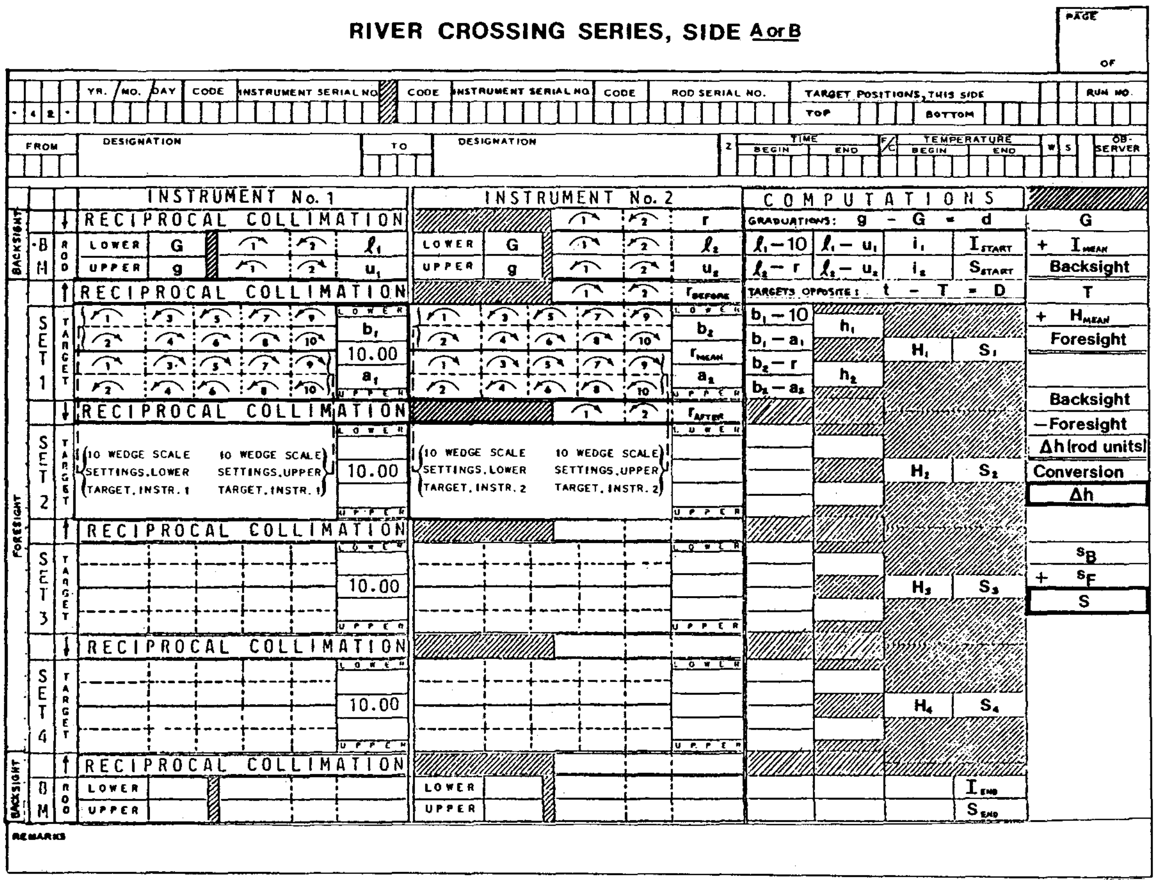

Figure 4-14.—Annotated recording form for the series of observations from the nearby bench mark to the opposite target station.

e. Compute the mean backsight interval, I:

$$I=(i_1+i_2)/2.$$

f. Compute the backsight distance, s_B, in meters. First compute the mean of l_1-u_1 and l_2-u_2. Then,

$$s_B=[d_{rod\ units}\times \text{conversion factor}_{\text{m/rod unit}}]/$$

$$[(l-u)_{\text{mean}}\times 0.0001_{\text{radian/wedge unit}}].$$

For d in half-centimeter units:

$$s_B=(50\times d)/[(l-u)_{\text{mean}}].$$

For d in centimeter units:

$$s_B=(100\times d)/[(l-u)_{\text{mean}}].$$

8. Turn both instruments toward ("↑") or away from ("↓") the opposite target station, as indicated on the recording form. On each instrument, displace the bubble in the circular level toward the station. Then, carefully recenter it.

9. Repeat the reciprocal collimation, step 5.

10. FORESIGHT: Before beginning each set of target observations, coordinate the activites with the observing team on the opposite side, using radios or visual signals. The two observing teams must begin each set simultaneously. For each set, carefully follow the instructions given here, first with instrument 1. Then, repeat the procedure with instrument 2. Make all final settings with the wedge knob in fine mode.

a. Point the instrument at the opposite target station and observe the lower target. Turn the wedge knob clockwise until the middle reticle line appears slightly below the lower target. If the middle line is already below the lower target, move it to a position substantially above the target before starting.

b. Reverse the knob and, in fine mode, intercept the target. Record the scale reading.

c. Continue turning the knob (counterclockwise) until the middle line appears slightly above the target. The knob should remain in fine mode. Release your fingers and grasp the knob in another place. Reverse it and, in fine mode, intercept the target. Record the scale reading.

d. Repeat steps 10a through 10c until ten settings (five counterclockwise and five clockwise) have been made on the lower target. Compute the mean, b.

e. Observe the upper target. Turn the wedge knob counterclockwise until the middle reticle line appears slightly above the upper target.

f. Reverse the knob and, in fine mode, intercept the upper target. Record the scale reading.

g. Continue turning the knob (clockwise) until the middle line appears slightly below the target. Release your fingers and grasp the knob in another place. Reverse it and, in fine mode, intercept the target. Record the scale reading.

h. Repeat steps 10e through 10g until ten settings (five clockwise and five counterclockwise) have been made on the upper target. Compute the mean, a.

11. Repeat steps 8 through 10 to obtain a total of four sets of target observations. In step 8, prior to successive sets, point the instruments alternately toward ("↑") and away ("↓") from the opposite target station. The correct positions are indicated, as a reminder, on the recording form (fig. 4-14). Regardless of the direction in which the instruments are pointed, always displace the bubbles toward the opposite target station.

12. This step may be performed after all the crossing observations are completed. Compute the mean foresight interval, H, for each set as follows (fig. 4-18):

a. Compute the mean value for the reciprocal collimation of Instrument 2 from the values observed before and after each set:

$$r_{mean}=(r_{before}+r_{after})/2.$$

b. Compute the angles of tilt from the settings on the lower target to the collimated lines of sight: b_1-10, for instrument 1; and b_2-r, for instrument 2.

c. For each instrument, compute the angle of tilt from the lower target to the upper target:

$$b_1-a_1 \text{ and } b_2-a_2.$$

d. For each instrument, compute the distance, h, from the lower target to the intercept of the mean line of sight, in rod units. Recall that D is the difference in the position of the opposite targets, $t-T$:

$$h_1=[D\times(b_1-10)]/(b_1-a_1),$$

$$h_2=[D\times(b_2-r)]/(b_2-a_2).$$

e. Compute the mean foresight interval, H:

$$H=(h_1+h_2)/2.$$

RIVER CROSSING SERIES, SIDE A EXAMPLE

PAGE 3 of 7

· 4 2 ·	YR./MO./DAY	CODE	INSTRUMENT SERIAL NO.	CODE	INSTRUMENT SERIAL NO.	CODE	ROD SERIAL NO.	TARGET POSITIONS, THIS SIDE	RUN NO
	800708	232	125385	232	125760	316	241603	TOP 120 BOTTOM 0	1

FROM	DESIGNATION	TO	DESIGNATION	Z	TIME BEGIN	TIME END	F/C	TEMPERATURE BEGIN	TEMPERATURE END	W	S	OBSERVER
0133	CAISSON USE	0002	HEIGHT STUD B	Q	1000	1205	C	7.5	8.1	1	2	KWS

