Chapter 3
Surveying Measurements

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3.1 General

Basic field operations performed by a surveyor involve linear and angular measurements. Through application of mathematics (geometry and trigonometry) and spatial information knowledge, the surveyor converts these measurements to the horizontal and vertical relationships necessary to produce maps, plans of engineering projects, or Geographical Information System/Land Information System (GIS/LIS).

The highway surveyor must be adept at making the required measurements to the degree of accuracy required. Various types of engineering works require various tolerances in the precision of the measurements made and the accuracies achieved by these measurements.

The use of common sense and development of good surveying practice in all phases of a survey cannot be overemphasized. All conditions that may be encountered in the "real world" during the actual field survey cannot be covered in any manual. A manual may specify certain techniques, such as a certain number of repeated operations, to achieve a required accuracy. The surveyor must then often use judgment based on the equipment being used and the field conditions encountered, to modify those techniques. Some field conditions (heat waves or wind for example) may make it impossible to perform some operations to a consistent degree of accuracy.

3.2 Accuracy and Precision

3.2.1 Accuracy

Accuracy is the degree of conformity with a standard or accepted value. Accuracy relates to the quality of the result. It is distinguished from precision that relates to the quality of the operation used to obtain the result. The standard used to determine accuracy can be:

A. An exact known value, such as the sum of the three interior angles of a plane triangle is 180°.

B. A value of a conventional unit as defined by a physical representation thereof, such as the international meter.

C. A survey or map value determined by superior methods and deemed sufficiently near the ideal or true value to be held constant for the control of dependent operations.

Although they are known to be not exact, higher order NGS control points are deemed of sufficient accuracy to be the control for all other less exact surveys.

3.2.2 Precision

Precision is the degree of refinement in the performance of an operation (procedures and instrumentation) or in the statement of a result. It is a measure of the uniformity or reproducibility of the result.

3.2.3 Accuracy Versus Precision

The accuracy of a field survey depends directly upon the precision of the survey. Although through luck (compensating errors, for example) surveys with high order closures might be attained without high order precision, such accuracies are meaningless. Therefore, all measurements and results should be quoted in terms that are commensurate with the precision used to attain them. Similarly, all surveys must be performed with a precision that ensures that the desired accuracy is attained. However, surveys performed to a precision that excessively exceeds the requirements are costly and should be avoided.

3.3 Errors and Classification of Accuracy

3.3.1 General

Statistically speaking, field observations and the resulting measurement are never exact. Any observation can contain various types of errors. Often some of these errors are known and can be eliminated by applying appropriate corrections. However, even after all known errors are eliminated, a measurement will still be in error by some unknown value. To minimize the effect of errors, the surveyor has to use utmost care in making the observations and utilizing only calibrated equipment. However, a measurement is never exact, regardless of the precision of the observations.

Although this manual contains many guidelines and standards, the ultimate responsibility for providing surveys that meet desired accuracies remains with the field personnel. To fulfill this responsibility, the crew chief and his or her assistants must understand errors, including but not limited to:

A. The various sources of errors.

B. The effect of possible errors upon each observation, each measurement, and the entire survey.

C. Economical procedures which will eliminate or minimize errors and result in surveys of the desired accuracies.

3.3.2 Blunders

Many textbooks on surveying refer to a blunder as a gross error. One can easily make a case for a blunder to be considered an error. However, a blunder is really an unpredictable gross mistake made by the surveying team. It is not a hidden error that will go unnoticed, but usually it becomes apparent that something is wrong with the measurements. Examples of blunders are:

- Transposing two numbers (in field notes or computer input.)
- Misplacing decimal point.
- Incorrect reading (i.e. the foot value on a leveling rod.)
- Inadvertently altering set instrument constants in the middle of a project.
- Placing sighting device or the instrument at a wrong point.
- Misunderstanding verbal instructions or reading announcements (call out).
- Neglecting to level an instrument.
- Using the incorrect coordinates or benchmark values.

Blunders are caused by carelessness, misunderstanding, confusion, or poor judgment. They are, for the most part, avoided by alertness, common sense, and good judgment.

Blunders are detected and eliminated by using proper procedures, such as:

- Checking each recorded and calculated value.
- Making independent and redundant measure check observations and measurements.
- Making redundant measurements that allow closure computation of sections of the entire survey.

Small blunders are more difficult to detect and correct especially if the number of redundant measurements is too small. Therefore, surveys must be carried out with sufficient redundancy to prevent a blunder from going undetected. All blunders must be eliminated prior to correcting and adjusting a survey for errors.

3.3.3 Definition of Error

Error is the difference, after blunders have been eliminated, between a measured or calculated value of a quantity and the true or established value of that quantity.

3.3.4 Types of Errors

Excluding gross errors, which were discussed above, there are two general types of errors, systematic and random.

3.3.4.1 Systematic Errors

A systematic error is an error that will always have the same magnitude and the same algebraic sign under the same conditions.

In most cases, systematic errors are caused by physical and natural conditions that vary in accordance with known mathematical or physical laws. Systematic errors are caused by:

- Equipment out of calibration
- Use of insufficiently accurate computation equations (too few terms in a series.)
- Failure to apply necessary geometric reductions of measurements.
- Failure to apply necessary reductions of measurements due to weather related conditions.
- Personal biases of the observer.
- Use of incorrect units (feet instead of meters.)

A systematic error of a single kind is cumulative. However, several kinds of systematic errors occurring in any one measurement could compensate for each other. Some examples of systematic errors are:

- EDM that measures 99.95 feet while indicating a measurement of 100.00 feet.
- Refraction in vertical angles.
- Observer’s tendency to sight on near or distant sights in a slightly different manner.

Although some systematic errors are difficult to detect, the surveyor must recognize the conditions that cause such errors. Once the conditions are known, the effect of these errors can be minimized as follows:

- Turning angles (with theodolite or total station) in direct and reverse modes.
- Balancing (maintaining similar distances between level and rod) foresights and backsights.
- Calibrating all surveying equipment.

- Calibrating EDM's yearly at a baseline calibration site.

When systematic errors cannot be eliminated by procedural changes, corrections are applied to the measurements. These corrections are documented in the user manuals of the equipment or in surveying textbooks.

Undeterminable systematic errors can also be modeled into the adjustment computation, but surveyors should not rely on this. They must eliminate all the known systematic errors prior to proceeding with any adjustment of the survey data.

