SUBCOURSE EN0593 | EDITION A

UNITED STATES ARMY ENGINEER SCHOOL

SURVEYING III (TOPOGRAPHIC AND GEODETIC SURVEYS)

"LET US TRY"

THE ARMY INSTITUTE FOR PROFESSIONAL DEVELOPMENT

ARMY CORRESPONDENCE COURSE PROGRAM

SURVEYING III (TOPOGRAPHIC AND GEODETIC SURVEYS)

Subcourse EN0593

EDITION A

United States (US) Army Engineer School
Fort Leonard Wood, MO 65473

22 Credit Hours

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SUBCOURSE OVERVIEW

This is one of a series of subcourses intended to assist enlisted personnel with improving their proficiency in job requirements for military occupational specialty (MOS) 82D, Topographic Surveyor. This subcourse covers the fundamental principles and applications of topographic and geodetic surveying.

This subcourse is designed to teach the student the techniques necessary to determine the precise position, azimuth, or elevation of a point. Additionally, this publication will provide information concerning the description and standardization procedures for performing reconnaissance, preparing station descriptions, and reporting and briefing survey projects.

There are no prerequisites for this subcourse, although it is highly recommended that the student complete subcourses EN0591 and 0592 before beginning this subcourse.

This subcourse reflects the doctrine which was current at the time it was prepared. In your own work situation, always refer to the latest publications.

The material in this subcourse is applicable, without modification, to all geodetic-survey projects in all environments (prebattle, conventional war, low-intensity conflicts, and postbattle). This subcourse complies with the Army doctrine and international precision surveying practices. It does not provide previously published surveying doctrine or theory and may be supplemented with commercially available text or previous editions of technical literature.

Unless otherwise stated, the masculine gender of singular pronouns is used to refer to both men and women.

TERMINAL LEARNING OBJECTIVE:

ACTION: You will learn the techniques necessary to determine the precise position, azimuth, or elevation of a point; how to describe and standardize procedures for performing a reconnaissance; and how to prepare station descriptions, reports, and briefings of survey projects.

CONDITION: You will be given this subcourse, a calculator, and an Army Correspondence Course Program (ACCP) examination response sheet.

STANDARD: To demonstrate competency of this course, you must achieve a minimum score of 70 percent on the subcourse examination.

TABLE OF CONTENTS

LESSON 1

GEODESY FOR THE LAYMAN

OVERVIEW

LESSON DESCRIPTION:

In this lesson, you will learn to identify the different datums and ellipsoids that encompass the earth.

TERMINAL LEARNING OBJECTIVE:

ACTION: You will learn to identify the different datums and ellipsoids.

CONDITION: You will be given the material contained in this lesson.

STANDARD: You will correctly answer all practice questions following this exercise.

REFERENCES: The material contained in this lesson was derived from FM 3-34.331.

INTRODUCTION

When Eratosthenes made his meridional arc measurement some twenty-two hundred years ago, the basic foundation for modern geodesy was established. Through the years, refinements have been made, necessary values of precision and accuracy have been determined, and a scientific attitude has evolved about solving the problem of size and shape. Geodesy is defined as the science of precise positioning of points on the earth's surface and the determination of the earth's exact size and shape. It also involves studying variations in the earth's gravity and how these variations apply to precise measurements of the earth. The geodesist's role is to put things on the surface of the earth in their exact places. In this lesson, you will learn how to perform this task.

PART A - FIGURES OF THE EARTH

1-1. Three Figures of the Earth. Before any type of measurement can take place, the surface on which we measure must be defined. Generally, we can assume the following three figures of the earth: topographic, mathematic, and geoidal.

a. Topographic. The surface most apparent is the actual topographic surface of the earth. This includes the mountains, valleys, and other continental and oceanic forms. The surveyor makes the actual measurements on these surfaces, but because of the irregularities of the land, this figure is not suitable for mathematical computations. This surface generally concerns the topographer and the hydrographer but interests the geodesist only with regard to the effect of the terrain features on gravity.

b. Mathematic. It is convenient to adopt a simple mathematical surface, resembling the actual earth, to permit simplified computations of positions on the earth's surface. We might select a simple sphere; however, the sphere is only a rough approximation of the true figure of the earth. We can and do use a spherical form to solve most astronomical problems and for navigation. The sphere is used to represent the earth because it is a simple surface that is easy to deal with mathematically.

c. Geoidal. The geoid is the equipotential surface within or around the earth where the plumb line is perpendicular to each point on the surface. The geoid is considered a mean-sea-level (MSL) surface that is extended continuously through the continents. The geoidal surface is irregular due to mass excesses and deficiencies with the earth. The figure of the earth is considered a sea-level surface that extends continuously through the continents. The geoid (which is obtained from observed deflections of the vertical) is the reference surface for astronomical observations and geodetic leveling. The geoidal surface is the reference system for orthometric heights.

1-2. Ellipsoid. Refer to Table 1-1 for ellipsoid data. The shape of the earth is more precisely represented mathematically by an ellipsoid of revolution, which is made by rotating an ellipse around its minor axis. The radius of the equator usually designates the size of an ellipsoid. The radius is called the semimajor axis (Figure 1-1). The shape of the ellipsoid is given by a flattening, which indicates how well an ellipsoid approaches the shape of a sphere. Figure 1-2, page 1-3, shows the flattening of various figures. The ellipsoid, which represents the earth very closely, approaches a sphere since it has a flattening of 1/300. An ellipse with such a small flattening is almost a perfect circle. Spheroid and ellipsoid of revolution are accurate terms; however, ellipsoid has become the more accepted term.

Table 1-1. Ellipsoid Data

Name	Equatorial Radius	Flattening	Where Used
Krassowsky (1940)	6,378,245 m	1/298.3	Russia
International (1924)	6,378,388 m	1/297.0	Europe
Clarke (1880)	6,378,249 m	1/293.5	France, Africa
Clarke (IM)	6,378,206 m	1/295.0	North America
Bessel (1941)	6,377,397 m	1/299.15	Japan
Airy (1830)	6,376,542 m	1/299.3	Great Britain
Everest (100)	6,377,276 m	1/300.8	India
New International (1967)	6,378,160 m	1/298.247	Worldwide

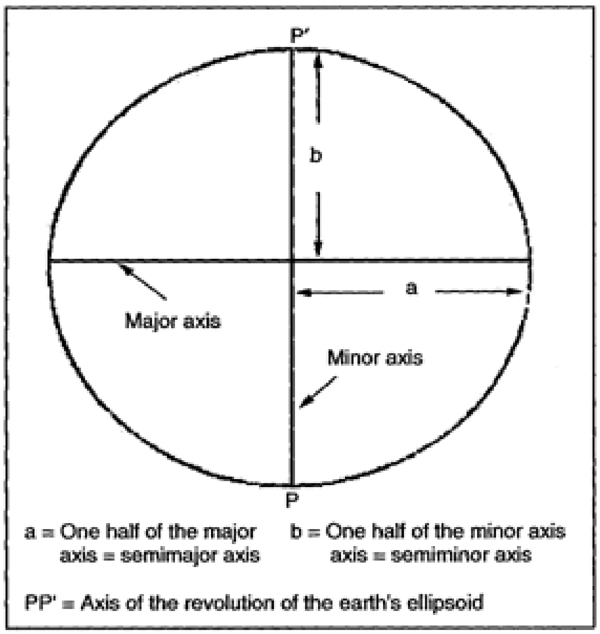


Figure 1-1. Components of an Ellipsoid

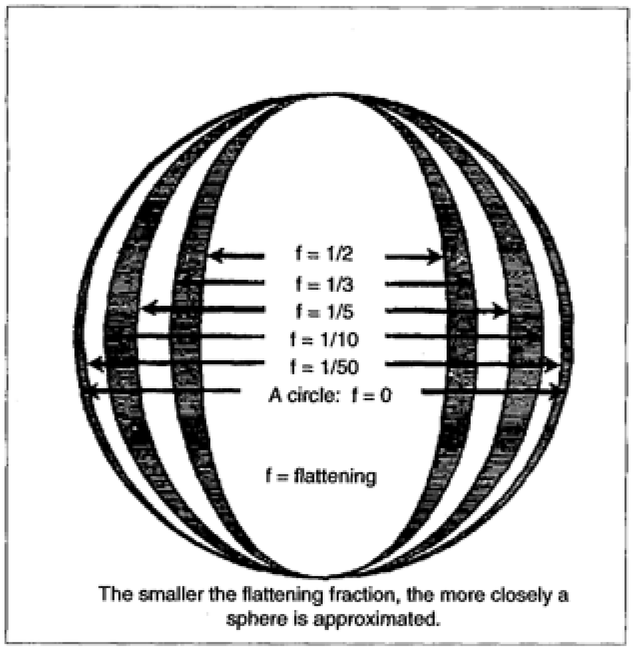


Figure 1-2. Flattening Fraction

a. Types of Ellipsoids. The two ellipsoids currently in use are named for the individuals who derived them. The differences between the two types are very small; however, there are a great number of ellipsoids actually in use. Generally, they describe or fit the portion of the earth where they are used.

(1) The international ellipsoid was developed by John F. Hayford in 1910 and adopted by the International Union of Geodesy and Geophysics (IUGG), which recommended it for international use.

(2) The new international ellipsoid was developed by William M. Kaula in 1964 and adopted by the IUGG, which recommended it for worldwide gravity surveys, updating of the World Geodetic System (WGS) and scientific survey requirements.

b. Axis of Rotation. The axis of rotation of an ellipsoid is always parallel to the axis of rotation of the earth. Ideally, the center of the ellipsoid should coincide with the earth's center of gravity. The bubble level on a surveyor's instrument shows only the general direction of the earth's center of gravity. The solution to this problem and its importance will be shown later. The ellipsoid is a reference surface that has been selected as a mathematical convenience to represent the figure of the earth.

c. Geodetic Latitude and Longitude. Points on the ellipsoid can be defined in terms of latitude and longitude. These coordinates are called geodetic latitude and longitude. These coordinates are the same as those that appear on charts and maps. There is only one value of geodetic latitude and longitude, which locate a single point to the ellipsoid.

In Figure 1-3, the geodetic latitude of point P has been formed by the angle of the semimajor axis and a perpendicular (or plumb) from point P on the edge of the ellipse. The geodetic longitude of point P has been formed by the angle between the plane of the Greenwich meridian and the meridian plane of point P.

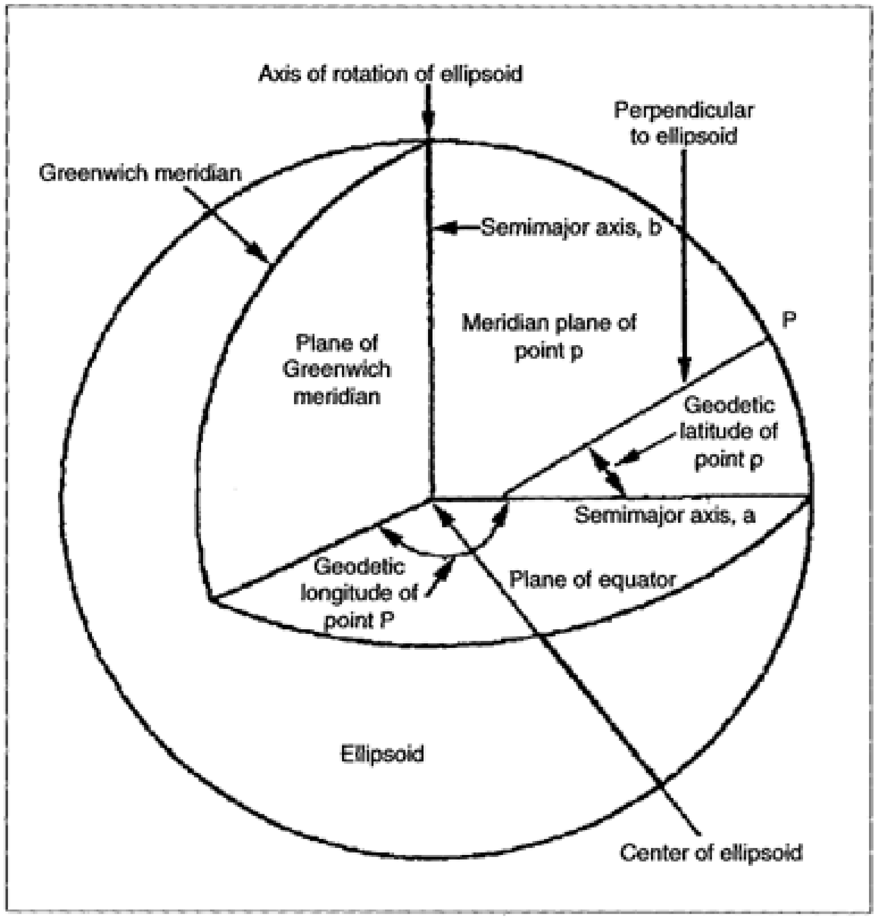


Figure 1-3. Geodetic Coordinates

d. Defining the Earth's Equator. For years, scientists have discussed the possibility that the earth's equator is an ellipse rather than a circle and, therefore, that the ellipsoid is triaxial. Until recently, the study has been slowed due to insufficient data. Modern technological developments have furnished new and rapid methods for data collection, and since the launching of the first Russian sputnik, orbital data has been used to investigate the theory of ellipticity. However, it will be some time, even at today's rate of data collection, before an exact conclusion is made.

e. A Second Theory. A second, more complicated theory than triaxiality proposes that satellite orbital variations indicate additional flattening at the south pole, accompanied by a bulge of the same degree at the north pole. It is also argued that the northern middle latitudes are slightly flattened, and the southern middle latitudes are bulged in a similar amount. This new concept suggests a slightly pear-shaped earth and has been the subject of much discussion.

1-3. Geoid. In geodesy, precise computations are made by using an ellipsoid. Unfortunately, measurements made on the earth's surface are not made on a mathematical ellipsoid. The surface is called a geoid.

a. General. The geoid is the surface which the ocean waters of the earth would conform to if they were free to adjust to the forces acting on them. The ocean waters would conform to the surface under the continents if allowed to flow freely through sea-level canals. The forces acting on the oceans include the actual attraction of the earth's mass, attractions due to density differences in the earth's crust, and centrifugal force due to the earth's rotation. The component of centrifugal force opposing the attraction of gravity is greater at the equator than near the poles. Since terrain features such as mountains, valleys, and ocean islands exert gravity forces, they also affect the shape of the geoid. The geoid can also be defined as the actual shape of a surface at which the gravity potential is the same. While this surface is smoother than the topographic surface, the geoid still has bumps and hollows.

b. Characteristics. There are two very important characteristics of the geoid. First, the gravity potential in the geoid is the same everywhere, and the direction of gravity is perpendicular to the geoid. Second, whenever you use an optical instrument with level bubbles, properly adjusted, the vertical axis of the instrument should coincide with the direction of gravity and is, therefore, perpendicular to the geoid. The second factor is very important because the attraction of gravity is shown by the direction of the plumb lines.

c. Deflection of the Vertical. Since the ellipsoid is a regular surface and the geoid is irregular, the two surfaces do not coincide. However, they do intersect, forming an angle between the two surfaces. Geometry has taught us that the angle between the two surfaces is also the angle formed between the perpendicular to the ellipsoid and the geoid plumb line. This angle is called the *deflection of the vertical*. The word *normal* is sometimes used to describe the perpendicular to the ellipsoid and the geoid since a normal is a line perpendicular to the tangent at a curve. In less precise language, this is known as perpendicular to a curve (Figure 1-4).

d. Separations. The separations between the geoid and the ellipsoid are called undulations of the geoid, geoid separations, or geoid heights. The geoid height reveals the extent to which an ellipsoid fits the geoid and thus helps to determine the bestfitting ellipsoid. For purposes of illustration, the undulations of the geoid in Figure 1-4 and other figures are highly exaggerated.

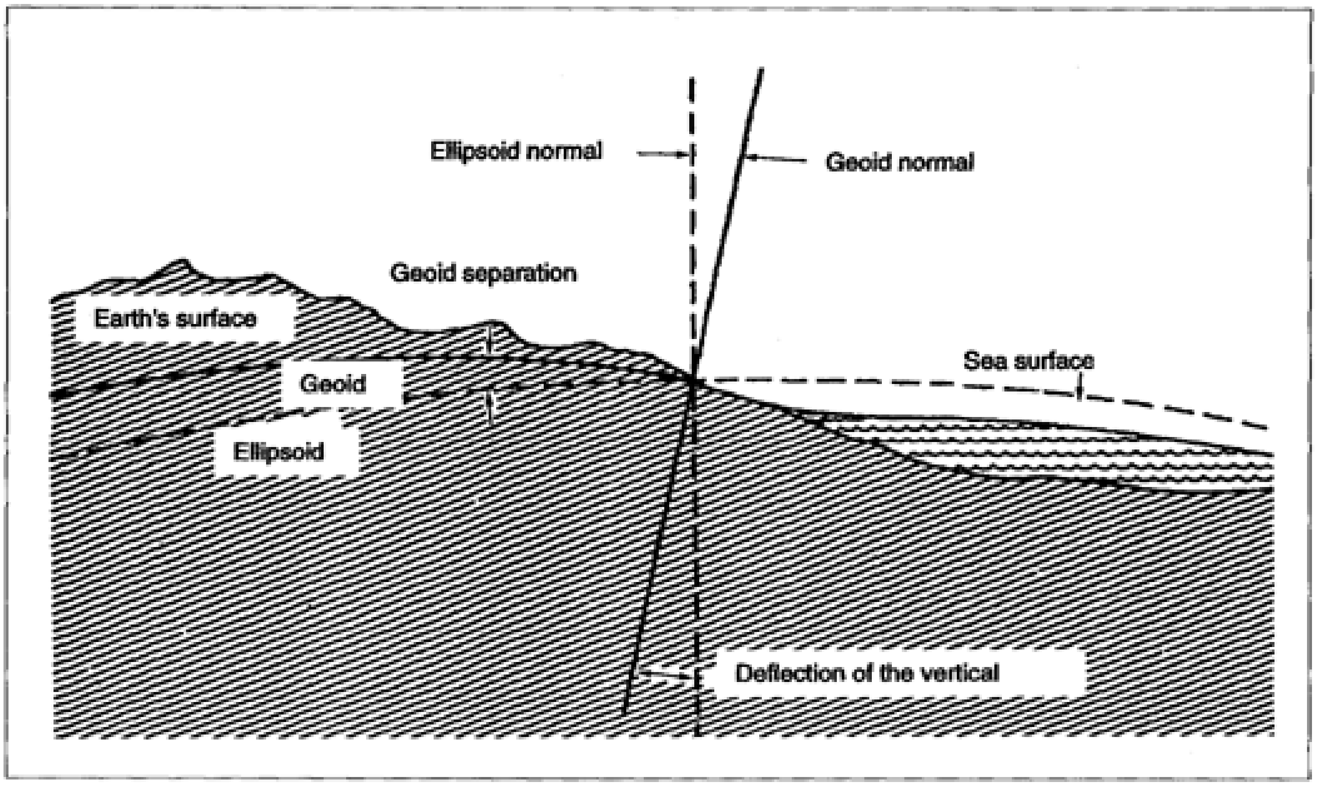


Figure 1-4. The Geoid and the Ellipsoid Rarely Coincide

PART B - GEODETIC SURVEYING

1-4. General. One of the purposes of geodesy is to determine the precise position of points on the earth's surface. The techniques used to do this can be grouped into an astronomic observation, horizontal control, or vertical control by gravimetric. These techniques are described here so that you can examine the methods to join the continents and insure their proper placement on earth; establish correct boundary lines, ending old disputes; make flying safer by identifying the correct placement and elevation of mountain peaks; ensure the correct missile launch information and target locations in this country and in foreign countries; establish exact control-point locations for accurate maps; and aid in the establishment of a WGS. These techniques should also help to visualize how these measurements are used to determine the earth's size and shape.

1-5. Astronomic Observation. The position of a point can be obtained directly by observing the stars. Astronomic positioning is the oldest positioning technique. It has been used for many years by mariners and, more recently, by airmen for navigational purposes. Explorers often used the astronomic technique to locate themselves in uncharted areas. Geodesists must use astronomic positions along with other geodetic survey data to establish precise positions. As the name implies, astronomic positions are obtained directly by measuring the angles between the plumb line at the point and a star, or series of stars, and recording the precise time at which the measurements are made. After combining the data with information obtained from star catalogues, the position is computed.

a. While geodesists use elaborate and very precise techniques for determining astronomic latitude, the simplest method is to measure the angle between the Polaris and the horizon of the observer. Astronomic latitude is defined as the elevation of the Polaris above the horizon, or the angle between the perpendicular to the geoid at point P and the plane of the equator (semimajor axis). The angle of elevation of the Polaris is about equal to the astronomic latitude. The astronomic longitude shown in Figure 1-5 is the angle between the plane of the Greenwich meridian (prime meridian) and the astronomic meridian of the point.

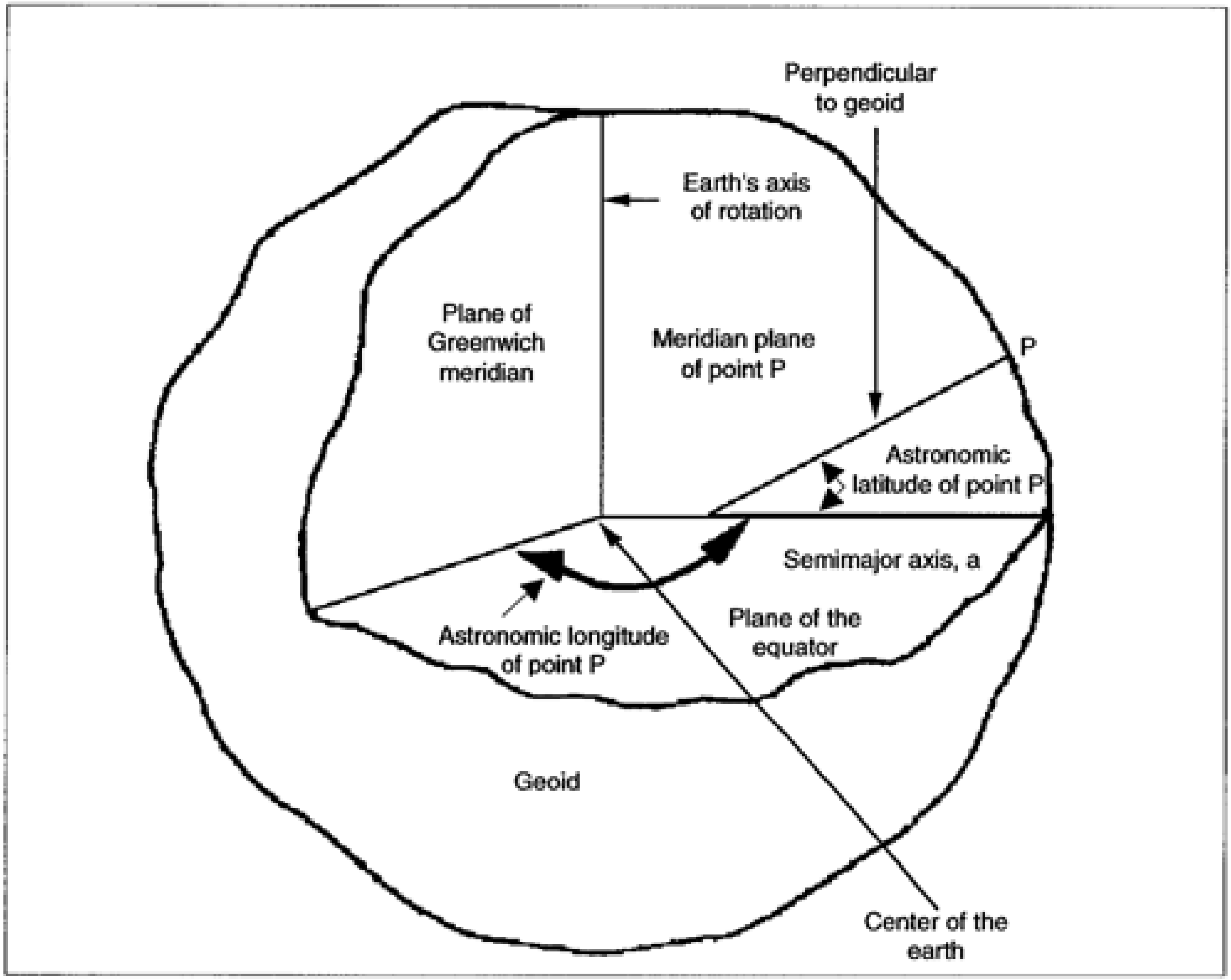


Figure 1-5. Astronomic Coordinates

b. It is necessary to always reference a system of horizontal measurements with respect to the earth, with the exception of surveys with very limited scope, such as engineering plans or city surveys. To obtain this reference, make astronomic observations to obtain astronomic coordinates for points on the earth's surface. You will observe the astronomic latitude and longitude at a starting point known as the origin of the survey. You should also determine an azimuth or directional control to another point in the survey to provide the directional control needed in the survey network. The observation of the longitude and azimuth is frequently repeated in the survey to give control to the geodetic network. When combined with other geodetic measurements, astronomic observations provide a method for determining the deflection of the vertical. This can help in determining the figure of the earth. Places at which astronomic longitude and azimuth are measured are called Laplace stations.

c. When making astronomic observations with an optical instrument containing a leveling device, the vertical axis of the instrument is perpendicular to the geoid. This means that the axis coincides with the direction of the force of gravity or a plumb line at the observation site. Therefore, an astronomic observation gives a direction with respect to the geoid.

d. The normal to the ellipsoid defines geodetic latitude and longitude. Since the direction of gravity rarely coincides with the normal of the ellipsoid, astronomic coordinates do not represent positions on the ellipsoid (Figure 1-6). This difference, the deflection of the vertical, can be used to help determine the accuracy of the survey technique and the suitability of the ellipsoid for use in a particular surveyed area.

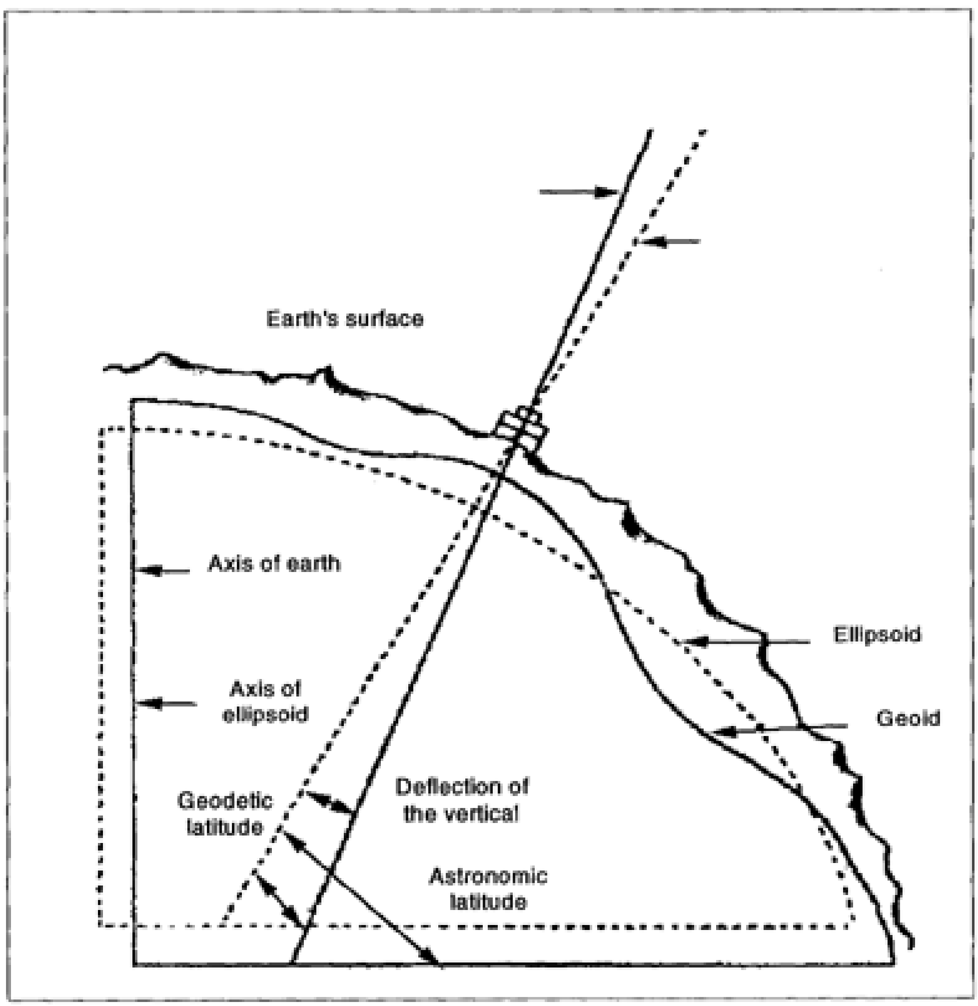


Figure 1-6. Deflection of the Vertical

e. Astronomic observations provide angular relationships only. They provide information on the shape of the earth, not its size. In order to determine distances between astronomic stations, the size of the earth must be determined by horizontal surveying techniques.

1-6. Horizontal Control Technique. Geodesy not only involves determining the shape of the earth; it also involves surveying--measurements on the surface of the earth. Surveys may be plane or geodetic. While plane surveys are restricted to small geographic areas and are computed as though the surface is level or plane, geodetic surveys are used over large geographic areas and require consideration not only of the curvature of the earth but also of the geoid. This is not the only difference between the two types of surveys. Instruments used in geodetic surveying are more accurate. Instrument errors are either removed or predetermined, and the procedures are more rigorous in order to minimize observation errors. The extension of horizontal control can be carried out in various ways. Basically, the procedure starts from some point or points of a known position, the distance and angles to new points are measured, and the positions of these points are computed by means of the measured data. The most common methods of establishing horizontal control are traverse, triangulation, and trilateration. Other methods include celestial techniques. Horizontal angles measured on the earth's surface can be used on the ellipsoid without any additional correction. There may be a very small difference between horizontal angles on the earth's surface and the corresponding angle on the ellipsoid, but the correction is usually smaller than your ability to measure the angle and should be ignored, except in the case of first-order surveys.

a. Traverse. A traverse begins at a position with a known azimuth to some other point, and the angles and distances along the line of survey points are measured (Figure 1-7).

(1) Closed Traverse. A closed traverse starts and ends at the same point or at points which have known relative horizontal positions. The measurements can be adjusted by computations to minimize the effect of accidental errors made in the measurements. Mistakes (blunders) can be detected by rechecking the computations.

(a) Loop Traverse. A loop traverse forms a continuous loop enclosing an area. The computed circuit closure, or error of closure, for a loop traverse normally indicates whether a large mistake was made in the measurements. When mistakes are eliminated, the error of closure will indicate the size of the accidental errors. However, systematic errors will seldom show in the error of closure. For example, when the tape used for distance measurements is longer than its nominal length, all of the recorded lengths will be proportionally too small and will cause little or no change in the computed error of closure of the traverse.

(b) Connecting Traverse. A connecting traverse starts and ends at separate points whose relative positions have been determined by a survey of an equal or higher order accuracy. A connecting traverse of third-order accuracy, for example, may be run and adjusted between two stations whose relative positions were determined by a first-, second-, or third-order traverse or triangulation.

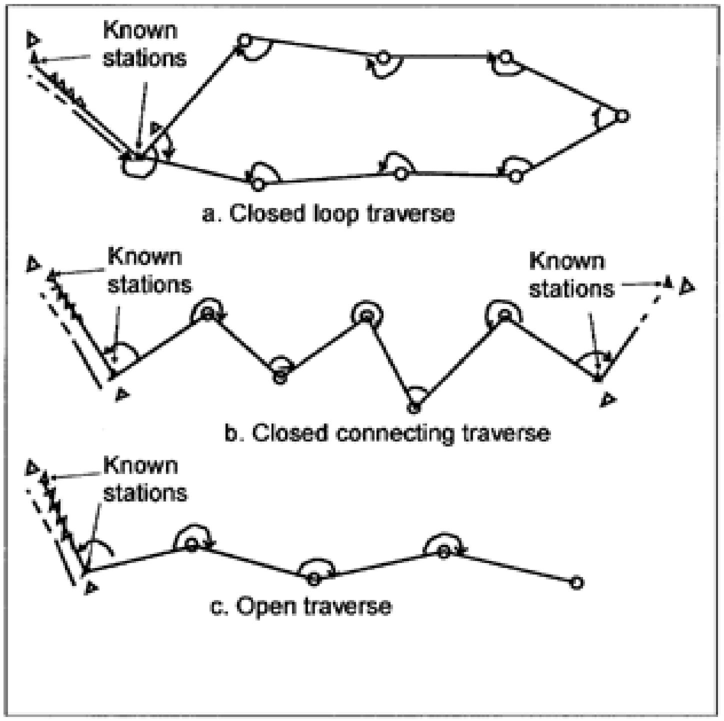


Figure 1-7. Principles of Traverse

(2) Open Traverse. An open traverse ends at a station whose relative position is not previously known. Unlike a closed traverse, an open traverse provides no check against mistakes and large errors. An open traverse usually starts at a station determined by a closed traverse or by triangulation but may start at an assumed position. An open traverse is often used for a preliminary survey for a road or railroad. When the centerline location traverse is run, ties to the preliminary traverse form a series of closed-loop traverses. A random traverse is a special adaptation of an open traverse. It is run between two stations to determine their relative positions. Running the computed line between the two points usually closes the traverse. One example of a random traverse is the measuring of the distance between points on opposite sides of a lake. This is done by running a traverse line around the shore. In this example, the angles of the loop may be closed if a sight is determined across the lake from the final to the initial point. This is not considered a closed-loop traverse, since the distance between the last and the first point is not measured directly. If an electronic distance-measuring device is able to measure the distance, the open traverse becomes a closed traverse.

NOTE: By measuring the angles between intermediate survey points and the length of the traverse sides, you can compute not only the direction of the traverse but also the position of each control point. Traverse control also uses astronomic observations for azimuth measurements.

b. Triangulation. Triangulation is the measurement of angles of triangles. It is the most common type of geodetic survey. If you accurately measure the distance along one side of a triangle and all of the angles, you can compute the other two sides (Figure 1-8, page 1-12). Also, if you know the latitude and longitude of one end of the side plus the length and direction of the side, you can compute the latitude and the longitude of the other end of the side.

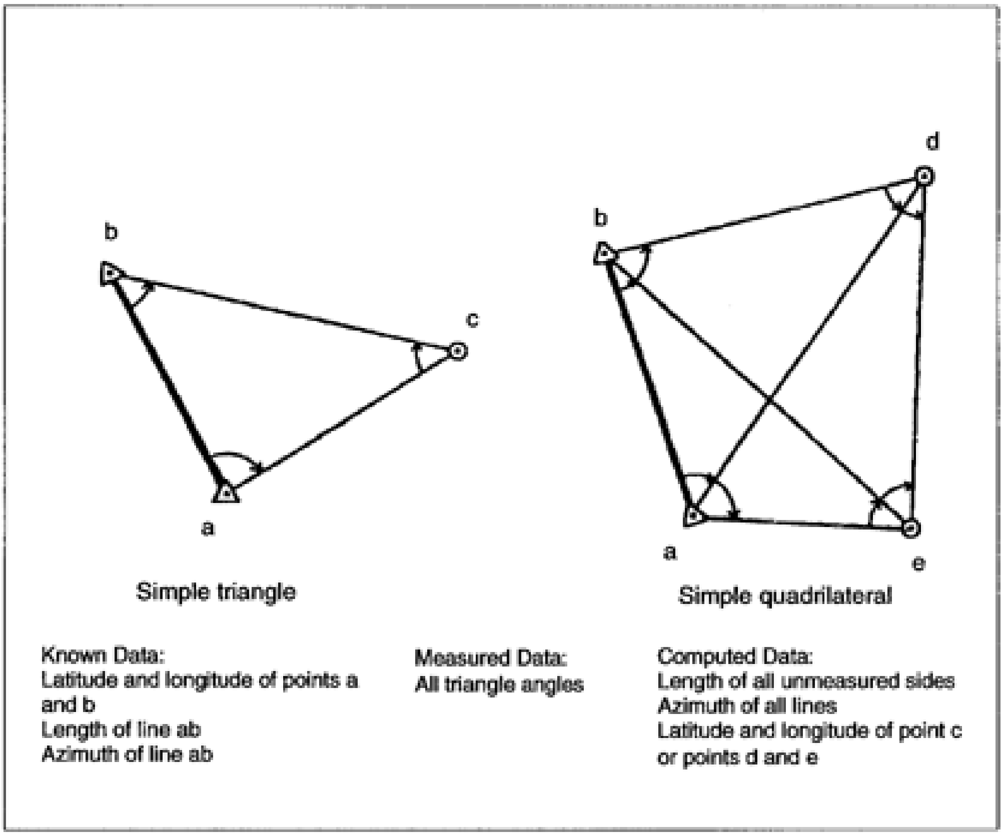


Figure 1-8. Principles of Triangulation

(1) The side of the triangle that is measured is called the baseline. It should be measured very carefully using a bar, a chain, or tape, all of which should be calibrated and periodically checked by the Bureau of Standards. It is not easy to reduce the baseline to the ellipsoid because our knowledge of both the geoid and the ellipsoid is so limited. Until recently, it was impossible because the geoid heights were not known. A compromise is usually necessary, and the baseline is reduced to MSL. The error in doing this is small in the baseline itself; however, since the baseline establishes the scale of the control network, this error may propagate throughout the network. While this source of error may be small, increased knowledge of the undulations of the geoid permits further improvement in the accuracy of horizontal positioning. With comparatively new instruments, such as the tellurometer (a microwave system that employs line-of-sight conditions and is capable of measuring as far as 20 to 25 miles) and the geodimeter (a system that uses light as a carrier and is capable of measuring as far as 2 to 3 miles by day and 15 to 20 miles by night), you can measure distances much faster but not necessarily with greater accuracy than with the conventional methods.

(2) To establish an arc of triangulation between two widely separated locations, you must measure a baseline at each location, connect these measured areas by a series of adjoining triangles forming quadrilaterals, and compute the latitude and longitude for the vertex of each triangle. The vertex is then called a triangulation or geodetic-control station. The end of the baseline is often called the initial point, and the directions of the sides of the triangles are called azimuths (Figure 1-9).

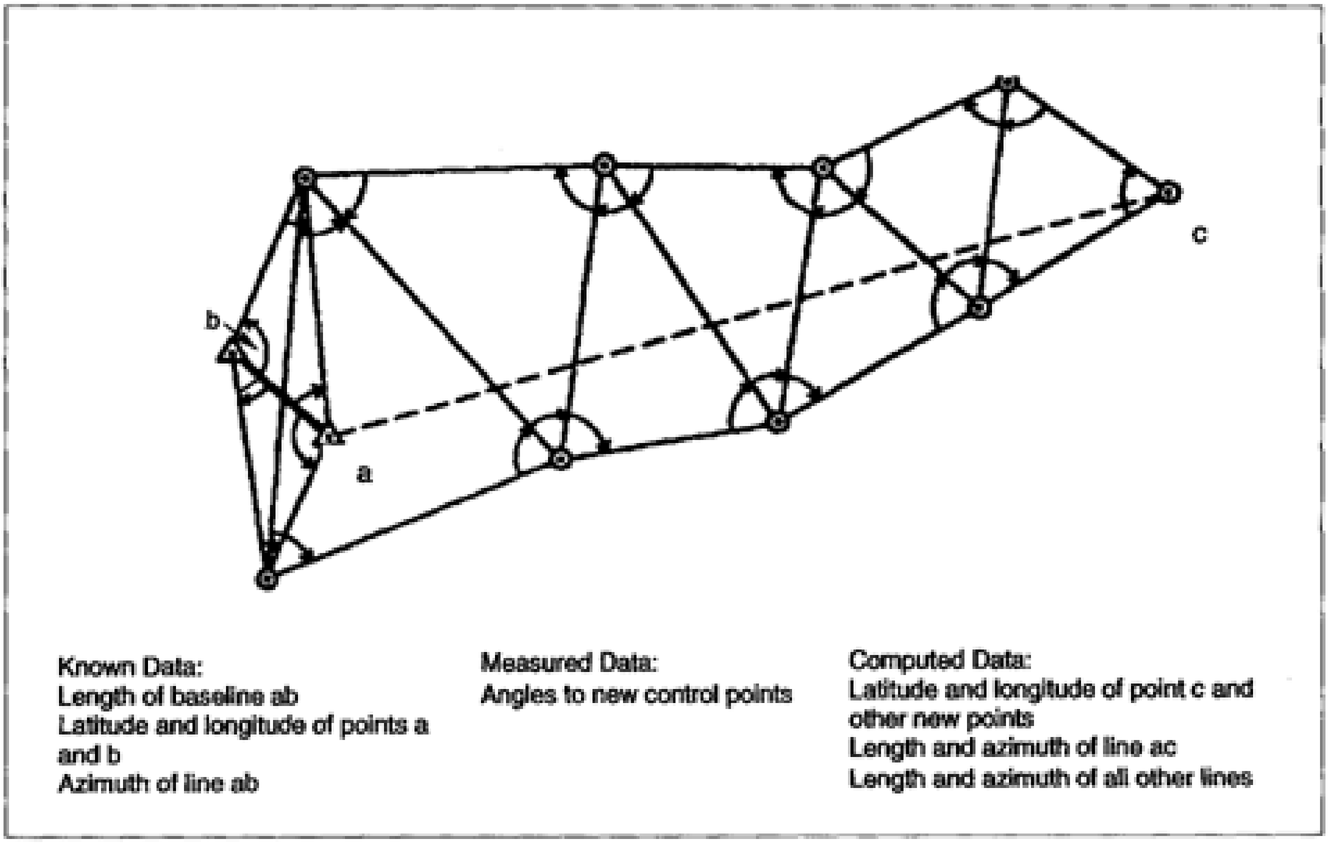


Figure 1-9. Simple Triangulation Net - Example 1

(3) Triangulation may be extended to cover larger areas by connecting and extending a series of arcs that form a network. The network, which forms the triangulation system, is mathematically adjusted to reduce all errors to a minimum.

c. Trilateration. Recently, electronic and light wave distance-measuring equipment has been applied to geodetic surveys in a technique known as trilateration. Trilateration is the measurement of the sides of a triangle. Each side of the triangular net is measured repeatedly to ensure precision. The angles of the triangle and geodetic positions are computed, as in triangulation (Figure 1-10, page 1-14). These systems also permit surveying over large water barriers, allowing the connection of islands and continents. Traditional electronic systems only measure distance.

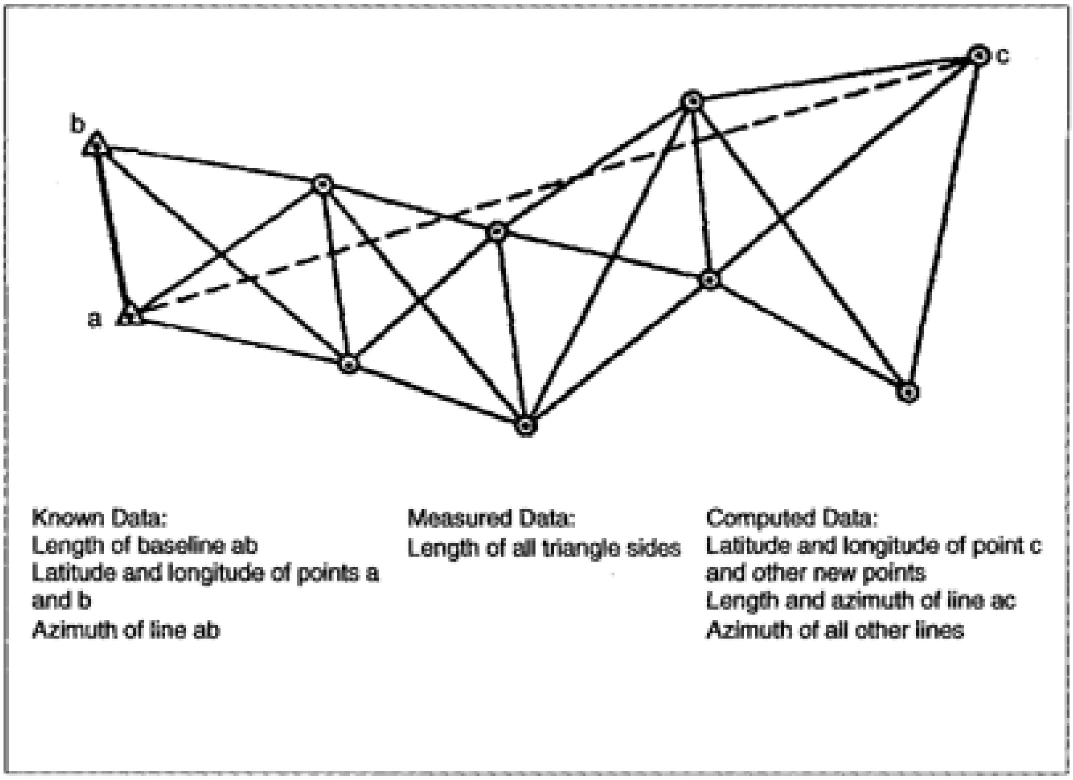


Figure 1-10. Simple Triangulation Net - Example 2

d. Celestial Techniques. Celestial triangulation methods permit the extension of long arcs across oceans and inaccessible terrain. All of these methods--the star occultation, the moon-position camera, and the observation and tracking of artificial earth satellites possess one common characteristic--observed data is not affected by the direction of the vertical at the observation point. The solar eclipse method has been discontinued due to the infrequency of total solar eclipses and the inconvenience and frequent inaccessibility of suitable observation sites located along the narrow path of the moon's shadow.

(1) The star occultation method provides information similar to that obtained through a solar eclipse from which the distance between observation points can be computed. This method is used primarily for connecting isolated islands in the ocean (Figure 1-11). Occultation may be defined as the passing of a larger celestial body in front of a star. The moon, which is used as the occulting body for geodesy, revolves around the earth about once a month and in its orbit blocks out certain stars. Using telescopes fitted with electronic observing and recording devices and a recording chronometer for precise time determination, it is possible to record the exact instant of the occultation of a star. For determining a position of a point on the earth, at least two pairs of observations are needed. A pair consists of occultation measurements at one known station and at another station whose position should be determined on the same datum as the known station. The same star must be observed on about the same point of the moon's limb at both stations. An approximate geographic position of the point, whose geodetic position has to be determined, is known for the prediction of the occultation. The observations yield a correction to the assumed position of the unknown point.

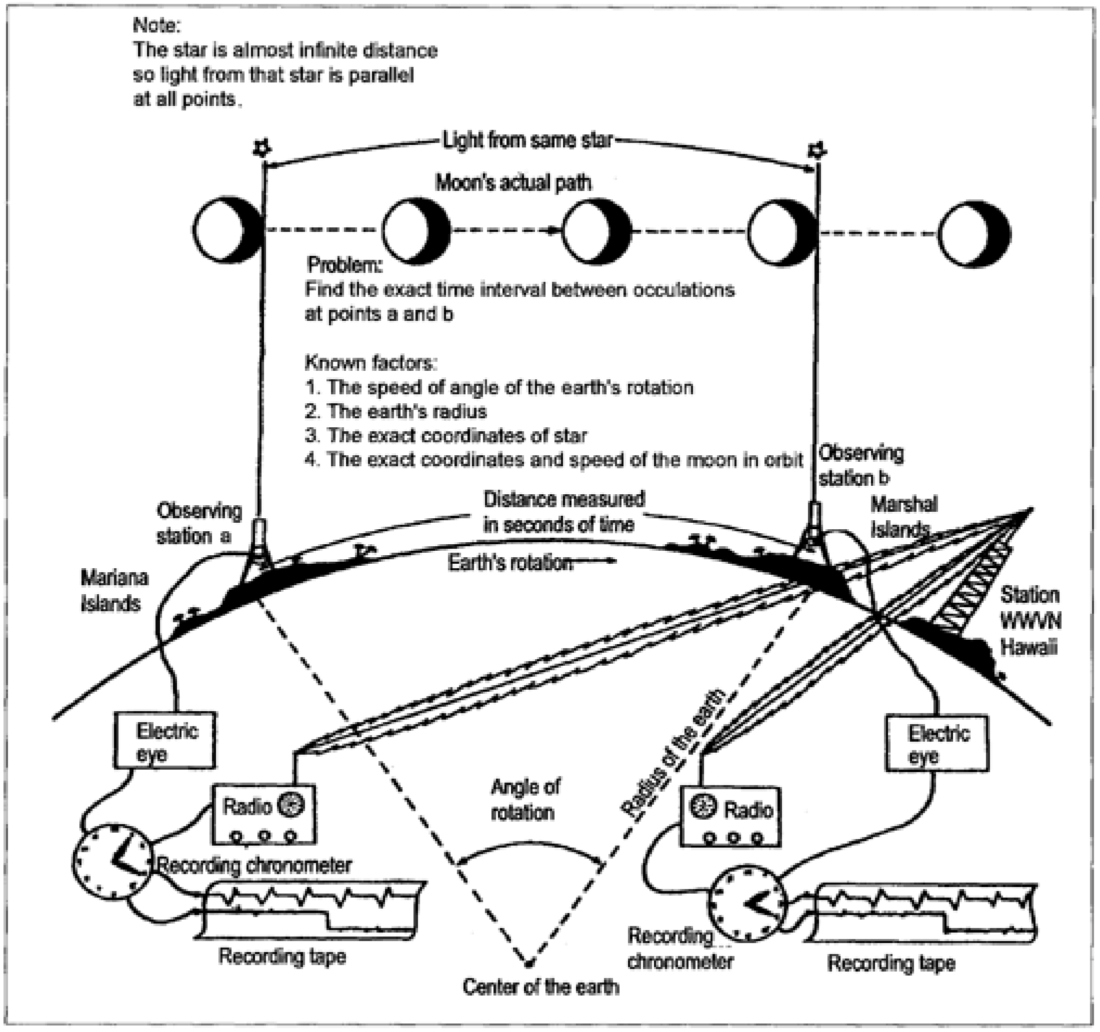


Figure 1-11. Arc Distance Determination by Star Occultation

(2) The moon-position camera method involves photographing the moon against a background of stars. The camera eliminates the problem of the great difference in the motion of the moon and the stars. Important geodetic data can be obtained from plates exposed at well-distributed stations. During the first international geophysical year (1 July 1957-31 December 1958), observations were made at several stations distributed throughout the world. The observed data is still being processed.

(3) The observation and tracking of artificial earth satellites shows the most promise for obtaining valuable geodetic data in the future. After the satellite is in orbit around the earth, it is possible to track it optically using specially constructed ballistic cameras or electronically by using the sequential collation of ranges (SECOR) system.

(a) For optical tracking, the satellite is photographed from known and unknown observation points. The satellite, which is designed specifically for optical observation, contains a light source which can be activated electronically for photographic purposes. Large satellites with no independent source of illumination can be photographed at dusk or dawn when reflected sunlight illuminates the vehicle. The smaller angular distances between the images of the satellite and the star background are measured, and these measurements are used to determine the direction of the satellite from the observation point.

(b) The SECOR system employs a series of transmitting and receiving stations on the ground which contact a specially equipped satellite as it passes within range. Command signals are transmitted from the ground to the satellite. When it receives the command signal, the satellite transmits a ranging signal back to the ground station. The distance to the satellite can thus be assured by comparing the outgoing signal with the incoming signals at the ground station. Various corrections must be made for atmospheric effects. Four practice stations are used on the ground. Three of these stations are located on a known datum, and the fourth at an unknown position (Figure 1-12). A series of simultaneous measurements are made from each ground station during each pass of the satellite. These measurements, when converted, establish the position of the satellite and the position of the unknown station. The unknown station can be located from 100 to 1,500 miles from the known stations. Although the SECOR system is still in the improvement stage, results of reduced data already indicate that it will be a first-order geodetic tool.

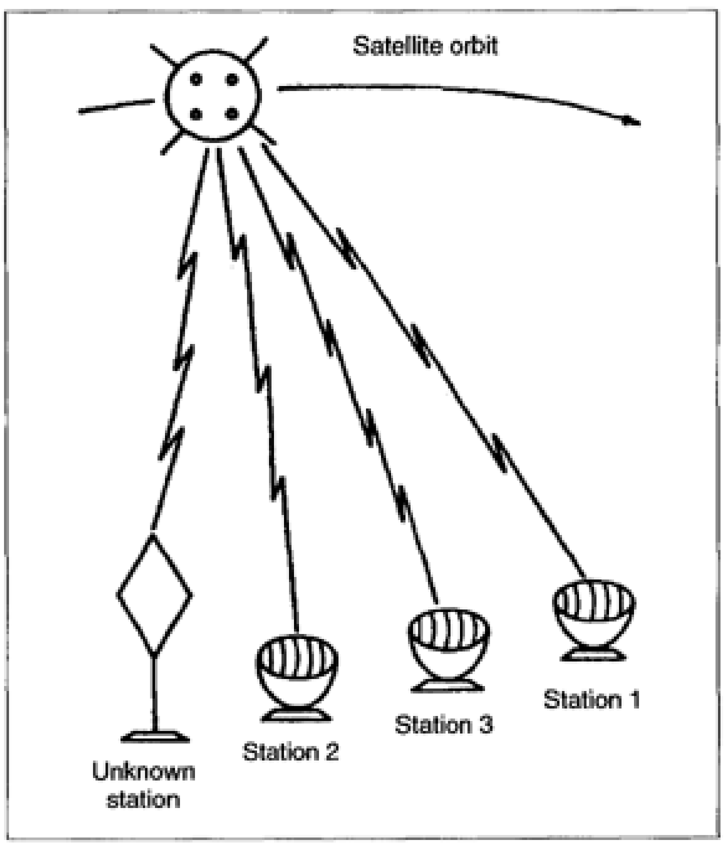


Figure 1-12. SECOR Ranging of Satellite

(c) Because of the high altitudes of satellites, it is possible to triangulate across the ocean and to determine relative positions between continents using the appropriate tracking method. In addition to obtaining the geodetic position, knowledge of the exact period of the satellite's orbit gives a measure of the earth's flattening. To be useful for extensive geodetic work, a satellite must be placed in a very stable orbit so that its spatial position at any moment can be accurately predicted.

1-7. Vertical Control by Gravimetric Technique. The term applied to the operation of determining differences in elevation of points on the earth's surface is known as vertical control or geodetic leveling. The highly accurate leveling instrument used is aligned perpendicular to the geoid, and the line of levels follow the geoid's curvature. The purpose of leveling is to determine the elevation above MSL of any number of vertical control points from which the elevation of any other point in a survey can be computed.

a. The determination of the acceleration of gravity over the earth's surface provides a method of determining the shape of the earth. In using the earth's gravity field to determine the earth's shape, the acceleration of gravity is measured at or near the earth's surface.

b. The theory behind gravitation acceleration depends directly on Newton's law of universal gravitation. The story of Newton and the falling apple has led toward the law that governs the universe. Newton reasoned that the force that pulled the apple down was the same force that holds the moon in its orbit around the earth. He also reasoned that the force diminished as the distance from the earth increased. From this reasoning he arrived at his inverse square law which states that the force is inversely proportional to the square of the distance from the center of the earth. When the distance is doubled, the force of gravity decreases by the square of two or four. Newton also deduced that the intensity of the force of gravity depended on the mass of an object. The force of mutual attraction exerted by two bodies was directly proportional to the masses of the pair--the larger the mass, the stronger the attraction. From these deductions Newton arrived at the following law of universal gravitation: every particle of matter in the universe attracts every other particle with a force directly proportional to the product of the masses and inversely proportional to the square of the distance between them. In mathematical form this law can be expressed in the following equation:

$$F = G \frac{m1m2}{r2}$$

F is the attractive or gravitational force between two bodies of masses (m1m2), with m1 and m2 expressing grams; r representing the distance between them; and G showing the constant of gravitation. To put Newton's equation to use, it was necessary for scientists to find the exact value of G from laboratory experiments. Newton presented only a rough estimate of its value from astronomical observations.

c. The universal gravitational constant (or G) is so named because its value is the same everywhere in the universe. An English scientist named Henry Cavendish completed the first accurate laboratory measurement of G in 1798. His device consisted of a horizontal rod with a small sphere of equal mass at each end. The dumbbell-shaped rod was suspended at the center by a sensitive twistable fiber. Large lead spheres of known mass were placed at the ends of the rod, near the small spheres, in such a way that the gravitational pull between the small and large masses caused the rod to rotate horizontally. Knowing the weights, the distances of the masses, and the torsion of the twisted fiber, Cavendish computed the value of G to be 6.754×10^{-8} or, written out, 0.00000006754 centimeter-gram-second (cgs) units. This constant is the force in dynes that is exerted between two masses of 1 gram each with centers 1 centimeter apart. Cavendish's value is very close to the latest accepted value of 6.673×10^{-8} *cgs units* determined in 1942 by Dr. Paul R. Heyl at the National Bureau of Standards. Dr. Heyl used a refined version of Cavendish's device.

d. Using the following formula we can determine the acceleration (a) of mass m2 due to the attraction of mass ml by dividing F, the force of attraction, by mass m2. (In Newton's second law of motion, force is the product of mass times acceleration). Any force acting on a unit mass creates acceleration, which gives a measure of the gravitational attraction at any point on the earth's surface.

$$a = \frac{F}{m2} = G \ \frac{m1}{r2}$$

e. The pull of gravity is usually expressed in terms of the acceleration of a freely falling body, expressed as the rate of increase of velocity per unit of time. A G-force of one is the force required to accelerate any freely movable body at the rate of about 32.16 feet per second per second. A G meter measures the G force in aircraft. A G is equal to approximately one thousand gals, a unit of gravity measurement named after Galileo. The gal has an acceleration of one centimeter per second per second *(1 cm/sec2)*. The acceleration of gravity over the earth's surface ranges from approximately *983 cm/sec2 or 983 gals* at the poles to *978 cm/sec2 or 978 gals* at the equator. Theoretically, if you were to jump from an airplane above the equator, you would fall faster and faster, gaining speed at the rate of 978 gals (about 32 feet) every second. At the end of two seconds, you would be falling at the rate of approximately 1,956 centimeters (about 65 feet) per second. After each succeeding second, the speed increases at a rate of 978 gals per second.

f. A smaller unit of measurement used in gravity measurements is the milligal, or one-thousandth part of a gal. It is used when dealing with variations in acceleration equal to one-millionth of one G. Modern instruments go even beyond this, measuring acceleration changes of one-billionth of a G or to one-thousandth part of a milligal.

g. Gravity can be defined as the attraction that the earth has for every particle on its surface. The force of gravity holds all objects on the earth in place and prevents them from flying off into space as the earth rotates. Gravity is what brings back to the ground a golf ball hit into the air and makes water flow downhill. It is because of gravity that all things on earth possess weight. An average person will weigh about 1 pound more at the poles where the attraction is greater than he will weigh at the equator where the attraction is weaker. The variation of the earth's gravity, with respect to latitude, is caused by two factors--the rotation of the earth and the earth's ellipsoidal shape. The attraction of gravity increases at the poles because the centrifugal acceleration due to the earth's rotation decreases to zero and the mass attraction is greater since it is closer to the center of the earth where the mass is considered concentrated. The attraction of gravity at the equator decreases since the distance to the center of the earth's mass is greater. Additionally, the component of centrifugal force opposing gravity is greater at the equator.

h. Additional variations in gravity can be caused by elevation, surface density, and topography. Oceans, plains, and mountains distort the surface, and the density (the amount of mass in a given volume) and elevation of these topographic features create regional differences in gravitational attraction. Furthermore, buried masses lying below the earth's crust exert additional gravitational pull on surface objects.

i. The variations in gravity, inferred at first from surveying discrepancies and later determined by actual observations, led to the concept of isostasy. The term isostasy, meaning equal pressure, was first proposed by C. E. Dutton in a paper written in 1889.

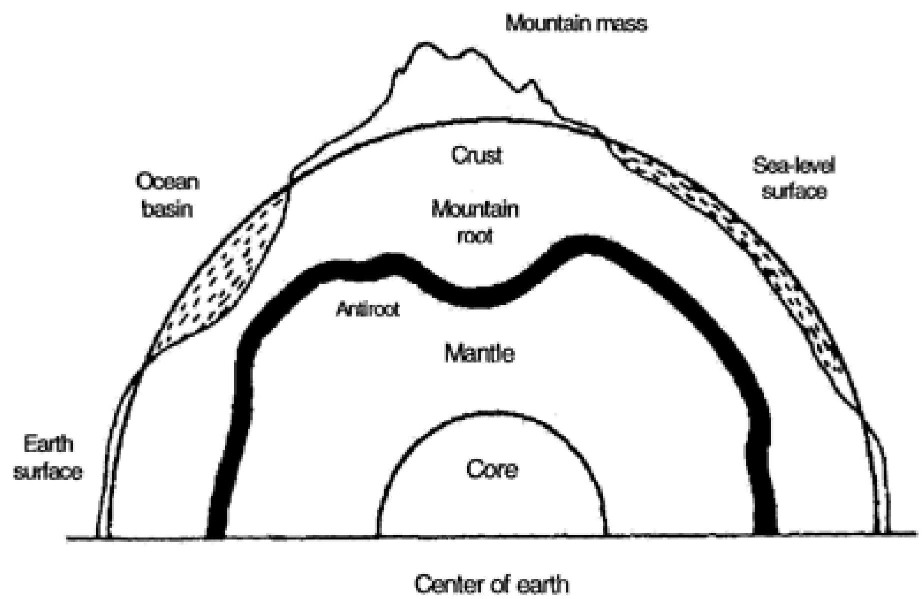
(1) Isostasy is a condition of approximate equilibrium in the outer part of the earth, such that the gravitational effect of masses extending above the surface of the geoid in continental areas nearly counterbalanced by a deficiency of density in the material beneath those masses, while the effect of deficiency of density in ocean waters is counterbalanced by an excess of density in the material under the oceans.

(2) The basic principle of isostasy is that the masses of prismatic columns of the outer part of the earth, extending to some constant depth below the surface of the geoid, are proportional to the areas of their sea-level sections regardless of their surface elevations. The depth below sea level to which these hypothetical columns extend is known as the depth of isostatic compensation and is somewhere between 96.56 and 112.65 kilometers. The area of the sea-level section of a unit hypothetical column for which isostatic compensation is ordinarily complete has not been determined; it may be uniform for all parts of the earth, or it may vary with the character of the relief in the same continental region.

(3) While isostasy is generally accepted as a proven principle, there are several theories as to the relative distribution of the matter producing this condition of equilibrium. The two principal theories are those of Pratt and Airy. The fundamental difference between the two theories is that Pratt postulated uniform depth with varying density, while Airy postulated uniform density with varying depth.

- The Pratt theory, announced by J. H. Pratt in 1855, assumed that the continents and islands project above the average elevation of the solid surface of the earth because of the material of less density beneath them--the higher the surface, the less the density below it. Under the Pratt theory, complete equilibrium exists at some uniform depth below sea level--the same depth for ocean areas as for landmasses.

- The Airy theory, announced by G. B. Airy in 1855, proposed that continents and islands are resting hydrostatically on highly plastic or liquid material, with roots or projections penetrating the inner material of the earth just as icebergs extend downward into the water (Figure 1-13). The greater the elevation of mountain masses above the earth, the deeper the penetration of the roots. It has been called the Roots of Mountain Theory, and has the support of some geologists.



The crustal thickness increases under a mountain mass and decreases under an ocean basin. These *roots* and antiroots bring about isostatic equilibrium.

Figure 13. Airy's Theory of Compensation

j. The theoretical value of gravity at a point on the ellipsoid depends on the size and shape of the ellipsoid and the observed value of gravity at the equator (978 gals). It also varies with the latitude of the observation point, assuming that the earth is a surface without mountains and oceans, having no variations in rock densities or in the thickness of the crust. The theoretical value of gravity represents the force of the earth's attraction due to gravitation, minus the centrifugal force due to the rotation of the earth.

k. Since the earth is not an ellipsoid and there are variations in both the crust material and the terrain, the observed gravity of the geoid varies from point to point. Gravity observations must be made so that the distance between the geoid and the ellipsoid can be computed. This undulation of the geoid, or distance, must be observed point by point; it cannot be computed directly from other data.

l. The difference between observed gravity and theoretical gravity at the same point is called a gravity anomaly. Gravity anomalies can be either positive or negative. They are positive in areas of mass surplus, where the observed gravity is greater than the theoretical value. In areas of mass deficiency, where the observed gravity is smaller than the theoretical value, the anomaly is negative.

m. The physical basis of gravimetric studies lies in the fact that the gravity anomalies, the deflection of the vertical, and the undulations all result from disturbing masses. The gravity anomalies can be measured and, from these measurements, the important geodetic quantities, undulations of the geoid, and deflections of the vertical can be computed (Figure 1-14).

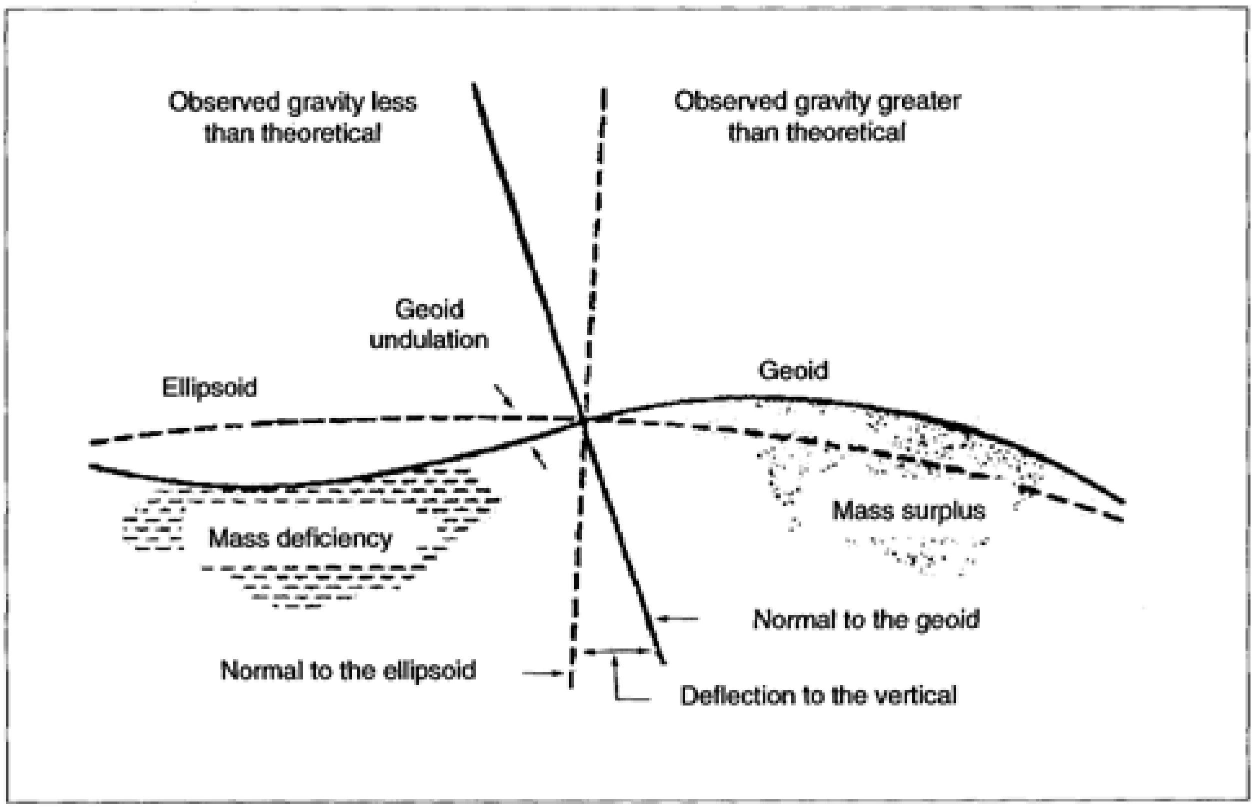


Figure 1-14. Effects of Mass Anomalies on the Geoid

(1) The method for determining undulation of the geoid was developed in 1854 by a British scientist named Sir George G. Stokes. In more recent years, a Dutch scientist named Vening Meinesz developed the formula for computing the gravimetric deflection of the vertical (Figure 1-15, page 1-22).

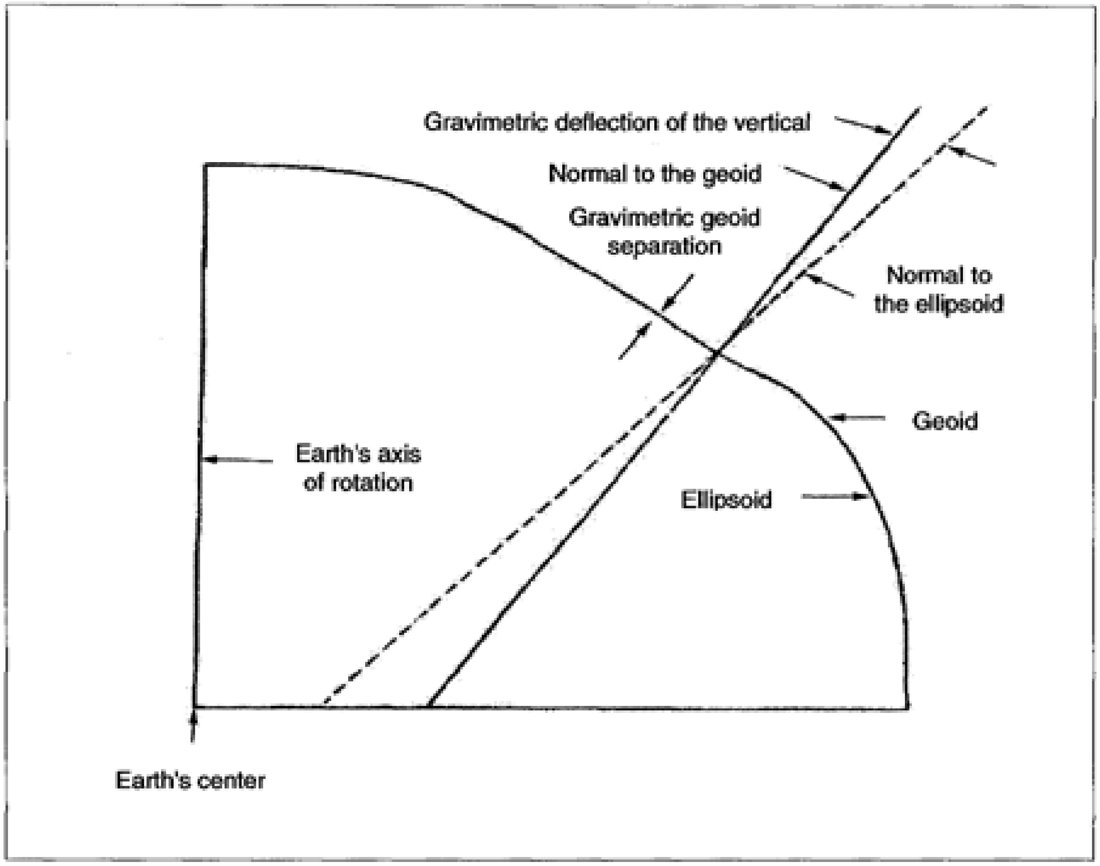


Figure 1-15. Products of the Gravimetric Method

(2) Computing the undulations of the geoid and deflections of the vertical requires extensive gravity observations. The areas immediately surrounding the computing point require a dense coverage of gravity observations. Detailed data must be obtained from distances out to 500 miles. A less dense network is required for the rest of the earth. The deflections and undulations computed from information obtained from these extensive observations are considered absolute values when referred to an earth-centered, referenced ellipsoid. In other words, the axis of rotation for the ellipsoid coincides with the rotation axis of the earth, and the geometrical center of the ellipsoid coincides with the earth's center of gravity (Figure 1-15).

n. The collection of gravity observations requires measuring the gravity at or near the earth's surface. The ultimate goal is to obtain a good distribution of gravity observations over the entire surface of the earth. Since 70 percent of the earth's surface is water, this is not an easy task. Consequently, geodesists are forced to use Stokes's theorem method to determine geoid separation in areas where gravity observations have not been made.

o. The absolute acceleration of gravity is usually measured with a pendulum apparatus. The swinging period of a pendulum is accurately timed, and from this time interval the value of gravity is computed. Pendulum measurements require lengthy observations, and the average period of swing must be determined from several thousand actual swings of the pendulum. This type of measurement requires several hours of observation, and the equipment involved is complex and cumbersome. Consequently, absolute gravity is usually measured at a limited number of reference stations called gravity base stations.

p. Gravity measuring instruments, which use base stations for referencing, are generally called *gravimeters*. Since these instruments do not measure absolute gravity, the results are known as relative measurements. With a gravimeter, gravity at different places is compared with the value of gravity at a pendulum base station. In recent years, mostly geophysical-prospecting companies have developed several different types of gravimeters. Some of the instruments are compact and light and permit rapid movement from place to place. Relative gravity measurements are usually sufficient for geophysical prospecting, but these measurements must be properly connected to a gravity base station before the results can also be used for geodetic purposes.

(1) Gravimeters are so sensitive that they respond to and register the gravitational attraction of the sun and the moon. Since the waters on earth have no rigidity, they are raised and lowered by forces of attraction known as tides. These same tidal forces exert a pull on the land surface. Evaluations on tidal effects shows that the moon's attraction is more than twice that of the sun. Although a point on the surface of the earth does not rise or fall as much as would a ship on the surface of the ocean due to tidal force, the slight movement of a point on the land surface causes small variations of gravity. The amount of variation depends upon the latitude, the time of the month, and the year. These variations are a source of error and must be computed and eliminated when obtaining readings with the gravimeter. This computation is known as the *earth tide correction*.

(2) Gravity observations over ocean areas can be made from submarines. Such measurements still require a considerable length of time, and special consideration must be given to account for the motion of the measuring vessel. Several different types of underwater gravimeters have been built by oil-prospecting companies to perform gravity surveys in shallow coastal areas.

(3) Gravity surveys have been made on many areas of the earth; however, there are still vast areas with only a few or no gravity observations. An airborne gravimeter and other new developments in gravity measuring devices should help to extend our knowledge of the earth's gravity field.

1-8. Determining the Earth's Size and Shape. In order to establish a WGS, it is essential that the size and shape of the selected ellipsoid of revolution closely approximate the true figure of the earth. While Eratosthenes' measurement method has not been changed in principle to the present day, certain very important refinements have been made. A long arc is measured between two points on the surface of the earth. The angle, which subtends this measured arc, is determined from the difference in the latitude or the longitude of the two astronomically-observed points. The value of the ellipsoid's semimajor axis and the flattening is then computed using the appropriate mathematical formulas.

a. If the shape of the earth was exactly that of an ellipsoid of revolution, the measurement of one arc on the surface of the earth should give its dimensions. In practice, numerous arcs from different areas of the earth are used to obtain a truer ellipsoid figure. As previously pointed out, astronomic observations are referred to the geoid. This causes errors in arc measurements, which must be eliminated by using gravity anomaly corrections and gravimetric computations. By mathematically removing the effects of the deflection of the vertical at the ends of the measured arcs, a more refined and truer ellipsoid can be determined.

b. Measurements can be solved to find both the size and the shape of the earth. Reductions are often accomplished by the use of the flattening value obtained by some other method. This simplifies the arc problem to one in which only the size of the earth is sought.

(1) The shape of the earth (not its size) can be obtained from gravity anomalies. Since the theoretical gravity formula depends on the assumed flattening of the earth, gravity anomalies are used to find the gravity formula which best satisfies certain assumptions about the structure of the earth's crust. The flattening that corresponds to this corrected gravity formula can be found by working backwards.

(2) Artificial earth satellites can also provide a good means of measuring the flattening of the earth. This method will assume greater importance as tracking techniques with special geodetic satellites improve.

c. To determine the size and the shape of an ellipsoid for use in a WGS, many methods should be used based on as large a group of observations as possible. Failure to do this would result in an ellipsoid appropriate for a small area of the earth but not necessarily an adequate reference for the entire earth.