		INSTRUMENT No. 1						INSTRUMENT No. 2						COMPUTATIONS				
BACKSIGHT	↓	RECIPROCAL COLLIMATION								1048	1045		10.46	GRADUATIONS: 102 - 97 = 5				97.
BM	ROD	LOWER	097	1696	1700		16.98	LOWER	097	1572	1574		15.73	6.98	12.92	2.70	2.32	+ 2.33
		UPPER	102	0405	0408		04.06	UPPER	102	0224	0225		02.24	5.27	13.49	1.95	18.9	99.33 hcm
FORESIGHT	↑	RECIPROCAL COLLIMATION								1060	1060		10.60	TARGETS OPPOSITE: 120 - 0 = 120				0.
SET 1	TARGET	1710	1711	1712	1712	1715	LOWER 17.12	1600	1598	1594	1599	1596	LOWER 15.97	7.12	97.76			+ 84.83
		1711	1710	1710	1713	1715	10.00	1602	1595	1595	1592	1596	10.58	8.74		85.06	678.7	84.83 hcm
		0831	0841	0830	0835	0840	08.38	0705	0709	0700	0701	0699	07.03	5.39	72.35			
		0840	0850	0832	0838	0842	UPPER	0706	0708	0700	0700	0698	UPPER	8.94				99.33
	↓	RECIPROCAL COLLIMATION								1055	1058		10.56					− 84.83
SET 2	TARGET	1719	1709	1705	1705	1702	LOWER 17.07	1585	1589	1589	1591	1584	LOWER 15.88	7.07	97.74			14.50 hcm
		1710	1708	1702	1706	1702	10.00	1588	1590	1592	1586	1582	10.63	8.68		84.26	682.6	÷ 200 hcm/m
		0839	0842	0842	0841	0836	08.39	0700	0702	0698	0698	0698	06.98	5.25	70.79			+0.07250 m
		0840	0840	0840	0835	0832	UPPER	0699	0701	0695	0699	0695	UPPER	8.90				
	↑	RECIPROCAL COLLIMATION								1070	1071		10.70					
SET 3	TARGET	1718	1715	1709	1708	1702	LOWER 17.09	1585	1591	1587	1591	1590	LOWER 15.88	7.09	97.91			19.0 m
		1719	1710	1708	1702	1703	10.00	1590	1589	1589	1588	1585	10.62	8.69		84.54	683.4	+684.0 m
		0840	0840	0840	0842	0838	08.40	0700	0698	0699	0702	0707	07.01	5.26	71.16			703.0 m
		0842	0839	0845	0835	0838	UPPER	0699	0695	0701	0708	0703	UPPER	8.87				
	↓	RECIPROCAL COLLIMATION								1053	1055		10.54					
SET 4	TARGET	1715	1712	1712	1711	1708	LOWER 17.11	1576	1579	1580	1580	1575	LOWER 15.77	7.11	98.75			
		1710	1715	1717	1708	1707	10.00	1579	1577	1579	1578	1572	10.52	8.64		85.46	691.2	
		0850	0845	0849	0845	0846	08.47	0105	0702	0703	0707	0701	07.04	5.25	72.16			
		0845	0851	0848	0845	0845	UPPER	0704	0703	0708	0706	0702	UPPER	8.73				
BACKSIGHT	↑	RECIPROCAL COLLIMATION								1050	1050		10.50					
BM	ROD	LOWER	097	1700	1702		17.01	LOWER	097	1580	1580		15.80	7.01	12.92	2.71	2.34	
		UPPER	102	0408	0410		04.09	UPPER	102	0242	0243		02.42	5.30	13.38	1.98	19.0	

REMARKS

Figure 4-15.—Series from the nearby bench mark to the opposite target.

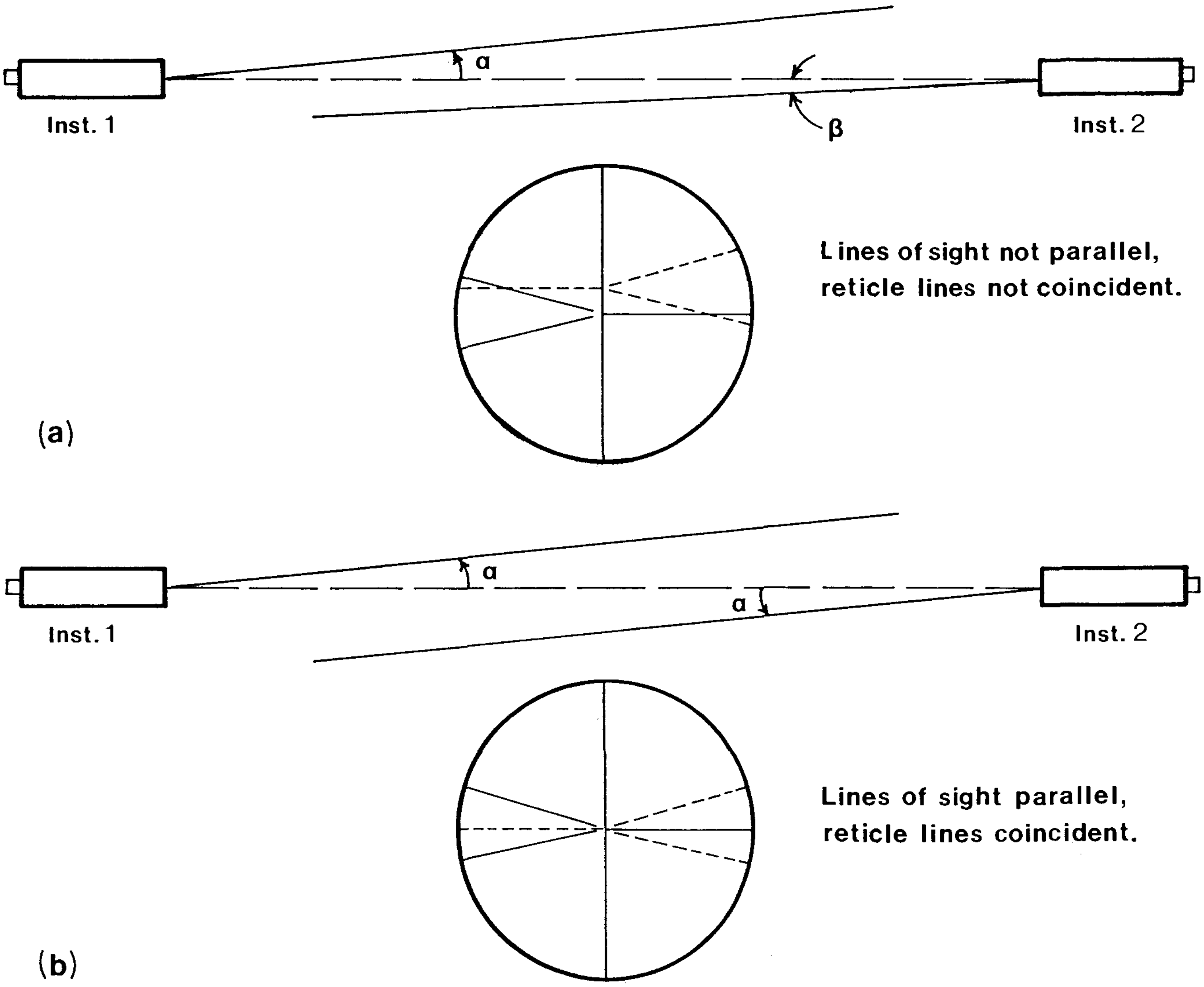

Figure 4-16.—Reciprocal collimation, before (a) and after (b).

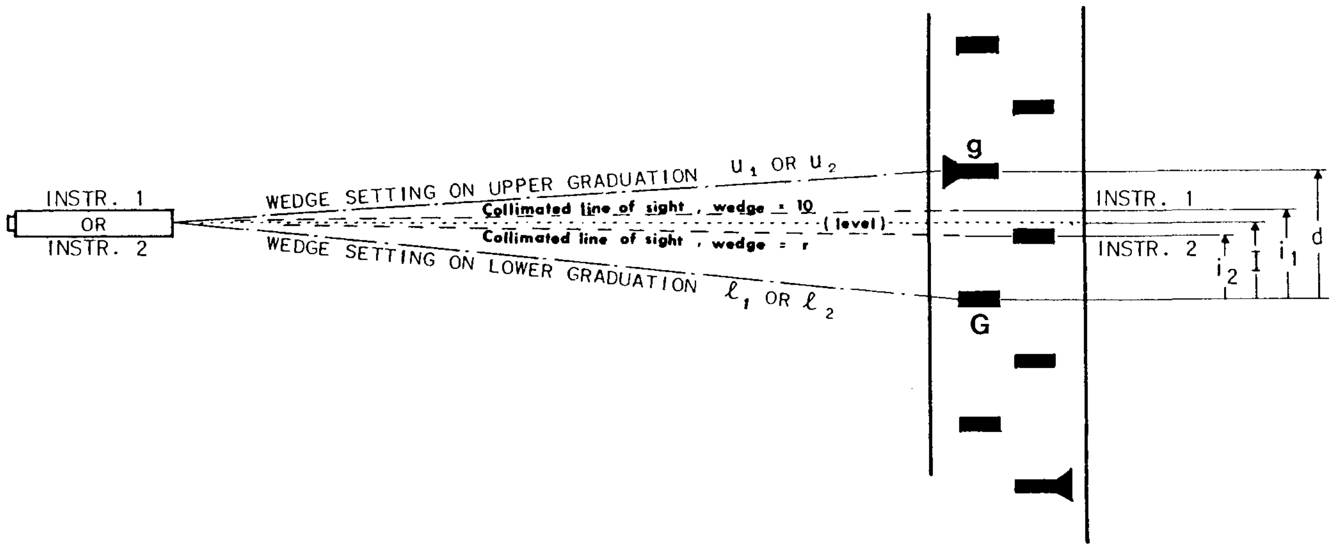

Figure 4-17.—Backsight, wedge settings on rod graduations.