3.3.4.2 Random (Accidental) Errors

A random error (or accidental error) is an error produced by irregular causes that are beyond the control of the observer. They do not follow any established rule which can be used to compute the error for a given condition or circumstance of the observation. The occurrence, magnitude, and algebraic sign of a random error is truly random and cannot be predicted. For a single measurement, it is the error remaining in the measurement after all possible systematic and gross errors are eliminated. An important characteristic of the random error is that if we repeat the same measurement many times, the sum of all these errors tends to be zero. This is yet another good reason to make extra measurements beyond the required minimum.

An example of a random error is the personal reading error of any scale. An observer estimates the final reading that can be either high or low in estimation since exactness cannot occur.

Unlike systematic errors, corrections for random errors cannot be computed directly. Random errors must be compensated by adjustments. The adjustment process computes adjusted observations for the actual ones in such a way that the remaining random errors are minimized. An example of such a process is computing an average distance from several measurements. The average represents the adjusted value for the distance for which the random error is minimized.

Random error obey the laws of chance or the random theory of statistics. Therefore, they are analyzed by applying the laws of probability. A complete discussion on the mathematical laws of probability is beyond the scope of this manual. The reference list at the beginning of this manual cites some excellent publications concerning the topic.

3.3.5 Sources of Error

Errors in measurements stem from three sources: personal, instrumental, and natural.

3.3.5.1 Personal Errors

Personal errors are caused by the physical limitations of the human senses of sight and touch. An example of a personal error is an error in the measured value of a horizontal angle, caused by the inability to hold a range pole perfectly in the direction of the plumb line. Personal errors can be either systematic or random. Personal systematic errors are caused by an observer tendency to react the same way under the same conditions. When there is no such tendency, the personal errors are considered to be random.

Common sense, self-calibration (estimating personal errors by experiments and experience) and attention to proper procedures generally keep such errors to a minimum.

3.3.5.2 Instrument Error

Instrumental errors are caused by imperfections in the design, construction, and adjustment of instruments and other equipment. Instruments can be calibrated to overcome these imperfections. Examples of instrument error are:

- Imperfect linear or angular scales.
- Instrument axes are not perfectly parallel or perpendicular to each other.
- Misalignment of various part of the instrument.
- Optical distortions causing “what you see is not exactly what you are supposed to see”.

Most instrumental errors are eliminated by using proper procedures, such as observing angles in direct and reverse modes, balancing foresights and back sights and repeating measurements. Since not all instrument errors can be eliminated by procedures, instruments must be periodically checked, tested and adjusted (or calibrated.) Instruments must be on a maintenance schedule to prevent inaccurate measurements.

3.3.5.3 Natural Errors

Natural errors result from natural physical conditions such as atmospheric pressure, temperature, humidity, gravity, wind, and atmospheric refraction. Examples of natural errors are:

- A steel tape whose length varies with changes in temperature.
- Sun spots activity and its impact on the ionosphere, hence on GPS surveying.

Natural errors are mostly systematic and should be corrected or modeled in the adjustment. Some natural errors such as the effect of curvature and refraction can be eliminated by a procedure. The leveling procedure to eliminate curvature and refraction corrections is to average foresights and backsights.

3.4 Observation vs. Measurement

An observation is a single, unadjusted determination of a linear or angular value. A single reading of an angle or a single reading of an EDM is an observation. An observed value is a quantity that is obtained by instrumental measurement of the quantity. A direct observation is an observation of the desired quantity while an indirect observation is a quantity computed from direct observations. For example, rod readings in leveling are direct observations and the elevation difference between two points that is computed from these rod readings is an indirect observation.

A measurement is the entire process of obtaining a desired quantity. A measurement entails performing a physical operation that usually consists of several more elementary operations such as preparations (instrument calibration and setup), pointing, matching, and comparing (reading). The result of these physical operations renders a numerical value that is called a “measurement”.

Surveys should be considered as measurements not as observations. With the advent of electronic readouts of linear or angular quantities everyone can make an observation. It requires a surveyor to make a measurement.

3.5 Linear Measurement

This section covers two methods of obtaining linear measurement: taping and EDM. Linear measurements with GPS are discussed in Chapter 4 of this manual.

3.5.1 Taping

EDM instruments have largely replaced steel tapes in practically all measurements, except lower order work, such as close staking out, building measurements, etc. Wherever feasible, distances over 30 meters (100 feet) should be measured with an electronic measuring device. Accurate distances under 30 meters can be obtained with a calibrated steel tape.

Many surveyors believe that third order accuracy is a natural result of taping a distance. This is not true. Taped measurements will produce a linear accuracy of one part in 7,500 and yield a position closure of one part in 5,000 only if correct procedures are used. Such correct procedures would include standardization of tape, application of temperature correction, application of correct tension (particularly if tape is suspended), correct horizontal and vertical alignment of tape, and careful plumbing procedures.

It is anticipated that taping will not be used in critical measurements and a detailed explanation of taping procedures will not be included in this discussion.

3.5.2 Electronic Distance Measuring

General Detailed operating instructions, instrument specifications and field adjustment information are included in the Operation Manual furnished with each instrument. The instrument manual should be kept with the instrument at all times.

Training Prior to performing field surveys, each operator should be thoroughly trained in the care and use of the measuring device and the allied equipment used therewith. The operator should be made fully aware of the instrument's limitations, possible causes of measurement errors, and have a thorough knowledge of the various functions performed by the instrument.

Checking Instrument When an instrument is received by a crew, whether new or transferred from another crew, the instrument should be checked on a known distance base line with the reflectors to be used with that instrument. All EDMs should be checked periodically, particularly prior to starting an important survey.

NGS calibrated base lines are currently available in different locations within the state. Each calibrated base line has permanent monuments set to test instruments at several distances. A description of the test areas is included in Appendix B. Caution should be exercised to insure use of the most current data. Current data is available through the NJ DOT Geodetic Survey office or the WWW site of NGS "www.ngs.noaa.gov"

Atmospheric Correction Atmospheric correction must be calculated and entered according to the instrument manufacturer's directions. Directions are usually supplied in the operator's manual for any instrument.

3.5.3 Common Sources of EDM Error

A. **Setting Up** The heavier EDM equipment puts an added strain on tripods and instrument stands. The tripods used to support EDM equipment should be sturdy and in good condition. Therefore, hinge and foot screws should be checked for tightness quite regularly.

B. **Tribrachs** Plumbing errors cannot be eliminated by measuring procedures. Therefore, tribrachs must be checked for adjustment (bubble and optical plummet) frequently. This includes not only the tribrachs used for the EDM instrument, but also those used with the reflectors. If a tribrach is accidentally bumped, dropped, or knocked over, it should be checked before any additional measurements are made.