PART C - GEODETIC DATUMS

1-9. General. The geodetic positions of points lying on the earth's surface are determined with respect to a group of specific initial quantities that form a geodetic system or datum. Since the relationship between geodetic positions remains true only so long as they are on the same geodetic datum. Positions derived from different datums are not directly comparable in computations. Consequently, the desired data, such as distance and direction, will be different. The difference will depend on the errors in the initial quantities of the datums.

a. Since a datum can be defined as any numerical or geometrical quantity or set of such quantities, a datum is a starting point. In geodesy, the following two datums must be considered: a horizontal datum that forms the basis for the computations or horizontal control surveys in which curvature is considered and a vertical datum to reference heights.

(1) Horizontal geodetic datum consists of a starting point and an ellipsoid on which to compute. Use the following five conditions or parameters in determining the computations: latitude, longitude, azimuth, equatorial radius, and flattening. Computations are made from the latitude and longitude of the initial point. The azimuth of a line from the initial point gives a direction with which the computations are made. A change in any one of these five quantities will change the datum; hence, the coordinates of all points on the datum would also change.

(2) The vertical datum, to which the elevations of the points are referenced, is usually the MSL surface. However, it may be any arbitrary level surface defined by an assumed elevation for some point.

b. Geodetic-datum orientation may be established using a single astronomical point and one astronomical azimuth, an astronomical-geodetic method, or a gravimetric method.

(1) Single Astronomical Point and One Astronomical Azimuth. After a reference ellipsoid is determined or established, the other three parameters or factors must be defined. The simplest means is to select a first-order triangulation station located at the center of the surveyed area. At this point, the astronomic coordinates are observed and the astronomic azimuth from this point to another control station in the net is determined. In this method, it is assumed that the normal (perpendicular) to the ellipsoid coincides with the plumb line or the normal (perpendicular) to the geoid (Figure 1-16). Therefore, the observed astronomical coordinates and azimuth are adopted, without any corrections, as coordinates and azimuth on the ellipsoid. The two components are defined as zero--the deflection of the vertical and the undulation of the geoid.

(a) Reference Figure 1-16 and note that a datum oriented by a single astronomical point produces large geoid separations. Also, it is not earth-centered, and the rotational axis of the ellipsoid does not coincide with that of the earth. The inconvenience of such an orientation is that the positions derived from different datums are not directly comparable in any geodetic computation.

(b) This type of orientation could be used locally for positions in the net, although large errors would probably be introduced as the survey is expanded because the entire net would be shifted relative to the axis of the earth.

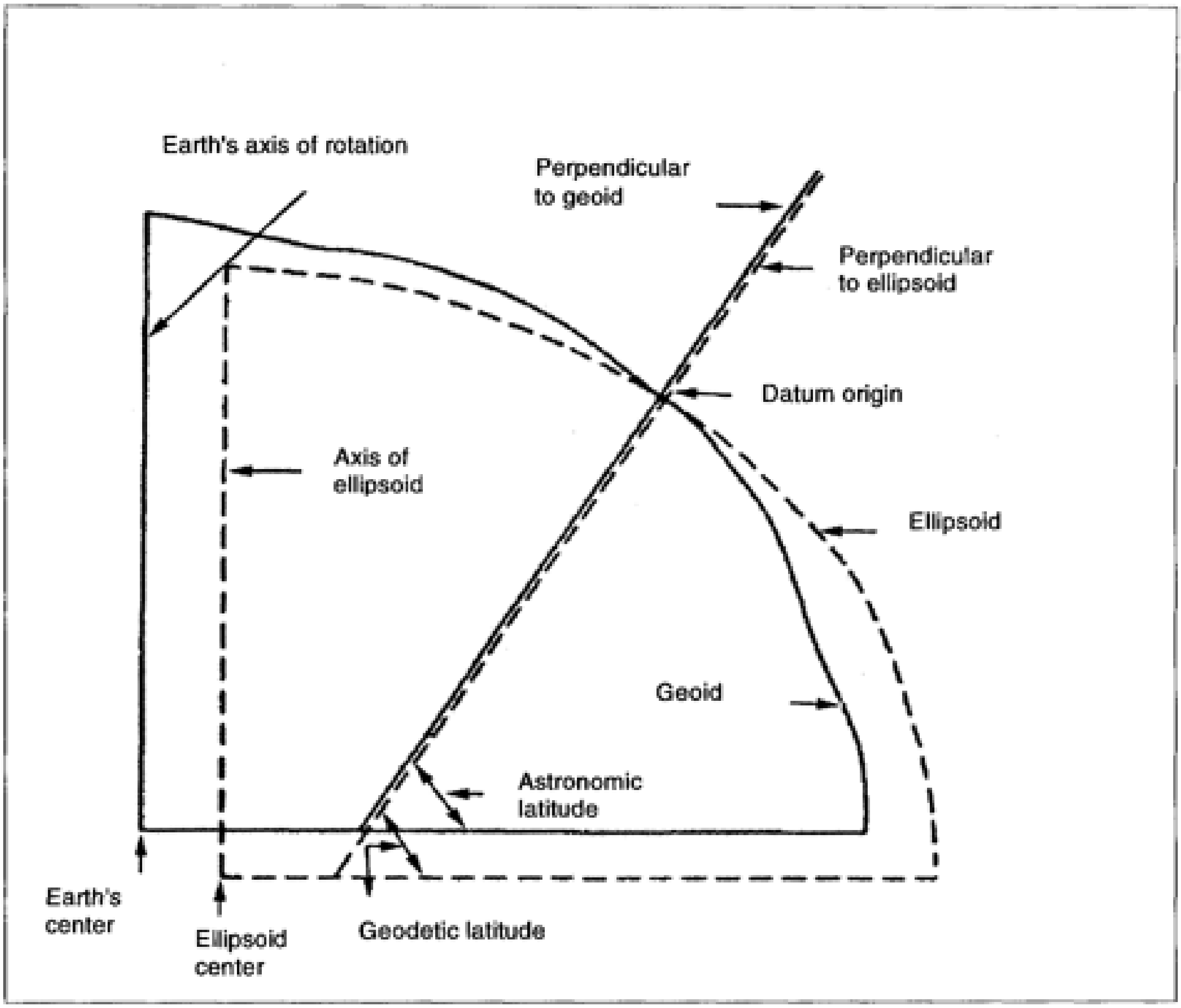


Figure 1-16. Single Astronomic Station Datum Orientation

(2) Astronomic-Geodetic Method. When comparing the geodetic and astronomic coordinates of a point, discrepancies will normally be found between the two sets of values. The deflection of the vertical angle, formed by the intersection of the ellipsoid and geoid normals, is usually expressed in terms of north-south and east-west. The north-south component, frequently called the *meridional component*, is equal to the difference between geodetic and astronomic latitude. The east-west component is proportional to the difference between the geodetic and the astronomic longitude and proportional to the difference between the geodetic and the astronomic azimuth. Consequently, the east-west component can be found in two ways. From the two ways of expressing the east-west component, we derive what is known as the Laplace equation.

(a) Since the Laplace equation is derived from certain mathematical relationships which involve the components of the deflection of the vertical along with astronomical and geodetic latitude, longitude, and azimuth, the following equations shown in Table 1-2, page 1-28, may be more enlightening:

Table 1-2. Deviation of the Laplace Equation

$\xi = \phi' - \phi$ (1)
$\eta = (\lambda' - \lambda) \cos - \phi$ (2)
$\eta = (A' - A) \cot - \phi$ (3)

Where:

ξ	=	the north-south component of the deflection of the vertical
η	=	the east-west component of the deflection of the vertical
ϕ'	=	astronomic latitude
λ'	=	astronomic longitude
A'	=	astronomic azimuth
ϕ	=	geodetic latitude
λ	=	geodetic longitude
A	=	geodetic azimuth

The Laplace equation is derived by equating (2) and (3) above:

$(A' - A) \cot \phi = (\lambda' - \lambda) \cos \phi$ (4)

or

$A' - A = (\lambda' - \lambda) \sin \phi$ (5)

(b) Refer to Table 1-2, and note that equations (1), (2), and (3) use the gravimetric method to obtain the components of the deflection of the vertical (ξ and η). If the astronomical coordinates and azimuth have been observed, the geodetic coordinates can be computed. The largest computed difference in position between astronomic and geodetic observations is approximately 1,000 feet. The average difference between the computed positions is somewhere in the neighborhood of 300 feet. These differences in position look very small. However, we are looking for a precise position; consequently, the geodesist must indicate the exact pinpoint position, not an approximation. This is especially critical at the initial point of any weapons system since any deviation will influence the final location of the impact area.

(c) The Laplace equation (5) in Table 1-2 is important because it gives the relationship between the azimuth and longitude differences without knowing the components of the deflection of the vertical. In survey networks this fundamental relationship is used as a check to determine the accuracy of the observed geodetic and astronomic data.

(d) The astro-geodetic deflections of the vertical are only relative since the deflections of the vertical are computed with respect to a specific ellipsoid. If the ellipsoid is changed, the deflections of the vertical also change (Figure 1-17). It is necessary to assume a specific orientation of the reference ellipsoid with respect to the geoid before computing the astro-geodetic deflections. This orientation is fixed by the initial values of the datum point from which the geodetic coordinates were computed. Any change in these initial values, therefore, changes the deflection of the vertical at each point. Consequently, the astro-geodetic deflections of the vertical have the same restrictions as the geodetic positions. They are related to the geodetic datum.

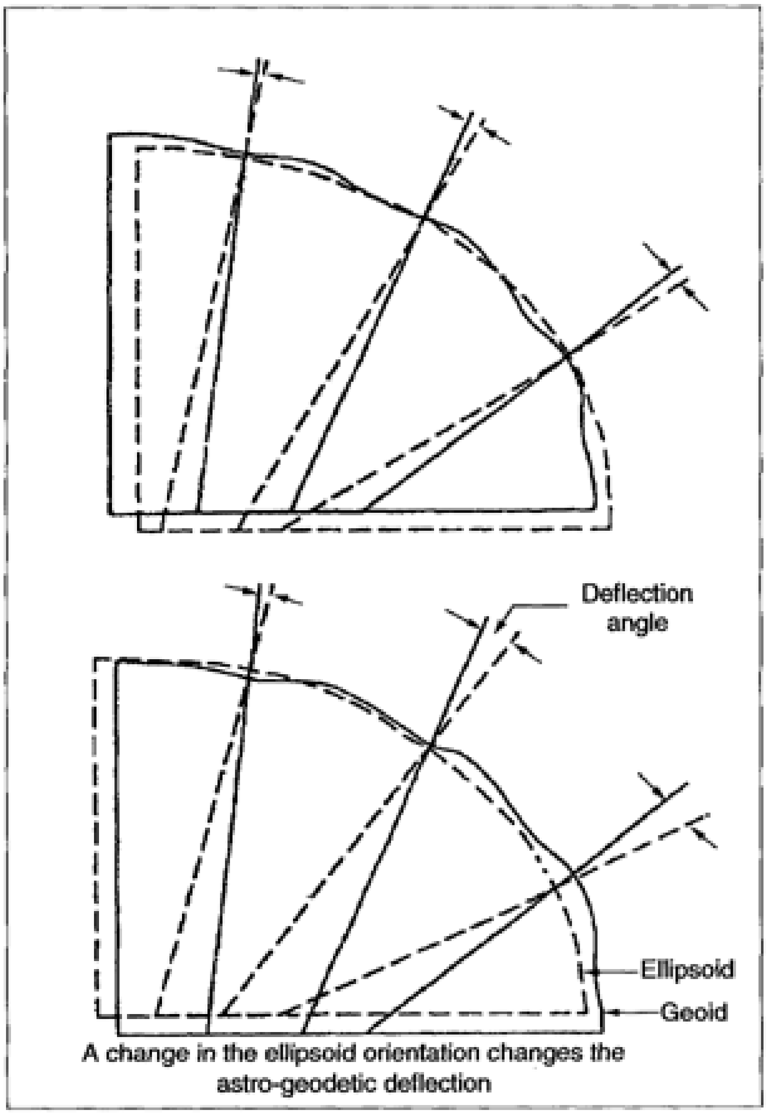


Figure 1-17. Astro-Geodetic Deflection of the Vertical

(e) It is possible to produce a profile of the geoid with respect to the chosen reference ellipsoid if a certain number of Laplace stations are given in a geodetic net. This profile can then answer the question of whether the adopted reference ellipsoid properly represents the curvature of the earth over the area being surveyed. The profile in Figure 1-18 indicates that between 50° north and 25° north, along the 98th meridian, there is nothing more than random deviations from the referenced Clarke ellipsoid. However, at 40° south the difference between the geoid and the ellipsoid approaches 200 meters.

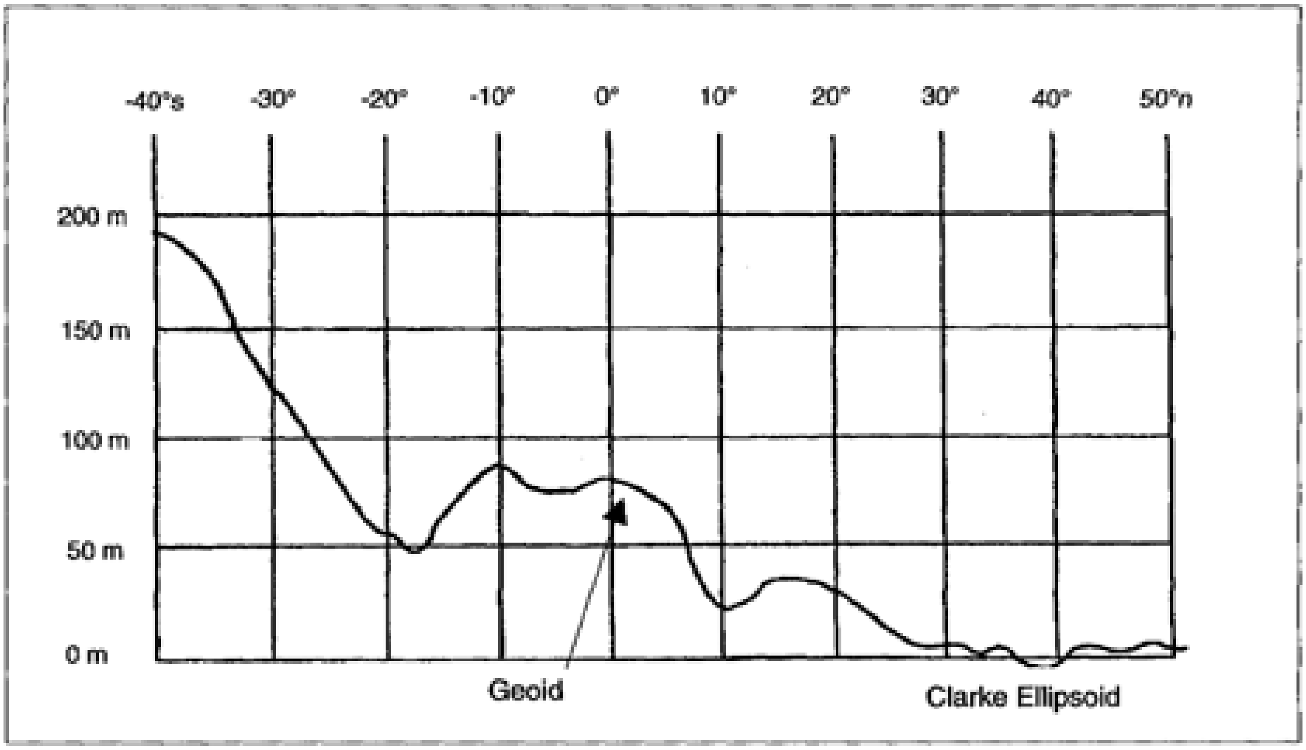


Figure 1-18. Geoid Profile of the 98th Meridian

(f) Astro-geodetic datum orientation is obtained from the deflections of the vertical at a number of selected Laplace stations (Figure 1-19). In this orientation method, the geoid and the ellipsoid are oriented so that the sum of the squares of the deflections of the vertical at the Laplace stations is made as small as possible. One of the Laplace stations in the adjustment is arbitrarily selected as the origin. Consequently, instead of having a zero deflection as with the single-point orientation, there is a deflection of the vertical at the origin point. Through a similar process, the geoid separation at the starting point can be found. This procedure reorients the ellipsoid so that it provides the best average fit at all the Laplace stations. The use of geodetic data produced by this method is still limited to relatively small areas since the deflections of the vertical still remain relative. They are not comparable with similar deflections derived from another adjustment. As in the single astronomic method, the datum is not earth-centered and the rotational axis of the ellipsoid does not coincide with the axis of the earth.

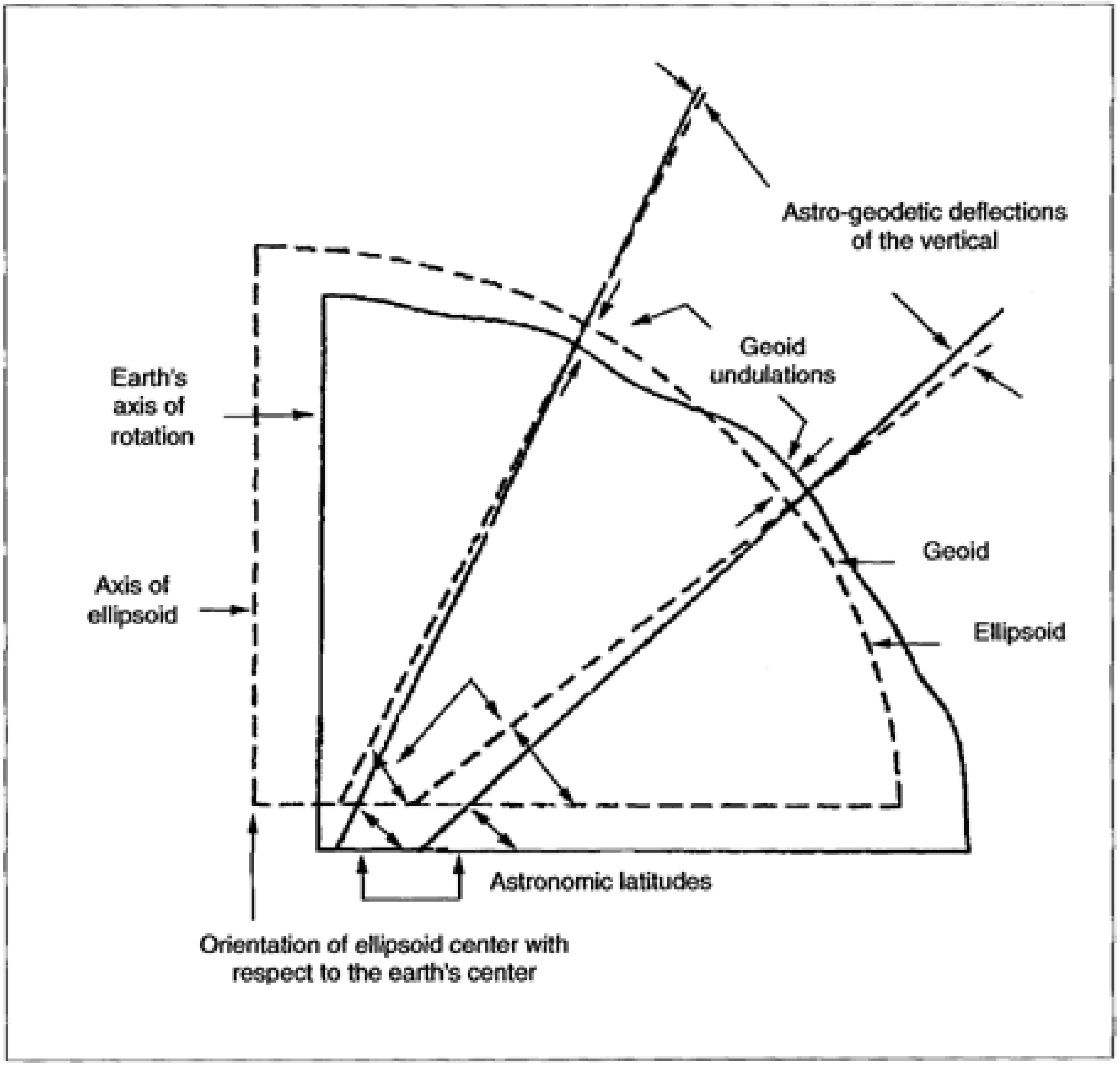


Figure 1-19. Astro-Geodetic Datum Orientation

(3) Gravimetric Method. If the relative deflections of the vertical at the initial point and the Laplace stations of a triangulation net could be replaced by absolute deflection components, the contradictions with similarly corrected adjoining surveys would be eliminated. Thus, the positions of unconnected surveys would be coordinated. The absolute deflections of the vertical and the absolute undulation of the geoid can be determined by the gravimetric method.

(a) Use the following procedures to obtain the absolute gravimetric orientation for a triangulation net:

- Compute the geoid's absolute gravimetric height and deflection of the vertical for the initial point of the geodetic system. To obtain a more reliable result, the same absolute quantities should be computed for several Laplace stations in the vicinity of the origin.

- The correction to be applied to the relative deflection of the vertical at the initial point is determined by the mean of the differences between the absolute gravimetric and relative astro-geodetic deflection components (Figure 1-20).

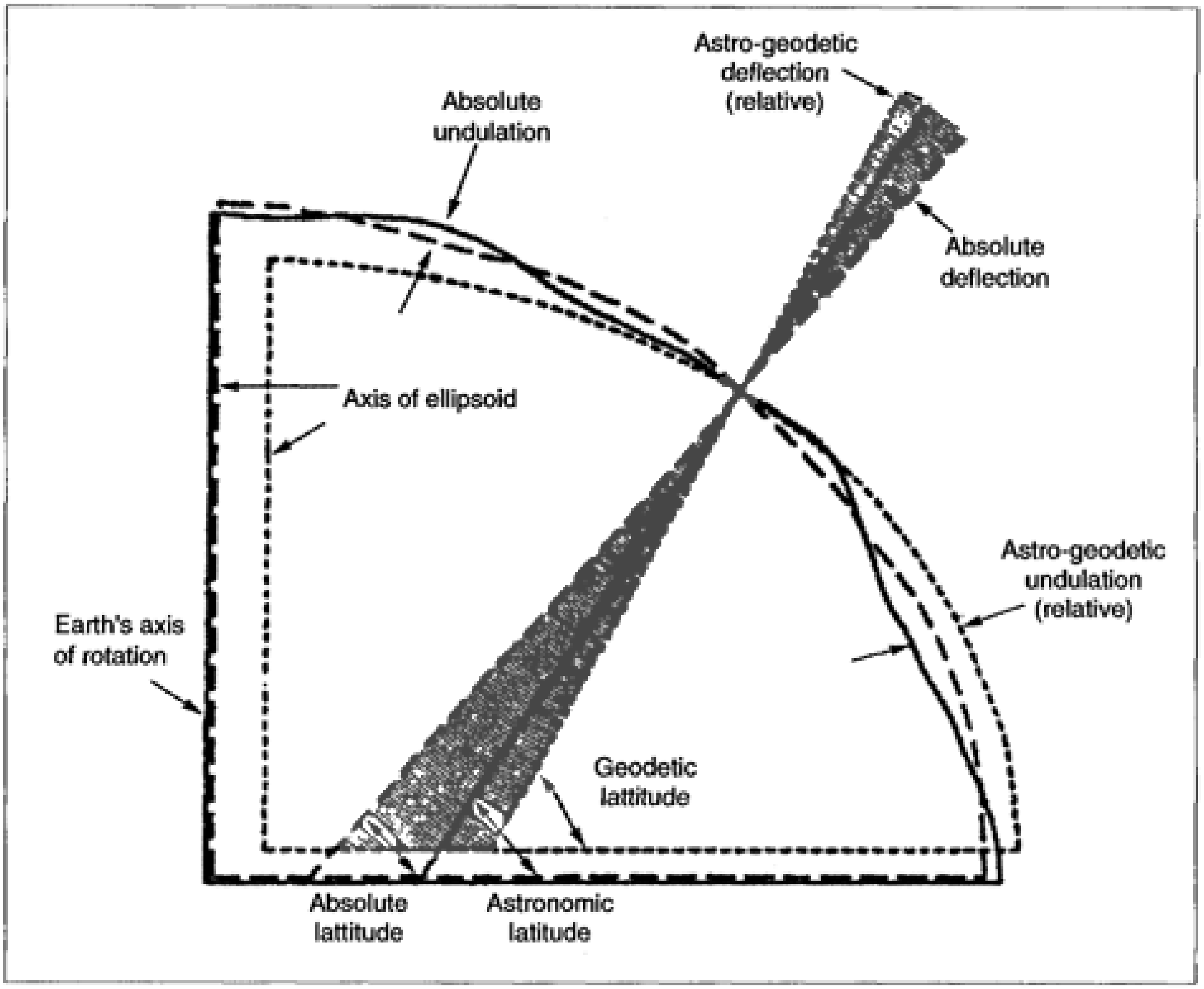


Figure 1-20. Gravimetric Datum Orientation

- Due to this correction at the origin, the relative deflections of the vertical components at the other Laplace stations are corrected. The relation between the corrections at the initial point and those of the other Laplace stations can then be computed.

- Readjusting the triangulation stations of the net, by holding fixed the corrected Laplace stations, provides the correct position of the net.

(b) Before such a gravimetric adjustment is possible, a general study of the earth's gravity field is required. A detailed study of the gravity field is required in the area of the points where the gravimetric deflection of the vertical is to be computed.

(c) The significance of a gravity-oriented datum is the absolute nature of the coordinates obtained (See Figure 1-21). If the astronomic observations and triangulation are correct, the positions derived by this method will be absolute positions with respect to the ellipsoid chosen. Assuming that we have an ellipsoid that is a good fit to the geoid, an absolute position is given by the direction of the normal to the ellipsoid whose axis coincides with the earth's axis of rotation.

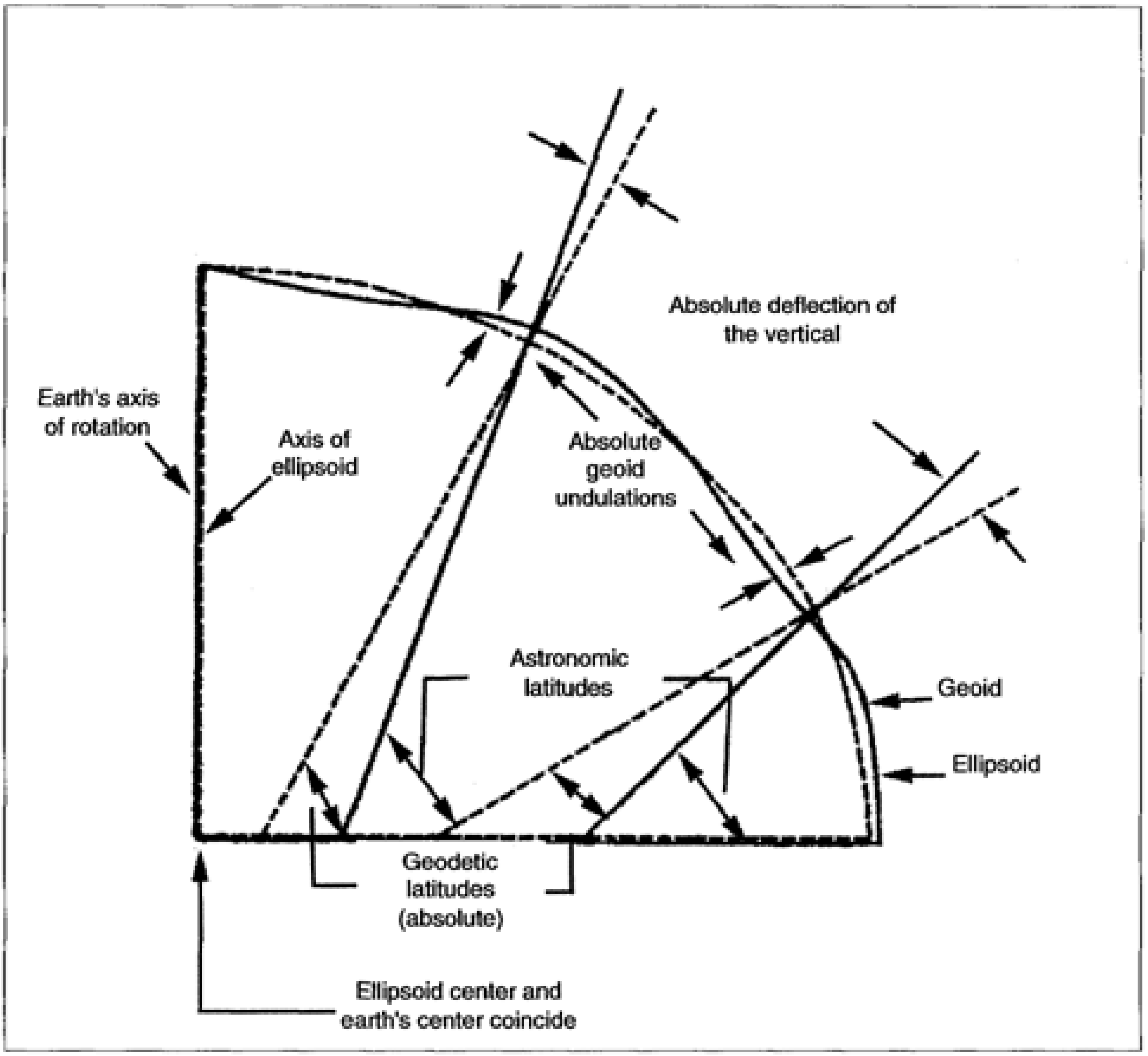


Figure 1-21. Astro-Gravimetric Datum Orientation

(d) Because the gravimetrically-oriented ellipsoid is earth-centered and aligned with the earth's axis of rotation, various gravimetrically-adjusted datums will all be earth-centered and have the minor axis of the ellipsoid coinciding with the rotational axis of the earth. This method of orientation will be especially significant when discussing the unification of various geodetic datums to a single worldwide system.

PART D - CONNECTING HORIZONTAL DATUMS

1-10. Discrepancies. When geographic coordinates of common points are derived from different datums, differences occur because of the different ellipsoids used, as well as the relative deflections of the vertical at their initial points. The latitude and longitude components of the absolute deflection at the initial points result in a parallel shift between two systems. This shift is caused because the minor axes of the reference ellipsoids do not coincide with the axis of the earth. Furthermore, deflection errors in azimuth cause a relative rotation between the two systems. Since using different ellipsoids causes a difference in the scale of horizontal control, a stretch occurs in the corresponding lines of the various geodetic nets. These discrepancies are generally larger for a datum oriented by a single astronomic point than they are for a datum oriented by the astro-geodetic method. However, even in the astro-geodetic nets, the deflections of the vertical are only relative and the system cannot be considered absolute. Consequently, it is impossible to determine the discrepancies between various systems unless direct observations can be made.

1-11. Unconnected Geodetic Systems. Without more information, the computation of geodetic information from one datum to an unconnected datum is impossible. Regardless of how accurate the individual datum may be for computations within themselves, there is no accurate way to perform computations for distances or azimuth between unconnected geodetic systems. Since the modern military requires geodetic computations between previously unconnected datums, the major geodetic datums of the earth must be unified. The methods used to accomplish this task are the datum transformation method, the arc measuring method, and the gravimetric method.

a. Datum Transformation Method. The datum transformation method is restricted to surveys of a limited scope. It consists of a systematic elimination of discrepancies between two overlapping triangulation networks. This is done through mathematical processes involving moving the origin, rotating and stretching networks to fit another. While this method is usually used to connect small local surveys to a national network, it can also be applied when extending control for detailed mapping purposes. However, the datum transformation method can only be used where control exists for common points in the different systems.

b. Arc Measuring Method. The arc measuring method can establish survey ties between unconnected systems. Arcs across relatively narrow waters and land areas inconvenient for ground surveys can sometimes be obtained by electronic distance measurements. Thus, high-precision super-range navigation (HIRAN) radar trilateration provides a method for measuring distances over areas where ground stations can be established within 400 or 500 miles of each other. This offers an operational method of connecting separate geodetic datums. The HIRAN trilateration has become a standard tool, within its capacity and range, in the coordination of geodetic systems. As mentioned earlier, celestial triangulation methods also permit the establishment of arc distances over oceans and inaccessible terrain.

c. Gravimetric Method. Components of the gravimetric method were previously discussed in this chapter. By using a single reference ellipsoid and determining the absolute deflections of the vertical and geoid separations for each initial point of all datums, the datums can be joined more accurately to a single unified absolute system.

PART E - GEODETIC SYSTEMS

1-12. General. The amount of work and time involved in establishing a single unified absolute system of geodetic datums is tremendous. For military purposes, we must know the distance and directions between widely separated points. Therefore, we must undertake this task. In this lesson, we discussed how geodetic datums are established and some methods by which the various datums can be connected. We also discussed the tools and techniques of geodesy. Now let's examine some of the problems.

1-13. Major Datums Before World War II. Every technically advanced nation has developed its own national geodetic system or systems, depending on its technical, industrial, or military requirements. The systems were developed by the expansion and unification of smaller local systems or by new nationwide surveys to replace the outdated local ones. Neighboring countries normally did not use the same geodetic datum, since the cost of establishing a separate datum was small, and the nation's military interest was against the use of a common datum. International surveys on the same datum were restricted to the measurements of long arcs across international boundaries in order to determine the size and shape of the earth's ellipsoid. The result of this policy was that many systems developed which differed from one another remarkably, as did the map series based upon them. (An example of the many datums in Southeast Asia is illustrated in Figure 1-22.) Due to the relatively small areas covered by these national datums, military operations requiring geodetic information were restricted to fairly short distances. The development of long-range rockets and guided missiles during the war illustrated the need for more extensive geodetic information.

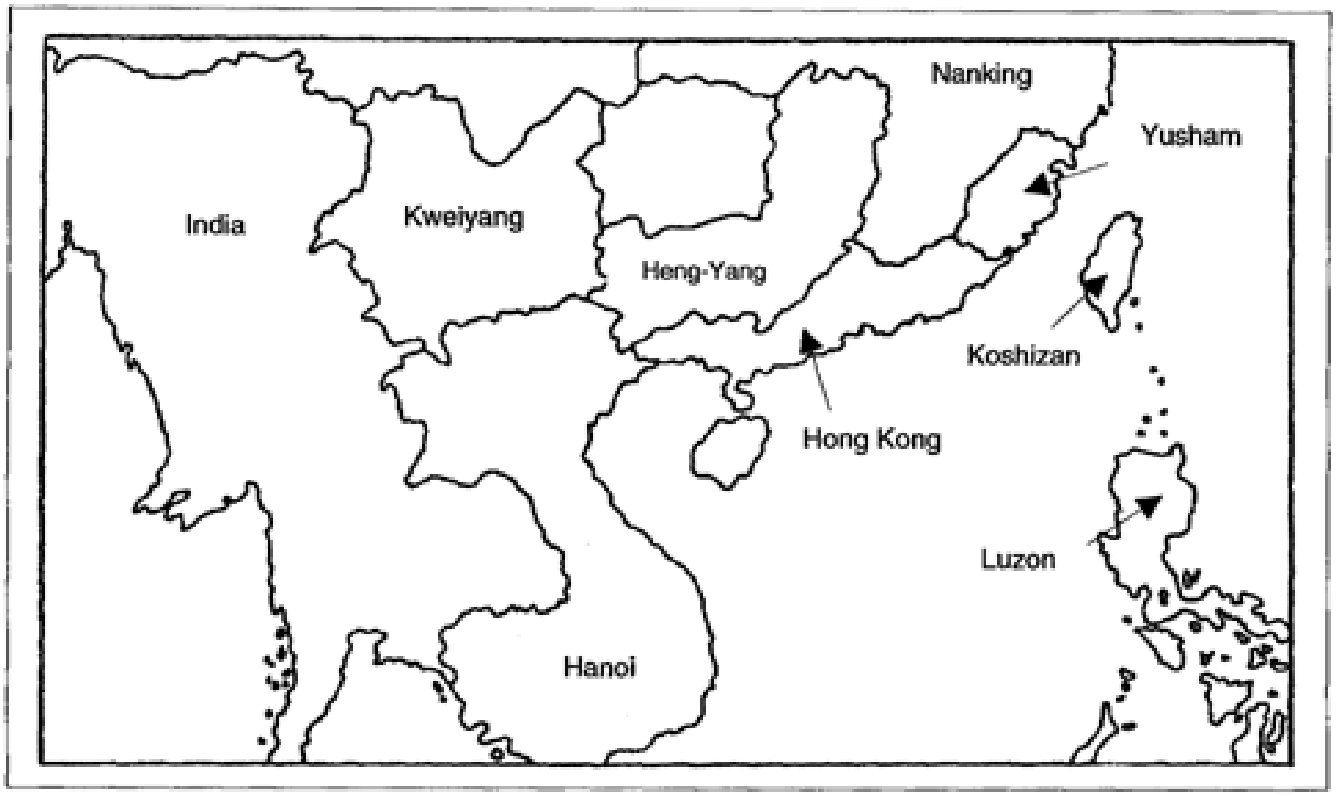


Figure 1-22. Major Datums in Southeast Asia

1-14. The Preferred Datums. After World War II, considerable advancement was achieved in the reduction of the different national datums and more toward large unified geodetic systems. This was achieved by readjusting existing surveys and making direct connections between the different systems. Large geodetic systems such as the North American, European, Russian, and Tokyo systems are generally limited to the continental boundaries or smaller areas. These large systems are generally called preferred datums. In military situations, which are limited to single continents, the preferred datums usually provide the necessary distance and direction information. Thus, for many types of air-to-surface or intermediate-range missiles, sufficient geodetic coverage is provided by the preferred datums.

1-15. The Four Principal Datums. The four principal datums into which many existing major or local geodetic systems are presently united are the North American Datum of 1927, the European Datum, the Tokyo Datum, and the Indian Datum. Numerous minor surveys have been combined into these four datums to provide geodetic systems for intermediate-range weapons. The general coverage of these systems is illustrated in Figure 1-23.

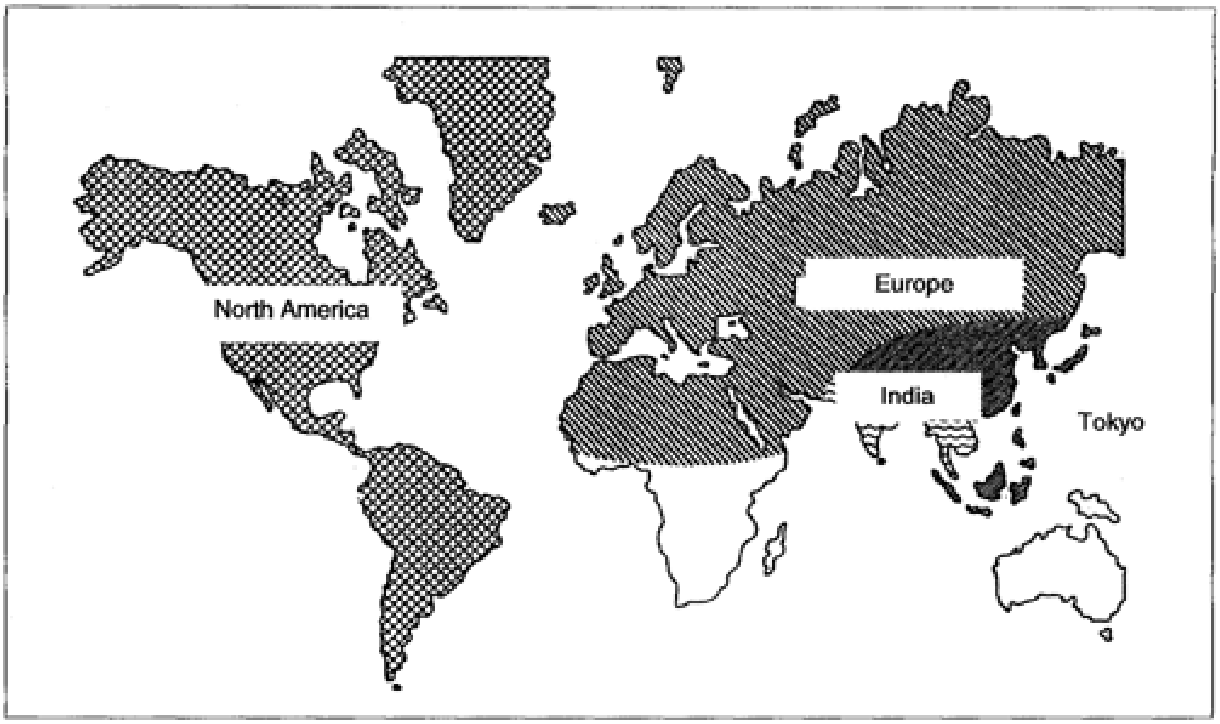


Figure 1-23. Preferred Datums

a. The North American Datum of 1927 is used in the United States. It has its origin at Meade's Ranch, Kansas and is based on Clarke's ellipsoid of 1866.

b. The European Datum's initial point is located in Potsdam, Germany. Numerous national systems have been joined into a large datum based upon the international ellipsoid. The National Imagery and Mapping Agency (NIMA) has connected the European and African triangulation chains and filled the gap of the African arc measurement from Cairo to Cape Town. Thus, all of Europe, the Republic of South Africa, and north Africa are molded into one system. Through common survey stations, it was also possible to convert data from the 1932 Russian Pulkova (Pulkovo) system to the European Datum. As a result, it includes triangulation as far east as the 84th meridian. Additional ties across the Middle East have permitted connection of the Indian and European Datums.

c. The Tokyo Datum's origin in Tokyo is defined in terms of Bessel's ellipsoid. By means of triangulation ties through Korea, the Japanese Datum is connected with the Manchurian Datum. The Tokyo Datum is oriented by means of a single astro-geodetic station. Unfortunately, Tokyo is situated on a steep geoid slope, and the single station orientation results in large systematic geoid separations as the system extends from its initial point.

d. The India Datum is accepted as the preferred datum for India and several adjacent countries in southeast Asia. It is computed on the Everest ellipsoid, with its origin at Kalianpur in central India. The Everest ellipsoid, derived in 1830, is the oldest of the ellipsoids in common use, although it is much too small. As a result of the small ellipsoid, the datum cannot be extended too far from the origin or very large geoid separations will occur. For this reason, and because the ties between local triangulation in southeast Asia are typically weak, the India datum is probably the least satisfactory of the preferred datums.

1-16. Deficiencies. The preferred datums, though an improvement over the older national datums, have serious deficiencies. The effect of these deficiencies is that the preferred systems do not provide the geodetic information required for intercontinental-range missiles. It is true that the HIRAN North Atlantic tie permits connecting the European Datum and the North American Datum. However, this does not completely solve the problem, for both the North American and European Datums are relative (Figure 1-24). While in each case the ellipsoid chosen is an adequate fit in the area of the origin, neither provides a good fit for the entire earth. Also, the process of connecting various datums by means of intervening datums or triangulation ties allows for errors and does not always provide agreement with newly observed data. The surveys joining India to the European and the Tokyo Datums also present such a problem. Since the preferred datums are relative and the few existing connections between the preferred datums provide weak ties, it is not possible to compute geodetic distances and directions between the systems, and the requirement for long-range geodetic information is still unsolved.

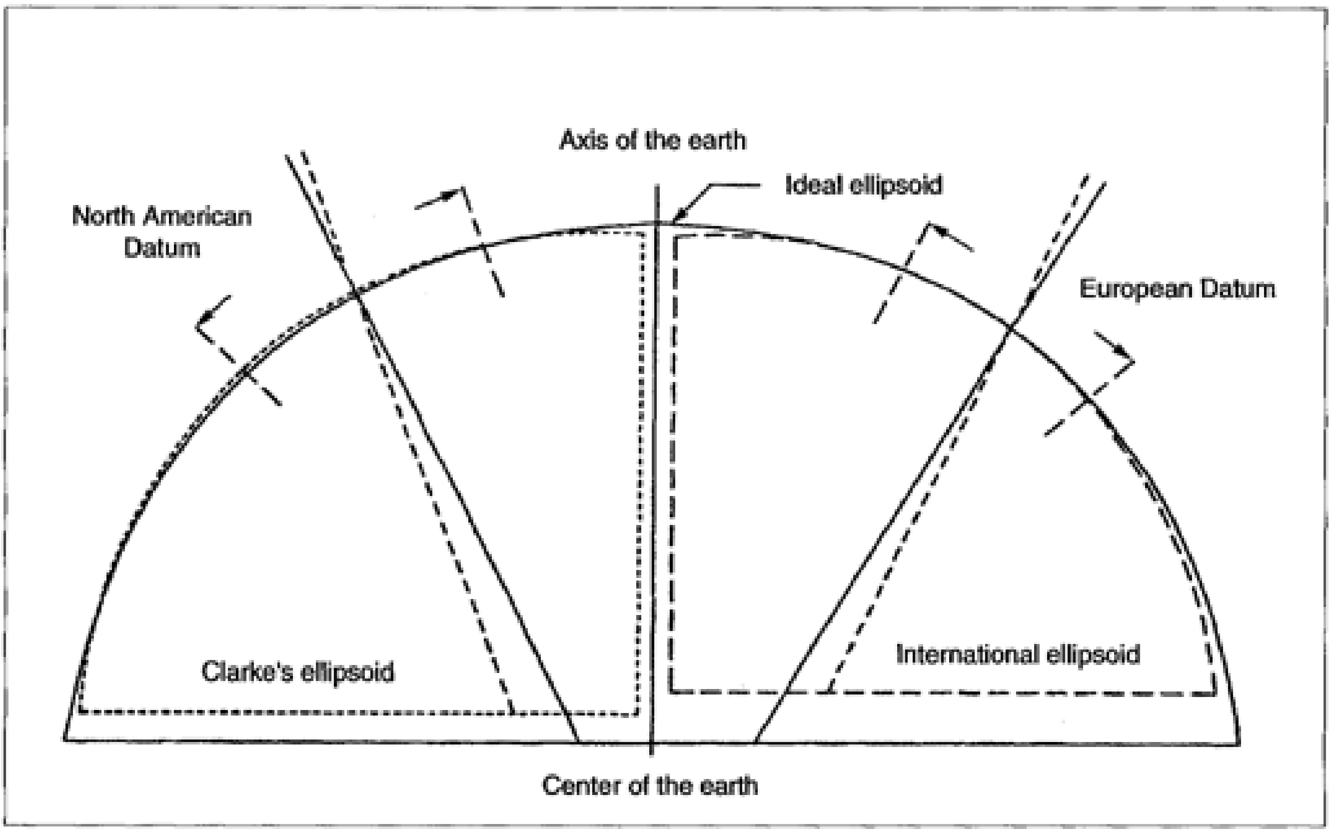


Figure 1-24. The Preferred Datums are Relative Datums

1-17. The World Geodetic System. Because local or even preferred systems fail to provide intercontinental geodetic information, a unified world system has become essential. To establish a world system, all of the available observed data must be considered. This will determine the absolute reference system which best fits the entire earth. Once such a system is established, computed geodetic coordinates will be comparable worldwide. To develop such a system for our military purposes, all branches of the military have actively participated in a program to develop a Department of Defense (DOD) WGS.

a. Because of its intercontinental-range missile capability, the United State Air Force (USAF) has played an active role in the development of a WGS. Due to the wide scope of the project, USAF participation has extended over a 20-year period in order to obtain sufficient data upon which to base a world system.

b. Active participation by various USAF agencies has permitted the gathering of extensive data needed to formulate a unified WGS. An important phase of this preparatory work is the HIRAN trilateration net which spans the North Atlantic Ocean from the eastern coast of Canada to Norway. This tie permits connection of the North American and European Datums. However, since both of these datums are based on ellipsoids which provide a good fit only in the areas of their origins, connecting the two datums through the HIRAN tie overextends the usefulness of either ellipsoid. We have seen how large, systematic geoid separations can occur when a geodetic datum is extended far from its origin. Nevertheless, ties made with HIRAN will provide effective checks when the two major datums are oriented to a common absolute ellipsoid. Another significant example of HIRAN trilateration is the West Indies loop.

c. A second major area in which preparatory work has been done in anticipation of a WGS is in the collection and analysis of gravity observations. Through an extensive program, several thousand gravity reference stations have been interconnected throughout the world. The connection of these base stations by airplane and gravimeter has permitted the reduction of numerous observations to a common usable system and, in effect, has established a world gravity system. This has permitted application to geodetic problems of data collected for oil prospecting and other geophysical purposes.

(1) The collection of extensive gravity data permitted preparation of mean gravity anomalies for 1 x 1 and 5 x 5 degree geographical sectors that are needed for further gravimetric computations. It also permitted preparing the gravimetric geoid used in the orientation of the ellipsoid for the NIMA WGS. Much of the work in the collection and the analysis of gravity data was done by contracts with civilian universities.

(2) An additional phase of gravity work includes computing gravity anomalies for places in which there are no observations. As stated previously, gravimetric computations require knowledge of gravity anomalies over the entire earth. Some regions of the earth are completely void of observation points, and the best substitute must be found. A theoretical anomaly can be found by considering the topography near the point and making certain applications of our knowledge of the earth's crust. Although the computations for each point are tedious and involve consideration of the

entire earth, the theoretical anomaly is better than a blank observation square on a gravity anomaly map. Filling in gravity maps with theoretical anomalies is not a substitute for actual observations, and data continues to be gathered from both land and ocean gravity surveys.

d. The reference ellipsoid used in a WGS must be based on all available information. This includes considering the data reduced from arc measurements in the eastern and western hemispheres as well as the connecting tie from the HIRAN North Atlantic arc. These arcs provide the basis for determining the ellipsoid's size. Recently, artificial earth satellites have provided a good method for determining the shape and flattening of the earth. The flattening of the earth can be obtained from a long-lived satellite placed in a high orbit. For example, the 1958 Vanguard satellite was tracked several thousand times and indicated that a flattening of 1/298 is probable. This value was suggested before from the arc measurements of Krassowsky. Krassowsky used the arc measurements as a datum element for the 1942 Russian datum. Earth satellites placed into orbit for the specific purpose of contributing geodetic data help to confirm or revise this value of the earth's flattening.

e. Even with the best values obtainable for the size and the shape of the ellipsoid, the reference ellipsoid must still be oriented to an absolute system. As mentioned earlier in this lesson, this can be done by gravimetrically orienting the various preferred datum origins, using the same reference ellipsoid for all datums (Figure 1-25). While we can use gravity anomalies to compute the deflection of the vertical and geoid undulation for each datum, a new problem presents itself. The gravity anomalies that we have available today are defined in terms of the international ellipsoid, even though the dimensions of the WGS ellipsoid are different. As a result, the gravity formula must be revised to correspond to the new ellipsoid and must be corrected in all the gravity anomalies. With the gravity anomalies consistent with the reference ellipsoid, the gravimetric deflectors and undulations can be computed; and the major world datums can be connected in absolute orientation on a common ellipsoid. Using this procedure, the preferred datums are joined in the WGS. All that will remain to define a WGS will be the selection of an initial point for the datum, and the selection will be quite arbitrary.

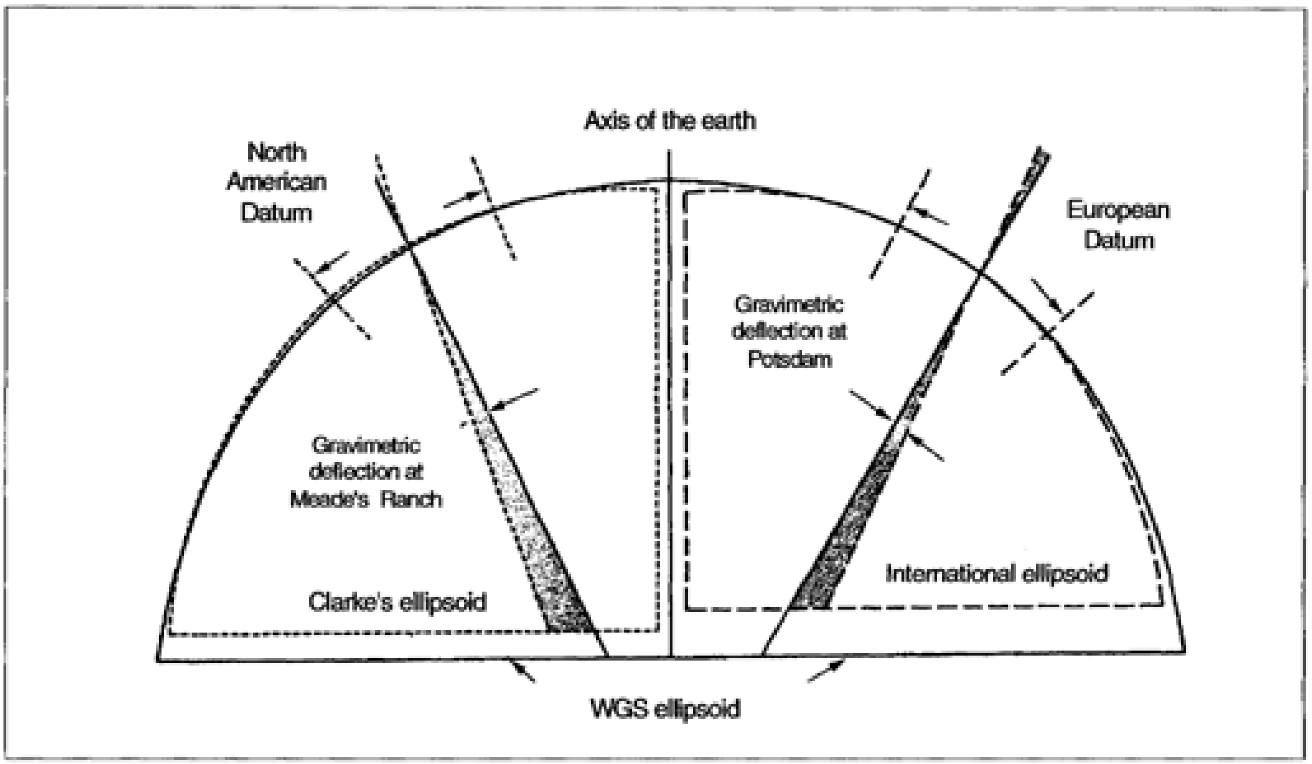


Figure 1-25. Using the Same Reference Ellipsoid for all Datums

f. A WGS will be established permitting the computation of distances and directions for use with intercontinental-range missiles. The effect of such a system will be more far-reaching than its application to intercontinental problems. Since the world system preserves the positional relationship of points within a local or preferred datum, the unified world system will also meet the requirements for geodetic information for short- or medium-range missiles. In this way, the air-to-ground missiles, the ground-based intermediate-range missiles, the ocean-based intermediate-range missiles, and the intercontinental-range missiles can all use the single WGS.

1-18. The National Imagery and Mapping Agency World System. NIMA is working to formulate a world reference system. The approach used by NIMA is different from the approach of the USAF, but it was vital that the fundamental concepts of methods were consistent.

a. The basic NIMA approach is to use the numerous arc measurements that span North and South America, Europe, and Africa. These arcs are used to determine the ellipsoid that provides the best fit for all the areas. In order to properly orient this ellipsoid with reference to the earth's axis of rotation and the center of the earth, a combined astro-geodetic deflection and gravimetric method will be used. By using numerous deflections in the western and eastern hemispheres, astro-geodetic undulations will be computed. These undulations will then be compared with absolute gravimetric undulations, and an adjustment will be made so that the difference between the astro-geodetic and gravimetric undulations is at a minimum. The end result of this method of orientation will be an absolute reference system in which all geodetic datums are unified. An important feature of this method in its dependence on gravimetric undulations based on observed gravity anomalies. The anomalies, in turn, depend on the gravity formula used. For the NIMA program, the international gravity formula and a flattening of 1/297 are used. Before these geoid undulations are used, corrections will be required for any flattening change adopted.

b. NIMA is participating in other important programs contributing to developing a WGS. These programs, including celestial triangulation and star occultations, have been used primarily to connect isolated islands to a world datum.

1-19. The Department of Defense World Datum. It is very probable that the world geodetic systems determined by NIMA and the USAF will not agree in every respect. While the fundamental principles involved in each approach are in harmony, the use of different groups of observed data is expected to cause minor differences in the final results. Since a unified world system is desired, the two systems obtained will be compared, and the major differences will be investigated by the DOD. However, the task does not end there. New information will be gathered and evaluated to permit further refinement of the world system. New geodetic methods, extended arc measurements, extended gravity surveys with new measuring methods, and data obtained from earth satellites will all contribute to the determination of a single world reference system that provides an increasingly close approximation to the true figure of the earth.

LESSON 1

PRACTICE EXERCISE

The following items will test your grasp of the material covered in this lesson. There is only one correct answer for each item. When you complete the exercise, check your answer with the answer key that follows. If you answer any item incorrectly, study again that part which contains the portion involved.

1. In geodesy, the earth is defined, for mathematical purposes, as a(n)

   A. Oblate spheroid
   B. Spherical geoid
   C. Sphere
   D. Ellipsoid of revolution

2. Points on the ellipsoid are defined in terms of ________________________________.

   A. Astronomic latitudes and longitudes
   B. Gravity determination
   C. Latitude and longitude
   D. Astronomic azimuths

3. The gravity potential on the geoid is ____.

   A. Perpendicular to the ellipsoid of revolution
   B. The same everywhere
   C. Parallel to the semiminor axis
   D. Equal to the centripetal force of the earth's rotation

4. The angle formed when the ellipsoid and geoid intersect is equal to the ____________________.

   A. Normal to the ellipsoid
   B. Deflection of the vertical
   C. Normal to the geoid
   D. Geoid separation

5. In geodetic surveying, the techniques used to determine precise points on the earth's surface are horizontal control techniques, vertical control techniques, and ____.

   A. Gravity surveys
   B. Astronomic observations
   C. The geodetic azimuth determination
   D. The determination of the deflection of the vertical

6. Laplace stations are geodetic positions on the earth's surface where ________________________.

A. Geodetic latitude, longitude, and directions are measured
B. The ellipsoid intersects the geoid
C. The normals to the geoid and ellipsoid are parallel
D. Astronomic longitude and azimuth are measured

7. Astronomic observations (latitude and longitude) give a direction with respect to the ____.

A. Normal to the ellipsoid
B. Ellipsoid
C. Spheroid
D. Geoid

8. The variation of the earth's gravity, with respect to latitude, is caused by two factors--the _____________________ and the earth's ellipsoidal shape.

A. Varying densities of the earth's masses
B. Differences in elevations on the earth's surface
C. Rotation of the earth
D. Makeup of the mantle

9. The theoretical value of gravity represents the ____.

A. Force of the earth's attraction due to gravity, minus observed gravity
B. Force of the earth's attraction due to gravity, minus the gravity anomaly
C. Force of the earth's attraction due to gravitation, minus the centrifugal force due to rotation of the earth
D. Observed gravity, minus the force of the earth's attraction due to gravity

10. In surveying, the pendulum apparatus is used to find the ________________________________.

A. Theoretical gravity at a station
B. Absolute acceleration of gravity
C. Early-tide correction factor
D. Gravitational constant

11. Which of the following is not a parameter to a horizontal geodetic datum?

A. Longitude
B. Flattening
C. Gravimetric data
D. Equatorial radius

12. Which of the following is assumed when determining the geodetic datum by a single astronomical point and one astronomical azimuth method?

A. The ellipsoid and geoid are congruent
B. The normal to the ellipsoid and the normal to the geoid are not in coincidence
C. The ellipsoid and geoid have the maximum geoidal separation
D. The perpendicular to the ellipsoid and the plumb line coincide.

13. The meridional component of the deflection of the vertical is the same as the difference between the ____.

A. Geodetic and astronomic latitude
B. Geodetic and astronomic longitude
C. Geodetic and astronomic azimuth
D. Theoretical and observed gravity

14. The Laplace equation is used to determine ________________________________.

A. The accuracy of observed geodetic and astronomic data
B. A Laplace station
C. The theoretical gravity at the point of origin
D. Gravity anomalies at the point of origin

15. What is one weakness in the preferred datums in use today?

A. Too few connections provide weak ties
B. There is only one absolute datum in use
C. All of the datums are relative
D. All of the datums are absolute and the Tokyo datum is relative

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LESSON 1

PRACTICE EXERCISE

ANSWER KEY AND FEEDBACK

Item		Correct Answer and Feedback
1.	D	Ellipsoid of revolution. The shape of the earth is more precisely represented mathematically by an… (page 1-2, para 1-2)
2.	C	Latitude and longitude Points on the ellipsoid can be defined… (page 1-4, para 1-2c)
3.	B	The same everywhere First, the gravity potential in… (page 1-6, para 1-3b)
4.	B	Deflection of the vertical This angle is called the… (page 1-6, para 1-3c)
5.	B	Astronomic observations The techniques used to do this… (page 1-8, para 1-4)
6.	D	Astronomic longitude and azimuth are measured Places at which astronomic longitude… (page 1-9, para 1-5b)
7.	D	Geoid Therefore, an astronomic observation… (page 1-10, para 1-5c)
8.	C	Rotation of the earth The variation of the earth's… (page 1-18, para 1-7g)
9.	C	Force of the earth's attraction due to gravitation, minus the centrifugal force due to rotation of the earth The theoretical value of gravity… (page 1-20, para 1-7j)
10.	B	Absolute acceleration of gravity The absolute acceleration of gravity… (page 1-22, para 1-7o)
11.	C	Gravimetric data Use the following five conditions… (page 1-24, para 1-9a[1])
12.	D	The perpendicular to the ellipsoid and the plumb line coincide In this method, it is assumed that… (page 1-25, para 1-9b[1])

13. A Geodetic and astronomic latitude
The north-south component… (page 1-26, para 1-9b[2])

14. A The accuracy of the observed geodetic and astronomic data
In survey networks this… (page 1-27, para 1-9b[2][c])

15. A Too few connections provide weak ties
Also, the process of connecting… (page 1-37, para 1-16)

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LESSON 2

PRELIMINARY SURVEY OPERATIONS

OVERVIEW

LESSON DESCRIPTION:

In this lesson, you will learn to identify the preliminary survey operations, reconnaissance, and signals that support the surveyor.

TERMINAL LEARNING OBJECTIVE:

ACTION:	You will identify the preliminary survey operations, reconnaissance, and different signals that support the surveyor.
CONDITION:	You will be given the material contained in this lesson, a number 2 pencil, and a calculator.
STANDARD:	You will correctly answer all practice questions at the end of this lesson.
REFERENCES:	The material contained in this lesson was derived from FM 3-34.331.

INTRODUCTION

When surveying large areas, the exact curvature of the sea-level surface of the earth must be taken into consideration. For this reason, many surveys conducted within the Army are called geodetic surveys. The purpose of these surveys is to establish precise horizontal control.

Horizontal control has many important uses. It is used in scientific research for studies of the figure of the earth, the earth's movement, and other geophysical research. It provides a rigid framework for all kinds of accurate charting projects; serves in locating national, state, and country boundaries; and perpetuates the surveys of private boundaries connected to it. Horizontal control enables surveys and mapping of all kinds to be fitted together into a DOD charting and mapping plan, serves to increase the accuracy of to provide checks for local and city surveys and surveys of lower order, and assists in the perpetuation of monuments established by such surveys. Horizontal control is of overall importance to the nation for all military and civilian activities concerned with long-range position control and accurate directions and distances.

Horizontal control data may be obtained using the triangulation or traverse method. This chapter discusses the field methods and required accuracies of high-order triangulation, precise instruments and equipment, baseline measurements, triangulation field records and computations, and astronomic observations and computations.

PART A - SURVEY MISSIONS

2-1. Use. Army topographic surveyors determine horizontal and vertical distances between objects, measure angles between lines, determine the direction of lines, and establish points of predetermined angular and linear measurements. After completing field measurements, surveyors use these measurements to compute a final report that is used for positioning by field artillery (FA), air defense artillery (ADA), aviation, intelligence, communications, and many other agencies.

a. FA agencies are primary users of precise positioning and orientation information in a wartime environment. Topographic-survey support, in accordance with FM 6-2, is provided to Multiple-Launch Rocket System (MLRS) units, general support units, and other nondivisional assets in the corps area. FA agencies require that topographic surveyors provide monumental survey control points (SCPs) (horizontal and vertical) and azimuth references for conventional and inertial FA survey teams. FA agencies sometimes require topographic surveyors to augment FA survey sections.

b. ADA agencies require positioning and orientation information for ADA systems. ADA and FA agencies have an agreement that FA surveyors (MOS 82C) provide direct ADA survey support.

c. The NIMA geodetic-survey division maintains Army topographic surveyors as part of their survey force structure. These surveyors are involved as team leaders, team members, and active members in the data-reduction process. Additionally, the Army surveyors are used in areas or situations where NIMA civilian personnel are not authorized (such as Saudi Arabia and Somalia). NIMA has the responsibility to provide earth-orientation data for the Navigation Satellite Timing and Ranging (NAVSTAR) Global-Positioning System (GPS). NIMA provides correlated World Geodetic System 1984 (WGS 84) airfield surveys and geographical and aeronautical database information needed to support the aviation approach requirements. NIMA also determines transformation parameters between geodetic systems. In many areas of the world, the transformation parameters are uncertain or unreliable. During times of conflict, Army topographic surveyors may be required to collect data to enable NIMA to better formulate these transformation parameters.

d. The United States Army Aeronautical Services Agency (USAASA) requires periodic airfield and navigational-aid (NAVAID) surveys and airport obstruction charts (AOCs) according to AR 95-2. These surveys are extensive field survey operations that provide aeronautical and other information to support a wide range of National Airspace System (NAS) activities. AOC surveys provide source information on position, azimuth, elevation, runways and stop ways, NAVAIDs, FAR-77, obstructions, aircraft-movement and apron areas, prominent airport buildings, selected roads and other traverse ways, cultural and natural features of landmark value, and miscellaneous and special-request items.

e. The positioning and orientation information for NAVAIDs is required to certify the airfield instrument-landing approaches. AOC surveys also establish geodetic control in the airport vicinity, consisting of permanent survey monuments accurately connected to the National Spatial Reference System (NSRS). This control and the NSRS connection ensure accurate relativity between surveyed points at the airport and between these points and other surveyed points in the NAS, including the navigation satellites.

f. The USAF requires positioning and orientation data for the initialization of inertial navigation systems (INSs), INS test pedestals, NAVAIDs, and compass roses. The USAF relies on NIMA to satisfy all of its positioning and orientation requirements. Army topographic surveyors are currently assigned to assist NIMA in their mission to establish survey control for the USAF.

g. The US Army intelligence and signal elements require positioning information for remote vehicles, remote sensing and imaging systems, antenna systems for geological location and direction, inertial navigation initialization, situation awareness, and combat identification. This information includes the following:

(1) Accuracy. The accuracy requirement for intelligence and signal elements is similar to the accuracy expressed by FA and ADA. In many cases, intelligence and signal units can use the SCPs established for the FA and ADA.

(2) Frequency and Timeliness. The number of SCPs and the timeliness are dependent on the battlefield and the mission.

(3) Distribution. This survey information is distributed to each intelligence and signal battalion's Operations and Training Officer (U.S. Army) (S3). Topographic surveyors are responsible for notifying the S3 of the various datum within the area of operation (AO). Topographic surveyors should provide the S3 with the necessary parameters and instructions on how to transform local coordinates to a predefined common grid.

h. During operations involving deployments to joint-level commands, topographic surveyors may be tasked to perform a number of different missions. Topographic surveyors are capable of providing support to allied nations for any of the aforementioned defined areas.

i. Additional support for other topographic missions is defined in AR 115-11, FM 5-105, unit table (s) of organization and equipment (TOE), and directives from higher headquarters (HQ). These missions--

- Provide precise positioning to support updating the MOS 81T (Terrain Analyst) database.
- Support construction surveyors (when projects require real-world coordinates).
- Establish and extend basic control for field surveys.
- Allow survey data and station description cards to be forwarded to NIMA, the organization's survey information center (SIC), and collocated terrain analyst teams upon request.

PART B - SURVEY OPERATIONS

2-2. General. The actual shape of the earth's solid mass is referred to as topography. A geoid is defined as the surface of the earth's gravity (attraction and rotation) which coincides with the MSL in the open undisturbed ocean. A spheroid (an ellipsoid of revolution) appears as a figure that is flattened at the poles and bulging at the equator. It can be described using a mathematical formula that defines a part of the surface of the geoid. However, because of the great variations in topography, many different spheroids exist. Because the earth's surface is irregular and pieces of mathematical computations are unreliable, the type of survey conducted depends on the purpose or the level of accuracy required.

PART C - SURVEY TYPES

2-3. General. In plane surveys, all points are referenced to a flat plane with the curvature completely or partially ignored. In geodetic surveys, all established points are referenced to the curved surface of a spheroid and, in all computations, the effect of curvature is computed.

a. Plane Survey. Plane surveys ignore the actual shape of the earth and apply the principles of plane geometry and trigonometry. These surveys are treated as if the measurements were made on a flat plane, with all lines being straight. When the survey area is less than 250 square kilometers and less accuracy is needed, curvature can be ignored. Most localized construction projects (highway and railroad) and boundary projects use plane surveys.

b. Geodetic Survey. Geodetic surveys take into account the size and shape of the earth. Since the stations in geodetic surveys are routinely spaced over extended distances, more precise instruments and techniques are required than for plane surveys. All observations are made on the actual curved surface of the earth. This curvature is later corrected through computations.

PART D - SURVEY METHODS

2-4. General. Topographic surveyors use theodolites, levels, and distance measuring equipment (DME). The automated integrated survey instrument (AISI) provides topographic surveyors with the capability to extend control through the use of a total station. On the other hand, the NAVSTAR GPS is capable of determining accurate positional, velocity, and timing information. The GPS provides positional and navigational data to civilian and military communities in the form of two positional services. The Standard Positioning Service (SPS) encompasses the civilian user and the United States Coast Guard (USCG). When using a single GPS receiver (absolute positioning), the SPS users are denied the high accuracy and instantaneous positioning capability of the GPS. The Precise-Positioning Service (PPS) consists of military users and authorized representatives. The PPS user can obtain high accuracy and instantaneous positioning if the receiver is capable of accepting the necessary cryptologic variables.

2-5. Global Positioning System. Absolute and differential (relative) positioning methods, use the GPS, provide accurate and timely positional data. The method of choice depends on the accuracy required, the equipment available, and the logistical requirements. At present, the PPS GPS receiver capable of performing relative positioning is the global positioning system-survey (GPS-S) differential global positioning system (DGPS).

a. Absolute positioning uses a single GPS receiver and does not require known survey control. Absolute positions can provide instantaneous (real-time) or postprocessed positions. Known survey control is unreliable or nonexistent in immature theaters. Topographic surveyors can establish SCPs by using absolute positioning.

b. Differential positioning uses two or more GPS receivers. One GPS receiver (reference receiver) is resident over a known SCP. The remaining receivers (remote) are used to position points of interest. Differential positioning can be performed in real time or through postprocessing. If real-time-positioning results are required, a communications link that is capable of transmitting digital data must be established at the reference and remote-receiver locations. This method supports distances up to 100 kilometers between the reference and remote stations. Topographic engineer battalions within the Army have PPS GPS receivers that are capable of real-time and postprocessed differential positioning and provide accuracy of about 1 centimeter.

2-6. Accuracy. The accuracy of a GPS is dependent on the user's classification (precise lightweight GPS receiver [PLGR] or GPS-S) and the surveying method employed (absolute or differential).

2-7. Status. At present, topographic surveyors have standardized PPS GPS receivers. These receivers have improved the efficiency and productivity of topographic surveyors and provided the Defense Mapping School (DMS) and the United States Army Engineer School (USAES) a background on the training, operational, and research and development requirements necessary to successfully field the GPS. The new GPS-S provides adequate absolute-positioning results and is designed to provide protection in a jamming/spoofing environment. The requirement for a PPS GPS receiver that is capable of performing DGPS when using military's authorized, encrypted pseudorandom noise (PRN) code has been met. This receiver satisfies the positional accuracy requirements of the Army, the DOD, and joint-level commands.

PART E - SURVEY CLASSIFICATIONS

2-8. General. Topographic surveyors are capable of conducting and supporting a wide variety of surveys.

a. Artillery surveys are conducted to determine the relative positions of weapons systems to targets. These surveys do not require the accuracy of geodetic-surveying techniques despite the relatively large areas and long distances. The requirements, methods, and techniques used by military FA surveyors are detailed in FM 6-2. ADA weapon systems require accuracies that are obtainable only from geodetic-surveying techniques.

b. Basic-control surveys provide horizontal and vertical positions of points. Supplementary surveys may originate from, and can be adjusted to, these surveys. The basic-control survey of the US provides geographic positions and plane coordinates of triangulation/traverse stations and the elevations or benchmarks (BMs). This information is used as the basis for the control of the US national topographic survey; many state, city, and private surveys; and hydrographic surveys of coastal waters. The techniques and methods used by military geodetic surveyors are discussed in this manual.

c. Satellite surveys determine high-accuracy, three-dimensional (3D) point positions from signals received by NAVSTAR GPS satellites. GPS-derived positions may be used to provide primary reference-control monument locations for engineering and construction projects from which detailed site plans, topographic mapping, boundary demarcation, and construction-alignment work may be performed using conventional surveying instruments and techniques.

d. Construction surveys provide data for planning and cost estimating. They are essential to locate or lay out engineering works and are recorded on engineer maps. Plane surveys are normally used for construction projects. The methods and techniques used by military construction surveyors are detailed in FM 5-233.

e. Airfield-engineering and NAVAID surveys are used to determine any combination of the following:

- The location of obstacles within 10 nautical miles of an airfield center.
- The dimensions of runways and taxiways, the height of flight towers, and the availability of NAVAIDs.
- The safe approach angles to runways and the minimum safe glide angle.
- The elevation of the barometer on an airfield.
- The positions and azimuths of points designated for INS checkpoints.
- The requirements of the Federal Aviation Administration (FAA), the USAASA, or an equivalent military activity.
- The information used to assist a military-aircraft crash or disaster incident investigation.

f. Hydrographic surveys are performed on large bodies of water to determine the channel depths for navigation and the location of rocks, sandbars, lights, and buoys. In rivers, these surveys are performed to support flood-control projects, power development, navigation, water supplies, and water storage.

g. Field classification and inspection surveys can help to identify features not normally revealed using a compiler, such as political-boundary lines, names of places, road classifications, and buildings obscured by trees. These surveys can also clarify aerial photographs by using comparisons with actual ground conditions.

h. Land surveys are used to locate boundaries and tracts of land on a city, county, state, national, or international level.

i. Inertial surveys are used to determine relative positions and azimuths. The Position and Azimuth Determination System (PADS) is now being used extensively to support artillery surveys.

PART F - SURVEY NETWORKS

2-9. General. Each survey has a fundamental classification of control points called a *network.* There are several different types of networks. A network of control areas usually establishes horizontal and vertical SCPs within a country. These areas are all referenced to a single datum and are related in position or elevation to each other. Networks are called basic, supplementary, and auxiliary. All horizontal networks in the US are referenced to the North American Datum of 1927 (NAD 27) and North American Datum of 1984 (NAD 84) with coordinates currently published in both. (North American Datum of 1983 [NAD 83] and WGS 84 are the same). Mean Sea Level of 1929 (MSL 29) is used for vertical control points. The continental United States (CONUS) uses the following terms:

- *Basic horizontal-control networks* are usually established by first-order geodetic triangulation, precise traverse, or GPS. The lines of the basic network are spaced throughout a country at intervals of about 96 kilometers.

- *Basic vertical-control networks* are established throughout the country by first-order differential leveling in areas spaced from 90 to 160 kilometers apart. Permanent benchmarks (PBMs) are spaced at intervals of about 3 kilometers on these lines.

- *Supplementary horizontal-control networks* are usually established by second-order survey techniques. These supplementary networks are used to fill in the areas between the basic control lines. Ultimately, either a basic or a supplementary network station will be spaced at intervals of about 6 to 16 kilometers.

- *Supplementary vertical-control networks* are established by second-order differential leveling. These lines are run within the basic control arcs to provide planned control-line spacing at intervals of about 10 kilometers. PBMs are emplaced on these lines at intervals of about 2 kilometers.

- *Horizontal auxiliary-control networks* are usually established by second- or third-order survey techniques. They provide localized control to be used by surveyors for artillery control, construction engineering surveys, mapping projects, or other positioning requirements. As more states and other agencies require geodetic accuracy for boundary and property surveys, they will use these networks.