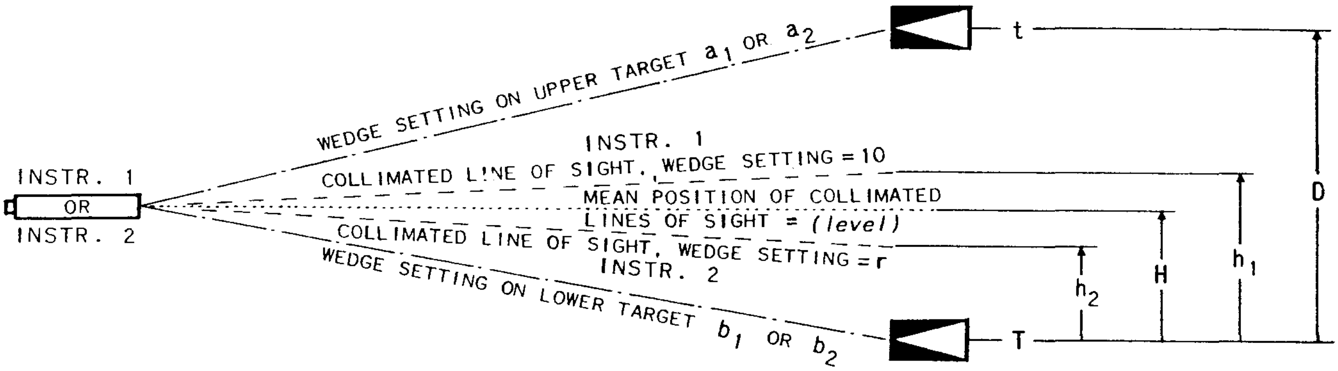

Figure 4-18.—Foresight, wedge settings on targets.

f. Compute the foresight distance, s_F, in meters. First compute the mean of $b_1 - a_1$ and $b_2 - a_2$. Then,

$$s_F = (d_{\text{rod units}} \times \text{conversion factor}_{\text{m/rod unit}}) / [(b-a)_{\text{mean}} \times 0.0001 \text{ radian/wedge unit}].$$

For D in half-centimeter units:

$$s_F = (50 \times D) / [(b-a)_{\text{mean}}].$$

For D in centimeter units:

$$s_F = (100 \times D) / [(b-a)_{\text{mean}}].$$

13. Repeat step 4, with the instruments pointed toward ("T") the target station. Follow this with a reciprocal collimation, step 5.

14. BACKSIGHT: Conclude the observations of the series by repeating a set of observations to the leveling rod, steps 6 and 7. Intercept the same graduations used for the first set. If this is not possible, the instrument station has moved and the series must be started over.

15. Prepare the ending series record (line *42*). Include the time, temperature, wind code, and sun code.

16. This step can be performed after all the crossing observations are completed. Compute the elevation difference for the series, as follows:

a. Compute the backsight intercept, in rod units:

$$\text{Backsight} = G + (I_{\text{start}} + I_{\text{end}})/2.$$

b. Compute the foresight intercept, in rod units:

$$\text{Foresight} = T + (H_1 + H_2 + H_3 + H_4)/4.$$

c. Compute the elevation difference, Δh, for the series, in meters. For half-centimeter rod units:

$$\Delta h = (\text{Backsight} - \text{Foresight})/200.$$

For centimeter rod units:

$$\Delta h = (\text{Backsight} - \text{Foresight})/100.$$

d. Compute the backsight distance, s_B, in meters:

$$s_B = (S_{\text{start}} + S_{\text{end}})/2.$$

e. Compute the foresight distance, s_F, in meters:

$$s_F = (S_1 + S_2 + S_3 + S_4)/4.$$

f. Compute the total distance for the series: $s_B + s_F$.

Leveling, nearby target station to nearby bench mark. As before (sec. 4.4.3), double run the section from the height stud of the nearby target station to the nearby bench mark. Check for closure with the previous leveling.

4.3.4 Final Computations

If time permits and if data are recorded on a computer, make the final computations for each running while at the crossing site. In any case, all observation records should be submitted to the project office at the end of the day's work.

Each completed crossing should include the records listed in table 4-2. Compute the elevation difference on a cover sheet for each running, as follows (fig 4-19):

1. From the leveling data for side B, enter the elevation differences, Δh, and distances, S, observed from the height stud of the nearby target station to the nearby bench mark. Use the data from the two sections leveled before and the two sections leveled after the series. Mean the elevation differences.

2. From the series data for side A, enter the elevation difference, Δh, and distance, S, observed from the nearby bench mark to the height stud on the opposite target station.

3. Add the leveling results from side B to the series results from side A. The values obtained are the observed elevation difference, Δh_A, and the distance, S_A, from bench mark A to bench mark B.

4. Repeat steps 1 through 3 with the leveling results from side A and the series results from side B. The values obtained are the observed elevation difference, Δh_B, and the distance, S_B, from bench mark B to bench mark A. Do not expect Δh_B to agree with Δh_A, since neither has been corrected for curvature or refraction.

Table 4-2.—Data set for a complete crossing with Zeiss River Crossing Equipment

Running record	Section	Form
1 River crossing record	BM A to BM B, forward	Cover sheet
Side A		
Survey equipment record (line *40*)	Collimation checks	NOAA 76-191 or 76-189
Beginning and ending running records (line *41*)	Height stud A to BM A, forward	NOAA 76-191 or 76-189
Beginning and ending running records (line *41*)	Height stud A to BM A, backward	NOAA 76-191 or 76-189
Series records (line *42*)	BM A to height stud B	River crossing series
Beginning and ending running records (line *41*)	Height stud A to BM A, forward	NOAA 76-191 or 76-189
Beginning and ending running records (line *41*)	Height stud A to BM A, backward	NOAA 76-191 or 76-189
Side B		
Survey equipment record (line *40*)	Collimation checks	NOAA 76-191 or 76-189
Beginning and ending running records (line *41*)	Height stud B to BM B, forward	NOAA 76-191 or 76-189
Beginning and ending running records (line *41*)	Height stud B to BM B, backward	NOAA 76-191 or 76-189
Series records (line *42*)	BM B to height stud A	River crossing series
Beginning and ending running records (line *41*)	Height stud B to BM B, forward	NOAA 76-191 or 76-189
Beginning and ending running records (line *41*)	Height stud B to BM B, backward	NOAA 76-191 or 76-189
2 River crossing record	BM A to BM B, forward	Cover sheet
Side A		
Series records (line *42*)	BM A to height stud B	River crossing series
Beginning and ending running records (line *41*)	Height stud A to BM A, forward	NOAA 76-191 or 76-189
Beginning and ending running records (line *41*)	Height stud A to BM A, backward	NOAA 76-191 or 76-189
Side B		
Series records (line *42*)	BM B to height stud A	River crossing series
Beginning and ending running records (line *41*)	Height stud B to BM B, forward	NOAA 76-191 or 76-189
Beginning and ending running records (line *41*)	Height stud B to BM B, backward	NOAA 76-191 or 76-189

If runs are conducted on separate days, the data set for each day should be as presented for running 1 above.

5. Compute the mean elevation difference for the running, in the forward direction along the line:

$$\Delta H=(\Delta h_A-\Delta h_B)/2.$$

As explained in subchapter 4.1, curvature and refraction error are largely eliminated from this result.

Also compute the mean sighting distance for the running:

$$S=(S_A+S_B)/2.$$

Closing the crossing. Check the runnings for closure, as described in section 3.7.7. Make additional runnings until the section is closed within the tolerance for the order and class of the survey (table 3-1). All records should be filed with the other observation records for the line of leveling.

4.4 Instructions for Ordinary Leveling Instruments

In 1929 the Coast and Geodetic Survey published an observing routine suitable for performing river crossings with the Fischer level, the leveling instrument in use at the time. The routine employs the basic procedures necessary for a precise crossing: multiple observations of targets, simultaneous reciprocal observations from two instruments, and reciprocal collimation by interchanging the instruments from one side to the other. Since the instruments must be interchanged and since the targets are mounted on leveling rods directly over the bench marks, the routine requires that the site meet criteria (sec. 4.2.1) which are somewhat less flexible than those for the Zeiss River Crossing Equipment. The routine is presented here for use (with suitable leveling equipment) when less than first-order precision is required and the Zeiss equipment is not available.

4.4.1 Equipment and Setups

At each side of the crossing, one leveling instrument, one leveling rod, and two targets should be available. Position each instrument across from the rod on the bench mark at the opposite side. A correct pair of setups is illustrated in figure 4-20. The foresights from each instrument to the opposite rod must be as equal in length as possible and within 1 m (3.3 ft) in height.

Instrument. After performing a collimation check (sec 3.3.7), set up the instrument as usual, placing the

RIVER CROSSING
Zeiss Valley Crossing Equipment

YR.	MO.	DAY
80	07	08

42 800708

FROM 0133 CAISSON USE (BM A) TO 0900 WINDSOR (BM B) RUN 1

Z	TIME BEGIN	TIME END
9	1000	1205

LEVELING At Side B:		Δh	S	LEVELING At Side A:		Δh	S
Before	F	−1.41618	40.5	Before	F	+0.48137	12.4
Before	B	+1.41620	40.5	Before	B	−0.48165	12.0
After	F	−1.41622	40.3	After	F	+0.48150	12.4
After	B	+1.41620	40.3	After	B	−0.48167	12.8
Height Stud B to BM B		−1.41620	(Mean) 40.4	Height Stud A to BM A		+0.48155	(Mean) 12.4

SERIES At Side A: BM A to Height Stud B	+0.07250	703.0	SERIES At Side B: BM B to Height Stud A	+0.43665	693.7
BM A to BM B = Δh_A =	−0.96675	S_A = 743.4	BM B to BM A = Δh_B =	+0.91820	S_B = 706.1

$$\Delta H = \tfrac{1}{2} \times (\Delta h_A - \Delta h_B) = -0.94248 \text{ m}$$

$$S = \tfrac{1}{2} \times (S_A + S_B) = 0.72 \text{ km}$$

Figure 4-19.—Computation of river crossing with Zeiss River Crossing Equipment.

tripod firmly in the ground to ensure that it does not move throughout the crossing routine. The instrument must be equipped with a mechanism for precisely measuring the tilt of the line of sight. The mechanism of the Fischer level is an example of the type required.