C. **Range Pole Mounted Prisms** Range pole mounted prisms will seldom be used for measurements on the control traverse. When such prisms are used for tying in supplemental points and topographic features or staking out, an out of adjustment pole level can be the source of considerable error. Pole levels should be checked frequently and when in use should be attached securely to the pole in a position that can easily be viewed by the holder.

D. **Reflections From Extraneous Objects** Under most circumstances, an EDM measurement will be within the accuracy specified for that instrument even if the line of sight passes through leaves, fences, or other obstructions. However, such objects can sometimes reflect or interrupt the light rays and cause erroneous measurements. This occurs usually when the object is relatively close to the instrument. Roadside reflectors, windows, or other reflective objects in the path of or behind the prism can often cause erroneous measurements. When the line of sight cannot be cleared, such conditions should be recorded. Then, if poor closures result, those distances can be isolated and rechecked. When measuring various distances along a straight line, only one reflector should face the instrument. Otherwise, the instrument may be measuring to the wrong reflector.

E. **Light Wave Skip** All EDMs have the inherent capability of false readings due to light wave skips. The skip is generally in increments of 1, 10, 20 or more meters. Generally, the skip is of sufficient magnitude to alert the operator that an erroneous measurement is being made. However, at some distances the skip will be small and difficult to eliminate. Repeat measurements are often successful, but not always.

Quite often the small one meter skip will not be evident until the survey is closed. An analysis of the traverse can sometimes indicate the false measurement.

A routine method of checking for skip is by eccentric measurement. The reflector can be moved two or three feet on line and a check measurement taken. If the check measurement is near the eccentric difference, in all likelihood there is no skip present.

F. **Improper Prisms or Preset Prism Constants** Most EDM prisms have built in cross lens constants of 30 millimeters (0.098 feet). Each EDM manufacturer provides for their instruments' direct measurements by the combination of an internal adjustment within the instrument, and position of the prism in relation to the vertical axis of the prism.

In the final analysis, extreme caution should be exercised to make sure that the EDM prism constant setting coincides with that of the prisms being used. Be sure to check the EDM operator’s manual and the specifications for the type prisms currently being used.

3.5.4 Number of Measurements

The number of measurements that should be made is a function of the characteristics of the survey. NGS has a set of standards and specifications for control surveys and the American Congress on Surveying and Mapping/ American Land Title Association (ACSM/ALTA) has another set of specifications for land surveying work. Other organizations developed their own standards and specifications. These standards and specifications outline the number of linear measurements that are to be performed. Under some circumstances, the distance between two points may be determined only on one line and only in one direction. In other words, reciprocal measurements or measurements to eccentric points may not necessary. A good rule of thumb to follow is that any time an angular direction is measured to a prism pole with a total station, a distance should be measured as well. There is very little extra effort involved in measuring the extra distance while the redundant measurement can provide a valuable check for the quality of the survey.

All base and control measurements should be the average of at least three measurements made in the standard (normal) measurement mode. Measurements made to set construction control stakes or points of equal importance should also be made in the standard mode.

Measurements made for topographic surveys, spot elevations, etc., can be made in the tracking mode. In order for the tracking mode to operate at the speed required, the instrument rounds off the displayed measurement.

3.6 Angular Measurement

3.6.1 Purpose

Points on the ground or on a map are related to each other through a horizontal distance and a horizontal angle (or direction.) Horizontal angular measurements are made between survey lines to determine the angle between the lines. A horizontal angle is the difference between two measured directions. Horizontal angles are measured on a plane perpendicular to the vertical axis (plumb line).

Vertical angular measurements are measured to determine slope of survey lines from the horizontal plane (level line). When the vertical angle is applied to the slope distance, the horizontal and vertical distances may be calculated. Vertical angles are measured on a plane passing through the vertical axis perpendicular to the horizontal plane. In order to facilitate the trigonometric calculations of horizontal and vertical distance, the reference or zero angle is on the vertical axis directly above the instrument, which is termed the zenith angle.

In the United States, the sexagesimal system of angular measurement is used. In the sexagesimal system, there are 360° in the circumference of a circle. The basic unit is the degree (°), which is further divided into 60 minutes (60'), and the minute is subdivided into 60 seconds (60"), and decimals thereof. Other angular unit systems utilize 400^g (grads) or 2π Radians per complete circle (360°).

3.6.2 Terms

The following terms are defined specifically for angular measurement. Their meanings may differ slightly in other contexts.

A. A pointing consists of a single sighting and circle reading on a single object.

B. An angular observation is a single, unadjusted determination of the size of an angle. A single angular observation is derived by subtracting the value of a pointing on a reference object from the value of a pointing on an observed station.

C. An angular measurement is the final determination of the magnitude of an angle before adjustment. Minimum angular measurement is the mean of at least two observations, one in the direct mode and the other in the reverse mode.

D. A reference object (RO) is a survey point that is used as an initial sight for orientation when measuring horizontal angles and "directions". The term, RO, will be used interchangeably with backsight (BS) in this manual.

E. A direction is the value of a clockwise angle between a backsight and any other survey point. The reference direction to the backsight can be either arbitrary or set to a desired value.

F. Setting a position is the act of setting a specified horizontal circle reading while the telescope is pointed toward a reference object. Generally, either zero degrees or the calculated "back azimuth" is used.

G. A direct reading is with the telescope in the upright (normal) position. An inverted or reverse reading is with the telescope inverted or plunged.

H. Turning a position is the act of making one direct and one reverse observation on each survey point to which a direction is required.

I. A repetition is a single observation (in a series of observations) of a horizontal angle, made with a repeating theodolite. This type of theodolite is rarely used nowadays. Information about repeating theodolites can be found in surveying textbooks.

J. Indirect measurement of an angle is a computed value of the angle from other data. For example angles of a triangle can be computed from distance measurements of its three sides. Orientation of the triangle is established by selected sides whose directions are known or measured.

3.6.3 Errors, Corrections, and Precautions

Direct measurement of angles and line direction by total station, theodolite, compass, or transit is familiar to all surveyors. However, many surveyors are not completely familiar with specific procedures that will achieve specified results. This section discusses errors involved in angular measurements and outlines procedures that will enable the surveyor to achieve specified results.

As mentioned earlier, errors in a measurement stem from various sources. Generally, angular measurements can be impacted by four classes of errors. These four classes are: instrumental, personal, natural, and miscellaneous errors. In the subsequent sections these factors and how to minimize them are discussed in detail.

