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CE8351 SURVEYING NOTES

UNIT I FUNDAMENTALS OF CONVENTIONAL SURVEYING AND LEVELLING

Surveying

The practice of measuring angles and distances on the ground so that they can be accurately plotted on a map

Principles of Surveying

The fundamental principles upon which the surveying is being carried out are

- Working from whole to part.
- After deciding the position of any point, its reference must be kept from at least two permanent objects or stations whose position have already been well defined.

The purpose of working from whole to part is

- to localise the errors and
- to control the accumulation of errors

Classifications of Surveying

Based on the purpose (for which surveying is being conducted), Surveying has been classified into:

Control surveying : To establish horizontal and vertical positions of control points.

Land surveying : To determine the boundaries and areas of parcels of land, also known asproperty survey, boundary survey or cadastral survey.

Topographic survey : To prepare a plan/ map of a region which includes natural as well asand man-made features including elevation.

Engineering survey : To collect requisite data for planning, design and execution ofengineering projects. Three broad steps are

1) **Reconnaissance survey :** To explore site conditions and availability of infrastructures.

2) **Preliminary survey :** To collect adequate data to prepare plan / map of area to be used forplanning and design.

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3) **Location survey :** To set out work on the ground for actual construction / execution of theproject.

Route survey : To plan, design, and laying out of route such as highways, railways, canals, pipelines, and other linear projects.

Construction surveys : Surveys which are required for establishment of points, lines, grades, and for staking out engineering works (after the plans have been prepared and the structural design has been done).

Astronomic surveys : To determine the latitude, longitude (of_the observation station) and azimuth (of a line through observation station) from astronomical observation.

Mine surveys : To carry out surveying specific for opencast and underground mining purposes

Chain Survey

Chain survey is the simplest method of surveying. In this survey only measurements are taken in the field, and the rest work, such as plotting calculation etc. are done in the office. This is most suitable adapted to small plane areas with very few details. If carefully done, it gives quite accurate results. The necessary requirements for field work are chain, tape, ranging rod, arrows and some time cross staff.

Survey Station:

Survey stations are of two kinds

1. Main Stations
2. Subsidiary or tie

Main Stations:

Main stations are the end of the lines, which command the boundaries of the survey, and the lines joining the main stations re called the main survey line or the chain lines.

Subsidiary or the tie stations:

Subsidiary or the tie stations are the point selected on the main survey lines, where it is necessary to locate the interior detail such as fences, hedges, building etc.

Tie or subsidiary lines:

A tie line joints two fixed points on the main survey lines. It helps to checking the accuracy of surveying and to locate the interior details. The position of each tie line should be close to some features, such as paths, building etc.

Base Lines:

It is main and longest line, which passes approximately through the centre of the field. All the other measurements to show the details of the work are taken with respect of this line.

Check Line:

A check line also termed as a proof line is a line joining the apex of a triangle to some fixed points on any two sides of a triangle. A check line is measured to check the accuracy of the framework. The length of a check line, as measured on the ground should agree with its length on the plan.

Offsets:

These are the lateral measurements from the base line to fix the positions of the different objects of the work with respect to base line. These are generally set at right angle offsets. It can also be drawn with the help of a tape. There are two kinds of offsets:

1) Perpendicular offsets, and

2) Oblique offsets.

The measurements are taken at right angle to the survey line called perpendicular or right angled offsets.

The measurements which are not made at right angles to the survey line are called oblique offsets or tie line offsets.

Procedure in chain survey:

1. Reconnaissance:

The preliminary inspection of the area to be surveyed is called reconnaissance. The surveyor inspects the area to be surveyed, survey or prepares index sketch or key plan.

2. Marking Station:

Surveyor fixes up the required no stations at places from where maximum possible stations are possible.

3. Then he selects the way for passing the main line, which should be horizontal and clean as possible and should pass approximately through the centre of work.

4. Then ranging roads are fixed on the stations.

5. After fixing the stations, chaining could be started.

6. Make ranging wherever necessary.

7. Measure the change and offset.

8. Enter in the field the book.

CLASSIFICATION OF SURVEYING

Generally, surveying is divided into two major categories: plane and geodetic surveying.

PLANE SURVEYING

PLANE SURVEYING is a process of surveying in which the portion of the earth being surveyed is considered a plane. The term is used to designate survey work in which the distances or areas involved are small enough that the curvature of the earth can be disregarded without significant error. In general, the term of limited extent. For small areas, precise results may be obtained with plane surveying methods, but the accuracy and precision of such results will decrease as the area surveyed increases in size. To make computations in plane surveying, you will use formulas of plane trigonometry, algebra, and analytical geometry.

A great number of surveys are of the plane surveying type. Surveys for the location and construction of highways and roads, canals, landing fields, and railroads are classified under plane surveying. When it is realized that an arc of 10 mi is only 0.04 greater that its subtended chord; that a plane surface tangent to the spherical arc has departed only about 8 in. at 1 mi from the point of tangency; and that the sum of the angles of a spherical triangle is only 1 sec greater than the sum of the angles of a plane triangle for a triangle having an area of approximately 75 sq mi on the earth'ssurface, it is just reasonable that the errors caused by the earth'sconsideredcurvatureonlyinprecisesurveysbeof large areas.

In this training manual, we will discuss primarily the methods used in plane surveying rather than those used in geodetic surveying.

GEODETIC SURVEYING

GEODETIC SURVEYING is a process of surveying in which the shape and size of the earth are considered. This type of survey is suited for large areas and long lines and is used to find the precise location of basic points needed for establishing control for other surveys. In geodetic surveys, the stations are normally long distances apart, and more precise instruments and surveying methods are required for this type of surveying than for plane surveying.

The shape of the earth is thought of as a spheroid, although in a technical sense, it is not really a spheroid. In 1924, the convention of the International Geodetic and Geophysical Union adopted 41,852,960 ft as the diameter of the earth at the equator and 41,711,940 ft as the diameter at its polar axis. The equatorial diameter was computed on the assumption that the flattening of the earth caused by gravitational attraction is exactly 1/297. Therefore, distances measured on or near the surface of the earth are not along straight lines or planes, but on a curved surface.

Hence, in the computation of distances in geodetic surveys, allowances are made for the earth'sminorand major diameters from which a spheroid of reference is developed. The position of each geodetic station is related to this spheroid. The positions are expressed as latitudes (angles north or south of the Equator) and longitudes (angles east or west of a prime meridian) or as northings and castings on a rectangular grid.

The methods used in geodetic surveying are beyond the scope of this training manual

TOPOGRAPHIC SURVEYS

The purpose of a TOPOGRAPHIC SURVEY is to gather survey data about the natural and man-made features of the land, as well as its elevations. From this information a three-dimensional map may be prepared. You may prepare the topographic map in the office after collecting the field data or prepare it right away in the field by plane table. The work usually consists of the following:

1. Establishing horizontal and vertical control that will serve as the framework of the survey

2. Determining enough horizontal location and elevation (usually called side shots) of ground points to provide enough data for plotting when the map is prepared

3. Locating natural and man-made features that may be required by the purpose of the survey

4. Computing distances, angles, and elevations

5. Drawing the topographic map

Topographic surveys are commonly identified with horizontal and/or vertical control of third- and lower-order accuracies.

ROUTE SURVEYS

The term *route survey* refers to surveys necessary for the location and construction of lines of transportation or communication that continue across country for some distance, such as highways, railroads, open-conduit systems, pipelines, and power lines. Generally, the preliminary survey for this work takes the form of a topographic survey. In the final stage, the work may consist of the following:

1. Locating the center line, usually marked by stakes at 100-ft intervals called stations

2. Determining elevations along and across the center line for plotting profile and cross sections

3. Plotting the profile and cross sections and fixing the grades

4. Computing the volumes of earthwork and preparing a mass diagram

5. Staking out the extremities for cuts and fills

6. Determining drainage areas to be used in the design of ditches and culverts

7. Laying out structures, such as bridges and culverts

8. Locating right-of-way boundaries, as well as staking out fence lines, if necessary

SPECIAL SURVEYS

As mentioned earlier in this chapter, SPECIAL SURVEYS are conducted for a specific purpose and with a special type of surveying equipment and methods. A brief discussion of some of the special surveys familiar to you follows.

Land Surveys

LAND SURVEYS (sometimes called cadastral or property surveys) are conducted to establish the exact location, boundaries, or subdivision of a tract of land in any specified area. This type of survey requires professional registration in all states. Presently, land surveys generally consist of the following chores:

1. Establishing markers or monuments to define and thereby preserve the boundaries of land belonging to a private concern, a corporation, or the government.

2. Relocating markers or monuments legally established by original surveys. This requires examining previous survey records and retracing what was done. When some markers or monuments are missing, they are reestablished following recognized procedures, using whatever information is available.

3. Rerunning old land survey lines to determine their lengths and directions. As a result of the high cost of land, old lines are remeasured to get more precise measurements.

4. Subdividing landed estates into parcels of predetermined sizes and shapes.

5. Calculating areas, distances, and directions and preparing the land map to portray the survey data so that it can be used as a permanent record. 6. Writing a technical description for deeds.

Control Surveys

CONTROL SURVEYS provide "basic control" or horizontal and vertical positions of points to which supplementary surveys are adjusted. These types of surveys (sometimes termed and traverse stations and the elevations of bench marks. These control points are further used as References for hydrographic surveys of the coastal waters; for topographic control; and for the control of many state, city, and private surveys.

Horizontal and vertical controls generated by land (geodetic) surveys provide coordinated position data for all surveyors. It is therefore necessary that these types of surveys use first-order and second-order accuracies.

Hydrographic Surveys

HYDROGRAPHIC SURVEYS are made to acquire data required to chart and/or map shorelines and bottom depths of streams, rivers, lakes, reservoirs, and other larger bodies of water. This type of survey is also of general importance to navigation and to development of water resources for flood control, irrigation, electrical power, and water supply.

As in other special surveys, several different types of electronic and radio-acoustical instruments are used in hydrographic surveys. These special devices are commonly used in determining water depths and location of objects on the bottom by a method called taking SOUNDINGS. Soundings are taken by measuring the time required for sound to travel downward and be reflected back to a receiver aboard a vessel.

TYPES OF SURVEYING

OPERATIONS

The practice of surveying actually boils down to fieldwork and office work. The FIELDWORK consists of taking measurements, collecting engineering data, and testing materials. The OFFICE WORK includes taking care of the computation and drawing the necessary information for the purpose of the survey.

FIELDWORK

FIELDWORK is of primary importance in all types of surveys. To be a skilled surveyor, you must spend a certain amount of time in the field to acquire needed experience. The study of this training manual will enable you to understand the underlying theory of surveying, the instruments and their uses, and the surveying methods. However, a high degree of proficiency in actual surveying, as in other professions, depends largely upon the duration, extent, and variation of your actual experience.

You should develop the habit of STUDYING the problem thoroughly before going into the field, You should know exactly what is to be done; how you will do it; why you prefer a certain approach over other possible solutions; and what instruments and materials you will need to accomplish the project.

It is essential that you develop SPEED and CONSISTENT ACCURACY in all your fieldwork. This means that you will need practice in handling the instruments, taking observations and keeping field notes, and planning systematic moves.

It is important that you also develop the habit of CORRECTNESS. You should not accept any measurement as correct without verification. Verification, as much as possible, should be different from the original method used in measurement. The precision of measurement must be consistent with the accepted standard for a particular purpose of the survey.

Fieldwork also includes adjusting the instruments and caring for field equipment. Do not attempt to adjust any instrument unless you understand the workings or functions of its parts. Adjustment of instruments in the early stages of your career requires close supervision from a senior EA.

Factors Affecting Fieldwork

The surveyor must constantly be alert to the different conditions encountered in the field. Physical factors, such as TERRAIN AND WEATHER CONDITIONS, affect each field survey in varying degrees. Measurements using telescopes can be stopped by fog or mist. Swamps and flood plains under high water can impede taping surveys. Sights over open water or fields of flat, unbroken terrain create ambiguities in measurements using microwave equipment. The lengths of light-wave distance in measurements are reduced in bright sunlight. Generally, reconnaissance will predetermine the conditions and alert the survey party to the best method to use and the rate of progress to expect.

The STATE OF PERSONNEL TECHNICAL READINESS is another factor affecting field-work. As you gain experience in handling various surveying instruments, you can shorten survey time and avoid errors that would require resurvey.

The PURPOSE AND TYPE OF SURVEY are primary factors in determining the accuracy requirements. First-order triangulation, which becomes the basis or "control" of future surveys, is made to high-accuracy standards. At the other extreme, cuts and fills for a highway survey carry accuracy standards of a much lower degree. In some construction surveys, normally inaccessible distances must be computed. The distance is computed by means of trigonometry, using the angles and the one distance that can be measured. The measurements must be made to a high degree of precision to maintain accuracy in the computed distance.

So, then, the purpose of the survey determines the accuracy requirements. The required accuracy, in turn, influences the selection of instruments and procedures. For instance, comparatively rough procedures can be used in measuring for earthmoving, but grade and alignment of a highway have to be much more precise, and they, therefore, require more accurate measurements. Each increase in precision also increases the time required to make the measurement, since greater care and more observations will be taken. Each survey measurement will be in error to the extent that no measurement is ever exact. The errors are classified as systematic and accidental and are explained in the latter part of this text. Besides errors, survey measurements are subject to mistakes or blunders. These arise from misunderstanding of the problem, poor judgment, confusion on the part of the surveyor, or simply from an oversight. By working out a systematic procedure, the surveyor will often detect a mistake when some operation seems out of place. The procedure will be an advantage in setting up the equipment, in making observations, in recording field notes, and in making computations.

Survey speed is not the result of hurrying; it is the result of saving time through the following factors:

1. The skill of the surveyor in handling the instruments

2. The intelligent planning and preparation of the work

3. The process of making only those measurements that are consistent with the accuracy requirements

Experience is of great value, but in the final analysis, it is the exercise of a good, mature, and competent degree of common sense that makes the difference between a good surveyor and an exceptional surveyor.

Field Survey Parties

The size of a field survey party depends upon the survey requirements, the equipment available, the method of survey, and the number of personnel needed for performing the different functions. Four typical field survey parties commonly used in the SEABEEs are briefly described in this section: a level party, a transit party, a stadia party, and a plane table party.

LEVEL PARTY.- The smallest leveling party consists of two persons: an instrumentmanand a rodman. This type of organization requires the instrumentman to act as note keeper. The party may need another recorder and one or more extra rodmen to improve the efficiency of the different leveling operations. The addition of the rodmen eliminates the waiting periods while one person moves from point to point, and the addition of a recorder allows the instrumentman to take readings as soon as the rodmen are in position. When leveling operations are run along with other control surveys, the leveling party may be organized as part of a combined party with personnel assuming dual duties, as required by the work load and as designated by the party chief.

TRANSIT PARTY.- A transit party consists of at least three people: an instrumentman, ahead chainman, and a party chief. The party chief is usually the note keeper and may double as rear chainman, or there may be an additional rear chainman. The instrumentman operates the transit; the head chainman measures the hori-zontal distances; and the party chief directs the survey and keeps the notes.

STADIA PARTY.- A stadia party should consist of three people: an instrumentman, a notekeeper, and a rodman. However, two rodmen should be used if there are long distances between observed points so that one can proceed to a new point, while the other is holding the rod on a point being observed. The note keeper records the data called off by the instrumentman and makes the sketches required.

PLANE TABLE PARTY.- The plane table party consists of three people: a topographer orplane table operator, a rodman, and a computer. The topographer is the chief of the party who sets up, levels, and orients the plane table; makes the necessary readings for the determination of horizontal distances and elevations; plots the details on the plane table sheet as the work proceeds; and directs the other members of the party.

The rodman carries a stadia rod and holds it vertically at detail points and at critical terrain points in the plotting of the map. An inexperienced rodman must be directed by the topographer to each point at which the rod is to be held. An experienced rodman will expedite the work of the party by selecting the proper rod positions and by returning at times to the plane table to draw in special details that he may have noticed.

The computer reduces stadia readings to horizontal and vertical distances and computes the ground elevation for rod observations. He carries and positions the

umbrella to shade the plane table and performs other duties as directed by the topographer. At times, the computer may be used as a second rodman, especially when the terrain is relatively flat and computations are mostly for leveling alone.

Field Notes

Field notes are the only record that is left after the field survey party departs the survey site. If these notes are not clear and complete, the field survey was of little value. It is therefore necessary that your field notes contain a complete record of all of the measurements made during the survey and that they include, where necessary, sketches and narrations to clarify the notes. The following guidelines apply.

LETTERING.- All field notes should be lettered legibly. The lettering should be in freehand, vertical or slanted Gothic style, as illustrated in basic drafting. A fairly hard pencil or a mechanical lead holder with a 3H or 4H lead is recommended. Numerals and decimal points should be legible and should permit only one interpretation.

FORMAT.- Notes must be kept in the regular field notebook and not on scraps of paper for later transcription. Separate surveys should be recorded on separate pages or in different books. The front cover of the field notebook should be marked with the name of the project, its general location, the types of measurements recorded, the designation of the survey unit, and other pertinent information.

The inside front cover should contain instructions for the return of the notebook, if lost. The right-hand pages should be reserved as an index of the field notes, a list of party personnel and their duties, a list of the instruments used, dates and reasons for any instrument changes during the course of the survey, and a sketch and description of the project.

Throughout the remainder of the notebook, the beginning and ending of each day'swork should be clearly indicated. Where pertinent, the weather, including temperature and wind velocities, should also be recorded. To minimize recording errors, someone other than the recorder should check and initial all data entered in the notebook.

RECORDING.- Field note recording takes three general forms: tabulation, sketches, anddescriptions. Two, or even all three, forms may be combined, when necessary, to make a complete record.

In TABULATION, the numerical measurements are recorded in columns according to a prescribed plan. Spaces are also reserved to permit necessary computations.

SKETCHES add much to clarify field notes and should be used liberally when applicable. They may be drawn to an approximate scale, or important details may be exaggerated for clarity. A small ruler or triangle is an aid in making sketches. Measurements should be added directly on the sketch or keyed in some way to the tabular data. An important requirement of

a sketch is legibility. See that the sketch is drawn clearly and large enough to be understandable.

Tabulation, with or without added sketches, can also be supplemented with DESCRIPTIONS. The description may be only one or two words to clarify t he recorded measurements. It may also be quite a narration if it is to be used at some future time, possibly years later, to locate a survey monument.

ERASURES ARE NOT PERMITTED IN FIELD NOTEBOOKS. Individual numbers or lines recorded incorrectly are to be lined out and the correct values inserted. Pages that are to be rejected are crossed out neatly and referenced to the substituted pages. THIS PROCEDURE IS MANDATORY since the field notebook is the book of record and is often used as legal evidence. Standard abbreviations, signs, and symbols are used in field notebooks. If there is any doubt as to their meaning, an explanation must be given in the form of notes or legends.

1.5.**OFFICE WORK**

OFFICE WORK in surveying consists of converting the field measurements into a usable format. The conversion of computed, often mathematical, values may be required immediately to continue the work, or it may be delayed until a series of field measurements is completed. Although these operations are performed in the field during lapses between measurements, they can also be considered office work. Such operations are normally done to save time. Special equipment, such as calculators, conversion tables, and some drafting equipment, are used in most office work.

In office work, converting field measurements (also called reducing) involves the process of computing, adjusting, and applying a standard rule to numerical values.

Computation

In any field survey operation, measurements are derived by the application of some form of mathematical computation. It may be simple addition of several full lengths and a partial tape length to record a total linear distance between two points. It maybe the addition or subtraction of differences in elevation to determine the height of instrument or the elevation during leveling. Then again, it maybe checking of angles to ensure that the allowable error is not exceeded.

Office computing converts these distances, elevations, and angles into a more usable form. The finished measurements may end up as a computed volume of dirt to be moved for a highway cut or fill, an area of land needed for a SEABEE construction project, or a new position of a point from which other measurements can be made.

In general, office computing reduces the field notes to either a tabular or graphic form for a permanent record or for continuation of fieldwork.

Adjustment

Some survey processes are not complete until measurements are within usable limits or until corrections have been applied to these measurements to distribute accumulated errors. Small errors that are not apparent in individual measurements can accumulate to a sizeable amount. Adjusting is the process used to distribute these errors among the many points or stations until the effect on each point has been reduced to the degree that all measurements are within usable limits.

For example, assume that 100 measurements were made to the nearest unit for the accuracy required. This requires estimating the nearest one-half unit during measurement. At the end of the course, an error of + 4 units results. Adjusting this means each measurement is reduced 0.04 unit. Since the measurements were read only to the nearest unit, this adjustment would not be measurable at any point, and the adjusted result would be correct.

SIGNIFICANT FIGURES.- The term known to be exact.

In a measured quantity, the number of significant figures is determined by the accuracy of the measurement. For example, a roughly measured distance of 193 ft has three significant figures. More carefully measured, the same distance, 192.7 ft, has four significant figures. If measured still more accurately, 192.68 ft has five significant figures.

In surveying, the significant figures should reflect the allowable error or tolerance in the measurements. For example, suppose a measurement of 941.26 units is made with a probable error of ± 0.03 unit. The ± 0.03 casts some doubt on the fifth digit which can vary from 3 to 9, but the fourth digit will still remain 2. We can say that 941.26 has five significant figures; and from the allowable error, we know the fifth digit is doubtful. However, if the probable error were ±0.07, the fourth digit could be affected. The number could vary from 941.19 to 941.33, and the fourth digit could be read 1, 2, or 3. The fifth digit in this measurement is meaningless. The number has only four significant figures and should be written as such.

The number of significant figures in a number ending in one or more zeros is unknown unless more information is given. The zeros may have been added to show the location of the decimal point; for example, 73200 may have three, four, or five significant figures, depending on whether the true value is accurate to 100, 10, or 1 unit(s). If the number is written 73200.0, it indicates accuracy is carried to the tenth of a unit and is considered to have six significant figures.

When decimals are used, the number of significant figures is not always the number of digits. A zero may or may not be significant, depending on its position with respect to the decimal

and the digits. As mentioned above, zeros may have been added to show the position of the decimal point. Study the following examples:

0.000047 . . . . . .two significant figures

0.0100470 . . . . . .six significant figures

0.1000470 . . . . . .seven significant figures

2.0100470 . . . . . .eight significant figures

In long computations, the values are carried out to one more digit than required in the result. The number is rounded off to the required numbers of digits as a final step.

ROUNDING OFF NUMBERS.- Rounding off is the process of dropping one or more digits and replacing them with zeros, if necessary, to indicate the number of significant figures. Numbers used in surveying are rounded off according to the following rules:

1. When the digit to be dropped is less than 5, the number is written without the digit or any others that follow it. (Example: 0.054 becomes 0.05.)

2. When the digit is equal to 5, the nearest EVEN number is substituted for the preceding digit. (Examples: 0.055 becomes 0.06; 0.045 becomes 0.04.)

3. When the digit to be dropped is greater than 5, the preceding digit is increased by one. (Example: 0.047 becomes 0.05.)

4. Dropped digits to the left of the decimal point are replaced by zeros.

5. Dropped digits to the right of the decimal points are never replaced.

EXAMPLES:

2738.649	to five significant figures equals	2738.6
792.850	to four significant figures equals	792.8
792.750	to four significant figures equals	792.8
675823.	to four significant figures equals	675800
675863.	to four significant figures equals	675900
4896.3	to four significant figures equals	4896
4896.7	to four significant figures equals	4897

CHECKING COMPUTATIONS.- Most mathematical problems can be solved by more than one method. To check a set of computations, you should use a method that differs from the original method, if possible. An inverse solution, starting with the computed

value and solving for the field data, is one possibility. The planimeter and the protractor are also used for approximate checking. A graphical solution can be used, when feasible, especially if it takes less time than a mathematical or logarithmic solution. Each step that cannot be checked by any other method must be recomputed; and, if possible, another EA should recompute the problem. When an error or mistake is found, the computation should be rechecked before the correction is accepted

Drafting Used In Surveying

The general concept of drafting and the use of drafting instruments were discussed in chapters 2 through 5. By this time, you should be familiar with the use of various drafting instruments and with the elements of mechanical drawing. Draft-ing used in surveying, except for some freehand sketches, is generally performed by mechanical means; for example, the drawing of lines and surveying symbols is generally done with the aid of a straightedge, spline, template, and so on.

The drawings you make that are directly related to surveying will consist of maps, profiles, cross sections, mass diagrams, and, to some extent, other graphical calculations. Their usefulness depends upon how accurately you plot the points and lines representing the field measurements. It is important that you adhere to the requirements of standard drawing practices. Correctness, neatness, legibility, and well proportioned drawing arrangements are signs of professionalism.

In drawing a PROPERTY map, for example, the following general information must be included:

1. The length of each line, either indicated on the line itself or in a tabulated form, with the distances keyed to the line designation.

2. The bearing of each line or the angles between lines.

3. The location of the mapped area as referenced to an established coordinate system.

4. The location and kind of each established monument indicating distances from referencemarks.

5. The name of each road, stream, landmark, and so on.

6. The names of all property owners, including those whose lots are adjacent to the mapped area.

7. The direction of the true or magnetic meridian, or both.

8. A graphical scale showing the corresponding numerical equivalent.

9. A legend to the symbols shown on the map, if those shown are not standard signs.

10. A title block that distinctly identifies the tract mapped or the owner'snam.(It is required to contain the name of the surveyor, the name of the draftsman, and the date of the survey.)

Besides the above information, there are some other items that may be required if the map is to become a public record. When this is the case, consult the local office of the Bureau of Land Management or the local surveyors'societyfor the correct general information requirements to be included in the map to be drawn.

In drawing maps that will be used as a basis for studies, such as those to be used in roads, structures, or waterfront construction, you are required to include the following general information:

1. Information that will graphically represent the features in the plan, such as streams, lakes, boundaries, roads, fences, and condition and culture of the land.

2. The relief or contour of the land.

3. The graphical scale.

4. The direction of the meridian.

5. The legend to symbols used, if they are not conventional signs.

6. A standard title block with a neat and appropriate title that states the kind or purpose of the map. Again, the surveyor'snameandthat of the draftsman, as well as the date of survey, are to be included in the title block.
Maps developed as a basis for studies are so varied in purpose that the above information may be adequate for some but inadequate for others. The Engineering Aid, when in doubt, should consult the senior EA, the engineering officer, or the operations officer as to the information desired in the proposed map. The senior EA or the chief of the field survey party is required to know all these requirements before actual fieldwork is started.

A map with too much information is as bad as a map with too little information on it. It is not surprising to find a map that is so crowded with information and other details that it is hard to comprehend. If this happens, draw the map to a larger scale or reduce the information or details on it. Then, provide separate notes or descriptions for other information that will not fit well and thus will cause the appearance of overcrowding. Studying the features and quality

of existing maps developed by NAVFACENGCOM and civilian architects and engineers (A & E) agencies will aid you a great deal in your own map drawing.

MAGNETIC COMPASS

A magnetic compass is a device consisting principally of a circular compass card, usually graduated in degrees, and a magnetic needle, mounted and free to rotate on a pivot located at the center of the card. The needle, when free from any local attraction (caused by metal), lines itself up with the local magnetic meridian as a result of the attraction of the earth'smagnetic North Pole.

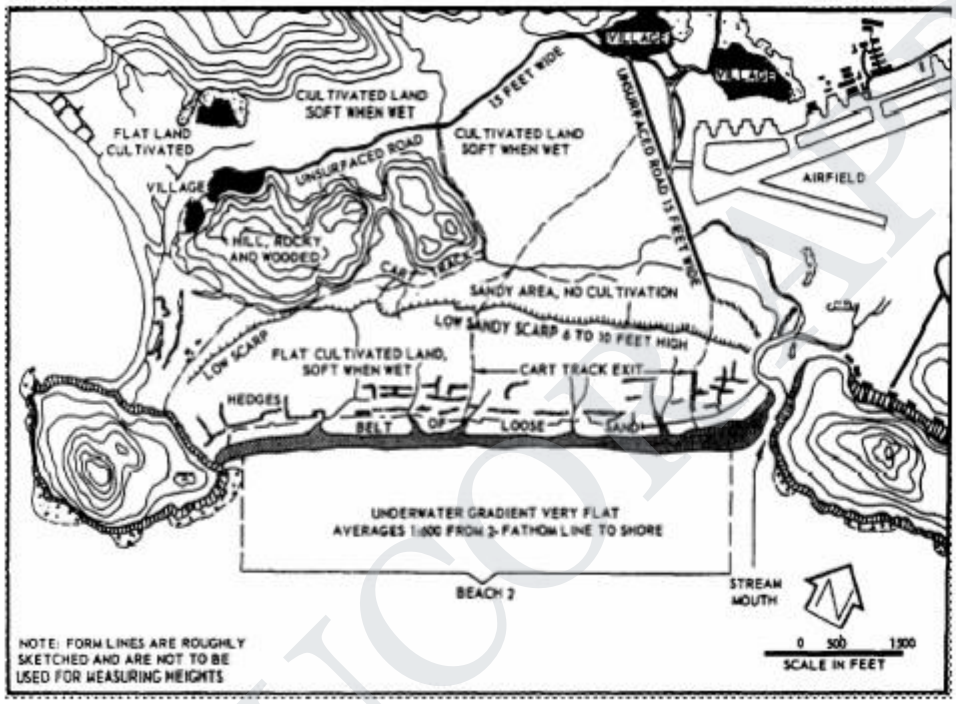


Figure 11-3.-Line map made by overlays from the aerial photograph in figure 11-2.

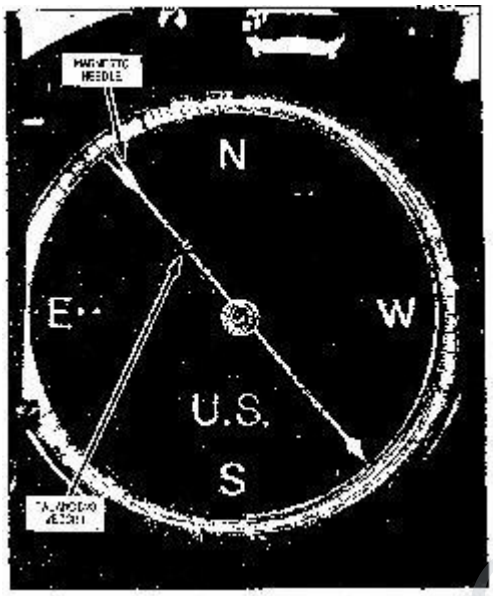
Figure 11-3.-Line map made by overlays from the aerial photograph in figure 11-2.

The magnetic compass is the most commonly used and simplest instrument for measuring directions and angles in the field. This instrument has a variety of both civilian and military applications. The LENSATIC COMPASS (available in your Table of Allowance) is most commonly used for SEABEE compass courses, for map orientation, and for angle direction during mortar and field artillery fires.

In addition to this type of compass, there are several others used exclusively for field surveys. The ENGINEER'STRANSIT COMPASS, located between the standards on the upper plate, is graduated from 0 o through 360 o for measuring azimuths, and in quadrants of 90 o for measuring bearings (fig. 11-4). Notice in figure 11-4 that the east and west markings are reversed. This permits direct reading of the magnetic direction.

The compass shown in figure 11-5 is commonly called the BRUNTON POCKET TRANSIT. This instrument is a combination compass and clinometer. It can be mounted on a light tripod or staff, or it may be cradled in the palm of the hand.

Other types of compasses can also be found in some surveying instruments, such as the theodolite and plane table.



A primary survey fieldwork consists of measuring horizontal and vertical angles or directions and extending straight lines. The instruments that can perform these functions have additional refinements (built-in) that can be used for other survey operations, such as leveling. Two types of instruments that fall into this category are the engineer'stransitand the theodolite. In recent years, manufacturing improvements have permitted construction of direct-reading theodolites that are soon to replace the vernier-reading transits. However, in most SEABEE construction, the engineer'ssit tranisstillthe major surveying instrument.

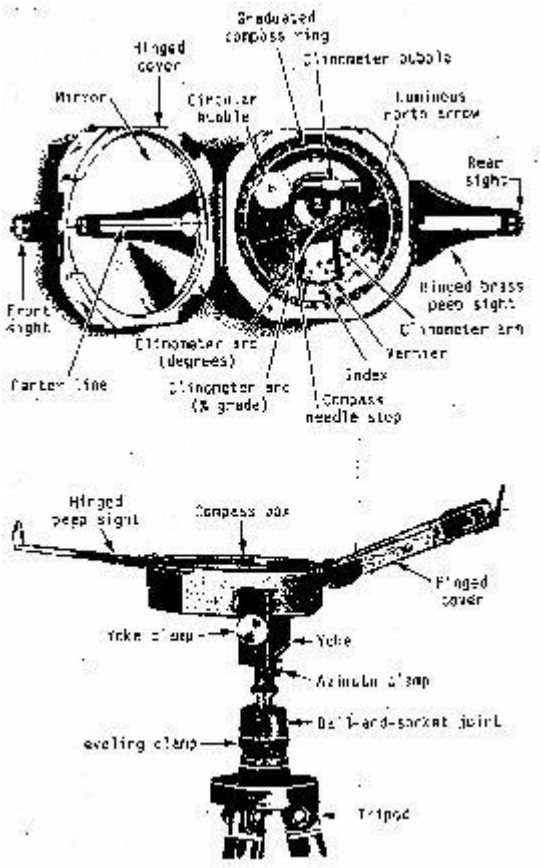


Figure 11-5.-A Brunton pocket transit.

The transit (fig. 11-6) is often called the universal survey instrument because of its uses. It may be used for measuring horizontal angles and directions, vertical angles, and differences in elevations; for prolonging straight lines; and for measuring distances by stadia. Although transits of various manufacturers differ in appearance, they are alike in their essential parts and operations.

The engineer'stransitcontains several hundred parts. For-descriptive purposes, these parts may be grouped into three assemblies: the leveling head assembly, the lower plate assembly, and the upper many plate or alidade assembly (fig. 11-7).

Leveling Head Assembly

The leveling head of the transit normally is the four-screw type, constructed so the instrument can be shifted on the foot plate for centering over a marked point on the ground.

Lower Plate Assembly

The lower plate assembly of the transit consists of a hollow spindle that is perpendicular to the

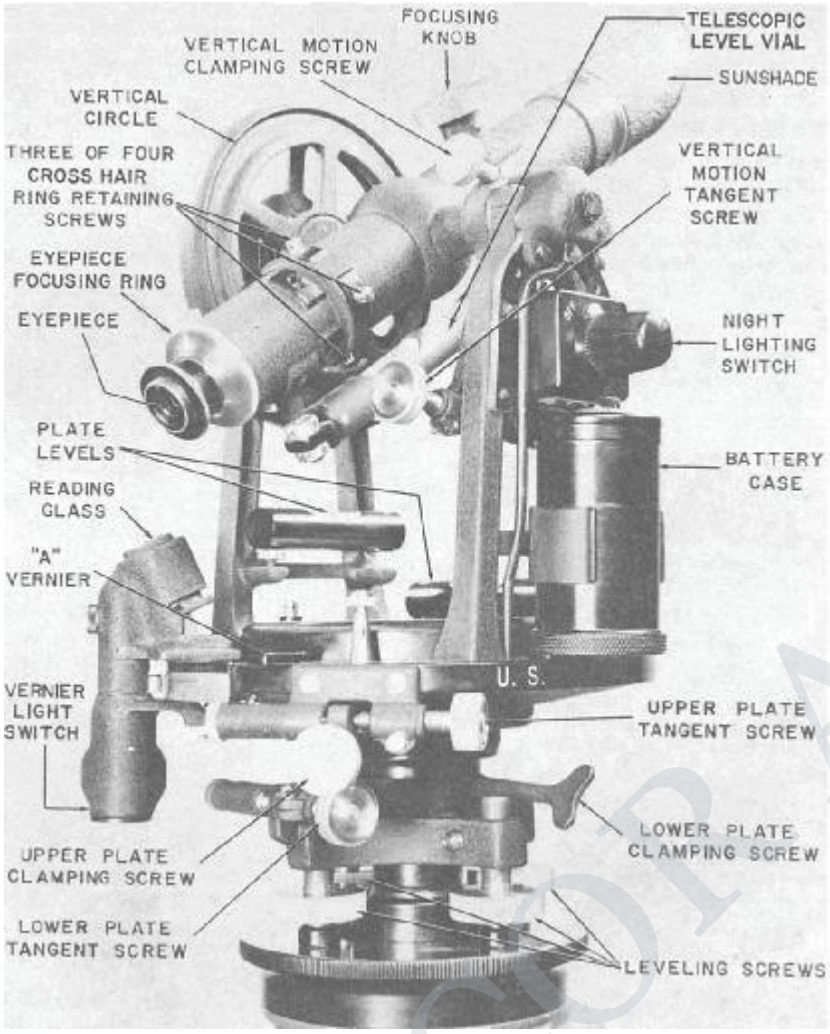


Figure 11-6.-An engineer's transit.

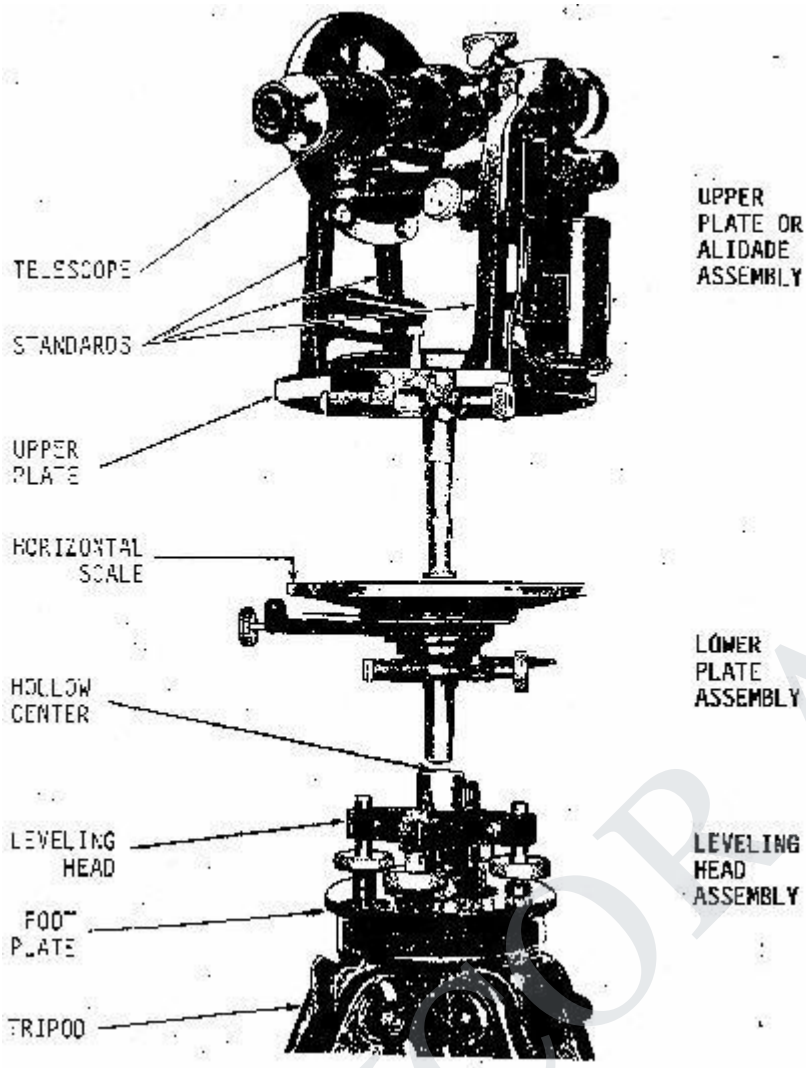


Figure 11-7.-An engineer's transit, exploded view.

center of a circular plate and accurately fitted the socket in the leveling head. The lower plate contains the graduated horizontal circle on which the values of horizontal angles are read with the aid of two verniers, A and B, set on the opposite sides of the circle. A clamp controls the rotation of the lower plate and provides a means for locking it in place. A slow-motion tangent screw is used to rotate the lower plate a small amount to relative to the leveling head. The rotation accomplished by the use of the lower clamp and tangent screw is known as the LOWER MOTION.

Upper Plate or Alidade Assembly

The upper plate, alidade, or vernier assembly consists of a spindle attached plate to a circular plate carrying verniers, telescope standards, plate-level vials, and a magnetic

compass. The spindle is accurately fitted to coincide with the socket in the lower plate spindle.
A clamp is tightened to hold the two plates together or loosened to permit the upper plate to rotate relative to the lower plate. A tangent screw permits the upper plate to be moved a small amount and is known as the UPPER MOTION.