The Fischer level (fig. 4-21) is equipped with a precise tilting screw, normally used to center the bubble in the level vial. The screw is graduated to serve as a micrometer, one unit representing one hundredth of a rotation. The position of rotation can be estimated to one tenth of a unit, which in the 1934 model corresponds to a tilt in the line of sight of approximately 0″.5. This precision is compatible with the precision with which the bubble may be centered. Thus, the tilting screw may be read to measure a tilt of the line of sight as small as ±0″.5.

A compensator instrument does not usually possess such a mechanism. The compensator cannot be "tilted" by any means analogous to the tilting screw. An alternative mechanism is the rotary wedge (sec. 4.3.1) but this attachment is not available for most instruments.

Rod. Set a leveling rod on the nearby bench mark. Either rod 1 or rod 2 of the standard pair may be used, since only the low scale is required. Plumb and guy the rod in place with three wires attached at the top center. Rotate it to face the instrument on the opposite side.

Targets. Use two targets that satisfy the requirements given in subchapter 4.1, "Intercepting targets." To permit precise measurement of the target's height against the rod scale, an index line should be clearly marked at the centerline of each target.

Mount the targets securely on the leveling rod. Position one target above and one below the estimated intercept of the horizontal line of sight from the instrument on the opposite side. The distance between the targets should be no more than 1.0 m (3.3 ft) and should be similar to the corresponding distance on the opposite side of the crossing. The angle of tilt from horizontal to each target should be at least ±100″ (20 units on the tilting screw of the Fischer level). Consult with the observer on the opposite side to ensure that these conditions are satisfied.

Enter the positions of the targets in rod units at the top top of the recording form for the first series. If possible, communicate these heights to the recorder on the opposite side.

4.4.2 Observing Routine for Two Targets (Three-Wire Leveling)

One running of this crossing routine includes two sets of simultaneous reciprocal observations. The locations of the instruments for the first set are reversed for the second set. The results of each set are meaned to compute the elevation difference for the running.

Two runnings are required for a complete crossing. They can usually be completed in one day. (See table 4-3 for a summary of the routine.) Conduct the following procedure simultaneously at each side of the crossing, recording the data on the standard forms for three-wire leveling (fig. 4-22).

1. Prepare the beginning series record (line *42*). Include the following: The survey-point serial numbers and stampings of the bench marks (designations if no stampings), running number, set number, local time zone and time, temperature units and temperature, wind code, and sun code.

2. BACKSIGHT: Level the instrument and point it toward the nearby rod. Observe and record the intercepts of the three reticle lines with the scale, as for three-wire leveling. Compute the backsight intercept and distance.

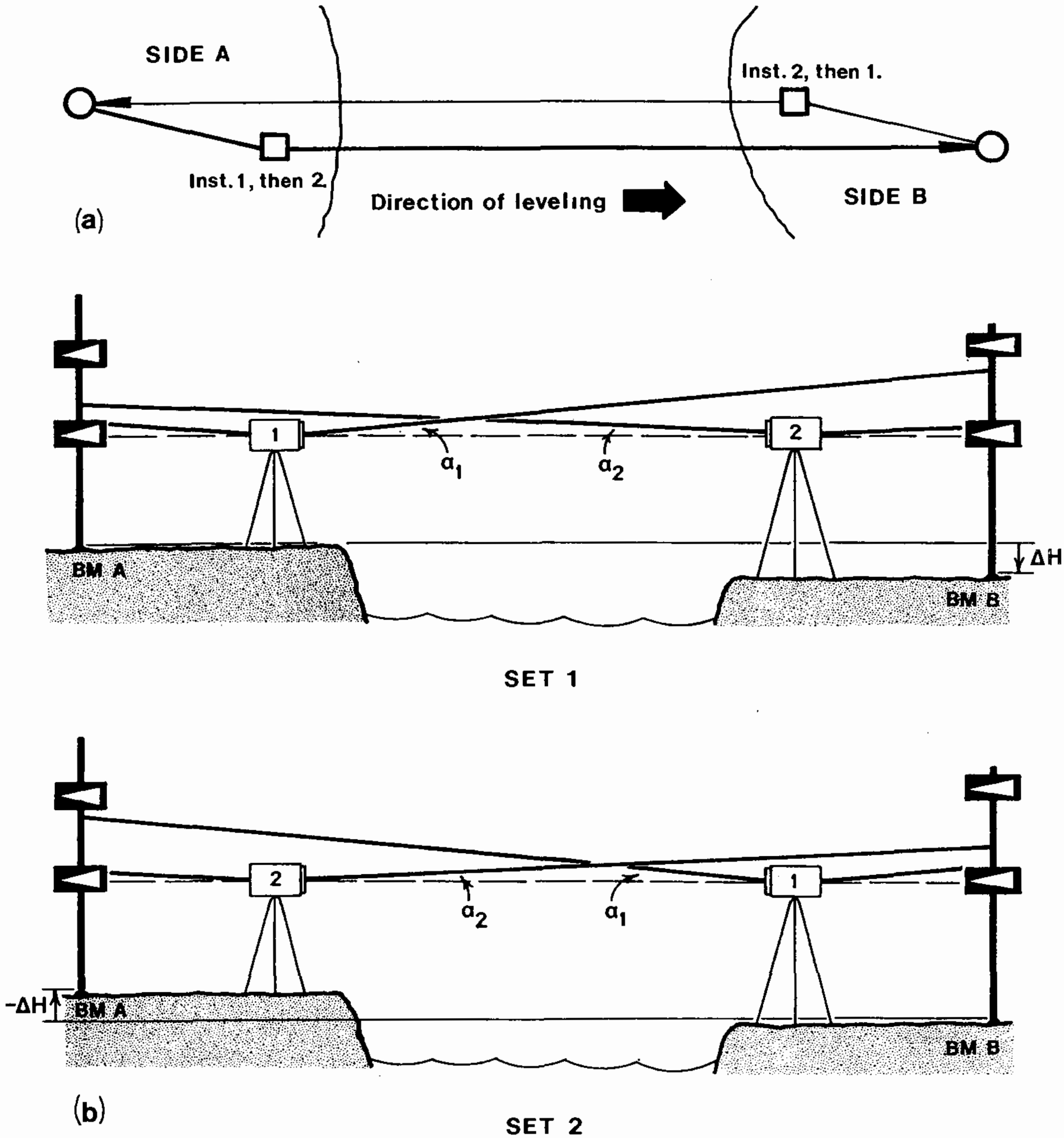

Figure 4-20.—Setups for simultaneous reciprocal observations with ordinary leveling instruments, top view (a) and side view (b). $\Delta H = \frac{1}{2}(\Delta h_{\text{Set 1}} + \Delta h_{\text{Set 2}})$.