3.6.3.1 Instrumental Factors

A. **Adjustment** Adjustments should be made at regular intervals and particularly before work on any control survey is started. Such adjustments should be made under the most ideal conditions available, normally in the highway yard or shop. Adjustment should be done in accordance with the user's manual of the specific instrument.

B. **Servicing** Instruments requiring major adjustments should be serviced at an authorized repair shop as specified by the Survey Operations Manager.

C. **Level Bubbles and Optical Plummet** Normal measuring procedures do not compensate for maladjustment of either the plate bubble(s) or the optical plummet. These components must be checked more frequently than others.

On base and control traverse projects, the optical plummet should be checked at least once each day. The plate bubbles should be routinely checked on each setup.

D. **Double Centering** Double centering compensates for lack of adjustment of almost all components of the instrument and should be standard practice for all angles measured (or laid off with a transit). Double centering consists of two repetitions (one direct and one reverse) with a transit.

E. **Parallax** Parallax occurs when the focal point of the eyepiece does not coincide with the plane of the cross hairs. The condition varies for each observer because the focal length depends in part on the shape of the observer's eyeball. Parallax is also a major concern in the optical plummet.

1. **When to Check** Parallax should be checked by each instrument person when beginning to operate a new instrument or one that has been operated by someone else. The optical plummet should be checked on every setup, particularly if the instrument height is significantly different from the last setup.

2. **How to Check** Focus the telescope on some well defined distant object. Slowly move the head back and forth, about an inch from the eyepiece, while watching the relationship of the object to the cross hairs. If the object appears to move, parallax exists.

3. **Eliminate** Rotate the knurled eyepiece ring until apparent object movement is no longer present. It may be necessary to refocus the cross hairs.

3.6.3.2 Personal Factors

A. Setting up Instrument

1. Be sure the tripod is in good condition and all hardware is snugly fitted.

2. Push the tripod shoes firmly into the ground. Pressure should be parallel to each leg. Keep your foot lightly on foot piece when adjusting leg lengths.

3. Place the legs in a position that will require a minimum of walking around the setup. In windy conditions, additional stability can be achieved if one leg is set downwind.

4. If the ground is soft or muddy, drive long 50 x 100 millimeter (2" x 4") wedges or iron pipes 19 millimeters x 1 meter (3/4" x 36") in the ground to support the tripod legs. Use duck boards to support the instrument man.

5. On warm asphalt pavement set the tripod shoes on stakes that have been nailed to the pavement. Shading tripod feet from direct sun may also be helpful.

6. Be sure that the instrument is exactly over the point.

7. Check the optical plummet after the instrument is set up and just before moving to another point. If the instrument has moved, check the angle just measured.

8. Recheck the instrument level. The bubble should hold one position when the instrument is smoothly turned through one circle.

9. Protect the instrument from direct exposure to the sun. Use a parasol if necessary.

B. Setting Sights

1. When tribrach mounted targets are used, take the same precautions as when setting up an instrument. With this equipment, "forced centering" between targets and theodolite (and vice versa) will greatly decrease the effects of plumbing errors in traverse closures. Forced centering is especially beneficial in short course traverses.

   Forced centering is the traverse procedure whereby backsight, instrument point and foresight are "leapfrogged". Once a tribrach is set over a point, it must stay mounted on the tripod over that point for all uses. The instrument and sights are transferred from point to point without disturbing the tribrach setup.

2. Before picking up the instrument or the target, check to see that the tribrach or the sighting device has not moved.

3. When setting a pole sight, plumb it with a precision equal to that required for the total survey. A twenty second error results from a sight that is 0.1 meters (0.32 feet) out of plumb at 1,000 meters (3280.8 feet) away.

4. If a sight is set near ground level, check the line of sight for obstructions or for excessive heat waves. When excessive heat waves are present, ground level sights are not advisable.

C. Pointing

1. **Tangent screw use** When moving sighting device onto a target, always make the last turn of the tangent screw clockwise. Clockwise movement increases the tension on the loading springs. A final turn counterclockwise releases tension and the spring can hang up, causing a “backlash" error.

2. **Cross Hair Use**

   a. **Consistency** Sight each object with the same part of the cross hair, preferably near the center of the field of view. This practice will minimize small residual adjustment errors.

   b. **Technique** The human eye can estimate the center of a wide object more accurately than it can line up two objects. For this reason, different pointing techniques should be used depending on the type and apparent size of the sight in the telescope.

When pointing on narrow sights, such as the center of a red and white target or distant range pole, straddle the sight with the double cross hairs. When pointing on wide sights, such as a lath or range pole at close range, split the sight with the single cross hair.

D. Split Bubble Image

A frequent error in zenith angle measurement is failure to adjust the split bubble image into coincidence before reading the angle. If coincidence is not made, the scale is not indexed to the vertical and significant errors can result. Always set the bubble before each reading.

E. Measuring Angles

Measure angles as rapidly as comfortably possible with a uniform rhythm. Take the first reading at an object, rather than fidgeting with the tangent screw trying to improve the pointing. Too much pointing time increases the probability of error through instrument settlement or atmospheric changes. Speed should not be cultivated at the expense of good results. Accuracy is more important than speed.

F. Reading the Circles and the Micrometer

1. Call Outs Carefully read and call out each reading to the recorder. Call out the entire reading each time so any large blunders will be caught. Have the recorder repeat the reading to the instrument man after it is recorded.

2. When using the old style instruments, always check the ten minute interval. This is done by counting the number of graduations between opposite, corresponding degree marks, after the circle has been brought into coincidence.

G. Analyzing Observation Sheets

Many recording errors and inconsistencies can be caught by carefully analyzing observation sheets. The following most important items should be checked in the field by the recorder and then rechecked in the office. They are:

1. Spreads between the seconds of direct and reverse readings should be consistent and in the same direction throughout the set.

2. Ten minute reading errors can frequently be located by examining a set of positions. This is the reason that it is highly recommended that the circle be closed (angle between foresight and backsight observed and recorded) for each observation. If one observation disagrees with all others by ten minutes, it is safe to assume that a reading or recording error was made.

3. When a direct and reverse observation of a position are in different minutes, be sure the average second value is coupled with the correct minutes value.

4. When checking the direction to a station and either the minutes or the seconds value of the backsight mean observation are greater than corresponding values to another station, be sure the subtraction is correct.

5. Using the entire set of averaged angles in the calculation will normally prevent errors in either of the above instances.

3.6.3.3 Natural Factors

A. **Differential Temperatures** Bright sunlight striking certain parts of the instrument may cause differential expansion of the metal components of the instrument, resulting in small errors. To minimize this error, it is recommended to work under a parasol.