- *Auxiliary vertical-control networks* are established by third-order differential leveling and are used to provide localized vertical control. They are also used to support artillery, construction, and engineering projects.

PART G - SURVEY EQUIPMENT

2-10. Conventional Survey Equipment. Topographic surveyors have theodolites, levels, and electronic distance-measuring equipment (EDME) in their equipment inventory. The AISI provides topographic surveyors with the capability to extend control in a timelier and more efficient manner. The AISI is a total station that combines angular, distance, and vertical measurements into a single electronic instrument that is designed to digitally record and transfer data into a computer.

2-11. Navigation Satellite Timing and Ranging Global Positioning System. The NAVSTAR GPS is capable of determining accurate positional, velocity, and timing information. The PPS consists of military users and authorized representatives. A PPS user can obtain high-accuracy instantaneous positioning if the receiver is capable of accepting the necessary cryptologic variables. When two or more receivers are used, it is known as DGPS surveying. The error values are determined and removed from the survey either by real time or postprocessing of the data. The type of DGPS survey used is dependent on accuracy requirements. There are two basic types of DGPS surveys--static and dynamic.

a. Static surveying uses a stationary network of receivers that collect simultaneous observations over a predetermined time interval. They yield the best accuracy.

b. Dynamic surveying uses one stationary receiver and any number of remote or roving receivers. It allows for rapid movement and collection of data over a large area. When operating in a real-time mode, the roving receiver provides very accurate positions almost instantaneously on the battlefield.

2-12. Computer Information Systems. Surveying has become a digital science. Modern survey systems work with software specifically designed to process field data, perform computations, and produce a precise product, whether it be a GPS network, a digital database, or computer-aided design (CAD) and computer-aided design and drafting (CADD). GPS-S computations require a computer system to process large amounts of mathematic variables. Efforts should be ongoing to obtain or upgrade to the fastest system available. Computer resources are standardized throughout TOE units with topographic surveyors. Application (such as databases or word processing) and functional (such as adjustment or CAD) software packages have increased the efficiency and productivity of topographic surveyors. The SIC collects and disseminates the positioning and orientation requirements of NIMA, FA, ADA, USAF, and armor units and maintains a digital database capable of archiving, querying, and manipulating survey control. Topographic surveyors are equipped with common GPS hardware and software and CAD and survey-application software.

PART H - FIELDWORK

2-13. General. Topographic surveyors perform most of their operational duties away from the parent unit, in constantly changing conditions. Survey fieldwork consists of making observations and measurements over a project area or battlefield; recording data; and returning the data to the computer or draftsman for computation, compilation, and dissemination.

2-14. Weather and Terrain. Weather and terrain can adversely affect field surveys. The effectiveness of optical and electro-optical instruments can be severely reduced by fog, mist, smog, or ground haze. Swamps and flood plains under high water can impede leveling operations. Signals from the GPS constellation generally require a clear line of sight to the sky. Urban and forested areas can mask or deflect the direct signal needed for accurate measurements. Good reconnaissance and proper planning can alert the field parties of the best times and methods to use.

2-15. Personnel. The rate of survey progress often varies in direct proportion to the training and experience level of the assigned personnel. The most effective method for training personnel is under real training conditions.

2-16. Equipment. Equipment reliability must be considered when establishing completion dates. Modern, well-maintained equipment can often increase the rate of progress. Older equipment, if properly maintained or adjusted, will yield accurate results. Having to stop to repair or replace broken instruments or parts can result in a slowdown or halt a field survey. All equipment should be calibrated during combat checks, before survey missions.

2-17. Survey Purpose. The purpose and the type of survey determine the accuracy requirements, and the accuracy requirements dictate both the equipment and the techniques used. For instance, comparatively rough techniques can be used for elevations in site surveys, but control-network leveling requires much more precise and expensive equipment and extensive, time-consuming techniques. First-order GPS, triangulation, traverse, or leveling for the control networks must have high-accuracy standards; cuts and fills for highways are much lower. In some surveys, distances to inaccessible points must be determined. High-accuracy distance and angle measurements are required so that these values, when used in trigonometric formulas, will yield acceptable results. This type of survey is directly dependent on the clearness of the atmosphere. Observing measurements for a single position can be delayed for days while waiting on good weather.

2-18. Errors. All measurements contain errors. Errors classified as systematic and accidental are the most common uncontrollable errors. Besides errors, measurements are susceptible to mistakes that arise from misunderstanding problems, poor judgment, confusion, or carelessness. The overall effect of mistakes can be greatly reduced by following a preestablished systematic procedure. This procedure is advantageous in all phases of a survey.

2-19. Progress. Rates of progress vary, depending on personnel experience and repetition. As skill and confidence increase, so will speed. Proper preparation and planning will reduce duplication of effort and increase efficiency.

2-20. Enemy. A hostile environment often forces a schedule adjustment. Night work requires greater speed, fewer lights, and increased security. Adding security forces increases the number of vehicles and personnel, which reduces efficiency and retards the time schedule.

2-21. Observations of Distances and Directions. Topographic surveyors observe distances and directions (angles) to establish the following:

- GPS, triangulation, and traverse stations for basic, supplementary, and auxiliary control networks.

- Gun and target positions for artillery batteries.

- Horizontal control to support PADS.

- Points and lines of reference for locating details (such as boundary lines, roads, buildings, fences, rivers, bridges, and other existing features).

- A stakeout point to locate roads, buildings, landing strips, pipelines, and other construction projects.

- Parallel lines or lines at right angles to other lines. Identify the tracts of land, measure inaccessible distances, or extend straight lines beyond obstacles.

- Picture points for databases.

- A need for work requiring the use of geometric or trigonometric principles.

2-22. Observations of Differences in Elevations. Topographic surveyors observe differences in elevations for the following reasons:

- For plotting projects and computing grade lines along a selected line.

- To stake out grades, cuts, and fills for earthmoving and other construction projects.

- For trigonometric elevations of triangulation and traverse stations for control nets and mapping projects.

- To establish gun and target positions for FA batteries.

2-23. Field Notes (DA Form 4446). Topographic surveyors record field notes to provide a permanent record of fieldwork. Even the best field survey is of little value if the field notes are not complete and clear. The field notes are the only records that are left after the survey party leaves the field site. Surveyors' notes must contain a complete record of all measurements or observations made during the survey. When necessary, sketches, diagrams, and narration should be made to clarify notes. Write-overs, erasures, or uses of correction tape or fluid are strictly forbidden. These actions, when prohibited by the unit's standing operating procedure (SOP), are cause for punishment under the Uniform Code of Military Justice. Recording errors should be lined out and initialed by the recorder, and the corrected reading should be entered on the recording form.

a. Record field notes using one of the following methods:

- A field recording booklet.

- A single-sheet recording form.

- A digital disk or device for automated data recording.

- A land survey plan.

- A property plan.

- Recovery and station description cards.

- A control diagram showing the relative location, method, and type of control established or recovered

b. Record field notes using the three general forms--tabulation, sketching, and description.

(1) Tabulation. Numerical data is recorded in columns following a prescribed format, depending on the type of operation, the instrument used, and the specifications for the type of survey.

(2) Sketching. Sketches add much to the clarity of field notes and should be used liberally. They may be drawn to scale (as in plane table surveys) or they can be drawn to an approximate scale (as in control cards). If needed, use an exaggerated scale to show detail. Measurements should be added directly on the sketch or keyed in such a way as to avoid confusion.

(3) Description. Tabulations with or without sketches can also be supplemented with narrative descriptions. The description may consist of a few words or it may be very detailed. Survey notes become a part of historic records, so a brief description entered at the time of the survey may be important and helpful in the future.

c. Ensure that the following qualities are applied to the completed field notes:

- The lettering conforms to the gothic style portrayed in FM 5-553. All entries should be formatted according to unit SOPs.

- There is only one possible interpretation of an entry. Decimal points and commas must be clear and distinct.

- Entries are complete and all resolved data is finished according to the unit's SOP. All entries should be recorded on the correct forms. Never record notes on scrap paper and transcribe them to a field recording form. If performing an underground survey, use a covered clipboard to protect the notes. Accurately describe the field experience. Sketches, diagrams, and notes help to reduce or eliminate questions.

- Loose-leaf sheets are numbered serially to ensure that all sheets are kept and turned in.
- The instructions for return of the notes or cassette tapes are clearly marked and any special-handling requirements are noted. An index of the field notes and a cross-reference to additional books or binders should also be shown.
- Party personnel and their duties and the project's beginning and ending dates are listed.
- Instruments used (including types, serial numbers, calibration dates, constant values, and dates used) are listed.
- A generalized sketch and description of the project are shown.
- The actual survey notes are shown on each page containing data.
- The completed heading, to include the station names (including the establishing agency and date), survey date, weather data, actual observed data, and any pertinent notes are listed.
- The observer's initials are marked in the bottom right corner of the recording form (indicating that the observer has checked all entries for correctness).

d. Standard abbreviations, signs, and symbols should be used in all survey notes and must be consistent with guidelines in such publications as AR 310-50 and FMs 21-31 and 101-5-1. Spell out words if there is any doubt about the meaning or interpretation of a symbol or abbreviation.

e. Field notes are considered legal documents and can be used in court proceedings. As such, no erasures or write overs are permitted. No position should be voided or rejected in the field, except in cases where the instrument or target was disturbed or the wrong target was observed. In either case, the position should be reobserved and the location of the reobserved data should be noted in the remarks section.

(1) All fieldwork should be done in black or blue-black ink (with no erasures) suitable for photocopying. The only exception is the field sheet of a plane-table survey.

(2) Field notes show what happened in the field. If a number is changed, make a single slanted line through the incorrect number, and insert the correct number directly above or next to the corrected value, creating the new entry. The person making the change should initial the changed entry. A note should be entered in the remarks column stating why the number was changed.

PART I - OFFICE WORK

2-24. General. Surveying procedures also consist of converting field measurements to a more usable form. Usually, the conversions or computations are required immediately to continue the fieldwork, but sometimes they must be held until a series of field measurements are completed. This procedure is known as office work, even though some of the operations may be performed in the field during lapses between measurements. Some office work requires the use of special equipment (calculators, computers, or drafting equipment) or extensive references and working areas. During survey operations, many field measurements require some form of mathematical computation

2-25. Computing. Office computing converts distances, angles, GPS measurements, and rod readings into a more usable form or adjusts a position of some point or mark from which other measurements can be made. This process involves the computation of--

a. Distances. The desired result is the horizontal distance between two points. In an electronic distance measurement (EDM), the distance is usually on a slope and has to be corrected to account for the temperature and barometric pressure and then reduced to the correct horizontal distance.

b. Azimuths and Bearings. In many operations, the observed angles are converted into directions of a line from north (azimuths) or north-south (bearings).

c. Relative Positions. The distance and direction of a line between two points determines the position of one point relative to the other point. If the direction is given as an azimuth bearing, a trigonometric formula, using the sine or cosine of the angle, multiplied by the distance, will result in a coordinate difference between the two points.

d. Adjusting. Some survey techniques are not complete until one or more of the following adjustments are performed. Adjusting is the determination and application of corrections to data. Adjusting provides a means of dealing with the random errors in a survey network. Adjustment causes the data to be consistent within itself and to a given set of references. Small errors that are not apparent during individual measurements can accumulate to a sizable amount. For example, in a linear adjustment, assume that 100 measurements were made to the nearest unit and required determining which unit monument was closer to the actual measurement. Adjusting the results requires reducing each measurement by the product that results from dividing the error by the number of measurements. Since the measurements were only read to the nearest unit, a single adjustment would not be measurable at any point, and the adjusted result would be correct. Some of the more precise surveys require least-square adjustments.

e. Global Positioning System Networks and Least-Square Adjustments. A least-square adjustment is the basis for correcting GPS (and traverse) networks that use automation to compute solutions in geometry and produce geodetic accuracy. A least-square adjustment in a survey network allows for the computation of a single solution for each station and minimizes the corrections made to the field observations. A least-square adjustment uses probability in determining the values for particular unknowns, independently weighs all field observations, highlights large errors and blunders that were overlooked before adjustment, and generates information for analysis after the adjustment (including estimates of the precision of its solutions).

f. Traverses. Traverse is the measurement of lengths and the determination of directions of a series of lines between known points. They establish the coordinates of the intermediate points. When computed, the accumulated closing error shows up as a position displacement of a known point. The displacement is corrected and distributed among the intermediate (traverse) points.

g. Elevations. Depending on the purpose, the elevations on some level lines are computed as the measurements are taken. When the line is closed, the difference in elevation (DE) between the measured and the known elevation is adjusted over all the stations in the line. In higher-order leveling, only the differences in elevation are recorded during the measuring, and all adjusting is done at the completion of the line. The error is then distributed among the various sections of the line.

2-26. Establishing Records. Office computations reduce the field notes to a tabular or graphic form. They become a permanent record and are stored for further use or subsequent operations. Many standardized forms are available and should be used. As long as the sheets are clearly identified and bound as a set, they are acceptable. Normally, all field notes should be abstracted and filed separately. The abstracts should be bound along with all computing forms into a single binder or folder and maintained on file for further reference. All pages should have the name and date of the person performing the work and at least one person who verified that page. Do not dispose of or destroy any of these records.

2-27. Performing Checks. Surveying involves a series of checks. The observer, the recorder, and the party chief should check the field notes before they are turned in for office work. Before computing, the assigned person should check the notes again. Most mathematical problems can be solved by more than one method. When checking a set of computations, it may be desirable to use a method that differs from the original computation method. An inverse solution (starting with the computed values and solving for the field data) or a graphic solution may be used. Each step that cannot be checked by any other means must be checked by a totally independent recomputation by another individual. Any errors or mistakes that are found must be resolved and rechecked before the computation is accepted.

PART J - SURVEY COMMUNICATION

2-28. General. Survey party members may find themselves in a situation where they become separated. The ability to communicate with each other could mean the difference between successfully completing a section of work or not. Even at short distances (as in site surveys or leveling operations), background noises can obscure direct voice contact. At longer distances, such as in EDM or direction-measurement operations, direct voice contact is impossible. Alternative types of communication are required.

2-29. Voice. On long lines, where hand signals are impossible, a radio must be used. Each theater of operations or Army command has published communications-electronics operation instructions (CEOI) that units must follow. Only frequencies obtained through the local signal officer may be used. All personnel must be familiar with the CEOI and the unit's communications SOP before using a radio. All radio communications must be kept as short and secure as possible.

a. Over shorter distances, during all types of site surveys, the AISI provides one-way voice communication. Two-way communication is the preferred method for short distances. Most units have some type of hand-held radios, although they are not TOE equipment. These radios should be able to communicate up to 5 kilometers and should not be limited to line of sight only. Portability, ease of operation, and frequency programmability should be considered when procuring this type of communication equipment. Military hand-held radios are readily available in most military communities.

b. TOE changes are resulting in frequency modulated (FM) radios being replaced with Single-Channel Ground-to-Air Radio System (SINCGARS) radios. The need to communicate across large distances is increasing in frequency. GPS surveys are conducted at distances of up to 25 kilometers and depend upon synchronization between receivers during data collection. Any disruption from a single station in a GPS network can result in a total loss of effort.

2-30. Digital. The primary focus of survey operations during wartime is to operate quickly over large distances. This requires the ability to transmit data digitally over the battlefield. The type of data will be largely or entirely GPS data. In order for a survey team to provide accurate positions where needed and in a timely manner, they need to operate in real time without having to process out the error code embedded in a GPS signal. The process of real-time GPS surveying begins with a base station receiver that broadcasts corrections to the signals emanating from the GPS satellites. Army surveyors have two means of transmitting this data.

a. Surveyors have an instrument known as a radio modem, designed primarily for broadcasting DGPS corrections or raw GPS data from a survey base station to one or more roving receivers for real-time differential or real-time kinematic (RTK) surveying. These radio modems require a line of sight between each instrument. They can be set up in a series of repeating stations that extend across the survey area. This system is effective only over a small, local area.

b. The primary system for distance data transmission on the battlefield is SINCGARS. DGPS is designed to transmit encrypted GPS data over SINCGARS. Any user that can receive the data will have a real-time correction to the broadcast GPS signal. This gives topographic surveyors the operational capability to perform their mission under circumstances where GPS signals are dithered or spoofed on the battlefield. A GPS signal can be retransmitted over a communication network to multiple users, which extends the range and capability of survey operations.

2-31. Miscellaneous. Mirrors and lights can be used for communication. A signal mirror can use the sun as a light source and a fairly accurate sighting device. Morse code or other prearranged signals can be used to effectively communicate during the day. At night, the same signals can be used with a light.

PART K - SYSTEM ACCURACIES AND REQUIREMENTS

2-32. Geodetic Triangulation. Triangulation in which the shape and size of the earth are taken into account is called geodetic triangulation. Geodetic triangulation is the highest form of survey engineering, necessitating extremely precise instrumental equipment and observational techniques, capable and conscientious personnel, and detailed computations. Geodetic triangulation is the most efficient method of controlling surveys over extensive areas of the earth's surface and is used in the basic horizontal control networks of the world. Triangulation is a method of surveying in which the stations are points on the ground at vertices of triangles forming chains or networks. In these triangles, the angles are observed instrumentally with highly accurate theodolites. The sides are determined by successive computations through the chains of triangles from selected triangle sides called baselines, the lengths of which are obtained from direct measurements on the ground.

2-33. Geodetic Triangulation Data. The data resulting from geodetic triangulation are primarily horizontal control data, which are expressed in the form of geodetic latitudes and longitudes (or equivalent plane coordinate values) of established points, including the distances and azimuths for all observed lines. In order to obtain horizontal control data, you must know the precise length of the starting line (either a measured base or an adjusted line of higher order), the latitude and longitude of at least one end of the starting line, and the azimuth of the starting line. These points are normally marked by bronze discs (most frequently set in concrete monuments) but may also include many prominent objects, such as church spires, water tanks, and radio towers.

a. From the known line, triangulation stations are placed at the desired intervals, forming a series of triangles. The angles are observed at the vertices of the triangles that form the triangulation stations. In each successive triangle, one length or side is known, and all three angles are measured. The other lengths or sides of the triangles are computed by applying the law of sines (sin) (Figure 2-1).

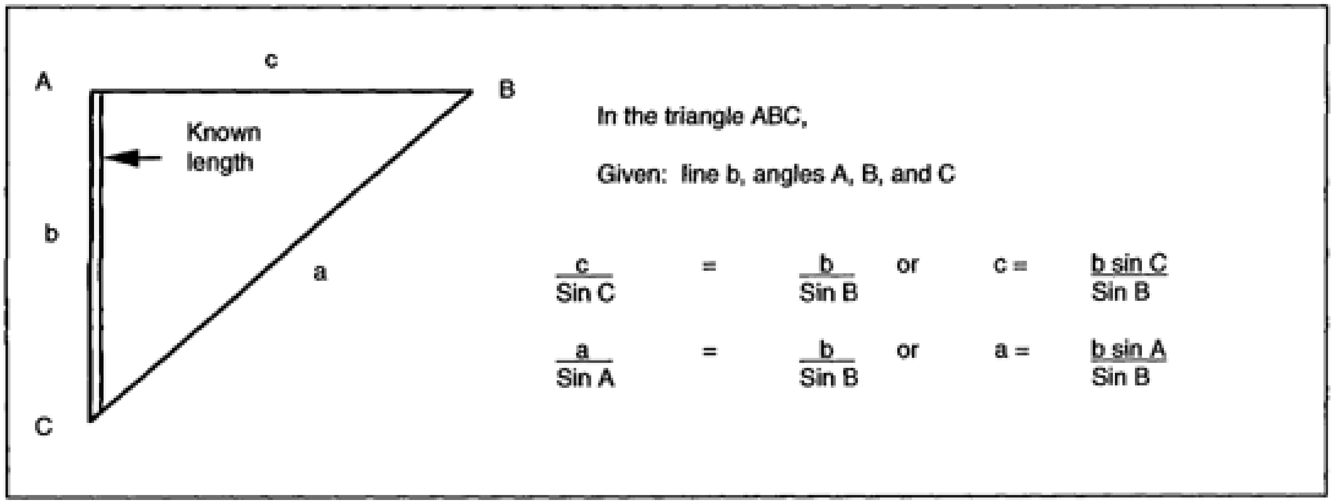


Figure 2-1. Solving Angles and Using the Law of Sines

b. To begin work on a net of triangulation, the length of the starting line is required. The final computation of the positions of the new stations, as well as the final azimuths, is done after the triangulation is complete from one line of known length to another and after the net has been adjusted. The azimuth, latitude, and longitude are obtained from astronomic observations made during or after the triangulation observations.

2-34. Classification of Triangulation. The basis of classification of triangulation is the accuracy with which the length and azimuth of a line of the triangulation are determined. Higher order triangulation is performed under two primary orders of accuracy. These orders are subdivided to give a total of five different degrees of precision. The principal criterion is that the discrepancy between the measured length of a baseline and its length, as computed through the triangulation net from the next preceding base, shall not, after adjustment, be greater than the length closure shown for each class.

a. First-Order Triangulation. There are three classes of first-order triangulation.

- Class I surveys are the most precise class of survey and must have a length closure of 1 part in 100,000. Its use is generally restricted to surveys for scientific purpose. These include establishing and testing missile and satellite systems, and performing studies of the shifting of the earth's crust and the tilting of landmasses. Class I is also the basis for accurate land surveys in highly developed areas.

- Class II must have a length closure of 1 part in 50,000. The basic national-control net should be composed of arcs of triangulation of this order, with a spacing of not more than 60 miles between arcs. The main effort today is toward this goal.

- The length closure in Class III is 1 part in 25,000, which corresponds to the classification formerly accepted as the most precise triangulation and used as a basis for extension of all surveys of the same and lower order. Old work in this class is being strengthened by additional baselines and by connection and adjustment to new Class I or Class II work. However, some Class III triangulation is still in demand in remote areas of the world.

(1) All first-order triangulation must start from stations of known position, baselines, and azimuths that have been established with the appropriate degree of accuracy. These nets must tie to baselines and azimuths of the same or higher order. Starting from and tying to two adjacent stations in a previously established and adjusted net of the same or higher order may meet these requirements. The criteria for length closures, after all side and angle conditions have been satisfied, have been previously given. Other criteria that should be considered are the maximum strength of figure (R1), side checks, inside equation tests, and the probable error and frequency of a check azimuth.

(2) The instrument used in running first-order triangulation should be a 0.2-second optical reading, direction theodolite, or equivalent. Since first-order triangulation is normally performed at night, signal lights (which will be discussed in a future chapter) are used as targets. In Class II triangulation, it is permissible to make daylight observations using the heliotrope or signal lights in areas where atmospheric conditions are stable. Daylight observations on standard cloth or wooden targets are not normally accepted.

(3) The methods used in observing first-order triangulation are designed to give a maximum triangle closure of ± 3 seconds of arc and an average triangle closure for the net not exceeding ± 1 second. This requirement can be met by observing the horizontal circle using a 0.2-second theodolite or equivalent. The rejection limit should be ± 4 seconds from the mean for any individual direction, with a set of 16 positions observed each of two nights, with a minimum of 2-hour separation between the two sets. The mean of the sets must agree within 1.5 seconds, or additional sets must be observed until two sets agree within 1.5 seconds.

b. Second-Order Triangulation. There are two classes of second-order triangulation.

- Second-order, Class I triangulation is used to subdivide areas between first-order control. It provides area networks and supplementary cross arcs in the primary scheme to be used in the extension of control for mapping, cadastral, and local land surveys.

- Second-order, Class I triangulation must start from and tie to lines of a first-order triangulation net or two adjacent stations of an adjusted second-order, Class I net. The length closures, as compared to these lines or measured baselines, must not exceed 1 part in 20,000 after all side and angle conditions have been satisfied.

(1) Second-order, Class II triangulation is used to subdivide areas of higher control and to provide networks for mapping, cadastral, and local surveys. Second-order, Class II triangulation must start from and tie to lines of a first-order triangulation net or to lines of an adjusted net of second-order, Class I or Class II. Length closures, as compared to these lines or measured baselines, must not exceed 1 part in 10,000 after all side and angle conditions have been satisfied.

(2) The 0.2-second theodolite is also used for second-order, Class I and Class II triangulation. Observations are made at night using a 1.0-theodolite and signal lamps. Daylight observations using heliotropes are also acceptable.

(3) The methods used in the observation of second-order, Class I and Class II, are designed to give a maximum triangle closure of ± 5 seconds, with an average triangle closure for the net that does not exceed the required average of ± 1.5 seconds for Class I or ± 3 seconds for Class II.

2-35. A Typical Triangulation Net. A typical triangulation net is shown in Figure 2-2. In first-order triangulation, the known lines AB and CD are always baselines measured to such a precision as to ensure that errors do not exceed 1 part in 300,000. For second-order triangulation, the known lines such as AB and CD are either second-order baselines or adjusted lines of first-order triangulation. The measured baseline AB is the known length. Once all the angles are observed, use the law of sines to compute the lengths of the other lines in the figure. There are various ways of computing line EF, such as through the triangle ABE, as shown in Figure 2-3. After computing the distance BE, which is then used as the known line of the triangle BEF, line EF is computed. Another route to compute the length of EF is through the triangles AFB and AFE. One of the routes is geometrically superior to the other, depending on the angles used. Carry this system through from one figure to another until the next base (CD) is reached. Obtain the check on the discrepancy between the measured length of that line and its length as computed through the triangulation net from the preceding base (AB).

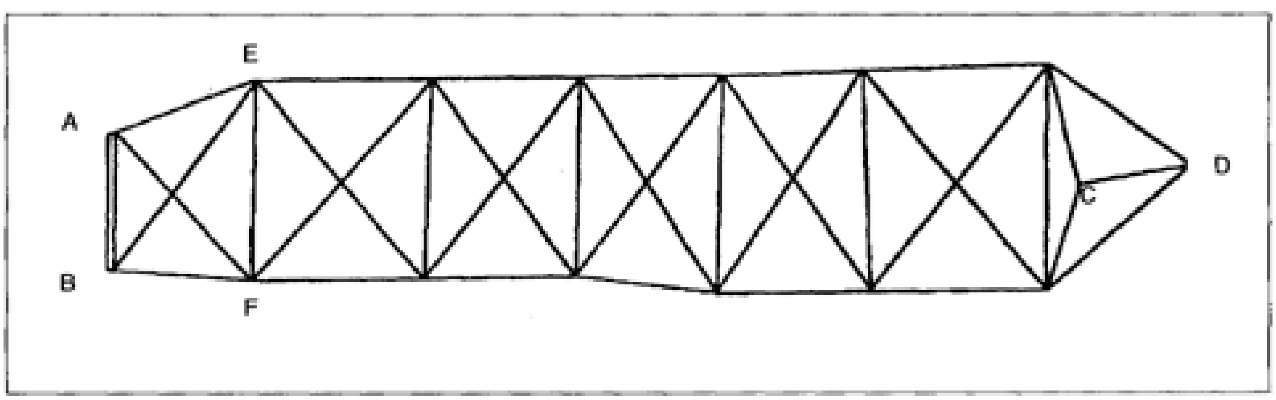


Figure 2-2. A Typical Triangulation Net

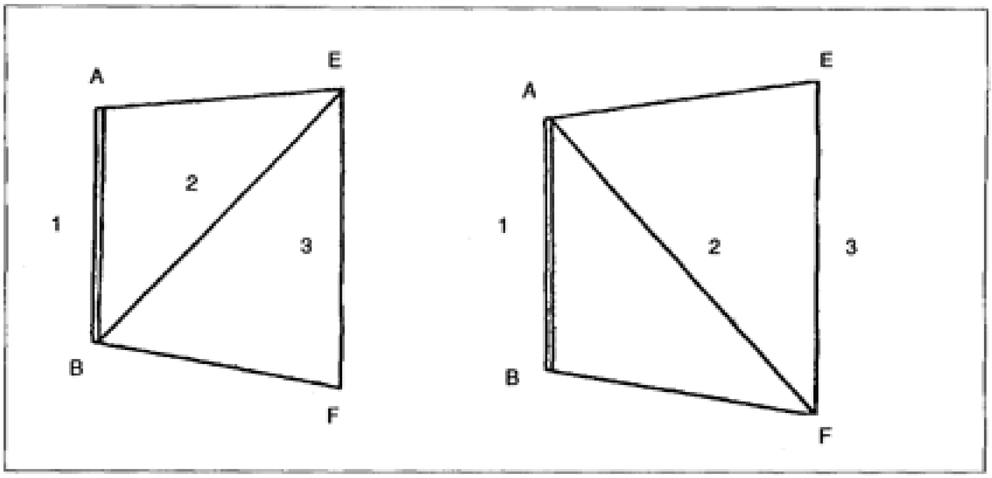


Figure 2-3. Methods Used to Compute Line EF

The illustrations in Figure 2-3 are quadrilaterals. This is the simplest and most economical figure used to give a double determination or check on the length of the line, as computed through the quadrilateral. This figure will be used as the *go forward* line or *base* for the succeeding figure. In first-order triangulation, the figures must be such that they afford a double check and all stations are occupied. Single triangles should not be used in first-order triangulation and only rarely in second-order triangulation.

2-36. Acceptable Error of Angle Measurements. Together with the criterion of length agreement between bases, and almost equal in importance, are the requirements limiting the acceptable error of angle measurements. The limits imposed on angular errors serve to maintain a uniform accuracy along the chain of triangles. The specifications for procuring the required accuracy make use of other criteria, such as the number and *strength* of the geometrical figures between adjacent bases, the observation of astronomical azimuths at specified intervals, and the accuracy of baseline measurements. All of these requirements are subsidiary to the controlling tests of the agreement between the measured and the computed length of a base and the limits specified for angle errors, and they are essential in securing sustained accuracy and control.

2-37. Standards of Accuracy. In review, the following standards of accuracy are rigidly applied to field observations in order to obtain consistent results in procuring the required precision. The requirements will vary, depending on the classification of the triangulation, be it first order or second order.

- Baseline-measurement accuracy.
- The strength of a single figure in a network.
- The sum of the strengths of figures (R1) between baselines.
- The maximum single-triangle closure.
- The average triangle closure between bases.

- The number of observations required for each angle.

- A check in each figure between the two values computed for the *go forward* line as obtained through the two strongest routes.

- A final check in the agreement between the computed length and the measured length of the base or the adjusted length of the check line (if the standards of accuracy have been adhered to).

a. To ensure that the desired accuracy is maintained and accidental errors are kept within the prescribed limits throughout the triangulation, give careful consideration to the capabilities and limitations of the surveying instruments and the accepted methods of observing. Under certain conditions, it is possible for the allowable error in the length of a line to be exceeded even when the triangulation meets the other specifications for that particular order of control. This may be caused by errors due to compensation to meet other specifications or by accumulative errors in which all the angular errors, although within the specified maximum allowed, are mostly of the same sign.

b. In the process of adjusting the final observations by the *method of least squares*, the results are consistent throughout; but all the errors cannot be removed. If the observational errors are small and indiscriminately plus and minus, then the adjustment will probably distribute them so that there is only a slight accumulation of errors. However, blunders, large accidental errors, and systematic errors of varying signs cannot be distributed correctly by the adjustment process; therefore, exercise great care in doing everything possible to avoid them.

PART L - SURVEY RECONNAISANCE

2-38. General. The reconnaissance party must consider special factors as determined by the objective of the survey and the methods, techniques, and equipment that will be employed.

2-39. Reconnaissance Requirements. A proper survey reconnaissance includes the following factors:

- The proper gathering of all existing survey data in reference to the target area.

- The proper testing and determination of the usability and visibility of existing stations.

- The selection of sites for the main and supplemental stations.

- The determination of monumental requirements.

- The collection of terrain and climatic information.

- Arrangements for access to private or government property.

- The availability of billeting, mess, medical, maintenance, and other required support.

2-40. Global Positioning System. Interreceiver visibility is not required for GPS surveying. Stations can be set according to network design principles rather than traversing around buildings or mountains. The only requirement for receiving GPS signals is a clear view of the sky. Choose a station with no obstructions above an incline of 15° to 20°. Draw a station-obstruction diagram to assist in the planning of GPS sessions. Verify the station accessibility, draw maps with directions to the stations, and mark each station clearly. The field crew will be in a hurry to set up when they arrive, and unmarked stations can waste valuable time.

2-41. Triangulation. When the need to locate the position of a point that cannot be occupied arises, as in a special survey, triangulation is necessary. This technique makes special demands on the reconnaissance party. The mathematical computations place stringent requirements on the size and shape of the geometric figures that are used to determine coordinates. For this reason, the location of the stations will normally be dictated to the field-reconnaissance party, based on the results of the office reconnaissance. The reconnaissance party must ensure that the observation stations, which form the baseline, are intervisible. A thorough knowledge of triangulation criteria is absolutely necessary.

2-42. Traverse. The demands for a traverse reconnaissance are less stringent than for triangulation. Ensure that both the rear and forward stations are visible from each proposed station. Wherever possible, distances between stations should be uniform. In control surveys that may become part of the United States National Control Network, the Standards and Specifications for Geodetic Control Networks (SSGCN) must be satisfied. Spacing between stations is dependent on the available EDME.

2-43. Electronic Distance-Measuring Equipment. An EDME traverse reconnaissance requires intervisibility between stations. The minimum and maximum allowable distances are based on the EDME characteristics and the clearance above possible obstructions. Use of infrared EDME is dependent on the weather.

2-44. Differential Leveling. Differential leveling should follow routes containing the least amount of change in elevation between BMs and individual setups. The routes frequently follow roads with moderate traffic, so care must be taken to ensure the safety of the leveling party.

2-45. Trigonometric Leveling. Trigonometric-leveling reconnaissance is performed during a traverse reconnaissance. If possible, select a relatively level area to perform the reconnaissance. Failure to accurately level the instrument could cause a greater error, especially in an elevated or depressed observation.

2-46. Other Control Methods. Reconnaissance for other control methods will vary according to the physical characteristics and limitations of the equipment or system used. No matter what system or equipment is used, the proposed station must be accessible and able to be included in the local survey control scheme. Stations occupied by PADS must not exceed the maximum distance and time from the initializing station.

2-47. Reconnaissance Party Composition. The reconnaissance party will vary in disposition and number according to the method of survey, the type of terrain, the available transportation, the extent of the survey, and the density of control required. The chief of the reconnaissance party is normally the section leader. The reconnaissance party usually consists of two to five personnel. At a minimum, the party will include the survey party chief and the section leader. It is also helpful to include personnel for instrument operations. The most qualified unit members should be assigned to the reconnaissance party. A properly designed reconnaissance will result in a survey project that is accurate, complete, and expeditious. The reconnaissance party should be thoroughly briefed on the project instructions and the specifications of the survey mission. Reconnaissance is accomplished in three phases--office reconnaissance, field reconnaissance, and reporting.

a. The office reconnaissance phase, which includes the gathering of existing data and a study of applicable maps, should be completed before the start of the field reconnaissance phase. The first step is to gather all existing data on the area to be surveyed. Depending on the area, there may be a number of sources that maintain some type of reliable survey data. The existing data will usually consist of trig lists, station-description cards, and aerial photographs or maps. Trig lists come in many forms, depending upon the publishing agency. A trig list may be compiled on a DA Form 1959, horizontal-control data booklets from the National Geodetic Survey (NGS) office, or a computer printout of coordinates. All trig lists (officially classified or not) must be safeguarded. Once secured, this information should be maintained as a database for that area since it may be necessary to conduct additional surveys in the same or an adjacent area. Sources of information for reliable data include the following units or agencies:

- Local Army units (such as map depots, FA target-acquisition units, SICs, and survey units).
- The NGS and the United States Geological Survey (USGS).
- United States Army Corps of Engineers (USACE) district offices.
- The Department of the Interior (DOI), Bureau of Land Management.
- State and local government civil-engineering or survey offices.

- Other nations. Existing data is sometimes received from the national agency charged with the mapping of that nation. Local municipalities and city governments also have survey information within engineering or land-planning offices.

Do not evaluate existing material until all material has been assembled and the information has been annotated on the available maps or aerial photographs. Plot the required SCPs from the project directive, and evaluate the usability of existing controls. Compare the required control method with the existing control method to determine if additional basic control is needed. It is possible that many required stations may be eliminated because adequate control already exists. For required stations that must be established, a tentative route of survey is annotated on the maps.

b. The field-reconnaissance phase is different for each survey project. A party chief must consider the lessons learned from previous projects and apply the methods and techniques to suit the conditions of the current project. A successful party chief employs the knowledge and ingenuity of the survey-party personnel. Time permitting, the party chief and one other person should conduct a preliminary field inspection of the area. When gathering information concerning the area to be surveyed, include the terrain; tree heights; the road width; the road surfaces; the spacing between roads; the microclimate, such as fog, haze, and heat waves; and any other factors that may have an effect on the distance measuring and intervisibility between the proposed stations. The inspection may be conducted using vehicles, helicopters, or airplanes. The results of the inspection determine the scheme and the route for the survey.

2-48. Recovery and Verification of Existing Control Stations. In areas where control is to be extended or established, there may be control stations from earlier surveys that must be recovered and verified. These stations should have been identified and annotated on overlays during the office reconnaissance phase and will serve as starting points for proposed GPS networks, traverse lines, or level lines. The existing stations should be located, described, and verified for accuracy before using them for extending control.

a. Recognize that existing control stations (and their establishing surveys) follow similar patterns. Recognizing and associating the patterns with the terrain types will assist the surveyor in locating existing stations. Triangulation stations are usually found on the highest point of a hill or a mountain. In areas of little relief, the stations may be located at prominent points or sites where a tower can be easily erected. BMs and traverse stations are typically located along roads, railroads, pipelines, or other transportation routes which permit intervisibility and accessibility. BMs and traverse stations may also be found along waterways, rivers, canals, and coastlines.

b. Evaluate all existing information. In some areas, urbanization has changed existing road or drainage information. In rural areas, land may have been cleared and cultivated, or fields may have become overgrown or reforested. Gather and consider all available information when searching for a station.

(1) Trig lists, control cards, and control bulletins contain brief descriptions and sketches of stations. The information may be outdated or insufficient for a final product but will permit surveyors to locate the general vicinity of the station. The final step in locating the station involves the use of distances and azimuths from the reference marks (RMs) to the station.

(2) Previous survey data may include survey schemes, overlays, or plots depicting the relative position of the stations in the general area. After one or more stations have been recovered, the other stations may be roughly plotted and located using a magnetic compass and either intersection or resection methods.

(3) Aerial photographs may be used if the station to be recovered can be identified in a photograph. Using features that are permanent and prominent on both the photograph and the ground will permit surveyors to reach the station site.

(4) Maps with the station's plotted coordinates permit surveyors to identify the route of travel to the station. Maps also assist surveyors in determining the station's accessibility.

(5) Local information sources include local surveyors, public-service officials, construction companies, and landowners. Local sources may be the only means of locating a station if the area has dramatically changed since the other sources of information were published.

c. Verify a station before using it. In a situation where only one other station is intervisible, a check-distance measurement can be performed using the GPS or a conventional method. When two or more stations are intervisible, check-angle observations or GPS measurements can be performed. After the measurements and observations have been performed and reduced, they will be compared to the published information. If the results within the overall specifications for the survey project agree, the stations may be used.

2-49. New Station Sites. New station sites will be selected after all existing stations have been recovered, described, and verified. The new stations will be placed where required to complete the scheme of the survey. The correct selection of a new station site will save time and expense and prolong the life of the new station. Take the following factors into consideration when selecting a new station site.

a. Monuments can be permanent or temporary.

(1) Permanent monuments are set in a relatively stable material or structure for the purpose of preserving the location of either horizontal or vertical control. Consider another site if the proposed site might experience disturbance or land development. Since there is a wide variety of possible situations that may be

encountered when setting a monument, it is impossible to address them all. The ultimate selection of the site is at the discretion of the monument setter.

(2) Temporary monumenters are the same as permanent monuments except that the required preservation time is less. Temporary monumenters consist of a 1-by 2-inch wooden hub (or larger) with adjacent guard stakes, a copper nail and washer, or a temporary spike that is set in relatively stable material.

b. Monument's are susceptible to damage or destruction. It is necessary to anticipate any construction that might occur in the area. Frequently, monuments that are set in asphalt surfaces are paved over. Monuments placed off the edge of an asphalt surface stand a better chance of survival.

c. Monuments should be accessable. If the monument cannot be found or conveniently occupied, its worth is questionable. Determine if there are nearby objects that can be used as references. Distances and directions from prominent reference objects should be used to locate a monument. These distances and directions are referred to as lines of position (LOPs). The prominent objects are referred to as origins. At least two LOPs are required to describe a point. The closer to perpendicular the angle at which the LOPs intersect, the more accurate a position can be described.

d. All monuments are subject to the effects of geologic and soil activity. Vertical-control monuments and BMs are particularly vulnerable because this activity results in vertical movements much more than horizontal motion. Selecting advantageous topographic features, such as on the crests of hills where the soil consistency tends to be firm, increases soil stability and decreases frost heave. When possible, choose a site with coarse-grained soils. Fined-grained soils (such as clays) are susceptible to high moisture content.

e. If a monument extends below the ground, there is a chance of encountering underground cables or pipes during installation. Evidence of underground utility lines can often be observed at the surface. Waterlines are marked by valve boxes, and in structures newer than 1960, the utilities are likely to be buried. Avoid digging near light poles, phone lines, or electric and gas junction boxes.

f. The ideal site provides maximum visibility above the horizon, plus 15°. Any obstruction above 15° could block satellite signals. The ideal site should have visibility in all directions above 15°; however, an obstruction in one or two directions may not affect the ability to use the site for GPS surveying. Existing BMs should be used as GPS monuments as often as possible. New monuments should be located as close as possible to a known vertical control. Maximum effort should be made to locate all GPS-type monuments within 100 feet of easy access to ground transportation.

2-50. Station Names. The customer normally assigns the station names. An example of a station name would be the project name or number followed by the sequence number of that station in the scheme-of-control extension. Names should be an alphanumeric symbol that is stamped on the respective disk monument. The name that appears on the control point for publication purposes should be the same as the name that actually appears on the monument. Old stations that are reestablished are given the previous name with a numerical suffix added. In the absence of guidance from the customer, adhere to the following rules:

- Use the name of a nearby geographical feature.
- Use short names with a maximum of 25 characters, including spaces.
- Include the name of the agency or unit that set the monument if it is not precast.
- Make sure the station name is spelled correctly on all documents.
- Do not use special characters such as periods, commas, slashes, or equal signs.
- Do not include nondescriptive terms or personal names.

2-51. Landowner Permission. Permission must be obtained before conducting a survey on any private land. The survey section sergeant or the party chief, working through the local Judge Advocate General (JAG), will contact and negotiate with landowners for access to prospective station sites. Written permission to enter the land is preferred because it is documented. The local JAG will assist in this matter and will help keep the military out of potential trouble.

a. Survey Land Within the United States. Reconnaissance and survey parties should have a right-of-entry letter from their headquarters to enter a selected area. This letter does not entitle the survey team to access private property or restricted areas without further permission. When the landowner is contacted, a full explanation of the work to be done is given without any attempt to conceal any inconveniences or damage that may arise. Government regulations concerning damage claims should be explained. In the case of an absentee owner, a letter explaining the work and requesting consent to access the property should be mailed.

b. Survey Land Outside of the United States. When working in other nations, the appropriate officer of the US embassy within that country will generally negotiate the right-of-entry letters for overall areas within that country. However, a right-of-entry letter or approval from the host nation is not always sufficient for access to all public lands within the national boundaries. It is sometimes necessary to contact the local officials where the work is to be performed. Agreements will be conducted according to local customs. Some countries consider an oral agreement, or any statement that could be construed to be an oral agreement, to be contractual and binding. Any transfer of assets (material or otherwise) requires close coordination with the JAG.

2-52. Station Description and Sketch. The reconnaissance party prepares a description and sketch of all newly established permanent, temporary, and recovered stations. Stations recovered, but not used, must also have a description completed. The description and sketch should be completed on a DA Form 1959, as shown in Figure 2-4, or in an appropriate field book. The field record should be completed in freehand using vertical gothic lettering. A final DA Form 1959 should be typed and kept with the official records.

COUNTRY	TYPE OF MARK	STATION	
Germany	70 Monument	Stone Kamp	
LOCALITY: Illesheim/L6528	STAMPING ON MARK: NA	AGENCY (CAST IN MARKS): NA	ELEVATION: 331.671 (FT) (M)
LATITUDE: 49°28'10.47467"	LONGITUDE: 10°23'10.92519"	DATUM: WGS 84	DATUM: Amsterdam
(NORTHING) (EASTING): 5,480,852.200 (FT) (M)	(EASTING) (NORTHING): 600,444.268 (FT) (M)	GRID AND ZONE: 32U	ESTABLISHED BY (AGENCY): 320th Engineer
(NORTHING) (EASTING) (FT) (M)	(EASTING) (NORTHING) (FT) (M)	GRID AND ZONE	DATE (YYYYMMDD): 2001 07 16 / ORDER: Third

TO OBTAIN GRID AZIMUTH, ADD ° ' " TO THE GEODETIC AZIMUTH

TO OBTAIN GRID AZ. (ADD) (SUB.) ° ' " TO THE GEODETIC AZIMUTH

OBJECT	AZIMUTH OR DIRECTION (GEODETIC)(GRID) (MAGNETIC)	BACK AZIMUTH	GEOD. DISTANCE (METERS) (FEET)	GRID. DISTANCE (METERS) (FEET)
	° ' "	° ' "		

The station is located on Storch Barracks, Illesheim, Germany.

To reach the station front gate of Storch Barracks (Grid 0082) go straight for 0.1 mile to four-way intersection. Turn right (west) and proceed 0.8 mile to the gate of the access road and a guard shack. Follow the access road around the perimeter of the airfield for 0.9 mile to the station site.

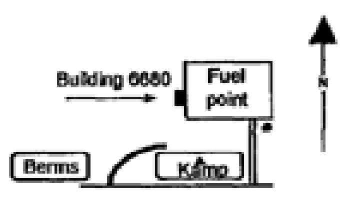
The station is a Type 70 monument protruding 20 cm above the ground and is located atop a burm.

The station is located 75.1 m at an azimuth of 160° from Building 6680, 82.3 m from the hot fuel point and 67 m from the fuel point sign.

Horizontal position was established by third-order class I traverse.

Elevation was established by third-order leveling procedures.

SAMPLE



SKETCH

DA FORM 1959, JUL 2001

REPLACES DA FORMS 1959 AND 1960, 1 FEB 57, WHICH ARE OBSOLETE.

DESCRIPTION OR RECOVERY OF HORIZONTAL CONTROL STATION
For use of this form, see FM 3-34.331; the proponent agency is TRADOC.

USAPA V1.00

Figure 2-4. Sample DA Form 1959 (Recovery Card, Horizontal Control)

2-53. Intervisibility of Stations. To execute reconnaissance for triangulation, use either of the following general methods or a combination. In the first method, which can be used in hilly or mountainous country, test the intervisibility of the stations by visiting each one. If the stations are not intervisible from the ground, clear obstructions on the line (such as trees) or adjust the tower heights to allow intervisibility from either or both ends of the line. Find the elevations of intervening high points by lowering a weighted tape from a helicopter or by using barometric or trigonometric leveling. In the second method, obtain the elevations of the stations and the intervening country from maps or other sources, and determine the intervisibility of points and the required heights of towers from the data. In actual practice, a combination of the two methods is generally used. The reconnaissance party should keep a total tower height for any one line to a minimum. Determining the instrument and signal heights is a matter of good judgment, trial computations, and experience.

a. The difference between the apparent and the true difference in elevation of two points is affected by two factors--the curvature of the earth's surface and the refraction of light by the earth's atmosphere. Because of the earth's curvature, distant points appear to fall below a level line of sight from the point of observation. Vertical refraction causes the line of sight to curve downward and makes the distant point appear higher than its true elevation.

b. The effect of refraction is about one eighth as much as the curvature and opposite in sign (refer to Table 2-1). In survey work, the two are combined and the approximate resultant correction is given by the following formula:

$$h \text{ (meters)} = K^2 \text{ (in kilometers)} \times 0.0676$$

Where h is the known quantity, the formula is:

$$K \text{ (in kilometers)} = \sqrt{h} \text{ (in meters)} \times 3.9478$$

Table 2-1 gives the corresponding values of K and h.

Table 2-1. Correction for the Earth's Curvature and Refraction

Distance Kilometers	Correction Meters	Distance Kilometers	Correction Meters	Distance Kilometers	Correction Meters
1	0.1	21	29.8	41	113.5
2	0.3	22	32.7	42	119.1
3	0.6	23	35.7	43	124.8
4	1.1	24	38.9	44	130.7
5	1.7	25	42.2	45	136.7
6	2.4	26	45.6	46	142.8
7	3.3	27	49.2	47	149.1
8	4.3	28	52.9	48	155.5
9	5.5	29	56.8	49	162.1
10	6.8	30	60.8	50	168.8
11	8.2	31	64.9	51	175.6
12	9.7	32	69.1	52	182.5
13	11.4	33	73.5	53	189.6
14	13.2	34	78.0	54	196.8
15	15.2	35	82.7	55	204.2
16	17.3	36	87.5	56	211.7
17	19.5	37	92.4	57	219.3
18	21.9	38	97.5	58	227.1
19	24.4	39	102.7	59	235.0
20	27.0	40	108.0	60	243.0

c. The general problem for intervisibility is determining how much a line of sight between two stations will clear or fail to clear an intervening obstruction. Use the following formula to make this determination:

$$h = h1 + (h2 - h1)\frac{d1}{d1 + d2} - 0.0676\ d1d2$$

h = The elevation of the line at obstruction (in meters)
h1 = The elevation of the lower station (in meters)
h2 = The elevation of the higher station (in meters)
d1 = The distance from the lower station to the obstruction (in kilometers)
d2 = The distance from the higher station to the obstruction (in kilometers)

The first two terms of the above formula are a solution of similar triangles. The last term is the curvature and refraction correction. If the two stations are at the same elevation, the obstruction can be cleared using signals of equal height, and the above formula becomes the following simple formula:

$h = hl - 0.0676\ d1d2$

The height of the signal at each end becomes the following formula:

$h1 = h + 0.0676\ d1d2$

d. For a given length (K) between stations, the elevation required at one station to see the other station across a level surface (such as water) is K2 X 0.0676. If both stations are raised to the same elevation, the height (h2,1) required at each station is $(K/2)^2$ x 0.0676, or one-fourth at one end only. In the field, the method generally amounts to a determination of an equal signal height for both stations so that the sight line clears the obstruction. Signal heights are nearly always computed in this way. Sometimes a higher signal is specified at one station for some other line than the one under consideration. This permits a lower signal at the opposite end of the line. The amount one signal may be reduced in height for a certain increase of the other is proportional to the distance of the two stations from the obstruction. The station nearest to the obstruction requires the least signal height to provide a clear line of sight. Figure 2-5 illustrates where h1 and h2 are signal heights required at stations A and B respectively, to clear obstruction H.

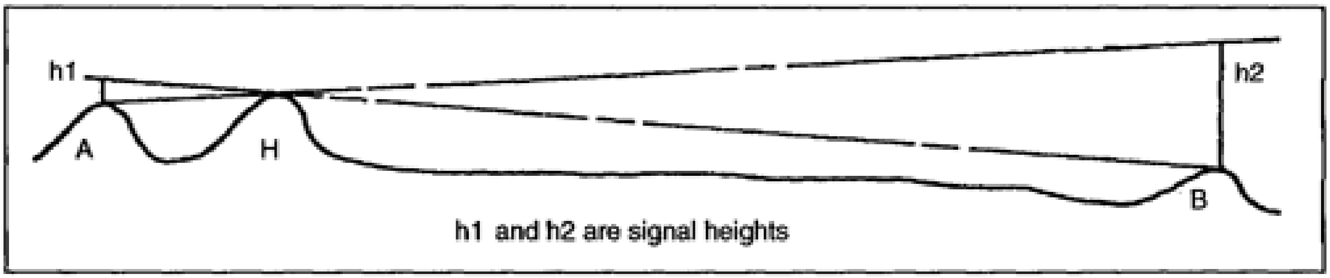


Figure 2-5. Clearance of Obstruction

e. Occasionally, a series of obstructions may occur along a line of sight that may not be detectable through inspection. A lower obstruction near the middle of the line may require higher signals to clear than a higher obstruction near the end. The simplest way to determine a critical obstruction is to compute the signal heights for each one in turn. The following examples illustrate typical problems:

Example 1. Two stations are at water level on opposite shores of a 30-kilometer-wide bay. What is the height (h) of equal signals required to make the line of sight graze the surface of the water? According to the formula, $h = (K/2)^2 \times 0.0676 = (30/2)^2 \times 0.0676 = 15.2$ *meters*. Figure 2-6 shows h as the distance of a line joining the stations (passing below the earth's surface). Because the obstruction is equidistant from the two stations, h will also be the required height of equal signals at the two ends needed to clear the line of sight. In actual practice, it is generally necessary to provide a certain amount of clearance over obstructions in order to reduce horizontal refraction.

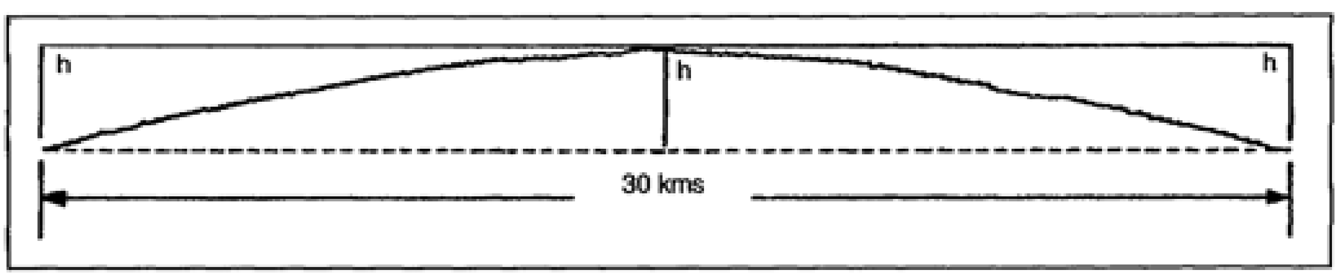


Figure 2-6. Signal Heights for a Line Over Water

Example 2. Figure 2-7 shows an example of two end stations not at the same elevation, with the obstruction not centered at a point midway between them. When applying the formula, first determine the amount of curvature at the obstruction at

B, and add this to the true elevation of B above the chord ac. *This is the effective elevation.* This is done using the following formula:

$h = 0.0676d1d2$
$h = 0.0676 \times 4 \times 6$
$h = 1.62$ *kilometers*

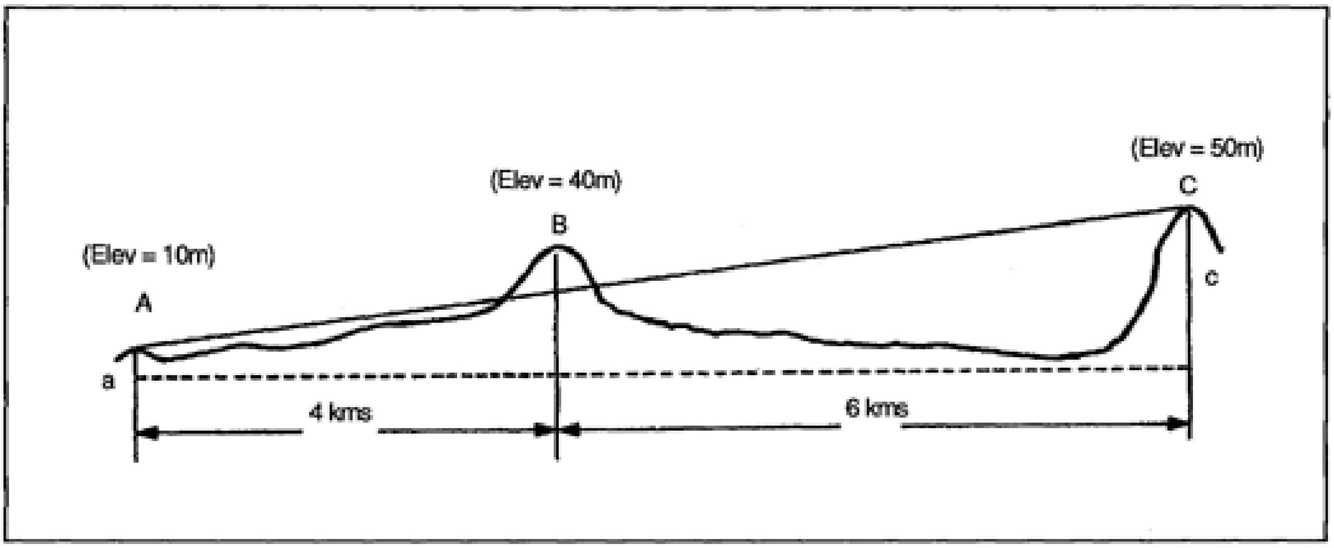


Figure 2-7. The Effect of Curvature at an Intermediate Obstruction

The effective elevation of B is *40 + 1.62 = 41.62 meters*. To perform the next step, compute the effective elevation of the grade line from the ground at A to the ground at C where it passes through the vertical of B. Use the principle of similar triangles as follows:

$$h = h1 + (h2 - h1) - 0.0676d1d2$$

$$h = 10 + (50 - 10\ )\frac{4}{4+6} - 0.0676(4)(6)$$

$$h = 10 + (40)x4 - 1.62$$

$$h = 10 + 16 - 1.62 = 24.38 \text{ meters}$$

Therefore, the amount of obstruction at B is *41.62 - 24.38 = 17.24 meters.*

f. The computation of obstruction is essentially the solution of similar triangles. In a), Figure 2-8, the elevations of the points are *A = 293 meters*, *B = 298* meters, and obstruction *H = 294 meters*. The distance from A to H is 8 kilometers and from B to H is 6 kilometers. Refer to Table 2-1, page 2-31, and locate the effective elevations of points H and B. The effective elevation of H is *294.0 - 4.3 = 289.7 meters* and B is *298.0 - 13.2 = 284.8 meters*.

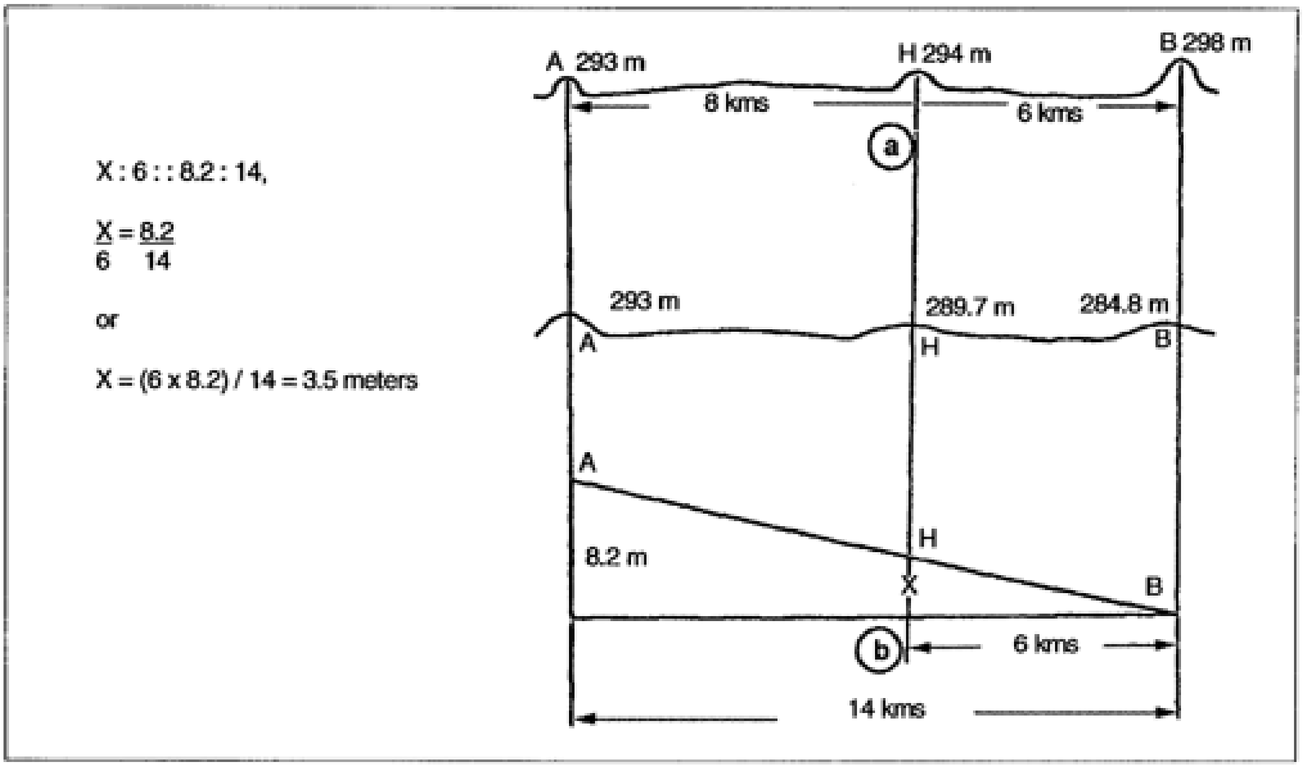


Figure 2-8. Similar Triangles Solution

(1) The effective elevation of the line at observation point H is now computed. In Figure 2-8, form the similar triangles using the effective elevation of B and H. Solving for X, we arrive at the formula shown on the left side of the figure.

(2) The effective elevation of the line at obstruction H is *284.8 + 3.5 = 288.3 meters*. The effective elevation of H is 289.7 meters, and the effective elevation of the line at H is 288.3; hence *288.3 -289.7 = -1.4 meters*. The line fails to clear the obstruction at H by 1.4 meters.

g. Some lines are affected by abnormal horizontal refraction. Where such lines are involved, you can only obtain good closures in the angles of triangles or distances measured by electronic equipment by use undue effort and expense, if at all. Therefore, the reconnaissance party should try to avoid lines likely to give refractive trouble. Layers or currents of unequally heated air along the line of sight cause horizontal refraction. Varying air densities produce a condition similar to that of a beam of light passing through a series of very flat prisms. Lines passing near the base of a mountain range or bluff may be affected by air currents flowing down side canyons and ravines (Figure 2-9). The kind of cover or vegetation, the range in temperatures between day and night, the direction and velocity of the wind, and the humidity are important factors.

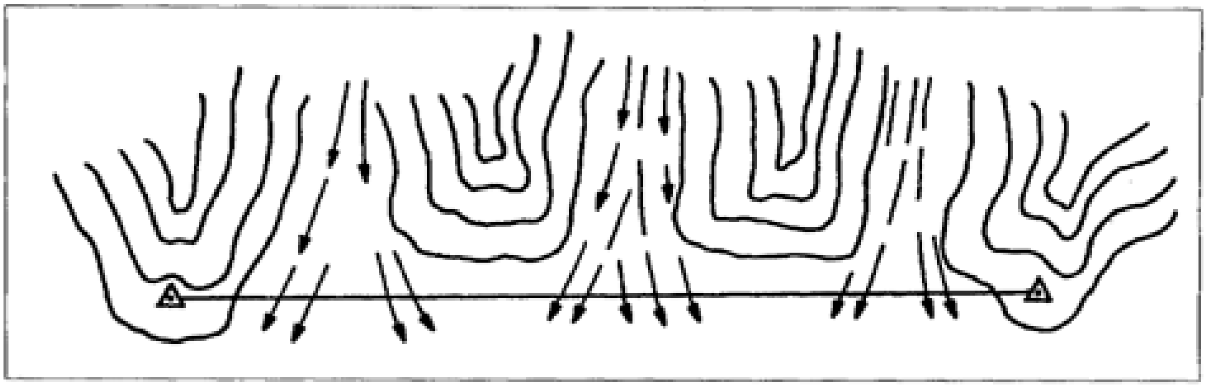


Figure 2-9. Horizontal Refraction Due to Air Currents From Ravines

h. Another example of serious refraction trouble is a valley through an open plain, bordered by bluffs. When it is impossible to avoid such a condition, place the stations on the tablelands as far away as possible (Figure 2-10). Lines between the headlands and parallel to the valley will give the least accurate results and should be avoided at the expense of additional signal building and stations.

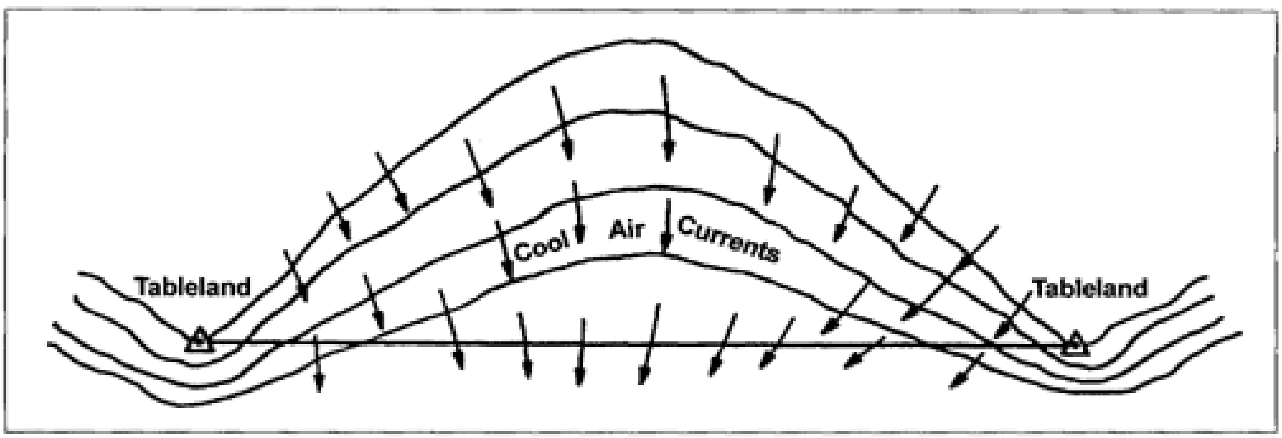


Figure 2-10. Horizontal Refraction Due to Air Currents Down a Slope or Hill

i. Horizontal refraction is much greater in barren or open country than in built-up areas. The effects of air currents are small over heavy timber. The reconnaissance chief may ignore the refraction effects in such areas.

j. On calm nights, the atmosphere tends to stratify into layers of different temperatures and densities, and the coefficient of vertical refraction becomes larger than normal, lifting distant objects above the horizontal. This abnormal nighttime refraction may vary between 3 to 5 minutes. A refraction line is a line that is normally obstructed but becomes visible at night. Directions observed over refraction lines are not unduly affected horizontally, and their use is sometimes permissible, though as a last resort. Stormy or windy nights may break up the air strata, and the abnormal vertical refraction may disappear entirely. Over sloping ground, the air strata may be inclined, in which case there will be a horizontal component of the refraction. Lines should always be kept clear of bare ground.

2-54. Height of Obstructions. It is often necessary to locate stations so that lines of sight will miss obstructions requiring signals to impractical heights. Blocks of higher timber will often be seen in heavily wooded areas. Plot the blocks on the map and ensure that all lines miss them. Intersections from water tanks, section corners, and other established stations give the positions of the obstructions. The heights of trees must be measured in wooded areas. The simplest and most accurate measurement is to lower a tape from the top of a representative tree. There are also various instrumental and improvised methods using an Abney hand level, a celluloid triangle, or a theodolite.

2-55. Clearance of Lines. The distance by which lines of sight must clear obstructions in order to avoid excessive refraction and dispersion of light varies with the type of vegetation cover and the other physical conditions of the line. The determining factor seems to be the amount of heat transferred to the atmosphere by the ground over which the line passes. Regions combining a bare ground surface with a large daily temperature range require the greatest amount of clearance, and areas heavily timbered and with a humid climate require the least.

a. Determining the amount of clearance required for a line is largely a matter of experience. The following information shows the average minimum values:

- Over water surfaces - 3 meters.

- Over open plains where the sun is hot during the day and the atmosphere is dry - 9 to 12 meters.

- Over cultivated land interspersed with wooded areas - 4.5 to 6 meters.

- Over treetops - 3 meters.

b. If the obstruction is a narrow ridge with relatively small capacity for heat radiation, the clearances may be safely reduced. If the line is parallel to the ground nearly the entire distance between stations, the clearance may have to be increased. Extreme heat and drought present special problems. The usual solution under unfavorable conditions is to specify the highest signal that the triangulation party is equipped to build economically.

2-56. Final Selection of Station Sites. The final selection of a station site requires a compromise of several requirements--the intervisibility of stations, the permanence of marks, the strength of figures, the wishes of property owners, and accessibility.

a. Intervisibility of stations is a very important requirement, along with the topics of curvature, refraction, and clearance. The possibility of securing visibility with a signal building at optional locations should also be investigated. The essential lines must be clear--this condition must govern the selected site.

b. With regard to permanence of monuments, no one can accurately predict what changes will occur at a given site over a period of years, but it should be taken into consideration during the site selection. You can be certain of a few things. All main roads will be widened, the grades will be reduced, and the curves will be eased. Most cities will continue to expand. Suburban areas will be enlarged, and residences will be built on what is now farmland, prairie, or wooded areas. All main-road intersections are desirable sites for gasoline stations and other establishments catering to the motoring public and should be avoided as station sites.

c. The right-of-way fence of an established highway is a good location for a station site, provided the road is full width. Since triangulation stations are naturally placed on high ground through which the highway may pass in a cut, it is necessary to keep back from the centerline. On the other hand, most railroad property is fairly well established and the right-of-way lines are permanent with the exception of abandoned branch and electric lines.

d. Cultivated land provides a fairly safe location but involves probable damages to crops each time the monument is used and precludes the use of a surface monument. Some landowners will permit the use of a surface monument on cultivated land. Never take advantage of such permission--at a later time, new owners may think differently. Line fences between farms are often good sites for stations, and signals can easily be built over them. Fences between fields are seldom permanent and are frequently removed for tractor cultivation.

e. In intensively cultivated sections, some of the best sites are in the vicinity of farm buildings or in groves maintained for shade and wind breaks. The buildings are generally located on knolls or fairly high ground. Public parks and buildings and the grounds of city and consolidated schools are generally in permanent locations.

f. In mountainous country, the natural physical conditions usually determine the permanence of monuments. These conditions include the quality of the rock, the amount of frost, and the rapidity of erosion. The tops of sand hills are often blown away, or blow pits, which may be started by the monument itself, may engulf the monuments. If it not possible to avoid such localities, use a long iron pipe monument.

g. During the final selection of station sites, it may be necessary to accept a weaker scheme in order to obtain a better station site, or vice versa. Good station sites are important, but the strength of figure must be maintained within the prescribed limits.

h. Accessibility is important both for the party establishing the station and all subsequent users. Unfortunately, the most accessible sites are often the most exposed, and the ease of access is less important than other conditions affecting the probable life of the monument.

i. The reconnaissance party assigns a name to each station selected in order to identify it and to aid in its recovery. Although the triangulation party is authorized to change the name, it seldom does. Ensure that a suitable name is selected, preferably a short one matching a geographical feature or political subdivision at or near the station. Never use duplicate names within the same triangulation system. In many incidences, the name of the landowner is given to the station. This normally stimulates interest on the part of the owner, helps to avoid duplication, and serves to locate the station quickly. Avoid using the names of men in the party, nicknames, names without meaning, or names arising from the incident. Ensure the correct spelling of the name.

PART M - SIGNALS AND TOWERS

2-57. Marking. Mark each triangulation station in some manner so that, as the survey progresses, an accurate triangulation net will be obtained. Signals must be visible from nearby stations.

a. If a station is one in which observations are taken, but is not occupied for the measurement of angles, it is not necessary to make provisions for setting the instrument. In this case, use a simple and inexpensive signal structure, such as the following:

- An object already in place, such as a flagpole, a chimney, or a telegraph pole.
- A pole set vertically in the ground or held firmly in a vertical position by a pile of stones, guy wires, or bracing (an excellent signal on a bare summit or in open country).

b. When taking angular observations at an instrument station, construct a signal with the instrument which has been placed directly over the station.

c. If a station will be used for any length of time, drive a pipe into the ground to mark the location. If the point is being used for a sight, a range pole or similar rod may be placed in the pipe. When the station is being occupied, the pole should be removed.

d. When a tall mast is necessary for visibility, use guy wires to anchor the sight. Provisions should be made for swinging the bottom of the mast to one side when placing an instrument at the station.

2-58. Signal Lamp. The most commonly used signal lamp has a 5-inch reflector. When used for sightings of less than 8 kilometers, the face of the light must be masked to reduce its size and brilliance.

a. Reflector. The reflector is frailly constructed and must be handled carefully to prevent tarnishing and scratching. It should be polished with a soft chamois skin or tissue, using only cleaning preparations recommended for this purpose. For a moderate cost, reflectors can be resilvered or replated with chromium. Typically, a lamp in service for a year will require resilvering.

b. Sighting Device. Signal lamps have a front sight on the reflector, in back of the lens frame and a notched rear sight fastened to a bracket on the rear of the reflector. The rear sight is adjustable both horizontally and vertically by means of knurled thumbscrews.

c. Focusing Device. The focusing device is a small knurled thumbscrew about 1/2 inch to the right of the rear of the bulb socket. This screw passes through a collar that encircles the bulb socket. Turning this screw in changes the position of the bulb in relation to the reflector.

d. Collar Wear. When a lamp is old, it frequently will not focus well because of collar wear. Repair the lamp by placing a nut or ring underneath the collar to make it longer. A lamp should be torn down and this focusing device examined to get a proper understanding of its operation.

e. Battery Connections. When the lamp is properly pointed and focused, the brilliancy of the light is dependant on the following three factors: the number of dry cells used, the method of connecting them, and the kind of bulb used.

(1) Figure 2-11 shows several cells connected in a series. The carbon pole of one cell is connected to the zinc pole of the next. When connected, each added cell increases the voltage. The total voltage is the sum of the voltages for the individual cells. There is a proper voltage for each lamp bulb. If the voltage is too low, a dim light will result; if the voltage is too high, the bulb will burn out. The total amperage is the ampere output of one cell. In a series circuit, the amperage does not increase with additional cells.

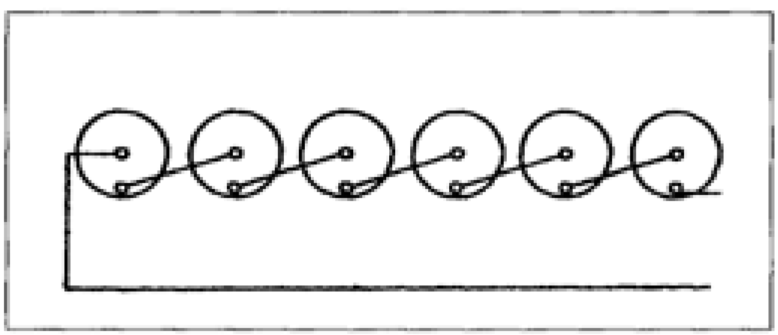


Figure 2-11. Cells Connected in a Series

(2) Figure 2-12, page 2-40, shows cells connected in parallel, with all carbon poles and zinc poles joined together. This method increases the amperage of the circuit, but the voltage is the same as that of a single cell. In this case, the six batteries act as one battery, with the same voltage as the individual cell but with amperage six times larger, thereby increasing the number of hours the lamp stays lit.

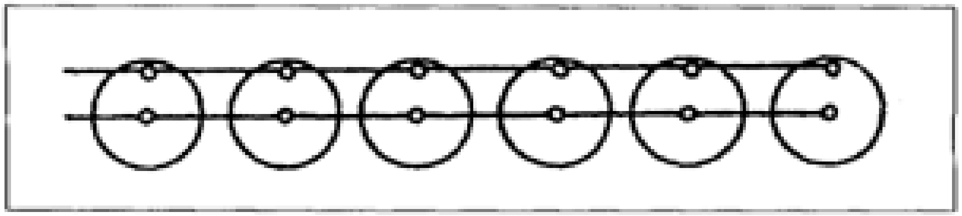


Figure 2-12. Cells Connected in Parallel

(3) When showing from three to six lights, the hookup ordinarily used is a set of 16 cells with four rows of four in parallel. The voltage may then be varied by using from one to four rows. When showing five or more lights over a period of 6 hours or more, the set of 16 cells should be backed up with a second set of 16 in parallel.