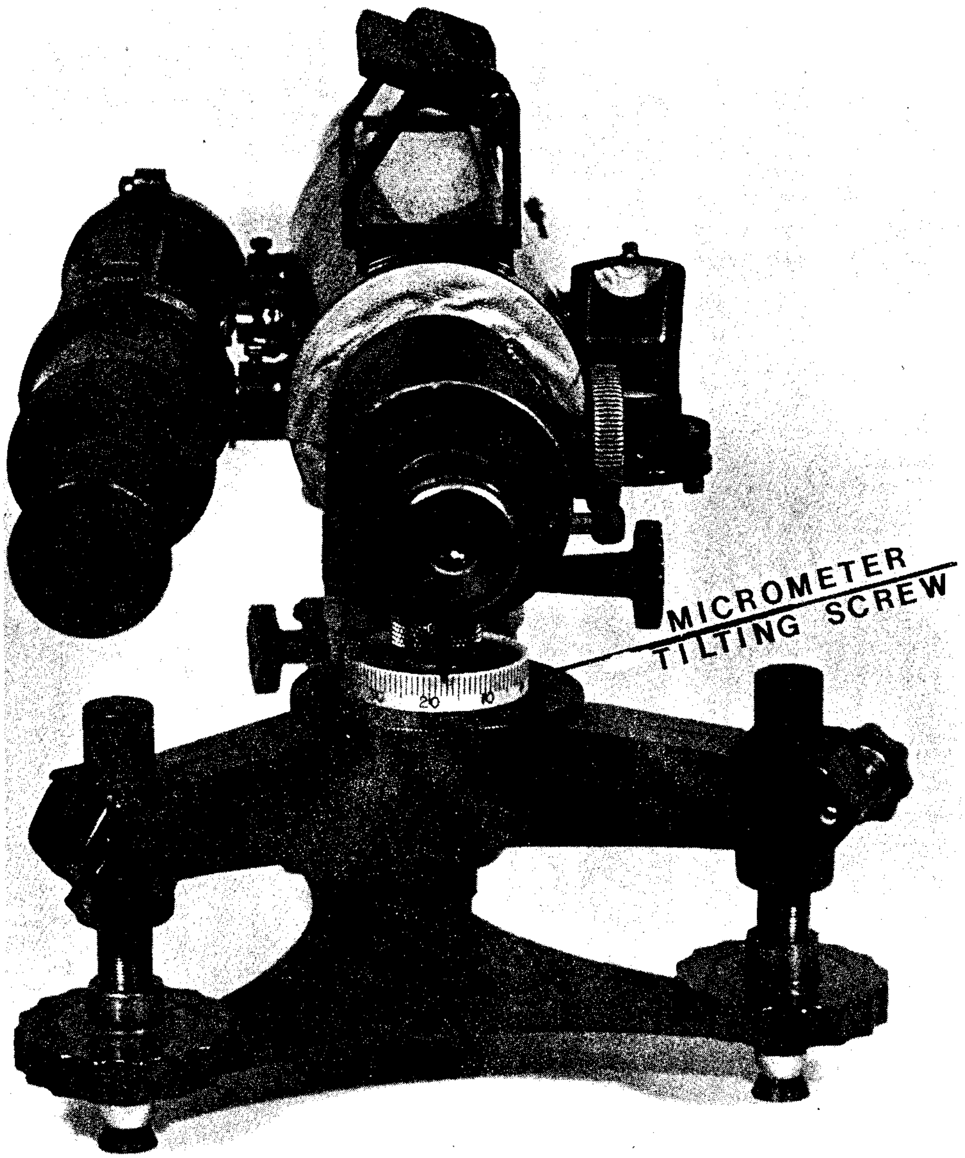

Figure 4-21.—Fischer level, with micrometer tilting screw.

3. FORESIGHT: Check with the observer on the opposite side to begin the foresight observations simultaneously. Point and focus toward the rod on the other side of the crossing. Make 25 sets of three tilt measurements, as follows:

a. Tilt the line of sight to intercept the center of the upper target with the middle reticle line. Read the tilting screw (or equivalent scale) and record the value to the nearest tenth of a unit.

b. Center the bubble in the level vial, bringing the line of sight to horizontal. Read and record the tilt measurement as before.

c. Tilt the line of sight to intercept the center of the lower target with the middle reticle line. Read and record the tilt measurement as before.

4. Compute the foresight intercept and distance:

a. Compute the mean tilts to the upper and lower targets and to horizontal (fig. 4-2): a, b, and r, respectively.

b. Obtain the heights of the opposite targets and compute the difference between them, D.

c. Compute the distance, h, from the lower target to the intercept of the horizontal line of sight:

$$h = D \times (b-r)/(b-a).$$

d. To h, add the height of the lower target to obtain the foresight intercept in rod units.

e. Compute the foresight distance by the formula:

$$s_F = [(b-a)_{\text{tilt units}} \times \text{conversion factor}_{\text{radian/tilt unit}}].$$

For example, with the 1934 Fischer level (0.000025 radian/tilt unit) and centimeter rod units:

$$s_F = (400 \times D)/(b-a).$$

Table 4-3.—Summary of the crossing routine for ordinary leveling instruments

During each running, on each side of the crossing

1. Collimation check.
2. Set up instrument station and leveling rod.
3. Position targets above and below level line of sight from opposite instrument.
4. Observe backsight to nearby rod (1 set of readings).
5. Observe foresight series to opposite rod (25 sets of readings).
6. Move instrument to opposite side. Do not change focus.
7. Observe foresight series to opposite rod (25 sets of readings).
8. Observe backsight to nearby rod (1 set of readings).

During one day

Side A	Side B		
COLLIMATION CHECK	COLLIMATION CHECK		
INSTRUMENT 1 INSTRUMENT 2	INSTRUMENT 2 INSTRUMENT 1	SET 1 SET 2	RUN 1
INSTRUMENT 2 INSTRUMENT 1	INSTRUMENT 1 INSTRUMENT 2	SET 1 SET 2	RUN 2

NOAA FORM 76-189 (4-77) TARGETS THIS SIDE (A) AT 143.0 CM AND 193.0 CM. U. S. DEPARTMENT OF COMMERCE NATIONAL OCEANIC AND ATMOSPHERIC ADMINISTRATION PAGE 2 OF 5

RUN 1
SET 1

GEODETIC LEVELING THREE-WIRE OBSERVATIONS EXAMPLE Second order, Class I RIVER CROSSING

* 4 0 *	YR. / MO. / DAY	CODE	INSTRUMENT SERIAL NO.	CODE	ROD SERIAL NO.	CODE	ROD SERIAL NO.		Z	RUN
* 4 3 *	8 0 0 2 2 8	2 1 1	74	3 1 2	368	3 1 2	387	Fischer Instrument ROD UNIT = centimeter		1

FROM	BM DESIGNATION	TO	BM DESIGNATION	Z	TIME BEGIN	TIME END	F/C	TEMPERATURE BEGIN	TEMPERATURE END	W	S	OBSERVER
0001	A 133	0002	B 133	Q	1022	1038	C	18.0	18.5	0	1	GWW

SET UP	BACKSIGHT UPPER / MIDDLE / LOWER Σ	MEAN Σ	BACK CENTER	S_U / S_L	(U+L) Σ	FORESIGHT UPPER / MIDDLE / LOWER Σ UPPER	MEAN Σ LEVEL	BACK CENTER LOWER	S_U / S_L	(U+L) Σ	COMMENTS OPPOSITE TARGETS
						22.8	49.0	72.8		UPPER	= 127.00
	84.4					22.8	48.8	72.8		Σ 9.1	
1	83.3	83.30	27.3	1.1		22.8	48.7	72.8		a = 22.36	
	82.2			1.1	2.2	22.8	48.8	72.8			
	249.9	83.30			2.2	22.7	48.6	72.5		LEVEL	
	÷ 3.				× 332	22.5	48.4	72.4		Σ 6.8	
	83.30				= 730.4	22.3	48.6	72.5		r = 48.27	
	− 100.02				÷ 100.	22.8	48.7	72.6			
	16.72			S_B	= 7.3	22.5	48.1	72.3		LOWER	= 75.00
	÷ 100.				+ 416.5	22.5	48.0	72.1		Σ 7.6	D = 52.00
	Δh = 0.1672 m.			S	= 423.8 m	22.2	48.1	72.0		b = 72.30	
						22.1	48.0	72.2			
						22.1	48.0	72.1		b − r =	
						22.1	48.1	72.0		24.03	× 52.00
						22.4	48.2	72.5		49.94	
						22.1	48.1	72.1		b − a =	= 25.02
						22.2	48.0	72.1			+ 75.00
						22.2	48.0	72.1			100.02
						22.1	48.0	72.1			
						22.1	48.1	72.1			52×400
						22.2	48.0	72.1			49.94
						22.3	48.1	72.1		S_F	= 416.5 m
						22.2	48.0	72.1			
						22.1	48.2	72.1			
						22.2	48.2	72.2			

Figure 4-22.—One side of a set of simultaneous reciprocal observations, three-wire leveling.

5. Prepare the ending series record (line *42*), which includes time, temperature, wind code, and sun code. Compute the elevation difference and the total distance in the same way as for a setup of three-wire leveling.

6. Leaving the tripod and the leveling rod in place, transport the instrument to the opposite side of the crossing. Do not change the focus on the instrument during this operation.

7. Observe and record a second series, this time observing the foresight (steps 3 and 4) and then the backsight (step 2). The leveling instrument should not have to be refocused to observe the foresight. In conjunction with the series from the opposite side, this set completes the first running.

8. Begin the first set of the second running from the same setup. After completing steps 1 through 5, transport the instrument back to the original side and observe a second set. Again, do not refocus the instrument between sets and begin the second set with the foresight.

9. Compute the results for each running on a cover sheet (fig. 4-23). First, mean the results from the two sides in each set. Then mean the results of the two sets to obtain the elevation difference for the running.

10. Check the runnings for closure, as explained in section 3.7.7. Obtain at least two runnings which close within the tolerance for the order and class of the survey.

REFERENCE

Pfeifer, L. and Morrison, N. L., 1980: *Input Formats and Specifications of the National Geodetic Survey Data Base*, vol. II: Vertical control data. Federal Geodetic Control Committee. National Geodetic Information Center, NOAA/NOS, Rockville, Md. 20852, 136 pp.

RIVER CROSSING
Simultaneous Reciprocal Observations

	YR.	MO.	DAY
42	77	11	28

FROM 0001 A 133 (BM A) TO 0002 B 133 (BM B) RUN 1

Z	TIME BEGIN	END
R	1000	1035

	Δh	S		Mean of Set	
Set 1					
Side A (Instrument 1) F	+0.1300	330.1	Δh_1 =	+0.1614	333.0
Side B (Instrument 2) B	-0.1928	336.5			
Set 2					
Side A (Instrument 2) F	+0.1415	335.2	Δh_2 =	+0.1712	337.6
Side B (Instrument 1) B	-0.2008	340.0			

ΔH = +0.1663 m S = 0.34 km

Figure 4-23.—Cover sheet for a running with ordinary equipment.