B. **Heat Waves** Heat waves can cause distortion of lines of sight near a reflecting surface. The use of a tower can reduce the effect of heat waves. Working at night is another alternative available. Otherwise, unless it is urgent, on control surveys the work should be postponed until better conditions exist.

C. **Phase** If a sight is not evenly lighted on both sides, the instrument man will tend to point toward one side. This phenomenon, called "phase", can be reduced by using a target with a flat surface pointed directly toward the instrument. "Y" shaped targets are useful in reducing phase.

D. **Diffraction** If a line of sight passes very near a solid object such as a pole, light rays from a distant target may bend or diffract around the object, causing the object to appear in the wrong place. The closer the obstacle is to the instrument, the greater the diffraction. Diffraction can occur in either horizontal or vertical observations.

To detect diffraction, move the focusing knob slowly back and forth while watching the target. If the target appears to move relative to the obstacle and the cross hairs, the obstacle is causing diffraction. Offset the line to correct this condition.

E. **Refraction** When light waves pass from a medium of one density into a medium of a different density, the rays change in direction (bend). The change in direction is called refraction. Since sight lines are light rays, they are refracted, or bent, by changes in the atmosphere, causing small errors in angular measurement.
Normally, the lateral refraction is insignificant in most surveys, but its effects can be further minimized by understanding and avoiding situations that generate the largest refraction of line of sight.

1. When the sun shines on a barren, dark surface, the surface warms relatively quickly. This warms the air and, if calm, it produces a column of warm, light air rising from the surface.

2. Other situations include:

- Dark, freshly plowed fields lying between lighter colored areas of growing crops.
- Clear areas between heavy forests.
- Large bodies of warm water between land areas.
- Open valleys bordered by bluffs on either side. If a line must pass over a valley, set the observation points as far back from the edges of the valley as possible.
- Air tends to layer parallel to the slopes of embankments or the base of foothills.

3. Minimizing Refraction When refraction is probable in angles to be measured, or is suspected in angles that have been measured, carefully examine the survey area and plan station locations to avoid problem conditions listed above.

Make your observations when a breeze keeps the air stirred and prevents layering, or on cloudy days or at night. Reobserve lines under different atmospheric conditions, preferably when the wind is from a different direction.

3.6.3.4 Miscellaneous Factors

A. **Trigonometric Functions** When trigonometric functions are used in computations, the precision of the computations is affected by the rates of change of the functions (the magnitude of the differences between the functions for a given angular increment). For example, to compute the impact of an angular error on the precision of the 'Sin' function at a particular angle, use the following equation:

$$\boxed{precision\_of\_\sin = \frac{\sin(angle + error) - \sin(angle)}{\sin(angle + error)}}$$

Example: The precision of a sin of an angle of 5° with an error of 5" is 1:3610.

$$\boxed{precision\_of\ \_\sin = \frac{\sin(5° + 5'') - \sin(5°)}{\sin(5° + 5'')} = 0.000277\ \ or\, 1:3610}$$

For a 'Cosine' and 'Tangent' replace the 'sin' in the above equation with the respective trigonometric function. The following table shows the angles at which trigonometric functions have the highest and lowest precision.

	Sin	Cos	Tan
Highest precision	90°	0°	45°
Lowest precision	0°	90°	0° or 90°

The precision of the angular measurements might have to be increased to compensate for the large rates of change in the trigonometric functions.

This problem can be lessened by careful reconnaissance which:

1. Minimizes the use of such angles in the computations.
2. Provides sufficient checks that will ensure that desired accuracy is maintained (for example, cross ties in traverse).

B. Relationship Between Angular and Linear Measurement

1. Consistency   When a survey involves both angular and linear measurements, maintain consistency (if practicable) between the precision of angular measurements and that of the linear measurements. In other words, keep the offsets in line, caused by errors in angular measurements, approximately equal to the errors in linear measurements. For example, if distances are to be measured to a precision of 1/10,000, measure the angles to the nearest 20 seconds. (A 20 second error will result in an error of 0.10 meters (0.32 feet) offset in 1,000 meters (3280.8 feet).)

2. Angular errors and their corresponding linear errors per unit of distance can be computed from:

   Linear error = Tan(angular error) x *(distance of the line)*

   Example: an angular error of 1" over a distance of 300meters (1000') is 1.5mm (0.005').

3. Inconsistency   Often it is not practical to maintain the same precision in angular and linear measurements. In this case the accuracy of each measurement should be estimated and properly recorded. During the computation (or adjustment) of this data, each measurement should be weighted in accordance to its precision.

Computation methods that do not accommodate for weighting measurements (such as the Compass Rule) should be avoided in this instance.

C. Curvature and Vertical Refraction

Zenith angles measured for horizontal and vertical reduction of long lines require curvature and refraction corrections.

The manufacturer's specifications for most total stations indicate that the earth's curvature and atmospheric refraction are internally computed and the corrected horizontal and vertical distances displayed. This correction can be eliminated (or balanced) by taking the mean of two reciprocal measurements. The reciprocal procedure means that a distance and a zenith angle are measured at point A to point B and then from point B back to point A. At longer distances (over 1000 meters (3280.8 feet)), it is better to use this averaging solution than the on board correction factors provided within these total stations, even when they are in perfect adjustment.

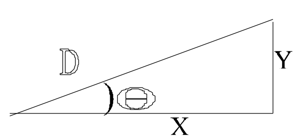
3.7 Vertical Measurement

3.7.1 Purpose

In addition to the horizontal position of points and features as determined by the previously described linear and angular measurement, the complete survey requires vertical measurements. Such vertical measurements establish the elevation of points in relation to a datum that extends through and beyond the project limits. In the coordinate survey, the horizontal position is described by X and Y coordinates and the elevation described by the letter Z (or H.) The complete description of a point being (X,Y,Z).

Vertical angular measurements are also required to reduce slope measured distances to horizontal distances. Most EDM instruments used by the Department are equipped with zenith sensing devices that provide the reduction internally. These instruments display the operator's choice of slope, horizontal or vertical distance.

Some older EDMs that are mounted on top of a theodolite measure the slope distance only. The theodolite is then used to make the angular measurement and the horizontal distance(X) is computed from the zenith angle(0) and slope distance(D). A correction for the earth's curvature and atmosphere must be applied to determine the true horizontal distance and difference in elevation (Y).



3.7.2 Importance

The determination of accurate elevations is an extremely important part of the information required for the design of highway projects. Grade lines, drainage structures and other highway features are designed in relation to existing and final elevations. Volumetric quantities are determined by preliminary (before) and final (as constructed) cross sections. Additionally, accurate elevations are very important in ensuring the reliability of photogrammetric mapping and orthophoto products.

Due to its importance in all other phases of the project development, vertical measurements to establish primary elevation control are made at an early stage of the survey. For example, in traversing, vertical angle elevations are established for all traverse control monuments to aid in the computation process. The control monuments become benchmarks. Subsequent fixed end loops between the control monuments established, again by differential leveling, are used to establish benchmarks for photogrammetric, preliminary, construction or other control networks.

3.7.3 Planning

By the time a project survey is completed from preliminary through the construction phase, each benchmark will have been used many times to provide the base for vertical measurement. Proper planning in anticipation of the future uses of vertical control benchmarks is as essential as that required for the horizontal control. Some considerations in that plan should be:

A. Location of the primary control (generally on the base traverse monuments).

B. Permanence (outside of anticipated construction limits), and type of monument set

- concrete monuments (permanent).
- Iron pins (semi permanent).
- wooden hubs (temporary

C. Accessibility (on the Right of Way or other accessible lands).

D. Spacing (generally at 300 meters (1.000 feet) or less).

E. Visibility.

3.7.4 Methods

Vertical measurements are made directly or indirectly. The choice of the method and its procedures depend on present and future accuracy requirements and the relative cost. Considerations in selecting the method and procedures should include:

- Classification of controlling benchmarks. (The precision of the survey should be compatible with the accuracy of the controlling monuments.)
- Type of equipment available.
- Future survey needs.

3.7.5 Direct Vertical Measurement

This method means "the direct reading of elevations or vertical distances”. The two common types of direct elevation determinations are readings from altimeters and from direct elevation rods. Tape (or Laser) measurement of a building height or depth of a mine shaft are also examples of direct measurements.

Elevations obtained by any of the above direct methods are less accurate and their application quite limited in highway surveys.

3.7.6 Indirect Vertical Measurement

Indirect vertical measurements require the use of calculations to determine elevations or vertical distances.

Most vertical measurements made in highway surveys are made by indirect methods, such as spirit level (differential) and trigonometric leveling. There are numerous procedural systems used in both methods to achieve various levels of accuracy versus the expenditure of field and office time. In recent years, vertical measurements are made by GPS as well. It has been proven that careful GPS leveling can yield accurate elevations that are comparable with those of differential or trigonometric leveling. GPS leveling will be discussed in Chapter 4 of this manual.

Prior to the development of the total station, almost all vertical measurements on highway projects were made by differential leveling. By strict adherence to distance limitations and other procedural methods, trigonometric measurements can often be the best option for making vertical measurements.

3.7.6.1 Accuracy

Tolerances for the various types of control points and other elevation points are a function of the needs of a given project. The general requirements are cited here for reference.

A. Base traverse and monumented control points:
- Second order Class II misclosure (mm) is 8 x square root of D(D = shortest length of section in kilometers) or
- Maximum loop misclosure (mm) is 8 x square root of E (E = perimeter of loop in kilometers).

B. Secondary control, such as construction benchmarks, horizontal vertical photo control hubs, etc.:
- Third order misclosure (mm) is 12 x square root of D (D = shortest length of section in kilometers) or
- Maximum loop misclosure of 12 x square root of E (E =perimeter of loop in kilometers).

C. Vertical only spot elevations, slope stakes, etc. ± 0.015 meters (0.05 feet).

3.7.6.2 Differential Leveling

3.7.6.2.1 General

A. **Equipment** The standard instrument for all differential leveling is the pendulum type (automatic) level.

The Department primarily uses Philadelphia two section, Lenker Rods, and fiberglass leveling rods. Each type of rod has its particular advantage under certain field conditions. Any rod used should be clean, "tight", and have properly indexed scales, Slip joint rods should be checked periodically for index.

B. **Instrument Setups**

1. Use a hand level in uneven terrain to aid in selecting setup and turning point sites.

2. Do not waste time by deeply imbedding tripod feet. Settlement is usually insignificant. Avoid setups on hot pavement or in spongy or muddy soil. If a setup in this area is unavoidable, be very careful around the instrument and constantly check level bubbles and backsights.

3. Set turning points so backsights and foresights are approximately equal. This compensates for curvature and refraction and for maladjustment of the instrument.

4. Use sight distances that best fit the terrain and are the most comfortable for the instrument person. Some operators can read the rod at much further distances than others. Sight distances should never exceed 75 meters (250 feet).

5. In steep terrain, place "turns" and instrument setups so they follow parallel paths (not along the same line).

6. Periodically test the level to be certain the pendulum compensator is working. Point on a "natural" sight with the telescope over a foot screw, and turn the screw back and forth, or lightly tap the instrument. If the cross hair dips and returns to its original position, the compensator is working properly.

3.7.6.2.2 Turning Points and Benchmarks

A. Establish benchmarks of the same physical quality as the technical quality of the leveling procedures. Set them in a stable, protected location.

B. Do not use spikes in utility poles. Do not use wooden stakes except as temporary benchmarks.

C. Make each turn stable and with a definite high point. If a turning point (TP) does not have a prominent point, mark the exact point with keel or paint. Depending on the soil, stakes driven at a slight angle make excellent TP's. A small piece of flagging will make them easily recoverable.

3.7.6.2.3 Rod Reading

A. Focus the eyepiece to eliminate parallax before any readings are made.

B. Do not deliberate over readings. Read and call them out in a moderate rhythm.

C. Turn through important points, rather than take "side shots". Benchmarks and photogrammetric control points should never be side shots.

D. Wherever possible, make it standard practice to plumb the rod with a rod bubble. In the absence of a rod bubble, the rodperson should slowly sway the rod at and away from the instrument. The observer reads and records the lowest reading. The rod must be set on a sharp or rounded projection; otherwise the rod will rise as it is rocked and will result in a false reading.

E. Avoid low, ground-skimming shots where refraction might become pronounced. Also, avoid sighting close to obstructions that might diffract the line of sight; if possible, not closer than 0.3 meters (1 foot) to obstruction.

3.7.6.2.4 Single - Wire Levels

Single-wire leveling is the most common and widely used method of vertical differential measurement. With proper attention to procedural consistency, and by the use of several variable methods of elevation checks, third-order accuracy may be achieved with single-wire leveling. Some of the various methods will be described in general terms only. Detailed procedures may be found in the appropriate referenced publications at the beginning of this manual.

Computations - Normally, single wire notes are reduced to height of instrument (HI) and turning point (TP) elevations as the survey progresses. To check the elevations of benchmarks (BM's) that are turned through, differences in elevations, delta elevations, are also calculated. Basically, the difference of the sum of the plus rods (backsights) and minus rods (foresights) should equal the difference in elevation between the BM's. All side shots must be eliminated from the calculation.

Adjustments Simple, single-wire level runs should be straight line adjusted. The closing error is prorated to each TP between two consecutive, controlling benchmarks.

$$C = E \cdot \frac{n}{N}$$

Where:

C – Correction applied at a TP.
E – Closure error of the leveling loop.
n Number of turns to a given TP.
N – Total number of TP's in the loop.

3.7.6.2.5 Double Turning Point (TP) Leveling

This technique uses two parallel, independent foresight and backsight TP's for each HI. It is usually used for third order leveling or very special circumstances. Each pair of TP's is set, if possible, at an appreciable difference in elevation (preferably 0.3 meters or more). They are also set a few feet apart so the level will have to be rotated slightly between the two rod readings.

From each setup, single wire plus shots are read on both backsight TP's; minus shots are read on both foresight TP's. Notes are kept separately for each line of levels.

The adjusted elevations from the two lines of TP's are averaged.

The system has some advantages in that the HI is determined from each of the two lines and misreading or misleveling blunders can be isolated immediately.

The system is time consuming and both lines are run in the same direction, which may not cancel natural systematic errors.

3.7.6.2.6 Double Height of Instrument (HI) Leveling

This technique is similar to double TP leveling and is used for the same reasons as double TP leveling. A double line of levels is run through a single line of TP's. At each setup site, two HI's are established, at approximately 0.3 meters difference. From each HI the rod is read on the single backsight TP and on the single foresight TP.

The system has approximately the same advantages and disadvantages as for double TP leveling except that the difference in elevation can be immediately checked between the two TP's. If the difference is more than 0.001 meters (0.002 feet), a third HI may be used.

3.7.6.2.7 Three Wire Leveling

When the distance between control benchmarks exceeds three kilometers, the system of three wire leveling can often be the most efficient method to establish project control benchmark elevations.

With this leveling technique, the cross hair and both stadia hairs are read to the nearest thousandth of a meter. (This system is widely used with an Invar rod and the readings taken to the nearest thousandth of a meter.) Stadia intercepts of plus and minus shots are accumulated. The running totals are constantly monitored so balance can be maintained between totals of foresight and backsight distances. The backsights and foresights should be balanced within 5 meters (20 feet) when setting BM elevations.

This technique is generally preferred over the two previously described methods in that it is faster and, to a large degree, self checking.

Since it is anticipated that this technique will be used most often in project control surveys, special requirements will be discussed in some detail.

A. **Equipment**

1. **Instrument** The Department generally uses pendulum levels with either a stadia ratio of 0.3 to 100 or 1.0 to 100. For precise levels, the 0.3 to 100 ratio is preferred because the stadia hairs are nearer the optical center. It also permits a greater elevation difference between the level and the TP while keeping all three cross hairs on the rod.

2. **Rods** If Invar rods are not available, it is recommended that the best Philadelphia rod available be used on these surveys. Even new rods should be checked upon delivery, as some rods have been found to be not calibrated.

B. **Procedures**

1. Check the instrument and rod before starting each day's run, test the level for collimation error. Test at or near the first setup of the day and record the process in the field notes. If the error exceeds 0.005 in 200, the instrument should be adjusted. Any time the instrument has a severe jolt or bump, it should be readjusted.

Check the rod in the raised position to ensure there is no index difference above and below the slip joint. The rod should be rechecked each time it is extended.

2. **Setups** Keep all sights within 75 meters (250 feet). When rejected readings average more than two in every ten, shorten the sighting distance.

3. **Turning Points** Railroad spikes, boat nails, wooden stakes or stakes may be used for TP's. If possible, all TP's should be left in place and flagged in case a complete or partial rerun is required. Permanent TP's should be numbered so they can be identified when recovered.

4. **Benchmarks** Establish all benchmarks prior to leveling. Check all found monuments that are to be incorporated in the level line for stability.

5. **Rod Readings** Plumb the rod with an accurate rod level. Start the rod reading with the top stadia wire, and progress to the bottom wire.

   Estimate readings to 0.002 meters. Read at moderate speed without deliberations. Do not turn or pick up the instrument until the note keeper has verified the spread. If the spread between top and center wire and bottom and center wire exceeds 0.002 meters (0.0065 feet), reread all three wires. The original readings should be crossed out with a single line and new readings entered on the next line of the Field Book.

6. **Reruns** The highest order of accuracy required will normally be met by a single run of three wire levels. If the run fails to close within the tolerance specified, one of two problems probably exist.

   a. There may be a discrepancy between the elevations of the beginning and closing benchmarks. This should be the first possibility checked. Be sure that the government elevation data sheet is the last published for the two points. (There is a possibility that one or both of the benchmark elevations have been readjusted.)

      If the last information was used, carry the survey to the next benchmark.

   b. If the above does not account for the discrepancy, the run should be releveled in the opposite direction. Releveling at a different time of day is preferable.

3.7.6.3 Trigonometric Vertical Measurement

Trigonometric vertical measurement is a procedure whereby vertical differences in elevation are computed from slope distance and zenith angle measurements.

The development and continuing refinement of total stations is making rapid changes in the use of trigonometric vertical measurements. These instruments are of varied vertical angle accuracy and certain procedural restrictions must be applied to equal the accuracy of differential leveling. Those restrictions required to meet the various tolerances will be discussed under each of the vertical traversing techniques described herein.

A. **Applicability** Vertical traversing is often the most practical (and economical) method for establishing elevations in rolling to steep terrain. It is useful for many types of surveys. Some of these are:

   1. Reconnaissance surveys.
   2. Control for aerial photography
   3. Check levels for long differential lines.
   4. Establishing elevations for datum adjustment.
   5. Establishing low order benchmarks by precise vertical traversing. This would be done when accuracy is difficult to attain at reasonable expense by differential leveling.
   6. Slope staking.
   7. Cross sectioning.

B. **Accuracy Attainable** The accuracy that can be obtained from vertical traverses is sufficient for surveying work. The accuracy attained depends on the individual measurement accuracies of:

   1. **Slope distance** With today's EDMS, slope distance accuracies are seldom significant in vertical measurement. For instance, with a zenith angle between 89 and 91 degrees, a slope distance error of one meter would result in a vertical error of less than 0.02 meter.

   2. **Zenith Angle** The effect of zenith angle depends on the size of the angle and slope distance. To consistently attain good results, proper procedures must be used. Reciprocal observations minimize zenith angle and refraction errors. An erroneous zenith angle will result if the target and reflector are not properly spaced to provide parallel sight lines.

   3. **Height of Instrument (HI) and Height of the Reflector(HR)** The major cause of error in trigonometric vertical measurement is the inaccurate determination of instrument and reflector heights. The net error that results from the two measurements produces a direct error in the difference in elevation. Aligning the height of the instrument and the reflector before making the measurements can reduce the effect of this error.

C. **Calculations** The calculation methods discussed herein provide satisfactory results. The methods are described in principle only, as all such systems require consistent good practices in equipment operation as detailed in other sections of this manual.

The equation for computing elevations using trigonometric leveling (without corrections for curvature and refraction) is:

$$H_B = H_A + S\,(\cos(Z)) + HI - HR$$

Where:

H_B – Elevation of point B (the new elevation to be established).
H_A – Elevation of point A (a point with known elevation on which the instrument is set up).
Z – Zenith angle from A to B.
S – Slope distance from A to B.

The term $S(\cos(Z))$ is the elevation difference component of the slope distance and it is denoted by DE.

There are three basic methods used in the computations to determine the elevation of a point from one or more points

1. **Difference in elevation without HI or HR**. This method is the most recommended one. It requires that the height of the rod and the height of the instrument are the same. As one can see from the above equation, if HI=HR then HI- HR=0. Thus, we do not need either of these heights. Most total stations have a special mark on the reflector as well as on the instrument that facilitates the alignment of these heights.

2. **Difference in elevation with HR only**. – When the instrument is set up at an arbitrary point (not a benchmark) and the reflector is held on a benchmark, HI can be computed adding HR and DE to the height of the benchmark. There is no instrument height measurement involved in this process. Once HI is established, additional points can be leveled ,even if HR is changed.

   The advantage of the above method is that the errors due to HI mismeasurement are eliminated. Once visualized, the systems are easy to use for remote wing points and spot elevations. They also allow the instrument to be set up at the best location for sight and distance.

3. **Difference in elevation with HI and HR**. This method requires the recording of the HI and HR for each observation. Calculations are done according to the above equation. While this is the least recommended procedure for trigonometric leveling, topographic and other constraints may dictate using this method.

3.7.6.3.1 Uses of Trigonometric Leveling

A. **General** – Elevation traversing, spot elevations, profiles of centerline and cross sections, drainage flow lines, or other required vertical information may be acquired by trigonometric leveling. Such elevations may be taken at identifiable points, such as staked centerline stations, or the horizontal position of said points may be determined during the same process.

B. **Elevation Traversing** – Trigonometric vertical measurements can be made at the same time that horizontal positions are surveyed or separately for vertical elevations only. Traverse point elevations should always be determined. Obtaining the elevations of traverse points require very little (if any) additional effort. The elevations of the traverse points can be used as a blunder check for more precise leveling of these points. They are also needed for reducing measurements to the state plane coordinate system,

C. **Vertical Only Traverse** On many occasions the elevation is required on points where the position is known (bridge ends, profile stations, etc.) or points that can be identified (wing points, etc.). Such elevations may be obtained by any of the three computation methods described previously. The accuracy required and position of the needed point relative to established horizontal and vertical control will generally determine the type of vertical traverse required.

D. **Centerline Profile** Centerline profiles are normally determined by differential leveling, but there are situations where trigonometric leveling may be the most economical. Much of the economy will depend on the location of control points in relation to the required profiles and the terrain involved. If the terrain is steep or rolling, requiring numerous "turns" with a level, trigonometric leveling may become the most economical method to obtain such elevations.

Any of the same procedures as described for vertical measurement to remote points may be used. If all the spot elevations required are not visible from a nearby control point, the instrument should be set where one or more control points, and all the required spot elevations, are visible. In order to increase the reliability of the elevations, foresights and back sights should not exceed 300 meters (1,000 feet). All DE's should be read at least three times.

E. **Roadway Cross Sections** The standard method of obtaining field roadway cross section elevations is normally the most efficient. There are situations, however, where trigonometric leveling may be the most economical method. Such conditions may be where cross sections have to be extended beyond efficient taping distance, or where the terrain is steep and level turns are required. In such cases, the most efficient method of trigonometric leveling would be to set up over each cross section station turning right angles and measuring the distances and DE's with the EDM. This may be measured from the profile station, or by measuring the DE from a control point. If a uniform rod height can be used for the entire cross section, the relative elevation can be obtained by sighting another profile station, or a control point. If setting over the centerline station is impractical because of traffic, the same procedures may be used from an offset centerline.

F. **Borrow Pit or Other Cross Sections** Field cross sectioning or grid elevations of borrow pits, building sights, or odd shaped areas by trigonometric leveling can be fast and economical survey procedures. Several options are available using either trigonometric positioning and leveling, trigonometric positioning and differential leveling, or various combinations of the three techniques. The shape of the area and terrain involved will generally be the factors that would most influence which method would be the most economical.

1. Grids by Coordinates (Radial) The trigonometric positioning and leveling technique can be used from a single setup for which the coordinate position and elevation are known, or specifically established for the survey.

   Coordinates for each grid point can be prefigured and the azimuth and distance from the instrument tabulated for positioning the rod person at each grid point. Differences in elevations (DE's) would be read at each point and an accurate grid elevation computed. Total station tracking mode speeds up the operation.

   The primary advantage of this system is that very few stakes or flags are required and much of the control work can be prefigured at the office.

   If the grid area is quite large, additional setup points may be required.

2. Grid by Double Base Lines The trigonometric positioning and leveling technique is advantageous where the shape and topography of the area provides good visibility for the rod person. Two rows of grid lath are placed along one side and one end of the area. The rod person can "eyeball" grid points for either trigonometric or differential leveling on the unstaked points.

   This system requires some time to set up the control lath, but speeds up the location of grid points for the remainder of the survey.

3. Cross Sectioning from Base Line(s) Cross sectioning requires the establishment of one or more baselines. Normally, the instrument would be set up at each station and distances and DE's taken at each break along the cross sectional profile. Proper positioning of the base line(s) for good coverage and sights is the most critical part of this type of survey.

   This system is most advantageous where the terrain is rough and grids would not reflect the true topography.

4. Contouring Other possible uses of trigonometric leveling, such as contouring by random spot elevations, could also be considered for each survey. The point in all of the above being that trigonometric leveling gives the surveyor many options to do the work efficiently and effectively.