(4) The most commonly used low-amperage bulb is rated at 0.6 ampere and 3.7 volts. This bulb is requisitioned through supply channels and requires that the amperage and voltage be marked on the base. When it is used in a signal lamp with a 5-inch reflector, it gives good results on lines around 25 miles in length. If it is necessary to have a brighter light due to a hazy atmosphere, use a 1.25-ampere bulb. In ordinary cases with a small lamp and a 0.6-ampere bulb, two units of new cells, each unit containing three cells connected in parallel, will provide sufficient light 6 hours a day for 10 days (Figure 2-13). Although rated at 3.7 volts, these bulbs will withstand 4.5 volts. Three units of new cells connected in series give a voltage of about 4.5 volts, since the voltage of an ordinary cell (or of a unit of two or more cells connected in parallel) is about 1.5 volts. Four units of new cells would have a voltage of 6, which would burn out a bulb of this type.

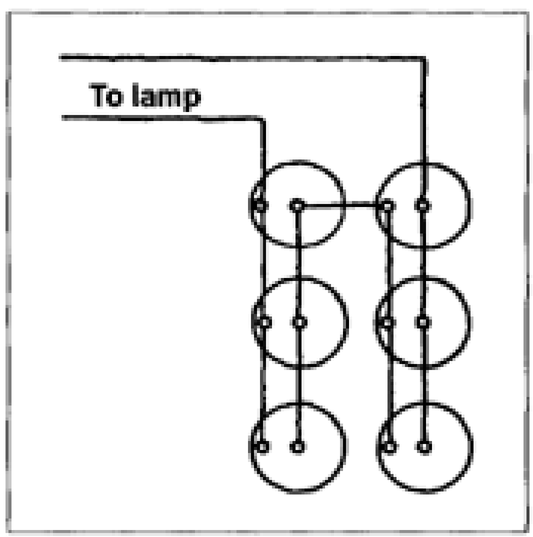


Figure 2-13. Series-Parallel Connection of Six Cells in Two Units of Three Cells Each

(5) After a period of use, the voltage supplied by the two units of three cells each decreases, and the cells no longer furnish a satisfactory light. A third unit and later a fourth unit should be added (Figures 2-14 and 2-15). When the four units no longer provide a sufficiently brilliant light, the two oldest units should be removed and replaced. Be careful not to connect too many cells and provide more voltage than the bulb can stand. After a period of rest, the removed cells may be used again with others of about the same degree of exhaustion. Very weak cells should not be used in combination with new ones.

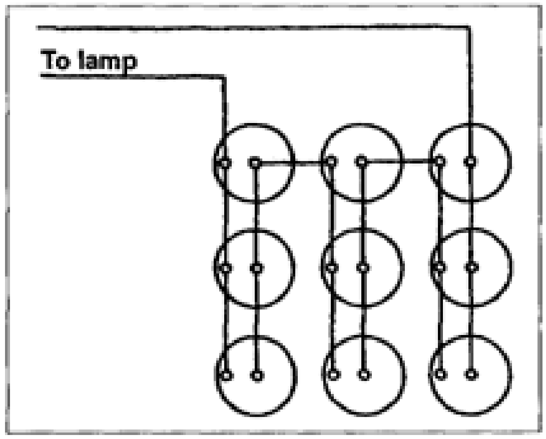


Figure 2-14. Series-Parallel Connection of Nine Cells in Three Units of Three Cells Each

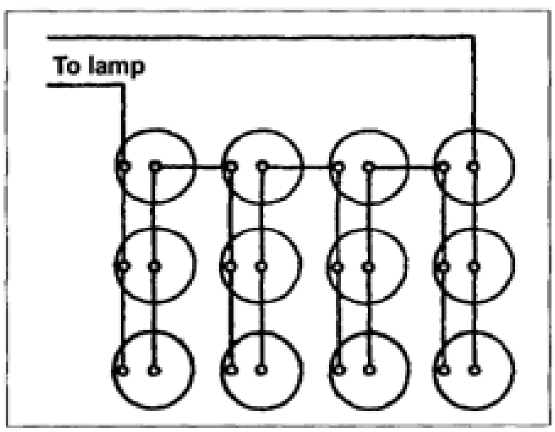


Figure 2-15. Series-Parallel Connection of Twelve Cells in Four Units of Three Cells Each

(6) Supply also furnishes a high-amperage bulb rated at 1.25 amperes and 3.7 volts. This bulb is the same size as the 0.6-ampere bulbs and can be used with the same battery. It is to be used in cases where the 0.6-ampere bulbs will not give a sufficiently powerful light. This bulb draws twice as much current as the 0.6-ampere bulbs and should not be used where the other will be satisfactory. When cells are new, two units connected in series should supply the 1.25-ampere bulb, with each unit consisting of three or four cells connected in parallel. Extra units should be

added as the cells weaken. Three units of new cells in series will give as great a voltage as the bulb will stand, since it has the same voltage rating as the 0.6-ampere bulb.

(7) Supply will also furnish, in special cases, a high-voltage bulb rated at 2 amperes and from 6 to 8 volts. This bulb consumes such a large amount of current that it is not economical or practical, except in cases where the observer desires not only a brilliant light but also a light with its beam slightly diffused. With fresh cells connected in five units of four cells each, the 2-ampere, 6-volt bulb should burn well for about 20 hours, after which its brilliancy will decrease noticeably. Add other units, one at a time, as needed. When eight units do not provide the required amount of light, dispose of the older cells and use fresh units. A comparison of the number of cells used with this bulb and with the smaller bulbs shows why this lamp is not practicable. It gives a beam only slightly more intense, but it requires a large number of cells, which in undesirable when weight is a serious consideration.

f. Testing Cells. Use a pocket ammeter to test the cells as they are used. Cells showing no energy should be thrown away. In general, cells of less than 4 or 5 amperes are of no use. Cold cells never test well and will show a poor light. Warm cold cells and test them before throwing them away. Freezing permanently injures dry cells.

g. Lamp Adjustments. Aside from the electric connections, there are only two adjustments needed for any of the lamps--one for focus and the other for the sighting device. These adjustments should be checked frequently.

(1) Properly focus the lamp at all times, regardless of how brilliant the filament of the bulb may be. The light will not be effective at any distance if it is not correctly focused. Each bulb will require a slightly different focus. This is true even of the same kind of bulbs of apparently the same size, because the position of the filament relative to the base of the bulb is rarely the same. Therefore, focus the lamp every time the bulb is changed and refocus after the lamp is transported since the vibration will likely cause a change. Make a focusing adjustment by turning the screw socket into which the bulb fits.

- Focus the lamp at night by directing the light upon a flat surface, such a tarpaulin or tent, about 100 feet away and turning the adjusting screw until the brightest part of the disk is a little larger than the lens of the lamp. As much light as possible should be concentrated within the area.

- Focus the lamp during the day by standing about 100 feet away from the lamp, with your eyes in the path of the beam. Have someone turn the focusing screw back and forth until the point is found where the light is the brightest, there are no black rings or spots on the reflector, and the spread of the bright beam is a little more than 1 foot (as found when moving the eye up and down and sidewise in the beam's path).

(2) When adjusting the lamp's height, point the center of the most brilliant part of the beam to the observer. If the sights are used in pointing, they must be parallel to the light beam. To adjust the sights by night, point the light to some object near enough to outline the central bright beam. After loosening the knurled nut, which holds the rear sight bracket, adjust it so that the sights point to a spot as far above the center of the beam as the sights above the center of the reflector. Tighten the nuts to hold the bracket and sight in that position. To adjust the sights during the day, place a stake in the ground in the path of the light and make a mark on the stake at the point where the reflector shows the brightest. This point may be found by moving the eye up and down and sidewise just back from the stake, in the same manner as when focusing the light. Adjust the rear sight so that the sights point to a spot the same distance above the mark on the stake as the sights are above the center of the reflector. Adjustments for the focus and sights may be made at the same time.

(3) Ensure that the adjustments are properly made, and the bright beam of the light goes to the observer (if the sights are pointed at him). At night, the pointing may be made over the observer's light by sighting accurately along the beam from directly above it for line measurements and from the side for elevation measurements. A light not properly pointed may either be invisible to the observer or may cause errors in the observing that may not be detected until all of the stations of the triangle have been occupied. This could cause great delay and expense.

(4) Be sure to point the light accurately. If at any time there is reason to believe that you may have disturbed the pointing of the lights, make the necessary adjustments, starting with the bottom light. Repoint accurately, both horizontally and vertically. Watch the pointings closely on windy nights, as a violent gust of wind could cause changes.

h. Lamp Condition. Keep the lamps in good condition and the reflectors polished. A dull reflector may result in a misread message. Never use a heater of any kind on the tower or stand, as heat waves from the heater will cause the light to appear wavy to the observer. Never light a lantern or fire around the tower or stand while observing, as the light may easily be mistaken by the observer as a signal light.

2-59. Wooden Signal Stands. Wherever elevation and unobstructed lines of sight permit, 4-foot signal stands are used to provide stability of signals for short-line triangulation and mountain areas. They are easily constructed in a minimum amount of time, easily transported to stations difficult to reach, and stable for observations. The design and construction details for a typical 4-foot stand are shown in Figure 2-16, page 2-44.

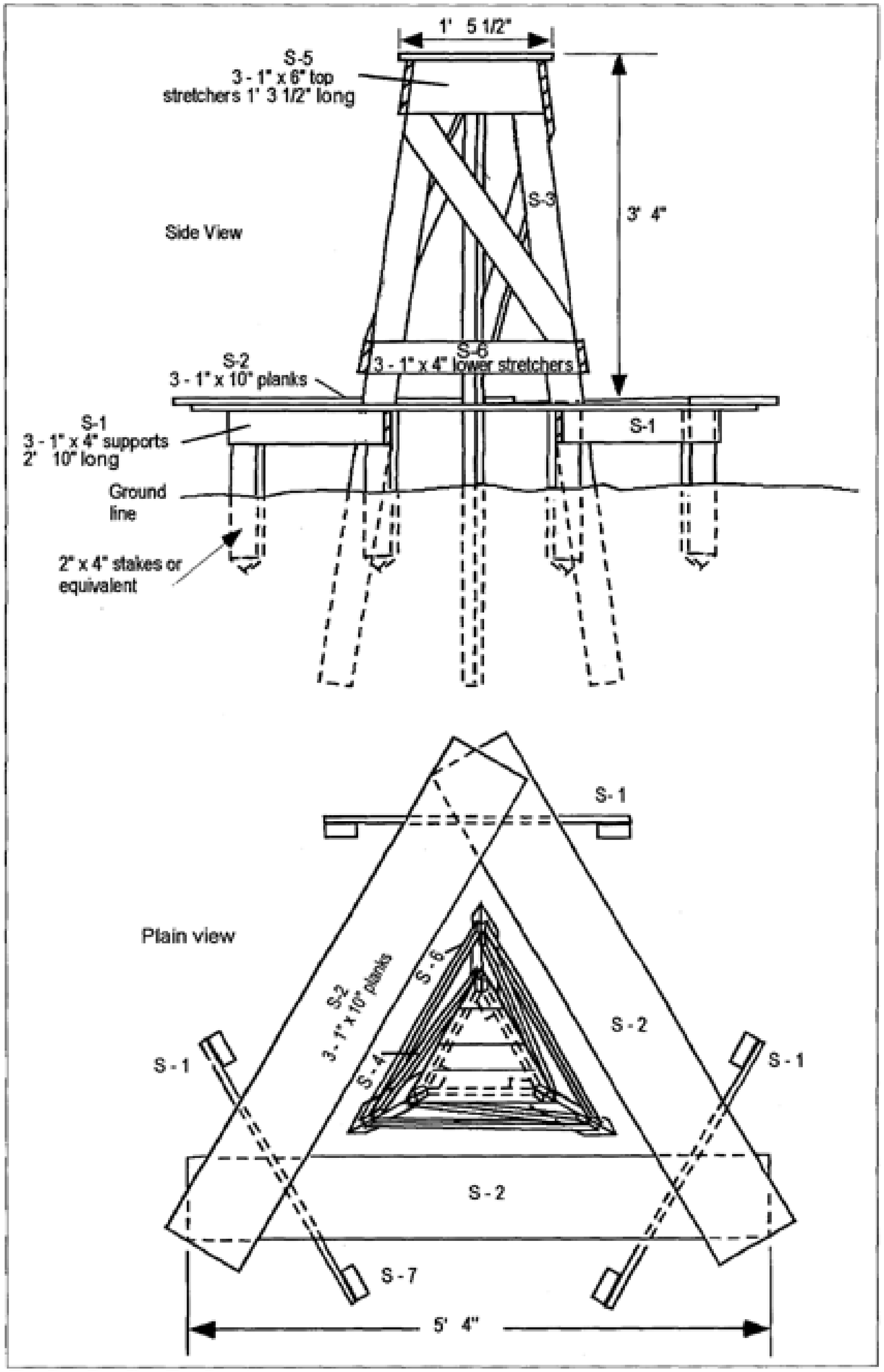
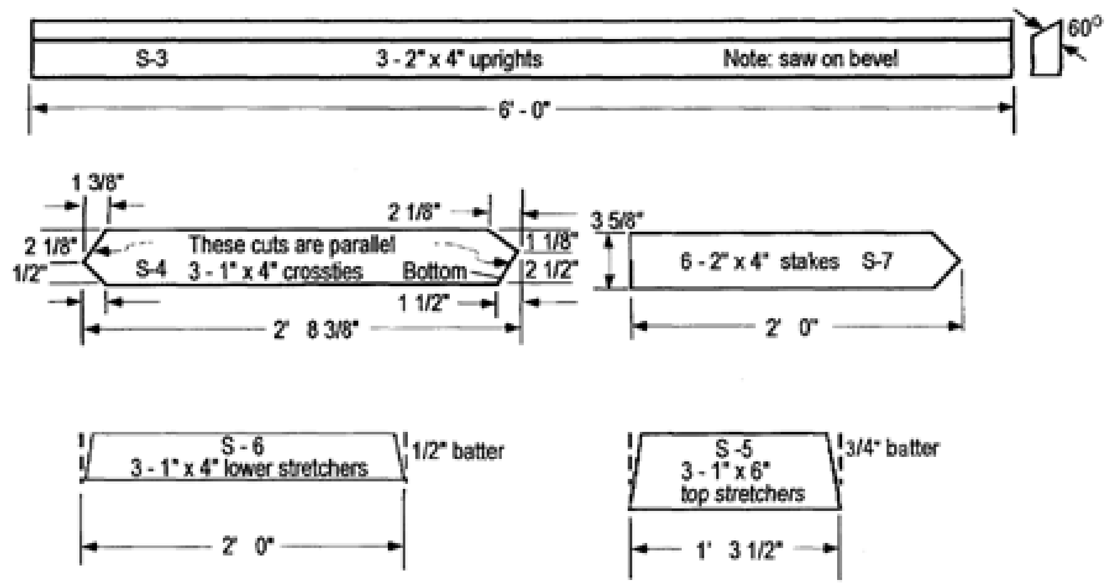


Figure 2-16. Construction Details for a 4-Foot Stand



Details

Material List

Note: Materials for one stand to be bundled.

No. of pieces		Length	Nominal	Length of stock required
S-1	3 pieces for supports	2' 10"	1" x 4"	1 piece 10'
S-2	3 platform planks	5' 4"	1" x 10"	3 pieces 6'
S-3	3 uprights	6' 0"	2" x 4"	3 pieces 6'
S-4	3 cross ties	2' 8 3/8"	1" x 4"	1 piece 10'
S-5	3 top stretchers	1' 3 1/2"	1" x 6"	1 piece 10'
S-6	3 lower stretchers	2' 0"	1" x 4"	1 piece 6'
S-7	6 stakes	2' 0"	2" x 4"	2 pieces 6'
S-8	1 board	4' 0"	1" x 4"	1 piece 4'

Figure 2-16. Construction Details for a 4-Foot Stand (continued)

a. Four-foot stands are tripods that are usually built of standard commercial sizes of lumber. The 2-inch by 4-inch legs on the low stands can often be purchased from the mill with a 60° bevel lengthwise along one edge, or 2-inch by 6-inch legs may be ripped or cut lengthwise at a 60° angle into two boards of an equal cross section. This helps to make a better nailing surface and a more stable stand. It is often advantageous to draw a pattern and cut the lumber to standard size in the base camp. Four-foot stands can often be made in advance as available men and the weather permit.

b. Dig holes for the legs of the tripod stand whenever possible. The tops of these holes should be left open to avoid transmitting surface disturbances to the stand. The stand should be securely staked in place.

c. Place footboards or a platform for the observer around the stand. This should be supported as far as practicable from the legs of the instrument stand.

2-60. Wooden Towers. When it becomes necessary to raise the line of sight to clear obstructions on longer lines, a stable tower must be built. It is necessary to construct an instrument stand and a separate platform to support the observer. The whole structure is referred to as a tower. Wooden towers are built at the station, using lumber or natural forest materials.

a. Towers should range between 6 and 25 feet high. It is not considered practical to build wooden towers higher than 25 feet. When greater heights are needed, aluminum or steel towers are used. The materials normally on hand and the easiest to handle are 2-inch x 4-inch x 16-foot, 1-inch x 4-inch x 16-foot, and 1-inch x 12-inch x 16-foot boards. For towers over 15 feet, 2-inch x 4-inch boards are usually doubled on the lower two-thirds or so of the tower's legs. The inner or instrument tower is built as a tripod, usually completed on the ground and set up in leg holes as one piece. The scaffold supporting the observer's platform is usually four legged, with two opposite sides put together on the ground and stood up around the tripod. All legs are laid out and marked on the ground. One side each of the tripod and the scaffold is used as a pattern to cut the horizontal ties and diagonals the same for all sides. The instrument tripod is not guyed during observation, but it may be weighted with rocks or other weights to increase its stability. In higher order work, no observations should be made on higher wooden towers during extremely high winds. However, the instrument tripod may be guyed when not in use to prevent damage from high winds.

b. The outer tower must be strong enough to be safe and must not touch the inner tower at any point. It should be provided with hand railings, as an observer often loses his sense of balance momentarily when taking his eyes away from the telescope. If necessary, the outer tower should be guy-wired at all four corners.

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LESSON 2

PRACTICE EXERCISE

The following items will test your grasp of the material covered in this lesson. There is only one correct answer for each item. When you complete the exercise, check your answer with the answer key that follows. If you answer any item incorrectly, study again that part which contains the portion involved.

1. Established points in geodetic surveys are referenced to as______________________________.

   A. Benchmarks
   B. The curved surface of a spheroid
   C. A plane surface
   D. Grid squares

2. Each survey has a fundamental classification of control points called______________________.

   A. Class
   B. Datum
   C. Order
   D. Network

3. To begin work on a net of triangulation, the ___________________ is required.

   A. Length of the starting line
   B. Azimuth of the baseline
   C. Astronomic coordinates of one end of the baseline
   D. Latitude and longitude of one end of the baseline

4. In first-order triangulation, Class I, the most accurate or most precise length of closure requires a ratio of 1 part in ____.

   A. 150,000
   B. 100,000
   C. 50,000
   D. 25,000

5. In Class I, first-order triangulation, the observation of station is normally performed ____.

   A. At night, using signal lights as targets
   B. At night, using the heliotrope or wooden targets as observation points
   C. At night, but may be performed in daylight using the heliotrope or signal lights as targets
   D. In daylight, using standard cloth or wooden targets

6. First-order triangulation requires that sets of observations made on different stations (normally 16 positions of the horizontal circle) be performed____.

A. Once a night for one night
B. Twice a night for two nights
C. Each of two nights, with a minimum of 2-hour separation between the two sets.
D. Each of two nights, with a minimum of 1-hour separation between the two sets.

7. The difference between the apparent and the true difference in the elevation of two points is affected by two factors--the curvature of the earth's surface and the ____.

A. Effect of gravity on the earth's surface
B. Refraction of light due to the earth's atmosphere
C. Distance above the earth's surface where the two points are located
D. Type of instrument used in observation between the two points

8. With respect to the curvature of the earth, the effect of refraction is about ______________as much and the __________________________ in sign.

A. One-fourth, same
B. One-fourth, opposite
C. One eighth, opposite
D. One-eighth, same

9. A refraction line is a line____.

A. Parallel to the horizontal axis of the instrument being used
B. Which conforms to the shape of the earth
C. That is normally obstructed but becomes visible at night
D. Perpendicular to the earth's surface

10. The determining factor for the amount by which the line of sight must clear the ground is the____.

A. Amount of heat transferred to the atmosphere by the ground
B. Height and number of obstructions in the line of sight
C. Direction and velocity of the wind
D. Viscosity or density of the vegetation directly below the line

11. The face of the 5-inch signal lamp must be masked when used for sightings of less than how many kilometers?

A. 8
B. 16
C. 30
D. 40

12. At what distance is a signal lamp focused when the light from the lamp is at its maximum concentration in a circle slightly larger than the lens of the lamp?

A. 25 feet
B. 75 feet
C. 100 feet
D. 150 feet

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LESSON 2

PRACTICE EXERCISE

ANSWER KEY AND FEEDBACK

Item		Correct Answer and Feedback
1.	B	The curved surface of a spheroid In geodetic surveys, all established...(page 2-4, para 2-3)
2.	D	Network Each survey has a fundamental...(page 2-7, para 2-9)
3.	A	Length of the starting line To begin work on a net of...(page 2-18, para 2-33b)
4.	B	100,000 Class I is the most precise...(page 2-18, para 2-34a)
5.	A	At night, using signal lights as targets Since first-order triangulation is...(page 2-19, para 2-34a[2])
6.	C	Each of two nights, with a minimum of 2-hour separation between the two sets. The rejection limit should...(page 2-19, para 2-34a[3])
7.	B	The refraction of light due to the earth's atmosphere The difference between the...(page 2-30, para 2-53a)
8.	C	One-eighth, opposite The effect of refraction...(page 2-30, para 2-53[b])
9.	C	That is normally obstructed but becomes visible at night A refraction line is...(page 2-35, para 2-53[j])
10.	A	Amount of heat transferred to the atmosphere by the ground The determining factor...(page 2-36, para 2-55)
11.	A	8 The most commonly used...(page 2-38, para 2-58)
12.	C	100 feet Focus the lamp at night...(page 2-42, para 2-58[1])

LESSON 3

HORIZONTAL MEASUREMENTS

OVERVIEW

LESSON DESCRIPTION:

In this lesson, you will learn to identify the different techniques of determining the traverse.

TERMINAL LEARNING OBJECTIVE:

ACTION: You will identify the different techniques of determining the traverse.

CONDITION: You will be given the material contained in this lesson, a number 2 pencil, and a calculator.

STANDARD: You will correctly answer all practice questions following this exercise.

REFERENCES: The material contained in this lesson was derived from FM 3-34.331.

INTRODUCTION

Geodetic triangulation is an efficient and accurate method of establishing control points over extensive areas of the earth's surface. In the past, as well as in many surveys today, triangulation and traverse have formed and do form the basic horizontal-control networks throughout the world. In geodetic triangulation, the survey and the computations start with a baseline whose length and azimuth are determined to a specified accuracy. This lesson will cover horizontal measurements in detail, to include methods of taking measurements, the types and characteristics of the instruments used and how they are read, and how the information is recorded in the field.

PART A - TRIANGULATION

3-1. Baseline Measurement. The measurement of an accurate baseline is necessary. Baselines are generally about one-sixth to one-fourth the length of the average line in a triangulation system. It is more important to be able to extend the baseline by well-conditioned figures than it is to have a baseline of great length.

a. A baseline, as used in triangulation, is the measured side of one of a series of connected triangles. Baselines are used as length control for triangulation, both as starting lines of known length and as check lines to ensure the required degree of accuracy of the triangulation. They are incorporated in the triangulation scheme to fulfill the requirements of the order and class of triangulation used.

b. Baseline measurements are conducted using methods designed to obtain high-accuracy readings. These methods require the use of standardized tapes with small coefficients of thermal expansion or high-precision, electronic-measuring devices; measurements with standard tape supports and tension; limiting conditions of alignment and grade; corrections for grade and temperature; and corrections for variations in alignment, tension, and support.

c. All first-order and second-order, Class I baselines must be accurately measured so that the computed probable error is not greater than 1 part in 1,000,000. Precautions used in the alignment, such as the lengths of marking tape, determination of grade corrections, tension, and temperature, should ensure that any errors in the length of the measured baseline does not exceed 1 part in 500,000. The total actual error must not exceed 1 part in 300,000. The actual error is the amount of deviation of each measurement from the mean of all measurements.

d. A baseline with the required degree of accuracy may be measured with tapes at any site where the grade does not exceed 10° and where there are no gullies, ravines, or similar obstacles over 50 meters wide to be crossed. It may be necessary to build stands or towers in rugged terrain to span ravines or reduce grades.

3-2. Tape Measurements. The principal baseline measuring tapes, called Invar™ tapes, are standardized 50-meter ribbon tapes made of nickel or steel. These tapes are used in sets of four, one of which is used as a standard for field comparison and substitute tape.

a. All base tapes should be standardized by the National Bureau of Standards (NBS) before the start of a project and immediately after the completion of the last baseline of the project. At the NBS, the 50-meter Invar tape length is compared with the US standard. The tape is placed, in a specified temperature, under a tension of 15 kilograms and supported at the 0-, 25-, and 50-meter points; the 0-, 12.5-, 37.5-, and 50-meter points; and on a flat, horizontal surface. The 30-meter steel tape, used for measuring short distances is standardized for each meter of tape on a flat, horizontal surface. The full-length value of the 50-meter tape for each of the three methods of support and the values of the meter intervals for the 30-meter tape described above are listed on certificates of standardization and returned with the tapes to the organization. Even though all baseline tapes are standardized in the field, make intercomparisons of the four tapes, using a test section at least four tape lengths long, immediately before and after measuring each baseline. The terminal points of the test section should be maintained while the baseline is measured. Use stakes for marking the test section; they should be braced in all four directions.

b. The spring balance used when stretching the tape should be tested and set to read correctly using the 15-kilogram test weight, as weighted at the NBS. Record this test before and after each day's work, at midday if possible, and more often if the temperature range is greater or if it is suspected that the position of the dial pointer has changed. There is an adjusting screw alongside the drawbar by which the dial can be reset with a screwdriver.

c. The special tape thermometers are tested at the NBS before being sent to the field to ensure that they are accurate to within 0.3° centigrade. Although no thermometer standardization correction is required in the field computations, use care in reading the thermometers to avoid thermometer lag, which is usually caused by rapid changes in temperature.

d. At least three tapes will be used in measuring each baseline. The portions measured with each tape should be about the same length. To accomplish this, divide the total length into 1-kilometer sections (one section may end up being longer or shorter). Group these sections in three divisions of about equal lengths. Each division must begin and end on either a kilometer mark or terminal station. Use a different pairing of the tapes on each of the three divisions in order to secure a complete intercomparison of the lengths. Run each tape forward on one division and backward on another. Only one forward and one backward measurement of each section should be made, unless the discrepancy between the two exceeds 10 millimeters $\sqrt{K}$ (where K is the length of the section in kilometers), in which case the measurements are repeated, preferably with the tapes originally used, until the forward and backward measurement of the section agrees within the limit.

3-3. Baseline Alignment. The alignment of the stakes (or tape ends) should be done with care so that the distance introduced does not exceed 1 part in 500,000. This requirement can be met by adhering to the following limitations:

- No marking strip or point marking a 50-meter tape end should be more than 2.5 centimeters out of line between the two adjacent marked points.
- The 2.5-centimeter tolerance should be decreased proportionally for distances less than 50 meters.
- No point between terminal stations should be more than 15 centimeters out of line.

A broken baseline (with more than one tangent) may be used where topographic conditions demand, provided the terminal stations are intervisible and the angles at each break and end station are measured to form a closed polygon. For all orders of baselines, no portion should deviate from the final projected line between terminal stations by more than 20° and, for first order, should be held to 12° or below when possible. The use of the broken baseline should be kept to a minimum, since it requires making angle measurements and adds another possibility for error.

3-4. Slope Measurements. The slope of a tape length in a baseline measurement should not exceed a 10-percent grade. Where this limit is approached or exceeded, use special care in determining the difference in the elevation of the tape ends or stands and performing horizontal taping. Measure the differences in elevation between the fiducial marks of the tapes using spirit levels. Obtain rod readings at all tape end supports and broken-grade intermediate supports. Read both the front (meter) and back (foot) sides of the rod at each point. If the available rods are graduated in only one unit, the spirit levels should run forward and backward over the line. Make a comparison of the observed differences of elevation for each tape length. Be careful when adjusting the level and when balancing the backsights and foresights for the observations. A maximum discrepancy between differences of elevation of ±3 millimeters should produce the desired 1/500,000 accuracy in the determination of the slope correction for slopes up to 10 percent.

3-5. Wind Effect. The error caused by the wind blowing the tape horizontally, such as errors in alignment, inclination, and friction of the tape on the supports, tends to make the measured length too long. No first-order or second-order, Class I baseline measurements should be made when the wind is strong enough to bend the tape more than 2 centimeters out of line. The wind would introduce an error of 1 part in 500,000.

3-6. Instruments and Appliances. It is possible to make satisfactory baseline measurements over somewhat rough and uneven ground if provisions are made to properly support and stretch the tape.

a. The following instruments and appliances are useful in running the baseline:

- Two marking scribes.
- Two pairs of dividers.
- One level, with rod.
- Two plumb bobs.
- Two range poles.
- Two 1/10-meter scales, boxwood, with a reading in millimeters.
- One stretch apparatus for tape, complete and consisting of two staves with loops and tape attaching the clip, two balances, and an apparatus for testing the balances.
- Copper strips for stake tops, of the same thickness as tape, 20 per kilometer.
- One 30-meter steel tape (standardized or tested in the field) for measuring setups and setbacks.
- One 50-meter steel or Invar tape (unstandardized) for marking out the baseline.

- Three Invar tapes (standardized).
- One theodolite.
- Three backed thermometers, for tapes.

b. The following tools and materials are used for staking a line:

- One hammer.
- One hatchet.
- One handsaw.
- Two mauls.
- Two machetes.
- 10d nails.
- Posts and stakes.
- Movable tripods.

3-7. Members of a Baseline Party. A baseline party should be staffed with six personnel to include the front contact man, the rear contact man, the recorder, the front stretcher man, the rear stretcher man, and the middleman. The party chief acts as the forward contact man unless other experienced personnel are available; otherwise, the party chief acts as the recorder. The general duties and responsibilities of the party members are briefly outlined below.

a. The front contact man ensures that conditions affecting the tape as a measuring unit are complied with. The tape must be in proper equilibrium and under proper tension and support before he makes the forward mark. He must also accurately read the front thermometer at each measurement.

b. The rear contact man makes the rear contact and reads the rear thermometer. As the tape is brought up to a new position, he steadies it as the rear staff places it in position and applies tension, ensuring that the tape does not drag over the rear stake. He advises the rear stretcher man to either ease off or take up the tape's tension.

c. The recorder should be an experienced man; otherwise, the party chief must frequently inspect the record, especially where broken grades or setups and setbacks are recorded.

(1) When the tape is advanced 50 meters, its forward end will occasionally fall short of the next metal strip. When this occurs, make a scratch on the rear metal

strip as close as possible to its forward end, and make rear contact at this point. Use a pair of dividers and a millimeter scale to measure the distance between the two scratches on the rear metal strip. Add this distance to the length of the baseline. This operation is known as *setting forward* or *setup*. The opposite situation occurs when the forward graduations overshoot their post. This operation is known as *setting backward* or *setback*.

(2) The recorder ensures that no blunders are committed, such as dropping or adding a tape length or recording a half-tape length as a full tape. He must check the chalked numbers on each stake as it is reached to ensure that he completes all the notes necessary for a definite and correct interpretation of the record. He must check the measurements of setups and setbacks and have the contact man check the entry in the record book.

d. The front stretcher man applies the proper tension to the tape by means of a tape stretcher with a spring balance attached to the forward end of the tape (Figure 3-1). A tape stretcher consists of two staves of steel tubing, pointed at the bottom, and wooden tops. A loose-fitting leather loop, with an attachment to receive the looped end of the tape, slips over the staff at the rear end of the tape. A frame for holding the spring balance is attached to the forward staff by means of a spring friction grip. In moving forward, the front stretcher man carries the front stretcher and balance, detached from the tape. As the tape is brought forward into position, he holds the spring balance hook in such a position that the tape can be quickly attached. As the tape is attached, he places the staff in line with the stakes, at the proper distance from the front stake, and applies the tension. The tension is applied rapidly at first and is increased gradually as he nears the 15-kilogram point. Under tension, the tape must clear the top of the forward stake and not drag over it; otherwise, the full tension will not be transmitted throughout the tape.

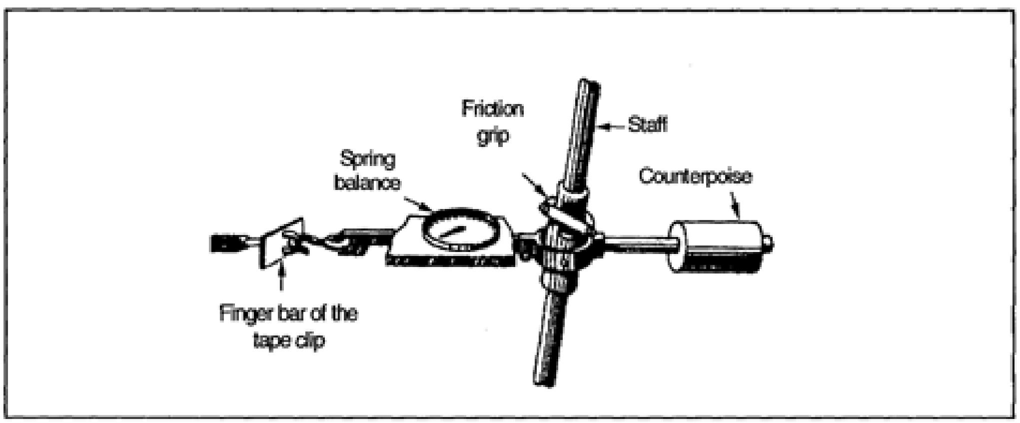


Figure 3-1. Tape Stretcher and Spring Balance

e. The rear stretcher man holds the rear stretcher in position while the tension is applied, so that the rear terminal mark on the tape is opposite to or slightly forward of the mark on the copper strip on the rear stake. He approaches the rear stake and places the staff firmly in the ground at the proper distance, directly in line with the other stakes, while he simultaneously slips on the leather loop to the proper height on the staff. To maintain steady pressure on the staff, the rear stretcher man should have the top back of the staff resting on one of his shoulders, with his body leaning forward. With full tension applied, the tape should be a few millimeters above the top of the stake.

f. The middleman is primarily concerned with the intermediate stakes along the baseline. He must move forward, carrying the tape, and place it on the middle support as tension is applied. If a nail is used as a middle support for the tape, he must rapidly and lightly tap the underside of the tape to reduce the friction at the moment of the reading. He must also ensure that the tape is not in contact with weeds, brush, or other obstructions and that the middle support is not more than 10 centimeters from the middle mark on the tape at the time of the reading.

The middleman also notifies the recorder of all middle supports marked *broken grade*. A broken grade is an intermediate support which is not on grade. If the intermediate support between the tape ends cannot be placed so that it will be down to grade with the terminal posts, it should be numbered with colored chalk and marked with a piece of cloth so that it is noticed and touched upon by the level man. All stakes should be numbered with colored crayon as they are driven. Intermediate supports above grade should be given a fractional number.

3-8. Run a Baseline. The top of a rail of a railroad track or the surface of a concrete roadway of uniform grade may be used for baseline measurements. However, these surfaces should not be used when their temperatures are appreciably different from that of the surrounding air.

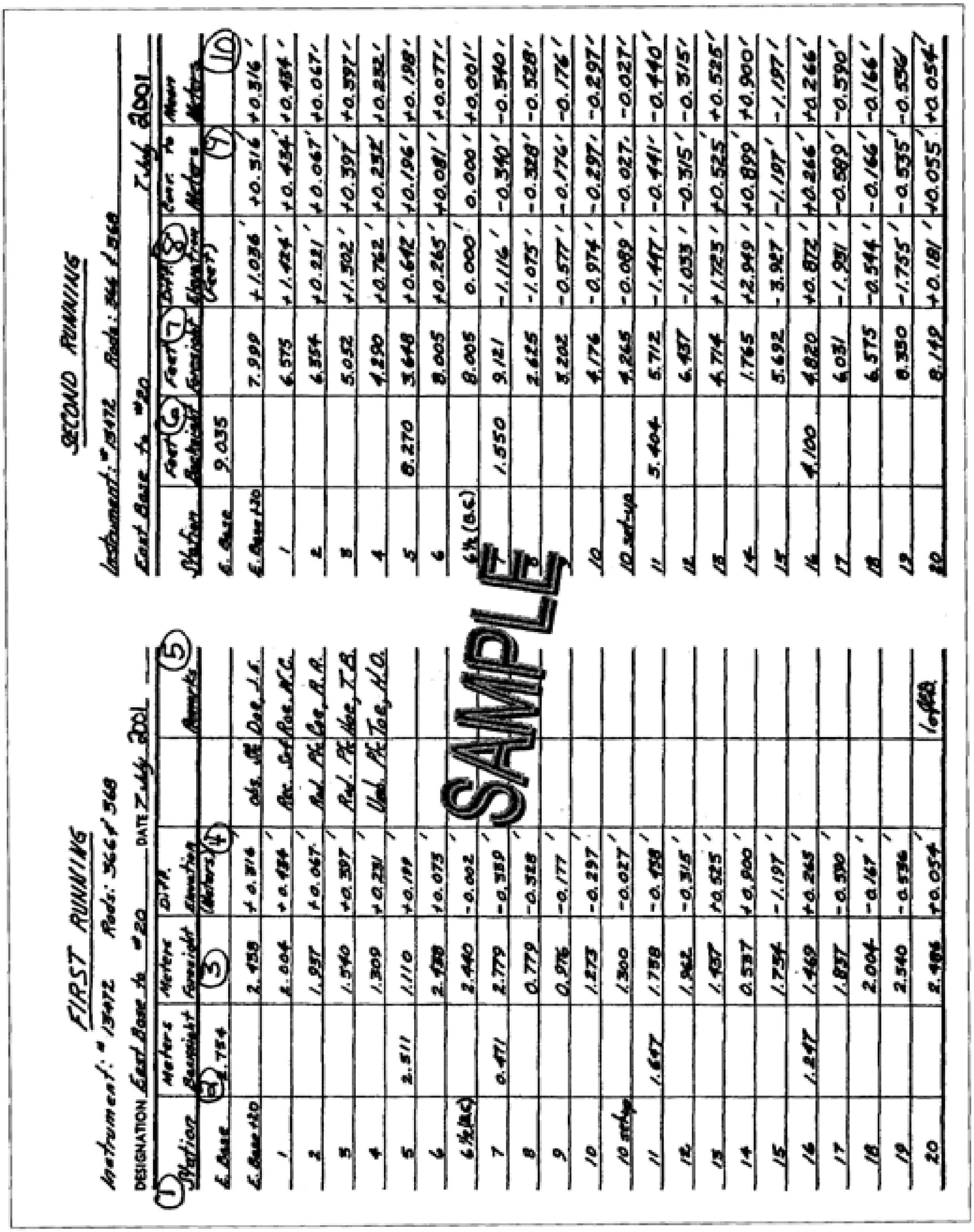
a. If the baseline is over uneven ground, provide supports for the tape using substantial posts, 2 by 4 inches or 4 by 4 inches, driven firmly in the ground. To run a baseline, the first task is to set these posts. Clear the baseline so that the ends are intervisible. Set up the theodolite at one end of the line, and set the sight (range pole) at the terminating end of the baseline. Carefully adjust the theodolite to the foresight. Drive the posts on the line, accurately spaced 50 meters between centers. The tops of the stakes should not be less than 10 inches or more than 24 inches above the surface of the ground and should be driven to a firm footing. Halfway (25 meters) between terminal posts, drive a 2- by 4-inch stake. Offset the face of the stake 1 inch from the line of sight. All stakes should be offset on the same side of the line of sight.

b. Sight along the tops of both of the 50-meter posts and locate and drive a fiducial nail horizontally into the face of the intermediate stake so that the nail is on line with the tops of the posts. These nails serve as intermediate supports between 50-meter posts during measurements. Nail a strip of copper or zinc on top of the 50-meter posts to provide a suitable surface on which to mark the tape lengths.

c. Ideal conditions may not be present in actual field practice. For example, at ravines and streams, it may be necessary to begin a tape length by placing a stake on the edge of a bank. If this stake is less than half a tape length from the previous tape end post, the number of which is 16, number the fractional-length post *16 setup*. Number the next terminal post *17*. If the fractional-length post is more than half a tape length from the next preceding tape end post, drive a 2- by 4-inch stake at the halfway mark and number it *16 1/2 setup*. Following this system will prevent uncertainty in the interpretation of the records.

d. With the reconnaissance made, the signal towers erected, and the station stakes set, it is time to measure the exact length of the baseline. Before doing this, run a line of levels over the line to determine the relative elevations of all 50-meter posts and broken-grade stakes.

3-9. Level the Baseline. The leveling sheet (DA Form 4446) in Figure 3-2 shows the level record when using a rod graduated in meters on one side and feet on the other. When using such a rod, run the levels only in one direction, but read both sides of the rod at each rod point. Record the reading in meters as the forward running and the reading in feet as the backward running. If the rod is graduated on only one side, make both forward and backward runnings. Be extremely careful to obtain readings on all broken grades and partial tape lengths, and plainly indicate the readings in the record. Mark the columns as either meters or feet, as applicable. Pay particular attention to the column headings in the level notes. Each item will eventually enter into the final calculations for the true length of the baseline. Refer to Figure 3-2 and the following information for required detailed column information:



FIRST RUNNING

Instrument: # 13472 Rods: 366 & 368

DESIGNATION East Base to #20 DATE 7 July 2001

1 Station	2 Meters Backsight	3 Meters Foresight	4 Diff. Elevation (Meters)		5 Remarks
E. Base	2.754				
E. Base +20		2.438	+0.316	Obs. Sfc	Doe, J.E.
1		2.004	+0.434	Rec. Sof	Roe, N.C.
2		1.937	+0.067	Rod. Pfc	Coe, R.R.
3		1.540	+0.397	Rod. Pfc	Hoe, T.B.
4		1.309	+0.231	Umb. Pfc	Toe, H.O.
5	2.511	1.110	+0.199		
6		2.438	+0.073		
6½ (BC)		2.440	-0.002		
7	0.471	2.779	-0.339		
8		0.779	-0.328		
9		0.976	-0.177		
10		1.273	-0.297		
10 set-up		1.300	-0.027		
11	1.647	1.758	-0.438		
12		1.962	-0.315		
13		1.437	+0.525		
14		0.537	+0.900		
15		1.734	-1.197		
16	1.247	1.469	+0.265		
17		1.837	-0.590		
18		2.004	-0.167		
19		2.540	-0.536		
20		2.486	+0.054		[illegible]

SAMPLE

SECOND RUNNING

Instrument: # 13472 Rods: 366 & 368

East Base to #20 7 July 2001

Station	6 Feet Backsight	7 Feet Foresight	8 Diff. Elevation (Feet)	9 Conv. to Meters	10 Mean Meters
E. Base	9.035				
E. Base +20		7.999	+1.036'	+0.316	+0.316'
1		6.575	+1.424'	+0.434'	+0.434'
2		6.354	+0.221'	+0.067'	+0.067'
3		5.052	+1.502'	+0.397'	+0.397'
4		4.290	+0.762'	+0.232'	+0.232'
5	8.270	3.648	+0.642'	+0.196'	+0.198'
6		8.005	+0.265'	+0.081'	+0.077'
6½ (B.C)		8.005	0.000'	0.000'	+0.001'
7	1.550	9.121	-1.116'	-0.340'	-0.340'
8		2.625	-1.075'	-0.328'	-0.328'
9		3.202	-0.577'	-0.176'	-0.176'
10		4.176	-0.974'	-0.297'	-0.297'
10 set-up		4.265	-0.089'	-0.027'	-0.027'
11	5.404	5.712	-1.447'	-0.441'	-0.440'
12		6.437	-1.033'	-0.315'	-0.315'
13		4.714	+1.723'	+0.525'	+0.525'
14		1.765	+2.949'	+0.899'	+0.900'
15		5.692	-3.927'	-1.197'	-1.197'
16	4.100	4.820	+0.872'	+0.266'	+0.266'
17		6.031	-1.981'	-0.589'	-0.590'
18		6.575	-0.544'	-0.166'	-0.166'
19		8.350	-1.755'	-0.535'	-0.536'
20		8.149	+0.181'	+0.055'	+0.054'

Figure 3-2. DA Form 4446 (Sample of Completed Leveling Notes, Precise Base)

- Column 1. Record the station number of each post set. This includes the stationing of all terminal end posts, intermediate posts, setups, setbacks, and broken grades.

- Column 2. Record all backsight shots with the reading from the meter side of the rod. A backsight reading indicates a change of position for the instrument. Try to balance the length of all backsights and foresights to reduce errors introduced by curvature and refraction.

- Column 3. Record the meter rod readings on all posts set on the baseline. Notice that readings are to the nearest 0.001 of a meter. Make the numbers clear and legible.

- Column 4. The value for each entry in this column is the difference between successive rod readings. For example, at station 4 the rod reading was 1.309 meters and at station 3 it was 1.540 meters. Therefore, the entry in column 4, opposite station 4 is *1.540 - 1.309*, or *0.231*. Since station 4 is higher than station 3 (indicated by a smaller rod reading at station 4), record it as *0.231*.

- Column 5. In the remarks section, list the name, rank, and position of each individual in the survey party. Also, add any other information pertinent to the survey, such as unusual wind conditions and temperature variations. The observer should initial the field notes after he has checked the recorder's figures.

- Column 6. Record all backsight shots from the reading on the foot side of the rod. Add this reading to the elevation of the known benchmark to establish the height of instrument (HI). You must know the HI in order to calculate the elevations of the posts set on the baseline (if available).

- Column 7. Record the foot rod readings on all posts on the baseline. Notice that the readings are rounded to the closest 0.001 of a foot.

- Column 8. The value for each entry in this column is the difference between successive rod readings. For example, at station 4 the rod reading was 4.290 feet and at station 3 it was 5.052 feet. Therefore, the entry in column 8, opposite station 4, is *5.052 - 4.290*, or *0.762*. Since station 4 is higher than station 3 (indicated by a smaller rod reading at station 4), record it as +*0.762*.

- Column 9. Take the difference in elevation in feet (column 8) and convert this figure to meters. This is done so that all figures in the final computation of the mean difference between the two runnings will be in the same unit of measure.

- Column 10. To obtain this value, take the entry in column 4 and column 9 and find their mean difference. For example, taken from station 4:

From column 9: *+0.232*
From column 4: *<u>+0.231</u>*
(+0.463)/2 = + 0.232 (mean difference)

Enter the following value in column 10, opposite station *4*: + *0.232*.

a. The field computations and entries for one complete station have been entered. Make the entries for all other stations in a similar manner. The surveyor should be primarily concerned with the data contained in columns 1, 2, 3, 6, and 7.

b. The object of profile leveling is to find the elevations of points at known distances apart and thus obtain the profile over a given line. The profile should show the elevations of the tops of all posts along the baseline, as they would be seen in from the side.

c. When a level is properly set up, the line of sight is perpendicular to the line of gravitation and revolves in a horizontal plane. The line of sight is the basis for determining elevations and establishing points.

d. In all surveys, the elevations are referred to some common datum that is designated as zero elevation. To obtain the elevation of any point not in the datum, add or subtract the point's vertical distance above or below that plane. When an elevation has not yet been established, an elevation (referred to as MSL, which is considered zero) simply assumes a datum and assigns an arbitrary elevation to the starting point. All level shots taken within the area are then relative to the arbitrary elevation assigned. At a later date, the assumed elevation of the starting point must be tied into a benchmark of known elevation.

e. Since the lines of a scheme of triangulation are reduced to their equivalent lengths at sea level, the length of any base must be likewise reduced to sea level before it can be used in adjusting the triangulation to which it is connected. This requires the connection of the baseline levels to a benchmark and the computation of the elevation above sea level of the tape supports in order to obtain a mean elevation for the base. This reduction to sea level is peculiar to triangulation work and is not necessary for ordinary level work.

3-10. Measure the Baseline. To accurately measure a baseline, all party members must function as a single unit at the precise moment of marking. The following procedures are recommended to obtain a successful measurement:

- The rear stretcher man, the middleman, and the front stretcher man are positioned as previously outlined in paragraph 3-7d and e, pages 3-6 and 3-7.

- The rear contact man stands directly opposite the mark, which is on the copper strip nailed to the top of the post. With one hand he firmly grasps the tape between the rear tape stretcher and the mark on the copper strip. With the other hand he lightly touches the tape on the opposite side of the stake to steady it. With the back of his hand, he brings the mark on the tape into coincidence with the mark on the copper strip and holds the marks in coincidence until the front contact man calls "Ready," the rear contact man calls "Right," and the front contact man answers, "Mark." Immediately following the call "mark," the rear contact man reads the thermometer.

- The recorder enters the reading in the record book.

- The front contact man assumes a position alongside the forward stake. With one hand he steadies the tape into its proper position just clear of the top of the stake, alongside the copper strip, and between himself and the strip. As the tension is perfected and the tape approaches equilibrium, he places the point of a sharp, symmetrically pointed scribe on the edge of the copper strip, next to the tape and in coincidence with the terminal mark on the tape. When he is satisfied that all conditions have been met, he checks the tension, glances down the tape to see that it is in alignment, and calls "Ready." When he hears the response "Right" from the rear contact man, he marks the copper strip with the scribe and calls, "Mark."

NOTE: Several precautions must be taken when making the mark. The scribe must be very sharp and should at no time touch the tape in the region of the terminal mark. The terminal mark of the tape, the axis of the scribe, and the eye of the man making the forward contact should be kept in about the same vertical plane. The contact man should mark the copper strip by moving the scribe away from him in order to keep constant any error due to parallax. The mark should be made at the very edge of the copper strip in order to make it easier for the rear contact man to make the contact when the tape is moved ahead.

- Immediately after making the mark, the front contact man reads the forward thermometer, and the recorder makes the notation in the record book.

NOTE: When recording the tape measures, two thermometer readings indicate a full 50-meter tape length, and one thermometer reading indicates a half-tape length or a setup. Each half-tape length or large setup should be recorded on a separate line, not on the same line as a full-tape length. The numbering of the stakes should plainly indicate the full-tape lengths and the partial lengths. Notes in the remarks column should clearly explain any unusual conditions. At the beginning of the day's work on the first page, and as often as changes occur, make entries in the remarks column giving the names and duties of the party chief, the recorder, and the two contact men. Also, a statement should be made as to the results of the comparison of the balances and the dial reading being used on the balances. All marginal notes and entries at the top of the page should be made as the measurement progresses. Inserting notes after leaving the site could result in errors.

a. A suggested method of keeping field notes for baseline measurements is shown in Figure 3-3, page 3-14. Complete the heading of the form in detail. The entries in columns 1 and 2 are the names of the stations and the numbers of the tape ends. Indicate a partial tape length on a separate line, using the number of the preceding tape end followed by the meters or the word *setup*. If a stake with a contact strip is set at the half-tape point and contact is made to it, designate it as a *+25 stake*.

b. In columns 3 and 4, the entry of two temperatures indicates that the measurement was made with a full 50-meter tape length; one thermometer reading indicates a half-tape length or a setup. In cases of partial tape lengths, include explanatory notes in the remarks column to indicate the part of the tape used in the measurement, such as 0 to 25, 25 to 50, and 0 to 10.

c. In columns 5 and 6, enter the setup and setback measurements, in meters, to 4 decimal places. These entries, in addition to various partial tape-length measurements, include rail-movement measurements (usually the mean of 10 per rail), and setup or setback measurements at the section ends and at intermediate stakes used to make tape ends fall on the copper strips. Enter the number of points of tape support in column 7. Enter a *T* when a tape is supported throughout. However, measurements are seldom made with a tape supported throughout because of friction.

d. Entries in the remarks column are very important for the proper interpretation of a record. Any unusual method of support should have an accompanying explanation in the remarks column. Note all broken grades. Indicate the part of the tape used in a partial tape-length measurement. Enter the number of the steel tape used opposite all measurements made using steel tape. Include remarks on any accidents to the tapes (noting the exact point of injury) or equipment and any weather changes.

e. When the forward mark has been accurately made, move the tape ahead and repeat the process until reaching the end of the baseline. If it is necessary to remeasure the baseline, the marks placed on the copper strips during the second measurement should be distinguished from the first markings by a bar across the scratch. Markings from a third measurement would have two scratches.

Forward Measurment Time : 0810

DESIGNATION East Base to Sta. #20 DATE 7 July 2001

Chief : SFC Doe J. Tape Nos 917 & 302

Thermo. Nos. 1518 & 73260 Weather: Warm, Clear & Calm

Station From	Station To	Temp Front °C	Temp Rear °C	Meter Set-up	Meter Set-back	Tape Support	Incl. Corr.	Temp. Corr.	Cal. Corr.	Remarks
E. Base	E. Base+20	20.0	—	20.0000		2				0-20
E. Base+20	1	21.3	21.0			3				
1	2	21.2	21.0			3				
2	3	20.6	20.0	0.0714		3				
3	4	21.2	21.0			3				
4	5	21.0	20.5			3				
5	6	20.0	19.8			3				
6	7	20.0	20.0		0.0214	3				B.G. at 6½
7	8	20.2	20.0			3				
8	9	20.4	20.5			3				
9	10	20.6	20.6			3				
10	10 Set-up	—	21.5	4.7000	0.0381	2				Steel tape 4150 Crossing Gully
10 Set-up	11	20.7	21.0			2				
11	12	20.8	21.0	0.0027		3				
12	13	20.8	21.0		0.0732	3				
13	14	21.0	21.0			3				
14	15	21.0	20.0			4				Supported at 0-12½-25-50
15	16	20.8	20.5			3				
16	17	20.5	20.3			3				
17	18	20.7	20.5			3				
18	19	20.8	20.4			3				
19	20	21.0	20.5			3				

Figure 3-3. DA Form 4446 (Sample of Completed Leveling Notes, Precise Taping)

PART B - MEASURING ANGLES

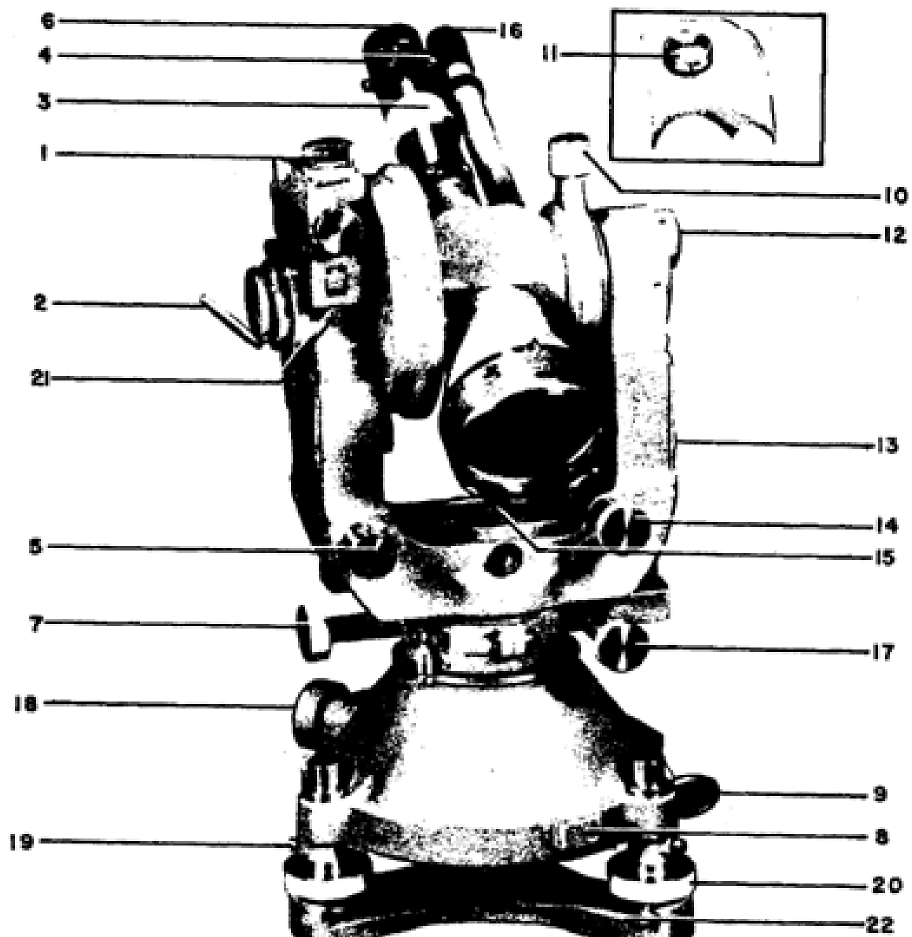
3-11. Triangulation. Triangulation affords a rapid and accurate method of surveying a large area by eliminating the necessity of measuring the length of every line. In any triangle, if we know the values of all three angles and the length of one side of the triangle, we can calculate the length of the other two sides. Having calculated the length of the unknown sides, we can use them as sides of known length for adjacent triangles. With this simple procedure, we could continue and calculate lengths throughout the entire series of connected triangles. However, we need not measure all three angles. The properties of a triangle are such that if we can measure any two angles, we can compute the value of the third.

a. This procedure does not guarantee that the calculated value of the unknown angle will be its true value. Any error in the original measurement will be reflected in the calculated angle; therefore, there would be no tangible reference for adjusting the triangulation on completion of the survey. Certain instruments and field techniques are employed which greatly reduce the possibility of error. Many of these techniques will be brought to your attention during the remainder of this lesson. Certain mathematical computations are applied to the data so that any error is distributed throughout the entire system. This is the primary task of the data computer. The persistent elimination of conditions likely to cause errors makes their occurrences rare and allows for isolation and immediate remedy.

b. Accuracy should be maintained throughout the triangulation. Use care when operating instruments, and pay strict attention to the methods. Observing the proper methods ensures that systematic and accidental errors are kept within the prescribed limits and that no part of the system exhibits undue weakness.

3-12. Theodolites. A theodolite is a precision surveying instrument used for the accurate measurement of angles. It consists of an alidade with a telescope. It is mounted on a base carrying an accurately graduated horizontal circle and equipped with necessary levels and reading devices. The alidade usually carries a graduated vertical circle. There are two general types of theodolites, repeating and direction. A repeating theodolite is designed so that successive measures of an angle may be accumulated on a graduated circle. The reading of the accumulated sum is divided by the number of repetitions to obtain the observed angle. On a direction theodolite, the circle remains fixed while the telescope is pointed on a number of signals in succession. The circle is read for each direction. The direction theodolite is the preferred instrument used in higher-order triangulation. The quality of a theodolite is not measured by its size or the minutes on the least reading of the micrometers, but by the beat measure of excellence in its performance in actual fieldwork.

3-13. Wild T-3 Precision Theodolite. Refer to Figure 3-4, page 3-16. The wild T-3 precision theodolite is a 0.2-second, prism microscope type of direction theodolite and is a satisfactory first-order instrument. The principal operating characteristic of this type of instrument is the method of reading the circles by means of an auxiliary telescope (microscope) alongside the sighting telescope. Both sides of the circle are reflected simultaneously in this reading microscope through a chain of prisms. A micrometer in this optical system is arranged so that movement of the micrometer screw brings the opposite sides of the circle into optical coincidence with the amount of movement read on the micrometer. Shifting the changeover knob on the side of the instrument allows either the vertical or horizontal circle to be read from the single microscope eyepiece. The vertical circle has a coincidence-type level.



1. Prism for altitude level
2. Illuminating mirror for vertical circle
3. Focusing ring for telescope
4. Adjusting screw for crosshair
5. Slow-motion screw for altitude level
6. Telescope eyepiece
7. Horizontal clamp
8. Fixing lugs for packing
9. Illuminating mirror horizontal circle
10. Vertical clamp
11. Knob for crosshair illumination (opposite side of telescope)
12. Micrometer milled head
13. Changeover knob
14. Vertical slow-motion screw
15. Plate spirit level
16. Reading eyepiece
17. Horizontal slow-motion screw
18. Milled head for circle setting
19. Adjusting screw for foot screw
20. Foot screw
21. Collimation level adjusting screw
22. Spring plate

Figure 3-4. Wild T-3 Precision Theodolite

a. Unpacking and Performing Instrument Setup.

(1) Erect the tripod over the station point, hang the plumb line by the hook on the central fixing screw, and center the tripod to 1 or 2 centimeters. For instrument stability, tread the feet of the tripod firmly into the ground. For precise measurements, see that both the tripod and the instrument are completely protected from the direct rays of the sun.

(2) Grasp the two ends of the leather strap of the instrument casing, and draw them forcibly outward so that the two lever fastenings open and the hood can be cautiously lifted and removed.

(3) Loosen the three black screws that hold the instrument to the bedplate of the casing and draw back the three slides. Lift the instrument using its two scope supports, and place it on the tripod in such a way that the illuminating mirror for the horizontal circle (9) can be turned toward the light. Screw the central fixing screw of the tripod into the footplate of the instrument and, after centering the instrument exactly over the point with the plumb line, tighten the fixing screw.

(4) To set up the instrument on an observation pillar, set the cast-iron base plate on the pillar and accurately center it using a centering pin with a circular level. Place the instrument on the base plate and affix it by turning the central fixing screw with the provided pin.

b. Instrument Leveling. Turn the upper part of the instrument into such a position that the plate spirit level (15) between the telescope supports lies parallel to a line joining two of the foot screws (20). Turn the two foot screws simultaneously but in opposite directions until the bubble in the level is centered. (The bubble sensitivity of the plate level is 5 to 7 seconds per 2-millimeter graduation.) Turn the upper part of the instrument through 90° and again center the bubble by turning the third foot screw. Turn the instrument through 180°. If the plate spirit level is correctly adjusted, the bubble will come to rest at the center. If the bubble is off center, move the bubble halfway back toward the center using the plate-level adjusting screw. Center the bubble in its vial by turning the foot screws. The resulting position of the bubble is called the adjustment position. It corresponds to a vertical position of the vertical axis. Repeat this test, making any additional required adjustments of the plate spirit level. The adjustment of the plate spirit level should be tested every time the theodolite is set up for use. It is essential that the plate spirit level be protected from the direct rays of the sun; otherwise, the position of the bubble will not remain stationary throughout a complete rotation.

c. Focusing. Direct the telescope toward the sky or a light source. Without observing the background, turn the edged, black diopter ring on the telescope eyepiece (6) until the crosshairs appear sharp and black. Note the setting on the numbered scale. This will remain constant for one observer. To focus to obtain a clear image of a sighted object, turn the focusing ring for the telescope (3). Test your absence of parallax between the crosshairs and the image by moving your eye laterally across the front of the eyepiece. If the crosshairs appears to move with respect to the observed object, parallax is present.

d. Parallax Removal. Parallax can be removed by turning the focusing ring (3) until the objective lens is the proper distance for the image to fall in the plane of the vertical crosshair. If the crosshairs appear fuzzy or dim, readjust the black diopter ring and repeat the process. Check for parallax periodically throughout the day, since the focal distance of your eyes changes as you tire. However, do not refocus the crosshairs during the observation of a position.

e. Circle Reading. Both circles are read in the microscope directly alongside the telescope's reading eyepiece (16). On the outer side of the right-hand support is the changeover knob (13) for the circle images. To make the horizontal circle visible, turn the knob clockwise as far as it will go; to make the vertical circle visible, turn the knob in the reverse direction. Simultaneously, on either of the circle images, the image of the scale of the seconds drum will be visible below the circle image. To bring the images to sharpness, turn the telescope's edged eyepiece (6). During the reading, the illumination of the circles should always be bright and uniform. Control the illumination by turning and tilting the illuminating mirrors (9 and 2). Do not alter the illumination during the angle measurement.

f. Horizontal-Circle Reading. When reading the horizontal circle, set the telescope on the object, and look in the microscope to view the images of the two diametrically opposite parts of the circle separated by a fine line. In the middle of the lower circle image is a fixed index mark. Bring the graduation lines of the upper and lower circle images into coincidence with each other in the immediate area of this fixed index mark by turning the micrometer milled head (12) on the right-hand bearing block. This causes the images to move relative to each other. The making of the coincidence must be done with all possible care, as it conditions the accuracy of the reading. The final movement of the milled head must always move clockwise.

(1) The unit of graduation of a circle is 4 minutes. When coincidence has been achieved, the index mark should be either on a graduation line or between two graduations. Read off the whole degrees from the upright number left of the index mark, and count the graduation intervals from the numbered degree line to the index mark. Each graduation interval represents 4 minutes. If the index mark lies in the middle of a graduation interval, count this as only half an interval (of 2 minutes). This reading is taken to 2 minutes on the circle itself. To this reading, add the reading of the seconds drum seen in the lower image. The seconds drum is divided into 60 numbered seconds. Each numbered second is further subdivided into tenths (10 intervals). The numbers represent seconds and tenths of seconds and are read directly. The drum only shows 60 seconds, and the circle reading always shows 2 minutes equal to 120 seconds, so the drum reading must be doubled. The best way to double the drum reading is to repeat the coincidence adjustment and read the seconds again, thus obtaining another reading for the seconds. The two are then added together. The circle reading plus the sum of the two drum readings give the correct reading. The plate circle of a 2/10-second theodolite is divided into 360° and each degree into 15 graduations or 4 minutes per graduation. The coincidence

method uses both the upright and inverted scales of the circle and allows readings to 2 minutes. On the older models, the 2-minute micrometer scale is divided into 600 units and each tenth part is numbered (Figure 3-5, page 3-20). The reading on this micrometer scale equals one-half of the true value, but instead of doubling this reading, procedure requires making a double coincidence and adding both coincidence readings for the final micrometer reading. On newer models the micrometer scale is divided into 0.2-second segments (Figure 3-6 and Figure 3-7, page 3-20). The micrometer is read directly, without adding the two coincidences, but procedure requires making a double coincidence and meaning the two coincidences for the final value. The procedure for reading the horizontal circle and micrometer for both models is shown in Figures 3-5, 3-6, and 3-7, page 3-20.

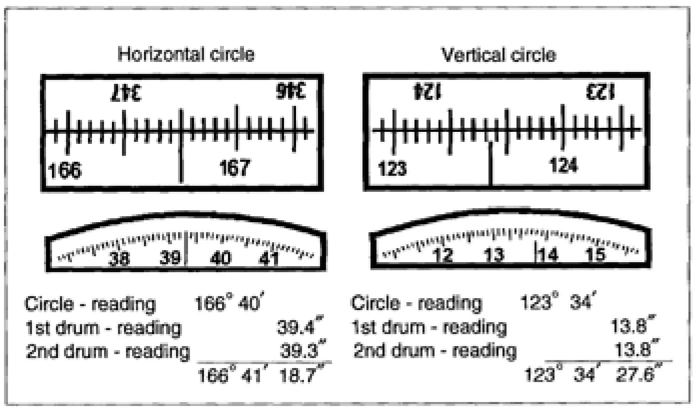


Figure 3-5. Circle Readings (Old Model)

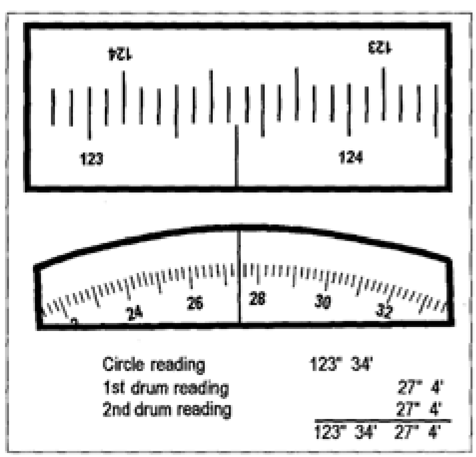


Figure 3-6. Vertical Circle and Micrometer (New Model)

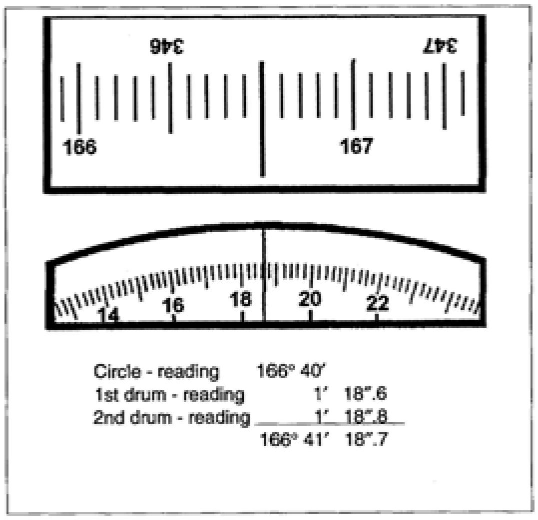


Figure 3-7. Horizontal Circle and Micrometer (New Model)

(2) If the instrument is slightly out of adjustment, the index mark may point halfway between the circle marks and cause confusion. In this case, obtain a reading by locating the degree graduation to the left of the index and counting to the right from the 4-minute subdivisions until reaching the diametrically opposite degree mark on the upper, inverted circle image. Multiply the number of these subdivisions by one-half their value to obtain the proper number of minutes read by the index.

(3) One method of reading the minute value of the horizontal circle uses both the upper and lower circles. If both circles are used, as shown in Figure 3-8, the number of divisions or graduations from the degree mark on the lower circle to the degree mark diametrically opposite (180°) on the upper circle, when multiplied by 2, gives the value of minutes.

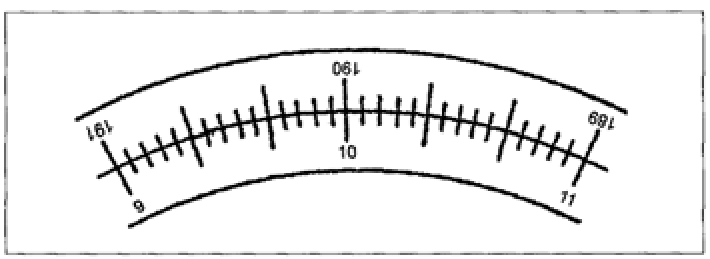


Figure 3-8. Determining the Minute Value

When the 10-percent mark on the lower circle coincides with the 190° mark on the upper circle, the minute value is 0. Counting to the right from the 9° mark on the lower circle to the 189° mark on the upper circle, there are 30 divisions. The actual value is *10°00'* which is equal to *9°60'*. Therefore, in 60 minutes, which is the minute value from 9°, there are 3 divisions. The minute value of each graduation using both circles is *60' / 30* or 2 minutes. There are several ways to read the minute value using both circles. These additional values are demonstrated and explained in the following examples:

Example 1: When both necessary degree marks show in the circle telescope and the degree mark of the upper circle is to the right of that of the lower circle, the value is *10° + (6 graduations x 2')*, which is equal to *10° + 12'* (Figure 3-9).

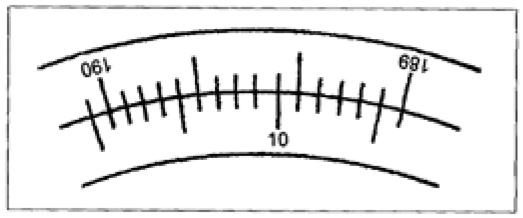


Figure 3-9. Determining the Minute Value - Example 1

When the correct or true value for the degrees does not show in the circle telescope, the reading for the minute value can be made in either of the following two ways.

Example 2: Since 190° from the value showing in the lower circle is to the left in Figure 3-10, the minute value must be subtracted from the visible degree value to give the true value of the direction. Therefore, *10° - (9 graduations x 2*') is equal *to 10° - 18'*, which is equal to *9° 42'.*

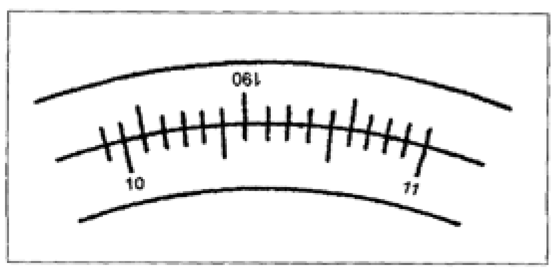


Figure 3-10. Determining the Minute Value - Example 2

Count to the right the number of divisions from the true degree value on the lower circle (which does not show) to the value 190° difference on the upper circle. Realizing that from the 9° mark to the 10° mark on the lower circle there are 15 divisions, count the number of divisions to the right from the 10° mark on the lower circle to the 189° mark on the upper circle.

9° + (15 graduations + 6 graduations) x 2'
9° + (21 graduations x 2') = 9° + 42'

(4) The previously described methods, using both circles, are slower than methods using the index mark. However, it is also possible for the index mark to be

out of adjustment. If the index mark is used, it should be checked each time the instrument is transported, as it may be thrown out of adjustment.

(5) In the first method, the presence of the index mark was completely disregarded. As previously explained, once it has been proven that the index mark is not out of adjustment, it may be used to determine the minute value of the reading. This is the faster and easier method of determining the minute value. The index mark always refers to the degree of the lower erect image or *lower circle*. Using the index mark in this way (in relation to only the lower circle), each graduation is equal to 4 minutes. When the coincidence is made between the images of both sides of the horizontal circle, or the upper and lower circles as they appear in the circle telescope, the index mark will always be either on a graduation line or halfway between two graduation marks. There are two ways to determine the minute value using the index marks on the lower circle. If the index is on a graduation line, the minute value is four times the number of graduations between the degree mark to the left. If the index is halfway between two marks, add 2 minutes to four times the whole number of graduations between the degree mark to the left and the index mark to obtain the minute value.

Example 1: When the true degree value shows in the circle telescope as does 10 in Figure 3-11, the value of the direction equals--

10° + (1 division x 4') + 2'
10° + 4' + 2'
10° 06'

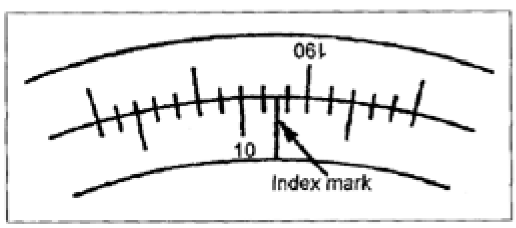


Figure 3-11. Determining the Minute Value Using the Index Mark (Visible True-Degree Value)

Example 2: Since the index mark in Figure 3-12 is to the left of the 10° value shown in the lower circle, the minute value must be subtracted from the visible 10° value in order to give the true value of the direction. Thus, when the true degree value (9) does not show in the circle telescope, as is the case in Figure 3-12, figure the minute value between the index mark and the degree mark to the right and subtract as follows:

= 10° - (3 graduations x 4') + 2'
= 10° - (12' +2')
= 10° - 14' = 9° 46'

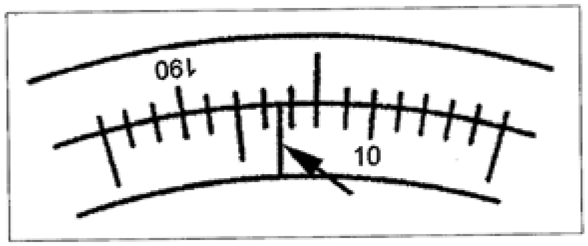


Figure 3-12. Determining the Minute Value Using the Index Mark (True-Degree Value Not Visible)

(6) As shown in the preceding illustrations and explanations, various methods can be used to read the horizontal circle to the nearest 2 minutes. To measure seconds, use the micrometer screw to bring the opposite sides of the circle into optical coincidence and read the amount of movement on the micrometer drum. The micrometer drum is divided into 60 seconds, with each second divided into 10 intervals. In a complete revolution of the micrometer drum, or a movement of 60 seconds, there is a corresponding change of 2 minutes (120 seconds) on the horizontal circle. The horizontal circle is double graduated in relation to the micrometer; therefore, you must read the seconds and tenths of a second twice, and add the two readings together to get the true value of the direction.

(7) To make a complete reading, bring the observed object near the vertical crosshair, and clamp the horizontal lock. Use the double crosshair for large objects and the single crosshair for normal objects. All lights should be such that the single hair can be used accurately. Using the horizontal tangent screw, put the vertical crosshair exactly on the center of the observed object.