This document is a new chapter (4.5) to be added to NOAA Manual NOS NGS 3 (hereinafter referred to simply as NOS 3). This new chapter provides an alternative method to normal leveling techniques for crossing rivers, valleys, highways, or other barriers.

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4.5 Instructions for Theodolites

In 2014, the National Geodetic Survey developed an alternate observing routine suitable for performing river/valley crossings. This alternate routine uses commonly available modern electronic theodolites. The routine addresses sources of error (atmospheric refraction, Earth's curvature, pointing and collimation) which are a function of long-sighting distances and unbalanced shots encountered during river/valley crossings. The routine also addresses instrument and observer errors associated with the use of theodolite instruments. The routine employs the fundamental procedures necessary for a precise crossing: double centering of measurements, multiple observations of targets and simultaneous reciprocal observations. The routine is presented here for use (with suitable theodolites) in crossings of up to 2 kilometers (1.2 miles) in length and up to first-order precision.

4.5.1 Instruments and Setups

Instrument. Suitable theodolites for this observing procedure must have: (1) a telescope that can be plunged, allowing for observations in both direct and reverse mode (2) automatic vertical circle indexing and (3) a minimal instrument least count of 0.1 to 1.0 arcseconds, depending on the desired leveling order and class and crossing width (see Table 4-4).

A measurement procedure known as double centering is used in this routine. Double centering consists of plunging the instrument's telescope between individual measurements and taking measurements to targets in both direct and reverse mode. The mean of two measurements taken this way is self-corrected for any collimation error.

This observing routine relies on the ability to measure precise vertical angles referenced to zenith ("zenith angles"). An error source occurs when a theodolite is not perfectly leveled, creating a misalignment between the instrument's vertical axis and the plumb line. Instruments with automatic tilt-axis compensation self-correct for this potential error source. Theodolite instruments suitable for this routine must have properly functioning automatic vertical-axis compensation.

The least count of an instrument is the smallest measurement increment directly displayed by the instrument. The least count is typically a smaller value than the standard deviation achieved by the measurements taken with the instrument. Table 4-4 identifies minimal least count requirements for instruments dependent on the desired order/class of leveling and crossing width.

Table 4-4.-Instrument Least Count Requirement

	Order/Class of Leveling				
	1/1	1/2	2/1	2/2	3
Crossing Width (meters)	Least Count (seconds)				
100 - 300	0.1	0.5	1.0	1.0	1.0
300 - 600	0.1	0.5	0.5	1.0	1.0
600 - 1000	0.1	0.1	0.5	0.5	1.0
1000 - 1500	0.1	0.1	0.5	0.5	1.0
1500 - 2000	0.1	0.1	0.1	0.5	0.5

The standard deviation of a given set of zenith angle measurements is influenced by both the achievable precision of the instrument and the ability of the observer to accurately point on a target. Table 4-5 represents standard deviations, based on the order and class of leveling being conducted and the width of the crossing, which if achieved should promote a successful crossing routine. Before attempting a crossing routine, you should verify your ability to achieve the standard deviations with your instrument by sighting a well-defined target at your intended crossing width under good sighting conditions, measuring eight sets of direct and reverse mode zenith angles, and then computing the standard deviation of the complete set.

Table 4-5.-Standard Deviation Recommendations

	Order/Class of Leveling				
	1/1	1/2	2/1	2/2	3
Crossing Width (meters)	Standard Deviation (seconds)				
100 - 300	1.4	2.2	3.1	3.9	6.2
300 - 600	1.1	1.4	2.3	2.8	4.2
600 - 1000	0.8	1.1	1.7	2.3	3.4
1000 - 1500	0.7	0.8	1.4	2.0	2.8
1500 - 2000	0.6	0.7	1.1	1.7	2.5

Target Assembly. The target assemblies recommended for this technique (see Figure 4-24) are essentially the same as used for other methods (see Figure 4-9). Each target assembly consists of a target column with two attached targets and an adapter on the foot plate for mounting to a tribrach and tripod. You should construct a set of target assemblies to meet your local needs using materials and methods that are practical and available. Each target assembly consists of four basic parts; a column, foot plate, and two targets. Most machine shops can fabricate target assemblies for this routine using commercially available materials.

The target column should be made of straight, rigid metal stock at least 0.8 meter long. The metal stock should allow for permanent mounting of sighting targets and a foot plate.

Figure 4-24. - Example of recommended target assembly.

The foot plate (see Figure 4-25) provides for mounting of the target assembly to a tribrach and tripod. Ideally, it should be at least 0.5 inches thick and 0.5 inches wider and longer than the bottom of the target column to allow approximately 0.25 inches of foot plate extension surrounding the target column. At its center, the foot plate should have a 5/8-11 inch tapped hole to accommodate a rotating tribrach adapter.

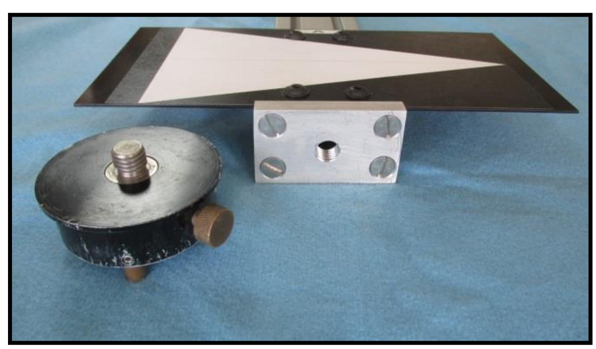

Figure 4-25. –Foot plate with 5/8-11 inch tapped hole and rotating tribrach adapter.

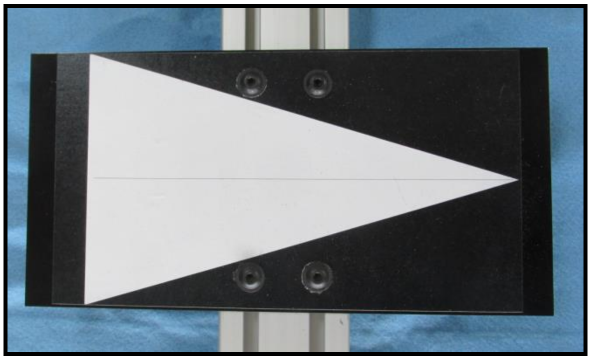

Figure 4-26. –Target, affixed to target column.

Targets (see Figure 4-26) can be constructed of sheet metal or a durable plastic material. Each target should be a 6-inch by 12-inch black rectangle with a 5-inch by 10-inch white isosceles triangle. Each triangle point should be set back from the edge of the rectangle approximately 1 inch to foster good sighting contrast and to protect the target point from incidental damage in handling. Targets should be affixed to the target column with the base of the isosceles triangle perpendicular to the target column. The triangle points should be separated by a distance of at least 0.6 meters and no more than 1.0 meter. It is important to securely attach the targets to the target column to ensure they do not move during field measurements.

After the targets have been attached to the target column, the separation distance between them must be measured to a precision of 0.0001 meter. The separation distance between targets does not have to be the same value for both target assemblies. See Appendix D for how this distance can be readily measured using a digital leveling system. You must measure target separation distances immediately before and after each use to confirm the targets did not shift during observations.

Each target assembly should be uniquely identified to eliminate confusion about which target assemblies were used in a survey or on which side of the crossing they were placed. Each target assembly should have a unique number, letter, or color. Ideally, you should be able to see the identity marking through the telescope eyepiece during the field observations.

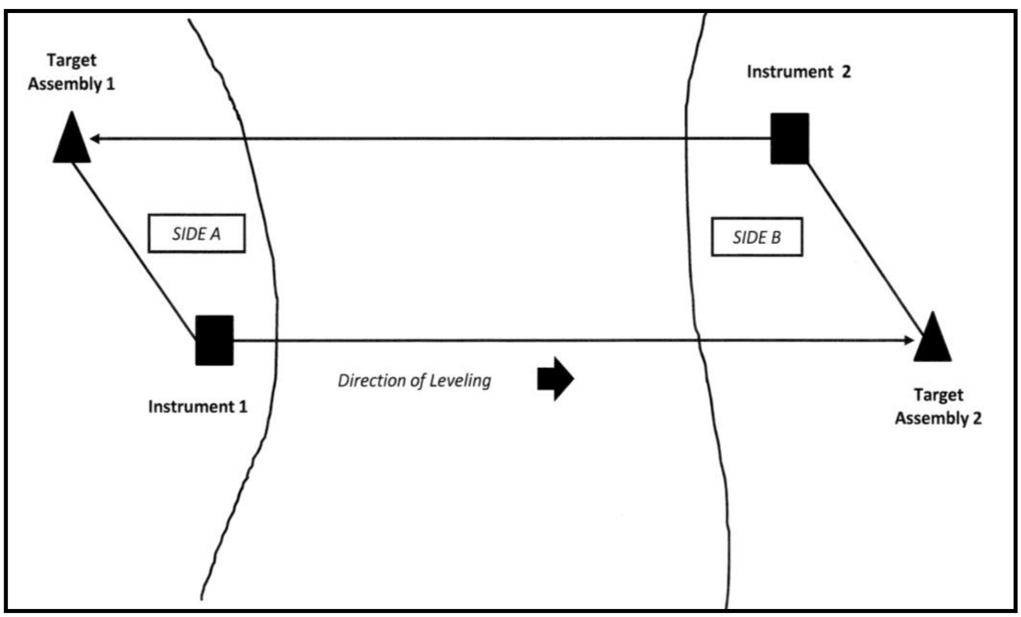

Figure 4-27. - Instrument and target assembly setup.