The standards support two pivots with adjustable bearings that hold the horizontal axis and permit the telescope to move on a vertical plane. The vertical circle moves with the telescope. A clamp and tangent screw are provided to control this vertical movement. The vernier for the vertical circle is attached to the left standard. The telescope is an erecting type and magnifies the image about 18 to 25 times. The reticle contains stadia hairs in addition to the cross hairs. A magnetic compass is mounted on the upper plate between the two standards and consists of a magnetized needle pivoted on a jeweled bearing at the center of a graduated circle. A means is provided for lifting the needle off the pivot to protect the bearing when the compass is not in use.

LEVEL VIALS.- Two plate level vials (fig. 11-6) are placed at right angles to each other. On many transits, one plate level vial is mounted on the left side, attached to the standard, under the

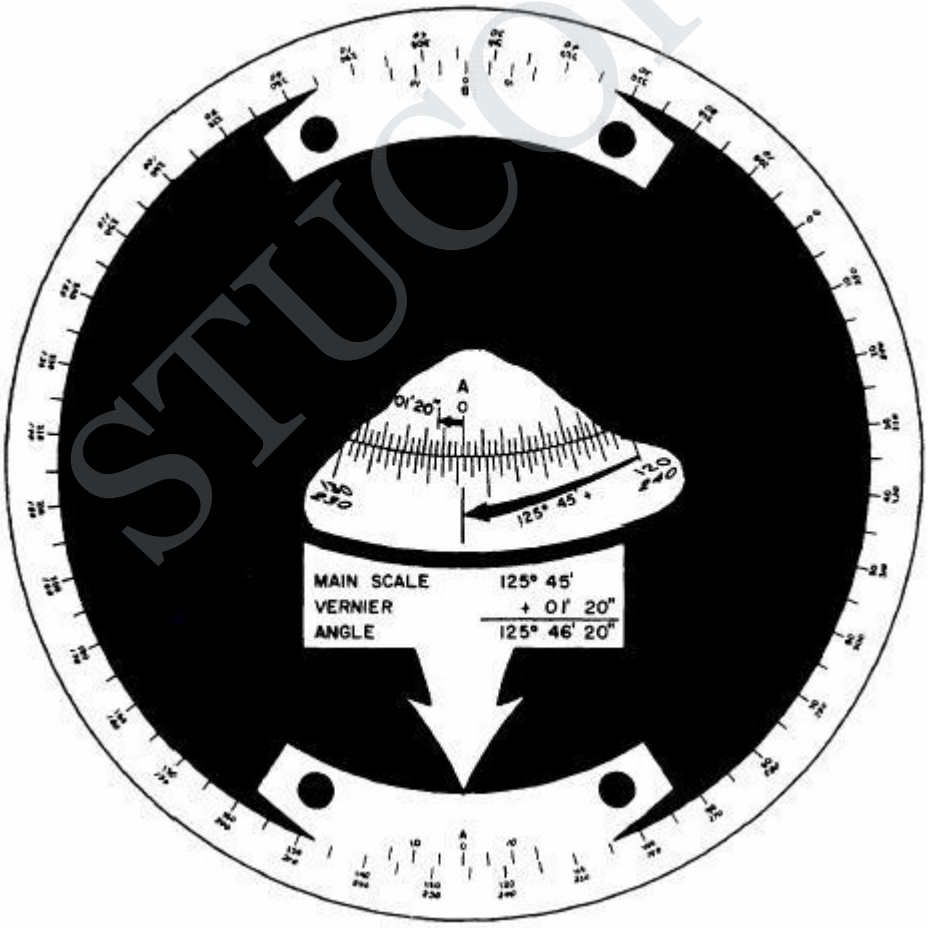


Figure 11-8.-Horizontal scales, 20 second transit.

vertical circle vernier. The other vial is then parallel to the axis of rotation for the vertical motion. The sensitivity of the plate level vial bubbles is about 70 sec of movement for 2 mm of tilt. Most engineer'stransitshave a level vial mounted on the telescope to level it. The sensitivity of this bubble is about 30 sec per 2-mm t i l t.

CIRCLES AND VERNIERS.- The hori-zontal and vertical circles and their verniers are the parts of the engineer'stransitbywhich the values of horizontal and vertical angles are determined. A stadia arc is also included with the vertical circle on some transits.

The horizontal circle and verniers of the transit that are issued to SEABEE units are graduated to give least readings of either 1 min or 20 sec of arc. The horizontal circle is mounted on the lower plate. It is graduated to 15 min for the 20-sec transit (fig. 11-8) and 30 min for the 1-min transit (fig. 11-9). The plates are numbered from 0 o to 360 o , starting with a common point and running both ways around the circle. Two double verniers, known as the A and B verniers, are mounted on the upper plate with their indexes at circle readings 180 o apart. A double vernier is one that can be read in both directions from the index line. The verniers reduce the circle graduations to the final reading of either 20 sec or 1 min.

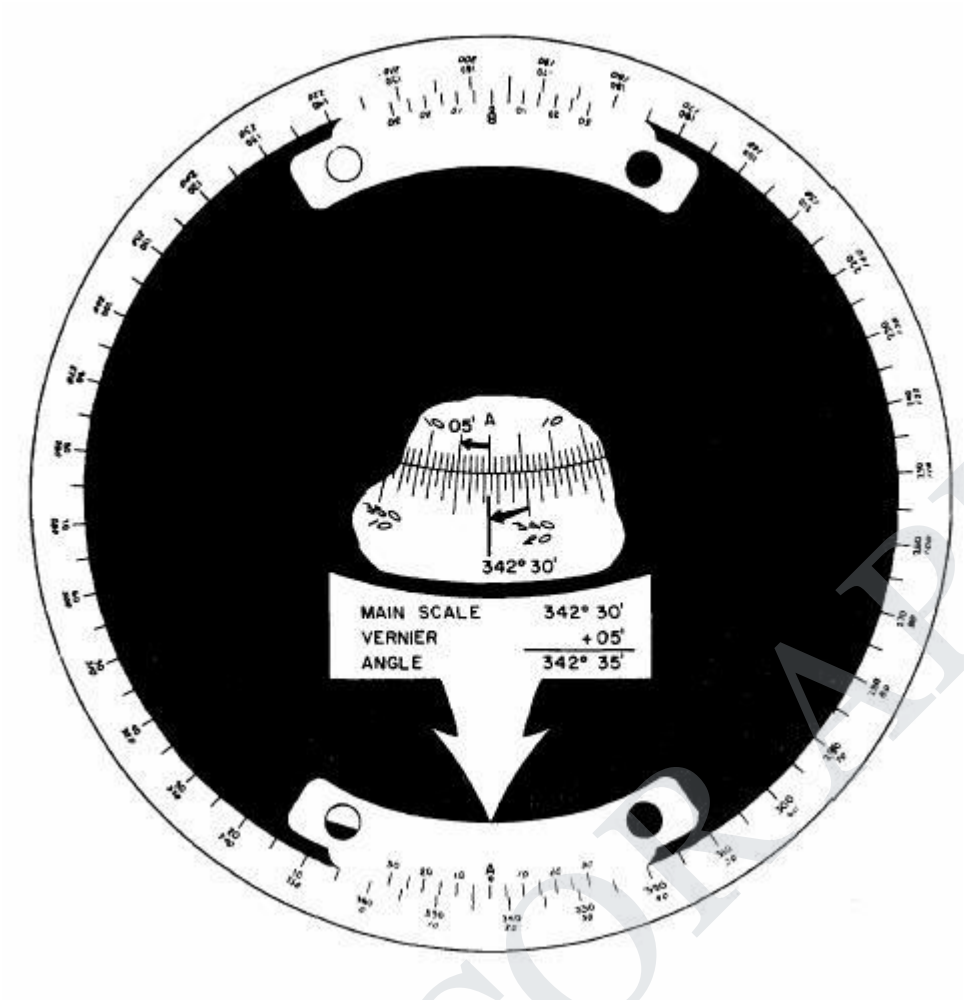


Figure 11-9.-Horizontal scales, 1-minute transit.

The A vernier is used when the telescope is in its normal position, and the B vernier is used when the telescope is plunged.

The VERTICAL CIRCLE of the transit (fig. 11-10) is fixed to the horizontal axis so it will rotate with the telescope. The vertical circle normally is graduated to 30 with 10 o numbering. Each quadrant is numbered from 0 o to 90 o ; the 00 graduations define a horizontal plane, and the 90 o graduations lie in the vertical plane of the instrument. The double vernier used with the circle is attached to the left standard of the transit, and its least reading is 1. The left half of the double vernier is used for reading angles of depression, and the right half of this vernier is used for reading angles of elevation. Care must be taken to read the vernier in the direction that applies to the angle observed.

In addition to the vernier, the vertical circle may have an H and V (or HOR and VERT) series of graduations, called the STADIA ARC (fig. 11-10). The H scale is adjusted to read 100 when the line of sight is level, and the graduations decrease in both directions from the level line.

The other scale, V, is graduated with 50 at level, to 10 as the telescope is depressed, and to 90 as it is elevated.

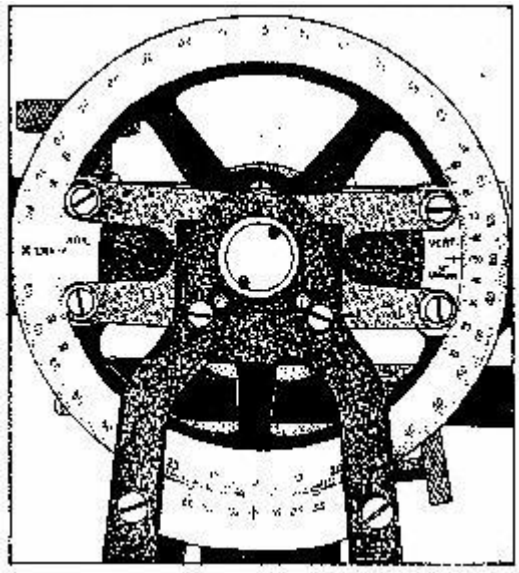


Figure 11-10.-Vertical circle with verniers, scales, and stadia arc.

The VERNIER, or vernier scale, is an auxiliary device by which a uniformly graduated main scale can be accurately read to a fractional part of a division. Both scales may be straight as on a leveling rod or curved as on the circles of a transit. The vernier is uniformly divided, but each division is either slightly smaller (direct vernier) or slightly larger (retrograde vernier) than a division of the main scale (fig. 11-11). The amount a vernier division differs from a division of the main scale determines the smallest reading of the scale that can be made with the particular vernier. This smallest reading is called the LEAST COUNT of the vernier. It is determined by dividing the value of the smallest division on the scale by the number of divisions on the vernier.

Different Types of Surveying Chains

Following are the various types of chain in common use:

1. Metric chains
2. Steel band or Band chain
3. Gunter's chain or surveyors chain
4. Engineers chain
5. Revenue chain

1. Metric Chain:

- Metric chains are made in lengths 20m and 30m. Tallies are fixed at every five-meter length and brass rings are provided at every meter length except where tallies are attached.

2. **Gunter's Chain**

- Length = 66' (22 yards), No of links = 100, Each link = .66'
- Used for measuring distances in miles or furlongs (220 yards), acres (Area).

3. **Engineer's Chain**

- Length = 100', No of links = 100, Each link = 1'
- Used in all Engineering Surveys.

4. **Revenue Chain**

- Length = 33', No of links = 16
- Commonly used for measuring fields in cadastral Survey.

Types of Measuring Tapes in Surveying

Tapes are made of different materials:

1. Cloth or linen tape

- Used for subsidiary measurements
- Very light, easy to handle
- May effect by moisture

2. Metric steel tape

- Made of steel
- Outer end is provided with a ring for holding

3. Invar tape

- Used for high precision work
- Made of alloy steel

4. Synthetic tape

- Made of glass fiber with PVC coating
- These are used for short measurements

Corrections. for Baseline

It is necessary to apply the following corrections to the field measurements of base line order its true length:

1.Correction for absolute length Adverti 2.Correction for temperature

3.Correction of tension or pull
4.Correction for Sag
5.Correction for slope of vertica

It may be noted that each section of a base line is separately corrected.

1. Baseline Correction for Absolute Length:

It is given by the formula

Where CA = Correction for absolute length L = measured length of base l = Nominal length of measuring unit
C = Correction to measuring unit

Sign of CA is the same as that of C

Nominal length: The designated length i.e 50 tape, 100 tape (30 m tape)

Absolute length: The actual length under specified conditions

2. Correction for Temperature:

It is given by the formula
Ct = Inf (Tm-To) -- > (B)

Where
Ct = Correction for temp

= co-efficient of thermal
Tm = Mean Temperature during

To = Temp at which the measuring is standardized

Steel = 0.0000099-0.000012/c

Steel = 0.0000055 - 0.0000070/ F

The sign of 'Ct' is plus or minus according to as 'Tm' is greater or less then 'to'

1. Correction for Pull or Tension

C_p = (Pm - Po)/AE . L -- > (c)

Where
CP = Correction for pull
Pm = Pull applied during measurement
Po = Pull at which the measurement unit (tape) is standardized
L = length measured
A = Cross-Sectional area of measuring unit
E = Modulus of elasticity of measuring unit

E steel = 21 x 105 kg / cm2

E steel = 30 x 106 /bs/in2

The sing of this correction is always plus (T) as the effect of pull is to increase the length of the tape and consequently to decrease the measured length of the base.
4. Correction for Sag:

Correction for sag is the deference in length between the are and its chord i.e b/t the curved length of the laps and the distance between the supports. It's us required only when the tape is

suspended during measurement. Since the effect of sag is to make the measured length too large, it is always subtractive.

It is given by the formula.

$Cs= L_1(WL_1^2)/24Pm^2$ -- > (D)

Where Cs = Correction for Sag

L1 = Distance b/t supports.

W = wt of tape per unit length

Pm = applied pull

W = wt of tape b/t supports.
If there are 'n' equal space per tape length,

L = nL1

Sag correction per tape length is
$C_s = nL_1(WL_1)^2/24Pm^2 = L(WL_1)^2/24Pm^2 = L(WL)^2/24nPm^2$

Total sag correction to measures length

Cs=N x Cs/ + Cs//

Where N = no of whole tape length
Cs/ = Sag correction per tape length.
Cs// = Sag correction for any tape length

Normal tension:

The normal tension of a tape is a tension which will cause the effects of pull and sag to neutralize each other. It may be obtained by equating the corrections for pull and sag

Pn = 0.0204W Rt(AE) / Rt(Pn-Po) -- > (E)

Where Pn = normal tension

W = wt of tape b/w supports

Pn is determine by trial

5. Correction for Slope or Vertical Alignment:

This correction is required when the points of supports are not exactly at the same level
L1, L2----------
= Successive length of uniform garages
B1, b2---------- = Difference of elevation b/t the extremities of each of these grades.
Cg = corr for slope

Cg = B1C1 = AC1 - AB1 = I -D

Cg =l-Rt(l^2-h^2) -- > Exact

This correction is always negative for measured length.

- If grades are given in terms in terms o

$$Cos\,\phi = \frac{D}{F}$$

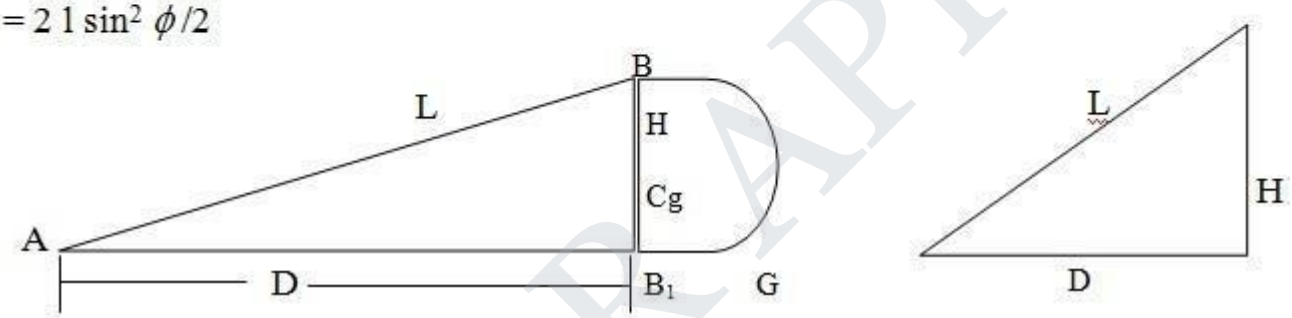
$$D = L\cos\phi$$

$$Cg = 1 - D$$

$$= 1 - 1\cos\phi$$

$$= 1\,(1 - \cos\phi)$$

$$= 2\,1\sin^2\phi/2$$



The INTERSECTION method.

This method is useful where it is not possible to measure the distances on ground as in case of a mountainous country. Hence, this method is employed for locating inaccessible points, the broken boundaries, rivers, fixing survey stations, etc. The procedure is as follows:

(i) Select two stations P and Q so that the points to be located on paper are easily seen from them.

(ii) Plot the line pq, which is known as the base line, on paper. This can be done in one of the two ways:

a. The table can be centered and leveled at station P and then after orienting at station Q, the distance PQ can be accurately measured and put up to some scale on the paper.

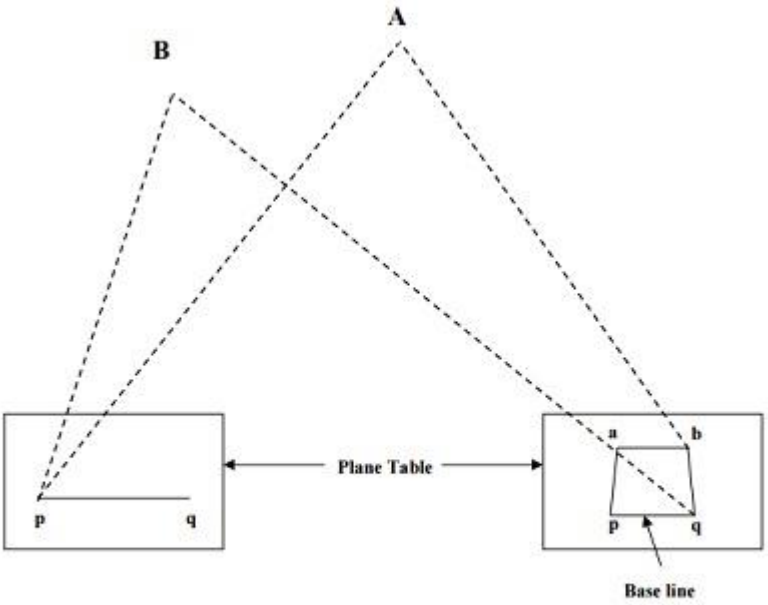
b. The line pq can be drawn to some scale on the paper and then the board can be adjusted from station P by back sighting at station Q.

(iii) From station P, draw rays for stations A, B, etc.

(iv) Shift the table to station Q and after proper orientation, take rays of stations A, B etc.,

(v) The intersection of rays from stations P and Q will give points a, b etc. on paper, as shown in figure.

For checking the accuracy of work, measure the distance AB on ground and compare it with its corresponding length ab on paper.



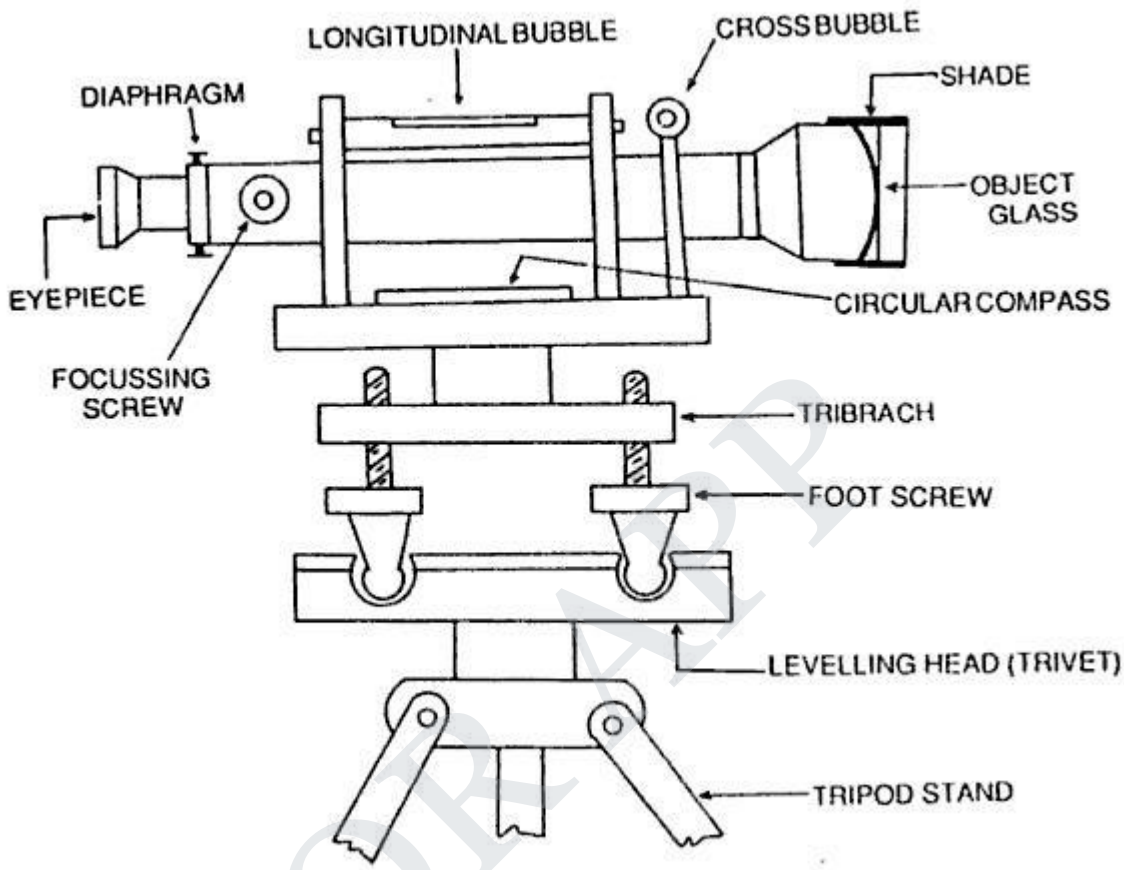
Intersection method

Different types of levels (Levelling Instrument) with neat sketches

Dumpy Level, Wye Level, Reversible Level, Tilting Level, and Digital level

1. Dumpy level:

A schematic diagram of an engineer's level is shown in. An engineer's level primarily consists of a telescope mounted upon a level bar which is rigidly fastened to the spindle. Inside the tube of the telescope, there are objective and eye piece lens at the either end of the tube. A diaphragm fitted with cross hairs is present near the eye piece end. A focussing screw is attached with the telescope. A level tube housing a sensitive plate bubble is attached to the telescope (or to the level bar) and parallel to it. The spindle fits into a cone-shaped bearing of the leveling head. The leveling head consists of tribrach and trivet with three foot screws known as leveling screws in between. The trivet is attached to a tripod stand. It is simple compact and stable. The telescope is rigidly fixed to its support therefore cannot be rotated about its longitudinal axis. A long bubble tube is attached to the top of telescope. Dumpy literally means short and thick.



Telescope : used to sight a staff placed at desired station and to read staff reading distinctly.

Diaphragm : holds the cross hairs (fitted with it).

Eye piece : magnifies the image formed in the plane of the diaphragm and thus to read staff during leveling.

Level Tube : used to make the axis of the telescope horizontal and thus the line of sight.

Leveling screws : to adjust instrument (level) so that the line of sight is horizontal for any orientation of the telescope.

Tripod stand : to fix the instrument (level) at a convenient height of an observer.

2. Wye level:

The essential difference between the dumpy level and the Wye level is that in the former case the telescope is fixed to the spindle while in the Wye level, the telescope is carried in two vertical Wye supports. The Wye support consists of curved clips.The clips are raised, the

telescope can be rotated in the Wyes, or removed and turned end for end. When the clips are fastened the telescope is held from turning about its axis by a lug on one of the clips. The bubble tube may be attached either to the telescope or to the stage carrying the wyes.

3. Reversible level:

A reversible level combines the features of bioth the dumpy level and the Wye level. The telescope is supported by two rigid sockets into which the telescope can be introduced from either end and then fixed in position by a screw. The sockets are rigidly connected to the spindle through a stage.

4. Tilting level :

It consists of a telescope attached with a level tube which can be tilted within few degrees in vertical plane by a tilting screw.

The main peculiarity of this level is that the vertical axis need not be truly vertical, since the line of collimation is not perpendicular to it. The line of collimation, is, however, made horizontal for each pointing of telescope by means of tilting screw. It is mainly designed for precise leveling work.

5. Digital level

There are fundamentally two types of automatic levels.

First, the optical one whose distinguishing feature is self-leveling i.e., the instruments gets approximately leveled by means of a circular spirit level and then it maintains a horizontal line of sight of its own.

Second, the digital levels whose distinguishing features are automatic leveling, reading and recording.

The different types of leveling

I) Direct Leveling : Direct measurement, precise, most commonly used;

Types:

(1) Simple leveling : One set up of level. To find elevation of points. When the difference of level between two points is determined by setting the leveling instrument midway between the points , the process is called simple leveling.

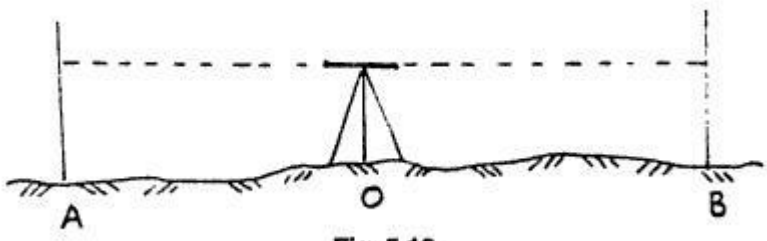


Fig. 5.10

2. Differential leveling :

Differential leveling is adopted when : (i) the points are at a great difference apart, (ii) the difference of elevation between the points is large, (iii) there are obstacles between the points. To find elevation of non-intervisible points.

This method is called compound leveling or continuous leveling.

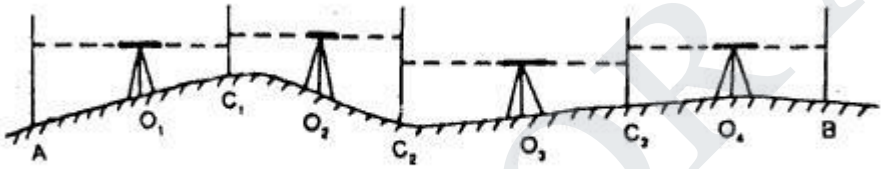
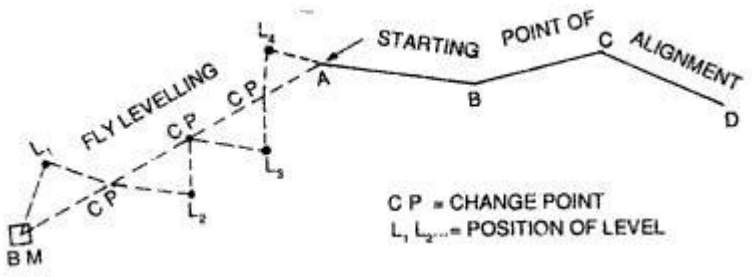


Fig. 5.11

3. Fly leveling :

When differential leveling is done in order to connect a bench mark to the starting point of the alignment of any project, it is called fly leveling. Fly leveling is done to connect the BM to any intermediate point of the alignment for checking the accuracy of the work. Only back sight and fore sight readings are taken at every set up of the level and no distances are measured along the direction of leveling.

Low precision, to find/check approximate level, generally used during reconnaissance survey.



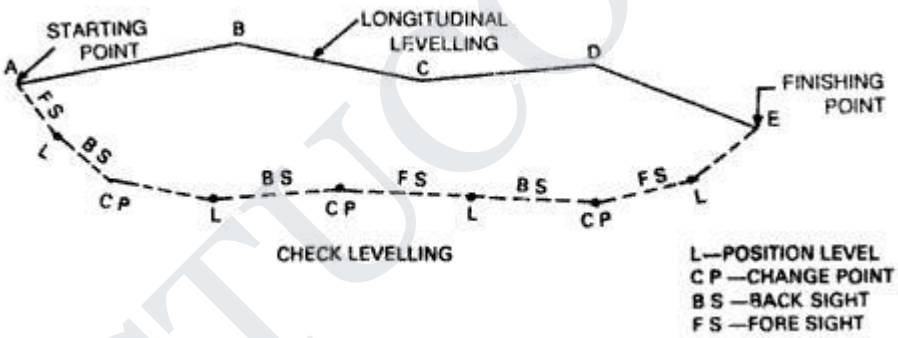
4. Precise leveling : Precise form of differential leveling.

5. Profile leveling : Finding of elevation along a line and its cross section.

6. Reciprocal leveling : Along a river or pond. Two level simultaneously used, one at either end.

7. Check leveling

The fly leveling is done at the end of day's work starting point on that particular day si known as check leveling.



(II) Indirect or Trigonometric Leveling : By measuring vertical angles and horizontal distance; Less precise.

(III) Stadia Leveling : Using tacheometric principles.

(IV) Barometric Leveling : Based on atmospheric pressure difference; Using altimeter; Very rough estimation

Differences between height of collimation method and rise and fall method

Sl.No	Height of collimation system	Rise and fall system
1	It is rapid as it involves few calculation	It is laborious involvin calcuation
2	There is no check on the RL of the intermediate sight	There is a check on the intermediate points
3	Errors in the intermediate RLs cannot be detected.	Errors in the intermediat be detected as all the correlated
4	There are two checks on the accuracy of RL calculation	There are three check accuracy of RL calculation
5	This system is suitable for longitudinal leveling where there are a number of intermediate sights	This system is suitabl leveling where there intermediate sights

There are two Methods of Levelling: 1. Height of collimation system 2. Rise and fall system

Differences between height of collimation method and rise and fall method

There are two Methods of Levelling:

1. Height of collimation system
2. Rise and fall system

Height of collimation system Vs Rise and fall system

1 It is rapid as it involves few Calculation

It is laborious involving several calcuation

2 There is no check on the RL of the intermediate sight

There is a check on the RL of the intermediate points

3 Errors in the intermediate RLs cannot be detected.

Errors in the intermediate RLs can be detected as all the points are correlated

4 There are two checks on the accuracy of RL calculation

There are three checks on the accuracy of RL calculation

5 This system is suitable for longitudinal leveling where number of intermediate sights

This system is suitable for fly there are a leveling where there are no intermediate sights

Height of collimation system

1 It is rapid as it involves few Calculation

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Rise and fall system

It is laborious involving several calcuation

There is a check on the RL of the intermediate points

Errors in the intermediate RLs can be detected as all the points are correlated

There are three checks on the accuracy of RL calculation

This system is suitable for fly there are a leveling where there are no intermediate sights

Temporary Adjustment of a Level

At each set up of a level instrument, temporary adjustment is required to be carried out prior to any staff observation. It involves some well defined operations which are required to be carried out in proper sequence.

The temporary adjustment of a dumpy level consists of (1)Setting , (2)Leveling and (3) Focusing .

During **Setting**, the tripod stand is set up at a convenient height having its head horizontal (through eye estimation). The instrument is then fixed on the head by rotating the lower part of the instrument with right hand and holding firmly the upper part with left hand. Before fixing, the leveling screws are required to be brought in between the tribrach and trivet. The bull's eye bubble (circular bubble), if present, is then brought to the centre by adjusting the tripod legs.
Next, **Leveling** of the instrument is done to make the vertical axis of the instrument truly vertical. It is achieved by carrying out the following steps:

Step 1: The level tube is brought parallel to any two of the foot screws, by rotating the upper part of the instrument.

Step 2: The bubble is brought to the centre of the level tube by rotating both the foot screws either inward or outward. (The bubble moves in the same direction as the left thumb.)

Step 3: The level tube is then brought over the third foot screw again by rotating the upper part of the instrument.

Step 4: The bubble is then again brought to the centre of the level tube by rotating the third foot screw either inward or outward.

Step 5: Repeat Step 1 by rotating the upper part of the instrument in the same quadrant of the circle and then Step 2.

Step 6: Repeat Step 3 by rotating the upper part of the instrument in the same quadrant of the circle and then Step 4.

Step 7: Repeat Steps 5 and 6, till the bubble remains central in both the positions. **Step 8:** By rotating the upper part of the instrument through 180 o , the level tube is brought parallel to first two foot screws in reverse order. The bubble will remain in the centre if the instrument is in permanent adjustment.

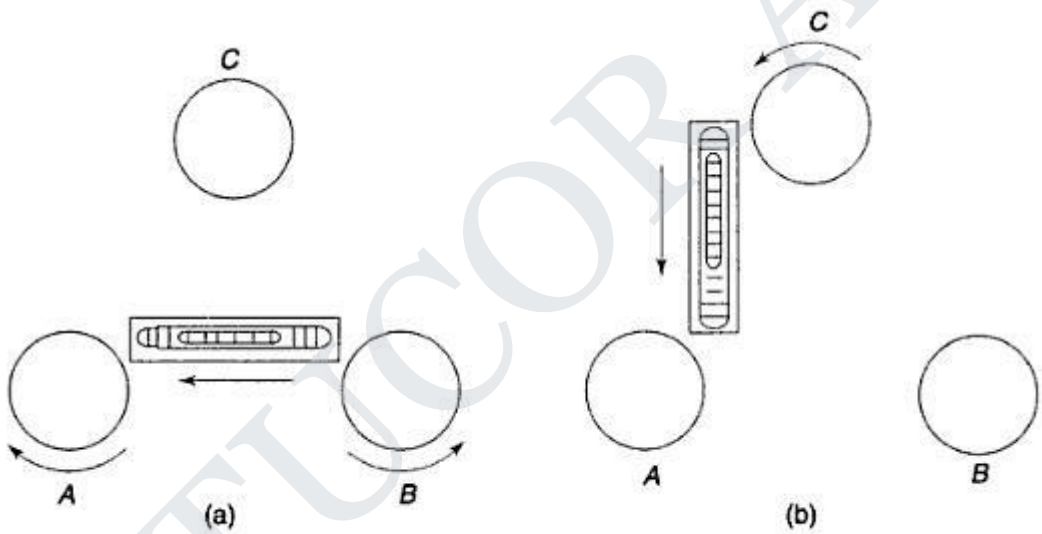


Fig. 6.22 *Levelling with three foot screws*

In the case of four foot screws the levelling is to be carried out as follows

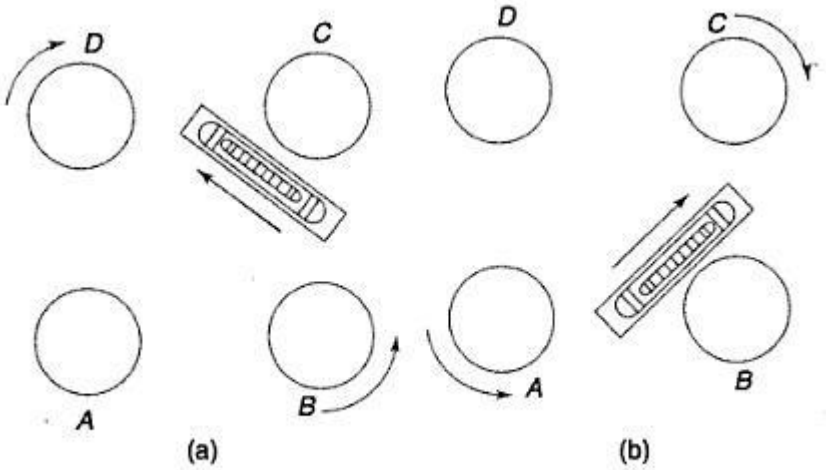


Fig. 6.23 *Levelling with four foot screws*

Focusing is required to be done in order to form image through objective lens at the plane of the diaphragm and to view the clear image of the object through eye-piece. This is being carried out by removing parallax by proper focusing of objective and eye-piece. **For focusing the eye-piece**, the telescope is first pointed towards the sky. Then the ring of eye-piece is turned either in or out until the cross-hairs are seen sharp and distinct. Focusing of eye-piece depends on the vision of observer and thus required whenever there is a change in observer.

For focusing the objective, the telescope is first pointed towards the object. Then, the focusing screw is turned until the image of the object appears clear and sharp and there is no relative movement between the image and the cross-hairs. This is required to be done before taking any observation.

Proceed profile leveling or longitudinal sectioning in the field.

Profile Leveling

Profile leveling is a method of surveying that has been carried out along the central line of a track of land on which a linear engineering work is to be constructed/ laid. The operations involved in determining the elevation of ground surface at small spatial interval along a line is called profile leveling.

Stations

The line along which the profile is to be run is to be marked on the ground before taking any observation. Stakes are usually set at some regular interval which depends on the topography, accuracy required, nature of work, scale of plotting etc. It is usually taken to be 10 meter. The beginning station of profile leveling is termed as 0+00. Points at multiples of 100m from this point are termed as full stations. Intermediate points are designated as pluses.

Procedure

In carrying out profile leveling, a level is placed at a convenient location (say I1) not necessarily along the line of observation. The instrument is to be positioned in such a way that first backsight can be taken clearly on a B.M. Then, observations are taken at regular intervals (say at 1, 2, 3, 4) along the central line and foresight to a properly selected turning point (say TP1). The instrument is then re-positioned to some other convenient location (say I2). After proper adjustment of the instrument, observations are started from TP1 and then at regular intervals (say at 5, 6 etc) terminating at another turning point, say TP2 . Staff readings are also taken at salient points where marked changes in slope occur, such as that at X.

The distance as well as direction of lines are also measured.

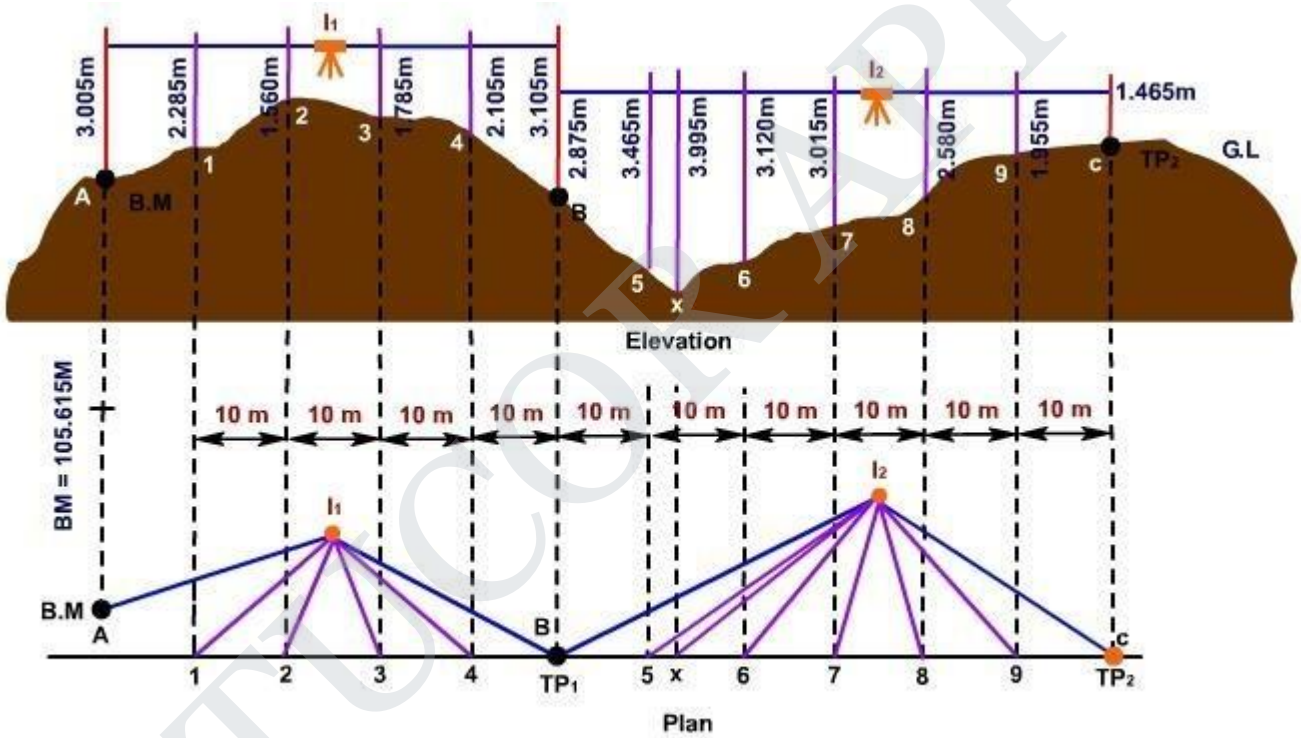


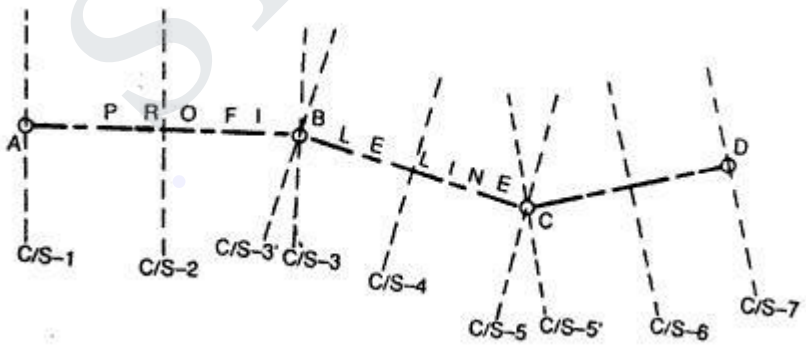
Figure 14.1 Profile Levelling

Field book for Reduction of Level

Field book for Reduction of Level

Pegs	Distance(m)	Direction	Staff Reading (m)			Difference in Elevation (m)		H.I (m)	R.L(m)	Remarks
			B.S	I.S	F.S	Rise	Fall			
A			3.005					108.620	105.615	B.M.
1	0+00			2.285		0.720			106.335	
2	0+10			1.560		0.725			107.060	
3	0+20			1.785			0.225		106.835	
4	0+30			2.105			0.320		106.515	
B	0+40		2.875		3.105		1.000	108.390	105.515	T.P.$_1$
5	0+50			3.465			0.590		104.925	
X	0+53.35			3.955			0.490		104.435	
6	0+60			3.120		0.835			105.270	
7	0+70			3.015		0.105			105.375	
8	0+80			2.580		0.435			105.810	
9	0+90			1.955		0.625			106.435	
C	1+00				1.465	0.490			106.925	T.P.$_2$
			5.880		4.570	3.935	2.625			

Cross sectioning done using a leveling instrument



Cross Sectioning

In many projects, terrain information transverse to the longitudinal section (through profile leveling) is also required such as for highways, railways, canals etc. In those cases, surveying is carried out at right angle to the central line, generally, at regular interval is being carried out and is termed as cross- sectioning. If, for any reason, a cross-section is run in any other direction, the angle with the centre line is required to be noted. The observations are then recorded as being to the left or right of the centre line. The notes of the readings are maintained as shown in for taking a cross-section along the stake point 4. Reduction of levels, Plotting etc. can be done as in case of profile leveling.

Pegs	Distance(m)	Direction	Staff reading (m)			Difference in elevation (m)		H.I (m)	R.L (m)	Remark
			B.S.	I.S.	F.S.	Rise (m)	Fall (m)			
A			3.005					108.620	105.615	B.M.
:										
4	0+30			2.105			0.320		106.515	0m
				1.850					106.770	2m left
				1.725					106.895	4m left
				1.680					106.940	6m left
				1.985					106.635	2m right
				1.875					106.745	4m right
				1.780					106.840	6m right
B	0+40		2.875		3.105		1.000	108.390	105.515	T.P.$_1$
:										

Reciprocal leveling done

In the case of an obstacle like river valley, it is not possible to set the up the level midway between two points on the opposite banks. In such cases the method of reciprocal leveling is adopted, which involv3es reciprocal observations from both banks of the river or valley. Two sets of staff readings are taken by holding the staff on both banks. In this case it is found that the errors are completely eliminated and the true difference of level is equal to the mean of the two apparent differences of level. The principle is explained as follows.

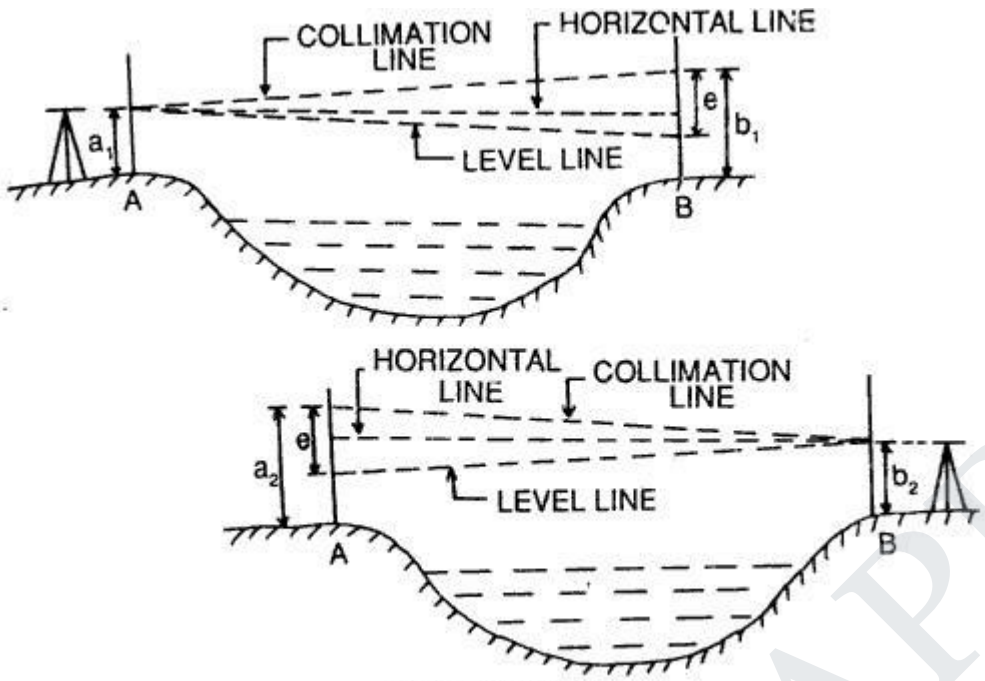


Fig. 5.23 (a) & (b)

up very near a and after proper temporary adjustment , staff readings are taken at A and B. Suppose the readings are a1 and b1.

The level is shifted and set up very near B and after proper adjustment , staff readings are taken aat A and B .Suppose the readings are a2 and b2.

Let h = true difference of level between A and B

e = Combined error due to curvature , refraction and collimation

Uses of contours maps

Contours provide valuable information about the nature of terrain. This is very important for selection of sites, determination of catchment area of a drainage basin, to find intervisibility between stations etc. Some of the salient uses of contours are described below

Nature of Ground

To visualize the nature of ground along a cross section of interest,

To Locate Route

Contour map provides useful information for locating a route at a given gradient such as highway, canal, sewer line etc.

Intervisibility between Stations

When the intervisibility between two points can not be ascertained by inspection of the area, it can be determined using contour map.

To Determine Catchment Area or Drainage Area

The catchment area of a river is determined by using contour map. The watershed line which indicates the drainage basin of a river passes through the ridges and saddles of the terrain around the river. Thus, it is always perpendicular to the contour lines. The catchment area contained between the watershed line and the river outlet is then measured with a planimeter

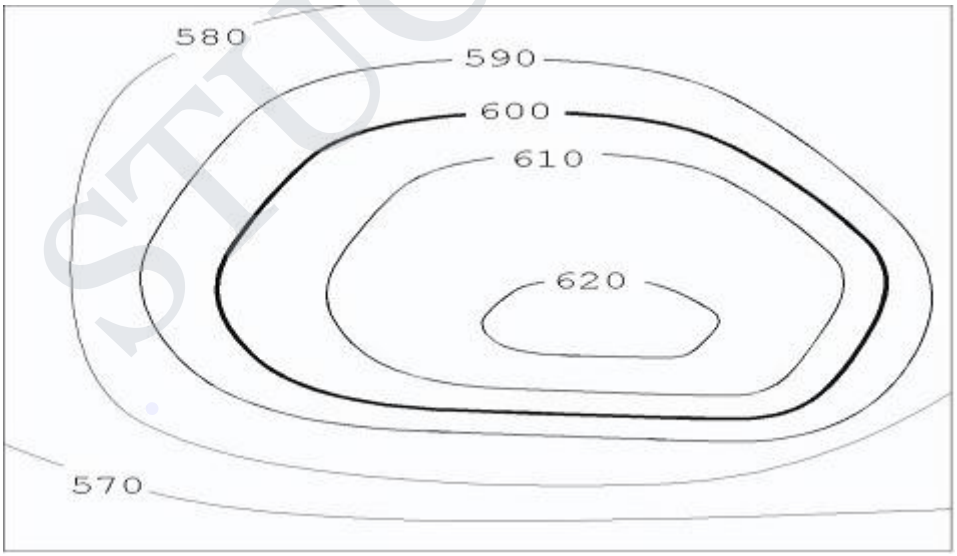
Storage capacity of a Reservoir

The storage capacity of a reservoir is determined from contour map. The contour line indicating the full reservoir level (F.R.L) is drawn on the contour map. The area enclosed between successive contours are measured by planimeter .The volume of water between F.R.L and the river bed is finally estimated by using either Trapezoidal formula or Prismoidal formula.

Characteristics of Contour

The principal characteristics of contour lines which help in plotting or reading a contour map are as follows:

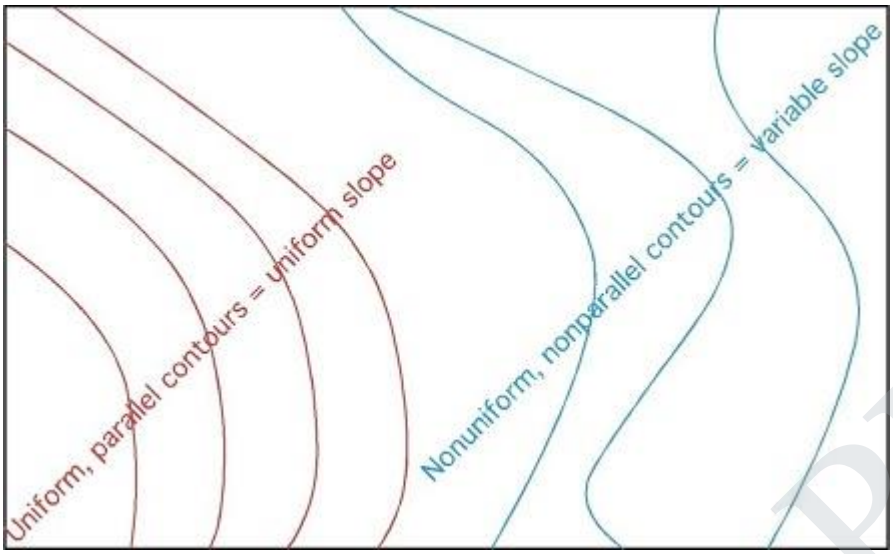
1. The variation of vertical distance between any two contour lines is assumed to be uniform. Contours are continuous.



(Fig: Contours are continuous)

The horizontal distance between any two contour lines indicates the amount of slope and varies inversely on the amount of slope. Thus, contours are spaced equally for

uniform slope ; closely for steep slope contours; widely for moderate slope



(Fig: Slope)

3. The steepest slope of terrain at any point on a contour is represented along the normal of the contour at that point . They are perpendicular to ridge and valley lines where they cross such lines.

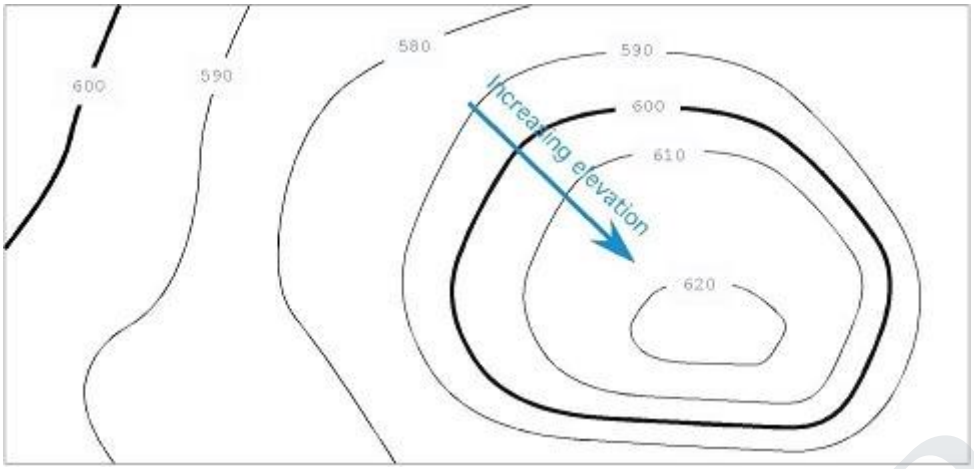
4. Contours do not pass through permanent structures such as buildings
5. Contours of different elevations cannot cross each other (caves and overhanging cliffs are the exceptions).

6. Contours of different elevations cannot unite to form one contour (vertical cliff is an exception).

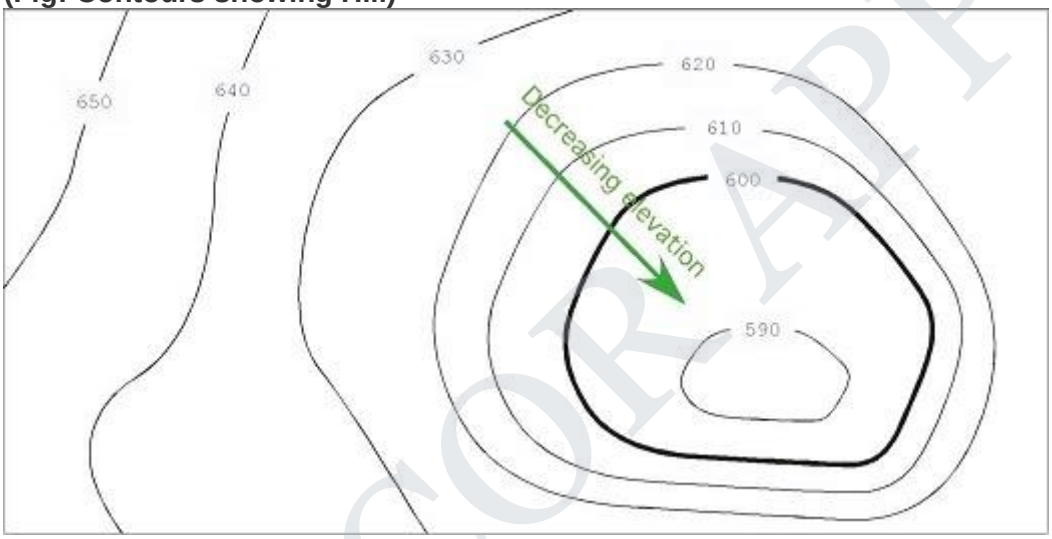
7. Contour lines cannot begin or end on the plan.

8. A contour line must close itself but need not be necessarily within the limits of the map.

A closed contour line on a map represents either depression or hill . A set of ring contours with higher values inside, depicts a hill whereas the lower value inside, depicts a depression (without an outlet).

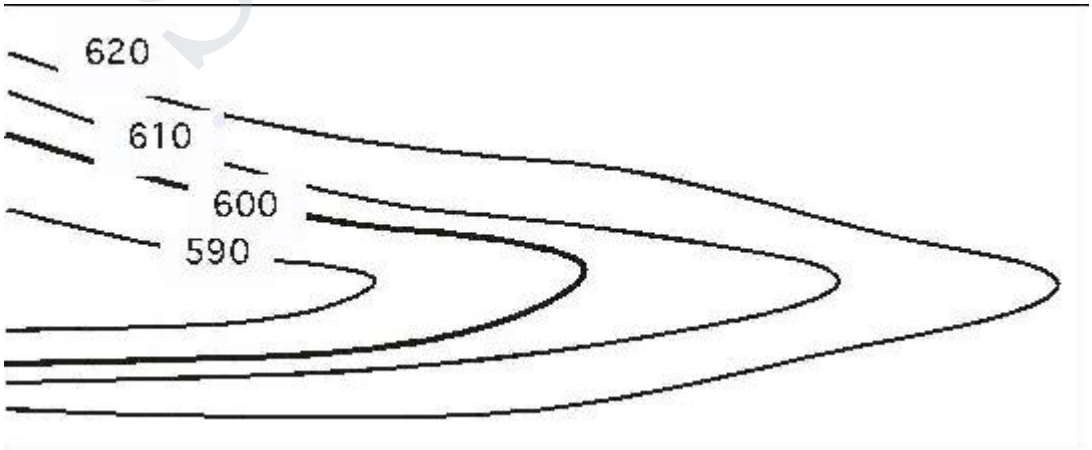


(Fig: Contours showing Hill)

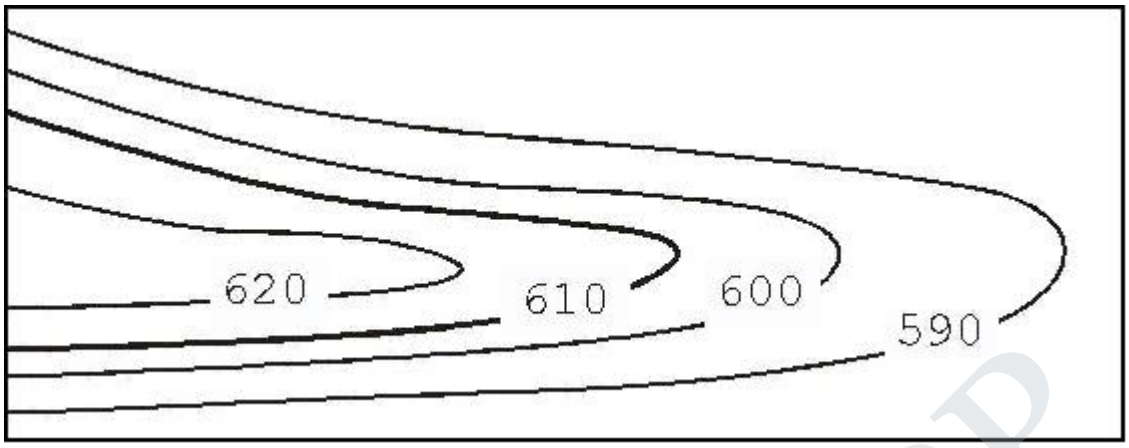


(Fig: Contours showing Depression)

10. Contours deflect uphill at valley lines and downhill at ridge lines. Contour lines in U-shape cross a ridge and in V-shape cross a valley at right angles. The concavity in contour lines is towards higher ground in the case of ridge and towards lower ground in the case of valley



(Fig: Contours showing valley)



(Fig: Contours showing Ridge)

7. Levelling across a rising ground or depression While levelling across high ground, the level should not be placed on top of this high ground, but on one side so that the line of collimation just passes through the apex.

While levelling across a depression, the level should be set up on one side and not at the bottom of the depression (Figs 5.35(a) and (b)).

UNIT II THEODOLITE AND TACHEOMETRIC SURVEYING

THEODOLITE

A theodolite is essentially a transit of high precision. Theodolites come in different sizes and weights and from different manufacturers. Although theodolites may differ in appearance, they are basically alike in their essential parts and operation. Some of the models currently available for use in the military are WILD (Herrbrugg), BRUNSON, K&E, (Keuffel & Esser), and PATH theodolites.

To give you an idea of how a theodolite differs from a transit, we will discuss some of the most commonly used theodolites in the U.S. Armed Forces.

One-Minute Theodolite

The 1-min directional theodolite is essentially a directional type of instrument. This type of instrument can be used, however, to observe horizontal and vertical angles, as a transit does.

The theodolite shown in figure 11-12 is a compact, lightweight, dustproof, optical reading instrument. The scales read directly to the nearest minute or 0.2 mil and are illuminated by either natural or artificial light. The main or essential parts of this type of theodolite are discussed in the next several paragraphs.

HORIZONTAL MOTION

Located on the lower portion of the alidade, and adjacent to each other, are the horizontal motion clamp and tangent screw used for moving the theodolite in azimuth. Located on the horizontal circle casting is a horizontal circle clamp that fastens the circle to the alidade. When this horizontal (repeating) circle clamp is in the lever-down position, the horizontal circle turns with the telescope. With the circle clamp in the lever-up position, the circle is unclamped and the telescope turns independently. This combination permits use of the theodolite as a REPEATING INSTRUMENT. To use the theodolite as a DIRECTIONAL TYPE OF INSTRUMENT, you should use the circle clamp only to set the initial reading. You should set an initial reading of 0 o 30 on the plates when a direct and reverse (D/R) pointing is required. This will minimize the possibility of ending the D/R pointing with a negative value.

VERTICAL MOTION

.- Located on the standard opposite the vertical circle are the vertical motion clamp and tangent screw. The tangent screw is located on the lower left and at right angles to the clamp. The telescope can be rotated in the vertical plane completely around the axis (360 o).

LEVELS.- The level vials on a theodolite are the circular, the plate, the vertical circle, and the telescope level. The CIRCULAR LEVEL is located on the tribrach of the instrument and is used to roughly level the instrument. The PLATE LEVEL, located between the two standards, is used for leveling the instrument in the horizontal plane. The VERTICAL CIRCLE LEVEL (vertical collimation) vial is often referred to as a split bubble. This level vial is completely built in, adjacent to the vertical circle, and viewed through a prism and 45^0

mirror system from the eyepiece end of the telescope. This results in the viewing of one-half of each end of the bubble at the same time. Leveling consists of bringing the two halves together into exact coincidence, as

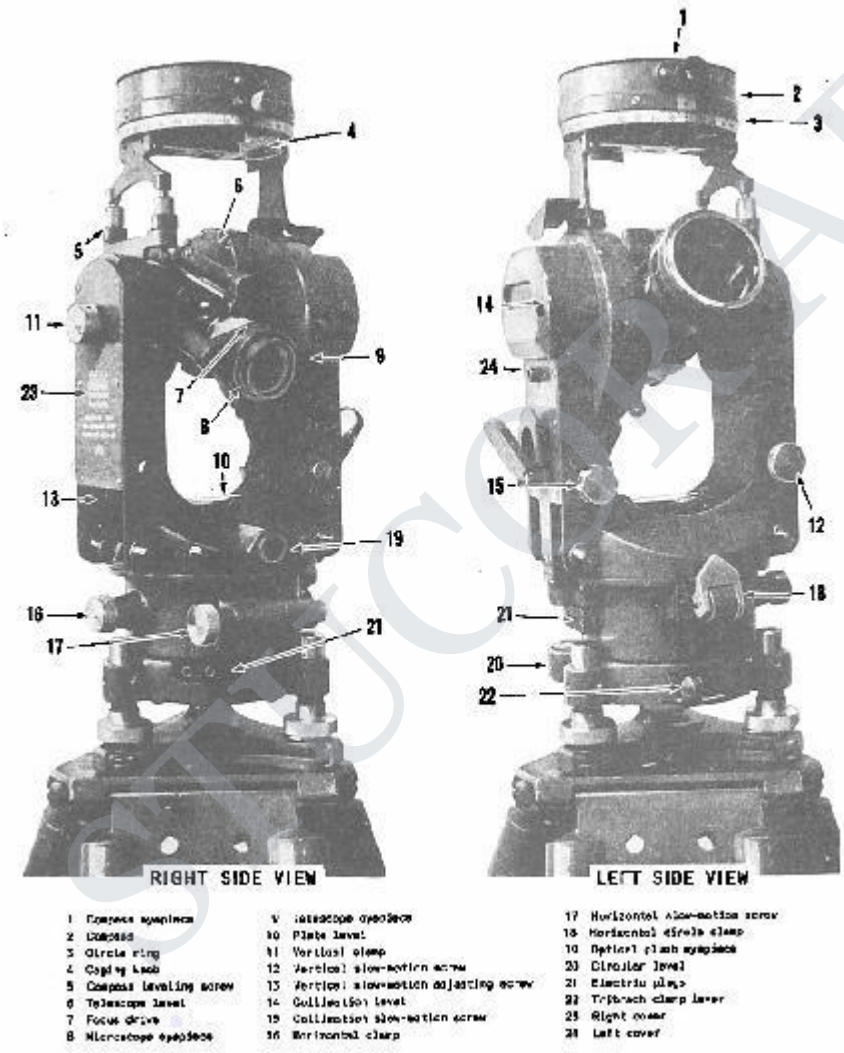


Figure 11-12.-One-minute theodolite.

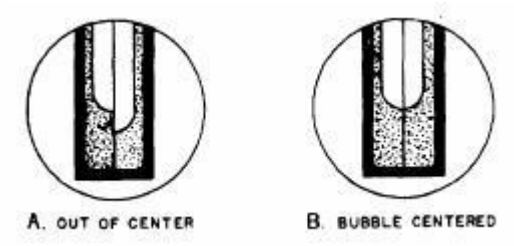


Figure 11-13.-Coincidence- type level.

shown in figure 11-13. The TELESCOPE LEVEL, mounted below the telescope, uses a

prism system and a 45^0 mirror for leveling operations. When the telescope is plunged to the reverse position, the level assembly is brought to the top.

TELESCOPE.- The telescope of a theodolite can be rotated around the horizontal axis for direct and reverse readings. It is a 28-power instrument with the shortest focusing distance of about 1.4 meters. The cross wires are focused by turning the eyepiece; the image, by turning the focusing ring. The reticle (fig. 11-14) has horizontal and vertical cross wires, a set of vertical and horizontal ticks (at a stadia ratio of 1:100), and a solar circle on the reticle for

making solar observations. This circle covers 31 min of arc and can be imposed on the sun's image (32 min of arc) to make the pointing refer to the sun'scenter. One-half of the vertical line is split for finer centering on small distant objects.

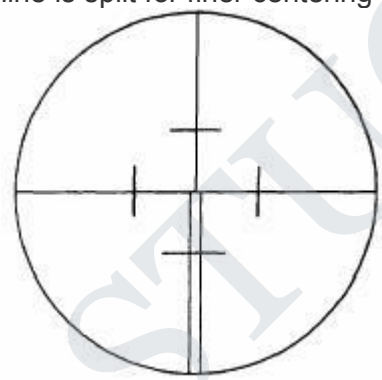


Figure 11-14.-Theodolite reticle.

The telescope of the theodolite is an inverted image type. Its cross wires can be illuminated by either sunlight reflected by mirrors or by battery source. The amount of illumination for the telescope can be adjusted by changing the position of the illumination mirror.

TRIBRACH.- The tribrach assembly (fig. 11-15), found on most makes and models, is a detachable part of the theodolite that contains the leveling screw, the circular level, and the optical plumbing device. A locking device holds the alidade and the tribrach together and permits interchanging of instruments without moving

the tripod. In a "leapfrog" method, the instrument (alidade) is detached after observations are completed. It is then moved to the next station and another tribrach. This procedure reduces the amount of instrument setup time by half.

CIRCLES.- The theodolite circles are read through an optical microscope. The eyepiece is located to the right of the telescope in the direct position, and to the left, in the reverse. The microscope consists of a series of lenses and prisms that bring both the horizontal and the

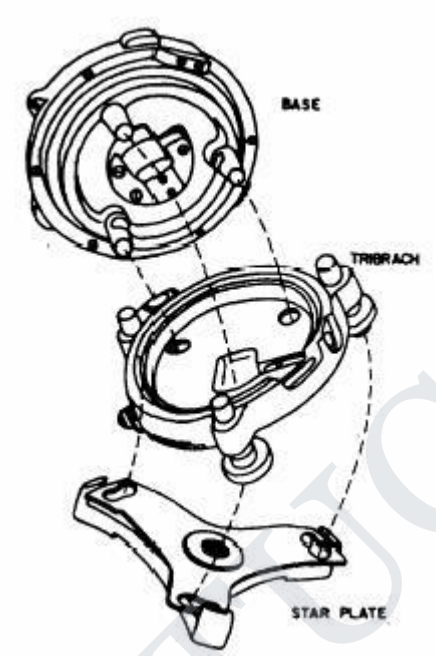


Figure 11-15.-Three-screw leveling head.

vertical circle images into a single field of view. In the DEGREE-GRADUATED SCALES (fig. 11-16), the images of both circles are shown as they would appear through the microscope of the 1-min theodolite. Both circles are graduated from 0 o to 360 o with an index graduation for each degree on the main scales. This scale'sgraduation appears to be superimposed over an auxiliary that is graduated in minutes to cover a span of 60 min (1 o). The position of the degree mark on the auxiliary scale is used as an index to get a direct reading in degrees and minutes. If necessary, these scales can be interpolated to the nearest 0.2 min of arc.

The vertical circle reads 0 o when the theodolite'stelescope is pointed at the zenith, and 180 o

when it is pointed straight down. A level line reads 90 o in the direct position and 270^0 in the reverse. The values read from the vertical circle are referred to as ZENITH DISTANCES and not vertical angles. Figure 11-17 shows how these zenith distances can be converted into vertical angles.

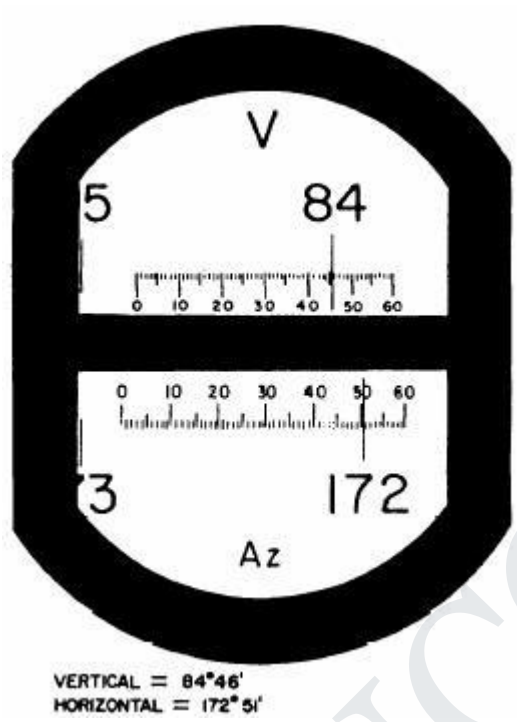


Figure 11-16.-Degree-graduated scales.

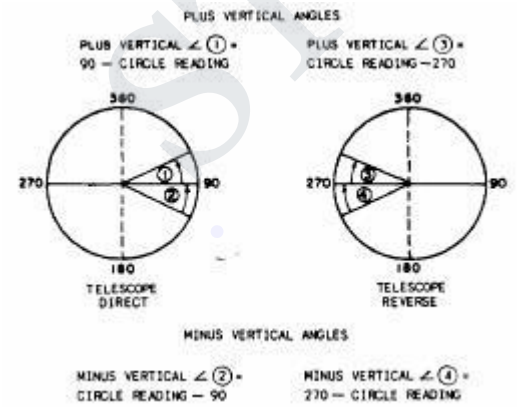


Figure 11-17.-Converting zenith distances into vertical angles (degrees).

In the MIL-GRADUATED SCALES (fig. 11-18), the images of both circles are shown as they would appear through the reading micro-scope of the 0.2-mil theodolite. Both circles are graduated from 0 to 6,400 mils. The main scales are marked and numbered every 10 mils, with the

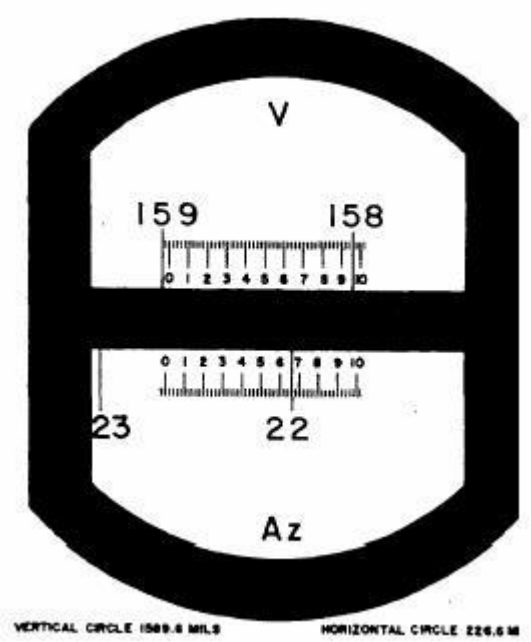


Figure 11-18.-Mil-graduated scales.

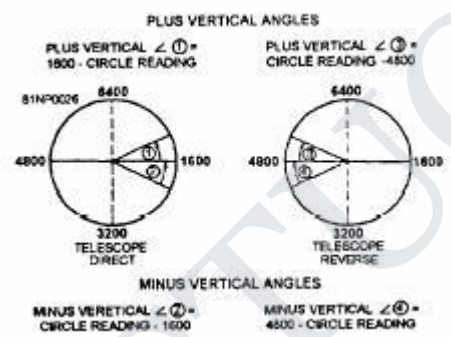


Figure 11-19.-Vertical angles from zenith distances (mils).

last zero dropped. The auxiliary scales are graduated from 0 to 10 roils in 0.2-mil increments. Readings on the auxiliary scale can be interpolated to 0.1 mil. The vertical circle reads 0 mil when the telescope is pointed at the zenith, and 3,200 mils when it is pointed straight down. A level line reads 1,600 roils in the direct position and 4,800 roils in the reverse. The values read are zenith distances. These zenith distances can be converted into vertical angles as shown in figure 11-1

The excavation of material in underwater areas is called dredging, and a dredge is an excavator afloat on a barge. A dredge may get itself into position by cross bearings, taken from the dredge on objects of known location on the beach, or by some other piloting method.

Many times, however, dredges are positioned by survey triangulation. The method of determining direction angles from base line control points is the same as that just described.

LAND SURVEYING

Land surveying includes surveys for locating and monumenting the boundaries of a property; preparation of a legal description of the limits of a property and of the area included; preparation of a property map; resurveys to recover and remonument property corners; and surveys to subdivide property. It is sometimes necessary to retrace surveys of property lines, to reestablish lost or obliterated corners, and to make ties to property lines and corners; for example, a retracement survey of property lines may be required to assure that the military operation of quarry excavation does not encroach on adjacent property where excavation rights have not been obtained. Similarly, an access road from a public highway to the quarry site, if it crosses privately owned property, should be tied to the property lines that are crossed so that correctly executed easements can be obtained to cross the tracts of private property.

EAs may be required to accomplish property surveys at naval activities outside the continental limits of the United States for the construction of naval bases and the restoration of such properties to property owners. The essentials of land surveying as practiced in various countries are similar in principle. Although the principles pertaining to the surveys of public and private lands within the United States are not necessarily directly applicable to foreign countries, a knowledge of these principles will enable the EA to conduct the survey in a manner required by the property laws of the nation concerned.

In the United States, land surveying is a survey conducted for the purpose of ascertaining the correct boundaries of real estate property for legal purposes. In accordance with federal and states laws, the right and/or title to landed property in the United States can be transferred from one person to another only by means of a written document, commonly called a deed. To constitute a valid transfer, a deed must meet a considerable number of legal requirements, some of which vary in different states. In all the states, however, a deed must contain an accurate description of the boundaries of the property.

A right in real property need not be complete, outright ownership (called fee simple). There are numerous lesser rights, such as leasehold (right to occupancy and use for a specified term) or easement (right to make certain specified use of property belonging to someone else). But in any case, a valid transfer of any type of right in real property usually involves an accurate description of the boundaries of the property.

As mentioned previously, the EA may be required to perform various land surveys. As a survey team or crew leader, you should have a knowledge of the principles of land surveys in order to plan your work accordingly.

PROPERTY BOUNDARY DESCRIPTION

A parcel of land may be described by metes and bounds, by giving the coordinates of the property corners with reference to the plane coordinates system, by a deed reference to a description in a previously recorded deed, or by References to block and individual property numbers appearing on a recorded map.

By Metes and Bounds

When a tract of land is defined by giving the bearings and lengths of all boundaries, it is said to be described by metes and bounds. This is an age-old method of describing land that still forms the basis for the majority of deed descriptions in the eastern states of the United States and in many foreign lands. A good metes-and-bounds description starts at a point of beginning that should be monumented and referenced by ties or distances from well-established monuments or other reference points. The bearing and length of each side is given, in turn, around the tract to close back on the point of beginning. Bearing may be true or magnetic grid, preferably the former. When magnetic bearings are read, the declination of the needle and the date of the survey should be stated. The stakes or monuments placed at each corner should be described to aid in their recovery in the future. Ties from corner monuments to witness points (trees, poles, boulders, ledges, or other semipermanent or permanent objects) are always helpful in relocating corners, particularly where the corner markers themselves lack permanence. In timbered country, blazes on trees on or adjacent to a boundary line are most useful in reestablishing the line at a future date. It is also advisable to state the names of abutting property owners along the several sides of the tract being described. Many metes-and-bounds descriptions fail to include all of these particulars and are frequently very difficult to retrace or locate in relation to adjoining ownerships.

One of the reasons why the determination of boundaries in the United States is often difficult is that early surveyors often confined themselves to minimal description; that is, to a bare statement of the metes metesToday, good practice requires that a land surveyor include all relevant information in his description.

In preparing the description of a property, the surveyor should bear in mind that the description must clearly identify the location of the property and must give all necessary data from which the boundaries can be reestablished at any future date. The written description contains the greater part of the information shown on the plan. Usually both a description and a plan are prepared and, when the property is transferred, are recorded according to the laws of the county concerned. The metes-and-bounds description of the property shown in figure 10-34 is given below.

"All that certain tract or parcel of land and premises, hereinafter particularly described, situate, lying and being in the Township of Maplewood in the County of Essex and State of

New Jersey and constituting lot 2 shown on the revised map of the Taylor property in said township as filed in the Essex County Hall of Records on March 18, 1944.

"Beginning at an iron pipe in the northwesterly line of Maplewood Avenue therein distant along same line four hundred and thirty-one feet and seventy- one-hundredths of a foot north-easterly from a stone monument at the northerly corner of Beach Place and Maplewood Avenue; thence running (1) North forty-four degrees thirty-one and one-half minutes West along land of. . ."

Another form of a lot description maybe presented as follows:

"Beginning at the northeasterly corner of the tract herein described; said corner being the intersection of the southerly line of Trenton Street and the westerly line of Ives Street; thence running S6 o 29$\diamond$54$\diamond\diamond$E bounded easterly by said Ives Street, a distance of two hundred and twenty-seven one hundredths (200.27) feet to the northerly line of Wickenden Street; thence turning an interior angle of 89 o 59$\diamond$16$\diamond\diamond$ and run-ning S83 o 39$\diamond$50$\diamond\diamond$W bonded southerly by said Wickenden Street, a distance of one hundred and no one-hundredths (100.00) feet to a corner; thence turn-ing an interior angle of. . ."

You will notice that in the above example, interior angles were added to the bearings of the boundary lines. This will be another help in retracing lines

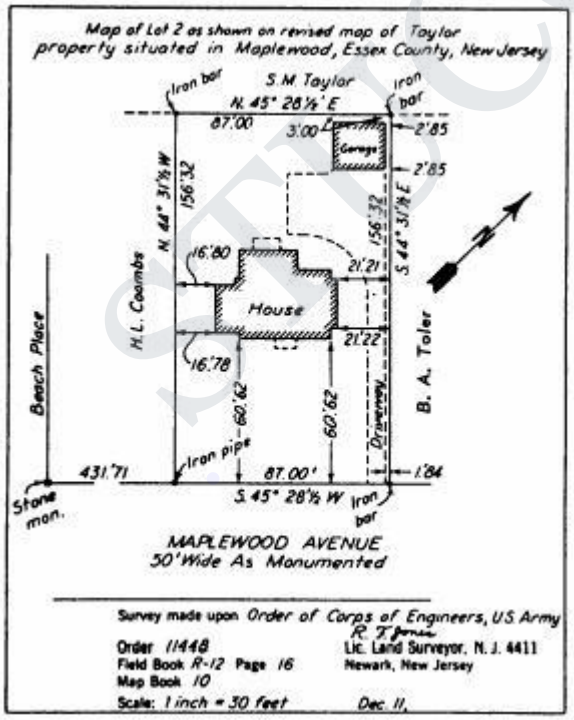


Figure 10-34.-Lot plan by metes and bounds.

INTRO TO ANTIQUE SURVEY INSTRUMENTS

First, some basics about their composition and finish... most instruments were made of wood, brass, or aluminum, although you will find whole instruments or instrument parts made of iron, steel, ebony, ivory, celluloid, and plastic. It is important to remember that many surveying instruments were "needle" instruments and their magnetic needles would not seek north properly if there were local sources of interference, such as iron. The United States General Land Office issued instructions requiring brass Gunters chains to be used in close proximity to the magnetic needle. (They soon changed that requirement to steel brazed link chains; the brass chain could not stand up to the type of wear and tear a chain received.

In American surveying instruments, wood was common until about 1800; brass instruments were made approximately 1775 to 1975, and aluminum instruments from 1885 to the present.

The finish of instruments has changed. Early wooden instruments were generally unfinished and were usually made of tight grained woods which resisted water well. Early brass instruments were usually unfinished or polished and lacquered to retain the shine. In the mid-1800s American instrument makers began finishing brass instruments with dark finishes for two reasons: first, that the dark finish reduced glare and as a result reduced eyestrain, and secondly, that the dark finish helped to even out the heating of an instrument in the sunlight and as a result reduced collimation problems caused by the heating. Beware of being taken in by polished and lacquered brass instruments; prior to 1900 that may have been the original finish for the instrument, but after 1900 , bright brass finishes are usually not original finishes.

There are three kinds of surveying instruments that are rather unique to North American surveying. They are the compass, the chain and the transit. In addition, the engineer's or surveyor's level contributed very strongly to making the United States the leading industrial nation in the world by virtue of the highly efficient railroad systems it helped design in the mid 1800's. I take a great deal of satisfaction in pointing out that in this country it was the compass and chain that won the west, not the six-shooter!

The following is a list of antique surveying instruments and tools with a brief and basic description of how they were used.

ABNEY HAND LEVEL - Measures vertical angles.

ALIDADE - Used on a Plane Table to measure vertical and horizontal angles & distances.

ALTAAZIMUTH INSTRUMENT - Measures horizontal and vertical angles; for position "fixing".

ASTRONOMIC TRANSITS - Measures vertical angles of heavenly bodies; for determining geographic position.

BAROMETER, ANEROID - Measures elevations; used to determine vertical distance.

BASE-LINE BAR - Measures horizontal distances in triangulation and trilateration surveys.

BOX SEXTANT - Measures vertical angles to heavenly bodies.

CHRONOGRAPH - Measures time.

CHRONOMETER - Measures time.

CIRCUMFERENTER - Measures horizontal directions and angles.

CLINOMETER - Measures vertical angles.

COLLIMATOR - For adjusting and calibrating instruments.

COMPASSES - Determines magnetic directions; there are many kinds, including plane, vernier, solar, telescopic, box, trough, wet, dry, mariners, prismatic, pocket, etc.

CROSS, SURVEYORS - For laying out 90 and 45 degree angles.

CURRENT METER - Measures rate of water flow in streams and rivers.

DIAL, MINER'S - A theodolite adapted for underground surveying; measures directions as well as horizontal and vertical angles.

GONIOMETER - Measures horizontal and vertical angles.

GRADIOMETER - Also known as Gradiometer level, it measures slight inclines and level lines-of-sight.

HELIOGRAPH - Signalling device used in triangulation surveys.

HELIOSTAT - Also known as a heliotrope, it was used to make survey points visible at long distances, particularly in triangulation surveys.

HORIZON, ARTIFICIAL - Assists in establishing a level line of sight, or "horizon".

HYPSOMETER - Used to estimate elevations in mountainous areas by measuring the boiling points of liquids. This name was also given to an instrument which determined the heights of trees.

INCLINOMETER - Measures slopes and/or vertical angles.

LEVEL - Measures vertical distances (elevations). There are many kinds, including Cooke's, Cushing's, Gravatt. dumpy, hand or pocket, wye, architect's, builder's, combination, water, engineer's, etc.

LEVELLING ROD - A tool used in conjunction with a levelling instrument.

LEVELLING STAVES - Used in measuring vertical distances.

MINER'S COMPASS - Determines magnetic direction; also locates ore.

MINER'S PLUMMET - A "lighted" plumb bob, used in underground surveying.

MINING SURVEY LAMP - Used in underground surveying for vertical and horizontal alignment.

OCTANT - For measuring the angular relationship between two objects.

PEDOMETER - Measures paces for estimating distances.

PERAMBULATOR - A wheel for measuring horizontal distances.

PHOTO-THEODOLITE - Determines horizontal and vertical positions through the use of "controlled" photographs.

PLANE TABLE - A survey drafting board for map-making with an alidade.

PLUMB BOB - For alignment; hundreds of varieties and sizes.

PLUMMETS - Same as plumb bob.

QUADRANT - For measuring the angular relationship between two objects.

RANGE POLES - For vertical alignment and extending straight lines.

SEMICIRCUMFERENTER - Measures magnetic directions and horizontal angles.

SEXTANTS - Measures vertical angles; there are many kinds, including box, continuous arc, sounding, surveying, etc.

SIGNAL MIRRORS - For communicating over long distances; used in triangulation surveys.

STADIA BOARDS - For measuring distances; also known as stadia rods.

STADIMETER or STADIOMETER - For measuring distances.

TACHEOMETER - A form of theodolite that measures horizontal and vertical angles, as well as distances.

TAPES - For measuring distances; made of many materials, including steel, invar, linen, etc. Also made in many styles, varieties, lengths, and increments.

THEODOLITE - Measures horizontal and vertical angles. Its name is one of the most misused in surveying instrument nomenclature, and is used on instruments that not only measure angles, but also directions and distances. There are many kinds, including transit, direction, optical, solar, astronomic, etc.

TRANSIT - For measuring straight lines. Like the theodolite, the transit's name is often misused in defining surveying instruments. Most transits were made to measure horizontal and vertical angles and magnetic and true directions. There are many kinds, including astronomic, solar, optical, vernier, compass, etc.

WAYWISER - A wheel for measuring distances

Traverse (surveying)

Traverse is a method in the field of surveying to establish control networks. It is also used ingeodetic work. Traverse networks involved placing the survey stations along a line or path of travel, and then using the previously surveyed points as a base for observing the next point. Traverse networks have many advantages of other systems, including:

- Less reconnaissance and organization needed;
- While in other systems, which may require the survey to be performed along a rigid polygon shape, the traverse can change to any shape and thus can accommodate a great deal of different terrains;
- Only a few observations need to be taken at each station, whereas in other survey networks a great deal of angular and linear observations need to be made and considered;
- Traverse networks are free of the strength of figure considerations that happen n triangular systems;

- Scale error does not add up as the traverse is performed. Azimuth swing errors can also be reduced by increasing the distance between stations.

The traverse is more accurate than triangulateration (a combined function of the triangulation and trilateration practice).

Types

Frequently in surveying engineering and geodetic science, control points (CP) are setting/observing distance and direction (bearings, angles, azimuths, and elevation). The CP throughout the control network may consist of monuments, benchmarks, vertical control, etc.

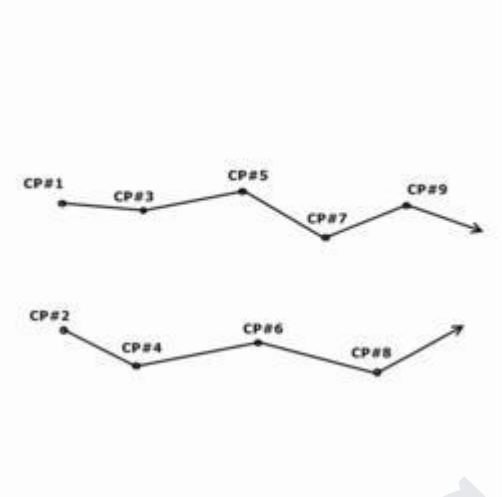


Diagram of an open traverse

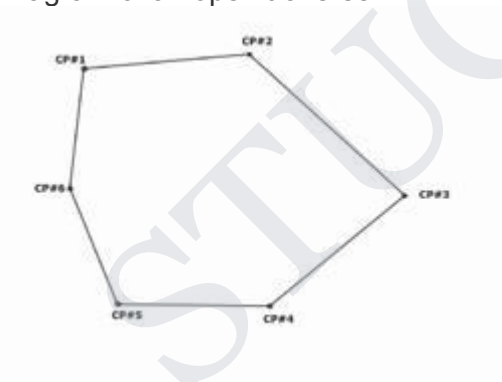


Diagram of a closed traverse

Open/Free

An open, or free traverse (link traverse) consist of known points plotted in any corresponding linear direction, but do not return to the starting point or close upon a point of equal or greater order accuracy. It allows geodetic triangulation for sub-closure of three known points; known as the "Bowditch rule" or "compass rule" in geodetic science and surveying, which is the

principle that the linear error is proportional to the length of the side in relation to the perimeter of the traverse

- Open survey is utilised in plotting a strip of land which can then be used to plan a route in road construction. The terminal (ending) point is always listed as *unknown* from the observation point.

Closed

A closed traverse (polygonal, or loop traverse) is a practice of traversing when the terminal point closes at the starting point. The control points may envelop, or are set within the boundaries, of the control network. It allows geodetic triangulation for sub-closure of all known observed points.

- Closed traverse is useful in marking the boundaries of wood or lakes. Construction and civil engineers utilize this practice for preliminary surveys of proposed projects in a particular designated area. The terminal (ending) point closes at the starting point.

- **Control point -** the primary/base control used for preliminary measurements; it mayconsist of any known point capable of establishing accurate control of distance and direction (i.e. coordinates, elevation, bearings, etc).

1. ***Starting -***It is the initial starting control point of the traverse.

2. ***Observation -***All known control points that are setted or observed within the traverse.

3. ***Terminal -***It is the initial ending control point of the traverse; its coordinates are*unknown*

Earthwork Computations

Computing earthwork volumes is a necessary activity for nearly all construction projects and is often accomplished as a part of route surveying, especially for roads and highways. Suppose, for example, that a volume of cut must be removed between two adjacent stations along a highway route. If the area of the cross section at each station is known, you can compute the average-end area (the average of the two cross-sectional areas) and then multiply that average end area by the known horizontal distance between the stations to determine the volume of cut.

To determine the area of a cross section easily, you can run a planimeter around the plotted outline of the section. Counting the squares, explained in chapter 7 of this traman, is another way to determine the area of a cross section. Three other methods are explained below.

AREA BY RESOLUTION.- Any regular or irregular polygon can be resolved into easilycalculable geometric figures, such as triangles and*ABH* and *DFE,* and two trapezoids, *BCGH* and*CGFD.* For each of these figures, the approximate dimensions have been determined by the scale of the plot. From your knowledge of mathematics, you know that the area of each triangle can be determined using the following formula:

$A\ [s(s-a)(s-b)(s-c)]^{1/2}$

s = one half of the perimeter of the triangle,

and that for each trapezoid, you can calculate the area using the formula: Where:
$A = \frac{1}{2}(b_1+b_2)h$

When the above formulas are applied and the sum of the results are determined, you find that the total area of the cross section at station 305 is 509.9 square feet.

AREA BY FORMULA.- A regular section area for a three-level section can be more exactly determined by applying the following formula:

Figure 10-4.-A cross section plotted on cross-section paper.

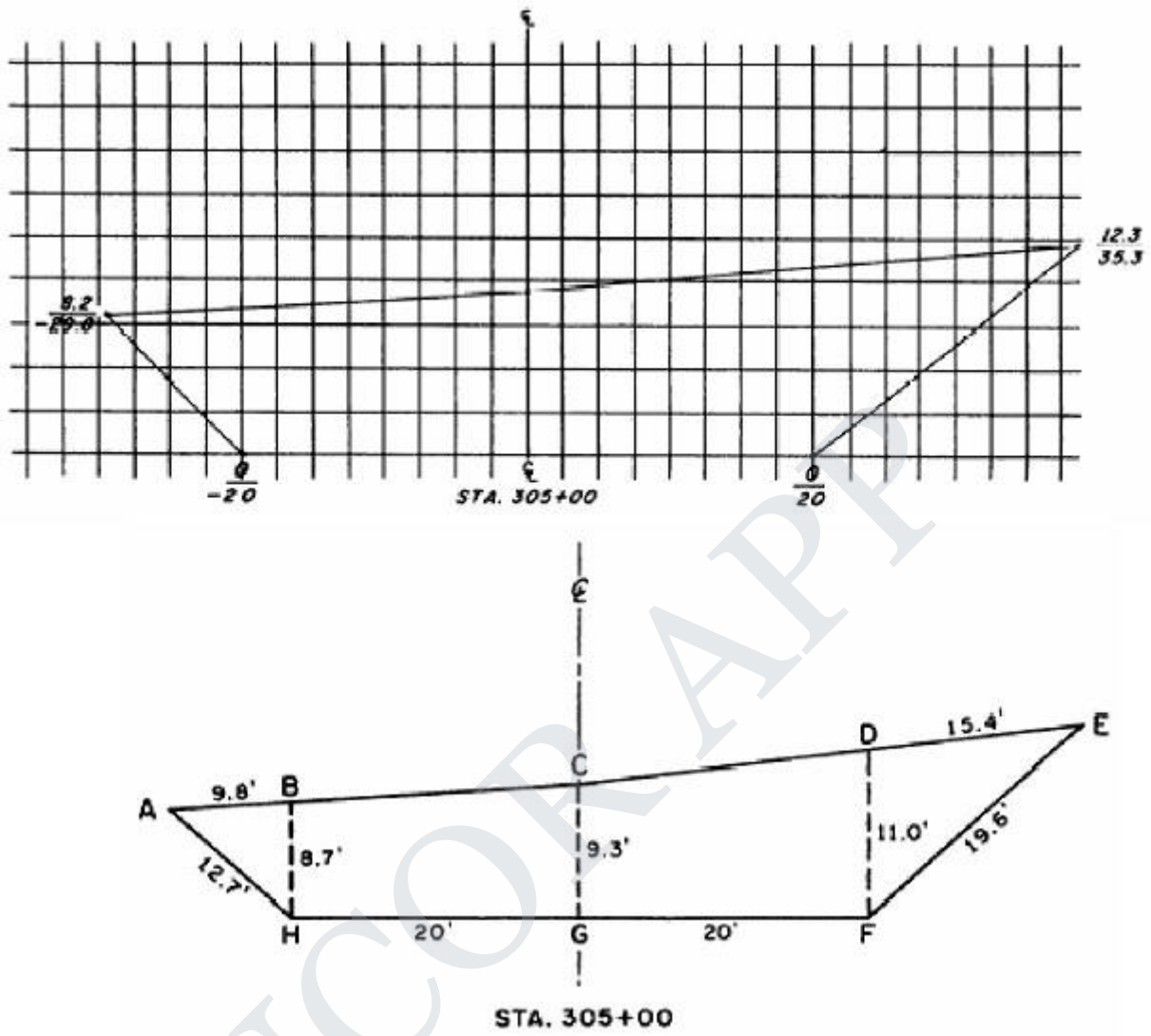


Figure10-6 forirregularsections

determine the area of sections of this kind, you should use a method of determining area by coordinates. For explanation purpose, let'sconsider station 305 (fig. 10-6). First, consider the point where the center line intersects the grade line as the point of origin for the coordinates. Vertical distances above the grade line are positive *Y* coordinates; vertical distances below the grade line are negative *Y* coordinates. A point on the grade line itself has a *Y* coordinate of 0. Similarly, horizontal distances to the right of the center line are positive *X* coordinates; distances to the left of the center line are negative *X* coordinates; and any point on the center line itself has an *X*coordinate of 0.

Plot the cross section, as shown in figure 10-7, and be sure that the *X* and *Y* coordinates have their proper signs. Then, starting at a particular point and going successively in a clockwise direction, write down the coordinates, as shown in figure 10-8.

After writing down the coordinates, you then multiply each **upper** term by the **algebraic**difference of the **following** lower term and the **preceding** lower term, as indicated by the direction of the arrows (fig. 10-8). The algebraic sum of the resulting products is the **double**area of the cross section. Proceed with the computation as follows:

Figure 10-5.-Cross section resolved into triangles and trapezoids.

$$A=w/4 \cdot (h_1+h_2) \quad + \quad C/2.(d_1+d_2)$$

the formula for station 305 + 00 (fig. 10-4), you get the following results:

A = ***(40/4)(8.2+ 12.3)+ (9.3/2)(29.8+ 35.3)= 507.71*** square feet.

AREA OF FIVE-LEVEL OR IRREGULAR SECTION.- Figures 10-6 and 10-7 are thefield notes and plotted cross sections for two irregular sections. To

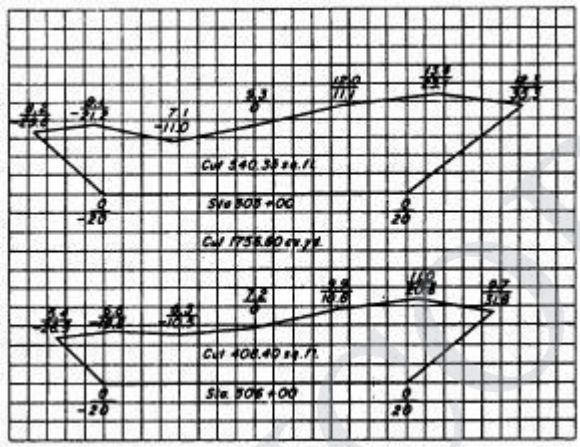


Figure 10-7.-Cross-section plots of stations 305 and 306 noted in figure 10-6.

Figure 10-6.-Field notes for irregular sections.

determine the area of sections of this kind, you should use a method of determining area by coordinates. For explanation purpose, let'sconsider station 305 (fig. 10-6). First, consider the point where the center line intersects the grade line as the point of origin for the coordinates. Vertical distances above the grade line are positive *Y* coordinates; vertical distances below the grade line are negative *Y* coordinates. A point on the grade line itself has a *Y* coordinate of 0. Similarly, horizontal distances to the right of the center line are positive *X* coordinates; distances to the left of the center line are negative *X* coordinates; and any point on the center line itself has an *X* coordinate of 0.

Plot the cross section, as shown in figure 10-7, and be sure that the *X* and *Y* coordinates have their proper signs. Then, starting at a particular point and going successively in a clockwise direction, write down the coordinates, as shown in figure 10-8.

After writing down the coordinates, you then multiply each **upper** term by the **algebraic**difference of the **following** lower term and the **preceding** lower term, as indicated by the direction of the arrows (fig. 10-8). The algebraic sum of the resulting products is the **double**area of the cross section. Proceed with the computation as follows:

8.2[–21.2 – (–20.0)]	=		–9.8
9.1[–11.0 – (–29.8)]	=	+171.1	
7.1[0 – (–21.2)]	=	+150.5	
9.3[11.1 – (–11.0)]	=	+205.5	
12.0[23.1 – 0]	=	+277.2	
13.4[35.3 – 11.1]	=	+324.3	
12.3[20.0 – 23.1]	=		–38.1
		+1,128.6	–47.9
		– 47.9	
		1,080.7	

Since the result (1,080.70 square feet) represents the **double** area, the area of the cross section is one half of that amount, or 540.35 square feet. By similar method, the area of the cross section at station 306 (fig. 10-7) is 408.40 square feet.

EARTHWORK VOLUME.- As discussed previously, when you know the area of two cross sections, you can multiply the average of those cross-sectional areas by the known distance between them to obtain the volume of earth to be cut or filled. Consider figure 10-9 that shows the plotted cross sections of two sidehill sections. For this figure, when you multiply the average-end area (in fill) and the average-end area (in cut) by the distance between the two stations (100 feet), you obtain the estimated amount of cut and fill between the stations. In this case, the amount of space that requires filling is computed to be approximately 497.00 cubic yards and the amount of cut is about 77.40 cubic yards.

MASS DIAGRAMS.- A concern of the highway designer is economy on earthwork. Hewants to know exactly where, how far, and how much earth to move in a section of

road. The ideal situation is to balance the cut and fill and limit the haul distance. A technique for balancing cut and fill and determining the

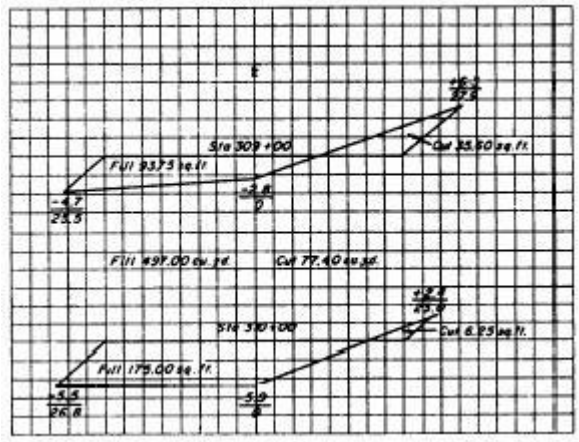


Figure 10-9.-Plots of two sidehill sections.

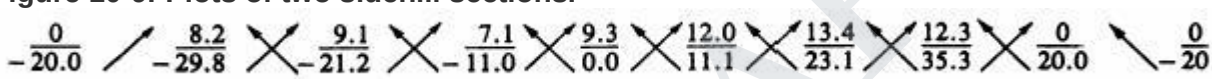


Figure 10-8.-Coordinates for cross-section station 305 shown in figure 10-7. economical haul distance is the **mass diagram** method.

A **mass diagram** is a graph or curve on which the algebraic sums of cuts and fills are plotted against linear distance. Before these cuts and fills are tabulated, the swells and compaction factors are considered in computing the yardage. Earthwork that is in place will yield more yardage when excavated and less yardage when being compacted. An example of this is sand: 100 cubic yards in place yields 111 cubic yards loose and only 95 cubic yards when compacted. Table 10-1 lists conversion factors for various types of soils. These factors should be used when you are preparing a table of cumulative yardage for a mass diagram. Cuts are indicated by a rise in the curve and are considered positive; fills are indicated by a drop in the curve and are considered negative. The yardage between any pair of stations can be determined by inspection. This feature makes the mass diagram a great help in the attempt to balance cuts and fills within the limits of economic haul.

The limit of economic haul is reached when the cost of haul and the cost of excavation become equal. Beyond that point it is cheaper to waste the cut from one place and to fill the adjacent hollow with material taken from a nearby borrow pit. The limit of economic haul will, of course, vary at different stations on the project, depending on the nature of the terrain, the availability of equipment, the type of material, accessibility, availability of manpower, and other considerations.

Components and their functions of Theodolite

A compass measures the direction by measuring the angle between the line and a reference direction, which is the magnetic meridian. A compass can measure angles up to an accuracy of 30□ and by judgement up to an accuracy of 15□. The principle of working of the compass is based on the property of the magnetic needle, which when freely suspended, takes the north-south direction. Compass measurements are thus affected by external magnetic influences and therefore a compass is unsuitable in some areas. In this here, we will discuss another method of measuring directions of lines; a theodolite is very commonly used to measure angles in survey work.

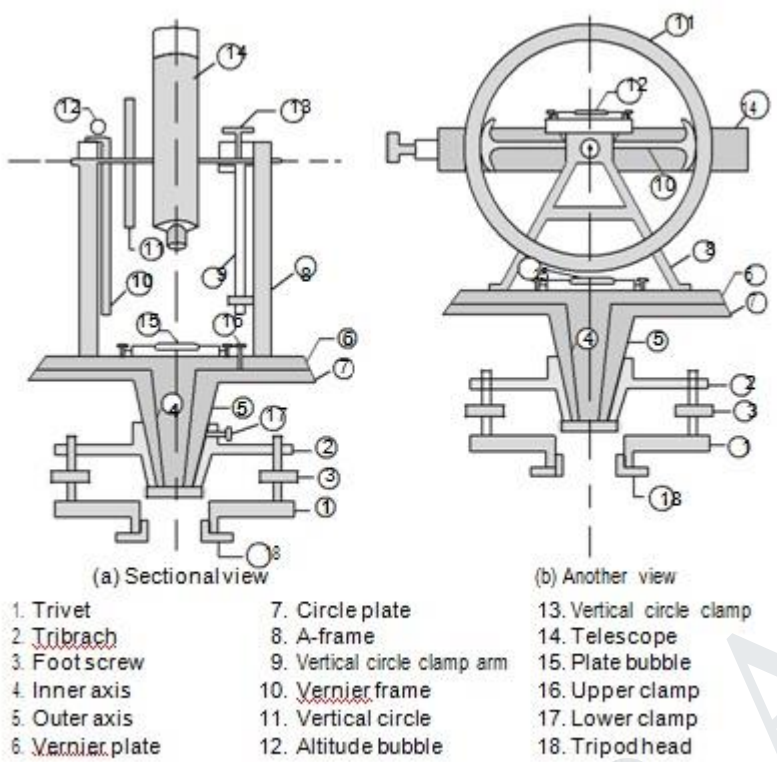
There are a variety of theodolites-vernier, optic, electronic, etc. The improve-ments (from one form to the other) have been made to ensure ease of operation, better accuracy, and speed. Electronic theodolites display and store angles at the press of a button. This data can also be transferred to a computer for further processing. We start our discussion with the simplest theodolite-the vernier theodolite.

The vernier theodolite is a simple and inexpensive instrument but very valuable in terms of measuring angles. The common vernier theodolite measures angles up to an accuracy of 20□ in a compass, where the line of sight is simple, restricting its range, theodolites are provided with telescopes which provide for much greater range and better ac-curacy in sighting distant objects. It is, however, a delicate instrument and needs to be handled carefully. The theodolite measures the horizontal angles between lines and can also measure vertical angles. The horizontal angle measured can be the included angle, deflection angle or exterior angle in a traverse. The vertical angle is the angle in a vertical plane between the inclined line of sight of the instrument and the horizontal. In the following sections we will discuss the vernier theodolite as well as its applications in surveying.

Vernier Theodolite

The vernier theodolite is also known as a *transit*. In a transit theodolite or simply transit the telescope can be rotated in a vertical plane. Earlier versions of theodolites were of the non-transit type and are obsolete now. Only the transit theodolite will be discussed here.

Two different views of a vernier theodolite are shown in Figs 6.1(a) and (b). The instrument details vary with different manufacturers but the essential parts remain the same. The main parts of a theodolite are the following.



	(a) Sectional view		(b) Another view		
1.	Trivet	7.	Circle plate	13.	Vertical circle clamp
2.	Tribrach	8.	A-frame	14.	Telescope
3.	Foot screw	9.	Vertical circle clamp arm	15.	Plate bubble
4.	Inner axis	10.	Vernier frame	16.	Upper clamp
5.	Outer axis	11.	Vertical circle	17.	Lower clamp
6.	Vernier plate	12.	Altitude bubble	18.	Tripod head

Fig. 6.1 Vernier theodolite

Levelling head The levelling head is the base of the instrument. It has the provi-sion to attach the instrument to a tripod stand while in use and attach a plumb bob along the vertical axis of the instrument. The levelling head essentially consists of two triangular plates kept a distance apart by levelling screws. The upper plate of the levelling head, also known as the *tribrach*, has three arms, each with a foot screw. Instruments with four foot screws for levelling are also available. In terms of wear and tear, the three-foot-screw instrument is preferable. The lower plate, also known as the*trivet*, has a central hole and a hook to which a plumb bob can be attached. In modern instruments, the base plate of the levelling head has two plates which

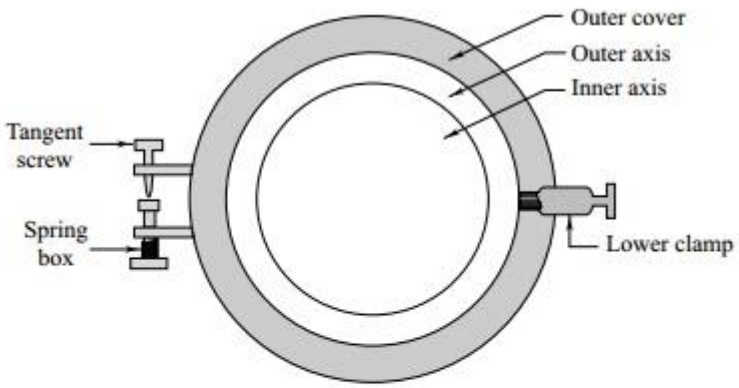
can move relative to each other. This allows a slight movement of the level-ling head relative to the tripod. This is called a *shifting head*and helps in centring the instrument over the station quickly. The functions of the levelling head are to support the upper part of the instrument, attach the theodolite to a tripod, attach a plumb bob, and help in levelling the instrument with the foot screws.

Lower plate The lower plate, also known as the *circle plate*, is an annular,
Horizontal plate with a beveled graduated edge fixed to the upper end of a hollow cylindrical part. The graduations are provided all around, from 0 o to 360 o , in the clockwise direction. The graduations are in degrees divided into three parts so that each division equals 20►. An axis through the centre of the plate is known as the outer axis or the centre. Horizontal angles are measured with this plate. The diameter of the lower plate is sometimes used to indicate the size of or designate the instrument; for example, a 100-mm theodolite.

Upper plate The upper plate is also a horizontal plate of a smaller diameter attached to a solid, vertical spindle. The bevelled edge of the horizontal part carries two verniers on diametrically opposite parts of its circumference. These verniers are generally designated A and B. They are used to read fractions of the horizontal circle plate graduations. The centre of the plate or the spindle is known as the inner axis or centre. The upper and lower plates are enclosed in a metal cover to prevent dust accumulation. The cover plate has two glass windows longer than the vernier length for the purpose of reading. Attached to the cover plate is a metal arm hinged to the centre carrying two magnifying glasses at its ends. The magnify-ing glasses are used to read the graduations clearly.

Two axes or centres The inner axis as mentioned earlier is the axis of the conical spindle attached to the upper or vernier plate. The outer axis is the centre of the hollow cylindrical part attached to the lower or circle plate. These two axes coin-cide and form the vertical axis of the instrument, which is one of the fundamental lines of the theodolite.

Clamps and tangent screws There are two clamps and associated tangent or slow-motion screws with the plates. The clamp screws facilitate the motion of the instrument in a horizontal plane. The lower clamp screw locks or releases the lower plate. When this screw is unlocked, the lower and upper plates move together. The associated lower tangent screw allows small motion of the plates in the locked position. The upper clamp screw locks or releases the upper vernier plate. When this clamp is released (with the lower clamp locked), the lower plate does not move but the vernier plate moves with the instrument. This causes a change in the reading. The upper tangent screw allows for a small motion of the vernier plate for fine adjustments. When both the clamps are locked, the instrument cannot move in the horizontal plane. The construction of the clamp and tangent screws is shown in Fig. 6.2.



(a) Clamp and tangent screw

(b) Vertical circle graduations (20′ main scale)

Plate level The plate level is a spirit level with a bubble and graduations on the glass cover. A single level or two levels fixed in perpendicular directions may be provided. The spirit level can be adjusted with the foot screw of the levelling head. The bubble of the spirit level can be moved with the foot screws of the levelling head, which is a very fundamental adjustment required for using the theodolite. A small circular bubble may be provided for rough adjustment before levelling.

Index frame The index frame, also known as a T-frame or vernier frame, is a T-shaped metal frame. The horizontal arm carries at its ends two verniers, which remain fixed in front of the vertical circle. These verniers are generally designedC and D. The vertical leg of the T-frame, known as the clipping arm, has clipping screws with which the frame can be titled. The altitude level is generally fixed on top of this frame. When the telescope is rotated in a vertical plane, the vertical circle moves and vertical angles are measured on the vertical circle with the help of these verniers.

Standard or A-frame Two standards in the shape of the letter A are attached to the upper plate. The horizontal axis of the instrument is attached to these stan-dards. The clipping arm of the index frame and the arm of the vertical circle clamp are also attached to the A-frame. The A-frame supports the telescope and the vertical circle.

Telescope The telescope is a vital part of the instrument. It enables one to see stations that are at great distances. The essential parts of a telescope are the eye-piece, diaphragm with cross hairs, object lens, and arrangements to focus the tele-scope. A focusing knob is provided on the side of the telescope. Earlier, external focusing telescopes were used. Today, only

internal focusing telescopes are used in theodolites. These reduce the length of the telescope. The telescope may carry a spirit level on top in some instruments.

Vertical circle The vertical circle is a circular plate supported on the trunnion or horizontal axis of the instrument between the A-frames. The vertical circle has a bevelled edge on which graduations are marked. The graduations are generally quadrantal, 0 o -90 o in the four quadrants as shown in Fig. 6.2. The full circle system of graduations can also be seen in some instruments. The vertical circle moves with the telescope when it is rotated in a vertical plane. A metal cover is provided to protect the circle and the verniers from dust. Two magnifying glasses on metal arms are provided to read the circle and verniers. The cover has glass or plastic windows on which the magnifiers can be moved.

Vertical circle clamp and tangent screw The vertical circle is provided with a clamp and tangent screw as in the case of the horizontal plate, Upon clamping the vertical circle, the telescope cannot be moved in a vertical plane. The tangent screw allows for a slow, small motion of the vertical circle.

Altitude level is used for levelling, particularly when taking vertical angle observations.

A circular or trough magnetic compass is generally fitted to the odolite for measuring the magnetic bearing of lines. It is fitted on the cover of the horizontal plates. Two plates with graduations are provided in the compass box for ensuring that the needle ends are centred. The needle can be locked or released by a pin. When released, the telescope can be turned in azimuth to make the north end of the needle point to the north by making it read zero.

Tripod One accessory essential with the theodolite is the tripod on which it is mounted when it has to be used. The tripod head is screwed onto the base or the lower part of the levelling head. Its legs should be spread out for stability. The legs of the tripod are also used for rough levelling.

Plumb bob A heavy plumb bob on a good string with a hook at the end is required for centring the theodolite over a station. The plumb bob is fixed to the hook or other device projecting from the centre of the instrument in a central opening in the levelling head.

Main circle and vernier graduations In most of the instruments, the vernier enables readings up to 20? of the arc. This is made possible by marking the gradu-ations on the circle and the vernier suitably as follows. As shown in Fig. 6.2(b), the main circle is graduated into degrees and each degree is divided into three parts. Each main scale division thus represents 2◆. For the vernier, 59 main scale divisions are taken and divided into 60 parts. 59 main scale divisions form 59 ◆ 20◆. Therefore, each vernier scale division represents 59 ◆ 20/60 minutes. As you would have studied earlier, least count of the vernier = difference between a main scale division and a vernier scale division = main scale division - vernier scale division. Hence, in this case,

Least count = 20◆ - 59 ◆ 20/60 = 1/3 = (1/3) ◆ 60? = 20?

Thus the least count of the vernier in common theodolites is 20?.

Terminology of Theodolite

It is important to clearly understand the terms associated with the theodolite and its use and meaning. The following are some important terms and their definitions.

Vertical axis It is a line passing through the centre of the horizontal circle and perpendicular to it. The vertical axis is perpendicular to the line of sight and the trunnion axis or the horizontal axis. The instrument is rotated about this axis for sighting different points.

Horizontal axis It is the axis about which the telescope rotates when rotated in a vertical plane. This axis is perpendicular to the line of collimation and the verti-cal axis.

Telescope axis It is the line joining the optical centre of the object glass to the centre of the eyepiece.

Line of collimation It is the line joining the intersection of the cross hairs to the optical centre of the object glass and its continuation. This is also called the line of sight.
Axis of the bubble tube It is the line tangential to the longitudinal curve of the bubble tube at its centre.

Centring Centring the theodolite means setting up the theodolite exactly over the station mark. At this position the plumb bob attached to the base of the instrument lies exactly over the station mark.

Transiting It is the process of rotating the telescope about the horizontal axis through 180 o . The telescope points in the opposite direction after transiting. This process is also known as *plunging*or *reversing.*

Swinging It is the process of rotating the telescope about the vertical axis for the purpose of pointing the telescope in different directions. The right swing is a rotation in the clockwise direction and the left swing is a rotation in the counter-clockwise direction.

Face-left or normal position This is the position in which as the sighting is done, the vertical circle is to the left of the observer.

Face-right or inverted position This is the position in which as the sighting is done, the vertical circle is to the right of the observer.

Changing face It is the operation of changing from face left to face right and vice versa. This is done by transiting the telescope and swinging it through 180 o .

Face-left observation It is the reading taken when the instrument is in the normal or face-left position.

Face-right observation It is the reading taken when the instrument is in the inverted or face-right position.

Temporary Adjustments of Theodolite

Theodolite has two types of adjustments-temporary and permanent. Temporary adjustments are to be done at every station the instrument is set up. Permanent adjustments deal with the fundamental lines and their relationships and should be done once in a while to ensure that the instrument is properly adjusted. The funda-mental lines and their desired relationships are explained later in this chapter and the permanent adjustments are explained in detail in Chapter 4. In this section we will discuss temporary adjustments.

The temporary adjustments are the following: (a) setting up and centring,

(b) levelling, (c) focusing the eyepiece, and (d) focusing the objective.

Setting Up and Centring

The following procedure is adopted for this operation.

1. Remove the theodolite from its box carefully and fix it onto a tripod kept over the station where the instrument is to be set up. The tripod legs should be well apart and the telescope should be at a convenient height for sighting.

2. Tie a plumb bob onto the hook provided at the base. If there is no shifting head in the instrument, centre it by adjusting the tripod legs and shifting the instru-ment as a whole to bring the plumb bob over the station mark.
3. To centre the plumb bob, shift the tripod legs radially as well as circumf-erentially. *Moving any leg radially shifts the plumb bob in the direction of the leg*. This does not affect the level status of the instrument. *Moving any leg circumferentially does not appreciably shift the plumb*. However, this movement tilts the instrument and affects the level of the plate bubbles. By moving the legs the plumb bob is brought over the station mark at the same time ensuring that the instrument is approximately level. This saves a lot of time for the next operation of levelling.
4. If the instrument has a shifting head with a clamp, first centre the instrument using legs. Make the final adjustment by loosening the clamp and shifting the head (or the instrument as a whole) to bring the plumb bob over the station mark. In all operations, the starting step should be to first bring the plumb bob very close to the mark and then make the final adjustment using the legs or the shifting head.

Levelling

After setting up and centring the instrument, levelling is done. Levelling has

to
be done at every station the instrument is set up. By levelling the instrument, it is ensured that as the instrument is swung about the vertical axis, the horizontal plate moves in a horizontal plane. The instrument may have a three-screw or a four-screw levelling head. The levelling operations differ slightly in these two cases as detailed in the following sections. Most instruments have only one bubble tube, but some instruments have two bubble tubes set at right angles over the plates.

Three-screw levelling head

When the theodolite has a three-screw levelling head, the following procedure is adopted.

1. Swing the theodolite and bring the plate bubble parallel to any two of the foot

   screws. Centre the bubble by rotating the foot screws. To do this, hold the foot screws by the uniband o reinge o each ch hand and *rotate both either inwards or outwards* [see Fig. 6.3(a)]. Also note that the bubble moves in the direction of movement of the left thumb during this operation.

Once the bubble traverses (or comes to the central position from the gradua-tion of the tube), swing the instrument and bring the bubble over the third foot screw. In this position, the bubble tube is at right angles to the earlier position. Centre the bubble by rotating the third foot screw alone.

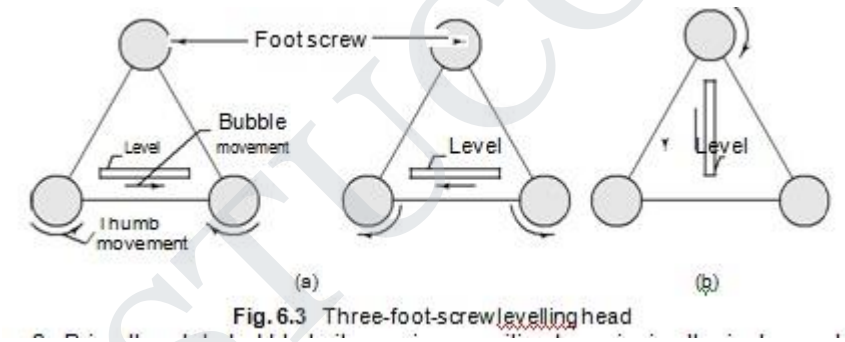


Fig. 6.3 Three-foot-screw levelling head

3. Bring the plate bubble to its previous position by swinging the instrument back. Check whether the bubble traverses. If it does not traverse, bring the bubble to the centre using the two foot screws as before.

4. Repeat the procedure till the bubble traverses in both these positions.

5. Swing the instrument through 180 o and check whether the bubble traverses. The bubble should traverse in all positions if the instrument has been properly adjusted.

If two plate bubbles are provided [see Fig. 6.3(b)], the procedure is the same except that swinging the instrument through 90 o is not required. When one plate level is kept parallel to a pair of foot screws, the other plate level is over the third foot screw (in a perpendicular direction). The third foot screw is adjusted alternately by the same process using the foot screws over which they are paral-lel.

Four-screw levelling head

When the theodolite has a four-screw levelling head, the following procedure is adopted.

1. After setting up and centring the theodolite, bring the plate level parallel to any one pair of diagonally opposite foot screws. Operate these foot screws to centre the bubble (Fig. 6.4).

2. Swing the instrument to bring the plate level parallel to the other pair of foot screws. Centre the bubble.

3. Swing it back to the previous position. Check whether the bubble traverses. If it does not, centre it with the foot screws to which the level is parallel.

4. Swing it back, check the position of the bubble, and repeat the procedure.

5. Once the bubble traverses in the two orthogonal positions, swing it through 180 o . The bubble should traverse in this position or in any other position.

If two plate levels are provided, the procedure is the same. Bring one plate level parallel to a pair of opposite foot screws. The other pair will be parallel to the remaining pair of foot screws. There is no need to swing the instrument. Bring the bubble to the central position alternately and check in the other positions.

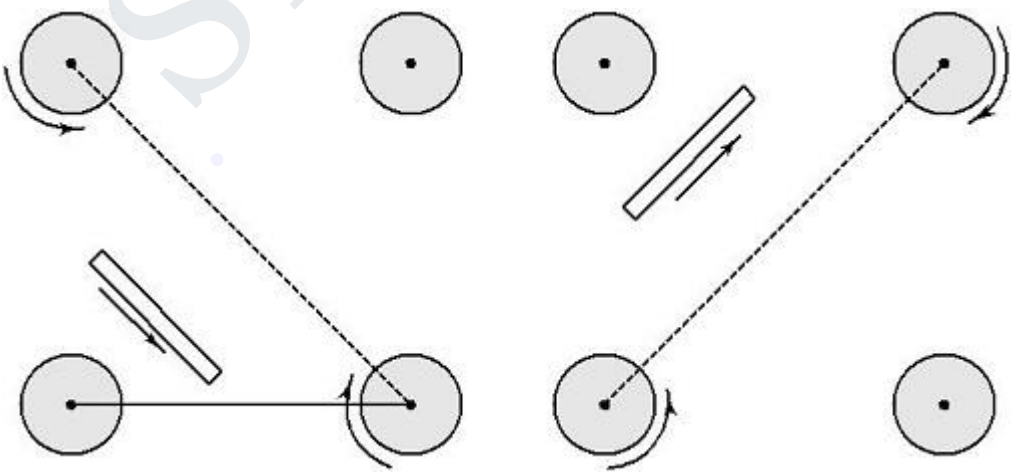


Fig. 6.4 Four-foot-screw levelling head

Focusing the Eyepiece

Focusing the eyepiece is the operation of bringing the cross hairs to focus. The focusing position varies with the eyesight of the observer. If the same observer is

taking the readings, this has to be done only once. To focus the eyepiece, use the following procedure.

1. Keep a piece of white paper in front of the telescope or direct the telescope towards a clear portion of the sky.

2. Looking through the telescope, adjust the vision by rotating the eyepiece till the cross hairs come into sharp and clear view.

3. If the eyepiece has graduations, note the graduation at which you get a clear view of the cross hairs. This can help in later adjustment if required.

Measurement of Horizontal and Vertical Angles in Theodolite

The objective lens has to be focused whenever an object is sighted, as this depends upon the distance between the instrument and the object. A focusing screw on the side of the telescope is operated to focus the objective. This operation brings the image of the object in the plane of the cross hairs. This helps to exactly bisect the object, be it a ranging rod or an arrow. To focus the objective, swing the instrument to bring the object into view by looking over the telescope. Rotate the focusing knob till the object is in sharp view along with the cross hairs.

Using the Theodolite

The theodolite is mainly used to measure horizontal and vertical angles, even though many other operations can be done with the instrument. It is a delicate and sensi-tive instrument and needs to be handled carefully. The following points should be noted while using the instrument.

1. The theodolite should be set up and levelled at every station. This is a funda-mental, necessary operation and should be carried out carefully.
2. In measuring horizontal angles, the inclination of the telescope is not significant The line of sight is arranged to bisect the object clearly.

3. The graduated circle plate gives the outer axis and the vernier plate provides the inner axis. Both the axes coincide if the instrument is properly adjusted and form the vertical axis.

4. There are three clamp screws each with its own tangent screw. The *lower clamp* screw releases the lower plate, the *upper clamp* screw releases the upper ver-nier plate, and the third vertical circular clamp releases the vertical circle. One should be familiar with the location of the clamp screws and the corresponding tangent screws.

5. Each clamp screw releases one plate. The lower plate is released by the lower clamp screw. When this plate is released, swinging the instrument or rotating it in a horizontal plane causes no change in the reading of the circle, as both the plates move together. This is used when an object has to be sighted with the zero setting of the circle or with any other reading without changing the reading.

6. Both the clamp screws should not be released together. When the lower clamp is tight and the upper clamp screw is released, the upper plate moves relative to the lower plate and the reading changes. This is done when one has to measure an angle.
The clamp screws should be tightened very near to their final postion so that only a a very small movement has to be effected by the tangent screw. For each clamp screw, the corresponding tangent screw should be for final adjustment.

8. To set the instrument to zero at the plate circle, release the upper clamp and rotate the instrument about the vertical axis. On the vernier A, make the zero of the circle coincide with the zero of the vernier. Tighten the upper clamp and using the upper tangent screw, make the zeros exactly coincide. This can be verified by look in g th rough the magnifying gl assessand seeing that the gradua tions on either side are symmetrical. Verify the condition on vernier B as well, where the 180 o graduation should coincide with the zero of the vernier.

9. While bisecting the signals or setting the zero reading, keep the line of sight in such a position that the tangent screw moves the sight in the same direction as the movement of the instrument. If the movement is clockwise, then the
tangent screw is adjusted to move the cross hairs from left to right.

10. Operate a tangent screw only after clamping the corresponding clamp screw.

11. The magnifying glasses are so fixed that they can be moved along the circle.
Read the circle by bringing the glass over the reading and looking directly over the reading to avoid any parallax error.

12. While bisecting stations with the theodolite, the station mark should be very clear and must be a point. Bisect either the cross marks on pegs at their inter-section or the ranging rod and arrow at their lowest pointed end.

13. Clamp screws and tangent screws need careful handling. Do not apply great force on these screws and handle them delicately during survey work.

Measuring Horizontal Angles

To measure the horizontal angle between two lines, the following procedure is adopted.

1. Referring to Fig. 6.5, the angle POQ is to be measured. Set up the theodolite at O.

2. Set the instrument to read 0 o 00 00?. This is not strictly required, as the angle can be determined as the difreadings. However, it is convenient to make the initial reading zero. For this, release the upper clamp and O rotate the instrument to make the Q reading approximately zero. Clamp the upper plate and using the upper Fig. 6.5 Measuring a horizontal angle tangent screw, make the readingcompassexactly zero. Vernier A reads zero and vernier B reads 180 o 00 00?.

3. Release the lower plate and rotate the instrument to bisect the station P. After approximately bisecting it, clamp the lower plate and using the lower tangent screw, bisect the signal exactly. The readings on the plates do not change as both the plates move together in this operation. Check that the readings on vernier A and B are zero and 180 o , respectively.

4. Release the upper plate by loosening the upper clamp. Rotate the instrument to screw, exactly bisect the signal at Q. Read both the verniers A and B. The reading at A will give the angle directly. The reading at B will be 180 o + -POQ.

5. If there is any difference, take the average of the two values as the correct angle.

Horizontal angles are measured this way for ordinary work. The accuracy can be improved by reading the angles with face-left and face-right observations and taking the average of the two. For more precise work, the angles are repeatedly measured with both the faces and the average taken. This method is known as the *repetition method* and is described below.

Method of Repetition in Theodolite

In the method of repetition, the horizontal angle is measured a number of times and the average value is taken. It is usual to limit the number of repetitions to three with each face except in the case of very precise work. With large number of rep-etitions, errors can also increase due to bisections, reading the verniers, etc. Very large number of repetitions necessarily do not lead to a more precise value of the angle. However, a number of errors are eliminated by the repetition method. The procedure is as follows (Fig. 6.6).

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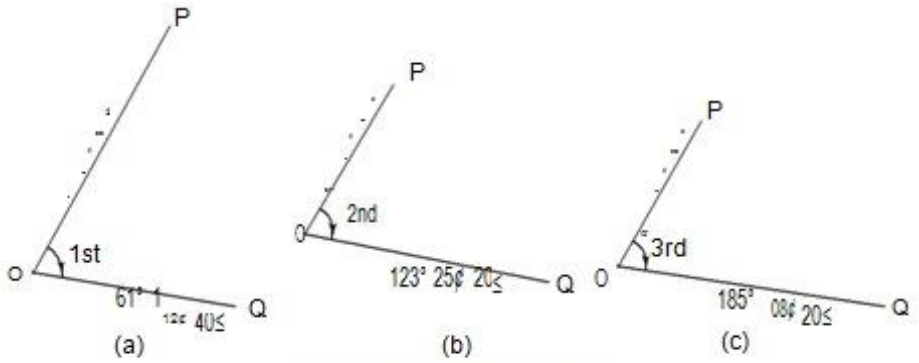

Fig. 6.6 Repetition method

1. Angle POQ is to be measured. Set up, centre, and level the theodolite at O. Ensure that the instrument is in the normal position, i.e., face left.
2. Set the instrument to read 0 o 00 00?. For this release the upper clamp and bring the zero of the vernier (at vernier A) very close to the zero of the circle. Clamp the upper plate and using the upper tangent screw, coincide the two zeros exactly.

3. Loosen the lower clamp and rotate the instrument so that the left signal at P is approximately bisected. Tighten the lower clamp and using the lower tangent screw, bisect the signal at P exactly. Read the verniers at A and B. The reading should not change and they should read zero and 180 o .

4. Loosen the upper clamp and rotate the instrument clockwise to bisect the right signal at Q. Using the upper tangent screw, biscect the signal at approximately Q exactly.

5. Read the verniers at A and B. The reading at A gives the value of the angle directly. The reading on the vernier at B will be 180 o + the angle. Record both the readings.

Release the lower clamp and rotate the instrument clockwise to bisect the sig- mal at the left station P again. Using the lower tangent screw, bisect the signal

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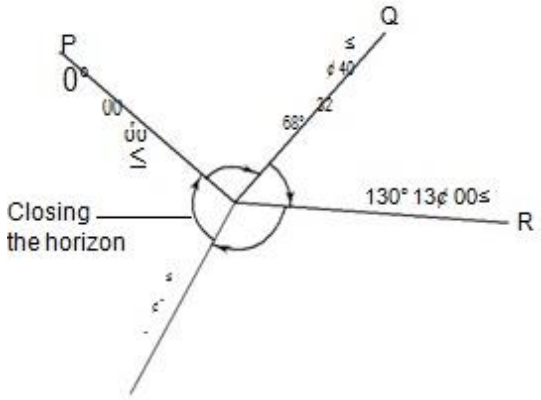

The method of repetition helps to eliminate the following errors.

(a) Errors caused by the eccentricity of the centres and verniers, by reading both the verniers and averaging.

(b) Graduation errors by reading from different parts of the circle.

(c) Imperfect adjustment of the line of collimation and horizontal axis by face-left and face-right observations.

(d) Observational errors and other errors tend to be compensated by the large number of readings.

However, the errors due to levelling cannot be compensated. This has to be done by permanent adjustment. Also a large number of repetitions tend to increase the wear of clamp and tangent screws.

Therefore, from the two sets,

Mean value of the angle = (1/2)(61 o 42 47? + 61 o 42 40?) = 61 o 42 44?

Method of Reiteration in Theodolite

The method of reiteration is another method of measuring horizontal angles. This method is useful when a number of angles are to be measured at one point. In Fig. 6.7, let O be the point where the instrument is set up and P, Q, R, and S be the stations. Angle POQ, QOR, and ROS are to be measured. In the retiration method, each angle is measured in succssion and finally the line of sight is brought back to P, i.e., the line of sight is made to close the horizon. The instrument is turned through 360 o . Obviously, the instrument should read, upon closing the ho-rizon, the same reading set initially at P. The procedure is as follows.

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1. Set up and level the theodolite at O. Keep the instrument in the normal posi-tion, i.e., face left. Set the vernier at A to read zero using the upper clamp and upper tangent screw. Check that the vernier at B reads 180 o .

2. Loosen the lower clamp and swing the instrument to bisect the station mark P Tighten the swrew and using the lower tangent screw finaly bisect the signal
3. at at P. Check that the verniers at A and B read zero and 180 o , respectively.
4. Release the upper plate with the upper clamp, swing the instrument clockwise to bisect the signal at Q. Tighten the clamp and using the upper tangent screw, bisect the mark at Q exactly.

5. Read the verniers at A and B and record both the readings.

6. Release the upper clamp screw, bisect the signal at R. Tighten the clamp and bisect the mark at R exactly with the upper tangent screw. Read the verniers at A and B and record the readings.

Continue the procedure with other stations.

-POQ = 68 o 32 30?, -QOR = 61 o 41 10?, -ROS = 102 o 54 20?

Measuring Vertical Angles in Theodolite

A vertical angle is made by an inclined line of sight with the horizontal. The line of sight may be inclined upwards or downwards from the horizontal. Thus one may have an angle of elevation or depression. See Fig. 6.8. For measuring vertical angles, the theodolite is levelled with respect to the altitude bubble.

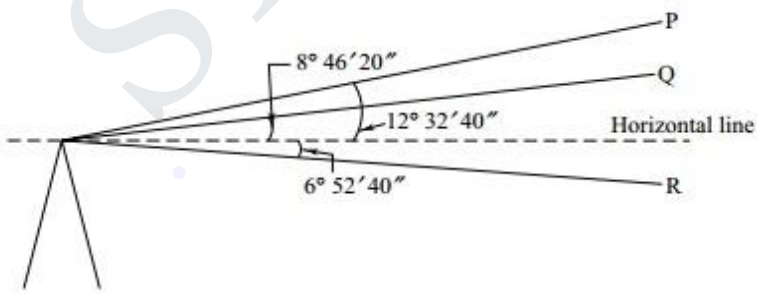

Fig. 6.8 Measuring vertical angles

1. Set up the theodolite at the station from where the vertical angle is to be mea-sured. Level the instrument with reference to the plate bubble.

2 . further level the instrument with respect to the altitude level ixed on the index rm. This bubble is generally more sensitive. The procedure for levelling is the same. Bring the altitude level parallel to two foot screws and level till the bubble traverses. Swing through 90 o to centre the bubble again with the third foot screw. Repeat till the bubble traverses.

3. Swing the telescope to approximately direct the line of sight towards the signal at P. Loosen the vertical circle clamp screw and incline the line of sight to bisect P. Clamp the vertical circle and bisect the signal exactly with the horizontal cross hair.

4. Read the verniers C and D. The average of these readings gives the value of the angle.

This procedure assumes that the instrument is properly adjusted. If there is an index error, the instrument does not read zero when the bubble is in the centre and the line of sight is horizontal, the adjustment is done by the clip screw. There may be a small index error, which can be accounted for in the value of angle. The readings can be recorded as shown in Table 6.3.

Measuring Vertical Angle Between Two Points

The two points may be above the horizontal or below the horizontal or one may be above and the other below. In all cases, the vertical angles between the instrument and the points are measured. If the points lie on the same side of the horizontal, the vertical angle between the points is the difference between the measured angles. If they lie on either side of the horizontal through the instrument, the vertical angle between the points is the sum of the angles measured.

Table 6.7 Recording of observations Face left

Instrument at	Sight to	Reading on vernier					Angle	Horizontal angle		
		A	B							
		°	¢	≤	¢	≤		°	¢	≤
O	P	00	00	00	00	00				
	Q	62	43	40	43	40	POQ	62	43	40
	R	120	38	00	38	00	QOR	57	54	20
	S	192	53	40	53	40	ROS	72	15	40
	T	273	15	00	15	20	SOT	80	21	10
	P	359	59	40	59	40	TOP	86	44	50

Interconversion of Angles

The theodolite measures the whole circle bearings of lines. These can be converted to reduced bearings by the methods discussed in Chapter 3. Also, one can calculate included angles from bearings and vice versa. Included angles can also be calculated from deflection angles and vice versa.

The following relationships of the angles of a closed traverse are known from geometry:

(a) sum of the interior angles = (2*n* - 4) right angles

(b) sum of exterior angles = (2*n* + 4) right angles
(c) sum of the deflection angles = 4 right angles

It is desirable to draw a rough sketch of the traverse before attempting to solve problems. The following examples illustrate these principles.

Locating Landscape Details with the Theodolite

We have discussed so far methods to survey the main frame or the skeleton of the survey. In most surveys, it is necessary to locate details such as buildings, railway lines, canals, and other landmarks along with the survey. A transit with a steel tape is used to locate details, and many methods are available, as the transit is an angle-measuring instrument. The following methods can be used.

Angle and distance from a single station

A point can be located with an angle to the station along with the distance from that station as shown in Fig. 6.28(a). The angle is preferably measured from the same reference line to avoid confusion. A sketch with the line and the distance and angle measured will help in plotting later. A road can be located as shown in Fig. 6.28(b). Angles to a number of points are measured and with each angle two distances are measured to locate the road.

Angle from one station and distance from another

If for any reason, it is not possible to measure the angle and distance to an object from the same point, it may be possible to locate the point by measuring angles from one station and distances from the other. The recorded data should clearly indicate the stations from which the angle and distance are measured. Figure 6.28(c) shows this method of measuring. The angle is measured from station A to point P. When the instrument is shifted to B, the distance to point P is measured from B with a steel tape.

Angles from two stations

If for some reason, it is not possible to measure distances, then angles from two stations are enough to locate a point. As shown in Fig. 6.28(d), the point P is located by measuring angles to point P from stations A and B.

The following are the fundamental lines.

1. The vertical axis
2. The horizontal or trunnion axis
3. The line of collimation or line of sight
4. Axis of altitude level
5. Axis of plate level

The meaning of these terms has been discussed earlier. The axes are shown in Fig. 6.29. When the instrument is properly adjusted, the relationships between these axes are the following.

(a) The horizontal axis must be perpendicular to the vertical axis.

(b) The axis of the plate level must be perpendicular to the vertical axis.

(c) The line of collimation must be at right angles to the horizontal axis.

(d) The axis of the altitude level (and telescope level) must be parallel to the line of collimation.

(e) The vertical circle vernier must read zero when the line of sight is horizon-tal.

Each one of these relations gives conditions for accurate measurement.

(a) When the horizontal axis is perpendicular to the vertical axis, the line of sight generates a vertical plane when transited.

(b) When the axis of the plate level is perpendicular to the vertical axis, the vertical axis will be truly vertical when the bubble traverses.

(c) When the line of collimation is at right angles to the horizontal axis, the tele-scope when rotated about the horizontal axis will move in a vertical plane.

(d) When the line of collimation and the axis of altitude level are parallel, the vertical angles will be measured without any index error.

(e) The index error due to the displacement of the vernier is eliminated when the vernier reads zero with the line of collimation truly horizontal.

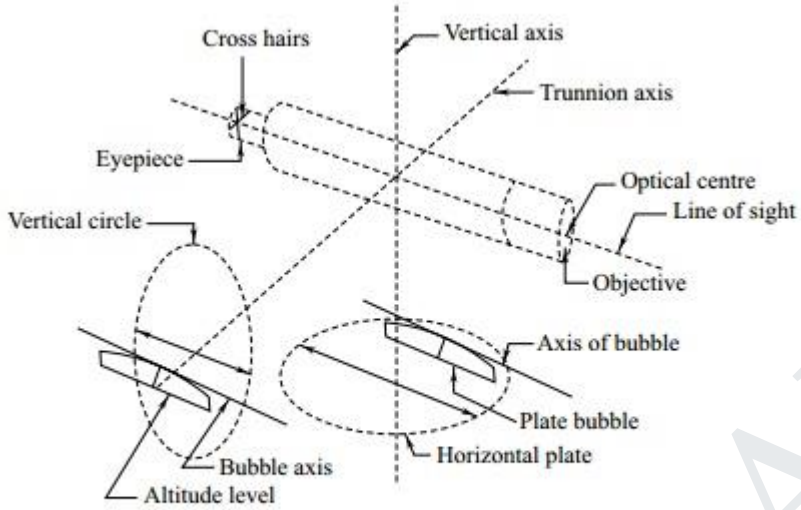

Fig. 6.29 Fundamental lines of a theodolite

UNIT III CONTROL SURVEYING AND ADJUSTMENT

HORIZONTAL CONTROLS & ITS METHODS

The horizontal control consists of reference marks of known plan position, from which salient points of designed structures may be set out. For large structures primary and secondary control points are used. The primary control points are triangulation stations. The secondary control points are reference to the primary control stations.

Reference Grid

Reference grids are used for accurate setting out of works of large magnitude. The following types of reference grids are used:

1. Survey Grid
2. Site Grid
3. tructural Grid
4. Secondary Grid

Survey grid is one which is drawn on a survey plan, from the original traverse. Original traverse stations form the control points of the grid. The site grid used by the designer is the one with the help of which actual setting out is done. As far as possible the site grid should be actually the survey grid. All the design points are related in terms of site grid coordinates. The structural grid is used when the structural components of the building are large in numbers and are so positioned that these components cannot be set out from the site grid with sufficient accuracy. The structural grid is set out from the site grid points. The secondary grid is established inside the structure, to establish internal details of the building, which are otherwise not visible directly from the structural grid.

VERTICAL CONTROL & ITS METHODS:

The vertical control consists of establishment of reference marks of known height relative to some special datum. All levels at the site are normally reduced to the nearby bench mark, usually known as master bench mark.

The setting of points in the vertical direction is usually done with the help of following rods:

1. Boning rods and travelers

2. Sight Rails

3. Slope rails or batter boards
4. Profile boards

A boning rod consist of an upright pole having a horizontal board at its top, forming a 'T 'shaped rod. Boning rods are made in set of three, and many consist of three 'T' shaped rods, each of equal size and shape, or two rods identical to each other and a third one consisting of longer rod with a detachable or movable 'T' piece. The third one is called traveling rod or traveler.
Sight Rails:

A sight rail consist of horizontal cross piece nailed to a single upright or pair of uprights driven into the ground. The upper edge of the cross piece is set to a convenient height above the required plane of the structure, and should be above the ground to enable a man to conveniently align his eyes with the upper edge. A stepped sight rail or double sight rail is used in highly undulating or falling ground. Slope rails or Batter boards:

These are used for controlling the side slopes in embankment and in cuttings. These consist of two vertical poles with a sloping board nailed near their top. The slope rails define a plane parallel to the proposed slope of the embankment, but at suitable vertical distance above it. Travelers are used to control the slope during filling operation.

Profile boards:

These are similar to sight rails, but are used to define the corners, or sides of a building. A profile board is erected near each corner peg. Each unit of profile board consists of two verticals, one horizontal board and two cross boards. Nails or saw cuts are placed at the top of the profile boards to define the width of foundation and the line of the outside of the wall.

An instrument was set up at P and the angle of elevation to a vane 4 m above the foot of the staff held at Q was 9 o 30?. The horizontal distance between P and Q was known to be 2000 metres. Determine the R.L. of the staff station Q given that the R.L. of the instrument axis was 2650.38.

Solution:

Height of vane above the instrument axis

= D tan ? = 2000 tan 9 o 30?

= 334.68 m

Correction for curvature and refraction

C = 0.06735 D^2 m, when D is in km
= 0.2694 ? 0.27 m (+ ve)

Height of vane above the instrument axis

= 334.68 + 0.27 = 334.95

R.L. fo vane = 334.95 + 2650.38 = 2985.33 m

R.L. of Q = 2985.33 - 4 = 2981.33 m

An instrument was set up at P and the angle of depression to a vane 2 m above the foot of the staff held at Q was 5 o 36?. The horizontal distance between P and Q was known to be 3000 metres. Determine the R.L. of the staff station Q given that staff reading on a B.M. of elevation 436.050 was 2.865 metres.

Solution:

The difference in elevation between the vane and the instrument axis

= D tan ?

= 3000 tan 5 o 36? = 294.153

Combined correction due to cuvature and refraction

C = 0.06735 D^2 metres , when D is in km

= 0.606 m.

Since the observed angle is negative, the combined correction due to curvature and refraction is subtractive.

Difference in elevation between the vane and the instrument axis

= 294.153 - 0.606 = 293.547 = h.

R.L. of instrument axis = 436.050 + 2.865 = 438.915
R.L. of the vane = R.L. of instrument aixs - h

= 438.915 - 293.547 = 145.368

R.L. of Q = 145.368 - 2

= 143.368 m.

In order to ascertain the elevation of the top (Q) of the signal on a hill, observations were made from two instrument stations P and R at a horizontal distance 100 metres apart, the station P and R being in the line with Q. The angles of elevation of Q at P and R were 28 o 42? and 18 o 6? respectively. The staff reading upon the bench mark of elevation 287.28 were respectively 2.870 and 3.750 when the instrument was at P and at R, the telescope being horizontal. Determine the elevation of the foot of the signal if the height of the signal above its base is 3 metres.

Solution:

Elevation of instrument axis at P = R.L. of B.M. + Staff reading

= 287.28 + 2.870 = 290.15 m

Elevation of instrument axis at R = R.L. of B.M. + staff reading

= 287.28 + 3.750 = 291.03 m

Difference in level of the instrument axes at the two stations

S =291.03 - 290.15 = 0.88 m.

? -- = 28 o 42 and ? ---- = 18 o 6?

s cot ?--- = 0.88 cot 18 o 6? = 2.69 m

= 152.1 m.

h-- = D tan ?-- = 152.1 tan 28 o 42? = 83.272 m

R.L. of foot of signal = R.L. of inst. aixs at P + h-- - ht. of signal

= 290.15 + 83.272 - 3 = 370.422 m.

Check : (b + D) = 100 + 152.1 m = 252.1 m

h-- = (b + D) tan ?-- = 252.1 x tan 18 o 6?

= 82.399 m

R.L. of foot of signal = R.L. of inst. axis at R + h--+ ht. of signal = 291.03 + 82.399 - 3 = 370.429 m.

CLASSIFICATION OF TRIANGULATION SYSTEM

The basis of the classification of triangulation figures is the accuracy with which the length and azimuth of a line of the triangulation are determined. Triangulation systems of different accuracies depend on the extent and the purpose of the survey. The accepted grades of triangulation are:

1. First order or Primary Triangulation
2. Second order or Secondary Triangulation
3. Third order or Tertiary Triangulation

1 FIRST ORDER OR PRIMARY TRIANGULATION:

The first order triangulation is of the highest order and is employed either to determine the earth's figure or to furnish the most precise control points to which secondary triangulation may be connected. The primary triangulation system embraces the vast area (usually the whole of the country). Every precaution is taken in making linear and angular measurements and in performing the reductions. The following are the general specifications of the primary triangulation:

1. Average triangle closure : Less than 1 second
2. Maximum triangle closure : Not more than 3 seconds
3. Length of base line : 5 to 15 kilometers
4. Length of the sides of triangles : 30 to 150 kilometers
5. Actual error of base : 1 in 300,000
6. Probable error of base : 1 in 1,000,000
7. Discrepancy between two measures of a section : 10 mm kilometers
8. Probable error or computed distance : 1 in 60,000 to 1 in 250,000
9. Probable error in astronomic azimuth : 0.5 seconds

2 SECONDARY ORDER OR SECONDARY TRIANGULATION

The secondary triangulation consists of a number of points fixed within the framework of primary triangulation. The stations are fixed at close intervals so that the sizes of the triangles

formed are smaller than the primary triangulation. The instruments and methods used are not of the same utmost refinement. The general specifications of the secondary triangulation are:

1. Average triangle closure : 3 sec
2. Maximum triangle closure : 8 sec
3. Length of base line : 1.5 to 5 km
4. Length of sides of triangles : 8 to 65 km
5. Actual error of base : 1 in 150,000
6. Probable error of base : 1 in 500,000
7. Discrepancy between two measures of a section : 20 mm kilometers
8. Probable error or computed distance : 1 in 20,000 to 1 in 50,000
9. Probable error in astronomic azimuth : 2.0 sec

3 THIRD ORDER OR TERTIARY TRIANGULATION:

The third-order triangulation consists of a number of points fixed within the framework of secondary triangulation, and forms the immediate control for detailed engineering and other surveys. The sizes of the triangles are small and instrument with moderate precision may be used. The specifications for a third-order triangulation are as follows:

1. Average triangle closure : 6 sec
2. Maximum triangle closure : 12 sec
3. Length of base line : 0.5 to 3 km
4. Length of sides of triangles : 1.5 to 10 km
5. Actual error of base : 1 in 75, 0000
6. Probable error of base : 1 in 250,000
7. Discrepancy between two Measures of a section : 25 mm kilometers
8. Probable error or computed distance : 1 in 5,000 to 1 in 20,000
9. Probable error in astronomic Azimuth: 5 sec.

Explain the factors to be considered while selecting base line.

The measurement of base line forms the most important part of the triangulation operations. The base line is laid down with great accuracy of measurement and alignment as it forms the basis for the computations of triangulation system. The length of the base line depends upon the grades of the triangulation. Apart from main base line, several other check bases are also measured at some suitable intervals. In India, ten bases were used, the lengths of the nine bases vary from 6.4 to 7.8 miles and that of the tenth base is 1.7 miles.

Selection of Site for Base Line. Since the accuracy in the measurement of the base line depends upon the site conditions, the following points should be taken into consideration while selecting the site:

1. The site should be fairly level. If, however, the ground is sloping, the slope should be uniform and gentle. Undulating ground should, if possible be avoided.

2. The site should be free from obstructions throughout the whole of the length. The line clearing should be cheap in both labour and compensation.

3.The extremities of the base should be intervisible at ground level.

4. The ground should be reasonably firm and smooth. Water gaps should be few, and if possible not wider than the length of the long wire or tape.

5. The site should suit extension to primary triangulation. This is an important factor since the error in extension is likely to exceed the error in measurement.

In a flat and open country, there is ample choice in the selection of the site and the base may be so selected that it suits the triangulation stations. In rough country, however, the choice is limited and it may sometimes be necessary to select some of the triangulation stations that at suitable for the base line site.

Standards of Length. The ultimate standard to which all modern national standards are referred is the international meter established by the Bureau International der Poids at Measures and kept at the Pavilion de Breteuil, Sevres, with copies allotted to various national surveys. The meter is marked on three platinum- iridium bars kept under standard conditions. One great disadvantage of the standard of length that are made of metal are that they are subject to very small secular change in their dimensions. Accordingly, the meter has now been standardized in terms of wavelength of cadmium light.

TYPES OF ERROR

Errors of measurement are of three kinds: (i) mistakes, (ii) systematic errors, and (iii) accidental errors.

(i) Mistakes. Mistakes are errors that arise from inattention, inexperience, carelessness and poor judgment or confusion in the mind of the observer. If mistake is undetected, it produces a serious effect on the final result. Hence every value to be recorded in the field must be checked by some independent field observation.

Systematic Error. A systematic error is an error that under the same conditions will always be of the same size and sign. A systematic error always follows some definite mathematical or physical law, and a correction can be determined and applied. Such errors are of constant character and are regarded as positive or negative according as they make the result too great or too small. Their effect is therefore, cumulative. If undetected, systematic errors are very serious. Therefore:

(1) All the surveying equipments must be designed and used so that whenever possible systematic errors will be automatically eliminated and (2) all systematic errors that

cannot be surely eliminated by this means must be evaluated and their relationship to the conditions that cause them must be determined. For example, in ordinary levelling, the levelling instrument must first be adjusted so that the line of sight is as nearly horizontal as possible when bubble is centered. Also the horizontal lengths for back sight and foresight from each instrument position should be kept as nearly equal as possible. In precise levelling, every day, the actual error of the instrument must be determined by careful peg test, the length of each sight is measured by stadia and a correction to the result is applied.

(iii) Accidental Error. Accidental errors are those which remain after mistakes and systematic errors have been eliminated and are caused by a combination of reasons beyond the ability of the observer to control. They tend sometimes in one direction and some times in the other, i.e., they are equally likely to make the apparent result too large or too small.

An accidental error of a single determination is the difference between (1) the true value of the quantity and (2) a determination that is free from mistakes and systematic errors. Accidental error represents limit of precision in the determination of a value. They obey the laws of chance and therefore, must be handled according to the mathematical laws of probability.

The theory of errors that is discussed in this chapter deals only with the accidental errors after all the known errors are eliminated and accounted for.

THE LAW OF ACCIDENTAL ERRORS

Investigations of observations of various types show that accidental errors follow a definite law, the law of probability. This law defines the occurrence of errors and can be expressed in the form of equation which is used to compute the probable value or the probable precision of a quantity. The most important features of accidental errors which usually occur are:

(i) Small errors tend to be more frequent than the large ones; that is they are the most probable.

(ii) Positive and negative errors of the same size happen with equal frequency ; that is, they are equally probable.

(iii) Large errors occur infrequently and are impossible.

PRINCIPLES OF LEAST SQUARES

It is found from the probability equation that the most probable values of a series of errors arising from observations of equal weight are those for which the sum of the squares is a minimum. The fundamental law of least squares is derived from this. According to the

principle of least squares, the most probable value of an observed quantity available from a given set of observations is the one for which the sum of the squares of the residual errors is a minimum. When a quantity is being deduced from a series of observations, the residual errors will be the difference between the adopted value and the several observed values,

Let $V1$, $V2$, $V3$ etc. be the observed values x = most probable value

LAW OF WEIGHTS

From the method of least squares the following laws of weights are established:

(i) The weight of the arithmetic mean of the measurements of unit weight is equal to the number of observations.

For example, let an angle A be measured six times, the following being the values:

A	Weight	A	Weight
30 o 20? 8'	1	30 o 20? 10'	1
30 o 20? 10'	1	30 o 20? 9'	1
30 o 20? 7'	1	30 o 20? 10'	1

Arithmetic mean

= 30 o 20? + 1/6 (8' + 10' + 7' + 10' + 9' + 10')

= 30 o 20? 9'.

Weight of arithmetic mean = number of observations = 6.

(2) The weight of the weighted arithmetic mean is equal to the sum of the individual weights.

For example, let an angle A be measured six times, the following being the values :

A	Weight	A	Weight
30 o 20? 8'	2	30 o 20? 10'	3
30 o 20? 10'	3	30 o 20? 9'	4
30 o 20? 6'	2	30 o 20? 10'	2

Sum of weights = 2 + 3 + 2 + 3 + 4 + 2 =16

Arithmetic mean = 30 o 20? + 1/16 (8'X2 + 10' X3+ 7'X2 + 10'X3 + 9' X4+ 10'X2)

= 30 o 20? 9'.

Weight of arithmetic mean = 16.

(3) The weight of algebric sum of two or more quantities is equal to the reciprocals of the individual weights.

For Example angle A = 30 o 20? 8', Weight 2
B = 15 o 20? 8', Weight 3
Weight of A + B =
(4) If a quantity of given weight is multiplied by a factor, the weight of the result is obtained by dividing its given weight by the square of the factor.

(5) If a quantity of given weight is divided by a factor, the weight of the result is obtained by multiplying its given weight by the square of the factor.

(6) If a equation is multiplied by its own weight, the weight of the resulting equation is equal to the reciprocal of the weight of the equation.

(7) The weight of the equation remains unchanged, if all the signs of the equation are changed or if the equation is added or subtracted from a constant.

DISTRIBUTION OF ERROR OF THE FIELD MEASUREMENT

Whenever observations are made in the field, it is always necessary to check for the closing error, if any. The closing error should be distributed to the observed quantities. For examples, the sum of the angles measured at a central angle should be 360 o , the error should be distributed to the observed angles after giving proper weight age to the observations. The following rules should be applied for the distribution of errors:

(i).The correction to be applied to an observation is inversely proportional to the weight of the observation.

(2) The correction to be applied to an observation is directly proportional to the square of the probable error.

(3) In case of line of levels, the correction to be applied is proportional to the length.
The following are the three angles ?, ? and y observed at a station P closing

the horizon, along with their probable errors of measurement. Determine their corrected values.

Solution.
? = 78 o 12? 12' ◆ 2'

? = 136 o 48? 30' ⊕ 4'
y = 144 o 59? 08' ⊕ 5'
Sum of the three angles = 359 o 59? 50'
Discrepancy = 10'

Hence each angle is to be increased, and the error of 10' is to be distributed in

proportion to the square of the probable error.

Let c1, c2 and c3 be the correction to be applied to the angles ?, ? and y respectively.

c1 : c2 : c3 = (2)⊕ : (4)⊕ : (5)⊕ = 4 : 16 : 25 ⊕ (1)

Also, c1 + c2 + c3 = 10' ⊕ (2)

From (1), c2 = 16 /4 c1 = 4c1

And c3 = 25/4 c1

Substituting these values of c2 and c3 in (2), we get c1 +

4c1 + 25/4 c1 = 10'

or c1 (1 + 4 + 25/4)= 10'

c1 = 10 x 4/45 = 0'.89

c2 = 4c1 = 3'.36

And c3 = 25 /4 c1 = 5'.55

Check: c1 + c2 + c3 = 0.'89 + 3'.56 + 5'.55 = 10'

Hence the corrected angles are

? = 78 o 12? 12' + 0'.89 = 78 o 12? 12'.89

? = 136 o 48? 30' + 3'.56 = 136 o 48? 33'.56

and y = 144 o 59? 08' + 5'.55 = 144 o 59? 13'.55

Sum = 360 o 00? 00'+ 00

An angle A was measured by different persons and the following are the values :

Angle	Number of measurements
65 o 30? 10' ⊕	2
65 o 29? 50' ⊕	3
65 o 30? 00' ⊕	3
65 o 30? 20' ⊕	4
65 o 30? 10' ⊕	3

Find the most probable value of the angle. Solution.

As stated earlier, the most probable value of an angle is equal to its weighted

arithmetic mean.

65 o 30? 10' x 2 = 131 o 00? 20'

65 o 29? 50' x 3 = 196 o 29? 30'

65 o 30? 00' x 3 = 196 o 30? 00'

65 o 30? 20' x 4 = 262 o 01? 20'

65 o 30? 10' x 3 = 196 o 30? 30'

Sum = 982 o 31? 40' ? weight = 2 + 3 + 3 + 4 + 3 = 15

Weighted arithmetic mean

= 982 o 31? 40'

= 65 o 30? 6'.67

Hence most probable value of the angle = 65 o 30? 6'.67

The telescope of a theodilite is fitted with stadia wires. It is required to find the most probable values of the constants C and K of tacheometer. The staff was kept vertical at three points in the field and with of sight horizontal the staff

intercepts observed was as follows.

Distance of staff from tacheometer D(m)	Staff intercept S(m)
150	1.495
200	2.000
250	2.505

Solution:

The distance equation is

D = KS + C

The observation equations are

150 = 1.495 K + C

200 = 2.000 K + C

$$250 = 2.505\,K + C$$

If K and C are the most probable values, then the error of observations are:

$$150 - 1.495\,K - C$$

$$200 - 2.000\,K - C$$

$$250 - 2.505\,K - C$$

By the theory of least squares

$(150 - 1.495\,K - C)^2 + (200 - 2.000\,K - C)^2 + (250 - 505\,K - C)^2 = \text{minimum}$---(i)

For normal equation in K,

Differentiating equation (i) w.r.t. K,

$2(-1.495)(150 - 1.495\,K - C) + 2(-20.)(200 - 2.000\,K - C)$

$+2(-2.505)(250 - 505\,K - C) = 0$

$208.41667 - 2.085\,K - C = 0$ -------- (2)

Normal equation in C

Differentiating equation (i) w.r.t. C,

$2(-1.0)(150 - 1.495\,K - C) + 2(-1.0)(200 - 2.000\,K - C)$

$+2(-1.0)(250 - 505\,K - C) = 0$

$200 - 2\,K - C = 0$ --------- (3)

On solving Equations (2) and (3)

$K = 99.0196$

$C = 1.9608$

The distance equation is:

$D = 99.0196\,S + 1.9608$

The following angles were measured at a station O as to close the horizon.

$\angle AOB = 83^\circ 42'28''.75$	**weight 3**
$\angle BOC = 102^\circ 15'43''.26$	**weight 2**
$\angle COD = 94^\circ 38'27''.22$	**weight 4**
$\angle DOA = 79^\circ 23'23''.77$	**weight 2**

Adjust the angles by method of Correlates.

Solution:

$\angle AOB = 83^\circ 42'28''.75$	Weight 3
$\angle BOC = 102^\circ 15'43''.26$	Weight 2
$\angle COD = 94^\circ 38'27''.22$	Weight 4
$\angle DOA = 79^\circ 23'23''.77$	Weight 2

Sum = $360^\circ 00'03''.00$

Hence, the total correction E = 360° - (360°0'3")

= -3"

Let e_1, e_2, e_3 and e_4 be the individual corrections to the four angles respectively. Then by the condition equation, we get

$e_1 + e_2 + e_3 + e_4 = -3''$ -------- (1)

Also, from the least square principle, $\Sigma(we^2) =$ a minimum

$3e_1^2 + 2e_2^2 + 4e_3^2 + 2e_4^2 =$ a minimum ------- (2)

Differentiating (1) and (2), we get

$\delta e_1 + \delta e_2 + \delta e_3 + \delta e_4 = 0$ --------- (3)

BOC	= 102°15'43".26 – 0".95	= 102°15'42".31
COD	= 94°38'27".22 – 0".47	= 94°38'26".75
DOA	= 79°23'23".77 – 0".95	= 79°23'22".82
		360°00'00".00

The following round of angles was observed from central station to surrounding station of a triangulation survey.

A = 93°43'22"	**weight 3**
B = 74°32'39"	**weight 2**
C = 101°13'44"	**weight 2**
D = 90°29'50"	**weight 3**

In addition, one angle $\overline{(A+B)}$ was measured separately as combi angle with a mean value of 168°16'06" (wt 2).

Determine the most probable values of the angles A, B, C and D.

Solution:

A + B+C+D = 359°59'35".

Total correction E = 360° - (359° 59' 35")

= + 25°

Similarly, $\overline{(A+B)}$ = (A+B)

Hence correction E' = A + B - $\overline{(A+B)}$

= 168°16'01" – 168°16'06"

= -5"

Let e_1, e_2, e_3, e_4 and e_5 be the individual corrections to A, B, C, D $\overline{(A+B)}$ respectively. Then by the condition equation, we get

$e_1 + e_2 + e_3 + e_4 = -25"$ -------- (1(a))

$e_5 - e_1 - e_2 = -5"$ -------- (1(b))

Also, from the least square principle, $\Sigma(we^2)$ = a minimum

$3e_1^2 + 2e_2^2 + 2e_3^2 + 3e_4^2 + 2e_5^2$ = a minimum ------- (2)

Differentiating (1a) (1b) and (2), we get

$\delta e_1 + \delta e_2 + \delta e_3 + \delta e_4 = 0$ --------- (3a)

$\delta e_5 - \delta e_1 - \delta e_2 = 0$ ---------- (3b)

BOC	= 102°15'43".26 – 0".95	= 102°15'42".31
COD	= 94°38'27".22 – 0".47	= 94°38'26".75
DOA	= 79°23'23".77 – 0".95	= 79°23'22".82
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= + 25°

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Hence correction E' = A + B - $\overline{(A+B)}$

= 168°16'01" – 168°16'06"

= -5"

Let e_1 ,e_2, e_3 ,e_4 and e_5 be the individual corrections to A, B, C, D $\overline{(A+B)}$ respectively. Then by the condition equation, we get

$e_1 + e_2 + e_3 + e_4 = -25"$ -------- (1(a))

$e_5 - e_1 - e_2 = -5"$ -------- (1(b))

Also, from the least square principle, $\Sigma(we^2) =$ a minimum

$3e_1^2 + 2e_2^2 + 2e_3^2 + 3e_4^2 + 2e_5^2 =$ a minimum ------- (2)

Differentiating (1a) (1b) and (2), we get

$\delta e_1 + \delta e_2 + \delta e_3 + \delta e_4 = 0$ --------- (3a)

$\delta e_5 - \delta e_1 - \delta e_2 = 0$ ---------- (3b)

ering

$$3e_1\delta e_1 + 2e_2\delta e_2 + 2e_3\delta e_3 + 3e_4\delta e_4 + 2e_5\delta e_5 = 0 \qquad \text{--------- (4)}$$

Multiplying equation (3a) by $-\lambda_1$, (3b) by $-\lambda_2$ and adding it to (3), we get

$$\delta e_1(3e_1-\lambda_1+\lambda_2) + \delta e_2(2e_2-\lambda_1+\lambda_2) + \delta e_3(2e_3-\lambda_1) + \delta e_4(3e_4-\lambda_1)$$
$$+\delta e_5(-\lambda_2+2e_5) = 0 \qquad \text{--------- (5)}$$

Since the coefficients of $\delta e_1, \delta e_2, \delta e_3, \delta e_4$ etc. must vanish independently, we have

$$-\lambda_1+\lambda_2+3e_1=0 \quad \text{or} \quad e_1=\frac{\lambda_1}{3}-\frac{\lambda_2}{3}$$

$$-\lambda_1+\lambda_2+2e_2=0 \quad \text{or} \quad e_2=\frac{\lambda_1}{2}-\frac{\lambda_2}{2}$$

$$-\lambda_2+2e_3=0 \quad \text{or} \quad e_3=\frac{\lambda_1}{2} \qquad \text{---------- (6)}$$

$$-\lambda_1+3e_4=0 \quad \text{or} \quad e_4=\frac{\lambda_1}{3}$$

$$-\lambda_2+2e_5=0 \quad \text{or} \quad e_5=-\frac{\lambda_2}{2}$$

Substituting these values of e_1, e_2, e_3, e_4 and e_5 in Equations (1a) and (1b)

$$\frac{\lambda_1}{3}-\frac{\lambda_2}{3}+\frac{\lambda_1}{2}-\frac{\lambda_2}{2}+\frac{\lambda_1}{2}+\frac{\lambda_1}{3}=25 \quad from(1a)$$

$$\text{or} \quad 5\frac{\lambda_1}{3}-\frac{5}{6}\lambda_2=25$$

$$\frac{\lambda_1}{3}-\frac{1}{6}\lambda_2=5 \qquad \text{-------- (I)}$$

$$\frac{\lambda_2}{2}-\frac{\lambda_1}{3}+\frac{\lambda_2}{3}-\frac{\lambda_1}{2}+\frac{\lambda_2}{32}=-5 \quad from(1b)$$

$$4\frac{\lambda_2}{3}-\frac{5}{6}\lambda_1=-5 \qquad \text{--------- (II)}$$

Solving (I) and (II) simultaneously, we get

$$\lambda_1 = +\frac{210}{11}$$

$$\lambda_2 = +\frac{90}{11}$$

Hence $$e_1 = \frac{1}{3}\cdot\frac{210}{11} - \frac{1}{3}\cdot\frac{90}{11} = +\frac{40''}{11} = +3''.64$$

$$e_2 = \frac{1}{2}\cdot\frac{210}{11} - \frac{1}{2}\cdot\frac{90}{11} = +\frac{60''}{11} = +5''.45$$

$$e_3 = \frac{1}{2}\cdot\frac{210}{11} = +\frac{105''}{11} = +9''.55$$

$$e_4 = \frac{1}{3}\cdot\frac{210}{11} = +\frac{70''}{11} = +6''.36$$

Total = +25".00

Also

$$e_5 = \frac{1}{2}\cdot\frac{90}{11} + 4''09.$$

Hence the corrected angles are

A = 93°43'22" + 3".64 = 93°43'25".64

B = 74°32'39" + 5".45 = 74°32'44".45

C = 103°13'44" + 9".55 = 101°13'53".55

D = 90°29'50" + 6".36 = 90°29'56".36

Sum = 360°00'00".00

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UNIT IV ADVANCED TOPICS IN SURVEYING

ADVANCED TOPICS IN SURVEYING

INTRODUCTION

Photogrammetry - Introduction

- Terrestial and aerial Photographs
- Stereoscopy
- Parallax
- Electromagnetic distance measurement
- Carrier waves
  - Principles - Instruments
- Trilateration

Hydrographic Surveying

- Tides
- MSL
- Sounding methods
- Location of soundings and methods
- Three point problem
- Strength of fix
- Sextants and station pointer
- River surveys
- Measurement of current and discharge

Cartography

- Cartographic concepts and techniques
- Cadastral surveying
  - Definition
  - Uses
  - Legal values
- Scales and accuracies.

PHOTOGRAMMETRIC SURVEYING

Photogram metric surveying or photogrammetry is the science and art of obtaining accurate measurements by use of photographs, for various purposes such as the construction of

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planimetric and topographic maps, classification of soils, interpretation of geology, acquisition of military intelligence and the preparation of composite pictures of the ground.

PRINCIPLES BEHIND TERRESTRIAL PHOTOGRAMMETRY.

The principle of terrestrial photogrammetry was improved upon and perfected by Capt. Deville, then Surveyor General of Canada in 1888. In terrestrial photogrammetry, photographs are taken with the camera supported on the ground. The photographs are taken by means of a photo theodolite which is a combination of a camera and a theodolite. Maps are then compiled from the photographs.

PHOTOGRAMMETRIC SURVEYING

Photogram metric surveying or photogrammetry is the science and art of obtaining accurate measurements by use of photographs, for various purposes such as the construction of planimetric and topographic maps, classification of soils, interpretation of geology, acquisition of military intelligence and the preparation of composite pictures of the ground. The photographs are taken either from the air or from station on the ground. Terrestrial photogrammetry is that Brach of photogrammetry wherein photographs are taken from a fixed position on or near the ground. Aerial photogrammetry is that branch of photogrammetry wherein the photographs are taken by a camera mounted in an aircraft flying over the area. Mapping from aerial photographs is the best mapping procedures yet developed for large projects, and are invaluable for military intelligence. The major users of aerial mapping methods are the civilian and military mapping agencies of the Government.

The conception of using photographs for purposes of measurement appears to have originated with the experiments of Aime Laussedat of the Corps of the French Army, who in 1851 produced the first measuring camera. He developed the mathematical analysis of photographs as perspective projections, thereby increasing their application to topography. Aerial photography from balloons probably began about 1858. Almost concurrently (1858), but independently of Laussedat, Meydenbauer in Germany carried out the first experiments in making critical measurements of architectural details by the intersection method in the basis of two photographs of the building. The ground photography was perfected in Canada by Capt. Deville, then Surveyor General of Canada in 1888. In Germany, most of the progress on the theoretical side was due to Hauck.

In 1901, Pulfrich in Jena introduced the stereoscopic principle of measurement and designed the stereo comparator. The stereoaitograph was designed (1909) at the Zeiss workshops in Jena, and this opened a wide field of practical application. Scheimpflug, an Australian captain, developed the idea of double projector in 1898. He originated the theory of perspective transformation and incorporated its principles in the photoperspecto graph. He

also gave the idea of radial triangulation. His work paved the way for the development of aerial surveying and aerial photogrammetry.

In 1875, Oscar Messter built the first aerial camera in Germany and J.W.Bagloy and A.Brock produced the first aerial cameras in U.S.A. In 1923, Bauersfeld designed the Zeiss stereoplanigraph. The optical industries of Germany, Switzerland, Italy and France, and later also those of the U.S.A and U.S.S.R. took up the manufacture and constant further development of the cameras and plotting instruments. In World War II, both the sides made extensive use of aerial photographs for their military operations. World War II gave rise to new developments of aerial photography techniques, such as the application of radio control to photoflight navigation, the new wide-angle lenses and devices to achieve true vertical photographs.

PRINCIPLES BEHIND TERRESTRIAL PHOTOGRAMMETRY

The principle of terrestrial photogrammetry was improved upon and perfected by Capt. Deville, then Surveyor General of Canada in 1888. In terrestrial photogrammetry, photographs are taken with the camera supported on the ground. The photographs are taken by means of a photo theodolite which is a combination of a camera and a theodolite. Maps are then compiled from the photographs.

The principle underlying the method of terrestrial photogrammetry is exactly similar to that of plane table surveying, i.e. if the directions of same objects photographed from two extremities of measured base are known, their position can be located by the intersection of two rays to the same object. However, the difference between this and plane tabling is that more details are at once obtained from the photographs and their subsequent plotting etc. is done by the office while in plane tabling all the detailing is done in the field itself.

Thus in Fig , A and B are the two stations at the ends of base AB. The arrows indicate the directions of horizontal pointing (in plan) of the camera. For each pair of pictures taken from the two ends, the camera axis is kept parallel to each other. From economy and speed point of view, minimum number of photographs should be used to cover the whole area and to achieve this, it is essential to select the best positions of the camera stations. A thorough study of the area should be done from the existing maps, and a ground reconnaissance should be made. The selection of actual stations depends upon the size and ruggedness of the area to be Surveyed. The camera should be directed downward rather than upward, and the stations should be at the higher points on the area.

The terrestrial photogrammetry can be divided into two branches:

(i) Plane-table photogrammetry.

(ii) Terrestrial stereo photogrammetry

The plane table photogrammetry consists essentially in taking a photograph of the area to be mapped from each of the two or three stations. The photograph perpendiculars may be oriented at any angle to the base, but usually from an acute angle with the latter. The main difficulty arises in the identifications of image points in a pair of photographs. In the case of homogeneous areas of sand or grass, identification becomes impossible. The principles of stereo photogrammetry, however, produced the remedy.

In terrestrial stereo photogrammetry, due to considerable improvement of accuracy obtained by the stereoscopic measurement of pairs of photographs, the camera base and the angles of intersection of the datum rays to the points to be measured can be considerably reduced since the camera axes at the two stations exhibit great similarity to each other. The image points which are parallactically displaced relative to each other in the two photographs are fused to a single spatial image by the stereoscopic measurement.

shore line survey?

The shore line surveys consist of:

(i) Determination or delineation of shore lines,

(ii) Location of shore details and prominent features to which soundings may be connected,

(iii) Determination of low and high water lines for average spring tides,

The determination or delineation of shore lines is done by traversing along the shore and taking offsets to the water edge by tape, or stadia or plane table. If the river is narrow, both the banks may be located by running a single line of traverse on one bank. For wide rivers, however, transverse may be run along both the banks. The traverse should be Connected at convenient intervals to check the work. Thus, the Fig. two traverses XY and X

- Y-- along the two opposite shores may be checked by taking observations from A and B to the points C and D. When the instrument is at B, angles ABC and ABD can be measured. From the measured length of AB and the four angles, the length CD can be calculated. If this agrees with the measured length of CD, the work is checked. Sometimes, a triangulation net is run along a wide river. In sea shore survey, buoys anchored off the shore and light houses are used as reference points and are located by triangulation.

In the case of tidal water, it is necessary to locate the high and low water lines. The position of high water line may be determined roughly from shore deposits and marks on rocks. To determine the high water line accurately, the elevation of mean high water of ordinary

spring tide is determined and the points are located on the shore at that elevation as in direct method of contouring. The low water line can also be determined similarly. However, since the limited time is available for the survey of low water line, it is usually located by interpolation from soundings.

Sounding and the methods employed in sounding.

The measurement of depth below the water surface is called sounding. This corresponds to the ordinary spirit leveling in land surveying where depths are measured below a horizontal line established by a level. Here, the horizontal line or the datum is the surface of water, the level of which continuously goes on changing with time. The object of making soundings is thus to determine the configuration of the sub aqueous source. As stated earlier, soundings are required for:

(i) Making nautical charts for navigation;

(ii) Measurement of areas subject to scour or silting and to ascertain the quantities of dredged material;

(iii) Making sub-aqueous investigations to secure information needed for the construction, development and improvement of port facilities.

For most of the engineering works, soundings are taken form a small boat. The equipment needed for soundings are:

(i) Sounding boat (ii) Sounding rods or poles

(iii) Lead lines (iv) Sounding machine

(v) Fathometer.

Sounding boat

A row-boat for sounding should be sufficiently roomy and stable. For quiet water, a flat bottom boat is more suitable, but for rough water round-bottomed boat is more suitable. For regular soundings, a row boat may be provided with a well through which sounds are taken. A sounding platform should be built for use in smaller boat. It should be extended far enough over the side to prevent the line from striking the boat. If the currents are strong, a motor or stream launch may be used with advantage.

Sounding rods or poles

A sounding rod is a pole of a sound straight-grained well seasoned tough timber usually 5 to 8 cm in diameter and 5 to 8 metres long. They are suitable for shallow and quiet

waters. An arrow or lead shoe of sufficient weights fitted at the end. This helps in holding them upright in water. The lead or weight should be of sufficient area so that it may not sink in mud or sand. Between soundings it is turned end for end without removing it from the water. A pole of 6 m can be used to depths unto 4 meters.

Lead lines

A lead line or a sounding line is usually a length of a cord, or tiller rope of Indian hemp or braided flax or a brass chain with a sounding lead attached to the end. Due to prolonged use, a line of hemp or cotton is liable to get stretched. To graduate such a line, it is necessary to stretch it thoroughly when wet before it is graduated. The line should be kept dry when not in use. It should be soaked in water for about one hour before it is used for taking soundings. The length of the line should be tested frequently with a tape. For regular sounding, a chain of brass, steel or iron is preferred. Lead lines are usually used for depths over about 6 meters.

Sounding lead is a weight (made of lead) attached to the line. The weight is conical in shape and varies from 4 to 12 kg depending upon the depth of water and the strength of the current. The weight should be somewhat streamlined and should have an eye at the top for attaching the cord. It often has cup-shaped cavity at the bottom so that it may be armed with land or tallow to pick up samples from the bottom. Where the bottom surface is soft, lead-

filled pipe with a board at the top is used with the lead weight. The weight penetrates in the mud and stops where the board strikes the mud surface.

Suggested system of marking poles and lead lines

The U.S. Coast and Geodetic survey recommends the following system of marking the poles and the lead lines :

Poles : Make a small permanent notch at each half foot. Paint the entire pole white and the spaces between the 2- and 3-, the 7- and 8-and the 12- and 13-ft marks black. Point ⬚" red bands at the 5- and 10-ft marks, a ⬚ " in black band at each of the other foot marks and ⬚" bands at the half foot marks. These bands are black where the pole is white and vice versa.

Lead Lines : A lead line is marked in feet as follow :

Feet	Marks
2, 12, 22 etc	Red bunting
4, 14, 24 etc	White bunting
6, 16, 26 etc	Blue bunting
8, 18, 28 etc	Yellow bunting
10, 60, 110 etc	One strip of leather
20, 70, 120 etc	Two strips of leather

30, 80, 130 etc Leather with two holes
40, 90, 140 etc Leather with one holes
50 Star-shaped leather
100 Star-shaped leather with one hole
The intermediate odd feet (1,3,5,7,9 etc.) are marked by white seizing.
Sounding Machine

Where much of sounding is to done, a sounding machine as very useful. The sounding machine may either be hand driven or automatic. Fig.4.3. show a typical hand driven Weddele's sounding machine.

The lead weight is carried at the end of a flexible wire cord attached to the barrel and can lowered at any desired rate, the speed of the drum being controlled by means of a break.

The readings are indicated in two dials-the outer dial showing the depth in feet and the inner showing tenths of a foot. A handle is used to raise the level which can be suspended at any height by means of a paul and ratchet. The sounding machine is mounted in a sounding boat and can be used up to a maximum depth of 100 ft.

Fathometer: Echo-sounding

A Fathometer is used in ocean sounding where the depth of water is too much, and to make a continuous and accurate record of the depth of water below the boat or ship at which it is installed. It is an *echo-sounding* instrument in which water depths are obtained be determining the time required for the sound waves to travel from a point near the surface of the water to the bottom and back. It is adjusted to read depth on accordance with the velocity of sound in the type of water in which it is being used. A fathometer may indicate the depth visually or indicate graphically on a roll which continuously goes on revolving and provide a virtual profile of the lake or sea.

What are the components of echo sounding instrument? Briefly explain the advantages of echo sounding.

The main parts of an echo-sounding apparatus are:

1. Transmitting and receiving oscillators.

2. Recorder unit.

3. Transmitter / Power unit.

Figure illustrates the principal of echo-sounding. It consists in recording the interval of time between the emission of a sound impulse direct to the bottom of the sea and the reception of

the wave or echo, reflected from the bottom. If the speed of sound in that water is v and the time interval between the transmitter and receiver is t, the depth h is given by

h = ⍰ vt

Due to the small distance between the receiver and the transmitter, a slight correction is necessary in shallow waters. The error between the true depth and the recorded depth can be calculated very easily by simple geometry. If the error is plotted against the recorded depth, the true depth can be easily known. The recording of the sounding is produced by the action of a small current passing through chemically impregnated paper from a rotating stylus

to an anode plate. The stylus is fixed at one end of a radial arm which revolves at constant speed. The stylus makes a record on the paper at the instants when the sound impulse is transmitted and when the echo returns to the receiver.

Advantage of echo-sounding

Echo-sounding has the following advantages over the older method of lead line and

rod:

1. It is more accurate as a truly vertical sounding is obtained. The speed of the vessel does deviate it appreciably from the vertical. Under normal water conditions, in ports and harbors an accuracy of 7.5 cm may be obtained.

2. It can be used when a strong current is running and when the weather is unsuitable for the soundings to be taken with the lead line.

3. It is more sensitive than the lead line.

4. A record of the depth is plotted immediately and provides a continuous record of the bottom as the vessel moves forward.

5. The speed of sounding and plotting is increased.

6. The error due to estimation of water level in a choppy sea is reduced owing to the instability of the boat.

7. Rock underlying softer material is recorded and this valuable information is obtained more cheaply than would be the case where sub-marine borings are taken.

Making the soundings

If the depth is less than 25 m, the soundings can be taken when the boat is in motion. In the case of soundings with rod the leadsman stands in the bow and plunges the rod at a forward angle, depending on the speed o the boat, such that the rod is vertical when the boat reaches the point at which soundings is being recorded. The rod should be read very quickly. The nature of the bottom should also be recorded at intervals in the note-book.

If the sounding is taken with a lead, the leadsman stands in the bow of the boat and casts the lead forward at such a distances that the line will become vertical and will reach the bottom at a point where sounding is required. The lead is withdrawn from the water after the reading is taken. If the depth is great, the lead is not withdrawn from the water, but is lifted between the soundings.

The water surface, which is also the reference datum, changes continuously. It is, therefore, essential to take the readings of the tide gauges at regular interval so that the soundings can be reduced to a fixed datum. To co-relate each sounding with the gauge reading, it is essential to record the time at which each sounding is made.

Methods employed in locating soundings

The soundings are located with reference to the shore traverse by observations made

(i) entirely from the boat, (ii) entirely from the shore or (iii) from both.

The following are the methods of location

1. By cross rope.
2. By range and time intervals.
3. By range and one angle from the shore.
4. By range and one angle from the boat.
5. By two angles from the shore.
6. By two angles from the boat.
7. By one angle from shore and one from boat.
8. By intersecting ranges.
9. By tacheometry.

Range.

A range or range line is the line on which soundings are taken. They are, in general, laid perpendicular to the shore line and parallel to each other if the shore is straight or are arranged radiating from a prominent object when the shore line is very irregular.

Shore signals.

Each range line is marked by means of signals erected at two points on it at a considerable distance apart. Signals can be constructed in a variety of ways. They should be

readily seen and easily distinguished from each other. The most satisfactory and economic type of signal is a wooden tripod structure dressed with white and coloured signal of cloth. The position of the signals should be located very accurately since all the soundings are to be located with reference to these signals.

Location by Cross-Rope

This is the most accurate method of locating the soundings and may be used for rivers, narrow lakes and for harbours. It is also used to determine the quantity of materials removed by dredging the soundings being taken before and after the dredging work is done. A single wire or rope is stretched across the channel etc. as shown in Fig.4.6 and is marked by metal tags at appropriate known distance along the wire from a reference point or zero station on shore. The soundings are then taken by a weighted pole. The position of the pole during a sounding is given by the graduated rope or line.

In another method, specially used for harbours etc., a reel boat is used to stretch the rope. The zero end of the rope is attached to a spike or any other attachment on one shore. The rope is would on a drum on the reel boat. The reel boat is then rowed across the line of sounding, thus unwinding the rope as it proceeds. When the reel boat reaches the other shore, its anchor is taken ashore and the rope is wound as tightly as possible. If anchoring is not possible, the reel is taken ashore and spiked down. Another boat, known as the sounding boat, then starts from the previous shore and soundings are taken against each tag of the rope. At the end of the soundings along that line, the reel boat is rowed back along the line thus winding in the rope. The work thus proceeds.

Location by Range and Time Intervals

In this method, the boat is kept in range with the two signals on the shore and is rowed along it at constant speed. Soundings are taken at different time intervals. Knowing the constant speed and the total time elapsed at the instant of sounding, the distance of the total point can be known along the range. The method is used when the width of channel is small and when great degree of accuracy is not required. However, the method is used in conjunction with other methods, in which case the first and the last soundings along a range are located by angles from the shore and the intermediate soundings are located by interpolation according to time intervals.

Location by Range and One Angle from the Shore

In this method, the boat is ranged in line with the two shore signals and rowed along the ranges. The point where sounding is taken is fixed on the range by observation of the angle from the shore. As the boat proceeds along the shore, other soundings are also fixed by the observations of angles from the shore. Thus B is the instrument station, A1 A2 is the range along which the boat is rowed and ?1, ?2, ?3 etc., are the angles measured at B from points 1, 2, 3 etc. The method is very accurate and very convenient for plotting. However, if the angle at the sounding point (say angle ?) is less than 30 o , the fix becomes poor. The nearer the intersection angle (?) is to a right angle, the better. If the angle diminishes to about 30 o a new instrument station must be chosen. The only defect of the method is that the surveyor does not have an immediate control in all the observation. If all the points are to be fixed by angular observations from the shore, a note-keeper will also be required along with the instrument man at shore since the observations and the recordings are to be done rapidly. Generally, the first and last soundings and every tenth sounding are fixed by angular observations and the intermediate points are fixed by time intervals. Thus the points with round mark are fixed by angular observations from the shore and the points with cross marks are fixed by time intervals. The arrows show the course of the boat, seaward and shoreward on alternate sections.

To fix a point by observations from the shore, the instrument man at B orients his line of sight towards a shore signal or any other prominent point (known on the plan) when the reading is zero. He then directs the telescope towards the leadsman or the bow of the boat, and is kept continually pointing towards the boat as it moves. The surveyor on the boat holds a flag for a few seconds and on the fall of the flag, the sounding and the angle are observed simultaneously.

The angles are generally observed to the nearest 5 minutes. The time at which the flag falls is also recorded both by the instrument man as well as on the boat. In order to avoid acute intersections, the lines of soundings are previously drawn on the plan and suitable instrument stations are selected.

Location by Range and One Angle from the Boat.

The method is exactly similar to the previous one except that the angular fix is made by angular observation from the boat. The boat is kept in range with the two shore signals and is rowed along it. At the instant the sounding is taken, the angle, subtended at the point between the range and some prominent point B on the sore is measured with the help of sextant. The telescope is directed on the range signals, and the side object is brought into coincidence at the instant the sounding is taken. The accuracy and ease of plotting is the

same as obtained in the previous method. Generally, the first and the last soundings, and some of the intermediate soundings are located by angular observations and the rest of the soundings are located by time intervals.

As compared to the previous methods, this method has the following advantages :

1. Since all the observations are taken from the boat, the surveyor has better control over the operations.

2. The mistakes in booking are reduced since the recorder books the readings directly as they are measured.

3. On important fixes, check may be obtained by measuring a second angle towards some other signal on the shore.

4. The obtain good intersections throughout, different shore objects may be used for reference to measure the angles.

Location by Two Angles from the Shore

In this method, a point is fixed independent of the range by angular observations from two points on the shore. The method is generally used to locate some isolated points. If this method is used on an extensive survey, the boat should be run on a series of approximate ranges. Two instruments and two instrument men are required. The position of instrument is selected in such a way that a strong fix is obtained. New instrument stations should be chosen when the intersection angle (?) falls below 30 o . Thus A and B are the two instrument stations. The distance d between them is very accurately measured. The instrument stations A and B are precisely connected to the ground traverse or triangulation, and their positions on plan are known. With both the plates clamped to zero, the instrument man at A bisects B ; similarly with both the plates clamped to zero, the instrument man at B bisects A. Both the instrument men then direct the line of sight of the telescope towards the leadsman and continuously follow it as the boat moves. The surveyor on the boat holds a flag for a few seconds, and on the fall of the flag the sounding and the angles are observed simultaneously. The co-ordinates of the position P of the sounding may be computed from the relations:

The method has got the following advantages:

1. The preliminary work of setting out and erecting range signals is eliminated.

2. It is useful when there are strong currents due to which it is difficult to row the boat along the range line.

The method is, however, laborious and requires two instruments and two instrument.

Location by Two Angles from the Boat

In this method, the position of the boat can be located by the solution of the three-point problem by observing the two angles subtended at the boat by three suitable shore objects of

known position. The three-shore points should be well-defined and clearly visible. Prominent natural objects such as church spire, lighthouse, flagstaff, buoys etc., are selected for this purpose. If such points are not available, range poles or shore signals may be taken. Thus A, B and C are the shore objects and P is the position of the boat from which the angles ? and ? are measured. Both the angles should be observed simultaneously with the help of two sextants, at the instant the sounding is taken. If both the angles are observed by surveyor alone, very little time should be lost in taking the observation. The angles on the circle are read afterwards. The method is used to take the soundings at isolated points. The surveyor has better control on the operations since the survey party is concentrated in one boat. If sufficient number of prominent points are available on the shore, preliminary work o setting out and erecting range signals is eliminated. The position of the boat is located by the solution of the three point problem either analytically or graphically.

Location by One Angle from the Shore and the other from the Boat

This method is the combination of methods 5 and 6 described above and is used to locate the isolated points where soundings are taken. Two points A and B are chosen on the shore, one of the points (say A) is the instrument station where a theodolite is set up, and the other (say B) is a shore signal or any other prominent object. At the instant the sounding is taken at P, the angle ?at A is measured with the help of a sextant. Knowing the distance d between the two points A and B by ground survey, the position of P can be located by calculating the two co-ordinates x and y.

Location by Intersecting Ranges

This method is used when it is required to determine by periodical sounding at the same points, the rate at which silting or scouring is taking place. This is very essential on the harbors and reservoirs. The position of sounding is located by the intersection of two ranges, thus completely avoiding the angular observations. Suitable signals are erected at the shore. The boat is rowed along a range perpendicular to the shore and soundings are taken at the points in which inclined ranges intersect the range. However, in order to avoid the confusion, a definite system of flagging the range poles is necessary. The position of the range poles is determined very accurately by ground survey.

Location by Tacheometric Observations

The method is very much useful in smooth waters. The position of the boat is located by tacheometric observations from the shore on a staff kept vertically on the boat. Observing the staff intercept s at the instant the sounding is taken, the horizontal distance between the instrument stations and the boat is calculated by

The direction of the boat (P) is established by observing the angle (?) at the instrument station B with reference to any prominent object A The transit station should be near the water level so that there will be no need to read vertical angles. The method is unsuitable when soundings are taken far from shore.

Reduction of soundings with a example

The reduced soundings are the reduced levels of the sub-marine surface in terms of the adopted datum. When the soundings are taken, the depth of water is measured with reference to the existing water level at that time. If the gauge readings are also taken at the same time, the soundings can be reduced to a common unvarying datum. The datum most commonly adopted is the 'mean level of low water of spring tides' and is written either as

L.W.O.S.T. (low water, ordinary spring tides) or

M.L.W.S. (mean low water springs). For reducing the soundings, a correction equal to the difference of level between the actual water level (read by gauges) and the datum is applied to the observed soundings, as illustrated in the table given below :

Gauge Reading at L.W.O.S.T. = 3.0 m.

Time	Gauge (m)	Distance	Soluction (m)	Correction	Reduced sounding (m)	Remarks
8.00 A.M.	3.5	10	2.5	-0.5	2.00	
		20	3.2		2.7	
		30	3.9		3.4	
		40	4.6		4.1	
8.10 A.M.	3.5	50	5.3	-0.5	4.8	
		60	5.4		4.9	
		70	5.1		4.6	
		80	4.7		4.2	
		90	3.6		3.1	
8.10 A.M.	3.5	100	2.1	-0.5	1.6	

What is three point problem ?How it can be solved ?

Given the three shore signals A, B and C, and the angles ? and ? subtended by AP, BP

and CP at the boat P, it is required to plot the position of P

1. Mechanical Solution

(i) By Tracing Paper

Protract angles ? and ? between three radiating lines from any point on a piece of tracing paper. Plot the positions of signals A, B, C on the plan. Applying the tracing paper to the plan, move it about until all the three rays simultaneously pass through A, B and C. The apex of the angles is then the position of P which can be pricked through.

(ii) By Station Pointer :

The station pointer is a three-armed protractor and consists of a graduated circle with fixed arm and two movable arms to the either side of the fixed arm. All the three arms have beveled or fiducial edges. The fiducial edge of the central fixed arm corresponds to the zero of the circle. The fiducial edges of the two moving arms can be set to any desired reading and can be clamped in position. They are also provided with verniers and slow motion screws to set the angle very precisely. To plot position of P, the movable arms are clamped to read the angles ? and ? very precisely. The station pointer is then moved on the plan till the three fiducial edges simultaneously touch A, B and C. The centre of the pointer then represents the position of P which can be recorded by a prick mark.

2. Graphical Solutions

(a) First Method :

Let a, b and c be the plotted positions of the shore signals A, B and C respectively and let ? and ? be the angles subtended at the boat. The point p of the boat position p can be obtained as under :

1. Join a and c.
2. At a, draw ad making an angle ? with ac. At c, draw cd making an angle ? with ca. Let both these lines meet at d.
3. Draw a circle passing through the points a, d and c.
4. Join d and b, and prolong it to meet the circle at the point p which is the required position of the boat.

Proof. From the properties of a circle,

apd = acd = ? and cpd = cad = ?

which is the required condition for the solution.

(b) Second Method :

Join ab and bc.

1. From a and b, draw lines ao1 and bo1 each making an angle (90 o - ?) with ab on the side towards p. Let them intersect at 01.
2. Similarly, from b and c, draw lines ========== each making an angle (90 o - ?) with ab on the side towards p. Let them intersect at --.

With - as the centre, draw a circle to pass through a and b. Similarly, with - as the centre draw a circle to pass through b and c. Let both the circles intersect each other at a point p. p is then the required position of the boat.

Proof. ao1b = 180 o - 2 (90 o - ?) = 2?

apb = ◆ ao1b = ?

Similarly, bo2c = 180 o - 2 (90 o - ?) = 2?

And bpc = ◆ bo2c = ?.

The above method is sometimes known as the method of two intersecting circles.

(c)Third Method :

1. Join ab and bc.
2. At a and c, erect perpendiculars ad and ce.
3. At b, draw a line bd subtending angle (90 o - ?) with ba, to meet the perpendicular through a in d.
4. Similarly, draw a line be subtending an angle (90 o - ?) with bc, to meet the perpendicular through c in e.
5. Join d and e.
6. Drop a perpendicular on de from b. The foot of the perpendicular (i.e. p) is then the required position of the boat.

Tides and its types and formation

All celestial bodies exert a gravitational force on each other. These forces of attraction between earth and other celestial bodies (mainly moon and sun) cause periodical variations in the level of a water surface, commonly known as tides. There are several theories about the tides but none adequately explains all the phenomenon of tides. However, the commonly used theory is after Newton, and is known as the equilibrium theory. According to this theory, a force of attraction exists between two celestial bodies, acting in the straight line joining the centre of masses of the two bodies, and the magnitude of this force is proportional to the

product of the masses of the bodies and is inversely proportional to the square of the distance between them. We shall apply this theory to the tides produced on earth due to the force of attraction between earth and moon. However, the following assumptions are made in the equilibrium theory :

1.The earth is covered all round by an ocean of uniform depth.

The ocean is capable of assuming instantaneously the equilibrium , required by the tide producing forces. This is possible if we neglect (i) inertia of water, (ii) viscosity of water, and (iii) force of attraction between parts of itself.

1. The Lunar Tides

shows the earth and the moon, with their centres of masses $O1$ and $O2$ respectively. Since moon is very near to the earth, it is the major tide producing force. To start with, we will ignore the daily rotation of the earth on its axis. Both earth and moon attract each other, and the force of attraction would act along $O1O2$. Let O be the common centre of gravity of earth and moon. The earth and moon revolve monthly about O, and due to this revolution their separate positions are maintained. The distribution of force is not uniform, but it is more for the points facing the moon and less for remote points. Due to the revolution of earth about the common centre of gravity O, centrifugal force of uniform intensity is exerted on all the particles of the earth. The direction of this centrifugal force is parallel to $O1O2$ and acts outward. Thus, the total force of attraction due to moon is counter-balanced by the total centrifugal force, and the earth maintains its position relative to the moon. However, since the fore of attraction is not uniform, the resultant force will very all along. The resultant forces are the tide producing forces. Assuming that water has no inertia and viscosity, the ocean enveloping the earth's surface will adjust itself to the unbalanced resultant forces, giving rise to the equilibrium. Thus, there are two lunar tides at A and B, and two low wate positions at C and D. The tide at A is called the superior lunar tide or tide of moon's upper transit, While tide at B is called inferior or antilunar tide.

Now let us consider the earth's rotation on its axis. Assuming the moon to remain stationary, the major axis of lunar tidal equilibrium figure would maintain a constant position. Due to rotation of earth about its axis from west to east, once in 24 hours, point A would occupy successive position C, B and D at intervals of 6 h. Thus, point A would experience regular variation in the level of water. It will experience high water (tide) at intervals of 12 h and low water midway between. This interval of 6 h variation is true only if moon is assumed stationary. However, in a lunation of 29.53 days the moon makes one revolution relative to sun from the new moon to new moon. This revolution is in the same direction as the diurnal rotation of earth, and hence there are 29.53 transits of moon across a meridian in 29.53 mean solar days. This is on the assumption that the moon does this revolution in a plane passing through the equator. Thus, the interval between successive transits of moon or any meridian will be 24 h, 50.5 m. Thus, the average interval between successive high waters would be about 12 h 25 m. The interval of 24 h 50.5 m between two successive transits of moon over a meridian is called the tidal day.

2. The Solar Tides

The phenomenon of production of tides due to force of attraction between earth and sun is

similar to the lunar tides. Thus, there will be superior solar tide and an inferior or anti-solar tide. However, sun is at a large distance from the earth and hence the tide producing force due to sun is much less.

Solar tide = 0.458 Lunar tide.

Combined effect : Spring and neap tides

Solar tide = 0.458 Lunar tide.

Above equation shows that the solar tide force is less than half the lunar tide force. However, their combined effect is important, specially at the new moon when both the sun and moon have the same celestial longitude, they cross a meridian at the same instant.

Assuming that both the sun and moon lie in the same horizontal plane passing through the

equator, the effects of both the tides are added, giving rise to maximum or spring tide of new moon. The term 'spring' does not refer to the season, but to the springing or waxing of the

moon. After the new moon, the moon falls behind the sun and crosses each meridian 50 minutes later each day. In after 7 ⅟₂ days, the difference between longitude of the moon and that of sun becomes 90 o , and the moon is in quadrature. The crest of moon tide coincides with the trough of the solar tide, giving rise to the neap tide of the first quarter. During the neap tide, the high water level is below the average while the low water level is above the

average. After about 15 days of the start of lunation, when full moon occurs, the difference between moon's longitude and of sun's longitude is 180 o , and the moon is in opposition.

However, the crests of both the tides coincide, giving rise to spring tide of full moon. In about 22 days after the start of lunation, the difference in longitudes of the moon and the sun becomes 270 o and neap tide of third quarter is formed. Finally, when the moon reaches to its new moon position, after about 29 ½ days of the previous new moon, both of them have the same celestial longitude and the spring tide of new moon is again formed making the beginning of another cycle of spring and neap tides.

4. Other Effects

The length of the tidal day, assumed to be 24 hours and 50.5 minutes is not constant because of (i) varying relative positions of the sun and moon, (ii) relative attraction of the sun

and moon, (iii) ellipticity of the orbit of the moon (assumed circular earlier) and earth, (v) declination (or deviation from the plane of equator) of the sun and the moon, (v) effects of the

land masses and (vi) deviation of the shape of the earth from the spheroid. Due to these, the high water at a place may not occur exactly at the moon's upper or lower transit. The effect of

varying relative positions of the sun and moon gives rise to what are known as priming of tide and lagging of tide.

At the new moon position, the crest of the composite tide is under the moon and

normal tide is formed. For the positions of the moon between new moon and first quarter, the high water at any place occurs before the moon's transit, the interval between successive high

Water is less than the average of 12 hours 25 minutes and the tide is said to prime. For positions of moon between the first quarter and the full moon, the high water at any place occurs after the moon transits, the interval between successive high water is more than the average, and tide is said to lag. Similarly, between full moon and 3rd quarter position, the tide primes while between the 3rd quarter and full moon position, the tide lags. At first quarter, full moon and third quarter position of moon, normal tide occurs.

Due to the several assumptions made in the equilibrium theory, and due to several other factors affecting the magnitude and period of tides, close agreement between the results of the theory, and the actual field observations is not available. Due to obstruction of land masses, tide may be heaped up at some places. Due to inertia and viscosity of sea water, equilibrium figure is not achieved instantaneously. Hence prediction of the tides at a place must be based largely on observations.

MEAN SEA LEVEL AND ITS USED AS DATUM

For all important surveys, the datum selected is the mean sea level at a certain place. The mean sea level may be defined as the mean level of the sea, obtained by taking the mean of all the height of the tide, as measured at hourly intervals over some stated period covering a whole number of complete tides, The mean sea level, defined above shows appreciable variations from day to day, from month to month and from year to year. Hence the period for which observations should be taken depends upon the purpose for which levels are required. The daily changes in the level of sea may be more. The monthly changes are more or less periodic. The mean sea level in particular month may be low while it may be high in some other moths. Mean sea level may also show appreciable variations in its annual values. Due to variations in the annual values and due to greater accuracy needed in modern geodetic

levelling, it is essential to base the mean sea level on observations extending over a period of about 19 years. During this period, the moon's nodes complete one entire revolution. The

height of mean sea level so determined is referred to the datum of tide gauge at which the observations are taken. The point or place at which these observations are taken is known as a tidal station. If the observations are taken on two stations, situated say at a distance of 200 to 500 kms on an open coast, one of the station is called primary tidal station while the other is called secondary tidal station. Both the stations may then be connected by a line of levels.

ASTRONOMICAL SURVEYING

Celestial Sphere.

The millions of stars that we see in the sky on a clear cloudless night are all at varying distances from us. Since we are concerned with their relative distance rather than their actual distance from the observer. It is exceedingly convenient to picture the stars as distributed over the surface of an imaginary spherical sky having its center at the position of the observer. This imaginary sphere on which the star appear to lie or to be studded is known as the celestial sphere. The radius of the celestial sphere may be of any value - from a few thousand metres to a few thousand kilometers. Since the stars are very distant from us, the center of the earth may be taken as the center of the celestial sphere.

Zenith, Nadir and Celestial Horizon.

The Zenith (Z) is the point on the upper portion of the celestial sphere marked by plumb line above the observer. It is thus the point on the celestial sphere immediately above the observer's station.
The Nadir (Z') is the point on the lower portion of the celestial sphere marked by the plum line below the observer. It is thus the point on the celestial sphere vertically below the observer's station. Celestial Horizon. (True or Rational horizon or geocentric horizon): It is the great circle traced upon the celestial sphere by that plane which is perpendicular to the Zenith-Nadir line, and which passes through the center of the earth. (Great circle is a section of a sphere when the cutting plane passes through the center of the sphere).

Terrestrial Poles and Equator, Celestial Poles and Equator.

The terrestrial poles are the two points in which the earth's axis of rotation meets the earth's sphere. The terrestrial equator is the great circle of the earth, the plane of which is
at right angles to the axis of rotation. The two poles are equidistant from it.
If the earth's axis of rotation is produced indefinitely, it will meet the celestial sphere in two points called the north and south celestial poles (P and P'). The celestial equator is the

great circle of the celestial sphere in which it is intersected by the plane of terrestrial equator.

1 CO-ALTITUDE OR ZENITH DISTANCE (Z) AND AZIMUTH (A).

It is the angular distance of heavenly body from the zenith. It is the complement or the altitude, i.e z = (90 o - ?).
The azimuth of a heavenly body is the angle between the observer's meridian and the vertical circle passing through the body.

Determine the hour angle and declination of a star from the following data:
(i) Altitude of the star = 22 o 36'
(ii) Azimuth of the star = 42 o W
(iii) Latitude of the place of observation = 40 o N.

Solution.

Since the azimuth of the star is 42 o W, the star is in the western hemi-sphere.

In the astronomical DPZM, we have

PZ = co-latitude = 90 o - 40 o = 50 o ;

ZM = co-altitude = 90 o - 22 o 36 o = 67 24' ; angle A = 42 o

Knowing the two sides and the included angle, the third side can be calculated from

the cosine formula

Thus, cos PM = cos PZ . cos ZM + sin PZ. Sin ZM. cos A

= cos 50 o . cos 67 o 24' + sin 50 o . sin 67 o 24'. cos 42 o

= 0.24702 + 0.52556 = 0.77258

\ PM = 39 o 25'

\Declination of the star = d= 90 o - PM = 90 o - 39 o 25' = **50 o 35' N.**
Similarly, knowing all the three sides, the hour angle H can be calculated from Eq.

1.2

$$\cos H = \frac{\cos ZM - \cos PZ.\cos PM}{\sin PZ.\sin PM} = \frac{\cos 67°24' - \cos 50°.\cos 39°25'}{\sin 50°.\sin 39°25'}$$

$$= \qquad = \frac{0.38430 - 0.49659}{0.48640} = -0.23086$$

\ cos (180 o - H) = 0.23086

\ 180 o - H = 76 o 39'

H = 103 o 21'.

UNIT V MODERN SURVEYING

TOTAL STATION: BASIC PRINCIPLE

Although taping and theodolites are used regularly on site - total stations are also used extensively in surveying, civil engineering and construction because they can measure both distances and angles.

A typical total station is shown in the figure below

Fig 3.1 Total Station

Fig 3.1 Total Station

Because the instrument combines both angle and distance measurement in the same unit, it is known as an integrated total station which can measure horizontal and vertical angles as well as slope distances.
Using the vertical angle, the total station can calculate the horizontal and vertical distance components of the measured slope distance.

As well as basic functions, total stations are able to perform a number of different survey tasks and associated calculations and can store large amounts of data.

As with the electronic theodolite, all the functions of a total station are controlled by its microprocessor, which is accessed thought a keyboard and display.

To use the total station, it is set over one end of the line to be measured and some reflector is positioned at the other end such that the line of sight between the instrument and the reflector is unobstructed (as seen in the figure below).

-The reflector is a prism attached to a detail pole

-The telescope is aligned and pointed at the prism

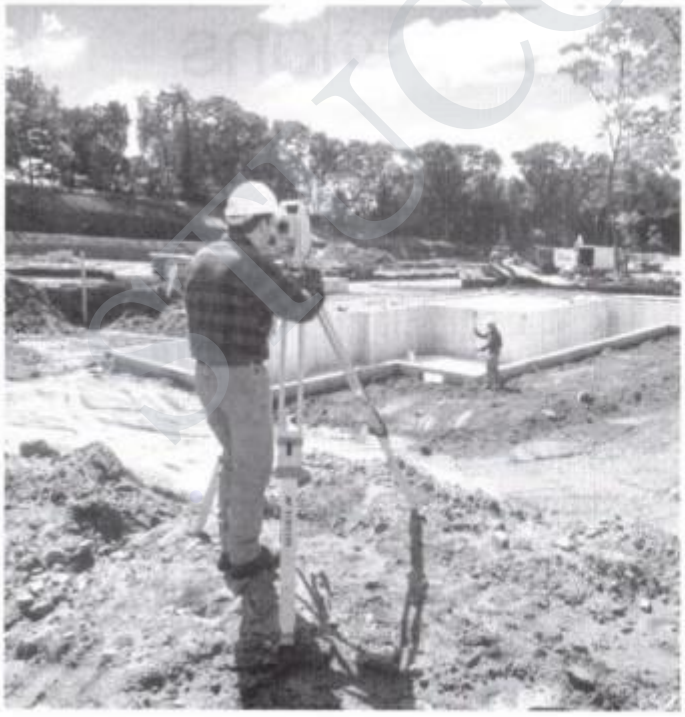
-The measuring sequence is initiated and a signal is sent to the reflector and a part of this signal is returned to the total station

-This signal is then analysed to calculate the slope distance together with the horizontal and vertical angles.

-Total stations can also be used without reflectors and the telescope is pointed at the point that needs to be measured

-Some instruments have motorised drivers and can be use automatic target recognition to search and lock into a prism - this is a fully automated process and does not require an operator.

-Some total stations can be controlled from the detail pole, enabling surveys to be conducted by one person.



Measuring with a total station

Robotic total station

Fig 3.2 Measuring with a Total Station

Most total stations have a distance measuring range of up to a few kilometres, when using a prism, and a range of at least 100m in reflector less mode and an accuracy of 2-3mm at short ranges, which will decrease to about 4-5mm at 1km.

Although angles and distances can be measured and used separately, the most common applications for total stations occur when these are combined to define position in control surveys.

As well as the total station, site surveying is increasingly being carried out using GPS equipment. Some predictions have been made that this trend will continue, and in the long run GPS methods may replace other methods.

Although the use of GPS is increasing, total stations are one of the predominant instruments used on site for surveying and will be for some time.

Developments in both technologies will find a point where devices can be made that complement both methods.

CLASSIFICATION OF TOTAL STATIONS

1 ELECTRO- OPTICAL SYSTEM

1.1DISTANCE MEASUREMENT

When a distance is measured with a total station, am electromagnetic wave or pulse is used for the measurement - this is propagated through the atmosphere from the instrument to reflector or target and back during the measurement.

Distances are measured using two methods: the phase shift method, and the pulsed laser method.

This technique uses continuous electromagnetic waves for distance measurement although these are complex in nature, electromagnetic waves can be represented in their simplest from as periodic waves.

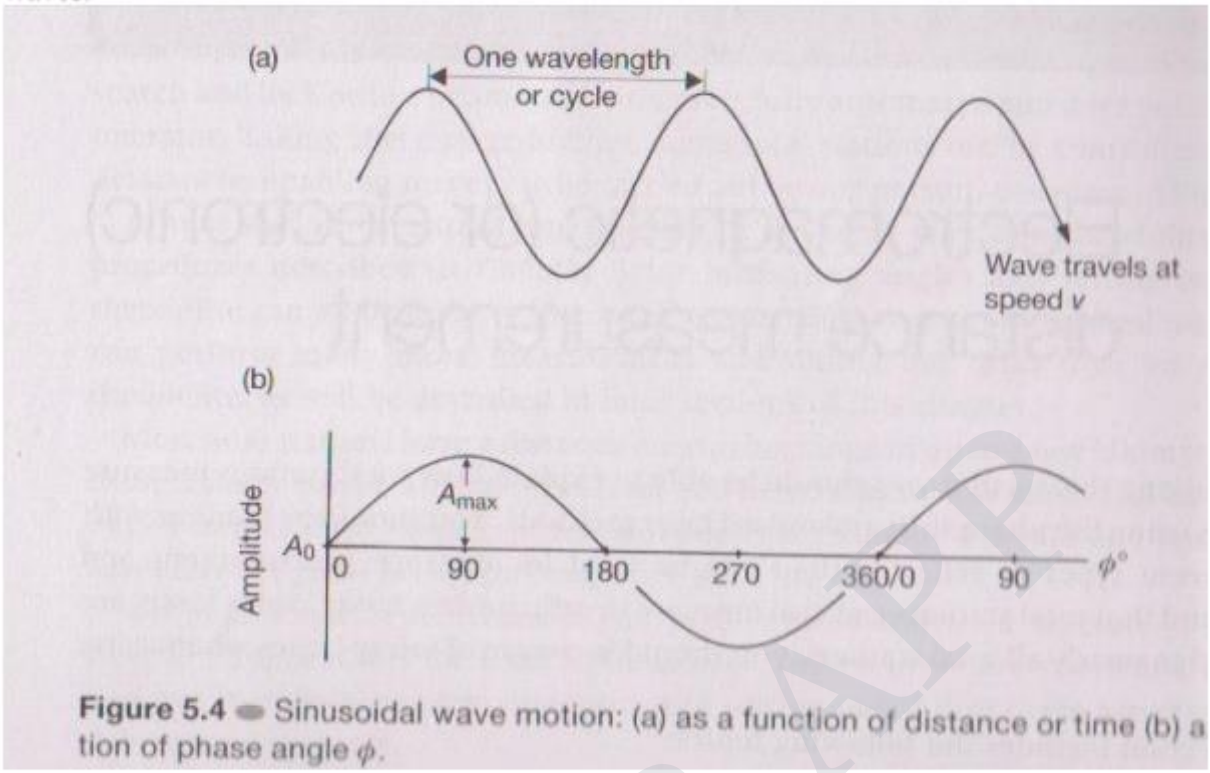


Fig 3.3 Sinusoidal wave motion

Fig 3.3 Sinusoidal wave motion

The wave completes a cycle when moving between identical points on the wave and the number of times in one second the wave completes the cycle is called the frequency of the wave. The speed of the wave is then used to estimate the distance.

2 LASER DISTANCE MEASUREMENT

In many total stations, distances are obtained by measuring the time taken for a pulse of laser radiation to travel from the instrument to a prism (or target) and back. As in the phase shift method, the pulses are derived an infrared or visible laser diode and they are transmitted through the telescope towards the remote end of the distance being measured, where they are reflected and returned to the instrument.

Since the velocity v of the pulses can be accurately determined, the distance D can be obtained using $2D = vt$, where t is the time taken for a single pulse to travel from instrument - target - instrument.

This is also known as the timed-pulse or time-of-flight measurement technique.

The *transit time t* is measured using electronic signal processing techniques. Although only a single pulse is necessary to obtain a distance, the accuracy obtained would be poor. To improve this, a large number of pulses (typically

20,000 every second) are analysed during each measurement to give a more accurate distance.

The pulse laser method is a much simpler approach to distance measurement than the phase shift method, which was originally developed about 50 years ago.

SLOPE AND HORIZONTAL DISTANCES

Both the phase shift and pulsed laser methods will measure a slope distance *L* from the total station along the line of sight to a reflector or target. For most surveys the horizontal distance *D* is required as well as the vertical component *V* of the slope distance.

Horizontal distance *D* = *L* cos? = *L* sin z

Vertical distance = *V* = *L* sin? = *L* cos z

Where ? is the vertical angle and z is the is the zenith angle. As far as the user is

concerned, these calculations are seldom done because the total station will

either display *D* and *V* automatically or will dislplay *L* first and then *D* and *V* after pressing buttons

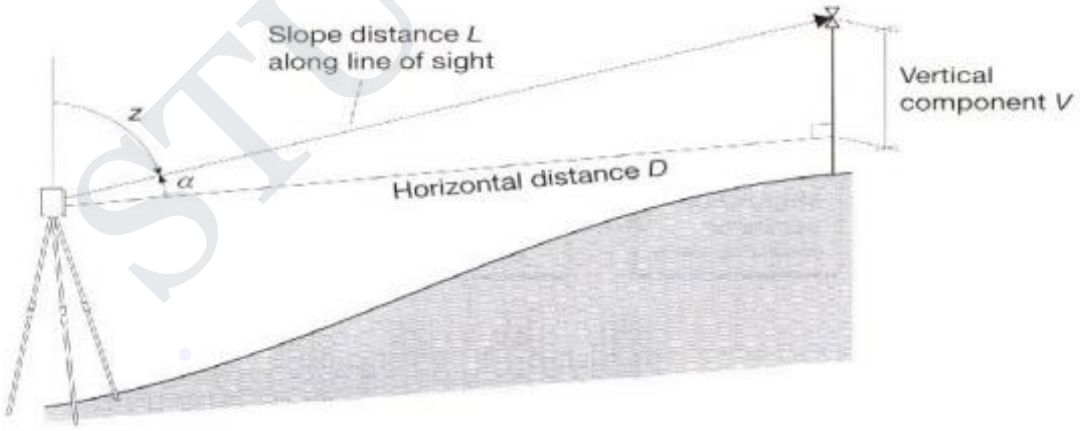


Fig 3.4 Slope and Distance Measured

Fig 3.4 Slope and Distance Measured

How accuracy of distance measurement is specified

All total stations have a linear accuracy quoted in the form

$\pm$(a mm + b ppm)

The constant a is independent of the length being measured and is made up of internal sources within the instrument that are normally beyond the control of the user. It is an estimate of the individual errors caused by such phenomena as unwanted phase shifts in electronic components, errors in phase and transit time measurements.

The systematic error b is proportional to the distance being measured, where 1 ppm (part per million) is equivalent to an additional error of 1mm for every kilometre measured.

Typical specifications for a total station vary from $\pm$(2mm + 2ppm) to $\pm$(5mm + 5 pmm).

For example: $\pm$(2mm + 2ppm), at 100m the error in distance measurement will be

$\pm$2mm but at 1.5km, the error will be $\pm$(2mm + [2mm/km * 1.5km]) = $\pm$5m m

Reflectors used in distance measurement

Since the waves or pulses transmitted by a total station are either visible or infrared, a plane mirror could be used to reflect them. This would require a very accurate alignment of the mirror, because the transmitted wave or pulses have a narrow spread.

To get around this problem special mirror prisms are used as shown below.

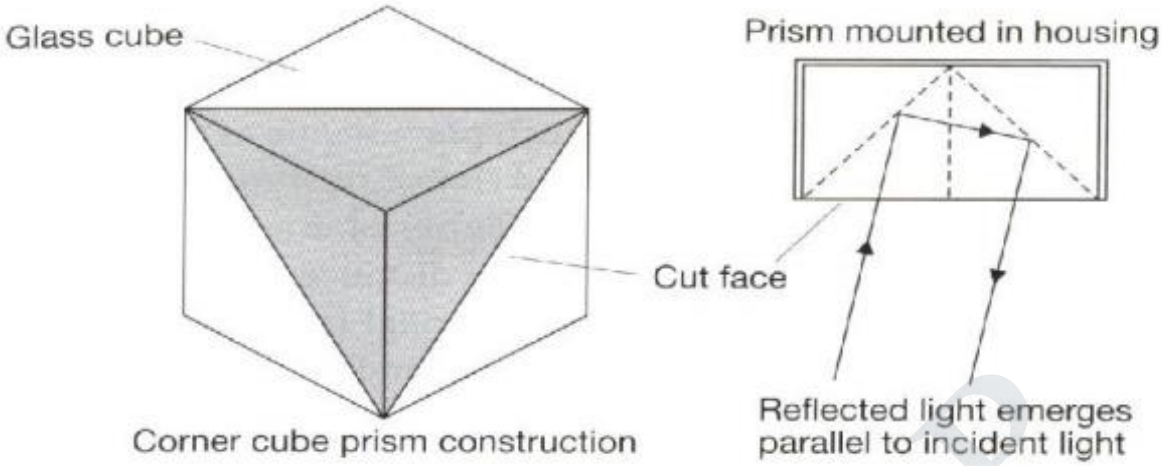


Fig 3.5 Reflector used in total station

Fig 3.5 Reflector used in total station

FEATURES OF TOTAL STATIONS

Total stations are capable of measuring angles and distances simultaneously and combine an electronic theodolite with a distance measuring system and a microprocessor.

ANGLE MEASUREMENT

All the components of the electronic theodolite described in the previous lectures are found total stations.

The axis configuration is identical and comprises the vertical axis, the tilting axis and line of sight (or collimation). The other components include the tribatch with levelling footscrews, the keyboard with display and the telescope which is mounted on the standards and which rotates around the tilting axis.

Levelling is carried out in the same way as for a theodolite by adjusting to centralise a plate level or electronic bubble. The telescope can be transited and used in the face left (or face I) and face right (or face II) positions. Horizontal rotation of the total station about the vertical axis is controlled by a horizontal clamp and tangent screw and rotation of the telescope about the tilting axis.
The total station is used to measure angles in the same way as the electronic theodolite.

Distance measurement

All total stations will measure a slope distance which the onboard computer uses, together with the zenith angle recorded by the line of sight to calculate the horizontal distance.

For distances taken to a prism or reflecting foil, the most accurate is precise measurement.

For phase shift system, a typical specification for this is a measurement time of about 1-2s, an accuracy of (2mm + 2ppm) and a range of

3-5km to a single prism.

Although all manufacturers quote ranges of several kilometres to a single prism.

For those construction projects where long distances are required to be measured, GPS methods are used in preference to total stations. There is no standard difference at which the change from one to the other occurs, as this will depend on a number of factors, including the accuracy required and the site topography.

Rapid measurement reduces the measurement time to a prism to between 0.5 and 1's for both phase shift and pulsed systems, but the accuracy for both may degrade slightly.

Tracking measurements are taken extensively when setting out or for machine control, since readings are updated very quickly and vary in response to movements of the prism which is usually pole-mounted.In this mode, the distance measurement is repeated automatically at intervals of less than 0.5s.

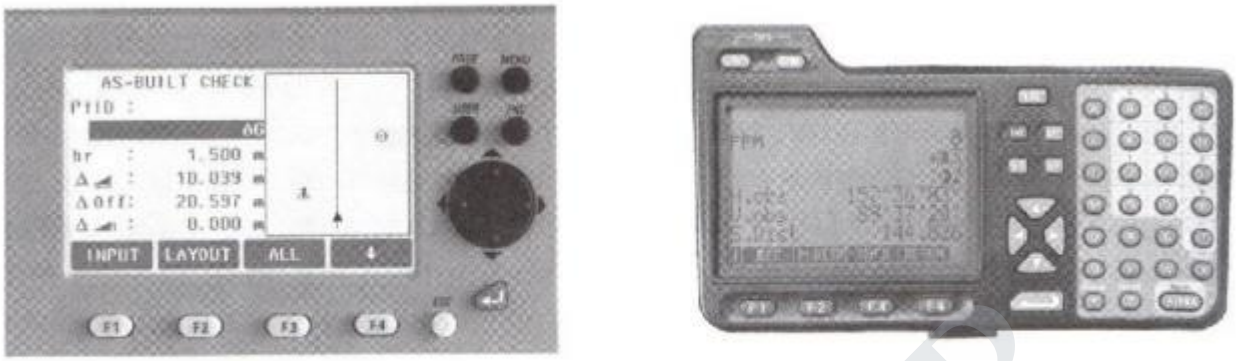
For reflector less measurements taken with a phase shift system, the range that can be obtained is about 100m, with a similar accuracy to that obtained when using a prism or foil.

KEYBOARD AND DISPLAY

A total station is activated through its control panel, which consists of a keyboard and multiple line LCD. A number of instruments have two control panels, one on each face, which makes them easier to use.

In addition to controlling the total station, the keyboard is often used to code data generated by the instrument - this code will be used to identify the object being measured.

On some total stations it is possible to detach the keyboard and interchange them with other total stations and with GPS receivers.This is called integrated surveying .



(a) (b)

Fig 3.6 Key Board and Display

SOFTWARE APPLICATIONS

The microprocessor built into the total station is a small computer and its main function is controlling the measurement of angles and distances. The LCD screen guides the operator while taking these measurements.

The built in computer can be used for the operator to carry out calibration checks on the instrument.

The software applications available on many total stations include the following:

Slope corrections and reduced levels

Horizontal circle orientation

Coordinate measurement

Traverse measurements

Resection (or free stationing)

Missing line measurement

Remote elevation measurement

areas

Setting out.

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SOURCES OF ERROR FOR TOTAL STATIONS

1 CALIBRATION OF TOTAL STATIONS

To maintain the high level of accuracy offered by modern total stations, there is now much more emphasis on monitoring instrumental errors, and with this in mind, some construction sites require all instruments to be checked on a regular basis using procedures outlined in the quality manuals.

Some instrumental errors are eliminated by observing on two faces of the total station and averaging, but because one face measurements are the preferred method on site, it is important to determine the magnitude of instrumental errors and correct for them.

For total stations, instrumental errors are measured and corrected using electronic calibration procedures that are carried out at any time and can be applied to the instrument on site. These are preferred to the mechanical adjustments that used to be done in labs by technician.

Since calibration parameters can change because of mechanical shock, temperature changes and rough handling of what is a high-precision instrument, an electronic calibration should be carried our on a total station as follows:

Before using the instrument for the first time

After long storage periods

After rough or long transportation

After long periods of work

Following big changes in temperature

Regularly for precision surveys

Before each calibration, it is essential to allow the total station enough to reach the ambient temperature.

2 HORIZONTAL COLLIMATION (OR LINE OF SIGHT ERROR)

This axial error is caused when the line of sight is not perpendicular to the tilting axis. It affects all horizontal circle readings and increases with steep sightings, but this is eliminated by observing on two faces. For single face measurements,

an on-board calibration function is used to determine *c,* the deviation between the actual line of sight and a line perpendicular to the tilting axis. A correction is then applied automatically for this to all horizontal circle readings.

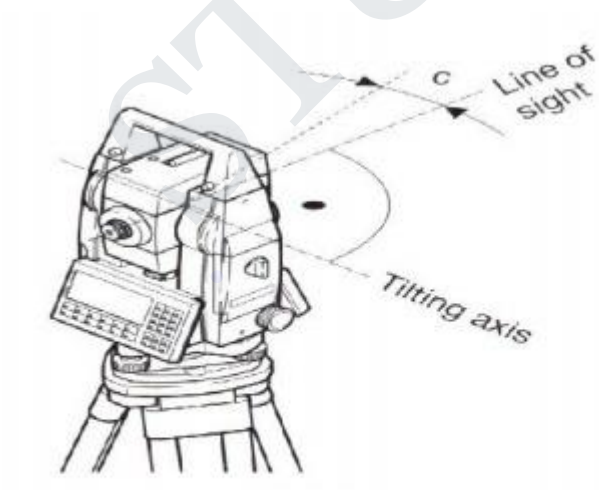


Fig 3.7 Line of Sight error

3 TILTING AXIS ERROR

This axial errors occur when the titling axis of the total station is not perpendicular to its vertical axis. This has no effect on sightings taken when the telescope is horizontal, but introduces errors into horizontal circle readings when the

telescope is tilted, especially for steep sightings. As with horizontal collimation error, this error is eliminated by two face measurements, or the tilting axis error a is measured in a calibration procedure and a correction applied for this to all horizontal circle readings - as before if a is too big, the instrument should be returned to the manufacture.

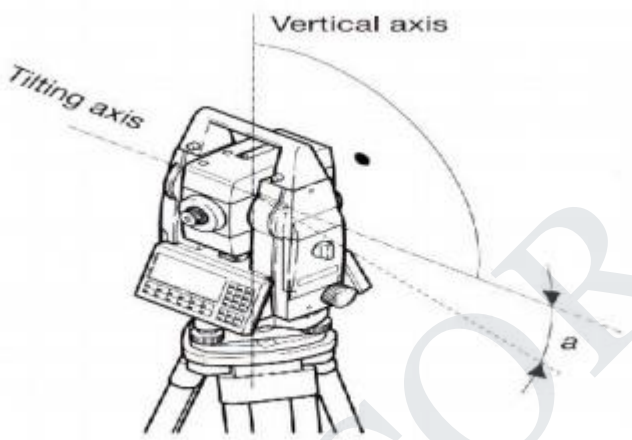


Fig tilting axis error

4 COMPENSATOR INDEX ERROR

Errors caused by not levellinga theodolite or total station carefully cannot be eliminated by taking face left and face right readings. If the total station is fitted with a compensator it will measure residual tilts of the instrument and will apply corrections to the horizontal and vertical angles for these.

However all compensators will have a longitudinal error *l* and traverse error *t* known as zero point errors. These are averaged using face left and face right readings but for single face readings must be determined by the calibration function of the total station.

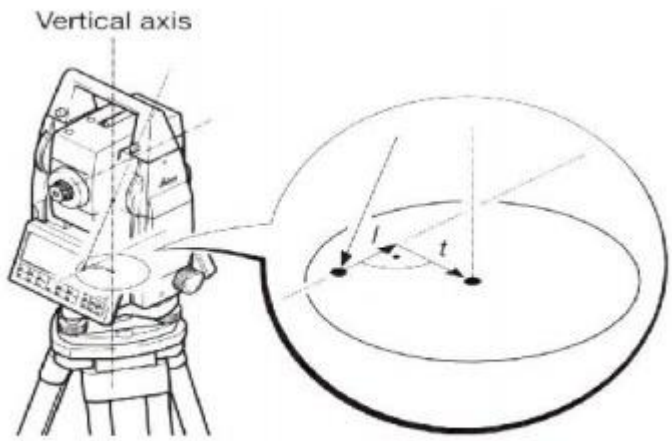
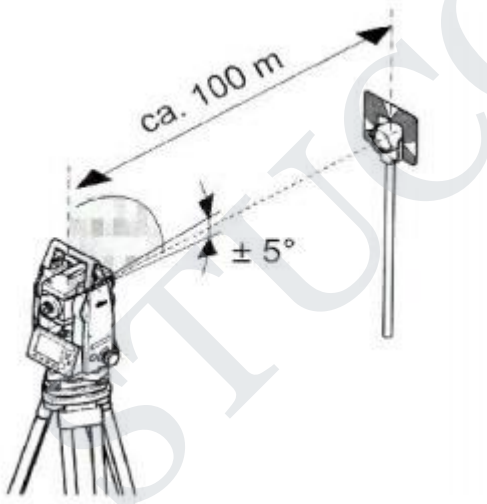


Fig 3.8 Compensator Index Error

A vertical collimation error exists on a total station if the 0° to 180° line in the vertical circle does not coincide with its vertical axis. This zero point error is present in all vertical circle readings and like the horizontal collimation error, it is eliminated by taking FL and FR readings or by determining i

For all of the above total station errors (horizontal and vertical collimation, tilting axis and compensator) the total station is calibrated using an in built function. Here the function is activated and a measurement to a target is taken as shown below.



Following the first measurement the total station and the telescope are each rotated through 180o and the reading is repeated.

Any difference between the measured horizontal and vertical angles is then quantified as an instrumental error and applied to all subsequent readings automatically. The total station is thus calibrated and the procedure is the same for all of the above error type.

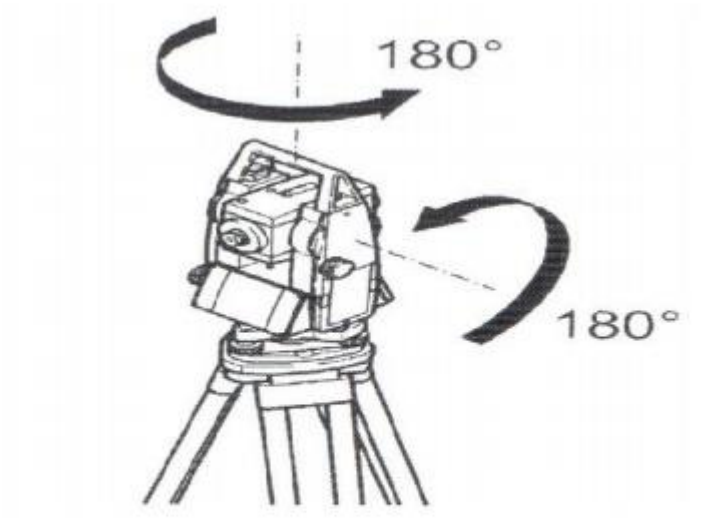


Fig 3.9 Compensator Index Error

Fig 3.9 Compensator Index Error

GPS SURVEYING

INTRODUCTION

Traditional methods of surveying and navigation resort to tedious field and astronomical observation for deriving positional and directional information. Diverse field conditions, seasonal variation and many unavoidable circumstances always bias the traditional field approach. However, due to rapid advancement in electronic systems, every aspect of human life is affected to a great deal. Field of surveying and navigation is tremendously benefited through electronic devices. Many of the critical situations in surveying/navigation are now easily and precisely solved in short time.

Astronomical observation of celestial bodies was one of the standard methods of obtaining coordinates of a position. This method is prone to visibility and weather condition and demands expert handling. Attempts have been made by USA since early 1960's to use space based artificial satellites. System TRANSIT was widely used for establishing a network of control points over large regions. Establishment of modern geocentric datum and its relation to local datum was successfully achieved through TRANSI T. Rapid improvements in higher frequently transmission and precise clock signals along with advanced stable satellite technology have been instrumental for the development of global positioning system.
The NAVSTAR GPS (Navigation System with Time and Ranging Global Positioning System) is a satellite based radio navigation system providing precise three-dimensional position, course and time information to suitably equipped user.

GPS has been under development in the USA since 1973. The US department of Defence as a worldwide navigation and positioning resource for military as well as civilian use for 24 hours and all weather conditions primarily developed it.

In its final configuration, NAVSTAR GPS consists of 21 satellites (plus 3 active spares) at an altitude of 20200 km above the earth's surface (Fig. 1). These satellites are so arranged in orbits to have atleast four satellites visible above the horizon anywhere on the earth, at any time of the day. GPS Satellites transmit at frequencies L1=1575.42 MHz and L2=1227.6 MHz modulated with two types of code viz. P-code and C/A code and with navigation message. Mainly two types of observable are of interest to the user. In pseudo ranging the distance between the satellite and the GPS receiver plus a small corrective

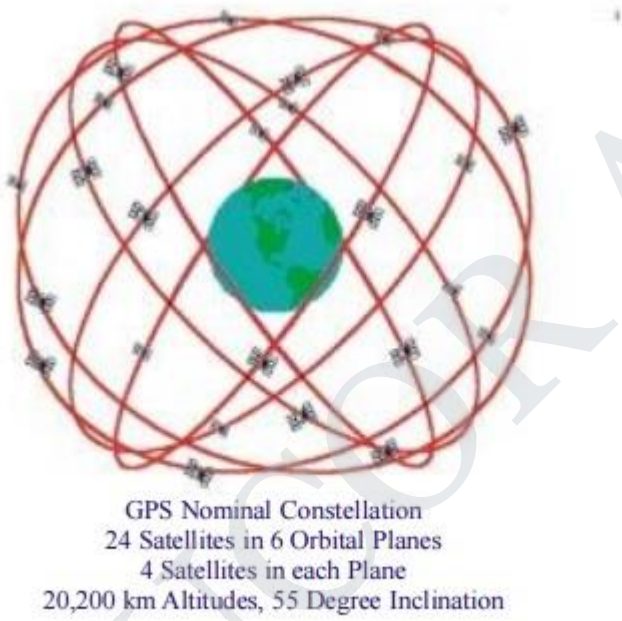


Fig 4.1 The Global Positioning System (GPS), 21-satellite configuration

Fig 4.1 The Global Positioning System (GPS), 21-satellite configuration

term for receiver clock error is observed for positioning whereas in carrier phase techniques, the difference between the phase of the carrier signal transmitted by the satellite and the phase of the receiver oscillator at the epoch is observed to derive the precise information.

The GPS satellites act as reference points from which receivers on the ground detect their position. The fundamental navigation principle is based on the measurement of pseudoranges between the user and four satellites (Fig.)
2). Ground stations precisely monitor the orbit of every satellite and by measuring the travel time of the signals transmitted from the satellite four distances between receiver and satellites will yield accurate position, direction and speed. Though three-range measurements are sufficient, the fourth observation is

essential for solving clock synchronization error between receiver and satellite. Thus, the term 'pseudoranges' is derived. The secret of GPS measurement is due to the ability of measuring carrier phases to about 1/100 of a cycle equaling to 2 to 3 mm in linear distance. Moreover the high frequency L1 and L2 carrier signal can easily penetrate the ionosphere to reduce its effect. Dual frequency observations are important for large station separation and for eliminating most of the error parameters.

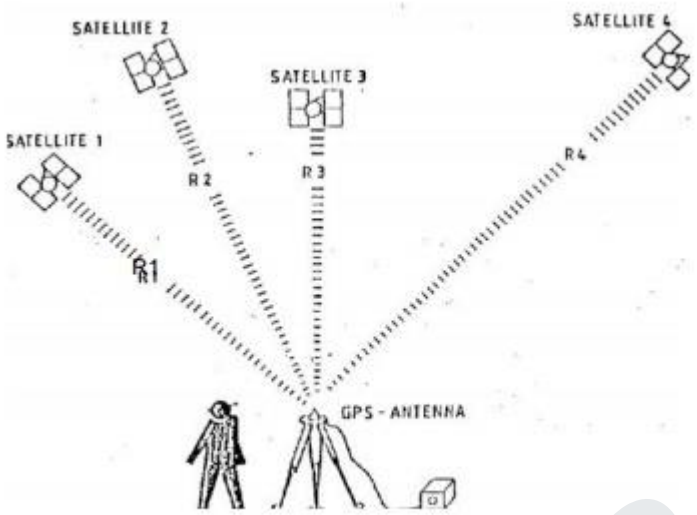


Figure 4.2: Basic principle of positioning with GPS

Figure 4.2: Basic principle of positioning with GPS

There has been significant progress in the design and miniaturization of stable clock. GPS satellite orbits are stable because of the high altitudes and no atmosphere drag. However, the impact of the sun and moon on GPS orbit though significant, can be computed completely and effect of solar radiation pressure on the orbit and tropospheric delay of the signal have been now modeled to a great extent from past experience to obtain precise information for various applications.

Comparison of main characteristics of TRANSIT and GPS re veal technological advancement in the field of space based positioning system (Table1).

Table 1. TRANSIT vs GPS

Details	TRANSIT	GPS
Orbit Altitude	1000 Km	20,200 Km
Orbital Period	105 Min	12 Hours
Frequencies	150 MHz 400 MHz	1575 MHz 1228 MHz
Navigation data	2D : X, Y	4D : X,Y,Z, t velocity
Availability	15-20 minute per pass	Continuously
Accuracy	ñ 30-40 meters (Depending on velocity	ñ15m (Pcode/No. SA 0.1 Knots
Repeatability	—	ñ1.3 meters relative
Satellite	4-6	21-24
Geometry	Variable	Repeating
Satellite Clock	Quartz	Rubidium, Cesium

GPS has been designed to provide navigational accuracy of ◆1 0 m to ◆15 m. However, sub meter accuracy in differential mode has been achieved and it has been proved that broad varieties of problems in geodesy and geodynamics can be tackled through GPS.

Versatile use of GPS for a civilian need in following fields have been successfully practiced viz. navigation on land, sea, air, space, high precision kinematics survey on the ground, cadastral surveying, geodetic control network densification, high precision aircraft positioning, photogrammetry without ground control, monitoring deformations, hydrographic surveys, active control survey and many other similar jobs related to navigation and positioning,. The outcome of a typical GPS survey includes geocentric position accurate to 10 m and relative positions between receiver locations to centimeter level or better.

GPS Surveying

Traditional methods of surveying and navigation resort to tedious field and astronomical observation for deriving positional and directional information. Diverse field conditions, seasonal variation and many unavoidable circumstances always bias the traditional field approach. However, due to rapid advancement in electronic systems, every aspect of human life is affected to a great deal. Field of surveying and navigation is tremendously benefited through electronic devices. Many of the critical situations in surveying/navigation are now easily and precisely solved in short time.

SEGMENTS OF GPS

For better understanding of GPS, we normally consider three major segments viz. space segment, Control segment and User segment. Space segment deals with GPS satellites systems, Control segment describes ground based time and orbit control prediction and in User segment various types of existing GPS receiver and its application is dealt .

Table 2 gives a brief account of the function and of various segments along with input and output information.

Table 2. Functions of various segments of GPS

Table 2. Functions of various segments of GPS

Segmen	Input	Function	Output
Space	Navigation message	Generate and Transmit code and carrier	P-Code C/A Code L1,L2
Control	P-Code Observations Time	Produce GPS time predict ephemeris	Navigation message
User	Code observation Carrier phase observation	Navigation solution Surveying	Position velocity time

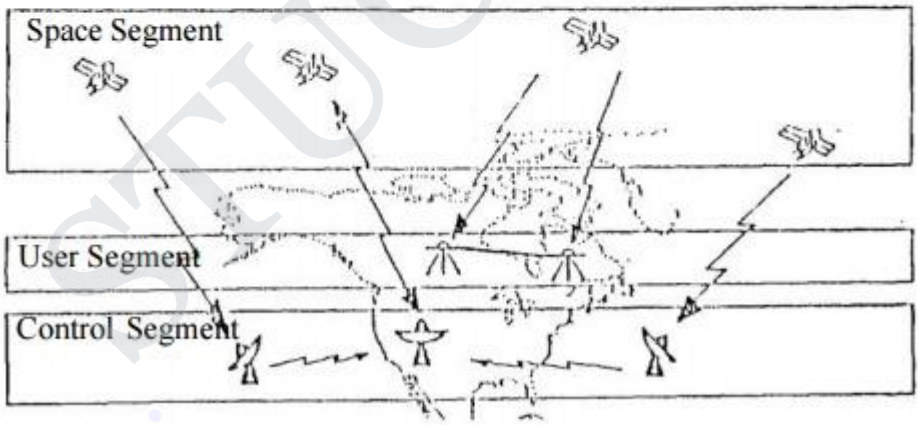


Figure 4.3: The Space, Control and User segments of GPS

Figure 4.3: The Space, Control and User segments of GPS

GLONASS (Global Navigation & Surveying System) a similar system to GPS is being developed by former Soviet Union and it is considered to be a valuable complementary system to GPS for future application.

GPS Surveying

SPACE SEGMENT

Space segment will consist 21 GPS satellites with an addition of 3 active spares. These satellites are placed in almost six circular orbits with an inclination of 55 degree. Orbital height of these satellites is about 20,200 km corresponding to about 26,600 km from the semi major axis. Orbital period is exactly 12 hours of sidereal time and this provides repeated satellite configuration every day advanced by four minutes with respect to universal time.

Final arrangement of 21 satellites constellation known as 'Primary satellite constellation' is given in Fig. 4. There are six orbital planes A to F with a separation of 60 degrees at right ascension (crossing at equator). The position of a satellite within a particular orbit plane can be identified by argument of latitude or mean anomaly M for a given epoch.

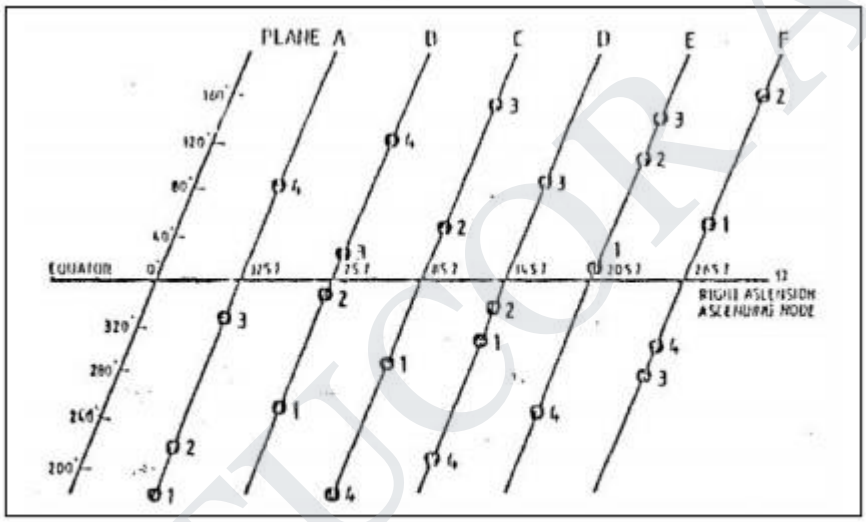


Figure 4. 4: Arrangement of satellites in full constellation

Figure 4. 4: Arrangement of satellites in full constellation

GPS satellites are broadly divided into three blocks: Block-I satellite pertains to development stage, Block II represents production satellite and Block IIR are replenishment/spare satellite.

Under Block-I, NAVSTAR 1 to 11 satellites were launched before 1978 to 1985 in two orbital planes of 63-degree inclination. Design life of these prototype test satellites was only five years but the operational period has been exceeded in most of the cases.

The first Block-II production satellite was launched in February 1989 using channel Douglas Delta 2 booster rocket. A total of 28 Block-II satellites are planned to support 21+3 satellite configuration. Block-II satellites have a designed lifetime of 5-7 years.

To sustain the GPS facility, the development of follow-up satellites under Block-II R has started. Twenty replenishment satellites will replace the current block-II satellite as and when necessary. These GPS satellites under Block-IR have additional ability to measure distances between satellites and will also compute ephemeris on board for real time information gives a schematic view of Block-II satellite. Electrical power is generated through two solar panels covering a surface area of 7.2 square meter each. However, additional battery backup is provided to provide energy when the satellite moves into earth's shadow region. Each satellite weighs 845kg and has a propulsion system for positional stabilization and orbit maneuvers.

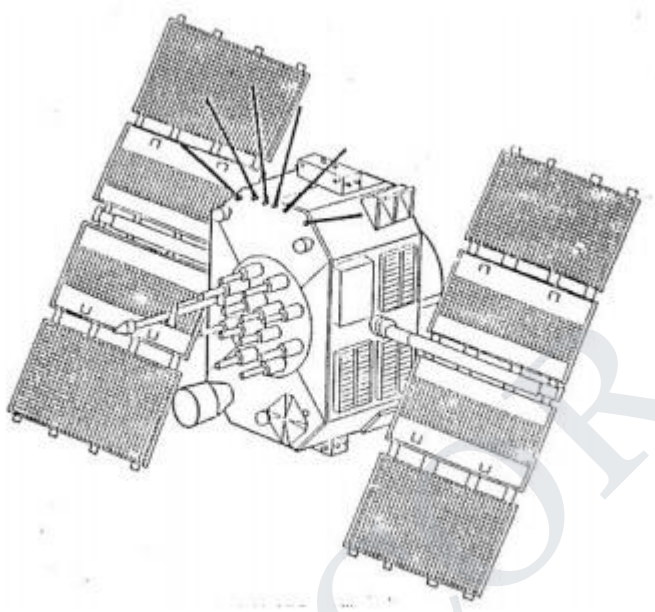


Fig 4.5 Schematic view of a Block II GPS satellite

Fig 4.5 Schematic view of a Block II GPS satellite

GPS satellites have a very high performance frequency standard with an accuracy of

between $1X10^{-12}$ to $1X10^{-13}$ and are thus capable of creating precise time base. Block-I satellites were partly equipped with only quartz oscillators but Block-II satellites have two cesium frequency standards and two rubidium frequency standards. Using fundamental frequency of 10.23 MHz, two carrier frequencies are generated to transmit signal codes.

OBSERVATION PRINCIPLE AND SIGNAL STRUCTURE

NAVSTAR GPS is a one-way ranging system i.e. signals are only transmitted by the satellite. Signal travel time between the satellite and the receiver is observed and the range distance is calculated through the knowledge of signal propagation velocity. One way ranging means that a clock reading at the transmitted antenna is compared with a clock reading at the receiver antenna. But since the two clocks are

not strictly synchronized, the observed signal travel time is biased with systematic synchronization error. Biased ranges are known as pseudoranges. Simultaneous observations of four pseudoranges are necessary to determine X, Y, Z coordinates of user antenna and clock bias.

Real time positioning through GPS signals is possible by modulating carrier frequency with Pseudorandom Noise (PRN) codes. These are sequence of binary values (zeros and ones or +1 and -1) having random character but identifiable distinctly. Thus pseudoranges are derived from travel time of an identified PRN signal code. Two different codes viz. P-code and C/A code are in use. P means precision or protected and C/A means clear/acquisition or coarse acquisition.

P- code has a frequency of 10.23 MHz. This refers to a sequence of 10.23 million binary digits or chips per second. This frequency is also referred to as the chipping rate of P-code. Wavelength corresponding to one chip is 29.30m. The P-code sequence is extremely long and repeats only after 266 days. Portions of seven days each are assigned to the various satellites. As a consequence, all satellite can transmit on the same frequency and can be identified by their unique one-week segment. This technique is also called as Code Division Multiple Access (CDMA). P-code is the primary code for navigation and is available on carrier frequencies L1 and L2.

The C/A code has a length of only one millisecond; its chipping rate is 1.023 MHz with corresponding wavelength of 300 meters. C/A code is only transmitted on L1 carrier.

GPS receiver normally has a copy of the code sequence for determining the signal propagation time. This code sequence is phase-shifted in time step- by-step and correlated with the received code signal until maximum correlation is achieved. The necessary phase-shift in the two sequences of codes is a measure of the signal travel time between the satellite and the receiver antennas. This technique can be explained as code phase observation.

For precise geodetic applications, the pseudoranges should be derived from phase measurements on the carrier signals because of much higher resolution. Problems of ambiguity determination are vital for such observations.

The third type of signal transmitted from a GPS satellite is the broadcast message sent at a rather slow rate of 50 bits per second (50 bps) and repeated every 30 seconds. Chip sequence of P-code and C/A code are separately combined with the stream of message bit by binary addition ie the same value for code and message chip gives 0 and different values result in 1.

The main features of all three signal types used in GPS observation viz carrier, code and data signals are given in Table 3.

GPS Satellite Signals

GPS Satellite Signals

Atomic Clock (G, Rb) fundamental	10.23. MHz
L1 Carrier Signal	154 X 10.23 MHz
L1 Frequency	1575.42 MHz
L1 Wave length	19.05 Cm
L2 Carrier Signal	120 X 10.23 MHz
L2 Frequency	1227.60 MHz
L2 Wave Length	24.45 Cm
P-Code Frequency (Chipping Rate)	10.23 MHz (Mbps)
P-Code Wavelength	29.31 M
P-Code Period	267 days : 7
C/A-Code Frequency (Chipping Rate)	1.023 MHz (Mbps)
C/A-Code Wavelength	293.1 M
C/A-Code Cycle Length	1 Milisecond
Data Signal Frequency	50 bps
Data Signal Cycle Length	30 Seconds

The signal structure permits both the phase and the phase shift (Doppler effect) to be measured along with the direct signal propagation. The necessary bandwidth is achieved by phase modulation of the PRN code as illustrated in Fig. 6.

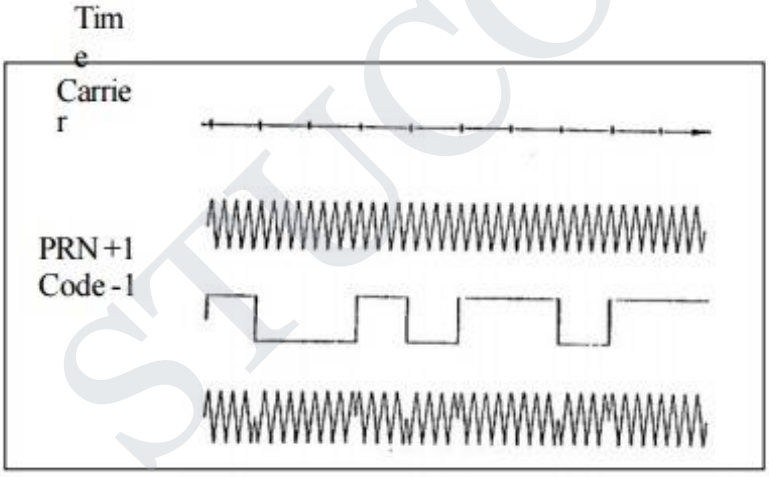


Fig 4.6 Generation of GPS Signals

Fig 4.6 Generation of GPS Signals

STRUCTURE OF THE GPS NAVIGATION DATA

Structure of GPS navigation data (message) is shown in Fig. 7. The user has to decode the data signal to get access to the navigation data. For on line navigation purposes, the internal processor within the receiver does the decoding. Most of the

manufacturers of GPS receiver provide decoding software for post processing purposes. With a bit rate of 50 bps and a cycle time of 30 seconds, the total information content of a navigation data set is 1500 bits. The complete data frame is subdivided into five subframes of six-second duration comprising 300 bits of information. Each subframe contains the data words of 30 bits each. Six of these are control bits. The first two words of each subframe are the Telemetry Work (TLM) and the C/A-P-Code Hand over Work (HOW). The TLM work contains a synchronization pattern, which facilitates the access to the navigation data. Since GPS is a military navigation system of US, a limited access to the total system accuracy is made available to the civilian users. The service available to the civilians is called Standard Positioning System (SPS) while the service available to the authorized users is called the Precise Positioning Service (PPS). Under current policy the accuracy available to SPS users is 100m, 2D- RMS and for PPS users it is 10 to 20 meters in 3D. Additional limitation viz. Anti-Spoofing (AS), and Selective Availability (SA) was further imposed for civilian users. Under AS, only authorized users will have the means to get access to the P-code. By imposing SA condition, positional accuracy from Block-II satellite was randomly offset for SPS users. Since May 1, 2000 according to declaration of US President, SA is switched off for all users.

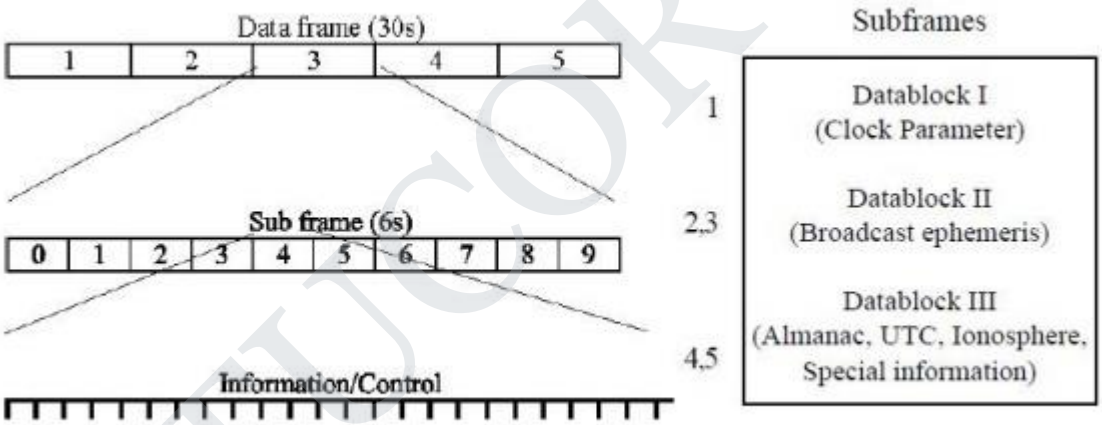


Fig 4.7 Data block

Fig 4.7 Data block

The navigation data record is divided into three data blocks:

Data Block I appears in the first subframe and contains the clock coefficient/bias.

Data Block II appears in the second and third subframe and contains all necessary parameters for the computation of the satellite coordinates.

Data Block III appears in the fourth and fifth subframes and contains the almanac data with clock and ephemeris parameter for all available satellite of the GPS

system. This data block includes also ionospheric correction parameters and particular alphanumeric information for authorized users.

Unlike the first two blocks, the subframe four and five are not repeated every 30 seconds.

International Limitation of the System Accuracy

The GPS system time is defined by the cesium oscillator at a selected monitor station. However, no clock parameter are derived for this station. GPS time is indicated by a week number and the number of seconds since the beginning of the current week. GPS time thus varies between 0 at the beginning of a week to 6,04,800 at the end of the week. The initial GPS epoch is January 5, 1980 at 0 hours Universal Time. Hence, GPS week starts at Midnight (UT) between Saturday and Sunday. The GPS time is a continuous time scale and is defined by the main clock at the Master Control Station (MCS). The leap seconds is UTC time scale and the drift in the MCS clock indicate that GPS time and UTC are not identical. The difference is continuously monitored by the control segment and is broadcast to the users in the navigation message. Difference of about 7 seconds was observed in July, 1992.

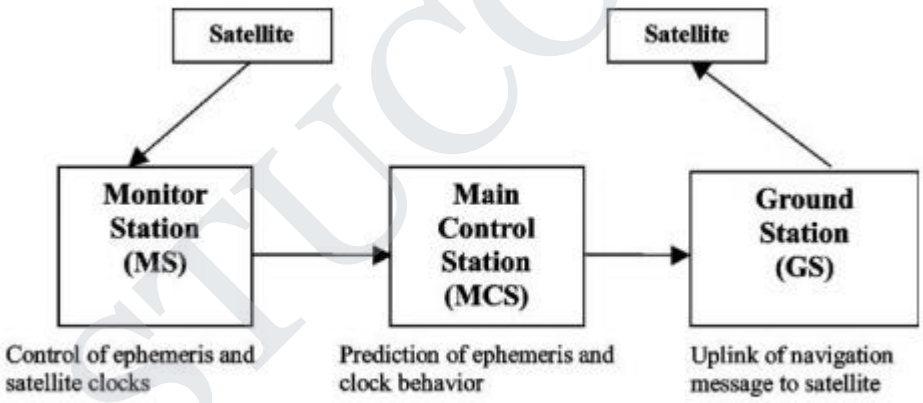


Figure 4.8 Data Flow in the determination of the broadcast ephemeris

Figure 4.8 Data Flow in the determination of the broadcast ephemeris

GPS satellite is identified by two different numbering schemes. Based on launch sequence, SVN (Space Vehicle Number) or NAVSTAR number is allocated. PRN (Pseudo Random Noise) or SVID (Space Vehicle Identification) number is related to orbit arrangement and the particular PRN segment allocated to the individual satellite. Usually the GPS receiver displays PRN number.

CONTROL SEGMENT

Control segment is the vital link in GPS technology. Main functions of the control segment.

- Monitoring and controlling the satellite system continuously
- Determine GPS system time
- Predict the satellite ephemeris and the behavior of each satellite clock.
- Update periodically the navigation message for each particular satellite.

For continuos monitoring and controlling GPS satellites a master control stations (MCS), several monitor stations (MS) and ground antennas (GA) are located around the world (Fig. 9). The operational control segment (OCS) consists of MCS near Colorado springs (USA), three MS and GA in Kwajaleian Ascension and Diego Garcia and two more MS at Colorado Spring and Hawai.

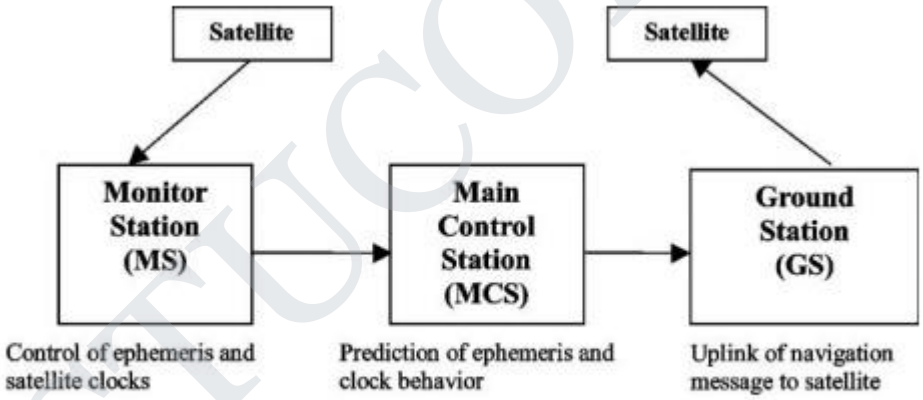


Figure 4.8 Data Flow in the determination of the broadcast ephemeris

GROUND CONTROL SEGMENT

The monitor station receives all visible satellite signals and determines their pseudorages and then transmits the range data along with the localmeteorological data via data link to the master control stations. MCS then precomputes satellite ephemeris and the behaviour of the satellite clocks and formulates the navigation data. The navigation message data are transmitted to the ground antennas and via S-band it links to the satellites in view. Fig. 9 shows this process schematically. Due to systematic global distribution of upload antennas, it is possible to have atleast three contacts per day between the control segment and each satellite.

USER SEGMENT

Appropriate GPS receivers are required to receive signal from GPS satellites for the purpose of navigation or positioning. Since, GPS is still in its development phase, many rapid advancements have completely eliminated bulky first generation user equipments and now miniature powerful models are frequently appearing in the market.

BASIC CONCEPT OF GPS RECEIVER AND ITS COMPONENTS

The main components of a GPS receiver are shown in Fig. 10. These are:

- Antenna with pre-amplifier
- RF section with signal identification and signal processing
- Micro-processor for receiver control, data sampling and data processing
- Precision oscillator
- Power supply
- User interface, command and display panel
- Memory, data storage

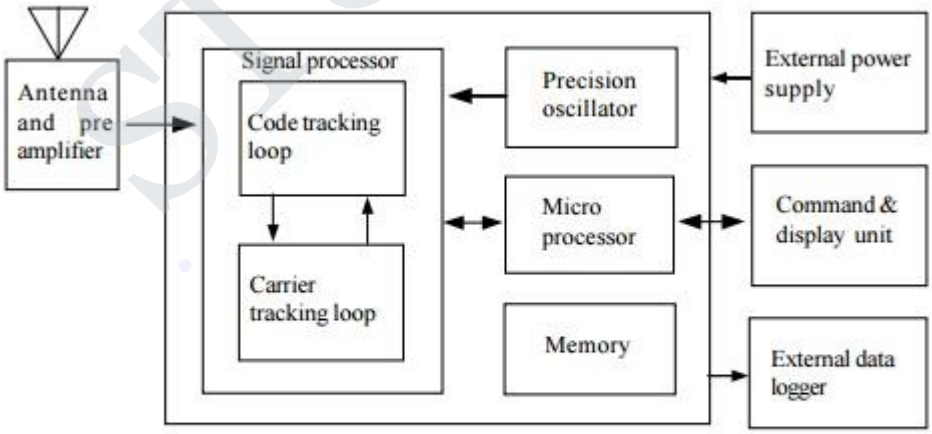
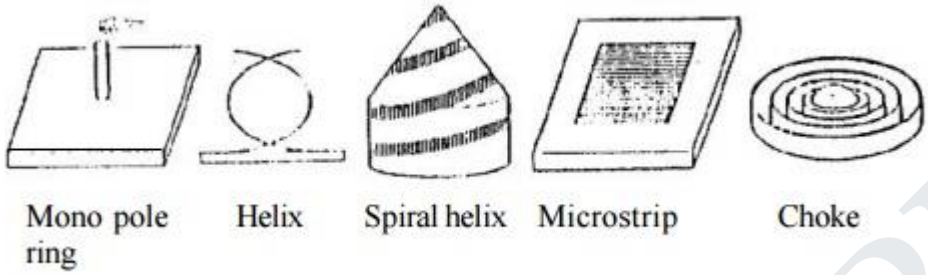


Fig 4.9 Major components of a GPS receiver

ANTENNA

Sensitive antenna of the GPS receiver detects the electromagnetic wave signal transmitted by GPS satellites and converts the wave energy to electric current] amplifies the signal strength and sends them to receiver electronics.

Several types of GPS antennas in use are mostly of following types (Fig.).



Mono pole Helix Spiral helix Microstrip Choke ring

Types of GPS Antenna

- Mono pole or dipole
- Quadrifilar helix (Volute)
- Spiral helix
- Microstrip (patch)
- Choke ring

Microstrip antennas are most frequently used because of its added advantage for airborne application, materialization of GPS receiver and easy construction. However, for geodetic needs, antennas are designed to receive both carrier frequencies L1 and L2. Also they are protected against multipath by extra ground planes or by using choke rings. A choke ring consists of strips of conductor which are concentric with the vertical axis of the antenna and connected to the ground plate which in turns reduces the multipath effect.

RF Section with Signal Identification and Processing

The incoming GPS signals are down converted to a lower frequency in the RS section and processed within one or more channels. Receiver channel is the primary electronic unit of a GPS receiver. A receiver may have one or more channels. In the parallel channel concept each channel is continuouslymfranking one particular

satellite. A minimum of four parallel channels is required to determine position and time. Modern receivers contain upto 12 channels for each frequency.

In the sequencing channel concept the channel switches from satellite to satellite at regular interval. A single channel receiver takes atleast four times of 30 seconds to establish first position fix, though some receiver types have a dedicated channel for reading the data signal. Now days in most of the cases fast sequencing channels with a switching rate of about one-second per satellite are used.

In multiplexing channel, sequencing at a very high speed between different satellites is achieved using one or both frequencies. The switching rate is synchronous with the navigation message of 50 bps or 20 milliseconds per bit. A complete sequence with four satellites is completed by 20 millisecond or after 40 millisecond for dual frequency receivers. The navigation message is continuous, hence first fix is achieved after about 30 seconds.

Though continuous tracking parallel channels are cheap and give good overall performance, GPS receivers based on multiplexing technology will soon be available at a cheaper price due to electronic boom.

Microprocessor

` To control the operation of a GPS receiver, a microprocessor is essential for acquiring the signals, processing of the signal and the decoding of the broadcast message. Additional capabilities of computation of on-line position and velocity, conversion into a given local datum or the determination of waypoint information are also required. In future more and more user relevant software will be resident on miniaturized memory chips.

Precision Oscillator

A reference frequency in the receiver is generated by the precision oscillator. Normally, less expensive, low performance quartz oscillator is used in receivers since the precise clock information is obtained from the GPS satellites and the user clock error can be eliminated through double differencing technique when all participating receivers observe at exactly the same epoch. For navigation with two or three satellites only an external high precision oscillator is used.

Power Supply

First generation GPS receivers consumed very high power, but modern receivers are designed to consume as little energy as possible. Most receivers have an internal rechargeable. Nickel-Cadmium battery in addition to an external power input. Caution

of low battery signal prompts the user to ensure adequate arrangement of power supply.

Memory Capacity

For port processing purposes all data have to be stored on internal or external memory devices. Post processing is essential for multi station techniques applicable to geodatic and surveying problems. GPS observation for pseudoranges, phase data, time and navigation message data have to be recorded. Based on sampling rate, it amount to about 1.5 Mbytes of data per hour for six satellites and 1 second data for dual frequency receivers. Modern receivers have internal memories of 5 Mbytes or more. Some receivers store the data on magnetic tape or on a floppy disk or hard-disk using external microcomputer connected through RS-232 port.

Most modern receivers have a keypad and a display for communication between the user and the receivers. The keypad is used to enter commands, external data like station number or antenna height or to select a menu operation. The display indicates computed coordinates, visible satellites, data quality indices and other suitable information. Current operation software packages are menu driven and very user friendly.

CLASSIFICATION OF GPS RECEIVERS

GPS receivers can be divided into various groups according to different criteria. In the early stages two basic technologies were used as the classification criteria viz. Code correlation receiver technology and sequencing receiver technology, which were equivalent to code dependent receivers and code free receivers. However, this kind of division is no longer justifiable since both techniques are implemented in present receivers.

Another classification of GPS receivers is based on acquisition of data types

e.g.

- C/A code receiver
- C/A code + L1 Carrier phase
- C/A code + L1 Carrier phase + L2 Carrier phase
- C/A code + p_code + L1, L2 Carrier phase

- L1 Carrier phase (not very common)
- L1, L2 Carrier phase (rarely used)

Based on technical realization of channel, the GPS receivers can be classified as:

- Multi-channel receiver
- Sequential receiver
- Multiplexing receiver

GPS receivers are even classified on the purpose as:

- Military receiver
- Civilian receiver
- Navigation receiver
- Timing receiver
- Geodetic receiver

For geodetic application it is essential to use the carrier phase data as observable. Use of L1 and L2 frequency is also essential along with P-code.

Examples of GPS Receiver

GPS receiver market is developing and expanding at a very high speed. Receivers are becoming powerful, cheap and smaller in size. It is not possible to give details of every make but description of some typical receivers given may be regarded as a basis for the evaluation of future search and study of GPS receivers.

Classical Receivers

Detailed description of code dependent T1 4100 GPS Navigator and code free Macrometer V1000 is given here:
T1 4100 GPS Navigator was manufactured by Texas Instrument in 1984. It was the first GPS receiver to provide C/A and P code and L1 and L2 carrier phase observations. It is a dual frequency multiplexing receiver and suitable for geodesist, surveyor and navigators. The observables through it are:

- P-Code pseudo ranges on L1 and L2
- C/A-Code pseudo ranges on L1
- Carrier phase on L1 and L2

The data are recorded by an external tape recorder on digital cassettes or are downloaded directly to an external microprocessor. A hand held control display unit (CDU) is used for communication between observer and the receiver. For navigational purposes the built in microprocessor provides position and velocity in real time every three seconds. T1 4100 is a bulky instrument weighing about 33 kg and can be packed in two transportation cases. It consumes 90 watts energy in operating mode of 22V - 32V. Generator use is recommended. The observation noise in P-Code is between 0.6 to 1 m, in C/ A code it ranges between 6 to 10 m and for carrier phase it is between 2 to 3 m.

T1 4100 has been widely used in numerous scientific and applied GPS projects and is still in use. The main disadvantages of the T1 4100 compared to more modern GPS equipment's are

- Bulky size of the equipment
- High power consumption
- Difficult operation procedure
- Limitation of tracking four satellites simultaneously
- High noise level in phase measurements

Sensitivity of its antenna for multipath and phase centre variation if two receivers are connected to one antenna and tracking of seven satellites simultaneously is possible. For long distances and in scientific projects, T1 4100 is still regarded useful. However, due to imposition of restriction on P- code for civilian, T1 4100 during Anti Spoofing (AS) activation can only be used as a single frequency C/A code receiver.

The MACROMETER V 1000, a code free GPS receiver was introduced in 1982 and was the first receiver for geodetic applications. Precise results obtained through it has demonstrated the potential of highly accurate GPS phase observations. It is a single frequency receiver and tracks 6 satellites on 6 parallel channels. The complete system consists of three units viz.

- Receiver and recorder with power supply

- Antenna with large ground plane

- P 1000 processor

The processor is essential for providing the almanac data because the Macrometer V 1000 cannot decode the satellite messages and process the data. At pre determined epoches the phase differences between the received carrier signal and a reference signal from receiver oscillator is measured. A typical baseline accuracy reported for upto 100 km distance is about 1 to 2 ppm (Parts per million).

Macrometer II, a dual frequency version was introduced in 1985. Though it is comparable to Macrometer V 1000, its power consumption and weight are much less. Both systems require external ephemeredes. Hence specialized operators of few companies are capable of using it and it is required to synchronize the clock of all the instruments proposed to be used for a particular observation session. To overcome above disadvantages, the dual frequency Macrometer II was further miniaturized and combined with a single frequency C/A code receiver with a brand name MINIMAC in 1986, thus becoming a code dependent receiver.

Examples of present Geodetic GPS Receivers

Few of the currently available GPS receivers that are used in geodesy surveying and precise navigation are described. Nearly all models started as single frequency C/A-Code receivers with four channels. Later L2 carrier phase was added and tracking capability was increased. Now a days all leading manufacturers have gone for code-less, non- sequencing L2 technique. WILD/ LEITZ (Heerbrugg, Switzerland) and MAGNAVOX (Torrance, California) have jointly developed WM 101 geodetic receiver in 1986. It is a four channel L1 C/A code receiver. Three of the channels sequentially track upto six satellites and the fourth channel, a house keeping channels, collects the satellite message and periodically calibrates the inter channel biases. C/A-code and reconstructed L1 carrier phase data are observed once per second.

The dual frequency WM 102 was marketed in 1988 with following key features:

- L1 reception with seven C/A code channel tracking upto six satellites simultaneously.

- L2 reception of up to six satellites with one sequencing P- code channel Modified sequencing technique for receiving L2 when P-code signals are encrypted.

The observations can be recorded on built in data cassettes or can be transferred on line to an external data logger in RS 232 or RS 422 interface.

Communication between operator and receiver is established by alpha numerical control panel and display WM 101/102 has a large variety of receiver resident menu driven options and it is accompanied by comprehensive post processing software.

In 1991, WILD GPS system 200 was introduced. Its hardware comprises the Magnavox SR 299 dual frequency GPS sensor, the hand held CR 233 GPS controller and a Nicd battery. Plug in memory cards provide the recording medium. It can track 9 satellites simultaneously on L1 and L2. Reconstruction of carrier phase on L1 is through C/A code and on L2 through P-code. The receiver automatically switches to codeless L2 when P-code is encrypted. It consumes 8.5 watt through 12-volt power supply.

TRIMBLE NAVIGATION (Sunny vale, California) has been producing TRIMBLE 4000 series since 1985. The first generation receiver was a L1 C/ A code receiver with five parallel channels providing tracking of 5 satellites simultaneously. Further upgradation included increasing the number of channels upto tweleve, L2 sequencing capability and P-code capability. TRIMBLE Geodatic Surveyor 4000 SSE is the most advanced model. When P-Code is available, it can perform following types of observations, viz.,

- Full cycle L1 and L2 phase measurements
- L1 and L2, P-Code measurements when AS is on and P-code is encrypted
- Full cycle L1 and L2 phase measurement
- Low noise L1, C/A code
- Cross-correlated Y-Code data

Observation noise of the carrier phase measurement when P-code is available is about ⊕ 0-2mm and of the P-code pseudoranges as low as ⊕ 2cm. Therefore, it is very suitable for fast ambiguity solution techniques with code/ carrier combinations.

ASHTECH (Sunnyvale, California) developed a GPS receiver with 12 parallel channels and pioneered current multi-channel technology. ASHTECH XII GPS receiver was introduced in 1988. It is capable of measuring pseudoranges, carrier phase and integrated dopler of up to 12 satellites on L1. The pseudoranges measurement are smoothed with integrated Doppler. Postion velociy, time and navigation informations are displayed on a keyboard with a 40-characters display. L2 option adds 12 physical L2 squaring type channels.

ASHTECH XII GPS receiver is a most advanced system, easy to handle and does not require initialization procedures. Measurements of all satellites in view are carried out automatically. Data can be stored in the internal solid plate memory of 5 Mbytes capacity. The minimum sampling interval is 0.5 seconds. Like many other receivers it has following additional options viz.

- 1 ppm timing signal output
- Photogrammetric camera input
- Way point navigation
- Real time differential navigation and provision of port processing and vision planning software

In 1991, ASHTECH P-12 GPS receiver was marketed. It has 12 dedicated channels of L1, P-code and carrier and 12 dedicated channels of L2, P-code and carrier. It also has 12 L1, C/A code and carrier channels and 12 code less squaring L2 channels. Thus the receiver contains 48 channels and provides all possibilities of observations to all visible satellites. The signal to noise level for phase measurement on L2 is only slightly less than on L1 and significantly better than with code-less techniques. In cases of activated P-code encryption, the code less L2 option can be used.

TURBO ROGUE SNR-8000 is a portable receiver weighing around 4 kg, consumes 15-watt energy and is suitable for field use. It has 8 parallel channels on L1 and L2. It provides code and phase data on both frequencies and has a codeless option. Full P-code tracking provides highest precision phase and pseudo rages measurements, codeless tracking is automatic 'full back' mode. The code less mode uses the fact that each carrier has identical modulation of P-code/Y-code and hence the L1 signal can be cross-correlated with the L2 signal. Results are the differential phase measurement (L1-L2) and the group delay measurement (P1-P2)

Accuracy specifications are :
P-Code pseudo range 1cm (5 minutes integration) Codeless pseudo range 10cm (5 minutes integration) Carrier phase 0.2 - 0.3 mm
Codeless phase 0.2 - 0.7 mm

One of the important features is that less than 1 cycle slip is expected for 100 satellite hours.

Navigation Receivers

Navigation receivers are rapidly picking up the market. In most cases a single C/A code sequencing or multiplexing channel is used. However, modules with four or five

parallel channels are becoming increasingly popular. Position and velocity are derived from C/A code pseudoranges measurement and are displayed or downloaded to a personal computer. Usually neither raw data nor carrier phase information is available. Differential navigation is possible with some advanced models.

MAGELLAN NAV 1000 is a handheld GPS receiver and weighs only 850 grams. It was introduced in 1989 and later in 1990, NAV 1000 PRO model was launched. It is a single channel receiver and tracks 3 to 4 satellites with a
2.5 seconds update rate and has a RS 232 data port.

The follow up model in 1991 was NAV 5000 PRO. It is a 5-channel receiver tracking all visible satellites with a 1-second update rate. Differential navigation is possible. Carrier phase data can be used with an optional carrier phase module. The quadrifilar antenna is integrated to the receiver. Post processing of data is also possible using surveying receiver like ASHTECH XII located at a reference station. Relative accuracy is about 3 to 5 metres. This is in many cases sufficient for thematic purposes.
Many hand held navigation receivers are available with added features. The latest market situation can be obtained through journals like GPS world etc.

For most navigation purpose a single frequency C/A code receiver is sufficient. For accuracy requirements better than 50 to 100 meters, a differential option is essential. For requirement below 5 meters, the inclusion of carrier phase data is necessary. In high precision navigation the use of a pair of receivers with full geodetic capability is advisable. The main characteristics of multipurpose geodetic receiver are summarized in Table 4.

Table 4. Overview of geodetic dual-frequency GPS satellite receiver (1992)

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Receiver	Channel		Code		Wavelen		Anti-spoofing
	L1	L2	L1	L2	L1	L2	
TI 4100	4	4	P	P			Single
MACROMET	6	6	-	-		/2	No influence
ASHTECH	12	12	C/A	-		/2	No influence
ASHTECH P	12	12	C/A,	P			Squaring
TRIMBLE	8-12	8-12	C/A	-		/2	No influence
TRIMBLE	9-12	9-12	C/A,	P			Codeless SSE
WM 102	7	1	C/A	P			Squaring
WILD GPS	9	9	C/A	p			Codeless
TURBO	8	8	C/A,	P			Codeless

Some of the important features for selecting a geodetic receiver are :

- Tracking of all satellites
- Both frequencies
- Full wavelength on L2
- Low phase noise-low code noise
- High sampling rate for L1 and L2
- High memory capacity
- Low power consumption
- Full operational capability under anti spoofing condition

Further, it is recommended to use dual frequency receiver to minimize ion-spherical influences and take advantages in ambiguity solution.

GPS Surveying Differential Theory

Differential positioning is technique that allows overcoming the effects of environmental errors and SA on the GPS signals to produce a highly accurate position fix.

ACCURACY

In general, an SPS receiver can provide position information with an error of less than 25 meter and velocity information with an error less than 5 meters per second. Upto 2 May 2000 U.S Government has activated Selective Availability (SA) to maintain optimum militar y effectiveness. Selective Availability inserts random errors into the ephemeris information broadcast by the satellites, which reduces the SPS accuracy to around 100 meters.

For many applications, 100-meter accuracy is more than acceptable. For applications that require much greater accuracy, the effects of SA and environmentally produced errors can be overcome by using a technique called Differential GPS (DGPS), which increases overall accuracy.

DIFFERENTIAL THEORY

Differential positioning is technique that allows overcoming the effects of environmental errors and SA on the GPS signals to produce a highly accurate position fix. This is done by determining the amount of the positioning error and applying it to position fixes that were computed from collected data.

Typically, the horizontal accuracy of a single position fix from a GPS receiver is 15 meter RMS (root-mean Square) or better. If the distribution of fixes about the true position is circular normal with zero mean, an accuracy of 15 meters RMS implies that about 63% of the fixes obtained during a session are within 15 meters of the true position.

TYPES OF ERRORS

There are two types of positioning errors: correctable and non-correctable. Correctable errors are the errors that are essentially the same for two GPS receivers in the same area. Non-correctable errors cannot be correlated between two GPS receivers in the same area.

CORRECTABLE ERRORS

Sources of correctable errors include satellite clock, ephemeris data and ionosphere and tropospheric delay. If implemented, SA may also cause a correctable positioning error. Clock errors and ephemeris errors originate with the GPS satellite. A clock error is a slowly changing error that appears as a bias on the pseudorange measurement made by a receiver. An ephemeris error is a residual error in the data used by a receiver to locate a satellite in space.

Ionosphere delay errors and tropospheric delay errors are caused by atmospheric conditions. Ionospheric delay is caused by the density of electrons in the ionosphere along the signal path. A tropspheric delay is related to humidity, temperature, and altitude along the signal path. Usually, a tropospheric error is smaller than an ionospheric error.

Another correctable error is caused by SA which is used by U.SDepartment of Defence to introduce errors into Standard Positioning Service (SPS) GPS signals to degrade fix accuracy.

The amount of error and direction of the error at any given time does not change rapidly. Therefore, two GPS receivers that are sufficiently close together will observe the same fix error, and the size of the fix error can be determined.

NON-CORRECTABLE ERRORS

Non-correctable errors cannot be correlated between two GPS receivers that are located in the same general area. Sources of non-correctable errors include receiver noise, which is unavoidably inherent in any receiver, and multipath errors, which are environmental. Multi-path errors are caused by the receiver 'seeing' reflections of signals that have bounced off of surrounding objects. The sub-meter antenna is multipath-resistant; its use is required when logging carrier phase data. Neither error can be eliminated with differential, but they can be reduced substantially with position fix averaging. The error sources and the approximate RMS error range are given in the Table.

Error Sources

Error Sources

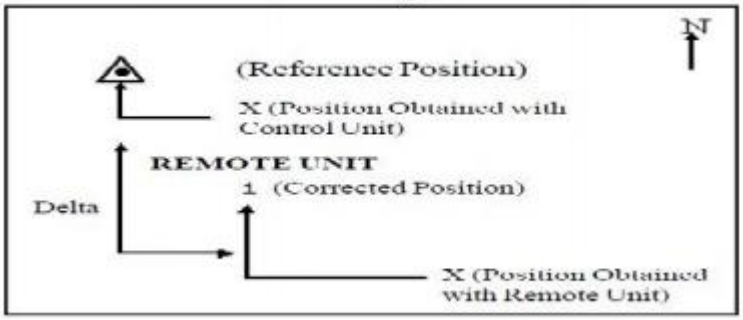
Error Source	Approx. Equivalent Range Error (RMS) in meters
Correctable with Differential	
Clock (Space Segment)	3.0
Ephemeris (Control Segment)	2.7
Ionospheric Delay (Atmosphere)	8.2
Tropospheric Delay (Atmosphere)	1.8
Selective Availability (if implemented)	27.4
Total	28.9
Non-Correctable with Differential	
Receiver Noise (Unit)	9.1
Multipath (Environmental)	3.0
Total	9.6
Total user Equivalent range error (all sources)	30.5
Navigational Accuracy (HDOP = 1.5)	45.8

DIFFERENTIAL GPS

Most DGPS techniques use a GPS receiver at a geodetic control site whose position is known. The receiver collects positioning information and calculates a position fix, which is then compared to the known co-ordinates. The difference between the known position and the acquired position of the control location is the positioning error.

Because the other GPS receivers in the area are assumed to be operating under similar conditions, it is assumed that the position fixes acquired by other receivers in the area (remote units) are subject to the same error, and that the correction computed for the control position should therefore be accurate for those receivers. The correction

is communicated to the remote units by an operator at the control site with radio or cellular equipment. In post-processed differential, all units collect data for off-site processing; no corrections are determined in the field. The process of correcting the position error with differential mode is shown in the Figure .



The difference between the known position and acquired position at the control point is the DELTA correction. DELTA, which is always expressed in meters, is parallel to the surface of the earth. When expressed in local co- ordinate system, DELTA uses North-South axis (y) and an East-West axis (x) in 2D operation; an additional vertical axis (z) that is perpendicular to the y and x is used in 3D operation for altitude.

Applications of GPS

z Providing Geodetic control.

z Survey control for Photogrammetric control surveys and mapping.

z Finding out location of offshore drilling.
z Pipeline and Power line survey.
z Navigation of civilian ships and planes. z Crustal movement studies.

z Geophysical positioning, mineral exploration and mining. z Determination of a precise geoid using GPS data.

z Estimating gravity anomalies using GPS.
z Offshore positioning: shiping, offshore platforms, fishing boats etc.

Astronomical observation of celestial bodies was one of the standard methods of obtaining coordinates of a position. This method is prone to visibility and weather condition and demands expert handling. Attempts have been made by USA since early 1960's to use space based artificial satellites. System TRANSIT was widely used for establishing a network of control points over large regions. Establishment of modern

geocentric datum and its relation to local datum was successfully achieved through TRANSI T. Rapid improvements in higher frequently transmission and precise clock signals along with advanced stable satellite technology have been instrumental for the development of global positioning system.

The NAVSTAR GPS (Navigation System with Time and Ranging Global Positioning System) is a satellite based radio navigation system providing precise three-dimensional position, course and time information to suitably equipped user.