(8) Look into the circle telescope, and bring the opposite sides of the circle into optical coincidence using the micrometer screw. Mark the degree and minute readings on the horizontal circle, and dictate these readings to the recorder while making the first coincidence with the micrometer drum. When the coincidence is made, read the micrometer drum, but do not dictate the seconds and tenths of a second to the recorder until the final coincidence reading is made.

(9) Throw the horizontal circle out of coincidence by rotating the micrometer screw about one-half turn. Bring it back into coincidence, and read the micrometer drum again. This micrometer reading must be within 0.3 of a second of the first micrometer reading. If it is not within these limits, repeat this procedure until the two readings agree within 0.3 of a second. Dictate both of the readings to the recorder exactly as they are on the micrometer drum.

g. Horizontal Directions. There are 16 positions that are necessary for first-order triangulation. A position consists of two points or readings on each object observed. Therefore, if there are three objects or stations, there will be 16 positions times 3 objects times 2 readings each (direct and reverse), which equals 96 readings or points. These 16 positions constitute a *set* of observations.

(1) In order to avoid instrumental errors caused from using the same portion of the horizontal circle and the micrometer to make all of the readings for the 16 positions, the readings are proportionately distributed. This is accomplished by changing the *initial setting* of the circle and the micrometer on the first object or station observed in each position. The reason for this procedure is to avoid the possibility of obtaining all readings from one portion of the circle or the micrometer, which could result in erroneous graduations.

(2) Circle and micrometer settings are shown in Table 3-1. For the circle settings at each position, add 11° to the final degree reading of the previous position, with the exception of the 5th, 9th, and 13th positions. In these positions, add 12° to the final degree reading of the previous position. The micrometer settings must be made within 5 seconds of those shown in Table 3-1.

Table 3-1. Circle Settings for First-Order and Second-Order, Class I Work With the Wild T-3

Position Number	Telescope	Circle Degrees	Circle Minutes	Micrometer Seconds
1	D	0	00	10
2	R	191	00	25
3	D	22	00	35
4	R	213	00	50
5	D	45	00	10
6	R	236	00	25
7	D	67	00	35
8	R	258	00	50
9	D	90	00	10
10	R	281	00	25
11	D	112	00	35
12	R	303	00	50
13	D	135	00	10
14	R	326	00	25
15	D	157	00	35
16	R	348	00	50

(3) When observing a position, place the vertical hair on the first observed object, with the telescope in the required position (D is direct, with the *circle telescope* on the left; reverse, R is on the right). Using the micrometer screw, obtain the required *seconds* setting on the micrometer drum. Refer to the circle and micrometer seconds in Table 3-1. Using the circle setting knob, set the required *degree* setting on the circle. Make the final coincidences with the micrometer screw, ensuring that the two micrometer readings are within 0.3 of a second before dictating them to the recorder.

(a) Continue this procedure on all objects to the right, with the telescope remaining in the same position, reading the circle and the micrometer (do not touch the circle setting knob). After the reading has been made on the last object to the right, rotate the instrument and the telescope so that the telescope is on the opposite side of the object. Mark the reading here in the prescribed manner. The reading should be about 180° different from the previous reading. Go through the reading procedure to the left, making readings on all objects until the *initial* or first station is observed once again. This terminates the *position*. Recheck the pointing on the initial object, and increase the *seconds* and *degree* setting for the next position. Repeat the same procedure for all remaining positions.

(b) When all 16 positions are complete, calculate the mean of all observations. Reject any of the positions that are not within ± 4 seconds of this mean. Reject on sight any position that is obviously bad (one which is 3 or 4 seconds above or below any other direction in the set) and do not include it in the trial mean. This eliminates the possibility of an abnormal direction causing a good direction to be rejected. You may reject a direction at the time of observation due only to a kicked tripod, wrong light, or flaring light. All rejected positions must be reobserved using the required circle and micrometer settings for those positions. Observe only the objects involved in the rejected positions, since it is unnecessary and timewasting to reobserve positions that are within the limits.

(c) Make every possible effort to obtain 16 positions within the limits of retention of ± 4 seconds from the mean. However, it is permissible to use a minimum of 12 good positions for first-order work if it is impossible to obtain the full 16 positions. Once you reject an observation, it remains rejected even though a new mean computed from the reobservation would bring it within the retention limit.

(4) For second-order, Class I triangulation supplemental to or adjoining first-order triangulation, observe with a T-3, and use the same procedures as described for first-order, Class I (in this case, the limits of retention are ±5 seconds from the mean.) For Class II, 8 positions within 5 seconds of the mean are recommended to avoid reoccupations, especially when the stations are not easily accessible or when the observer is somewhat inexperienced. When recording for Class II, show the degrees and minutes for all odd-numbered stations.

h. Vertical Circle Reading. The vertical circle (Figure 3-6, page 3-19) is read in exactly the same manner as the horizontal circle. To read the vertical circle, turn the changeover knob (Figure 3-4, page 3-16 [13]) counterclockwise as far as it will go to bring the vertical circle into view in the reading microscope. A coincidence of the graduations is made as described in the horizontal-circle reading section. The collimation level is centered exactly. Due to the method of graduation, the same figures will appear upright and inverted. The plate circle (from the numbered value farthest left to the same figure inverted on the right) and double coincidence drum values are read as described in paragraph 3-13, f (1), page 3-18, and totaled as 123° 34' 27.4″ (as shown in Figure 3-6). This constitutes one reading. For a reverse reading, plunge the telescope, reverse the instrument, and read the circle.

i. Vertical-Angle Computation. A vertical-angle computation is unique to this instrument. The vertical circle is set so that a perfectly level line reads 90° in the direct and reverse positions.

(1) To differentiate between direct and reverse positions, the nomenclature circle left (direct) and circle right (reverse) are used normally. This refers to the

location of the vertical circle on the standard, which is on the left when the instrument is direct and on the right when in reverse. Elevating the telescope in a circle left (direct) position increases the circle reading and in a circle right (reverse) decreases it.

(2) The value read on the vertical circle equals only one-half of the vertical angle added to or subtracted from 90° and is not the vertical angle added to or subtracted from 90°. However, to compute the vertical angle, the value of one reading is not doubled, but circle left (direct) and a circle right (reverse) readings are taken. The smaller one is subtracted from the larger, and the sign of the angle is determined from the circle left (direct) reading (plus the vertical angle, if above 90°; minus the vertical angle, if below 90°).

(3) The zenith distance is the vertical angle from a point directly overhead (zenith) to the sighted point. To compute the zenith distance, the vertical angle is algebraically subtracted from 90 if plus and added to 90 if minus. A more direct method uses *(circle right plus 90°) - circle left*.

j. Horizontal Collimation Correction. Making corrections to a horizontal collimation error ensures that the line of collimation (line of sight) of the telescope is perpendicular to the horizontal axis of the telescope. After setting up the instrument for observation, make a horizontal observation on a clearly defined object, and read the horizontal circle. Reverse the telescope, and repeat the pointing and reading on the same point. These two readings should be exactly 180° apart. Any other reading indicates that the line of sight is out of adjustment, and the difference is equal to twice the collimation error.

Refer to Figure 3-4, page 3-16. To correct the collimation error, turn the micrometer milled head (12) until the seconds drum shows the mean value found. Restore the coincidence of the circle graduations by turning the horizontal slow-motion screw (17). Move the vertical crosshair sideways with the adjusting screws (4) until it is again on the object. The three adjusting screws, one of which is horizontal, are the pull-action type and are set on the eyepiece 120° apart. To move the crosshair to the right, loosen the horizontal adjustment screw on the left-hand side of the eyepiece and gently tighten, by equal amounts, the other two obliquely spaced adjustment screws on the right-hand side of the eyepiece. Ensure that the screws are not overtightened. To move the crosshair to the left, first loosen, by equal amounts, the two obliquely spaced adjusting screws on the right, and tighten the left horizontal screw (Figure 3-4 shows only two of the three screws [17]). Continue this movement until the object appears exactly on the vertical crosshair. Repeat these measurements and corrections until the collimation error is brought within the allowable limits for the survey.

k. Vertical Circle Correction. Making corrections to the level of the vertical circle reduces the index error of vertical-angle observations. Any necessary horizontal collimation correction must be done before correcting the level of the vertical circle. Refer to Figure 3-4 and perform the following steps.

(1) To test the theodolite for proper vertical collimation, level the instrument carefully, bring the image of the vertical circle into view by turning the changeover knob (13), sight the horizontal crosshair onto a clearly defined object, and record the reading. Reverse the telescope, resight the object, and record the second reading. Halve the second reading and add 90 minutes. This should result in exactly the same reading obtained in the first position. If it does not, a collimation error or index error of the vertical circle exists.

(2) To adjust the vertical circle to the proper reading, place the telescope in the first position. Set the micrometer scale to the proper seconds reading obtained in the vertical collimation check. Direct the telescope on the object. Use the vertical slow-motion screw (14) and bring into coincidence the graduation lines which give the proper reading. Turn the collimation level adjusting screw (21) to bring the level bubble to center. Repeat the measurement of the vertical angle, as with the collimation check, and add the two readings. The sum of the readings should now be within 10 seconds of 180 minutes. If it is not, repeat the entire correction procedure.

PART C: TRAVERSING AND TRAVERSE TYPES

3-14. General. Traversing is a form of control survey that is used in a wide variety of surveys. Traverses are a series of established stations linked together by the angle and distance between adjacent points. The angles are measured by theodolites, and the distances are measured by EDME. The AISI total station combines both of these functions. Detailed information pertaining to traverse design, data collection, and limitations are discussed in the SSGCN.

a. Starting Control. The purpose of a traverse is to locate points relative to each other on a common grid. Surveyors need certain elements of starting data such as the coordinates of a starting point and an azimuth to an azimuth mark. Surveyors should make an effort to use the best data available to begin a traverse. Survey control data is available in the form of existing stations (with the station data published in a trigonometric list) or new stations that were established by local agencies who can provide the station data.

b. Open Traverse. An open traverse (Figure 3-13, page 3-28) originates at a starting station, proceeds to its destination, and ends at a station whose relative position is not previously known. The open traverse is the least-desirable traverse type, because it does not provide the opportunity for checking the accuracy of the fieldwork. All measurements must be carefully collected, and every method for checking position and direction must be use. The planning of a traverse should always provide for closure of the traverse.

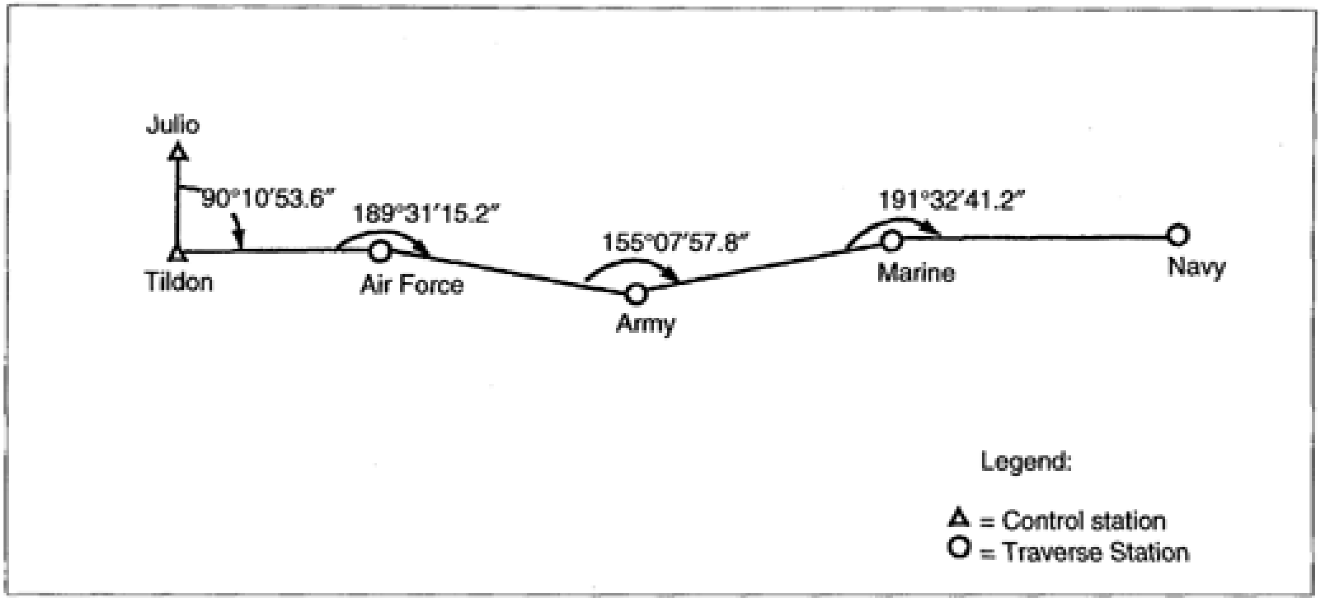


Figure 3-13. Open Traverse

c. Closed Traverse. A closed traverse (Figure 3-14) either begins and ends on the same point or begins and ends at points whose coordinates have been previously determined and verified. In both cases, the angles can be closed, and closure accuracy can be mathematically determined.

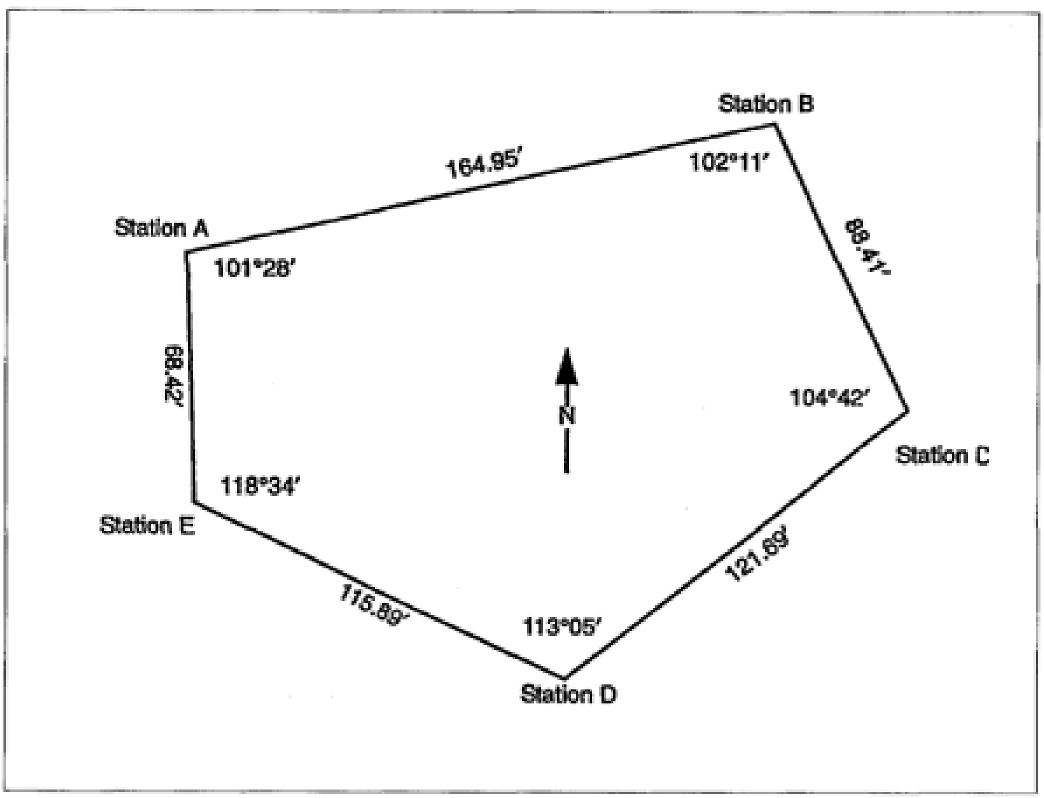


Figure 3-14. Closed Traverse (Loop)

(1) Closed Traverse on a Starting Point. A traverse which starts at a given point, proceeds to its destination, and returns to the starting point without crossing itself in the process is referred to as a loop traverse (as shown in the example in Figure 3-14). Surveyors use this type of traverse to provide control if there is little existing control in the area and only the relative position of the points is required. While the loop traverse provides some check of the fieldwork and computations, it does not ensure detection of all the systematic errors that may occur in a survey.

(2) Traverse Closed on a Second Known Point. A traverse closed on a second known point begins at a point of known coordinates, moves through the required point(s), and terminates at a second point of known coordinates. Surveyors prefer this type of traverse because it provides a check on the fieldwork, computations, and starting data. It also provides a basis for comparison to determine the overall accuracy of the work.

d. Fieldwork. In a traverse, three stations are considered to be of immediate significance. These stations are the rear, the occupied, and the forward. The rear station is the station that the surveyors who are performing the traverse have just moved from or a point to which the azimuth is known. The occupied station is the station at which the party is located and over which the instrument is set. The immediate destination of the party is the forward station or the next station in succession.

e. Horizontal Angles. Measure horizontal angles at an occupied station by sighting the instrument at the rear station and measuring the clockwise angles to the forward station. To measure horizontal angles, make instrument observations to the clearest, most defined, and repeatable point of the target, marking the rear and forward stations. Measurements should be repeated according to the required specifications.

f. Distance. Use EDME to measure the distance in a straight line between occupied and forward stations. Measurements are repeated according to the required specifications.

g. Traverse Stations. Surveyors select sites for traverse stations as the traverse progresses. Surveyors locate the stations in such a way that at any one station both the rear and forward stations are visible.

h. Station Selection. The number of stations in a traverse should be kept to a minimum to reduce the accumulation of instrumental errors and the amount of required computing. Short traverse legs require the establishment and use of a greater number of stations and may cause excessive errors in azimuth. Small errors in instrument centering, station marking equipment, and instrument pointings are magnified and absorbed in the azimuth closure as errors in angle measurements.

i. Station Markers. Station markers are usually 2- by 2-inch wooden stakes, 6 inches or more in length. These stakes (or hubs) are driven flush with the ground. The center of the top of the hub is marked with a surveyor's tack or an *X* to designate the exact point of reference for angular and linear measurements. To assist in recovering a station, surveyors drive a reference (witness) stake into the ground so that it slopes toward the station. Surveyors must write the identification of the station on the reference stake or attached tag using a lumber crayon or a china-marking pencil. Signal cloth may also be tied to the reference stake to further assist in identifying or recovering a station.

j. Station Signals. A signal must be erected over survey stations to provide a sighting point for the instrument operator. The survey target set is the most commonly used signal.

k. Traverse-Party Organization. The number of personnel available to perform survey operations depends on the unit's TOE. The organization of these people into a traverse party and the duties assigned to each member will depend on the unit's SOP. The organization and duties of a traverse party are based on the functional requirements of the traverse.

l. Team Members. The party chief selects and marks the locations of the traverse stations and supervises the work of the other party members. The party chief also assists in survey reconnaissance and planning. Additional team members perform the following duties:

- The instrument operator measures the horizontal angles and distances at each traverse station.

- The recorder keeps the field notes in a field notebook and records the measured angles and distances and all other information pertaining to the survey.

- The rodman assists the party chief in marking the traverse stations, removes the target from the rear station (when signaled by the instrument operator), and moves the target forward to the next traverse station.

m. Azimuth Computations. The azimuth of a line is the horizontal angle (measured clockwise) from a base direction to the line in question. To compute a traverse, surveyors determine an azimuth for each traverse leg (section). The azimuth for each succeeding leg is determined by adding the value of the measured angle at the occupied station to the value of the azimuth from the occupied station to the rear station. Upon occupation of each successive station, the first step is to compute the back azimuth of the preceding leg (the azimuth from the occupied station to the rear station).

n. Azimuth Adjustment. Surveyors must determine the need for adjustment before beginning final-coordinate computations. If the angular error of closure (AEC) falls within the computed allowable error (AE), the azimuths of the traverse may be adjusted. Use the following formula to determine the allowable AEC for third-order, Class I traverse:

$$AEC = 10'' \sqrt{N}$$

Where— $10'' = AE$

N = *the number of traverse segments*

If the azimuth error does not fall within the AEC, surveyors must reobserve the station angles of the traverse in the field.

o. Azimuth Corrections. Before determining a correction, surveyors compute the actual azimuth error. The azimuth error is obtained by subtracting the computed closing azimuth from the known closing azimuth. This difference provides the angular error with the appropriate sign. By reversing this sign, the azimuth correction (with the appropriate sign) is obtained. If the azimuth correction falls within allowable limits, surveyors may compute the error and the correction.

(1) Traverse adjustment is based on the assumption that errors have accumulated gradually and systematically throughout the traverse. The azimuth correction is applied accordingly. The correction is distributed systematically among the angles of the traverse.

(2) After the angles are adjusted, surveyors compute the adjusted azimuth of each leg by using the starting azimuth and the adjusted angles at each traverse station. Surveyors should compute the adjusted azimuth throughout the entire traverse and check against the correct azimuth to the closing azimuth mark before beginning any further traverse computations.

p. Azimuth-Bearing Angle Relationship. The trigonometric functions (sine, cosine, tangent, and so on) of the azimuth and the bearing are numerically the same. Surveyors may use either the azimuth or the bearing to compute the traverse. The choice depends upon the computer and the available equipment.

q. Azimuth and Bearing. If a calculator with angular functions is available, the use of the azimuth is easier since it eliminates the need to compute the bearing. If the functions must be determined from tables, it is necessary to first compute the bearing angles since the tabulation of functions is normally published for angles of 0° to 90°. The bearing of a line is the acute angle (less than 90°) formed by the line in question and the north-south line through the occupied point. This illustrates the relationship between the azimuth of a line and its bearing.

r. Quadrants. The manner for computing bearing angles from a given azimuth depends on the quadrant in which the azimuth lies. When the azimuth is in the first quadrant (0° to 90°), the bearing is equal to the azimuth. When the azimuth is in the second quadrant (90° to 180°), the bearing is equal to 180° minus the azimuth. When the azimuth is in the third quadrant (180° to 270°), the bearing is equal to the azimuth, minus 180°. When the azimuth is in the fourth quadrant (270° to 360°), the bearing is equal to 360° minus the azimuth. Since the numerical values of the bearings repeat in each quadrant, surveyors must label them and indicate into which quadrant they fall. The label must indicate whether the bearing angle is measured from the north or south line and whether it is east or west of that line. For example, a line with an azimuth of *341° 12' 30"* falls in the fourth or northwest quadrant and its bearing is *N 18° 47' 30 "W*.

s. Coordinate Computations. If the coordinate of a point and the azimuth and distance from that point to a second point are known, surveyors can compute the coordinate of the second point. The azimuth and distance from Station A to Station B are determined by measuring the horizontal angle from the azimuth mark to Station B and the distance from Station A to Station B.

(1) A grid is a rectangular system in which the easting and northing lines form right angles at the point of intersection. The computation of the difference in northing (dN)(side Y) and the difference in easting (dE)(side X) uses the formulas for the computation of a right triangle. The distance from Station A to Station B is the hypotenuse of the triangle, and the bearing angle (azimuth) is the known angle. The following formulas are used to compute dN and dE:

dN = cosine azimuth x distance
dE = sine azimuth x distance

(2) If the traverse leg falls in the first (northeast) quadrant, the value of the easting increases as the line goes east, and the value of the northing increases as it goes north. The product of the dE and the dN are positive and are added to the easting and northing of Station A to obtain the coordinate of Station B.

(3) When surveyors use calculators with trigonometric functions to compute the traverse, the azimuth is entered and the calculator provides the correct sign of the function, the dN, and the dE. If the functions are taken from tables, the computer provides the sign of the function based on the quadrant. Lines going north have positive dNs, and lines going south have negative dNs. Lines going east have positive dEs, and lines going west have negative dEs. The formulas shown in the following examples are used to determine the dN and the dE:

Example 1: Given an azimuth from Station A to Station B of *70° 15' 15"* and a distance of 568.78 meters (this falls in the first [northeast] quadrant), compute the dN and the dE.

dN = cosine 70° 15' 15" x 568.78 meters
= +0.337848 x 568.78 meters = +192.16 meters

dE = sine 70° 15' 15" x 568.78 meters
= +0.941200 x 568.78 = +535.34 meters

Example 2: Given an azimuth from Station B to Station C of *161° 12' 30"* and a distance of 548.74 meters (this falls in the second [southeast] quadrant), compute the dN and the dE.

dN = cosine 161° 12' 30" x 548.74 meters
= -0.946696 x 548.74 meters = -519.49 meters

dE = sine 161° 12' 30" x 548.74 meters
= +0.322128 x 548.74 meters = +176.76 meters

Example 3: Given an azimuth from Station C to Station A of *294° 40' 45"* and a distance of 783.74 meters (this falls in the fourth [northwest] quadrant), compute the dN and the dE.

dN = cosine 294° 40' 45" x 783.74 meters
= 0.417537 x 783.74 = +327.24 meters

dE = sine 294° 40 '45" x 783.74 meters
= -0.908660 x 783.74 meters = -712.15 meters

t. Accuracy and Specifications. The overall accuracy of a traverse depends on the equipment, the methods used in the measurements, the accuracy achieved, and the accuracy of the starting and closing data. An accuracy ratio of 1:5,000 is the minimum accuracy sought in topographic surveying. In obtaining horizontal distances, an accuracy of at least 2 millimeters per 100 meters must be obtained. When using a 1-second theodolite, surveyors turn the horizontal angles twice in each position (two direct and two reverse observations) with an angular closure of 10 seconds per station.

u. Linear Error. To determine the acceptability of a traverse, surveyors must compute the linear error of closure (LEC) (using the Pythagorean theorem), the AE, and the accuracy ratio. The first step in either case is to determine the linear error in dN and dE. In the case of a loop traverse, the algebraic sum of the dNs should equal zero. Any discrepancy is the linear error in dN. The same is true for dEs. Surveyors then compute the AE using the appropriate accuracy ratio (1:5,000 or better) and the total length of the traverse. Compare this to the LEC. If the AE is greater than the LEC, the traverse is good and can be adjusted. If it is not good, it must be redone.

v. Accuracy Ratio. The accuracy ratio provides a method of determining the traverse accuracy and comparing it to established standards. The accuracy ratio is the ratio of the LEC (after it is reduced to a common ratio and rounded down) to the total length of the traverse. If the accuracy ratio does not fall within allowable limits, the traverse must be redone. It is very possible that the measured distances are correct and that the error can be attributed to large, compensating angular errors.

w. Coordinate Adjustment. Surveyors make adjustments to the traverse using the compass rule. The compass rule states that on any leg of the traverse, corrections to the dN or the dE are also corrections to the total correction for the dN or the dE, as the length of the leg is to the total length of the traverse. The total correction for the dN or the dE is numerically equal to the error in northing (En) or the error in easting (Ee) but with the opposite sign.

When adjusting a traverse that starts and ends on two different stations, surveyors compute the coordinates before the error is determined. The correction (per leg) is determined in the same manner, but is applied directly to the coordinates. The correction to be applied after computing the first leg is equal to the correction computed for the first leg. The correction to be applied after computing the second leg is equal to the correction computed for the first leg plus the correction computed for the second leg. The correction for the third leg equals the correction computed for the first, second, and third leg. This method continues throughout the traverse. The final correction must be equal to the total correction required.

x. Special Instructions. A good observer consistently secures accurate results, as permitted by the capabilities of the instrument used. To attain proficiency, the surveyor must study the instrument carefully, exercise good judgment, and carefully study all factors affecting the accuracy of theodolite observations. The different classes of errors should also be taken into consideration. In order to obtain the best results, observe the following precautions:

- Adjust and level the instrument carefully before proceeding with the measurement.
- Keep away from the tripod when making a measurement.
- Turn the plates gently. Hold the plates or limbs, not the telescope.
- Work steadily and carefully. Do not rush.
- Do not screw on the tripod legs too tightly.
- Do not turn clamps or leveling screws too tightly.
- Leave the horizontal axis lightly clamped during horizontal-angle observation.
- Leave the lower motion unclamped during vertical-angle observations.
- Never carry the theodolite by its tripod.
- Direct the recorder to immediately record all readings in the notebook, while calling back the figures clearly and promptly.
- Cultivate the impersonal attitude toward results, and read the angles without bias. An angle forced in order to give a good triangle closure will often cause a large angle and side correction when the adjustment is made.

3-15. Traits of a Good Recorder. A good recorder must be able to work quickly and accurately. Recorders should be thoroughly familiar with all phases of reading angles and recording them. They should have sufficient mathematical knowledge to make field computations. The recorder is considered to be an understudy of the instrument man.

a. Under favorable conditions for observing, readings are made so rapidly that the recorder must be methodical in order to not hinder the observations. Mental arithmetic must be infallible, as there is not time for setting down readings and figuring by means of longhand during the steps of recording, computing, and transcribing.

b. All of the records and blank forms should be neat and clear. The form is a valuable part of the permanent record of the survey and may have to be referred to by personnel other than the observing party.

c. As the work progresses, the recorder, under the direction of the observer, performs the following functions:

- Completes records of all observations.
- Prepares the title page and index for the field book.
- Cross-references, when necessary.
- Describes all stations.
- Makes transcripts of the results.
- Records triangle closures.

d. Before going into the field, the recorder should be familiar with all notes and forms used in the work. Headings and column rulings in the notebook should be prepared in advance.

e. All observations should be clearly and distinctly recorded on the proper forms. The numbers must be written so that there is no chance of misunderstanding them. Erasures must never be made in the original records. If it is necessary to change a figure, it should be lightly crossed out, and the correct figure should be placed above it or to one side. The original entry should still be legible. Ensure that each reading is recorded on the appropriate line so that no changes to the record are required.

LESSON 3

PRACTICE EXERCISE

The following items will test your grasp of the material covered in this lesson. There is only one correct answer for each item. When you have completed the exercise, check your answer with the answer key that follows. If you answer any item incorrectly, study again that part which contains the portion involved.

1. An actual error in baseline measurements is defined as ________________________________.

   A. An amount which is mathematically computed to be the error of a single measurement
   B. The actual error is the amount of deviation of each measurement from the mean of all measurements.
   C. The amount of error that is inherent in the measuring tape at any given degree
   D. The amount of error which occurs when traversing over land of highly contrasting relief

2. The 30-meter steel tape is standardized by the NBS __________________________________.

   A. For each meter of tape on a flat, horizontal surface
   B. In the supported position at the 0-, 15-, and 30-meter points
   C. In the supported position at the 0-, 7.5-, 15.0- and 22.5-meter points
   D. For its overall length (at the 30-meter point) in accuracy

3. What is the maximum distance a marking strip or point marking a 50-meter tape end on a baseline can be out of line between the two adjacent marked tape end points?

   A. 2.5 centimeters
   B. 15 centimeters
   C. 10 millimeters
   D. 4 millimeters

4. A survey party is measuring a second-order, Class I baseline and the wind begins to increase. At what point should the taping be halted?

   A. When the tape bends more than 5 centimeters out of line
   B. When the tape bends more than 10 centimeters out of line
   C. When the tape bends more than 50 millimeters out of line
   D. When the tape bends more than 2 centimeters out of line

5. If there is an experienced forward contact man in a baseline survey party, the party chief may act as the ____.

A. Front stretcher man
B. Rear contact man
C. Middleman
D. Recorder

6. When two thermometer readings are recorded in the field notebook, this indicates a ____.

A. Half-tape length
B. Full-tape length (50 meters)
C. Setup
D. New station

7. When measuring, a tape is seldom supported throughout its length due to ____.

A. Irregularities on the surface of the earth
B. Friction
C. A much higher or lower temperature on the surface on which it is laid
D. Its low coefficient of thermal expansion

8. The wild T-3 precision theodolite is a ________________________________.

A. 0.2 second, direction theodolite
B. 0.2 second, repeating theodolite
C. 2.0 second, direction theodolite
D. 2.0 second, repeating theodolite

9. Parallax exists when ____.

A. The instrument is not completely leveled
B. You move your eye from side to side and the observed object also moves
C. The crosshair appears to move with respect to the observed object
D. The plate level bubble has been in correctly calibrated

10. When reading horizontal-circle conditions, the accuracy of the reading is dependant on what factor?

A. The adjustment of the parallax
B. The illumination of the horizontal circle
C. The edged eyepiece of the microscope
D. The making of the coincidence

11. The unit of graduation on a horizontal-circle reading is ________________________________.

A. 4 minutes
B. 2 seconds
C. 1 second
E. 0.1 second

12. One method of reading the minute value of the horizontal circle uses both the upper and lower circles. If both circles are used, the number of divisions or graduations from the degree mark on the lower circle to the degree mark diametrically opposite (180°) on the upper circle, when _____________________, gives the value of minutes

A. Divided by 2
B. Divided by 4
C. Multiplied by 2
D. Multiplied by 4

13. What is the least desirable type of traverse?

A. An open traverse closed on a known point
B. An open traverse
C. A closed traverse
D. A closed traverse on a starting point

14. When using a 1-second theodolite, what angular closure per station is expected?

A. 1 second
B. 5 seconds
C. 10 seconds
D. 20 seconds

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LESSON 3

PRACTICE EXERCISE

ANSWER KEY AND FEEDBACK

Item		Correct Answer and Feedback
1.	B	The actual error is the amount of deviation of each measurement from the mean of all measurements. The actual error...(page 3-2, para 3-1c)
2.	A	For each meter of the tape on a flat, horizontal surface The 30-meter steel tape...(page 3-2, para 3-2a)
3.	A	2.5 centimeters No marking strip or point...(page 3-3, para 3-3)
4.	D	When the tape bends more than 2 centimeters out of line No first-order and second-order...(page 3-4, para 3-5)
5.	D	Recorder The party chief acts...(page 3-5, para 3-7)
6.	B	Full-tape length (50 meters) When recording...(page 3-12, para 3-10 [note])
7.	B	Friction However, measurements...(page 3-13, para 3-10c)
8.	A	0.2 second, direction theodolite The wild T-3 precision theodolite...(page 3-15, para 3-13)
9.	C	The crosshair appears to move with respect to the observed object If the crosshairs appears...(page 3-18, para 3-13c)
10.	D	The making of the coincidence The making of the coincidence...(page 3-18, para 3-13f)
11.	A	4 minutes Each graduation interval...(page 3-18, para 3-13f[1])
12.	C	Multiplied by 2 If both circles are used...(page 3-20, para 3-13f[3])

13.	B	An open traverse The open traverse is the...(page 3-27, para 3-14b)
14.	C	10 seconds When using a 1-second theodolite...(page 3-33, para 3-14t)

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LESSON 4

LEVELING

OVERVIEW

LESSON DESCRIPTION:

In this lesson, you will learn to identify the different techniques of leveling.

TERMINAL LEARNING OBJECTIVE:

ACTION:	You will identify the different techniques of leveling.
CONDITION:	You will be given the material contained in this lesson, a number 2 pencil, and a calculator.
STANDARD:	You will correctly answer all practice questions following this exercise.
REFERENCES:	The material contained in this lesson was derived from FM 3-34.331.

INTRODUCTION

In previous lessons we learned to triangulate from a known point to an unknown point in order to find its position and tape between two known points to establish a baseline for a triangulation net, but this is not all that is necessary to know about surveying or establishing known points on the earth's surface. In order to completely define the location of any point on the earth, we must also determine its exact elevation. This can be done through geodetic leveling.

Geodetic leveling is the operation of measuring precise vertical distances above or below a reference or datum. The datum universally used is MSL and is considered to be zero meters. MSL is the average height of the sea for all stages of the tide. The vertical distance above or below this datum then becomes the elevation of that point. It is the surveyors job to run the levels in lines to determine the differences in elevation between points whose elevations are known and those whose elevations are required.

For higher order surveys, various leveling methods and instruments can be used to accomplish this task. The spirit level and graduated rods are used for precise differential leveling. The theodolite, other measuring equipment, and the trigonometric functions of a triangle are all used for trigonometric leveling. In point of accuracy leveling, trigonometric leveling is inferior to precise spirit leveling, particularly in flat areas. In mountainous country, the trigonometric method is of great value, and the results are comparatively more accurate.

There are many accuracy classifications and requirements for leveling. This lesson begins by discussing the orders of accuracy and requirements and continues by describing the equipment, methods, procedures, records, and computations for precise differential leveling. The final portion of this chapter covers the process of determining elevations by the indirect method of trigonometric leveling.

PART A - ACCURACY AND REQUIREMENTS

4-1. Spirit Level Lines. Spirit level lines usually determine elevations referred to the datum. These lines are run according with rigid specifications that prescribe the conditions under which the line may be run and the tolerances that are permissible for the recognized orders of accuracy. When planning new level lines, the surveyor must take into consideration the ties to all previously determined elevations of acceptable orders of accuracy, the best distribution of new elevations for the contemplated survey, the nature of the terrain involved, and the most practical routes to run the new line. All pertinent information must be assembled for study, and suitable guide maps must be prepared to show the routing of the proposed lines. Generally, the level man is not responsible for such planning, but he should understand the established requirements as he will be in a position to apply sound judgment if local conditions force slight deviations from the initial plan.

4-2. Benchmarks. The value of a line of levels depends upon the accuracy, distribution, and permanency of established benchmarks or those tied in during leveling. Benchmarks are permanent objects, natural or artificial, bearing a marked point whose elevation above or below the adopted datum is known. Temporary benchmarks (TBMs) are intended to serve for short time periods.

4-3. Leveling. Leveling is classified as the degree of accuracy according to the methods and instruments used and the closure specified. The orders of accuracy are first, second, and third order. First order is the most precise. This lesson will only address the required accuracies of first- and second-order leveling.

a. First-Order Leveling. Since the uses of first-order leveling are so important, the criteria are very strict. First-order leveling is used to establish an area's main level net and to provide basic vertical control for the extension of level nets of the same or lower accuracy. This order of leveling provides data for mapping projects, special site construction projects, cadastral and local surveys, and geodetic research involving earth sciences.

(1) First-order level lines must start and end on proven, existing benchmarks of the same order. New levels must be run between the starting

benchmark and at least one other existing benchmark and must show there is no change in their relative elevations.

(2) All first-order lines are divided into 1- to 2-kilometer sections. Each section must be run forward and backward. The two runnings of a section must not differ by more than ±3 millimeters $\sqrt{K}$, where K is the length of the section in kilometers. On all sections that are 0.25 kilometers or less in length and which require two or more setups in each direction, a discrepancy of not more than 2.0 millimeters between the backward and forward measurement is considered a satisfactory check. In the case of a single-setup section, the checks between forward and backward runnings seldom exceed 1.0 millimeters, averaging 0.6 millimeters or less.

(3) When additional runs are made due to excessive divergence, the indiscriminate mean of all measured differences in elevation for that section is computed, excluding obvious blunders. If any measurement of the difference in elevation for that section is more than ±6.0 millimeters $\sqrt{K}$ from the indiscriminate mean, that observation is rejected. No rejection should be made on because of a residual smaller than ±6.0 millimeters $\sqrt{K}$, unless there is some good reason for suspecting an error in that particular measurement. In such cases, the reason for the rejection must be stated in the record. After the rejection has been completed, the mean of all *remaining forward measurements* is computed and compared with the mean of all *remaining backward measurements* to determine if the required accuracy has been obtained. The *mean* of the *means* is then the final value.

(4) The method and equipment used in first-order levels are designed to yield a maximum of ±1 millimeter for the probable accidental error and ±0.2 millimeter for the probable systematic error in a distance of 1 kilometer.

b. Second-Order Leveling. The uses of second-order leveling are also quite important, so the criteria are only slightly less strict than those for first-order leveling. Second-order leveling is used to subdivide nets of first-order leveling and to provide basic control for the extension of levels of the same or lower accuracy. This order leveling is used to provide data for mapping projects, local surveys, and special projects, which include the positioning of radar equipment and stellar camera pads. Second-order leveling is also used for initial missile site surveys. Second-order levels are divided into two classes, Class I and Class II.

(1) Class I is used in remote areas where the line must be longer than 40 kilometers due to the unavailability of routes, for the development of additional or higher order networks, and for spur lines. All lines must start on previously established benchmarks of first or second order. New levels must be run between the starting benchmark and at least one other existing benchmark to prove that they have not changed their relative elevations. Failure to check within the limit of ±8.4 millimeters $\sqrt{K}$ (where K is the distance between benchmarks in kilometers) may indicate that at least one or both of the benchmarks must be tied in to prove the starting elevation. All Class I lines are divided into 1- to 2-kilometer sections which are run both forward and backward. The discrepancy between these runs must not exceed ±8.4 millimeters $\sqrt{K}$, where K is the length of the section in kilometers. When

additional runs are made due to excessive divergence in a second-order section of a level line, the rejection limit of individual values from the indiscriminate mean of all measurements is ±9 millimeters $\sqrt{K}$.

(2) Class II is used for the development of nets in more accessible areas. The criteria for Class II are the same as those for Class I except that Class II lines are single run (in one direction only). The AE formula is still ±8.4 millimeters $\sqrt{K}$, where K is the length of the line in kilometers, the line being the distance between ties to existing benchmarks.

PART B: PRECISE DIFFERENTIAL LEVELING

4-4. General. In principle there is no difference between precise and ordinary differential leveling (Figure 4-1). In ordinary differential leveling, the distances run between checks are relatively short and with the usual precautions, and the results are sufficiently accurate for general-purpose surveys. Since very small errors cannot be detected, the relative coarseness of the determinations may, through compensation of errors, give some degree of accuracy. In precise leveling, the circuits may be of considerable length; therefore, operations must be conducted so that the uncertainty of each individual determination, as well as the actual closing error, is reduced to a minimum. This necessitates the use and proper handling of precision instruments and precise rods.

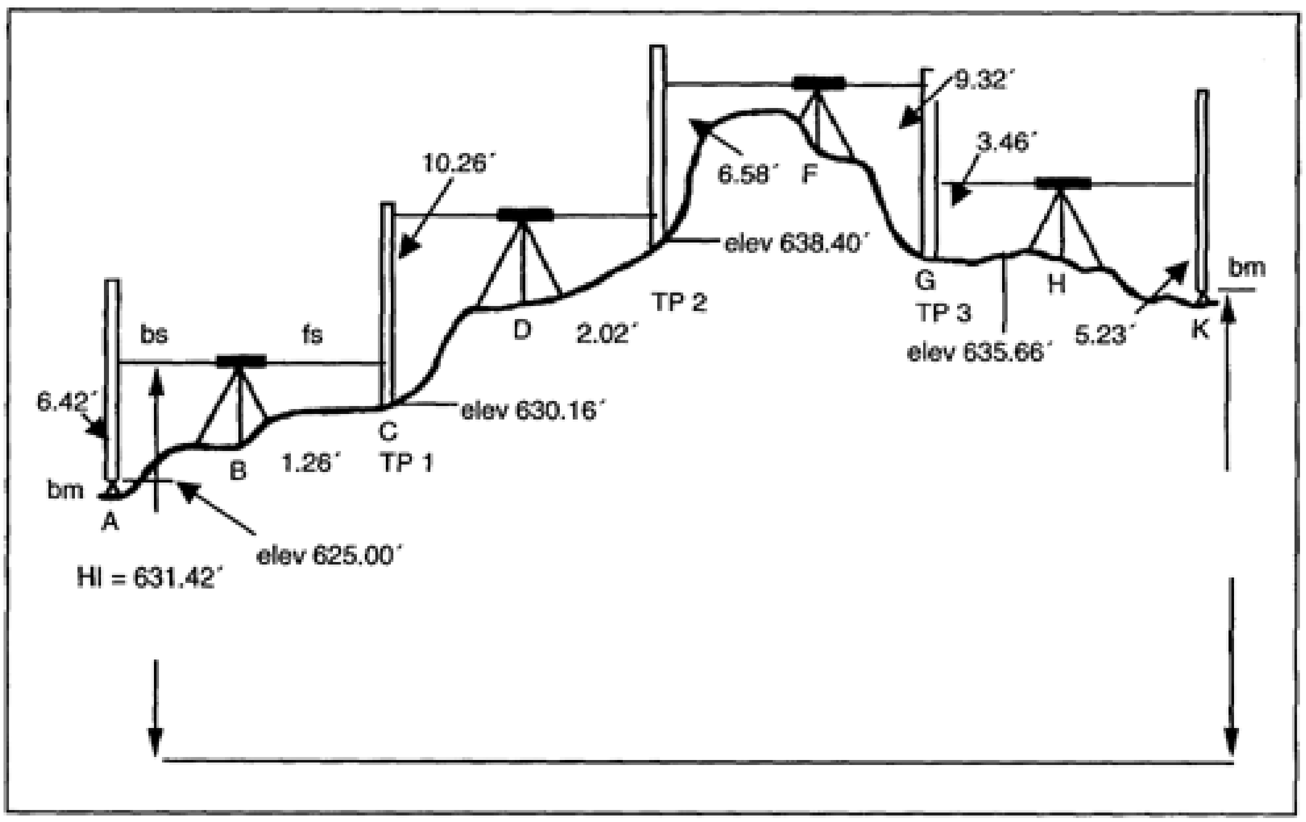


Figure 4-1. Direct Differential Leveling

4-5. Precise Leveling. The instrument used in first- and second-order leveling is a precise level. The instrument is constructed of materials that have a low coefficient of thermal expansion. It is of the tilting type and is equipped with three horizontal hairs (often referred to as threads or wires) for determining the rod reading and length of sights.

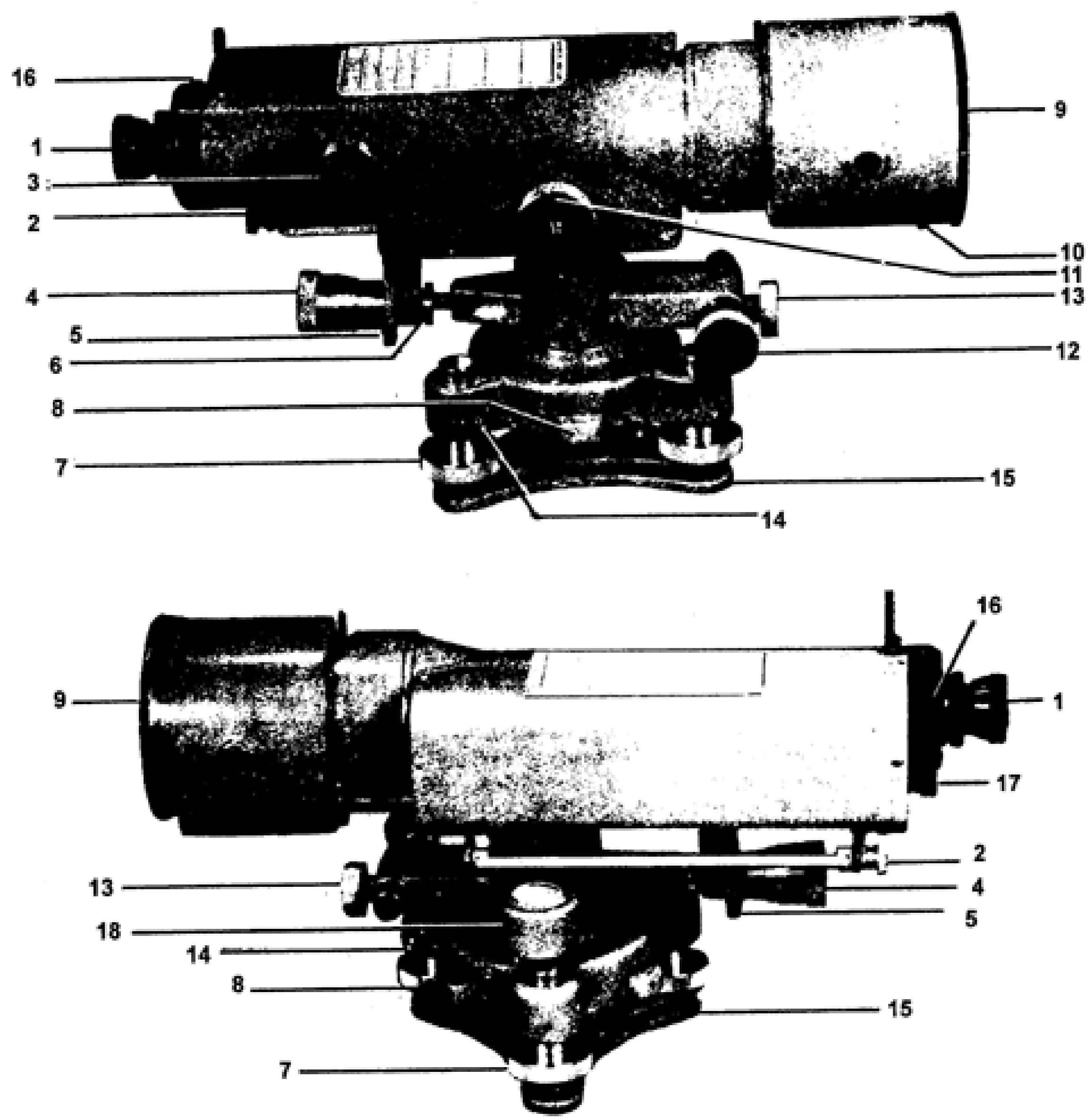
a. A graduated micrometer is built into the instrument to allow readings to the nearest 0.001 of a unit. The sensitivity of the tubular vial, the prismatic device for centering the bubble; the telescopic power; the focusing distance; and the size of the objective lens are factors in determining the precision of the level. These factors may vary individually, but when one factor is weakened, other factors must be strengthened to maintain accuracy. Levels are tested and rated according to their ability to maintain the specified order of accuracy. Only those rated as precise geodetic levels may be used for first- and second-order work. There are various levels available which are rated as precise-leveling instruments. One of these, the wild N-3 precision level, is shown in Figure 4-2, page 4-6. Refer to this figure to identify instrument parts.

(1) Three foot screws (7) rest on a triangular base plate (15), which eliminates the necessity of inserting slots in the tripod head and permits instrument use with a theodolite tripod. You can regulate the motion of the foot screws on their threads using an adjusting screw (14). The lug (8) secures the instrument to its case.

(2) After loosening the azimuth clamp (13), the instrument can swing completely around on its vertical axis. After tightening the screw, fine movements can be made using the azimuth tangent screw (12). By turning the tilting screw (4), the telescope can be inclined to a small degree. A precise setting is assured by transmission through a special level system. The tilting screw is connected to a drum scale (5) bearing a graduation, each interval of which corresponds to a tilt of 0.01 of the distance between the instrument and the leveling rod. The tilting screw can be regulated using the adjusting nut (6). A similar adjusting nut is placed behind the knob of the horizontal azimuth tangent screw.

(3) The micrometer knob (11) controls the optical micrometer. The most important feature of the optical micrometer is its plane-parallel glass plate, which is placed in front of the objective lens and turns on a horizontal axis. The movement of the glass plate causes upward or downward displacement of the telescope's line of sight. The amount of displacement can be read directly on a scale. Since the total possible displacement amounts to 1 centimeter, it is possible to bring the middle thread of the reticule into coincidence with the centimeter line of the leveling rod.

(4) The micrometer knob (11) is fitted to a horizontal shaft attached to a cog, which engages the teeth of a horizontal guide rod. One end of this rod fits, with a ball, into the socket of a lever attached to the glass plate. The other end bears a fine glass scale divided into 100 intervals. This scale can be read using the eyepiece (17), which is movable and can be adjusted to the correct focus. One interval on the scale corresponds to a vertical displacement of 0.1 millimeter on the line of sight. Estimation readings can be made to an accuracy of 0.01 millimeter.



1. Telescope eyepiece
2. Reflector knob
3. Focusing knob
4. Tilting screw
5. Drum scale
6. Adjusting nut for screw 4
7. Foot screws
8. Lug
9. Protective glass mounting
10. Locking screw for protective glass
11. Micrometer knob
12. Azimuth tangent screw
13. Azimuth clamp
14. Adjusting screw
15. Triangular base plate
16. Eyepiece for tubular level
17. Eyepiece for micrometer scale
18. Circular level

Figure 4-2. Wild N-3 Precision Leveling Instrument

(5) The focusing knob (3) is used to focus the telescope. The telescope eyepiece (1) is provided with a diopter scale, enabling individual focusing preferences. Through the eyepiece for the tubular level (16), which magnifies approximately 2 1/2 times, two half images of the level bubble may be observed (as shown in Figure 4-3). A reflector provides illumination. One side bears a mirror, while the other side is lacquered white. The reflector can be turned using the reflector knob (2) to achieve the best possible reflection. The micrometer scale receives its illumination from the same reflector, so if the level is well lighted, good illumination is automatically assured. The interval lines of the scale stand out sharply against a light background and are usually well defined.

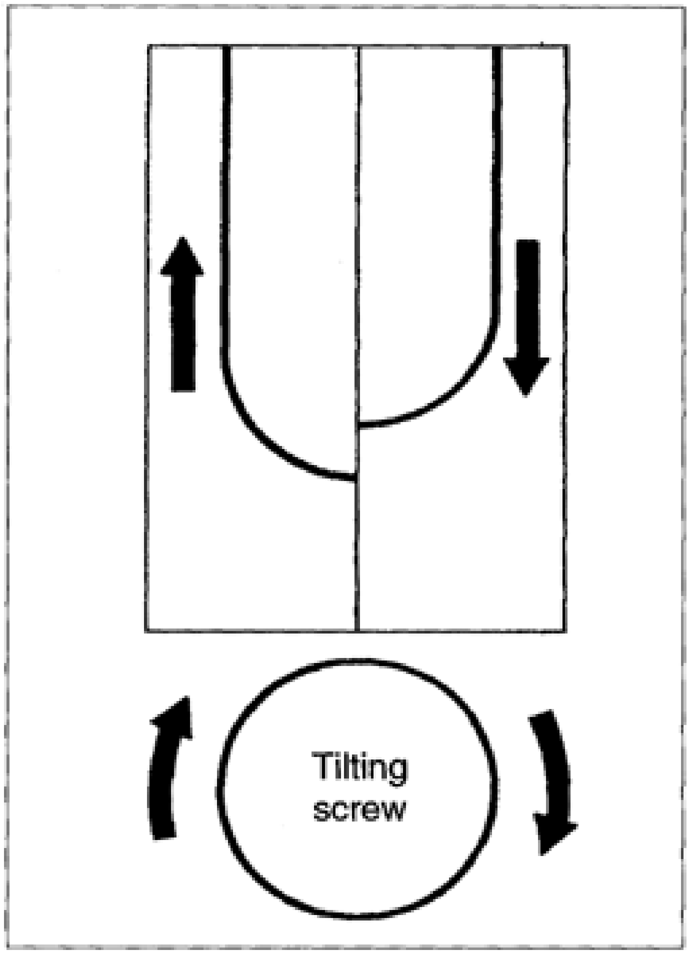


Figure 4-3. Tubular Coincident Level Bubble and Fine-Adjustment Screw

(6) The circular level (18) is situated beneath the reflector. In cases where it is advantageous to erect the instrument in as high a position as possible, the level can be observed in the mirror, entirely free from any parallax.

(7) Four adjusting screws are inserted on the objective side of the level, which can be used to adjust the tubular level. To allow for an easy field adjustment, the protective glass in front of the objective is cut in the form of a wedge. After loosening the locking screw (10), the protective glass rim may be turned, permitting the line of sight to be set horizontally (following the centering of the level bubble).

(8) Loosen the azimuth clamp (13), and swing the instrument around to the most convenient position for observing the circular level. Turn the large illuminating mirror until the bubble can be clearly observed. Center the circular bubble by adjusting the foot screws (7).

(9) Direct the telescope along the line of sight by means of the open sights on top of the instrument. Lightly tighten the azimuth clamp (13), and turn the telescope eyepiece (1) until the crosshairs of the reticule appear sharp and black. Obtain the proper focusing using the focusing knob (3), and test for parallax by moving your eye up and down before the eyepiece to ensure no movement of the crosshairs on the rod. If the crosshairs appear to move up and down on the rod, correct or eliminate this movement by moving the objective lens in or out. Readjust the eyepiece if the crosshairs appear fuzzy, and note the reading obtained on the diopter scale. For subsequent readings, reset the eyepiece at its original focused position. Bring the vertical centerline of the reticule to the center of the leveling rod by means of the azimuth tangent screw (12) located beside the azimuth clamp (13).

(10) Observe the tubular level through the eyepiece (16), and turn the reflector until the images of the bubble ends stand out with uniform clearness. To obtain the most satisfactory illumination, ensure that the white lacquered side of the reflector is uppermost and outwardly inclined. Turn the tilting screw (4), directly beneath the eyepiece, until the two half images of the level bubble form a single, continuous curve. Note that the left and right halves of the level bubble image move in the same direction as the left and right sides of the tilting knob (Figure 4-3, page 4-7 and the upper left side of Figure 4-4).

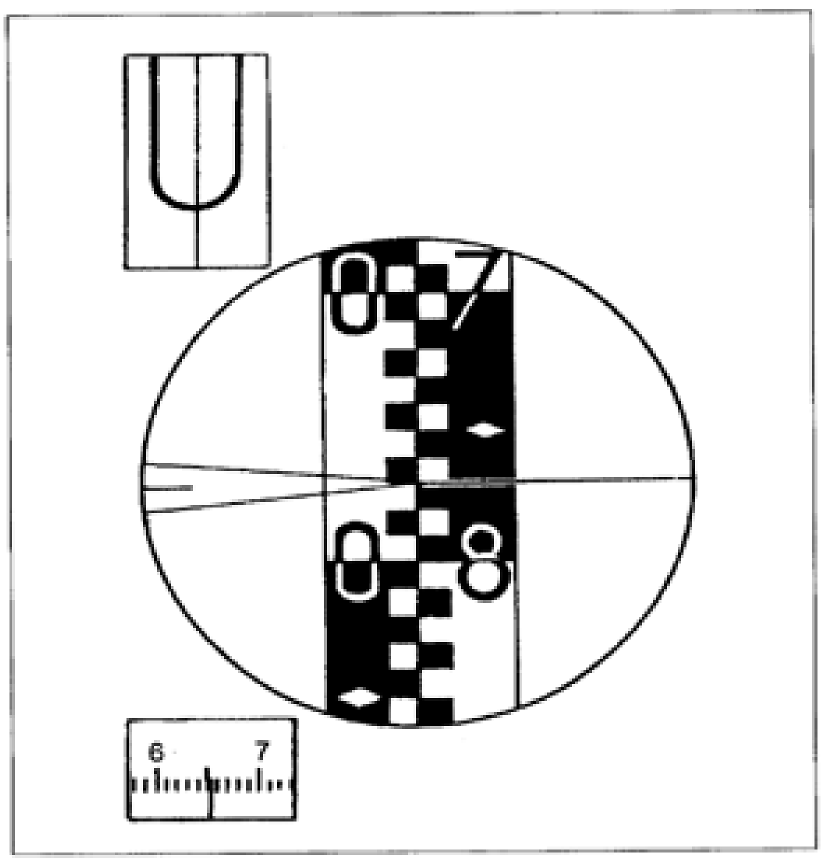


Figure 4-4. Rod Reading

(11) The optical micrometer can be used for all rod readings. Turn the micrometer knob (11) until the wedge lines of the reticule enclose an interval line of the rod (as shown in Figure 4-4). The interval gives the reading in centimeters. Observe the micrometer reading through the micrometer scale eyepiece (17). As shown on the lower left in Figure 4-4, the scale runs from left to right. (In older N-3 models, the scale runs from right to left.) The figures on it represent millimeters, while the smaller intervals represent tenths of millimeters. Estimations may be made to hundredths of a millimeter. The reading obtained on the micrometer is added directly as a decimal fraction to the rod reading. The middle thread reading in Figure 4-4 is 77.653 centimeters (0.77653 meter) and would be recorded as 077653.

b. Most surveying organizations within the Army and the majority of other well-known federal surveying agencies lock the micrometer with a micrometer setting lock. When installed, this lock is located just forward and in from the front end of the reflector. The rod is read directly to the nearest centimeter, and the reading is estimated and recorded to the nearest millimeter. When reading the rod by this direct method, the readings of the three crosshairs (as shown in Figure 4-4) are as follows: Read the bottom hair first (0.807 millimeter), and record it as *0807* at the top of the page in the field book. Read the middle hair (0.770 millimeter), and record it as *0770*. Read the top hair last (0.733 millimeter), and record it as *0733.* This procedure speeds the leveling process and provides required accuracies and checks with minimum effort by the observer and recorder.

4-6. Precise Rods. A precise rod is required in first- and second-order leveling. This rod consists of a graduated metal strip, 25 millimeters wide by 1 millimeter thick, with a low coefficient of thermal expansion. The strip is rigidly attached to a metal foot piece. The foot piece is fastened to a wooden backing which supports the strip. The backing is about 2.8 centimeters thick by 7.6 centimeters wide, with an overall length of 3.3 meters.

a. The front of the rod is graduated in meters and decimeters. The Invar strip is graduated in centimeters. The rear of the rod is graduated in feet and tenths of feet. Mounted on the back of the rod is a circular or bull's-eye level and thermometer. The bull's-eye bubble is used to plumb the rod. The thermometer, located above the bull's-eye bubble, is used to obtain the rod temperature to correct for the expansion or contraction of the metal strip.

b. Rods are standardized by the NBS (or their foreign counterpart), and their index and length corrections are determined. The rods are paired and marked according to similar characteristics and their convenience in applying various corrections. The Invar strips require periodic checks against the standard to determine any changes that may affect the accuracy of readings.

4.7. Miscellaneous Equipment. In addition to a precise level and rod, a leveling party requires the following equipment:

- A surveyor's umbrella to shade the instrument.
- Turning pins or plates to set the rod on during observation.
- Hammers to set the turning pins.
- A locator's hand level to estimate the lines of sight.
- Hand-held counters to tally paces for the approximate balancing of the lines of sight.
- Recording books and conversion tables for note keeping.

4-8. Field Tests and Adjustments. To maintain the required accuracy, certain tests and adjustments must be made before and after the level is used.

a. Determine the Stadia Constant. Carefully determine the stadia constant of the instrument before it is used. It is the factor by which the totals of the cumulative sums of the total intervals for both backsights and foresights are multiplied to obtain the lengths of the sections. These results, in turn, are used in the formulas for computing the allowable divergences (errors) between the forward and backward runnings of the various sections. Comparing the stadia intervals observed over a known course will aid in the determination.

(1) Select a suitable stretch or reasonably level track, roadway, or sidewalk. Place nails or other marks in a straight line at distances 75, 85, 95, 105, 115, and 125 meters from an initial-point 0.2 meter to the back of the center of the N-3 level. Using the direct method, read the three thread readings on the rod at each of these stations, and record the readings on a DA Form 5820, as shown in Figure 4-5. The split-level bubble of the N-3 need not be accurately centered for these readings, but it should be free of the ends of the tube. Compute the half-thread intervals as a check against erroneous readings, and compute the sum of the total intervals for the six readings. The stadia constant is the sum of the measured distances (300 meters) divided by the sum of the six total thread intervals (*300 - 898 = 0.334*).

THREE-WIRE LEVELING

For use of this form, see FM 3-34.331; the proponent agency is TRADOC.

INST. OP. INT.	1st COMP. INT.	2nd COMP. INT.

PROJECT	LOCATION	ORGANIZATION
30/FBV/2LL/B168	Ft. Leonard Wood, MO	30th Engr Bn

OBSERVER	RECORDER	INSTRUMENT	SUN	WIND	WEATHER
SFC J. Doe	SPC W. Roe	Wild N-3 #11268	7	0 to 1	Warm, breezy, clear

FROM	TO	DATE (YYYYMMDD)	TIME	LINE OR NET	PAGE NO.	NO. OF PAGES
		20010709	1000	Stradia constant	1	1

STATION	BACKSIGHT FACE OF ROD	MEAN	BACK OF ROD	INTERVAL	SUM OF INTERVALS	FORESIGHT FACE OF ROD	MEAN	BACK OF ROD	INTERVAL	SUM OF INTERVALS	REMARKS
	1337										Rod #778
25	1300			37'							
	1262			38'	75'			25' =		0.3333'	
								75'			
	1359										
35	1307			52'							
	1254			53'	105'			35' =		0.3333'	
					180'			105'			
	1377										
45	1310			[illegible]							
	1242			[illegible]	[illegible]			45' =		0.3333'	
					[illegible]			135'			
	1325										
55	1243			82'							
	1160			83'	165'			55' =		0.3333'	
					480'			165'			
	1262										
65	1165			97'							
	1068			97'	194'			65' =		0.3335'	
					674'			194'			
	1218										
75	1106			112'							
	0994			112'	224'			75' =		0.3335'	
					898'			224'			
Total=300'										2.002'/	6=
		300'	=	0.334'							
		898					Check			0.334'	

DA FORM 5820, JUL 2001 — EDITION OF AUG 89 IS OBSOLETE. — USAPA V1.00

Figure 4-5. Sample DA Form 5820 (Determination of the Stadia Constant)

(2) To check for gross errors in the measurement of the six rod readings, compute each separate observation. The average of the results of the six separate computations will serve as a numerical check. If any of the six individual stadia constant values is more than 0.002 from the mean of these six values, redo the rod readings or remeasure the distance. Any tendency for the values to vary in one direction is good evidence that there is some error in distance between the point 0.2 meter behind the center of the instrument and the first of the rod points. It is possible that all six measured distances could be in error by an even 10 meters.

(3) There are two stadia constants for the N-3 level--1/100 and 1/300. The constant depends upon the spacing of the crosshairs in the type of reticle installed in the instrument. The fraction constant 1/100 means that the distance between the lower and upper crosshairs is 1/100 of the focal length of the instrument, and the constant 1/300 means that the distance is 1/300 of the focal length of the instrument. The stadia constant determined in Figure 4-5, page 4-11 was 1/310 or 0.300. (Remember, this value is not the decimal equivalent of 1/300 but the determined value of the table.) The crosshairs for a 1/100 constant are spaced to represent a horizontal distance of about 100 meters for every meter intercepted on the rod between the lower and upper crosshairs. This is a ratio of 1:100. For a 1/300 constant, the closer spacing of the crosshairs represents a horizontal distance of about 300 meters for every meter intercepted on the rod. The latter spacing has the advantage that, at longer distances over uneven terrain, all three hairs can be observed on the rod. After the stadia constant has been determined, any rod intercept (the distance between the upper and lower crosshairs) during the day's work may be multiplied by that decimal constant to obtain the stadia distance.

b. Determine the *C Factor* and Instrument Adjustment. The most common cause of instrument error is due to tubular level adjustment. When in perfect adjustment, the axis of the tubular level is parallel to the line of sight. The line of sight on the telescope is horizontal when the bubble is centered. When it is not in adjustment, the line of sight will fall above or below the horizontal when the bubble is centered. This type of error becomes evident when, over the course of several setups, it is not possible to obtain equal lengths of backsights and foresights. This type error can be minimized or eliminated by first determining the *C-factor* correction, in which *C* represents the coefficient of the collimation error. The actual inclination error is a function of the distance and the *C factor*.

(1) The *C factor* should be computed before each day of leveling is begun and any time the instrument is jolted. Set two turning pins about 75 meters apart and at about the same elevation. Set the level between the two pins 10 meters from the rear point. With the instrument carefully leveled, read the rods on both points, using all three hairs. Record the results. Move the instrument 10 meters from the front point and repeat the procedure. The bull's-eye bubble on the rod must be accurately centered while the rods are read.

(2) All figures and computations should be recorded on a DA Form 5820, as shown in Figure 4-6. The circled numbers shown on the figure correspond to the procedures for completing the required entries and computations.

THREE-WIRE LEVELING For use of this form, see FM 3-34.331; the proponent agency is TRADOC.	INST. OP. INT.	1st COMP. INT.	2nd COMP. INT.

PROJECT	LOCATION	ORGANIZATION
30/US/2LL/5/69 (1)	Ft. Leonard Wood, MO	30th Engr Bn

OBSERVER	RECORDER	INSTRUMENT	SUN	WIND	WEATHER
SFC J. Doe	SPC W. Roe	WildN-3 #1268	T	0	Clear, calm and warm

FROM	TO	DATE (YYYYMMDD)	TIME	LINE OR NET	PAGE NO.	NO. OF PAGES
		20010707	0800	"C" factor check	2	2

STATION	BACKSIGHT FACE OF ROD (2)	MEAN (4)	BACK OF ROD	INTERVAL (5)	SUM OF INTERVALS (6)	FORESIGHT FACE OF ROD (3)	MEAN (7)	BACK OF ROD	INTERVAL (8)	SUM OF INTERVALS (9)	REMARKS
A	1613					2802					Rod #778 (R)
	1565 (R)	15650'		48'		2337	23370'		465'		Rod #780
	1517			48'		1872			465'		
	4695'				96'	7011	(10)	Ref=	-0.6'	930'	
											Stradia
B	1775					1428					Constant =
	1717	17167'		58'		0974 (R)	09743'		454'		0.100
	1658				117'	0521			453'	907'	
	9845	32817			213'	9934'	33113'	Ref=	-0.6'	1837'	
							- 1.2' =	Tot ref=		-213'	
							33101'			1624'	
							32817'				
							- 28.4'	/1624' =	-0.017' = C	(11)	
After	Adjustment										
B	1774					1411					
	1715	17153'		59'		0959 (R)	09583'		452'		
	1657			58'		0505			454'		
	5146'				117'	2875'		Ref=	-0.6'	906'	
A	1602					2759					
	1546 (R)	15467'		56'		2303	23033'		456'		
	1492			54'	110'	1848			455'	911'	
	9786'	32620'			227'	9785'	32616'	Ref=	-0.6'	1817'	
		32604'					-1.2' =	Tot ref		-227'	
		+1.6	/1590' =	+0.001' =C			32604'			1590'	

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Figure 4-6. Sample DA Form 5820 (Recording and Computation of the C Factor)

(a) (1) Begin by completing the information in the page heading.

(b) (2) Record the readings from the backsight (near) rods (keep a running total).

(c) (3) Record the readings from the foresight (distant) rods (keep a running total).

(d) (4) Enter the means of each set of three thread readings on the near rod and the sum of the means. (The decimals are not usually shown when recording the mean of the three thread readings; however, they have been inserted here since millimeters and tenths of a millimeter will be used to help determine the *C factor*.)

(e) (5) Enter the thread interval on the near rod. This reading is obtained by subtracting the middle thread reading from the upper thread reading and subtracting the lower thread reading from the middle thread reading for each backsight.

(f) (6) Enter the sum of the intervals for each backsight and the sum of all backsights.

(g) (7), (8), and (9) Apply the same procedure used to obtain columnar values for the backsights, using the readings in column (3).

(h) (10) The curvature and refraction correction (Table 4-1) must be taken out and applied to the distant-rod readings. Use each foresights separately, and determine the curvature and refraction value. Sum these values and apply this sum to the sum of the mean values in column (7).

Table 4-1. Total Correction for Curvature and Refraction

Distance (Meters)	Correction to Rod (Meters)
0 to 27	-0.0
27.0 to 46.8	-0.1
46.8 to 60.4	-0.2
60.4 to 71.4	-0.3
71.4 to 81.0	-0.4
81.0 to 89.5	-0.5
89.5 to 97.3	-0.6
97.3 to 104.5	-0.7

(i) (11) The required constant, C, is the ratio of the required rod readings to the corresponding subtended interval and can be determined using the following formula:

$$C = \frac{\textit{(Sum of near - rod readings) - (Corrected sum of distant - rod readings)}}{\textit{(Sum of distant - rod intervals) - (Sum of near - rod intervals)}}$$

Substituting the values from the sample into the above formula, the new formula becomes--

$$C = \frac{3281.7 - 3310.1}{1837 - 213} \quad \text{or} \quad C = \frac{-28.4}{1624} \quad \text{Therefore} \quad C = -0.017$$

(3) If the total of the mean readings in column (7) is larger than the total of the mean readings in column (4), C carries a minus value. If the opposite occurs, C carries a plus value. Compute the value of C to the nearest unit in the third decimal place.

(4) The maximum permissible *C factor* varies with the stadia constant of the instrument. If C exceeds 0.010 when using the N-3 level with a stadia constant of 1/300, adjust the instrument by .007.

(5) If an adjustment of the level is made, immediately redetermine the *C factor*. Adjust the level by raising or lowering the time of sight by an amount equal to C times the total foresight rod interval (-4 millimeters in the example). The N-3 instrument is adjusted by loosening the locking screw for the protective glass (10) (Figure 4-2, page 4-6). Turn the rim of the protecting glass in the appropriate direction until the middle thread intercepts the distant rod at the desired reading. If care is exercised, the level bubble will remain in coincidence. Complete the adjustment by tightening the locking screw. If the instrument receives a severe jolt, which might disturb its adjustment, immediately make a new determination of the level error and adjust the level if the *C factor* exceeds 0.010. Always note the time and the date of *C factor* determinations, as this information is essential to the accuracy of the computations.

c. Adjust the Leveling Rods. At least once a month and any time it receives a severe shock during leveling, the leveling rod should be tested and the plumbing level adjusted. If the deviation from the vertical in either direction exceeds 10 millimeters on a 3-meter rod, adjust the rod level. Record the method and results in the level notebook under test of rod verticality.

4-9. Methods and Procedures. When all instrument tests and adjustments are complete, the instrument man is ready to begin leveling. The methods and procedures discussed here are applicable to both first- and second-order leveling.

a. Precise Leveling. Precise leveling uses the three-wire technique. The program of observation at each instrument station uses the following procedures:

- Set up and level the instrument.

- Read all three crosshairs as projected on the face of rod number 1, with the bubble continuously in coincidence.

- Read and record each crosshair to the nearest 0.001 meter, as shown in the example in Figure 4-7, columns (7) and (13). Always read the three crosshairs in their order of magnitude on the rod, beginning with the highest value. These values are averaged for the final values. In this process, the half (the difference between the top and the middle and the middle and the bottom crosshairs) and the total stadia intervals are determined and recorded. Any time that the half-thread intervals do not agree to within 0.003 meter, reobserve all readings. If the difference after reobservation is exactly 3 millimeters, the divergence is acceptable. A note should be added in the remarks column that the foresight or backsight observation was repeated.

(1) Check the readings taken on the Invar strip on the front of the rod by reading the center crosshair on the back of the rod to the nearest hundredth of a foot. The mean of the three thread readings, converted to feet, must check within 0.02 foot.

(2) Instrument stations are numbered consecutively for a day's work. At odd-numbered instrument stations, take the readings at the backsights first; at even-numbered stations, take the foresight reading first. In this method, the same rod is held on a turning point for both the foresights and backsights, so the same rod is read first at each setup. It is the one used for the backsights at the first instrument station of the day's work. Use leveling platforms or metal pins driven into the ground for turning points.

(3) Sections are named in their forward direction. *Forward* or *backward* denotes the direction in which the level line is run. The backward measurement on any section of a line running in both directions should be made under different atmospheric conditions from those on the forward measurement.

(4) Shade the instrument from the direct rays of the sun at all times. Since the wind and the sun are possible sources of error, record their direction and intensity in the level book in the spaces provided at the top of the page (as shown in Figure 4-7, (2) and (4)). An explanation of the numbers used after the words sun and wind is shown in Figure 4-8, page 4-18.

THREE-WIRE LEVELING

For use of this form, see FM 3-34.331; the proponent agency is TRADOC.

INST. OP. INT.	1st COMP. INT.	2nd COMP. INT.

PROJECT	LOCATION	ORGANIZATION
30/FBV/2LL	Ft. Leonard Wood, MO	30th Engr Bn

OBSERVER	RECORDER	INSTRUMENT	SUN 2	WIND 4	WEATHER
SFC J. Doe	SPC W. Roe	Wild N3 #1268	1	0 to 1	Calm and warm

FROM 3	TO	DATE (YYYYMMDD) 1	TIME 5	LINE OR NET	PAGE NO.	NO. OF PAGES
RM S-1	TBM S-1	20010709	0900	Main line Net "S"	4	4

STATION 6	BACKSIGHT FACE OF ROD 7	MEAN 8	BACK OF ROD 9	INTERVAL 10	SUM OF INTERVALS 11	FORESIGHT FACE OF ROD 13	MEAN 14	BACK OF ROD 15	INTERVAL 16	SUM OF INTERVALS 17	REMARKS 12 18
RM S-1	1292			273'		3031					Rod #778 (R)
	1019 (R)	10193'	3.34'	272'	545'	2750	27503'	9.02'	281'		Temp 27°C
	0747					2470			280'	561'	
											Rod #780
	0718					2779					Temp 26°C
	0487	04873'	1.60'	231'		2556 (R)	25563'	8.39'	223'		
	0257			230'	461'	2334			222'	445'	
	19	15066'	20 4.94'		21 1006'		53066'	17.41'		1006'	
	3001					0955					
	2692 (R)	26917'	8.83'	309'		0619	06193'	2.03'	336'		
	2382			[illegible]	619'	0284			335'	671'	
		41983'	13.[illegible]		[illegible]		59259'	19.44'		1677'	
	2896					1870					
	2536	25360'	8.32'	360'		[illegible]02 (R)	15020'	4.93'	368'		
	2176			360'	[illegible]	1134			368'	736'	
		67343'	22.09'		2345'		74279'	24.37'		2413'	
	2977					0861					Rod #778
	2612 (R)	26117'	8.57'	365'		0530	05300'	1.74'	331'		Temp 28°C
	2246			366'	731'	0199			331'	662'	Rod #780
22	28038'	93460'	30.66'		3076'	23874' 26	79579' 29	26.11' 31		33 3075'	Temp 27°C
		23	24		25	28038'	93460'	30.66'		3076'	TBM S-1 35
					3/	+4164'	+1.3881	4.55'		6151' =	615.1m' 34
					27	+1.3880' 28	30	1.39' 32			
					Stradia	constant	=	0.100'			

JD

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Figure 4-7. Sample DA Form 5820 (Recording Procedure, Main Line, 0.100 Stadia Constant)

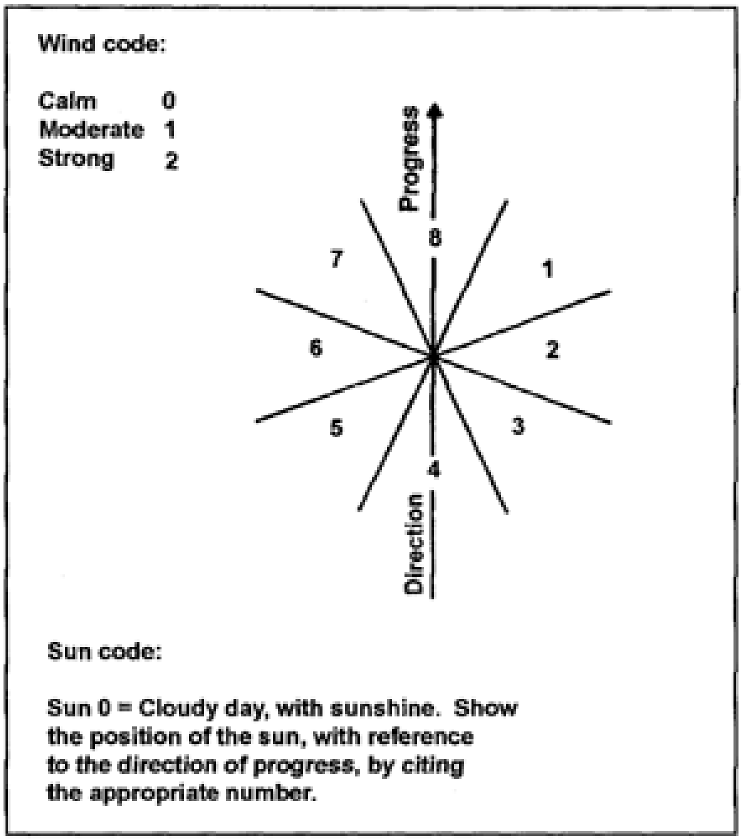


Figure 4-8. Sun and Wind Code

(5) The temperature, to the nearest degree centigrade as shown by the rod thermometer, is read and recorded for the rod coming up from the first backsight at the beginning of each section (as long as the leveling is continuous). When leveling is interrupted for a considerable length of time, make an additional temperature reading and record it at the end of the continuous leveling. If the interruption occurs during the running of a section, read and record the temperature when the leveling is resumed. If the thermometer readings of the two rods differ, record the number and readings of both rods, beginning with the rod in the backsight position.

(6) The maximum length of sight is 75 meters, and this length is used only under the most favorable conditions. The length of sight should be kept as long as possible and still stay within closure limits. Short lines of sight require additional setups and slow progress, but long sight lines increase closure errors and require reruns. A convenient rule to remember when fixing the length of a sight is to shorten the sights when difficulty is encountered in keeping the upper and lower intervals subtended on the rod within 0.003 meter.

(7) The difference in length between the foresight and the backsight distances at any instrument station must not exceed 10 meters. The continuous sum of these distances must not differ by more than 20 meters for any section or at any tie point. Lines of sight should be close enough in length so that it is not necessary to refocus between the two sights of an instrument station. Errors due to imperfect level adjustment can be nearly eliminated if the lengths of the foresight and the backsight of each section are nearly equal.

(8) When a leveling line is complete to a point which is to hold the elevation of the line overnight or longer, establish at least two points with at least one instrument setup apart. When the leveling is continued from or to such a pair of points, set up the instrument midway between them, and take rod readings on each point to determine if either of them has been disturbed. If the new difference does not agree with the first determination, leveling must be extended to other points to verify the stability of the point used in carrying the elevation ahead. When establishing a pair of points, decide which one is to be the TBM and which one is to be the checkpoint. To avoid confusion, carry the elevation forward over the point selected as the TBM. The difference in elevation between the points of the pair should be at least 0.3 meter. Then, should confusion occur in carrying the elevation ahead through the line, any blunder would be so large that it would stand little or no chance of remaining undetected when the line is tied out or a circuit is closed.

(9) Permanent benchmarks should be established at about 2-kilometer intervals throughout the line. When needed, TBMs should be placed between PBMs. Depending upon the terrain and the control to be extended from the line, it may be necessary to break up the line into shorter sections so that it will only be necessary to rerun a small section of the line if it busts.

b. Recording Precise (Spirit) Leveling Data. All observations of either first- or second-order leveling are recorded in the field book, as shown in Figure 4-7, page 4-17. The circled numbers in the figure correspond to the numbers shown in the following recording procedures:

(1) Enter the date.

(2) Enter the direction of the sun, with respect to the direction of the running of the line. If conditions are cloudy, the abbreviation should be 0 (refer to Figure 4-8).

(3) Enter the designation of the level-line section.

(4) Enter the strength of the wind (refer to Figure 4-8).

(5) Enter the time.

(6) Number the instrument stations (not turning points) consecutively throughout the day.

(7) Record the backsight readings to the nearest 0.001 meter.

(8) Carry the mean reading to one more place than the thread reading. The mean is the average of the three readings for each rod reading.

(9) Enter the reading of the middle thread on the back of the rod to the nearest 0.01 foot.

(10) Enter the interval between the upper and middle threads and the interval between the middle and lower threads.

(11) Enter the total interval between the upper and lower threads or the sum of the two half intervals.

(12) Record the rod number and temperature for the first backsight in each section. If it is found that the readings of the two thermometers differ, the number of each rod and its temperature should be recorded, with the reading for the backsight recorded first (specify Centigrade or Fahrenheit).

(13) Record the foresight readings to the nearest 0.001 meter.

(14) Enter the mean of the three thread readings for each foresight.

(15) Enter the reading of the middle thread on the back of the rod to the nearest 0.01 foot.

(16) Perform the procedures as for (10) above, but for the foresight readings.

(17) Perform the procedures as for (11) above. This sum must not differ from the backsight sum (11) of any one instrument setup by more than 30 intervals (when using a level with a stadia constant of 1/300) or by more than 33 intervals (when using a level with a stadia constant of 1/333). The 30-interval and 33-interval figures are equivalent to 10 meters of distance *(30 x 0.333 = 9.99 or 10 meters; 33 x 0.300 = 9.9 or 10 meters)*.

(18) Enter any other pertinent information in this column.

(19) Enter the sum of the mean thread readings continuously for columns (8) and (14).

(20) Enter the sum of the back of the rod readings continuously for columns (9) and (15).

(21) Enter the sum of the intervals continuously down columns (11) and (17).

(22) Enter the sum of all backsight readings.

(23) Enter the sum of mean backsight readings.

(24) Enter the sum of the back of rod backsight readings.

(25) Enter the final sum of the backsight thread intervals.

(26) Enter the sum of all foresight rod readings.

(27) Enter the difference between (22) and (26), and divide this difference by 3. In this case, the backsight readings are larger; otherwise, the value in (22) would be carried over to the right-hand page and subtracted from (26).

(28) Enter the quotient. The quotient is a check against the difference of elevation obtained by subtracting (29) from (23) and should agree within one or two millimeters.

(29) Enter the sum of the mean foresight readings.

(30) Enter the DE obtained by subtracting (29) from (23) with proper sign (backsight - foresight = DE).

(31) Enter the sum of the back of the rod readings for the foresight rods.

(32) Enter the difference in elevation in feet. Obtain this figure by subtracting (31) from (24) and multiplying by 0.3048 to get the difference in meters. This value should agree approximately with the DE shown in (30).

(33) Enter the final sum of the foresight thread intervals. This sum must not differ from the backsight sum (25) by more than 20 meters (60 intervals for a level with a stadia constant of 1/300 and 67 intervals for a level with a stadia constant of 1/333).

(34) Enter the length of section in kilometers. This is the sum of (33) and (25) multiplied by the stadia constant of the instrument and divided by 1000.

(35) Enter the final foresight. This reading was taken on benchmark 2.

c. Instructions for Obstacle Crossings. When crossing a river, ravine, or other obstacle requiring a line of sight greater than 75 meters, make simultaneous reciprocal observations from each of two points, one on each side of the obstacle. Ensure that backsights and foresights are balanced, and establish a TBM near the edge of the obstacle. Establish another TBM on the far side of the obstacle, at about the same elevation. Place rods, equipped with two targets each, on each of the TBMs. Place the instruments, one on each side of the obstacle, where they can be directed on both rods and form a closed-circuit crossing as shown in Figure 4-9, page 4-22.

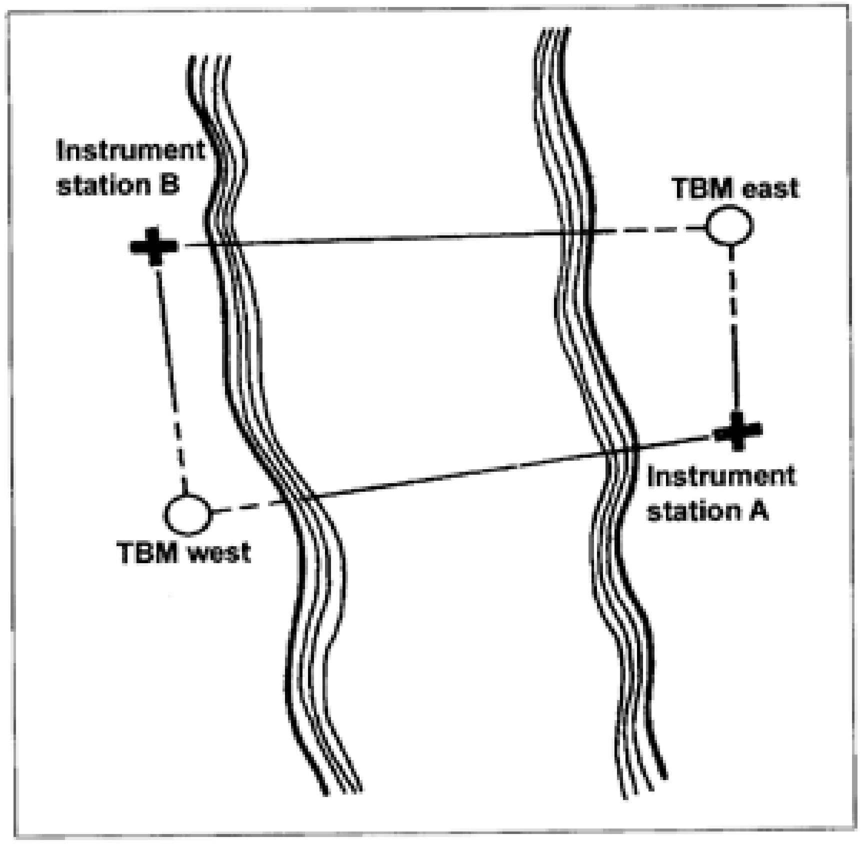


Figure 4-9. Closed-Circuit River Crossing

(1) Two targets are set on each rod, one above and the other below the point on the rod intersected by the center crosshair of the instrument when the bubble is leveled. Read the drum scale with the telescope in three positions: first, with the middle thread bisecting the top target; second, with the telescope level (bubble centered); and third, with the middle thread bisecting the bottom target. The reading on the rod for a level line of sight can be determined, since the distance from the level line of sight to the two targets will be proportional to the corresponding differences in the readings on the drum scale.

(2) In Figure 4-10, A and B are the positions of the upper and lower targets on the rod, and C is the point where the leveled line of sight intersects the rod. Assume that the readings of the drum scale are 25.8 and 40.2 for A and B respectively, and the reading at C (telescope level) is 30.6. If the targets (A to B) are 0.4000meter apart, the distance from target A to point C can be found in the following formula:

$$\frac{30.6 - 25.8}{40.2 - 25.8}$$

$$(.4000) = 0.1333 \text{ meter}$$

If the setting of target A is 1.8930 meters, then the line of sight C intersects the rod at 1.7597 meters (1.8930 - 0.1333).

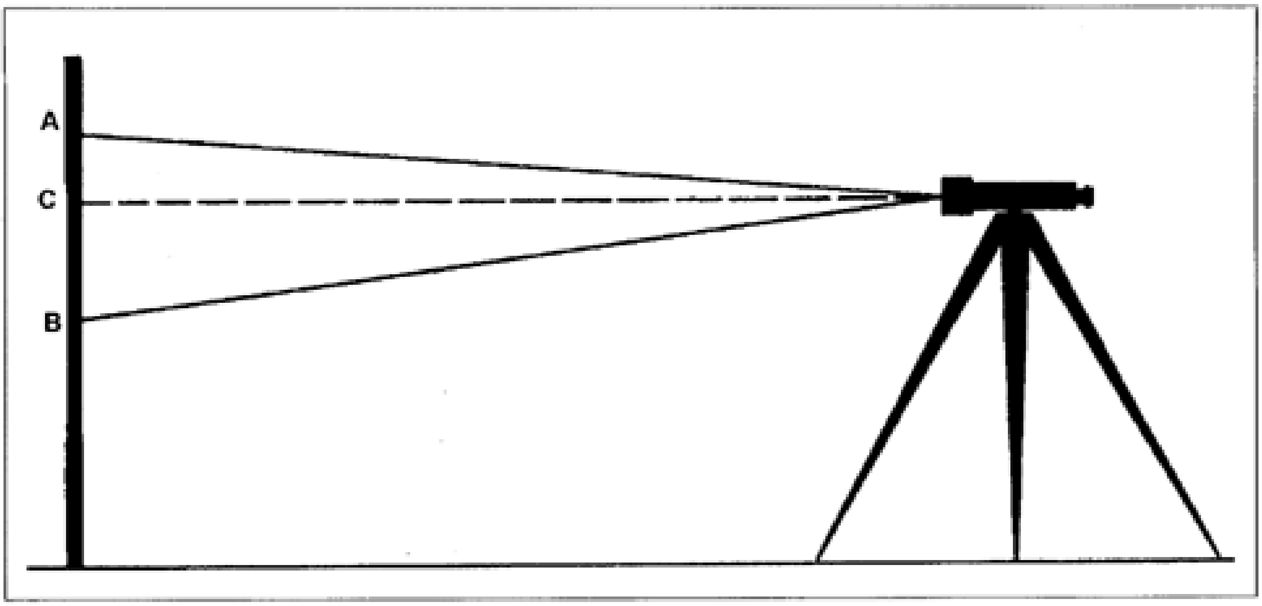


Figure 4-10. Principle of Method For Crossing Obstacles

(3) The instruments should be adjusted as accurately as possible. Refer to the sample DA Form 5820 at Figure 4-11, and use the following procedures to ensure accurate readings:

(a) Read and record the near rod once for each set of 25 readings on the distant rod. For each of the 25 readings, the drum scale is read three times and recorded. First, set the center thread on the top target (A) and read the scale. Second, center the bubble and read the scale at C. Lastly, set the center thread on the bottom target (B) and read the scale. These three readings of the drum scale should be repeated in the above order until at least 25 have been obtained. The observers then change places, taking their instruments with them but leaving the tripods.

(b) Two days' observations are required. Each day's observations consist of four sets, two per observer at each station. These sets should be obtained in the following order: Assume that A and B are the two stations (refer to Figure 4-9, page 4-22). Observer one takes a set of readings at station A, transports his instrument to station B, takes two sets, and returns to station A to take another set. Observer 2 carries out a similar program, beginning at station B.

THREE-WIRE LEVELING

For use of this form, see FM 3-34.331; the proponent agency is TRADOC.

INST. OP. INT.	1st COMP. INT.	2nd COMP. INT.

PROJECT	LOCATION	ORGANIZATION
30/FBV/2LL	Ft. Leonard Wood, MO	30th Engr Bn

OBSERVER	RECORDER	INSTRUMENT	SUN	WIND	WEATHER
SFC J. Doe	SPC W. Roe	Wild N3 #1268	1	0	Calm and warm

FROM	TO	DATE (YYYYMMDD)	TIME	LINE OR NET	PAGE NO.	NO. OF PAGES
BM 5-6	BM 5-7	20010707	1630	River crossing, line 5	1	1

STATION	BACKSIGHT FACE OF ROD	MEAN	BACK OF ROD	INTERVAL	SUM OF INTERVALS	FORESIGHT FACE OF ROD	MEAN	BACK OF ROD	INTERVAL	SUM OF INTERVALS	REMARKS
BM 5-6	0844					1	22.8	49.0	72.8		Rod #718 (red)
	0833R	08330	2.73	11		2	22.8	48.8	72.8		Temp 24°C
	0822			11	22	3	22.8	48.7	72.8		
	2499					4	22.8	48.8	72.8		Rod #780
						5	22.7	48.6	72.5		Temp 21°C
						6	22.5	48.4	72.4		
						7	22.3	48.6	72.5		
						8	22.8	48.7	72.6		
						9	22.5	48.1	72.3		
Bottom target	at 1.430m					10	22.5	48.0	72.1		
						11	22.2	48.1	72.0		
Top target at	1.930m					12	22.1	48.0	72.2		
						13	22.1	48.0	72.1		
						14	22.1	48.1	72.0		
						15	22.4	48.2	72.5		
BM 5-6	(East Side	River)				16	22.1	48.1	72.1		
BM	(West Side	River)				17	22.2	48.1	72.1		
						18	22.1	48.1	72.1		
						19	23.1	48.1	72.1		
						20	23.1	48.1	72.3		
						21	23.2	48.1	72.0		
						22	22.3	48.0	72.1		
						23	22.2	48.0	72.1		
						24	22.1	48.2	72.1		
						25	22.2	48.2	72.2		
						26	22.1	48.0	72.0		
						Σ	9.2	6.7	7.6	BM	
							22.354	48.258	72.292	5-7	

DA FORM 5820, JUL 2001 EDITION OF AUG 89 IS OBSOLETE. USAPA V1.00

Figure 4-11. Sample DA Form 5820 (Recording an Obstacle-Crossing Observation

(4) The program of observation is arranged in order to avoid all unnecessary changing of focus. At the first station, the observer should read the near rod before taking the first set of 25 observations. At the second station, the observer should make the near rod reading between the two sets of observations. The day's program for each observer is as follows: read the near rod and take a set of observations; move to the second station, take a set of observations, read the near rod, and take a set of observations; return to first station, take a set of observations, and read the near rod.

(5) Observations should begin and end at the same time. A faster observer should continue reading until the slower observer has 25 readings. Both observers should stop at the same time.

(6) The DE for each set of 25 observations per observer is computed, and the comparable sets from both observers are averaged. Sets 1 and 2, and sets 3 and 4 are then paired for a mean. The mean of pairs must agree within the AE for the length of section, as previously discussed under the criteria for first- and second-order leveling. The means of pairs are combined for a mean of sets (one day's observations), which must agree with the second day's mean of sets, within the AE. The two days' observations (mean of sets) are averaged to obtain the difference in elevation between the TBMs.

PART C: TRIGONOMETRIC LEVELING

4-10. General. Based upon the accurately determined elevation of an initial point, the elevations of all stations in a triangulation system can be determined by measuring the vertical angles between the stations. Apply the fundamentals of trigonometry, and use these vertical angles to compute the differences in elevations. This process is known as *trigonometric leveling*.

a. Trigonometric leveling is usually a part of triangulation or traverse work, with the transit or theodolite used to determine the vertical angle and the measured distance. The determinations by trigonometric leveling are checked at reasonable intervals by a connection with lines run by instrumental leveling. In order to provide starting and check elevations, a complete scheme of vertical-angle observations should be carried through all triangulation.

b. This complete scheme consists of a continuous series of vertical angles measured through the main scheme of triangulation, observations made on each line over which horizontal angles are observed (the observations over each line to be made in both directions if both ends of the line are occupied), and observations of vertical angles upon all supplementary and intersection stations corresponding to the horizontal angles measured upon such stations.

c. As you recall, a vertical angle, as measured with a transit, is the angle measured vertically up or down from a horizontal plane of reference. When the telescope is pointed in the horizontal plane (level), the value of the vertical angle is 0. When the telescope is pointed up at a higher feature (elevated), the vertical angle increases from 0 and is called a plus vertical angle. These values increase from 0° to +90° when the telescope is pointed straight up. As the telescope is depressed (pointed down), the angle also increases in numerical value. A depressed telescope reading showing that it is below the horizontal plane is called a minus vertical angle. These numerical values increase from 0° to -90° when the telescope is pointed straight down.

d. To determine the difference in elevation between two points, set up and level the instrument at one point. Hold the rod at another point. Point the telescope at an easily read value (a full meter) on the rod, and measure the vertical angle (Figure 4-12). Determine the horizontal distance (ED) between the instrument and rod by taping, obtaining a stadia reading, or by triangulation. You may also determine the slope distance (EB) using EDME. Whichever distance (ED or EB) is selected, one side and one angle (α) of a right triangle should be determined. Using this information, compute the other sides and angle. For trigonometric leveling, only the side opposite the measured angle, the difference in elevation, is computed.

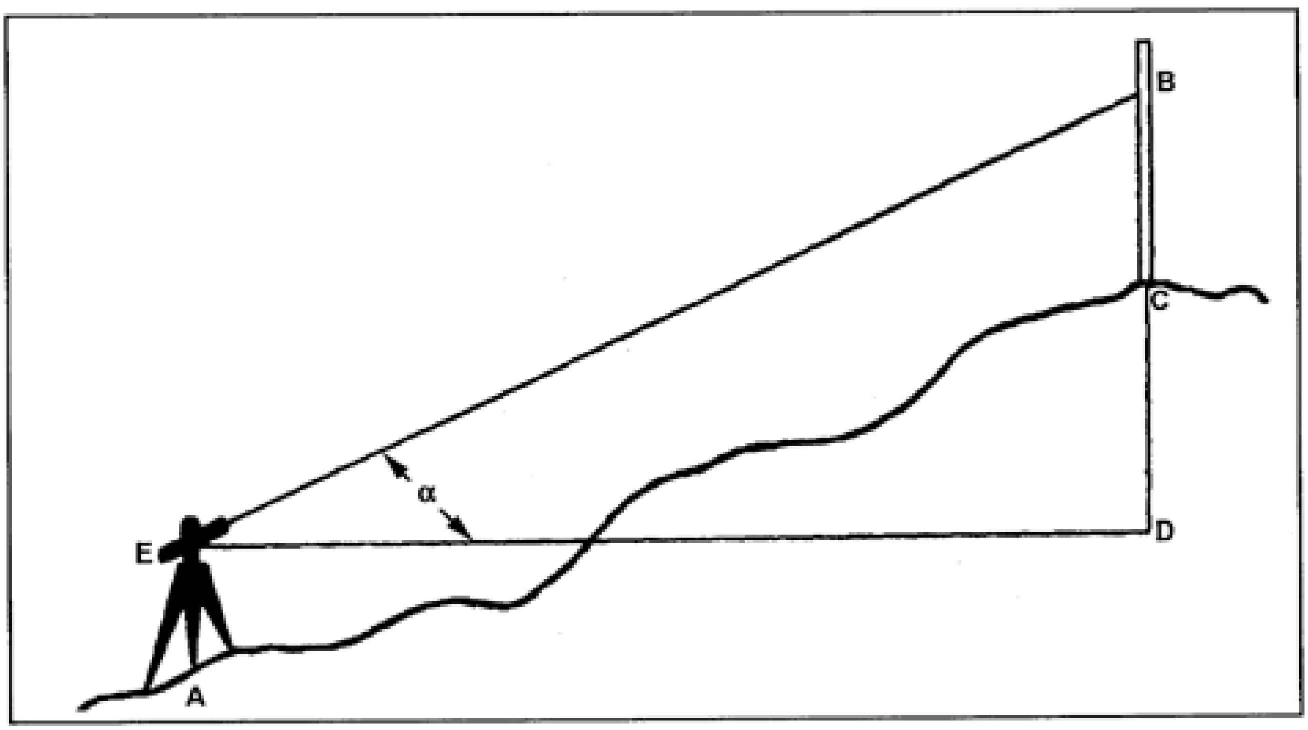


Figure 4-12. Trigonometric Leveling With the Telescope in an Elevated Position

(1) The computation consists of multiplying the measured distance by the proper trigonometric function of the measured angle (sine, if the slope distance [EB] is measured; tangent, if the horizontal distance [ED] is measured). Where AE is the height of the instrument above point A, and BC is the height of the line of sight above point C, the difference in elevation between A and C is AE + BD - BC.

(2) This method of determining the difference in elevation should be limited to horizontal distances less than 300 meters when moderate precision is sufficient and to proportionately shorter distances when higher precision is required.

Beyond 300 meters, the effect of curvature and refraction must be considered and the necessary corrections applied.

4-11. Methods of Trigonometric Leveling. The difference in elevation between two stations, whose distance apart is known, can be determined using one of the following methods:

a. Reciprocal Observations. During the method of reciprocal observations, observations are performed at both stations, either simultaneously or not simultaneously. First, determine the difference in elevation from vertical angles observed from one station to the other. The object of such reciprocal observations is to remove the effect of uncertainty regarding the value of the coefficient of refraction. This effect will balance out when the observations are taken in both directions and are more complete if the observations are taken simultaneously. On triangulation work where each station is occupied consecutively and additional observers are not available, this method of reciprocal observations is usually used, but no attempt is made to observe in both directions simultaneously. The measurements are sometimes made at the time of minimum refraction on different days but with less accurate results.

b. Nonreciprocal Observations. During nonreciprocal observations, the difference of elevation from the vertical angle measured at only one of the stations is commuted. The value of the refraction must be known. This method is available to determine the elevation of the station occupied by observation to a point of known elevation and elevations of inaccessible and intersected points.

4-12. Zenith Distances With the T-3 Theodolite. In first- and second-order triangulation, zenith distances are obtained using a Wild T-3 theodolite or a theodolite of similar accuracy.

a. The *zenith* is an imaginary point overhead where an extension of the plumb line intersects an assumed sphere on which the stars appear projected. The equivalent point that is directly below the zenith is called the *nadir*. The zenith distance is the vertical angle from the zenith, or a point directly overhead, to the sighted point. The zenith permits reading angles in a vertical plane without using a plus or minus. Vertical-angle measurements with the T-3 read elevation (plus) angles as values less than 900 and depression (minus) angles as values greater than 900.

b. Observe zenith distances between 1200 and 1600 hours whenever possible, as vertical refraction is at its minimum and more constant during these hours. Confine angle observations to this period, the duration of which varies in area. In general, good observations are possible from 1000 to 1600 hours.

c. Over longer distances, the preferred method of obtaining observations is with heliotropes or other visible objects at the stations. When it is not possible to observe zenith distances during the recommended hours, make observations at night using lights. To obtain the best results, make reciprocal observations from two stations. However, since all stations cannot be occupied with instruments simultaneously, it in recommended that zenith distances be observed at about the same hour. In this way, the observations are made from both stations under conditions of refraction that are similar. It is especially important to adhere to this procedure as closely as possible when making observations at night, as the effects of refraction are large and variable. Combining day and night observations as reciprocals usually produces poor results and should be avoided whenever possible.

d. Zenith distances should be observed on all intersection stations and computed as nonreciprocal observations.

e. Due to the peculiar method used in the graduation of the vertical circle of the T-3, the manner in which the zenith distance is determined is difficult to visualize and understand. To make the procedure more understandable, a vertical circle is illustrated in Figure 4-13. Circle right or circle left refers to the position of the vertical circle in relation to the observer.

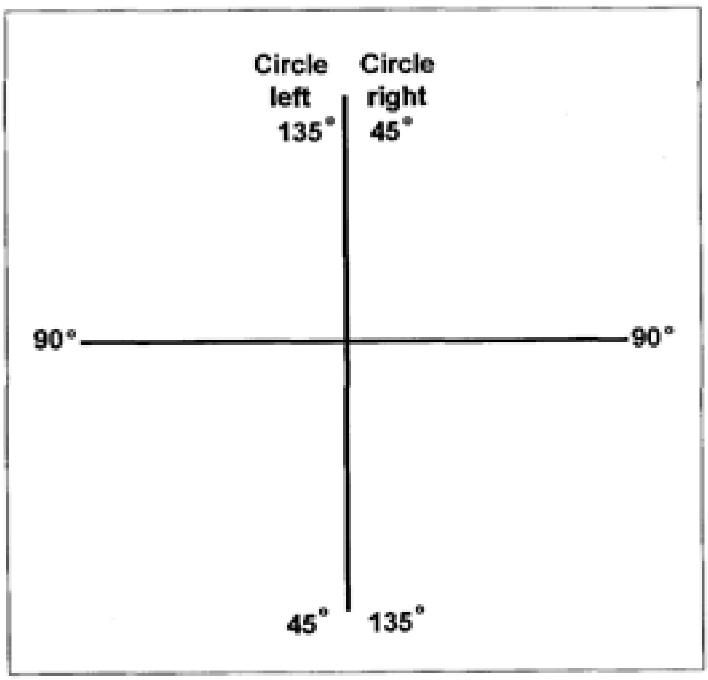
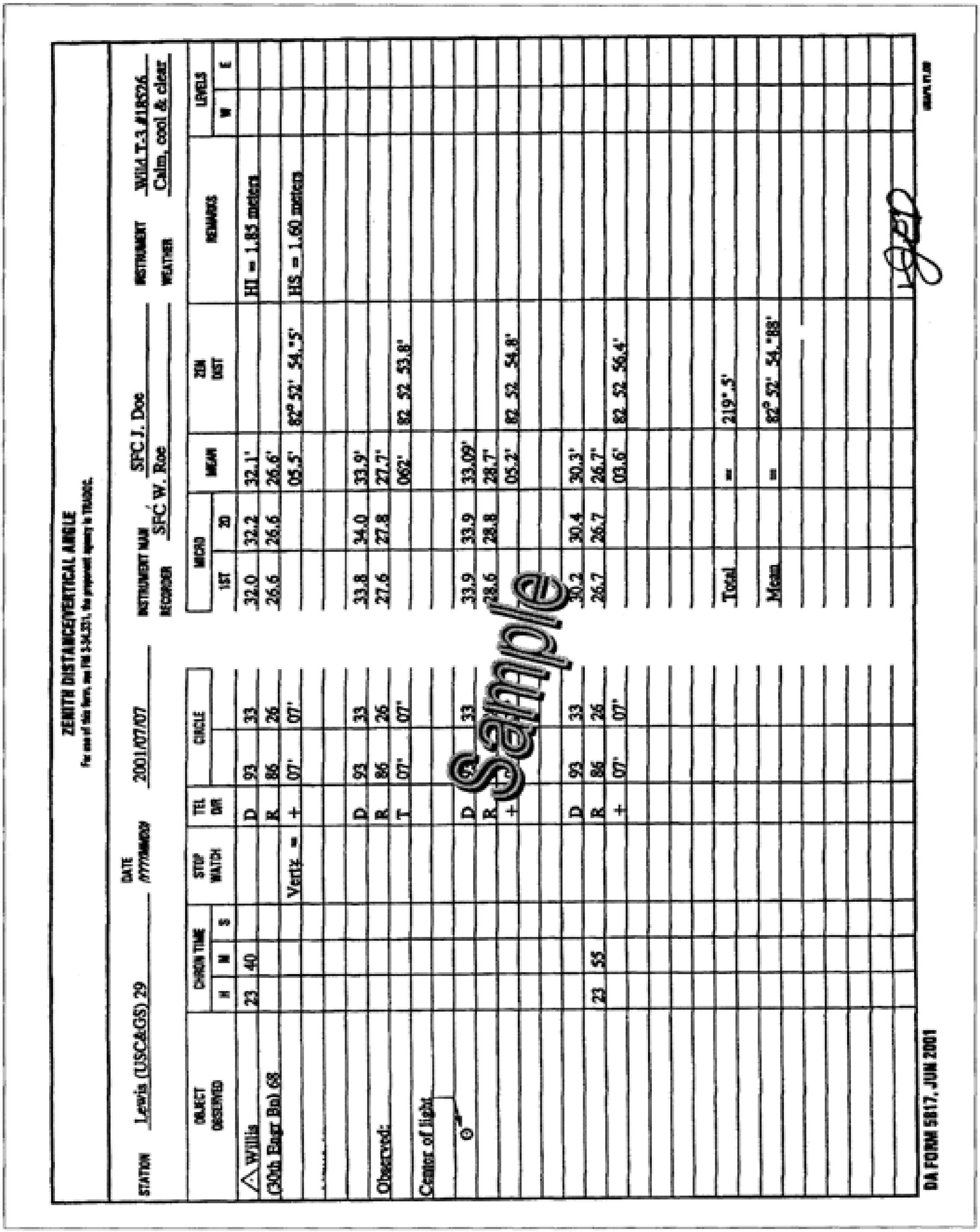


Figure 4-13. Vertical Circle

The vertical circle is half numbered. The arrangement of the graduations for both the left and right positions give the following formulas for the zenith distance and the vertical angle from the horizontal plane.

Zenith distance = *(circle right + 90°) - circle left*
Vertical angle = *circle left - circle right*

4-13. Recording Zenith Observations. A sample DA Form 5817 of recorded zenith observations is shown in Figure 4-14. Refer to this figure and follow the procedures for completing the entries.



ZENITH DISTANCE/VERTICAL ANGLE

For use of this form, see FM 3-34.331, the proponent agency is TRADOC.

STATION Lewis (USC&GS) 29 DATE (YYYYMMDD) 2001/07/07 INSTRUMENT MAN SFC J. Doe INSTRUMENT Wild T-3 #18526

RECORDER SFC W. Roe WEATHER Calm, cool & clear

OBJECT OBSERVED	CHRON TIME H	M	S	STOP WATCH	TEL D/R	CIRCLE		MICRO 1ST	2D	MEAN	ZEN DIST	REMARKS	LEVELS W	E
△ Willis	23	40			D	93	33	32.0	32.2	32.1'		HI = 1.85 meters		
(30th Engr Bn) 68					R	86	26	26.6	26.6	26.6'				
				Vertz =	+	07'	07'			05.5'	82° 52' 54."5'	HS = 1.60 meters		
					D	93	33	33.8	34.0	33.9'				
Observed:					R	86	26	27.6	27.8	27.7'				
					T	07'	07'			062'	82 52 53.8'			
Center of light ⊙					D		33	33.9	33.9	33.09'				
					R			28.6	28.8	28.7'				
					+					05.2'	82 52 54.8'			
					D	93	33	30.2	30.4	30.3'				
	23	55			R	86	26	26.7	26.7	26.7'				
					+	07'	07'			03.6'	82 52 56.4'			
								Total		=	219".5'			
								Mean		=	82° 52' 54."88'			

DA FORM 5817, JUN 2001

Figure 4-14. Sample DA Form 5817 (Recorded Zenith Observations)

a. Complete the heading, as shown.

b. In the object observed column, identify the name of the station and the type of target observed (such as light or heliotrope).

c. Record the time the observations began in the time column.

d. Record the readings of the circle right (reverse) and circle left (direct). Read the vertical circle, record the degrees and minutes in the circle column, and make and record the two coincidence readings of the micrometer drum under the columns headed first and second. Add the two coincidence readings, and record the average in the mean column. Compute the value of the zenith distance by adding 90° to the total circle right value and subtracting the total circle left value. The result should be noted in the zenith distance column. This constitutes one position. At least three positions within a range of 10 seconds are required. The instrument should be reversed between circle left pointing and circle right pointing for *each* position. The recorder should remind the observer to level the vertical circle bubble before each pointing.

e. Record the heights of the stand and the instrument in the remarks column. If the heights of the lights or other equipment change during the occupation of the station, they should be noted and dated. Note any other pertinent data.

f. Check all computations and measured distances, and initial each page.

4-14. Abstract of Zenith Distances. An example of an *abstract of zenith distances* is based on the quadrilateral shown in Figure 4-15. A sample DA Form 1943 is shown at Figure 4-16. The necessary data for the abstract are transcribed from the record books, with the exception of steps (6), (7), and (10), which are computed. The circled numbers in Figure 4-16 correspond to the following steps:

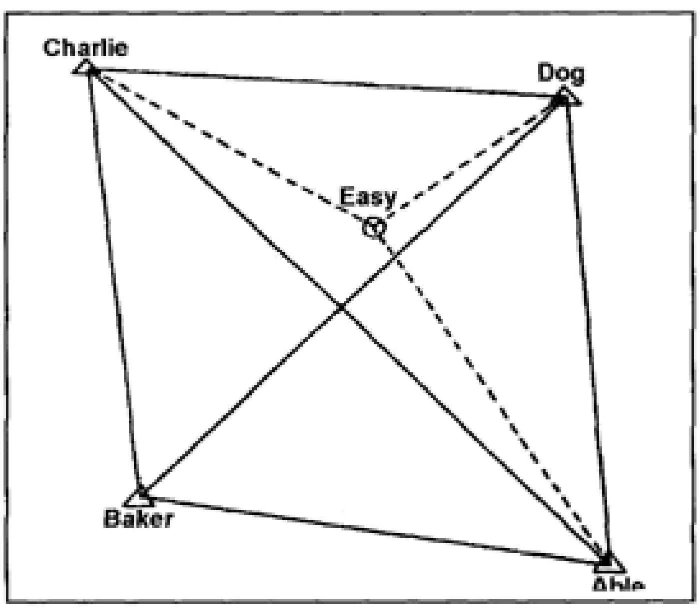


Figure 4-15. Quadrilateral

PROJECT 30E 15371TR12					ABSTRACT OF ZENITH DISTANCES For use of this form, see FM 3-34.331; the proponent agency is TRADOC.		
LOCATION VA		INSTR. (TYPE) (NO.) T-3 #5406		STATION Lewis (USC&GS) 29			
ORGANIZATION 30th Engr Bn		OBSERVER SFC J. Doe				HEIGHT OF STAND 1.60M	

DATE (YYYYMMDD) ①	TIME ②	OBJECT OBSERVED ③	OBJECT ABOVE STATION + - (Meters) ④	TELESCOPE ABOVE STATION + - (Meters) ⑤	DIFF. OF HEIGHTS + - (Meters) ⑥	REDUCTION TO LINE JOINING STATIONS ⑦	OBSERVED ZENITH DISTANCE ⑧	CORRECTED ZENITH DISTANCE
20010714	2340	Willis	0.61	1.85	+1.24	-6.84	82 52 54.5	° ′ ″
		(30th Engr) 68					53.8	
		Observed Center					54.8	
	2355	of light				⑨	56.4	⑩
						MN	82 52 54.88	82 52 48.04
20010714	0005	King	1.93	1.85	-0.08	+0.53	97 04 54.5	
		(30th Engr) 68					53.5	
		Observed Center					55.1	
	0020	of light					52.3	
						MN	97 04 53.85	97 04 54.38
20010714	0030	James	9.37		52	+82.36	90 06 58.6	
		(USC&GS)					(07 05.7)	Ro
		Observed Center					06 54.5	
		of light					06 55.7	
	0045						06 56.8	
						MN	90 06 56.40	90 08 16.76

Sample

DATE (YYYYMMDD)	LIGHT SHOWN TO STATION ⑪	HEIGHT OF LIGHT* ABOVE STATION (Meters)	DATE (YYYYMMDD)	LIGHT SHOWN TO STATION	HEIGHT OF LIGHT* ABOVE STATION (Meters)
20010714	Willis	0.83	20010715	Willis	2.65
"	King	2.36	"	King	2.65
"	James	8.76	"	James	8.76

*Height of Light (or object above station) should also be entered on Abstract of Zenith Distances of station to which light was shown.

COMPUTED BY	DATE (YYYYMMDD)	CHECKED BY	DATE (YYYYMMDD)
SFC W. Coe	20010715	SFC J. Doe	20010715

DA FORM 1943, JUL 2001 — EDITION OF FEB 57 IS OBSOLETE. — USAPA V1.00

Figure 4-16. Sample DA Form 1943 (Abstract of Zenith Distances)

a. (1) Enter the date, as in the sample.

b. (2) Enter the time.

c. (3) Enter the name of the observed station and indicate whether the object sighted was a light, a heliotrope, the top of the instrument stand, the top of the tower, the top of a mountain, the top of a stack, a tank, a cupola, or spires of intersection stations.

d. (4) Enter the height of light, above or below the monument, at the observed station.

e. (5) Enter the height of the telescope above the monument at the observing station.

f. (6) Compute the difference of heights by subtracting step (4) from step (5).

g. (7) The *reduction* to line-joining stations is applied to all reciprocal observations and is computed using the following formula:

$$r = \frac{t - o}{S \sin 1''}$$

where—

r = *reduction in seconds*
$t - o$ = *difference of heights*
S = *distance in meters between the stations involved*

For example:

Station occupied = *ABLE*
Station observed = *CHARLIE*

$t - o$ = *3.781 meters*
S = *22,584 meters*
$\sin 1''$ = *0.000004848*

therefore— r = $34''.5$

The sign of the *r* value in this column is opposite from the sign of step (6) when zenith distances are used.

h. (8) Enter the observed zenith distance values directly from the record book.

i. (9) Enter the mean value of the observed angles in step (8). At least three values must be obtained within a range of 10 seconds.

j. (10) Obtain this value by applying the *reduction* from step (7) to the mean value in step (9).

k. The data on the bottom portion of the form shows the dates and heights for all objects and lights shown to other stations from the observation station in the heading. These values are obtained from the field books.

Transferred data and computations from the record book to the abstract must be checked and initialed. Whenever nonreciprocal observations are made, the computation of the reduction to line and the determination of corrected zenith distance are not required.

LESSON 4

PRACTICE EXERCISE

The following items will test your grasp of the material covered in this lesson. There is only one correct answer for each item. When you have completed the exercise, check your answer with the answer key that follows. If you answer any item incorrectly, study again that part which contains the portion involved.

1. As in all topographic or geodetic surveys, the observations and computations have a known basis to work on or have a reference datum on which to work. In geodetic leveling this reference datum, which is universal, is known as ______________.

   A. Clarke's ellipsoid
   B. The North American datum
   C. The MSL
   D. The European datum

2. _______________________ are permanent material objects, natural or artificial, bearing a marked point whose elevation above or below the adopted datum

   A. Triangulation stations
   B. TBMs
   C. Benchmarks
   D. LaPlace stations

3. The primary difference between second-order, Class I and Class II lines is that Class II lines ____.

   A. Are run in one direction only
   B. Have an AE of ±8.4
   C. Are used for distances under 40 kilometers
   D. Are used in easily accessible areas

4. Precise levels (first- and second-order instruments) have the ability to allow a reading to the nearest ____.

   A. 0.001 of a unit
   B. 0.01 of a unit
   C. 0.1 of a unit
   D. 1.0 of a unit

5. In a three-wire, precise leveling operation always____.

A. Read the instrument scale first
B. Read the top hair first
C. Read the middle hair first
D. Read the bottom hair first

6. The stadia constant is a numerical value calculated for each instrument. It is used to____.

A. Find the limits of the instrument's telescopic capability
B. Find the amount of error inherent in the instrument
C. Obtain the length of the sections being observed
D. Determine if the precise level rods have any gross error and how much

7. In determining the stadia constant, if any of the individual stadia constant computations (one at each of the six stations) are not within the allowable limits of the mean of the six constants, the readings must be taken again. What is the limit?

A. 2.0 from the mean
B. 0.2 from the mean
C. 0.02 from the mean
D. 0.002 from the mean

8. The value given as the stadia constant of an instrument is entirely dependent on the____.

A. Power of the telescope
B. Spacing of the crosshairs
C. Type of precise rod used on the survey
D. Type of reticle installed in the instrument

9. If the deviation from the vertical in either direction on a precise level rod exceeds _____________________________ on a 3-meter length, adjust the rod level.

A. 4 millimeters
B. 8 millimeters
C. 10 millimeters
D. 12 millimeters

10. In three-wire leveling, the half-thread (wire) intervals must agree within what distance____.

A. 0.002 inch
B. 0.003 meter
C. 0.02 meter
D. 0.03 meter

11. The maximum length of site to observe first- and second-order, three-wire leveling is____.

A. 75 feet
B. 75 meters
C. 150 yards
D. 150 meters

12. Balancing the lengths of backsights and foresights is very desirable in leveling because it eliminates errors due to____________________________________.

A. Parallax
B. Reading the rod incorrectly
C. Refraction
D. Imperfect level adjustment

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LESSON 4

PRACTICE EXERCISE

ANSWER KEY AND FEEDBACK

Item		Correct Answer and Feedback
1.	C	MSL The datum universally…(page 4-1, Introduction)
2.	C	Benchmarks Benchmarks are permanent…(page 4-2, para 4-2)
3.	A	Are run in one direction only The criteria for Class II…(page 4-4, para 4-3b[2])
4.	A	0.001 of a unit A graduated tilting screw…(page 4-5, para 4-5a)
5.	D	Read the bottom wire first Read the bottom hair…(page 4-9, para 4-5b)
6.	C	Obtain the length of the sections being observed It is the factor by which…(page 4-10, para 4-8a)
7.	D	0.002 from the mean If any of the six individual…(page 4-12, para 4-8a[2])
8.	B	Spacing of the crosshairs The constant depends upon…(page 4-12, para 4-8a[3])
9.	C	10 millimeters If the deviation from…(page 4-15, para 4-8c)
10.	B	0.003 meter Any time that the…(page 4-16, para 4-9a, third bullet)
11.	B	75 meters The maximum length…(page 4-18, para 4-9a[6])
12.	D	Imperfect level adjustment Errors due to imperfect level…(page 4-18, para 4-9a[7])

LESSON 5

NAVIGATIONAL SATELLITE AND TIMING GLOBAL POSITIONING SYSTEM

OVERVIEW

LESSON DESCRIPTION:

In this lesson, you will learn to identify the different techniques of GPS.

TERMINAL LEARNING OBJECTIVE:

ACTION:	You will identify the different signals that support the surveyor.
CONDITION:	You will be given the material contained in this lesson, a number 2 pencil, and a calculator.
STANDARD:	You will correctly answer all practice questions following this exercise.
REFERENCES:	The material contained in this lesson was derived from FM 3-34.331.

INTRODUCTION

This chapter provides a general overview of the basic operating principles of the NAVSTAR GPS. The NAVSTAR GPS is a passive, satellite-based navigation system operated and maintained by the DOD. Its primary mission is to provide passive global positioning or navigation for air-, land-, and sea-based strategic and tactical forces. A GPS receiver is simply a range-measurement device. Distances are measured between the receiver antenna and the satellites, and the position is determined from the intersections of the range vectors. These distances are determined by a GPS receiver, which precisely measures the time it takes a signal to travel from the satellite to the station. This measurement process is similar to that used in conventional pulsing marine-navigation systems and in phase-comparison EDM land-surveying equipment.

PART A - GLOBAL POSITIONING SYSTEM OVERVIEW

5-1. Operating and Tracking Modes. There are two general operating modes from which GPS-derived positions can be obtained--absolute and relative or differential positioning. Within each of these two modes, range measurements to the satellites can be performed by tracking either the phase of the satellite's carrier signal or PRN codes that are modulated on the carrier signal. In addition, GPS positioning can be performed with the receiver operating in a static or dynamic (kinematic) environment. This variety of operational options results in a wide range of accuracy levels, which may be obtained from the NAVSTAR GPS. Accuracies can range up to 100 meters and down to less than a centimeter. Increasing the accuracy to less than a centimeter requires additional observation time and can be achieved in real time. Selection of a particular GPS operating and tracking mode (such as absolute, differential, code, carrier, static, kinematic, or combinations thereof) depends on the user application. Topographic survey applications typically require differential positioning using carrier phase tracking. Absolute modes are rarely used for geodetic surveying applications, except when worldwide reference control is established.

5-2. Absolute Positioning. Absolute positioning is the most commonly used military and civil application of NAVSTAR GPS for real-time navigation. When operating in this passive, real-time navigation mode, a single receiver, placed at a point where a position is desired, can observe ranges to NAVSTAR GPS satellites. This receiver may be positioned to be stationary (static) over a point or in motion (kinematic), such as on a vehicle, aircraft, missile, or backpack. Two levels of absolute-positioning accuracy may be obtained from the NAVSTAR GPS--SPS and PPS. With certain specialized GPS receiving equipment, data-processing refinements, long-term static observations, and absolute positional coordinates can be determined to accuracy levels of less than one meter. These applications are usually limited to worldwide geodetic reference surveys.

5-3. Differential Positioning. Differential positioning is a process of measuring the differences in coordinates between two receiver points, each of which is simultaneously observing or measuring satellite code ranges and carrier phases from the NAVSTAR GPS constellation. The process measures the difference in ranges between the satellites and two or more ground observing points. The range measurement is performed by a phase-difference comparison, using the carrier phase or the code phase. The basic principle is that the absolute-positioning errors at the two receiver points will be about the same for a given instant. The resultant accuracy of these coordinate differences is at the meter level for code phase observations and at the centimeter level for carrier phase tracking. These coordinate differences are usually expressed as 3D baseline vectors, which are comparable to conventional survey azimuth and distance measurements. DGPS positioning can be performed in either a static mode or a kinematic mode.

5-4. System Configuration. The NAVSTAR GPS consists of three distinct segments--the space segment (satellites), the control segment (ground tracking and monitoring stations), and the user segment (air-, land-, and sea-based receivers).

a. Space Segment. The space segment consists of all GPS satellites in orbit. The first generation of satellites was the Block I or developmental. Several of these are still operational. A full constellation of Block II or production satellites is now in orbit. The full constellation consists of 24 Block II operational satellites (21 primary with 3 active on-orbit spares). There are four satellites in each of six orbital planes inclined at 55° to the equator. The satellites are at altitudes of 10,898 nautical miles and have 11-hour, 56-minute orbital periods. The three active spares are transparent to the user on the ground. The user is not able to tell which are operational satellites and which are spares. A procurement action for Block IIR (replacement) satellites is underway to ensure full system performance through the year 2025.

b. Control Segment. The control segment consists of five tracking stations located throughout the world (Hawaii, Colorado, Ascension Island, Diego Garcia, and Kwajalein). The information obtained from tracking the satellites is used in controlling the satellites and predicting their orbits. Three of the stations (Ascension Island, Diego Garcia, and Kwajalein) are used for transmitting information back to the satellites. The master control station is located at Colorado Springs, Colorado. All data from the tracking stations are transmitted to the master control station where they are processed and analyzed. Ephemerides, clock corrections, and other message data are then transmitted back to the three stations responsible for subsequent transmittal back to the satellites. The master control station is also responsible for the daily management and control of the GPS satellites and the overall control segment.

c. User Segment. The user segment represents the ground-based receiver units that process the satellite signals and arrive at a user position. This segment consists of both military and civil activities for an almost unlimited number of applications in a variety of air-, land-, or sea-based platforms.

5-5. Broadcast Frequencies and Codes. Each NAVSTAR satellite transmits signals on two L-band frequencies, designated as L1 and L2. The L1 carrier frequency is 1575.42 megahertz and has a wavelength of about 19 centimeters. The L2 carrier frequency is 1227.60 megahertz and has a wavelength of about 24 centimeters. The L1 signal is modulated with a precision code (P-code) and a coarse-acquisition code (C/A-code). The L2 signal is modulated with only the P-code. Each satellite carries precise atomic clocks to generate the timing information needed for precise positioning. A navigation message is transmitted on both frequencies and contains ephemerides, clock corrections and coefficients, satellite health and status information, almanacs of all GPS satellites, and other general information.

5-6. Pseudorandom Noise Codes. The modulated C/A- and P-codes are referred to as PRN codes. These PRN codes are actually a sequence of very precise time marks that permit the ground receivers to compare and compute the time of transmission between the satellite and the ground station. The range to the satellite can be derived from this transmission time. This is the basis behind GPS range measurements. The C/A-code pulse intervals are about every 300 meters. The more accurate P-code pulse intervals are every 30 meters.

5-7. Pseudorange. A pseudorange is the time delay between the satellite clock and the receiver clock, as determined from C/A- or P-code pulses. This time difference equates to the range measurement but is called a pseudorange since at the time of the measurement, the receiver clock is not synchronized to the satellite clock. In most cases, an absolute 3D real-time navigation position can be obtained by observing at least four simultaneous pseudoranges.

5-8. Carrier Phase Measurements. Carrier frequency tracking measures the phase differences between the Doppler-shifted satellite and receiver frequencies. The phase differences are continuously changing due to the changing satellite earth orbit geometry. However, such effects are resolved in the receiver and subsequent data postprocessing. When carrier phase measurements are observed and compared between two stations (differential mode), 3D baseline vector accuracy between the stations below the centimeter level is attainable. New receiver technology and processing techniques have allowed for carrier phase measurements to be used in real-time centimeter positioning.

5-9. Broadcast Messages and Ephemeris Data. Each NAVSTAR GPS satellite periodically broadcasts data concerning clock corrections, system and satellite status and, most critically, position or ephemeris data. There are two basic types of ephemeris data--broadcast and precise.

a. Broadcast Ephemerides. Broadcast ephemerides are predicted satellite positions broadcast within navigation messages (transmitted from satellites in real time). The ephemerides can be acquired in real time through a receiver capable of acquiring either a C/A- or P-code. Broadcast ephemerides are computed using the past tracking data of satellites. The satellites are continuously tracked by the monitor stations to obtain current data for use in orbit predictions. The data are analyzed by the master control station, and new parameters for the satellite orbits are transmitted back to the satellites. This upload is performed daily, and the new predicted orbital elements are transmitted every hour by the navigation message.

b. Precise Ephemerides. Precise ephemerides are based on actual tracking data that are postprocessed to obtain more accurate satellite positions. These ephemerides are delayed for processing but are more accurate than the broadcast ephemerides because they are based on actual tracking data and not predicted data. Nonmilitary users can obtain this information from the NGS or private sources that maintain their own tracking networks. For most survey applications, broadcast ephemerides are adequate to obtain needed accuracies.

PART B - ABSOLUTE PRECISE POSITIONING

5-10. General. Absolute positioning involves the use of a single passive receiver at one station location to collect data from multiple satellites to determine the station's location. It is not sufficiently accurate for precise surveying and positioning uses. It is, however, the most widely used GPS positioning method for real-time navigation and location.

5-11. Accuracy. The accuracy of absolute positioning is dependent on the user's authorization. The SPS user can obtain real-time point-positional accuracy of 100 meters. The lower level accuracy using SPS is due to intentional degradation of the GPS signal by DOD selective availability (S/A). The PPS user (usually a DOD-approved user) can use a decryption device to achieve a 3D accuracy in the range of 10 to 16 meters with a single-frequency receiver. Accuracy to less than one meter can be obtained from absolute GPS measurements when special equipment and postprocessing techniques are employed.

5-12. Ephemerides. By using broadcast ephemerides, the user is able to use pseudorange values in real time to determine absolute point positions with an accuracy of between 3 meters in the best of conditions to 80 meters in the worst of conditions. Using postprocessed ephemerides (precise), the user can expect absolute point positions with the accuracy of near 1 meter in the best of conditions and 40 meters in the worst of conditions.

5-13. Pseudorange. When a GPS user performs a GPS navigation solution, only an approximate range or pseudorange to selected satellites is measured. In order to determine the user's precise GPS location, the known range to the satellite and the position of those satellites must be known. Using pseudoranging, the GPS user can measure an approximate distance between the antenna and the satellite through correlation of a satellite-transmitted code and a reference code created by the receiver. It is not necessary to factor in corrections for errors in synchronization between the clock of the transmitter and the receiver. The distance the signal travels is equal to the velocity of the transmission of the satellite multiplied by the elapsed time of transmission. Satellite signal velocity changes due to tropospheric and ionospheric conditions are also considered.

5-14. Accuracy. The accuracy of the positioned point is a function of the range-measurement accuracy and the geometry of the satellites, reduced to spherical intersections with the earth's surface. The dilution of precision (DOP) is a description of the geometrical magnification of uncertainty in a GPS-determined point position. Repeated and redundant range observations will generally improve range accuracy; however, the DOP remains the same. In a static mode (where the GPS antenna remains stationary), range measurements to each satellite can be continuously remeasured over varying orbital locations of the satellites. The varying satellite orbits cause varying positional intersection geometry. Additionally, simultaneous range observations to numerous satellites can be adjusted using weighting techniques based on the reliability of the elevation and pseudorange measurements.

5-15. Number of Pseudorange Observations Needed. Four pseudorange observations are needed to resolve a GPS 3D position. Only three pseudorange observations are needed for a two-dimensional (2D) (horizontal) location. There are often more than four pseudorange observations due to the need to resolve the clock biases contained in both the satellite and the ground-based receiver. In computing the X-Y-Z coordinates of a point, a fourth unknown (clock bias) must also be included in the solution. The solution of a 3D position point is simply the solution of four pseudorange observations containing four unknowns (X, Y, Z, and time).

5-16. Absolute-Positioning Error Sources. There are numerous sources of measurement errors that influence GPS performance. The sum of all systematic errors or biases contributing to the measurement error is referred to as a range bias. The observed GPS range, without removal of biases, is referred to as a biased range or pseudorange. Principal contributors to errors in the final range and overall GPS readings include errors in the ephemeris, satellite clock and electronics accuracies, tropospheric and ionospheric refraction, atmospheric absorption, receiver noise, and multipath effects. Other errors include those induced by DOD S/A and antispoofing (AS). In addition to these major errors, GPS also contains random observation errors (such as unexplainable and unpredictable time variations). These errors are impossible to model and correct. The following paragraphs discuss errors associated with absolute GPS positioning modes. Many of these errors are either eliminated or significantly minimized when GPS is used in a differential mode because the same errors are common to both receivers during simultaneous observing sessions.

a. Ephemeris Errors and Orbit Perturbations. Satellite ephemeris errors are errors in the prediction of a satellite's position, which may then be transmitted to the user in the satellite data message. Ephemeris errors are satellite dependent and are very difficult to completely correct or compensate for because the many forces acting on the predicted orbit of a satellite are difficult to measure directly. Because direct measurement of all forces acting on a satellite orbit is difficult, it is nearly impossible to compensate or accurately account for those error sources when modeling the orbit of a satellite. The previously stated accuracy levels are subject to performance of equipment and conditions. Ephemeris errors produce equal error shifts in the calculated absolute-point positions.

b. Clock Stability. GPS relies heavily on accurate time measurements. GPS satellites carry rubidium and cesium time standards that are usually accurate to 1 part in 10 trillion and 1 part in 100 trillion, respectively. Most receiver clocks are actuated by a quartz standard accurate to 1 part in 100 million. A time offset is the difference between the time recorded by the satellite clock and the time recorded by the receiver. Range error observed by the user as the result of time offsets between the satellite and receiver clock is a linear relationship and can be approximated.

c. Ionospheric Delays. GPS signals are electromagnetic signals and are nonlinearly dispersed and refracted when transmitted through a highly charged environment like the ionosphere. Dispersion and refraction of the GPS signal is referred to as an ionospheric range effect because dispersion and refraction of the signal result in error to the GPS range value. Ionospheric range effects are frequency dependent.

d. Tropospheric Delays. GPS signals in the L-band level are not dispersed by the troposphere, but are refracted. The tropospheric conditions causing refraction of the GPS signal can be modeled by measuring the dry and wet components.

e. Multipath Effects. Multipath describes an error in positioning which occurs when a receiver receives a signal from more than one path. This is caused by the reflection of the GPS signal by a nearby object, which produces a false signal at the GPS antenna. Multipath normally occurs near large reflective surfaces, such as buildings with reflective surfaces, chain-link fences, and antenna arrays. GPS signals received as a result of multipath give inaccurate GPS positions when processed. Newer receiver and antenna designs and thorough mission planning can aid in minimizing these errors. Averaging GPS signals over a period of time can also reduce multipath effects.

f. Receiver Noise. Receiver noise includes a variety of errors associated with the ability of the GPS receiver to measure a finite time difference. These errors include signal processing, clock and signal synchronization and correlation methods, receiver resolution, and signal noise.

g. Selective Availability and Antispoofing. S/A purposely degrades a satellite signal to create position errors by dithering the satellite clock and offsetting the satellite orbits. The effects of S/A can be eliminated by using differential techniques. AS is implemented by interchanging the P-code with a classified, encrypted P-code called a Y-code. This code denies access to users who do not possess an authorized decryption device. Manufacturers of civil GPS equipment have developed methods such as squaring or cross correlation to make use of the P-code when it is encrypted. The absolute value of range accuracies obtainable from GPS is largely dependent on which code is used to determine positions. The range accuracy (for example, user-equivalent range error [UERE]), when coupled with the geometrical relationships of the satellite during the position determination (for example, DOP), result in a 3D ellipsoid that depicts uncertainties in all three coordinates. Given the changing satellite geometry and other factors, GPS accuracy is time and location dependent. Error propagation techniques are used to define nominal accuracy statistics for a GPS user.

h. Root-Mean-Square (RMS) Error Measures. Two-dimensional GPS positional accuracies are normally estimated using an RMS radial error statistic. A 1-sigma RMS error equates to the radius of a circle in which the position has a 63 percent probability of falling. A circle of twice this radius (2 sigmas) represents about a 97 percent probability that the position is within the circle. This 97 percent probability circle, or 2D RMS, is the most common positional-accuracy statistic used in GPS surveying. In some instances, a 3D RMS or 99 percent or more probability is used. This RMS error statistic is also related to the positional-covariance matrix. An RMS error statistic represents the radius of a circle and therefore is not preceded by a ± sign.

i. Probable Error Measures. The accuracy of 3D GPS measurements is commonly expressed by the spherical error probable (SEP). The SEP represents the radius of a sphere with a 50 percent confidence or probability level. This spheroid radial measure only approximates the actual 3D ellipsoid representing the uncertainties in the geocentric coordinate system. A circular error probable (CEP) statistic is commonly used in 2D horizontal positioning, particularly in military targeting. The CEP represents the radius of a circle containing a 50 percent probability of position confidence.

5-17. Accuracy Comparisons. It is important that GPS measurements clearly identify the statistic from which they are derived. A 100-meter or positional variance-covariance matrix is meaningless unless it is identified as being either 2D or 3D, along with the applicable probability level. For example, a PPS 16-meter 3-deviation accuracy is, by definition, an SEP (50 percent). This 16-meter SEP equates to a 28-meter 3D, 95 percent confidence spheroid. If transformed to 2D accuracy, the SEP equates roughly to a 10-meter CEP, a 12-meter RMS, a 2-meter 2-deviation RMS, or a 36-meter 3-deviation RMS. Table 5-1 shows additional information on GPS measurement statistics. Additionally, absolute GPS point-positioning accuracies are defined relative to an earth-centered coordinate system or datum. This coordinate system differs significantly from local or construction datums. Nominal GPS accuracies may also be published as design or tolerance limits, and accuracies achieved can differ significantly from these values.

5-18. Dilution of Precision. The final positional accuracy of a point, determined using absolute GPS-S techniques, is directly related to the geometric strength of the configuration of satellites observed during the survey session. GPS errors resulting from satellite constellation geometry can be expressed in terms of DOP. In mathematical terms, DOP is a scalar quantity used in an expression of a ratio of the positioning accuracy. It is the ratio of the standard deviation of one coordinate to the measurement accuracy. DOP represents the geometrical contribution of a certain scalar factor to the uncertainty (such as standard deviation) of a GPS measurement. DOP values are a function of diagonal elements of the covariance matrices of the adjusted parameters for the observed GPS signal. DOP values are used in point formulations and determinations. In general terms, DOP is a scalar quantity of the contribution of the configuration of satellite constellation geometry to the GPS accuracy or a measure of the strength of the satellite constellation geometry. The more satellites that can be observed and used in the final solution, the better the solution. Since DOP can be used as a measure of geometrical strength, it can also be used to selectively choose four satellites in a particular constellation that will provide the best solution.

Table 5-1. Representative GPS Error-Measurement Statistics for Absolute Point Positioning

Error-Measure Statistic	Probability %	Relative Distance (ft)[1]	GPS Precise-Positioning Service (m)[2]		GPS Standard-Positioning Service (m)[2]	
1D Measures			σ_N or σ_E	σ_U	σ_N or σ_E	σ_U
Probable error	50.00	0.6745 σ	±4.0	±9.0	±24.0	±53.0
Average error	57.51	0.7979 σ	±5.0	±11.0	±28.0	±62.0
1σ standard error/deviation[3]	68.27	1.0000 σ	±6.3	±13.8	±35.3	±78.0
90% probability (map accuracy standard)	90.00	1.6450 σ	±10.0	±23.0	±58.0	±128.0
95% probability/confidence	95.00	1.9600 σ	±12.0	±27.0	±69.0	±153.0
2σ standard error/deviation	95.45	2.0000 σ	*±12.6*	*±27.7*	*±70.7*	*±156.0*
99% probability/confidence	99.00	2.5760 σ	±16.0	±36.0	±91.0	±201.0
3σ standard error (near certainty)	99.73	3.0000 σ	±19.0	±42.0	±106.0	±234.0
2D Measures[4]			Circular Radius		Circular Radius	
1σ standard error circle[5]	39.00	1.0000 σ_c	6.0		35.0	
CEP[6]	50.00	1.1770 σ_c	7.0		42.0	
1-deviation RMS (1DRMS)[3, 7]	63.00	1.4140 σ_c	9.0		50.0	
Circular map accuracy standard	90.00	2.1460 σ_c	13.0		76.0	
95% 2D positional confidence circle	95.00	2.4470 σ_c	15.0		86.0	
2-deviation RMS (2DRMS)[8]	98.00	2.8300 σ_c	*17.8*		*100.0*	
99% 2D positional confidence circle	99.00	3.0350 σ_c	19.0		107.0	
3.5σ circular near-certainty error	99.78	3.5000 σ_c	22.0		123.0	
3-deviation RMS (3DRMS)	99.90	4.2400 σ_c	27.0		150.0	
3D Measures			Spherical Radius		Spherical Radius	
1σ spherical standard error[9]	19.90	1.0000 σ_s	9.0		50.0	
SEP[10]	50.00	1.5400 σ_s	13.5		76.2	
Mean radial spherical error (MRSE)[11]	61.00	1.7300 σ_s	16.0		93.0	
90% spherical accuracy standard	90.00	2.5000 σ_s	22.0		124.0	
95% 3D confidence spheroid	95.00	2.7000 σ_s	24.0		134.0	
99% 3D confidence spheroid	99.00	3.3700 σ_s	30.0		167.0	
Spherical near-certainty error	99.89	4.0000 σ_s	35.0		198.0	

Table 5-1. Representative GPS Error-Measurement Statistics for Absolute Point Positioning (continued)

Error-Measure Statistic	Probability %	Relative Distance (ft)[1]	GPS Precise-Positioning Service (m)[2]	GPS Standard-Positioning Service (m)[2]

[1]Valid for 2- and 3-deviation only if $\sigma_N = \sigma_E = \sigma_U$. ($\sigma_{minimum}/\sigma_{maximum}$) generally must be ≥ 0.2. Relative distance used unless otherwise indicated.
[2]Representative accuracy based on 1990 Federal Radio Navigation Plan (FRNP) simulations for PPS and SPS (FRNP estimates shown in bold italics) and that $\sigma_N \approx \sigma_E$. SPS may have significant short-term variations from these nominal values.
[3]Statistic used to define USACE hydrographic survey depth and positioning criteria.
[4]The 1990 FRNP also proposes SPS maintain, at minimum, a 2D confidence of 300 meters @ 99.99 percent probability.
[5]$\sigma_C \cong 0.5\ (\sigma_N + \sigma_E)$—approximates standard error ellipse.
[6]CEP $\cong 0.589\ (\sigma_N + \sigma_E) \approx 1.18\ \sigma_C$.
[7]1DRMS $\cong (\sigma_N^2 + \sigma_E^2)^{½}$.
[8]2DRMS $\cong 2\ (\sigma_N^2 + \sigma_E^2)^{½}$.
[9]$\sigma_S \cong 0.333\ (\sigma_N + \sigma_E + \sigma_U)$.
[10]SEP $\cong 0.513\ (\sigma_N + \sigma_E + \sigma_U)$.
[11]MRSE $\cong (\sigma_N^2 + \sigma_E^2 + \sigma_U^2)^{½}$.

LEGEND:
σ_c = approximate standard error ellipse
σ_s = nominal standard error

NOTES:
1. Most commonly used statistics are shown in bold-face type.
2. Estimates are not applicable to differential GPS positioning. Circular/spherical error radii do not have ± signs.
3. Absolute positional accuracies are derived from GPS-simulated user range errors/deviations and the resultant geocentric-coordinate solution (X-Y-Z) covariance matrix, as transformed to a local datum (N-E-U or Φ-λ-h). GPS accuracy will vary with GDOP and other numerous factors at time(s) of observation. The 3D covariance matrix yields an error ellipsoid. Transformed ellipsoidal dimensions given (for example, σ_N, σ_E, or σ_U) are only average values observed under nominal GDOP conditions. Circular (2D) and spherical (3D) radial measures are only approximations to this ellipsoid, as are probability estimates.

PART C - DIFFERENTIAL PRECISE POSITIONING

5-19. Reducing Errors. Absolute positioning, as discussed earlier, will not provide the accuracies needed for most survey control projects due to existing and induced errors. To eliminate errors and obtain higher accuracies, use GPS in a differential-positioning mode. The terms relative and differential have similar meanings. Relative is used when discussing one thing in relation to another. Differential is used when discussing the technique of positioning one thing in relation to another. Differential positioning requires that at least two receivers be set up at two stations (usually one is known) to collect satellite data simultaneously to determine coordinate differences. This method positions the two stations relative to each other (hence the term relative positioning) and can provide the accuracy required for basic land surveying.

5-20. Code Pseudorange Tracking. Differential positioning (using code pseudorange) is performed similarly to code-pseudorange tracking for absolute positioning. Some of the major uncertainties are effectively eliminated or minimized. This process results in absolute coordinates of the user on the earth's surface. Errors in range are directly reflected in resultant coordinate errors. Differential positioning is not so concerned with the absolute position of the user but with the relative difference between two user positions, that are simultaneously observing the same satellites. Since errors in the satellite position and atmospheric-delay estimates are effectively the same at both receiving stations, the errors cancel each other to a large extent.

5-21. Correcting Errors. A pseudorange correction (PRC) can be generated for each satellite being observed. If a second receiver is observing at least four of the same satellites and is within a reasonable distance, it can use these PRCs to obtain a relative position to the known control point since the errors will be similar. Thus, the relative distance (coordinate difference) between the two stations is relatively accurate regardless of poor absolute coordinates. For example, if the true pseudorange distance from a known control point to a satellite is 100 meters, and the observed or measured pseudorange distance is 92 meters, then the pseudorange error or correction is 8 meters for that particular satellite. In effect, the GPS-observed baseline vectors are no different from azimuth/distance observations. As with a total station, any type of initial-coordinate reference can be input to start the survey.

5-22. Global Positioning System Coordinates. The GPS coordinates will not coincide with the user's local-project datum coordinates. Since differential-survey methods are only concerned with relative coordinate differences, disparities with a global reference system used by the NAVSTAR GPS are not significant for topographic purposes. Therefore, GPS coordinate differences can be applied to any type of local-project reference datum (for example, NAD 27 or NAD 83).

5-23. Carrier Phase Tracking. Differential positioning using carrier phase tracking uses a formulation of pseudoranges. The process becomes somewhat more complex when the carrier signals are tracked so that range changes are measured by phase resolution. In carrier phase tracking, an ambiguity factor is added, which must be resolved to obtain a derived range. Carrier phase tracking provides a more accurate range resolution due to the short wavelength (about 19 centimeters for L1 and 24 centimeters for L2) and the ability of a receiver to resolve the carrier phase down to about 2 millimeters. This technique has primary application to engineering, topographic, and geodetic surveying and may be employed with either static or kinematic surveys. There are several techniques which use the carrier phase to determine a station's position. These surveying techniques include static, rapidstatic, kinematic, stop-and-go kinematic, pseudokinematic, and on-the-fly (OTF) kinematic/RTK. Table 5-2 lists these techniques and their required components, applications, and accuracies.

Table 5-2. Carrier Phase Tracking

Concept	Requirements	Applications	Accuracy
Static (postprocessing)	L1 or L1/L2 GPS receiver 386 or 486 computer for postprocessing 45-minute to 1-hour minimum observation time[1]	Control surveys (that require high accuracy)	Subcentimeter level
Rapid static (postprocessing)	L1/L2 GPS receiver 5- to 20-minute observation time[1]	Control surveys (that require medium to high accuracy)	Subcentimeter level
Kinematic[2] (postprocessing)	L1 GPS receiver with kinematic-survey option 386 or 486 computer for postprocessing	Continuous topographic surveys Location surveys	Centimeter level
Stop-and-go kinematic[2] (postprocessing)	L1 GPS receiver 386 or 486 computer for postprocessing	Medium-accuracy control surveys	Centimeter level
Pseudokinematic[2] (postprocessing)	L1 GPS receiver 386 or 486 computer for postprocessing	Medium-accuracy control surveys	Centimeter level
RTK/OTF kinematic[3] (real-time or postprocessing)	Real-time processing: Internal or external processor (one 386 or 486 computer with dual communication ports) Minimum 4800 baud radio/modem data-link set Postprocessing: L1/L2 GPS receiver 386 or 486 computer	Real-time high-accuracy hydro surveys Location surveys Medium-accuracy control surveys Photo control surveys Continuous topographic surveys	Subdecimeter level

[1]Dependent on the satellite constellation and the number of satellites in view.
[2]An initialization period is required and the loss of satellite lock is not tolerated.
[3]No static initialization is necessary, integers are gained while moving, and the loss of satellite lock is tolerated.

a. Static surveying is the most widely used differential technique for control and geodetic surveying. It involves long observation times (1 to 2 hours, depending on the number of visible satellites) to resolve the integer ambiguities between the satellite and the receiver. Accuracies of less than a centimeter can be obtained using this method.

b. Rapid-static surveying measures baselines and determines positions in the centimeter level with a short observation time (5 to 20 minutes). The observation time is dependent on the length of the baseline and the number of visible satellites. Loss of the satellite lock, when moving from one station to the next, can occur since each baseline is processed independently.

c. Kinematic surveying allows the user to rapidly and accurately measure baselines while moving from one point to the next. The data are collected and postprocessed to obtain accurate positions to the centimeter level. This technique permits only partial loss of lock during observation and requires a brief period of static initialization. The OTF technology, both real time and postprocessed, could eventually replace standard kinematic procedures for short baselines.

d. Stop-and-go kinematic surveying involves 1 to 2 minutes of data collection at each station (after a period of initialization) to gain the integers. This technique does not allow for loss of lock during the survey. If loss of lock does occur, a new period of initialization must take place. This method should be performed with two fixed or known stations to provide redundancy and improve accuracy.

e. Pseudokinematic surveying is similar to standard kinematic and static procedures combined. The differences are that there is no static initialization and there is a longer period of data collection at each station (1 to 5 minutes). Each point must be revisited after one hour, and the loss of lock is acceptable. Pseudokinematic surveying is less acceptable for establishing baselines because the positional accuracy is less than for kinematic or rapid-static surveying.

f. OTF/RTK kinematic surveying uses GPS technology to allow positioning to less than a decimeter in real time. This system determines the integer number of carrier wavelengths from the GPS antenna to the GPS satellite, transmitting them while in motion and without static initialization. The basic concept behind the OTF/RTK kinematic system is kinematic surveying without static initialization (integer initialization is performed while moving) and allowances for loss of lock. Other GPS techniques which can achieve this kind of accuracy require static initialization while the user is not moving and does not allow for loss of lock while in motion.

5-24. The Impact of the Global Positioning System. The impact of GPS on geodetic-control surveys has been immense. In the past, surveyors relied upon line-of-sight instrumentation to develop coordinates. With GPS, ground station intervisibility is no longer required, and much longer lines can be surveyed. Different instruments and survey techniques were previously used to measure horizontal and vertical coordinates, leading to two different networks with little overlap. GPS, on the other hand, is a 3D system.

5-25. Vertical Measurements with GPS. GPS is not recommended for third-order or higher vertical-control surveys. It is not recommended as a substitute for standard differential leveling, but rather for small-scale topographic mapping or similar projects. The height component of GPS measurements is the weakest plane because the orbital geometry of the X-Y-Z position determination. Thus, GPS-ellipsoidal height differences are usually less accurate than the horizontal components. GPS-derived elevation differences do not meet third-order standards as those obtained using conventional levels. Accordingly, GPS-derived elevations must be used with caution.

5-26. GPS Height Determinations. GPS positioning, whether operated in an absolute or differential positioning mode, can provide heights (or height differences) of surveyed points. The height or height difference obtained from GPS is in terms of height above or below the WGS-84 ellipsoid. The ellipsoid heights are not the same as orthometric heights or elevations, which are obtained from conventional differential leveling. This distinction between ellipsoid heights and orthometric elevations is critical to many engineering and construction projects. GPS users must exercise extreme caution in applying GPS height determinations to projects that are based on conventional orthometric elevations.

The heights obtained from GPS are in a different height system than those historically obtained with geodetic leveling. GPS data can be readily processed to obtain the ellipsoidal height. This is the height above or below a simple ellipsoid model of the earth. Geodetic leveling gives rise to a orthometric height, often known as the height above the MSL. These heights are found on topographic maps, stamped on markers, or stored in innumerable digital and paper data sets. To transform between these height systems requires the geoid height. These height systems are related by the following equation:

$$h = H + N$$

Where--

h = *ellipsoidal height*
H = *orthometric height*
N = *geoid height*

5-27. Differential Error Sources. Error sources encountered in using differential GPS positioning techniques are the same as for absolute positioning. In addition to these error sources, the user must ensure that the receiver maintains satellite lock on at least three satellites for 2D positioning and four satellites for 3D positioning. When loss of lock occurs, a cycle slip (discontinuity of an integer number of cycles in the measured carrier beat phase as recorded by the receiver) may occur. In GPS absolute surveying, if satellite lock is not maintained, positional results will not be formulated. In GPS static surveying, if satellite lock is not maintained, positional results may be degraded resulting in incorrect formulations. In GPS static surveying, if the observation period is long enough, postprocessing software may be able to average out loss of lock and cycle slips over the duration of the observation period and formulate positional results that are adequate. If this is not the case, reoccupation of the stations may be required. In all differential-surveying techniques, if loss of lock does occur on some of the satellites, data processing can continue easily if a minimum of four satellites are tracked. Generally, the more satellites tracked by the receiver, the more insensitive the receiver is to loss of lock. In general, cycle slips can be repaired.

5-28. Differential Accuracies. There are two levels of accuracy obtainable from GPS using differential techniques. The first level is based on pseudorange formulations, while the other is based on carrier beat phase formulations.

a. Pseudorange formulations can be developed from either the C/A-code or the more precise P-code. Pseudorange accuracies are generally accepted to be 1 percent of the period between successive code epochs. Use of the P-code, where successive epochs are 0.1 millisecond apart, produces results that are about 1 percent of 0.1 millisecond (about 1 nanosecond). Multiplying this value by the speed of light gives a theoretical-resultant range measurement of around 30 centimeters. If using pseudorange formulation with the C/A-code, results can be ten times less precise (a range-measurement precision of around 3 meters). Point-positioning accuracy for a differential pseudorange solution is generally found to be in the range of 0.5 to 10 meters. These accuracies are largely dependent on the type of GPS receiver used.

b. Carrier beat phase formulations can be based on the L1, the L2, or both carrier signals. Accuracies achievable using the carrier beat phase measurement are generally accepted to be 1 percent of the wavelength. Using the L1 frequency where the wavelength is around 19 centimeters, the theoretical resultant range measurement is 1 percent of 19 centimeters (about 2 millimeters). The L2 carrier can only be used with receivers that employ cross correlation, squaring, or another technique to get around the effects of AS.

PART D - PLANNING PRECISE-POSITIONING SURVEYS

5-29. General. Using differential carrier phase surveying to establish control for military projects requires operational and procedural specifications for a project-specific function of the control being established. To accomplish these surveys in the most efficient and cost-effective manner and to ensure that the required accuracy criteria are obtained, a detailed survey-planning phase is essential. This section defines global positioning system-survey (GPS-S) design criteria and other specifications that are required to establish control for survey projects.

5-30. Planning a Control Survey. The first step in planning a control survey is to determine the ultimate accuracy requirements. Survey accuracy requirements are a direct function of the project's functional needs--the basic requirements needed to support the planning, engineering design, maintenance, and operations. This is true for GPS or conventional surveying in order to establish project control. Most military activities require relative accuracies (accuracies between adjacent control points) ranging from 1:1,000 to 1:50,000, depending on the nature and scope of the project. Few topographic projects demand positional accuracies higher than the 1:50,000 level (second-order, Class I). Although a GPS-S may be designed and performed to support lower-accuracy project-control requirements, the actual results could generally be several magnitudes better than the requirement. Although higher accuracy levels are easily achievable with GPS, it is important to consider the ultimate use of the control on the project in planning and designing GPS control networks. Thus, GPS-S adequacy evaluations should be based on the project's accuracy standards, not those theoretically obtainable with GPS.

a. Project Functional Requirements. Project functional requirements must include planned and future design and mapping activities. Control density within a given project is determined from factors such as planned construction, site plan mapping scales, master plan mapping scales, and artillery/aviation survey positioning requirements. The relative accuracy for project control is also determined based on such things as mapping scales, design needs, and project types.

b. Project Control Surveys. Project control surveys should be planned, designed, and executed to achieve the minimum accuracy demanded of the project's functional requirements. To most efficiently use resources, control surveys should not be designed or performed to achieve accuracy levels which exceed the project requirements. For instance, if a third-order, Class I accuracy standard (1:10,000) is required for most of the topographic-survey control on a project, field survey criteria should also be designed to meet this minimum standard.

5-31. Network Design Factors. Some of the factors to be considered in designing a GPS network and subsequent observing procedures are the--

a. Project Size. The extent of the project will affect the GPS-S network shape.

b. Required Density of Control. The type of GPS-S scheme used will depend on the number and spacing of points to be established, which is a project-specific requirement. Maximum baseline lengths between stations or existing control are also prescribed. A combination of GPS and conventional-survey densification is often the most effective approach.

c. Absolute GPS Reference Datums. Coordinate data for baseline observations are referenced and reduced relative to the WGS-84 earth-centered earth-fixed (ECEF) coordinate system (X, Y, and Z). For all practical purposes, this system is not directly referenced to (though closely related) the Geodetic Reference System of 1980 (GRS 80) upon which NAD 83 is related (for CONUS work). Data reduction and adjustment are normally performed using the WGS-84 ECEF coordinate system, with baseline vector components measured relative to the ECEF coordinate system. The baseline vector components are denoted by delta (Δ) X, ΔY, and ΔZ.

d. Connections to Existing Control. For most static and kinematic GPS horizontal-control work, at least two existing control points should be connected for referencing and adjusting a new GPS-S. Table 5-3 shows GPS-S design, geometry, connection, and observing criteria. Existing points may be part of the NGRS or in place project control that has been adequately used for years. Additional points may be connected if practical. In some instances, a single existing point may be used to generate spurred baseline vectors for supplemental construction control.

Table 5-3. GPS-S Design, Geometry, Connection, and Observing Criteria

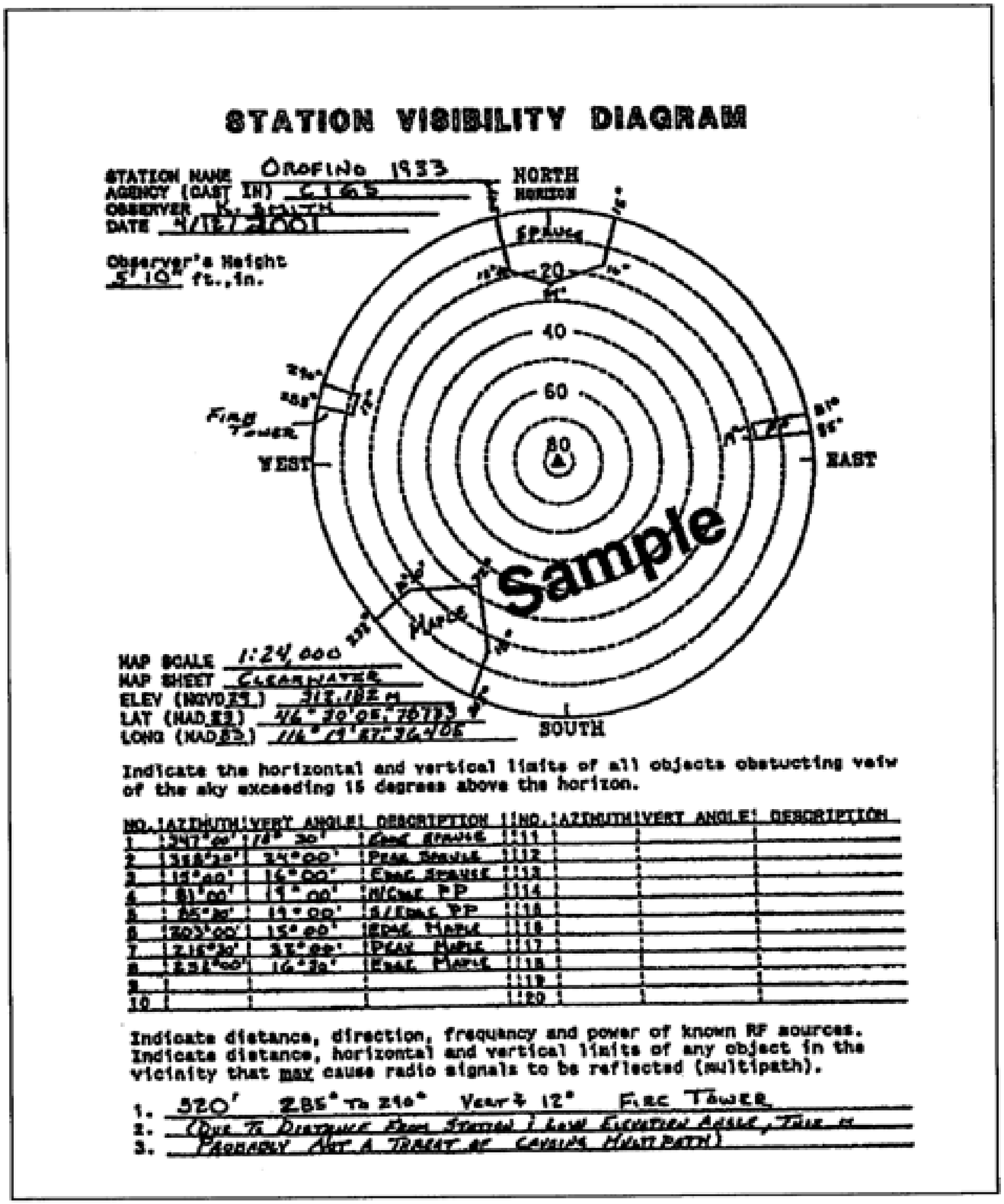
Criterion	Classification Order			
	2nd, I	2nd, II	3rd, I	3rd, II
Relative accuracy:				
ppm	20	50	100	200
1 part in	50k	20k	10k	5k
NGRS network (local project network) (W/F/P)	Yes	Yes	Yes	Yes
Baseline observation check required over existing control	Yes	W/F/P	W/F/P	No
Number of connections with existing network (NGRS or local project control):				
Minimum	2	2	2	2
Optimum	3	3	2	2
New point spacing not less than (m)	1,000	500	200	100
Maximum distance from network to nearest control point in project (km)	50	50	50	50
Minimum network control quadrant location (relative to project center)	2	N/R	N/R	N/R
Master of fiducial stations required	W/F/P	No	No	No
Loop closure criteria:				
Maximum number of baselines/loop	10	20	20	20
Maximum loop length not to exceed (km)	100	200	N/R	N/R
Loop misclosure not less than (ppm)	20	50	100	200
Single spur baseline observations:				
Allowed per order/class	No	No	Yes	Yes
Required number of sessions/baseline	NA	NA	2	2
Required tie to NGRS	NA	NA	No	No
Field-observing criteria (static GPS-Ss):				
Required antenna phase height measurement per session	2	2	2	2
Meteorological observations required	No	No	No	No
Two frequency L1/L2 observations required:				
< 50-km lines	No	No	No	No
> 50-km lines	Yes	Yes	Yes	Yes
Recommended minimum observation time (per session) (min)	60	45	30	30
Minimum number of sessions per GPS baseline	1	1	1	1
Satellite quadrants observed (minimum number)	3 W/F/P	N/R	N/R	N/R
Minimum obstruction angle above horizon (deg)	15	15	15	15

Table 5-3. GPS-S Design, Geometry, Connection, and Observing Criteria (continued)

Criterion	Classification Order			
	2nd, I	2nd, II	3rd, I	3rd, II
Maximum HDOP/VDOP during session	N/R	N/R	N/R	N/R
Photograph and/or pencil rubbing required	A/R	No	No	No
Kinematic GPS surveying:				
Allowable per survey class	Yes	Yes	Yes	Yes
Required tie to NGRS	W/F/P	W/F/P	No	No
Measurement time/baseline (follow manufacturer's specifications)	A/R	A/R	A/R	A/R
Minimum number of reference points	2	2	1-2	1
Preferred references	2	2	2	1
Maximum PDOP	15	15	15	15
Minimum number of observations from each reference station	2	2	2	2
Adjustment and data submittal criteria:				
Approximate adjustments allowed	Yes			
Contract acceptance criteria:				
Type of adjustment	Free (unconstrained)			
Evaluation statistic	Relative distance accuracies			
Error-ellipse sizes	(not used as criteria)			
Histogram	(not used as criteria)			
Reject criteria:				
Statistic	Normalized residual			
Standard	±3 • SEUW			
Optimum/nominal weighting:				
Horizontal	±5 + 2 ppm			
Vertical	±10 + 2 ppm			
Optimum variance of unit weight	Between 0.5 and 1.5			
GPS station/session data recording format	Field-survey book or form			
Final station descriptions	Standard DA form			
Written project/adjustment report required	Yes			

LEGEND:
W/F/P = where feasible and practical
N/R = no requirement for this specification (usually indicates variance with provisional FGCC GPS specifications)
A/R = as required in specific project instructions or manufacturer's operating manual
SEUW = standard error of unit weight

e. Location Feasibility and Field Reconnaissance. A good advance reconnaissance of all marks within the project is crucial to the expedient and successful completion of a GPS-S. The site reconnaissance should be completed before the survey is started. The surveyor should also prepare a site sketch and a brief description on how to reach the point since the individual performing the site reconnaissance may not be the same surveyor that returns to the station. The azimuths and vertical angles should be determined using a compass and inclinometer and recorded by the individual performing the site reconnaissance. Because obstructions such as trees and buildings cause the GPS signal transmitted from the GPS satellite to be blocked, it is also important to know the type of obstruction to determine if the multipath might be a problem. Site obstruction data are needed to determine if the survey site is suitable for GPS surveying. Obstruction data should be plotted in a station visibility diagram. GPS surveying requires that all stations have an unobstructed view 15º above the horizon. Satellites below 10º should not be observed. A site reconnaissance report form may be used in lieu of a standard field-survey book. A sample of obstruction data plotted in a station-visibility diagram is shown in Figure 5-1.



STATION VISIBILITY DIAGRAM

STATION NAME Orofino 1933
AGENCY (CAST IN) C 165
OBSERVER K. Smith
DATE 4/18/2001

Observer's Height 5'10" ft.,in.

MAP SCALE 1:24,000
MAP SHEET Clearwater
ELEV (NGVD 29) 313.182 m
LAT (NAD 83) 46° 30' 05.70783" N
LONG (NAD 83) 116° 19' 87.36406" W

Indicate the horizontal and vertical limits of all objects obstucting veiw of the sky exceeding 15 degrees above the horizon.

NO.	AZIMUTH	VERT ANGLE	DESCRIPTION	NO.	AZIMUTH	VERT ANGLE	DESCRIPTION
1	347°00'	18° 30'	[illegible]	11			
2	358°30'	24°00'	Peak Spruce	12			
3	15°00'	16°00'	Edge Spruce	13			
4	81°00'	19°00'	N/Edge PP	14			
5	85°30'	19°00'	S/Edge PP	15			
6	203°00'	15°00'	Edge Maple	16			
7	215°30'	33°00'	Peak Maple	17			
8	231°00'	16°30'	Edge Maple	18			
9				19			
10				20			

Indicate distance, direction, frequency and power of known RF sources.
Indicate distance, horizontal and vertical limits of any object in the vicinity that may cause radio signals to be reflected (multipath).

1. 520' 285° to 290° Vert ≈ 12° Fire Tower
2. (Due to Distance from Station & Low Elevation Angle, This is
3. Probably Not a Threat of Causing Multipath)

Figure 5-1. Sample Station-Visibility Diagram

f. Multiple/Repeat Baseline Connections. Table 5-3, pages 5-17 and 5-18 shows the recommended criteria for baseline connections between stations, repeat baseline observations, and multiple station occupations. Many of these standards were developed by the Federal Geodetic Control Subcommittee (FGCS) for performing high-precision geodetic-control surveys.

g. Loop Requirements. A loop (traverse) provides the mechanism for performing field data validation as well as final-adjustment accuracy analysis. Since loops of GPS baselines are comparable to traditional EDME/taped traverse routes, misclosures and adjustments can be handled similarly. Most GPS-S nets (static or kinematic) end up with one or more interconnecting loops that are either internal from a single fixed point or external through two or more fixed network points. Loops should be closed off at the spacing, as indicated in Table 5-3, pages 5-17 and 5-18. Loop closures should meet the criteria specified in Table 5-3, based on the total loop length.

5-32. Network Design and Layout. A wide variety of survey configuration methods may be used to densify project control using GPS-S techniques. Unlike conventional triangulation and EDME traverse surveying, the shape or geometry of the GPS network design is not as significant. The following guidelines for planning and designing proposed GPS-Ss are intended to support lower-order (second-order, Class I or 1:50,000 or less accuracy) military control surveys where relative accuracies at the centimeter level or better are required over a small project area.

a. Incorporate Newly Established Global Positioning System Control. Newly Established GPS control may or may not be incorporated into the NGRS, depending on the adequacy of the connection to the existing NGRS network or whether it was tied only internally to existing project control.

b. Develop a Network Design. When developing a network design, it is important to obtain the most economical coverage within the prescribed project accuracy requirements. The optimum network design provides a minimum amount of baseline/loop redundancy without an unnecessary amount of observation. Obtaining this optimum design (cost versus accuracy) is difficult and changes constantly due to evolving GPS technology and satellite coverage.

c. Plan a Global Positioning System Survey Network. Planning a GPS-S network is similar to planning for conventional triangulation or traversing. The type of survey design used is dependent on the GPS technique and the user requirements. A GPS network is proposed when established survey control is used in precise-network densification (1:50,000 to 1:100,000). When the networking method is selected, the surveyor should devise a survey network that is geometrically sound. Figure 5-2 shows an example of a step-by-step method to build a GPS-S network.

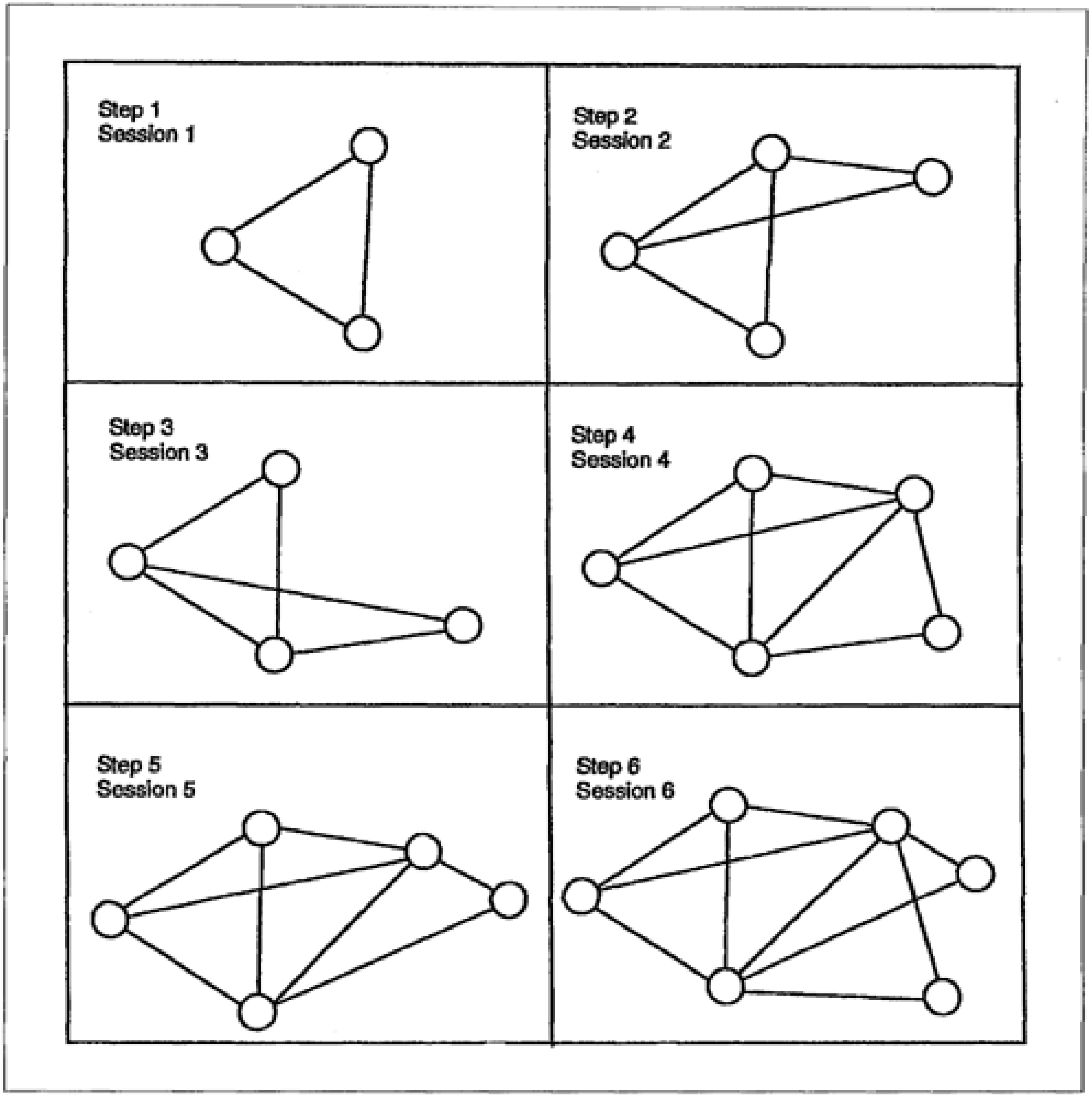


Figure 5-2. GPS-S Network Design

d. Determine a Global Positioning System Survey Technique. After a GPS network has been designed and laid out, a GPS-S technique needs to be considered. Choose the most efficient method--one which minimizes time and cost while still providing the necessary accuracy requirements of a given project. Once a technique is chosen, the equipment requirements, observation schedules, session's designations, and planning functions can be determined.

e. Determine the Type of Global Positioning System Instrumentation. The type of GPS instrumentation used on a project depends on the accuracy requirements, the GPS-S technique, the project size, and economics. Dual-frequency receivers are recommended as baseline lengths approach or exceed 50 kilometers. The length may also vary depending on the amount of solar activity during the observation period. Using a dual-frequency receiver permits the user to solve possible ionospheric and tropospheric delays, which can occur as the signal travels from the satellite to the receiver antenna.

f. Determine the Number of Required Receivers. The minimum number of receivers required to perform a differential GPS-S is two. The actual number used on a project depends on the project size and the number of available instruments and operators. Using more than two receivers will often increase productivity and field-observation efficiency. Some kinematic applications require two reference receivers (set at known points) and at least one rover.

g. Determine Personnel Requirements. Personnel requirements are also project-dependent. Most GPS equipment is compact and lightweight and only requires one person per station setup. However, when a station is not easily accessible or requires additional power for a data link, two individuals may be required.

h. Determine Vehicle Requirements. Normally, one vehicle is required for each GPS receiver used. Vehicles should be equipped to handle the physical conditions that may be encountered while performing the field observations. In most cases, a two-wheel-drive vehicle should be adequate. If adverse site conditions exist, a four-wheel-drive vehicle may be required. Adequate and reliable transportation is important when the observation schedule requires moving from one station to another between observation sessions.

i. Determine Electrical Requirements. Adequate power should be available for all equipment (such as, receivers, computers, and lights) that will be used during the observations. PCs, software, and data-storage devices should be available for on-site field data reduction. Other equipment should include tripods, tribrachs, tape measures, flags, flashlights, tools, equipment cables, a compass, and an inclinometer. A data link is also needed if real-time positioning is required.

j. Determine Observation Schedules. Planning a GPS-S requires that the surveyor determine when satellites will be visible for the given survey area. The first step in determining observation schedules is to plot the satellite visibility for the project area. Even when the GPS becomes fully operational, a full two-hour coverage of at least four satellites may not be available in all areas. Station occupation during each session should be designed to minimize travel time and to maximize the overall efficiency of the survey. Determination of session times is based mainly on the satellite-visibility plan, with the following factors taken into consideration:

- The time required for safe travel between survey sites.
- The time to set up and take down the equipment before and after the survey.
- The time of the survey.

- The possible loss of observation time due to unforeseeable problems or complications.

k. Determine a Suitable Site. A satellite sky plot (Figure 5-3) and a positional dilution of precision (PDOP) versus time plot (Figure 5-4, page 5-24) may be run before a site recon. The output files created by the satellite prediction software are used in determining if a site is suitable for GPS surveying.

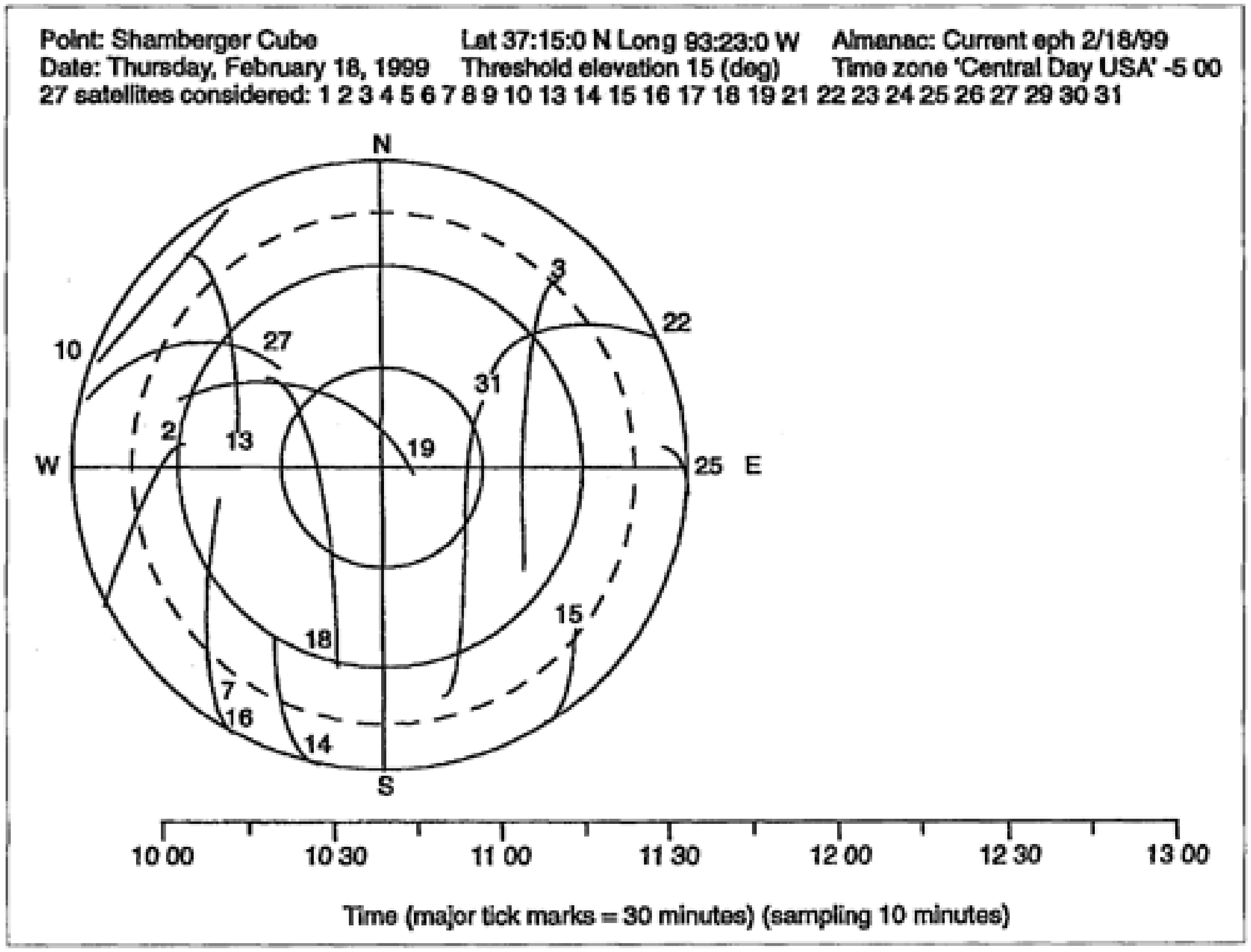
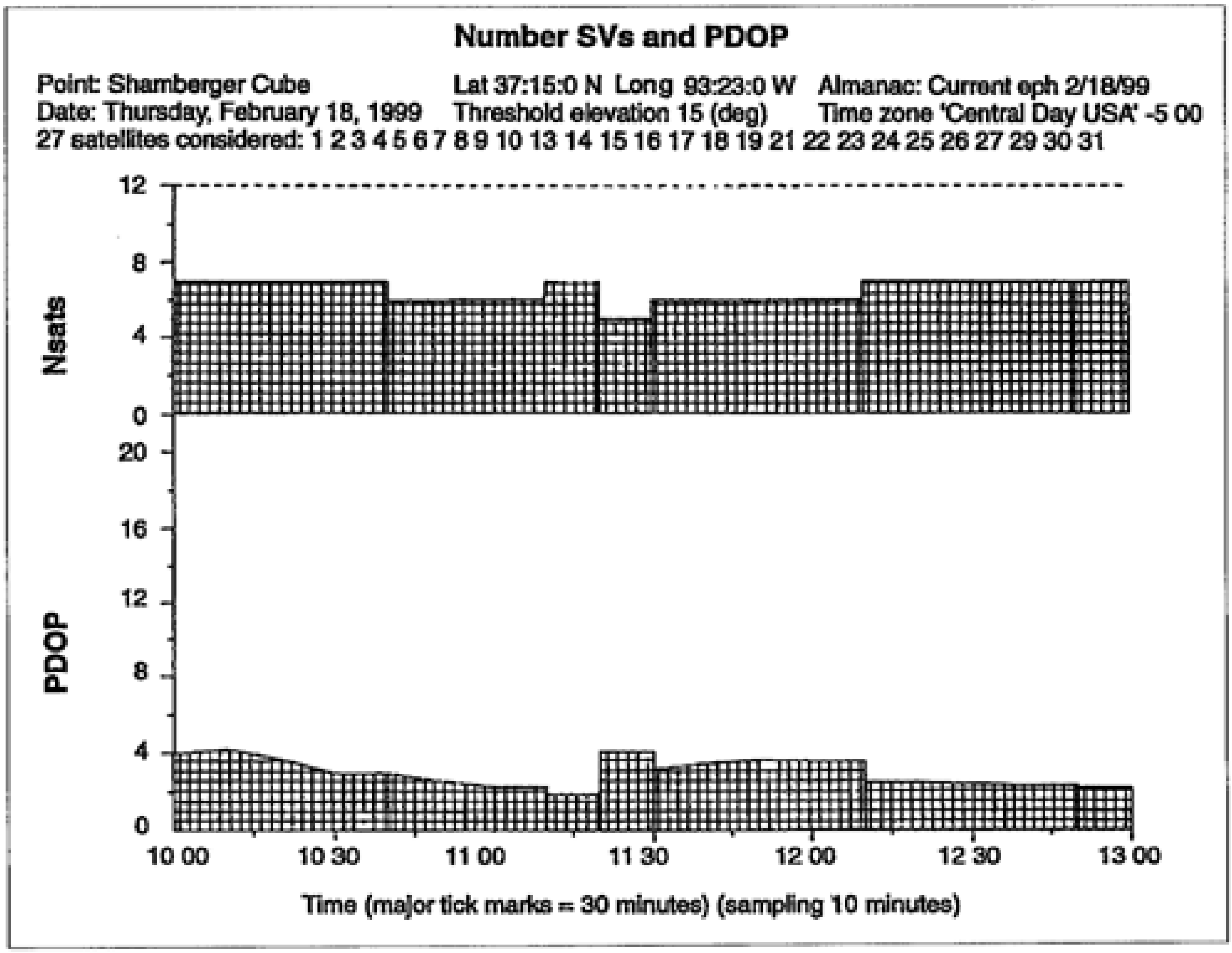


Figure 5-3. Sample Satellite Sky Plot



Figures 5-4. PDOP Versus Time Plot

PART E - CONDUCTING PRECISE-POSITIONING SURVEYS

5-33. General. This section presents guidance to field personnel performing GPS-Ss for all types of projects. The primary emphasis in this section is on static and kinematic carrier phase DGPS measurements. The following are some general GPS field procedures that should be performed at each station, observation, or session on a GPS-S.

a. Receiver Setup. GPS receivers should be set up according to manufacturers' specifications before to beginning any observations. To eliminate the possibility of missing the beginning of the observation session, all equipment should be set up with power supplied to the receivers at least 10 minutes before to the beginning of the observation session. Most receivers will lock on to satellites within 1 to 2 minutes of power-up.

b. Antenna Setup. All tribrachs should be calibrated and adjusted before to beginning each project. Since centering errors represent a major error source in all survey work (not just GPS surveying), use both optical plummets and standard plumb bobs.

c. Height of Instrument. HI refers to the correct measurement of the distance of the GPS antenna above the reference monument over which it is placed. HI measurements should be made before and after each observation session. The HI should be made from the monument to a standard reference point on the antenna. The standard reference points for each antenna should be established before to the beginning of the observations. All measurements should be made both in meters and feet for redundancy and blunder detection. Determine the measurement to the nearest millimeter in metric units and the nearest 0.01 foot in US units. Note whether the HI is vertical or diagonal.

d. Field Global Positioning System Observation Recording Procedures. Field-recording books, log sheets, or log forms should be completed for each station and session. Any acceptable recording media may be used. Recording sheets or forms will be used for archiving purposes. The amount of recording detail will depend on the project. Low-order geographic mapping points do not need as much descriptive information as permanently marked, primary control points. Unit commands may require that additional data be recorded. These requirements are contained in the project instructions. The following data may be included on the field log records:

- Project name, directive number, observer name(s), and unit name.
- Station designation number.
- Station file number.
- Date and weather conditions.
- Session start and stop time (local and universal time, coordinated [UTC]).
- Receiver; antenna; data-recording unit; and tribrach make, model, and serial number.
- Antenna height (vertical or diagonal measurements in metric or US units).
- Satellite vehicle (SV) designation and number.
- Station-location sketch.
- Geodetic location and elevation (approximate).
- Problems encountered.

e. Field Processing and Verification. It is strongly recommended that GPS data processing and verification be performed in the field (when applicable) so that any problems may be identified and corrected before returning from the field.

5-34. Absolute Positioning. The accuracy obtained by GPS point positioning is dependent on the user's authorization. The SPS user can provide an accuracy of 80 to 100 meters. SPS data are most often expressed in real time; however, the data can be postprocessed if the station occupation took place over a period of time. Postprocessing produces a best-fit point position. Although this will provide a better internal approximation, the effects of S/A (when activated) still degrade positional accuracy up to 80 to 100 meters. The PPS user requires a decryption device within the receiver to decode the effects of S/A. The PPS provides an accuracy reading between 10 to 16 meters when a single-frequency receiver is used for observation. Dual-frequency receivers using the precise ephemeris may produce an absolute positional accuracy on the order of 1 meter or better. These positions are based on the absolute WGS-84 ellipsoid. The PPS that uses the precise ephemeris requires the data to be postprocessed. The military uses a GPS-S receiver that is capable of meter-level GPS point positioning without postprocessing.

5-35. Differential Positioning. Differential GPS surveying is the determination of one location with respect to another location. When using this technique with the C/A- or P-code, it is called differential code phase positioning. Differential code phase positioning has limited application to detailed engineering surveying and topographic site mapping applications. Exceptions include general reconnaissance surveys and operational military or geodetic-survey support functions. Additional applications for differential code phase positioning have been on the increase as positional accuracy has increased. The code phase tracking differential system is a functional GPS-S for positioning hydrographic-survey vessels and dredges. It also has application for topographic, small-scale mapping surveys. The collected data is used as input for a geographic information system database. A real-time dynamic DGPS positioning system includes a reference station, a communication link, and remote-user equipment. If results are not required in real time, the communication link can be eliminated and the positional information is postprocessed.

a. Accuracy of Differential Global Positioning System Surveys. Differential code phase surveys can obtain accuracies of 0.5 to 0.05 meter. This type of survey can be used for small-scale mapping or as input to a geographic information system (GIS) database.

b. Reference Station. A reference station is placed on a known survey monument. It is an area with an unobstructed view of at least four satellites, 10° above the horizon. It consists of a GPS receiver, a GPS antenna, a processor, and a communication link (if real-time results are desired). The reference station measures the timing and ranging information broadcast by the satellites and computes and formats range corrections for broadcast to the user equipment. Using differential pseudoranging, the position of a survey vessel is found relative to the reference station. The pseudoranges are collected by the GPS receiver and transferred to the processor where PRCs are computed and formatted for data transmission. Many manufacturers have incorporated the processor within the GPS receiver, eliminating the need for an external processing device. The recommended data format is that proposed by the Radio Technical Commission for Maritime (RTCM) Services Special Committee (SC). The processor should be capable of computing and formatting PRCs every 1 to 3 seconds.

c. Communication Link. A communication link is used as a transfer media for differential corrections. The main requirement of the communication link is that the transmission be at a minimum rate of 300 bits per second. The type of communication system is dependent on the user's requirements.

5-36. Differential Global Positioning System Carrier Phase Horizontal-Positioning Techniques. Differential GPS carrier phase surveying is used to obtain the highest precision from GPS and has direct application to most military topographic and engineering surveys. Manufacturers' procedures should be followed for conducting a GPS field survey. The following six basic DGPS techniques are in use:

- Static.
- Stop-and-go kinematic.
- Kinematic.
- Pseudokinematic.
- Rapid static.
- OTF/RTK.

Project horizontal-control densification can be performed using any one of these techniques. Procedurally, all six techniques are similar in that each measures a 3D baseline vector between a receiver at one point (usually of known local-project coordinates) and a second receiver at another point, resulting in a vector difference between the two occupied points. The major distinction between static and stop-and-go kinematic baseline measurements is the method by which the carrier-wave integer-cycle ambiguities are resolved; otherwise, they are functionally the same.

- Cycle ambiguity (or integer ambiguity) is the unknown number of whole carrier wavelengths between the satellite and receiver. Successful ambiguity resolution is required for successful baseline formulations. In static surveying, instrumental error and ambiguity resolution can be achieved through long-term averaging and simple geometrical principles, resulting in solutions to a linear equation that produces a resultant position. Ambiguity resolution can also be achieved through a combination of the pseudorange and carrier beat measurements, made possible by knowledge of the PRN modulation code.

- All carrier phase relative-surveying techniques, except OTF/RTK, require postprocessing of the observed data to determine the relative baseline vector differences. OTF/RTK can be performed in real time or in the postprocessed mode. Postprocessing of observed satellite data involves the differencing of signal phase measurements recorded by the receiver. The differencing process reduces biases in the receiver and satellite oscillators and is performed in a computer. When contemplating the purchase of a receiver, the computer requirements necessary to postprocess the GPS data must be considered. Most manufacturers require a personal computer (PC) with a math coprocessor. All baseline reductions should be performed in the field (if possible) to allow an onsite assessment of the survey adequacy.

a. Static Surveying. Static surveying is the most common method of densifying project network control. Two GPS receivers are used to measure a GPS baseline distance. The line between a pair of GPS receivers from which simultaneous GPS data have been collected and processed is a vector referred to as a baseline. The station coordinate differences are calculated in terms of a 3D ECEF coordinate system that utilizes X, Y, and Z values based on the WGS-84 ellipsoid. These coordinate differences are then subsequently shifted to the local-project coordinate system. GPS receiver pairs are set up over stations of either known or unknown locations. Typically one of the receivers is positioned over a point whose coordinates are known (or have been carried forward as on a traverse) and the second is positioned over another point whose coordinates are unknown, but desired. Both GPS receivers must receive signals from the same four (or more) satellites for a period of time ranging from a few minutes to several hours, depending on the conditions of observation and the precision required.

(1) Station occupation time is dependent on the baseline length, the number of satellites observed, and the GPS equipment. A good approximation for a baseline occupation of 1 to 30 kilometers is 30 minutes to 2 hours. A rough guideline for estimating station occupation time is shown in Figure 5-5.

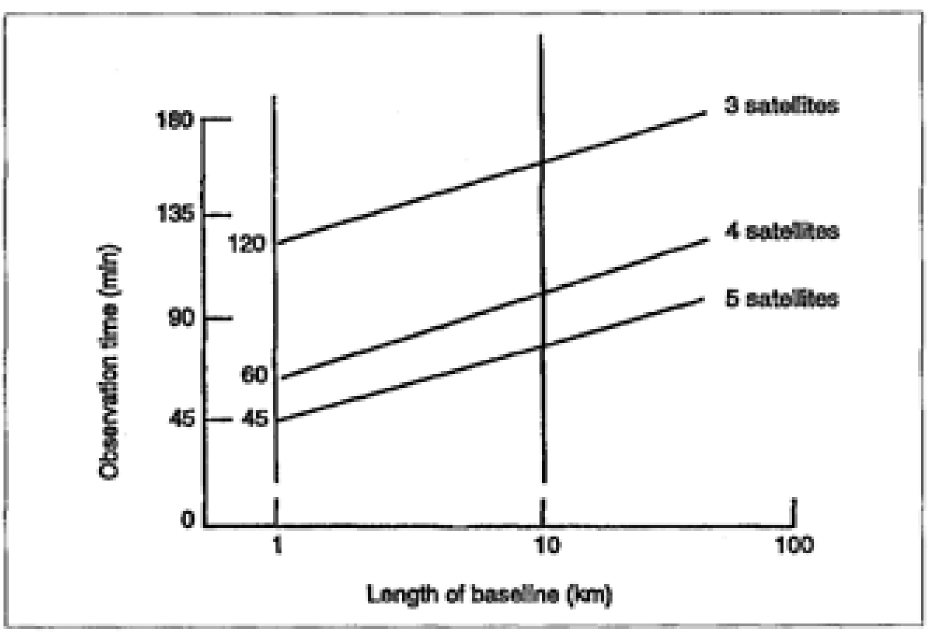


Figure 5-5. Station Occupation Time

(2) The stations that are selected for survey must have an unobstructed view of the sky 15° or greater above the horizon during the *observation window*. An observation window is the period of time when observable satellites are in the sky and the survey can be successfully conducted.

(3) It is critical for a static survey baseline reduction or solution that the receivers simultaneously observe the same satellites during the same time interval. For instance, if a receiver observes a satellite constellation during the time interval 1,000 to 1,200 and another receiver observes that same satellite constellation during the time interval 1,100 to 1,300, only the period of common observation (1,100 to 1,200) can be processed to formulate a correct vector difference between these receivers.

(4) After completing the observation session, the GPS signals from both receivers are processed in a computer to calculate the 3D baseline vector components between the two observed points. From these vector distances, local or geodetic coordinates may be computed or adjusted.

(5) Static baselines may be extended from existing control using the control densification method. This method includes networking, traverse, spur techniques, or combinations thereof. Specific requirements are normally contained in project instructions.

(6) Specific receiver operation and baseline data postprocessing requirements are manufacturer-dependent. The user should consult and study the manufacturer's manual (including the baseline data reduction examples).

(7) Accuracy of static surveys usually exceeds 1 part per million (ppm). Static is the most accurate of all GPS processing methods and can be used for any order survey.

b. Stop-and-Go Kinematic Surveying. Stop-and-go kinematic surveying is similar to static surveying in that each method requires at least two receivers simultaneously recording observations. A major difference between static and stop-and-go surveying is the amount of time required for a receiver to stay fixed over a point of unknown position. In stop-and-go surveying, the first receiver (the home or reference receiver) remains fixed on a known control point. The second receiver (the rover receiver) collects observations statically on a point of unknown position for a period of time (usually a few minutes) and then moves to subsequent unknown points to collect signals for a short period of time. During the survey, at least four (preferably five) common satellites need to be continuously tracked by both receivers. Once all required points have been occupied by the rover receiver, the observations are postprocessed by a computer to calculate the baseline vector and coordinate differences between the known control point and points occupied by the rover receiver during the survey session. The main advantage of this method over static surveying is the reduced occupation time required over the unknown points. Because stop-and-go surveying requires less occupation time over known points, the time spent and the cost of conducting the survey are significantly reduced. The achievable accuracies typically equal or exceed third order.

(1) Stop-and-go surveying is performed similarly to a conventional EDM traverse or electronic total station radial survey. The system is initially calibrated by performing either an antenna swap with one known point and one unknown point or by performing a static measurement over a known baseline. This calibration process is performed to resolve initial cycle ambiguities. This known baseline may be part of the existing network or can be established using static GPS-S procedures. The remote roving receiver then traverses between unknown points as if performing a radial topographic survey. Typically, the points are double-connected, or double-run, as in a level line. Optionally, two fixed receivers may be used to provide redundancy on the remote points. With only 1 1/2 minutes at a point, production of coordinate differences is high and limited only by satellite observation windows, travel time between points, and overhead obstructions.

(2) During a stop-and-go kinematic survey, the rover station must maintain satellite lock on at least four satellites during the survey period (the reference station must be observing at least the same four satellites). Loss of lock occurs when the receiver is unable to continuously record satellite signals or the transmitted satellite signal is disrupted and the receiver is not able to record it. If satellite lock is lost, the roving receiver must reobserve the last control station surveyed before loss of lock. The receiver operator must monitor the GPS receiver when performing the stop-and-go survey to ensure loss of lock does not occur. Some manufacturers have now incorporated an alarm into their receiver that warns the user when loss of lock occurs.

(3) Survey site selection and the route between rover stations to be observed are critical. All sites must have a clear view (a vertical angle of 15º or greater) of the satellites. The routes between rover occupation stations must be clear of obstructions so that the satellite signal is not interrupted. Each unknown station to be occupied should be occupied for a minimum of 1 1/2 minutes. Stations should be occupied two or three times to provide redundancy between observations.

(4) Although the antenna swap procedure can be used to initialize a survey before to a stop-and-go survey, it can also be used to determine a precise baseline and azimuth between two points. An unobstructed view of the horizon must be maintained at both occupied stations and the path between them. A minimum of four satellites (although more than four satellites are preferred) and a maintainable satellite lock are required. One receiver or antenna is placed over a point of known control and the second receiver or antenna is placed a distance of 10 to 100 meters away from the other receiver. The receivers at each station collect data for about 2 to 4 minutes. The receiver or antenna locations are then swapped. The receiver or antenna at the known station is moved to the unknown site while the other receiver or antenna is moved to the known site. Satellite data are again collected for 2 to 4 minutes, and the receivers are swapped back to their original locations. This completes one antenna swap calibration. If satellite lock is lost during the procedure, the procedure must be repeated.

(5) Accuracy of stop-and-go baseline measurements will usually well exceed 1 part in 5,000; therefore, third-order classification for horizontal control can be effectively, efficiently, and accurately established using this technique. For many projects, this order of horizontal accuracy will be more than adequate; however, field procedures should be designed to provide adequate redundancy for *open-ended* or *spur* points. Good satellite geometry and minimum multipath are also essential in performing acceptable stop-and-go surveys.

c. Kinematic Surveying. Kinematic surveying using differential carrier phase tracking is similar to stop-and-go kinematic and static differential carrier phase GPS surveying because it also requires two receivers recording observations simultaneously. Kinematic surveying is often referred to as dynamic surveying. As in stop-and-go surveying, the reference receiver remains fixed on a known control point while the roving receiver collects data on a constantly moving platform (such as a vehicle, a vessel, an aircraft, or a backpack). Unlike stop-and-go surveying, kinematic surveying techniques do not require the rover receiver to remain motionless over the unknown point. The observation data are later postprocessed with a computer and the relative vector or coordinate differences to the roving receiver are calculated.

(1) A kinematic survey requires two L1 receivers. One receiver is set over a known point (reference station) and the other is used as a rover (moved from point to point or along a path). Before the rover receiver can move, a period of static initialization or antenna swap must be performed. This period of static initialization is dependent on the number of satellites visible. Once this is done, the rover receiver can move from point to point as long as satellite lock is maintained on at least four common satellites (common with the known reference station). If loss of lock occurs, a new period of static initialization must take place. It is important to follow manufacturers' specifications when performing a kinematic survey.

(2) Kinematic data-processing techniques are similar to those used in static surveying. When processing kinematic GPS data, the user must ensure that satellite lock is maintained on four or more satellites and that cycle slips are adequately resolved within the data recorded.

(3) Differential (carrier phase) kinematic survey errors are correlated between observations received at the reference and rover receivers, as in differential static surveys. Experimental test results indicate kinematic surveys can produce results in centimeters. Test results from an experimental full-kinematic GPS-S conducted by Topographic Engineer Center (TEC) personnel at White Sands Missile Range, New Mexico, verified (under ideal test conditions) that kinematic GPS surveying could achieve centimeter-level accuracy over distances up to 30 kilometers.

d. Pseudokinematic Surveying. Pseudokinematic GPS surveying is similar to stop-and-go surveying except that loss of lock is tolerated when the receiver is transported between occupation sites (the roving receiver can be turned off during movement between occupation sites, but this is not recommended). This feature provides the surveyor with a more favorable positioning technique since obstructions such as a bridge overpasses, tall buildings, and overhanging vegetation are common. A loss of lock resulting from these obstructions is more tolerable when pseudokinematic techniques are employed.

(1) Pseudokinematic techniques require that one receiver be placed over a known control station. A rover receiver occupies each unknown station for 5 minutes. About 1 hour after the initial station occupation, the same rover receiver must reoccupy each unknown station.

(2) Pseudokinematic techniques require that at least four of the same satellites are observed between the initial station occupations and the requisite reoccupation. For example, the rover receiver occupies Station A for the first 5 minutes and tracks satellites 6, 9, 11, 12, and 13; then 1 hour later, during the second occupation of Station A, the rover receiver tracks satellites 2, 6, 8, 9, and 19. Only satellites 6 and 9 are common to the two sets, so the data cannot be processed because four common satellites were not observed between the initial station occupation and the requisite reoccupation.

(3) Mission planning is essential in conducting a successful pseudokinematic survey. Especially critical is the determination of whether or not common satellite coverage will be present for the desired period of the survey. During the period of observation, one receiver (the base receiver) must continuously occupy a known control station.

(4) Pseudokinematic survey satellite data records and resultant baseline-processing methods are similar to those performed for static GPS-Ss. Since the pseudokinematic technique requires each station to be occupied for 5 minutes and then reoccupied for 5 minutes about one hour later, this technique is not suitable when control stations are widely spaced and the transportation between stations within the allotted time is impractical. Pseudokinematic survey accuracies are similar to kinematic survey accuracies of a few centimeters.

e. Rapid-Static Surveying. Rapid-static surveying is a combination of stop-and-go kinematic, pseudokinematic, and static surveying methods. The rover receiver spends only a short time on each station (loss of lock is allowed between stations) and accuracies are similar to static surveying. However, rapid-static surveying does not require reobservation of remote stations like pseudokinematic surveying. The rapid-static technique requires the use of dual-frequency GPS receivers with either cross correlation, squaring, or any other technique used to compensate for AS.

(1) Rapid-static surveying requires that one receiver be placed over a known control point. A rover receiver occupies each unknown station for 5 to 20 minutes, depending on the number of satellites and their geometry. Because most receiver operations are manufacturer specific, following the manufacturers' guidelines and procedures is important.

(2) Data collected in the rapid-static mode should be processed according to the manufacturers' specifications. Accuracies are similar to static surveys of a centimeter or less. This method can be used for medium- to high-accuracy surveys up to 1:1,000,000.

f. On-the-Fly/Real-Time Kinematic Surveying. OTF/RTK surveying is similar to kinematic surveying because it requires two receivers that record observations simultaneously and allows the rover receiver to move. Unlike kinematic surveying, OTF/RTK surveying uses dual-frequency GPS observations and can handle loss of lock. Since OTF/RTK surveying uses the L2 frequency, the GPS receiver must be capable of tracking the L2 frequency during AS. There are several techniques used to obtain L2 during AS, including the squaring and cross-correlation methods.

(1) Successful ambiguity resolution is required for successful baseline formulations. The OTF/RTK technology allows the remote to initialize and resolve these integers without a period of static initialization. If loss of lock occurs, reinitialization can be achieved while the remote is in motion. The integers can be resolved at the rover within 10 to 30 seconds, depending on the distance from the reference station. OTF/RTK surveying uses the L2 frequency transmitted by the GPS satellites in the ambiguity resolution. After the integers are resolved, only the L1 C/A-code is used to compute the positions.

(2) OTF/RTK surveying requires dual-frequency GPS receivers. One of the GPS receivers is set over a known point and the other is placed on a moving or mobile platform. If the survey is performed in real time, a data link and a processor (external or internal) are needed. The data link is used to transfer the raw data from the reference station to the remote.

(a) Internal Processor. If OTF/RTK surveying is performed with an internal processor (built into the receiver), follow the manufacturer's guidelines.

(b) External Processor. If OTF/RTK surveying is performed with external processors (a notebook computer), the PC at the reference station collects and formats the raw GPS data and sends it via a data link to the rover. The notebook computer at the rover processes the raw data from the reference and remote receivers to resolve the integers and obtain a position.

(3) OTF/RTK surveys are accurate to within 10 centimeters when the distance from the reference to the rover does not exceed 20 kilometers. Testing by TEC personnel has produced accuracies of less than 10 centimeters.

PART F - PROCESSING PRECISE-POSITIONING SURVEY DATA

5-37. General. GPS baseline solutions are usually generated through an iterative process. From approximate values of the positions occupied and observation data, theoretical values for the observation period are developed. Observed values are compared to computed values and an improved set of positions occupied is obtained using least squares minimization procedures and equations that model potential error sources. This section discusses general postprocessing issues. Due to the increasing number and variety of software packages available, consult the manufacturer's guidelines when appropriate. Processing time is dependent on the accuracy required, the available software, the computer hardware, the data quality, and the amount of data. In general, high-accuracy solutions, crude computer software and hardware, low-quality data, and high volumes of data will cause longer processing times. The user must take special care when attempting a baseline formulation with observations from different brands of GPS receivers. It is important to ensure that observables being used for the formulation of the baseline are of common format. The common data-exchange formats required for a baseline formulation exist only between receivers produced by the same manufacturer, even though there are some exceptions.

5-38. Processing Techniques. The capability to determine positions using GPS is dependent on the ability of the user to determine the range or distance of the satellite from the receiver located on the earth. There are two general techniques used to determine this range--pseudoranging and carrier beat phase measurement.

a. Pseudoranging. The observable pseudorange is calculated from observations recorded during a GPS-S. The observable pseudorange is the difference between the time of signal transmission from the satellite (measured in the satellite time scale) and the time of signal arrival at the receiver antenna (measured in the receiver time scale). When the differences between the satellite and the receiver clocks are reconciled and applied to the pseudorange observables, the result is corrected pseudorange values. The value found by multiplying this time difference by the speed of light is an approximation of the true range between the satellite and the receiver. The value can be determined if ionosphere and troposphere delays, ephemeris errors, measurement noise, and unmodeled influences are taken into account during pseudoranging calculations. The pseudorange can be obtained from either the C/A-code or the more precise P-code.

b. Carrier Beat Phase Measurement. The observable carrier beat phase is the phase of the signal remaining after the internal oscillated frequency generated in the receiver is differenced from the incoming carrier signal of the satellite. The observable carrier beat phase can be calculated from the incoming signal or from observations recorded during a GPS-S. By differencing the signal over a period or epoch of time, the number of wavelengths that cycle through the receiver during any given specific duration of time can be counted. The unknown cycle count passing through the receiver over a specific duration of time is known as the cycle ambiguity. There is one cycle-ambiguity value per satellite/receiver pair as long as the receiver maintains continuous phase lock during the observation period. The value found by measuring the number of cycles going through a receiver during a specific time, when given the definition of the transmitted signal in terms of cycles per second, can be used to develop a time measurement for transmission of the signal. Again, the time of transmission of the signal can be multiplied by the speed of light to yield an approximation of the range between the satellite and receiver. The biases for carrier beat phase measurements are the same as for pseudoranges, although a higher accuracy can be obtained using the carrier. A more exact range between the satellite and receiver can be formulated when the biases are taken into account during derivation of the approximate range between the satellite and the receiver.

5-39. Baseline Solution by Linear Combination. The level of accuracy achievable by pseudoranging and carrier beat phase measurement in both absolute and relative positioning surveys can be improved through processing that incorporates differencing of the mathematical models of the observables. Processing by differencing takes advantage of correlation of error (such as GPS signal, satellite ephemeris, receiver clock, and atmospheric-propagation errors) between receivers, satellites, and epochs, or combinations thereof, to improve GPS processing. Through differencing, the effects of the errors common to the observations being processed are greatly reduced or eliminated. There are three broad processing techniques that incorporate differencing--single, double, and triple. Differenced solutions generally proceed in the following order: differencing between receivers, between satellites, and between epochs.

a. Single Differencing. There are three general single-differencing processing techniques--between receivers, between satellites, and between epochs.

(1) Between Receivers. Single differencing the mathematical models for pseudorange (C/A- or P-code) carrier phase observable measurements between receivers will eliminate or greatly reduce satellite clock errors and a large amount of satellite orbit and atmospheric delays.

(2) Between Satellites. Single differencing the mathematical models for pseudorange or carrier phase observable measurements between satellites eliminates receiver clock errors. Single differencing between satellites can be done at each individual receiver during observations as a precursor to double differencing and to eliminate receiver clock errors.

(3) Between Epochs. Single differencing the mathematical models between epochs takes advantage of the Doppler shift (apparent change in the frequency of the satellite signal by the relative motion of the transmitter and the receiver). Single differencing between epochs is generally done in an effort to eliminate cycle ambiguities. Three forms of single-differencing techniques between epochs are intermittently integrated Doppler, consecutive Doppler counts, and continuously integrated Doppler.

b. Double Differencing. Double differencing is a differencing of two single differences. Double-difference processing techniques eliminate clock errors. There are two general double-differencing processing techniques--receiver time and receiver satellite.

(1) Receiver Time. This technique uses a change from one epoch to the next in the between receiver single differences for the same satellite. This technique eliminates satellite-dependent integer cycle ambiguities and simplifies the editing of cycle slips.

(2) Receiver Satellite. There are two different methods that can be used to compute a receiver satellite double difference. One method involves using two between-receiver single differences and a pair of receivers that record different satellite observations between two satellites. The second method involves using two between-satellite single differences and a pair of satellites, but with different receivers, which differences the satellite observations between the two receivers.

c. Triple Differencing. There is only one triple-differencing processing technique--receiver satellite time. All errors eliminated during single- and double-differencing processing are also eliminated during triple differencing. When used in conjunction with carrier beat phase measurements, triple differencing eliminates initial cycle ambiguity. During triple differencing, the data are automatically edited by the software to delete any data ignored during the triple-difference solution. This feature is advantageous because of the reduction in the editing of data required; however, degradation of the solution may occur if too much data are eliminated.

5-40. Baseline Solutions by Cycle-Ambiguity Recovery. The resultant solution (baseline vector) produced from carrier beat phase observations when differencing resolves cycle ambiguity is called a fixed solution. The exact cycle ambiguity does not need to be known to produce a solution. If a range of cycle ambiguities is known, then a float solution can be formulated from the range of cycle ambiguities. It is desirable to formulate a fixed solution. However, when the cycle ambiguities cannot be resolved, which occurs when a baseline is between 20 to 65 kilometers, a float solution may actually be the best solution. The fixed solution may be unable to determine the correct set of integers (fix the integers) required for a solution. Double-differenced fixed techniques can effectively be used for positional solutions over short baselines of less than 20 kilometers. Double-differenced float techniques normally can be effectively used for positional solutions of medium-length lines between 20 and 65 kilometers.

5-41. Data Processing and Verification. Baselines should be processed daily in the field to identify any problems that may exist. Once baselines are processed, the field surveyor should review each baseline output file. The procedures used in baseline processing are manufacturer-dependent. Certain computational items within the baseline output are common among manufacturers and may be used to evaluate the adequacy of the baseline observation in the field. The triple-difference float solution is normally listed. The geodetic azimuth and the distance between the two stations are also listed. The RMS is a quality factor that helps identify which vector solution (triple, float, or fixed) to use in the adjustment. The RMS is dependent on the baseline length and the length of baseline observation. Table 5-4 provides guidelines for determining the baseline quality. If the fixed solution meets the criteria in this table, the fixed vector should be used in the test. If the vector does not fit into the network adjustment, the surveyor should use the float vector in the adjustments or check to make sure that the stations were occupied correctly.

Table 5-4. Guidelines for Determining the Baseline Quality

Distance Between Receivers (km)	RMS Criteria Formulation (d = Distance Between Receivers)	Formulated RMS Range (Cycles)	Formulated RMS Range (m)
0 -10	$\leq[0.02 + (0.0040 \bullet d)]$	0.020 - 0.060	0.004 - 0.012
10 - 20	$\leq[0.03 + (0.0030 \bullet d)]$	0.060 - 0.090	0.012 - 0.018
20 - 30	$\leq[0.04 + (0.0025 \bullet d)]$	0.090 - 0.115	0.018 - 0.023
30 - 40	$\leq[0.04 + (0.0025 \bullet d)]$	0.115 - 0.140	0.023 - 0.027
40 - 60	$\leq[0.08 + (0.0015 \bullet d)]$	0.140 - 0.170	0.027 - 0.032
60 - 100	≤ 0.17	0.170	0.032
>100	≤ 0.20	0.200	0.040

NOTES:
1. These are general postprocessing criteria that may be superseded by GPS receiver/software manufacturers' guidelines; consult those guidelines when appropriate.
2. For lines longer than 50 kilometers, dual-frequency GPS receivers are recommended to meet these criteria.

5-42. Postprocessing Criteria. Generally, postprocessing software will provide three solutions--a triple difference, a double-difference fixed solution, and a double-difference float solution. In addition to the relative dilution of precision (RDOP) as a measurement of the quality of data reduction, two methods that can be used to gauge the success of an observation session (based on data processing done by a differencing process) are RMS and repeatability.

a. An RMS is a measurement (in units of cycles or meters) of the quality of the observation data collected during a particular time period. RMS is dependent on the line length, the observation strength, the ionosphere, the troposphere, and multipath effects. In general, the longer the line and the more signal interference by the ionosphere, the troposphere, multipath effects, and other electronic gear, the higher the RMS. A good RMS factor (between 0.01 and 0.2 cycles) may not always indicate good results but is one indication to be taken into account. RMS can generally be used to judge the quality of the data used in the postprocessing and the quality of the postprocessed baseline vector.

b. Redundant lines should agree to the level of accuracy that GPS is capable of measuring. For example, if GPS can measure a 10-kilometer baseline to 1 centimeter ± 1 ppm, the expected ratio of misclosure would be:

$$\frac{1\,cm \pm ppm}{baseline} =$$

$$\frac{0.01\,m \pm 0.01\,m}{10,000\,m} = 1{:}500,000$$

NOTE: Repeated baselines should be near the corresponding ratio.

c. A baseline solution typically includes the following information:

- The filename.
- The type of solutions (single-, double-, or triple-difference).
- The satellite availability during the survey for each station occupied.
- The ephemeris file used for the solutions.
- The type of satellite selection (manual or automatic).
- The elevation mask.
- The minimum number of satellites used.
- Meteorological data (for example, pressure, temperature, or humidity).
- The session date and time.
- The data-logging start and stop time.
- Station information (such as the location, the serial number, and the antenna serial number and height).
- The RMS.
- The epoch intervals.
- The solution files (ΔX, ΔY, and ΔZ between stations, the slope distance between stations, Δlatitude and Δlongitude between stations, the horizontal distance between stations, and Δheight).
- The number of epochs.

d. Sample static-baseline formulations are shown in Figure 5-7. The baseline formulations compensate for the height differences between antennas.

			U S and NATO Military Forces
Project Name:	Belvoir		
Processed:	Sunday, October 19, 1997 16:59		
	WAVE 2.10		
Solution Output File :	00000272.SSF	IMPORTED	
From Station:	DTP4		
Data file:	DTP40722.DAT		
Antenna Height (meters):	1.608 True Vertical	1.618 Uncorrected	
Position Quality:	Fixed Baseline Solution		
WGS 84 Position:	38° 41' 23.838157" N		X 1109965.311
	77° 08' 03.891696" W		Y -4859774.737
	6.725		Z 3965514.263
To Station:	FB09		
Data file:	FB090722.DAT		
Antenna Height (meters):	1.611 True Vertical	1.621 Uncorrected	
WGS 84 Position:	38° 41' 42.125849" N		X 1108978.939
	77° 08' [illegible]0" W		Y -4859637.075
	5.16[illegible]		Z 3965953.432
Start Time:	3/[illegible] 16:46:00.00 GPS	(844 233160.00)	
Stop Time:	[illegible]/12/96 17:50:15.00 GPS	(844 237015.00)	
Occupation Time Meas. Interval [illegible]	01:04:15.00	15.00	
Solution Type:	L1 fixed double difference		
Ephemeris:	Broadcast		
Met Data:	Standard		
Baseline Slope Distance Std. Dev. (meters):	1088.462	0.000185	

		Forward	Backward
Normal Section Azimuth:		301° 12' 27.087988"	121° 12' 03.005418"
Vertical Angle:		-0° 05' 13.855491"	0° 04' 38.662749"

Baseline Components (meters):	dx	-986.371	dy	137.662	dz	439.169
Standard Deviations (meters):		0.000215		0.000458		0.000422
	dn	563.974	de	-930.956	du	-1.656
		0.000174		0.000166		0.000613
					dh	-1.563
						0.000613

Aposteriori Covariance Matrix:	4.611597E-008		
	-6.156637E-008	2.097716E-007	
	4.545004E-008	-1.653685E-007	1.777377E-007
Variance Ratio:	76.5		
Reference Variance:	0.880		
Observable Count/Rejected RMS:	L1 phase	1005/0	0.004

Figure 5-7. Sample Static Baseline Formulations

5-43. Loop Closure Checks. Postprocessing criteria are aimed at an evaluation of a single baseline. To verify the adequacy of a group of connected baselines, one must perform a loop closure on the baselines formulated. When GPS baseline traverses or loops are formed, their linear (internal) closure should be determined in the field. If the job requires less than third-order accuracy (1:10,000 or 1:5,000) and the internal loop/traverse closures are very small, a formal (external) adjustment may not be warranted.

a. The internal closure determines the consistency of the GPS measurements. Internal closures are applicable for loop traverses and GPS networks. It is required that one baseline in the loop be independent. An independent baseline is observed during a different session or on a different day. Many of the better postprocessing software packages come with a loop closure program. Refer to the manufacturers' postprocessing user's manual for the particulars of the loop closure program included with the hardware. If the postprocessing software package does not contain a loop closure program, the user can perform the following loop closure computation:

Step 1. List the ΔX, ΔY, ΔZ, and the distance components for all baselines used in the loop closure.

Step 2. Sum the ΔX, ΔY, ΔZ, and the distance components for all baselines used in the loop closure.

Step 3. Add the square of each of the summations together and take the square root of this sum.

This resultant value is the misclosure vector for the loop.

Step 4. The loop misclosure ratio is calculated as follows:

Loop misclosure ratio = m/L

Where--

m = misclosure for the loop
L = total loop distance (perimeter distance)

The resultant value can be expressed as 1:loop misclosure ratio. All units for the expressions are stated in terms of the units used in the baseline formulations (such as meters, feet, or millimeters).

b. External closures are computed in a manner similar to internal loops. External closures provide information on how well the GPS measurements conform to the local coordinate system. Before the closure of each traverse is computed, the latitude, the longitude, and the ellipsoid height must be converted to geocentric coordinates (X, Y, and Z). If the ellipsoid height is not known, geoid-modeling software can be used with the orthometric height to get an approximate ellipsoid height. The external closure aids the surveyor in determining the quality of the known control and how well the GPS measurements conform to the local network. If the control stations are not of equal precision, the external closures will usually reflect the lower-order station. If the internal closure meets the requirements of the job, but the external closure is poor, the known control is probably deficient and an additional known control point should be included in the system.

5-44. Data Archival. The raw data is the data recorded during the observation period. Raw data should be stored on an appropriate medium (such as a compact disk, a portable hard drive, or magnetic tape). The raw data and the hard copy of the baseline reduction (resultant baseline formulations) should be stored at the discretion of each unit's command.

PART G - ADJUSTING PRECISE-POSITIONING SURVEYS

5-45. General. Differential carrier phase GPS-S observations are adjusted the same as conventional survey observations. Each 3D GPS baseline vector is treated as a separate distance observation and adjusted as part of a network. A variety of techniques may be used to adjust the observed GPS baselines to fit existing control. Since GPS-S networks often contain redundant observations, they are usually adjusted by some type of rigorous least squares minimization technique. This section describes some of the methods used to perform horizontal GPS-S adjustments and provides guidance in evaluating the adequacy and accuracy of the adjustment results.

5-46. Global Positioning System Error Measurement Statistical Terms. To understand the adjustment results of a GPS-S, it is necessary to understand the following statistical terms:

- Accuracy. Accuracy is how close a measurement or a group of measurements is in relation to a true or known value.

- Precision. Precision is how close a group or sample of measurements is to other groups or samples. For example, a low standard deviation indicates high precision. A survey or group of measurements can have a high precision but a low accuracy (for example, measurements are close together but not close to the known or true value).

- Standard Deviation. Standard deviation is a range of how close the measured values are from the arithmetic average. A low standard deviation indicates that the observations or measurements are close together.

5-47. Adjustment Considerations. This section deals primarily with the adjustments of horizontal control established using GPS observations. Although vertical elevations are carried through the baseline reduction and adjustment process, the relative accuracy of these elevations is normally inadequate for engineering and construction purposes. Special techniques and constraints are necessary to determine approximate orthometric elevations from relative GPS observations.

a. The baseline reduction process provides the raw relative position coordinates that are used in a 3D GPS network adjustment. Depending on the manufacturer's software, each reduced baseline contains various orientation parameters, covariance matrices, and cofactor or correlation statistics that may be used in weighting the final network adjustment. Most least squares adjustments use the accuracy or correlation statistics from the baseline reduction; however, other weighting methods may be used in a least squares or approximate adjustment.

b. The adjustment technique employed (and the time devoted to it) must be commensurate with the project's accuracy requirements. Care must be taken to prevent the adjustment process from becoming a project in itself.

c. There is no specific requirement that a rigorous least squares adjustment be performed on topographic surveys, whether conventional, GPS, or mixed observations. Traditional approximate-adjustment methods may be used in lieu of the least squares method and provide comparable, practical accuracy results.

d. Commercial software packages designed for higher-order geodetic-densification surveys often contain a degree of statistical sophistication that is unnecessary for engineering survey control densification (such as second-order or less). The distinction between geodetic surveying and engineering surveying must be fully considered when performing GPS-S adjustments and analyzing the results.

e. Connections and adjustments to existing control networks, such as the National Geodetic Reference System (NGRS), must not become independent projects. It is far more important to establish dense and accurate local-project control than to consume resources tying into first-order NGRS points that are miles from the project. Engineering, artillery, construction, and property/boundary referencing requires consistent local control with high relative accuracies. Accurate connections and references to distant geodetic datums are of secondary importance (the exception being projects in support of military aviation operations). The advent of GPS-S technology has provided a cost-effective means of tying previously poorly connected projects to the NGRS and simultaneously transforming the projects to the newly defined NAD 83. When performing (adjusting) these connections, do not distort or warp long-established project reference points.

5-48. Survey Accuracy. The accuracy of a survey (whether performed using conventional or GPS methods) is a measure of the difference between observed and true values (such as coordinates, distance, or angle). Since the true values are rarely known, only estimates of survey accuracy can be made. These estimates may be based on internal observation closures (such as on a loop traverse) or connections with previously surveyed points assumed to have some degree of reliability.

a. A loop traverse originating and ending from a single point will have a misclosure when observations (such as EDM traverse angles or distances or GPS baseline vectors) are computed forward around the loop and back to the starting point. The forward-computed misclosure provides an estimate of the relative or internal accuracy of the observations in the traverse loop, or more directly, the internal precision of the survey. This is perhaps the simplest method of evaluating the adequacy of a survey. These point misclosures (usually expressed as ratios) are not the same as relative distance accuracy measures.

b. The coordinates (and reference orientation) of the single, fixed starting point will also have some degree of accuracy relative to the network in which it is located. This external accuracy (or inaccuracy) is carried forward in the traverse loop or network; however, any such external variance (if small) is generally not critical to engineering and construction projects. When a survey is conducted relative to two or more points on an existing reference network, misclosures with these fixed control points provide an estimate of the absolute accuracy of the survey. This analysis is usually obtained from a final adjustment (usually a fully constrained least squares minimization technique) or by another recognized traverse adjustment method.

5-49. Internal Versus External Accuracy. Geodetic surveying is largely concerned with absolute accuracy or the best fit of intermediate surveys between points on a national network, such as the NGRS. In engineering and construction surveying and, to a major extent, in relative or local boundary surveying, accuracies are more critical to the project at hand. Thus, the absolute NAD-27 or NAD-83 coordinates (in latitude and longitude) relative to the NGRS datum reference are of less importance; however, accurate relative coordinates over a given project reach are critical to design and construction.

a. For example, when establishing basic mapping and construction layout control for a military installation, developing a dense and accurate internal relative-control network is far more important than the values of these coordinates relative to the NGRS. Surveys performed with GPS-S and the final adjustment thereof, should be configured or designed to establish accurate relative (local) project control. This is of secondary importance in connection with NGRS networks.

b. Although reference connections with the NGRS are desirable and recommended and should be made where feasible and practicable, it is critical that such connections (and subsequent adjustments) do not distort the internal accuracy of intermediate points from which design, construction, or project boundaries are referenced. Connections and adjustments to distant networks (such as NGRS) can result in mixed datums within a project area, especially if not all existing project control has been tied in. This can lead to errors and contract disputes during both design and construction. On existing projects with long-established reference control, connections and adjustments to outside reference datums or networks should be performed using caution. The impacts on legal property and project-alignment definitions must also be considered prior to such connections.

c. On newly authorized projects, or on projects where existing project control has been largely destroyed, reconnection with the NGRS is highly recommended.

This will ensure that future work will be supported by a reliable and consistent basic network, while minimizing errors associated with mixed datums.

5-50. Internal and External Adjustments. GPS-Ss are usually adjusted and analyzed relative to their internal consistency and external fit with existing control. The internal consistency adjustment (such as free or minimally constrained) is important from a mission compliance standpoint. The final (or constrained) adjustment fits the GPS-S to the existing network. This is not always easily accomplished since existing networks often have lower relative accuracies than the GPS observations being fit. Evaluation of a survey's adequacy should not be based solely on the results of a constrained adjustment

a. Internal Adjustment. An internal adjustment (also referred to as a free adjustment) is made to determine how well the baseline observations fit or internally close. Other EDM distances or angles may also be included in the adjustment. This adjustment provides a measure of the internal precision of the survey.

(1) In a simplified example, a conventional EDM traverse that is looped back to the starting point will misclose in both azimuth and position. Classical approximate-adjustment methods will typically assess the azimuth misclosure, proportionately adjust the azimuth misclosure (usually evenly per station), recompute the traverse with the adjusted azimuths, and obtain a position misclosure. This position misclosure (in X and Y) is then distributed among all the points on the traverse using various weighting methods (such as distance, latitude, or departure). Final adjusted azimuths and distances are then computed from grid inverses between the adjusted points. The adequacy and accuracy of such a traverse is evaluated based on the azimuth misclosure and the position misclosure after an azimuth adjustment (usually expressed as a ratio to the overall length of the traverse).

(2) A least squares adjustment of the same conventional loop traverse will end up adjusting the points similarly to the approximate methods traditionally employed. The only difference is that a least squares adjustment simultaneously adjusts both the observed angles (or directions) and the distance measurements. A least squares adjustment also allows for variable weighting to be set for individual angle or distance observations, which is a somewhat more complex process when approximate adjustments are performed. Additionally, a least squares adjustment will yield more definitive statistical results of the internal accuracies of each observation or point, rather than just the final closure. This includes estimates of the accuracies of individual station coordinates, relative azimuths, and relative distances.

(3) A series of GPS baselines forming a loop off a single point can be adjusted and assessed similarly to a conventional EDM traverse loop described above. The baseline vector components may be computed (accumulated) around the loop with a resultant 3D misclosure at the starting point. These misclosures (in X, Y, and Z) may be adjusted using either the approximate or least squares methods. The

method by which the misclosure is distributed among the intermediate points in the traverse is a function of the adjustment weighting technique.

(4) In the case of a simple EDM traverse adjustment, the observed distances (or position corrections) are weighted as a function of the segment length and the overall traverse length (compass rule) or to the overall sum of the latitudes or departures (transit rule). Two-dimensional EDM distance observations are not dependent on their direction (a distance's X and Y components are uncorrelated). GPS baseline vector components (in X, Y, and Z) are correlated due to the geometry of the satellite solution. Since satellite geometry is continuously changing, remeasured baselines will have different correlations between the vector components. Such data are passed down from the baseline reduction software for use in the adjustment.

(5) The magnitude of the misclosure of the GPS baseline vectors at the initial point provides an estimate of the internal precision or geometric consistency of the loop (survey). When this misclosure is divided by the overall length of the baselines, an internal relative-accuracy estimate results. This misclosure ratio should not be less than the relative-distance accuracy classification intended for the survey. For example, if the position misclosure of a GPS loop is 0.08 meter and the length of the loop is 8,000 meters, then the loop closure is 0.08 divided by 8,000, which equals 1:100,000.

(6) When an adjustment is performed, the individual corrections or adjustments made to each baseline (residual errors) provide an accuracy assessment for each baseline segment. A least squares adjustment can also provide relative-distance accuracy estimates for each line, based on standard error propagation between adjusted points. This relative-distance accuracy estimate is most critical in engineering and construction work and represents the primary basis for assessing the acceptability of a survey.

b. External Adjustment. An external (or fully constrained) adjustment is the process used to best fit the survey observations to the established reference system. The internal adjustment provides adjusted positions relative to a single, often arbitrary, fixed point. Most conventional surveys or GPS-Ss are connected between existing stations on some predefined reference network or datum. These fixed stations may be existing project-control points (on NAD 27) or stations on the NGRS (NAD 83). In locales outside the continental United States (OCONUS), other local or regional reference systems may be used.

(1) A simple, conventional EDM traverse between two fixed stations best illustrates the process by which comparable GPS baseline vectors are adjusted. Unlike a loop traverse, the azimuth and position misclosures are not wholly dependent on the internal errors in the traverse--the fixed points and their azimuth references are not absolute but contain relative inaccuracies with respect to one another.

(2) A GPS-S between the same two fixed points also contains a 3D position misclosure. Due to positional uncertainties in the two fixed points, this misclosure

may (and usually does) far exceed the internal accuracy of the raw GPS observations. As with a conventional EDM traverse, the 3D misclosures may be about adjusted by proportionately distributing them over the intermediate points. A least squares adjustment also accomplishes the same thing. For example, if the GPS-S is looped back to the initial point, the free-adjustment misclosure at the initial point may be compared with the apparent position misclosure with the other fixed point. A free-adjustment loop misclosure is 1:100,000, whereas the misclosure relative to the two network control points is only 1:5,000. Thus, the internal relative accuracy of the GPS-S is about 1:100,000 (based on the misclosure). If the GPS-baseline observations are constrained to fit the existing control, the 0.6-meter external misclosure must be distributed among the individual baselines to force a fit between the two end points.

If the intent of the above example was to establish 1:20,000 relative-accuracy control, connecting between these two points will not provide that accuracy given the amount of adjustment that must be applied to force a fit. For example, if one of the individual baseline vectors was measured at 600 meters and the constrained adjustment applied a 0.09-meter correction in this sector, the relative accuracy of this segment would be roughly 1:6,666. This distortion is not acceptable for subsequent design/construction work.

(3) Most GPS-S networks are more complex than a simple traverse. They may consist of multiple loops and may connect with any number of control points on the existing network. Additionally, conventional EDM, angles, and differential-leveling measurements may be included with the GPS baselines, resulting in a complex network with many adjustment conditions.

5-51. Partially Constrained Adjustments. In the previous example of a simple GPS traverse, holding the two network points rigidly caused an adverse degradation in the GPS-S because of the differences between the free (loop) adjustment and the fully constrained adjustment. Another alternative is to perform a partially constrained adjustment of the net. In a partially constrained adjustment, the two network points are not rigidly fixed but are only partially fixed in position. Partially constrained adjustments are not practicable using approximate adjustment techniques.

a. For example, if the relative-distance accuracy between the two fixed points is about 1:10,000, it can be equated to a positional uncertainty between these points. Depending on the type and capabilities of the least squares adjustment software, the higher-accuracy GPS baseline observations can be best fit between the two end points. The end points of the GPS network are not rigidly constrained to the original and two control points but will end up falling near them.

b. Adjustment software allows relative weighting of the fixed points to provide a partially constrained adjustment. Any number of fixed points can be connected, and these points may be given partial constraints in the adjustment.

c. Performing partially constrained adjustments takes advantage of the inherent higher-accuracy GPS data relative to the existing network control. Less warping of the GPS data (due to poor existing networks) will occur.

d. A partial constraint also lessens the need for performing numerous trial-and-error constrained adjustments in attempts to locate poor external control points causing high residuals. Fewer ties to the existing network are needed if the purpose of such ties is to find a best fit on a fully constrained adjustment.

e. When connections are made to the NAD 83, relative accuracy estimates of NGRS stations can be obtained from the NGS. Depending on the type of adjustment software, these partial constraints may be in the form of variance-covariance matrixes, error ellipses, or circular-accuracy estimates.

5-52. Rigorous Least Squares Adjustment. Adjustment of GPS networks on PCs is typically a trail-and-error process for both the free and the constrained adjustments. When a least squares adjustment is performed on a network of GPS observations, the adjustment software will provide 2D or 3D coordinate accuracy estimates, variance-covariance matrix data for the adjusted coordinates, and related error ellipse data. Most software programs provide relative accuracy estimates (length and azimuth) between points. Analyzing these various statistics is not easy, and they are also easily misinterpreted. Arbitrary rejection and readjustment to obtain a best fit should be avoided. The original data-reject criteria must be established and justified in a final report document.

a. When a series of loops are formed relative to a fixed point or off another loop, redundant conditions are formed. This is comparable to loops formed in conventional differential level nets. These different loops allow forward baseline-vector position computations to be made over different paths. From the different routes (loops) formed, different positioning closures at a single, fixed point result. These variances in position misclosures from the different routes provide additional data for assessing the internal consistency of the network, in addition to checking for blunders in the individual baselines. The number of different paths or conditions is partially related to the number of degrees of freedom in the network.

b. Multiple baseline observations provide additional redundancy or strength to a line or network since they are observed at two distinct times of varying satellite geometry and conditions. The amount of redundancy required is a function of the accuracy requirements of a particular survey.

c. Performing a free adjustment on a complex network containing many redundancies is best performed using the least squares methods. Adjustment methods are difficult to evaluate when complex interweaving networks are involved. Least squares adjustment software will output various statistics from the free adjustment to assist in detecting blunders and residual outliers in the free adjustment. Most commercial packages display the normalized residual for each observation (for example, GPS, EDM, angle, or elevation), which is useful in detecting and rejecting residual outliers. The variance of unit weight is also important in evaluating the overall adequacy of the observed network. Other statistics (such as chi-square, confidence levels, or histograms) are usually not significant for lower-order engineering projects, and become totally insignificant if the user is not well versed in statistics and adjustment theory. Use of these statistics to reject data (or report the results of an adjustment) without understanding their derivation and source within the network adjustment is not advised.

d. Relative-distance and positional accuracy estimates resulting from the free adjustment of a GPS network are usually excellent in comparison to conventional surveying methods. Loop misclosures and relative-distance accuracies between points commonly exceed 1:100,000.

e. Relative distance accuracy estimates between points in a network are determined by error propagation in the positional standard errors at each end of the tie. Relative-accuracy estimates may be derived for resultant distances or azimuths between the points. The relative-distance accuracy estimates are those typically used to assess the free and constrained accuracy classifications, expressed as a ratio (such as 1:80,000). Since each point in the network has particular position variances, the relative-distance accuracy propagated between any two points will also vary throughout the network.

f. The minimum value (or the largest ratio) will govern the relative accuracy of the overall project. This minimum value (from a free adjustment) is compared with the intended relative-accuracy classification of the project to evaluate compliance. However, relative-distance accuracy estimates should not be rigidly evaluated over short lines (less than 500 meters). Depending on the size and complexity of the project, large variances in the propagated relative-distance accuracies can result. When a constrained adjustment is performed, the adequacy of the external fixed stations will have a major impact on the resultant, propagated distance-accuracies, especially when connections are made to weak control systems. Properly weighted, partially constrained adjustments will usually improve the propagated distance accuracies.

g. The primary criteria for assessing the adequacy of a particular GPS-S is based on the relative-distance accuracy results from a minimally constrained free adjustment, not a fully constrained adjustment. This is due to the difficulty in assessing the adequacy of the surrounding network. If the propagated relative accuracies fall below the specified level, reobservation is warranted.

h. Most adjustment software will output the residual corrections to each observed baseline vector component. These residuals indicate the amount by which each segment was corrected in the adjustment. A least squares adjustment minimizes the sum of the squares of these baseline residual corrections. Commercial least squares adjustment software packages are available. These software packages adjust GPS networks using standard PCs. Sample adjustment statistics summaries from the software package used by Army topographic surveyors are shown in Figure 5-8, page 5-50.

```
              ADJUSTMENT STATISTICS SUMMARY
              NETWORK = Belvoir
              TIME = Mon Oct 20 20:20:26  2000


                          ADJUSTMENT SUMMARY


                Network Reference Factor = 0.88
                Chi-Square Test (α = 95%) = PASS
                      Degrees of Freedom = 105.00


                     GPS OBSERVATIONS
                         Reference Factor = 0.88
                                        r = 105.00

GPS Solution    1 Reference Factor =   1.11    r =  2.83
GPS Solution    2 Reference Factor =   0.37    r =  1.90
GPS Solution    3 Reference Factor =   1.41    r =  [illegible]
GPS Solution    4 Reference Factor =   1.58    [illegible]
GPS Solution    5 Reference Factor =   [illegible]  1.87
GPS Solution    6 Reference Fact[illegible]  =  2.92
GPS Solution    7 Reference Fact[illegible]  =  1.88
GPS Solution    8 Reference Factor     0.55    r =  2.42


GPS Solution   48 Reference Factor =   0.71    r =  2.97

                          WEIGHTING STRATEGIES:

     GPS OBSERVATIONS:
          Scalar Weighting Strategy:
User-Defined Scalar Set  Applied Globally =  11.50

          No summation weighting strategy was used

          Station Error Strategy:
HI  error = 0.0020
--------------------------------------------------------------------------------
                Esc=EXIT        ↑↓=SCROLL        PgUp        PgDn
              COORDINATE ADJUSTMENT SUMMARY
              NETWORK = Belvoir
              TIME = Mon Oct 20 20:20:26  2000


Datum = WGS-84
Coordinate System = Geographic
Zone = Global

Network Adjustment Constraints:
  Inner constraints in y
  Inner constraints in x
  Inner constraints in H


POINT   NAME       OLD COORDS        ADJUST       NEW COORDS          1.96σ

   1 DTP4
        LAT=  38° 41' 23.839155" +0.000007"  38° 41' 23.839163"    0.002593m
        LON=  77° 08' 03.890845" +0.000017"  77° 08' 03.890828"    0.002305m
     ELL HT=            6.7551m   +0.0062m              6.7613m    0.005981m
   ORTHO HT=            0.0000m   +0.0000m              0.0000m    NOT KNOWN

   2 FB09
        LAT=  38° 41' 42.126953" -0.000003"  38° 41' 42.126950"    0.002604m
        LON=  77° 08' 42.414329" -0.000003"  77° 08' 42.414331"    0.002182m
     ELL HT=            5.1959m   +0.0008m              5.1967m    0.005981m
   ORTHO HT=            0.0000m   +0.0000m              0.0000m    NOT KNOWN
```


Figure 5-8. Example of an Adjustment Statistics Summary

i. Relative GPS baseline standard errors can be obtained from the baseline reduction output and, in some software programs, can be directly input into the adjustment. These standard errors, along with their correlations, are given for each vector component (X, Y, and Z). They are converted to relative weights in the adjustment. The following input weighting is commonly used:

- Fixed. ± 3 millimeters (latitude) ± 5 millimeters (longitude) + 1 ppm ± 10 millimeters (height) + 1 ppm.

- Float. ± 6 millimeters (latitude) ± 10 millimeters (longitude) + 2 ppm ± 10 millimeters (height) + 2 ppm.

These optimum standard errors have been found to be reasonable in standard work where extremely long baselines are not involved. Use of these optimum values is recommended for the first adjustment iteration.

j. Error ellipses or 3D error ellipsoids generated from the adjustment variance-covariance matrixes for each adjusted point are also useful in depicting the relative positional accuracy. The scale of the ellipse may be varied as a function of the 2D deviation. A 2.45 sigma (or 95 percent) probability ellipse is usually selected for output. The size of the error ellipse's relative distance or the azimuth accuracy estimate between two adjacent points is a direct function of the size of these positional ellipses.

5-53. Evaluation of Adjustment Results. Surveys should be classified based on their horizontal-point closure ratio or vertical-elevation difference standard (Table 5-5, page 5-51).

a. The horizontal-point closure ratio is determined by dividing the linear-distance misclosure of the survey into the overall circuit length of the traverse, loop, or network line or circuit. When independent directions or angles are observed (such as a conventional survey [traverse or triangulation]), these angular misclosures may be distributed before assessing positional misclosure. In cases where GPS vectors are measured in geocentric coordinates, the 3D positional misclosure is assessed.

b. The vertical accuracy of a survey is determined by the elevation misclosure within a level section or level loop. For conventional-differential or trigonometric leveling, section or loop misclosures (in millimeters) should not exceed the limits shown in Table 5-5, page 5-52, where the line or circuit length is measured in kilometers. Fourth-order accuracies are intended for construction grading work. Procedural specifications or restrictions pertaining to vertical-control surveying methods or equipment should not be over restrictive.

Table 5-5. Point-Closure Standards for Horizontal- and Vertical-Control Surveys

Horizontal	
Classification	**Point-Closure Standard (Ratio)**
Second order, Class I	1:50,000
Second order, Class II	1:20,000
Third order, Class I	1:10,000
Third order, Class II	1:5,000
Fourth order (construction layout)	1:2,500 - 1:20,000
Vertical	
Classification	**Point-Closure Standard (mm)**
Second order, Class I	$6\sqrt{\text{distance in km}}$
Second order, Class II	$8\sqrt{\text{distance in km}}$
Third order	$12\sqrt{\text{distance in km}}$
Fourth order (construction layout)	$24\sqrt{\text{distance in km}}$

5-54. Final-Adjustment Reports. A variety of free- or constrained adjustment combinations may be specified for a GPS-S. Specific stations may be held fixed or a contractor may be instructed to determine the optimum adjustment, including the appropriate weighting for constrained points. When fixed stations are to be partially constrained, appropriate statistical information must be provided. Either variance-covariance matrixes or relative positional accuracy estimates may be converted as approximate variance-covariance matrixes in the constrained adjustment. All rejected observations should be clearly indicated, along with the criteria and the reason used for the rejection.

a. When different combinations of constrained adjustments are performed due to indications of one or more fixed stations causing undue biasing of the data, an analysis should be made as to a recommended solution that provides the best fit for the network. Any fixed control points requiring readjustment should be clearly indicated in the final recommendation.

b. The final, adjusted horizontal- or vertical-coordinate values are assigned an accuracy classification based on the statistical results. This classification should include both the resultant geodetic or Cartesian coordinates and the baseline-differential results. The final, adjusted coordinates shall state the 95 percent confidence region of each point and the accuracy in the part(s) per million between all points in the network. The datum should be clearly identified for all coordinate listings.

c. Final-report coordinate listings may be required on hard copy as well as on a specified computer media. A scaled plot should be submitted with the adjustment report showing the proper locations and designations of all established stations.

LESSON 5

PRACTICE EXERCISE

The following items will test your grasp of the material covered in this lesson. There is only one correct answer for each item. When you have completed the exercise, check your answer with the answer key that follows. If you answer any item incorrectly, study again that part which contains the portion involved.

1. In a NAVSTAR GPS, distances are measured between the receiver antenna and the satellites, and the position is determined from the intersections of the ____________________________________.

   A. Guideline
   B. Redundant
   C. Line Distance
   D. Range vector

2. NAVSTAR GPS consist of three distinct segments, which factor is not included?

   A. Time segment
   B. Space segment
   C. Control segment
   D. User segment

3. The L1 signal is modulated under what code(s)?

   A. P-code
   B. Y-code
   C. C/A-code and Y-code
   D. P-code and C/A-code

4. The PPS user can use a decryption device to achieve a 3D accuracy in the range of ____ with a single-frequency receiver.

   A. 10 to 16 meters
   B. 8 to 12 meters
   C. 5 to 10 meters
   D. 3 to 5 meters

5. Simultaneous range observations to numerous satellites can be adjusted using ____.

   A. Different field procedures
   B. GPS-related software
   C. Weighting techniques
   D. Different time frames

6. Given the changing satellite geometry and other factors, GPS accuracy is dependent on ____.

A. Coordinates' order
B. Time or location
C. Equipment used
D. Computer software

7. How many receivers are required for differential positioning?

A. Four
B. Three
C. Two
D. One

8. What is the less acceptable GPS technique for establishing baselines?

A. Pseudokinematic
B. RTK/OTF
C. Kinematic
D. Rapid Static

9. Geodetic leveling gives rise to a height called __________________, often known as the height above MSL.

A. Datum height
B. Geoid height
C. Orthometric height
D. Ellipsoid height

10. The first step in planning a control survey is to ____.

A. Gather the equipment to be utilized
B. Establish a design that will fit the situation
C. Obtain the visibility diagrams of each station
D. Determine the ultimate accuracy requirements

11. GPS surveying requires that all stations have an unobstructed view of 15° above the horizon and satellites below __________ should not be observed.

A. 5°
B. 10°
C. 12°
D. 15°

12. When the networking method is selected, the surveyor should devise a survey network that is ____.

A. Algebraically stabled
B. Geometrically sound
C. Open
D. Closed

13. HI measurements should be determined to the nearest millimeter in metric units and the nearest ______________________________________ in US units.

A. 1.0 foot
B. 0.1 foot
C. 0.01 foot
D. 0.001 foot

14. The main advantage of a stop-and-go kinematic survey over a static survey is the ____.

A. Reduced occupation time required
B. Accuracy
C. Fewer number of satellites required
D. Fewer number of personnel required

15. Under ideal test conditions, kinematic GPS can achieve what kind of accuracy level over distances up to 30 kilometers?

A. Millimeter-level
B. Centimeter-level
C. Decimeter-level
D. Meter-level

16. How long is a rover receiver required to stay at each unknown station to achieve rapid static?

A. 30 to 45 minutes
B. 10 to 25 minutes
C. 5 to 20 minutes
D. 1 to 10 minutes

17. Position misclosures (in X and Y) are distributes among the points on the traverse using ____.

A. Various weighting methods
B. Various criteria
C. Algebraic formulas
D. Common sense

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LESSON 5

PRACTICE EXERCISE

ANSWER KEY AND FEEDBACK

Item		Correct Answer and Feedback
1.	D	Range vector Distances are measured between...(page 5-1, Introduction)
2.	A	Time segment The NAVSTAR GPS consists of...(page 5-2, para 5-4)
3.	D	P-code and C/A-code The L1 signal is modulated...(page 5-3, para 5-5)
4.	A	10 to 16 meters The PPS user...(page 5-5, para 5-11)
5.	C	Weighting techniques Additionally, simultaneous range...(page 5-5, para 5-14)
6.	B	Time or location Given the changing satellite...(page 5-7, para 5-16g)
7.	C	Two Differential positioning requires...(page 5-10, para 5-19)
8.	A	Pseudokinematic Pseudokinematic surveying is the...(page 5-13, para 5-23e)
9.	C	Orthometric height Geodetic leveling gives rise...(page 5-14, para 5-26)
10.	D	Determine the ultimate accuracy requirements The first step in planning a control...(page 5-15, para 5-30)
11.	B	10° Satellites below 10° should not...(page 5-19, para 5-31e)
12.	B	Geometrically sound When the networking method is...(page 5-20, para 5-32c)
13.	C	0.01 foot Determine the measurement to the...(page 5-25, para 5-33c)

14.	A	Reduced occupation time required The main advantage of this method...(page 5-29, para 5-36b)
15.	B	Centimeter-level Test results from an...(page 5-31, para 5-36c[3])
16.	C	5 to 20 minutes A rover receiver occupies...(page 5-32, para 5-36e[1])
17.	A	Various weighting methods This position misclosure...(page 5-44, para 5-50a[1])

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LESSON 6

AIRFIELD OBSTRUCTION AND NAVIGATIONAL-AID SURVEYS

OVERVIEW

LESSON DESCRIPTION:

In this lesson, you will learn the terminology and requirements for airfield obstruction and NAVAID engineering surveys. Due to vast differences in airfield instrumentation, customer requirements, and FAA regulations, the content is general in nature.

TERMINAL LEARNING OBJECTIVE:

ACTION: You will become familiar with the necessary terminology and requirements for the Army surveyor to perform airfield obstruction and NAVAID engineering surveys.

CONDITION: You will be given the material contained in this lesson, a number 2 pencil, and a calculator.

STANDARD: You will correctly answer all practice questions following this exercise.

REFERENCES: The material contained in this lesson was derived from FM 3-34.331.

INTRODUCTION

AOCs and NAVAID surveys are extensive field or photogrammetric operations that are required by an agreement between the FAA and the USAASA and are specified in AR 95-2. Airfield obstruction and NAVAID surveys embrace surveying operations involved in obtaining accurate and complete NAVAID and associated airport/heliport-obstruction and geodetic-positioning data. A precise geographic position of these navigational facilities is required to support the FAA and a wide range of NAS activities. AOC surveys provide source information on--

- Runways and stopways.

- NAVAIDs.

- FAR-77 obstructions.
- Aircraft-movement aprons.
- Prominent airport buildings.
- Selected roads and other traverse ways.
- Cultural and natural features of landmark value.
- Miscellaneous and special-request items.

AOC surveys also establish or verify geodetic control in the airport vicinity that is accurately connected to the NSRS. This control and the NSRS connection ensure accurate relativity between the points on the airport and other surveyed points in the NAS, including GPS navigational satellites. AOC data is used to--

- Develop instrument-approach and -departure procedures.
- Determine the maximum takeoff weights.
- Certify airports for certain types of operations.
- Update official aeronautical publications.
- Provide geodetic control for engineering projects related to runway/taxiway construction, NAVAID positioning, obstruction clearing, and other airport improvements.
- Assist in airport planning and land-use studies.
- Support activities such as aircraft accident investigations and special-purpose projects.

PART A - FEDERAL AVIATION ADMINISTRATION AND FEDERAL AVIATION REGULATION STANDARDS

6.1 Requirements. FAA Publication 405 (FAA 405) and FAR-77 outline the required standards for AOC surveys. Various areas, surfaces, reference points, dimensions, and specification requirements are used in airfield surveys.

a. Runways. All length and width measurements should be determined to the nearest foot. If the runway threshold is displaced, give the distance from the beginning of the runway surface. Determine the coordinates (latitude and longitude) of the runway threshold and stop end at the runway centerline. Elevations at the runway threshold, the stop end, and the highest elevation (within the first 3,000 feet of each runway touchdown zone elevation [TDZE]) should be determined to the nearest 0.1 foot from the MSL. Additionally, runway profiles should be prepared that show elevations listed above the runway's high and low points, grade changes, and gradients. The elevation of a point on the instrumented runway centerline nearest to the instrument landing system (ILS) and the glide-path transmitter are determined to the nearest 0.1 foot from the MSL.

b. Navigational Aids. Airports requiring airfield obstruction and NAVAID surveys are instrumented runways. The exact point on the radar, the reflectors, the runway intercepts, or the ILS and microwave landing system (MLS) components, depends on the type, location, and required accuracy. The requirement to verify existing systems, their proper description, and all components on or near the runway is mandatory. With help from airfield operations, maintenance sections, and control-tower personnel, all information may be obtained for locating and describing all airfield features.

(1) NAVAIDs located on airports include the--

- ILS.
- MLS.
- Precision approach radar (PAR).
- Airport surveillance radar (ASR).

(2) NAVAIDs not located on airports include--

- Tactical air navigation (TACAN).
- Very-high-frequency omnidirectional range (VOR).
- Nondirectional beacon (NDB) radios.
- Very-high-frequency omnidirectional range and tactical air navigation (VORTAC).

6-2. Obstructions. An obstruction is an object or feature protruding through or above any navigational imaginary surface that poses a threat to the safe operation of aircraft. Navigational imaginary surfaces or obstruction identification surfaces (OISs) are defined in FAR-77. Figures 6-1 through 6-3, pages 6-4 through 6-6, show some definitions and samples.

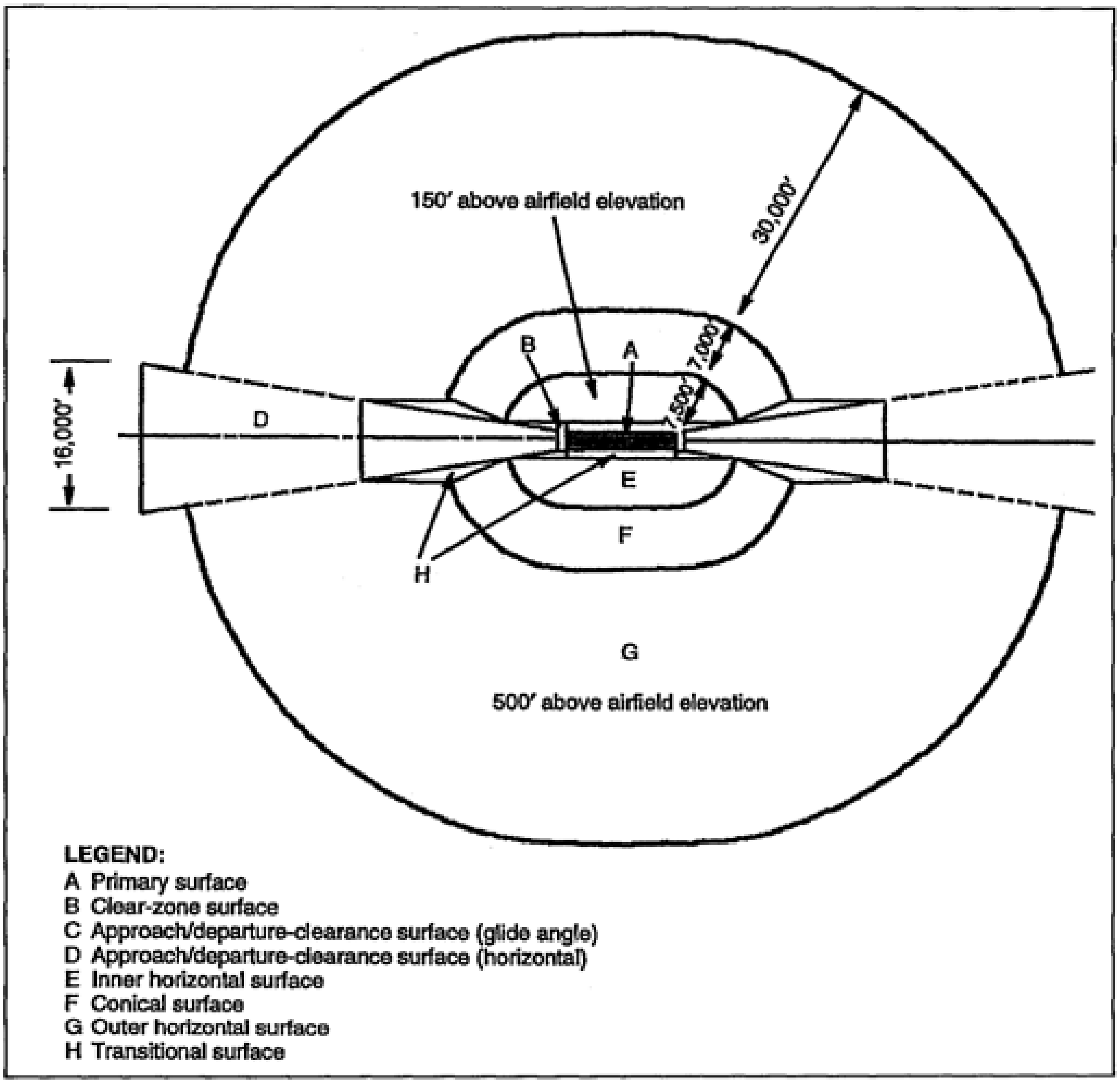


Figure 6-1. Imaginary Surfaces

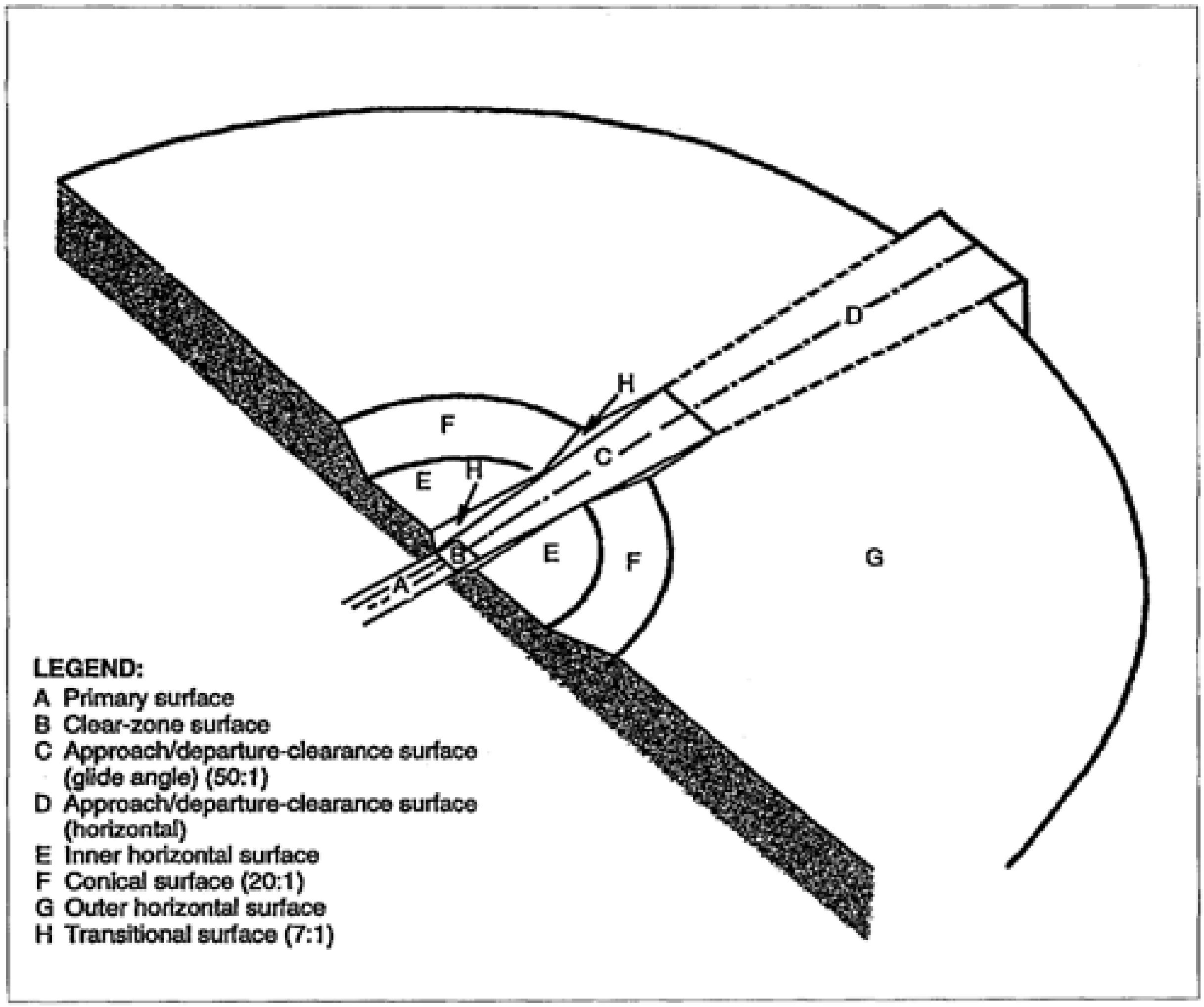


Figure 6-2. OIS Dimensions

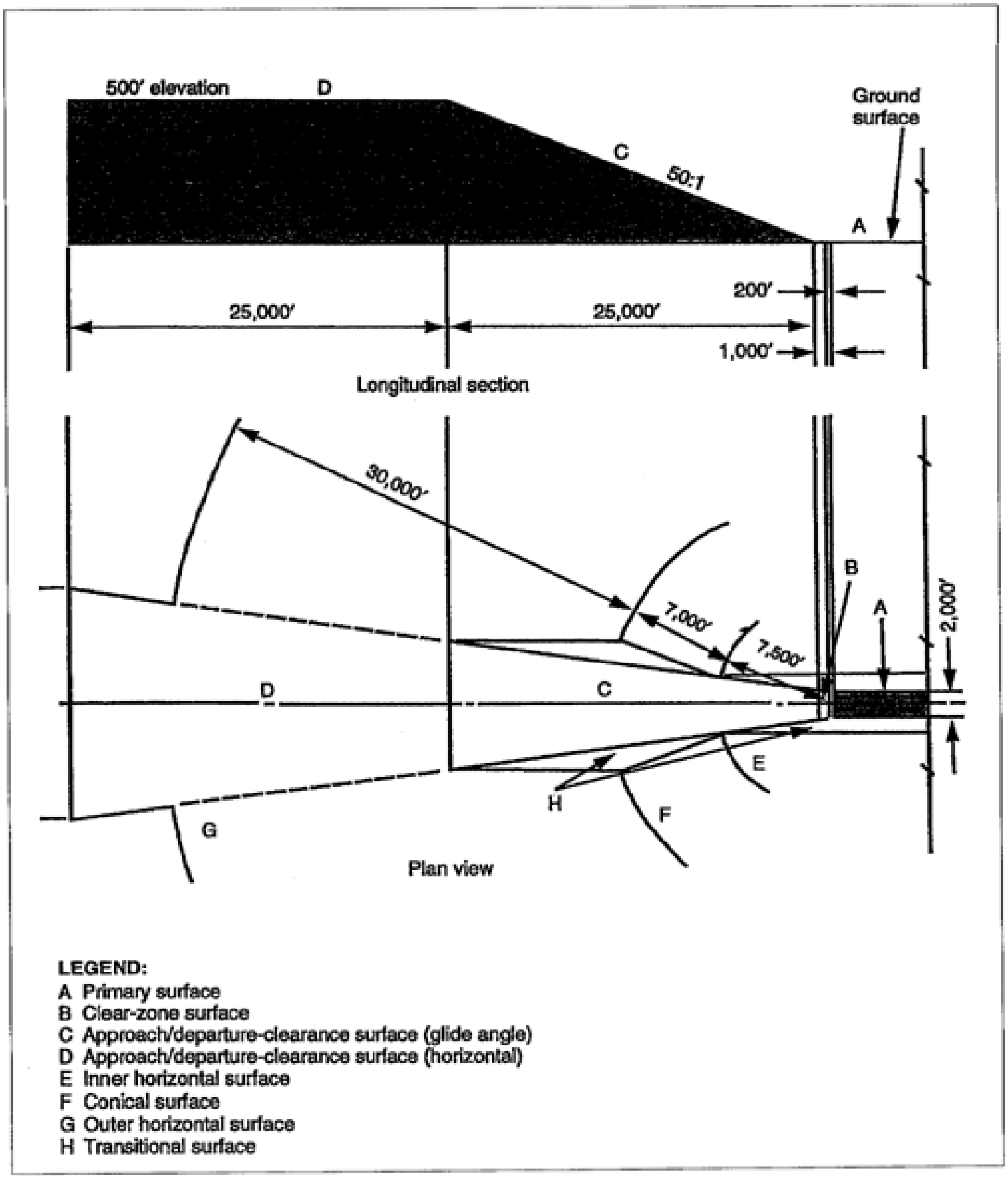


Figure 6-3. Approach Surfaces

a. Federal Aviation Regulation, Part-77, Section 77.28, Military Airport Imaginary Surfaces. These surfaces apply to all military airports. For the purpose of this section, a military airport is any airport operated by an armed force of the US.

(1) Related to Airport Reference Points (ARPs).

- An inner horizontal surface is an oval plane that is at a height 150 feet above the established airfield elevation. The plane is constructed by scribing an arc with a radius of 7,500 feet at the centerline at the end of each runway. The arcs are then interconnected with tangents.

- A conical surface is a surface extending from the periphery of the inner horizontal surface outward and upward at a slope of 20:1. The horizontal distance is 7,000 feet, with a height of 500 feet above the established airfield elevation.

- An outer horizontal surface is a plane, 500 feet above the established airfield elevation, extending outward from the outer periphery of the conical surface for a horizontal distance of 30,000 feet.

(2) Related to Runways.

- A primary surface may be located on the ground or the water. The primary surface is longitudinally centered on each runway and is the same length as the runway. The width of the primary surface for runways is 2,000 feet; however, at established bases where substantial construction has taken place according to previous lateral-clearance criteria, the 2,000-foot width may be reduced to the former criteria.

- A clear-zone surface may be located on the ground or on water at each end of the primary surface. The length is 1,000 feet, and the width is the same as the primary surface.

- An approach/departure-clearance surface is an inclined plane, symmetrical at the extended runway centerline, beginning 200 feet beyond each end of the primary surface at the centerline elevation of the runway end and extending for 50,000 feet. The slope of the approach-clearance surface is 50:1 along the runway-centerline and extended until it reaches an elevation of 500 feet above the established airport elevation. It then continues horizontally at this elevation to a point 50,000 feet from the point of beginning. The width of this surface at the runway end is the same as the primary surface, then it flares uniformly and the width at 50,000 feet is 16,000 feet.

- A transitional surface connects the primary surfaces, the first 200 feet of the clear-zone surfaces, and the approach/departure-clearance surfaces to the inner horizontal surface, the conical surface, the outer horizontal surface, or other transitional surfaces. The slope of the transitional surface is 7:1 outward and upward at right angles to the runway centerline.

b. Federal Aviation Regulation-77, Section 77.29, Airport Imaginary Surfaces for Heliports. These surfaces apply to all military heliports. For the purpose of this section, a military heliport is any heliport operated by an armed force of the US.

- A heliport primary surface is an area of the primary surface which coincides in size and shape with the designated takeoff and landing area of a heliport. This surface is a horizontal plane at the elevation of the established heliport elevation.

- A heliport approach surface begins at each end of the heliport primary surface, is the same width as the primary surface, and extends outward and upward for a horizontal distance of 4,000 feet where its width is 500 feet. The slope of the approach surface is 8:1 for civil heliports and 10:1 for military heliports.

- A heliport transitional surface extends outward and upward from the lateral boundaries of the heliport primary surface and from the approach surfaces at a slope of 2:1 for a distance of 250 feet measured horizontally from the centerline of the primary and approach surfaces.

c. Federal Aviation Regulation-77, Section 77.5, Kinds of Objects Affected. This section further defines an obstruction and applies to--

- Any object of natural growth, terrain, or permanent or temporary construction or alteration (including equipment or materials used therein) and apparatus of a permanent or temporary character.

- The alteration of any permanent or temporary existing structure by a change in its height (including appurtenances) or lateral dimensions (including equipment or materials used therein).

6-3. Airport Data. The ARP location (in degrees, minutes, and seconds of longitude and latitude) is determined according to FAA 405. Field elevation is the highest point on any airport landing surface.

PART B - AIRFIELD DATA ACCURACY REQUIREMENTS

6-4. Requirements. All contiguous CONUS, Alaskan, and Caribbean coordinates are determined based on NAD 83 and/or WGS 84. Geodetic accuracy of orthometric heights is referenced to the North American Vertical Datum of 1988 (NAVD 88). The coordinates for the points on the airport require different degrees of accuracy. Tables 6-1 through 6-5, pages 6-9 and 6-10, are examples of different accuracy standards for airfield data. FAA 405 contains the complete requirements.

Table 6-1. Control-Station Accuracy Requirements

Item	Vertical (Values are in Centimeters)			
	Horizontal	Orthometric	Ellipsoidal	Above Ground Level
Primary airport control station (PACS)[1]	5	25.0	15	NA
Secondary airport control station (SACS)[2]	3	5.0	4	NA
Wide-area augmentation system (WAAS)[1]	5	10.0	10	NA
WAAS reference station[3]	1	0.2	2	NA

[1]Accuracies are relative to the nearest NGS-sanctioned continuously operating reference station.
[2]Accuracies are relative to the PACSs and other SACSs at the airport.
[3]Accuracies are relative to the other WAAS *reference station at the site*.

Table 6-2. Electronic-NAVAID Accuracy Requirements

Item	Vertical (Values are in Feet)			
	Horizontal	Orthometric	Ellipsoidal	Above Ground Level
Air route surveillance radar (ARSR)	(1)	100	100	NA
ASR	(1)	10	10	NA
DME:				
Frequency paired with localizer	1	1	1	NA
Frequency paired with MLS azimuth guidance	1	1	1	NA
Frequency paired with NDB	(1)	NA	NA	NA
Frequency paired with VOR	(1)	NA	NA	NA

NOTE: The horizontal accuracy requirement for items coded (1) is 20 feet when located on a public-use airport or military field and 50 feet for all other locations.

Table 6-3. Visual-NAVAID Accuracy Requirements

Item	Vertical (Values are in Feet)			
	Horizontal	Orthometric	Ellipsoidal	Above Ground Level
Airport beacon	(1)	NA	NA	NA
Visual glide slope indicators	20	NA	NA	NA
Runway edge indicator light	20	NA	NA	NA
Approach lights	20	NA	NA	NA

NOTE: The horizontal accuracy requirement for items coded (1) is 20 feet when located on a public-use airport or military field and 50 feet for all other locations.

Table 6-4. Airport Obstruction Accuracy Requirements

Item		Horizontal	Vertical (Values are in Feet)		
			Orthometric	Ellipsoidal	Above Ground Level
Nonman-made objects and man-made objects less than 200 feet above ground level that penetrate the following OISs:	A primary surface	20	3	3	NA
	Those areas of an approach surface within 10,200 feet of the runway end	20	3	3	NA
	Those areas of a primary transitional surface within 500 feet of the primary surface	20	3	3	NA
	Those areas of an approach transitional surface that are both within 500 feet of the approach surface and within 2,766 feet of the runway end	20	3	3	NA

Table 6-5. Airport Runway Accuracy Requirements

Item	Horizontal	Vertical (Values are in Feet)		
		Orthometric	Ellipsoidal	Above Ground Level
Physical end	1	0.25	0.2	NA
Displaced threshold (DT)	1	0.25	0.2	NA
TDZE	NA	0.25	0.2	NA
Supplemental profile points	20	0.25	0.2	NA

a. Horizontal Requirements. Horizontal accuracy requirements can be met through third-order, Class II traverse, GPS, or two-point intersection methods.

b. Vertical requirements. Vertical accuracy requirements dictate a minimum of third-order differential-leveling methods.

PART C - REPORTING

6-5. General. The required reporting for airfield-related surveys is not significantly different from that required for other survey operations. The parent unit will normally require all of the reports listed in FM 3-34.331. In addition to these routine reports, a special report will be required to submit the final data. This report will be completed according to AR 95-1, AR 95-2, FAA 405, and FAR-77. For quick reference, the required documentation is listed below.

a. Airport Obstruction Chart. An AOC is a 1:12,000-scale graphic depicting FAR-77 guidance. An AOC represents objects that penetrate airport imaginary surfaces, aircraft movement and apron areas, NAVAIDs, prominent airport buildings, and a selection of roads and other planimetric details in the airport vicinity. Also included are tabulations of runway and other operational data. AOC data is current as of the date of the field survey. The AOC consists of four sections:

- The airport plan (AP).

- Runway plans and profiles (RPP).

- Tabulated operational data (TOD).

- Notes and legends (NL).

Each section (all contents, portrayal, and general format) should conform to the sheet style (obstruction chart [OC] 000) represented in FAA 405. The AOC is published on E50 chart paper (or equivalent) with border dimensions of 30 x 42 or 30 x 48 inches. The long dimension may be either in the north-south or east-west direction and should have a 34-inch space between the border and the trim line. In cases where the AP and RPP will not fit on the front of the chart, the RPP is printed on the back.

b. Airport Plan. The depiction of the AP depends on the surface type and whether an obstruction survey was accomplished. A detailed explanation of what pertinent information to depict is included in FAA 405, Section 10.1.3. For example, an AP for a specially prepared hard surface (SPHS) runway will include information on the--

- Runway length and width.

- DTs.

- Physical end of the runway.

- Airport elevation.

- TDZE.

- Magnetic bearing.

- Runway numbers.

- Obstructions.

- NAVAIDs.

- Meteorological apparatus.

- ARPs.

c. Runway Plans and Profiles. A detailed explanation of what pertinent information to depict is included in FAA 405, Section 10.1.4. The RPP should include--

- The proper angular orientation.

- A horizontal scale of 1:12,000 and a vertical scale of 1 inch equal to 100 feet.

- An adequate coverage area of the primary and approach surfaces.

- A plan view of the runway as on the AP.

- A profile view of objects carried in the plan view.

- A profile view of objects penetrating the approach surfaces.

- The correct approach surface or precise-instrument-runway (PIR) surface.

- The correct numbering scheme of objects in the profile.

- A north arrow.

NOTE: A PIR has an existing instrument approach procedure using an ILS or PAR. A PIR also refers to a runway for which a precision approach system is planned and is so indicated by an FAA-approved airport layout plan, a military service military airport layout plan, any other FAA planning document, or a military service military airport planning document.

d. Tabulated Operational Data. The TOD should show--

- The airport location point (ALP) listed in degrees and minutes.

- The ARP listed in degrees, minutes, and three-decimal-place seconds.

- A runway data table with runway numbers, appropriate latitude and longitude coordinates, and TDZE elevations.

- The geodetic azimuth from approach end to stop end, reckoned from the north.

- Additional information pertaining to runways with DTs.

e. Notes and Legends. A detailed explanation of what pertinent information to depict is included in FAA 405, Section 10.1.6. The NL should include the--

- Horizontal and vertical data.

- Map projection.

- Airport elevation.

- Legend.

- Graphic horizontal and vertical scales.

f. Forms. In addition to the AOC, each airfield report requires the completion of a--

- DA Form 5821.

- DA Form 5822.

- DA Form 5827.

DA Form 5821 is a tabulation of all the information obtained from the survey. Figure 6-4, page 6-14 shows an example of a completed airfield compilation report. Table 6-6, page 6-15 includes instructions for completing this form.

g. Precision Approach Radar or Ground-Controlled Approach (GCA) Data. Enter this data on DA Form 5822. The completion of this form is self-explanatory. A sample form is show at Figure 6-5. page 6-16.

h. Instrument Landing System Data. Enter this data on DA Form 5827. The completion of this form is self-explanatory. A sample form is shown at Figure 6-6, page 6-17.

AIRFIELD COMPILATION REPORT

For use of this form, see FM 3-34.331; the proponent agency is TRADOC.

SURVEY AGENCY: 30th Engineer Battalion			
AIRPORT NAME: McCoy Army Airfield			IDENTIFIER: CMY
CITY: Fort McCoy	STATE: Wisconsin	EDITION: #1	SURVEY DATE (YYYYMMDD): 2001 07 15
AIRPORT REFERENCE POINT: ARP	LATITUDE: 43°57'33.458"N	LONGITUDE: 90°44'14.641"W	Δ CL OR θ ANGLE: -01°34'15.6"
AIRPORT LOCATION POINT	LATITUDE	LONGITUDE	DECLINATION
AIRPORT ELEVATION (feet): 837.3 MSL	LOCATED: EOR 01	CONTROL TOWER FLOOR ELEVATION (feet): 871.2 MSL	
DATUM: WGS-84		POSITION CODE - 1. Field Survey 2. Photogrammetric 3. Other	

SAMPLE

AIRPORT DATA	ELEVATION	LATITUDE	LONGITUDE	YR-CODE	REMARKS	OFFICE CODE
NDB (CMY)	1,020.1	43°[illegible]16.1"N	90°38'30.3"W	01/01		
Windsock (1)	860.4	43°[illegible]"N	90°43'58.5"W	01/01		
Beacon (13)	896.4	43°57'14.[illegible]	[illegible]05.8"W	01/01		
WDI	845.8	43°57'35.7"N	90°43'[illegible].1"W	01/01		
Tetrahedron	834.7	43°57'36.1"N	90°43'58.[illegible]	01/01		
Control Tower (9)	911.6	43°57'22.5"N	90°44'05.9"W	01/01		
Maltese Cross #1	830.5	43°57'30.8"N	90°43'59.7"W	01/01		
Maltese Cross #2	829.8	43°57'26.4"N	90°44'15.9"W	01/01		
Maltese Cross #3	832.9	43°57'22.8"N	90°43'51.6"W	01/01		

RUNWAY	DSPLCD THR LENGTH	RWY END ELEVATION	LATITUDE	LONGITUDE	WIDTH / LENGTH	GEODETIC AZ. (M) / MAG. BEARING (N)	OFFICE CODE
EOR 29 TDZE	NA	831.8	43°57'27.478"N	90°43'48.699"W	100.00 4,211.00	292°09'26.2" 290°58'26.2"	
EOR 11	NA	822.4	43°57'43.164"N	90°44'42.017"W	75.00 4,211.00	112°09'25.8" 110°58'25.8"	
TDZE 11/29	NA	829.5	43°57'32.027"N	90°44'03.899"W	NA	NA	
EOR 19	NA	824.7	43°57'44.922"N	90°44'08.802"W	90.00 2,962.90	195°25'26.5" 194°14'26.5"	
EOR 01	NA	837.3	43°57'16.715"N	90°44'19.574"W	90.00 2,962.90	15°25'27.3" 14°14'27.3"	
EOR 01 DT	1,326.7	835.7	43°57'04.089"N	90°44'24.408"W	60.00 1,326.70	NA	

DA FORM 5821, JUL 2001 EDITION OF AUG 1989 IS OBSOLETE. USAPA V1.00

Figure 6-4. Sample DA Form 5821 (Airfield Compilation Report)

Table 6-6. Instructions for Completing DA Form 5821

Block Name	Instruction
Survey agency	The agency conducting the field survey.
Airport name	The official airport name as determined by the FAA.
Identifier	The airport location-identifier designator as listed in FAA Publication 7350.5-U.
City	Self-explanatory.
State	Self-explanatory.
Edition	The number of times the airfield has been surveyed by the agency listed in the survey agency block. Identify the original survey as 1; identify subsequent surveys as 2, 3, and so on.
Survey date	The year of declination.
ARP	The physical location of the ARP.
Latitude	The latitude of the ARP.
Longitude	The longitude of the ARP.
Delta azimuth or theta angle	The grid convergence for the ARP.
ALP	The physical location of the ALP.
Latitude	The latitude of the ALP.
Longitude	The longitude of the ALP.
Declination	The magnetic declination of the ARP.
Airport elevation	The airport elevation (in feet)
Located	A short narrative description (include the latitude and the longitude).
Control-tower floor elevation	Self-explanatory.
Datum	Identify the survey datum.
Airport data	Object or airfield feature observed (use additional sheets as required).
Elevation	Self-explanatory.
Latitude	The latitude of the elevation.
Longitude	The longitude of the elevation.
Year code	The year and month surveyed (for example, April 87 is written 8704).
Remarks	Self-explanatory.
Office code	Leave blank (may be used by other offices).
Runway	The numerical designation of the runway.
DT length	Self-explanatory.
Runway-end elevation	Self-explanatory.
Latitude	The latitude of the runway-end elevation.
Longitude	The longitude of the runway-end elevation.
Width/length	The physical width and length of the runway surface.
Geodetic azimuth/magnetic bearing	Self-explanatory.
Office code	Leave blank.

PRECISION APPROACH RADAR (GCA) DATA
For use of this form, see FM 3-34.331; the proponent agency is TRADOC.

AIRPORT NAME McCoy Army Airfield		
CITY Fort McCoy	STATE Wisconsin	SURVEY DATE (YYYYMMDD) 2001 07 15

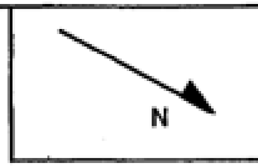
PAR COMPONENTS AND PERTINENT RUNWAY DATA Numbered items correspond to the diagram below.	LATITUDE (1/ 100 Second)	LONGITUDE (1/ 100 Second)	ELEVATION (1/ 10 Foot)
1. PAR Antenna	43°57'14.51"	90°44'05.79"	896.4
2. Touchdown Reflector	43°57'36.10"	90°43'58.40"	834.7
3. The point on runway C/L closest to the Touchdown Reflector (item 2).	43°57'36.33"	90°43'58.40"	831.4
4. Runway C/L End.	43°57'43.16"	90°44'42.02"	829.5
5. Runway C/L End. EOR 11	43°57'27.48"	90°43'48.70"	831.8
6. The point on runway C/L closest to PAR Antenna.			
7. Displaced Threshold (if applicable).			

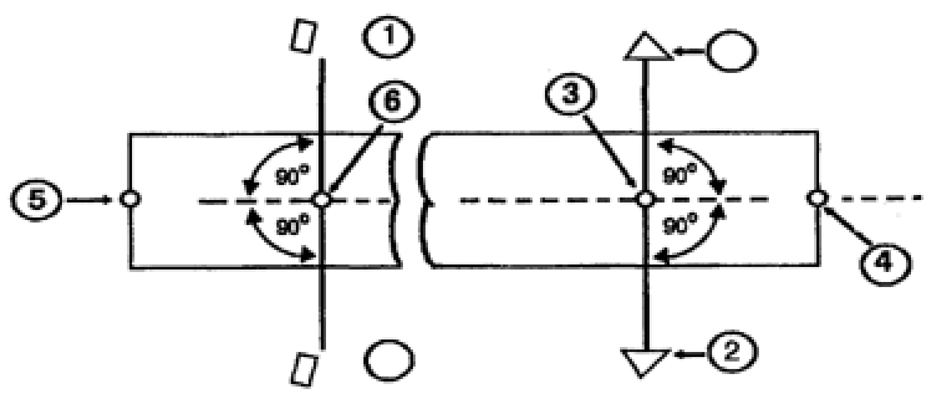
SAMPLE

PAR Antenna - Enter Numeral 1 in circle to indicate PAR Antenna Position.
Touchdown Reflector - Enter Numeral 2 in circle to indicate Touchdown Reflector.

PAR - GROUND DISTANCE			
3 to 7 FEET (If applicable)	1 to 6 FEET	2 to 3 FEET	GEODETIC AZIMUTH SOUTH ° ' "
	3 to 6 FEET	3 to 4 FEET	4 to 5

ADD APPLICABLE NUMBERS TO CIRCLES AND RUNWAY ENDS. SHOW NORTH ARROW.





DA FORM 5822, JUL 2001 EDITION OF AUG 1989 IS OBSOLETE. USAPA V1.00

Figure 6-5. Sample DA Form 5822 (PAR Data)

INSTRUMENT LANDING SYSTEM DATA For use of this form, see FM 3-34.331; the proponent agency is TRADOC.		
AIRPORT NAME McCoy Army Airfield		
CITY Fort McCoy	STATE Wisconsin	SURVEY DATE (YYYYMMDD) 2001 07 15

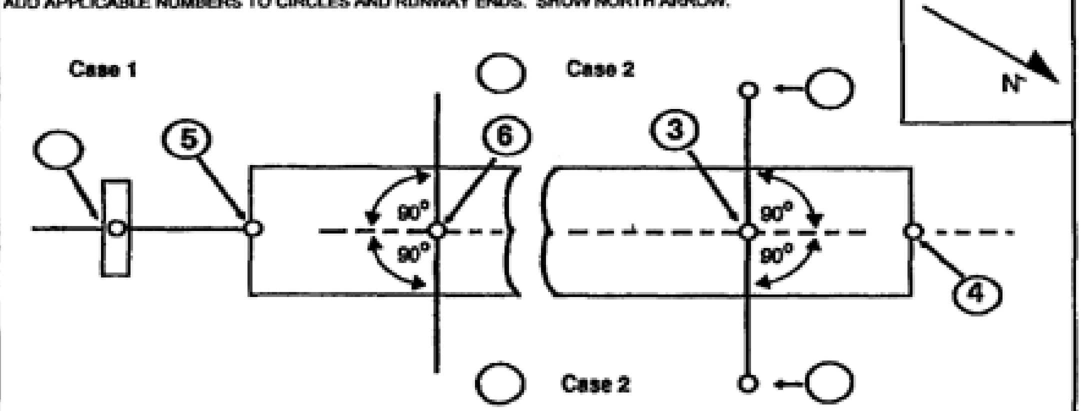
ILS COMPONENTS AND PERTINENT RUNWAY DATA Numbered items correspond to the diagram below.	LATITUDE (1/ 100 Second)	LONGITUDE (1/ 100 Second)	ELEVATION (1/ 10 Foot)
1. Localizer Antenna (Course Array): Point on ground beneath the localizer antenna.	43°57'14.51"	90°44'05.79"	896.4
2. Glide Slope Indicator (GSI): Center of the base supporting the antenna.	43°57'36.10"	90°43'58.40"	834.7
3. The point on runway C/L closest to the base of the Glide Slope Indicator Antenna (Item 2).	43°57'36.33"	90°43'58.40"	831.4
4. Runway C/L End.	43°57'43.16"	90°44'42.02"	829.5
5. Runway C/L End.	43°57'27.48"	90°43'48.70"	831.8
6. The point on runway C/L closest to the base of the offset localizer (Case 2).			

SAMPLE

MARKERS	LATITUDE (1/10 Second)	LONGITUDE (1/10 Second)	GROUND DISTANCE TO END OF RUNWAY
INNER OR B.C. MARKER (RUNWAY END)			
MIDDLE MARKER (RUNWAY END)			
OUTER MARKER (RUNWAY END)			

LOCALIZER - GROUND DISTANCE Case 1 (normal)	LOCALIZER - GROUND DISTANCE Case 2 (offset)		
1 to 5 FEET	1 to 6 FEET	2 to 3 FEET	GEODETIC AZIMUTH SOUTH ° ' "
	5 to 6 FEET	3 to 4 FEET	4 to 5

ADD APPLICABLE NUMBERS TO CIRCLES AND RUNWAY ENDS. SHOW NORTH ARROW.



DA FORM 5827, JUL 2001 EDITION OF AUG 1989 IS OBSOLETE. USAPA V1.00

Figure 6-6. Sample DA Form 5827 (ILS Data)

LESSON 6

PRACTICE EXERCISE

The following items will test your grasp of the material covered in this lesson. There is only one correct answer for each item. When you have completed the exercise, check your answer with the answer key that follows. If you answer any item incorrectly, study again that part which contains the portion involved.

1. Elevations at the threshold, stop end, and highest elevation should be determined to the nearest ______________________________.

   A. 0.001 foot
   B. 0.01 foot
   C. 0.1 foot
   D. 1.0 foot

2. Where are the navigational imaginary surfaces or obstruction identification surfaces defined?

   A. FAR-77
   B. DMS special test 031
   C. FAA 405
   D. Survey SOPs

3. What surface extends from a periphery of a surfaces outward and upward at a slope of 20:1?

   A. Primary
   B. Outer horizontal
   C. Transitional
   D. Conical

4. What is the slope of the approach-clearance surface?

   A. 100:1
   B. 50:1
   C. 20:1
   D. 7:1

5. Horizontal accuracy requirements can be met using what order traverse?

   A. Third order, Class II
   B. Third order, Class I
   B. Second order
   C. First order

6. What scale is used for AOCs?

   A. 1:5,000
   B. 1:8,000
   C. 1:10,000
   D. 1:12,000

7. What type of paper is used to publish the AOC?

   A. Scaled
   B. Graphic
   C. E50 chart
   D. Specialty sized, 30 x 30 inches

LESSON 6

PRACTICE EXERCISE

ANSWER KEY AND FEEDBACK

Item		Correct Answer and Feedback
1.	C	0.1 foot Elevations at the runway threshold...(page 6-3, para 6-la)
2.	A	FAR-77 Navigational imaginary surfaces...(page 6-3, para 6-2)
3.	D	Conical A conical surface is a surface...(page 6-7, para 6-2a[1])
4.	B	50:1 The slope of the approach-clearance...(page 6-7, para 6-2a[2])
5.	A	Third order, Class II Horizontal accuracy requirements...(page 6-10, para 6-4a)
6.	D	1:12,000 An AOC is a 1:12,000-scale…(page 6-10, para 6-5a)
7.	C	E50 chart The AOC is published on E50 chart...(page 6-11, para 6-5a)

APPENDIX A

LIST OF COMMON ACRONYMS

°	degree
α	angle
/	divided by
′	minute(s)
″	second(s)
#	number
1DRMS	1-deviation root-mean-square
1st	first
2DRMS	2-deviation root-mean-square
2nd	second
2D	two dimensional
3D	three dimensional
3DRMS	3-deviation root-mean-square
ACCP	Army Correspondence Course Program
ADA	air defense artillery
AE	allowable error
AEC	angular error of closure
AIPD	Army Institute for Professional Development
AISI	automated integrated survey instrument
ALP	airport location point

AMEDD	Army Medical Department
AO	area of operation
AOC	airport obstruction chart
AP	airport plan
APO	Army Post Office
AR	Army regulation
ARP	airport reference point
ARSR	air route surveillance radar
AS	antispoofing
ASR	airport surveillance radar
Aug	August
AUTOVON	automatic voice network
AV	automatic voice network
AWR	answer weight reference
az	azimuth
b	backward
BC	basic control
bm	benchmark
bn	battalion
bs	backsight
C	Celsius
C/A-code	course-acquisition code
C/L	centerline
CAD	computer-aided design

CADD	computer-aided design and drafting
cal	caliber
CEOI	communications-electronics operation instruction
CEP	circular error probable
CHRON	chronometer
coml	commercial
comp	computation
CONUS	continental United States
coords	coordinates
corr	correction
d	direct
D	ratio of side/sine
DA	Department of the Army
DD	day
dE	difference in easting
DE	difference in elevation
deg	degree(s)
DETC	Distance Education and Training Council
dev	development
DGPS	differential global-positioning system
DINFOS	Defense Information School
DIS	distance
DME	distance measuring equipment
DMS	Defense Mapping School

dN	difference in northing
DOD	Department of Defense
DOI	Department of Interior
DOP	dilution of precision
DSN	Defense Switched Network
dsplcd	displaced
DT	displaced threshold
E	east
ECEF	earth centered earth fixed
EDM	electronic distance measurement
EDME	electronic distance-measuring equipment
Ee	error in easting
elev	elevation
ell	ellipsoidal
E-mail	electronic mail
En	error in northing
EN	engineer
engr	engineer
eph	ephemeris (ephemerides)
esc	escape
f	forward
FA	field artillery
FAA	Federal Aviation Administration
FAA 405	Federal Aviation Administration Publication 405

FAR	Federal Aviation Regulation
FAR-77	Federal Aviation Regulation, Part 77
FGCC	Federal Geodetic Control Committee
FGCS	Federal Geodetic Control Subcommittee
FM	field manual
FM	frequency modulated
FRNP	Federal Radio Navigation Plan
fs	foresight
ft	fort
G	gravity
GCA	ground-controlled approach
GD	ground
GDOP	geometric dilution of precision
GIS	graphic information system
GPS	global positioning system
GPS-S	global positioning system-survey
GRS 80	Geodetic Reference System of 1980
GSI	glide slope indicator
H	orthometric height
HDOP	horizontal dilution of precision
HI	height of instrument
HIRAN	high-precision super-range navigation
HS	height of stand
ht	height

HQ	headquarters
ICE	Interservice Correspondence Exchange
ILS	instrument landing system
incl	including
INS	inertial navigation system
int	initial
inst	instrument
IPD	Institute for Professional Development
IUGG	International Union of Geodesy and Geophysics
JAG	Judge Advocate General
JFK	John F. Kennedy
Jul	July
Jun	June
km	kilometer(s)
lat	latitude
LEC	linear error of closure
lon	longitude
long	longitude
LOP	line of position
m	meter(s)
mag	magnetic
meas	measured
met	meteorological
MI	middle initial

micro	micrometer
min	minute(s)
MLS	microwave landing system
MM	month
mn	mean
MO	Missouri
Mon	Monday
MOS	military occupational specialty
MRSE	mean radial spherical error
MSL	mean sea level
MSL 29	Mean Sea Level of 1929
n	north
NAD	North American Datum
NAD 27	North American Datum of 1927
NAD 83	North American Datum of 1983
NAD 84	North American Datum of 1984
NAS	National Airspace System
NAVAID	navigational aid
NAVD 88	North American Vertical Datum of 1988
NAVSTAR	Navigation Satellite Timing and Ranging
NBS	National Bureau of Standards
NDB	nondirectional beacon
NGRS	National Geodetic Reference System
NGS	National Geodetic Survey

NL	notes and legends
No.	number
NSATS	number of satellites
NSRS	National Spatial Reference System
OC	obstruction chart
OCONUS	outside the continental United States
Oct	October
OIS	obstruction identification surface
op	operator
ortho	orthometric
OTF	on the fly
PACS	primary airport control station
PADS	Position and Azimuth Determination System
PAR	precision approach radar
PBM	permanent benchmark
PC	personal computer
P-code	precision code
PDOP	positional dilution of precision
pgdn	page down
pgup	page up
PIR	precise instrument runway
PLGR	precise lightweight GPS receiver
ppm	part(s) per million
PPS	Precise-Positioning Service

PRC	pseudorange correction
PRN	pseudorandom noise
r	reverse
R	degrees of freedom
R	replacement
R1	reject value, use first mean value
R2	reject value, use second mean value
ref	reference
RCOAC	Reserved Component Officer's Advanced Course
RDOP	relative dilution of precision
REIL	runway-end identifier light
ref	refraction
RM	reference mark
RMS	root-mean-square
RPP	runway plans and profiles
RTCM	Radio Technical Commission for Maritime
RTK	real-time kinematic
rwy	runway
RYE	retirement year ending
S3	Operations and Training Officer (U.S. Army)
S/A	selective availability
SACS	secondary airport control station
SC	special committee
SCP	survey control point

SEP	spherical error probable
SFC	sergeant first class
SIC	survey information center
sin	sine
SINCGARS	Single-Channel Ground-to-Air Radio System
SOP	standing operating procedure
SPC	specialist
SPHS	specially prepared hard surface
SPS	Standard Positioning Service
SSGCN	Standards and Specifications for Geodetic Control Networks
SSN	social security number
sta	station
std	standard
SV	satellite vehicle
TACAN	tactical air navigation
TBM	temporary benchmark
TDZE	touchdown zone elevation
TEC	Topographic Engineering Center
tel	telescope
temp	temperature
thermo	thermometer
thr	threshold
TOD	tabulated operational data

TOE	table(s) organization and equipment
tot	total
TP	temporary point
TRADOC	United States Army Training and Doctrine Command
UERE	user-equivalent range error
US	United States
USA	United States of America
USAASA	United States Army Aeronautical Services Agency
USACE	United States Army Corps of Engineers
USAES	United States Army Engineer School
USAPA	United States Army Publishing Agency
USCG	United States Coast Guard
USC&GS	United States Coast and Geodetic Survey
USGS	United States Geological Survey
UTC	universal time, coordinated
VA	Virginia
VDOP	vertical dilution of precision
vert	vertical
VHF	very-high frequency
VOR	very-high-frequency omnidirectional range
VORTAC	very-high-frequency omnidirectional range and tactical air navigation
W	west
WASS	wide-area augmentation system
WGS	World Geodetic System

WGS 84 World Geodetic System 1984

YYYY year

ZEN zenith

APPENDIX B

RECOMMENDED READING LIST

The following publications provide additional information about the material in this subcourse. You do not need these materials to complete this subcourse.

AR 115-11. *Army Topography.* 30 November 1993.
AR 310-50. *Authorized Abbreviations and Brevity Codes.* 15 November 1985.
AR 95-1. *Flight Regulations.* 1 September 1997.
AR 95-2. *Air Traffic Control, Airspace, Airfields, Flight Activities, and Navigation Aids.* 10 August 1990.
DA Form 1942. *Computation of Levels.* May 2001.
DA Form 1943. *Abstract of Zenith Distances.* July 2001.
DA Form 1959. *Description or Recovery of Horizontal Control Station.* July 2001.
DA Form 4446. *Level, Transit, and General Survey Record Book.* 1 November 1975.
DA Form 5817. *Zenith Distance/Vertical Angle.* June 2001.
DA Form 5820. *Three-Wire Leveling.* July 2001.
DA Form 5821. *Airfield Compilation Report.* July 2001.
DA Form 5822. *Precision Approach Radar (GCA) Data.* August 1989.
DA Form 5827. *Instrument Landing System Data.* July 2001.
FAA 405. *Standards for Aeronautical Surveys and Related Products, Forth Edition.* September 1996.
FAR-77. *Objects Affecting Navigable Airspace.* 15 Jul 1996.
FM 101-5-1. *Operational Terms and Graphics.* 30 September 1997.
FM 21-31. *Topographic Symbols.* 19 June 1961.
FM 3-34.230. *Topographic Operations.* 03 August 2000.
FM 3-34.331. *Topographic Surveying.* To be published within six months.
FM 5-233. *Construction Surveying.* 4 January 1985.
FM 5-553. *General Drafting.* 6 January 1984.
FM 6-2. Tactics, Techniques, and Procedures for Field Artillery Survey. 23 September 1983.

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APPENDIX C

LEVELING SPECIFICATIONS

Table C-1. Leveling Specifications

	First Order	Second Order		Third Order
		Class I	Class II	
*Spacing of lines and crosslines	72 kilometers	40 kilometers	10 kilometers	Not specified
Average spacing of permanently-marked stations not to exceed	2 kilometers	2-5 kilometers	2-5 kilometers	5-8 kilometers
Length of the section	1-2 kilometers	1-2 kilometers	1-2 kilometers	1-2 kilometers
Check between forward and backward running between fixed elevations to loop closures, not to exceed.	3 mm $\sqrt{K}$	8.4 mm $\sqrt{K}$	8.4 mm $\sqrt{K}$	12 mm $\sqrt{K}$
K is the distance in kilometers. *In areas outside of the US, this criteria may be changed to conform to the situation.				

Table C-2. Triangulation Specifications

Criterion for—	First Order			Second Order		Third Order
	Class I Special	Class II Optimum	Class III Standard	Class I	Class II	
Principal uses	Urban surveys, scientific studies.	Basic network.	All others.	Area networks and supplemental cross arcs in the national net.	Coastal areas, inland waterways, and engineering surveys.	Topographic mapping.
*Spacing of arcs or principal stations	Stations: 1 to 8 kilometers or greater as required.	Arcs: 96 kilometers Stations: 16 to 234 kilometers	Stations: 16 to 24 kilometers	Stations: 6 to 16 kilometers	As required.	As required.
Strength of figure ΣR_1, between bases:						
Desirable limit	25	60	80	80	100	125
Maximum limit	30	80	110	120	130	175
Single figure: Desirable limit:						
R_1	5	10	15	15	25	25
R_2	10	30	50	70	80	120
Maximum limit:						
R_1	10	25	25	25	40	50
R_2	15	60	80	100	120	170
Base measurement Actual error not to exceed**	1 part in 300,000	1 part in 300,000	1 part in 300,000	1 part in 300,000	1 part in 150,000	1 part in 75,000
Probable error not to exceed	1 part in 1,000,000	1 part in 1,000,000	1 part in 1,000,000	1 part in 1,000,000	1 part in 500,000	1 part in 250,000
Triangle closure						
Average not to exceed	1"	1"	1"	1"5	3"	5"
Maximum seldom to exceed	3"	3"	3"	5"	5"	10"
Side checks Ratio of maximum difference of logs of the sides to the tabulated difference for 1 inch of log sine of smallest angle	1.5	1.5	2	2-4	4	7.5 - 10
OR Inside equation test, average correction to directions not to exceed	0"3	0"4	0"4	0"6	0"8	2"

Table C-2 Triangulation Specifications (continued)

Astronomic azimuths Spacing figures Probable error	6-8 0"3	6-8 0"3	8-10 0"3	8-10 0"3	10-12 0"5	12-15 2"0
Closure in length (Also position when applicable) After side and angle conditions have been satisfied, should not exceed	1 part in 100,000	1 part in 50,000	1 part in 25,000	1 part in 20,000	1 part in 10,000	1 part in 50,000

*Additional stations of the same accuracy may be interspersed among principal stations
** The "actual error" is the difference between the true value and the measured value

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APPENDIX D

MENSURAL CONVERSION CHARTS

This appendix complies with current Army directives which state that the metric system will be incorporated into all new publications. Table D-1 is a conversion chart.

Table D-1. Metric Conversion Chart

US Units	Multiplied By	Equals Metric Units	Metric Units	Multiplied By	Equals US Units
Length					
Inches	2.5400	Centimeters	Centimeters	0.39370	Inches
Inches	25.4001	Millimeters	Millimeters	0.03937	Inches
Feet	0.3048	Meters	Meters	3.28080	Feet
Yards	0.9144	Meters	Meters	1.09360	Yards
Miles	1.6093	Kilometers	Kilometers	0.62140	Miles
Miles, Nautical	1.8532	Kilomete s	Kilometers	0.53960	Miles, Nautical
Area					
Square miles	2.590	Square kilometers	Square kilometers	0.38500	Square miles
Volume					
Gallons	3.7854	Liters	Liters	0.26420	Gallons
Mass (Weight)					
Pounds	0.4536	Kilograms	Kilograms	2.20460	Pounds

Table D-2. Temperature, Angle, and Time Conversion Chart

Units	Multiplied By	Equals	Units	Multiplied By	Equals
Temperature					
Degrees (F) - 32	0.5556	Degrees (C)	Degrees (C) + 17.8	1.8000	Degrees (F)
Angle					
Degrees (angular)	17.7778	Mils	Mils	0.0562	Degrees (angular)
Time					
Seconds	0.001	Milliseconds	Milliseconds	1,000	Seconds
Seconds	0.000000001	Nanoseconds	Nanoseconds	1,000,000,000	Seconds