Equipment Setups. Two theodolites and two target assemblies are required for this routine. Each theodolite is set up directly across from a target assembly on the opposite side of the crossing. Observations from instruments to target assemblies should be balanced, or as equal in length as possible, and over similar terrain and conditions. A correct configuration of instrument setups is illustrated in Figure 4-27. Target assemblies should be set up near a benchmark, ideally within one setup, included in the level line associated with the crossing.

Prior to conducting a crossing routine, position each target assembly such that its upper target is above and its lower target is below the intercept of the horizontal line of sight for the level instruments located on either side of the crossing.

Both target assemblies should be located in similar environmental conditions i.e., both in full sun or full shade. The theodolite instruments should be shaded during the observations. Place tripods firmly in the ground to ensure there is no movement of the target assemblies during the crossing routine. The lower target triangle point for both sides of crossing will be treated as a temporary benchmark, so the target assembly must remain stable throughout the crossing routine.

Immediately before beginning a crossing routine, conduct double-run leveling for the leveling order and class associated with the crossing routine, between the nearby bench mark and the bottom target triangle point on both sides A and B of the crossing (See chapter 3.7.5 "Leveling to Awkward Control Points.") When measuring directly to a feature, such as a triangle point with a digital level, the reticle of the level instrument being used must be coincident with its horizontal line of sight, or a systematic error source will be introduced into the measurement. Appendix E provides specific instructions for checking a digital level to ensure the reticle is coincident with the horizontal line of sight.

4.5.2 Observing Routine

One running of the crossing routine includes a minimum of eight sets of observations from both instrument setups to their respective nearby target assemblies and eight sets of simultaneous reciprocal observations from both instrument setups to their respective distant target assemblies. You must record all associated metadata on the appropriate form. (See Appendix C for an example observing form for this new procedure). Take beginning and ending temperature information at the height of the target assemblies. For a complete crossing, two independent runnings are required. Leaving the target assemblies in place, break down the instrument setups, and reset them in the same location before beginning the second running. You should only attempt the crossing routine when environmental conditions are conducive to favorable results. (See subchapter 4.2, "Environmental Conditions").

1. Each observer should set their instrument up in a safe and secure location, so that lines of sight to both target assemblies are clear and unobstructed and measurements can be taken from a comfortable observing height. The instrument should be shaded from the sun. You must set up each instrument so that its horizontal line of sight intercepts or falls somewhere between the upper and lower targets of both side A and side B target assemblies. For the most precise results, the angular difference between upper and lower targets should be between 100 and 250 arcseconds and target separation distance should not exceed 1.0 meter. This is possible at sighting distances of 2.0 kilometers or less.

2. Record beginning information metadata (see Table 4-6).

3. Remove parallax and sharply focus on targets. With the telescope in direct pointing mode, observe zenith angles to the nearby upper and lower targets. Plunge the scope and rotate the instrument to put the telescope in reverse pointing mode and repeat the zenith angle observations. This completes one set of zenith angles. At a minimum, observe eight sets of zenith angles with the telescope in direct and reverse mode to the nearby targets. With the telescope in direct pointing mode, observe zenith angles to the opposite side upper and lower targets. Plunge the scope, rotating the instrument to put the telescope in reverse pointing mode and repeat the zenith angle observations. At a minimum, observe eight sets of zenith angles with the telescope in direct and reverse mode to the opposite side targets. Targets should be observed in a logical pattern. Be sure each observer follows the same observing pattern. Observers must synchronize sets of zenith angle measurements taken to opposite side targets to within one minute of each other to adequately correct for the effects of refraction. Failure to synchronize measurements may result in failure of the crossing routine. Record associated metadata for each set of measurements (see Table 4-6).

4. Record ending information metadata (see Table 4-6).

5. Reset both instrument setups and conduct a second running, following steps 1 through 4. The target assemblies should not be reset.

6. At the completion of the second running, verify the target assemblies did not settle during the crossing routine. Recheck the target assembly plumbing bubbles, etc.

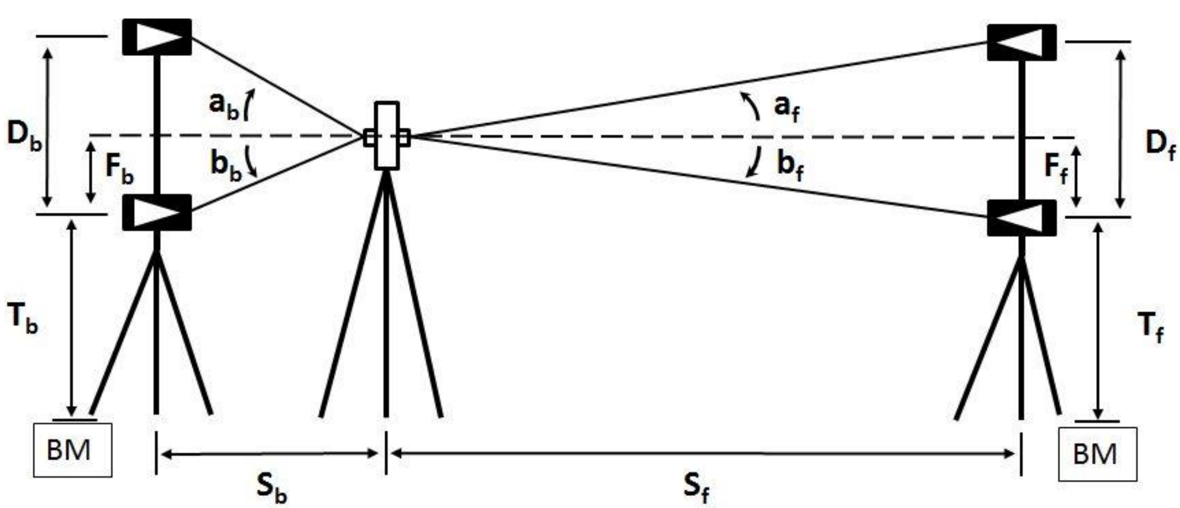

Figure 4 -28. Computational components

4.5.3 Computation Sequence

1. For the first running and for both sides of the crossing, compute a backsight target intercept height F_b and a backsight stadia distance S_b, (see figure 4 -28):

a. Using measured zenith angles, compute elevation angle a_b to upper targets and depression angle b_b to bottom target for each set of measurements.

b. Compute distance F_b, from the lower target triangle point to the intercept of the horizontal line of sight using the formula:

$$F_b = \mathrm{D_b}\ (b_b\ /(b_b - a_b))$$

c. Compute backsight stadia distance S_b using the formula:

$$S_b \approx \mathrm{D_b}\ /(b_b - a_b)$$

This formula is an approximation, but is suitable for computing the stadia distance when using small tilt angle values.

2. For the first running and for both sides of the crossing, compute a foresight target intercept height F_f and a foresight stadia distance S_f, (see figure 4-28):

a. Using measured zenith angles, compute elevation angle a_f to upper targets and depression angle b_f to bottom target for each set of measurements.

b. Compute distance F_f, from the lower target triangle point to the intercept of the horizontal line of sight using the formula:

$$F_f = \mathrm{D_f}\ (b_f\ /(b_f - a_f))$$

Compute foresight stadia distance S_b using the formula:

$$S_f \approx \mathrm{D_f}\ /(b_f - a_f)$$

3. For both side A and side B of the first running, combine target intercept heights F along with leveled height differences T to compute a section height difference H between bench marks:

$$\mathrm{H} = (\mathrm{T_b} + \mathrm{F_b}) - (\mathrm{F_f} + \mathrm{T_f})$$

and combine stadia distances S for a mark-to-mark stadia distance:

$$\mathrm{S} = \mathrm{S_b} + \mathrm{S_f}$$

4. Mean the two height differences and stadia distances computed in step 3, computing an average height difference and stadia distance for the section between bench marks for the first running.

5. Compute an average section height difference and stadia distance for the second running by completing steps 1 through 4.

6. Check the mean height differences of the first and second runnings computed in steps 4 and 5 to make sure they meet section closure requirements for the leveling order and class being undertaken, as explained in section 3.7.7. If the height differences do not meet closure requirements, additional runnings will be necessary, as described in detail in section 3.7.7.

7. Create an Elevation Difference file (HGZ) for the river crossing, and merge it with the HGZ file associated with the level line being continued through the crossing.

Table 4-6.- Required Metadata

When to Record	Required metadata
Before Measurements	Side of Instrument setup (A or B)
Before Measurements	Level line number
Before Measurements	Date of observations
Before Measurements	Start time of observations
Before Measurements	5-Digit Weather Code
Before Measurements	Observer and Recorder names
Before Measurements	Temperature and temperature units
Before Measurements	Instrument model and serial number
Before Measurements	BM stamping and SSN (side A and B)
Before Measurements	Target Assembly identifier and separation distances (side A and B)
Before Measurements	Leveled Height diff. from BM to lower target (side A and B)
During Measurements	Time at start of each set of measurements
During Measurements	Target being observed for each set of measurements (lower/upper)
During Measurements	Instrument face (D or R) for each set of measurements
During Measurements	Zenith angles (Degrees, minutes and seconds)
After Measurements	5-Digit Weather Code
After Measurements	Ending time
After Measurements	Ending temperature and temperature units

Appendix D –
Instructions for Measuring Target Separations Using a Digital Leveling System

Purpose

This appendix describes how a digital leveling system consisting of a digital bar-code level and a one-piece staff can be used to measure the separation distance between target assembly target triangle points to a precision of +/- 0.0001 meters.

Specific Instructions

Set up a level staff on top of a turning pin using the brace poles. Be sure the turning pin and staff are secure and will not move throughout the entire procedure.

Secure a target assembly with tribrach onto a fixed-leg tripod, taking care to finely plumb the target column. Locate the target assembly less than one meter to the side of the level staff and perpendicular to the intended instrument line of sight.

Set up the level approximately 5 meters away so that the distances from the level to the staff and from the level to the target assembly are close to the same length.

The target assembly should be set up to conveniently sight both the upper and lower targets (as shown in Figures D-1 and D-2 respectively).

Figure D-1. Sighting on the upper target triangle point.

Step 1 – Upper Target Staff Reading

By trial and error, sight the upper target triangle point as finely as possible. Adjust the tripod legs first, and later use the foot screws of the level to raise and lower the instrument line of sight. Once the triangle target point has been finely bisected with the horizontal crosshair of the level, rotate the level to read the adjacent staff. Record the staff reading. Repeat this entire step at least three times to be sure you have obtained consistent sighting of the target point and rod readings. Average the readings to obtain a staff reading for the upper target.

Figure D-2. Sighting on the lower target triangle point.

Step 2 – Lower Target Staff Reading

By trial and error, sight the lower target triangle point as finely as possible. At first, adjust the tripod legs, and later use the foot screws of the level to raise and lower the line of sight. Once the target point has been finely bisected with the horizontal crosshair of the level, rotate the level to read the adjacent bar-code staff. Record the staff reading. Repeat this entire step at least three times to be sure you have obtained consistent sighting of the target point and rod readings. Average the readings to obtain a staff reading for the lower target.

Step 3 – Computing Target Separation

Subtract the staff reading of the lower target from the staff reading of the upper target. This height difference represents the target separation distance. The value should be recorded to 0.00001 meters.

Step 4 – Target Separation Distance

Repeat setups 1 through 3 after resetting the turning pin and finely plumbing the level staff. This second separation distance should agree with the first distance recorded to +/- 0.0001 meter. If it does, the mean of these two measured distances is the final separation distance. The final separation distance should be recorded for use in future computations. If it does not, you must repeat steps 1 through 4.

Appendix E.
Instructions for Verification of Instrument Reticle Coincidence with Horizontal Line of Sight

This appendix describes a procedure for verifying that an instrument's reticle is coincident with its horizontal line of sight. When measuring directly to a feature, such as a target triangle point, the horizontal crosshair of the level instrument's reticle must be coincident with its horizontal line of sight. If not, a systematic error source will be introduced into the measurement.

Specific Instructions

Step 1 – Construct a suitable adapter.
Install a footplate using instructions provided in chapter 4.5.1, "Instruments and Setups," onto the top of one of the two target assembly columns (see figure E-1). Place an invar strip parallel to the edge of the back side of the target column and near its center. Drill two small-diameter holes through the invar strip and target column. Secure the invar strip to the target column using two self-tapping metal screws appropriately sized for the drill holes.

Step 2 – Identify an "edge."
Approximately at the midpoint on an invar strip, arbitrarily select a top or bottom edge of a black stripe or block. Mark this "edge" so it can be identified for future measurements. The edge has a unique and precise digital value recognized by the level instrument. It will be repeatedly observed optically and digitally with the strip both erect and inverted. Through an iterative process, the instrument's horizontal crosshair will be checked, and if necessary, aligned to the edge.

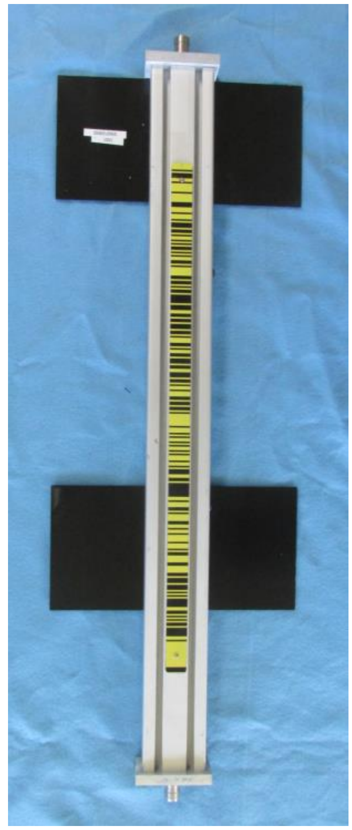

Figure E-1. Sighting target with invar strip attached.

Step 3 – Set up the equipment.
Mount the target assembly to a tribrach and tripod. Take care to finely plumb the invar strip. Adjust the tripod height so the midpoint of the invar strip is at a comfortable observing height. Set up the level instrument nearby. The instrument will need to be adjusted up or down very precisely. An adjustable-height instrument stand would be a good choice here, however a standard tripod and the instrument's footscrews can also be used.

Step 4 – Sight on the invar strip.

Point the instrument on the invar strip, adjusting it up or down so the horizontal crosshair intercepts the edge identified in step 2. Take time to be as precise as possible. When you are satisfied with the pointing, read the rod digitally, and record the value. Repeat this two more times, and calculate the standard deviation. Repeat as necessary until the standard deviation of the series of readings meets an acceptable tolerance of 0.0001m. Determine the average value of the series of readings.

Step 5 – Invert the invar strip and sight on the edge identified in step 2.

Without moving the tripod and tribrach setup, invert the target assembly and remount it in the tribrach. Set the level to read an “inverted” rod and repeat step 4.

Step 6 – Check for misalignment.

Compare the erect and inverted average values. Half of the difference between these two values is the amount of misalignment in the horizontal crosshair with the instrument’s line of sight. If the misalignment is less than 0.0001 meter, the check is complete and no adjustment is necessary. If the misalignment exceeds 0.0001 meter, then an adjustment of the horizontal crosshair will be necessary.

Step 7 – Adjust the horizontal crosshair.

Using the instrument footscrews, adjust the instrument up or down while taking a series of test readings until the mean height determined in step 6 is displayed. Once this is accomplished, use the appropriate tool and the manufacturer’s instructions to adjust the horizontal crosshair to the edge identified in step 2. Check the new alignment by repeating steps 4 and 5.

Observing Form for Optical Total Station
River Crossing Routine

Optical Total Station River Crossing Routine	Side:		Page __of __

Level Line #	Date	Start Time:	Start 5 digit Weather Code:
Observer:		Recorder:	
Instrument model	Instrument serial#	Temperature units:	Beginning Temperature :
BM Stamping:	BM SSN#	BM Stamping:	BM SSN#
Target Assembly #	Target Sep.	Target Assembly #	Target Sep.
Height diff. - BM to Target Assembly lowe		Height diff. - BM to Target Assembly lower	

Backsight (nearby target assembly)

Set #	Target	Face	Time	Zenith Angle Deg.	Min.	Sec.
1	*Lower*	*D*				
		R				
	Upper	*R*				
		D				
2	*Upper*	*D*				
		R				
	Lower	*R*				
		D				
3	*Lower*	*D*				
		R				
	Upper	*R*				
		D				
4	*Upper*	*D*				
		R				
	Lower	*R*				
		D				
5	*Lower*	*D*				
		R				
	Upper	*R*				
		D				
6	*Upper*	*D*				
		R				
	Lower	*R*				
		D				
7	*Lower*	*D*				
		R				
	Upper	*R*				
		D				
8	*Upper*	*D*				
		R				
	Lower	*R*				
		D				

Mean Z angle upper target	
Mean Z angle lower target	
Lower target deflection angle	radians:
Upper target elevation angle	radians:
DH - lower target to Intercept	
Stadia dist. - Inst. To Target	
End Time:	Temperature units:
End weather code:	Ending Temp:

Foresight (opposite side target assembly)

Set #	Target	Face	Time	Zenith Angle Deg.	Min.	Sec.
1	*Lower*	*D*				
		R				
	Upper	*R*				
		D				
2	*Upper*	*D*				
		R				
	Lower	*R*				
		D				
3	*Lower*	*D*				
		R				
	Upper	*R*				
		D				
4	*Upper*	*D*				
		R				
	Lower	*R*				
		D				
5	*Lower*	*D*				
		R				
	Upper	*R*				
		D				
6	*Upper*	*D*				
		R				
	Lower	*R*				
		D				
7	*Lower*	*D*				
		R				
	Upper	*R*				
		D				
8	*Upper*	*D*				
		R				
	Lower	*R*				
		D				

Mean Z angle upper target	
Mean Z angle lower target	
Lower target deflection angle	radians:
Upper target elevation angle	radians:
DH - lower target to Intercept	
Stadia dist. - Inst. To Target	
Ht diff lower to lower target	
Ht diff BM to BM	

Comments: